BUILDING VIBRATIONS DUE TO DEEP VIBRO PROCESSES

M Achmus, Leibniz University of Hannover, Germany
J Wehr, Keller Grundbau Holding GmbH, Germany
T Spannhoff, Spannhoff Consultant Engineers, Germany

Abstract

Vibro compaction and vibro replacement methods are effective and widely used techniques for ground improvement. However, deep vibro methods cause vibrations which may have negative effects on adjacent buildings. In this paper, prediction equations for foundation and ground vibration intensities derived from the evaluation of measurement data are presented. With prediction formulas for average and "worst case" values, taking the vibrator energy and the distance of the vibrator from a foundation into account, the range of peak particle velocities to be expected can be well described. In combination with design values of admissible vibration intensities from relevant standards and transfer coefficients for the propagation of vibrations inside a building, a risk assessment method for building damage due to deep vibration processes is established. The potential settlement of the foundation soil due to the vibration impact is also considered. Recommendations are given regarding the minimum distance of deep vibro methods from buildings. The prediction equations and the risk assessment method can be used to identify potential risks and can help to decide on collateral measures such as vibration measurements or building state documentation.

Keywords: Buildings, Dynamics, Risk & Probability Analysis

1. Introduction

For the compaction of deep soil layers, the "vibro compaction" (VC) and "vibro replacement" (VR) methods are widely used and effective techniques for ground improvement.

Vibro compaction is a method used to densify granular soils using a depth vibrator, which is a steel cylinder with diameters between 30 and 40cm and lengths between 3 and 5m. The horizontal vibrations are produced by a rotating eccentric weight which is driven by an electrical motor mounted within the vibrator.

The vibrator is lowered into the ground under vibration with the aid of water jets. Once the required depth is reached, the water jets are turned off and the vibrator is retracted with a gradual up- and down-movement. Due to the vibration, the interparticle friction between the soil grains is reduced, rearranging the particles to a denser state. At the end of the process a column of compacted granular soil has been produced. The construction process is elucidated in Figure 1.

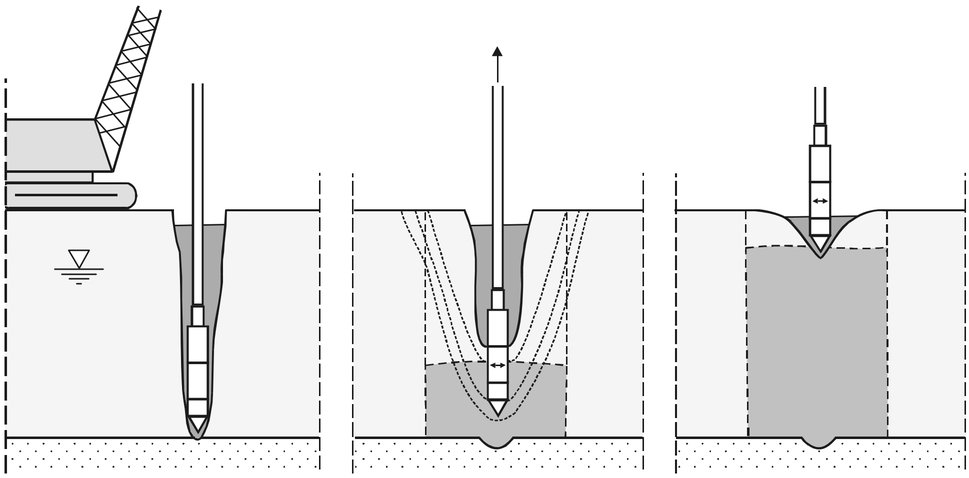

Fig. 1 Construction process of the vibro compaction method

In the Vibro Replacement method, stone columns are installed in weak soils to improve their load bearing and settlement behavior. To construct the stone columns, the vibrator is lowered into the soil to the design depth under action of vibrations and a jetting medium (compressed air or water). The cavity formed is filled with hard inert stone material, free of clay and silt fines. The required interaction between the stone columns and the surrounding soil is developed by the stone infill being introduced and compacted in stages, each layer of stone being thoroughly compacted by the vibrator. In temporarily stable soils, the stones can be filled in by a top feed process (successive withdrawal of the vibrator, filling in a layer of stones, placement of the vibrator in the hole and compaction of the infill) or by a bottom feed process (vibrator stays in place and infill material is placed through a material lock and a feeder pipe).

As with all dynamic construction methods, with deep vibratory compaction a portion of the vibration energy is emitted to the surrounding soil as vibratory waves. These waves transmit energy through the soil and may cause vibrations in adjacent buildings.

The magnitude of groundborne vibrations may be quantified in terms of acceleration, velocity or displacement. For assessing building vibrations, the peak particle velocity (PPV) is usually considered as a measure of vibration intensity. For the purpose of predictions in most cases equations of the following type are used:

$$PPV = K\frac{\sqrt{E}}{r} \qquad (1)$$

Here E is the theoretical energy, usually taken as the maximum nominal energy of the vibratory device, r is the distance of the device from the point under consideration, and K is a factor whose quantity or bandwidth is to be determined empirically by evaluation of vibration measurements.

For instance, in annex 3 of Eurocode 3 [1] a K-value of 22 is recommended for vibratory pile driving. In this approach, the energy per cycle is derived from the nominal power W of the vibrator and the frequency of vibration f by

$$E = \frac{W}{f} \qquad (2)$$

Using equation (1) and K = 22, the PPV is obtained in mm/s when E is applied in kNm and r in m. Similar prediction formula have been presented by Achmus et al. [2] for pile driving and soil compaction by vibration plates and rollers. In this study, the bandwidth of K-values derived from measurements of the vibrations of foundations was described by different K-values for 50% probability of exceeding (P=50%) and for 2.25% probability of exceeding (P=2.25%). A comparison of measured and predicted PPVs of the foundations is given in Fig. 2 for pile vibrators with nominal energies between 3.0 and 4.6 kNm.

Fig. 2 shows that the K-value for P=50% is an indicator for the average value to be expected, and the K-value for P=2.25% is a good indicator for the “worst case” PPV. The K-values are smaller than the value given in Eurocode 3, because the prediction applies to the vibration components measured at foundations, whereas the Eurocode 3 prediction applies to the resultant ground vibrations. Usually, the transfer of the ground vibration to a foundation is accompanied by a significant decrease in vibration intensity.

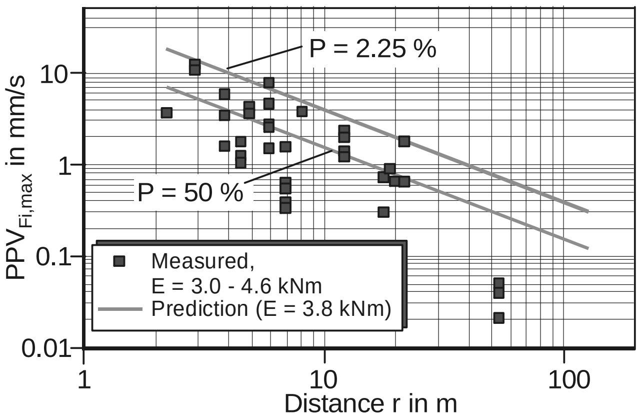

Fig. 2 Comparison of measured and predicted foundation PPVs induced by vibratory pile driving according to Achmus et al. [2]

Using the same conceptual approach, vibrations induced by deep vibro processes were considered. The results of vibration measurements carried out on several deep vibration construction sites were thoroughly analysed. The data stem from construction sites of the Keller Grundbau company, at which the four vibrator types given in Table 1 were used.

Table 1 Data of the Keller deep vibrators considered in the study

Vibrator type		T	M	L	S
Length	m	4.75	3.15 – 3.55	3.10	3.00 – 4.30
Diameter	mm	290	290	320	400 – 470
Weight	t	2.00	1.52 – 2.20	1.82 – 2.60	2.45 – 4.20
Power	kW	35-50	50	100	120
Operation frequency	Hz	50	50	60	20 - 40
Max. Energy	kNm	0.7 – 1.0	1.0	1.67	3.0 – 6.0

With these data, for the M and T vibrators with vibration frequencies of 50Hz energies of maximum E=1kNm, for the L vibrator with a frequency of 60Hz E=1.7kNm and for the S vibrator with a frequency of 30Hz E=4.0kNm apply. It should be mentioned that the T vibrator is today no longer in use, but of course the data could be used in the study.

2. Evaluation results

Regarding the quality of data to be analysed, it was required that the measurements and boundary conditions were documented thoroughly and that at least the vertical and one horizonal component of the ground and/or the foundation vibration velocities had been measured. A total of 21 vibration measurement reports with these criteria were available and were analysed.

In Figure 3 the measurement results for the vibrations of the foundations are presented. In the upper part the maximum of the three or at least two components of the peak particle velocity is given, and in the lower part the vertical component is depicted separately, dependent on the distance *r* of the deep vibrator from the foundation under consideration. The vertical component of the *PPV* is of special importance, since in many cases vertical vibrations of building slabs, which depend on the vertical foundation vibration, are decisive in the assessment of vibration intensities.

For each measured *PPV*, with regard to equation (1) a *K*-value can be determined. Assuming a Gaussian distribution, *K*-values belonging to different probabilities of exceeding can be derived for the set of measured data. In the evaluation *PPV* values less than 0.3mm/s for distances less than 20m (which probably correspond to situations in which the vibrator did not work at full capacity) were not taken into account. With the remaining data, the following *K*-values were determined:

- *K*-values for the maximum component of the foundation *PPV* ($PPV_{Fi,max}$):
  50% probability of exceeding: $K_{F,\max}^{P=50\%}$ = 10.3,
  2.25% probability of exceeding: $K_{F,\max}^{P=2.5\%}$ = 23.1.
- *K*-values for the vertical (*z*-)component of the foundation *PPV* ($PPV_{F,z}$):
  50% probability of exceeding: $K_{F,z}^{P=50\%}$ = 7.3,
  2.25% probability of exceeding: $K_{F,z}^{P=2.5\%}$ = 17.0.

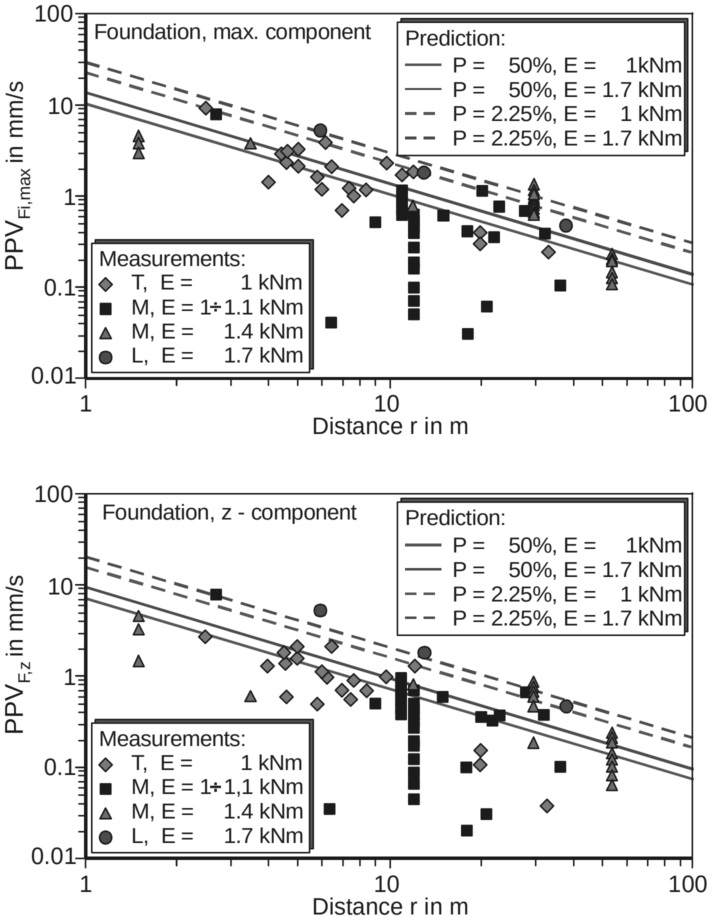

Fig. 3 Results for foundation *PPV*'s. Top: maximum of the components measured; bottom: vertical component

The prediction lines for energies of 1 kNm and 1.7 kNm resulting from these values are also shown in Fig. 3. Obviously, the bandwidth of possible *PPV*s can be well described with the prediction curves for 50% and 2.25% probability of exceeding.

In Fig. 4 measurement and prediction results for resultant ground *PPV*s ($PPV_{G,res}$) are shown. In the available measurement reports, usually only the maximum values of the velocity components were documented. To derive the resultant velocity, the following equation was used:

$$PPV_{res} = \sqrt{PPV_x^2 + PPV_y^2 + PPV_z^2} \qquad (3)$$

Since the maximum components do not necessarily occur at the same time, this formula gives an upper limit. Moreover, if only one horizontal component was measured, this value was assumed to be the same for the second (perpendicular) horizontal component.

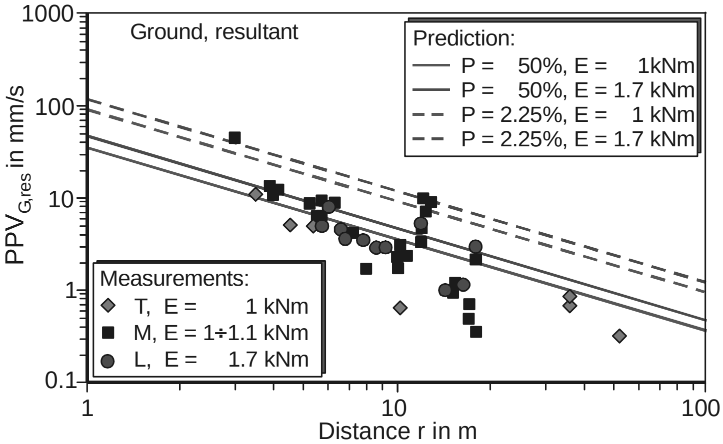

Fig. 4 Results for resultant ground *PPV*s

From the analysis of the ground *PPV*s, the following *K*-values were determined:

- *K*-values for the resultant ground *PPV ($PPV_{G,res}$)*:
  50% probability of exceeding: $K_{G,res}^{P=50\%}$ = 37.2,
  2.25% probability of exceeding: $K_{G,res}^{P=2.5\%}$ = 95.2.

Again, the prediction lines for energies of 1 and 1.7kNm are shown together with the measured data in Fig. 4.

In two data series, also measurements during deep vibro compaction with S vibrators (power 120kW, operation frequency 30Hz, cf Table 1) were reported. However, in these series only the maximum *PPV* component or only the maximum horizontal component was measured, so that with respect to the data quality criteria given above these series were not used in the evaluation. In spite of that, in Fig. 5 the maximum components measured are compared with the prediction lines for the resultant *PPV* according to the $K_{G,res}$-values given above. It is found that also for vibrators with greater energies the prediction gives fairly good results.

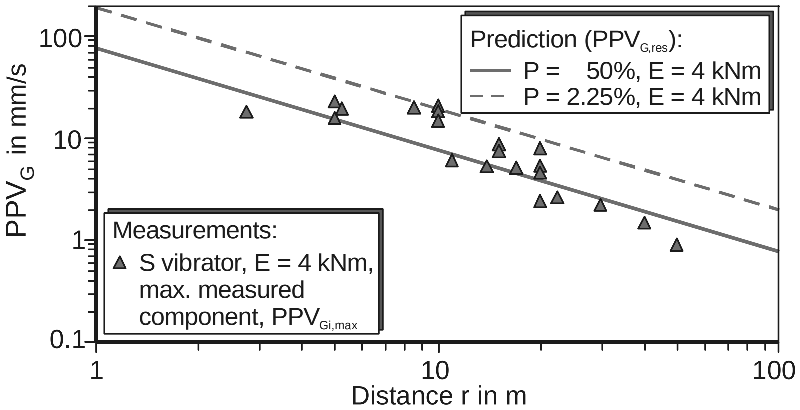

Fig. 5 Results for ground PPVs induced by S vibrators

All in all, it can be stated that equations of the type (1) are suitable for predicting *PPV*s to be expected during deep vibro methods. By means of the *K*-values presented, the bandwidth of both foundation and ground *PPV*s can be estimated.

3. Assessment of the influence of vibrations on buildings

The maximum *PPV* is usually applied as a reference value for assessing possible damage to buildings due to vibrations. In relevant standards or guidelines, e.g. DIN 4150-3 [3] in Germany, *PPV* design or threshold values are given which do not lead to damage in normal buildings. If the actual *PPV*s are less than these design values, no damage is to be expected. These values were evaluated from numerous measurements on buildings of different kinds. Of course, the use of these values assume the conditions of the building to be normal (i.e. good), since a very sensitive building may exhibit damage such as fissures even from very small impacts. However, in these cases the poor condition of the building is the main reason for the damage, and not the (low) vibrations. Normally, it can be assumed that the damage would have occurred over time even without any vibration impact from construction work.

The German regulation DIN 4150-3 distinguishes between steady-state and transient vibrations. Vibrations due to deep vibro methods are classified as steady-state vibrations. For this case, the DIN 4150-3 gives design values for the horizontal *PPV* ($DPPV_h$) acting at the top floor level of a building. Different values are given for industrial buildings, residential buildings and very sensitive (e.g. historic) buildings (Table 2). In addition, as a design value for the maximum vertical *PPV* of floor slabs $DPPV_z$ = 10mm/s is given.

In contrast to the case of transient vibration, DIN 4150-3 gives no design values for the foundation *PPV*s due to steady-state vibrations. But, in the Swiss regulation SN 640312a [4] design values for resultant *PPV*s of construction elements are given (Table 3). For vibratory compaction the values for frequent vibrations apply, which yields design values between 6 and 12mm/s, dependent on the frequency of the vibration impact.

Table 2 Design values of DIN 4150-3 for the maximum horizontal *PPV* at the top floor level of a building due to steady-state vibration

Building type	Design values for the horizontal *PPV* of construction elements at the top floor level, $DPPV_h$ in mm/s
Industrial buildings	10
Residential buildings	5
Very sensitive buildings	2,5

Table 3 Design values of SN 640312a for the maximum resultant *PPV* of construction elements of buildings

Sensitivity class	Frequency class	Design values for the resultant *PPV* of construction elements, $DPPV_{res}$ in mm/s		
		f < 30Hz	*f* = 30 to 60Hz	*f* > 60Hz
Normal sensitivity (e.g. usual residential buildings, office buildings)	occasional	15	20	30
	frequent	6	8	12
	permanent	3	4	6
Little sensitivity (e.g. industrial buildings)		Up to two times the respective values for normally sensitive buildings		
Increased sensitivity (e.g. new residential buildings, historic buildings)		Between 100 and 50% of the respective values for normally sensitive buildings		

Transferring vertical vibrations from the foundation to floor slabs an increase of the maximum *PPV* may occur. This can be described by a transfer coefficient, which is defined as the ratio of the floor slab *PPV* to the foundation *PPV*. Transfer coefficients are dependent on the type of building and the floor slabs and in particular on the vibration frequency. With vibrator operation frequencies around 50Hz (T, M and L vibrators, cf Table 1) resonance can be excluded, and transfer coefficients are then always less than 2.0 (see [2], [5]). Since the design value $DPPV_z$ for vertical floor slab vibrations is 10mm/s, the respective design value for the foundation can be estimated at 5mm/s. Other values apply if the operation frequency of a vibrator lies in the range of the eigenfrequency of a floor slab. Then resonance transfer coefficients of 10 to 15 can occur, which means that much smaller design values for the foundation *PPV* have to be used. However, eigenfrequencies of floor slabs are rarely more than 25Hz. Thus, resonance may only occur with S vibrators, which operate at a frequency of 30Hz (cf Table 1).

Regarding horizontal vibrations, normally no resonance is to be expected in the transfer of the vibration from the foundation to walls or floors inside a building. As an approximation, the design values for the horizontal *PPV*s at the foundation can be set equal to the design values for construction elements at the top floor level (Table 2). Considering the above, for residential buildings a design value of 5 mm/s applies for the maximum horizontal *PPV* at the foundation.

Building damage caused by construction works can also occur indirectly, if the soil beneath the foundation of an adjacent building settles due to the vibration effects. In the German DIN 4150-3 it is explicitly stated that in special cases this effect should be taken into account. Loose fine-grained, but non-cohesive, soils like sands or silts are particularly sensitive to vibration.

Settlements of such soils can occur due to a temporal reduction of shear strength, which causes a rearrangement of particles to a denser state. As the most relevant parameter in that respect, the resultant peak particle acceleration (*PPA*) is usually considered. As a design value for sensitive soils (i.e. loose or loose to medium dense sand), Funk [5] recommends one third of the gravity acceleration *g*:

$$PPA_{G,res} \leq DPPA_{G,res} = \frac{1}{3} g = 3{,}3\, m/s^2 \qquad (4)$$

If this value is kept, settlements can be excluded even for very sensitive soils.
For steady-state vibration with a frequency *f*, the *PPA* is connected with the *PPV* through the following equation:

$$PPA = 2\pi\, f\, PPV \qquad (5)$$

This means, if the design *PPA* according to equation (4) is used, that the design values for the resultant ground *PPV* are 10.5mm/s for an operation frequency of 50Hz (T, M and L vibrators) and 17.5mm/s for an operation frequency of 30 Hz (S vibrator).

4. Risk assessment for deep vibro methods adjacent to industrial and residential buildings

Combining the prediction equations for soil and foundation *PPV*'s and the design values for admissible vibration intensities presented in the foregoing sections, a risk assessment method for deep vibro methods can be established.

In the first step of this method the *PPV*s of the foundation and of the ground beneath the foundation are determined by means of the developed prediction equations. To describe the range of possible values, both the average values (P=50%) and the "worst case" values (P=2.25%) shall be considered. From the resultant ground *PPV*, the *PPA* can be easily calculated by equation (5).

In the second step, the transfer coefficients for the propagation of the vibration in the building are to be estimated. If no resonance effects are to be expected, which is the normal case, the maximum vertical *PPV* of floor slabs can be estimated by two times the vertical foundation *PPV*, and the horizontal *PPV* at the top floor level can be set equal to the horizontal *PPV* at foundation level.

Finally, the predicted PPVs have to be compared with the design values from relevant regulations like the DIN 4150-3 or the SN 640312a. If even the "worst case" values are less than the design values, no building damage is to be expected. If the prediction values and especially the average (P=50%) values exceed the design values, caution is necessary. In such cases vibration measurements should be carried out during the deep vibro works or, if the prediction values are much larger than the design values, alternative methods should be considered.

The risk assessment method is summarized in Table 4. In the following, an example is given.

Table 4 Risk assessment method regarding building damage due to deep compaction works

Step	Description / Formula
1	Prediction of foundation and ground vibration intensities: Foundation, horizontal: $PPV_{F,h}^{P=50\%} = 10.3\frac{\sqrt{E}}{r}$ $PPV_{F,h}^{P=2.25\%} = 23.1\frac{\sqrt{E}}{r}$ Foundation, vertical: $PPV_{F,z}^{P=50\%} = 7.3\frac{\sqrt{E}}{r}$ $PPV_{F,z}^{P=2.25\%} = 17.0\frac{\sqrt{E}}{r}$ Foundation resultant: $PPV_{F,res} \approx \sqrt{PPV_{F,z}^2 + 2\,PPV_{F,h}^2}$ Ground, resultant: $PPV_{G,res}^{P=50\%}$; $PPV_{G,res}^{P=2.25\%}$ → $PPA_{G,res} = 2\pi\, f\, PPV_{G,res}$
2	Transfer coefficients $TC = PPV/PPV_F$: For horizontal vibrations of walls and floors: $TC_h \approx 1.0$ For vertical vibrations of floor slabs, if no resonance is to be expected: $TC_v \approx 2.0$ → $PPV_h = TC_h\, PPV_{F,h}$ $PPV_z = TC_z\, PPV_{F,z}$
3	Comparison with design values / Assessment: $PPV_{F,res}^{P=2.25\%} \le DPPV_{F,res}$? $PPV_{F,res}^{P=50\%} \le DPPV_{F,res}$? $PPV_h^{P=2.25\%} \le DPPV_h$? $PPV_h^{P=50\%} \le DPPV_h$? $PPV_z^{P=2.25\%} \le DPPV_z$? $PPV_z^{P=50\%} \le DPPV_z$? $PPA_{G,res}^{P=2.25\%} \le DPPA_{G,res}$? $PPA_{G,res}^{P=50\%} \le DPPA_{G,res}$?

Example

A ground improvement by the vibro replacement method is planned at a minimum distance of 6m from a residential building. A Keller M-vibrator (maximum energy 1kNm, operation frequency 50Hz, cf Table 1) is to be used. From the prediction equations given in Table 4, the following values are obtained:

- Horizontal vibration of foundation: $PPV_{F,h}^{P=50\%}$ = 1.72mm/s, $PPV_{F,h}^{P=2.25\%}$ = 3.86mm/s.
- Vertical vibration of foundation: $PPV_{F,z}^{P=50\%}$ = 1.22mm/s, $PPV_{F,z}^{P=2.25\%}$ = 2.84 mm/s.
- Resultant vibration of foundation: $PPV_{res} \approx \sqrt{PPV_z^2 + 2\,PPV_h^2}$, i.e. $PPV_{F,res}^{P=50\%}$ = 2.67mm/s, $PPV_{F,res}^{P=2.25\%}$ = 6.15mm/s.

- Resultant ground vibration: $PPV_{G,res}^{P=50\%}$ = 6.2mm/s, $PPV_{G,res}^{P=2.25\%}$ = 15.9mm/s.

  Resultant ground acceleration: $PPA_{G,res}^{P=50\%}$ = 1.95m/s^2, $PPA_{G,res}^{P=2.25\%}$ = 4.98mm/s^2.

Concerning the resultant *PPV* of the foundation, the “worst case” value is 6.15mm/s, which is less than the relevant design value according to SN 640312a of 8 mm/s. With an operation frequency of 50Hz, no resonance effects are to be expected in the transfer of the vibration to the floor slabs. Thus, a transfer coefficient of 2.0 can be estimated, and the vertical *PPV*s of the floor slabs are 2.44mm/s (P=50%) and 5.68mm/s (P=2.25%), respectively. Both values are much smaller than the design value according to DIN 4150-3 of 10mm/s.

The transfer coefficient for horizontal vibrations can be assumed to be 1.0, so the horizontal *PPV*s to be expected at the top floor level are 1.72mm/s (P=50%) and 3.86 mm/s (P=2.25%), respectively, and are also both smaller than the design value, which is 5mm/s for a residential building.

Concerning settlement, the ground accelerations have to be assessed. The *PPA* for P=50% is much smaller than the design value of 3.33m/s^2. However, the “worst case” *PPA* (P=2.25%) exceeds this value. This means that in sensitive soils and under unfavourable conditions some settlement cannot be excluded. However, it should be noted that buildings are normally founded on soils with appropriate bearing capacity, which are normally not very sensitive to vibration settlement.

5. Conclusions

Based on the prediction equations derived, in normal cases no damage to residential buildings is to be expected if a deep vibro method with Keller T, M or L vibrators is carried out at a distance of 5m or more. Here an admissible maximum component of the foundation *PPV* of 5mm/s is assumed, and the “worst case” prediction is applied. If average prediction values are used, a distance of only 3m is possible. It should be noted that with shorter distances the design values are not necessarily exceeded. However, in such cases vibration measurements are strongly recommended.

Even closer distances are possible for industrial buildings, but greater distances are necessary if the adjacent building is classified as sensitive.

Concerning ground settlement, for vibrators with 50Hz (T, M and L vibrators) operation frequency, a design value of the resultant ground *PPV* of 10.5 mm/s applies. To totally exclude settlements adjacent to T, M and L vibrators a distance of up to 10m (“worst case” analysis) is required. However, the measurement values given in Fig. 4 show that in most cases a distance of 4 to 5m is enough to avoid exceeding the threshold *PPA*. However, if the building is founded on soil sensitive to vibration, special care should be taken.

Keller S vibrators operate with greater machine power and lower frequencies. Greater power leads to greater vibration intensities, and lower frequencies can lead to resonance effects with regard to the vertical vibration of floor slabs. Thus, the required minimum distances from buildings can be much greater than for T, M and L vibrators.

Vibration assessments based on empirical relations independent of special site-specific conditions can of course give only a rough estimate of the bandwidth of vibration intensities to be expected. But they at least enable the design engineer to identify and assess potential risks and they can give a basis for deciding whether a documentation of the state of the building should be carried out before starting the construction measures or whether vibration measurements simultaneous to the construction works should be carried out. In this respect, the presented prediction equations are recommended for application in practice.

6. References

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