Vibrations from driven concrete piles in layered soft soils close to a railway embankment

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Abstract. Driven concrete piles are a well-known method for foundations for various structures on soft soils. In Aalborg, Denmark, the area around an old liquor factory is to be re-developed. Due to soft soils in the upper 15 to 30 m, all buildings are to be built on foundations based on driven, concrete piles. On the western site boundary is a railway embankment, which is also placed on soft soils. Vibrations from driven piles are a well-known problem and can lead to damage to nearby structures.

The procedure for installing piles under the new buildings called for a desk study to be performed concerning the surface vibrations, caused by the pile driving. Moreover, a field test programme was set up, measuring surface vibrations due to pile driving on the site. A total of 24 piles were monitored during driving, logging peak point velocity used to quantify the vibration magnitude at various distances. All data was processed, and a model was set up to predict magnitude of vibrations from pile driving as a function of distance from the pile. This paper presents the derived model and comparisons with other published models.

1. Introduction

In Aalborg, Denmark, the contractor and developer A. Enggaard A/S is re-developing a site which was formerly used for production of liquor. The site is known as "Spritten". A. Enggaard is planning to develop common housing and flats on the site along with buildings for cultural uses such as museums, some of which will be as high as 17 stories. The site is located east of an existing railway line, which marks the western boundary of the construction site. A map of the construction site is illustrated in Figure 1.

The upper 15-30 m soils in the area are of post glacial origin, consisting of interchanging layer of fine sands and gyttja, which is why the planned buildings are to be founded on driven, precast concrete piles. As the preliminary geotechnical investigations indicated a high degree of variability on site, it was decided to perform a test installation of a total of 24 concrete piles. The piles were driven to depths between 20 and 30 m below the terrain level into firm clay or gravel layers. The purpose of the test pile installation was primarily to assess the bearing capacity of piles to enable optimization of the detailed design comprising several hundreds of piles, some up to 40 m in length.

The railway line running along the western boundary of the site is laid out on an embankment, elevated approx. 5 m above the average terrain level on the Spritten site. The railway embankment was constructed in the 1930s and is laid out directly on the natural terrain without strengthening by, e.g., rigid inclusions under the embankment. As such, the railway line is sensitive towards settlements of the soft soil layers below the embankment fill. Consequently, the 24 test piles were also used to evaluate any potential influence on the railway embankment during vibrations from pile driving.
2. Geotechnical site conditions

A series of geotechnical investigations were performed on site. In general, these investigations showed an increasing depth to good bearing stratum when going north on the site. A 3D geological model was set up and selected cross sections were analysed to evaluate the geological conditions at the site. The good bearing stratus consists of firm clay or an old beach ridge, depending on the specific location on site. The soils on top of these layer are randomly mixed layers of ‘pure’ gyttja, and layers of sand and gyttja in varying ratios. The upper two to four metres are fill material of relatively clean sands. A representative north-south cross section (under the railway track) is presented in Figure 2, and an east-west cross section is illustrated in Figure 3, both compiled from the Danish national database for soil investigations.
Figure 2: Cross section A-A’, (south-north). As illustrated, the soft deposits (presented in yellow) increase in thickness when going north. Grey soils indicate fill materials and represent the railway embankment, which ends in approx. chainage 250 m with an abutment before a bridge crossing the Limfjord.

Figure 3: Typical cross section going east-west. Note the fill layer (grey) under the railway, and the local top of good bearing stratum (brown material). Yellow material signifies soft soils. Note the railway embankment in chainage 50 m.

A typical borehole from site is presented in Figure 4, representing a borehole performed as part of the design of the pile foundations. From the boring profile, it is evident that very soft layers are present below an upper fill layer of approx. 2 m thickness. The soft layer extends to approx. 24 m below terrain level and consists of an irregular mixture of sand and gyttja in various measures. As may be observed from the boring profile in Figure 4, the water contents in the organic layer is significant, which explains the very low shear vane test strengths encountered. Expectedly, the soft soil is normally consolidated with little or no increase of strength over depth, as observed from the performed shear vane tests. Approx. 24 m below terrain level, a harder clay layer was encountered, which constitutes the bearing stratum.
Figure 4. Representative boring profile for the area where the test drive was performed. Shear wane tests are presented with dots connected with full lines. Natural water content is presented with open dots connected with a dashed line. Note the low strengths to a significant depth.

3. Planning of the pile driving tests
A total of 24 piles were planned to be installed. The length of the piles varied across the site, generally the piles being longer to the north. A total of 5 piles with a total length of 20 m, 3 piles of 22 m, 9 piles of 26 m and 7 piles of 30 m below the terrain were installed. All piles were smeared with bitumen to minimize negative skin friction in the soft layers. For the 20 and 23-metre piles, the upper approx. 11.5 m was smeared, for the 26 and 30-metre piles, the upper 20.5 and 22 m, respectively, was smeared with bitumen. A piling plan is presented in Figure 5.
To minimize the effects of vibrations on the railway track, it was planned to use a heavy hammer and a small drop height during pile driving. The pile driving crew was instructed to push down the pile as far as possible, and the harder fill layers in the top of the profiles were predrilled prior to piling. The predrilling was primarily performed in the fill layers at the top of the profiles, as illustrated in Figure 4.

3.1. Installation of vibration sensors

Two vibration sensors were installed at the soil surface and were monitored during pile driving. The monitors were installed in a steel spike installed in compacted gravel to achieve good contact with the soil surface. The location of the sensors was planned to achieve varying distances to the piles, allowing for an assessment of peak point velocity as a function of distance to pile driving. The monitors were named "north" and "south" and logs were kept for installation of all piles. Thus, peak point velocities in two horizontal directions along with vertical were logged against time for installation of all 24 test piles, totalling 48 data sets with varying distance between pile and vibration sensor. Each data set consists of the measured vibrations in three directions at one of the vibration monitors for each pile.

During piling, it became necessary to move one of the vibration sensors as the piling rig would otherwise crush the sensor when piling a pile.

Figure 5: Piling plan for the 24 test piles installed. Note the relative short distance to the railway tracks to the west (left in the figure).
3.2. Planning of the pile driving tests.
A total of 24 test piles were planned to be installed. The length of the piles varied across the site, and generally the piles were driven several metres into the firm soils below the soft layers. For most piles, it was possible to push the pile statically to a relatively large depth (5-10 m) before impact energy from the hammer became necessary.

To acquire approval from the Danish Railway authorities (Banedanmark/Rail Net Denmark), an estimation of the potential peak point velocity at the track level was performed based on the British Standard BS5228-2 [1]. At the railway track, a peak point velocity, $PPV = 5$ mm/s, was set as allowable maximum for safe train operation during piling. An empirical prediction is presented in [1] for ground borne vibrations from percussive piling, and the following expression was used in present study:

$$v_{res} \leq k_p \left( \frac{\sqrt{W}}{r^{1.3}} \right)^{1.5}$$  

(1)

In Eq. (1), $W$ denotes hammer energy, $r$ the distance to pile toe and $k_p$ a scaling factor based on soil type. To keep predictions on the conservative side, $k_p = 5$ was selected, corresponding to piles at refusal, $r$ was simplified to the distance to the pile (i.e. horizontal distance to pile axis). These assumptions were made to assess worst-case scenario – based on the description in [1], a realistic value of $k_p = 1$ should match the ground conditions. Using these worst-case parameters, the peak point velocities calculated at various intervals from the pile are presented in Figure 6. As the shortest distance between a pile and the railway was 17 m, the installation of test piles was initiated as the peak point velocity estimate was acceptable. A brief discussion on the validity of the assumptions made is presented in section 5.

In general, the piles farthest away from the railway track were installed first to allow experience from these pile drives to be utilized for piles closer to the railway tracks.

![Figure 6: Prediction of peak point velocity during pile driving, based on BS5228-2.](image)

4. Data from pile driving tests
At all time during pile driving, the loggers were kept active. During the first day of piling, the vibrations were watched on site to closely monitor potential impact on the rails and verify the predictions performed under the planning phase. Start-up and completion of piling were logged manually and all vibrations were monitored and assessed against requirements DIN4150 [2]. These
requirements were defined as threshold values during the phase of obtaining Rail Net Denmark’s final approval for test driving near the railway embankment. Thus, all peaks above the $PPV = 5 \text{ mm/s}$ threshold at frequencies between 0 and 50 Hz, i.e. 'line 2' after DIN4150, were flagged. According to DIN4150, 'line 2' is the recommended threshold for common structures and private dwellings. An example of data logged during pile driving is illustrated in Figure 7. As may be seen from the figure, test pile 14 was pushed several metres through the soft soil, which is marked by a local minimum in $PPV$ measurements from 5 to 6 min.

![Figure 7: Data logged during installation of test pile 14. Maximal velocities are plotted against time during driving. Driving is initiated at 0 min.](image)

As a general trend, the measured $PPV$ was largest for piles closest to the vibration loggers – which was expected. Thus, the piles violating the acceptance criterion at the vibration logger positions (and not at the railway tracks) were the piles closest to the loggers. The transgressions for logger positions "north" and "south" are presented in Figure 8. As illustrated in the figure, most of the transgressions for the northern logger point were in the frequency range 5-15 Hz, whereas vibration transgressions at higher frequencies were recorded at the southern position. Note how the measurement points from PP11 and PP13 for the northern logger and PP5, PP6 and PP8 and PP9 caused the highest peak point velocities logged during pile driving, as these piles were closest to the loggers.
5. Discussion
During the piling, it was observed that vibration levels increased when the piles reached the dense layer of sand under the soft deposits, which corresponds to the phenomenon observed by many others. In relatively soft soils, a large amount of the impact energy is used in advancing the pile, and relatively low levels of vibrations occur. In relatively stiff soils, a larger amount of the impact energy is dissipated as elastic deformation of the soil and penetration are small, resulting in relatively higher vibrations levels.

The model presented in [1] and [3], i.e. Eq. (1) used to predict the peak point velocities before the piling, underestimated the measured peak point velocities, especially at short distances between pile and the point of interest. The empirical model is based on a number of different cases, a range for the pile length between 1 and 27 m, a horizontal distance range between 1 and 111 m, and a hammer energy range from 1.5 to 85 kJ [1]. Both the pile length and the hammer energy used at the Spritten site are at the upper limit of the range, which could explain the deviation between Eq. (1) and observations at the site. However, as discussed above, the parameters used for estimating vibrations using Eq. (1) were all chosen to be conservative (i.e. assuming piles at refusal, all pile tips 1 m in soil at refusal etc.).

Ground conditions can affect vibrations levels arising from piling. At the current test site, there is roughly a three-layered soil structure. A stiff top and bottom layer and a soft layer in-between. This combination may affect the vibration levels by ‘capturing’ and maintaining the wave energy between these two stiff layers, limiting the energy dissipation with distance as the hard layer on top of the profiles may reflect the waves originating from the deeper seated hard layer.

From the observation at the site during the live observation of the first few test piles, it was clear that penetrating the hard fill layer on top of the softer layers causes significant peak point velocities at terrain level. This is most likely caused by relatively soft support of the upper layer, causing large vibrations in this layer at the pile tip, which are then transferred through the harder layers to a relatively large area. Based on assessments on site, it was agreed to pre-augur the test piles to limit vibrations during penetration of the upper stiff layer. Pre-augering this layer significantly diminished the vibrations associated with penetrating the fill layers and was thus chosen as the preferred method of piling through the fill layers, meaning that all piles were preaugered prior to driving.

The combination of the specific ground conditions at the site and the limited parameter range for the used model is considered to affect the first predictions and most likely explains the observed difference. Thus, based on the current case and in line with the [BS 5228], the authors recommend a site-specific calibration to be performed in cases where a more precise prediction is needed.

To identify the shortest distance allowable for pile driving without violating the acceptance criterion, all data sampled during installation of all piles were reviewed and maximal peak point
velocities were plotted against the distance between pile and vibration logger. Maximum $PPV$ recorded during pile driving are plotted against the distance between pile and vibration logger in Figure 9. As may be noted from the figure, a general trend of decreasing logged $PPV$ with increasing distance from pile driving is evident. As a general tendency, piles installed where the strong fill layers in top of the soil profiles were not preaugered also yielded larger $PPV$ values compared to piles installed through preaugered top soil. Also evident from the data is that the model presented in [1] for the current ground conditions underestimates the $PPV$ caused by pile driving.

To set up a model for the actual ground conditions and piling equipment, a power fit was employed to all measured maximal $PPV$ values plotted against distance. With a coefficient of determination of 0.70, the model for estimating $PPV$ from pile driving was found:

$$PPV = 49.797x^{-0.88}$$

In Eq. (2), $x$ denotes the distance between pile driving and the point of interest for evaluating potential peak point velocity. The model presented in Eq. (2) was successfully employed at the specific site, where several hundreds of piles were installed without affecting the stability of the railway embankment. Other works, e.g. [4], suggest a power-function to evaluate ground borne vibrations from pile driving, indicating different functions depending on installation method. Thus, the model presented in present paper aligns nicely with literature.

6. Conclusion
To evaluate the capacity of driven piles for a large redevelopment of the area around Spritten, a former liquor factory in Aalborg, Denmark, 24 test piles were driven. As the area is bordered by a railway...
embankment to the west, the magnitude of vibrations induced by pile driving was of great interest and subject to an upper limit to allow safe continuation of train traffic during piling. Thus, all 24 piles were monitored with vibration loggers during driving.

Based on full monitoring of a total of 48 measurements of pile driving, an empirical model was derived and presented in present paper. Compared to the model presented in BS 5228 [1], the model derived in present paper yields higher peak point velocities, especially at short distances between pile and the point of interest.

The model presented in the current paper was successfully employed to evaluate allowable pile driving sites at the Spritten site, where several hundred piles were installed without affecting the railway embankment. Given that the proposed model is a power fit, it generally aligns with literature and may be expected to be successfully employed at other sites, providing proper calibration to specific site conditions.

Acknowledgements
A. Enggaard A/S is acknowledged for allowing publication of the information in the present paper.

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