The Specificities of Foundation Ground Vibrations Induced by Pile Driving Under Conditions of Infill Construction in an Existing City Block

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Abstract –**Under condition of infill construction pile driving causes an appreciable impact on adjacent buildings, which should be accounted during the works. The work is the first to present the materials obtained in 1990s. The ground was represented by soft and plastic clay with the thickness from 4.5 to 13.0 meters. We considered the specificities of vibrations induced by pile driving into pre-drilled well with smaller diameter, an axially symmetric source. The frequency of vertical vibrations amounted to 18–20 Hz, while horizontal ones amounted to 13–15 Hz. The amplitude of vertical vibrations in the near zone amounted to 0.11–0.413 mm, while in the immediate vicinity, it amounted to 0.0024–0.0061 mm. Besides the solution of the direct problem of effect estimation on existing facilities, the vibration shielding also was considered. The impact with and without the shield was calculated.**

Keywords –*pile driving; soils; seismic vibrations; spectral content; absorption; site effect.*

I. INTRODUCTION

The engineering macroseismic survey of territories in the epicentral area of strong and destructive earthquakes shows that real manifestation of the intensity often differs from forecasted ones. Generally, it is explained by the neglection of a number of effects, including non-linear effects in soils under strong earthquakes, peculiarities of deep structure, relief, type and physical condition of soils, etc. [1–11].

The instrumental measurements are used to estimate the specificities of slight soil movement and forecast the effect of strong ones [12–14]. In this connection, the analysis of soil vibrations induced by pile driving is of great interest [15–18]. Under the conditions of infill construction, the process affects the nearest buildings [19–21].

Fig. 1. Modern residence building built in the experimental spot (image courtesy of S.A. Gogmachadze, 2019).

The Institute of construction mechanics of the Academy of Sciences of Georgia in 1991 has carried out field works on the analysis of vibrations induced by pile driving in the soils of building foundations in Kaloubanskaya and Trialetskaya streets in Tbilisi.

The results are still of relevance. Indeed, the number of accidents and damage events during the construction conditioned by serious miscalculations and errors is of the same amount.

II. IMPACT OF PILE DRIVING ON SOIL

The intensity of vibrations induced by artificial sources, along with other factors, depends on the distance to the impact location. In our case, the smallest distance from piles of constructed building (Fig. 1) altered from 12 to 16 m. The pile driving and soil vibration were measured on the construction site of a high-rise building (Fig. 1).

According to the recommendations [22–23], the impact of soil vibrations on buildings and the value of critical acceleration vary depending on the soil type and their physical condition.

In our case, the soil is presented by soft plastic clay, and its critical acceleration—as per the recommendations—is $a = 150$–220 cm/s^2.

The acceleration in this case is determined from a known relation:

$$a = 4\pi^2 f^2 A, \qquad (1)$$

where f is vibration frequency;

A is vibration amplitude.

The piles were driven by a S-995 kick-atomizing pipe hammer (Fig. 2) with the following parameters: total hammer mass Q = 2500 kg, hammer impact force q = 1250 kg.
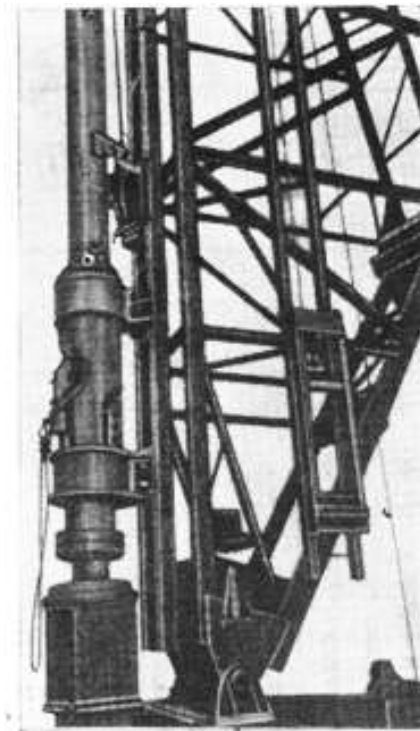

Fig. 2. Kick-atomizing pipe hammer S-995.

A. *Measurement and recording equipment used in the experiment*
B. *Geological profile*

As the residential and designed houses (Fig. 1) are located on even relief III of left-bank terrace of Kura river. The site surface is formed as embankment with the thickness of up to 1.5 m, then follow quaternary deposits represented by soft plastic clay. The thickness of clay varies from 4.5 m in the area of the residence house in Trialetskaya street to 10.5 m in the area of the residence house in Kaloubanskaya street and to 13.0 m in the intersection of the two streets. The noted soil is spread under by bedrock represented by argillites and sandstones weathered on the surface down to 2.5–3.5 meters.

To record the vibrations, conventional engineering and seismometric equipment was used including two light-beam oscilloscopes N-700, ten seismic detectors VEGIK and two seismic detectors K-001. The Seismic detectors with reflecting galvanometer GB-III-3 measure the displacement depth. The measurement profile consisted of 5 measuring sites, each of them including vertical and horizontal sesmic detectors. Site 5 was located two meters away from the hammering location; site 4 was six meters away; site 3 was 16 meters away (sites 3–5 were mounted on the ground). Site 2 was located on a step of the building foundation (a smaller seismic detector K-001 was used and fixed by gypsum to the foundation), while site 1 was located on the building roof at the side edge of the building (Fig. 3).
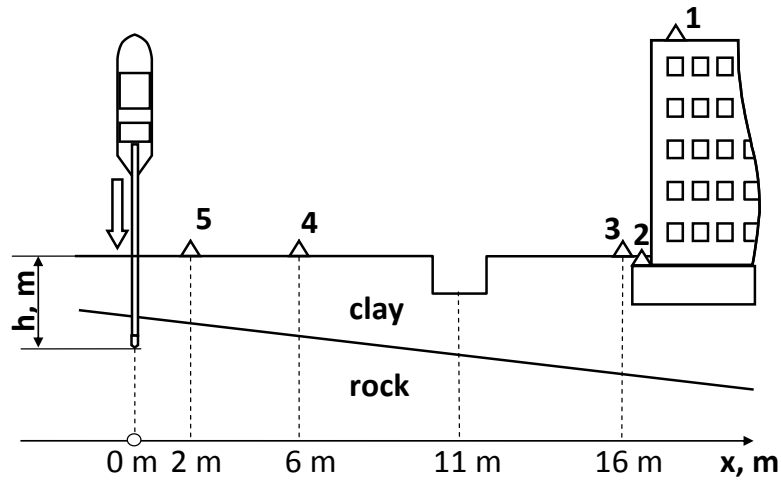
Fig. 3. Arrangement scheme of source and recording equipment

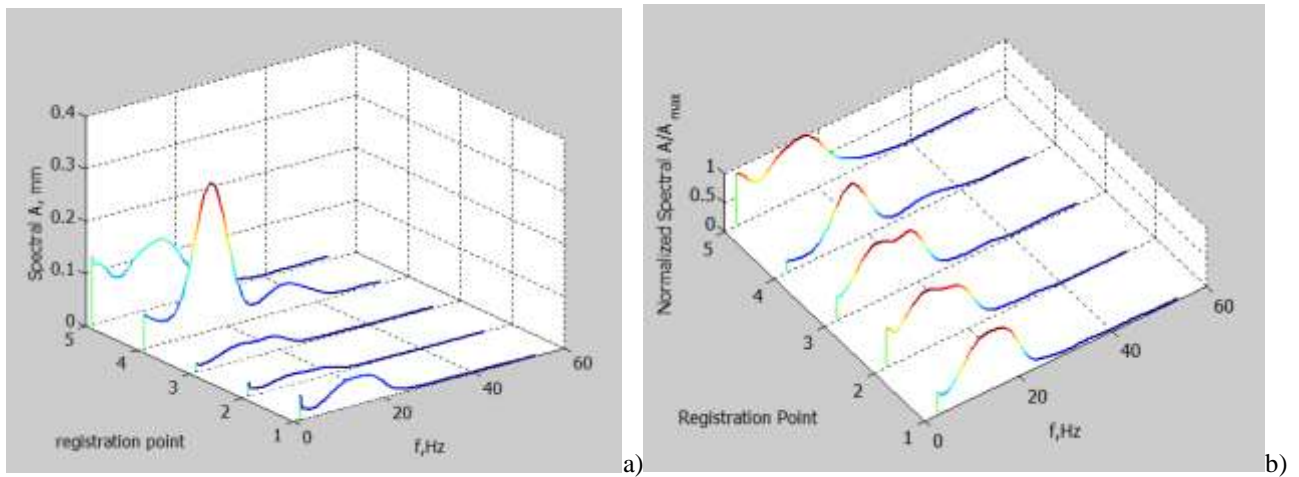Fig. 4. Digitized record, pile no. 2, depth of 4 m, vertical component

Fig. 5. Spectrum (a) and normalized spectrum (b) of digitized record, pile no. 2, depth of 4 m, vertical component

TABLE I. MAXIMUM AMPLITUDE OF VERTICAL VIBRATIONS (MM)

Measurement site No.	Depth of pile driving h [m]						
	First pile			Second pile			
	2	4	5	2	4	6	>6
1	0.0029	0.0021	0.0049	0.0029	0.0023	0.0025	0.0059
2	0.0018	0.0011	0.0018	0.0016	0.0009	0.0011	0.0027
3	0.003	0.0024	0.0027	0.003	0.0024	0.0048	0.0061
4	0.03	0.033	0.024	0.038	0.00362	0.110	0.023
5	0.145	0.156	0.11	0.18	0.18	0.413	0.263

TABLE II. MAXIMUM AMPLITUDE OF HORIZONTAL VIBRATIONS [MM]

Measurement site No.	Depth of pile driving h [m]						
	First pile			Second pile			
	2	4	5	2	4	6	>6
1	0.0056	0.0034	0.0034	0.0034	0.0046	0.0046	0.0047
2	0.0021	0.0018	0.0018	0.0036	0.0033	0.0030	0.0036
3	0.0079	0.0092	0.0066	0.0088	0.0088	0.0147	0.014
4	0.144	0.203	0.0867	0.084	0.1294	0.179	0.241

III. MEASUREMENT RESULTS

The seismic records were digitized (Fig. 4) and amplitude spectra were plotted (Fig. 5). Total measurement count amounted to 121 (e.g. Tables 1 and 2). The measurement channels were adjusted. The average frequency of vertical vibrations amounted to 18–20 Hz. The average frequency of horizontal vibrations amounted to 13–15 Hz. In the area adjacent to the pile driving locations, the maximum amplitude of vertical vibrations (site 5) amounted to 0.11–0.413 mm, 0.024–0.116 mm in measurement site 4, 0.0024–0.0061 mm in measurement site 3, 0.001–0.0027 mm in measurement site 2, i.e on the foundation, 0.0021–0.0059 mm in measurement site 1 (floor slab of the 5th floor). The maximum amplitude of horizontal vibrations in site 5 was 0.084–0.241 mm, 0.006–0.0147 mm in site 3, 0.0018–0.0036 mm in site 2, 0.0034–0.0057 mm in site 1. We should note the presence of clear non-linearity in the clay soil under strong impact (site 5) which outstands by the shape of the spectral curve (Fig. 5a).

In the area adjacent to the pile (site 5), the acceleration of vertical vibrations amounts to 530 cm/s^2, (A = 0.413 mm, f = 18 Hz), which denotes the occurrence of residual deformations in the soil [24–25]. In the next observation point (No. 4), the acceleration amounts to 113 cm/s^2, (A = 0.127 mm, f = 15 Hz). In other words, at the distance of 6 meters away from the pile driving location, there are no residual deformations. Then (No. 3), we determined the acceleration directly in the soil under the building foundation, unlike the vibration of the free soil surface. Here it amounted to 31 cm/s^2 (A = 0.0048 mm, f = 14 Hz).

For the horizontal component in site 4, the acceleration amounted to 310 cm/s^2 (A = 0.241 mm, f = 18 Hz), i.e. on the basis of acceleration of the horizontal component at the distance of 6 meters, the residual deformations are also present and two times exceed the lower limit of the critical aceleration, which is also quite high. In the next observation point No.3, the acceleration amounts to 93 cm/s^2, (A = 0.014 mm, f = 13 Hz).

Let us determine the vibration decrement using the following formula:

$$\delta = \frac{1}{m} ln \frac{a_n}{a_{n+m}} \qquad (2)$$

where m is the number of cycles;

a_n, a_{n+m} is the amplitude of corresponding oscillation cycles.

The values of the decrement varied for the vertical component from $\delta = 0.47 - 1.87$ to $\delta_{av} = 1.2$, for horizontal from $\delta = 0.31 - 1.15$ to $\delta_{av} = 0.5$. The value of δ increases with the penetration depth.

Interestingly, the vibration decrement is maximum for high amplitudes and in soils with residual deformations. For small amplitudes, the decrement is considerably lower.

IV. PROPAGATION OF SEISMIC WAVES AND THEIR ABSORPTION

When analyzing the obtained results, one should note the specificities of the observed process. First, these are vibration amplitude, their frequency and absorption by the environment of the initial signal. The absorption depends on both distance and time. The second absorption type is when the law of wave shape alteration is harmonic $A = A_0 e^{-ht} cos\omega t$ (h is attenuation factor, A_0 is vibration amplitude in the vibration origin). We have already assessed this type of absorption (vibration decrement) and $\delta = \frac{K}{f}$ (f is wave frequency). Consequently, the average signal attenuation coefficient (signal in this soil) will amount for vertical and horizontal components $K_v = 21.6$ and $K_h = 7$, correspondingly.

Taking into account that we use pulsed source, it could be represented by a spherical axially symmetric source (together with connected soil mass). However, the presence of preliminarily bored well creates so-called "long linear source" for considered distances (Fig. 6a).

It is a source of cylindrical fronts (Fig. 6b), which will give us cylindrical divergence, which changes the intensity of corresponding waves in the following way:

$$\frac{J_2}{J_1} = \frac{r_1}{r_2} \qquad (3)$$

Hence, taking into account that the intensity is the amplitude squared $I = A^2$, we derive the amplitude change in a remote point (A^2) relatively to the initial one (A_1):

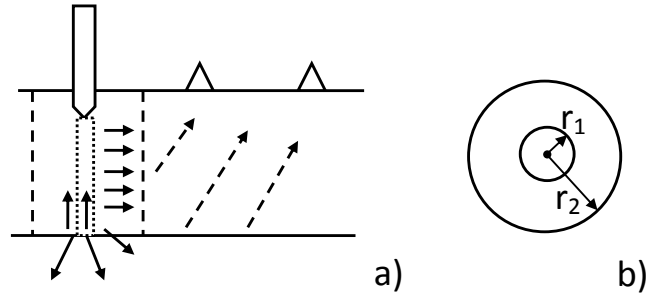$$\frac{A_2}{A_1} = const = \sqrt{\frac{r_1}{r_2}} \qquad (4)$$

Fig. 6. Scheme of sources with connected mass of soil

By substituting values $r_1 = 2$ m and $r_2 = 6$ m gives $\frac{A_2}{A_1} = 0.57$. At distances $r_1 = 2$ m and $r_2 = 6$ m: $\frac{A_2}{A_1} = 0.35$.

For site 4, the decrease of the signal connected with cylindrical divergence will amount to 0.57, and 0.35 for site 3. With distance from the source it remains the same. This also increases the contribution to the change of the initial signal due to purely absorbing properties of the soil.

Taking into account that the geological profile of the plot is represented by two layers, let us estimate the coefficients of transmission T and reflection R of seismic waves by known equations [3]:

$$T = \frac{2V_1\rho_1}{V_1\rho_1 + V_2\rho_2}, \qquad R = \frac{V_2\rho_2 - V_1\rho_1}{V_2\rho_2 + V_1\rho_1}. \qquad (5)$$

In the upper layer, the wave velocity $V_1 = 200$ m/s, density $\rho_1 = 1.8 \cdot 10^3$ kg/m³; in lower layer, $V_2 = 1800$ m/s and $\rho_2 = 2.3 \cdot 10^3$ kg/m³. Then T = 0.16 and R = 0.84.

Taking into account the above, the amplitude of waves induced in the soil will change as follows:

$$A = \sqrt{\frac{r_1}{r_2}} \cdot T \cdot A_0 e^{-\eta x} \qquad (6)$$

where A_0 is the amplitude in the initial point.

η is the absorption coefficient in the medium\

x is the distance between reception points.

The vibrations of the slab of the 5-storey building in general repeat the oscillation of foundation soil, but exceed the soil vibration amplitude 2.2. times. This also complies with reference [26].

The base near the foundation of the residence house for this source represents a far zone. The absorption coefficient can be determined as follows [2]:

$$\alpha = 2.4 \cdot f \cdot 10^{-3} \qquad (8)$$

where f is vibration frequency.

V. PROPAGATION OF SEISMIC WAVES WITH SHIELDING

At the distance of 11 meters from the source, at the depth of 3 meters in a reinforced-concrete rectangular duct, there is a metal heat pipeleing with diameter δ = 400 mm (Fig. 1) which virtually is a shield from propagating waves [27]. Thus, we have performed calculations with and without accounting the shield.

For site 4 in relation to site 5: A = 0.11, $\sqrt{\frac{r_1}{r_2}} = 0,57$, $A_0 = 0.413$ m; R = 0.84; T = 0.16 (one reflection).

For site 3 in relation to site 4: A = 0.0048, $\sqrt{\frac{r_1}{r_2}} = 0.35$, R = 0.52; T = 3·0.16 (three reflections).

a) without shield (the trapezium is the pipeline duct)
$$0.11 = 0.57 \cdot 0.84 \cdot 0.413 e^{-4\eta}$$
$$\eta_1 = 0.15$$
and
$$0.0048 = 0.35 \cdot 0.52 \cdot 0.413 e^{-14\eta}$$
$$\eta_2 = 0.2$$
here $\eta_2 > \eta_1$

b) with shield (concrete E = 200 000 kg/cm, ρ = 2.5 T/m³, μ = 0.18, T = 0.125 and V_s = 2170 m/s)
$$\eta_1 = 0.15$$
and
$$0.0048 = 0.35 \cdot 0.27 \cdot 0.413 e^{-14\eta}$$
$$\eta_2 = 0.15$$
[T = 4×0.16 + 2×0.125 = 0.73; R = 0.27]

$\eta_1 = \eta_2$ i.e. absorption is similar in virtually similar soil.

In other words, with accounting of reflections from the duct walls $\eta_1 = \eta_2$, which testifies the adequateness of the data.

c) Then, let us calculate the amplitude that would be without the duct in measurement site No. 3.
$$A = 0.35 \cdot 0.52 \cdot 0.413 e^{-14 \cdot 0.15}$$
$A = 0.0092$ mm

and acceleration $a = 39.4384 \cdot 11^2 \cdot 0.0092 = 4.4$ cm/s². Let us determine the acceleration magnitude at the distance of 11 m:

$$4.4 = 0.35 \cdot 0.52 \cdot A_0' e^{-5 \cdot 0.15}$$

$A_0' = 51$ cm/s^2

This, the acceleration of vertical oscillations is almost 3 times lower than the critical one.

Vertical component H = 2 m.

a) without shield $0.03 = 0.57 \cdot 0.84 \cdot 0.145 e^{-4\eta}$

$$\eta = 0.21$$

$$0,003 = 0.35 \cdot 0.52 \cdot 0.145 e^{-14\eta}$$

$$\eta = 0.155 \approx 0.16$$

b) with shield

$$0.003 = 0.35 \cdot 0.27 \cdot 0.145 e^{-14\eta}$$

$$\eta = 0.11$$

Hence, with H = 2, the obstacle will not impede the wave propagation and the duct has no effect.

Horizontal component H = 2 m.

a) without shield $0.0147 = 0.35 \cdot 1 \cdot 0.19 e^{-14\eta}$

$\alpha = 0.104$ m^{-1}

b) with shield $0.0147 = 0.35 \cdot 0.715 \cdot 0.179 e^{-14\eta}$
with the obstacle

$\alpha = 0.08$ m^{-1}

Let us estimate A at the distance of 11 m from the source:

$$0.0147 = 0.35 \cdot 0.715 \cdot A_0' e^{-5 \cdot 0.008}$$

$A_0' = 0.088$ mm

$A_0' = 78$ cm/s$^2 \approx 80$ cm/s^2

Horizontal component H > 6

a) without shield $0.0102 = 0.35 \cdot 1 \cdot 0.226 e^{-14\eta}$

$$\eta = 0.146 \approx 0.15$$

b) with shield $0.0102 = 0.35 \cdot 0.715 \cdot 0.226 e^{-14\eta}$

$$\eta = 0.122 \approx 0.12$$

c) worst-case scenario. $\eta = 0.12$

$$A = 0.35 \cdot 0.715 \cdot 0.226 e^{-5 \cdot 0.12}$$

$$A = 0.031$$

$$a = 28 \, cm/s^2 = 30 \, cm/s^2$$

Thus, the study results have demonstrated that during pile driving, the induced vibrations in the foundation soil and in the building itself do not reach values posing danger for the integrity of buildings most closely located to the pile driving location and are quite safe. Indeed, the construction of the high-rise building was consequently realized under total absence of any deformations in existing buildings.

VI. CONCLUSIONS

The average frequency of vertical vibrations amounted to 18–20 Hz, while that of horizontal ones amounted to 13–15 Hz.

The amplitudes of horizontal vibrations, as a rule, exceed those of vertical vibrations 1.5–2 times. The only exclusion is the site located at distance R = 6 m from the source. There the exceeding reaches 5 times. This, evidently, is explained by high-power non-linear deformations in horizontal direction at introduction of the pile into the bored well and extension of its edges as a side thrust.

The acceleration of vertical vibrations amounted at the distance of 2 m from the source A = 530 cm/s2 and 113 cm/s^2 at the distance of 6 m from the source. Thus, at the distance of 6 meters, the amplitude of soil vibration acceleration is less that critical (150 cm/s^2) and leaves no residual deformation.

The analysis of data has shown that during the pile driving for high-rise building foundation, the induced vibrations in the foundation soil and in the closest 5-storey building are rather safe. Indeed, the building of the high-rise building was realized with total absence of any deformations in the existing buildings surrounding the tested construction site.

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