Non-destructive determining of foundation pile length variability for reliability analysis

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Abstract. Contemporary design methods of foundation piles are more and more focused on sophisticated methods based on reliability analysis. Such methods, on different level of complicity, demand precise description of all data under study, not only by means of their average or characteristic values but, on higher levels of reliability analysis, standard deviations of basic variables, possible correlations and type of probability distribution. Most of the basic variables related to the ground were cautiously studied during last decades. The geometry of the pile itself (diameter and length) was usually assumed to be guaranteed and considered as non variable. The work tends to emphasize the role of pile length and present some examples of its non-destructive evaluation. Basic concepts of law strain integrity evaluation, elementary methods of reliability and sensibility analysis are presented and proceeded by results of excessive field testing programs. Some remarks on selection of basic random variables for pile capacity computation is given in discussion section. The results of the study confirm intuition that pile’s length variability plays a secondary role in reliability assessment of pile’s capacity.

1. Introduction – the scope of the study
The quality control of foundation piles requires information accessible only from the pile head. Both: load testing, where the ultimate capacity of the subsoil is derived from load-displacement relationship, and quality assessment, where pile’s length and integrity is examined, must be performed from the ground level above the embedded pile. In the case of high strain dynamic load testing and integrity and length control by means of low energy non-destructive methods, the proper assumption of wave velocity in pile material (concrete) is of primary importance for both: pile capacity and length. As the wave velocity changes in time (similarly to compressive strength and elastic modulus), there is a need for reliable references or recommendations for every type of piling technology, concrete type and its setting conditions throughout first 4 weeks after concreting. Proposed methodology and acquired data may form the basis for proper basic random variable selection in the case of computation of reliability of foundation piles [1,2]. Traditional quality control methods of pile or barrette (segment of diaphragm wall) are limited to random capacity evaluation by means of Static or Dynamic Load Testing. The range of performed tests is usually determined by technical conditions, ground conditions and the class (importance) of the construction. Pile testing technologies are described according to the time of pile loading in course of testing procedure. Static load test seem to be the most reliable method with long time of load exposure simulating real loading conditions. Dynamic testing, due to its relative simplicity (short time) becomes more and more popular, however it requires creditable data about pile’s geometry and wave velocity within pile material at the moment of dynamic testing. Last decades brought an immense development of hybrid testing technologies that make it possible for prolonging the time of loading by means of explosions or systems of strings.
Quality control is regarded as a separate issue. In general, it is never assumed that the contractor could produce shorter piles, however some variations are possible and impossible to detect from the level of pile head. The risk of breaking in course of neighbouring pile driving is reduced by the steel reinforcement designed for most of the displacement technologies. It must be underlined that most of the modern piling technologies bring the risk of discontinuity when the concrete supply is stopped in course of lifting of the drill. As deep foundations become a standard in contemporary construction the need for relevant control procedures becomes crucial.

The authors of studies popularizing application of reliability theory methods to geotechnical design always endeavour to get the more objective and optimum measure of safety [3]. For the sake of considerable discrepancy of the methods used to estimate the failure probability in the case of axial or lateral pile testing [1,2,4,5], the results reached are however often as incomparable as those global safety coefficients assumed on the basis of civil engineering practice [6-9]. A very simple method is given by Duncan [10] by examples of verifying the angular retaining wall and slope stability for displacement. Which is characteristic, the discussion about this study [10] was three times larger than the study itself. Endeavours are also made to calculate the failure probability using other methods based on Monte Carlo simulation or stochastic finite element method [11]. However, due to their limited range, the odds in favour of more common applying are very low.

2. Materials and Methods

2.1. Data acquired from integrity testing - review of publications and former author’s experience

Measurements of acoustic wave velocity in concrete are generally related to the search for correlation between this velocity and the strength of concrete in concrete (or reinforced concrete) components, or to the identification of defects in these elements. These examinations aim at evaluating the quality of the concrete itself, whereas wave velocity is calculated on the basis of the time the wave needs to cover the segment of known length. The tests made on concrete samples in successive days elapsed from concrete work confirm that the relation of velocity versus time, until concrete attains its full strength [12], is of logarithmic type and it stabilizes after about 4 weeks at about 4,000 m/s.

Final velocity depends on various factors such as: concrete grade, additives used, maturing conditions, etc. An extensive review of non-destructive diagnostic methods for concrete elements can be found in the publications [13,14]. Regarding foundation piles, estimation of acoustic wave velocity is attributable to their non-destructive diagnostics, both in the sense of pile capacity (high strain testing) and their quality (length and integrity) in low strain testing. As far as the dynamic (high strain) testing of capacity is concerned, it is justified, for analysing the strain wave, to take the velocity determined for specific concrete strength class after the completion of concrete setting (testing is usually performed after 4 weeks from pile completion) [15-22]. However, for continuity testing with a low strain method, pile material is not required to attain its full strength. Integrity testing is admitted as early as “7 days after casting or after concrete strength achieves at least 75% of its design strength, whichever occurs earlier” as recommended by ASTM D 5882 – 07 and equipment/software providers.

In practice, this period is additionally shortened by the will to progress quickly at the construction site. Functional relations reported which describe changes in acoustic wave velocity in concrete are usually prepared for testing calibration for a single construction site. Hence, they are distinguished by small diversification of random sample, resulting from small number of piles under testing, short testing time and – what could be the most important – pile homogeneity as far as their length, diameter and grade of concrete are concerned. The proposed velocity-versus-time relationships for bored foundation piles for time t elapsed from pile completion were published by numerous authors and may be juxtaposed in Table 1. All publications confirm that velocity rises in time and stabilizes after some 4 weeks. It is necessary to stress that the testing results differ significantly due to various piling technologies, concrete recipes and, in general, different time of observations. The results of author’s experience [20], collected from the sample of about 140 bored piles made of C30/C37 concrete, as shown in Figure 1, are the closest to the findings of Thasnanipan [17] and are as follows:

\[ c = 281 \cdot \ln(t) + 2891 \] for \( 12 < t \leq 33 \) days
Table 1. Variability coefficient for soil properties acc. to [11].

| Autor(s) (year)          | Formulae                                      | Additional information |
|-------------------------|------------------------------------------------|------------------------|
| Amir (1988) [15]        | $c = 3946 \cdot \log(t + 1) \cdot f_c / 30$ | $f_c$ - concrete strength in MPa |
| Finno, Gassman (1998) [16] | $c = 217 \cdot \ln(t) + 3339$                    | for $1 < t \leq 14$ [days] |
|                         | $c = 12 \cdot \ln(t) + 3887$                    | for $t > 14$ [days]     |
| Thasnanipan et al. (2000) [17] | $c = 204 \cdot \ln(t) + 3235$                | for $5 < t \leq 62$ [days] |
| Niederleithinger, Taffe (2006) [18] | $c = 182 \cdot \ln(t) + 3497$                | for $3 < t \leq 25$ [days] |
| Rybak, Schabowicz (2012) [19] | $c = 380 \cdot \ln(t) + 2607$                | for $5 < t \leq 12$ [days] |
| Rybak (2014) [20]       | $c = 281 \cdot \ln(t) + 2891$                | for $12 < t \leq 33$ [days] |

These results confirm both the rising trend in time domain and relatively large variability of velocity for particular piles. Attention is attracted to large scatter of the results obtained. It is all the more striking that the measurements were taken from merely three construction sites and the piles were made with the same piling machine. It indicates that mistakes could be made in evaluating the length of piles, and – what is worse – in evaluating the quality of placed concrete. Independent testing for 131 concrete columns made in CFA process with C20/25 concrete is provided in the paper [19]. The functional relationship was then as follows: $c = 380 \cdot \ln(t) + 2607 \text{ m/s}$ for $5 < t \leq 12$ days.

Average values of acoustic wave velocity in the CFA piles determined by the author in the second week from concreting were significantly lower than the values found in literature and the author’s own testing (for bored piles) summarized in figure 2. It could result from such factors as:

- a unique composition of concrete mixture (sand concrete in the CFA piles),
- lower strength class of concrete (C20/25) than that in the remaining research works,
- testing was run in winter season, which could affect the quality of concrete mixture and the conditions of its curing within the pile head zone.

Significant discrepancies of formulae describing the rise of acoustic wave velocity taken versus concrete ageing time justified more complex statistical analysis aimed at finding, for the statistical sample as broad as possible, both the relationship describing the increment in wave velocity and the variability coefficient for this feature. This coefficient could be an indication of a possible scatter of results and in this way, it might define the accuracy of the method [21,22]. The first attempts of such estimation were given in the work of Amir (1988) [15], where standard deviation was found at the level of 160 m/s, which represented about 4% of the measured velocity value. That finding brings us to a conclusion that the combined effect of length irregularity and velocity estimation standard deviation does not exceed that value and that finding will be checked in Results section of this work.

2.2. Variability of data in geotechnical calculations

As the soil features are of random nature, application of statistical methods in evaluating the safety requires the soil medium to be treated as a spatial random field. Knowing a vast set of field test results,
it is possible to determine the measure of the set for soil properties: average values, extreme values, scatter values, correlation values, etc. The methodology of determining the measures of sets for geotechnical parameters is extensively described in bibliography [6,7,23-27]. Some difficulty occurs with their application as such terms as average value, the most probable value, eigenvalue or derived value [28] have been defined differently and are still a subject of discussion. The application of sophisticated reliability methods is limited, because the determination of the type of probability distribution for a soil feature, for generally low number of samples, is encumbered with large potential error.

In general, it can be assumed that geotechnical parameters have log-normal or normal distribution (alternatively the “cut-off” normal, to ensure always non-negative value of soil properties). The papers [1-5,23,26] provide analyses how the distribution type assumed affects the reliability calculations. Which is characteristic – the hypothesis about the type of distribution is usually much less important than the accuracy of estimating the measures of sets. The work [6] provides an example of how assuming the log-normal distribution for the safety factor leads to conclusions inconsistent with intuition: the safety coefficient (in the quotient form) equal to $F = 1.10$ (for the variability coefficient $V_F = 50\%$) gives the failure probability over 50%. However, in general, the problem refers to the cases which are not applicable in design practice.

The variability coefficients for soil properties available in bibliography were summarized by Cardoso and Fernandes [28]. Lancelotta in work [29] provides variability coefficients for non-cohesive soils of the order 5-15% (recommended is 10%) for internal friction angle and 1-10% (recommended is 3%) for bulk density. Cohesive soils feature higher variability coefficients of the order 10-55%, especially for the cohesion.

Table 2. Variability coefficient for soil properties acc. to [28].

| $X_i$          | $X_{mi}$ (average) | $V_{X_i}$ | Examinations                      |
|---------------|--------------------|-----------|-----------------------------------|
| $< 50$ kPa    | 0.26 – 0.82        | Kulhavy (1991); Cherubini (1992) |
| 50-150 kPa    | 0.19 – 0.66        |           |                                    |
| 150-300 kPa   | 0.19 – 0.53        |           |                                    |
| $> 300$ kPa   | 0.13 – 0.41        |           |                                    |
| $c_u$         | 0.12 – 0.85        |           |                                    |
| without division |                   | 0.34      | Becker (1996)                      |
| $< 30^\circ$  | 0.03 – 0.15        | Kulhavy (1991); Cherubini (1992) |
| 30 - 40$^\circ$ | 0.10 – 0.22    |           |                                    |
| $\phi'$       | 0.05 – 0.25        |           |                                    |
| without division |                   | 0.13      | Becker (1996)                      |
| $\tan\phi'$  | 0.07 – 0.15        |           |                                    |

2.3. Levels of reliability calculations

Detailed information is included in the canonical works [7, 29-31] and book [32]. Depending on the measures used, the reliability methods are divided into the following levels (rows):

- **Level I methods** – they make use of a single „characteristic“ value of each unstable parameter; load or material coefficients are the examples.

- **Level II methods** – use the two first moments of each random parameter (average and variance) in conjunction with correlation between the parameters (usually the covariance). An example of these methods is the reliability index method used in this work (study). Calculations need not require the probability distributions of random variables, which are in so far essential that due to exiguous statistical information, selection of the distribution type can result in considerable error.
• Level III methods – the measure of reliability is the probability of failure $p_f$, hence they require knowledge of joint distributions of all random vectors.
• Level IV methods – are used to determine the structure reliability in economical aspects, taking into account expenditures and profits, maintenance, overhauls, damage effects, social values; such structures can be individual facilities, among other the highway overpasses, transmission towers, nuclear power plants.

2.4. Measures of reliability
A traditional, deterministic evaluation of reliability insists in determining the safety margin $SM$ as a boundary function: $SM = R - S$, where: $R$ – strength (resistance to influences exerted on the structure), $S$ – load acting on the structure. Here, the safety condition is in the form $SM = g(r, s) = r - s$, and in general: $SM = g(Z_i)$. Another traditional approach is to estimate the safety coefficient $F=R/S$ (then, $g(Z_i) = F(Z_i)-1$). Such determination of reliability allows only for ascertaining that the structure is in safe state or it suffered a stipulated breakdown. However, it does not specify to which degree the structure is safe, nor it allows for comparing the safety degree for various structures. The value of safety margin depends on the way the model description, and even on the selection of units. A series of examples illustrating ambiguity of the above methods as used in designing retaining structures are given by Cherubini [23]. A dimensionless reliability index was introduced to get more objective degree of structure safety. A series of methods were developed to determine this index depending on input data available, such as the type of structure boundary state function and difficulties in establishing the function. The idea of safety index determination method lies in presentation of all variables affecting the structure reliability by means of expected values (the first moment) and covariance (the second moment) of introduced parameters (random variables $Z_i$). Calculations of this method are “distribution-free”, i.e. they require no hypothesis about the type of probability distributions. According to Cornell, the reliability index takes the form:

$$
\beta_c = \frac{E[R] - E[S]}{\sqrt{Var[R] + Var[S]}} = \frac{\mu_R - \mu_S}{\sqrt{\sigma^2_R + \sigma^2_S}}
$$

(1)

The index of this form can be introduced when $R$ and $S$ are independent, $M = R - S$ and when the boundary surface is a plane, so it has linear form

$$
g(Z_i) = a_0 + \sum_{i=1}^{n} a_i \cdot Z_i
$$

(2)

2.5. Sensitivity coefficients
Numerous essential information can be gained from the knowledge of reliability index sensitivity to variability of a given random parameter. It allows, for instance, for proper defining fundamental random variables to be taken to calculations (elimination of variables, which randomness is negligible for safety, from being included in calculations) and makes it possible for the design engineer to focus on better description of such variables. When we cannot simply define the function describing the boundary state surface, while at the same time fundamental random variables are independent (which most often is true when designing a retaining structure), the sensitivity analysis can be made using Duncan method [6] as follows:
• determine the demanded value (e.g. safety coefficient $F$ or safety margin $SM$) for average values $\bar{X}_k$ of all random variables taken into calculations,
• determine the demanded value (e.g. $F$) by substituting successive variables for $\bar{X} \pm \sigma_X$ (in total $2 \times N$ calculations when the number of variables is $N$), $\bar{X}$ is the average value of random variable, $\sigma_X$ is the standard deviation.
• specify the change of demanded value depending on changes of particular parameters, e.g. 
  \( \Delta F_k = F(\overline{X}_k + \sigma X_k, \overline{X}_i) - F(\overline{X}_k - \sigma X_k, \overline{X}_i) \)

• calculate the standard deviation of demanded value using the formula (3):
  \[
  \sigma_F = \sqrt{\sum \left( \frac{\Delta F_k}{2} \right)^2}
  \] (3)

and the coefficient of variation (4)
  \[
  V_F = \frac{\sigma_F}{F(\overline{X})}
  \] (4)

• estimating the average value of demanded value and estimating its standard deviation enables (when it is e.g. F or SM) to estimate the reliability index, from the following formulae (5), (6), respectively:
  \[
  \beta(F) = \frac{\ln \left( \frac{F(\overline{X})}{\sqrt{1 - V^2}} \right)}{\sqrt{\ln(1 + V^2)}}
  \] (5)
  \[
  \beta(SM) = \frac{SM(\overline{X})}{\sigma_{SM}}
  \] (6)

The values of reliability index received are obviously dependent on the adopted hypothesis about the distribution of F or SM, but they allow estimating the safety more objectively than when the values F or SM are considered.

It may be noticed, that the calculations carried out allow for calculation, in a simplified way, the parameter defining sensitivity of determined value (e.g. safety coefficient or safety margin) to the variability of k-fundamental variable. The sensitivity parameters are given in formula (7) and (8):
  \[
  \alpha_{Fk} = \frac{\Delta F_k}{2 \cdot \sigma_F}
  \] (7)
  \[
  \alpha_{SMk} = \frac{\Delta SM_k}{2 \cdot \sigma_{SM}}
  \] (8)

The sensitivity coefficients proposed in this way have the properties \( \sum \alpha_{Fk}^2 = 1 \) and \( \sum \alpha_{SMk}^2 = 1 \), and simultaneously it can be shown, for simple geotechnical situations like pile capacity, that they are not dependent on the way how they were determined (for F or SM). Another definition of sensitivity coefficients is given by Lancelotta [29]. If the condition of boundary state is written as \( \gamma_R \cdot E(R) - \gamma_S \cdot E(S) = 0 \) then we can introduce the sensitivity coefficients (9):
  \[
  \alpha_R = \frac{\sigma_R}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad \alpha_S = \frac{\sigma_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}
  \] (9)

and prove that the partial coefficients are (10):
  \[
  \gamma_R = 1 + \alpha_R \cdot \beta \cdot \nu_R \quad \gamma_S = 1 + \alpha_S \cdot \beta \cdot \nu_S
  \] (10)
3. Results of performed testing
Tests and results presented in subsequent subsections show the range of pile length variability derived from large number of low strain integrity testing. However one cannot distinguish the real variability of the length and the accuracy of its control method, the results may be very helpful in determining the upper bound of standard deviations (or coefficients of variability) and consequently select basic random variables for reliability computation of foundation pile capacity.

3.1. Repeated series of CFA pile testing in the second week of their aging
This paragraph presents only one research program from thousands of tests carried out so far, as part of which 12 piles were being screened in course of 8 subsequent days – on 6th to 13th day after the concreting of piles. The tests were performed on 12 CFA piles, with the diameters of 800 mm and the length of 12.5 m. Figure 3 shows the view of the pile testing site. The signals were recorded by means of the PIT device (figure 4). Owing to favorable weather conditions, it was possible to adhere the sensor (accelerometer) to the smoothed surface of the head of each pile under test. In order to generate the impulse, a 1.3-kilo hammer was used. A sample signal (before the noise reduction) was shown in figure 3. One may notice a distinct echo from the pile bottom at the depth of about 12.5 m below the pile head. Acoustic wave velocity corresponding with that pile length, in a pile tested on the 9th day after concreting, equaled 3178 m/s.

Figure 3. The view of the pile testing site a) pile heads prepared for inspection, b) Pile Integrity Tester – pile head prepared for testing

Analogical analysis was carried out for 96 signals (12 piles on the subsequent 8 days). The velocity test results, calculated on the basis of the PIT signals and the time elapsed since concreting, confirming the expected increase of velocity in time, have been juxtaposed in Table 3. Despite the visible variablity in the observations on subsequent days, there is a distinct upward trend, and the results dispersion, measured by means of the coefficient of variation presented in Table 4, falls within the range of 3-4%. Such variability is confirmed by the results of tests carried out on other construction sites [19,20]
It must be stressed that the observed result dispersion may also arise from imperfection of piling works as far as the piling depth is concerned, as well as the absence of marking the level of hacking of the pile heads. The 8 pairs of results (obtained on the basis of Table 3) of the dependence of the average elastic wave velocity on the time lapsed after the concreting of the pile has been presented in Table 4, figure 5 (along with the trend line) and figure 6 (histogram of velocities).

![Figure 5](image1.png)  
**Figure 5.** The change of acoustic wave velocity in the concrete on subsequent days

![Figure 6](image2.png)  
**Figure 6.** Histogram of the observation of velocity on selected days (intervals of 100 m/s)

Equation (2) illustrates the surprising (by its linear shape) dependence of the acoustic wave velocity on time, between 6th and 13th day after the pile concreting, interpreted out of the “cloud of results” in Table 4 (or graph in figure 5):

\[ c = 3311 + 28.43 \cdot t \quad [m/s] \quad t \quad [\text{days}] \]  

(8)

Figure 6 shows the histograms of velocity for the 6, 10 and 13-day old piles (the proportion of piles for which the wave velocity was calculated in every interval of 100 m/s within a range 3000-4000 m/s. It seems that the observed distribution of the identified velocities on subsequent days after concreting are similar to a normal distribution, and such thesis were confirmed on a larger statistic sample [20].

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**Table 3.** Acoustic wave velocity calculated on the basis of the PIT

| Pile No. | Length [m] | Subsequent days after the concreting of the piles |
|---------|------------|--------------------------------------------------|
|         |            | day 6  | day 7  | day 8  | day 9  | day 10 | day 11 | day 12 | day 13 |
| 13      | 12.52      | 3298   | 3375   | 3313   | 3368   | 3471   | 3461   | 3395   | 3545   |
| 14      | 12.52      | 3220   | 3362   | 3381   | 3362   | 3309   | 3427   | 3316   | 3588   |
| 15      | 12.52      | 3296   | 3295   | 3405   | 3486   | 3471   | 3401   | 3496   | 3485   |
| 16      | 12.58      | 3525   | 3504   | 3634   | 3529   | 3566   | 3572   | 3578   | 3692   |
| 17      | 12.45      | 3134   | 3138   | 3193   | 3178   | 3246   | 3285   | 3238   | 3413   |
| 18      | 12.55      | 3320   | 3249   | 3314   | 3350   | 3441   | 3432   | 3412   | 3515   |
| 19      | 12.52      | 3448   | 3478   | 3423   | 3449   | 3605   | 3653   | 3769   | 3733   |
| 20      | 12.55      | 3441   | 3260   | 3295   | 3451   | 3359   | 3481   | 3479   | 3497   | 3422   |
| 21      | 12.52      | 3448   | 3468   | 3551   | 3539   | 3564   | 3579   | 3507   | 3662   | 3696   |
| 22      | 12.38      | 3486   | 3585   | 3649   | 3607   | 3584   | 3594   | 3674   | 3633   |
| 23      | 12.52      | 3374   | 3350   | 3469   | 3558   | 3529   | 3507   | 3662   | 3696   |
| 24      | 12.52      | 3235   | 3452   | 3377   | 3359   | 3501   | 3525   | 3506   | 3373   |

**Table 4.** Statistic values on the subsequent days after the concreting

|                           | day 6  | day 7  | day 8  | day 9  | day 10 | day 11 | day 12 | day 13 |
|---------------------------|--------|--------|--------|--------|--------|--------|--------|--------|
| Average [m/s]             | 3360   | 3367   | 3393   | 3403   | 3449   | 3470   | 3506   | 3562   |
| Deviation [m/s]           | 102.44 | 120    | 130    | 127    | 127    | 146    | 117    |
| Coefficient of variability [%] | 3.05%  | 3.57%  | 3.82%  | 3.73%  | 3.67%  | 3.65%  | 4.17%  | 3.28%  |
3.2. Velocity test results for large number sample of various concrete piles

The velocity test results, calculated on the basis of the PIT signals and the time elapsed since concreting, confirming the expected increase of velocity in time, have been juxtaposed in Figure 7. Despite the visible variability in the observations on subsequent days, there is a distinct upward trend, and the results dispersion, measured by means of the coefficient of variation falls within the range of 3-4%. Such variability can also be partially confirmed by the similar results of tests carried out in laboratory conditions [9]. It must be stressed that the observed result dispersion arises, to some extent, as a joined effect of the imperfection of piling works (as far as the piling depth is concerned), as well as from the absence of marking the level of hacking of the pile heads. The functional relationship,

\[ V = 351 \ln(t) + 2682 \text{ [m/s]} \]

for \( 5 < t \leq 36 \) days obtained from a cloud of results which describe wave velocity \( c \) versus time \( t \) can be used to evaluate the pile length with the PIT method. This function can be also applied in sonic logging testing (type CHSL) or dynamic pile capacity testing.

![Figure 7. The increase of acoustic wave velocity in time.](image1)

![Figure 8. Histogram of velocity on selected days](image2)

Hence, examinations of C20/25 concrete CFA piles at construction site in Poland prove that specific composition of concrete mixture used in these piles affects significantly the evaluations of their length with PIT method over the first days after concreting. Also the conditions of concrete curing differ, in many cases, from typical laboratory environment, especially when constant temperature is considered. These factors influence the delay of concrete strength rise in the pile. What seemed however important, it has no significant effect on final value of velocity determined after 5 weeks, being in general somewhat over 4,000 m/s. Figure 9 finally provides data based on review of almost 1,000 PIT tests for CFA piles compared with earlier relationships analysed and obtained mainly for bored piles.
4. Final remarks

I short words, the above-shown, relatively small, variation of standard deviation calculated for measurements of wave velocity in concrete, made in successive days (see Figure 10), is very close to the value of 160 m/s presented by Amir [11]. When the relation which approximates the standard deviation is referred to the formerly proposed functional relationship, describing variation of acoustic wave velocity during concrete curing, a virtually constant coefficient of variation at the level of 4.4 % is attained for a joined effect of inaccuracy of piling works and accuracy of low strain pile length evaluation.

Results from more uniform data set presented in section 3.1. (Table 4) provide even smaller values of coefficient of variation. Compare to data juxtaposed in Table 2, where the variety and no uniformity of ground properties (cohesion, friction angle) reaches 10-50% or more, the pile embedment depth seem to be negligible in reliability analysis of pile capacity. Such finding is partially confirmed in work [7], where uniform distribution of pile depth was assumed. Currently, still some authors [1] consider pile length as basic random variable in reliability calculations. Such practice (based on subjective assessment (judgement) may lead do high underestimation of reliability index and consequently to overestimation of failure probability. Improper selection of basic random variables does not allow focusing on more important factors and decreases the quality of final analysis.

As the presented results were mainly gathered from large number of CFA and some bored piles, a detailed survey should be started for other piling technologies. Especially new displacement piling technologies and concrete columns should be excessively examined by means of non destructive methods. One of the reasons is their sensitivity to possible damage during construction but also because limited drivability may result in much higher coefficients of length variation. That is a planned direction of the future research.

References

[1] Wyjadłowski M, Bagińska I and Rainer J 2018 Probabilistic assessment of pile capacity based on CPTu probing including random pile foundation depth. MATEC Web Conf., 196, 01058
[2] Bauer J, Puła W and Wyjadłowski M 2018 Pile load test results as a basis for reliability calculation with parabolic response surface. Tehnički Vjesnik – Tech. Gazette, 25 (2), 558-64
[3] Wyjadłowski M, Puła W and Bauer J 2015 Reliability of diaphragm wall in serviceability limit states. Archives of Civil and Mechanical Engineering, 15 (4), 1129-37
[4] Kozubal J; Puła W and Stach M 2014 Reliability assessment of a single pile in unsaturated substrate under climate factors influence. Procedia Engineering, 91, 310-6
[5] Kozubal J; Puła W, Wyjadłowski M and Bauer J 2013 Influence of varying soil properties on evaluation of pile reliability under lateral loads. Journal of Civil Engineering and Management, 19 (2), 272-84
[6] Tamrazyan A G and Fedorova N V 2016 Reliability assessment of reinforced concrete structures, strengthening by external reinforcement with carbon fiber. Izvestiya Vysshikh Uchebnykh Zavedenii, Seriya Teknologiya Tekstil'noi Promyshlennosti, 2016-Jan. (6), 226-30
[7] Tamrazyan A G 2016 The Assessment of Reliability of Punching Reinforced Concrete Beamless Slabs under the Influence of a Concentrated Force at High Temperatures. Procedia Engineering, 153, 715-20
[8] Tamrazyan A G, Fedorova N V and Dekhterev D S 2018 Assessment of ponderability of constructional parameters of platform joint of panel buildings on reliability of connection by method of linearization. Izvestiya Vysshikh Uchebnykh Zavedenii, Seriya Teknologiya Tekstil'noi Promyshlennosti, 2018-Jan.(1), 155-161
[9] Howiacki T, Sienko R and Sykora M 2019 Reliability analysis of serviceability of long span roof using measurements and FEM model. AIP Conference Proceedings, 2116 (1), 450078
[10] Duncan J M 2002 Factors of safety and reliability in geotechnical engineering. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 126 (4)
[11] Pula W 1991 On some methods in structural reliability. Studia Geotechnica et Mechanica, 13 (1-2), 21-35
[12] Amir E I and Amir J M 2008 Statistical analysis of a large number of PEM tests on piles. Proc. of 6th Conf. on the application of stress wave theory to Piles, Lisbon, Portugal, IOS Press, 671-5

[13] Schabowicz K 2010 Empirical relations for nondestructively determined strength of concrete on different days of its maturing. NDE for Safety : 40th international conference and NDT exhibition : proceedings, November 10-12, 2010, Pilsen, Czech Republic, 255-62

[14] Hola J and Schabowicz K 2010 State-of-the-art non-destructive methods for diagnostic testing of building structures - anticipated development trends. Archives of Civil and Mechanical Engineering, 10 (3), 5-18

[15] Amir J M 1988 Wave velocity in young concrete. Proc 3rd Intl Conf on Application of Stress-wave Theory to Piles, Ottawa, 911-12

[16] Finno R J and Gassman S L 1998 Impulse response evaluation of drilled shafts. Journal of Geotechnical and Geoenvironmental Engineering, 124 (10), 965–75

[17] Thasnanipan N, Maung A W, Navaneethan T and Aye Z Z 2000 Non-destructive integrity testing on piles founded in Bangkok subsoil. Proceedings of 6th Conference on the application of stress wave theory to Piles, Sao Paulo, Brazil, Balkema, Rotterdam, 171-7

[18] Niederleithinger E and Taffe A 2006 Early stage elastic wave velocity of concrete piles. Cement & Concrete Composites, 28, 317–20

[19] Rybak J and Schabowicz K 2010 Acoustic wave velocity tests in newly constructed concrete piles, NDE for Safety : 40th international conference and NDT exhibition : proceedings, November 10-12, 2010, Pilsen, Czech Republic, 247-54

[20] Rybak J 2014 Stress wave velocity tests in early-stage of concrete piles. Concrete solutions : proceedings of Concrete Solutions, 5th International Conference on Concrete Repair, Belfast, Northern Ireland, 1-3 September 2014, 571-76

[21] Restrepo C 2000 Stress wave propagation velocity at early ages. Proceedings of 6th Conference on the application of stress wave theory to Piles, Sao Paulo, Brazil, Balkema, Rotterdam

[22] Yoo J K and Ryu D W 2008 A study of the evaluation of strength development property of concrete at early ages. 3rd ACF International conference - ACF/VCA

[23] Cherubini C 1997 Data and consideration on the variability of geotechnical properties of soils. Proceedings of the ESREL 97, Lisbon, 2, 1583-91

[24] Cherubini C 2000 Probabilistic approach to the design of anchored sheet pile walls. Computers and Geotechnics, 26, 309-30

[25] Cherubini C, Giasi C I and Rehati L 1993 The coefficients of variation of some geotechnical properties. Proc. of the conf. on Probabilistic Methods in Geotechnical Engineering. Li. Lo. Editors, Canberra, 179-83

[26] Phoon K K and Kulhawy F H 1999 Characterization of geotechnical variability. Canadian Geotechnical Journal, 36, 612-24

[27] Phoon K K and Kulhawy F H 1999 Evaluation of geotechnical property variability. Canadian Geotechnical Journal, 36, 625-39

[28] Cardoso A S and Fernandes M M 2001 Characteristic values of ground parameters and probability of failure in design according to Eurocode 7. Géotechnique, 51 (6), 519-31

[29] Lancellotta R 1995 Geotechnical engineering. Balkema, Rotterdam

[30] Meyerhof G G 1993 Development of geotechnical limit state design. International Symposium. Limit state design in geotechnical engineering. Danish Geotechnical Society, Copenhagen, 1

[31] Meyerhof G G 1982 Limit states design in geotechnical engineering. Structural Safety, 1

[32] Orr T L L and Farrell E R 1999 Geotechnical Design to Eurocode 7. Springer, London