ISSN 1644-0633
www.acta.media.pl

ACTA Scientiarum Polonorum

Acta Sci. Pol. Architectura 15 (2) 2016, 23–32

DERIVING A RELIABLE CPT CONE RESISTANCE VALUE FOR END-BEARING CAPACITY CALCULATION OF PILES

Witold Bogusz
Building Research Institute, Warsaw

Abstract. The necessity of optimising the foundation design encourages the use of methods based on in situ test results in the design of pile foundations. For this purpose, a cone penetration test (CPT) is commonly used. Design methods developed over the last few decades may differ not only in factors used for correlating the cone resistance with bearing capacity of a given type of pile foundation, but also in adoption of the representative cone resistance value for the calculations. Contrary to the design of pile foundation in relatively homogeneous ground conditions, in the case of heterogeneous stratification, the influence of the quality of test itself and the adoption of the average cone resistance for the calculation may have significant impact on the result. This is especially important issue when weaker strata, which may affect the end-bearing capacity, is present just below the pile, in its area of influence. The article presents some of the methods of averaging the measured cone resistance, how they may affect the obtained results and other factors affecting mainly end-bearing capacity of a single compressed pile.

Key words: pile foundations, CPT, average cone tip resistance, bearing capacity

INTRODUCTION

Design methods based on the 'model pile' procedure introduced by Eurocode 7 [2008] and described by Frank et al. [2004] allow for calculating ultimate compressive resistance based on individual tested profiles. Methods based on direct application of CPT results offer better estimation of pile ultimate capacity due to continuous q_c profiling. Reducing the effect of possible biased interpretation of the results due to a human factor is an additional advantage of these methods. However, the conversion of CPT results to pile bearing capacity, generally, is not straightforward.

Corresponding author: Witold Bogusz, Building Research Institute, Building Structures and Geotechnics Department, 21 Ksawerów St., 02-656 Warsaw, Poland, e-mail: w.bogusz@itb.pl

PILE DESIGN BASED ON IN-SITU TESTS RESULTS

Classic approach to the calculation of pile compressive resistance considers its capacity as a sum of shaft and base resistance separately (1):

$$R_{c;k} = R_{b;k} + R_{s;k} = A_b \cdot q_{b;k} + \sum A_s \cdot q_{s;i;k} \tag{1}$$

where: $R_{c;k}$ – total characteristic compressive resistance of the pile,
$R_{b;k}$ – characteristic base resistance of the pile,
$R_{s;k}$ – characteristic shaft resistance of the pile,
A_b – pile base area,
$q_{b;k}$ – characteristic value of base resistance per unit area,
A_s – pile shaft area,
$q_{s;i;k}$ – characteristic value of shaft resistance per unit area.

According to Lunne et al. [1997] and Tomlinson and Woodward [2008], in fine-grained (cohesive) soils, shaft resistance has greater importance for compressive pile bearing capacity. Shaft resistance can be calculated directly from CPT results (q_c or f_s – depending on the applied method) by means of correlation factors dependent on the type of soil and the pile installation method. Additionally, some of the methods impose a limiting values on maximum unit shaft and unit base resistance. According to Bond et al. [1997], it is usually explained by reduction in horizontal stresses due to arching at greater depth.

On the other hand, in coarse-grained (cohesionless) soils, usually pile base capacity predominates. Calculation of pile base resistance is subjected to higher uncertainty due to larger area of the subsoil influencing its actual value, especially in case of heterogeneous soil conditions.

INFLUENCE AREA OF THE PILE BASE

According to Eurocode 7 [2008] requirements, the influence area below and above the pile base must be considered in calculation of pile end-bearing capacity. This area can extend over the distance of few diameters above and below the pile end (Fig. 1). Bond et al. [1997] concluded that, in layered soils, the end-bearing resistance depends on the relative strength of the layers and the position of the toe in relation to the boundary between them. Different authors report the distance of transition zone from 2D (D – pile diameter) up to 15D depending mainly on soil conditions and overburden pressure. Eslami and Fellenius [1997] stated that in theory the expansion of rupture surface in a homogeneous soil does not exceed 1,5D below pile toe, however, due to the greater weight of soil resistance below pile toe and to account for existence of weaker strata, larger depth should be considered.

The problem of influence area in pile design is often addressed by imposing additional construction rules on minimum embedment of the pile in the bearing stratum. Furthermore, keeping minimum distance of the pile end above soft strata is required in order to avoid punch through failure (Fig. 1). According to previous Polish standard PN-83/B-

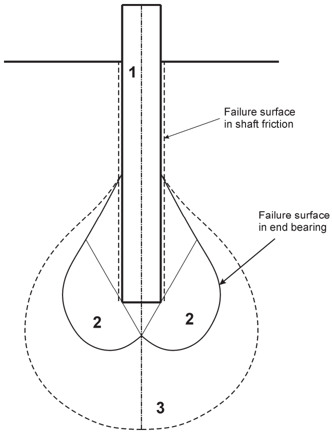

Fig. 1. Influence area of the pile base (after Tomlinson and Woodward [2008] and Wiłun [2007]): 1 – pile, 2 – plastic deformation zone, 3 – active compression zone

-02482 [1983], the minimum embedment depth of up to 2 m was required, depending on soil conditions. The same standard required at least 5D distance between the pile end and upper surface of the soft or organic soil strata. The same value was suggested by Hobbs [1992] (after Bond et al. [1997]) to avoid punch-through failure. This approach is consistent with current Eurocode 7 [2008] requirements, stating that punch through failure must be considered if soft stratum is present at depth of less than 4D beneath pile end.

AVERAGING METHODS FOR CONE RESISTANCE IN THE INFLUENCE AREA

The cone resistance value used for calculation is usually taken as average value over the zone influenced by stresses imposed by the toe of the pile [Tomlinson and Woodward 2008]. Averaging methods for cone resistance are often associated with specific methods of pile bearing capacity calculation.

Most methods assume arithmetical average over the influence area. Eslami and Fellenius [1997] proposed using geometrical average instead. However, they argued for differentiation of influence area above the pile toe in the case of pile installation through a dense stratum, to avoid giving it too much weight; however, no clear solution was proposed. Similar approach has been presented by Gwizdała [2011], who proposed three possible diagrams of relative pile end location to cone resistance values of different strata. If weaker stratum is present below pile toe, an area of 4D below is considered instead of 1D. On the other hand, if dense strata are present above, area of 2D is taken into account instead of 4D.

Bustamante and Gianeselli [1983] proposed a method of calculation ultimate bearing capacity considering the zone of 1.5D below and 1.5D above the pile end using additional filtration of the results. However, no additional requirements concerning larger influence area were given.

Schmertmann [1978] proposed a method based on the minimum path rule (Fig. 2) as a mean of deriving weighted average value. More importance is given to the area below pile tip as it is predominant in calculation of end-bearing capacity. Additionally, it takes into account the possibility of punch through failure if weaker soils stratum is present at depth of less then 4D from the pile tip. This method of deriving average cone tip resistance for calculation of bearing capacity of compressive piles has been presented in a slightly modified form in informative annex D.7 of PN-EN 1997-2 [2007], based on the method used in the Netherlands [NEN 9997-1, 2012]. Mean value of cone resistance in the influence zone is calculated according to equation:

$$q_{c;\mathrm{mean}} = 0,5 \cdot \left(\frac{q_{c;I;\mathrm{mean}} + q_{c;II;\mathrm{mean}}}{2} + q_{c;III;\mathrm{mean}} \right) \tag{2}$$

where: $q_{c;I;\mathrm{mean}}$ – is the mean of the q_c values over the depth running from the pile base level to a level which is at least 0.7 times and at most 4 times the equivalent pile base diameter *D* deeper, MPa,

$q_{c;II;\mathrm{mean}}$ – is the mean of the lowest q_c values over the depth going upwards from the critical depth to the pile base, MPa,

$q_{c;III;\mathrm{mean}}$ – is the mean value of the q_c values over a depth interval running from pile base level to a level of 8 times the pile base diameter higher.

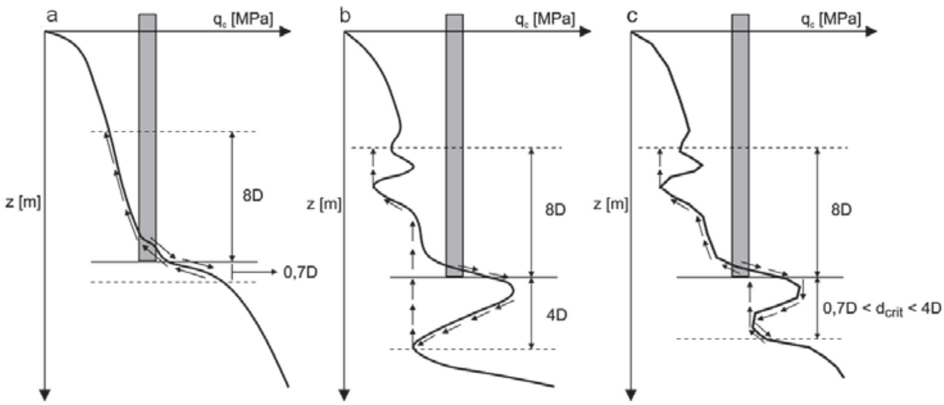

Fig. 2. Calculation of the average cone tip resistance – description in the text (adopted after Schmertmann [1978])

If q_c values increase to a depth of 4D below the pile tip, average value is determined over 0.7D below the pile (Fig. 2a). If decrease in cone resistance between 0.7–4D was measured, the lowest calculated average value over this depth shall guide the design (Fig. 2b, c) and corresponding distance from the pile toe is considered as the critical depth. Firstly, an average value in downward direction is calculated based on measured cone resistance profile to a critical depth. Secondly, determining the average value using the minimum path rule continues from the lowest q_c value used in averaging below pile toe. Following upward direction, envelope is drawn over the q_c values, either remaining constant or decreasing, up to 8D above pile toe. For each case (Fig. 2a–c), this envelope is presented with arrows following the direction of calculations of an average cone resistance value.

Meigh [1987] (after Tomlinson and Woodward [2008]) describes additional factors affecting calculated average value. First of all, q_c values higher than 30 MPa should be disregarded. Additionally, De Ruiter and Beringen [1979] (after Lunne et al. [1997]) limit the pile end-bearing capacity to the maximum of 15 MPa due to lack of evidence of higher values from the static pile load tests. Secondly, sharp peak depressions can be ignored provided that they are not clay bands in a sand deposit.

ADDITIONAL FACTORS AFFECTING THE AVERAGE CONE RESISTANCE AND PILE END-BEARING CAPACITY IN CALCULATIONS

Quality of geotechnical investigation

First and foremost, the quality of data should be considered before the design of pile foundation. It is necessary that boreholes and cone penetration tests include strata to a sufficient depth below the pile end. Usually, it is sufficient if site geotechnical investigation is performed in accordance with PN-EN 1997-1 [2008] and PN-EN 1997-2 [2007] requirements. Especially, depth of geotechnical investigation should be sufficient in order to avoid extrapolation of data previous to averaging procedure.

However, caution is required with identification of stratification based purely on CPT results or if it differs significantly from data obtained from the boreholes. Additionally, if mechanical cone is used for fine-grained (cohesive) soil, Schmertmann [1978] (after Salgado and Lee [1999]) recommended reduction of the base resistance obtained from Dutch method [De Ruiter and Beringen 1979] by 60%. In such case, geotechnical investigation report should clearly state whether presented data were corrected due to the use of mechanical cone. Nevertheless, since calculated bearing resistance using direct application of CPT results is strongly dependant on the cone resistance values, it is recommended to avoid using mechanical cone. Instead, the cone of accuracy class 1 (according to EN-ISO 22476-1 [2012]) is preferable, namely CPTU.

Pile installation technique

Unlike in the case of analytical calculation models for pile foundations, which assess pile resistance based on soil strength parameters derived from test results, the majority of methods correlating pile capacity directly to cone resistance are regarded as semi-empirical models. These methods and correlation coefficients provided with them are related to specific pile types and installation techniques. This factor may have an impact on the average value of cone resistance; especially, when negative influence of unforeseen circumstances or human error has significant probability of occurrence during execution of piles. However, it is usually addressed by adherence to proper execution standards and supervision at the construction site. The possible positive influence, namely densification of loose coarse-grained (cohesionless) soils in the case of driven piles, is usually taken into account by the calculation model.

For example, the method presented in Annex D of PN-EN 1997-2 [2007] requires the assumption of limiting value of 2 MPa for cone resistance above pile toe in the case of calculating end-bearing capacity of CFA piles, unless CPT test was performed at distance of less than 1 m from the pile after its installation. NEN 9997-1 [2012] attributes this limitation to possible distortion of sand at the pile end due to the start of lifting of the auger, which may results in cone resistance locally decreasing to 2 MPa.

Over-consolidation ratio

Some methods of calculating end-bearing capacity of the pile are imposing limit on its value in order to avoid overestimation. Moreover, it is advisable to reduce the end-bearing capacity in the case of driving the pile into over-consolidated strata and to take into account grain size of the soil (informative annex D.7 PN-EN 1997-2 [2007]). Suggested upper limiting value is presented at Figure 3. Although over-consolidation ratio (OCR) is rarely known in coarse-grained (cohessionless) soils, Tomlinson and Woodward [2008] suggested that normally consolidated soils show low penetration values at the surface increasing with depth linearly. While high values, sometimes decreasing at lower levels, can be observed at shallow depths in over-consolidated soils.

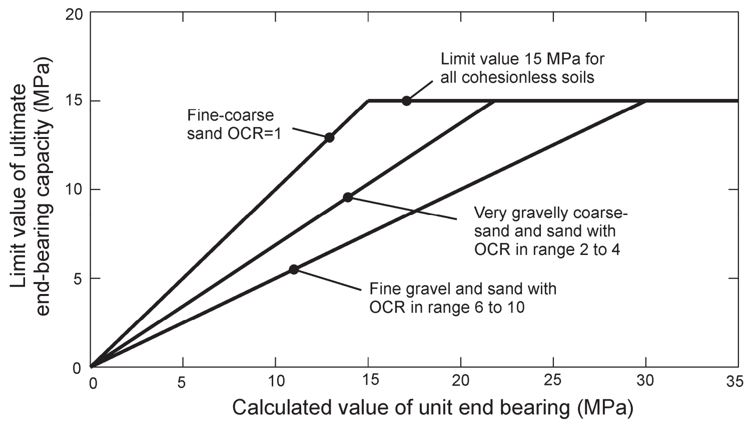

Fig. 3. Limiting values of pile end-bearing resistance (De Ruiter and Beringen [1979], after Lunne et al. [1997])

Alternatively, NEN 9997-1 [2012] recommends the reduction of cone resistance used in the calculations, due to over-consolidation, by the following relationship:

$$q_{c;z;NC} = q_{c;z;OC} \cdot \sqrt{\frac{1}{OCR}} \tag{3}$$

where: OCR – over-consolidation ratio,
$q_{c;z;NC}$ – reduced cone resistance taking into account overconsolidation, MPa,
$q_{c;z;OC}$ – measured cone resistance in overconsolidated soil, MPa.

Change in overburden pressure due to unloading

Apart from the influence of over-consolidation, the effect of change due to unloading should be taken into account. It is of utmost importance in the case of short piles and deep excavations. Such piles are usually designed based on q_c values from in-situ CPT tests, while change in the overburden pressure may result in significant reduction of cone tip resistance, especially, at a shallow depth below the bottom of the excavation.

According to NEN 9997-1 standard [2012], cone tip resistance of undisturbed soil in the case of excavation in sand or gravel, without vibration during driving of the pile, can be corrected as follows:

$$q_{c;z;\mathrm{excav}} = q_{c;z} \cdot \frac{\sigma'_{v;z;\mathrm{excav}}}{\sigma'_{v;z;0}} \tag{4}$$

where: $q_{c;z;\mathrm{excav}}$ – corrected calculated cone resistance at depth z, MPa,
$q_{c;z}$ – measured cone resistance at depth z, MPa,
$\sigma'_{v;z;\mathrm{excav}}$ – effective vertical stress at depth z, [kPa]
$\sigma'_{v;z;0}$ – initial effective vertical stress at depth z, [kPa].

This approach can be regarded as simplified and conservative. In practice, a reduction of cone resistance values and a change in soil parameters, due to unloading, can be observed; however, the influence of complete loading history, including preconsolidation, makes the issue more complex and demanding further research after gathering additional data regarding this phenomenon.

Examples of deriving an average value using different methods

Figure 4a presents an example of cone penetration test performed in order to validate results assumed in the design of pile foundation in alluvial sands. Second test showed a decrease in cone resistance just below the depth of primary investigation.

At Figure 4b a difference between average values calculated by using different methods is presented. Comparison has been made between arithmetical, geometrical, and ‘minimum path rule’ average calculated iteratively at different depths. A diameter of 60 cm was assumed in the example. If pile base is located more than 4D above the upper surface of weak soil, resulting difference in average values is relatively small. However, when pile base approach the stratum underlying the stronger strata, the average value for ‘minimum path rule’ method decreases rapidly. Even though the averaging methods are usually associated with different methods of ultimate bearing capacity calculation, the resulting cone resistance value has direct influence on the calculation results. Not taking into account the possibility of punch-through failure at the stage of deriving representative cone resistance average value can results in significant overestimation of the pile capacity, unless it is addressed directly by the requirements concerning minimum clearance between pile end and soft strata, following a specific code or based on sound engineering judgement.

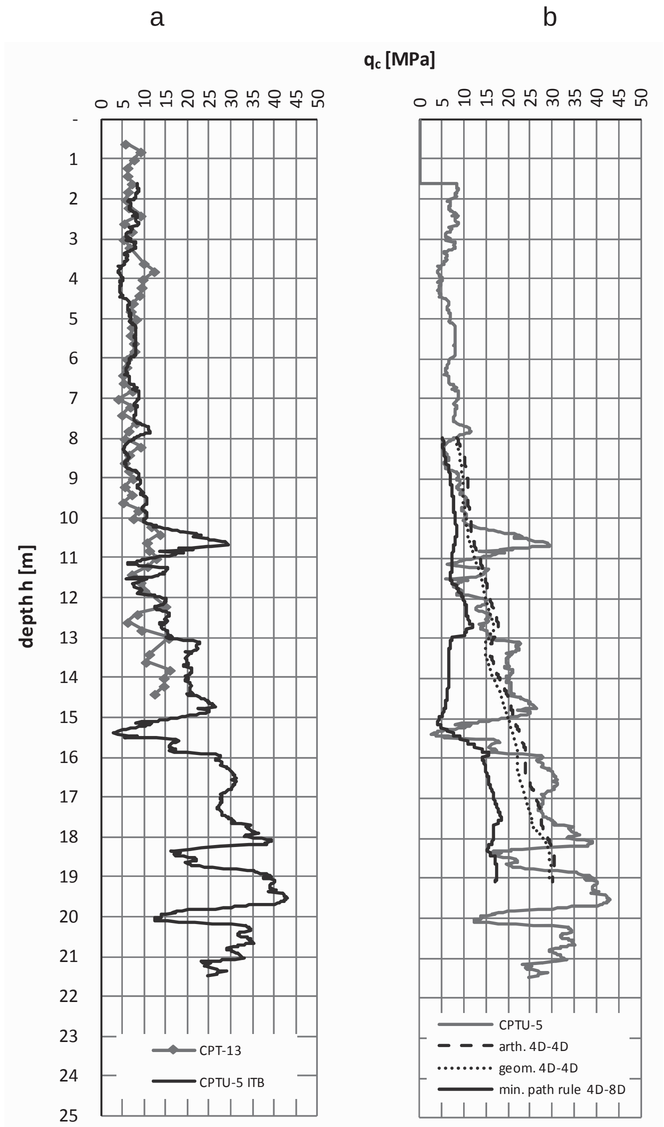

Fig. 4. Example of insufficient depth of investigation (a), example of difference between results of averaging procedures (b)

CONCLUSIONS

Deriving an average cone penetration resistance for the purpose of pile design is usually associated with specific method of bearing capacity calculation. However, a geotechnical engineer responsible for the design must be aware of all the factors influencing the final result. The influence of quality of geotechnical investigation, pile installation technique, over-consolidation and change in pre-overburden pressure should be considered.

Most averaging methods are highly dependable on the interpretation of the engineer. On the other hand, the 'minimum path rule' method is optimised for the use of numerical methods in calculations as it requires iterative procedure to asses critical depth in range of 0.7–4D below the pile tip. Methods that do not consider the possibility of punch through failure, if weaker stratum is present in influence area below pile tip, must be used with caution. Additional rules concerning minimal embedment length into bearing strata and minimum distance from upper layers of weaker strata should be considered.

Comparison between different methods of calculating ultimate bearing capacity is going to be the subject of future study by the author, thus this paper covers only the difference in the approach to deriving average cone resistance values.

REFERENCES

Bond, A.J., Hight, D.W., Jardine, R.J. (1997). Design of Piles in Sand in the UK Sector of the North Sea, HSE, Norwich.

Bustamante, M., Gianeselli, L. (1983). Calcul de la capacité portante des pieux ŕ partir des essais au pénétromčtre statique. Bull. Liaison Labo. P. Ch., 127, 73–80.

EN-ISO 22476-1:2012. Geotechnical investigation and testing. Field testing. Part 1: Electrical cone and piezocone penetration test.

Eslami, A., Fellenius, B.H. (1997). Pile capacity by direct CPT and CPTu methods applied to 102 case histories. Canadian Geotechnical Journal, 34, 886–04.

De Ruiter, J., Beringen, F.L. (1979). Pile foundations for large North Sea structures, Marine Geotechnology, 3, 3, 267–314.

Frank, R., Bauduin, C., Driscoll, R., Kavvadas, M., Krebs Ovesen, N., Orr, T., Schuppener, B. (2004). Designers' Guide to EN 1997-1. Eurocode 7: Geotechnical design – General rules, Thomas Telford Publishing, London.

Gwizdała, K. (2011). Fundamenty palowe. Technologie i obliczenia. Tom 1. Wydawnictwo Naukowe PWN, Warszawa.

Hobbs, R. (1992). A review of the design and certification of offshore piles, with reference to recent axial pile load tests. Proc. Int. Conf. on Offshore Site Investigation & Fdn Behaviour, 729–750.

Lunne, T., Robertson, P.K., Powell, J.J.M. (1997). Cone Penetration Testing in Geotechnical Practice. Blackie Academic and Professional.

Meigh, A.C. (1987). Cone penetration testing. CIRIA – Butterworth.

NEN 9997-1:2012. Geotechnical design of structures – Part 1: General rules.

PN-83/B-02482. Foundations. Bearing capacity of piles and pile foundations.

PN-EN 1997-1:2008. Eurocode 7: Geotechnical design – Part 1: General rules.

PN-EN 1997-2:2007. Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing.

Salgado, R., Lee, J. (1999). Pile Design Based on Cone Penetration Test Results. Publication FHWA/IN/JTRP-99/08. Indiana Department of Transportation and Purdue University, West Lafayette, Indiana.

Schmertmann, J.H. (1978). Guidelines for Cone Penetration Test, Performance and Design, U.S. Department of Transportation, FHWA-TS-78-209.

Tomlinson, M., Woodward, J. (2008). Pile design and construction practice. Taylor & Francis, London.

Wiłun, Z. (2007). Zarys geotechniki. WKŁ, Warszawa.

USTALANIE MIARODAJNEJ WARTOŚCI OPORU STOŻKA CPT NA POTRZEBY WYZNACZANIA NOŚNOŚCI GRANICZNEJ PALI

Streszczenie. Konieczność optymalizacji posadowienia konstrukcji wymusza w projektowaniu fundamentów palowych stosowanie metod opartych na bezpośrednich wynikach badań podłoża. W tym celu wykorzystuje się najczęściej sondowania statyczne CPT. Metody projektowania rozwijane przez ostatnie kilkadziesiąt lat różnią się nie tylko współczynnikami korelującymi opór stożka z wartościami jednostkowego oporu podstawy dla danego typu fundamentu palowego, ale także samym sposobem przyjęcia wartości oporu stożka, reprezentatywnej dla prowadzonych obliczeń. W odróżnieniu od realizacji pali we względnie jednorodnym ośrodku gruntowym, w przypadku podłoża uwarstwionego wpływ jakości przeprowadzonego badania i przyjęcia wartości średniej oporu stożka może mieć istotne znaczenie dla uzyskiwanych rezultatów. Ma to głównie znaczenie, gdy poniżej podstawy pojedynczego pala występują grunty słabsze, mogące istotnie wpływać na jego nośność. W artykule porównano sposoby uśredniania wartości oporu stożka oraz ich potencjalny wpływ na uzyskiwane rezultaty obliczonych nośności granicznych pojedynczych pali wciskanych.

Słowa kluczowe: pale fundamentowe, CPT, uśredniony opór stożka, nośność

Accepted for print: 12.06.2016

For citation: Bogusz, W. (2016). Deriving a reliable CPT cone resistance value for end-bearing capacity calculation of piles. Acta Sci. Pol. Architectura, 15 (2), 23–32.