BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iași
Volumul 63 (67), Numărul 4, 2017
Secția
CONSTRUCȚII. ARHITECTURĂ

STUDIES REGARDING THE VERTICAL SOIL PRESSURE ASSESSMENT ON RIGID PIPES BURIED IN STRAIGHT DITCH

BY

MIHAI VRABIE[1,*], SERGIU-ANDREI BĂETU[1] and ANGELICA TOMA[2]

[1]“Gheorghe Asachi” Technical University of Iasi,
Faculty of Civil Engineering and Building Services,
[2]S.C. Apa Vital S.A. Iasi, Romania

Received: October 27, 2017
Accepted for publication: November 30, 2017

Abstract. For the analysis of buried pipe networks different parameters must be considered, from which the most important are the proprieties of the pipe material and the soil characteristics around the pipe.

The main exterior forces which push on the rigid pipes are caused by the soil and by the traffic and, in this case, a relative horizontal pressure can be neglected.

The calculation expression of the acting load on a rigid pipe buried in soil is based on the Marston load theory. In this paper are presented two situations of buried pipe location, namely: in a ditch (trench) and in a tunnel (at great depths).

For each of the two locations of the pipe, the load calculation expression from vertical soil pressure is detailed, in which is inserted a load coefficient. This coefficient is customized according to different parameters and in the paper are given tables with numerical values and variation graphics which are useful in the design of the rigid buried pipes.

A case study was done, where the vertical pressure of the soil is calculated above a buried pipe in five types of soils, at different depths H, in a trench with diverse widths B_d.

Keywords: rigid buried pipe; soil vertical pressure; Marston load theory; load coefficient; parametric studies.

[*]Corresponding author: *e-mail:* mihai.vrabie@tuiasi.ro

1. Introduction

The pipes represent the main elements of an underground network, being vital for actual human communities, namely: main water supply, sewage networks, heat distribution networks, Gas networks, networks for transporting oil products etc.

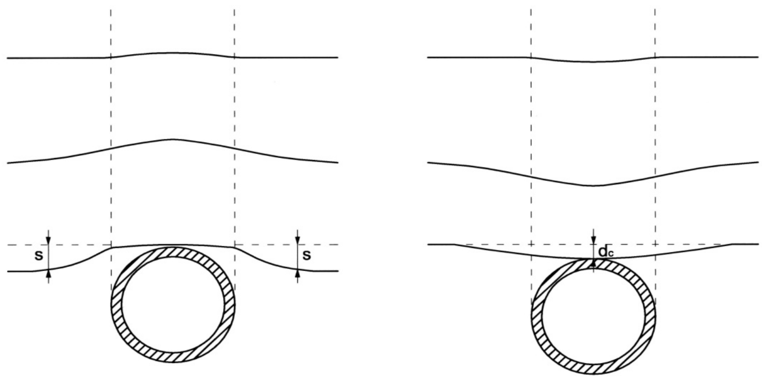
In order to fulfill its designed functions, the pipes must be sustainable over the entire design life and must pose enough strength and stiffness to resist the forces which acts over them.

In the designing of the buried pipe networks must be taken in consideration diverse parameters, of which, the pipe constitutive material and the soil characteristics have an overwhelming importance.

According to the design codes (NP 133/1-2013; EN 1993-4-3; BS EN 12201:1; EN 1998-4) the constitutive materials of the pipes are divided in two main categories: rigid (concrete, asbestos, gray cast iron, ceramics etc.) and flexible (steel, cast iron, plastics, reinforced composites etc.).

The concept of flexible pipe is associated with its ability to deform at least 2% without structural cracks. The pipes made from materials which don't fulfill the above criteria are considered to be rigid (Moser, 2001; Watkins & Anderson, 2000). Between the two main classes, some authors identified an intermediary class of semi-rigid or semi-flexible pipes, made from ductile cast iron, high density polypropylene (PEHD) or even aluminum (Moser, 2001; Carte tehnică - Valrom; Campino de Carvalho; Tohda *et.al.*, 1997).

The effect of soil compaction over the pipelines shown in Fig. 1, illustrates even better the concepts of rigid and flexible pipe.



a *b*

Fig. 1 − The effect of soil settlement on the buried pipe: *a* – rigid pipe (*s* is the settlement of backfill); *b* – flexible pipe (d_c – the vertical displacement of the pipe due to the earth pressure).

Fig. 1 shows that each type of pipe has different behavior to earth pressure and therefore, in the design process should be considered different performance criteria.

In conclusion, the engineer must know the design criteria for each different product because not all of them are designed in the same manner. As an initial part of the designing process, it is very important to evaluate correctly the loads.

2. Exterior Loads and Their Effects on Buried Pipes

The exterior loads are exerting directly on the buried pipes by the soil around them, but also indirectly, by other causes from or above the soil. The soil-structure interaction has a high importance in the designing of underground structures, because the soil around them transfers the gravitational and surface forces to the structure (Tohda *et. al*., 1997; Yoo & Kang, 2007).

In the soil-structure analysis there are considered as variables the following: the soil type, the density, the humidity and the location depth of the structure. Also, in the analysis of these structures with finite element method, many of the characteristics listed above are required as input data of the numerical model. These soil properties are usually determined by laboratory triaxial shear tests. The testing methods for the soil classifications and for the determination of different properties are standardized by international and national organizations (ISO, CEN, ASTM, AASHTO, BSI, ASRO etc.).

The soils have a diverse physical and chemical structure, but according to the Unified Soil Classification System (USCS), can be divided in five groups: gravel, sand, mud (river deposits), clay and organic soil (Moser, 2001). There are also other methods to classify the soils, of which, with a great interest for engineers are the ones related to the ability to improve the structural performance of the pipes inserted in that soil.

The loadings which act on the buried pipes depend on the stiffness proprieties of the pipe and also of the surrounding soil, this leading to a statically undetermined problem. Thus, the soil pressure on the pipe produces displacements which influence the soil pressure (Moser, 2001; Lester, 2008; Napolitano & Parlato, 2016).

For flexible pipes, the vertical loads causes a displacement of the pipe, from which results a relative horizontal pressure in the sideways soil of pipe. The rigid pipes are mainly affected by the vertical pressure caused by the soil and by the traffic and, in this case, a reactive horizontal pressure is neglected.

Also, the location of pipe in the soil can affect its behavior, namely the pipe can be situated in ditches (trenches), in embankments and in tunnels (undisturbed soil).

The structural principle according to the idea that "stiffer elements will attract greater proportions of shared loads than those that are more flexible" leads to the conclusion that "the same well-compacted soils surrounding the pipe, the more flexible pipe attracts less crown load than the rigid pipe of the same outer geometry" (Lester, 2008).

Considering the importance of the soil vertical pressure on the rigid pipes, there is a constant preoccupation for a good estimation and development of experimental techniques to measure this pressure (Liu *et al.*, 2013, Talesnick *et al.*, 2011, Rogers, 1986). In the literature, there is also a preoccupation to reduce this pressure by creating arching soil effect, using EPS Geofoam (Vaslestad *et al.*, 2011; Witthoeft & Kim, 2016) or geogrid reinforcement (Ahmed *et al.*, 2015).

3. The Soil Pressure Estimation for Buried Pipes

3.1. General Considerations

The external soil pressure is among the loading to be known, in order to design the buried pipes. Vertical soil pressure at the top of the pipe is caused by (Watkins & Anderson, 2000):

1. dead load, P_d, the weight of soil at the top of the pipe;
2. live load, P_l, the effect of surface live loads at the top of the pipe.

In the design process, the sum of the two pressures is made and results the total vertical pressure, P, at the top of the pipe (Fig. 2):

$$P = P_d + P_l \tag{1}$$

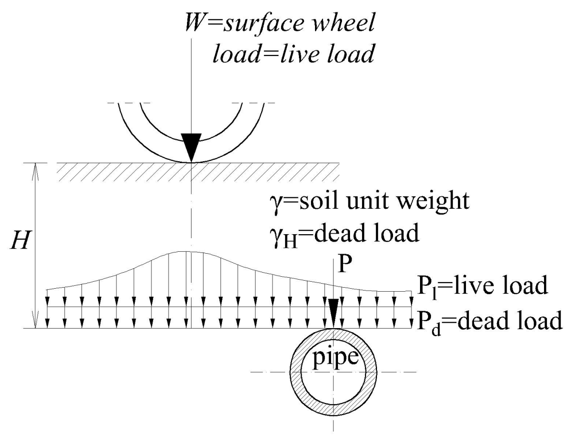


Fig. 2 – Vertical soil pressure P at the level of the top of buried pipe (adaptation from source: Watkins & Anderson, 2000)

The concept illustrated by the eq. (1) and Fig. 2 is useful just for the rigid pipes, which are design in this manner if a load factor is included.

Vertical pressure P_d is the weight of the soil, including its water content, at the level H (the height of soil cover over a pipe).

In the technical books (Watkins & Anderson, 2000; Moser, 2001) can be found explanations regarding this pressure calculation in different special cases (more different layers, the underground water table, the divers soil degree of compaction, etc.). For instance, in the paper (Watkins & Anderson, 2000) is made the specification that "if the embedment about a buried pipe is densely compacted, vertical soil pressure at the top of the pipe is reduced by arching action of the soil over the pipe, like a masonry arch, that helps to support the load. To be conservative, arching action is usually ignored".

Examining the soil pressure assessment methods and techniques on buried pipes, in the paper (Liu *et al.*, 2013) is made a classification of these methods in five categories, as it is follows:

1. Represented by Marston's theory of limit equilibrium calculation model based on limit equilibrium condition.
2. Adopt the earth pressure concentration coefficient method.
3. Theory formula based on deformation and the elasticity theoretical solution.
4. Soil column method which assumes that the earth pressure is proportional to the height of backfill.
5. The unloading arch theory which assumes that the "unloading arch" exists in the tube top fill.

Although is the oldest, the Marston theory represents the basis of the most theories proposed later, with limit equilibrium base. The subsequent revisions of Marston's theory depend on the various working conditions from the design, the building and the exploitation practice of buried pipes.

3.2. The Marston Theory of the Soil Pressure Estimation for Buried Pipes

3.2.1. *Pipes Placed in a Ditch*

The Marston load theory (Moser, 2001; Watkins & Anderson, 2000) is based on the earth prism concept, which loads the pipe placed into a narrow ditch, dug in undisrupted soil (Fig. 3).

According to this theory, the vertical pressure, V, on an embankment horizontal plane can be determined with the equation:

$$V = \frac{\gamma B_d^2}{2K\mu'}\left(1 - \mathrm{e}^{-2K\mu'(h/B_d)}\right) \quad (2)$$

where: γ is the specific weight of the embankment; B_d – the width of the ditch; μ' – internal friction coefficient between embankment and the ditch borders; h – plane height to which V pressure is calculated; e – natural logarithms base; K – Rankine ratio (between the unity of active lateral pressure and the unity of vertical pressure).

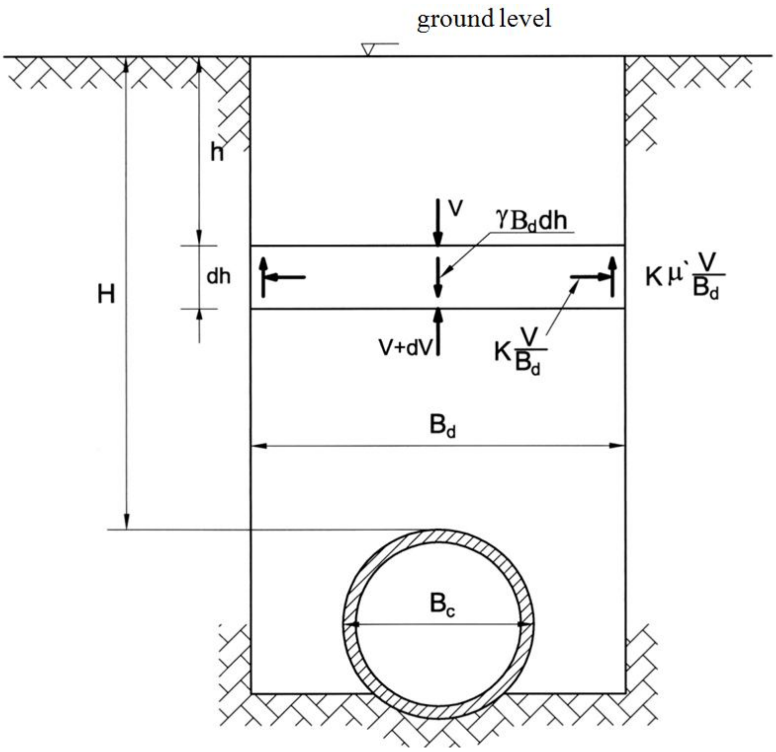


Fig. 3 – The illustration of the Marston load theory (adaptation from source: Moser, 2001).

Customizing eq. (1) for $h = H$ can be obtained the maximum loading on the trench pipe, which can be written in a compact form as follows:

$$W_d = C_d \gamma B_d^2, \tag{3}$$

where: C_d is the loading coefficient and is defined by the equation:

$$C_d = \frac{1 - \mathrm{e}^{-2K\mu'(H/B_d)}}{2K\mu'}. \tag{4}$$

The parameters γ, K and $\mu = \mu' = \tan \varphi$ (φ – internal friction angle of soil/embankment) have been experimentally determined by Martson and some typical values (Moser, 2001) are given in the Table 1.

In Fig. 4 is shown the variation graphics of the loading coefficient C_d according to the ratio H/B_d, for different types of soils, differentiated by the maximum value of the product $K\mu$ (indicative A, B, C, D, E in the Table 1).

Table 1
Approximate Values for the Parameters γ, K and $\mu = \mu'$

Soil type	γ, [kN/m^3]	K	$\mu = \mu'$	max $K\mu$	Indicative/ description
Granular materials without cohesion	–	–	–	0.1924	A – granular materials
Dry sand	15.9	0.33	0.50	0.165	B – sand and gravel
Wet sand	19.1				
Partially compacted damp topsoil	14.3				
Saturated topsoil	17.5	0.37	0.40	0.15	C – saturated topsoil
Partially compacted damp clay	15.9	0.33	0.40	0.13	D – ordinary clay
Saturated clay	19.1	0.37	0.30	0.111	E – saturated clay

For $H/B_d < 2$ ratio, the values of C_d are read on the bunch of curves from the right (using the bottom scale), and for $H/B_d > 2$ ratio, C_d are read on the bunch of curves from the left (which are in the extension with the ones from the right), using the top scale. The two arrows indicate the two parts of the same curve, C.

3.2.2. *Pipe Placed Into Undisturbed Soil (Tunnel Construction)*

In order to determine the soil loading on a jacked pipe or placed in undisturbed soil, the Martson theory leads to the following equation:

$$W_t = C_t B_t (\gamma B_t - 2c), \tag{5}$$

where the loading coefficient, C_t, is obtained in the same way as C_d (Eq. 3); B_t – is the maximum width of the tunnel or the exterior diameter of the jacked pipe; c – the soil cohesion coefficient, determined through laboratory tests on undisturbed samples. In the Table 2 are shown cohesion values, c, recommended in the paper (Moser, 2001).

Table 2
Recommended Safe Values of Cohesion c

Nr. Crt.	Material (Soil type)	Values of c, [kPa]
1.	Clay, very soft	2
2.	Clay, medium	12
3.	Clay, hard	50
4.	Sand, loose, dry	0
5.	Sand, silty	5
6.	Sand, dense	15

Because the load calculation theory on tunnel pipes (or jacketed on site in unperturbed soil) it is almost identical with the theory related to the pipes placed in ditch, the graphics from Fig. 3 can be used, in the same manner, in order to determine the C_t coefficient according to the ratio H/B_t.

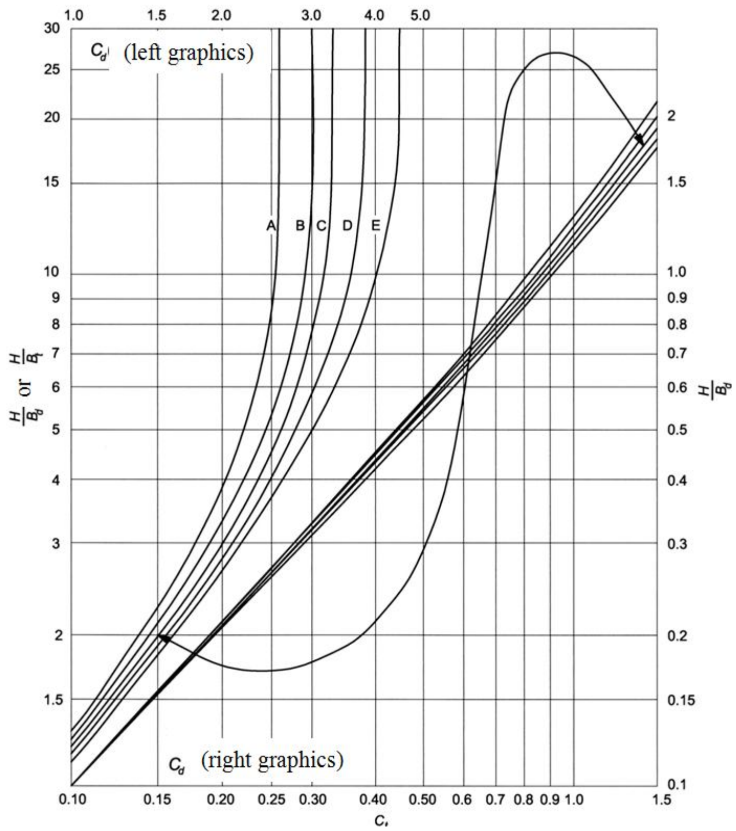


Fig. 4 – The diagram variation of the loading coefficient C_d (or C_t) (adaptation from source: Moser, 2001)

3.2.3. *Alternative Formulation*

Campino de Carvalho gave an alternative formulation to the original Marston's equation (Eq. 3). Thus, the loading on the pipe burried in trench is in relation to the earth prism weigth (γHB_d), using a unit load coefficient, α_v, which can be obtained from the following equality:

$$\alpha_v \gamma H B_d = C_d \gamma B_d^2 \quad \Rightarrow \quad \alpha_v = C_d \frac{B_d}{H}. \tag{6}$$

The equation (6) is physically easier to understand since they result from the application of factors directly to the weight of the volume of soil.

For graphical representation of the α_v coeficient depending on the ratio H/B_d (Fig. 5), it was used the complete equation:

$$\alpha_v = C_d \frac{B_d}{H} = \frac{1 - \mathrm{e}^{-2K\mu'(H/B_d)}}{2K\mu'} \cdot \frac{B_d}{H}, \tag{7}$$

where for the product $K\mu'$ were used the maximum values from the Table 1.

The values of C_d and α_v coefficients for each five type of soil, at different ratios H/B_d, are presented in Table 3.

Table 3

C_d and α_v coefficients for each type of soil

Soil type	Loading coefficients	H/B_d							
		0.25	0.5	0.75	1	5	10	15	20
A	C_d	0.2384	0.4548	0.6515	0.8301	2.2193	2.5433	2.5907	2.5976
	α_v	0.9534	0.9097	0.8686	0.8301	0.4439	0.2543	0.1727	0.1299
B	C_d	0.2400	0.4609	0.6644	0.8517	2.4483	2.9185	3.0088	3.0262
	α_v	0.9599	0.9219	0.8859	0.8517	0.4897	0.2919	0.2006	0.1513
C	C_d	0.2410	0.4648	0.6726	0.8656	2.6093	3.2033	3.3385	3.3693
	α_v	0.9639	0.9295	0.8968	0.8656	0.5219	0.3203	0.2226	0.1685
D	C_d	0.2419	0.4684	0.6804	0.8789	2.7760	3.5176	3.7157	3.7686
	α_v	0.9677	0.9368	0.9072	0.8789	0.5552	0.3518	0.2477	0.1884
E	C_d	0.2432	0.4732	0.6909	0.8968	3.0200	4.0153	4.3433	4.4514
	α_v	0.9728	0.9465	0.9212	0.8968	0.6040	0.4015	0.2896	0.2226

4. Case Study

In order to highlight the influence of the parameters H, B_d, γ and $k\mu'$, the vertical soil pressure is calculated on a buried rigid pipe. Taking into account the parameters H and B_d there are two distinct cases:

Case 1: H = 2 m (constant); B_d = (0.5; 1.0; 1.5; 2.0; 2.5; 3.0) m;

Case 2: B_d = 2 m (constant); H = (0.5; 1.0; 1.5; 2.0; 2.5; 3.0) m.

It is also considered different types of soil in which the pipe is buried, as it follows:

A. granular material without cohesion (γ = 12 kN/m^3; max $k\mu'$ = 0.1924);
B. partially compacted damp top soil (γ = 14.3 kN/m^3; max $k\mu'$ = 0.165);
C. saturated top soil (γ = 17.5 kN/m^3; max $k\mu'$ = 0.15);
D. partially compacted damp clay (γ = 15.9 kN/m^3; max $k\mu'$ = 0.13);
E. saturated clay (γ = 19.1 kN/m^3; max $k\mu'$ = 0.111).

The loading coefficients calculation C_d, α_v, the earth prism weight situated on the top of the pipe, P_d, and the vertical pressure, W_d, are shown in the Tables 4 and 5. The graphic processing of the obtained results in the two tables is depicted in Figs. 6,...,8.

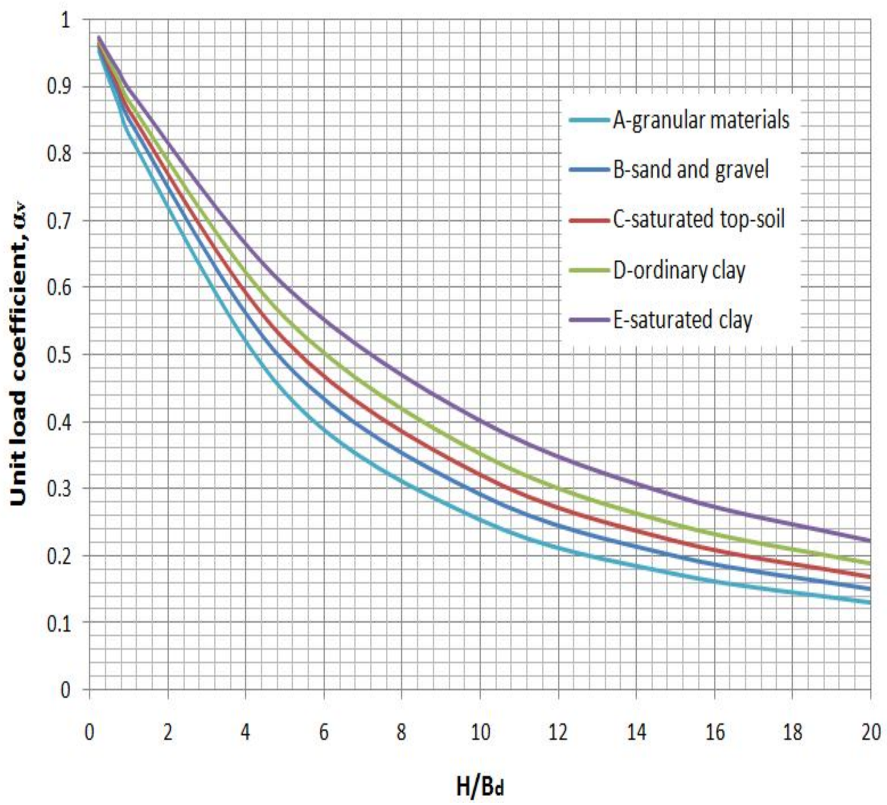


Fig. 5 – The graphic variation of the α_v coefficient.

Table 4

Calculation of the Vertical Soil Pressure for H=2m and Variable B_d

Soil type	B_d[m] Parameters	0.5	1	1.5	2	2.5	3
A	C_d	2.0412	1.3950	1.0430	0.8301	0.6886	0.5880
	α_v	0.5103	0.6975	0.7822	0.8301	0.8607	0.8820
	P_d, [kN/m]	12.0000	24.0000	36.0000	48.0000	60.0000	72.0000
	W_d, [kN/m]	6.1235	16.7402	28.1608	39.8436	51.6438	63.5064
B	C_d	2.2208	1.4641	1.0787	0.8517	0.7031	0.5984
	α_v	0.5552	0.7320	0.8090	0.8517	0.8789	0.8976
	P_d, [kN/m]	14.3000	28.6000	42.9000	57.2000	71.5000	85.8000
	W_d, [kN/m]	7.9394	20.9364	34.7064	48.7199	62.8405	77.0177
C	C_d	2.3294	1.5040	1.0989	0.8639	0.7112	0.6042
	α_v	0.5823	0.7520	0.8242	0.8639	0.8891	0.9063
	P_d, [kN/m]	17.5000	35.0000	52.5000	70.0000	87.5000	105.0000
	W_d, [kN/m]	10.1909	26.3193	43.2705	60.4757	77.7919	95.1664
D	C_d	2.4867	1.5595	1.1268	0.8806	0.7223	0.6121
	α_v	0.6217	0.7798	0.8451	0.8806	0.9029	0.9181
	P_d, [kN/m]	15.9000	31.8000	47.7000	63.6000	79.5000	95.4000
	W_d, [kN/m]	9.8847	24.7966	40.3100	56.0043	71.7766	87.5897
E	C_d	2.6510	1.6150	1.1541	0.8968	0.7330	0.6197
	α_v	0.6628	0.8075	0.8656	0.8968	0.9162	0.9295
	P_d, [kN/m]	19.1000	38.2000	57.3000	76.4000	95.5000	114.6000
	W_d, [kN/m]	12.6585	30.8469	49.5979	68.5138	87.5001	106.5229

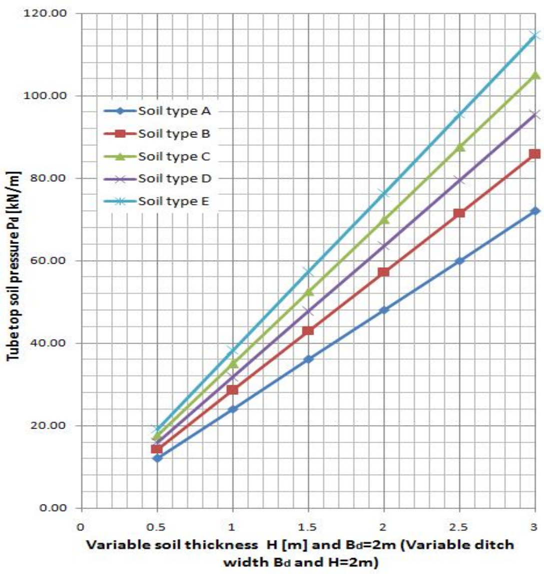


Fig. 6 – The graphic variation of the dead loading, P_d, from earth prism weight.

Table 5

Calculation of the vertical soil pressure for B_d =2m and variable H

Soil type	H[m] Parameters	0.5	1	1.5	2	2.5	3
A	C_d	0.2384	0.4548	0.6515	0.8301	0.9923	1.1396
	α_v	0.9534	0.9097	0.8686	0.8301	0.7938	0.7598
	P_d, [kN/m]	12.0000	24.0000	36.0000	48.0000	60.0000	72.0000
	W_d, [kN/m]	11.4409	21.8324	31.2709	39.8436	47.6302	54.7025
B	C_d	0.2400	0.4609	0.6644	0.8517	1.0243	1.1831
	α_v	0.9599	0.9219	0.8859	0.8517	0.8194	0.7887
	P_d, [kN/m]	14.3000	28.6000	42.9000	57.2000	71.5000	85.8000
	W_d, [kN/m]	13.7260	26.3651	38.0033	48.7199	58.5878	67.6744
C	C_d	0.2409	0.4643	0.6716	0.8639	1.0424	1.2079
	α_v	0.9634	0.9286	0.8955	0.8639	0.8339	0.8053
	P_d, [kN/m]	17.5000	35.0000	52.5000	70.0000	87.5000	105.0000
	W_d, [kN/m]	16.8599	32.5015	47.0129	60.4757	72.9658	84.5534
D	C_d	0.2420	0.4689	0.6814	0.8806	1.0672	1.2421
	α_v	0.9682	0.9377	0.9085	0.8806	0.8538	0.8281
	P_d, [kN/m]	15.9000	31.8000	47.7000	63.6000	79.5000	95.4000
	W_d, [kN/m]	15.3943	29.8197	43.3374	56.0043	67.8741	78.9969
E	C_d	0.2432	0.4732	0.6909	0.8968	1.0916	1.2758
	α_v	0.9728	0.9465	0.9212	0.8968	0.8732	0.8505
	P_d, [kN/m]	19.1000	38.2000	57.3000	76.4000	95.5000	114.6000
	W_d, [kN/m]	18.5796	36.1562	52.7839	68.5138	83.3945	97.4719

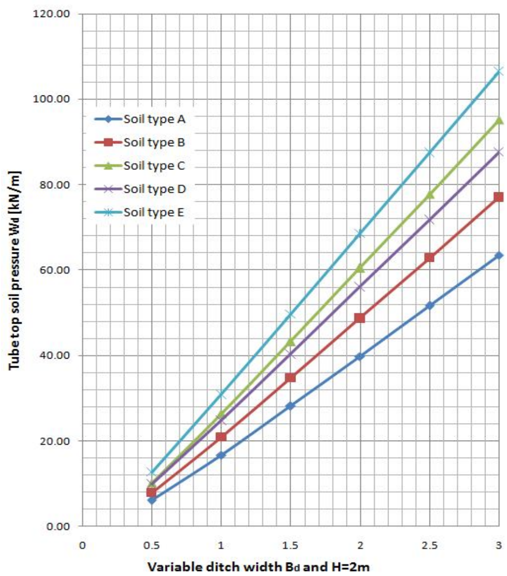


Fig. 7 – The graphic variation of the vertical pressure, W_d, for H = 2 m and B_d variable.

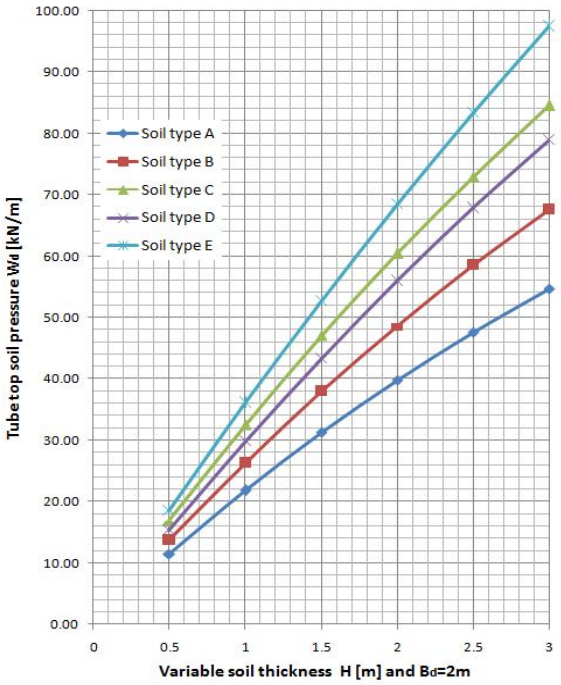


Fig. 8 – The graphic variation of the vertical pressure, W_d, for B_d = 2 m and variable H.

5. Conclusions and Observations

The analysis of loading coefficient, C_d or C_t, graphically represented in Fig. 4, permits the formulation of some useful conclusions in the design of such structures, namely:

- at subunitary values for ratio H/B_d, the loading increase (through the C_d coefficient) is almost linear and doesn't depend considerable of the type of soil (all the five curves are almost overlaid);
- the loading coefficient gets stable (remain constant), for large values of the ratio H/B_d (starting with values greater than 10 for curve A – granular materials without cohesion, till values greater than 15 for curve E – saturated clay);
- for high values of the ratio H/B_t, namely for tunnel pipes placed at great depths, the C_t coefficient gets to the limit value $1/2K\mu'$;
- the loading in the tunnel condition, W_t, will be more reduced than in ditch condition due to the soil cohesion;

- since the loading W_t cannot be negative, in eq. 5, the term 2c cannot be less than γB_t.

The subunitary value of the α_v alternative coefficient indicates that the vertical soil pressure on a rigid buried pipe in trench is always smaller than the earth prism weight situated above the pipe.

The obtained results from case study may be used in designing process of the rigid buried pipes.

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STUDII CU PRIVIRE LA EVALUAREA PRESIUNII VERTICALE A SOLULUI ASUPRA CONDUCTELOR RIGIDE ÎNGROPATE ÎN ŞANŢ DREPT

(Rezumat)

În proiectarea reţelelor de conducte îngropate trebuie luaţi în considerare diverşi parametri, dintre care o importanţă covârşitoare o au proprietăţile materialului conductei şi caracteristicile solului care o înconjoară.

La conductele rigide principala acţiune exterioară este presiunea verticală cauzată de sol şi de trafic şi, în acest caz, o presiune relativă orizontală este neglijabilă.

Relaţia de calcul a încărcării pe o conductă rigidă amplasată în sol se bazează pe teoria de încărcare a lui Marston. În lucrare se prezintă două situaţii de amplasare a conductei şi anume: în şanţ sau în tranşeu şi în tunel (la mare adâncime). Pentru fiecare din cele două situaţii de amplasare a conductei se detaliază relaţia de calcul a încărcării din presiunea verticală a solului, relaţie în care apare un coeficient al încărcării. Acesta este particularizat funcţie de diverşi parametri şi se dau tabele cu valori numerice şi grafice de variaţie, utile în proiectarea curentă a conductelor rigide îngropate.

A fost realizat un studiu de caz în care a fost calculată presiunea verticală la partea superioară a unei conducte îngropate în cinci tipuri de sol, la diverse adâncimi H, în tranşeu având diverse lăţimi B_d.