Numerical Investigations of the Stressed-Deformed State of Buried Pipelines Made of Reinforced-Concrete Pipes

Nikolai Shepelevich*, Aleksei Molchan

Republican Unitary Research Enterprise for Construction "Institute BelNIIS" ("Institute BelNIIS", RUE)
15 "B", F. Skoriny str. 220114 Minsk, Belarus

*Corresponding author: **shepelevich@belniis.by**

http://dx.doi.org/10.5755/j01.sace.9.4.7420

The method for determining the internal forces in longitudinal sections of the pipe wall based on the numerical analysis method has been proposed. The two-dimensional finite-element model simulating the annular section of the pipe and surrounding soil body are used. The external loads are presented as vertical pressures applied at the level of the pipe top +0.5 m. The calculation is performed by the iterative method with the account of the deformation parameters of the elements. The computation results conform well with the experimental data.

KEYWORDS: reinforced-concrete pipes, numerical simulation, loads, bending moment, deformations.

Introduction

One of the main problems arising when computing and designing the reinforced-concrete pipes for buried pipelines consists in determining the internal forces (bending moments) occurring in longitudinal sections of the pipe walls under the action of external loads. Here the maximum values of the bending moments depend not only on the external loads, but also on the geotechnical conditions of lying the pipes.

The valid regulatory system of the CIS countries contains only the technique provided in the Construction Norms and Regulations (SNiP) 2.05.03-85(ЦИТП Госстроя СССР, 1985), according to which the loads and internal forces in the pipes to be laid in the road bodies. The main scope of application of reinforced-concrete pipes is construction of the water-supply and sewage pipelines, where the pipes are laid in trenches and their stressed-deformed state differs considerably from that of the pipes laid in the road body.

In 1975, the project "Instruction for Determining the Loads upon the Buried Pipelines" (CH00075) based on the use of semi-empirical dependencies obtained by G.K.Klein (Клейн Г.К. 1968) was developed in the USSR. However, this instruction was neither approved nor published. The GOST 6482-88 (Издательство стандартов, 1989), where the drawings of pipes (including the reinforcement) and design conditions of their laying were provided, was valid in the USSR and

Russia till recently. This solved the problem of absence of a normative document for computing and designing the pipes, but constrained the development of their new engineering solutions and efficient manufacturing technologies.

The investigation is aimed at the development of the technique of computing the reinforced-concrete pipes laid in the soil based on the use of the computer-generated simulation.

Methods

The geometrical dimensions of the reinforced-concrete pipes (diameter/length) are those that the pipes laid in the soil may be considered as annular elements being in the planar-deformation conditions. In the SNiP 2.5.03-85, the computational pattern presented in **Fig. 1** is used for determining the bending moments in longitudinal sections of the pipe wall.

The bending moments M_1, M_2, M_3, are determined from the formulae:

$$M_{1,2,3} = \delta \cdot (p_v - p_n) \times r_m^2 \cdot b; \tag{1}$$

or

$$M_{1,2,3} = \delta \cdot p_v (1 - \lambda) \times r_m^2 \cdot b; \tag{2}$$

where:

δ is the coefficient taking into account the section position, bed type and angle of contact of the pipe in the bed;

r_m and b are the average pipe radius and design section width, respectively;

λ is the lateral soil pressure coefficient.

As seen from (1)-(2), the value of the bending moments M_1, M_2, M_3 is affected considerably by the "passive" lateral soil pressure p_n, which depends on the type of the soil and degree of its compaction. In the project CH00075, this factor was taken into consideration by applying the three degrees of compaction of the soil in the gaps between the pipes and the trench walls (in the pipeline project): the uncontrolled, increased and dense one (compaction by alluvion), to each of which a certain value of the lateral pressure coefficient for the given kind of soil λ corresponded.

Besides, the computational pattern (see **Fig.1**) does not take into account the occurrence of the additional "reactive" lateral soil pressure when increasing the horizontal diameter (warping of the annular section of the pipe) during the formation of the cracks in the "dangerous" sections of the pipe wall.

This problem can be solved using the method of computer-generated

Fig. 1

Computational pattern and distribution diagram of the bending moments in longitudinal sections of the pipe wall
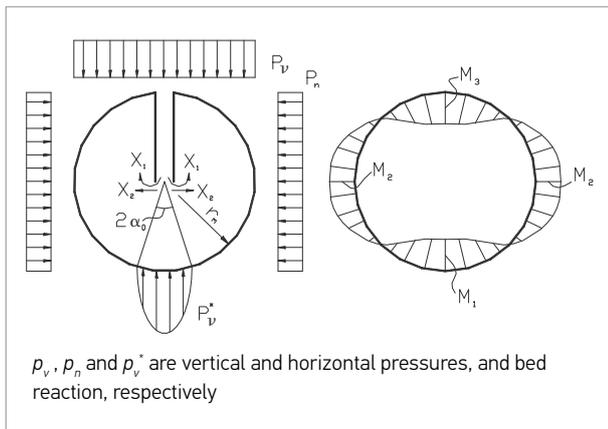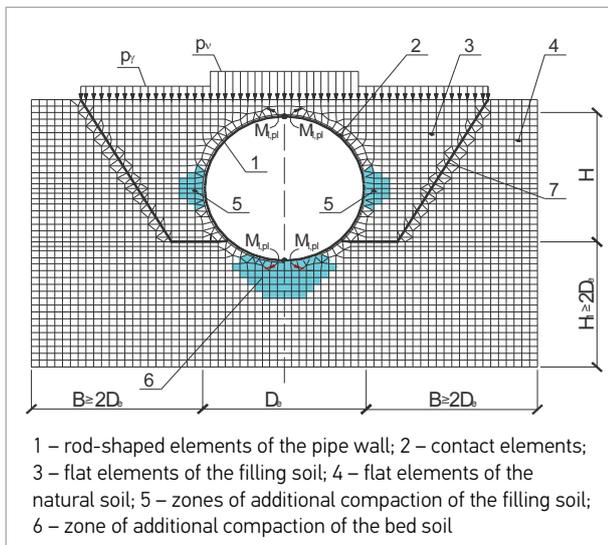

p_v, p_n and p_v^* are vertical and horizontal pressures, and bed reaction, respectively

Fig. 2

Computational finite-element model of the pipe

1 – rod-shaped elements of the pipe wall; 2 – contact elements; 3 – flat elements of the filling soil; 4 – flat elements of the natural soil; 5 – zones of additional compaction of the filling soil; 6 – zone of additional compaction of the bed soil

simulation. To solve this problem, the two-dimensional finite-element model (see **Fig. 2**) is used. This model is formed as follows:

_ the annular section of the pipe (of a single width) is formed by the rectilinear rod-shaped finite elements 1 conjugated rigidly with one another and arranged over the perimeter of the middle surface of the pipe;

_ the length of the rod-shaped elements (l_0) is assumed to be 0.03...0.05 of the length of the circumference of the medial surface and their modulus of elasticity (E) according to the deformation curve for concrete of the specified strength class;

_ the zone of contact between the pipe elements and the soil body is formed by the grid of rod-shaped contact elements 2 (with the compressive stiffness only);

_ the soil for filling the gaps between the pipes and the trench walls is formed by flat rectangular and triangular (in the contact zones) soil elements 3 having its own weight;

_ the modulus of deformation of the soil elements is adopted depending on the degree of compaction of the filling soil (coefficient K_y): 5 MPa for normal (uncontrolled) compaction degree; 15 MPa – for increased (up to $0.93 < K_y < 0.95$) compaction degree; 25 MPa – for high (up to $K_y > 0.95$) compaction degree.

_ the soil outside the trench (natural) is formed by the flat rectangular and triangular (in the zone of contact with the filling soil) soil elements 4, the modulus of deformation of which is to be assumed depending on the kind of the soil of the natural (undisturbed) structure surrounding the trench;

_ the zone of contact of the filling soil with the soil of undisturbed structure is formed by special contact elements with finite shear modulus simulating the friction of the filling soil on the natural soil;

_ the external load on the pipe (to be determined using the technique provided in the SNiP 2.05.03-85) is applied as uniform vertical pressures p_v and p_γ, applied at the level of the pipe top +0.5 m,

where p_v and p_γ are the total vertical pressure (including the variable load on the surface) and the pressure from the filling soil, respectively, computed on the basis of the existing techniques.

The boundaries of the computational domain are assumed depending on the outer diameter of the pipe D_e for the purpose of excluding the effect of the external loop on the computation results.

For solving this problem, the computational software packages, such as "LYRE", "NASTRAN", etc. taking into account the physical and geometrical non-linearity can be used (Городецкий А.С., Евзеров И.Д. 2009).

When describing the finite elements, the following is assumed:

_ modulus of deformation of the soil: E_{soil} = 50 MPa – for simulating the soil of the undisturbed structure and E_{soil} =5 MPa (15; 25 MPa) – for the filling soil;

_ Poisson ratio ν = 0.3;

_ cohesion R_c =22 kPa – for simulating the soil of the undisturbed structure and R_c =1...5 kPa – for simulating the backfilling soil;

_ soil density ρ = 17.7 kN/m^3;

_ angle of internal friction φ = 30°;

_ geometrical parameters B and H;

_ diagrams of the material deformation.

The computation is performed using the iterative method, by the step-by-step application of the external load (at least 10 stages). At each loading stage, the change in the stiffness of the rod-

shaped finite elements of the pipe wall in the cracking zones as well as change in the deformation module of the filling soil in the additional compaction zones 5 and 6.

This computation method makes it possible to determine the values of the internal forces of the bending moments in the rod-shaped elements simulating the pipe wall as well as the values of warping of the annular section of the pipe (changes in the vertical and horizontal diameter).

Results

Fig. 3

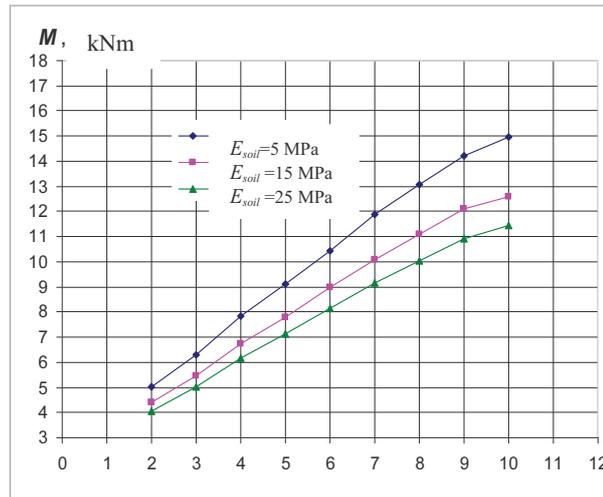M_{max} vs. H dependence for the 1000 mm diameter pipes

Fig. 4

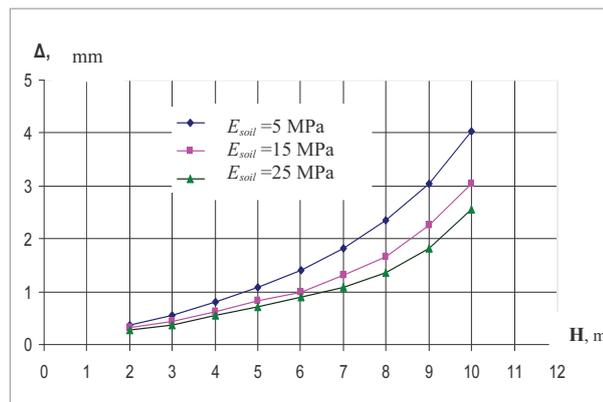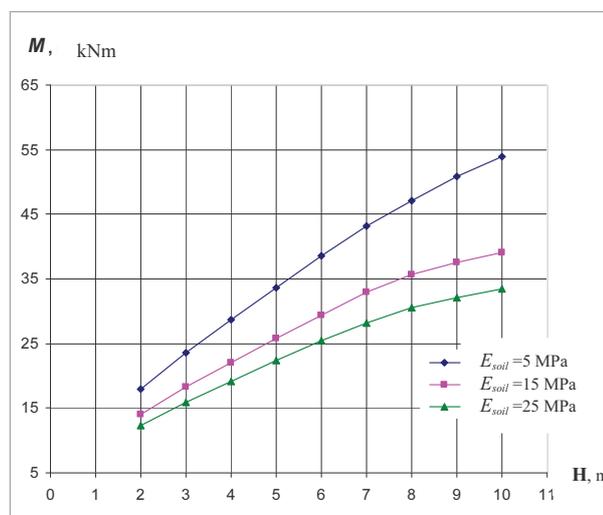Δ vs. H dependence for the 1000 mm diameter pipes

Fig. 5

M_{max} vs. H dependence for the 2000 mm diameter pipes

Figures 3-6 present the graphical dependencies of the bending moment values M_{max}, in the walls of pipes Ø1000 mm and 2000 mm and values of changes in the horizontal pipe diameter Δ on the filling height H (to be used for computing the p_v and p_y). The modulus of deformation of the soil filling the gaps between the pipes and the trench walls E_{soil}. is assumed to be 5; 15; 25 MPa (averaged values from Project CH00075), to which various extent of its compaction (uncontrolled, increased and dense) corresponds).

As seen from the graphs, the increase of the modulus of deformation of the soil filling the gaps between the pipes and the trench walls (for example, by using the sandy soil with steeping and layer-by-layer compaction) leads to the reduction of the maximum bending moments in the unsafe sections of the pipe wall by 20...35%, depending on the geometrical parameters of the pipes and depth of their burial. Here the warping of the annular section of the pipe (Δ) is reduced by 1.5...2.

This phenomenon is conditioned by the action of the lateral soil pressure occurring as a reaction to warping of the annular section (Δ). Here no lateral pressure (p_n) (see the diagram in **Fig. 1**) was applied explicitly to the pipe.

It is obvious that the value of the reactive lateral soil pressure depends on the stiffness of the annular section of the pipe. In the reinforced-concrete pipes, the stiffness of the annular section decreases considerably in the process of formation and opening of the cracks. The cracks are formed in the longitudinal sections of the pipe

wall in the zones of action of the maximum bending moments (see the distribution diagram M in **Fig. 1**). To reveal the influence of this factor on the stressed-deformed state of the pipes, their computation without accounting this phenomena was performed.

Tables 1-2 presents the comparative values of the maximum bending moment M_{max} and warping of the annular section Δ for the 1000 and 2000 mm diameter pipes with the modulus of deformation of the filling soil E_{soil}=15 MPa obtained from the linear-elastic and iterative computations.

As the load grows (the filling height H increases), the results of computation of the pipes using the linear-elastic and iterative model become considerably different so that the heavier is the load, the greater are the differences between the results.

This phenomenon is conditioned by the fact that as the cracks occur in the "unsafe" sections of the pipe wall (bottom line, soffit and at the horizontal diameter level), the warping of the annular section grows. The additional (reactive) lateral back pressure of the soil filling the gaps between the pipes and the trench walls that just causes the reduction of the bending moments M_{max}:

The most significant occurrence of this phenomenon takes place in the large-diameter pipes having lesser annular stiffness. So in the pipe with the diameter of 2000 mm at the filling height H=10 m, the computed value of M_{max} was reduced by 34% in comparison with the elastic computation.

To check the adequacy of the proposed computational model, the comparison of the computed values of warping of the annular section Δ of the pipes with the diameter of 2000 mm with the results of their measurements was performed. These pipes were used in

H, m	M_{max}, kNm/m		Δ, mm	
	Iterative	Linear-elastic	Iterative	Linear-elastic
2	4.39	4.54	0.33	0.27
3	5.46	5.74	0.44	0.34
4	6.74	7.22	0.6	0.42
5	7.81	8.52	0.78	0.5
6	8.97	9.86	1.03	0.57
7	10.1	11.2	1.33	0.65
8	11.1	12.6	1.66	0.73
9	12.1	14.2	2.27	0.83
10	12.6	15.7	3.07	0.91

Table 1
The comparative values of the maximum bending moment M_{max} and warping of the annular section Δ

H, m	M_{max}, kNm/m		Δ, mm	
	Iterative	Linear-elastic	Iterative	Linear-elastic
2	13.69	16.0	1.90	1.30
3	17.70	20.7	2.66	1.83
4	21.38	25.8	3.54	2.20
5	25.07	31.1	4.49	2.60
6	28.62	36.5	5.56	3.10
7	31.82	42.0	6.72	3.53
8	34.68	47.5	8.07	3.92
9	36.54	53.8	9.89	4.50
10	38.24	58.6	11.81	4.90

Table 2
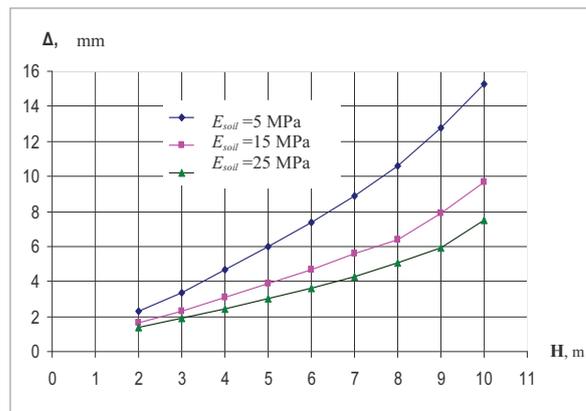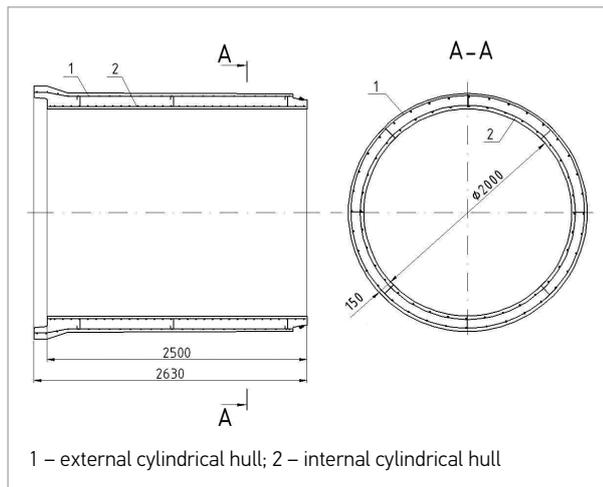The comparative values of the maximum bending moment M_{max} and warping of the annular section Δ

Fig. 6
Δ vs. H dependence for the 2000 mm diameter pipes

Fig. 7

Design solution of pipe

1 – external cylindrical hull; 2 – internal cylindrical hull
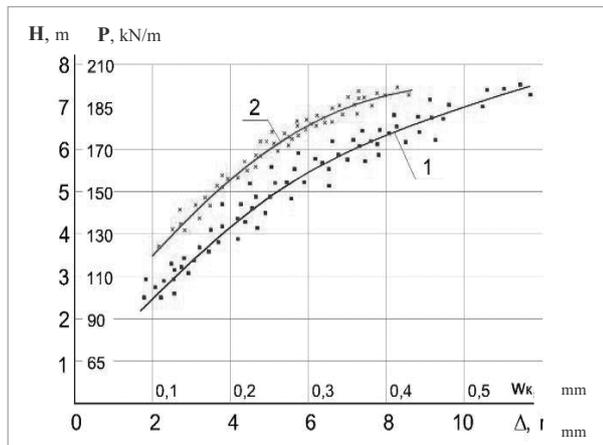
Fig. 8

Graphs: 1 – H vs.Δ; 2 – H vs wkcylindrical hull

construction of storm drains in the city of Minsk, their depth was from 2 to 7.5 m.

Basic parameters of the pipes:

_ inner diameter – 2000 mm;

_ useful length (without the hub) - 2,5 m;

_ type of pipe connection joint – hub with rubber sealing;

_ pipe wall thickness - 150 mm;

_ pipe wall is reinforced with double cylindrical hulls symmetrically placed at inner and outer faces;

_ design concrete class – C25/30.

Design solution of pipe is shown in **Fig. 7**.

Designers of these pipes (including authors of this article) were monitoring the technical condition of the pipes both immediately after filling as well as during the operation process. Degree of the pipe loading was determined by measuring changes of its horizontal diameter, and by presence of cracks in the crown and their opening width.

Fig. 8 presents the graphical dependencies (obtained by processing the results of pipe measurements after backfilling) of changes of the horizontal diameter Δ and maximum crack opening width wk at various filling heights (H) of the pipes. Ordinate axis also presents computed vertical (equivalent) linear load P according to Project CH00075 (for pipes laid in sandy soils on a natural profiled bed with angle of contact 2α=90° and with high degree of compaction of the soil filling the gaps between the pipes and the trench walls).

As seen from graph 1 (see **Fig. 8)**, the values of warping of the annular sections of the pipes Δ conform well with the data of Table 2 (in the iterative computation with the account of cracking).

Here the computed values M_{max} as per Table 2 (for iterative computation) conform also well with the respective values computed from the equivalent linear load P according to Project CH00075 for high degree of compaction (alluvion) of the soil filling the gaps between the pipes and the trench walls.

Conclusions

Thus, the technique for computing the buried pipelines made of reinforced-concrete pipes by the numerical simulation method using the finite-element model conforms well with the similar results of the computation according to Project CH 00075 and corresponds to the experimental data.

The advantage of the technique being proposed consists in the possibility of simulation of various geotechnical conditions of lying the pipes (in an embankment, trench, slit, etc.) as well as degree of compaction of the surrounding soil. In so doing, the computations take into account the change in the stiffness of the annular section of the pipe in the process of formation and opening of cracks.

The proposed technique can be recommended for computing and designing the buried pipelines made of reinforced-concrete pipes.

References

Shepelevich N.I., Molchan A.E. 2010. Experimental Investigation of Bearing Capacity of Reinforced Concrete Non-pressure 2000 and 2400 mm Diameter Pipes for Deep Burial. In Advanced Construction - Proceedings of the 2nd International Conference. Kaunas 2010. 107-112.

Городецкий А.С. Евзеров И.Д. 2009. Компьютерные модели конструкций [Computational model of structures]. Москва. Издательство АСВ, 36-39.

ГОСТ 6482-88 Трубы железобетонные безнапорные. Технические условия [Reinforced concrete non-pressure pipes. Specifications]. – М.: Издательство стандартов, 1989.

Клейн Г.К. 1968. Расчет подземных трубопроводов [Computation of the Buried Pipelines]. М., Стройиздат, 98 с.

Мосты и трубы: СНиП 2.05.03-85. – М.: ЦИТП Госстроя СССР, 1985.

About the authors

NIKOLAI SHEPELEVICH

Header of Laboratory

Scientific Research Laboratory, Republican Unitary Research Enterprise for Construction "Institute BelNIIS" ("Institute BelNIIS", RUE), Belarus

Main research area
Computing and designing the reinforced-concrete structures for buried engineering constructions

Address
15 "B", F. Skoriny str. 220114 Minsk, Belarus
Tel. (017) 267-92-26; (029) 679-89-18
E-mail: shepelevich@belniis.by

ALEKSEI MOLCHAN

Principal Engineer

Scientific Research Laboratory, Republican Unitary Research Enterprise for Construction "Institute BelNIIS" ("Institute BelNIIS", RUE), Belarus

Main research area
Computing and designing the reinforced-concrete structures for buried engineering constructions

Address
15 "B", F. Skoriny str. 220114 Minsk, Belarus
Tel. (017) 263-61-22
E-mail: m.a.e.81@mail.ru