International Journal of Engineering & Technology, 7 (4.8) (2018) 432-439

International Journal of Engineering & Technology

Website:www.sciencepubco.com/index.php/IJET

Research paper

Theoretical Bases of the Compatible Work of the Constraction of Stormwater Drainage Systems in the Regulation of Stormwater Runoff

Olexander Tkachuk[1]*, Yana Yaruta[2], Olha Shevchuk [3], Anna Azizova[4]

[1]National University of Water And Environmental Engineering, Ukraine
[2]National University of Water And Environmental Engineering, Ukraine
[3]National University of Water And Environmental Engineering, Ukraine
[4]Poltava National Technical Yuri Kondratyuk University, Ukraine
**Corresponding Author E-Mail:o.a.tkachuk@nuwm.edu.ua*

Abstract

Measures of flooding process on urban areas, urban stormwater sewerage systems and methods regulation of stormwater need to be rectification. The method of temporary interception stormwater at places of their precipitation with infiltration basins is reviewed.
Infiltration basins are used to reduce the peak load on existing stormwater sewerage facilities, prevent the causes of flooding by stormwater. The processes of stormwater formation on urban areas is analyzed. Analytical dependencies for the hydrograph construction of flow through sewers and their inflow to the accumulation points were obtained. Conditions for the regulation of stormwater on urban areas with its partial flow and temporary interception on infiltration basins are determined. A refined exponent formula with a coefficient taking into account the filling of the pipeline for hydraulic and optimization calculations of pressure and non-pressure sewerage pipelines are proposed. Numerical values of coefficients and exponent of the specified formula for Ukrainian normative indicators of working conditions and calculations of sewerage pipelines are obtained. The full expediency of the refined power formula for practical use has been confirmed. The application of this formula for calculations of the compatible work of the pipelines and joint operation with the stormwater regulating construction to calculate the work of sewerage networks in the pressure and non-pressure conditions and identify possible areas of flooding with location of infiltration basins there.

Keywords: *rain water infiltration basin,. sewerage system,. regulation of stormwater, urban areas*

1. Introduction

In recent years, there are significant processes of flooding of urban territories. It is known that the causes of these phenomena are not only climate change, which contribute to the fallout of high-intensity rains. Another reason is the increase in areas with impervious pavement and disruption in the operation of sewerage systems and their imperfections [[6], [10], [11]]. As a rule, flooding of urban areas is temporary (usually no more than 0,5-1 hours). But the losses from it are significant (flooding cars, basements of buildings, washed away of road pavements, destruction of green planting, sanitary pollution of territories, etc.). The flooding of the areas is observed for a longer time, and on separate plots constantly. It also leads to the flooding of basements of buildings, the destruction foundations of buildings, engineering communications, road pavements, green planting, etc. Practice shows that flooding of territories takes place even when the system of rain drainage works properly, but the urban territory has a rugged terrain [[10], [11]]. In many cases, at rains are of high intensity, when the networks operate in the pressure regime, flooding of the territories is carried out through stormwater and inspection wells [[11]]. An analysis of such conditions of flooding on urban areas showed that they are due to the imperfection of urban stormwater systems, caused by the permissibility of pressure regime in sewers [[1]]. Pressure regime is not sufficiently substantiated for hydraulic calculations of stormwater systems. Separately, valid methods of calculating stormwater sewerage [[1], **Error! Reference source not found.**, [7], [17]] do not envisage the joint operation of all sections along the length of the sewer of stormwater sewerage. In recent years, construction and sewerage networks or their unsatisfactory condition only complicate the problem and become widespread.
The problems of study the conditions for the forming stormwater on urban areas, the operation of stormwater sewerage systems and their calculations were undertaken by such leading scientists as Bolshakov VO, Zhuk VM, Konstantinov Yu.M, Panteleit GS, Tkachuk SG, Abramov LT, Alekseev MI, Belov NN, Gorbachev PF, Dikarevsky VC, Kurganov AM, Molokov MV [**Error! Reference source not found.**-[7], [16]], Ferguson B. Horton R., Królikowska J., Królikowski A., Mays L.W., Rossman L.A., Weitman D., Weinberg A., Goo R. [[18]-[21], [23]], etc. These scientists have made a significant contribution to the study of the processes of forming the estimated discharges of stormwater by various methods. For example, the limiting intensity, the research of individual methods and modes operation of construction for the regulation of stormwater runoff. However, the issues of optimizing the operation of sewerage systems are not solved, based on a systematic approach to the regulation (collection and discharge) and the treatment of stormwater. In recent years, in the National University of Water and Environmental Engineering in

Rivne has started comprehensive research on solving these issues [[11]-[15]].

Today, the methods of regulating stormwater, which allow to accumulate stormwater directly in the places of precipitation, have a great importance. This allows to keep water balance, prevent flooding of territories, reduce the maximum load on existing sewerage systems and prevent pollution. One of such methods is the installation of infiltration area with the subsequent drainage of detained stormwater in the pipelines of the sewerage system [[11] [13]]. The use of various types of permeable pavement can significantly increase the infiltration area, and therefore the volumes of temporarily delayed stormwater [[2]]. For this purpose, such city territories as parking for cars, various kinds of roads, lawns, etc. can be used. The normative base [[1]-[3], [21]] of such regulation of stormwater requires a generalization and theoretical estimation.

2. Main Body

A comprehensive solution to the problems of stormwater management in urban areas involves conducting research to improve methods of calculations and engineering approaches to the arrangement of separate constructions of urban stormwater sewerage systems. Among the main areas of such research should be distinguished:

- formation of the estimated discharges and volumes of stormwater in the places of their accumulation and spillway;
- calculations of the compatible work of stormwater sewerage facilities for the most typical modes of stormwater inflow;
- accumulation of stormwater and treatment of sediment and pollution.

2.1. Formation of estimated discharges and volumes of stormwater

The process of formation of stormwater depends on many natural (intensity and duration of rain and between rainy periods, terrain, hydrological regimes of reservoirs) and technical (the state of the stormwater sewerage system, the characteristics of the coverings and soils in urban areas, the conditions for the release of stormwater in the reservoir, etc.) factors. According to existent norms [[1], [3]] estimated runoff discharges of stormwater are determined by the method of limiting intensities; changes in their magnitudes in time and based on the graphs of their inflow to the calculated control point, hydrographs of runoff [**Error! Reference source not found.**, [5]]. The scientific basis for constructing hydrographs of stormwater from the urban area provides for the construction of a runoff model with the determine the dependence of rainfalls and the formation of runoff, restrictions on their application and the adoption of reasonable assumptions and simplifications without which the construction of the model will be complicated or impossible at all [[9], [16], [23]].

The basis of runoff model is the method of limiting intensities. By this method, the calculated duration of the rainfall takes equal to the duration of the flow of surface water from the most remote point of the basin to the settlement reference point of the system, and the inflow of stormwater which is proportional to the area of flow. In this case, for sewerage sewers, the inflow of stormwater in their length is taken uniform, and the flow area which evenly located along the sewer [**Error! Reference source not found.**, [5]]. From this point of view, the sewer itself should be divided into separate plots, for each of which the flow area can be considered evenly located along the sewerThe scheme of inflow of stormwater to such a separate section of the sewer is shown in Fig. 1. According to the intensity of the inflow of stormwater to the control point ***K*** section of sewer's, the area of water collection has three zones: an intensive, stable and slow growth of the collection of stormwater.

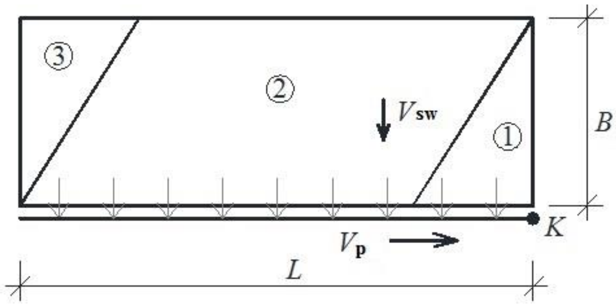


Fig. 1: Estimated scheme forming stormwater inflow at the control point ***K*** to the sewer: 1, 2 and 3 - zones of increase of stormwater collection, respectively, intensive, stable and slow

In the adopted model formation of stormwater for each section of the sewer (at the ***i***-th control point), the estimated discharge of stormwater q_i, in l/s, was determined according to the norm [[1]]. In this case, the rain duration $t_{r.i}$ is assumed to be equal sum of time flow of stormwater on the surface the built-up area t_{sw} (time of surface concentration and in the trays) and the time of flow through the pipeline t_p from the most remote point of the basin of flow to the calculated control point. The consumpltion of rain runoff on each (***i***-th) section q_i is represented in dimensionless coordinates, dividing them into the expense at the end point q_K. Relative consumpltion q'_i is calculated by the formula

$$q_i^{'} = \frac{q_i}{q_K} = \frac{z_{mid} \cdot A^{1.2}}{t_{r.i}^{1,2n-0,1}} \cdot \frac{t_{r.K}^{1,2n-0,1}}{z_{mid} \cdot A^{1,2}} \cdot \frac{F_i}{F_\Sigma} = \frac{F_i^{'}}{t_{r.i}^{'\ 1,2n-0,1}} = q_{0i}^{'} \cdot F_i^{'}, \quad (1)$$

where F_i' is the relative area of the drain for an ***i***-th area determined by the ratio of the area F_i to F_Σ ; $t'_{r.i}$ is relative duration of the rain for the *i*-th section of the sewer, which is equal to the ratio of the duration of the discharge water to the control point on the *i*-th section tr to the duration of the discharge of water at the end point of the sewer $t_{r.K}$, through which all the volumes of stormwater pass; q'_{0i} is relative specific intensity of the arrival of stormwater for the *i*-th section of the sewer

$$q_{0i}^{'} = \frac{1}{t_{r.i}^{'\ 1,2n-0,1}} = \left(\frac{t_{r.K}}{t_{r.i}}\right)^{1,2n-0,1}. \quad (2)$$

Each zone formation of stormwater has its own characteristic patterns, which include:

- increase in the inflow of stormwater is proportional to the area of the runoff with an increase in the time of water flow to the control point;
- increase of the area of runoff in time is determined by its differential dF at infinitely small distance dx (Fig. 2): $dx = V_p \cdot dt$;
- changes in the relative discharges of stormwater in the *i*-th area dq'_i at the control point of the sewer were recognized by formula 3;
- changes in relative discharges of stormwater q'_i were recognized in the time from the beginning of the rain by integrating type dependencies (3);
- changes in relative volumes of stormwater dW'_i, passing through the control point of the sewer were recognized by formula 4;
- values of the relative volumes of stormwater W'_i were recognized in the time from the beginning of the rain by integrating type dependencies (4).

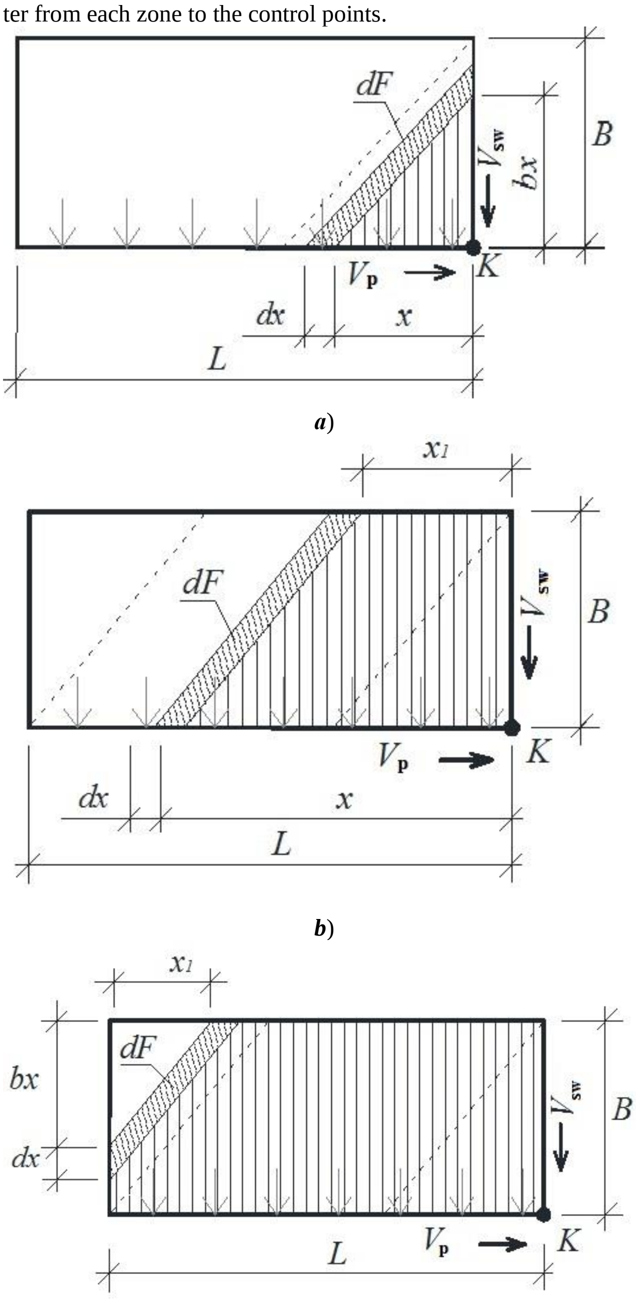
$$dq_i^{'} = q_{0i}^{'} \cdot dF^{'} = q_{0i}^{'} \cdot \frac{\partial(F_i^{'})}{\partial t^{'}} \cdot dt^{'}, \quad (3)$$

where $\frac{\partial(F_i^{'})}{\partial t^{'}}$ is derivative function of changing the relative runoff area in the corresponding zone; V_{sw} is average velocity of runoff on the surface (roofs, sidewalks, trays, etc.); V_p is average velocity of water in the pipeline.

$$dW_i^{'} = q_i^{'} \cdot dt^{'}, \tag{4}$$

where q'_i is relative discharge of rain runoff of the ***i***-th zone at the control point of the sewer, which was determined by the dependence obtained on the basis of the formula (3).

The characteristics of each zone formation of rain runoff depend on the form area of runoff and kinds dependencies of type (3), as well as the limiting values of the duration of the discharge of water from each zone to the control points.



a)

b)

c)

Fig. 2: Schemes formation of the inflow stormwater in the zones of growth of their runoff: ***a)*** intensive***; b)*** stable***; c)*** slowed down

In accordance with these features and characteristic regularities of flow, we obtained the following formulas for the values of relative consuptions of stormwater q'_i and relative volumes of stormwater W'_i in time from the beginning of the rain:

a) in the zone of intense growth of runoff (Fig. 2*a*)**:** $0 < t < t_1$; $(0 < t' < t'_1)$; $t'_1 = t'_{sw}$;

$$q_i^{'} = \frac{q_{0i}^{'}}{2 \cdot t_p \cdot t_{sw}} \cdot \left(t^{'}\right)^2; \tag{5}$$

$$W_i^{'} = \frac{q_{0i}^{'}}{6 \cdot t_p \cdot t_{sw}} \cdot \left(t^{'}\right)^2 = q_i^{'} \cdot \frac{t^{'}}{3}; \tag{6}$$

b) in the zone of stable growth of runoff (Fig. 2*b*)**:** $t'_1 < t < t_2$; $(t'_1 < t' < t'_2)$; $t'_1 = t'_{sw}$; $t'_2 = t'_p$;

$$q_i^{'} = q_{1i}^{'} + \frac{q_{0i}^{'}}{t_p^{'}} \cdot \left(t^{'} - t_1^{'}\right), \tag{7}$$

where q'_{1i} is relative water discharge at the end of the 1-st area of the runoff for the *i*-th zone $t' = t'_1$;

$$W_i^{'} = W_{1i}^{'} + q_{1i}^{'} \cdot \left(t^{'} - t_1^{'}\right) + \frac{q_{0i}^{'}}{2 \cdot t_{mp}^{'}} \cdot \left(t^{'} - t_1^{'}\right)^2, \tag{8}$$

where W'_{1i} is relative volume of water passing through the control point of the sewer after the end of the ***1***-st area of runoff, at $t' = t'_1$;

c) in the zone of slow growth of runoff (Fig. 2*c*)**:** $t'_2 < t < t_3$; $(t'_2 < t' < t'_3)$; $t'_2 = t'_p$; $t'_3 = t'_{r.i}$;

$$q_i^{'} = q_{2i}^{'} - \frac{q_{0i}^{'}}{2 \cdot t_{sw}^{'} \cdot t_p^{'}} \cdot \left(\left(t_{r.i}^{'} - t^{'}\right)^2 - \left(t_{r.i}^{'} - t_2^{'}\right)^2\right); \tag{9}$$

$$W_i^{'} = W_{2i}^{'} + q_{2i}^{'} \cdot \left(t^{'} - t_2^{'}\right) + \frac{q_{0i}^{'}}{2 \cdot t_{sw}^{'} \cdot t_p^{'}} \times \left(\frac{1}{3} \cdot \left(\left(t_{r.i}^{'} - t^{'}\right)^3 - \left(t_{r.i} - t_2^{'}\right)^3\right) + \left(t_{r.i}^{'} - t_2\right)^2 \cdot \left(t^{'} - t_2^{'}\right)\right), \tag{10}$$

where q'_{2i} i W'_{2i} is relative discharge and volume of water passing through the control point of the sewer after the end of the ***2***-nd area of runoff, at $t' = t'_2$.

The relative discharge at the end of the ***3***-th zone (the time of the completion of the rain, $t' = t'_{r.i}$), which is the maximum discharge of the hydrograph, is equal to

$$q_{\partial.i}^{'} = q_{2i}^{'} + \frac{q_{0i}^{'}}{2 \cdot t_{sw}^{'} \cdot t_p^{'}} \cdot \left(t_{\partial.i}^{'} - t_2^{'}\right)^2, \tag{11}$$

and the relative volume of water after the completion of the rain (at $t' = t'_{r.i}$) is

$$W_{\partial.i}^{'} = W_{2i}^{'} + q_{2i}^{'} \cdot \left(t_{r.i}^{'} - t_2^{'}\right) + \frac{q_{0i}^{'} \cdot \left(t_{r.i} - t_2^{'}\right)^3}{3 \cdot t_{sw}^{'} \cdot t_p^{'}}. \tag{12}$$

At the completion of the rain (при $t' > t'_{r.i}$) discharges and volumes of water will be formed during the time of discharge of water from the most remote to the control point the basin of flow. The whole area of the basin of the flow 1-th area will be discharged from stormwater during the time $t'_{r.i} = t'_{p.i} + t'_{sw}$, and from the beginning of the rain $2t'_{r.i}$. Having calculated the relative discharges q'_i by formulas (2) and (3), the relative volumes of water $W'_{a.i}$ by integrating formula (4); the relative areas of flow F'_i separately for each of the zones of reduction the area of the flow having a mirror position relative to the growth zones of the runoff (Fig. 1), the following formulas are obtained for eachzones of the runoff:

a) zone of intense decrease of runoff (in fig. 1 - zone 3)**:** $t_{r.i} < t < t_a$; $(t'_{r.i} < t' < t'_a)$; $t'_a = t'_{r.i} + t'_{sw}$;

$$q'_i = q'_{r.i} - \frac{\left(t'_{r.i}\right)^{0.1-1.2\cdot n}}{2\cdot t'_{sw}\cdot t'_p}\cdot\left(t' - t'_{r.i}\right)^2, \tag{13}$$

where $q'_{r.i}$is relative water discharge on the i-th zone at the completion of the rain;

$$W'_i = W'_{r.i} + q'_{r.i}\cdot\left(t' - t'_{r.i}\right) - \frac{\left(t'_{r.i}\right)^{0.1-1.2\cdot n}}{6\cdot t'_{sw}\cdot t'_p}\cdot\left(t' - t'_{r.i}\right)^3, \tag{14}$$

where $W'_{r.i}$ is relative volume of water passing through the control point of the i-th section of the sewer at the completion of the rain, which is calculated by the formula (12);

b) zone of stable decrease of runoff (in fig. 1 - zone 2): $t'_a < t < t_b$; $(t'_a < t' < t'_b)$; $t'_a = t'_{r.i} + t'_{sw}$; $t'_b = t'_{r.i} + t'_p$;

$$q'_i = q'_{a.i} - \frac{\left(t'_{r.i}\right)^{0.1-1.2\cdot n}}{t'_p}\cdot\left(t' - t'_a\right), \tag{15}$$

where $q'_{a.i}$ is relative water discharge at the end of the previous runoff zone for i -th area, which is calculated by the formula (15) at $t' = t'_a = t'_{r.i} + t'_{sw}$;

$$W'_i = W'_{a.i} + q'_{a.i}\cdot\left(t' - t'_a\right) - \frac{\left(t'_{r.i}\right)^{0.1-1.2\cdot n}}{2\cdot t'_p}\cdot\left(t' - t'_a\right)^2; \tag{16}$$

where $W'_{a.i}$ is relative volume of water passing through the control point of the sewer at the completion of the previous runoff zone, which is calculated by the formula (14) at $t' = t'_a = t'_{r.i} + t'_{sw}$.

c) zone of slow decrease of runoff (in Figure 1 zone 1): $t'_b < t < t_c$; $(t'_b < t' < t'_c)$; $t'_b = t'_{r.i} + t'_p$; $t'_c = t'_{r.i} + t'_{sw} + t'_p$;

$$q'_i = q'_{b.i} + \frac{\left(t'_{K.i}\right)^{0.1-1.2\cdot n}}{2\cdot t'_p\cdot t'_{sw}}\cdot\left(\left(t'_c - t'\right)^2 - \left(t'_c - t'_b\right)^2\right), \tag{17}$$

where $q'_{b.i}$ is relative water discharge at the end of previous runoff zone on the i-th section calculated by formula (15) for $t' = t'_b = t'_{r.i} + t'_p$;

$$W'_i = W'_{b.i} + q'_{b.i}\cdot\left(t' - t'_b\right) - \frac{\left(t'_{r.i}\right)^{0.1-1.2\cdot n}}{2\cdot t'_p\cdot t'_{sw}}\times$$
$$\times\left(\frac{1}{3}\cdot\left(\left(t'_c - t'\right)^3 - \left(t'_c - t'_b\right)^3\right) + \left(t'_c - t'_b\right)^2\cdot\left(t' - t'_c\right)\right), \tag{18}$$

where $W'_{b.i}$ is relative volume of water passing through the control point of the sewer at the completion of the previous runoff zone, which is calculated by the formula (16) at $t' = t'_b = t'_{r.i} + t'_p$.

Hydrographs of relative consumptions and volumes are shown in Fig. 3 and 4 for a sewer of three sectors with control points at the end of each of them K_1, K_2 i K_3. When designing the hydrographs, it was accepted that the relative time flow of the water along the surface is t'_{sw} = 0,25, at each of the sections $t'_{p.i}$ = 0,25, along the pipeline t'_p = 0,75, and the estimated relative duration rain for the entire sewer t'_r = 1,0.

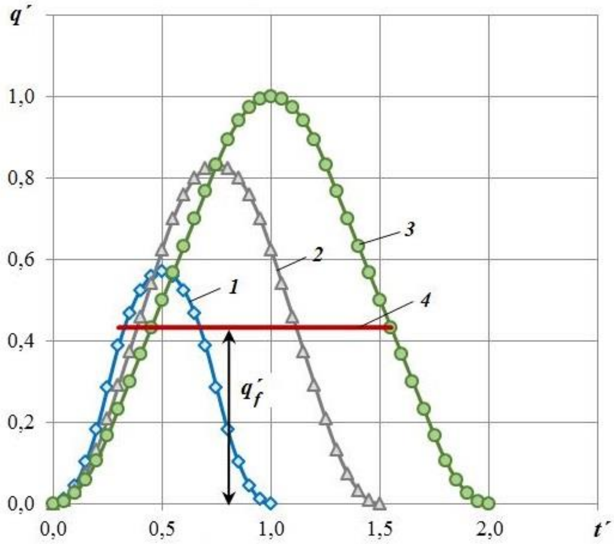


Fig. 3: Hydrographs of the relative discharges of the sewer: 1, 2 and 3 - estimated hydrographs for the 1st, 2nd and 3rd sections of the sewer; 4 - marginal discharge of the throughput of sewer

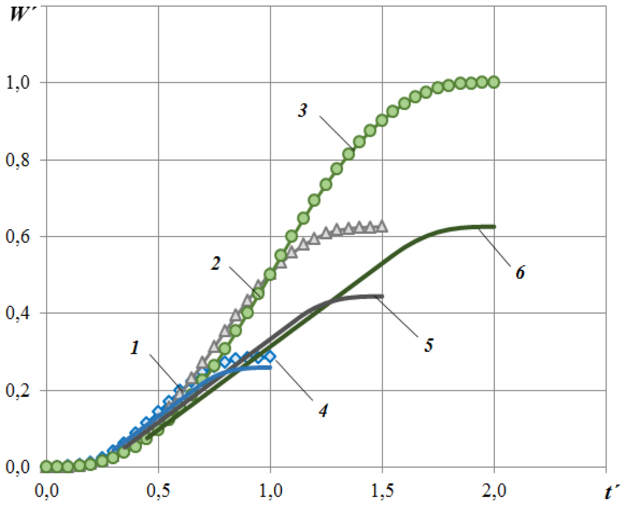


Fig. 4: Hydrographs of relative volumes of stormwater passed through control points of the sectors of sewer: 1, 2 and 3 - estimated hydrographs for the 1st, 2nd and 3rd sections of the sewer; 4, 5 and 6 - hydrographs of throughput capacity of sections of the sewer

The obtained hydrographs (Fig. 3 and 4) show that by the method of limiting intensities, the growth of the estimated discharges on section of sewer occurs less intensively than the volumes of stormwater passing through these sections. In real conditions, the sewer is able to pass through the discharges of no more than a certain value q'_f (Fig. 3), which is determined by the results of hydraulic calculations or actual measurements. On separate sections of the sewer, or at all, these consuptions are less than the calculated maximum $q'_f < q'_{max.i}$. This leads to the flooding of the urban territories were such areas are located. Volumes of stormwater discharged through the sewer's sections are determined by the hydrographs of their throughput, and the difference between them and the calculated hydrographs corresponds to the volumes of stormwater that will flood the urban areas. To prevent flooding of urban areas, these volumes of stormwater should be detained by the structures of regulation of stormwater, in infiltration basins.

2.2. The stormwater accumulation

The numerical modeling of the change of stormwater drainage hydrographs for q'_f = 0,1 – 1,0 in the values range t'_{sw} = 0,2 ÷ 0,3 and t'_p = 0,7 ÷ 0,8 was conducted to determine the dependence of the volume of stormwater that should be detained in the infiltration basins W'_{reg}, of the admeasurement of the actual discharge reduction q'_f . The simulation results are shown in Fig. 5.

According to the simulation results, a formula for determining of the relative regulating volumes W'_{reg} from the relative actual discharges q'_f, is obtained. The adequacy of equation is confirmed by high correlation bonds with numerical data (R^2 > 0.998).)

$$W'_{reg} = t'_p \cdot \left(q'_f - 1\right)^2 - t'_{sw} \cdot \left(q'_f - 1\right). \tag{19}$$

The data of Fig. 5 show that the calculations results of the obtained formula (19) are practically concurred with the corresponding values of numerical simulation. For practical calculations, it is necessary to determine the flow capacity of the sewer (in relation to the maximum flow rate of the hydrograph at its control point of calculation) and the duration of surface runoff concentration and flow of water through the sewer to the calculated point (in relation to the estimated runoff time).

Thus, the calculation of sewer's capacity during the construction of drainage hydrographs depends on many parameters. Among them there is to determine the values of the stormwater volume formated in each point of the stormwater experimental watershed. There is also to estimate the actual sewer's capacity without area flooding and the regulating storage volume that it is necessary to detain in infiltration basins.

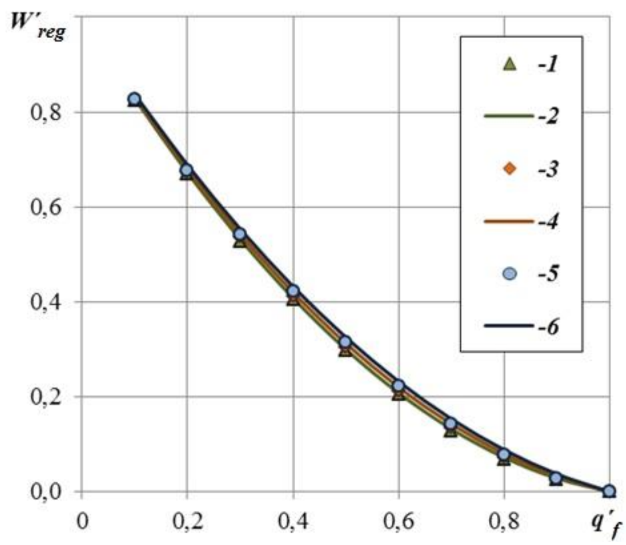


Fig. 5: The graph of dependence of relative regulating storage volume W'_{reg} of relative colector's capacity q'_f: 1 – points of numerical simulation for t'_{sw} = 0,2 і t'_p = 0,8; 2 – the same for t'_{sw} = 0,25 і t'_p=0,75; 3 – the same for t'_{sw} = 0,3 і t'_p=0,7; 4, 5 і 6 – curves of design values according to the formula (19)

2.3. Compatible work of drainage facilities

The capacity of the existing or project sewers of the stormwater drainage system have to be evaluated on the calculations of the compatible operation of all interacting spillways construction for the most typical conditions of stormwater inflow. In this case, detached sewers operate in both, nonpressure and pressure conditions [[1], **Error! Reference source not found.**, [5], [11], [17]]. According to their hydraulic calculations, it is necessary to determine the flow discharges and velocities, depending on the sewer slope, but also the piezometric markers in the nodes, as in the case of nonpressure, and, especially, pressure runoff sewer transportaion. It is necessary not only for leveling accommodation and section of sewers, but also for the analysis of compatible operation of stretch of the sewerage network, optimization of its individual parameters, identification of possible zones of flooding of urban areas, etc. Taking into account that the most of the sewerage drainage networks of modern cities are pipelines, for their hydraulic calculations, an equation (20) represented in [[14], [15]] is recommended

$$I = k \cdot \frac{q^{\beta}}{d^m} \cdot k_{h/d}, \tag{20}$$

where k, β and m are coefficient and exponents, which depend on the roughness of the inner surface of the pipes, their material, their quantity and type of sediments on the walls, etc; q is calculating runoff discharges, in m^3/s, or l/s; d is calculating inner diameter or other size of sewer, in m, або mm; $k_{h/d}$ is coefficient, which depends on the pipeline impoundment and estimates by the formula

$$k_{h/d} = a + b \cdot \left(\frac{h}{d}\right)^{\gamma}. \tag{21}$$

Numerical values of coefficients and exponents in formulas (20) and (21) depend on the roughness of the inner surface of the pipes and operating range of hydraulic conditions [[7], [8] [14][17]]. The coefficient k also depends on the dimension of the parameters q and V. The structure of formula (20) is similar to the widespread in the world practice formulas for calculations of pressure pipelines [7], [19], [21] [22], [23].

For practical application, the values of coefficients and exponents in formulas (20) and (21) were determined by approbation of numerical data of an array of hydraulic slopes I, calculated according to the recommended standarts and guidelines [1] [3] by the formulas of the given parameters ranges: velocities V = 0,5-4,0 m/s; diameters d = 0,15-2,5 m; the kinematic viscosity $\nu = 1{,}3 \cdot 10^{-6}$ m^2/s, which is typical for sewage with a temperature t = 11-12°C with the amount of sediment suspension up to 400 mg/l; the roughness of the inner surface of the pipes Δ_e = 0,002 m (n = 0,014; a_2 = 100), which is most common in the disposal sewer, in particular concrete and reinforced concrete pipes [1], **Error! Reference source not found.**, [5][8] [17]. In addition, for determining the parameters a, b and γ there was taken into account not only the dependence of hydraulic parameters (hydrological radius, wetted perimeter, etc.) [14], [15] from the impoundment values of the pipelines h/d, but also the influence of hydrodynamic characteristics on the value of runoff discharges in the pipelines with their partial impoundment [7][15]. Figure 6 shows the calculated values of the coefficient $k_{h/d}$, which takes into account the impoundment value of the pipeline h/d. As a result, the following values of search parameters were obtained: k = 0,002087; β = 1,96; m = 5,23; a = 0.74; b = 0.74; γ = -3.92 [7][14], [15].

A comparative analysis of the results of calculations by the formulas (20) and (21) with the reference formulas recommended by the current Ukrainian standards and guidelines, shows that under the same conditions of sewerage pipelines discharge (roughness of the inner surface of pipes, runoff viscosity, etc.), their accuracy is proportional to the accuracy of the hydraulic calculations of pressure pipelines [14], [22], and does not exceed + 5% (Fig. 7). At the same time, changes in the conditions of the pipelines make divergence in hydraulic calculations up to + 10%. Thus, only the change in the roughness of the surface of the pipes with Δ_e = 2.0 mm on Δ_e = 1.35 mm in the normative formula [1] gives an error in the calculated hydraulic slopes -12%. It is obvious that in practice these differences will be greater. Therefore, slight deviations from the "reference" formulas in the range from -5% to + 3% (Figure 7) can be considered quite acceptable.

The current practical recommendations for hydraulic calculations of sewage networks [1], [3], [7], [17] are based on different operation conditions of sewage pipelines, which is obviously connected with different initial conditions of study. It makes sense to use in the exponent formula for the accepted numerical values of the parameters that correspond to the most common operating conditions of the stormwater pipelines.

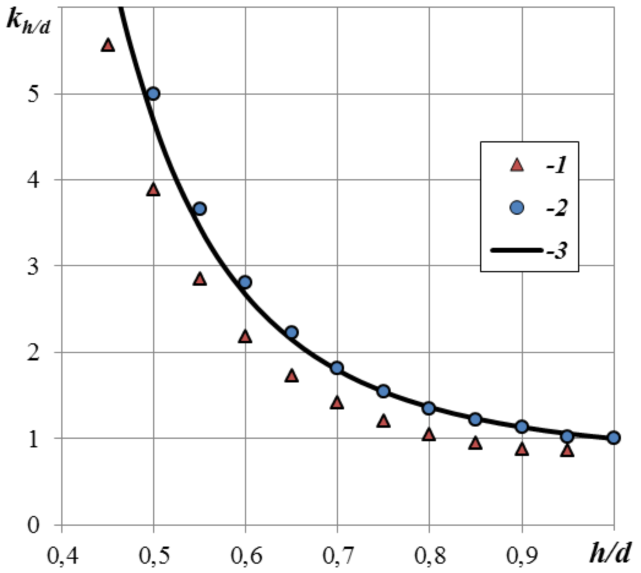


Fig. 6: The dependence of coefficient $k_{h/d}$ of pipeline impoundment h/d and influence of hydrodynamic characteristics of the flow: 1 – the points of numerical simulation without the influence of the hydrodynamics of the flow; 2 – the same with this infuence;3 – calculating value $k_{h/d}$, received by the formula (21)

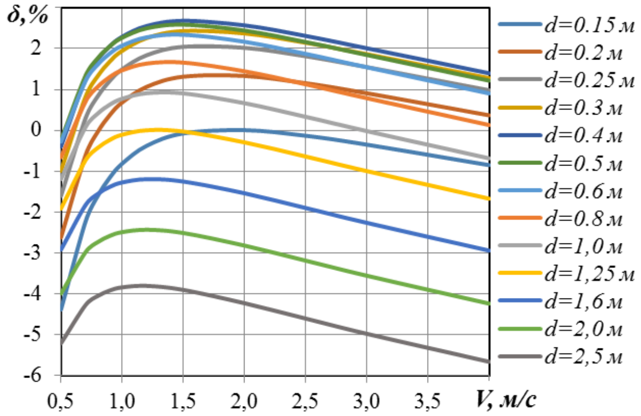


Fig. 7: The divergence of calculating of hydraulic slopes by the formula (20) in comparison with calculation according to the reference formulas [14]]

The obtained dependences (20) and (21) allow to take into account the compatible operation for different sections of a single sewer or an extensive network of several sewers, but also connected regulating construction or reservoirs. To do this, with received watersheds hydrographs of each section, the estimated maximum runoff discharges according to this sewer area capacity have to be determined. Taking into account that in the adopted model of stormwater formation, the estimated discharges of rainfall q_i, , l/s, were determined according to the norm [[1]], for the further calculations may be applied formula (22). In this case, the duration of rainfall $t_{r.i}$ is reasonable to assume equal to its average value for this local region. Usually, for the conditions of Ukraine, this value is 30-90 minutes.

$$q_i = z_{mid} \cdot \frac{A^{1.2}}{t_{r.i}^{1.2n-0.1}} \cdot F_i = q_o \cdot F_i, \qquad (22)$$

where z_{mid} is the average value of the runoff coefficient, which depends on the types of coverage of different parts of the watershed area and their particles of runoff; A and n are parameters taking into account the geographical location of the city and period P, years, one-time probability of the calculated intensity of rainfall; q_0 is unit rainfall intensity, l/s/ha.

This approach allows to receive one-time calculation load on the entire sewage network and its hydraulic calculations are carried out not begining from the starting points of the network, as is the case in the current methods [1], **Error! Reference source not found.**, [5], [17]. It allows to begin from its endpoints, where by way of spillways stormwater flow into sewage treatment facilities, tank or reservoirs. This makes possible to take into account values of runoff level in these constructions which are familiar and independent of the results of hydrological calculations of the network. Sewer sections with pressure and nonpressure disposal conditions, as well as potential areas of flooding are accurate evaluated. Based on the hydraulic calculations, it is sufficient to obtain the calculated values of the hydraulic slopes I in each section of the network and the piezometric mark in its design points (Fig. 8). In the calculations of each section of the sewer, the first priority belongs to equation of the hydraulic slope I in the sewer to the slope of its tray I_l. Then the formula (20) for $I = I_l$ determines the value of the coefficient $k_{h/d}$. Taking into account that the sewer impoundment coefficient is in the range $k_{h/d}$ > 1.0 (Fig. 6), the obtained value $k_{h/d}$ which is less than 1.0, indicates that the pipeline operates in the pressure conditions, and the hydraulic slope at this discharge will be greater than the slope of the sewer tray $I > I_л$. The calculating value of the hydraulic slope is determined by the formula (20) for $k_{h/d}$ = 1.0, which corresponds to the complete sewer impoundment h/d = 1.0. The value of h/d is determined from the formula (21) for $I = I_l$ and $k_{h/d}$ > 1.0.

The values of the piezometric marks at the beginning of each sewer section are calculated by the formula

$$P_{n.i} = P_{к.i} + I_i \cdot l_i, \qquad (23)$$

where $P_{k.i}$ is piezometric mark at the end of this section; I_i is hydraulic slope on the i-th section; l_i – geometric length of the i-th section.

The value of the piezometric mark at the end of the previous section of the rainfall flow is taken equal to the obtained value $P_{k.i}$. Figure 8 shows the diagram of a sewer which consists of 3 section. Calculations begin with the last sewer section (**3**rd) for runoff flow, taking the value of the end piezometric mark equals to the level of water in the construction or reservoir for the inflowing rainfall runoff ($P_{к.3} = Z_в$).

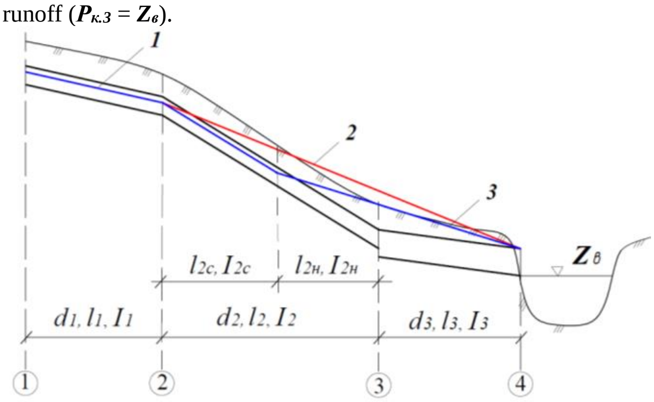


Fig. 8: The estimated scheme of the pressure-gravity condition of the operate of stormwater drainage sewer

1 – the estimated water level on the gravity section of the sewer;
2 – piezometric line of pressure condition of stormwater flow for area flooding; 3 – the same without area flooding; 1, 2, 3, 4 (in circles) – calculation points of sewer section

Calculations according to the sewer scheme, shown in Fig. 8, indicate that, when the process of exploitation the discharges in the section 3 (for example, due to the high density of the building and the increase of the urban area with impervious pavement) is growth, it will operate in the pressure condition. With increased flow of water around the calculated 3rd point, flood zone can be formated, and the piezometric mark at this point (line 2) will be the higher of ground surface. Thus, it is necessary to reduce the discharge on 3^{rd} section in such a way that the piezometric line (Fig. 8, line 3) throughout the sewer trail passes below the ground surface. In this case, some sewer section (in Fig. 8, section 2) will operate in gravity and pressure condition.

The new values of runoff discharges in detached sewer sections will correspond to the boundary discharge of the flow capacity of the corresponding sewer section (Fig. 3). In this case, redundant runoff discharges should be intercepted to stormwater regulating construction. Their storage volumes are calculated by the formula (19), and their locations determine above the possible flooding area of watershed. Figure 9 shows an example of determining of the infiltration basins' possible locations on the territory of the housing estate. In order to determine their exact location it is necessary to study the characterics of ground base layer to the required depth and flow the runoff from them by gravity in the existing drainage sewers.

Calculations of the compatible operation of the stormwatar drainage construction show that the emptying of infiltration basins after the end of the rainfall does not lead to flooding of urban areas, because the dischsrges that inflow from any stormwater regulating construction are always lower than those that inflow to it for temporary interception [13].

The runoff drainage by infiltration basin allows them to intercept a significant portion of sediments. The main amount of them is delayed in the upper base course layer of the basin, which necessarily includes the grass vegatation, layers of sand and geotextiles [12]. However, study in this direction is still continuing.

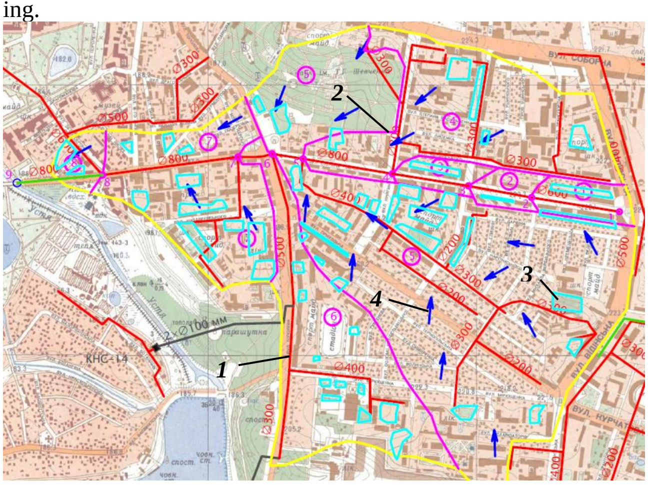


Fig. 9: An example of the location of infiltration basins in the residential estate: 1 – existing sewerage; 2 – the boundaries of the runoff watershed by sewer section; 3 – location of infiltration basins; 4 –rainfall flow on the area

3. Conclusions

Infiltration basins temporary accumulate rainfall directly at place of precipitation. This allowed to reduce the maximum water discharge that determine pipe diameters and sizes of construction of stormwater drainage system, to prevent urban flooding and inundation and to purificate stormwater.

Based on model of stormwater runoff on urban areas analytical dependencies of monitoring hydrographs of stormwater flow through sewers and their inflow to storage sites were obtained. The received hydrographs show that the growth of calculating discharge in sewer sections is less intense than the volume of stormwater flowing through these sections. The difference between the hydrographs of the stormwater inflow to the sewer and its capasity corresponds to the volume of stormwater that will flood the urban areas. To prevent flooding these volumes are proposed to accumulate and temprorary intersept on stormwater regulating construction, in particular infiltration basins.

It has been determined that the regulation of stormwater runoff in urban areas during its partial flow through infiltration basins depends on the intensity and duration of the rainfall, the capasity of existing drainage sewers and the location of infiltration basins.

A refined exponent formula with coefficient, which takes into account the filling of the pipeline for hydraulic and optimisation calculations, pressure and nonpressure drainage sewers, is estimated. Numerical values of formula's coefficients and exponenents are obtained.

Based on the refined exponent formula, a new approach to hydraulic calculations of stormwater drainage networks is proposed, which involves the calculation of their compatible work with joint facilities. This allows to identify places of possible flooding and inundation of urban areas and efficient location of stormwater regulating construction.

References

[1] DBN V.2.5-75:2013 (2013). Kanalizatsiia. Zovnishni merezhi ta sporudy. Osnovni polozhennia proektuvannia. [Sewerage. External networks and facilities. The main principles of the design], Kyiv, Minrehionbud Ukrainy, 211 p. [in Ukrainian]

[2] DSTU-N B V.1.1-38:2016 (2017). Nastanova shchodo inzhenernoho zakhystu terytoriy, budivel′ i sporud vid pidtoplennya ta za-toplennya. [Guidelines engineering protection of the territory, building and structure from water logging and flooding.], Kyiv, Minrehion Ukrainy, 198 p. [in Ukrainian]

[3] DSTU-N B V.2.5-61:2012 (2012). Nastanova z ulashtuvannya system poverkhnevoho vodovidvedennya. [Guidelines installation of storm drainage systems], Kyiv, Minrehion Ukrainy, 30 p. [in Ukrainian]

[4] Alekseev M., Kurhanov A. (2000) *Orhanyzatsyia otvedenyia poverkhnostnoho (dozhdevoho y taloho) stoka s urbanyzyrovannykh terrytoryi [Organization of runoff (rainfall and snowmelt) disposal from urban areas]*. Moscow: Publishing ASB; 352 p. [in Russian]

[5] Dikarevsky V., Kurganov A., Nechaev A. & Alekseev M. (1990), *Otvedeniye i ochistka poverkhnostnykh stochnykh vod [Withdrawal and treatmening of surface wastewater]*: a textbook for high schools, Leningrad, 224 p [in Russian].

[6] Zhuk V.M. (2011) Suchasni systemy upravlinnya doshchovym stokom na zabudovanykh terytoriyakh [Modern systems management of stormwater in built-up areas]. *Scientific and Technical Periodical of KNUCA. Problems of water supply, sewerage and hydraulics, Issue 17.* Kiev, pp. 26-31.

[7] Konstantinov Yu.M, Gizha O.O. (2006) *Inzhenerna hidravlika [Engineering Hydraulics]*. Publishing House "Word", Kyiv, 432 p.

[8] Lukinnyh A.A., Lukinnyh M.A. (1987) *Tablitsy dlya gidravlicheskogo rascheta kanalizatsionnykh setey i dyukerov po formulakh N.N. Pavlovskogo [Tables for the hydraulic calculation of sewage networks and ducers according to the academician's formula NN Pavlovsky].* Stroyizdat. Moscow, 159 p. [in Russian]

[9] Matlay I.I. (2013) Udoskonalennya metodiv rozrakhunku hidrohrafiv prytoku doshchovykh stichnykh vod [Improvement of the methods calculating of the hydrographs of flow stormwater]. *Author's abstract of dis. Candidate of Technical Sciences : 05.23.16.* Rivne, 20 p. [in Ukrainian]

[10] GDR report (2013) Provedennya doslidzhen′ shchodo propusknoyi spromozhnosti system poverkhnevoho vodovidvedennya v suchasnykh pryrodnykh umovakh [Conducting studies of the capacity of surface drainage systems in modern natural conditions]. Kharkiv: DTsIK UkrVODGEO. 60 p. [in Ukrainian]

[11] Tkachuk O., Salchuk V. & Oleksiyuk O. (2014) Otsinka prychyn zatoplennya kanalizovanykh miskykh terytoriy doshchovymy vodamy [Estimation of the causes of flooding of sewage urban areas by stormwater]. *Bulletin NUWMNRU. Technical sciences 1 (65).* Rivne, pp. 344 - 350. [in Ukrainian]

[12] Tkachuk O., Shevchuk O. (2016) Konstruktyvni osoblyvosti infil′tratsiynykh maydanchykiv z vodopronyknymy pokryttyamy [Design features of infiltration areas with permeable pavements].

Science and construction, Issue 1, SRICC. Kyiv, pp. 38-41. [in Ukrainian]
[13] Tkachuk O., Shevchuk O. (2016) Umovy formuvannya drenazhnykh vytrat, shcho nadkhodyat′ u systemu doshchovoho vodovidvedennya pislya infil′tratsiynykh maydanchykiv [Forming conditions of drainage discharge that enter to the rain drainage system after infiltration areas]. *Bulletin NUWMNRU. Technical sciences 1 (73).* Rivne, pp. 100-105. [in Ukrainian]
[14] Tkachuk O., Yaruta Ya. (2017) Utochneni formuly dlya rozrakhunkiv truboprovodiv merezh vodovidvedennya [Updated formulas for calculations of pipelines of sewerage networks]. *Periodical of the Odessa State Academy of Civil Engineering and Architecture. Issue 68.* Odessa, pp. 165-172. [in Ukrainian]
[15] Tkachuk O., Yaruta Ya. & Shuminsky V. (2016) Obgruntuvannya formul ta yikh parametriv dlya optymizatsiynykh rozrakhunkiv merezh doshchovoho vodovidvedennya [Justification of formulas and their parameters for the optimization calculations of networks of the rain sewerage]. *Periodical of NUWEE. Technical sciences 4 (76)*. Rivne, pp. 259-267. [in Ukrainian]
[16] Tkachuk S., Zhuk V. (2012) Rehuliuvannia doshchovoho stoku v systemakh vodovidvedennia [Stormwater regulation on drainage systems]. *Lviv: Publishing Lviv Polytechnic*, 216 p [in Ukrainian]
[17] Yakovlev S., Karelin Ya., Zhukov A. & Kolobanov S. (2012) Kanalizatsiya [Sewerage]. Moscow, 633 p. [in Russian]
[18] Ferguson B. Porous Pavement: Integrative studies in water management and land development. Boca Raton : CRC Press, 2005. 600 p.
[19] Mays L. (2001) Storm water collection systems design hand book. *McGraw-Hill Professional*, 1008 p.
[20] Królikowska J., Królikowski A. (2012) Wody opadowe. Odprowadzanie zagospodarowanie podczyszczanie. *Wydawnictwo Seidel-Przywecki*. Piaseczno, 115 s.
[21] Rossman, LA (2007) Storm Water Management Model. User's Manual. Version 5.0 : EPA/600/R 05/040. *Water Supply and Water Resources Division, National Risk Management Research Laboratory, U.S. Environmental Protection Agency*. Cincinnati, 265 p.
[22] Tkachuk A., Pilipaka L., Azizova A. (2018) Optimization of City Water Supply Networks on Their Structural and Functional Analysis Base. *International Journal of Engineering & Technology*, № 7 (3.2), P. 680-685.
[23] Weitman D., Weinberg A., Goo R. (2008) Reducing Storm water Costs through LID Strategies and Practices. *Low Impact Development for Urban Ecosystem and Habitat Protection.* Washington: American Society of Civil Engineers, P. 1-10.