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ANDRZEJ KOTOWSKI*, PATRYK WÓJTOWICZ**

THE NEW METHOD FOR LIMITING OUTFLOW FROM STORM OVERFLOWS

On the basis of experimental data, a mathematical model is proposed to describe the performance of a side weir with a high-elevated overflow edge and equalizing unit located behind the weir chamber with throttled outflow by system built from elbows or bends. A method for hydraulic dimensioning of a non-conventional storm overflow with a new throttling system is described. The paper presents the results of model studies, including the procedure as well as an example of hydraulic dimensioning of proposed non-conventional storm overflows with flow stilling chambers. A new method of throttling the outflow to the wastewater treatment plant is proposed as an alternative to commonly used throttling devices.

DENOTATIONS

d_{th} – throttling pipe diameter, m,
D – inlet channel diameter, m,
D_o – outlet channel diameter, m,
Fr – Froude number,
g – gravitational acceleration, m/s^2,
h_m – effective height of liquid layer above the weir edge, m,
i_t – terrain slope, ‰,
l_{ax} – axial length of piping, m,
l_{cr} – length of the weir crest, m,
l_e – equivalent length of piping, m,
l_s – length of the stilling chamber, m,
n_{id} – initial dilution coefficient,
p – height of the weir crest, m,
Q – rate of outflow to recipient, m^3/s,
q_{fi} – flush intensity, dm^3/s·ha,
q_r – coefficient of flow division in the weir,
Q_{in} – maximal rate of inflow, m^3/s,

* Institute of Environmental Protection Engineering, Wrocław University of Technology, pl. Grunwaldzki 9, 50-370 Wrocław, Poland; e-mail: andrzej.kotowski@pwr.wroc.pl

** Faculty of Environmental Engineering, Wrocław University of Technology, Wybrzeże Wyspiańskiego 27, 50-370 Wrocław, Poland; e-mail: patryk.wojtowicz@pwr.wroc.pl

$Q_{\lim}$ – limiting rate of inflow to the weir, m^3/s,
Q_o – rate of outflow to treatment plant, m^3/s,
Q_r – effective rate of inflow to the weir, m^3/s,
Q_s – flow rate of municipal sewage, m^3/s,
Re – Reynolds number,
λ – friction factor,
μ – side weir discharge coefficient,
υ – mean flow velocity in cross-section, m/s,
ζ – minor loss coefficient,
ψ – mean run-off coefficient.

1. INTRODUCTION

Volumetric separators of rainfall sewage (generally referred to as storm overflows) are used in combined sewage systems [1]–[4] mainly to protect a wastewater treatment plant against hydraulic overloading during torrential rains. Another benefit of using these separators is that they allow the interceptor size to be reduced. When used in semi-separate and separate sewage systems, storm overflows are to discharge a certain portion of rainfall sewage to the recipient streams or directly to the environment. At maximal rate of sewage inflow (Q_{in}) the object of the weir is to split this discharge, in assumed proportions, into two streams: one (denoted by Q) entering, directly or indirectly, the recipient, and the other (referred to as Q_o) passing to the wastewater treatment plant (figure 1).

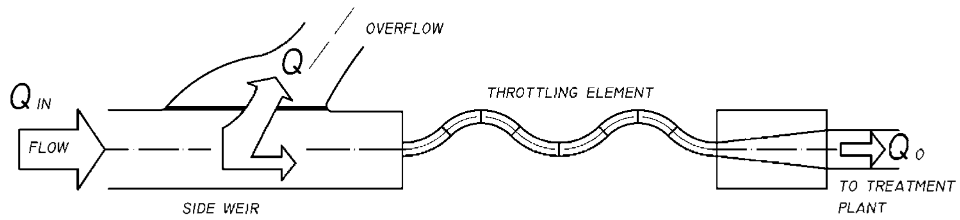

Fig. 1. Situation of storm overflow with throttled outflow to sewage system

In hydraulic terms, the storm overflows functioning as volumetric separators of rainfall sewage can be divided into two types: those with a low weir edge, with no devices that throttle the outflow of the sewage to the treatment plant [5] and those with a high weir edge (with throttling devices) [6]. Side weirs with a high overflow edge and controlled (throttled) outflow are preferred to the side weirs with a low weir edge. The adopted conditions of weir operation can be maintained via the regulating units that are in use now, i.e., throttling pipes of appropriate length and diameter, gates with adjustable openings, or hydrodynamic regulators of various types, with properly selected flow characteristics. The application of throttled sewage outflow facilitates the use of sewer system retention and reduces the frequency of overflow operation throughout the year, even at the limit rate of flow ($Q_{\lim}$). Throttling the outflow (Q_o) to the treatment plant at maximal swollen inflow of the sewage ($Q_{in\ sw}$) to the unit upgrades the hydraulic efficiency of the side weir and thus enables the length of the overflow to be shortened, in most instances to several meters.

The paper presents the examples of model test results, the procedure for (and an example of) hydraulic dimensioning of non-conventional storm overflows with high-weir edges and stilling chambers behind the weir chamber. A new method of throttling the outflow to the treatment plant is proposed.

2. METHODS FOR THE REGULATION OF SEWER STRUCTURES

Depending on the throttling method, the available structures for controlling the flow rate in sewer structures, like storm overflows, settling or retention tanks, can be divided into two groups:

- linear throttling devices, e.g., rectilinear sections of pipelines with an adequate diameter, length and wall roughness, referred to as throttling pipes,
- local throttling devices such as orifices, reducing pipes, gates and gate valves, hydrodynamic flow regulators, etc.

The new flow rate control method involves pressure flow of the sewage through a properly selected system of elbows or bends made of plastics (e.g., PVC-U or PP). In principle, it applies to the group of local throttling devices, which display the features of advanced structures (without reducing the internal area of the channel), but are much cheaper than the conventional (steel) throttling pipes. A major drawback of the throttling pipe is its considerable length, which often reaches several dozen meters. Equivalent to the resistance of a throttling pipe of such considerable length, the hydraulic resistance of an appropriately selected throttling system occurs within several meters of axial length (or piping length), when the system consists of (n) elbows, or within approximately a dozen meters, when the system is made of (n) bends. The selected throttling system consists of sinusoidal waves made of elbows with relative radius of curvature $R/d \le 1$ in the case of slightly polluted liquids. Bends with a curvature radius $R/d > 1$ are used for highly polluted liquids (raw sewage), both (elbows and bends) with the same diameter (d) as that of the throttling pipe. This becomes evident if we compare the ("local") head loss of such systems and the corresponding frictional head loss in the conventional throttling pipe:

$$\zeta_{(n)} \frac{\upsilon^2}{2g} = \lambda(Re) \frac{l_e}{d} \frac{\upsilon^2}{2g} . \tag{1}$$

Hence, the equivalent (substitute) length of a rectilinear throttling pipe becomes

$$l_e = \frac{\zeta_{(n)}}{\lambda(Re)} d . \tag{2}$$

Since the local loss coefficient (the table) of the systems built from (n) elbows or (n) bends ($\zeta_{(n)}$) is many times higher than the friction factor of the rectilinear throttling pipe ($\lambda(Re)$), the equivalent length (l_e) of the throttling pipe must be many times the axial length ($l_{ax(n)}$) of the system made of (n) elbows or (n) bends ($l_e >> l_{ax(n)}$).

Table

Loss coefficient $\zeta_{(n)}$ of a throttling system built from (n) plastic elbows or (n) segmental bends (β_i) [7]

System		A	B	C
Version		R/d = **4.25**	R/d = **2.25**	R/d = **1.75**
No.	Description β_i (β_{sum})	System built from bends β = **15°**	System built from bends β = **30°**	System built from bends β = **45°**
		ζ / l_{ax} / (l_{piping})	ζ / l_{ax} / (l_{piping})	ζ / l_{ax} / (l_{piping})
1	4 bends or elbows 90° (360°)	**0.90** 26.7d (17.0d)	**1.0** 14.1d (9.0d)	**1.5** 11.0d (7.0d)
2	4 bends 60°(240°)	**0.65** 17.8d (14.7d)	**0.83** 9.4d (7.8d)	–
3	4 bends 45°(180°)	**0.47** 13.3d (12.0d)	–	**0.91** 5.5d (4.9d)
4	4 bends 30°(120°)	**0.30** 8.9d (8.5d)	**0.44** 4.7d (4.5d)	–
5	8 bends or elbows 90°(720°)	**1.9** 53.4d (34.0d)	**2.0** 28.3d (18.0d)	**3.0** 22.0d (14.0d)
6	8 bends 60°(480°)	**1.4** 35.6d (29.4d)	**1.6** 18.8d (15.6d)	–
7	8 bends 45°(360°)	**1.0** 26.7d (24.0d)	–	**2.2** 11.0d (9.9d)
8	8 bends 30°(240°)	**0.65** 17.8d (17.0d)	**0.82** 9.4d (9.0d)	–
9	12 bends 60°(720°)	**2.1** 53.4d (44.2d)	**2.3** 28.3d (23.4d)	–
10	12 bends 45°(540°)	**1.5** 40.1d (36.1d)	–	**3.2** 16.5d (14.8d)

The example throttling system that consists of two sinusoidal waves built from eight alternatingly connected (segmental) bends ($n = 8$) with central angles $\beta_i = \beta = 45°$ (e.g., made of PVC-U) and a radius of curvature $R = 1.75d$ yields a "local" loss $\zeta_{(8x45°)} = 2.2$ (table 1: system 7, version C). With a Reynolds number of, e.g., Re = 1 000 000, we obtain a friction coefficient $\lambda(Re) = 0.012$ (PVC-U pipeline [8], [9]) and an equivalent length of the rectilinear throttling pipe $l_e = (2.2/0{,}012)d = 183.3d$. This is approximately 17 times the axial length ($l_{ax} = 11d$) of the example throttling system and about 19 times the (real) length of the piping: $l_{\text{piping}} = n \cdot \sin\beta_i°R = 9.8d$ (the table).

3. PROCEDURE AND EXAMPLE

3.1. INITIAL ASSUMPTIONS

The available methods for hydraulic dimensioning of side weirs with throttled outflow suffer from simplifications, such as the omission of variations in the liquid level ordinate along the length of the overflow and the assumption of a constant value for the weir discharge coefficient (depending on weir crest shape), by analogy with sharp-crested, non-submerged rectangular weirs. Furthermore, in the light of technological research [10], [11], the widespread standardization of such structures – without flow stilling chambers behind the side weir – is not a recommendable trend since the bottom wastes enter the recipient via overflow. It would be advisable to establish a new construction standard for the weirs under consideration and carry out relevant model tests [12]. It has been assumed that the shape of the cross-section for the overflow is identical to that of the stilling chamber – up to the level of the intersection axis. Above this level, the cross-sections are rectangular in shape (above $D/2$ the channel is circular, and above $2H_c/3$ the channel is egg-shaped, etc.), and have a width $b = D$. The stilling chamber length behind the side weir is $l_s = 2b = 2D$. The computational scheme for the weir is shown in figure 2.

The computational method proposed involved the following procedure:

- at the limiting rate of inflow (Q_{lim}) to the weir, an appropriate height of the overflow crest (p) is adopted, taking into account the hydraulic and operating conditions for the occurrence of (swollen) subcritical flow in the vicinity of the weir, and thereafter the throttling element (with an appropriate value of the loss coefficient $\zeta_{(n)}$ consisting of (n) segmental bends, connected in series, of an axial length $l_{ax} = l_{th}$) is selected;
- at the maximal rate of flow (Q_{in}) the desired flow division at the overflow is specified: for the assumed rate of outflow to the treatment plant $Q_o \in \langle 1.1Q_{\text{lim}}; 1.2Q_{\text{lim}}\rangle$, the hydraulic losses of the previously selected throttling system and the height of the liquid layer above the weir edge (h_c) at the overflow end are calculated according to ΔH_o;

• for the outflow to recipient $Q = Q_{in} - Q_o$ and the calculated height h_c, the necessary length of weir crest (l_{cr}) is iteratively determined by discrete change in the height of liquid layer above the weir edge h_a – at the beginning of the weir.

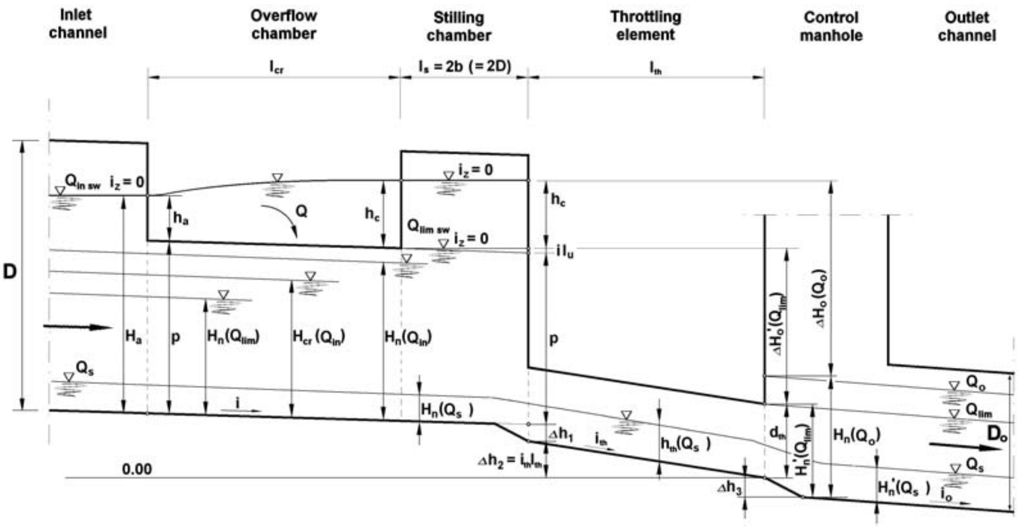

Fig. 2. Computational scheme for a non-conventional side weir with throttled sewage outflow to the treatment plant

3.2. INPUT PARAMETERS OF THE WEIR

Example drainage area is $A_{dr} = 100$ ha, having mean run-off coefficient $\psi = 0.3$ and terrain slope at the overflow location $i_t = 1.0‰$. The initial dilution coefficient $n_{id} = 3$ and the flush intensity $q_{fi} = 15$ dm^3/s·ha. Flow rate of municipal sewage into weir $Q_s = 0.150$ m^3/s, flow rate of rainfall sewage $Q_{in\,\max} = 2.500$ m^3/s.

Limiting rate of combined sewage inflow: $Q_{\lim} = Q_s + n_{id}Q_s = 0.600\ \mathrm{m^3/s}$ or $Q_{\lim} = Q_s + q_{fi}\psi A_{dr} = 0.600\ \mathrm{m^3/s}$. Effective (maximal) rate of inflow to the weir: $Q_r = Q_s + Q_{in\,\max} = 2.650\ \mathrm{m^3/s}$.

Inlet channel. For the effective rate of inflow $Q_{in} = 2.650$ m^3/s and the assumed bottom slope $i = 1.0$ ‰, a concrete channel of a diameter $D = 1.80$ m is selected. Calculations were carried out using the nomographs for the Manning equation (circular channels) at $n = 0.013$ s/m$^{1/3}$. But use can also be made of plastic channels; then the nomographs for the Darcy–Weisbach, Colebrook–White and Bretting equations at $k = 0.4$ mm [13] are applied.

Standard depth of flow for calculated flow rates: $H_n(Q_s) = 0.25$ m ($\upsilon = 0.73$ m/s), $H_n(Q_{\lim}) = 0.49$ m ($\upsilon = 1.00$ m/s), $H_n(Q_{in}) = 1.12$ m ($\upsilon = 1.60$ m/s).

Critical depth of flow: $H_{cr}(Q_{in}) = 0.77$ m ($\upsilon = 2.65$ m/s).

The weir crest. The height of the weir crest (p) should be assumed by including the following hydraulic conditions [6]:

$p > H_n(Q_{\lim})$, (3)

$p > H_{cr}(Q_{in})$, (4)

$p + h_a > H_n(Q_{in})$. (5)

In addition, consideration should be given to the following operating conditions [11], [14]:

$p > 0.6D$ (or H_c), (6)

$\upsilon_{\min}(Q_{\lim sw}) \geq 0.30$ m/s. (7)

The initial assumption was: $p = 1.30$ m ($p = 0.72D$), and condition (7) of minimal flow velocity in the overflow chamber was checked at the limiting flow rate, swelled to the height (p) of the weir crest. After substitution of the numerical values we obtain:

$$\upsilon_{\min}(Q_{\lim sw}) = \frac{Q_{\lim}}{\left[\frac{\pi D^2}{8} + \left(p - \frac{D}{2}\right)D\right]} = 0.30 \text{ m/s}.$$

If condition (7) is not satisfied, then the weir crest height (p) must be lowered.

3.3. CALCULATED PARAMETERS OF THE WEIR AT THE FLOW RATE Q_s

Throttling element. At the flow rate ($Q_s = 0.150$ m^3/s) of the municipal sewage, it is necessary to select the diameter of the throttling element (d_{th}) (consisting of n bends β_i, the axial length being $l_{ax(n\beta)} = l_{th}$) and to assume the depth of flow $h_{th}(Q_s)$, taking into account the following restrictions: $d_{th\,\min} = 0.20$ m, $h_{th}/d_{th} \leq 0.6$, $\upsilon_s(Q_s) \geq 1.0$ m/s.

If we assume that $d_{th} = 0.60$ m and $h_{th} = 0.30$ m, then the flow area of Q_s becomes $\pi d_{th}^2/8 = 0.141$ m^2, and the sewage flow velocity in the throttling system: $\upsilon_s(Q_s) = 1.06$ m/s.

The requirement for water level compensation (at the inlet to the throttling system) at the flow Q_s often necessitates the lowering of the throttling element bottom (at its beginning), in relation to the stilling chamber bottom (at its end), by the value of Δh_1 (figure 2): $\Delta h_1 = h_{th}(Q_s) - H_n(Q_s) = 0.30 - 0.25 = 0.05$ m.

3.4. CALCULATED PARAMETERS OF THE WEIR AT THE FLOW RATE $Q_{\lim}$

Choice of the throttling system. Using the Bernoulli equation, derived for the sections immediately before the inlet and just behind the outlet of the throttling system (at the flow rate $Q_{\lim}$), we obtain the following hydraulic loss equation (figure 2):

$$il_s + p + \Delta h_1 + \Delta h_2 - d_{th} \equiv \Delta H_o^{\mathrm{I}}(Q_{\lim}) = \zeta_{in}\frac{\upsilon_{\lim}^2}{2g} + \zeta_{(n\beta)}\frac{\upsilon_{\lim}^2}{2g} + \zeta_{\mathrm{out}}\frac{\upsilon_{\lim}^2}{2g}, \tag{8}$$

where $\upsilon_{\lim}$ is average velocity of flow at $Q_{\lim}$: $\upsilon_{\lim} = 4Q_{\lim}/(\pi d_{th}^2)$, m/s; ζ_{in} stands for the local loss coefficient at the inlet of the throttling system [15]: $\zeta_{in} = 0.45$; $\zeta_{(n\beta)}$ denotes the hydraulic ("local") loss coefficient for the throttling system consisting of (n) segmental bends (connected in series) with central angles β_i and axial length $l_{ax} = l_{th}$ (table 1), ζ_{out} is local loss coefficient at the outlet of the throttling system, assumed equal to the Coriolis coefficient: $\alpha = 1 + 2.93\lambda - 1.55\,\lambda^{3/2}$; for $\lambda \in \langle 0.0017;0.031\rangle$ we have $\alpha \in \langle 1.05;1.08\rangle$. Assumption for plastics [16]: $\zeta_{\mathrm{out}} = \alpha = 1.05$.

The required value of the loss coefficient $\zeta_{(n\beta)}$ should be calculated from the rearranged relation (8), neglecting the component Δh_2 in the first approximation. Thus we have:

$$\zeta_{(n\beta)} > \frac{\left(il_s + p + \Delta h_1 - d_{th} - (\zeta_{in} + \zeta_{\mathrm{out}})\dfrac{8Q_{\lim}^2}{g\pi^2 d_{th}^4}\right)}{\dfrac{8Q_{\lim}^2}{g\pi^2 d_{th}^4}}.$$

Substituting numerical values for the above equation we obtain $\zeta_{(n\beta)} > 1.78$. The throttling system characterized by the loss coefficient $\zeta_{(8x45°)} = 2.2 > 1.78$ was selected from table 1 (system no. 7, according to version C). The system consists of $n = 8$ bends connected in series, having central angles $\beta_i = 45°$ and a radius of curvature $R = 1.75d$, so the axial length is $l_{ax(8x45°)} = (n\beta_i^o/360°)2\pi R = 11.0d = 6.6$ m (the piping length being: $l_{\mathrm{piping}(8x45°)} = n{\cdot}\sin\beta_i^o R = 9.9d = 6.0$ m).

For the above mentioned parameters of the throttling system, the difference in the invert height between the inlet and outlet of the system ($\Delta h_2 = i_{th}l_{th}$, figure 2), which is equal to the hydraulic head loss at the flow rate Q_s, can be calculated from the following equation: $\Delta h_2(Q_s) = \zeta_{(8x45°)}\upsilon_s^2/2g = 0.13$ m.

Hence the invert slope of the throttling element (along the axial length) equals $i_{th} = \Delta h_2/l_{ax(8x45°)} = 0.13/6.6 = 0.020$ (the real slope along the piping length being: $0.13/6.0 = 0.022$).

The correction of the weir crest height (p) – from the rearrangement of equation (8) – includes the previously neglected component Δh_2 and takes the form:

$$p_{(1)} = (\zeta_{in} + \zeta_{(n\beta)} + \zeta_{\mathrm{out}})\frac{8Q_{\lim}^2}{g\pi^2 d_{th}^4} - il_{\mathrm{s}} - \Delta h_1 - \Delta h_2 + d_{th}. \tag{8a}$$

After substitution of numerical values $p_{(1)} = 1.27$ m.

The conditions (3)–(7) must be verified for the new value of $p_{(1)}$ (i.e., $\upsilon_{\min(1)}(Q_{\lim\,sw})$

= 0.31 m/s). If the conditions are not met, we have to select (from table 1) a throttling system with another value of the loss coefficient $\zeta_{(n\beta)}$, and then to calculate a new value of the head $\Delta h_2(Q_s)$ and the height of the weir crest $p_{(i)}$ using equation (8a).

Outlet channel (to the sewage treatment plant). For the flow rate $Q_{\lim}$ = 0.600 m^3/s and selected invert slope i_o = 1.0‰, the diameter of the outlet channel D_o = 1.0 m and the normal depth of flow $H_n^{\mathrm{I}}(Q_{\lim})$ = 0.67 m ($H_n^{\mathrm{I}}(Q_s)$ = 0.30 m) were selected. Mostly, in the case of

$$H_n^{\mathrm{I}}(Q_{\lim}) > d_{th}\,, \tag{9}$$

it is necessary to lower the inlet channel with respect to the assumed reference datum level by the value of $\Delta h_3 = H_n^{\mathrm{I}}(Q_{\lim}) - d_{th} = 0.07$ m (figure 2). If $H_n^{\mathrm{I}}(Q_{\lim}) < d_{th}$, we have to correct the parameter: i_o or D_o.

3.5. CALCULATED PARAMETERS OF THE WEIR AT THE FLOW RATE Q_{in}

Height of the liquid layer above the weir edge (h_c) at the end of the overflow. At maximal inflow to the weir $Q_{in} = Q_s + Q_{in\ \max} = 2.650$ m^3/s we should assume a value for the rate of outflow Q_o through the throttling element which is by 10 to 20% higher than that of $Q_{\lim}$ [15]:

$$Q_o \in \langle 1.1Q_{\lim}; 1.2Q_{\lim} \rangle\,. \tag{10}$$

For the assumed value of Q_o it is necessary to determine the normal depth of flow $H_n(Q_o)$ in the outlet channel, and then to calculate the hydraulic head loss $\Delta H_o(Q_o)$ in the throttling system (by virtue of the modified equation (8)):

$$\Delta H_o(Q_o) = (\zeta_{in} + \zeta_{(n\beta)} + \zeta_{\mathrm{out}})8Q_o^2 / g\pi^2 d_{th}^4 = 1.12 \text{ m}$$

assuming that $Q_o = 1.15Q_{\lim} = 0.690$ m^3/s. The normal depth of flow in the outlet channel is $H_n(Q_o)$ = 0.75 m (υ_o = 1.05 m/s).

The height of the liquid layer above the weir edge at the end of the weir should be calculated in terms of the following equation:

$$h_c = H_n(Q_o) + \Delta H_o(Q_o) - (i \cdot l_s + p + \Delta h_1 + \Delta h_2 + \Delta h_3)\,, \tag{11}$$

hence h_c = 0.35 m.

Length of the weir crest (l_{cr}). The length of the weir crest (single-sided) must meet the condition $l_{cr} \le 4D$. If $l_{cr} > 4D$ there is a need to apply a double-sided weir. The required length of the weir crest should be calculated iteratively, either by virtue of the dimensionless form of the differential equation of motion [12], or in terms of the following equations:

$$l_{cr} = \frac{Q}{(2/3)\,\mu\sqrt{2g}\ h_m^{3/2}}\,, \tag{12}$$

where:

Q is the rate of flow through the side weir: $Q = Q_{in} - Q_o$,

μ is the side weir discharge coefficient ($\mu \in \langle 0.50;0.60\rangle$):

$$\mu = 0.64 - 0.052q_r + 0.0088L_0 + 0.035W_0 - 0.075\,Fr_0 - 0.065K_0\,, \tag{13}$$

q_r is the coefficient of flow division in the weir ($q_r \in \langle 0.5;1\rangle$): $q_r = Q/Q_{in}$,

L_0 is a relative length of the weir crest ($L_0 \in \langle 1.8;5.1\rangle$): $L_0 = l_{cr}/H_a$, where $H_a = p + h_a$,

W_0 is a relative height of the liquid layer above the overflow edge in the initial part of the weir ($W_0 \in \langle 0.13;0.35\rangle$): $W_0 = h_a / H_a$,

Fr_0 is the Froude number in the initial cross-section of the overflow chamber ($Fr_0 \in \langle 0.1;0.5\rangle$), where $Fr_0^2 = Q_{in}^2/(A_0^2 g H_a)$ by definition [12],

K_0 is the shape factor of the channel bottom in the initial part of the overflow chamber ($K_0 \in \langle 1;1.2\rangle$): $K_0 = bH_a / A_0$,

h_m is an effective (weighted average) height of the liquid layer above the weir edge:

$$h_m = h_a + \frac{3}{5}(h_c - h_a)\,, \tag{14}$$

and

$$h_c = h_a + 0.9\frac{\alpha_{in}\upsilon_a^2}{2g}\,, \tag{15}$$

υ_a is the velocity of flow in the initial part of the overflow chamber:

$$\upsilon_a = \frac{Q_{in}}{A_0}\,, \tag{16}$$

A_0 is a cross-sectional area of flow in the initial part of the overflow chamber,

$$A_0(Q_{in\,sw}) = \frac{\pi D^2}{8} + \left(p + h_a - \frac{D}{2}\right)D\,, \tag{17}$$

α_{in} is a kinetic energy (Coriolis) coefficient in the inlet channel before the weir: $\alpha_{in} = 1.15$ for cylindrical channels ($\alpha_{in} = 1.20$ for prismatic channels).

In the side weir, the sewage swells along the weir length. It is therefore necessary to assume such a value for the height of the liquid layer above the weir edge at the beginning of the overflow (h_a) that will be by several centimetres lower than the value of h_c ($1.05 \le h_c/h_a \le 1.4$). In the first approximation, we assumed that $h_{a(1)} = 0.27$ m, hence the cross-sectional area of flow $A_{0(1)}$ ($Q_{in\ sw}$) in the initial part of the overflow chamber (by virtue of equation (17)) equals 2.42 m^2, and the flow velocity in the initial part of the overflow chamber (in terms of equation (16)) becomes $\upsilon_{a(1)} = 1.10$ m/s.

The height of the liquid layer above the edge of the weir at its end (by virtue of equation (15)) equals $h_{c(1)} = 0.33$ m.

From the calculated hydraulic head loss in the adopted throttling system (at the flow rate $Q_o = 0.690$ m^3/s) it follows that $h_c = 0.35$ m is higher than $h_{c(1)} = 0.33$ m from the first approximation (the difference being greater than 1 cm). Thus, in the second approximation it is necessary to adopt a new value for the height of the liquid above edge of the weir in its initial part (h_a), e.g., $h_{a(2)} = 0.29$ m. Finally, in the second approximation we shall have $A_{0(2)}(Q_{in\ sw}) = 2.46$ m^2, $\upsilon_{a(2)} = 1.08$ m/s and $h_{c(2)} = 0.35$ m.

The effective value of the height of the overflowing liquid in the side weir (by virtue of equation (14)) equals $h_m = 0.33$ m. Hence, assuming primarily that the value of the weir discharge coefficient is $\mu = 0.60$ for the stream $Q = Q_{in} - Q_o = 1.960$ m^3/s, the initial length of the side crest of weir (equation (12)) becomes $l_{cr(1)} = 5.84$ m. With the preliminarily established length of the weir crest ($l_{cr(1)}$), it is possible to calculate a value of the weir discharge coefficient, after having determined the dimensionless values of the factors q_r, L_0, W_0, Fr_0 and K_0 (similarity numbers [6]):

$q_r = Q / Q_{in} = 0.74$ (condition $q_r \in \langle 0.5;1 \rangle$);

$L_{0(1)} = l_{cr(1)} / H_a = 3.74$ (condition $L_0 \in \langle 1.8;5.1 \rangle$);

$W_0 = h_a / H_a = 0.19$ (condition $W_0 \in \langle 0.13;0.35 \rangle$);

$Fr_0 = Q_{in} /(A_0 \sqrt{gH_a}) = 0.28$ (condition $Fr_0 \in \langle 0.1;0.5 \rangle$);

$K_0 = bH_a / A_0 = 1.14$ (condition $K_0 \in \langle 1.0;1.2 \rangle$);

thus, the weir discharge coefficient calculated (by virtue of equation (13)) for $l_{cr(1)} = 5.84$ m equals $\mu_{(1)} = 0.54$ and enables the correction of side edge length of the weir (in terms of equation (12)) $l_{cr(2)} = 6.48$ m. After successive approximation, for $L_{0(2)} = l_{cr(2)}/H_a = 6.48/(1.27 + 0.29) = 4.15$, we obtain: $\mu_{(2)} = 0.55$ and $l_{cr(3)} = 6.37$ m.

Since the difference in the length of the weir between $l_{cr(2)}$ and $l_{cr(3)}$ (from the last approximation) is comparatively small (amounting to 10 cm), the calculations can be thought of as being completed. Hence, we adopted the single-sided weir of the side edge length $l_{cr} = 6.4$ m (the condition $l_{cr} \le 4D$ has been satisfied, $6.4 \le 7.2$).

4. CONCLUSIONS

The improved standard of side storm weir construction involves a new method of throttling, more effective compared to the classical throttling pipe. It also formulates principles of dimensioning, which allow the frequency of overflow dumps to be limited (owing to the retention capabilities of the channels situated above the overflow), and the quality of the storm dumps to be notably improved, thus protecting the sewage treatment plant against hydraulic overload.

The new construction standard and dimensioning method for side weirs (single and double-sided) with throttled outflow to the sewage treatment plant apply to the structures:

- with high-elevated overflow crests of practical shape (fulfilling the conditions: $p > H_n$ $(Q_{\lim})$; $p > H_{cr}(Q_{in})$; $p > 0.6D$ (or H_c); $p + h_a > H_n(Q_{in})$) and non-submerged operation condition ($H_n(Q) < p$),
- with a cylindrical cross-section of the overflow chamber up to the level of the horizontal axis, and rectangular (of a width $b = D$) above this level and with stilling chambers after the overflow (of the length $l_s = 2b = 2D$),
- with throttled sewage outflow, via in-series systems of segmental bends (in the form of sinusoidal waves), having a noticeably shorter piping length (compared to the equivalent length of the throttling pipe) and limiting the outflow rate (Q_o) of the sewage to the treatment plant to the predetermined value.

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NOWA METODA OGRANICZANIA ODPŁYWU Z PRZELEWÓW BURZOWYCH

Przedstawiono metodę hydraulicznego wymiarowania niekonwencjonalnych przelewów burzowych z odpływem dławionym za pomocą układu kolan bądź łuków. Na podstawie danych eksperymentalnych zaproponowano model matematyczny opisujący działanie przelewu burzowego o wysokich krawędziach przelewowych i z komorą uspokajającą umieszczoną za przelewem oraz nowy sposób dławienia odpływu do oczyszczalni ścieków jako alternatywę dla stosowanych urządzeń dławiących. Przedstawiono wyniki badań modelowych wraz z procedurą i przykładem obliczeniowym.