Environment Protection Engineering

Vol. 31 2005 No. 2

ANDRZEJ KOTOWSKI*, HENRYK SZEWCZYK**, ANDRZEJ PAWLAK*

HYDRAULIC MODELLING OF SEWAGE FLOW IN SEPARATORS OF PETROLEUM DISTRIBUTORS

The present paper deals with the hydraulic assessment of functions of petroleum derivative separators integrated with the settling tank and with the by-pass of the coalescence chamber (inside the separator). A hydraulic model of its functioning was developed – with a free discharge of liquid sewage from the settler chamber through a side overflow (into the by-pass channel) and with a throttled discharge of sewage from the coalescence chamber by means of siphon elbows with specific diameters (into the discharge channel). Calculations carried out confirmed that the sewage flow distribution inside the unit is as assumed by the manufacturer, both at the rated and maximum flow rates. The paper also specified the minimum bottom slopes of the inlet and outlet channels for a typical sewage separator, at the maximum stream.

DENOTATIONS

A – cross-sectional flow area, m^2,
b – liquid table width in the channel, m,
C – Chezy coefficient, m$^{1/2}$/s,
d – siphon diameter, m,
D_n – channel diameter, m,
g – acceleration due to gravity, m/s^2 ,
$h_{(n)}$ – standard depth of flow in the channel, m,
$h_{(p)}$ – height of sewage layer (over the overflow), m,
h_v – velocity head, m,
I – channel bottom slope,
l – length of overflow crest, m
M – height of overfall edge, m
n – channel roughness coefficient, $n = 0.013$ s/m$^{1/3}$,
R_h – hydraulic radius ($R_h = A/U$), m,
q_V – volume flow rate, 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 and andrzej.pawlak@pwr.wroc.pl

** Institute of Power Engineering and Fluid Mechanics, Wrocław University of Technology, Wybrzeże Wyspiańskiego 27, 50-370 Wrocław, Poland. E-mail: henryk.szewczyk@pwr.wroc.pl

$Q_{\text{by-pass}}$ – flow rate through the by-pass channel at $Q_{\max}$, m^3/s,
$Q_{\max}$ – maximum flow rate in the unit, m^3/s,
Q_n – rated flow rate in the unit, m^3/s,
Q_{storm} – flow rate through the coalescence chamber at $Q_{\max}$, m^3/s,
U – wetted perimeter, m,
V – sewage velocity, m/s,
z – hydraulic loss head, m,
α – Coriolis coefficient,
λ – friction factor,
μ – flow ratio,
ζ – local loss factor.

SUBSCRIPSTS

by-pass – channel by-pass,
in – channel inlet,
out – channel outlet.

1. INTRODUCTION

The present paper deals with the hydraulic assessment of functions performed by the petroleum derivative separators integrated with the settling tank, fulfilling the European standards: EN 858-1 and EN-858-2, with regard to separating units for light liquids, and DIN 1999, with regard to the volume of petroleum derivative dumps for separators with coalescence filters.

The separators – those with by-passes of the coalescence filters inside the unit – are used mainly for preliminary purification of rainfall sewage. The principle of operation of such units is based on two phenomena: gravitational separation of solids and floatation of petroleum derivatives. The separators in question consist of three principal elements contained in a steel (horizontal) drum, namely [1]:

- Settler chamber used for trapping and collecting solids.
- Coalescence chamber used for capturing and collecting petroleum derivatives and equipped with the automatic discharge cut-off system (at siphon elbows) preventing the collected substances from entering the environment.
- By-pass inside the unit used for relieving the coalescence chamber in case of torrential rains.

Such separators are used for preliminary purification of rainfall sewage from the road surface, car-parks, car handling yards, etc. The maximum volume of rainfall sewage inflow to the unit ($Q_{\max}$) is determined on the basis of the PN-EN 752-4 standard. The size of the unit is to be matched to this value assuming that about 1/5 of this flow rate is subject to a constant purification process from petroleum derivatives and deposits, including the so-called first wave – highly contaminated rainfall sewage washed from the drainage surface. This flow rate is known as rated flow (Q_n). The remaining inlet flow, i.e. about 4/5 $Q_{\max}$, only after preliminary purification from deposits is transferred directly to the by-pass channel (by-passing the coalescence chamber) and is introduced into the outlet channel.

2. PURPOSE AND SCOPE OF WORK

The present paper deals with hydraulic calculations which verify the sewage flow distribution assumed by the manufacturer inside the unit, that is, at the coalescence chamber inlet at Q_n, and that discharged through the by-pass channel at $Q_{\max}$. The calculations refer to a standard type of series of separators – with a side by-pass inside the unit, with detailed discussion of a standard type dimension, namely the separator with $Q_n = 55$ dm^3/s (figure 1).

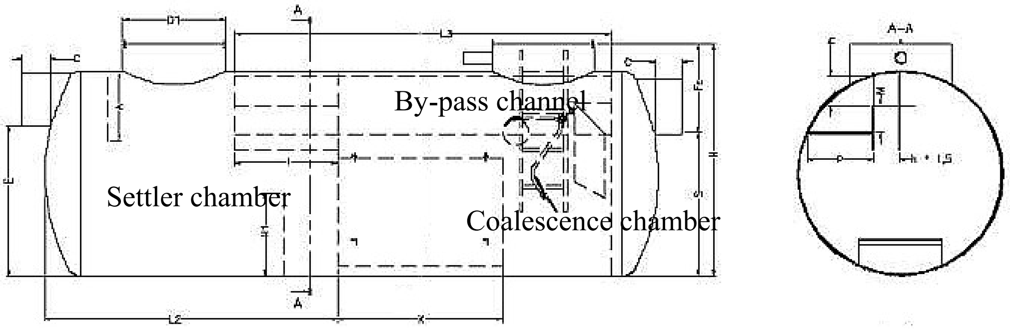

Fig.1. Separators with side (internal) by-pass

A typical separator with a side by-pass (figure 1) is designed for the rated flow of $Q_n = 55$ dm^3/s and the maximum flow of $Q_{\max} = 5Q_n = 275$ dm^3/s. The inner diameter of the inlet and outlet channels is $D_n = 500$ mm with the difference in ordinates of their bottom $h_o = E - S = 5$ cm (designations of structural variables are given in figure 1). Functioning of the unit:

- At the rate of flow below $Q_n = 55$ dm^3/s the rainfall sewage flows through the settling tank chamber and then through the coalescence chamber thus achieving full pre-cleaning (up to the degree assumed).
- At the rate of flow equal to Q_n the sewage in the settling tank chamber is swollen up to the ordinate of the side overflow threshold on the by-pass channel, that is, up to the height of M above the outlet channel located on the bottom level of the by-pass channel; rated discharge from the coalescence chamber takes place through two siphoned elbows, I.D. $d = 0.20$ m (with automatic arm-type closures) into the outlet channel, thus ensuring the assumed pre-cleaning process of this sewage flow.

• At the inlet flow rate $Q_{\max}$ the sewage in the settling tank chamber is swollen above the side overflow threshold height in the by-pass channel and is discharged at the rate of about Q_n through the coalescence chamber and at the rate of about $Q_{\max} - Q_n$ through the by-pass channel, directly to the receiver.

The by-pass channel has a total length L_3 and its side overflow edge (inside the settling tank chamber) has a length l. It is located at the side drum wall. The shape of the cross section area of this channel resembles an equilateral triangle of the height of $M + h_{(p)}$. The lengthwise slope of the by-pass channel is equal to zero (figure 1).

3. MODELLING OF SEWAGE FLOWS IN THE UNITS

3.1. INLET AND OUTLET CHANNELS

The flow rate q_V of sewage in the inlet and outlet channels was calculated from the de Chezy formula:

$$q_V = CA\sqrt{IR_h}\,, \tag{1}$$

where the velocity (Chezy) coefficient C was determined from the Manning–Strickler formula:

$$C = \frac{1}{n}\sqrt[6]{R_h}\,. \tag{2}$$

For a given depth of flow in the channel, the bottom slope required was calculated from the formula:

$$I = \frac{n^2 q_V{}^2}{A^2\sqrt[3]{R_h{}^4}}\,. \tag{3}$$

For instance, for a separator of type dimension $Q_n = 0.055$ m^3/s, assuming a full depth of flow in the inlet channel: $h = D_n = 500$ mm at the maximum inlet flow of rainfall sewage, $Q_{\max} = 5Q_n = 0.275$ m^3/s, the required bottom slope in the inlet channel shall be $I_{\text{in}} = 0.0055$. For such a bottom slope in the inlet channel, the normal depth of flow ($h_{(n)}$) at the rated flow of $Q_n = 0.055$ m^3/s may be determined from the equation:

$$\frac{nq_V}{\sqrt{I}} = A(h)R_h{}^{2/3}(h)\,, \tag{4}$$

where:

$$A(h) = \frac{D_n^{\ 2}}{4}\left[\arccos\left(1-\frac{2h}{D_n}\right)-\left(1-\frac{2h}{D_n}\right)\sqrt{1-\left(1-\frac{2h}{D_n}\right)^2}\,\right], \tag{4a}$$

and

$$R_h(h) = \frac{D_n}{4}\left[1-\frac{\left(1-\frac{2h}{D_n}\right)\sqrt{1-\left(1-\frac{2h}{D_n}\right)^2}}{\arccos\left(1-\frac{2h}{D_n}\right)}\right]. \tag{4b}$$

Solving equation (4) yielded $h_{(n)} = 0.15$ m.

For the outlet channel, assuming the depth of flow at about $h = 0.9D = 0.45$ m (the highest hydraulic efficiency of the channel), then for $Q_{\max} = 0.275$ m^3/s, from equation (3) one can obtain $I_{\mathrm{out}} = 0.0049$, and normal depth of flow at $Q_n = 0.055$ m^3/s will be then $h_{(n)} = 0.16$ m (from equations (4), (4a) and (4b)).

3.2. RATE OF FLOW DIVIDING AT NOMINAL DISCHARGE

Voluminal discharge q_V from the coalescence chamber of the separator (figure 1) under rated operating conditions (without involving the by-pass channel) was calculated from the equation:

$$q_V = n_s \frac{\pi d^2}{4}\sqrt{\frac{2g\Delta h}{\zeta + \frac{\lambda l_r}{d}}}, \tag{5}$$

where:

n_s – the number of outlet siphons in the coalescence chamber ($n_s = 2$),

d – the siphon pipe diameter ($d = 0.20$ m),

Δh – the difference between the levels of sewage in the settling tank chamber and and in the outlet channel,

ζ – the coefficient of flow resistance in the siphon (inlet into the pipe, 90° elbow and outlet from the pipe, $\zeta = 2.5$),

λ – the friction factor of the siphon pipe ($\lambda = 0.03$), length l_r.

The height of Δh was determined by assuming that the sewage may be swollen at most to the high overflow edge, on the inlet side of the separator, while on the outlet side its level is determined by the depth of the outlet channel during discharge with at most flow rate. On the basis of equation (5) one can determine the position of the overflow edge so that for $q_V \le Q_n$ the discharge would take place only through the coalescence chamber of the separator. Only when the voluminal discharge would exceed the rated value, some sewage would flow to the by-pass channel.

3.3. FLOW DIVISION AT THE MAXIMUM DISCHARGE

The voluminal discharge of sewage flowing through the coalescence chamber at $Q_{\max} = 5Q_n$ was also calculated from equation (5). The value of Δh was determined by assuming that:

- the inlet channel may be filled to the full (however, remaining an open channel in a hydraulic sense) – during maximum inflow of sewage at the rate of 5 Q_n,
- the outlet channel may be filled at most up to 0.9 D_n – also at 5 Q_n.

In view of the fact that in the separators under consideration the outlet channel lies deeper than the inlet channel by $h_0 = 5$ cm, it was assumed:

$$\Delta h = h_0 + 0.1\ D_n. \tag{6}$$

Assuming $q_V = Q_{\max}$ in formula (5) one can determine the actual rate of flow Q_{storm} through the coalescence chamber during torrential rainfalls (that is, at $Q_{\max} = 5\ Q_n$). This results in a rate of discharge through the by-pass channel – $Q_{\text{by-pass}} = Q_{\max} - Q_{\text{storm}}$ (the table). Later this would be a basis for checking the length of the side overflow into the by-pass channel. During the maximum rate of flow to the separator, in formula (5) one should assume another difference in levels Δh than during the rated flow since different levels at the inlet and outlet are established. Now these are determined by the depths of flow in the inlet and outlet channels (the position of the overfall edge has no influence). This means that it is not possible to select the parameters of that hydraulic unit so that under the rated (Q_n) and maximum ($Q_{\max}$) flow conditions the discharge through the separator coalescence chamber would be the same ($Q_n \neq Q_{\text{storm}}$). Only the rates of flow close each to other in their order of magnitude may be obtained. This applies to all hydraulic units which are not provided with adjustable control features, such as valves or gates which contain variable hydraulic resistance.

The problem of calculating the discharge of sewage through the by-pass channel is much more complicated than those discussed previously. The liquid flows to this channel through side overflows which are examined, above all, in cases where liquid flows towards the discharge [2]–[4]. The case under examination is an opposite one: the liquid is collected from the environment and by flowing over the side overfall crest reaches the by-pass channel. Therefore, one should distinguish two parts in the by-pass channel: the part containing an overflow and open channel (both parts have a horizontal bottom). The overflow part is extremely complicated taking account of its hydraulics. If it is to be treated as a regular overflow, then the flow rate may be calculated from the formula [2]–[6]:

$$q_V = \frac{2}{3}\mu l \sqrt{2gh_{(p)}^3}\ , \tag{7}$$

where:

μ – the flow ratio ($\mu = 0.6$),

l – the length of weir crest,

$h_{(p)}$ – the height of liquid layer over the overflow edge.

In such a case, calculations should be based upon the value of $h_{(p)}$ equal to the height of a cut-out in the by-pass channel wall (figures 1, 2, and 3). Since this height is somewhat smaller than the water rise head in the settling tank chamber (this results from the drum wall curvature), the difference found here (up to 2 cm) shall be treated as a value covering the losses in the inlet siphon to the overflow – which are not taken into account in calculations. However, during torrential rainfalls when the inlet channel is filled up ($q_V = 5Q_n$) one can also treat the inflow to the by-pass channel as a run-off through a large opening with partial throttling. Then the voluminal discharge should be calculated from the formula:

$$q_V = \mu l h_{(o)} \sqrt{2gz}\ , \tag{8}$$

where:

μ – the coefficient of discharge in large opening ($\mu = 0.9$),

l – the length of opening equal to the weir crest length,

$h_{(o)}$ – the height of opening – analogy to the liquid layer height over the overflow,

z – the pressure loss head during flow of liquid at q_V rate.

The numerical value of the discharge coefficient was selected from the literature [7] – as for a large opening with a similar throttling effect.

In the existing, hydraulically ambiguous, situation, the calculations were carried out according to formula (7) and it was checked whether the loss head z, calculated from formula (8), does not exceed the pressure difference available. The value of the height z is still to be calculated. To that end, one should first calculate the depth of flow in the by-pass channel at the end of overflow which is the beginning of the open channel. This may be carried out by means of the differential equation of motion [3], [4], [8]

$$\frac{dh}{dx} = -\frac{\dfrac{U}{C^2A^3}q_V^2}{1-\dfrac{\alpha b}{gA^3}q_V^2}\ , \tag{9}$$

where:

$h = h(x)$ – the depth of channel at the distance of x from the origin of coordinates (figure 3),

$\alpha = 1.3$ – the Coriolis coefficient ($\alpha \in [\,1.1, 1.3\,]$ [3], [5], [8]),

$b = b(h)$ – the width of liquid level at the distance of x,
$A = A(h)$, $C = C(h)$, $U = U(h)$ – the functions of the depth h in the by-pass channel.

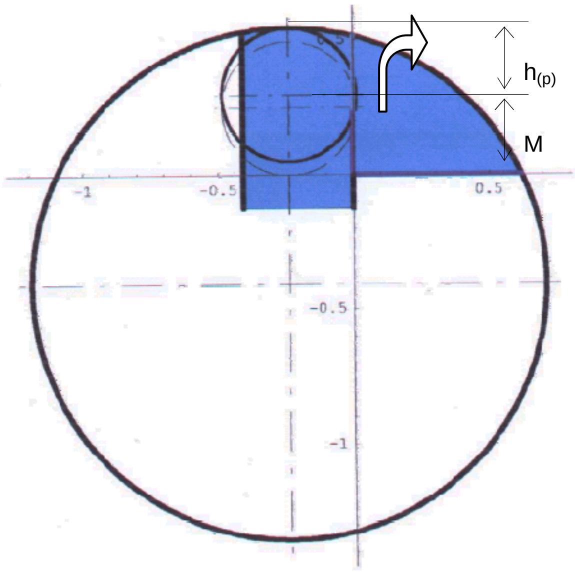

Fig. 2. Cross-section through the inlet siphon, overflow and by-pass channel of the separator type dimension $Q_n = 0.055$ m^3/s – with the flow rate $Q_{\max} = 5Q_n$

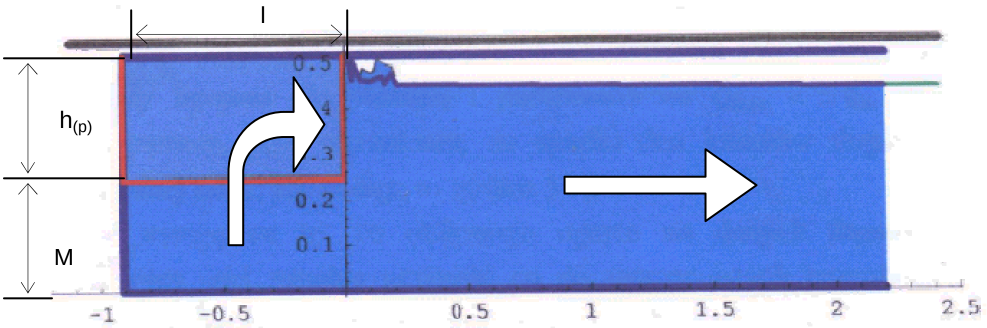

Fig. 3. Lengthwise profile of the by-pass channel in the separator type dimension $Q_n = 0.055$ m^3/s – with the flow rate of $Q_{\max} = 5Q_n$

Equation (9) should satisfy the initial condition that at the end of channel ($x = L_k$), the height h is 0.9 D_n (where $L_k = (L_3 - l)$ – length of channel from the end of overflow towards movement of liquid). The liquid level falls down towards the direction of flow (the bottom is horizontal) so, in order to estimate its maximum elevation, we should calculate the hypothetical value of this elevation in the middle of the overflow

($x = -l/2$), assuming a constant voluminal discharge rate in the channel and in the overflow in formula (9). In this way, the average sewage level elevation shall be estimated in advance and the value of z shall be estimated at the end, thus ensuring a certain safety margin for calculations. After performing calculations it appeared that the loss of energy determined from formula (8) was lower than the available pressures determined by means of the estimates described above.

Table

Calculated operating data of the separator type dimension $Q_n = 0.055$ m^3/s

Q_n	I_{in} (min)	I_{out} (min)	Q_{storm} (max)	$Q_{\max} = 5Q_n$	$Q_{\text{by-pass}}$
					(5)–(4)
dm^3/s	mm/m	mm/m	dm^3/s	dm^3/s	dm^3/s
1	2	3	4	5	6
55	5.5	4.9	58	275	217

It should be finally noted that an inlet siphon is before the overflow to the by-pass channel (figures 1 and 2) which is responsible for certain hydraulic losses. Discharge cross-sectional area of this siphon is 3/2 of the discharge cross sectional area through the overflow, and the velocity head (h_v) in this siphon for a typical separator is 1.8 cm:

$$h_v = \frac{V^2}{2g} = \frac{Q^2{}_{\text{by-pass}}}{2g\left(\frac{3}{2}h_{(p)}l\right)^2}. \tag{10}$$

A part of this energy (hardly assessable) should be considered lost but, as mentioned above, in calculations a certain energy margin was left for such losses.

Overall results of hydraulic modelling of the sewage flow distribution inside a typical unit, i.e. calculations verifying the functions of the separator type dimension $Q_n = 0.055$ m^3/s (with a side by-pass), are presented in the table.

4. CONCLUSIONS

1. On the basis of hydraulic calculations performed it was found that a typical separator (of a series of type with a side by-pass channel inside the unit) would ensure the rated flow capacity (Q_n) of the unit in the period when the by-pass is not working and the maximum flow capacity $Q_{\max} = 5Q_n$ and when a specific volume of sewage flows through the by-pass, provided that the slopes of the inlet and outlet channel bottoms are at least such as those shown in the table.

2. One should note that hydraulic modelling of sewage flows was based on reference data and one can be fairly sure as to the actual distribution of streams inside the

units under consideration only after performing model testing – as always in such cases. In this case, one should test the performance of the side overflow into the by-pass channel.

REFERENCES

[1] *Katalog stalowych separatorów substancji ropopochodnych*, 2004.

[2] KOTOWSKI A., *Modellversuche über Regenüberläufe mit gedrosseltem Ablauf*, GWF Wasser/Abwasser, 1990, Jg. 131, H. 3, S. 108–114.

[3] KOTOWSKI A., *Podstawy wymiarowania bocznych przelewów burzowych z rurą dławiącą*, Oficyna Wydawnicza Politechniki Wrocławskiej, Wrocław, 1998.

[4] KOTOWSKI A., *Verfahren und Beispiel der Dimensionierung des verbesserten Regenüberlaufs in Mischwasserkanalisation*, GWF Wasser/Abwasser, 2001, Jg. 142, H. 12, S. 843–850.

[5] MITOSEK M., *Mechanika płynów w inżynierii środowiska*, Oficyna Wydawnicza Politechniki Warszawskiej, Warszawa, 1997.

[6] SAWICKI J., *Przepływy ze swobodną powierzchnią*, Wydawnictwo Naukowe PWN, Warszawa, 1998.

[7] RABINOVICZ E.Z., *Gidravlika*, Gosudarstviennoe Izdatelstvo Fiziko-Matemat. Literatury, Moskva, 1961.

[8] JEŻOWIECKA-KABSCH K., SZEWCZYK H., *Mechanika płynów*, Oficyna Wydawnicza Politechniki Wrocławskiej, Wrocław, 2001.

MODELOWANIE HYDRAULICZNE PRZEPŁYWÓW ŚCIEKÓW W SEPARATORACH SUBSTANCJI ROPOPOCHODNYCH

Celem pracy była ocena hydrauliczna działania separatorów substancji ropopochodnych zintegrowanych z osadnikiem oraz bocznym kanałem obejściowym (*by-passem*) komory koalescencyjnej (wewnątrz separatora). Opracowano model hydrauliczny działania urządzenia ze swobodnym odpływem ścieków z komory osadnika przez boczny przelew (do kanału *by-passa*) oraz dławionym odpływem ścieków z komory koalescencyjnej za pomocą kolan syfonowych o określonej średnicy (do kanału odpływowego). Wykonane obliczenia potwierdziły założony przez producenta rozdział przepływów ścieków wewnątrz urządzenia zarówno podczas przepływu nominalnego, jak i maksymalngo. Określono też minimalne wymagane spadki dna kanałów dopływowego i odpływowego dla przykładowego separatora ścieków przy strumieniu maksymalnym.