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DESIGN FLOWS FOR PRESSURE SEWERS

Existing methods to estimate design flows of sewage in pressure mains have been compared. It was found that the number of simultaneously running identical cavity displacement (grinder) pumps (m out of n), connected to a pressure main is better described by the binomial (for $n \leq 30$) and normal distributions (for $n > 30$) than by the Poisson probability distribution. A new probabilistic approach to estimate the design flow which takes into account distinguishable (not identical) sources of sewage, disposing to a pressure main, is presented. All possible states of pumps, equal to the number of variations with repetitions, are considered. The method is elaborated for small systems equipped with cavity displacement pumps which have relatively steep characteristic curves. A new formula for the peaking factor, combining averaging over time and over ensemble (number of dwellings) is also presented. The new formula can be applied for individual homes, dwellings in apartment blocks and villages, serving by cavity displacement and/or centrifugal pumps.

1. DESCRIPTION OF THE PRESSURE SEWERAGE SYSTEM

Raw sewage from one or a few individual houses is flowing by gravity through building sewers of diameter 100–150 mm to a pump sump (vault). The pump sump is typically constructed as a cylindrical tank 60–90 cm in diameter. The storage volume between pump *turn-on* and *turn-off* levels for an individual dwelling is 30–150 dm^3. A reserve volume between pump's turn-on level and the alarm (high) levels, provided for emergency cases (pump failure, power black-outs), is equal to 30-100 dm^3 per person. Pumps of two types are typically applied in the system: cavity displacement (progressing cavity or semi-positive displacement pumps) and centrifugal pumps. The maximum head for a low-pressure pump is 35–42 m (0.35–0.4 MPa) and for a high-pressure up to 100 m of water column (1.0 MPa). Cavity displacement pumps have much steeper characteristic curves than the centrifugal pumps, thus hydraulic calculations are easier for the former ones. They function also as check valves, lowering the risk of back-flow.

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The pressure sewerage is dedicated for both flat and hilly terrains, sparsely populated areas with high ground water levels and rocky soils.

There are several methods of dimensioning pressure sewerage systems. The design flows can be defined as max*imum flow rates expected to occur once or twice per day, and are used to size the pressure sewer mains* [1]. The authors divided the design approaches into two categories: the rational method and the probability method.

2. RATIONAL METHODS

The rational method is called also a conventional or the peak effluent method. It is based on the following assumptions:

- the peak flow depends on the number of identical (typical) dwellings, e.g. inhabited by 2–4 persons,
- the minimum design flow is taken as a single pump capacity,
- specific water usage (sewage production) per an equivalent person is equal to 150–230 dm^3/d·e.p.,
- there is no storage in the sewer (elasticity of water and pipe wall material as well as gas bubbles in the pipe are neglected).

Summarizing six linear and non-linear empirical equations, the following simplified linear equation for both pressure and vacuum sewers can be obtained [1]:

$$Q = 76 + 1.9\, N_{ed} \tag{1}$$

where: Q – design peak flow, dm^3/min; N_{ed} – number of equivalent dwellings.

Judging by the dimension, one would assume that the formula concerns the maximum flow averaged over 1 min.

A similar equation has been presented recently in other American guidelines according to which the peak flow used in the pipeline design for alternative systems shall be equal to or greater than the following [2]:

$$Q = 57 + 1.9 N_{ed} \qquad \text{or} \qquad 57 + 0.57 N_{ep} \tag{2}$$

where: Q – design peak flow as an event that lasts about 15 min, dm^3/min; N_{ed} – number of equivalent consumption dwellings.

The constant in Eq. (2) is lower than that in Eq. (1) probably due to the lower specific water usage (sewage production) in the USA after 15 years. Anyhow, the difference in design flows calculated by both equations decreases as the number of dwellings increases. The discrepancy could be also brought about the time of averaging, however, the time can hardly be equal to 15 min, as for one dwelling the design flow would then generate the sewage volume 15×(57 + 1.9) = 883 dm^3, which is comparable to the whole daily outflow.

The ratio of the maximum to a long-term (yearly) average value is called a peaking factor. There are several empirical formulae for calculation of the peaking factors. One of the most popular one [3] is based on Harmon's (1918) measurements of conventional sewage flow, which is still used in the USA [4, 2]:

$$PF = \frac{Q}{Q_{ad}} = \frac{18 + \sqrt{N_{ep}}}{4 + \sqrt{N_{ep}}} = \frac{4 + \sqrt{N_{ep}} + 14}{4 + \sqrt{N_{ep}}} = 1 + \frac{14}{4 + \sqrt{N_{ep}}} \tag{3}$$

where: Q – peak flow of domestic sewage, dm^3/s, Q_{ad} – average daily sewage flow, dm^3/s, N_{ep} – population in thousands, i.e. number of equivalent persons ($0.1\cdot10^3 < N_{ep} < 100\times10^3$). For $N_{ep} = 100$, $PF = 4.24$ and for $N_{ep} = 100\,000$, $PF = 2.0$.

The peak flow Q in Eq. (3) is approximately equal to an hourly maximum rate of domestic sewage flow.

For small communities ($N_{ep} < 7000$) an empirical formula [5] is recommended:

$$PF = \frac{6.51}{N_{ep}^{0.38}} \tag{4}$$

where: N_{ep} – population in thousands.

Another rational formula on the design flow can be written as follows [6]:

$$Q = 1.5qN_{ep} \geq Q_p \tag{5}$$

where: q – specific sewage production, dm^3/s·e.p.; N_{ep} – number of equivalent persons; Q_p – capacity of a single pump.

The lower limit of validity of Eq. (5) is equal to the capacity of a single pump or two pumps in the case of a pressure main. The specific sewage flow recommended in [6] $q = 0.005$ dm^3/s·e.p. Thus, for the specific daily sewage production 150 dm^3/d·e.p., it corresponds to the total peaking factor $PF = 1.5\times86\,400\times0.005/150 = 4.3$, suggesting that the design flow is approximately equal to an hourly maximum flow. For $N_{ep} = 10$ and $Q_p = Q_{p\ \min} = 27$ dm^3/min Eq. (5) gives $Q = 27$ dm^3/min and for $N_{ep} = 1000$, $Q = 450$ dm^3/min; these values are 2.3 and 1.4 times lower flow values, respectively than those calculated by Eq. (2). It can be explained by a longer averaging time (an hour vs. a quarter), a higher sewage production in the USA than in Germany and hgher flow variability at lower numbers of served persons.

3. PROBABILITY METHODS

Under specific conditions, the flow in a pressure sewer main can be treated as a random variable, ranging between zero (all pumps waiting) and the maximum value (all pumps running). In many cases, the probability distribution of the peak flows can be approximated by a theoretical one. Then the design peak flow can be determined in terms of exceedance probability, typically in the form:

$$Q = Q_a + f_{Pr} S_{Qa} \tag{6}$$

where: Q_a – mean peak flow, f_{Pr} – frequency factor, i.e. a function dependent on the exceedance probability level Pr for a given probability distribution, S_{Qa} – standard deviation of the mean peak flows. Frequency factors can be taken from statistical tables. For the same exceedance probability $Pr < 0.1$, the normal distribution provides lower f_{Pr} values than the Gumbel distribution ($f_{PrN} < f_{PrG}$), thus the latter one is more conservative.

In pressure sewers, the flow variability depends mainly on the number of sewage sources, changeability of water usage and/or duration of the averaging time intervals.

The model of indoor water use assuming that residential water demands occur as a non-homogeneous Poisson-rectangular-pulse process was developed [7]. Later on, peaking factors of water demand were described using similar assumptions [8]. There the maximum demand is treated as a rare event, assuming that for a longer averaging period, pulses with different starting times may overlap. Then, the total water use is the sum of the joint intensities from the coincident pulses. Finally, they obtained a general formula of the form:

$$PF_e = A + \frac{B}{\sqrt{N}} \tag{7}$$

where A – a measure of the time weighted average total demand by a typical dwelling during the daily period of maximum water use, B – variability in water use at a typical dwelling, N – number of dwellings.

It has occurred that the peak indoor water use follows a Gumbel extreme value type I distribution. For the indoor use only, Eq. (7) can be applied to sewage flows, as well.

Equations (3), (4) and (7)] exemplify averaging over ensemble. Equations to calculate flow rates averaged over different time intervals have also been published. Based on observations, one can estimate [9]:

$$Q_{at} \cong \frac{Q_{am}}{\sqrt{t_a^*}} \tag{8}$$

where Q_{at} – mean peak demand averaged over time period t, Q_{am} – mean peak demand averaged over 1 min, t_a^* – number of minutes in averaging time period.

Assuming that the ergodicity principle (averaging over ensemble is equivalent to averaging over time) can be applied to our problem, and combining Eqs. (6), (7) and (8) [10] leads to the following formula:

$$PF_{t\&e} = \frac{Q}{Q_{ad}} = \frac{Q_{at} + f_{Pr} S_{Qat}}{Q_{ad}} = \left(1 + f_{Pr} C_{v1}\right)\sqrt{\frac{1440}{t_a^* + N_{ed} - 1}} \tag{9}$$

where Q_{ad} – mean daily flow, $C_{vt} = S_{Qat}/Q_{at}$ – the coefficient of variability of flow (for a typical dwelling $C_{v1} = S_{Qam}/Q_{am}$ = 0.3–0.5 [9] at t_a = 1 min; t_a^* = 1), t_a^* – number of minutes in averaging time period ($1 \le t_a^* \le 1440$), N_{ed} – number of dwelling units over which the total flow is averaged.

Equation (9) is valid for $t_a^* + N_{ed} \le 1441$. Plots of Eq. (9) for Pr = 1% (f_{PrG} = 3.14) and C_{v1} = 0.4 as well as of Eqs. (3) and (4) are shown in Fig. 1. The discrepancy between them is probably due to the routing of sewage "waves" in gravity sewers, which is inherent in empirical Eq. (3). The higher values obtained from the empirical formula for N_{ed} > 400 may be contributed to more differentiated objects in reality than in our theoretical model.

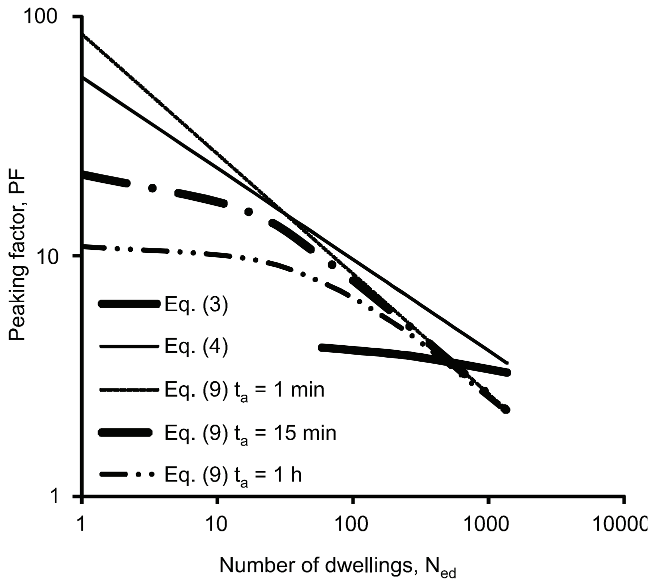

Fig. 1. Dependences of average flows
on averaging time and number of dwellings

Another German guidelines ATV-DVWK [11] recommend for N_{ep} < 5000 ($N_{ed} \le 1250$) the values of PF = 6–9, which are in a good agreement with Eq. (9) for t_a = 1.0 h (60 min) up to 100 dwellings.

More sophisticated is the probability (probabilistic) method (called also a statistical method) based on the probable number of simultaneously running pumps [12–14]. Its basic assumptions are as follows:

- discharge rate (capacity) of every individual pump is constant,
- pumps work only for a few minutes per the peak hour, thus there is a low probability that all connected pumps are running at the same time,
- the probability that a pump is running is equal to the ratio of the pump discharge rate and the inflow to the pump well [15],
- design flow is the product of the number of pumps running during a peak period and the pump discharge rate.

It was noted that the statistical method is better suited for pumps characterized by vertical or near vertical head-discharge curves, such as cavity displacement pumps [1]. Centrifugal pumps, having gradually sloping head-discharge curves, provide discharges dependent on pressures in the sewer main, thus it is hard to determine their discharge rates Q_p and run time periods t_r. Nevertheless, the sewage volume to be pumped is the same in both cases provided that the time period is sufficiently long, i.e. its contribution in the maximum hourly flow (or even in the maximum quarterly flow) is the same, independent of the pressure in the sewer main.

Simultaneous work of all pumps connected to a pressure main is extremely rare, therefore sizing the main for such a case would be hydraulically and economically unjustified. It can be assumed that the cumulative probability that m pumps out of n are running can be calculated according to the discrete distribution as follows [13, 12]:

$$F_x(m) = \sum_{x=0}^{m} P_{(x \le m)} = \sum_{x=0}^{m} \frac{n!}{x!(n-x)!} p^x (1-p)^{n-x} \tag{10}$$

where $x = 0, 1, 2, ..., m$ – number of simultaneously running pumps, p – probability that a pump is running in the peak hour.

The above described binomial distribution has been erroneously named the Poisson one [15, 16]. The value of p they recommend to estimate as:

$$p = \frac{Q_{in}}{Q_p} \tag{11}$$

where Q_{in} – the inflow to a single pump sump during 10 h of sewage production.

The analysis [17] for a constant influent rate in the peak hour showed that the best agreement with empirical data published by EPA [18] gave the equality $Q_{in} = Q_{h\max}$. It is equivalent to $p = t_r/3600$ s (where t_r – expected pump running time during the peak hour) and is consistent with Hunter's approach in which the peak hour is only considered. The probability p is not influenced by the operation storage (called also working volume) in the pump sump, as confirmed using a simulation model for a main with 100 pumps connected [19]. To avoid sewage septicity it is recommended to apply the smallest possible working volume. The storage volume from the pump turn-on level to the alarm level can be calculated based on the analysis of inflows lasting shorter than 1 h [1].

The exceedance probability level Pr = 1– $F_x(m)$ is called the failure rate [20]. The design probability is dependent on the type of pump but usually $Pr \approx 0.1$ [15]. Some authors assumed that for design purposes it is reasonably to take $Pr = 0.05$ [14]; a relevant curve is shown in Fig. 2 for $p = 0.1$. More precisely, the curve should be replaced by an ascending stepwise line, but then one can rarely find the exact value $Pr = 0.05$.

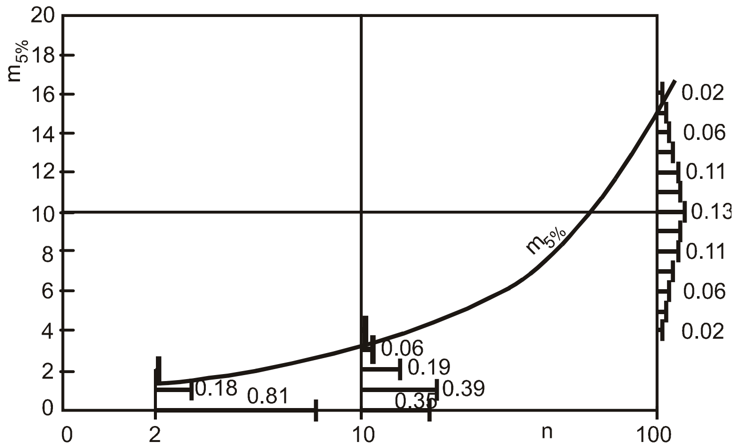

Fig. 2. Probable (F = 0.95) numbers of simultaneously running pumps $m_{5\%}$ out of n pumps

A similar approach was applied to describe black water flow rates in high-rise residential buildings in Hong-Kong [20]. Application of Eq. (10) is troublesome at hgh n ($n \geq 70$) due to difficulties in calculation of the factorial value. For $n > 30$, when p is not close to zero nor unity, i.e. for $np > 5$ and $n(1-p) > 5$, the binomial probability distribution is equivalent to the normal one and then [20]:

$$m_{Pr} = np + f_{Pr}\sqrt{np(1-p)} \tag{12}$$

where m_{Pr} is the number of simultaneously running pumps, which probability of exceedance is equal to Pr, np – mean value, f_{Pr} – frequency factor, i.e. the number of standard deviations from the mean to the maximum value ($f_{5\%N} = 1.645$ for $Pr = 0.05$ and $f_{0.1\%N} = 3.09$ for $Pr = 0.001$), $(np(1-p))^{1/2}$ – standard deviation.

Assuming constant capacity of the running pumps, one can calculate the maximum flow as

$$Q_{Pr} = m_{Pr}Q_p = \left[np + f_{Pr}\sqrt{np(1-p)}\right]Q_p = npQ_p\left(1 + f_{Pr}\sqrt{\frac{1-p}{np}}\right) \tag{13}$$

A comparison of design flow rates calculated for $Q_{p\min} = 0.45$ dm^3/s (the minimum available pump capacity), $n = N_{ed} = N_{ep}/3.5$, $p = 0.1$ and the specific daily sewage production = 150 dm^3/d·e.p. is shown in Fig. 3. The peak flows estimated using Eq. (13) are comparable with design flows calculated using rational equations (Eqs. (1), (4) and (5)). Equation (9) for the maximum peak flow averaged over 1 min shows intermediate results, and the Legg formula (Eq. 4) gives the highest values.

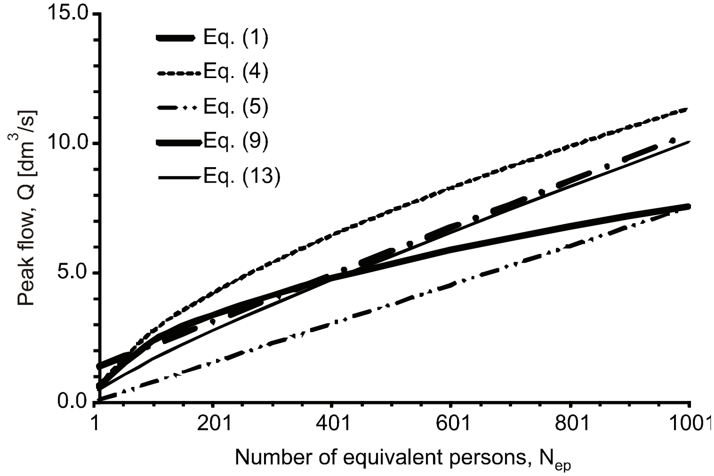

Fig. 3. Designed flows according to various models

There is also a question of spatial distribution of the running pumps. The worst case is when running pumps are located at the initial part of a given reach but probability of such a case is low. It is equal to a reciprocal of the number of combinations of the number of running pumps out of all installed ones: $m!(n - m)!/n!$, e.g. probability that $m = 5$ running pumps out of $n = 50$ is localized at the beginning of the reach is equal to 5×10^{-7}. Thus, more probable is an uneven distribution along the whole reach; consequently the head losses are lesser than in the worst case. An equivalent flow rate for calculation of a mean head loss (using the Darcy–Weisbach formula) over a reach with an initial flow Q_i and a final flow Q_f in the turbulent regime can be roughly calculated using

$$Q_{eq} \approx \sqrt{\frac{Q_i^2 + Q_i Q_f + Q_f^2}{3}} \approx Q_i + 0.55\left(Q_f - Q_i\right) \tag{14}$$

The accuracy of Eq. (14) increases with the number of running pumps m connected to the reach. Reduction in friction head loss using Eq. (14) for evenly spaced pumps can be significant, e.g. for $m > 10$, the head loss is reduced by more than 60% comparing to that calculated using Q_f only.

When n is large and p is small (i.e. $np < 7$), a good approximation to the binomial probability distribution is also the Poisson probability distribution. Considering variability of flow in time, the averaging interval $\{0, t\}$ can be regarded as made up of a large number of subintervals of the length Δt (say $\Delta t = 1$ s), each treated as an independent trial (measurement). The probability that m pumps will run simultaneously in time interval $\{0, t\}$ is

$$p(m,t) = \frac{(\lambda t)^m}{m!} e^{-\lambda t} \tag{15}$$

where λ – running intensity, s^{-1}, $\lambda = np/t = nQ_{h\max}/(tQ_p)$, t – time, s.

Introducing $\lambda t = nQ_{h\max}/Q_p$ into Eq. (15) gives

$$p(m,t) = \frac{1}{m!}\left(\frac{nQ_{h\max}}{Q_p}\right)^m \exp\left(-\frac{nQ_{h\max}}{Q_p}\right) \tag{16}$$

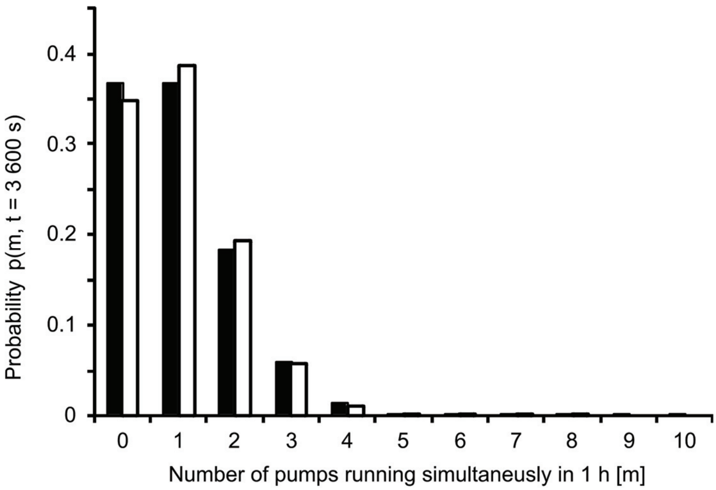

Fig. 4. Binomial (white bars) and Poisson (black bars) probability distributions for $n = 12$, $p = 0.1$, $\lambda = 1.2/3600 = 0.00033$ s^{-1}

Figure 4 shows a comparison of binomial distribution with the Poisson one. This discrepancy is due to a small number of pumps ($n = 12$). Application of the Poisson law needs fulfilling the following assumptions [21]:

- The probability of a random inflow in any subinterval t is independent (in the probability sense) of previous inflows (*independent increments*) and of absolute time (*stationarity*).

• The probability of an inflow in an interval Δt is $\lambda\Delta t$, where λ is a positive constant; the probability of no inflow $1 - \lambda\Delta t$, i.e. no more than one additional inflow is allowed in any single interval Δt (*unit jump*).

The assumption of stationarity is hardly fulfilled in practice, however one can consider the peak hours only and assume that there is no daily nor seasonally variability. Anyhow, it seems that instead of the Poisson distribution it is more correctly to use the binomial (for $n \leq 30$) and normal distributions (for $n > 30$). Another disadvantage of Eq. (15) is its implicit form.

4. FLOWS FROM NOT IDENTICAL OBJECTS

The case when pumps and/or their running phase durations are not equal can be observed in practice if several types of pumps and/or highly differentiated numbers of equivalent persons per one sump (connection) are connected to a common pressure main. In such a case we should assume that the connections are distinguishable. It is still possible to estimate the probability of simultaneous work of the pumps but the procedure is more time-consuming than in the case of indistinguishable pumps. In that case, instead of combinations (Eq. 10), one should consider variations with repetition. The number of variations with repetitions of the r-th order from n different elements (pumps) is given by:

$$V_r^n = r^n \tag{17}$$

In our case $r = 2$ because any given pump can be in two states: on and off. For all possible variations it holds:

$$\sum_{j=1}^{2^n} \prod_{i=1}^{n} \left[p_i^{\delta_{ij}} \left(1 - p_i\right)^{1-\delta_{ij}} \right] = 1 \tag{18}$$

where j – successive number of variation, $\delta_{ij} = 0$ when the i-th pump is waiting and $\delta_{ij} = 1$ when the i-th pump is running.

To find the probability distribution of the flow in the pressure sewer main, one has to divide the whole range of flows ($0 \leq Q_s \leq \sum Q_{pi}$) into l classes ($k = 0, 1, 2 \ldots, l$). Then the probability of occurrence of a flow in the sewer Q_s in the k-th class will be

$$P\left(Q_s = Q_{sk}\right) = \sum_{j \in J} \prod_{i=1}^{n} \left[p_i^{\delta_{ij}} \left(1 - p_i\right)^{1-\delta_{ij}} \right] \tag{19}$$

where

$$J = \left\{ j : \sum_{i=1}^{n} \left(\delta_{ij} Q_i = Q_{sk} \right) \right\}$$

The expected (mean) flow is

$$\overline{Q_s} = \sum_{j=1}^{2^n} \prod_{i=1}^{n} \left[p_i^{\delta_{ij}} Q_i^{\delta_i} \left(1 - p_i\right)^{1-\delta_{ij}} \right] \tag{20}$$

To demonstrate application of Eqs. (19) and (20), an example is given as follows. There are 8 different pump installations with parameters shown in Table 1.

Table 1

Input data for the example

Source (pump) No.	1	2	3	4	5	6	7	8
Capacity, Q_{pi}, dm^3/s	8	7	6	5	4	3	2	1
Probability, p_i	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

Totally, we have to consider $2^8 = 256$ variations of the pump states. Results of application of Eq. (19) are shown in Fig. 5. It follows from the figure that the median $Q_{50\%} = 11$ dm^3/s and the maximum $Q_{5\%} = 22$ dm^3/s. of Eq. (20) gives the mean flow $\bar{Q}_s = 12$ dm^3/s. The mass probability function is multimodal. The probability that all 8 pumps will run simultaneously, giving $Q = \Sigma Q_{pi} = 36$ dm^3/s, is equal to 0.04% but the least probable is $Q = 35$ dm^3/s (0.01%).

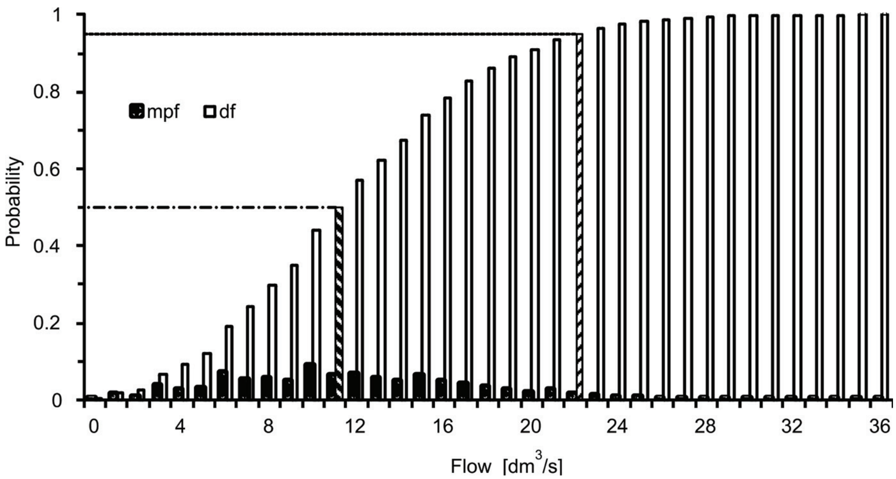

Fig. 5. Mass probability function (mpf) and distribution function (df = F) for flows Q_s generated by 8 different pumps (described in Table 1) in the pressure sewer main

The method described above is practically (using personal computers) acceptable for a relatively small number of pumps ($n < 20$). For $n = 20$, the number of variations exceeds 1o^6 and it increases in geometric progression.

4. CONCLUSIONS

- Values of peaking factors decrease upon increasing the averaging time and the number of served persons. For identical objects (e.g. dwellings) they converge to a constant, same value.
- The number of simultaneously running pumps (m out of n) is better described by the binomial (for $n \leq 30$) and normal distributions (for $n > 30$) than by the Poisson distribution.
- Probability that a pump will run in a peak hour is equal to the ratio of maximum hourly flow to the pump capacity,
- The expected value of a pump run time in a peak hour is independent of its sump working volume, thus the volume should be as small as possible to avoid sewage septicity,
- Probabilistic methods give results comparable with those obtained using rational methods and additionally they provide for information on frequency of exceedance of a given flow value, thus allow for a risk assessment.

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SYMBOLS

C_{vt} – coefficient of variability of flow averaged over time interval t
f_{Pr} – frequency factor
F – cummulative probability or distribution function
m_{Pr} – number of simultaneously running pumps, which probability of exceedance is equal to Pr
n – total number of pumps
N_{ep} – number of equivalent persons
N_{ed} – number of equivalent dwellings
p – probability that a pump is running in the peak hour
PF – peaking factor
Pr – probability of exceedance
q – specific sewage production
Q – design peak flow
Q_{at} – mean peak flow averaged over time $t_a < 24$ h (for $t_a = 24$ h, Q_{ad} is the mean daily flow)
Q_{eq} – equivalent peak flow to calculate head losses

Q_f – peak flow at the final (end) section of a sewer reach
$Q_{h\max}$ – maximum hourly inflow rate into the wet well
Q_i – peak flow at the initial section of a sewer reach
Q_{in} – inflow rate into the wet well
Q_p – pump discharge rate (capacity)
Q_s – flow in the pressure sewer main
r – order of variation with repetitions
S_{Qat} – standard deviation of peak flow averaged over time t_a
t_a – time of averaging
t_a^* – number of minutes in the time of averaging
t_r – pump's running time
t – time
V_r^n – number of variations with repetitions of the r-th order from n different elements
λ – pump's running intensity
δ_{ij} – switch factor (0 – off, 1 – on)

REFERENCES

[1] BOWNE W.C., NARET R.C., OTIS R.J, *Alternative Sewage Collection Systems*, Manual, US EPA, Washington, 1991.
[2] Washington State Dept. of Ecology. Criteria for Sewage Works Design, http://www.ecy.wa.gov/pubs/9837/c1.pdf, 2006
[3] CLARK J.W., VIESSMAN W., *Water Supply and Pollution Control*, Int. Textbook Co., Scranton, Pa., 1965.
[4] THRASHER D., *Design and Use of Pressure Sewer Systems*, Lewis Publ. Inc., Chelsea, 1987.
[5] LITTLE C.J., Water SA, 2004, 30 (5), 137.
[6] ATV, *Worksheet A 116. Special drainage systems. Vacuum-pressure drainage*, GFA, Henef, 1992 (in German).
[7] BUCHBERGER S.G., WU L., J. Hydraul. Eng., 1995, 121 (3), 232.
[8] ZHANG X., BUCHBERGER S.G., VAN ZYL J.E., *A theoretical explanation for peaking factors*, Proc. ASCE EWRI Conf., Anchorage, Alaska, May 15–19, 2005.
[9] JONES E.E., *Domestic water use in individual homes and hydraulic loading of and discharge from septic tanks*, Proc. of the National Home Sewage Disposal Symposium. ASAE, St. Joseph, Mi., 89–103, 1975.
[10] BLAZEJEWSKI R., *Peak domestic water use in individual homes, apartment blocks and villages*, Proc. of the 20th National and 8th Int. Conf. *Water Supply and Water Quality*, PZITS, Gniezno, 2008.
[11] ATV-DVWK, *Worksheet A 198. Harmonization and derivation of design values for sewage works*, GFA, St. Augustin, 2003 (in German).
[12] HUNTER R.B., *Methods of Estimating Loads in Plumbing Systems. Building Materials and Structures*, Report BMS65, Washington D.C., 1940.
[13] BIEDUGNIS S., Environ. Prot. Eng., 1981, 7 (1), 65.
[14] SZABO T., Korrespondenz Abwasser, 1989, 8, 869.
[15] SOEDERLUND P., JOENSSON L., NILSSON P., *Design and performance of pressure sewerage systems*, Dept. of Water Res. Eng., Lund University Report No. 7180, Lund, 1994.
[16] DOHSE C., ECKSTAEDT H., Water Sci. Technol., 2003, 47 (7–8), 351.
[17] BLAZEJEWSKI R., *Rural Sanitation*, PZITS, Poznań, 2003 (in Polish).

[18] EPA *Low pressure sewer systems. Design module WWBLDM07*. NSFC, West Virginia University, 1992.
[19] SIONKOWSKI T., ŚLIZOWSKI R., Environ. Prot. Eng., 2008, 34 (3), 21.
[20] WONG L.T., MUI K.W., Build. Environ., 2007, 42 (5), 2024.
[21] STARK R.M., NICHOLLS R.L., *Mathematical Foundations for Design: Civil Engineering Systems*, McGraw-Hill Book Co., New York, 1972.