Computer modeling and Design of Cost-Effective Pump Lines Systems for Different Thickness, Materials and Fluids

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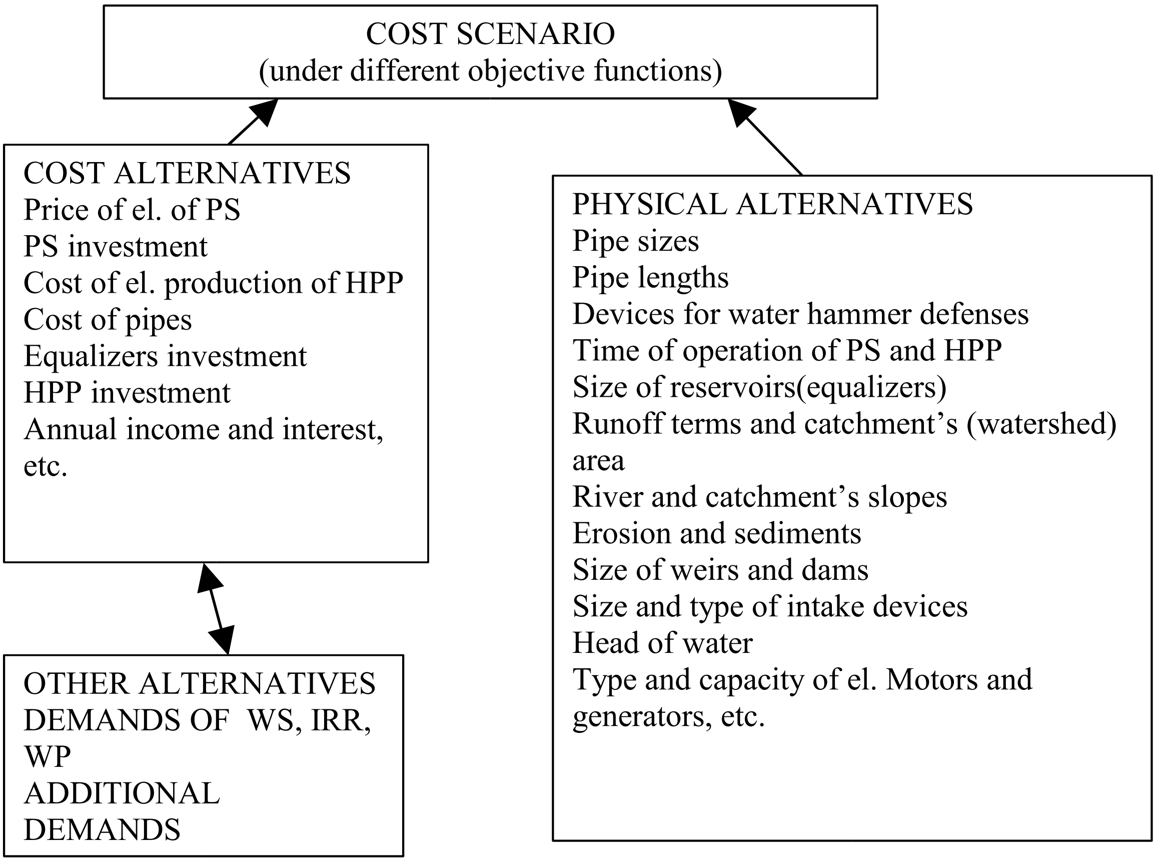
Abstract

A foundation of complete **Cost-Effective Pump Lines Systems** (CEPLS) is possible only due to the application of computer modeling. This complex problem is solved by authors' algorithm and software with the following main data: maximum operating water discharge (i.e. water discharge at normal head), head of water, hydrological and topographic data, cost-effective and technological parameters, etc. By means of these data and parameters optimal design elements of PLS are proposed at customer's choice as follows: runoff flow regulation and distribution using water intake devices, pipelines, upstream and downstream day equalizers as ponds or reservoirs, pumping stations (PS) with electrical consumption. After cost-effective analysis an additional structure as under dam Hydro Power Plant (HPP) could be included in the beginning of PLS.

Keywords: MOWD(maximum operating water discharge), CEPLS (cost-effective pump lines systems), PLS (pump lines systems), HPP (Hydro-Power Plant), PS (Pumping Station), ws (water supply), irr (irrigation), wp (water power), Object Oriented Design (OOD), Object Oriented Programming (OOP), operating time (OT), operating level maximum (OL_{max}), operating level minimum (OL_{min}).

Introduction

The optimal likely choice is studied consecutively through annual expenses on different elements having in mind: stream-flow regulation for 20-50 years as generated record's period and dependable outflow for ws, irr, wp in term of probability in percent as follows – $s_{ws} \leq 97\%$ for water supply, $s_{irr} \leq 75\%$ for irrigation, $s_{wp} \leq 90\%$ for water power to 50% by day equalizer with weir, seasonal, annual and long-term period of regulation(with dam for the last cases). Water intake devices are taken from river, ground water, spring water, mountain lakes. Pipeline materials are made of steel, reinforced concrete, glass- layer pipes HOBAS and fluid for conveyance is water or oil (naphtha crude). Under the circumstances the volume of upper and lower day equalizer, the number and the kind of pumps(with or without trimming), capacity and electrical consumption for operating time 8 h (nightly), 16 h or 24 h are estimated. Meanwhile economical diameters and wall-pipe thicknesses are determined for different cases. CEPLS are found on the bases of annual expenses using minimization of objective functions under various cost-effective scenarios:



Scheme 1. Cost Scenario Diagram

Firstly an algorithm will be presented for different elements separately.

1. Regulation of runoff according demands and pumping station operating time (Fig. 1).

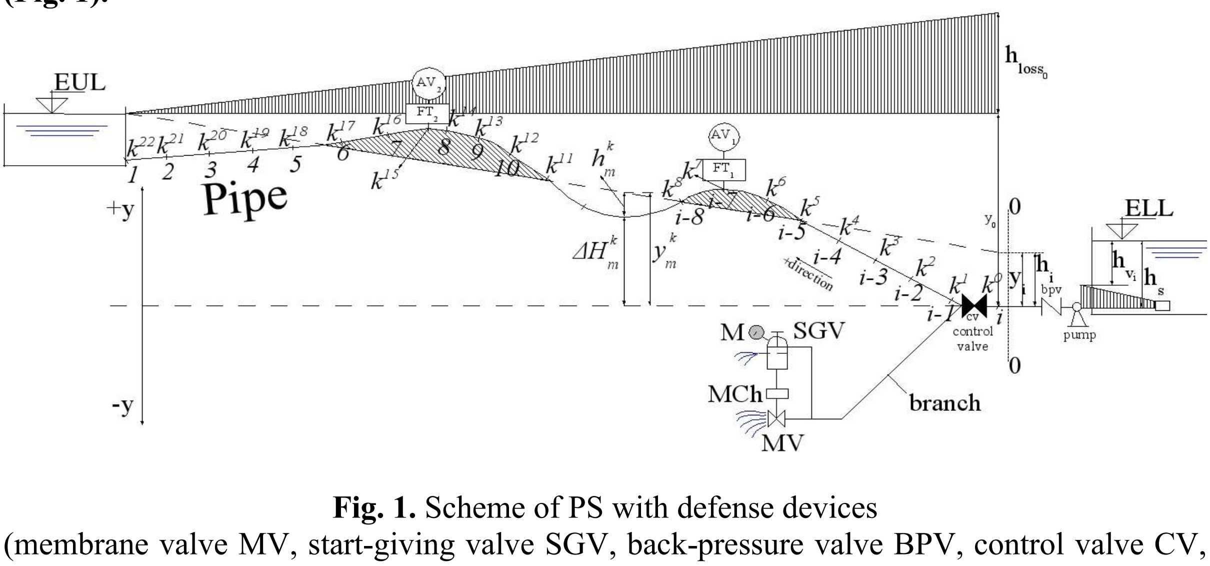


Fig. 1. Scheme of PS with defense devices
(membrane valve MV, start-giving valve SGV, back-pressure valve BPV, control valve CV, air-valves and feeder tanks, AV and FT)

On the basis of average monthly data for 25–50 years period, the runoff module coefficient k, the coefficient of variance c_v, standard deviation and skewness c_s, could be estimated. Then

the probability distribution $\overline{Q_s}$ (average annual discharge), Q_s^{month}, $Q_{\min}^{month}$ (average monthly and minimum monthly distributions with probability s) are got. The needed discharges with probabilities s_{ws}, s_{irr}, s_{wp} (see introduction) as well as maximum discharge Q_{max} with probability s_{max}= (0.01-20)% are defined by hydrological data and analysis. The calculated results are compared with recurrence intervals, given by experimental data, and then the coefficient of skewness c_s is specified if it is necessary. In case of luck of hydrological data the method of analogy is used and data from the nearest gage are transferred at a point of interest: water intake, culvert, etc. Having in mind hydrological data three baseline water discharges are taken in: $Q_{\min}^{month}, Q_s^{month}$, $\overline{Q_s}$ where the first and the second one are minimum and average monthly distributions for long term period of regulation and the last is given above. The following criteria for the kind of regulation are perceived:

1.1 Day equalizer (diurnal) regulation (Fig.1, Fig.2) – if $3Q_{\min}^{month} \geq Q_8$ otherwise the next condition must be tested.

1.2 Seasonal regulation (Fig.3, Fig.5) – if $3Q_s^{month} \geq Q_8$ otherwise 1.3

1.3 Annual regulation (Fig.4, Fig.5) – if $3\overline{Q_s} \geq Q_8$ otherwise 1.4

1.4 Long-term period of regulation (Fig.4, Fig.5) - if $3\overline{Q_s} \geq Q_8$ otherwise it is accepted $\overline{Q}_{50\%} \geq \overline{Q_s} \rangle \overline{Q}_{97\%(75\%)(90\%)}$ (see Introduction) and a higher dam is needed.

The following abbreviations are accepted:
$Q_8 = N.Q_{0_0}$, N is a pump numbers with discharges Q_{0_0} that the cheapest nightly electrical energy in night (8 h) is pumping. The following conditions for 8h, 16h, and 24hours must be fulfilled for annual regulation with probabilities s_{ws}, s_{irr}, s_{wp} :

$$3Q_{\min}^{month} \geq Q_8 = NQ_{0_0} \tag{1}$$

$$1.5Q_{\min}^{month} \geq Q_{16} \tag{2}$$

$$Q_{\min}^{month} \geq Q_{24} \tag{3}$$

The needed volumes of day equalizers are:

$$V_8 = 28800\left(Q_8 - Q_{\min}^{month}\right) \tag{4}$$

$$V_{16} = 57600\left(Q_{16} - Q_{\min}^{month}\right) \tag{5}$$

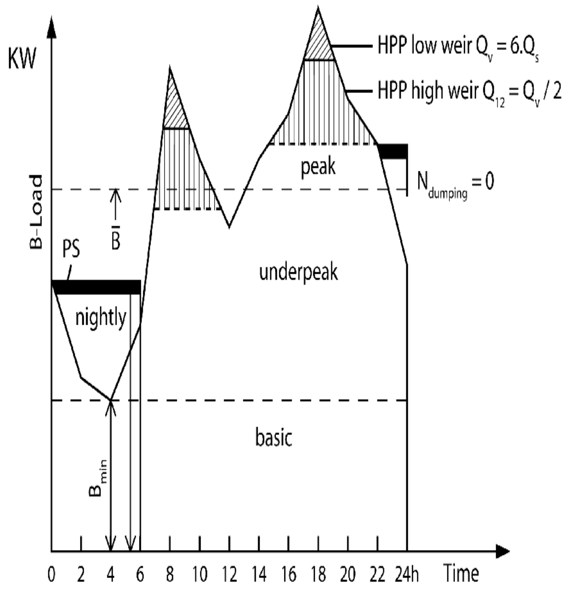
2. Technological and built-up strategy

The four kind of regulations mentioned above are studied according several possible built-up scenarios in the beginning of PLS:

2.1. Design with under dam HPP
2.2. Design with HPP combined only with weir (running water regulation)
2.3. Design without HPP

Depending on OT of PS six scenarios are available:

2.1.1. PS operates with one pump and under dam HPP
2.1.2. PS operates with two or more different kind of pumps and under dam HPP
2.2.1. PS operates with one pump on running water
2.2.2. PS operates with two or more different kind of pumps on running water
2.3.1. PS operates with one pump without HPP
2.3.2. PS operates with two or more different kind of pumps without HPP



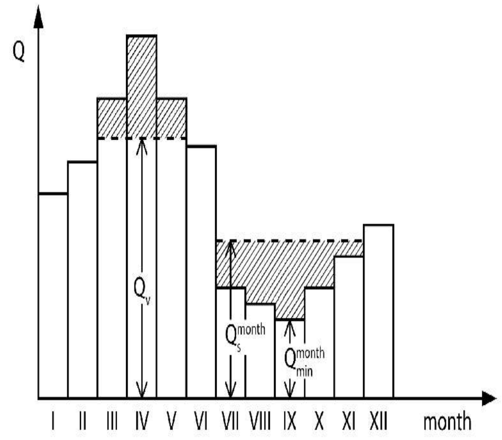


Fig. 3. Season regulation

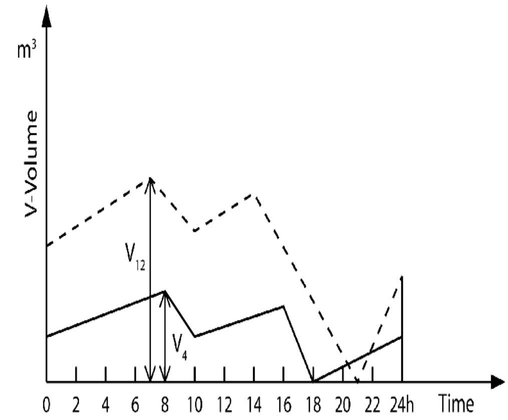


Fig. 2. HPP day equalizer

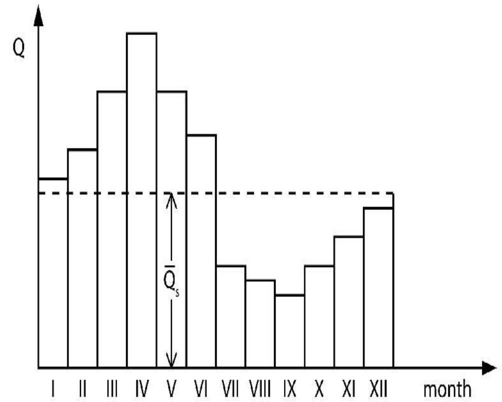


Fig. 4. Annual and long term period

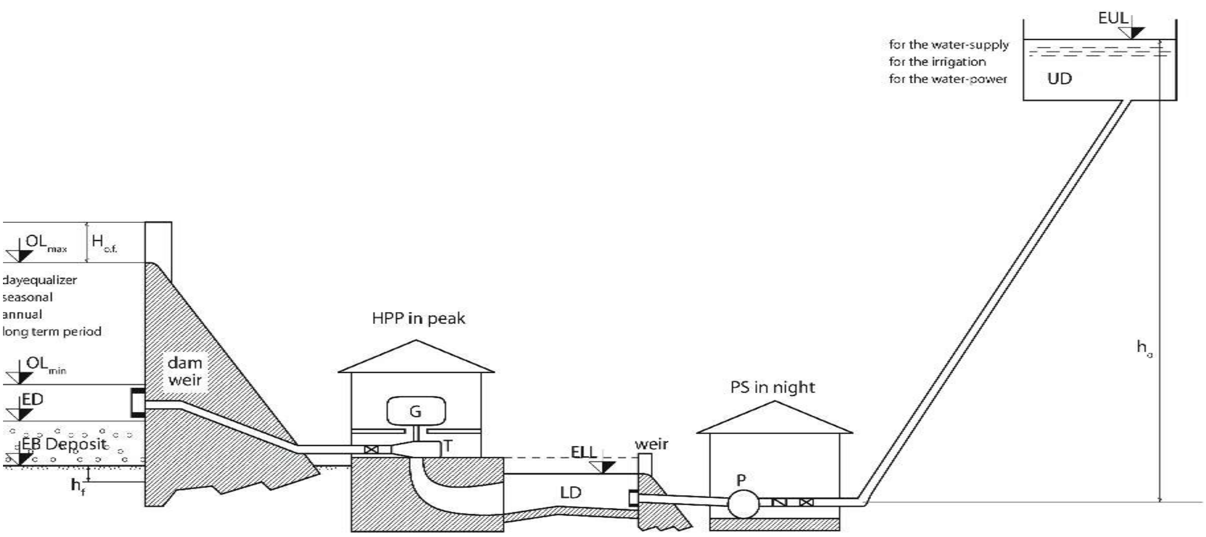


Fig. 5. OL_{max} - operating level max; OL_{min} - operating level min; ED - elevation deposit; EB - elevation bottom; HPP ; ELL - elevation lower level; LD - lower dayequallizer; PS ; EUL - elevation upper level; UD - upper dayequallizer; $H_{o.f.}$ - overflow head

Different kind of regulations is examined further according to this strategy following various criteria of regulations.

3. Diurnal regulation of runoff with weir (2.2)

3.1. Building of HPP and cost – effective criterion

HPP is built after (**2.2**) with day equalizer for peak, under peak and basic energy. The volume of reservoir is smaller – for 12 or 4 hours OT. Besides PS with LD and UD (Fig. 5) for OT 8h nightly is examined for ws, irr, wp needs. Monthly distribution of water discharges can be seen in Fig. 3 where Q_v notes MOWD without overflowing. The volume and capacity load of HPP are shown in Fig.2 and hydraulic characteristics are estimated by the next formulas:

Low weir HPP $V_4 = 6.Q_s^{month}.3600.4[m^3]$

High weir HPP $V_{12} = \dfrac{Q_v}{2}.3600.12[m^3]$; Q_v is MOWD; $Q_v = 1.5\overline{Q_s}$

PS →LD → $V_4 = 6.Q_s^{month}.3600.4[m^3]$;

PS →LD → $V_{12} = \dfrac{Q_v}{2}.3600.12[m^3]$

The head of broad crested weir is: $H_{w.t.} = \left(\dfrac{K_h N Q_{0_0}}{m.b_e\sqrt{2g}}\right)^{2/3}$;m=0.345; $v_0 = 0.8m/s = \dfrac{K_h N Q_{0_0}}{b_e H_{w.t}}$

$$OL_{\max} = EB + h_h + H_{w.t} + (1+\xi_{grid})\frac{v_0^2}{2g} + \sqrt{\frac{V_{12(4)} I_r}{S.0.5}};$$

$$OL_{\min} = OL_{\max} - \sqrt{\frac{V_{12(4)} I_r}{S.0.5}}$$

The overflow head of weir in LD is: $H_{o.f} = \left(\dfrac{Q_4}{m.S_{o.f}\sqrt{2g}}\right)^{2/3}$; m=0.44

The total depth is:

$$h_w^{LD} = \sqrt{\frac{V_{12(4)} I_r}{S'.0.5}} + h_h + d_{pi} + (1+\xi_{grid}+\xi_{en}+\xi_f)\frac{v^2}{2g} + h_f + H_{o.f}$$

Abbreviations given in the formulas are shown in Fig.5 where:

K_h is the coefficient of hour schedule of electrical consumption (1, 0.5,0.33 for 8,16,24h resp.); b_e and $S_{o.f}$ are widths of the entrance of weirs; m is a coefficient of water discharge in high and low weir resp. 0.345 and 0.44; v_0 is the velocity in entrance, approximately 0.8 m/s because of sediment; S, S' are widths of the high and low weirs, resp.; h_h, h_f are the height of threshold and foundation, resp; local(minor) losses coefficients are $\xi_{grid}, \xi_{en}, \xi_f$; d_{pi} is the diameter of pipe from LD to PS; $H_{w.t.}, H_{o.f}$ are overflowing heads in high and low weir resp.; h_w^{LD} is the total depth of LD.

The most effective decision can be obtained if the dumping energy of HPP is small or better equal to 0 (Fig.2). In this instance the income and expenses are:

$$I_{income}^{HPP} = 8.2.Q_v\left(\frac{OL_{\max}+OL_{\min}}{2} - ELL_{\min}\right).4000.s_{HPP}^{12} \text{ [\$/yearly]}$$

$$E_{\substack{expense\\additional}} = \frac{9.81.Q_8(OL_{\min}-ELL_{\min})}{\eta\eta_m}.8.365.s_{nightly} + \left(0.375.(h_w^{LD})^2.S'\right)C_l.p +$$

$$+8.2.Q_v\left(\frac{OL_{\max}+OL_{\min}}{2}-ELL_{\min}\right)C_{HPP}.p\ ;\qquad C_{HPP}=500[\$/kW]$$

where s_{HPP}^{12} is the price of produced energy for 12 month; C_{HPP} is the price of investment of HPP; $p=p_a+p_{\text{int}}$ are annual expenses for amortization and interest; $s_{nightly}$ is the price of el. energy nightly; $C_l[\$/m^3]$ is the cost of LD for building. The criterion for cost-effective design of HPP is given by inequality:

$$I_{income}^{HPP}\rangle E_{\substack{\exp ense\\ additional}} \tag{6}$$

The next step evaluates the operation of pumps and PLS.

3.2 Design of PLS with different materials and kind of pumps

Approximate choice of economical diameters is listed in Table 1, where $Q_{0_{pipe}}=Q_8=N.Q_{0_0}$ and N is a number of pumps.

Table 1. Economical diameters for different pipes

$Q_{0_{pipe}}$	0.0015	0.0033	0.0060	0.0100	0.0150	0.0300	0.0500	0.0720	0.1060	0.1450	0.1900
d [mm]	50	75	100	125	150	200	250	300	350	400	450
V [m/s]	0.75	0.75	0.76	0.82	0.85	0.95	1.02	1.05	1.10	1.15	1.20
$Q_{0_{pipe}}$	0.245	0.365	0.520	0.705	0.920	1.200	1.475	2.000	3.020	6.000	
D [mm]	500	600	700	800	900	1000	1100	1250	1500	2000	
V [m/s]	1.25	1.30	1.35	1.40	1.45	1.50	1.55	1.63	1.71	1.91	

Data from Table 1 were used for computer modeling by cubic splines and factor coefficients K_f for different materials: 1(steel), 1.07(reinforced concrete), 0.9(glass layer HOBAS). Different properties of pipes HOBAS used in Bulgaria are given in catalogue [1]. The example is complied with two pumps (N=2). Firstly by various catalogues of centrifugal pumps the type is chosen ensuring the maximum efficiency of each pump. Then the curves $Q=f(H)$ and $Q=f(\eta)$ are implemented by spline functions with 3 points in the field of maximum efficiency. The discharges of pumps for 8, 16, 24h are: for 8 h $Q_{0_p}=Q_{0_0}$;16 h $Q_{0_p}=0.5Q_{0_0}$;24 h $Q_{0_p}=0.33Q_{0_0}$ where Q_{0_0} is the selected discharge from catalogue in m^3/s. The total discharges and heads of pipeline are:

8 h, $Q_{0_{pipe}}=NQ_{0_0}\rightarrow H_{0_0}=h_0+h_{loss}=h_0+v^2\sum_{m=1}^{s}l_m/C^2R$ [m]

16 h, $Q_{0_{pipe}}=N0.5Q_{0_0}\rightarrow H_{0_0}=h_0+h_{loss}=h_0+v^2\sum_{m=1}^{s}l_m/C^2R$ [m]

24 h, $Q_{0_{pipe}}=N0.33Q_{0_0}\rightarrow H_{0_0}=h_0+h_{loss}=h_0+v^2\sum_{m=1}^{s}l_m/C^2R$ [m]

The velocity and losses are calculated by the formulas below with different roughness factors: $n=0.013; n=0.013; n=0.009$ for steel, reinforced concrete and glass layer, resp. Including Manning's coefficient and losses the result is:

$$F=\pi.d^2/4, v=N.Q_{0_p}/F, R=d/4, C=R^{1/6}/n, h_{loss}=v^2\sum_{m=1}^{s}l_m/C^2.R$$

By catalogue three values are selected in the field of maximum efficiency. Then cubic splines $Q = f(H)$ and $Q = f(\eta)$ are composed with one basic and three or less trimmed wheels according to the standard. The condition for effective selection of pump is:

- $Q^{\min} \le Q_{0_0} \le Q^{\max} \qquad H^{\min} \le H_{o_p} \le H^{\max}$

For the selected pump with discharge Q_{0_0} the head of pump is: $H_{0_p} \to \eta_i^{\max}$, where H_{0_p} is the pump head which has the nearest position to maximum efficiency. If $H_{0_0} \ne H_{0_p}$ the wheel is trimmed (only for one-stage pumps).

Example:

A rotary pump 450D90 with parameters: $Q_{0_0} = 0.450 m^3/s, H_{0_0} = 86 m, H_{0_p} = 91.02 m$; revolution n=1450 rev/min and diameter D_0=540 mm are selected. The head of pipeline is H_{0_0}. The pump is chosen in the field of maximum efficiency by catalogue. By similarity laws follows:

$$\frac{H_{0_p}}{H_{0_0}} = i_D^2 = 1.058429; \quad \frac{Q^{tr}}{Q_{0_0}} = i_D; \quad i_D = \frac{D_0}{D} = 1.0288 \qquad (7)$$

Hence the wheel must be trimmed and the new diameter is $D = \dfrac{540}{1.0288} = 525[mm]$

The result is presented in Table 2 for 8h OT according the similarity law for capacity and efficiency of selected pipe, here $i_D = \sqrt{1.058429} = 1.0288$.

Table 2. OT 8h; The wheel is trimmed

$i_D = 1.0288$	i = 1	i = 2	i = 3
$Q\ [m^3/s]$	$Q^{\min} = 0.360$	$Q = 0.450$	$Q^{\max} = 0.600$
H [m]	$H^{\max} = 96.5$	H=91.02	$H^{\min} = 66.5$
$\eta_{0_i}^{(0)}$	0.765	0.830	0.690
$Q_i^{tr} = \dfrac{Q}{i_D}[m^3/s]$	0.3499	0.4374	0.5832
$\left[Q_i^2\right]^{tr}$	0.122430	0.191319	0.340122
$H_i^{tr} = \dfrac{H_{0_i}}{i_D^2}[m]$	91.17	85.99	62.83
$\left[C_i^3\right]^{(j-1)} = \dfrac{\left[Q_i^2\right]^{tr}.\eta_{0_i}^{(0)}}{9.81.H_i^{tr}}$	1.05^{-04}	1.88^{-04}	3.81^{-04}
$\eta_i^{(1)} = 9.81.\left[C_i^3\right]^{(j-1)}.H_i^{tr}/\left[Q_i^2\right]^{tr}$	0.76704 8	0.828928	0.690404
$\left[C_i^3\right]^{(j)} = \dfrac{\left[Q_i^2\right]^{tr}.\eta_{0_i}^{(1)}}{9.81.H_i^{tr}}$	1.05^{-04}	1.88^{-04}	3.81^{-04}
$\eta_i^{(2)} = 9.81.\left[C_i^3\right]^{(j)}.H_i^{tr}/\left[Q_i^2\right]^{tr}$	0.76705	0.8289	0.690404

As can be seen from the results the problem is solved by iterations and changes of efficiencies are small in the field of optimal points. Further one another kind of pump could be chosen

from catalogues and the results are compared according to the optimal efficiency η_i. Thereupon the results are transferred for OT 16 h, 24 h. If the demands are bigger or other alternatives are accepted more than 2 pumps are selected. In all this cases one standard pump is chosen and three modified pumps could be the worst trimmed. In some particular cases the cubic splines have to be constructed through 9 points going to IV quadrant by extrapolation. This special case is requisite for studying of transient processes and water hammer in PLS.
The pump station investment are evaluated by the price of energy, amortization and interest. After scenarios **2.3.1** and **2.3.2** expenses for el. energy are:

- one pump without LD and HPP

$$K_{p.s} = \frac{9.81 K_h N Q_{0_0} H_{0_0}}{\eta \eta_m} b_{p.s} p \tag{8}$$

- two different pumps without LD and HPP

$$K_{p.s} = \frac{1}{2}\frac{9.81 K_h N Q_{0_0} H_{0_0}}{\eta \eta_m} b_{p.s} p + \frac{1}{2}\frac{9.81 K_h N Q_{0_0} \overline{H}_{0_0}}{\eta \eta_m} b_{p.s} p \tag{9}$$

Abbreviations above are:
$K_{p.s}$ is PS investment [\$]; $b_{p.s}$ is the price of investment of *PS* in \$/Kw ≈ 400 \$/Kw; $p = p_a + p_{\text{int}}$ are annual expenses for amortization and interest; K_h is the coefficient of hour schedule of el. consumption (1/8h, 0.5/16h, 0.33/24h).
If two pumps are used $K_{p.s}$ is shared in two parts as it can be seen by (9):

- one pump (Fig. 5)

$$H_{0_0} = h_0 + h_{loss} - (OL_{\min} - EP_{pump}) \tag{9'}$$

where EP_{pump} is the elevation at pump axis; h_{loss}, H_{0_0}, h_0 are explained above. In case of high weir or dam the head $h_{dam}^{s.r.}$ is bigger and the control valve (Fig. 1) has to be shut.

- two pumps (Fig. 5)

$$\overline{H}_{0_0} = h_0 + h_{loss} - \frac{(OL_{\min} + OL_{\max})}{2} + EP \tag{9''}$$

where EP is the elevation at axis of the different two pumps.

3.3. CEPLS with different sections, diameters, thicknesses and materials

The PLS can be built by different sections of pipelines (Fig. 1).

- **Steel pipes**

  The maximum total head is without restriction. The approximate choice of diameters, given in Table 1, could be specified by objective function for every section:

$$d_m = 11.904.\sqrt[6.167]{\frac{s_p.(K_h.NQ_{0_0})^3.n^2.t.t_d(\bar{t}_d)}{\gamma_f \eta \eta_m .b\delta_m .p.100}}\text{ ; m =1,...,s; } \delta_m = \frac{h_m.d_m}{2.\sigma_a} + \delta_s \tag{10}$$

$$C_m = R_m^{1/6}/n \text{ ; } \quad Q_{0_0} = \frac{Q_{0_{pipe}}}{K_h N}\text{ ; } h_{loss_1} = \sum_{m=1}^{s}\frac{l_m v_m^2}{C_m^2 R_m}\text{ , } h_{loss_m} = h_{loss_1}\frac{\sum_{m=2}^{s} l_m}{\sum_{m=1}^{s} l_m}$$

$$h_m = (EHL - E_{0_m}) + h_{loss_m}\text{ ; m = 2,...,s}$$

where m is the number of current section with maximum number s; δ_m is the thickness according to Mariot (kettle) formula with reserve and correction factors - δ_s, σ_a; h_m is the total head; n is the roughness coefficient as follows: 0.012 – for normal pipe, 0.013 – after long period; $p=(p_a+p_{\mathrm{int}})/100$ - investment for amortization and interest resp.; s_p is the price of el. consumption (nightly, diurnal, peak) [cent/Kwh]; γ_f, η, η_m are specific weight of the fluid and efficiencies of pump and motor resp.; b is the price of pipe material [$/tone]; t, t_d, $\bar{t}_d$ are OT in hours or months: $t=8\ h(16\ h)(24\ h)$; $t_d=1(4/12)1$ for ws, *irr, wp* resp.; $\bar{t}_d$ is dimensionless time per year for mixed use (Scheme 1); E_{o_m} is the elevation of pipe at section m; K_h is the coefficient of hour schedule of el. consumption $(1/8h, 0.5/16h, 0.33/24h)$.

The comprehensive assessment of δ_m, d_m is completed by formula (10). The computer modeling starts with the approximate choice of diameters – Table 1 with the minimum thickness $\delta_m=0.6cm$ for $m=1$. Afterwards the calculations are repeated for all sections and coefficients K_h: m= m+1,....,s and OT = 8/16/24h. In the end the wave spreading velocity (celerity) of fluid is determined:

$$a_m=\sqrt{\frac{\dfrac{\varepsilon.g}{\gamma_f}}{1+\dfrac{d_m.}{\delta_m}\dfrac{\varepsilon}{E}}}\quad ;\quad m=1,....,s \qquad (11)$$

Table 3. Physical values of parameters

Fluid		Pipe materials		
water	oil	steel	reinf. concrete	HOBAS
$\varepsilon[\mathrm{tm}^{-2}] / \gamma_f[tm^{-3}]$		$\gamma[tm^{-3}] / E[tm^{-2}]$		
$2.1x10^5 / 1$	$1.35x10^5 / 0.9$	$7.85 / 2.1x10^7$	$2.20 / 2.1x10^6$	$1.95 / 10^6$

The annual expenses for steel pipelines including the expenses for PLS, annual el. consumption for 8h OT, the investments for dayequalisers and for PS are:

$$\sum k_s=\sum_{m=1}^{s}\gamma\pi d_m\delta_m l_m bp+\frac{9.81K_hNQ_{0_0}H_{0_0}}{\eta.\eta_m}t.365.s_p+K_{u.d}.p+\left(0.375.h_w^2.S'\right)2.5.C_l.p+$$
$$+\frac{9.81.K_h.N.Q_{0_0}.H_{0_p}}{\eta.\eta_m}.b_{p.s}.p \qquad (12)$$

where $K_{u.d}$, C_l; p are the investments for dayequalisers and $p=(p_a+p_{\mathrm{int}})/100$; h_w and S' are the depth and with of lower day equalizer; s_p is the price of el. consumption; t is OT (8/16/24h). If the criterion (6) is fulfilled and after assessment of the additional expenses for the building, the formula (12) is changed. In this instance the sum for additional expenses has to be reduced with the incomes of HPP. This remark is referred to all cases with building of weirs and dams.

By analogy with (12) the expenses for 16h and 24h are evaluated and the cost-effective OT is determined as a result.

- **Reinforced concrete pipes**

The price is twice less than the steel pipes but they are built to the maximum head of 30 – 50 m. The objective function for d_m is similar to (10) and calculations are repeated for every sections according to (10) but one important restriction must be added - $h_m \le 50[m]$ for m=1,...s. The computer modeling starts with the minimum of thickness $\delta_m = 5[cm]$ and approximate diameters given from Table 1. The celerity of fluid is modified by the formula (13):

$$a_m = \frac{1425}{\sqrt{1+\dfrac{d_m}{m_e.\dfrac{\delta_m}{100}\left[1+(m_e-1)\kappa_m\right]}}} \text{; m=1,...., s,} \qquad (13)$$

$m_e = \dfrac{E}{E_{r.concrete}} = 10$; $\kappa_m = 3\left(1-\dfrac{50-h_m}{50}\right)\% \ge k_{\min}$ =0.5%. The coefficient m_e can be given from Table 3 from the values of the steel and reinforced concrete. The coefficient κ_m is the restriction for the head.

- **Glass layer pipes HOBAS**

The price is more than twice bigger than steel pipes but they are more stable on aggressive fluids and chemical influence. The maximum head is about 25atm. and the maximum diameter is Ø2.4m with maximum thickness $\delta = 47.9mm$. More properties of these pipes are given in catalogue [1]. According to authors investigation the celerity is about $550[ms^{-1}]$ approximately. The objective function is specified according to (10) and computer modeling follows as a logical consequence of the previous cases.
Following the criteria for regulations in **part 1** the next cases will be considered.

4. Seasonal regulation (1.2)

4.1. Regulation with under dam HPP and PS (2.1.2)

PS is used for *ws, irr, wp* with LD and UD (Fig. 3, Fig. 5). Under dam HPP could be turned to use for other kinds **1.3** and **1.4** after verification of relevant criteria and probabilities s_{ws}, s_{irr}, s_{wp}.

4.1.1. PS operates with one pump – formula (9')

If the head of dam $h_{dam}^{s.r.}$ is big the valve is shut (Fig. 1) and the pipe head is determined following (9').

4.1.2. PS operates with two different kinds of pumps

Two pumps are used as a rule for low and high dams, according to (9''). For seasonal regulation the storage dam per year is:

$$\left(Q^{month} - Q_s^{month}\right)\rangle 0 \text{ ; } V^{s.r.} = 86400\sum\left(Q^{month} - Q_s^{month}\right)t_{month-days} \qquad (14)$$

Diurnal storage volume with HPP is:

HPP $V_4 = 6.Q_s.3600.4[m^3]$, peak operation

HPP $V_{12} = \dfrac{Q_v}{2}.3600.12[m^3]$, under peak operation; Q_v - MOWD.

If there is not overflowing by the dam the volume of lower equalizer daily is the same:

PS →LD → $V_4 = 6.Q_s.3600.4[m^3]$; PS →LD →$V_{12} = \frac{Q_v}{2}.3600.12[m^3]$.

Overflow head of the dam with $s_{\max} = 0.1\%$ probability of the discharge is:

$$H_{o.f.} = \left(\frac{Q_{0.1\%}}{m.S_{o.f,} \sqrt{2g}} \right)^{2/3}, \text{ m=0.48; } h_w = \sqrt{\frac{V_{12(4)}.I_r}{S.0.5}} + h_h + h_f + H_{o.f.}$$

$$OL_{\max} = EB + h_{deposit} + d_{pipe} + \left(1 + \xi_{grit} + \xi_{en} + \xi_f\right)\frac{v^2}{2g} + \sqrt{\frac{V_{.}^{m.y.r.(y)(s)} I_r}{S.0.5}}$$

$$OL_{\min} = OL_{\max} - \sqrt{\frac{V_{.}^{m.y.r.(y)(s)} I_r}{S.0.5}} \text{ ; } h_w^{LD} = \sqrt{\frac{V_{12(4)} I_r}{S.0.5}}. + h_h + h_f + H_{w.t} + H_{o.f.}$$

The abbreviations are given in **3.1**. The annual income is:

$$I_{income}^{HPP} = 8.2.Q_v \left(\frac{OL_{\max} + OL_{\min}}{2} - ELL_{\min} \right).USE.s_{HPP}^{m.y.;(y);(s)} \quad [\$/\text{year}] \qquad (15)$$

where: USE is the annual hour use of HPP with the following parameters –

- seasonal regulation $Q_v = 3.\overline{Q}_s$, $USE_s = 4000h / year$;
- annual regulation $Q_v = 4.\overline{Q}_s$, $USE_a = 3000h / year$;
- long-term period of regulation $Q_v = 6.\overline{Q}_s$, $USE_{long} = 2000h / year$.

The annual expenses are described in paragraph **3.1**. The criterion for cost-effective design is the same i.e. inequality (6). When two different pumps are used a high dam can be built and equation (9’’) is valid. For average monthly distribution with probabilities s_{ws}, s_{irr}, s_{wp} and different hour use the volume is:

t=8 h; $V_8^{s.r.} = 28800 \sum_{3Q_{\min}^{months} \langle Q_8} \left(Q_8 - Q_{\min}^{months}\right).t_{month-days}$; $Q_8 = NQ_{0_0}$

t=16 h; $V_{16}^{s.r.} = 57600 \sum_{1.5Q_{\min}^{months} \langle Q_8} \left(Q_{16} - Q_{\min}^{months}\right).t_{month-days}$; $Q_{16} = 0.5.NQ_{0_0}$; $Q_{16} = 1.5.Q_s^{month}$

t=24 h; $V_{24}^{s.r.} = 86400 \sum_{Q_{\min}^{months} \langle Q_{24}} \left(Q_{24} - Q_{\min}^{months}\right).t_{month-days}$; $Q_{24} = 0.33.Q_{0_0}$; $Q_{24} = Q_s^{month}$

The height of dam is:

$$h_{dam-\max}^{s.r.} = \sqrt{\frac{V_8^{s.r.} I_r}{S'.0.5}} + \sqrt{\frac{V_{depos} I_r}{S'.0.5}} + d_{pi} + \left(1 + \xi_{grid} + \xi_{en} + \xi_f\right)\frac{v^2}{2g} + h_{foun} + H_{o.f.} \qquad (16)$$

where V_{depos} is the volume of sediment for 100 years ; $H_{o.f.}$ is the overflow head see **3.1.**

The expenses for seasonal regulation are determined by analogy with **3.1.** Electrical consumption for one or two pumps and the investment for the dam are:

$$\overline{E}_{el} = \frac{9.81.K_h.N.Q_{0_0}.H_{0_0}(\overline{H}_{0_0})}{\eta.\eta_m} t.365 \text{ ; } K_{dam} = \left(0.375 h_{dam-\max}^2 S\right) C_l$$

where H_{0_0} and $\overline{H}_{0_0}$ are the head of pipeline for one or two pumps, resp. defined by (9’) and (9’’). The minimum height of dam is:

$$h_{dam.\min}^{s.r.} = h_{deposit} + d_{pipe} + \left(1 + \xi_{grit} + \xi_{en} + \xi_f\right)\frac{v^2}{2g} + h_{foun} \qquad (17)$$

The criterion for total volume of dam is described by inequality:

$$V_8^{fill} = 86400.\sum_{3.Q^{month}\rangle Q_8} (Q_{24}^{month} - \frac{1}{3}Q_8).t_{month-days} \geq V_8^{s.r.} \quad (18)$$

5. Annual regulation (1.3)

5.1. Under dam HPP and PS with LD and UD – 2.1. (Fig.4, Fig.5)

HPP is built with tendency to annual or long-term period after verification of criteria given in **1.3** and **1.4**. The probabilities for complex use are mentioned in **Introduction.**

5.1.1. PS with one kind of pump – 2.1.1.

If the head of dam $h_{dam}^{s.r.}$ is big the valve is shut (Fig. 1) and the pipe head is determined following (9'). Electrical consumption is estimated by analogy with **4.1.1.**

5.1.2. PS with two different pumps – 2.1.2.

Two different kinds of pumps are necessary for low and high head and the valve is shut if the head is big. The total head of PLS is determined according to (9') and (9"). The size of LD and UD could be exacted by inheritance hierarchy of ponds (Fig.7) including the volume of sediment. The volume of dam storage is evaluated according to OT (8h/16h/24h) by the next formula :

$$V_{24}^{y.r} = 86400.\sum_{I.month.}^{XII.month}\left(Q^{month} - \overline{Q}_s\right)t_{month.days}[m^3]; \; V_{depos} = 1.855x10^6\,\overline{Q}.\rho_{turbidity} \quad (19)$$

The maximum height of dam is determined by analogy with formula (16) having in mind the overflowing after paragraph **3.1.** Electrical consumption for one or two different pumps is explained in paragraph **4.1.2.** Some variants for the volume of sediment and turbidity should be specified through the hierarchy of ponds – Fig. 7.

6. Long – term period of regulation

6.1. Under dam HPP and PS with LD and UD according to 2.1. (Fig.4, Fig.5)

The basic scenario is realized with one pump and control valve (cv) that can be shut – Fig.1. for high head of dam.

6.1.1. PS with one pump

6.1.2. PS with two different pumps

In the both cases the total head of PLS is determined by analogy with the previous cases. As a rule the total volume for full long – term regulation is twice bigger than average annual runoff. If the probabilities s_{ws}, s_{irr}, s_{wp} are known, the dam volume is modeling firstly by the formulas for annual regulation:

$$\left(Q^{month} - \overline{Q}_s\right)\rangle 0 \quad \text{and} \quad V^{y.r.} = 86400\sum\left(Q^{month} - \overline{Q}_s\right)t_{month-days} \quad (20)$$

Secondly the volume is specified including the periods of overflow and storage with OT for 25 to 50 years long-term period. The volume is:

$$V_{25-50}^{m.y.r.(y)(s)} = 86400\sum_{1.year}^{25-50year}\left[\left(\sum_{I.month}^{XII.month}\left(Q^{month} - \overline{Q}_s\right).t_{month-days}\right) \leq V^{m.y.r.(y)(s)}\right] \quad (21)$$

Equation (21) is valid for seasonal, annual and long- term period of regulation. In this way the usable volume can be established by means of the next inequalities for overflow and shortage yearly :

$$Overflow: \sum_I^{XII}\left(Q^{month} - \overline{Q}_s\right)t_{month-days}\rangle V^{m.y.r.(y)} \; ; \; shortage \; \sum_I^{XII}\left(Q^{month} - \overline{Q}_s\right)t_{month-days}\langle 0 \quad (22)$$

Obviously the shortage volume per year shows that other kind of regulation is needed or a bigger volume of dam has to be taken.

7. Object oriented implementation

The described system is modeled with OOD and implemented in C++. Fig.6 presented basic structure of program system.

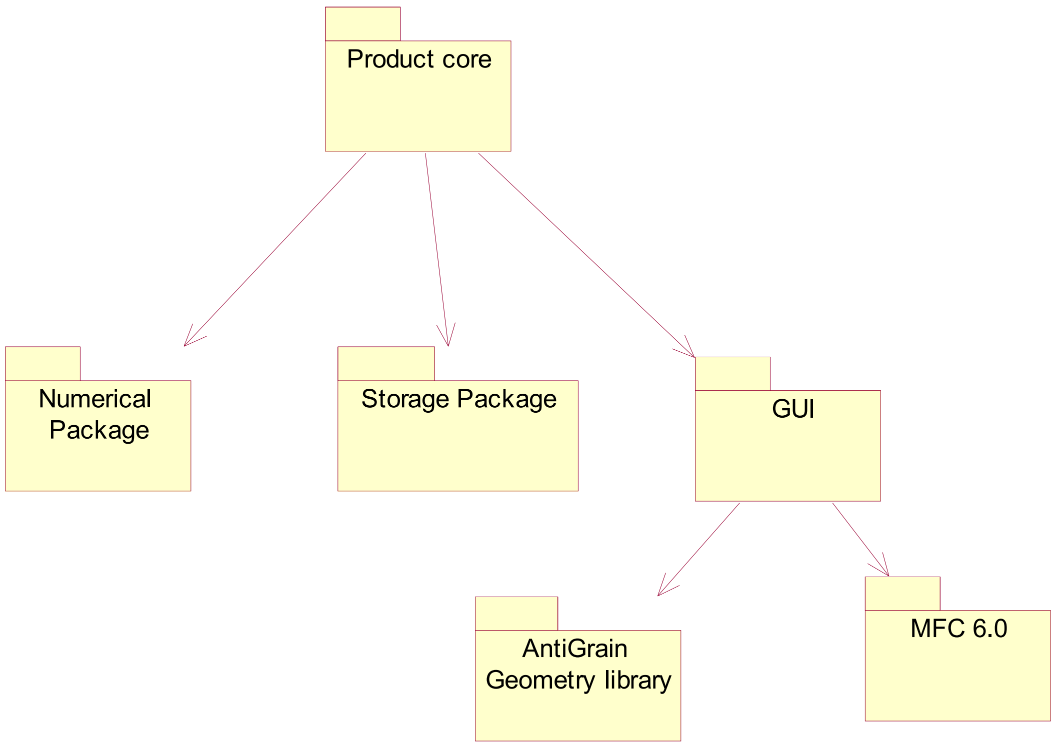


Fig. 6 Basic structure of software implementation

In general the product is decomposed on subsystems according to rules of Model/View/Controller approach. GUI package is responsible user interaction and data visualizing. It is built on the base of following libraries:

- Microsoft Foundation Classes used for general GUI organization.
- Anti Grain Geometric library used for producing high quality drawings.

The product core package contains Model and Communicator. Later we will see some of basic classes included into this subsystem.

Numerical Package is set of static classes which include mathematical and numerical computations methods like Cubic spline computation, Gaussian elimination, Least squares method, implementation of different statistical methods etc.

Storage Package provides services supplying persistent storage of all data related with a Fluid system project. This approach ensures independency of real storage system. It can be a file system, RDBMS, Object oriented database, or anything else. Fig   presents the design idea for building Core subsystem. The pipeline system is modelled with class named CFSPipline

which aggregates classes modelling different pipeline system components. CFSPond , CFSPipe, and SFLSPump are base classes describing ponds, pipes, and pumps which compose such a system. According to this figure the parameters of these components depend significantly from fluid which is modelled with CFSFluid class, as is shown on the figure.

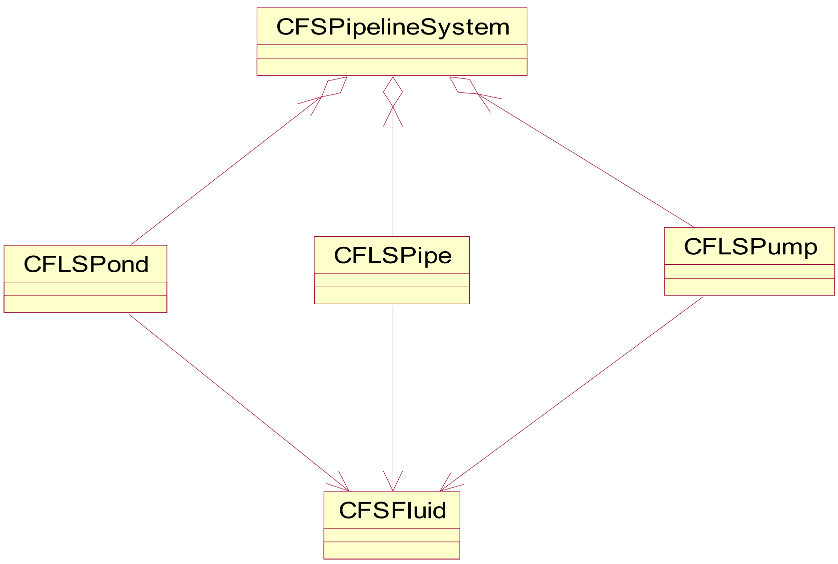


Fig. 7 General class structure of Pipeline system

Fig.8.a shows inheritance hierarchy of ponds. As it was shown in previous parts of this paper different optimal configuration is chosen between types of ponds. This approach ensures uniform treating of chosen pond using polymorphism.

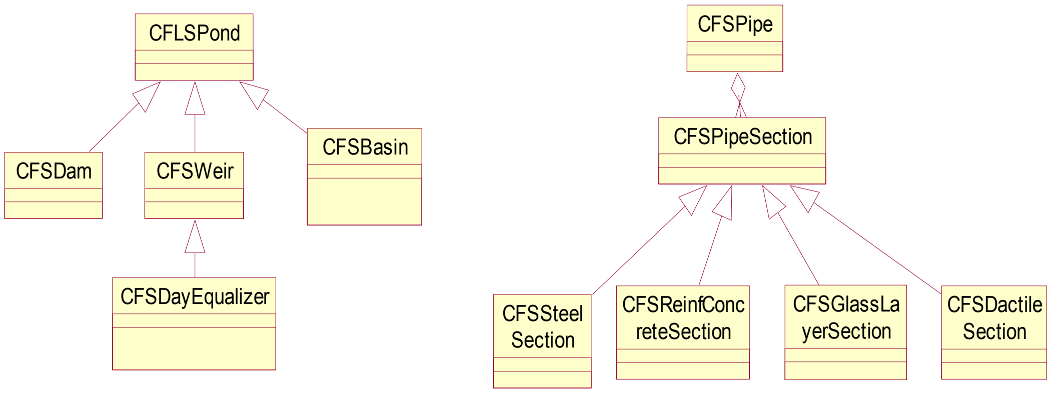


Fig. 8 **a)** Pond class hierarchy **b)** Pipe class hierarchy

Fig. 8.b describes the object structure of pipeline. According to this design approach a pipeline is a container class composed of sections. Each section is modelled with class CFSPipeSection, which is a generalization of sections made of different materials.

This approach allows seamless future extension of building components in general and spatially to create pipelines built from new materials.

Similar approach is used modeling the fluid. Currently water, oil, and condensed gas are considered. The system can easily be expanded to process other types of fluid.

8. Conclusion

On the basis of the worked out algorithm and software six scenarios for built- up strategy are examined about regulation of runoff for water supply needs, irrigation and water power. Cost-effective variants for the kind and number of pumps are proposed. Diameters, thickness of walls and material of pipeline are studied. Some additional devices as dam, weirs and ponds are examined in the beginning of HPP and PS. Operating time of pumping station and criterion for cost-effective design is proposed on the bases of objective function for diameters and annual expenses. The results can be used independently as well as for modeling of the next stage, i.e. transient processes and water hammer.

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