Chapter 6
Management issues

SOME GENERAL CONDITIONS FOR A SATISFACTORY OPERATION OF ON-DEMAND SYSTEMS

A large number of on-demand irrigation networks have been developed over the past thirty years. While the majority of them have been found to perform to the satisfaction of farmers and development authorities, there have been a number of cases where the design was below expectations.

A closer analysis of the cases with poor performance records shows that many of the criticisms leveled against on-demand systems are attributable to shortcomings in:

- *Unsuitable size of family holdings and*
- *Low levels of irrigation and farm management knowledge*
- *Non volumetric pricing of water*
- *Incorrect estimation of the design parameters*
- *High energy costs*
- *Lack of qualified staff for the operation and maintenance of the system*

Experience shows that for on-demand systems to give satisfactory results, some basic criteria are needed (FAO, 1990):

Suitable size of family holdings

On-demand systems are probably less suited to certain social environments such as large farms with abundant hired labour or very small holdings run by unskilled farmers. Due attention must therefore be paid to the application of on-demand systems where the size of the farm enterprise is predominantly coherent with the socio-economic context of the project.

In the case of the very large farms with abundant labour, the flexibility afforded by an on-demand system appears to be superfluous since detailed planning of irrigation tasks entrusted to personnel exclusively engaged in this work is possible. Distribution on a rotation basis seems better suited to such a situation, provided that the stream size is sufficient.

On-demand systems are often not well suited to the needs of very small holdings if the farmers have a low level of skill. Here the existence of a rotation provides a strict framework which assures a proper understanding of the quantity of water to be used, as well as the proper frequency of application. Furthermore the extension of the pipe network to every farm becomes expensive and cost repercussions are inevitably high.

Medium to high levels of irrigation and farm management knowledge

On-demand systems were originally placed at the disposal of family holdings on which diversified crops were grown and where farming experience was generally high. These farmers had a high standard of farm management and were eager to make the best possible use of this new tool which allowed for a more flexible integration of irrigation practices with other farming activities and the results were positive. However, other experiences have shown that, when such technology was made available to farmers with little or no knowledge of irrigation and reduced farming experience, long adaptations period with strong training efforts were required until the new technology was used in a satisfactory manner.

Volumetric pricing

Irrigation water must be priced on a volumetric basis if on-demand systems are to operate economically. If water meters are not installed, the farm hydrants will remain open for durations greater than designed and the system will not operate satisfactorily. Furthermore, a high recovery of the water charges is necessary to guarantee a recurrent flow of funds for the operation and maintenance of the system

Correct estimation of the design parameters

When the flows in on-demand irrigation networks are computed through the probabilistic approach proposed by Clément, a performance analysis of the system is strongly recommended during the design stage. Furthermore, three parameters need careful attention: the degree of freedom, e_h, the use coefficient, r, and the operation quality, $U(P_q)$.

The degree of freedom (or elasticity of the hydrants) defines the freedom afforded to farmers to organize their irrigation. The degree of freedom depends on criteria such as size and dispersion of plots, availability of labor, type of on-farm equipment, frequency of irrigation. Hydrants with capacities of one and a half to twice the value of the duty correspond to the lowest feasible degree of freedom. With smaller values, the probability of an hydrant being open becomes too great for the demand model to apply. Conversely, hydrant capacities should not exceed six to eight times the value of the duty. This corresponds to a high degree of freedom.

The use coefficient, r, was already defined and analysed in chapter 3. The values selected for the parameter r normally lie between 16/24 (r=0.67) and 22/24 (r=0.93). The performance analysis of existing systems is the most reliable approach for selecting the coefficient r best suited to a given irrigation network (see Annex 1).

The parameter $U(P_q)$ defines the "operation quality" of the network; it normally has values ranging from 0.99 to 0.95. A significant reduction of this parameter beyond these values can lead to the occurrence of unacceptable failures to satisfy the demand in certain parts of the network (Galand *et al*., 1975).

Reduced energy costs

Many on demand systems were designed at the time that energy costs were considerably lower than nowadays and, therefore, the pipe diameters were reduced as much as possible and compensated with high pressure (at that time cheaper to generate). Such systems have difficulties to operate under principles of financial autonomy and often the public sector has to sustain them with subsidies.

A related problem is that many of the pumping stations that were designed some 20 years ago are integrated by several identical motor pumps that pump the water to a reservoir of fixed height. The result of this configuration is the lack of adaptation of the pumping station (and related reservoir) to a variable piezometric demand that leads to wastage of energy.

The tendency at present is to reduce the pressure to values compatible with the need at the farm level and to optimize the pipe diameter accordingly. For this purpose the use of the tools included in the COPAM package may facilitate the task of adapting old networks to the new conditions.

Qualified staff for the operation and maintenance of the system

The daily operation of on-demand systems is relatively simple and can be mastered by well trained staff in relatively short time. However a deeper understanding of why the network is not functioning satisfactorily requires a good knowledge of some technical concepts that have a certain degree of complexity. A well qualified staff is required and that, in certain environments, may not be easy to obtain. The operation of and maintenance of the pumping stations with its related equipment also requires well qualified staff and a good system of preventive maintenance .

THE ON-DEMAND OPERATION DURING SCARCITY PERIODS

The ever-increasing demand of water for irrigation, together with the growing difficulties and costs for developing new resources, make it necessary to carry out field surveys aiming at a better management of irrigation systems as well as to evaluate specific operation and management decisions. When there are difficulties in matching supply and demand during peak periods managers of irrigation projects can change the delivery schedules from on demand to restricted frequency demand by applying different types of rotations among sectors within an irrigation district.

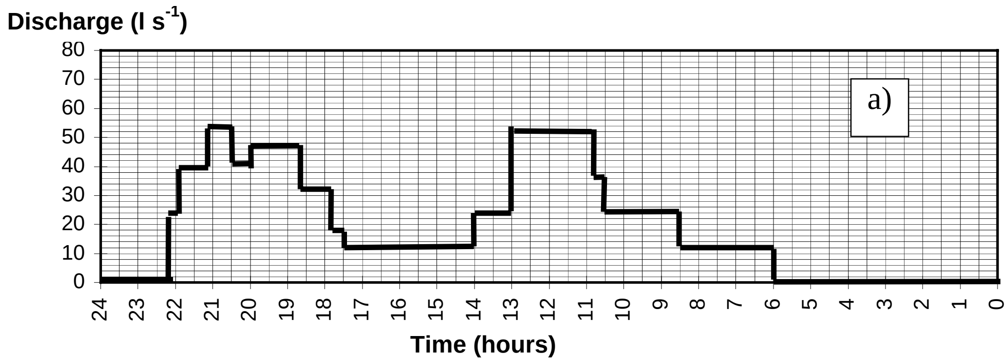
Changes in delivery schedules, however, modify the farmers' behaviour in relation to the withdrawal of water. Often, the rotation among sectors induces all the farmers to irrigate simultaneously when their sector is in charge. In this way, the demand may exceed the upper limit of the discharge allowed in the network, causing an increasing of head losses and, consequently, a reduction in the available pressure at each farm hydrant. Under these circumstances, the on-farm distribution uniformity and application efficiency and, in turn, the global performance of the whole system, is reduced. It is important to evaluate the changes in delivery schedules, particularly concerning the modifications induced in the daily irrigation demands.

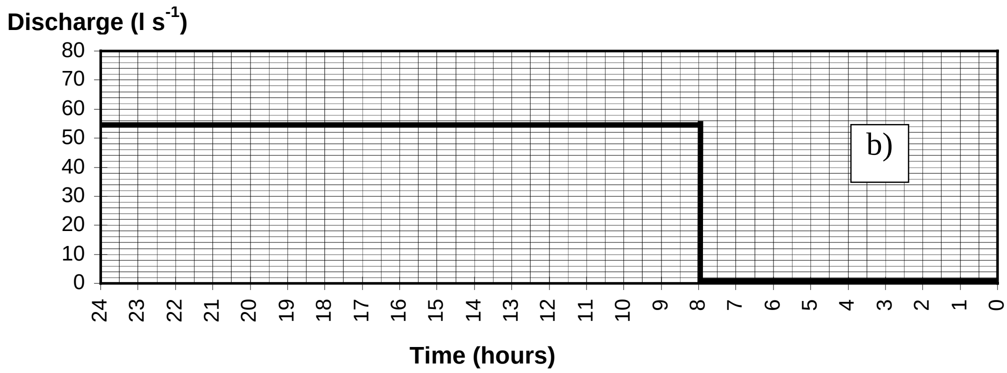
A very common cause inducing managers to modify the delivery schedule is the change of the cropping pattern with respect to the design hypothesis. In fact, when more water demanding crops are introduced in the system, the irrigation demand increases and supply is no longer sufficient to match demand. Under this circumstance, a restriction of deliveries is required. Usually, this consists of a rotation among the irrigation sectors of this district, closing the water supply, alternatively, to 50% of the sectors while maintaining free access to the water to the other 50%. This delivery schedule is called "restricted frequency demand", (Clemmens, 1987).

A survey and analysis were performed by Lamaddalena *et al*. (1996) on an Italian irrigation district. The conclusion was that, when a restricted frequency demand is imposed, the daily withdrawal exhibits abnormal behaviour of farmers in the use of water. On the contrary, this behaviour seems to be more regular during the periods when the operation of the district is on-demand.

In Figure 78, the demand hydrographs recorded at the upstream end of a typical network are reported. From these graphs, it is observed that during the on-demand operation farmers tend to irrigate when they need and according to their habit. On the contrary, when restricted frequency demand is applied, all farmers tend to irrigate simultaneously, during daytime and nighttime, by using the maximum discharge permitted by the network. This behaviour often leads farmers to over irrigate their fields because of uncertainty in water availability. Thus, operation under restricted demand does not necessarily induce water saving but rather an increase in water demand. To solve this problem, rather than maintaining the practice of changing delivery schedule, it is appropriate to develop new operation and management rules.

FIGURE 78
Typical demand hydrographs at the upstream end of an irrigation sector: a) on-demand operation; b) arranged demand operation





Shortages of the existing irrigation systems are satisfactorily, or at least acceptably, solved by inducing single farmers to modify the flow hydrographs, according to the global capacity of the irrigation system. This approach is expected to be better than the drastic 50% rotational-reduction between sectors.

In view of this, improving distribution and reducing water consumption by each farmer from the installation of new delivery equipment to avoid wastes or unuseful concentrated withdrawals without penalizing the on-demand operation is beneficial. In fact, these devices, opportunely programmed to limit the delivery volume and/or to limit withdrawals during the daily peak periods, may stimulate farmers to modify the flow hydrographs in such a way to be compatible with the system capacity.

Trials were made on some irrigation systems in Italy to check the reaction of farmers to such limitations. They responded very well and accept such shortage much more willingly than the drastic rotation among sectors. This approach is still under study for identifying exactly which hydrants need to be controlled. Also, the possibility of reducing the night-time and increasing the day-time tariffs is under consideration.

The extension service is vitally important for the success of introducing new technologies (Tollefson and Wahab, 1995). In fact, the extension service is responsible for simplifying research information and transfer it to farmers in an easy and understandable way. The extension service should also provide a feed-back mechanism to researchers on farmers' problems so that a two-way information exchange approach is maintained. Seminars between extension researchers, irrigation agencies industry representatives and farmers, together with cooperative research activities are needed for improving the two-way approach.

The extension service should carried out activities aimed at improving the use of the water resource through the simple and systematic dissemination of the information drawn from the best literature on irrigation, with special reference to more accurate estimates of crop water requirements by the collection of agro-meteorological data of the stations densely located on the territory. The agro-meteorological data should be archived in a data bank and disseminated by a responsible agency. For example, weekly bulletins may be prepared and adequately disseminated through the participation of associations, cooperatives, entrepreneurs and other technical offices.

A quality jump in the dissemination of information is possible through a dedicated INTERNET site supporting an interactive programme using the area of the farm, data on the crops to be irrigated, hydraulic characteristics of the on-farm system, the water volume to be supplied and, in the case of trickle irrigation systems, the irrigation time, on the basis of the agro-meteorological data taken from the station closest to the farm, may be obtained.

VOLUMETRIC WATER CHARGES

The use of volumetric water charges in on–demand systems is indispensable. Otherwise, farmers would tend to leave open the hydrants for a period greater than needed and this will hamper the operation of the system and eventually generate salinity and waterlogging problems that will reduce the productivity of the land. In order to be able to measure the water that has been used every hydrant is equipped with a measuring device or water meter that registers the volume of water consumed. Volumetric charges are not only important in terms of the functioning of the system but is a socially fair method in the sense that the farmer pays for what he/she uses and represents an incentive for water conservation.

In most of the on demand irrigation systems every farm is equipped with one (or more) hydrant but when the farms are relatively small providing an hydrant for every farm becomes too expensive and several farms use the same hydrant. In such cases the volumetric reading correspond to a group of farmers and the volume consumed has to be distributed among them The easiest way is that every farmer reads its consumption and provide this information to the management of the system. Usually there are some differences between the sum of the individual readings and the total reading for the period considered and the management distributes the difference among the group of farmers using some agreed criteria of proportionality (area or water consumed). In the rare cases that the group of farmers do not agree to provide their individual reading the management has no other practical option that to distribute the whole consumption according to some criteria (estimated consumption of the crops planted, area, or other).

Another way to solve the problem of sharing volumes among the farmers using the same hydrant is to use new delivery equipment where withdrawals are controlled by electronic card. A more detailed description of these devices is reported hereafter.

WATER METERS

Every hydrant is normally equipped with a water meter and other accessories (the pressure regulating valve, the flow limiting device and the cut-off valve). There are many types of water meters but those more commonly used in irrigation networks are of the propeller type where the turbine rotates at the a speed that is proportional to the flow velocity (Figure 79). The reader is normally of the cumulative type[1] so that the volume of water used in a given period is the result of the difference between the two readings (end and beginning of period). One possible problem of such system is that the farmer may realize at the end of the period that he/she has used more water than actually needed. To avoid overlooking the actual consumption he/she should annotated how much water is being used in every irrigation to avoid surprises at the end, however this is not a common practice.

The accuracy of the mechanical water meters is limited (normally with reading errors around 2 %) and the accuracy decreases with the time as the moving parts are subject to wear. For this reason water meters require frequent maintenance to keep their accuracy.

TYPES OF VOLUMETRIC CHARGES

The most common method of volumetric charging is to apply a unit value to the cubic meter of water provided such as US$ 0.10/m^3. Such value is normally calculated by dividing all the operation, maintenance and administration costs by the total number of cubic meters provided by the system. Obviously this value changes from year to year as both components of it can change considerably. Sometimes the unit value also includes an additional component which main purpose may be the creation of a reserve fund for rehabilitation or other purposes.

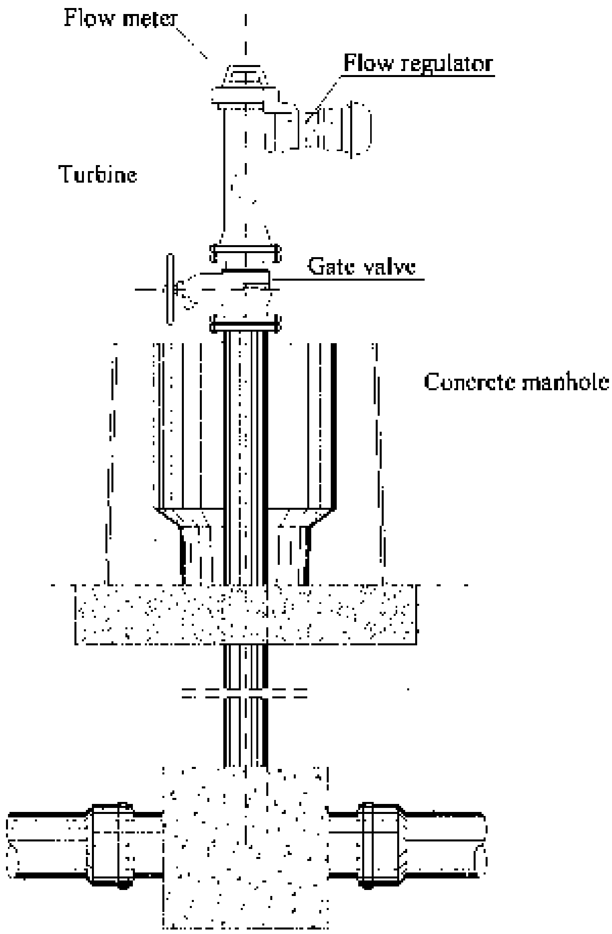
It is also frequent to apply *volumetric charges by blocks* in order to keep consumption within reasonable limits. When such method is applied the consumption which is considered normal is cost at a fair price, for instance for the first 5000 m^3 ha^{-1}, US$ 0.10/m^3 for consumption between 5000 and 7000 m^3 ha^{-1} the price may increase to US$ 0.13/m^3 and for any consumption beyond 7000 m^3ha^{-1} the price may jump exponentially for instance: US$ 0.25/m^3, to reduce the chance that farmers may use water beyond this threshold. However the application of such method requires a detailed analysis of the crops prevailing in the system as some crops may need more water than others and the farmer should not be penalized for it. Applying the volumetric block pricing indiscriminately may lead to changes in the cropping pattern which may, or may not, be desirable.

[1] Some water meters give the instantaneous flow as well.

RECOVERY OF INVESTMENTS THROUGH THE VOLUMETRIC CHARGE

Countries practices different policies for the recovery of investments made in he irrigation systems. Such policies vary from of total subsidy to those that try to recover all the costs. Assuming that a country desires to recover at least a part of the investments the question arising is whether such costs should be included or not in the volumetric unit value applied. From a theoretical point of view the answer is it should not be included because that would lead to the inconsistency that those farmers planting crops with high water requirements will end paying more for the investments than those planting crops with low requirements. In the extreme a farmer that would not use any water a given year he/she would contribute nothing to the recovery of investment. Nevertheless, there are some few cases where for reasons of simplicity the recovery of investments is included as one more component of the volumetric unit value applied.

FIGURE 79
Cross-section of a hydrant



A more rational approach would be to recover the investments though a charge that is applied on per hectare basis while the variable costs (operation, maintenance and administration) are recovered through unit value applied for the cubic metre of water.

THE OPERATION, MAINTENANCE AND ADMINISTRATION COSTS

In a traditional surface irrigation system most of the operation and maintenance costs are constituted by the staff costs. In fact in open canal systems a large number of staff must dedicate their time to the opening and closing gates either to regulate the flows or to distribute the water to the farms. In an on-demand system such expenditures are very much reduced and the staff concerned with the actual operation of the system is limited to the pump stations operators and some supervisory staff at the central level.

Another advantage of on demand systems with respect to the traditional surface irrigation systems is that practically eliminates the conflicts between the staff responsible for distributing the water and the farmers that should receive the water, as is the farmer the person that open and closes the hydrant.

On the other hand the largest expenditure of on-demand systems is represented by the energy bill as the whole irrigation pipe network must be under pressure. Few on-demand systems have the fortune of having available the necessary pressure under natural conditions. One such case is The South Conveyor System of Cyprus, which is designed for on-demand operation, and is connected to the dam outlet where the topographic elevation permit generates enough pressure to keep the network working under the necessary conditions. In the rest of the cases the energy bill remains the most important component reaching values that represent 55-70% of the total O&M costs. Therefore if savings can be made in the energy required without sacrificing the quality of the operation the net savings for the farmer can be important. In this sense the analysis of the performance described in Chapter 5 can be useful to diagnose how well designed the system is and in particular the effectiveness of the pumping station.

In general the water charges due to O&M are somewhat higher than in open canals systems but the quality of the water service is clearly superior. The fact that the farmer can apply the water when the crops needs it has a positive impact in the yields which clearly offset the greater water charges.

In any case, the impact of the water charges in the total production costs of a given crop are in general small (particularly if recovery of investments is not included) and often they are less than 5 % and rarely exceed 10 %.

THE MAINTENANCE OF THE PUMPING STATIONS AND RELATED DEVICES

The ordinary maintenance of the pumping station concerns the equipment and all civil engineering works in the plant. Some general rules to ensure an adequate maintenance are listed hereby.

Flow and pressure meters:

- Checking, at least once per day, the transmitter efficiency, to be sure that the quantity recorded by the recorder coincides with that measured by the instruments;
- Checking, at least once every three months, the calibration of equipment and execute adjustment if required;

Pumps:

- Regulation of the electric pump operation in relation to water discharge
- Inspection, at least once per day, of electric pumps to ascertain any misfunctioning (either clogging or blocking of the impeller, jamming of flap valve, leak in the delivery pipe, defective float switches, etc…)
- Checking, at least once per day, the regular position of floats or sensors as well as their proper operation to start and stop the electric pump itself;
- Checking, at least once per month, the degree of wear of the electric pump mechanical equipment and fittings
- Periodical cleaning has to be provided to ensure the proper operation of all lifting stations, including stand-by pumps.

Compressors:

- Inspecting, at least once per day, the electric compressors and the pipeline of pneumatic circuits to be sure that there are no air leaks;
- Checking, at least once per month, the perfect efficiency of electric and pneumatic circuits and of their organs with special reference to the pressure gage.
- Checking, at least once per month, the degree of wear of the electric compressor mechanical organs;
- Checking and cleaning, at least once per month, the suction filter.

Switchboards:

- Inspecting, at least once per month, the switchboards and execute repairing if required.
- If the inverter is installed, the switchboard has to be placed in a room with air conditioning system in order to avoid damages to it, due to air temperature.

Ancillary works:

- Annual painting of metal structures;
- Two-yearly painting of all building works;
- Periodical manoeuvres of existing gates and valves in the system to ensure their proper operation.

THE MAINTENANCE OF THE IRRIGATION NETWORK

In general the maintenance of the irrigation network does not require major maintenance works if the construction was done according to specifications and suitable materials were selected. Nevertheless possible problems arising from certain characteristics of the water quality (abrasion, high calcium carbonate, etc..) may need particular attention.

The most common problem of the pipe networks is the appearance of leaks mostly due weathering of the joints or misplacement of them. Accidental breakage of pipes sometimes happens due to the use of earth movement equipment in construction sites near the pipes.

The most critical point in the whole network is the hydrant and all the accessories included there. Hydrants must be reliable and robust to avoid the possible manipulations of farmers which may alter their characteristics.

The most common problem is their partial or total breakage due to wrong operations with tractors or other agricultural machinery. To avoid such problems is a good practice to protect them with some concrete structures placed around them.

TOWARDS THE USE OF TRICKLE IRRIGATION AT THE FARM LEVEL IN ON-DEMAND IRRIGATION SYSTEMS

Many on-demand irrigation systems originally designed for sprinkler irrigation are gradually being replaced by trickle irrigation methods. The implications for on-demand systems are several:

- the pressure head required is considerably lower than for sprinkler methods. It leads to improve the hydraulic performance of the system when pressure deficits occur at the hydrant level for on-farm sprinkler methods;
- trickle methods require daily operation and this has some implications in the design parameters of the system (see the analysis in Annex 1);
- the daily required discharge is generally lower than for sprinkler methods. It leads to lower head losses in the system and, consequently, higher pressure heads at the hydrant level.

This progressive change in many on-demand systems has positive implications for the operation of the system and the analysis of such situations can be facilitated by the COPAM package.

NEW TRENDS AND TECHNOLOGIES OF DELIVERY EQUIPMENT

The ever-increasing water demand for irrigation, together with the growing difficulties and costs of developing new resources, make it necessary to assist managers of irrigation systems for a better use of water. In this perspective, the new technologies may play an important role in improving both sound water use and management activities. In particular, new delivery devices have been developed in the last years (Antonello *et al*., 1996; Megli, 1998) based on microprocessor systems that allow to regulate water withdrawals. They can be programmed with a number of functions, are mechanically resistant, reliable and not expensive.

These delivery devices are installed in the field and can be activated by an electronic card used by the farmer.

The unit located in the field is composed of a microprocessor, a hydraulic group consisting of a water meter with pulse emitter and a hydrovalve, an impulsive electric valve, a ring flow limiter, a delivery connection pipe, a stainless-steel box (see Figure 80). It is supplied with a lithium battery having 10-15 year lifetime under normal operating conditions.

The electronic card is composed by a plastic box in which a microprocessor, a real time clock, an alphanumeric display and two selection buttons are located.

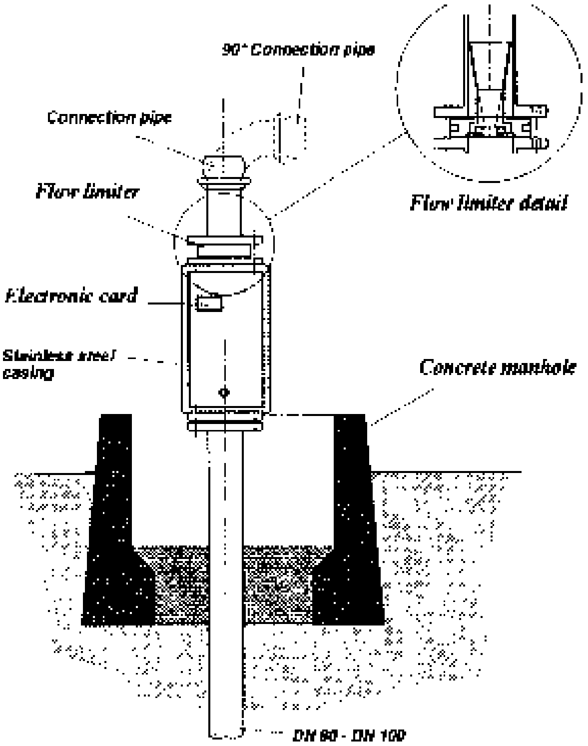
Each electronic card is programmed (by the manager, through a user-friendly software package) in the management office at the beginning of the irrigation season. The seasonal available water volume may be pre-loaded on each card, as well as the maximum daily volume to be withdrawn and/or the maximum daily operating time.

The card may be removed during irrigation after the opening signal is transmitted. The closure will be done automatically in case of using the maximum daily volume or in case of exceeding the maximum operating time. In this way, nobody can steal or remove the card during its operation.

When farmers tend to withdraw too much water during the peak hours, the operating daily time interval may be pre-fixed, as well as the turn at the hydrant level. It will lead farmers to modify the flow hydrographs in such a way to be compatible with the system capacity.

After each irrigation, the residual volume appears on the display of the card. In this way the farmer may realize immediately if he/she is using more water than that actually needed by the crops.

FIGURE 80
Scheme of a delivery device controlled by an electronic card



The opening and closing time of the delivery device may be recorded on the card. It allows to define different tariff rules, for example, by reducing the night-time water price with respect to the day-time one. This solution may lead to a better distribution of deliveries during the day by avoiding excessive withdrawals during the daily peak hours.

Each electronic card is coded and, therefore, several farmers can take water from the same group by using different cards with different codes. In this case, no problems of water sharing among farmers will occur.

Finally, when such technology is used, the management activity is strongly simplified. In fact, farmers have an interest to address to the managing agency (not the opposite, as it happens when classical hydrants are used) for declaring and showing on the card, the volume of water effectively used. Should they not address to the agency, they will be charged for the total volume pre-loaded on the card at the beginning of the irrigation season, even if they used less. Once the farmers exhaust the pre-loaded volume and need an additional one, they have to address again to the managing agency for requesting it. The manager can take the decision to supply the requested volume, if available, and he may also decide to charge a different price for it.

These devices are successful installed and used in some Southern Italy Irrigation schemes (Altieri *et al*., 1999).

These types of delivery devices may greatly contribute to solve the problem of continuous and systematic interfacing between Irrigation Authorities and farmers. Managers of irrigation systems may also benefit, especially under conditions of limited water availability. Furthermore, a large number of reliable data may be available through these devices and they may be used both by managers and researchers for improving their knowledge on irrigation systems behaviour.

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Annex 1
Validation of the 1st Clément model

The Clément model, like all other models, only offers a schematic representation of an actual network. Therefore, it must be adjusted and/or calibrated by introducing field data relative to existing networks. In particular, values of the parameter "r" should be, whenever possible, selected for homogeneous regions and for particular crops. An example of the field calibration of the Clément model, for an Italian irrigation network, is reported below, after a short presentation of the theoretical approach.

THEORETICAL APPROACH

Remembering the following relationship (see the first Clément model in chapter 3):

$$p = \frac{q_s A}{r R d} \tag{A1}$$

the relationship

$$Q_k = R\,p\,d + U(P_q)\sqrt{R\,p\,(1-p)\,d^2} \tag{A2}$$

becomes[1]:

$$Q_k = \frac{q_s A}{r}\left[1 + U(P_q)\sqrt{\frac{rd}{q_s A} - \frac{1}{R}}\right] \tag{A3}$$

1

$$Q_k = d\left[R\frac{q_s A}{rRd} + U(P_q)\sqrt{R\frac{q_s A}{rRd}\left(1 - \frac{q_s A}{rRd}\right)}\right] = d\left[\frac{q_s A}{rd} + U(P_q)\sqrt{\frac{q_s A}{rd}\left(\frac{rRd - q_s A}{rRd}\right)}\right]$$

$$= d\left[\frac{q_s A}{rd} + U(P_q)\sqrt{\frac{q_s A\, r\, Rd - (q_s A)^2}{r^2 d^2\, R}}\right] = d\left[\frac{q_s A}{rd} + U(P_q)\sqrt{\frac{q_s A}{r\,d} - \frac{(q_s A)^2}{r^2 d^2\, R}}\right]$$

$$= d\left[\frac{q_s A}{rd} + U(P_q)\sqrt{\frac{q_s A}{r\,d} - \left(\frac{q_s A}{r\,d}\right)^2 \frac{1}{R}}\right]$$ by taking out the term $\frac{q_s\ A}{r\ d}$ we have:

$$Q_k = d\frac{q_s A}{rd}\left[1 + U(P_q)\sqrt{\frac{r\,d}{q_s\ A} - \frac{1}{R}}\right]$$

This formula indicates that the discharge during the peak period fits a Gaussian distribution having mean

$$\mu_{th} = \frac{q_s A}{r} \qquad \text{(A4)}$$

and standard deviation

$$\sigma_{th} = \frac{q_s A}{r} \sqrt{\frac{r\,d}{q_s A} - \frac{1}{R}} \qquad \text{(A5)}$$

where the subscript "th" indicates these are theoretical values.

To verify the applicability of the 1st Clément formula a recording flow meter is installed at the upstream end of the network. From the discharges recorded in field during the peak period, selecting the classes of discharges and the appropriate time interval for the observations, it is possible to build up the histogram of frequencies (see examples in Figures A4 and A5).

A histogram gives the number NQ_i of discharges included in each class, whose central value is Q_i. Then the mean (μ_{exp}) and the standard deviation (σ_{exp}) are computed:

$$\mu_{exp} = \frac{\sum_{i=1}^{NQ_T} NQ_i \; Q_i}{NQ_T} \qquad \text{(A6)}$$

$$\sigma_{exp} = \sqrt{\frac{\sum_{i=1}^{NQ_T} NQ_i \; (Q_i \; - \; \mu_{exp})^2}{NQ_T - 1}} \qquad \text{(A7)}$$

where $NQ_T = \Sigma_i \, NQ_i$ and the subscript "exp" indicates that these are experimental values.

From all the discharges withdrawn in the field during the peak period, it is possible to identify one or more populations of discharges[1]. For the population which characterizes the peak water use, it is possible to estimate the parameter r. This parameter behaves like an adjusting parameter (CTGREF, 1977; Bethery 1990).

To compute r, it is assumed that

$$\mu_{th} = \mu_{exp} \qquad \text{(A8)}$$

$$\sigma_{th} = \sigma_{exp} \qquad \text{(A9)}$$

The following steps are required. First, μ_{exp} is computed from all experimental values relative to the whole peak period of 10 days and to the whole day (from 0 to 24 hours). This leads to a first estimation of the mean specific continuous discharge (q_s) withdrawn 24/24 hours. From the relationships (A4), (A6) and (A8) we have:

$$\left[\mu_{exp}\right]_0^{24} = \frac{\sum_{i=1}^{NQ_T} NQ_i \; Q_i}{NQ_T} = \frac{q_s A}{r} \qquad \text{(A10)}$$

---

[1] As for the case study reported hereafter two populations were identified, one concerning daytime irrigation, the other nighttime irrigation.

Once t'/t = 24 hours, in this first approach r = 1. Therefore, the specific continuous discharge is computed:

$$q_s = \frac{\left[\mu_{exp}\right]_0^{24}}{A} \tag{A11}$$

In a second step, the theoretical and the experimental standard deviations are calculated through the relationships (A5) and (A7) respectively. These deviations being different, their relative difference is then computed:

$$\delta\sigma = \frac{\sigma_{exp} - \sigma_{th}}{\sigma_{exp}} \tag{A12}$$

When $\delta\sigma >> 0$, since the objective is to characterize the operation of the system for the peak period, the night-time discharges can be excluded. A new histogram is then built for the discharges observed during a period smaller than 24 hours (i.e., between the hours t_{in}= 4 a. m. and t_{fin} = 12 pm, of each day during the peak period). Thus, new values for NQ_i and n result in a new $\left[\mu_{exp}\right]_{t_{in}}^{t_{fin}}$ (Eq. A6). Since q_s is now known (Eq. A11) it is possible to compute r:

$$r = \frac{q_s A}{\left[\mu_{exp}\right]_{t_{in}}^{t_{fin}}} = \frac{\left[\mu_{exp}\right]_0^{24}}{\left[\mu_{exp}\right]_{t_{in}}^{t_{fin}}} \tag{A13}$$

Then, new values for σ_{th} and σ_{exp} are computed (Eqs. A5 and A7 respectively) and a new $\delta\sigma$ is calculated. An iterative procedure is then performed until the minimum value for $\delta\sigma$ is obtained. The corresponding value for r is then retained as the best estimation for the adjustment of the Clément formula.

Subsequently, through the integral curve of the Gaussian distribution, it is possible to determine, at the maximum recorded field discharge, the value of the cumulative probability and, consequently, the experimental $U(P_q)$.

APPLICATION

The calibration of the first Clément formula was performed for an Italian irrigation system: the district 4 of the "Sinistra Ofanto" irrigation scheme (Lamaddalena and Ciollaro, 1993), in the province of Foggia (Italy), run by the Consorzio di Bonifica of Capitanata (CBC, 1984). The scheme (Fig. A1), covering an area of about 22500 ha, is approximately triangular-shaped, confined in the south by the Ofanto river and in the southeast by the town of Cerignola. The system is divided into seven irrigation districts (numbered 4 to 10) which are, in turn, subdivided into sectors with surface areas ranging from 20 ha to 300 ha.

The irrigation districts are served by a storage and daily compensation reservoir supplied by a conveyance conduit, which originates from the Capacciotti dam (Fig. A1). The pressurized irrigation network in each district originates from these reservoirs and is designed for on-demand delivery scheduling (Malossi and Santovito, 1975).

District 4 (Fig. A1) was chosen for the survey since a calibrated flow meter is available at the upstream end of the distribution network. District 4 has a topographic area of 3256 ha, and is supplied by a storage and daily compensation reservoir having a capacity of 28000 m^3 where the maximum water level is 143 m a.s.l. and the minimum water level is 139 m a.s.l.

FIGURE A1
The "Sinistra Ofanto" irrigation scheme

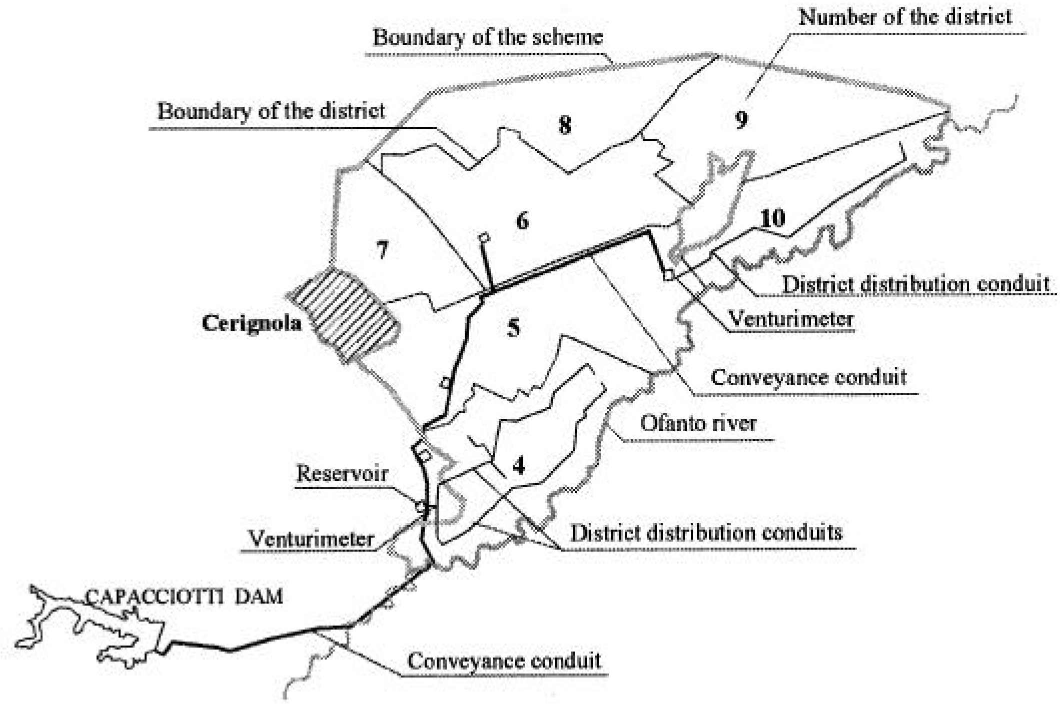
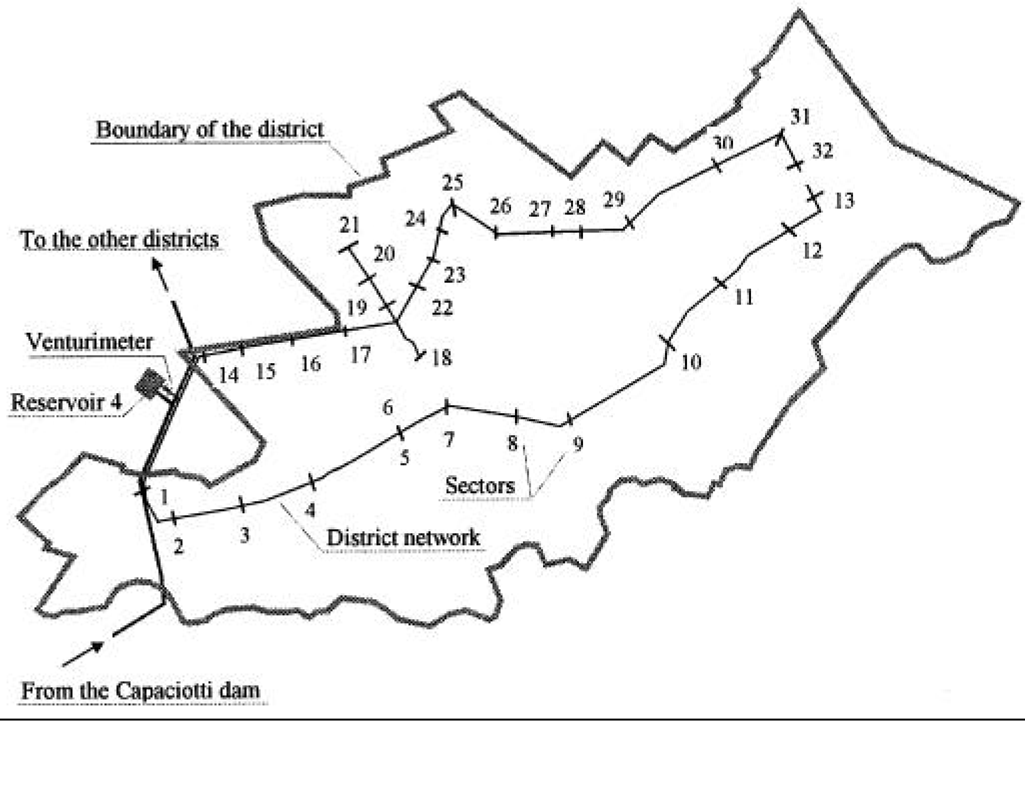


FIGURE A2
Layout of the district 4 network



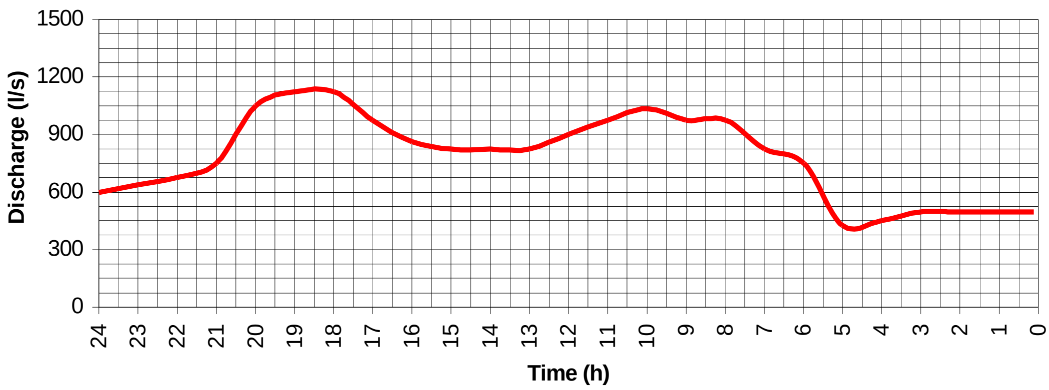
The district distribution conduits consist of underground steel pipes starting with a diameter of 1200 mm. This conduit supply 32 sectors distribution systems (Fig. A2). A control unit is installed at the head of each sector and consists of a gate, a Venturi meter with recorder and a flow regulator. The sectors distribution networks serve the farms with hydrants having nominal discharge of 10 l s^{-1}. The District 4 network is composed of 903 nodes of which 660 with hydrants.

The discharges have been calculated with the Clément model while the diameters of the district distribution conduit have been calculated by applying linear programming techniques.

The calibration was performed for the year 1991 because during this year the district 4 irrigation system operated on-demand.

Discharge measurements using the recording Venturi meter were performed (Figure A3) at the inlet cross-section of the network, immediately downstream of the reservoir .

FIGURE A3
Example of the discharges observed at the upstream end of the district 4



The irrigated areas and the corresponding crops grown during the 1991 irrigation season were calculated from the water user files available at the Irrigation Board. These data are reported in Table A1.

TABLE A1
Cropping pattern in the irrigation district during the 1991 irrigation season and scheduled in the year 1975

	SCHEDULED (1975)	SCHEDULED (1975)	1991	1991
CROPPING PATTERN	IRRIGATED AREA (ha)	IRRIGABLE AREA (ha)	IRRIGATED AREA (ha)	IRRIGABLE AREA (ha)
Vineyards	444.0	444.0	1325.9	1325.9
Olive trees	1149.0	1149.0	424.9	424.9
Orchards	21.0	21.0	71.2	71.2
Almond trees	------	------	5.0	5.0
Tomato	------	------	118.3	118.3
Potato	------	------	15.0	15.0
Asparagus	------	------	116.1	116.1
Vegetables	------	------	16.1	16.1
Wheat	416.0	925.0	-------	610.6
TOTAL	2030.0	2539.0	2092.5	2703.1

On the basis of the recorded discharges q(t), the daily volumes, V_d (m^3), withdrawn by farmers during the observed months were computed from the relationship

$$V_d = \int_0^{24} q(t)\,dt \qquad \text{(A14)}$$

In order to determine the seasonal peak period, the moving average method has been applied to the daily volumes, V_d, for periods of 5, 7 and 10 days. For these three time steps there are two peak periods falling at the end of June and at the first ten days of August. In both cases the delivered volumes (Table A2) are quite close, so in this case it was not possible to determine in a unique way the real peak period.

TABLE A2
Average value of the volumes (m^3) withdrawn during different peak periods

PEAK PERIODS	MOVING AVERAGE 5-DAY BASIS (m^3)	MOVING AVERAGE 7-DAY BASIS (m^3)	MOVING AVERAGE 10-DAY BASIS (m^3)
June 24 - July 3 August 3 - August 12			62223 61992
June 25 - July 1 August 4 - August 10		64030 64472	
June 25 - June 29 August 5 - August 9	64128 64719		

The method described in the previous section was applied to the 10-day peak periods identified in Table A2. After trying other values, the classes of discharges were defined for ranges of 60 l s^{-1} (0÷60; 60÷120; 120÷180; and so on). The histograms of frequencies for the two peak periods (Figures A4a and A5a) were developed considering hourly data (between 0 a.m. and 12 p.m.).

For the 10-day peak period from June 24 to July 3, the specific continuous discharge computed from the Equation (A11) is q_s = 0.340 l s^{-1} ha^{-1}. Results of computations for shorter intervals within the day are given in Table A3 and in Figure A4b. When $\delta\sigma$ reduced to 0.06 the parameter r became r=0.86. The peak specific discharge was $q_p = q_s/r$ = 0.395 l s^{-1} ha^{-1}.

Similar results have been obtained considering data within the seasonal 10-day peak period ranging from August 3 to August 12 (Figure A5). In this case, the specific continuous discharge resulted to be q_s = 0.350 l s^{-1} ha^{-1}. Results of computations for shorter intervals within the day are given in Table A4 and in Figure A5b. When $\delta\sigma$ reduced to 0.01 the parameter r became r=0.90. The peak specific discharge resulted to be $q_p = q_s/r$ = 0.389 l s^{-1} ha^{-1}.

TABLE A3
Value of the r parameter obtained for different time intervals. Period from June 24 to July 3 (q_s = 0.340 l s^{-1} ha^{-1})

Initial Time	Final Time	μ_{exp}	μ_{th}	σ_{exp}	σ_{th}	$\delta\sigma$	r
0	24	708.15	708.15	278.95	194.76	0.30	1.00
2	24	734.86	734.86	268.73	197.95	0.26	0.97
4	24	766.97	766.97	251.91	201.67	0.20	0.93
6	24	807.30	807.30	222.40	206.19	0.07	0.88
8	24	823.67	823.67	220.70	207.97	0.06	0.86
10	24	812.61	812.61	224.60	206.77	0.08	0.88

FIGURE A4
Discharge frequency histogram. Period from 24 June to 3 July

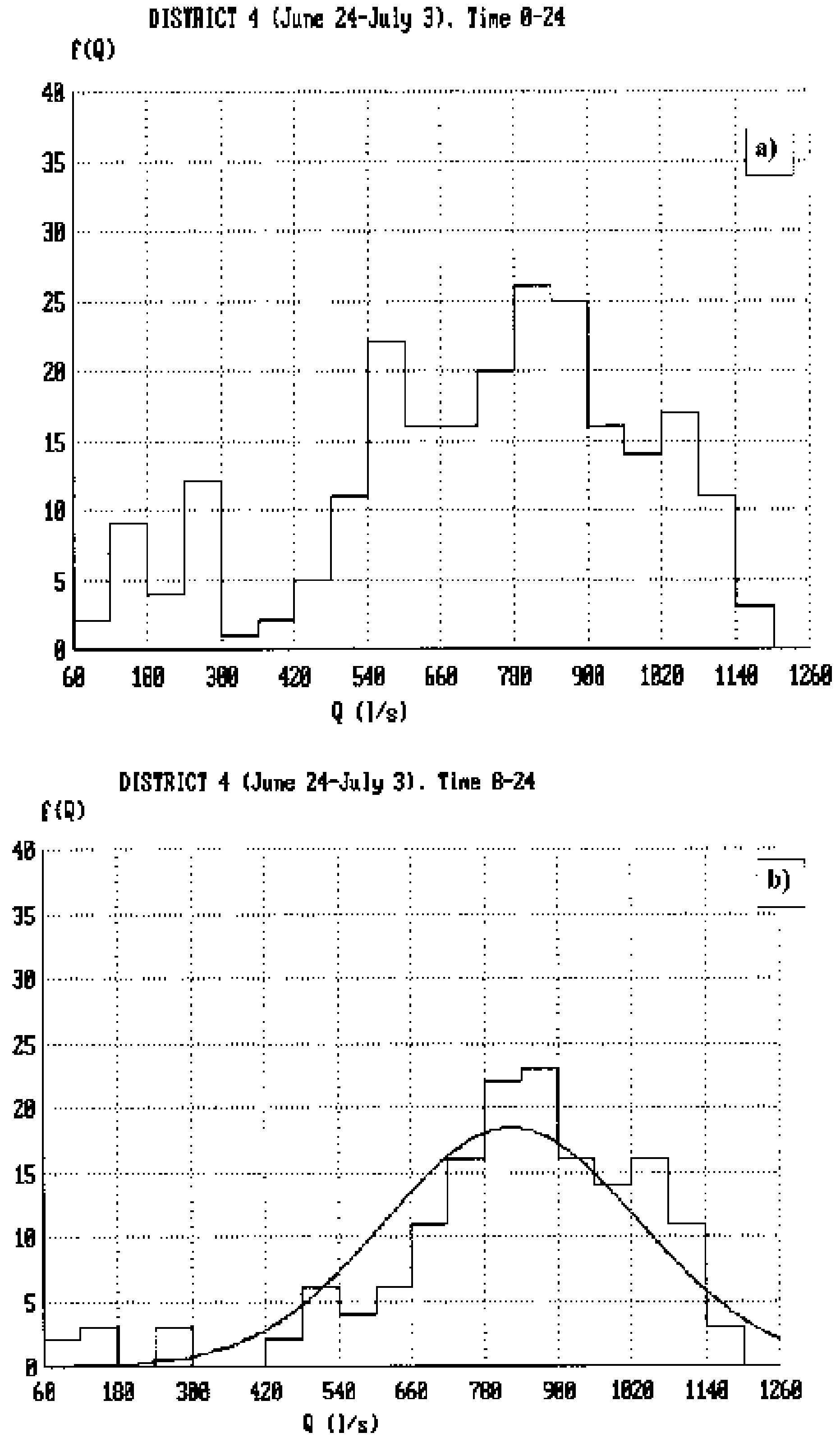


FIGURE A5
Discharge frequency histogram. Period from 3 August to 12 August

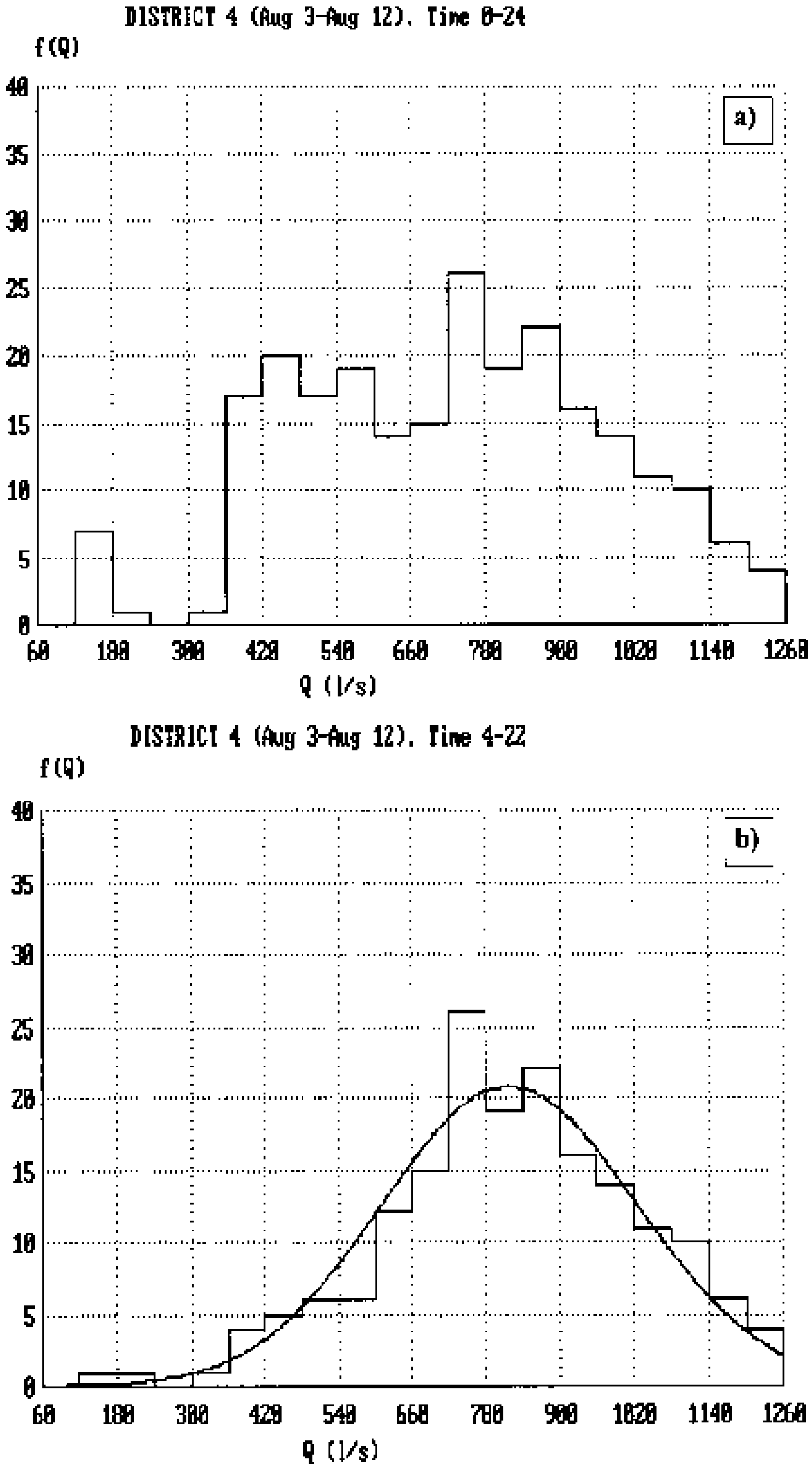
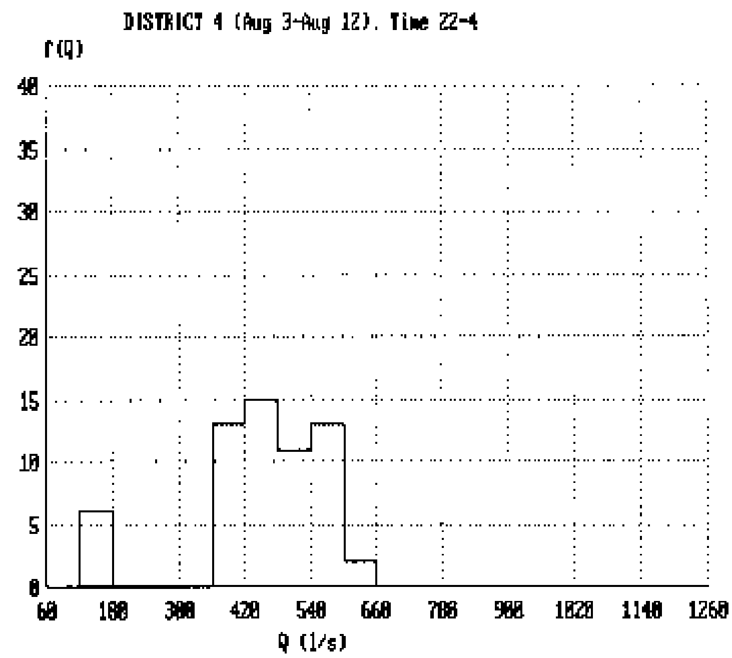


TABLE A4
Value of the r parameter obtained for different time intervals. Period from August 3 to August 12 (q_s = 0.350 l s^{-1} ha^{-1})

Initial Time	Final Time	μ_{exp}	μ_{th}	σ_{exp}	σ_{th}	$\delta\sigma$	r
0	24	724.90	724.90	247.73	196.77	0.21	1.00
2	24	751.64	751.64	239.24	199.91	0.16	0.97
4	24	783.47	783.47	225.19	203.54	0.10	0.93
6	24	811.01	811.01	214.93	206.59	0.04	0.90
8	24	803.21	803.21	219.09	205.74	0.06	0.91
4	22	817.04	817.04	207.87	207.25	0.01	0.90

FIGURE A6
Discharge frequency histogram. Period from June 24 to July 3. Night-time withdrawals



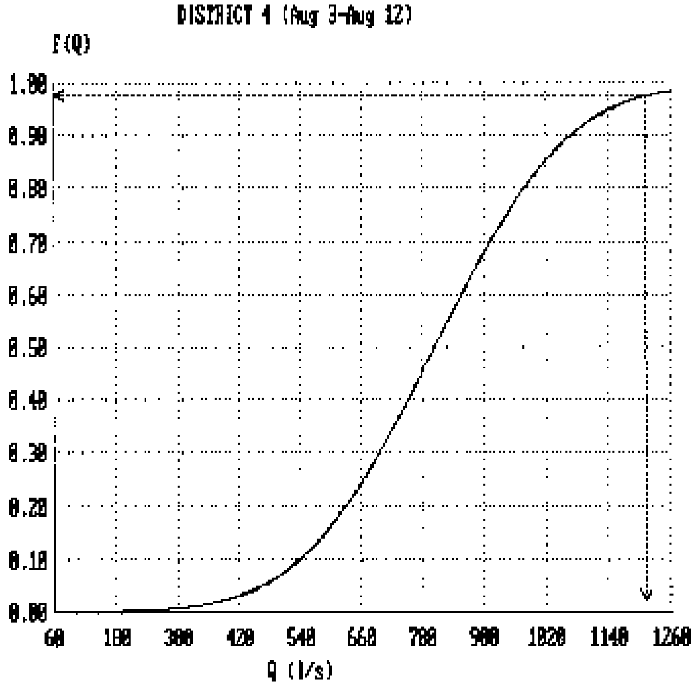
In particular, there is a good fit between the theoretical Gaussian curve and the histogram of frequencies obtained using the field data during daytime water withdrawals (Figures A4b and A5b). It means that the population of the discharges during this period is well represented by the Clément formula.

However, it is evident that the elimination of night-time data implies that night-time and daytime data belong to independent populations of discharge (which is evidenced when comparing Figures A5b and A6).

Considering that, in the study area, not all farms are equipped with automated irrigation systems and only few farmers prefer to irrigate during the night-time (for obtaining higher pressure), it is possible to consider the two populations as independent (night-time data and daytime data). Then the proposed approach seems valid. Clearly, it is necessary to verify every time through field data if the hypotheses on the basis of the Clément method are correct.

It is important to note that the minimum variation of standard deviation corresponds to a certain laps of time, t' (i.e., 16 hours for the 10-day peak period from June 24 to July 3),

FIGURE A7
Cumulated probability. Period from 3 to 12 August.



whereas the value of r does not correspond to the value of the ratio t'/24. Therefore, it is evident that the r coefficient should be intended only as a calibration coefficient aiming at understanding the farmer's behaviour. An irrigation system operating on-demand has to work 24 hours per day to allow for different withdrawals and to avoid dangerous simultaneously opening of hydrants.

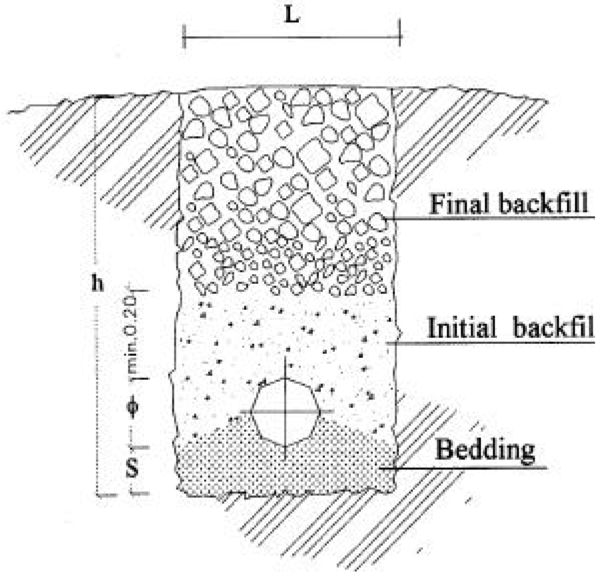
From the above analysis, it is possible to calculate the integral curve, P_q, of the Gaussian distribution (Figure A7) and, consequently, to determine the $U(P_q)$ parameter of the lément formula. For the peak period between August 3 and 10, i.e., the value of cumulated probability corresponding to the discharge of 1210 l s^{-1} (maximum discharge recorded in field) resulted to be P_q = 0.976 corresponding to $U(P_q)$ = 1.98 (Figure A7).

Using a field survey, the r coefficient was calculated to clarify it's meaning. For the studied case, r was close to 1 (r=0.86 and r=0.90 respectively for June 24 to July 3 and August 3 to August 12). The computed r is higher than the design value, r = 0.67. This means that a change occurred in the farmer's behaviour. In fact, the actual cropping pattern is different (Table A1) and more water is demanded than the designed pattern. Thus, the peak discharge should be higher. In order to avoid higher discharges in the network, with consequent failure of the hydraulic performance of the system, the farmers changed on-farm irrigation methods from sprinkler to trickle irrigation and have often adopted automated equipment. The water withdrawals became better distributed along the day (Figure A3), with less concentration in some hours. This is possible because the system operates 24 hours per day. Moreover, using field calculations, the Clément operation quality is $U(P_q)$=1.98. This corresponds to P_q = 97.6% (more than the designed value P_q = 95%), and it implies a lower probability of exceeding the maximum discharge.

Annex 2
Pipe materials and design considerations

There are several advantages in using pipelines for conveyance in water distribution projects as well as for distribution on farm. Because of their capacity to transmit pressure, pipelines facilitate the use of flexible schedules. In fact, a farmer can open a valve on his farm in response to his irrigation need. This action can initiate a distant reaction to start flow from a gravity source or to start up a pump to satisfy the need in frequency, rate and duration of water flow. Essentially a pressure pipeline system is an automated system for transmitting and carrying out precise instructions. Furthermore, the use of pipelines in a conveyance system considerably reduces evaporation and seepage losses. Because they do not have to follow contours, pipelines can be laid with straight alignments and go up and down hills. Also, with the use of pumps and lifting plants, pipelines can convey water to higher elevation. In addition, the required right-of way width is considerably less relative to canals. The trench cross section and some suggested trench dimensions are reflected in Figure A8 and in Table A5.

FIGURE A8
Trench cross-section



Since it is very difficult to make changes once pipelines have been installed, the designer must use great care when selecting the type of pipe, diameters and materials to insure that the initial installation is technically and economically acceptable and not limiting for the future. The following is a general overview on pipe types, materials and design considerations. Their main characteristics (nominal diameter, thickness, weight, working and breaking pressures) have not been included because of a great variability from country to country. Therefore the reader should obtain this information directly from concerned manufacturers.

PIPE MATERIALS AND DESIGN CONSIDERATIONS

With the advent of pipelines to convey irrigation water, a wide variety of materials were developed by the industry. Each different type of pipe has its own features. The engineer must

TABLE A5
Suggested trench cross-section dimensions

Nominal Diameter ϕ (mm)	Pipe Bed S (m)	Excavation Width L (m)	Excavation Depth h (m)
110	0.10	0.70	1.30
140	0.10	0.70	1.35
160	0.10	0.70	1.35
200	0.15	0.70	1.50
225	0.15	0.70	1.50
250	0.15	0.80	1.50
280	0.15	0.80	1.55
315	0.20	1.00	1.70
350	0.20	1.00	1.75
400	0.20	1.00	1.80
450	0.20	1.10	1.85
500	0.20	1.10	1.90
600	0.20	1.20	2.00
700	0.20	1.30	2.10
800	0.20	1.40	2.20

consider these characteristics in order to make sure that the selected type of pipe is suitable for that particular application. A wrong type of pipe can cause an increase of the construction costs for the Water Authority, a reduction of the system's life, higher annual maintenance and power costs, or can result in the system not working properly.

The characteristics of a pipe include:

- the material constituting the pipe;
- the nominal diameter, which is the one considered for hydraulic calculation;
- the length of pipe sections, which determines the number of joints for every km of pipeline;
- the wall thickness, which determines the maximum pressure of operation;
- the types of joints.

Different materials

Cast iron

Recently, cast iron pipes are produced by centrifugation of liquid cast iron in cylindrical shaped moulds (formerly called "shells"). Pipes are then fired to obtain a homogeneous texture. The so obtained pipes are coated with a thin zinc layer and finally sheltered with bituminous paint. When cast iron pipes have to be used in chemically aggressive soils, a polythene coating is applied. An internal mortar lining, 3 to 6 mm thick, is then applied by centrifugation in order to reduce roughness and to provide higher resistance to aggressive waters.

Cast iron pipes are commonly available in 60 mm through 700-mm diameters; higher diameters are also provided by manufacturers, when made to order. Thickness can range from 6 mm to 10.8 mm depending on the nominal pressure and diameter. Cast iron pipes are produced in 6-m length sections. The pipe sections are joined together by a bell and spigot joint (with a rubber ring gasket for sealing). Less frequently flanged joints are adopted. In fact, besides being more expensive, flanged joint does not allow for any altimetric or planimetric deflection. On the contrary, flanged joint is more suitable for systems with a high number of special parts.

Steel

Steel pipes are produced through a special rolling-mill process called “Mannesmann” process. The pipe is made from the hot rolling of a steel bar, without any welding. Steel pipes are also produced from large steel sheets, which are then welded along the generatrix or following a spiral shaped line.

Another process, the helical-shaped soldering, is used to produce steel pipes with very large diameters (2500 - 3000 mm). Steel pipes are commonly available in 40 mm up to 3000 mm diameters. Wall thickness can range from 2,6 mm for the 40 mm diameter (for a maximum PN of 14,5 Mpa) and 8,8 mm for the 900 mm diameter (for a maximum PN of 3,7 Mpa). The thickness is increased for pipes that are operated at high pressure. Sections are provided in 8 to 13,5 m lengths. Sections can be joined together both by bell and spigot joints and by flanged joints. Recently, due to the development of the electrical welding, the welded joining has become the most common type of joint for steel pipe sections.

Concerning the characteristics of the different types of joint, the bell and spigot joint is used for underground pipelines whereas flanged and welded joints are used for outdoor applications or for systems with a high number of special parts and devices.

Comparing cast iron pipes with steel pipes, cast iron pipes have higher wall thickness, with diameters and pressure being equal. This make cast iron pipes more suitable for soils with problem water and for areas where electrolytic corrosion is feared, as a result of stray electric currents. Besides, steel pipes have higher toughness and lower fragility, which reduce breakage. The higher specific traction strength of metal enables to produce pipes having lower wall thickness; therefore steel pipes are light-weight and cheaper than cast iron pipes. Steel pipes are more suitable for unsteady soils, as they stand ground movements without breaking.

Steel pipes are provided in greater lengths, which decreases the number of joints for every Km of aqueduct. Furthermore, their lightness, with regard to cast iron pipes, allows for easier and cheaper transportation. Thus, the selection between those two types of pipe is not an easy task. In the present circumstances, steel pipes are suitable for outdoor aqueducts. For underground applications, cast iron pipes are preferred, thanks to their higher resistance to electrolytic corrosion and when a high number of interruptions have to be made for diversions and for insertion of special parts and devices.

With regard to lining materials, very thick bituminous layers insure a satisfying inside protection of the steel pipes against strongly aggressive waters. Further, very good results have been accomplished by using epossidic paints.

An effective outside protection is obtained coating pipes with bituminous layers, glass wool or polyethylene. When welded joints are used, the heat resulting from welding destroys lining and coating near the joining edges. This can be a real drawback because the coating material can be easily replaced, but it is nearly impossible to restore the inside lining.

Concrete

Concrete pipe most commonly used for distribution systems in the past. Concrete pipes can be subdivided into pre-cast pipes and cast-in-place (monolithic) pipes. Pre-cast concrete pipes can be provided either with or without reinforcement. The latter are mainly utilized to convey water with a very low pressure (sewerage and drainage systems). In fact, concrete does not have high resistance against forces of traction resulting from the water pressure. Reinforced concrete pipes are suitable for much higher hydrostatic heads than the unreinforced ones. The range of internal pressure head they are designed for varies from 7,5 m to 37,5 m. Moreover, concrete pipes are good to use when an external loading of soil is placed above the top of the pipe. The thickness of the walls and the type of reinforcement are varied according to the hydrostatic pressure head and to the external loading of any different application. The steel reinforcement can be a single or a double cage, circular or elliptical shaped. Recently, according to new production methods, the concrete is vibrated, consolidated and compacted to obtain a high density material that is watertight and not subject to pinhole leaks through the pipe barrel.

Diffuse reinforcement pipe is another recent innovation. This type of pipes has both cross and longitudinal reinforcement that are made from a high number of steel wires, which provide high resistance to the forces of traction. Furthermore, the high specific resistance of the material allows for pipes with reduced wall thickness. The diffuse reinforcement pipes are available in 400 mm to 1 400 mm diameters (for PN of 1.5 - 2 Mpa) while normal reinforced concrete pipes are produced in 600 mm to 2700 mm diameters. Reinforced concrete pipes are available in 2.4 m to 3.6 m lengths and the pipe sections are commonly joined together by a bell and spigot joint with a rubber ring gasket for sealing.

Cast-in-place pipes are mainly used for low-pressure head applications and for very large diameters. This kind of pipe is made by pouring concrete into metal forms, previously set to obtain a given wall thickness, and removing the forms and struts after the concrete has reached a sufficient level of strength. The result of the above process is a continuous pipeline without any joint, except for the expansion ones. This type of pipe is not suitable for expansive soils as multiple cracks can occur as a result of ground movements.

A third type of concrete pipe is the “continuous reinforcement” concrete pipe, also called “concrete cylinder” pipe or “Bonna” pipe. It is made of a steel cylinder, wrapped by a steel wire. The cylinder is also coated on both sides by cement mortar. The result of this process is a concrete pipe with a continuous reinforcement (steel cylinder). Pipe sections are connected with a bell-and-spigot joint, with a rubber gasket for sealing. In addition, sealing pipe sections by welding is possible. Continuous reinforcement concrete pipe is available in 12 m length sections and for diameters ranging from 300 mm to 1 350 mm. This type is suitable for pressure heads ranging from 70 m up to 350 m. The use of this type of pipe is also suggested to convey low-quality water, as the concrete coating and lining make an alkaline environment providing resistance against the corrosion of the cylinder.

Whatever the construction method of the concrete pipe is, particular care has to be taken in selecting the type of cement, its dosage, the water/cement ratio and the type of seasoning of the material obtained. The final purpose is to obtain a material with particular characteristics of impermeability.

Concrete pipes are, however, heavy compared to steel and cast iron pipes. This can affect the installation costs because less pipe is installed per day and because of the specialized construction equipment required.

Asbestos cement

Asbestos cement pipes are a typical component of water distribution systems. Asbestos cement pipes do not contain steel reinforcement, but they are composed of a mixture of cement mortar and asbestos fibres. Those fibres provide resistance to traction forces, the same function performed by steel wires in the reinforced concrete material. Asbestos cement pipes are formerly, but improperly, known as “Eternit” pipes, from the name of the first company that developed this technology.

The pipe material is obtained mixing asbestos fibres, cement mortar and water; with the dosage of each component is made by electronic devices. The resulting material is then further mixed to obtain high homogeneity in the mixture. Later, the mixture is poured on a rotating steel cylinder, until the desired wall thickness is reached. After that, the steel cylinder is taken off and the pipe is maturated in a ventilated tunnel. Lastly, the pipe goes in a hydration chamber to get a high level of mechanical strength.

Due to the production method used, asbestos cement pipes have good resistance to internal pressure, as asbestos fibres mainly orient in the direction of the cylinder. Therefore, breaks can occur as a result of longitudinal forces. This means that pipes need careful placement without external loading of soil above the pipe.

Asbestos cement pipes are provided in 600 mm diameter and smaller sizes. The maximum allowable internal pressure (hydrostatic) can be as high as 1.72 Mpa (175 m). This type of pipe is available in 3.9 m section length and the pipes are commonly joined together using Gibault or Simplex joints. Alternatively, one end of each pipe is machined to form a spigot, the other being provided with a collar. The spigot of a pipe is inserted into the collar of the subsequent pipe and two rubber gaskets are used for sealing the joint.

Asbestos cement pipes have a low roughness coefficient, a good chemical inertia, a good resistance to aggressive agents and a fairly good mechanical strength. These features encouraged their use in the past. Limits to their usefulness are their high weight, which makes transport and assembly more difficult. This is a big problem when pipes are laid underground in soils with a high colloidal fraction (expansive soils). Furthermore, the increasing concerns over toxicity of asbestos fibres resulted in many companies restricting or even ceasing their production. Also, their use has been drastically reduced over the last ten years by the diffusion of PVC, polyethylene and glassfibre pipes. Asbestos cement pipes are now widely used only for those situations imposing surface assembling such as in rocky or very high colloidal soils.

Plastic pipes

Over the last twenty years, a wide variety of plastic materials, composed by macro-molecules of carbonic compounds, such as Polystyrol, Polythene, Polypropylene and Polyvinyl Chloride have evolved. Each of these materials is made of thermoplastic resins enveloping a continuous or discontinuous fibrous phase. The plastic materials most commonly used for pipe production are Polyvinyl Chloride (PVC), Polyethylene (PE) and glass fibre reinforced Polyester (GFRP). The technology used for producing these materials is based on a chemical reaction, promoted by a catalytic agent, providing a macro-molecular structure of carbonic atoms (polymerization). For instance, Polyethylene is obtained through the polymerization of the molecule of ethylene. The polymers, when pure, are powder or grain shaped. Then, they are mixed with additive compounds to make them plastic and malleable. The resulting mixture is heated and then shaped to produce the pipe. The pipe is then water-cooled.

Plastic pipe features are very different from the steel and concrete pipes, as regard density, elasticity, resistance and thermal expansion coefficient. The best characteristics of the plastic pipes concern their resistance to corrosion (which can be caused by acids, alkaline compounds and organic fluids), their chemical inertia, their very low roughness coefficient, their lightness and their easiness in transport and placement. Moreover, they are adaptable to difficult and expansive soils.

Polyvinyl Chloride (PVC). Due to their indisputable advantages, Polyvinyl Chloride pipes have gained large acceptance in recent years. Until recently, they were available only in diameters ranging from 150 mm to 375 mm and were suitable for low-pressure head applications (from 15 m to 30 m). Due to progress in the production technology, PVC irrigation pipes are now produced for high-pressure head applications (56 m up to 70 m) and in 150 mm through 675 mm diameters. Pipe sections are commonly available in 6 m and 12 m lengths. Sections are joined together using bell and spigot joints with rubber gasket seals. In some cases the bell and spigot joint is further sealed with a solvent welding joint.

Since PVC possesses high flexibility, extra care is needed in bedding and back-filling to avoid an excessive deflection and distortion of the pipes. Another limit to the use of this type of pipes is their vulnerability to atmospheric agents. When placed outdoors, it is advisable to protect them with a coating. In those cases, PVC plastified pipes are preferred because of their special external “carbon black” protection.

Polythene (PE). Polythene pipes are widely used thanks to their advantages (e.g., lightness, handiness, low cost, flexibility, chemical inertia and resistance to atmospheric agents). The latter feature is provided by the “carbon black” additive, which protects the material from the action of solar rays.

Although the cost per metre is slightly higher than the PVC, Polythene pipes are more flexible and hence more widely used. Small diameters are available in large rolls and, therefore, pipe-layers are able to cut the soil and place the pipe in a single passage.

Polythene pipes are produced both in low (LDPE) and high density (HDPE). Low density polythene pipes have a slightly higher thickness, but they are more flexible and handy. The high density pipes have lower weight and, therefore, lower placement costs. Polythene pipes are available up to 1 200 mm diameter, but also in greater sizes when made to order. Wall thickness varies according to the class of pressure head (PN 2,5; PN 4; PN 6; PN 10; PN 16). They are produced in 6 or 12 m length. Pipes are joined together both by welding and flanged or plug joints. A good alternative to welding is electric welding where sections are connected by a hose clamp made of high density polythene. The hose contains an electric resistor providing the heat necessary for melting the three pieces together.

Glass wool reinforced polyester (GWRP). This type of material is made by adding glass fibre to a plastic mixture. This method produces pipes with higher mechanical strength than other plastic pipes. GWRP pipes can be coated with a layer of siliceous sand to protect them against ultraviolet rays. Sections are produced in 6 m and 12 m lengths and up to 2000 mm diameter. Sections are joined together by using the same types of joints as for the other plastic pipes.

Glassfibre Glassfibre pipes are widely used in large networks requiring large diameters. However, due to their relative higher cost for smaller diameters, they are not used for small networks. Glassfibre pipes are lighter than other materials but the very scarce availability of special parts limits their use. This is the reason why PVC and polythene pipes are preferred for small and medium diameter pipes.

Annex 3

Example of input and output files[1]

******* Example of Network Input file *********

SN	IN	FN	Area	CQ_Hyd	Length	LandEl	ND	Hmin
24								
1	0	1	2.2	2	150	96.20	250	20
2	1	2	2.2	2	462	95.80	250	20
3	2	3	2.2	2	162	96.30	250	20
4	3	4	2.2	2	118	97.50	250	20
5	4	5	2.2	2	80	98.40	250	20
6	5	6	0.00	0	112	101.10	250	20
7	6	7	0	0	250	100.00	250	20
8	7	8	0	0	13	100.10	200	20
9	8	9	2.2	2	63	98.80	180	20
10	9	10	2.2	2	50	98.00	180	20
11	10	11	0	0	83	96.80	180	20
12	11	12	2.2	2	30	96.00	140	20
13	12	13	2.2	2	30	95.70	140	20
14	13	14	2.2	2	63	95.30	140	20
15	14	15	2.2	2	35	95.20	140	20
16	15	16	2.2	2	40	95.00	140	20
17	7	17	0	0	315	100.30	250	20
18	17	18	2.2	2	413	102.00	200	20
19	18	19	2.2	2	173	103.00	200	20
20	19	20	2.2	2	43	102.80	200	20
21	6	21	2.2	2	1	101.10	180	20
22	21	22	2.2	2	123	103.00	140	20
23	22	23	2.2	2	75	103.80	110	20
24	8	24	2.2	2	63	99.50	110	20

ND	Thickness(mm)	(γ Bazin)	Cost (ITL/m)
110	5.3	.06	14000
125	6.0	.06	17700
140	6.7	.06	22100
160	7.7	.06	29300
180	8.6	.06	36500
200	9.6	.06	55000
225	10.8	.06	65000
250	11.9	.06	80000
280	13.4	.06	90000
315	15.0	.06	105000
450	00.0	.16	137000
500	00.0	.16	160000
600	00.0	.16	258000
700	00.0	.16	400000
800	00.0	.16	500000

****

128	(upstream piezometric elevation, m a.s.l.)
0.327	(specific continuous discharge, l s^{-1} ha^{-1})
0.0	(uncultivated land, %)
0.667	(Use coefficient, r)
3	(Number of hydrants simultaneously operating)
20.0	(Minimum pressure head, m)
1.645	(Operation quality, U(Pq))

[1] Note that input and output files in the COPAM package are SCI files. Therefore they can be easily imported into other software (like Excel).

* Example of output file of the Clement optimization program *

****
*

SN	IN	FN	Area	N.Hydr	Qcl (l/s)	Diam (mm)	LandEl (m asl)	L (m)	H (m)	Y (m)	PiezEl (m asl)	v (m/s)
1	0	1	41.8	19	50.0	315	96.20	150.0	31.54	0.26	127.74	0.64
2	1	2	39.6	18	50.0	315	95.80	462.0	31.15	0.79	126.95	0.64
3	2	3	37.4	17	40.0	280	96.30	162.0	30.33	0.33	126.63	0.65
4	3	4	35.2	16	40.0	280	97.50	118.0	28.89	0.24	126.39	0.65
5	4	5	33.0	15	40.0	280	98.40	80.0	27.83	0.16	126.23	0.65
6	5	6	30.8	14	40.0	280	101.10	112.0	24.90	0.23	126.00	0.65
7	6	7	24.2	11	30.0	250	100.00	250.0	25.49	0.51	125.49	0.61
8	7	8	17.6	8	30.0	160	100.10	13.0	25.11	0.27	125.21	1.49
9	8	9	15.4	7	30.0	160	98.80	63.0	25.08	1.33	123.88	1.49
10	9	10	13.2	6	30.0	160	98.00	50.0	24.83	1.06	122.83	1.49
11	10	11	11.0	5	30.0	160	96.80	83.0	24.28	1.75	121.08	1.49
12	11	12	11.0	5	30.0	160	96.00	30.0	24.28	0.80	120.28	1.49
13	12	13	8.8	4	30.0	140	95.70	30.0	23.31	1.27	119.01	1.95
14	13	14	6.6	3	30.0	140	95.30	63.0	21.03	2.67	116.33	1.95
15	14	15	4.4	2	20.0	140	95.20	35.0	20.47	0.66	115.67	1.30
16	15	16	2.2	1	10.0	110	95.00	40.0	20.00	0.67	115.00	1.05
17	7	17	6.6	3	30.0	250	100.30	315.0	24.44	0.75	124.74	0.61
18	17	18	6.6	3	30.0	225	102.00	413.0	21.27	1.47	123.27	0.75
19	18	19	4.4	2	20.0	225	103.00	173.0	20.00	0.27	123.00	0.50
20	19	20	2.2	1	10.0	160	102.80	43.0	20.00	0.20	122.80	0.50
21	6	21	6.6	3	30.0	180	101.10	1.0	24.89	0.01	125.99	1.18
22	21	22	4.4	2	20.0	160	103.00	123.0	21.44	1.55	124.44	0.99
23	22	23	2.2	1	10.0	125	103.80	75.0	20.00	0.64	123.80	0.81
24	8	24	2.2	1	10.0	110	99.50	63.0	24.65	1.06	124.15	1.05

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****** ANALYSIS OF COSTS ******

DIAM (mm)	Length (m)	COST (ITL)
110	103.0	1442000
125	75.0	1327500
140	128.0	2828800
160	405.0	11866500
180	1.0	36500
225	586.0	38090000
250	565.0	45200000
280	472.0	42480000
315	612.0	64260000

****** TOTAL COST : 207531296.0 ******

SN= Section Number; IN= Initial Node; FN = Final Node;
Q_Hydr = Hydrant discharge; CQ_Hyd = Code Hydrant discharge;
LandEl = Land elevation; Qcl = Discharge;
Diam = Diameter; L=Length; H = Residual head; Y = Head losses;
PiezEl = Piezometric elevation; v= Flow velocity;
Area = area served by each section; N.Hydr = Number of hydrants
downstream each section; Hmin = Minimum pressure head at the node.