# Optimal Allocation of Water Withdrawals in River Basin[a]

## Discussion by Ashok K. Keshari[3]

The authors present an objective methodology using flow duration curves and a chance-constrained mathematical programming technique to determine the optimal allocation of streamflow to competing multiple users in a river basin. The discusser feels that the methodology presented by the authors is very interesting and valuable to field engineers and policymakers involved in water allocation decisions. The methodology has merits because of its simplicity, its ease in implementation, and its ability to help in understanding the impact of streamflow availability and user requirements on the streamflow allocation. However, the discusser feels that a few points associated with the formulation and application of the proposed general chance-constrained model for maximizing water allocation in a river basin for productive use must be addressed and clarified before the proposed model can be advocated for implementation in solving real-life problems.

### STREAMFLOW ALLOCATION MODELS

The inequality constraints defined by (2) and (3) in the proposed general chance-constrained model for maximizing water allocation in a river basin for productive use place restrictions on the permitted withdrawal quantity to ensure that water allocation at a site does not exceed its demand and in no case is it less than the existing permitted quantity. In fact, these inequality constraints represent the upper and lower bounds placed upon the amount of permitted withdrawals by users at various sites. Solutions are being sought to optimize total water withdrawals in a river basin to meet demands at various sites with its acceptable streamflow reliabilities. Taking this cognizance, the inequality constraints defined by (6) and (7) become redundant. The constraint set defined by (3) and (4) itself ensures that the decision variables $q_i$ and $r_i$ are nonnegative, where $q_i$ is the permitted withdrawal quantity for site $i$, and $r_i$ is the streamflow reliability for withdrawal $i$. If allocation is desired on the basis of only flow duration curves (FDCs) and demands at various sites, without putting any restriction of minimum withdrawal permit, solution can be obtained by assigning zero values to $p_i$ for all values of $i$, where $p_i$ is the existing permitted withdrawal at site $i$.

The second issue is associated with the range within which the water allocation at a particular site is allowed to take any value during the optimization search process to maximize the total water allocation in a river basin. The constraint set defined by (3) is nothing but the demands requested by various users estimated currently or previously. The constraint set defined by (2) is basically the projected demand by various users in the coming future. This conceptualization has a flavor of scheduling the permitted withdrawals at various sites. Thus, it may be appropriate even to cut down water allocation at some insensitive sites to values less than their existing permits in order to maximize the production function. This is relevant when the reductions are likely to occur in streamflow availability. In such cases, the optimal solution will be primarily based on the FDCs. The sensitivity herein is with respect to production function. In fact, the lower limit on the variable $q_i$ should be based on the bare minimum water requirement at site $i$ without adversely affecting production function. This will enable one to enhance the model resilience in obtaining the optimal water allocation policy. The higher priorities to more sensitive sites will further intensify the degree of flexibility in water allocation under different scenarios.

Further, the nonlinear streamflow availability constraints due to nonlinear FDCs are expressed as the chance-constrained equations in the author's proposed model. These constraints are transformed to linear constraints using a piecewise linearization technique. Linearization is achieved by defining $K$ segments, each having slope $s_{i,k}$, horizontal length $r_{i,k}$, and beginning at reliability $R_{i,k}$. The reliability $r_i$ appears to be erroneously defined by (11a) in the authors' paper. The reliability should be defined as

$$r_i = \sum_{k=1}^{K} [R_{i,k} + r_{i,k}]z_{i,k}; \quad \forall i = 1, 2, \ldots, N \tag{13}$$

where $z_{i,k}$ is an integer; and $N$ = number of withdrawals. The variable $z_{i,k}$ should satisfy the following equality constraints:

$$\sum_{k=1}^{K} z_{i,k} = 1; \quad \forall i = 1, 2, \ldots, N \tag{14}$$

### APPLICATION OF DEVELOPED METHODOLOGY

The developed methodology was applied to a hypothetical unregulated basin composed of two separate streams converging downstream to form a single stream. Three possible sites were considered with assumed values of reliabilities and corresponding streamflows for the linearized FDCs. The authors do not mention whether their FDCs are based on the daily, weekly, monthly, or yearly streamflows. Since the FDCs differ for different time bases, some discussion on how this could affect the streamflow allocation would have benefited the paper. Further, the FDCs could be constructed using a complete duration series or a partial duration series. A complete duration series consists of all the data available, whereas a partial duration series is a series of data selected so that their magnitude is greater than a predefined base value. A partial duration series may be an annual exceedence series, an annual maximum series, or an annual minimum series. The authors have indicated use of both types of series for constructing the FDCs. In general, the entire recorded available data is used in constructing the FDCs. Alternatively, the entire data can be grouped into three different series representing dry year, wet year, and normal year series, and one can construct the annual-based FDCs separately for these three rainfall patterns. The authors should explain how the streamflow allocation is going to be affected in such scenarios, and which one will be appropriate for evolving streamflow allocation policy.

In real-life situations, most often monthly streamflow allocation is desired in unregulated catchments. In such cases, FDCs are generally constructed with the entire monthly streamflow data over the period-of-record using the Weibull plotting position, as discussed by the authors. The other way of constructing the FDCs could be summarized as: (1) each year's average monthly streamflows are arranged in descending order and ranked separately for all the years over the period-of-record; (2) the average flow values corresponding to the wettest month, second wettest month, and so on up to the driest month are found by taking the arithmetic mean of all values of the same rank; and (3) these average values are then used for the computation of exceedence probability using the Weibull plotting position. The authors do not discuss the effect of different approaches that can be used for constructing the FDCs on the optimal withdrawal permit programs.

[a]November/December 1998, Vol. 124, No. 6, by Jennifer M. Jacobs and Richard M. Vogel (Paper 16130).
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The results reported in Table 2 show that input parameters taken for the instream flow requirements at three sites have been chosen such that it satisfies the continuity equation. This is good. But the discusser would like to state here that instream flow requirements would have been based on the environmental or navigational requirements and the continuity equations would have been incorporated into the model as equality constraints to simulate the mass balance. This would have increased the robustness of the model. Although this will not affect the results reported in the authors' paper for the considered input values for instream flow requirements at various sites, arbitrary input values for these parameters may yield imprecise streamflow allocation.

## Closure by Jennifer M. Jacobs[4] and Richard M. Vogel[5]

The writers thank the discusser for his careful, thoughtful, and useful comments on our paper. His comments have the potential to make our paper even more useful than we originally envisioned. The discusser's comments relate to both the formulation and application of our generalized optimization and flow duration curve formulation of water allocation problems.

### FORMULATION OF STREAMFLOW ALLOCATION MODEL

From the perspective of improving the general chance-constrained model, the writers agree with most of the discusser's comments. The discusser suggests that use of constraints (2) and (3) may not be appropriate for all allocation problems. The writers also observed this when we suggested that the allocation lower bound $p_i$ as defined in constraint (3), which limits the permitting agency to maintaining historical allocation quantities, is not appropriate for all permitting processes. The fourth extension to our model (see page 362) addressed this issue by suggesting that ''the constraint set (3) that protects existing permitted values could be relaxed. This would allow a more flexible analysis during permit renewals.'' The discusser formulates an alternative definition for this lower bound to constrain the streamflow $q_i$ allocated at each site $i$ rather than just at existing withdrawal sites. This approach requires that the existing permit value be set to zero for all new permit requests. The benefit of Keshari's approach is that it eliminates the need for nonnegative constraints on decision variables. However, confusion may result from this formulation, because the modeler is unable to distinguish a priori between a previously denied permit request and a new permit request.

The writers also appreciate the discusser's identification of an error in the allocation reliability formulation defined in (11*a*). However, the modification suggested by the discusser would result in a nonlinear model formulation. To maintain a linear formulation, the writers prefer the definition of streamflow reliability that appears below in the Erratum section.

### APPLICATION OF STREAMFLOW ALLOCATION MODEL

The discusser points out that the time basis for the flow duration curves used in our case study was not mentioned. We stated on page 358 that ''FDCs are defined for specific sites and flow measurement duration; daily streamflows are typically used, though other durations are possible (see Vogel and Fennessey 1994).'' The time duration of the FDC will depend on the overall goals and implementation of the permit program. For example, if the focus of the permit program is on monthly flows, then a monthly duration should be used to construct the FDC. The discusser suggests that FDCs can be constructed using either the complete series of streamflows or a partial duration series. This nomenclature is misleading, because normally FDCs are always obtained from the complete series of streamflow observations. Vogel and Fennessey (1994) introduced a new approach to the development and interpretation of FDCs when they suggested developing a FDC for each year, and using all $n$ years of FDCs to construct confidence intervals for an FDC and to construct a ''median annual FDC.'' If the intent of the permit program is water allocation during a *typical* year, than a ''median annual'' FDC would be the most appropriate FDC to use. One could also use the traditional ''period-of-record'' FDC for this application; however, that FDC will not reflect the probability of low flows during a typical year. Instead, the period-of-record FDC reflects the steady-state probability distribution of streamflow. If the permit program wishes to focus on a relatively severe drought year with a 20-year recurrence interval, than an FDC with a 20-year return period should be constructed, as is shown in figure 8 of Vogel and Fennessey (1994). The goals and implementation schedule of the permit program will dictate which reliability definition is most suitable, and, on that basis, the various methods for constructing FDCs outlined by Vogel and Fennessey (1994) can then be used to construct the most suitable FDC to suite the intended purpose. For a detailed discussion of the various reliability definitions and methodologies for constructing FDCs, see Vogel and Fennessey (1994).

Naturally, use of different FDCs within the streamflow allocation algorithm will lead to different optimal withdrawals. Users of this approach are encouraged to explore the tradeoffs that exist between the reliability definition of the flow duration curve and the optimal withdrawals that result.

The writers emphasize once again that the methodology introduced here is not a substitute for a water permitting program; rather, it provides a planning tool to assist policymakers and water managers to understand the impact that a proposed set of water use goals and constraints can have on streamflow allocation within a river basin.

**Erratum.** The chance constraint (11*a*) should appear as follows:

$$r_i = \sum_{k=1}^{K} [R_{i,k} z_{i,k} + r_{i,k}] \quad (11a)$$

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# MODIFIED PIPE NETWORK MODEL FOR INCORPORATING PEAK DEMAND REQUIREMENTS[a]

## Discussion by Ryszard Orłowski[4]

Two problems are discussed in the paper. The first is practical use of peak flow values, calculated using the PDD

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[a]September/October 1998, Vol. 124, No. 5, by Srinivasa Lingireddy, Don J. Wood, and Alan Nelson (Technical Note 16769).

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method, in the process of designing diameters of pipelines in the water-pipe network. This problem is presented in a wider aspect, pointing out the task fulfilled by these values in the diameter optimization procedure when designing branched networks. The second problem is the use of PDD for designing looped networks. Generally, this method models simultaneous water demand by users depending on their number, but irrelevant of the network type. In the paper, an algorithm is proposed for applying the PDD method in checking the looped network capacity.

## INTRODUCTION

The peak demand diversity (PDD) method, described in the above article, is based on the mathematical description of simultaneous consumption of water by users on a water-pipe network. General tendencies in this area have been well recognized, resulting in relatively accurate values of, for instance, one-hour nonuniformity and other factors assumed by designers for the water consumption. These tendencies can be briefly summed up: the bigger and more densely populated the regions supplied by the system are, the lower the factor values.

The mathematical model presented in the article is extremely valuable for correct determination of peak flows in the branched water-pipe network. However, some factors of relatively great importance should be pointed out:

1. The instantaneous peak flow requirement is strongly influenced by specific features of the water-pipe system and other environmental, technical conditions—which may, in some cases, dramatically change PDD relations or assumed experimental coefficients. Those features and conditions need to be taken into consideration when computing flows in the water-pipe network. These are:
   - The method of controlling the pumping station: ''on-off'' control with possible surge tank, or continuous pumping control at various possible values of controlling parameters
   - The presence or absence of flow or pressure governors in the network, as well as their location and actual values of controlling parameters
   - The strong nonlinearity of the water-pipe system with respect to the water consumption, connected with the relation between water consumption and network pressure, which reveals different spatial distribution in various systems; this nonlinearity is especially great when there are no pressure reducers on pipelines linking buildings to the network, which is common in Polish circumstances
   - Other factors influencing absolute values of averaged and peak water consumption, such as the dimensions of a building, the type of its internal water-pipe system, the method of hot water preparation, and principles of its possible circulation
2. It should be emphasized that, for the majority of systems, the PDD method does not make a sufficient basis for direct dimensioning of diameters of the network water pipes or for designing pumping station controls. Only familiarity, even relatively approximate, with the real variation of the water consumption in time and across the area makes it possible to optimize these diameters, along with possible optimization of the control system. The optimal project should take into consideration both the cost of water-pipe construction and the cost of water pumping, which changes in time, depending on actual demand and method of control. The network peak flow values, calculated using the PDD method, only determine one of the technical limitations in the above optimization task, i.e., they make it possible to check the maximum flow velocities in pipes and, if possible, to correct the designed, most economically favorable diameters.
3. In the article, a possible use of the PDD method for designing looped networks was mentioned, but the conclusion was that, generally, a traditional approach, based on the water consumption evaluated for each individual network section using a per capita method, i.e., via direct summation of instantaneous peak consumption values for all users connected to this section, the simultaneousness being neglected, is sufficient in these cases. Let us point out that the PDD formula models the simultaneous water consumption by users independently of the network type, so there is no reason for limiting its application to the branched networks only.

Due to the space limitations imposed on discussions, only two of the above problems—problems 2 and 3—will be developed herein.

## DESIGNING WATER-PIPE NETWORKS USING PDD

As mentioned, designing diameters of pipes constituting the water-pipe network takes into account not only peak flow rates and maximum flow velocities, but also certain economic aspects by trying to optimize total costs of network construction and operation. Here, the economic effectiveness coefficient, playing the role of a target function, takes into account the costs of both the construction and the operation of all pipelines and pumping stations in the water-pipe network. An essential component of the investment costs is the cost of construction of the pipe system, which depends on their diameters, while the vital part of the operating costs is the cost of pumping water, which also depends, in a certain way, on the diameters of pipes in the water-pipe network. Also, the cost of pumping water depends on time and spatial variability of water demand, and on the technology and adopted way of controlling the pumping stations, including the flow control, and the possible water storage in the system. These costs can be evaluated only with the aid of the extended period simulation (EPS) of the flows in the designed system, for an assumed time and spatial variability of water demand.

Here, the peak flow values, calculated using the PDD method, can be treated only as an additional method of controlling the maximum flows, with possible further correction of previously defined, most economically favorable diameters when the maximum velocities turn out to be too high for technological reasons.

A detailed presentation of mathematical models used for solving such an optimization task goes far beyond that possible in a discussion. Therefore, let us formulate only a few remarks in the subject. As is well known, the mathematical methods applied depend on the form of the target function (Findeisen 1985). Here, the following methods of optimizing network water pipe diameters can be named for an assumed diagram of links and method of pump operation and flow control:

1. The linear theory method, which consists in temporary linearization of parameters such as coefficients related to search optimum diameters $D_i^*$. In consecutive iterations, the mathematical formulas are linearized and model the pipeline construction costs as a function of their diameters. For pipes located at the critical area with respect to the pump delivery head, extra components are added for the cost of pumping water, integrated for particular pipes during EPS.
2. The nonlinear programming method, consisting in optimization of parameters of a functional with a number of excitation functions. Here, the objective function is the measure of economic efficiency for a water-pipe system,

taking into consideration, among other things, the cost of pipe network construction and the cost of pumping water. The output parameters to be found are the diameters $D_i$, while the excitation functions, components of the water consumption vector, changing with time and across area, are defined for a repeatable time period of interest. The functional is calculated via numeric integrating during EPS.

Note that the peak flows in the water-pipe network, corresponding to the maximum simultaneous water demand calculated using PDD relation, can also appear in the water-pipe network during EPS, but never at the same time in all network sections. Therefore, thus defined flows cannot be used for calculating pressure drops in the water-pipe network or for evaluating the delivery head of pumps and/or examining their work.

To sum up:

1. In the cases of large water-pipe systems and those for which the diameters are to be designed as the most favorable ones on an economic basis, the PDD method makes it possible to make an extra check on whether the flow capacity of the designed network is sufficient, i.e. whether the maximum possible flows in particular pipes do not lead to exceeding the maximum flow velocities permissible for technical reasons.
2. In the case of small water-pipe systems, or when the designing process is simplified—i.e., the only designing criterion assumed is securing the maximum hydraulic capacity of particular pipes, the delivery head being secured by continuous adjustment of revolutions—PDD may serve as the basic tool for designing diameters of the network pipes.

## USING PDD FOR DESIGNING LOOPED NETWORKS

As mentioned, the PDD method assumes implicitly simultaneous water consumption by users. This simultaneousness is independent of the type of network (branched or looped), and there are no reasons for using it only for branched networks. In the above procedure of designing diameters of water pipes for a branched network, two stages can be distinguished; (1) the economic optimization of diameters, taking into account technical restrictions, like extreme permissible flow velocities and pressure; and (2) the final check of hydraulic capacity of the designed network, done using PDD, with further possible correction of the earlier determined diameters.

For looped networks, the general design procedure remains the same. In the same optimization scheme, only hydraulic calculations in EPS are done using another algorithm. However, the question remains open: what is a proper algorithm for using PDD in checking the hydraulic capacity of the economically optimized network?

From the discusser's experience gained from work in this area, the following procedure has been the most profitable:

1. The area supplied with water by the network is to be divided into regions populated by residents of similar type with respect to the per capita water demand. Then the time periods of peak water demand are to be determined for those regions.
2. On the basis of the EPS results for the entire network, the hours are to be analyzed for which the peak water demand was recognized in each individual region. This is done in order to recognize real directions of flow in the network sections that supply those regions during the time periods of their peak water demand.
3. Through this procedure, certain trajectories of water supply are obtained for particular regions, which determine so-called branched regions, analyzed in a standard way using the PDD relation.

## CONCLUSIONS

The water consumption vector in the system, determined using the PDD method, does not correspond to any real time instant, as the maximum flows estimated for particular pipes do not occur at the same time in the entire network. This results in a lack of continuity in network centers for which the flows are calculated, with a further necessity for determining the water consumption vector in an artificial way to secure continuity in the calculations. Thus, generated flows may only be used, formally, for checking the network flow capacity related to maximum flow velocities in the pipes, for which diameters and control methods are designed, for instance: (1) on the basis of economic optimization criteria and with the aid of EPS; or (2) in simpler cases, taking into account only really possible extreme and averaged situations.

Since the PDD method models simultaneous consumption of water taken by the users depending on their number, irrespective of the network type (branched or ring), it can be used for controlling the flow capacity of both types of network. The procedure for applying the PDD method in this latter case has been briefly described in the paper.

## APPENDIX. REFERENCES


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# Discussion by Michael B. Sonnen,[5] Member, ASCE

The authors' Table 1, which gives tabulated values for "curves" 1, 2, and 3, should be understood not to represent smooth curves at all (fitted with the authors' coefficients, *A*, *B*, and *C*). The values are data from 1968. Fig. 7 shows the


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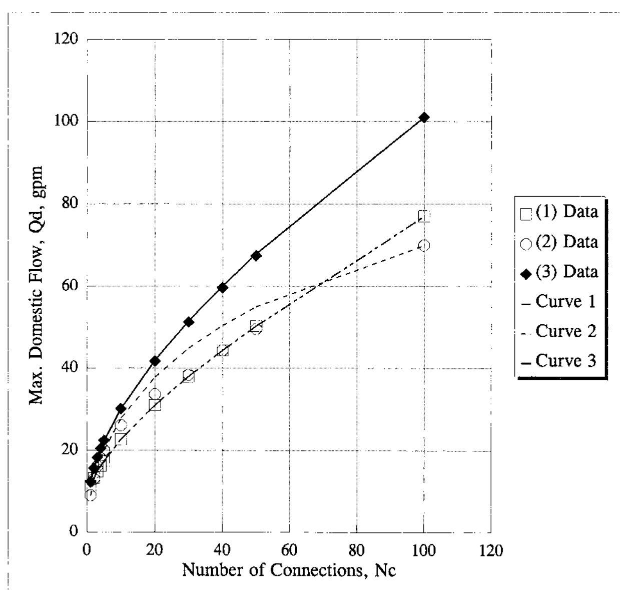


**FIG. 7. Eq. (1) Fitted to Data Given in Table 1 for "Curves" 1, 2, and 3**

curves and the data from Table 1. ''Curve 2'' does not fit (1) well with the coefficients the authors gave ($A$ = 9.78, $B$ = $-0.273$, and $C$ = $-0.51$).

Fig. 8 shows a closer fit to the ''curve 2'' data. However, as seen below, improving the fit to any of the Table 1 data sets does not improve the selection of water mains—either over previous fits *or* over the authors' described ''conventional approach.''

The typical approach for estimating peak hourly domestic flow when the authors' Table 1 data were collected (the 1960s) was multiplication of the average anticipated daily flow by a factor of 3 or thereabouts (Fair et al. 1963). This flow was used for the design sizing of such distribution system accoutrements as treatment plants and (some) pumps. It was not used for sizing distribution mains.

Main sizing, conventionally, was performed to accommodate anticipated fire-fighting water demands as follows:

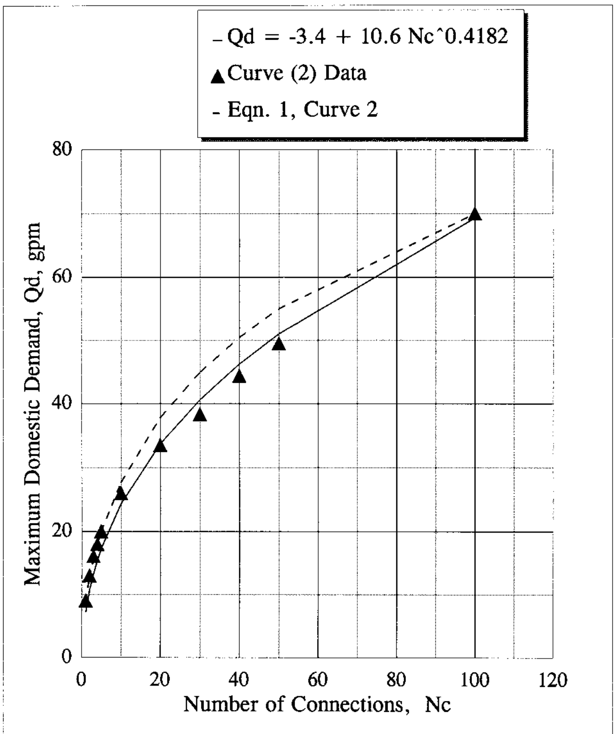


**FIG. 8. Improved-Fit Relationship between $Q_D$ and $N_C$ Compared with Authors' ''Curve 2''**

$$Q_{\text{fire}} = 510\sqrt{N_c} - 2.55N_c \tag{2}$$

as derived from a 1942 National Board of Fire Underwriters equation, and with the present assumption of four persons per connection (Fair et al. 1963). One modern text does not discuss domestic flow rates at all for water distribution modeling, but it references the following 1984 English equation for the controlling ''fire demand'' in L/min:

$$Q = 3{,}860\sqrt{\text{POP}}(1 - 0.01\sqrt{\text{POP}}) \tag{3}$$

Eq. (3) is essentially (2) converted from gallons to liters and with population (POP) instead of connections (Abdel-Magid et al. 1997).

The authors did not show (in Figs. 5 and 6) pipe lengths, diameters, or elevations. The discusser has inferred several from the flow and pressure data given. The resulting choices for four of the authors' pipes are shown in Table 3.

Two conclusions leap from the table: (1) pipes sufficiently large to deliver peak *domestic* flows are too small for adequate, safe, reliable service; and (2) virtually no difference in selected pipe size results from using what the authors allege is an ''inadequate'' conventional approach versus their preferred PDD approach. In the PDD approach, one 4 in. pipe *is* selected over a 2-in. pipe, but in another case a 4 in. pipe is selected over

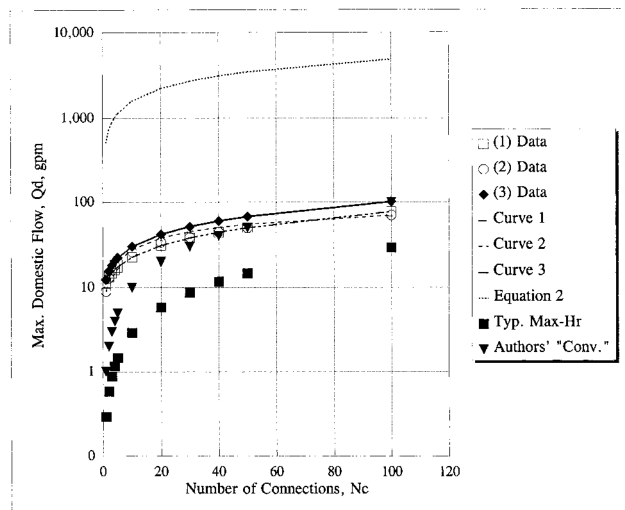


**FIG. 9. Comparison of Typical Flow Estimates for Municipal Water Systems with Authors' ''Conventional'' and PPD Approaches**

**TABLE 3. Alternative Possible Diameter-Length-Lift Combinations for ''Conventional'' (Conv) and Modified (Mod) Pipe Solutions Indicated in Figs. 5 and 6**

| | Pipe Numbers from Fig. 4(a) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 27 | | 29 | | 30 | | 43 | |
| Item (1) | Conv (2) | Mod (3) | Conv (4) | Mod (5) | Conv (6) | Mod (7) | Conv (8) | Mod (9) |
| Alternative diameters (in.) | 2[a] | 2 | 2 | 2 | 2[a] | 2[a] | 2[a] | 2[a] |
| | 4 | 4[a] | 4 | 4 | 4 | 4[b] | 4[b] | 4[b] |
| | 6 | 6 | 6[a] | 6 | 6 | 6 | 6 | 6 |
| Lengths (ft) | 800 | 665 | 400 | 2,390 | 1,000 | 1,100 | 5,280 | 1,320 |
| | 600 | 5,280 | 600 | 11,000 | 2,000 | 10,000 | 10,560 | 7,920 |
| | 9,000 | 41,500 | 1,000 | 1,000 | 4,000 | 25,000 | 999,999 | 52,825 |
| Elevation change (ft) | 0 | 0 | $-17.1$ | $-17.1$ | $-16.95$ | 0 | 1.24 | 3.1 |
| | 1.6 | 7.25 | $-16.6$ | 10 | $-15.1$ | 11.5 | 10.84 | 18.8 |
| | 1.6 | 7 | $-16.6$ | 15 | $-15$ | 14.8 | 2.28 | 19.1 |
| Flow (gpm) | 6 | 18 | 5 | 17 | 6 | 18 | 6 | 18 |
| | 6 | 18 | 5 | 17 | 6 | 18 | 6 | 18 |
| | 6 | 18 | 5 | 17 | 6 | 18 | 6 | 18 |

[a]Indicates apparently acceptable diameter for pipe at least 80 ft in length.
[b]Indicates second, roughly equivalent, acceptable solution.

**TABLE 4. Diameter-Length Combinations That Achieve Maximum Flows from 700–9,000 gpm**

| Diameter (in.) (1) | Length (ft) (2) | Maximum flow (gpm) (3) |
|---|---|---|
| 6 | 811 | 700 |
| 8 | 2,866 | 900 |
| 10 | 5,040 | 1,000 |
| 12 | 3,393 | 2,000 |
| 15 | 4,748 | 3,000 |
| 18 | 6,772 | 4,000 |
| 21 | 9,490 | 5,000 |
| 24 | 12,980 | 6,000 |
| 24 | 9,752 | 7,000 |
| 27 | 13,516 | 8,000 |
| 30 | 18,160 | 9,000 |

Note: In all cases, Hazen-Williams C factor = 110; lift = 50 ft; pressure = 80 psi upstream, 40 psi downstream.

a 6-in. pipe—and in two other cases 2 in. pipes appear to suffice.

As many as 10 domestic ''connections'' can be served their peak water demands through one or two garden-hose-sized pipes. In many parts of the world, that occurs. One could conclude that water conservation and sustainability of the resource could be achieved by installation of rather tiny, if sufficient, water pipes. The authors' example analysis demonstrates this.

But, as Fig. 9 and Table 4 make explicit, comparatively enormous flows—meeting fire demands while experiencing pressure drops unnoticeably small during peak *domestic* demands—can be achieved with water mains only slightly larger than the authors' analyses indicate. The ''conventional approach'' of virtually all municipal water departments and water service agencies *today* is to require that engineers design and contractors install—for connection to the agency's system—no pipe less than 6 inches in diameter. The convenience; reliability; evenness of flow; defenses against water hammer, backflow, or back-siphonage; and sustainable pressures at *all* flows are standard-of-living benefits far outweighing the added cost of pipe—particularly given the fixed cost of digging the trench in the first place.

## APPENDIX. REFERENCES


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