January 1979

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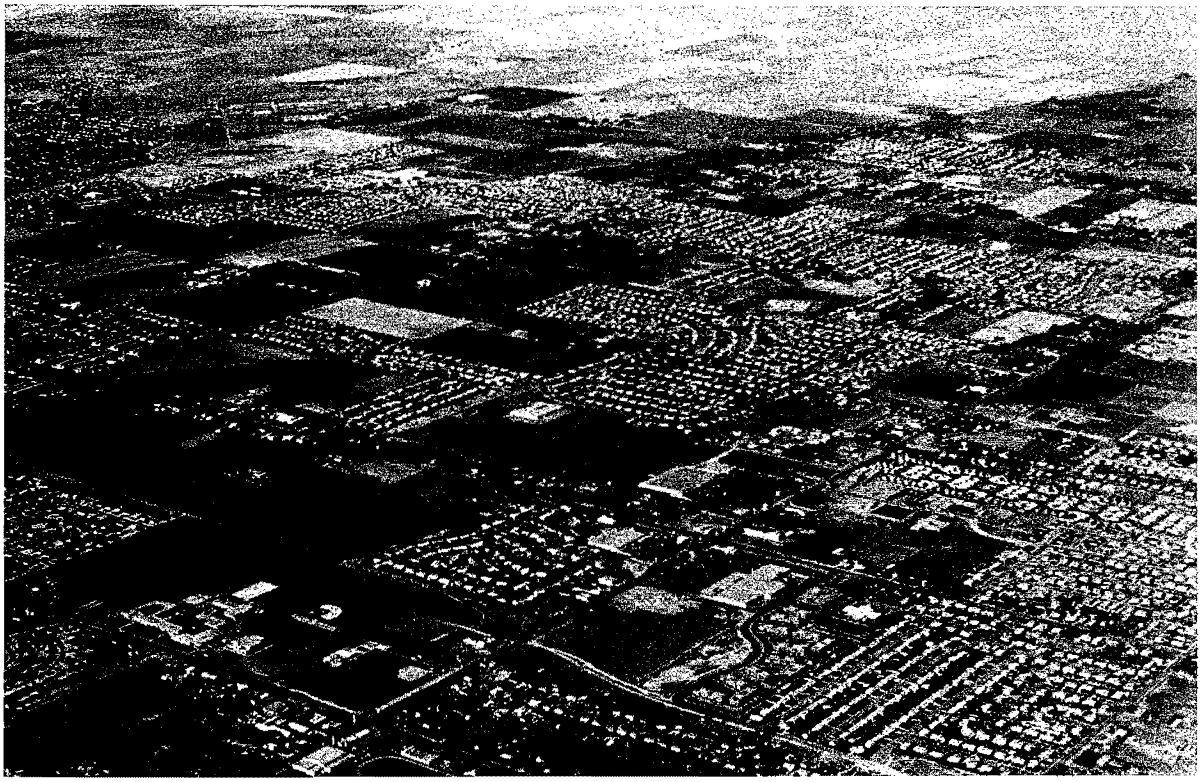

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WATER RESOURCES PLANNING SERIES
UWRL/P-79/04

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ABSTRACT

Multiple regression and frequency analysis of average month, peak month, peak day, and instantaneous water use by various water supply systems in Utah and Colorado are used to develop water demand functions. The research objective was to predict water demand as a function of a small number of independent variables for which data were easily obtainable and thereby provide an attractive method for use by consulting engineers in future planning studies. The independent variables which were significant for monthly and daily demands were cost of water and an outdoor use index which includes the effect of variation in landscaped area and accounts for use of supplementary ditch or pressure irrigation systems. The demand functions were developed with data from a sample of 14 systems varying in size from very small low density rural systems to Salt Lake City's water system. The correlation coefficients (R^2) vary from 0.80 to 0.95. The demand functions were validated by comparing calculated to measured water use for more than 40 other Utah systems. Instantaneous demands are determined for any desired recurrence interval as a function of number of connections.

The demand functions are presented both at best fit (expected value) levels for average month, peak month, and peak day and at recommended design levels for the same time durations. The design levels were calculated by adding to expected values an increment which was based upon standard deviation of the samples.

Instantaneous demand peaks which can be expected once in about 30 years in Utah systems are under 2 gpm per connection for lines serving more than 50 families, 3 gpm for lines serving 10 families, and 5 gpm for lines serving 4 connections.

ACKNOWLEDGMENTS

The authors are indebted to officials of all of the 14 cities or water districts which cooperated in providing historic water demand data, and particularly to the three which helped us obtain special meter readings for the short term peak study--namely Robert Hilbert and Terry Holzworth of the Salt Lake County Water Conservancy District, Ray Leonard of the Price City water system, and Wayne Boucher for the Brooklyn Tap data.

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INTRODUCTION

Nature of the Problem

Engineers customarily design municipal water supply distribution networks to handle peak hour demands; treatment plants and well pumps must meet 24 hour peaks; and raw water storage reservoirs are sized to provide seasonal peak demands. These design decisions are made daily by thousands of engineers. Despite the apparently routine nature of these decisions, we don't know nearly so much as we should about matching water supply facility capacities to future demand levels. If one examines the range of historic water use quantities per capita for a large number of systems even within the same general area, the striking characteristic of these quantities is usually the large variability between systems. For example, a recent statewide survey of municipal water use in Utah (Hansen et al., 1979) revealed the average water use levels in 50 Utah systems in Figure 1. The quantities vary from 93 to 505 gallons per capita per day (gpcd).

There have been many attempts at developing models which explain the large variability in water use by expressing demand as a function of various parameters which affect water use. Theoretical approaches to such models have not been very useful, except in determining which parameters may be significant demand determinants. The empirical approach which is commonly used is multiple regression analysis. For example, a recent study in Mississippi (Camp, 1978) produced equations which predict annual water use as a function of the following 13 variables: Occupants per household, age of head of household, market value of residence, irrigable lawn area, number of bathrooms, clothes washers, dishwashers, and swimming pools, race, average temperature, precipitation, price of water, and educational level.

Such models may be relatively successful in explaining historic water use but their use by consulting engineers in planning studies presents many problems including the following: (1) Many regression models (such as the previous example) require data that are simply not available for the entire study area without unreasonable data gathering costs. (2) Unless the model accuracy has been verified for systems in the particular geographic region of interest, planning engineers are not likely to have confidence in the results. (3) Most regression models include only average annual demand functions. This is usually the <u>least</u> important demand period to a design engineer. Peak day and peak hour demands must then be estimated by applying some multiplier to the average demand quantity. The multipliers suggested by most water supply textbooks are usually in the form of upper and lower range which are often very far apart. For example, Clark and Viessman (1966) suggest a peak day flow of 120 to 400 percent of average day and a peak hour of 150 to 1200 percent of average. This clearly leaves the designer with scant help in deciding on a single number for a particular system.

An additional problem is the question of whether a peak hour design provides adequate capacity for shorter term demands such as 5 or 10 minutes. Public health officials are traditionally concerned with the possibility of negative line pressures causing contamination by back siphoning during short term peaks. Very little information is available in the literature concerning instantaneous peaks. Conventional wisdom suggests that hydrographs in large municipal supply lines are stable enough that the peak hour parameter is adequate; however, little is known concerning very short term peaks in small rural systems or in the lines serving small portions of urban systems.

Scope of the Report

The original objective of this study was to develop demand functions for rural domestic water systems in Utah. One of the questions which previous research in this area raised was the difference between rural and urban water demand. That is, can municipal demand functions be used to plan rural, low density systems or are there significant differences between the peak period and average water requirements of rural and urban users. This project addressed that question and produced some surprising answers. The determination of the rural/urban difference question resulted in a substantial extension of the original scope of the study to include a rather extensive analysis of water demand in both large and small Utah municipalities as well as low density rural domestic systems.

The water parameters which were analyzed include the following: 1) average month; 2) peak month; 3) peak day; and 4) instantaneous.

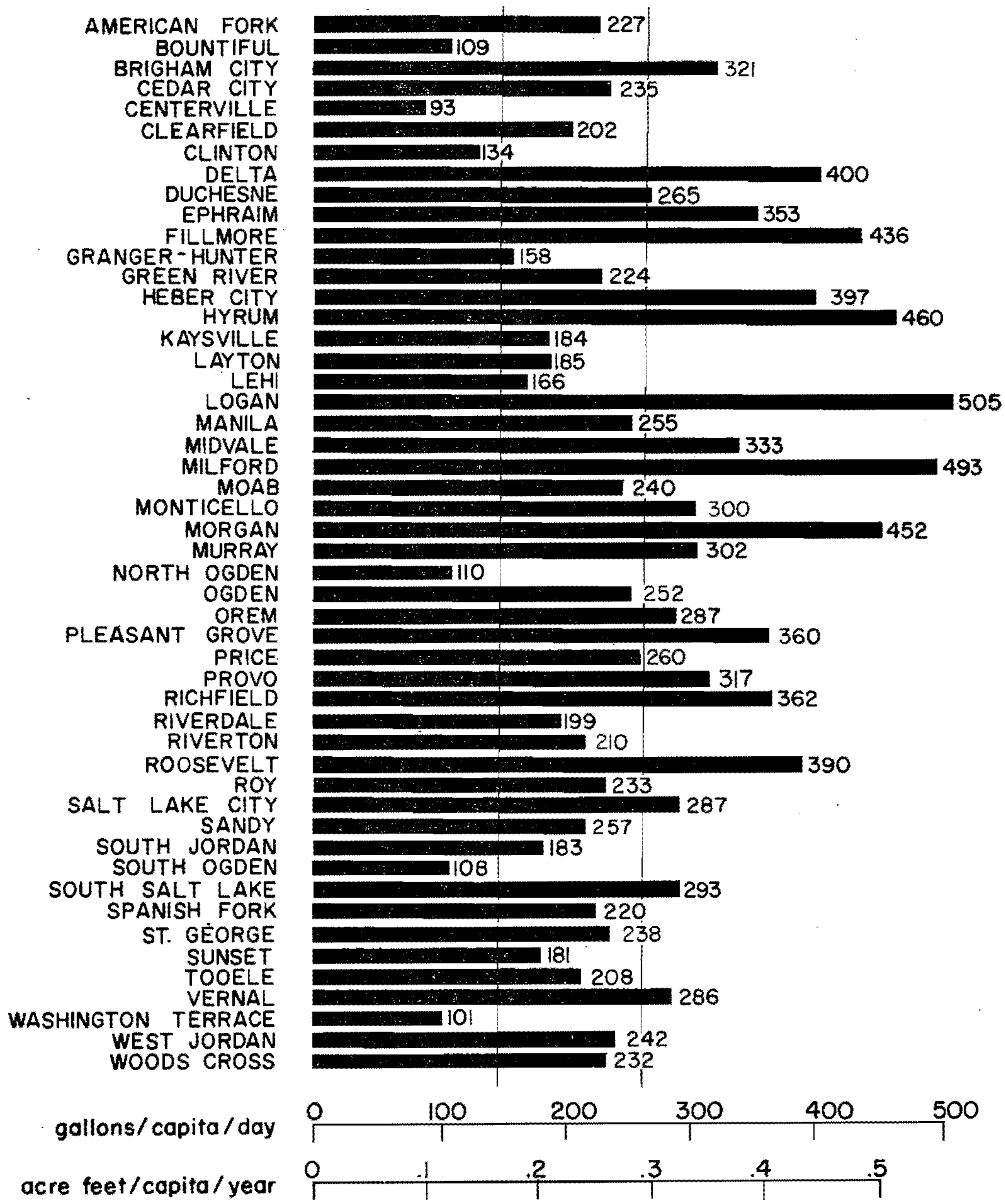

Figure 1. Daily per capita withdrawal rates (gcd) for 50 Utah municipal systems: average of 1974, 1975, and 1976.

All of these parameters are important to planning and design engineers because they each determine the size of a water right and/or various capital and operating costs of water systems as described in Table 1.

The monthly (peak and average) data and part of the daily data which were used in this study consisted of historic measurements provided by a representative sample of Utah water systems. Instantaneous data, however, are almost never collected routinely and therefore, were measured as part of this project. Measurements were taken of very short term flowrates (1 to 5 minutes) during the summer of 1977 at master meters on three Utah systems. Because of the extreme drought in Utah during that summer, these data were not considered to adequately represent peak flows and therefore, the measurements were repeated during the summer of 1978.

The longer term data gathering from a 14 system sample consisted of the three most

Table 1. Demand duration--system costs interactions.

Peak Flow Parameters	Costs Which are Determined by Peaks	
	Capital Investment	Operating Costs
Monthly (both average and peak)	Raw Water Reservoir, Water stock or right purchases	Monthly well, booster and treatment plant total pumping costs, annual water purchase charges
Daily	Treatment Plant, Well pumps, transmission conduits covered storage	Electrical demand charges for pumps. Tr. plant start-up costs
Instantaneous	Distribution Mains (within ranges where fire flow does not govern), Service Lines, In-Line Booster Pumps	Booster Pumps

recent years, excluding the drought (1974, 1975, and 1976). The sample was selected on the basis of several desired characteristics as follows:

1. Master meters. Reliable readings of master meters which were located so as to exclude or at least quantify the overflow at reservoirs were desired. The objective was to determine the actual use of water entering the distribution system not the total flow from water sources.

2. Geographic distribution. The objective was to include systems from all types of Utah climates and cultural settings.

3. Size of system. The objective was to compare differences between large and small municipal, rural, and urban systems.

4. Leakage. Leakage is an unavoidable component of demand in almost all systems but those with unusually large amounts of leakage were avoided in order to prevent distortion of the demand functions.

5. Cost of water. Since price elasticity was one of the variables to be analyzed, a sample was selected which covered a large range of variation in water price.

6. Individual meters. All systems in the sample were completely metered.

The 14 system sample was used to develop demand functions for average peak month, and peak day demand. Another UWRL project which was being conducted concurrently, however, produced a statewide survey of historic municipal water use, including data from some 150 Utah systems (Hansen et al., 1979). These data were also analyzed in the context of the objectives of this study and were used to test the reliability of the 14 system demand functions.

Format of Report

This report is divided into two basic sections as dictated by the two very different statistical techniques required for the various demand duration parameters. A multiple regression approach was adopted for annual, monthly, and daily demand functions while the instantaneous demand function was developed by a frequency analysis approach. The longer term demand analysis (average, peak month, and peak day) which is based upon data from a 14 city sample will be presented first. The following chapter will describe the instantaneous demand analysis and will compare instantaneous, peak hour, and peak day demands.

REVIEW OF LITERATURE

Format

The literature review will include three sections in order to relate various studies to the research objectives. The first category includes literature relating long term demand parameters (monthly and annual) to various demand determinants. These are typically multiple regression studies. The second category will report research on very short term demands (instantaneous to hourly). The final literature category will be a review of literature relating longer term parameters such as peak day and peak month to hourly and instantaneous peaks.

Monthly and Annual Demand Literature

Most of the water demand research has been focused in this area because monthly and annual water demand data are routinely available. Several of the studies have made use of regression analysis to relate the water demand to socio-economic factors. Price and income elasticities from several of the reports are summarized in Table 2. The four reports reviewed here have narrowed the scope of their studies to include only average monthly or yearly water consumption.

Israel--Darr et al. (1976)

In a report called "The Demand for Urban Water" seven different socio-economic factors were used to try to explain the variation in residential water use in Israel. A survey was conducted in several areas considered to be representative of Israel's urban, residential areas. The survey collected information about the following seven factors:

1. Household size
2. Income per capita
3. Urban area
4. Country of origin of the respondent
5. Type of metering
6. Education of the respondent
7. Density per room

Three dependent variables were of interest to the study group. The first dependent variable studied was per capita water consumption per annum. Also studied as dependent variables were within house water usage and the quantity of water used for outdoor sprinkling on a per capita basis. All three of the dependent variables were related to the seven socio-economic factors in linear and log-linear multiple regression equations. Equations were developed for each residential area surveyed. In each case

Table 2. Comparison of income and price elasticities from various studies.

Researcher	Year	Geographical Area	Form of Equation	Income Elasticity	Price Elasticity	R^2
Gottlieb	1963	Kansas	Logarithmic	0.28 to 0.89	-0.66 to -1.24	-
Gardner and Schick	1964	Utah	Linear	-	-	0.55
Gardner and Schick	1964	Utah	Logarithmic	-	-0.77	0.83
Howe and Linaweaver	1967	U.S.	Linear	0.31 to 0.38	-0.21 to -0.23	-
Beattie	1960	U.S.	Exponential	-	-	0.54
Beattie	1960	Western Plains	Exponential	0.63	-0.30 to -0.85[a]	0.74
Beattie	1970	U.S.	Exponential	-	-	0.58
Beattie	1970	Western Plains	Exponential	0.37	-0.39 to -0.81[a]	0.71
Wong	1970	Illinois	-	0.48 to 1.03	-0.26 to -0.82	-
Camp	1971	Mississippi	Linear	0.07	-0.03 to -0.29	0.58 to 0.60[b]
Camp	1971	Mississippi	Logarithmic	0.14	-0.35 to -0.40	0.54 to 0.59[c]
Darr	1975	Israel	Linear	-	-	0.42
Darr	1975	Israel	Logarithmic	0.30	-0.13	0.45

[a]This range of values is from plus one to minus one standard deviation from the mean value.

[b]Two of the regression equations developed from this study had all variables in linear form. This was the range of their R^2 values.

[c]Eight of the regression equations developed from this study had some linear and some logarithmic variables included. This was the range of their R^2 values.

the log-linear form of the equation provided the best fit except in the models for outdoor sprinkling. Regressions were also performed on all the data pooled together from the various sample areas. The analysis showed that income and the number of persons per household were the most significant variables. All coefficients had the expected sign with demand per capita increasing with increases in income and decreasing with increases in size of household.

The price of water was not considered significant. The study explained that, in Israel, water costs had "become such a small proportion of expenditure that consumption was not responsive to the prevailing range of prices."

U.S.--Beattie (1978)

In a recent report entitled "A Cross-Sectional Investigation of the Determinants of Urban Residential Water Demand in the United States" the price of water, income, precipitation, and persons per service connection were used to explain water consumption. A least squares regression analysis of the data from all over the U.S. provided price-exponential models predicting water consumption per service connection per annum on a regional basis. An aggregated model was also developed using all of the available data.

All of the models developed by this study agreed that water demand per annum did not vary with the size of the city but did vary from one region of the country to another. The signs of the coefficients were all as expected. Price and rainfall had negative coefficients and income and persons per residence had positive coefficients. The price elasticities in each region were computed and compared with other research done in each particular region. The report concludes that the demand responsiveness to price changes is small because most utilities sell water at a low price. The author believes that "doubling the real price of residential water would move us from an inelastic region of the water demand curve to an elastic region for most of the subregions of the United States." The report suggests that doubling the cost of water would reduce household demand by 30 to 50 percent.

Mississippi--Camp (1978)

This American Water Works Association Journal paper is titled "The Inelastic Demand for Residential Water: New Findings." It reports on residential water demands in Northern Mississippi. A sample of between 28 and 30 individual monthly demands were taken in each of 10 different cities. Along with the water demand data, the participating consumers were asked to answer a questionnaire. The survey obtained information on 13 economic, socio-economic, and climatic factors which were hypothesized as influencing the water demand in the area. The dependent variable was the annual demand per capita. The 13 explanatory variables were:

1. Persons per household
2. Age of household head
3. Market value of residence
4. Irrigable lawn area
5. Number of bathrooms
6. Number of clothes washers
7. Number of dishwashers
8. Existence of a swimming pool
9. Race
10. Average maximum temperature
11. Annual precipitation
12. Price of water
13. Educational level

With these variables, a regression equation was developed for each of the 10 cities. The equations were in both linear and in logarithmic form. Five of the 13 original variables proved to be significant in all 10 equations. These variables were the number of persons per household, number of clothes washers, pool, price of water, and educational level. The two climatic variables, temperature and precipitation, were not important in predicting demand. These two variables would be necessary when predicting demand across different climatic regions but, given this one specific area of the country, rainfall and temperature did not vary enough to influence demand. Another variable which has proven significant in studies done in semiarid areas is the irrigable lawn area. Because of the humid conditions which prevail in Mississippi, this variable was not significant. All of the significant variables had the expected signs.

This report suggested that utility authorities could increase revenue by raising the prices of water, but if discouraging consumption was their objective then, price changes would be an ineffective tool. Price elasticities are computed for each of the 10 equations. Although the report title emphasizes the inelasticity of demand, it reported some elasticities as high as 40 percent which were certainly significant.

Utah--Gardner and Schick (1964)

This report is entitled "Consumption of Urban Household Water in Northern Utah." It is of special interest to the proposed research because the geographic location of both studies overlap. Although the data were collected some 15 years earlier, the functional relationships should be similar to those observed during this research after correcting for inflation.

Gardner and Schick attempted to relate household water consumption in Northern Utah to several variables. The study objective was to explain the cross-sectional variations in the water consumption of average households among various communities. The authors did not attempt to explain variations within communities. Forty-three cities were used in the study, each with a population of at least 1000 people.

The authors used linear and logarithmic regression techniques to relate the average daily water consumption per capita to seven different variables, as follows:

1. Price of water
2. Average income
3. Median home value
4. Lot area
5. Percent of homes with complete plumbing
6. Precipitation
7. Temperature

In the linear model it was shown that only price and lot area were significant. The equation explained only 51 percent of the variation with these two variables. When a logarithmic fit was tried with the same two variables, the percent of the explained variation went up to 83 percent. Precipitation and temperature again showed no significance in predicting water consumption on a town to town basis. This was explained as being caused by the limited variation in the temperature and the precipitation between the 43 towns.

U.S.--McPherson (1976)

Another source of information on household water demand is "Household Water Use." This report will not be reviewed in detail here because it is a review of the literature itself. Nearly one hundred references are listed as well as water use data for nine different homes on a minute to minute basis.

Short Term Demand Literature

The bulk of water demand literature is focused upon monthly or annual water use, because that type of historic data is routinely recorded by almost all water utilities. Short term data such as hourly and shorter interval peaks are almost nonexistent except for that recorded during a few research projects. All of the literature reviewed in this section except Howe and Linaweaver (1967) adopt the concept of relating short term peaks per connection to number of connections served rather than to the socio-economic factors which are commonly regressed against longer term demands. Table 3 is a summary of results reported in this literature.

Kansas--Williams (undated)

This unpublished report was written by a Farmers Home Administration engineer in Kansas. Peak instantaneous demands from four separate water systems (16 to 185 connections) in rural Kansas were recorded during the summer of 1966 and 1968. No towns were included. All of the systems served farmers or non-farm tracts in rural areas. All of the systems purchased their water from cities. Master meters were located at the point of connection to the cities' source.

The peak monthly demand was found from the past records on monthly water usage. A survey determined the day of the week with maximum water usage and also the hour of the day. Peak flows were then found by reading the master meters at one minute intervals during the peak periods of several days.

The report concluded by stating the maximum instantaneous demand which had been observed on each of the four systems. These values are included in Table 3. The values of instantaneous demand reported seem very low (see Table 3). Only a few meter readings were taken at each site. It is possible that the engineer missed the actual peak demand by not recording more than one or two days of data from each system. It is also possible that these low water use values are typical of rural Kansas systems. The price of water at the time of the report was approximately $1/kgal.

Mississippi--Ginn, Corey, and Middlebrooks (1966)

This study objective was to determine instantaneous peak water demands in rural, northern Mississippi. Because of the poor economic status of rural northern Mississippi only very small peaks were observed. The researchers reasoned that as economic conditions improved, rural water demand would approach urban demand levels. Thus, an upper-middle class subdivision in the area was used to collect the demand data which were used to estimate future rural water demand.

Data for the study were taken by recording individual household meter readings from 15 households at one minute intervals during the daily morning and late afternoon peak demand periods. These peak demand periods had been previously determined by meter readings taken at 15 minute intervals throughout the day. These 15 individual daily demand hydrographs were then combined using probability to define an aggregate demand for any number of service connections.

Maximum peak instantaneous demand was never measured for the entire water system being used in this study. Therefore, no maximum value for this report is included in Table 3. However, the design criteria recommended by this research is shown in Table 3 for three different numbers of service connections.

Oklahoma--Goodwin (1973)

In this study, three separate laterals (34 to 39 connections) on the same water system were monitored for peak instantaneous water demands. Battery operated counters were attached to mutating disk water meters to count the number of seconds which elapsed between successive 100 gallon flow volumes. Data were analyzed on a monthly, daily, and instantaneous basis. Water usage characteristics were determined for several dif-

Table 3. Peak instantaneous and peak day demands for various studies.

Water System	Average Monthly Demand K gal/conn.	Number of Connections	Highest Measured Instantaneous Peak (gpm/conn.)	Highest Measured 24 Hour Peak (gpm/conn.)	Date of Measurements
Kansas (Montgomery #6)	4.5	185	0.32	0.165	1968
Kansas (Montgomery #3)	7.4	21	0.52	0.30	1966
Kansas (Montgomery #1)	5.8	36	-	0.25	1966
Kansas (Allen #6)	-	16	0.75	-	1968
Mississippi (Ginn et al.)[a]		10			1966
Mississippi (Ginn et al.)[a]		50			1966
Mississippi (Ginn et al.)[a]		100			1966
Oklahoma (District #3)	7.0	100	1.85	0.40	1974
Utah (Lapoint)	-	4	4.0	-	1975-76
Utah (Lapoint)	-	12	2.25	-	1975-76
Utah (Lapoint)	-	22	2.29	-	1975-76
Kansas (Johnson)	-	100	0.90	-	-
U.S. West (Howe and Linaweaver)	-	44 to 410	1.7 (mean)	0.68	1963-65
U.S. East (Howe and Linaweaver)	-	44 to 410	1.2 (mean)	0.54	1963-65

[a]The values given for the research done in Mississippi (Ginn et al., 1966) is computed from a probability function at a recurrence interval of 27 years.

ferent types of residences (residences were differentiated by apparent economic value) and for dairies.

One conclusion of the study was that no difference in demand was seen between most types of residences. Differences in demand did occur between dairies, the lowest economic level of residence, and all other residences. Values for peak instantaneous demand were recorded accurately on paper tape and are summarized in Table 3.

Utah--Hughes, Kono, and Canfield (1977)

This study measured instantaneous peaks on three laterals (4 to 22 connections) of one rural water system in eastern Utah for two summer periods. The daily peak instantaneous demands were analyzed statistically to determine the frequency distribution of peak demands. A principal conclusion of the study was that measured peak demands were far below Utah State Division of Health requirements but were in between FmHA average and minimum standards.

U.S.--Howe and Linaweaver (1967)

This report represents a large data base from which residential water demand can be estimated. The data were recorded by master meters at 41 residential areas across the U.S. These data were recorded on punch tapes at 15 minute intervals over three years.

Although the research primarily studied the effect of climate on demand, and price and income elasticities of demand on a monthly or annual basis, estimates of instantaneous demand were also made. These values are for urban residential areas, however, and not from rural settings. Average values for instantaneous demand over a range of number of connections are presented in Table 3.

Literature Relating Short Term to Long Term Peaks

Eight relatively recent (1955-1978) water supply textbooks were reviewed for design criteria recommended for short term peak design. All of these texts stated that there were many factors which influenced water demand. The most common factors mentioned were:

1. Community size
2. Geographic location
3. Standard of living
4. Water pressure
5. Water quality
6. Water rates
7. Percent of sewage connections
8. Percent of metered connections
9. Climate
10. Character of community

All of the authors indicated that as the size of the community increases the consumption per capita also increased. This tendency is considered to be related to the character of the community. As the size of a community increases, very often industry and commercial water uses also increase. The increase in industrial water use shows up as an increase in the quantity of water used per person (Steel, 1960).

Geographic location and climate can affect water consumption dramatically. It has been shown that the sprinkling demand during the summer months can be a major portion of the peak daily demand. In an arid region or an area experiencing hot, dry weather, this portion of the demand can become as much as 95 percent of the peak hourly demand (Clark and Viessman, 1966).

The economic status of the population can influence water demand. People who enjoy a higher standard of living can afford more water using appliances. Also, they are not as concerned about water rates. It has also been shown that an increase in quality of the water will bring about an increase in consumption (Walker, 1978).

Finally, the pressure maintained in the distribution system will significantly influence water consumption. A system under high pressure will use more water than one of a lower pressure. Leakage out of water systems is a major contributor to the consumption level. Higher pressure systems will have significantly higher leakages (10-15 percent leakage is not unusual) (Walker, 1978).

The primary objective of this section of the literature review was to determine what multiplying factors textbooks recommend for estimating peak day and peak hour water demand. The factors found in the eight textbooks are summarized in Table 4.

Most of the authors gave certain ranges that peak daily or peak hourly demand could be expected to fall into. Two of the texts, Clark and Viessman, and Hardenbergh and Rodie, gave examples of communities which far exceeded the normal range of peak hourly flow (700-1200 percent of average demand). The authors attributed the unusually high peaks to excessive usage of sprinkling water in small areas where the economic status of the population was high and lawn areas were large.

Table 4. A summary of estimating factors relating peak day and peak hour water demand to average daily water demand.

Authors	Year of Publication	Peak Day as % of Average Day	Peak Hour as % of Average Day
Babbitt Doland	1955	150-250	300-400
Linsley Franzini	1972	180	360
Steel	1960	180-200	270-300
Walker	1978	-	312
Twort Hoather Law	1974	150-200	< 400
Clark Viessman	1966	120-400	150-1200
Fair Geyer	1961	150	250
Hardenbergh Rodie	1970	150	250-300

DAILY AND MONTHLY DEMAND

Objectives

The overall objective of this portion of the research was to develop Utah domestic water demand functions for average month, peak month, and peak day parameters. The following specific objectives guided the research design.

1. The demand functions shall be in a form that is usable by consulting engineers in design situations. That is, the demand determinants used shall be limited to those for which data are easily obtainable in planning situations.

2. The data used shall be obtained from both low density rural and urban systems so that differences can be identified.

3. The data used to produce demand functions shall consist of measurements of water actually entering the distribution systems. That is, it will not be a summation of flows at individual residential meters (so that leakage and unmetered public uses will be included). Also, it will exclude such things as reservoir overflow from springs.

4. The degree to which separate irrigation systems are used to supplement the domestic water system shall be represented explicitly in the demand models.

Demand Determinants

As described in the literature review, most multiple regression approaches to estimating water demand functions include price of water, income of water users (or some surrogate such as appraised value of property), size of lot, and climate. The first three of these parameters seem to be justified by classical micro-economic theory which argues that the demand for any product is determined by price, income of the buyer, tastes, and prices of closely related commodities (Watson, 1968). Climate is an obvious additional demand determinant where any outdoor irrigation is involved. Taste and price of related commodities are normally deleted from water demand considerations because there are no closely related commodities and the fraction of income involved is so small that substitution effects are not quantifiable.

In Utah, the widespread use of separate irrigation systems appears to be extremely important in determining total demand from municipal and domestic systems but this factor has not been quantified in previous studies. Most Utah communities have either an irrigation ditch network or pressure pipelines which provide all or some fraction of outdoor demand to all or some fraction of residences within the service area. Since outdoor summer demand is the dominating component which determines required system capacities and since the fraction of that component which is served by the municipal systems in Utah varies from 0 to 100 percent, quantification of that fraction should be important in explaining variability among water systems. This hypothesis was tested as part of this study and indeed, the outdoor use index developed here plus the retail cost of water were the two variables which were best correlated with water demand.

Income was not used explicitly in this study as a demand determinant even though it has been found to be somewhat significant in previous studies (but less important than price of water). Beattie (1978) for example, calculates an income elasticity of 0.37 for all regions of the U.S. while price elasticity for the Plains and Rocky Mountains region (at 1970 price) averaged 0.60. Elasticity is defined as the percent change in water quantity per percent change in price (or income) as follows:

$$\varepsilon_p = \frac{\Delta D/D}{\Delta P/P}$$

Where D is quantity of water demand, ΔD is change in D, P is price and ε_p is price elasticity. Although income appears to be significant at the micro level in explaining differences between various neighborhoods within a city, few engineers actually gather the detailed data required to use different design criteria within a single city. Differences in aggregate income averages between Utah cities probably vary over a relatively small range. Also, one of the reasons income is significant is that it is highly correlated with lot size which is considered here indirectly by the outdoor use index. Another reason for deleting income is that it is not subject to management while lot size or its surrogate, outdoor index can be managed by zoning or by development of supplemental irrigation systems.

One of the short comings of much of the previous research in this field is that it produced demand functions which require data which are difficult to obtain in the field and results are often presented in a form focused upon micro-economic theory rather than design criteria with which most consulting engineers can be comfortable. The focus of this study is upon developing design equations which require data which are readily available and which is easy to use. During the search for these equations the following parameters were considered: Price of water, outdoor use index, size of system and persons per connection.

Multiple Regression--14 System Sample

Data

The data used in this analysis were all either obtained from the water utility managers or actually measured by the study team during visits to the sites. Project personnel also examined system layouts and meter locations. This avoided reliance on mailed questionnaire forms which may not have fit the actual site specific situations at each utility or which may have been interpreted incorrectly by respondents.

The various time related water demand functions were determined by examining possible correlation with the variables noted above. These variables and the measured water demands are included in Table 5. Average and peak month data were available for all 14 systems and peak day measurements were available for 10 systems. The water demand data used were 3 year averages (1974, 1975, and 1976). Additional data were available for most systems but were excluded because:

1. 1977 data were dramatically affected by drought related restrictions at most systems. These data were therefore excluded (but are being studied in connection with a drought impact research project).

2. The objective was to obtain data resulting from constant values of the variables which affect demand. Use of more than 3 years of data would have involved several changes in price of water during the period.

Water connections

The number of connections shown in Table 5 refer only to residential connections and these were represented by the averages during the 3 year period. The unit demands for each year, however, were determined by the number of connections for that year. The commercial and industrial demand was included as part of the total demand per residential connection or person. The number of residential connections in Salt Lake City and Bountiful are only estimates since neither utility distinguishes between residential and other connections. The number of persons per connection includes some error where census data were not available for the exact service area and also for Salt Lake City where the number of residential connections were only estimated. The large number of persons per connection in Salt Lake City and Bountiful was undoubtedly a reflection of the large number of multiple dwelling units served by a single service connection (not large families).

Because of the obvious distortion of demand per connection produced by large apartment buildings with only one connection, demand functions were developed on a per person as well as a per connection basis. This distinction appears to be unimportant in smaller systems but very significant in urban areas where number of water connections is significantly less than the number of households.

Price of water

The prices shown in Table 5 are average prices per thousand gallons (kgal). These prices were selected as being closest to representing the water users' perception of what they are paying for water. Marginal prices are approximately $0.20/kgal for the average system while average price was $0.83/kgal. This difference was caused by the minimum monthly charges which were prorated into the average unit prices.

The systems included in the sample were all Utah systems except for Penrose and Mancos which are small Colorado systems. These two utilities were included to provide data points in the high price range of the demand functions. They are close to Utah and experience similar climatic and social conditions.

Outdoor use index

It has long been known that outside use (principally yard irrigation) is a very important factor in summer water use and that this component of demand varies greatly among Utah systems. Some areas have supplemental pressure pipelines or ditch systems from which all or part of the outside demand is supplied. Another complicating factor is the great variation in area landscaped between rural and urban and between old and new developments within systems. Still another factor impacting on outside demand is climatic variations within the state such as between Utah's Dixie and the higher and wetter northern valleys.

An objective of this study was to develop a single, easy to use index which would account to the greatest extent possible for all factors which collectively determine outside demand from a municipal-domestic system. This index is shown in Table 6. It associates an integer between 1 and 9 with each of nine outdoor use category descriptions. These descriptions, although somewhat subjective, are reproducible and provide a reasonable easy means for defining the index number. The index number 1 is a system

Table 5. Multiple regression data.

Water System	Avg. Cost (P) ($1/1000 gal)	No. of Residential Conn.	Persons Per Conn.	Outdoor Use Index (I)	Average Demand[a]		Peak Month Demand		Ratio pm/avg	Peak Day Demand		Ratio Pd/avg
					$D_{avg/c}$	$D_{avg/p}$	$D_{pm/c}$	$D_{pm/c}$		$D_{pd/c}$	$D_{pd/p}$	
Chesterfield	0.30	519	3.33	4	21.1	211	39.9	399	1.89			
Draper	0.35	885	4.13	6	29.0	234	53.5	432	1.85	2485	529	2.26
So. Price	1.62	124	3.21	4	12.46	129	23.5	244	1.89			
Orangeville	0.57	227	3.43	5	18.6	181	21.5	209	1.16	1312	382	2.12
Brooklyn Tap	0.47	34	3.3	4	12.4	125	18.7	189	1.51	891	270	2.15
Monticello	0.30	525	3.2	9	29.7	309	59.7	622	2.01	2227	696	2.24
Bell Canyon	0.30	270	4.0	7	38.1	318	85.2	710	2.24			
Wellington	1.35	565	3.21	3	9.54	99.0	12.9	134	1.35			
Salt Lake City	0.18	63,000	4.55	8	29.2	214	70.9	519	2.42	2466	542	2.53
Bountiful	0.34	6,340	4.9	1	14.8	101	17.6	120	1.18	598	122	1.21
Penrose	1.86	680	3.3	2	5.07	61.3	9.50	97	1.58	461	140	2.28
Mancos	2.55	158	3.3	3	9.0	91.0	9.9	100	1.10	572	173	1.90
North Emery	0.85	497	3.43	3	11.5	112	17.3	168	1.50	720	210	1.87
Duchesne	0.52	459	3.61	5	16.5	152	24.6	227	1.49	1397	387	2.54
Averages	0.83		3.64		13.4	167	33.2	298	1.78	1283	345	2.06

[a]Notation: The symbols used in Table 5 and in subsequent demand functions are defined by the table column headings plus the following matrix.

Parameter	Thousand Gallons Per Month, Per Connection	Gallon Per Day: Per Conn.	Gallon Per Day: Per Person
Average	$D_{avg/c}$		$D_{avg/p}$
Peak Month	$D_{pm/c}$		$D_{pm/p}$
Peak Day		$D_{pd/c}$	$D_{pd/p}$

Table 6. Outdoor use index (I). (Principally irrigation but includes stockwater in rural areas.)

Index (I)	Categories Indicating Extent of Outdoor Demand From Domestic System
1.	No outdoor use from domestic system--everyone has connection to pressurized dual system.
2.	Almost no irrigation from domestic system--supplementary system is available which serves at least 85% of outside demand.
3.	Supplementary ditch system is available and landscaped areas are very small (average less than 1500 square feet).
4.	No supplementary system is available but landscaped areas are very small (average less than 1500 square feet).
5.	Ditch system available for gardens but lawns (over 60%) are irrigated from domestic system.
6.	Ditch or piped system available to some customers but most outside irrigation (over 75%) is from domestic system.
7.	All outside demand from domestic system--moderate amount of landscaping, average Utah climate.
8.	Large amount of landscaping and all from domestic system--average Utah climate.
9.	Large amount of landscaping and all from domestic system--hot and dry Utah climate.

which provides no outside water (such as Bountiful City) and increases as outside irrigation increases up to 9 which represents a city is which all of the outside demand is furnished by the municipal system and from which relatively large landscaped areas are irrigated in a hot-dry Utah climate (such as Monticello). The index numbers themselves are ordinal in nature; that is, they simply rank the outdoor water use in increasing order. There was no a priori reason, for example, to expect this component of demand in an I = 8 system to be twice as great as in an I = 4 system. However, the index numbers were used in the multiple regression analysis as if they did have a quantitative relationship and the results were surprisingly productive.

It is believed that despite the subjectivity of some portions of the index descriptions that any design engineer could determine an index from Table 6 for any given system and that the selection would vary only slightly (perhaps one integer) from that selected by another designer.

Results

Average demand

A scatter diagram of average water demand vs price with associated best fit demand functions for various outdoor use index values are shown in Figure 2. The average demand function expressed in thousands of gallons per connection per month (see Table 1 for notation) is:

$$D_{avg/c} = 4.60 - 5.40\, \ell n(P) + 2.39(1)$$

This semi-log form of equation implies that monthly use increases about 2400 gallons as outdoor use index increases by one integer. Since demand also varies inversely with the natural logarithm of price this function plots as a straight line on semi-log paper. This more convenient form of graphical representation is used in Figures 3 and 4 and for all subsequent semi-log functions. Figure 3 displays the average monthly demand in thousand gallons per connection. Figure 4 displays average demand in gallons per capita per day. This dual set of dimensions will also be used for the peak period functions for the following reasons:

1. Some engineers plan systems based upon future population projections. This makes the gallon per person dimension convenient.

2. Some engineers make their own projections based upon number of existing connections. This may be a more reliable parameter particularly when the utility's service area does not coincide with the population census boundary.

3. Trailer courts and apartment buildings where large number of people are served by a single water connection tend to distort the gallons per connection figure and the per capita functions may be more accurate in such areas. It should be noted, however, that for the 14 system sample, the degree of correlation was very similar for both sets of units. For example, Figures 3 and 4 indicate that for average demand the R^2 coefficients are identical at 0.83.

The F test for significance of the correlations for these and all other demand functions to be described later are summarized in Table 7. If the mean square ratio given in column (4) (the variance from the mean which is explained by the regression divided by the unexplained deviation) is greater than the F value given in column (5), this implies a statistically significant regression has been obtained (only 5 chances in 100 that the correlation was due to chance). However, column (4) should be considerably greater than column (5) for the function to represent a good predictive equation. Draper and Smith (1966) suggest that a ratio of 4 or greater is desirable for a good predictive equation. The ratios given in column (5) for the average demand func-

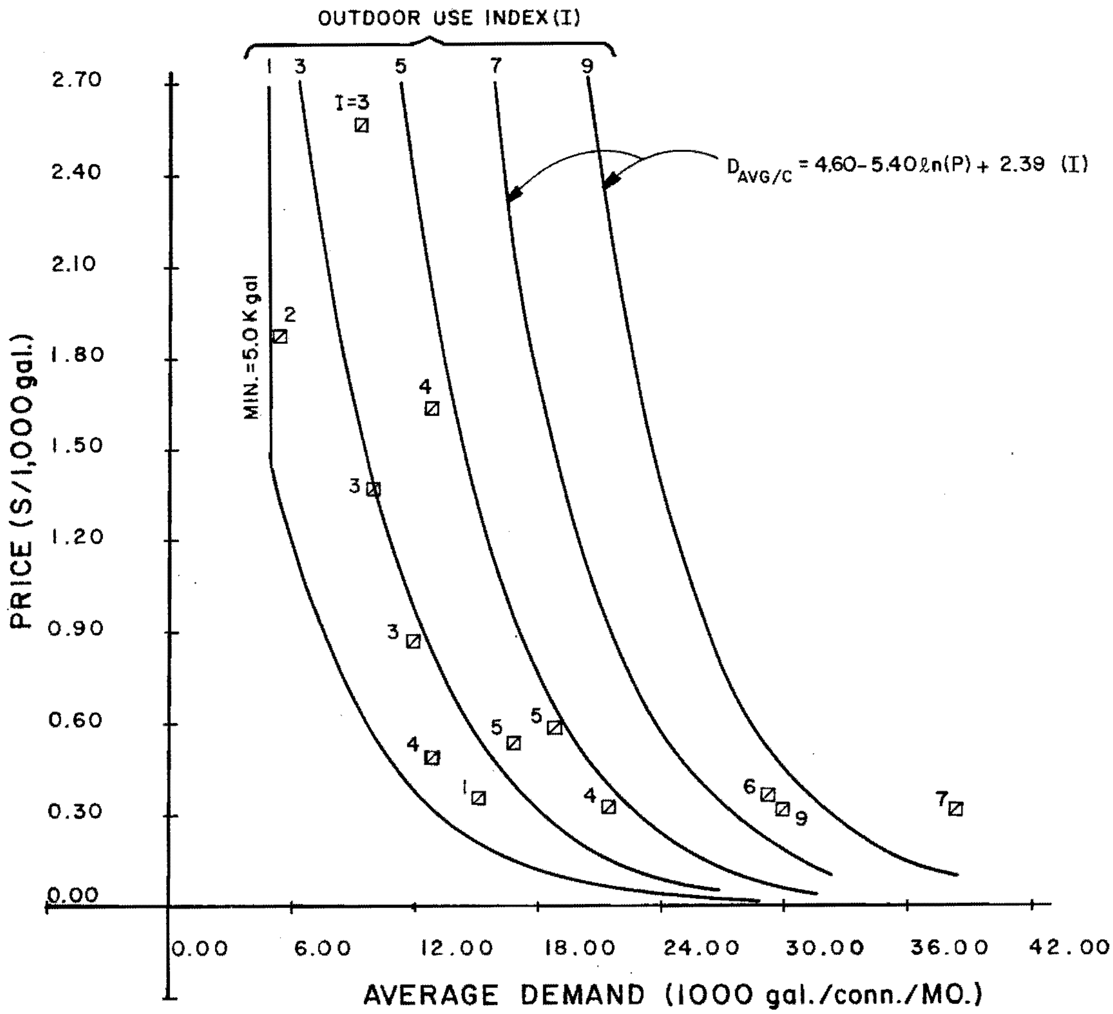

Figure 2. Average monthly demand and outdoor use index data.

tions are 6.1 and 5.7. They both qualify, therefore, as statistically good predictive equations.

The R^2 values can be thought of as the decimal fraction of total deviation from the mean which is explained by the regression function. All of the R^2 values in Table 7 are relatively high but this indicator should not be used alone as evidence of good correlation (particularly where the number of data points is small) but rather in conjunction with the F test (which considers degrees of freedom).

It should be noted that Figures 3 and 4 represent the expected value of average demand rather than a safe value for design purposes. The figures therefore include a note suggesting a 20 percent increase for design of water right purposes. Justification for the 20 percent figure will be discussed later.

Peak month demand

Sustained periods of high demand are sometimes important in determining storage capacity or required sustained yield of wells. Peak month demand can be estimated from Figures 5 or 7 in kgal per connection ($D_{pm/c}$) or from Figures 6 or 8 in daily gallons per person ($D_{pm/p}$). Figures 5 and 6 express peak month demand as a function of average demand. In planning situations where average demand is not reliably known, however, Figures 7 and 8 can be used to predict peak month requirements as a function of price and outdoor use index as before.

Any of these functions are statistically adequate predictive models, but as shown in Table 7, Figures 5 and 6 have particularly good correlation (mean square/F ratios of 41 and 35 compared to 5.4 and 6.2 for Figures 6 and 7). The R^2 ratios vary from 0.80 to 0.942 (1.0 implies perfect correlation). The

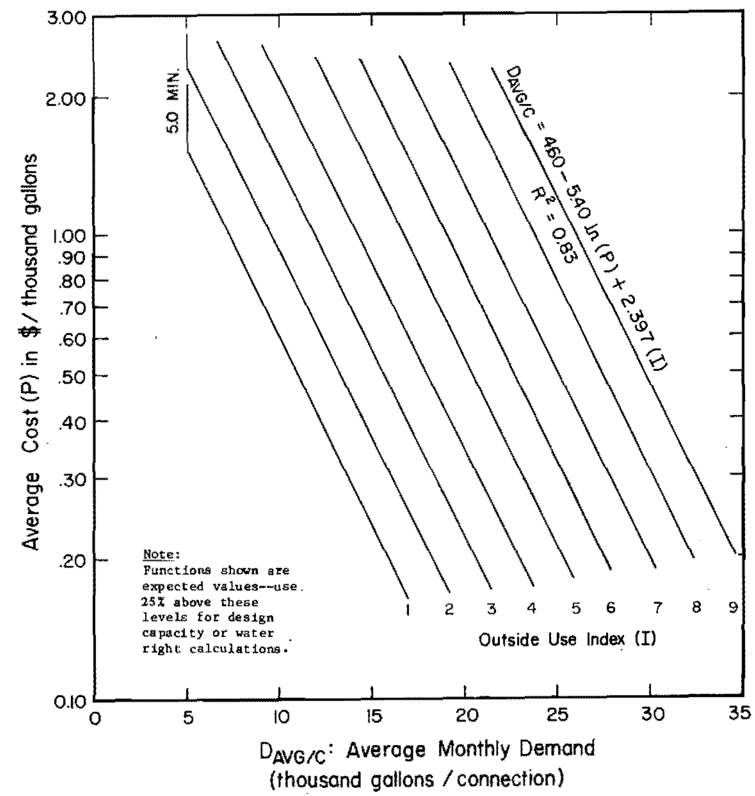

Figure 3. Average monthly demand per connection.

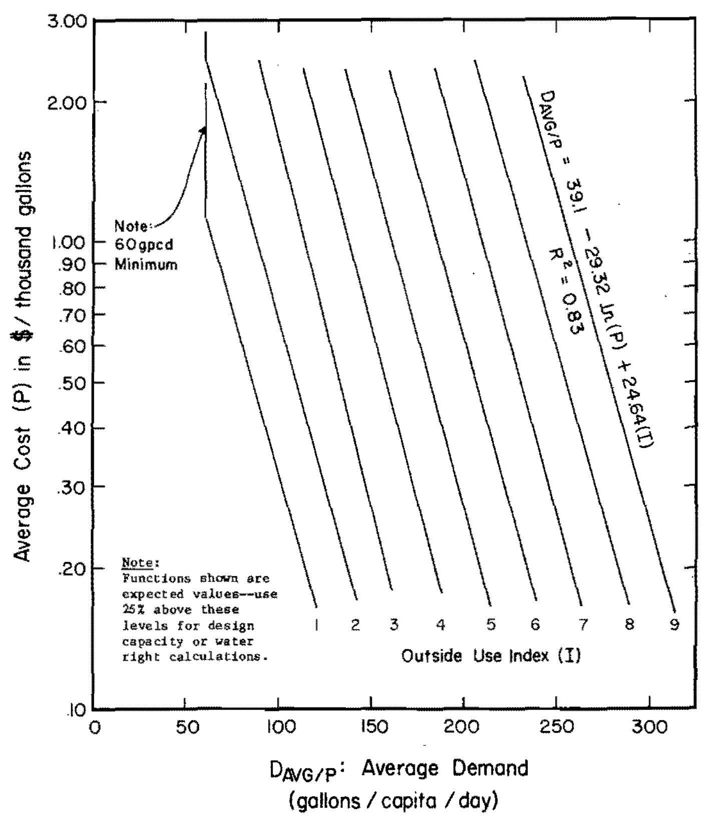

Figure 4. Average monthly demand per person.

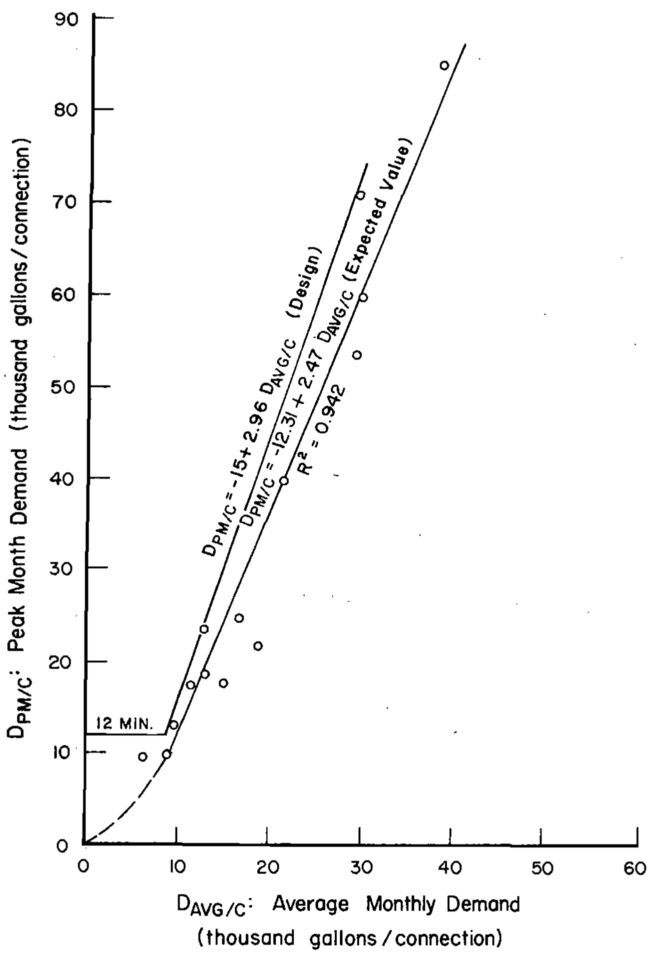

Figure 5. Peak month demand per connection as a function of average demand.

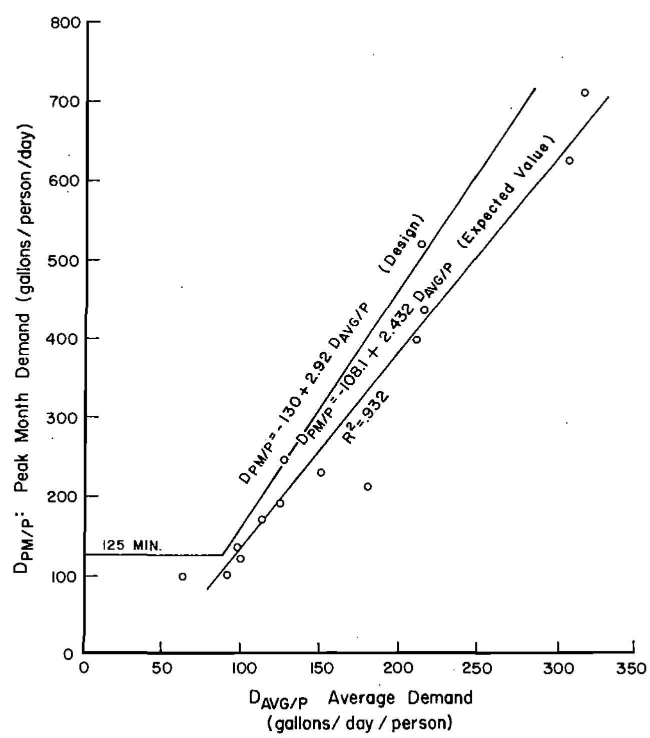

Figure 6. Peak month demand per person as a function of average demand.

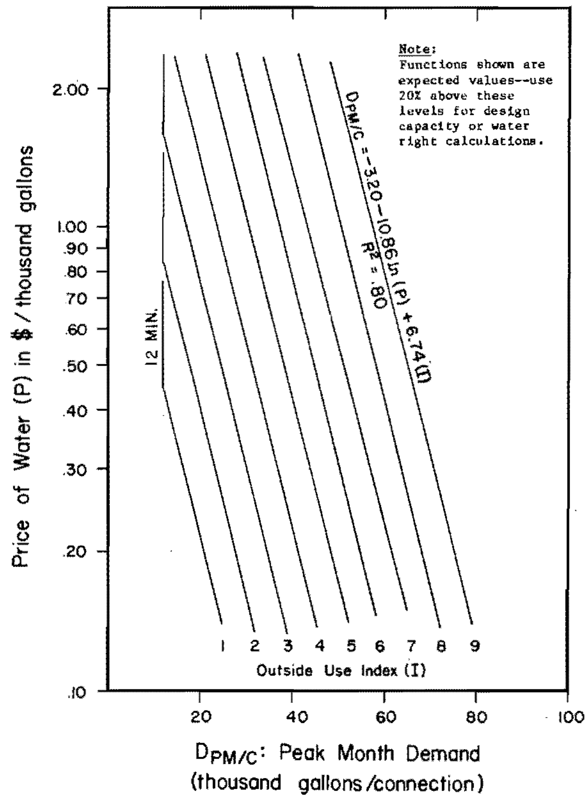

Figure 7. Peak month demand per connection as a function of price and outdoor use index.

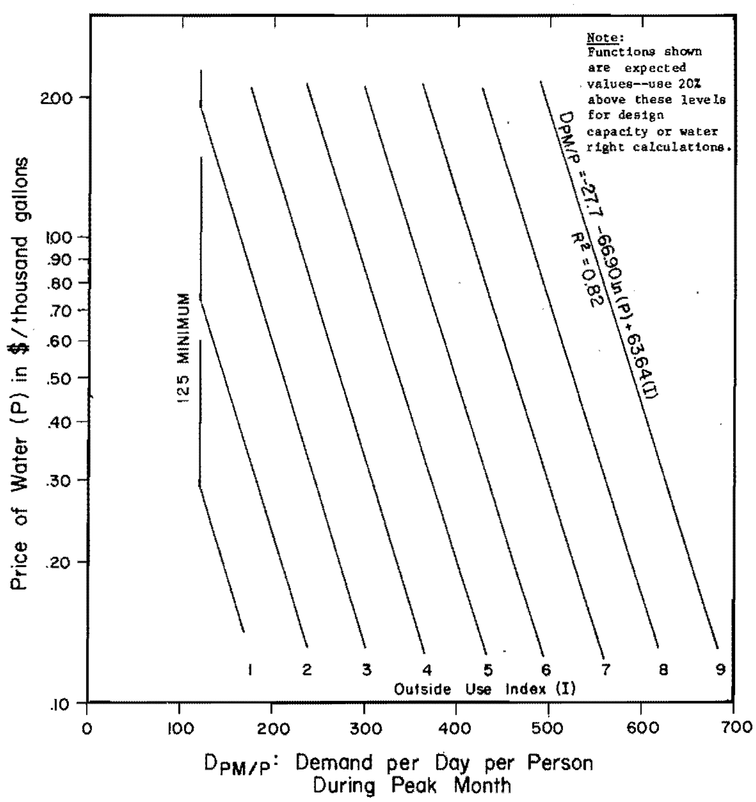

Figure 8. Peak month demand per person as a function of price and outdoor use index.

Table 7. Statistical significance of the demand function correlations.

Figure	(1) Demand Parameter (Dependent Variable)	(2) Independent Variables	(3) Correlation Coefficient (R^2)	(4) Measured Mean Square (F) Ratio (Model/Error)	(5) Tabular F Distribution Value at 95% Confidence Level $F(v_1, v_2, 0.95)$	(6) Ratio (4)/(5)
3	$D_{avg/c}$	P,I	0.83	27.0	3.98	6.80
4	$D_{avg/p}$	P,I	0.83	26.4	3.98	6.63
5	$D_{pm/c}$	$D_{avg/c}$	0.942	194.9	4.75	41.0
6	$D_{pm/p}$	$D_{avg/p}$	0.932	164.8	4.75	34.7
7	$D_{pm/c}$	P,I	0.80	21.4	3.98	5.4
8	$D_{pm/p}$	P,I	0.82	24.8	3.98	6.2
9	$D_{pd/c}$	$D_{avg/c}$	0.938	120.8	5.32	22.7
10	$D_{pd/p}$	$D_{avg/p}$	0.953	163.9	5.32	30.8
11	$D_{pd/c}$	I	0.853	46.5	5.32	8.7
12	$D_{pd/p}$	I	0.934	113.2	5.32	21.3

difference between the best fit lines (expected value) and the design functions will be discussed later.

Peak day demand

The capacities of many water supply facilities such as treatment plants, pump motors, equalizing reservoirs, etc., are determined by average demand during the peak 24 hour period. Figures 9 and 11 represent the best fit regression functions for peak day water demand in gallons per connection ($D_{pd/c}$) and Figures 10 and 12 give the same demand in gallons per person ($D_{pd/d}$). The first two (9 and 10) express the peak day requirement as a function of average demand. In situations where reliable data on average demand are not available Figures 11 and 12 can be used to estimate the peak day demand as a function of the outdoor use index.

Note that average and peak month demand are significantly correlated with price of water but peak day is not. This is to be expected since water users are billed on a monthly basis and are concerned about minimizing monthly costs but have no economic incentive to distribute their monthly demand more equally over various days in that month.

The statistical correlation again was extremely good between peak and average demands. The alternative procedure, expressing peak day requirements as a function of outdoor index only also produced excellent results, particularly when expressed on a per person basis.

Statewide survey analysis

Background. During 1977 and 1978 related research was conducted at UWRL which was financed by the U.S. Bureau of Reclamation in cooperation with the Utah State Divisions of Water Rights, Water Resources and Health. The objective of the related study was to gather and analyze historic municipal water use data from Utah systems and to estimate future water use. A detailed questionnaire was mailed to 450 Utah municipal and rural domestic water utilities from which 154 replies were received. The results of that survey are presented in detail in a separate report (Hansen et al., 1979).

That survey provided an expanded data base related to the average demand functions reported here including such determinants as water price, outdoor use, size of system, etc. The survey data were assumed to be less reliable than the 14 system sample because of possible distortions from such things as reported water use including reservoir overflows, unknown amounts of leakage, some water use estimates rather than actual Master Meter readings, etc. However, analysis of the survey data suggested that information from 41 systems (in addition to the 14 already studied) appeared complete enough to be of value in this study. These data are summarized in Table 8. The demand functions were developed from the 14 system sample and then tested on data from the 41 survey systems by comparing reported water use to quantities predicted by the functions.

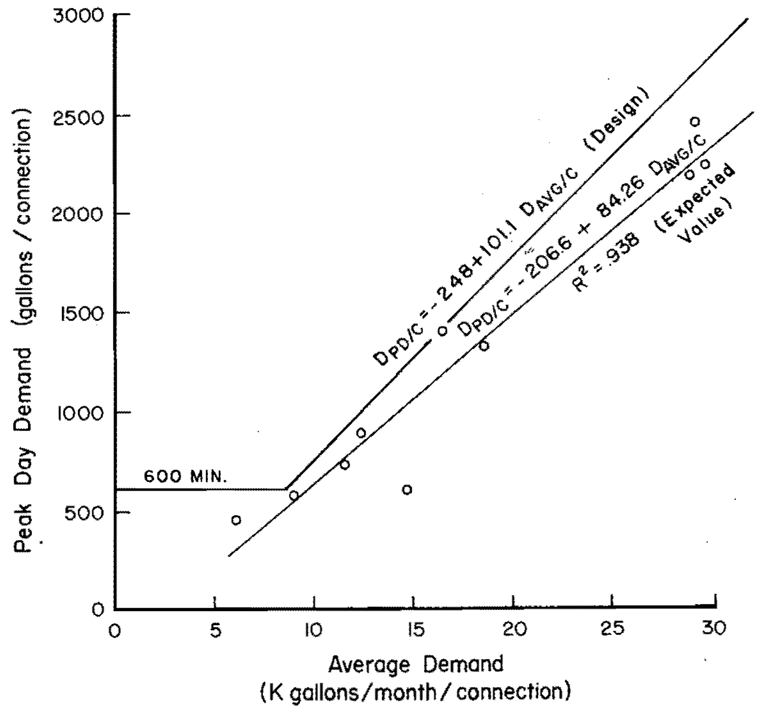

Figure 9. Peak day demand for connection as a function of average demand.

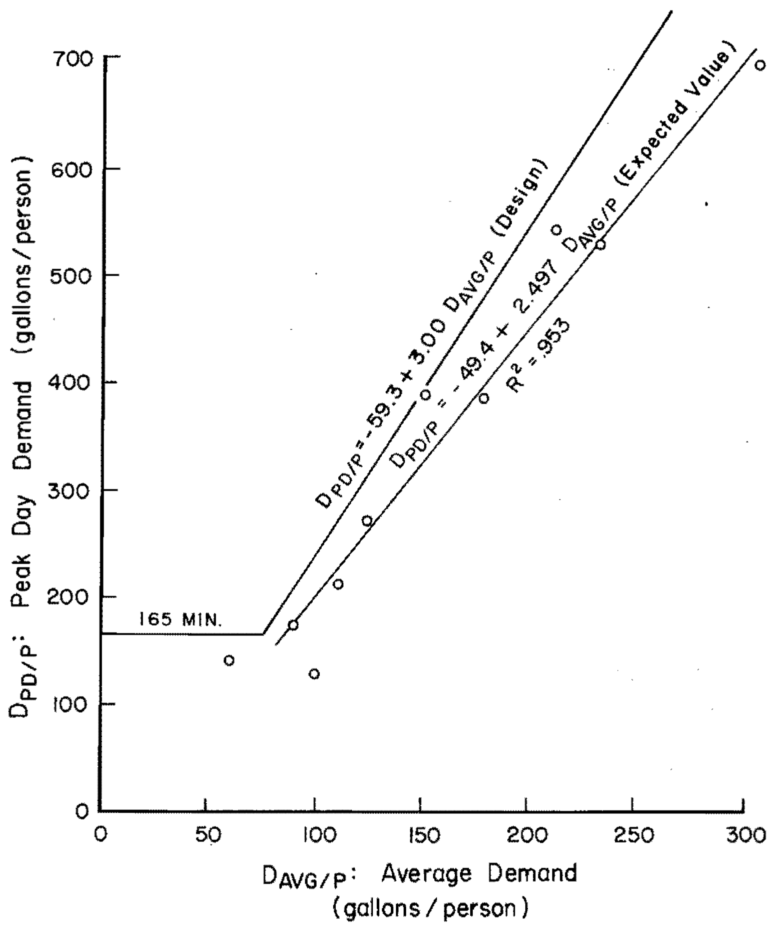

Figure 10. Peak day demand per person as a function of average demand.

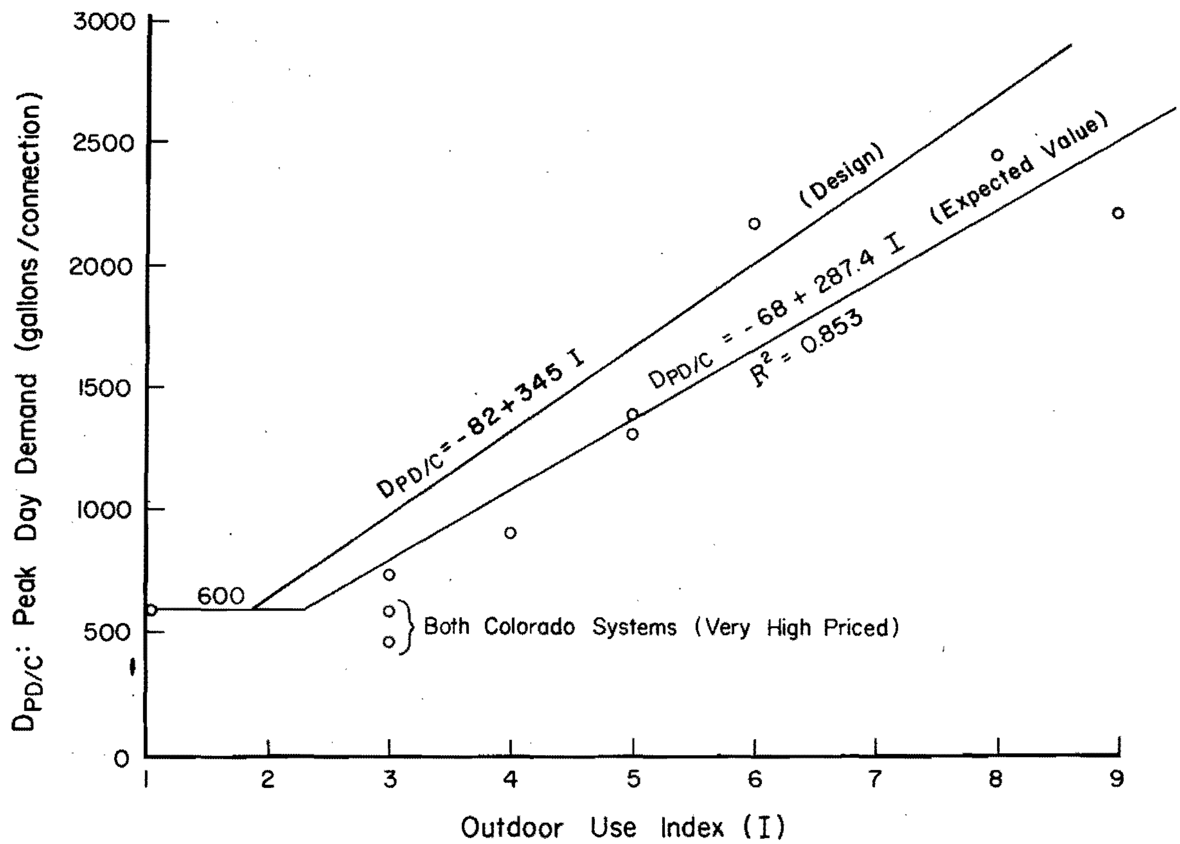

Figure 11. Peak day demand per connection as a function of outdoor use index.

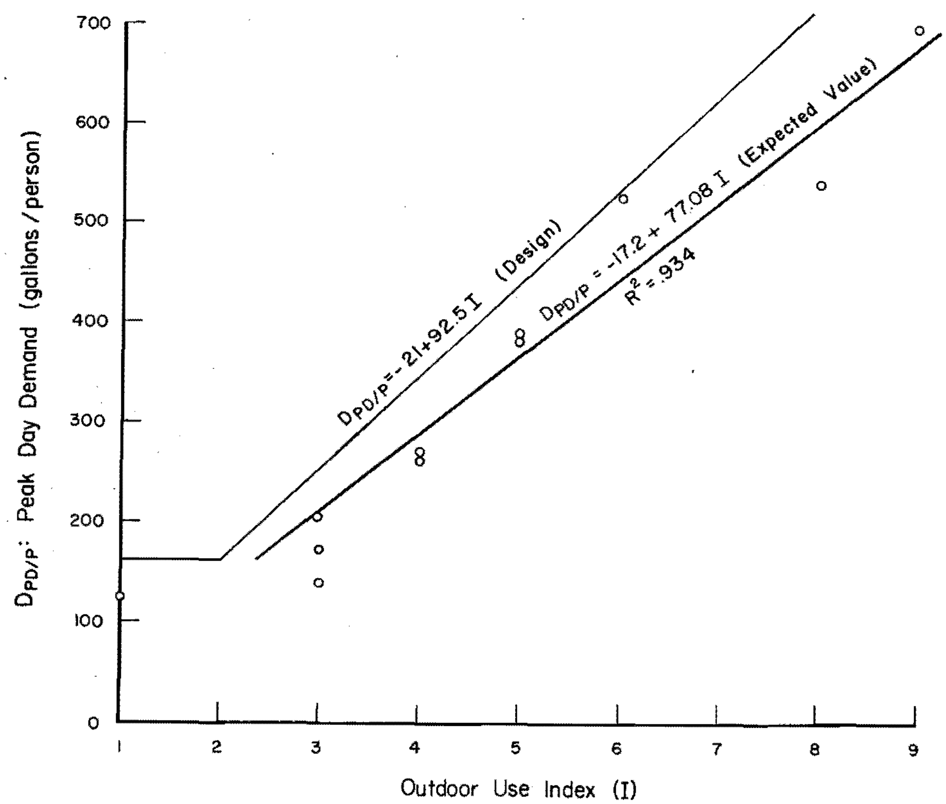

Figure 12. Peak day demand per person as a function of outdoor use index.

Table 8. Statewide survey data--average demand ($D_{avg/c}$).

System	Price ($/kgal)	Outdoor Index	Population	No. of Connections	D_{obs} (avg.)	$\hat{D}$ From Model	$\hat{D}/D_{obs}$ Ratio
American Fork	0.232	6.6	10,462	2,958	24.09	28.3	1.17
Bountiful	0.213	1.8	30,358	6,806	14.59	17.4	1.19
Brigham City	0.219	3.4	14,157	4,003	40.64	21.0	0.52
Centerville	0.387	1	5,198	1,200	12.08	12.4	1.03
Clearfield	0.168	6	13,416	2,625	30.97	28.6	0.92
Clinton	0.151	7.4	3,629	990	14.74	32.4	2.19
Delta	0.202	8	2,016	689	35.11	32.3	0.92
Duchesne	0.360	5.8	2,198	459	21.84	24.0	1.1
Ephraim	0.299	8	2,380	721	34.96	30.2	0.86
Fillmore	0.378	3.8	1,826	885	26.99	19.0	0.70
Green River	0.659	5.0	968	362	17.97	18.9	1.05
Hyrum	0.206	4.2	3,137	1,021	42.40	23.2	0.55
Kaysville	0.123	4.2	7,553	1,224	34.06	26.0	0.76
Layton	0.237	7.4	17,511	4,365	22.26	30	1.35
Lehi	0.274	6.6	5,736	1,686	16.94	27.3	1.61
Logan	0.185	5.8	23,810	6,025	59.87	27.6	0.46
Manila	1.184	3.4	345	200	13.20	12.0	0.91
Midvale	0.239	8	8,310	2,906	28.57	31.4	1.20
Moab	0.223	9	6,400	1,312	35.12	34.1	0.97
Monticello	0.301	9	1,726	612	26.14	32.5	1.24
Morgan	0.172	5	1,704	582	39.70	26.1	0.66
Murray	0.153	8	23,595	5,220	40.95	33.8	0.83
North Ogden	0.321	3.7	6,566	1,740	12.45	19.7	1.58
Ogden	0.285	6.3	68,978	19,424	27.06	26.4	0.97
Orem	0.216	8	35,584	9,334	32.82	30.5	0.93
Pleasant Grove	0.348	6.6	7,074	1,966	36.92	26.0	0.70
Price	0.474	6.3	10,310	4,124	19.50	25.4	1.30
Provo	0.164	7	55,593	10,788	44.99	31.0	0.69
Richfield	0.298	5.8	4,947	1,741	30.86	25.0	0.81
Roosevelt	0.393	8	3,943	1,250	36.91	28.7	0.78
Roy	0.173	3.4	16,781	3,982	29.46	22.3	0.76
Salt Lake City	0.18	8	275,000	73,349	32.28	32.9	1.02
Sandy	0.334	8	36,000	8,670	26.66	29.6	1.11
South Ogden	0.229	1	10,175	3,219	10.24	15.2	1.48
Spanish Fork	0.214	4.6	8,065	2,376	22.40	24.0	1.07
South Salt Lake	0.224	8	9,041	2,626	30.26	31.8	1.05
St. George	0.243	9	8,760	2,500	48.67	33.7	0.69
Sunset	0.241	8	6,300	1,478	23.14	31.3	1.35
Vernal	0.157	5.8	14,000	3,000	40.04	28.5	0.71
Washington Terrace	0.463	1.0	8,078	1,972	12.41	11.4	0.92
West Jordan	0.350	7.6	11,405	3,200	25.88	28.4	1.10
						Avg. Ratio	= 1.00

Results. A two dimensional scatter diagram of the 14 system plus the 41 system data relating average water use to price is given in Figure 13. The figure also shows a least squares function which best fit the 14 system data. This truncated form of the model (it does not include outdoor use index) produced an R^2 of 0.634. The points which are farthest from the best fit line are generally those with extreme outdoor use index values. For example, of a group of five systems in the lower right corner of the figure, four have an outdoor index of 1 (all outdoor use provided by a separate pressure irrigation system). Price elasticity calculations will be discussed in a separate section.

The 14 system average demand function was used to predict the 1974-76 demand ($\hat{D}$). Calculated demands were then compared to the water use reported by the survey (D_{obs}) by using a ratio of the two. This information is also displayed in Table 8. The average monthly demand function used in the comparison (from Figure 3) is:

$$\hat{D} = 4.60 - 5.40 \ln(P) + 2.4(I)$$

The expectation of the research team was that the survey data would be biased toward higher flows than the demand functions because of possible reservoir outflow and high leakage included in the survey data and because the 14 system sample included a

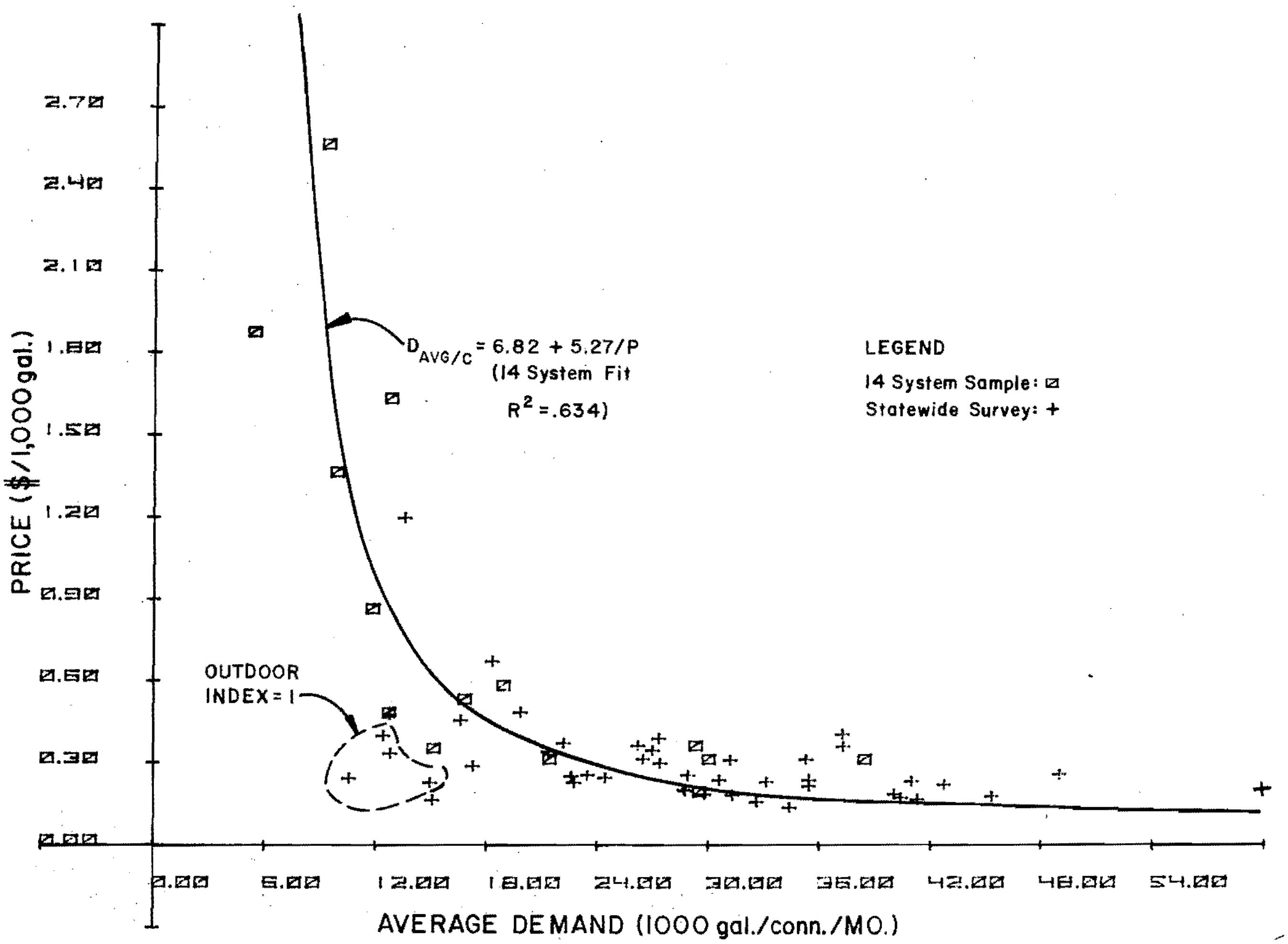

Figure 13. Average demand/price data for 55 Utah systems.

larger proportion of small rural systems. However, this did not occur. In fact, the average of the 41 ($\hat{D}/D_{obs}$) ratios was precisely unity--suggesting both reasonable reliability of survey responses and that the 14 system model represents the characteristics of all Utah systems reasonably well rather than being biased toward rural systems. This implies that in terms of average water use, there is little difference between rural and urban demand and both can be represented by a single demand function (if it includes an outdoor use index). The accuracy of the demand function in predicting the observed demands is summarized as follows:

- $\hat{D}$ was greater than D_{obs} for 19 of 41 systems
- $\hat{D}$ was less than D_{obs} for 22 of 41 systems
- $\hat{D}$ was within 10% of D_{obs} for 37% of the systems
- $\hat{D}$ was within 20% of D_{obs} for 50% of the systems
- $\hat{D}$ was not within 50% of D_{obs} for 9.7% of systems

Presumably, an engineer making a planning study for expansion of an individual system could obtain more reliable outdoor use data than that obtained via the statewide mailed questionnaire and therefore, application of the demand function in that setting should yield better results than those summarized above.

Price elasticity of Utah systems

Price elasticity can be calculated for a particular price level by evaluating the ratio of percent change in water quantity over percent change in price in that range of the function. A more convenient method, however, is to derive an equation expressing elasticity as a function of the specified demand equation being used as follows:

The form of monthly demand function selected from the regression analysis for both average and peak demand was:

$$D = b_0 + b_1 \ln(P) + b_2 I \quad . \quad . \quad . \quad . \quad (1)$$

In order to isolate the variation of D with price only, the outdoor index can be treated as a constant. If price elasticity is given in differential form rather than the finite (Δ) form given previously, elasticity is defined as

$$\varepsilon_p = \frac{\partial D/D}{\partial P/P} \text{ or } \frac{\partial D}{\partial P}\left(\frac{P}{D}\right) \quad . \quad . \quad . \quad . \quad (2)$$

Differentiating Equation 1 we have:

$$\frac{\partial D}{\partial P} = \frac{b_1}{P} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

therefore:

$$\varepsilon_p = \frac{b_1}{D} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Average demand price elasticity

The best fit equation for the Utah average month data was:

$$D_{avg/c} = 4.60 - 5.4 \ell n(P) + 2.4 I \quad . \quad . \quad (5)$$

This implies a price elasticity at average water use (18.4 kgal) of -0.293. Elasticity at one standard deviation (9.7) above and below the sample mean of water use are -0.19 and -0.61 respectively.

If a model giving average demand as a function only of water price is used (shown graphically in Figure 13) the best fit equation is:

$$D_{avg/c} = 6.817 - 5.274/P \quad . \quad . \quad . \quad . \quad (6)$$

Price elasticity for this form of equation is:

$$\varepsilon_p = \frac{-b_1}{PD} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

which at the sample average price and demand ($0.82 and 18.4 kgal) is -0.35. This slightly higher average elasticity is generated by allowing all of the variance explained by the model to be attributed to the price variable (there is not total independence between the price and outdoor index variables).

Peak month price elasticity

The best fit equation for peak month demand is:

$$D_{pm/c} = 3.20 - 10.86 \ell n(P) + 6.74(I)$$

This equation yields a price elasticity of -0.33 at an average use rate of 33.2 during the peak month. At a use rate of one standard deviation above this mean (57.7) elasticity would be -0.19 and -1.25 at one standard deviation below the mean (8.7). Other research has shown that outdoor demand is much more elastic than indoor. The elasticities calculated here for Utah systems verify that relationship (because the peak month includes relatively more outdoor use than the average month) but at mean use levels the peak and average month elasticities (0.29 and 0.33) are closer than anticipated.

Demand variations over time

It has been customary in water planning studies to make a small allowance for growth in unit demand over a period of several years. The rationale for this practice has been the increasing number of water using fixtures such as garbage disposals, more bathrooms per residence, etc. Analysis of data from Utah systems covering the last decade, however, do not support this concept. Total water use continues to increase due to population growth but demand per person has either stabilized or in many cases has definitely decreased (Hansen et al., 1979).

A recent study of water demand in Salt Lake County (Kirkpatrick, 1976) notes a very stable average water demand of 214 gallons/day/person. Possible reasons for this phenomenon include the increase in multiple unit dwellings and the trend of increasing water costs (at a higher rate than other consumer goods) which tends to counteract the previous expectation of ever increasing unit demand. In short, the era of mostly free water sources such as gravity flow directly from springs seems to be ending. As continual population growth and very high energy and importation costs produce ever increasing water treatment and conveyance costs almost uniformly over the state, there appears to be no justification for allowances for future growth in water demand per person.

The demand functions presented here which include price as a variable should be corrected for inflation in future years (using 1975 as the reference year). This has been demonstrated by comparing the average demand-price relationships for the data presented in Table 5 to price elasticity data from a previous Utah study (Gardner and Schick, 1964). The estimated demands are very different if compared directly but become almost identical if the 1964 dollars are inflated to 1975 price levels.

Design recommendations

Safety factor. All of the demand function figures show both the best fit (expected value) function for the empirical data and some indication of a recommended increase for design capacity. The expected value functions should be used for estimating average operating costs and the design functions should be used for capital investment decisions. The objective is to insure design capacities which will meet demand during nontypical demand periods. Actual demand levels can be expected to be greater than expected values approximately half the time and less than these quantities the balance of the time. Often the most difficult questions facing water supply planners are: How far above the expected values will the infrequent peak demand levels reach, and 2) is it possible to design for any peak however infrequent, or is a compromise between absolute confidence in ability to meet peaks and cost of the proposed system necessary?

Such questions in relation to instantaneous peaks were addressed by Hughes et al. (1977). The concept of a design based upon defining an acceptable recurrence interval for demand exceeding capacity which was developed in that report was also used in this study for the instantaneous peak design criteria. The nature of longer term average and peak parameters discussed in this section, however, do not allow so explicit a determination of the recurrence interval vs maximum demand levels. For example, it is possible to obtain a large amount of data on instantaneous peaks during a single summer. However, if average demand at a single system is being analyzed the approximately 30 data points required to develop a reasonable frequency analysis would require 30 years of data. During such a long period many other factors which must be treated as constants (such as price, income, type of plumbing, etc.) would have changed significantly. It is necessary therefore, to make some reasonable estimates of variance levels from a shorter term but multiple system record.

The approach used here is to calculate means and standard deviations of average and peak period data for a large number of systems and to reduce these data to a common basis by using a ratio of standard deviation divided by the mean ($S/\bar{D}$). This expresses an average variation from expected value as a percent of the mean. Then, assuming a normal distribution, inferences can be made about fractions of the time an average or peak period demand will exceed a particular level.

Average demand. Hansen et al. (1979) include means and standard deviations for annual per capita demands of 47 Utah communities (Hansen's Table 11). The data consisted of demand during the 1960 to 1976 period. The dimensionless ratio discussed previously ($S/\bar{D}$) for these data is 0.151. This suggests that if one is interested in estimating a level of flow which will be exceeded only during one of 20 events (one in 20 years), this would be estimated as (15.1)(1.65) = 24.9% above the mean. The calculation assumes a normal distribution and therefore 1.65 standard deviations as the 95 percent exceedance level. The actual design level suggested on Figures 3 and 4 for average demand is 25 percent above the mean (the expected value).

Peak month. The design level for peak month demand is recommended at 20 percent above expected value level (Figures 5, 6, 7, and 8). No reliable peak month standard deviation data were available for a large number of Utah systems. Therefore, this figure was selected by assuming that the $S/\bar{D}$ ratio for peak month flows could be expected to approximate the ratio for peak day demand or at least be within the range between the average month and peak day ratios.

Peak day. Standard deviations of peak day demand were characterized by using data gathered recently by the Utah State Division of Water Rights. Only a minority of Utah communities even record data on peak day flows. Eleven of these systems were used as a sample and means and standard deviations were calculated for the most recent 10 years of those data. The average $S/\bar{D}$ ratio was 0.1007. This suggests that the peak day demand during one out of 20 cases (years) would be approximately (10.07)(1.65) = 16.6%. However, an increase of 20 percent over expected value was selected as the recommended design criteria because of lack of knowledge about data reliability.

Comparison of demand functions and textbook multipliers

Peak day demand estimates are commonly expressed in the literature as a percent of average demand. The literature review included a summary of the ranges of such multipliers in current textbooks (Table 4). Most authors recommend a lower range of 150 to 180 percent of average while the upper range varies from 200 to 400 percent.

The per capita estimate of peak day demand suggested here for Utah systems (Figure 10) is the linear function:

$$D_{pd/p} = -49.4 + 2.5\, D_{avg/p}$$

This function produces a peak to average day ratio for the range of average demands encountered in Utah as follows:

Average Demand (gpcd)	Peak Day Demand (gpcd)	Peak/Avg
100	200	2.0
200	450	2.25
500	1200	2.4

The upper range recommended by five out of seven textbooks (Table 4) is lower than the factors suggested by peaks measured in Utah. This is another indication of the impact of outdoor demands in semiarid climates. It also demonstrates the danger of using textbook factors which may have been developed from the eastern U.S. experience (although not necessarily identified as such) to design water systems in more arid climates.

A discussion of peak hour to average demands will be given in a later section.

INSTANTANEOUS WATER DEMAND

Scope and Availability of Historic Data

Instantaneous and peak hour water demands are used to size distribution mains, and in-line booster pumps (Table 1). Fire flows govern pipe sizes in distribution systems only in high value districts or in residential areas where the pipe capacity is less than about 500 gpm (about 250 connections). Larger feeder line sizes are usually governed by residential demand.

Instantaneous or peak hour data are almost nonexistent except for that generated by a few research projects (see literature review). Most utilities have master meters on lines connecting water sources or treatment plants to finished water storage reservoirs. Such meters are necessary to monitor the performance of wells, springs, and treatment plants. But, having measured these inflows, very few utilities invest the rather substantial additional cost of master metering the much greater peak flows which occur below these equalizing reservoirs.

The reasons that local water system managers do not make these measurements on a routine basis are apparent if one examines the potential uses of these data. Continuous recorders are necessary for the data to be of value. (Daily and monthly readings of such meters would essentially duplicate the measurement of inflow to the reservoirs.) The continuously recorded data are not of particular value to the operator of an existing system (except for planning decisions which are well into the future) because decisions on main feeder line diameters have already been made. The real value of such data is in planning an expansion of a distribution system.

Another reason for the lack of incentive to collect such data is that a single measurement location such as immediately below an equalizing reservoir is not sufficient because the short term peaks per connection vary with the number of connections served and probably also with the economic level of the neighborhood being served. It would therefore be desirable to monitor peaks in various neighborhoods, but because of the looped nature of the pipe networks, and the possibility of reverse flows, etc., several recording meters would be required for even a single neighborhood. The cost of purchasing a continuous recording meter and installing it in a 6 to 10 inch line would be several thousand dollars. The use of nonrecording integrating meters would be of comparable expense because of the labor cost of meter readers who would have to work several hours each day during the peak demand periods.

Research Objectives

The overall objective of this portion of the research was to develop design criteria for the capacity of water supply components which are related to very short term peak flows. This objective was pursued by accomplishing the following subobjectives:

1. Gather instantaneous demand data from a sample of Utah systems covering a range of number of connections from 30 to 1000. All data were gathered from systems which have individual household water meters.

2. Analyze both the frequency of peak events and the duration of peaks above given levels in such a manner that statistical inferences can be made about the probability of future peak flows.

3. Examine the relationship between instantaneous peaks (1 to 5 minutes), peak hours, and peak days in order to: a) assess the validity of traditional factors being used to relate peak daily and peak hourly flows, and b) examine the validity of using peak hour rather than some short term peak for sizing distribution mains.

4. Develop recommendations for design criteria for capacity of water supply facilities which depend upon short term peak flows.

Data Collection Procedures

System selection

The problems associated with the gathering of instantaneous water demand data were discussed previously. Very few water systems have master meters and pipe network arrangements which allow gathering of instantaneous data. Criteria for identifying such water systems for use in this study were as follows:

1. The system must have a master meter (in working order) below any reservoir.

2. The only inflow to the distribution system is through the master meter. (Having more than one inflow would entail using a

second master meter and thus, would double meter reading costs.)

3. The systems should have no major leakage problems.

4. The system must have officials who agree to give researchers access to the master meters and to historic water use records.

5. The selected systems should cover the desired range of sizes (30-1000 connections).

During the spring of 1977, the research team met with representatives of both the Utah Division of Health and the Division of Water Resources Communities Loan Program. The meetings were called to 1) determine what instantaneous demand data if any were already available for Utah systems and 2) to determine which Utah systems would fit the selection criteria discussed above. Essentially, no instantaneous demand data were known to be available other than some spot checks of questionable accuracy. During the meetings a few potential water systems within Utah were identified.

The study team then visited each prospective community. Permission to collect the necessary demand data was obtained, and information about the characteristics of the population (areas of employment, degree of outdoor irrigation, number of multiple dwelling units, etc.) was gathered. Utility managers were questioned concerning water pricing policies, water use habits (weekend water use versus weekday water use, morning peaks or afternoon peaks or both, etc.), and any large water users (dairies, golf courses, etc.) which would affect the water demand. After the leakage level for each system was checked, the final system selection was made. The systems chosen were Brooklyn Tap, South Price, and Chesterfield. The three water systems selected met all of the selection criteria. It was impossible to find a suitable water system which served a high value urban residential area. The systems had a range of 84 to 790 connections but all were rural or semi-rural types of systems except Chesterfield, which is in an urban area but serves a relatively low socioeconomic area. The high value districts in Utah tend to be along the east bench of the Wasatch Front. These systems are tied into complicated distribution networks which either include reservoirs or would have required monitoring of several master meters to obtain the necessary data.

Determination of Peak Periods and Leakage

Near the end of July 1977, peak demand periods on each system were determined by observing master meter readings several times every hour for two 24 hour periods. The two day average daily demand hydrographs were drawn and peak demand periods were identified. These hydrographs are shown in Appendix A. During this preliminary data gathering period master meters were read between 1:00 a.m. and 4:00 a.m. to obtain an upper limit on the amount of leakage from all three systems. The Brooklyn Tap and the South Price systems showed no signs of leakage. The Chesterfield system would not reach a zero flowrate even during these late hours. The lowest flowrate recorded for the Chesterfield system was 80 gpm. This measurement was obtained at 2:30 a.m. on July 19, 1977. This flowrate could be considered an upper limit on the amount of water leaking from the system, and a correction could have been made to the peak demand data. The subtraction to be made from peak demand data would amount to 0.11 gpm/connection. This would lower the estimates of peak demand only slightly. Since there is no way of verifying that the entire 80 gpm was leakage this minor correction was not made. The Brooklyn Tap and the South Price systems showed no signs of leakage; therefore, no corrections to the raw data for leakage were necessary.

Data Gathering

Meter readers were hired and were asked to read and record the meter readings at short time intervals (1 to 5 minutes) each day during a specified peak demand period as determined by the two day average hydrographs in Appendix A. On each of the three systems this peak period occurred between 6:30 p.m. and 9:30 p.m. Communication with the water utility managers of each system determined that peak demands did not occur on the weekends in South Price or in Chesterfield. Therefore, meter readings were only recorded on weekdays for these two systems. The Brooklyn Tap system showed no difference in daily water demand between Saturday and any weekday, so data were recorded six days a week at that system. No readings were recorded on Sundays as water demand was well below weekday water demand rates on all three systems.

The data were recorded during part of July and all of August 1977. Meter readings were recorded at 1 minute intervals at Brooklyn Tap (84 connections), 2 minute intervals in South Price (124 connections), and at 5 minute intervals in Chesterfield (727 connections). It has been shown that as the size of a water system increases, the diversification in the water use habits of a large number of people tends to eliminate short term peaks in the demand (Linaweaver et al., 1966). Therefore, flow measurements at the intervals listed above were considered sufficient to show the peak instantaneous demands on each of the systems. This assumption is discussed in more detail in the section on peak flow durations.

During 1977, a personal problem of one of the meter readers resulted in a total lack of data from the South Price system. Also, during 1977, Utah was experiencing the worst one year drought in its recorded history. Because Chesterfield customers were asked to decrease their water consumption levels, the 1977 data were not considered to be a valid sample of peak flowrates. For these reasons the data gathering effort was repeated during the summer of 1978. The same three systems were used during the second summer and time intervals and peak daily demand periods remained the same. As much data as possible were recorded during July and August of 1978.

Description of the Systems Studied

Some Utah communities purchase their water from local water wholesalers. These wholesalers keep accurate flow measurements by means of a master meter at the point of delivery. Many of these communities have storage reservoirs of their own, thereby eliminating them from consideration for this research. The few systems with no reservoirs were candidates for this study. All three of the systems described here fall into this category.

Chesterfield

The Chesterfield Improvement District, the largest system studied, serves a suburb just west of Salt Lake City. The district purchases its water from the Salt Lake County Water Conservancy District. The community varied in size between 527 connections and 590 connections during the two year study period. Two hundred occupied mobile homes were located in two large trailer courts in the service area. The 200 mobile homes were added to the number of active connections for a total of 727 and 790 families served.

The Chesterfield outdoor water demand included both lawn and garden irrigation but the landscaped areas were relatively small. Most of the working population of Chesterfield drive to work somewhere in Salt Lake metropolitan area or they use the limited bus service available to them. The water system does supply water to approximately ten light industrial users and to several restaurants and small stores.

The 12 inch main distribution line servicing the Chesterfield system has two positive displacement meters in the meter box operating in parallel. During 1977, one of the two lines in the meter box was closed off and all of the flow was forced through one meter. This caused no problem with the water service and also facilitated the meter reading. In 1978, when there were no water use restrictions in effect, more water was being used and it was necessary to keep both meters operating in order to handle the peak demands.

South Price

South Price water system serves semi-rural type residences in central Utah. It purchases its water from the City of Price. Unlike the Chesterfield system, the South Price area has supplemental ditch water for irrigation available to nearly 100 percent of its residences. There is almost no commercial or industrial use of water within the system. The homes in the area vary in age with many being relatively new ones, built before a current moratorium on building went into effect. There were 124 individual connections at the time of the study. Most of the work force in South Price is employed in coal mining operations in the area.

The flow of water entering the South Price system passed through a dual positive displacement meter. When demand was low (0 to 25 gpm) the low flow meter recorded flow volumes. When the flowrate reached a certain level (approximately 25 gpm) the low flow meter shut off and the flow volumes were recorded on the high flow meter. Because the meter readers only recorded data during peak demand periods the low flow meter was seldom operating.

Brooklyn Tap

The Brooklyn Tap Water Users Association buys water from the town of Elsinore which is located in south central Utah. The system serves 35 connections, one of which is a trailer court supplying 50 occupied units. The 50 occupied trailers plus 34 residential connections represent a total of 84 connections.

As is the case with Chesterfield, the Brooklyn Tap users do not have access to secondary irrigation water. The average area irrigated is small, and the climate is hot and dry. The system does supply some stock and wash water for one dairy. Water is also supplied to a cement and gravel operation in the area. There is no other commercial or industrial water demand on the system. Brooklyn Tap has a positive displacement integrating type meter. This single meter measures both high and low flowrates.

Frequency Analysis of Data

Theoretical distribution of the data

A problem which is frequently confronted in hydrologic studies is using empirical data to make inferences concerning the probability of future events. In this study, frequency analysis is used to define the instantaneous demand flowrate (not a hydrologic parameter) with a probability, P, of being equaled or exceeded on any hot, summer day. This probability can also be expressed in terms of a return period, Tr (measured in days). Return period and probability are reciprocals.

In a previous study by Hughes (1977), peak daily flow data (gpm/connection) during hot, summer days were shown to be normally distributed. The small skew coefficients found for the five sets of data obtained during this study (Brooklyn Tap, 1977 and 1978; Chesterfield, 1977 and 1978; and South

Price, 1978) suggest that these daily peaks are also normally distributed.

The cumulative probability of normally distributed data may be represented graphically on normal-probability paper. For this study the ordinate represents the maximum daily instantaneous demand and the abscissa represents the probability, P, or the recurrence interval, Tr. The ordinate and abscissa scales are so designed that normally distributed data plot as a straight line. The reason for using the normal probability paper was to linearize the distribution so that plotted data can be extrapolated more accurately (Chow, 1964).

To plot data on normal-probability paper, a plotting position must be used. A study comparing several proposed plotting position formulas (Benson, 1962) revealed that on the basis of theoretical sampling from normal distributions, the Weibull formula provides the most consistent estimates:

$$t_r = \frac{N + 1}{m}$$

where:

N is the number of days of data

m is the order number of the daily peak demands arranged in descending magnitude (m = 1 for the largest peak demand)

The probability distribution of the five sets of daily peak data are shown in Figure 14 and Figure 15. The lines through the data were located by using the method of moments (Chow, 1964). By this method, the mean and the standard deviation of the data sets (see Table 9) are computed and used as estimates of the true population parameters in the normal probability function:

$$X = \overline{X} + SK$$

where:

X = the estimation of peak daily demand

$\overline{X}$ = the mean of the sample

S = the standard deviation of the sample

K = the number of standard deviations from the mean for a normal distribution. This value is available from standard normal tables for various probabilities.

The values computed for X at different probabilities plot as a straight line through the data on normal probability paper. This method of curve fitting requires the assumption that the estimates of means and standard deviation from the sample data equal the true means and standard deviations for these populations. This assumption introduces little error when the number of data points is large. But when the number of data points is small, the t-distribution may be a desirable alternative to the normal distribution (Ott, 1977). The t-distribution assumes a normal distribution but allows variability in the point estimates of the mean and standard deviation of the sample. It does this by considering variability as a function of degrees of freedom of the data. As the number of data points increases the t-distribution approaches a normal distribution. By allowing the randomness in the mean and standard deviations, peak flows predicted by the t-distribution are more conservative (are higher). In order to check the validity of the frequency analysis using the method of moments, t-distribution frequency analysis was also calculated.

The five data sets used in this study had between 12 and 29 days of peak demand data and therefore, 11 to 28 degrees of freedom. The peak demand at a given proba-

Table 9. Statistical parameters for daily maximum instantaneous flows in gallons per minute (gpm) per service connection.

Parameters	Water System				
	Brooklyn Tap		South Price	Chesterfield	
	1977	1978	1978	1977	1978
Number of Connections	84	84	124	727	790
Number of Days (N)	17	29	20	12	16
Mean Daily Maximum ($\overline{X}$)	1.10	1.30	0.51	1.17	1.30
Standard Deviation (S)	0.220	0.168	0.056	0.129	0.245
Skew Coefficient (g)	0.227	0.019	-0.225	-0.052	0.231
Maximum Measured Flow	1.48	1.67	0.60	1.39	1.70

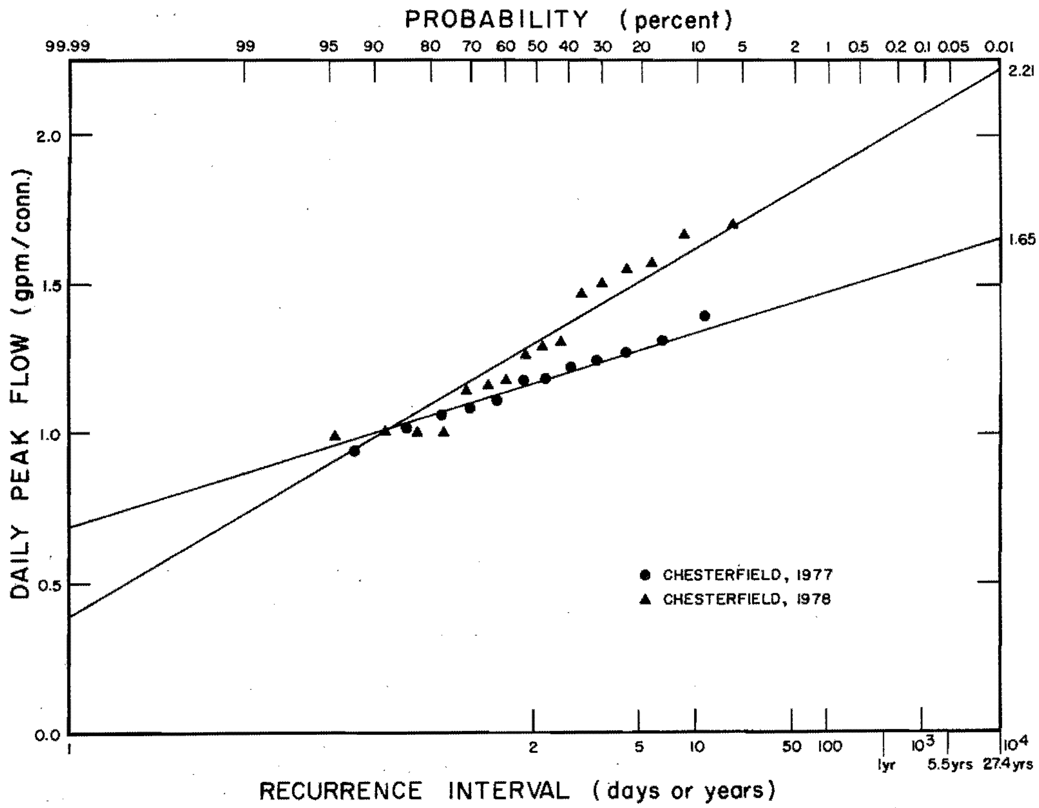

Figure 14. Frequency distribution of daily peak events for Chesterfield, Utah.

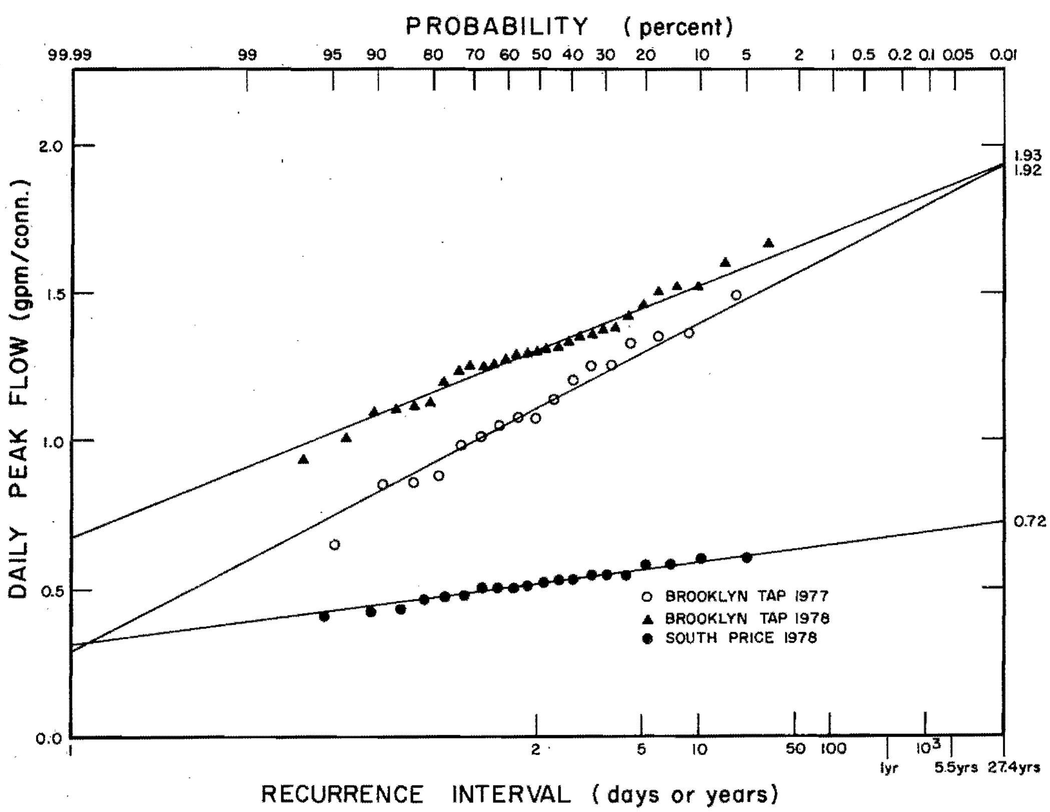

Figure 15. Frequency distribution of daily peak events for Brooklyn Tap and South Price, Utah.

bility of occurring on a hot summer day using the student t-distribution is (Kempthorne and Folks, 1971):

$$X = t_{(1-\alpha,\ N-1)}\ S\sqrt{\frac{N+1}{N}} + \overline{X}$$

where:

$\alpha = P = \frac{1}{T_r}$

t = students t

N = number of data points in sample

S = standard deviation of sample

$\overline{X}$ = mean of sample

X = expected peak flow

Values for expected peak flows for various probabilities for both the normal and the t methods are summarized in Table 10. Both methods produce similar results from a one day recurrence interval (P = 1) through about a three year recurrence interval (P = 0.0009). As the recurrence interval gets larger the t method estimates higher values of peak demand than does the normal method at the same probability. When there are more data and when the standard deviation of those data is small, the t method and the normal distribution give nearly the same etimates of peak demand all the way through a 27 year recurrence interval. The Brooklyn Tap data from 1978 are a good example of this (see Table 10). The Chesterfield data created the largest differences between the two methods. The Chesterfield 1977 data included only 12 days of peak demand. With so few degrees of freedom, the t-distribution deviates from a normal distribution much more than it does for say, the Brooklyn Tap 1978 data, where it had 28 degrees of freedom. In 1978, the Chesterfield data had a large standard deviation (0.245 gpm/connection). This standard deviation causes the t-distribution to deviate from normal more than, for example, the South Price data which actually has more data days and thus more degrees of freedom. A logical conclusion seems to be that instantaneous demand data are normally distributed and that deviation from the best fit straight line is significant only where the number of data was relatively small. Therefore the expected value of peak flows used throughout the remainder of the report will be those predicted by the method of moments line.

The number of usable data days are shown in Table 9 for each of the five data sets. The meter readers actually recorded more days of peak flow data. After all of the raw data were collected, weather records were checked for temperature and precipitation which may have occurred in the area of each system. When there were cooler temperatures (temperature < 85°F) and/or significant precipitation (precipitation $\geq$ 0.01 inch) water demand dropped off from the norm. These days were excluded as being not representative of peak demands.

Defining the Recurrence Interval

In Figure 14 and Figure 15 the probabilities of various peak flowrates being experienced on any hot, summer day are given. This prediction was based on a normal distribution of peak daily flows. Since peak daily flows used to make the prediction were recorded only on hot, summer days the recurrence interval for infrequent events (periods longer than one summer) requires some modifications. One might expect that if the peak instantaneous demands on a given system were known for each day of the year that a more reliable prediction of peak flows could be attained. However, this is not the case. Peak water use rates are correlated with time

Table 10. Comparison of unit demands (gpm/conn.) from t distribution and from linearized normal distribution.

Recurrence Interval	Parameter	Brooklyn Tap		Chesterfield		S. Price
		1977	1978	1977	1978	1978
P = 0.25	Q(t)	1.26	1.42	1.26	1.47	0.55
Tr = 4 Days[a]	Q(normal)	1.25	1.42	1.25	1.47	0.55
P = 0.01	Q(t)	1.69	1.73	1.53	1.96	0.66
Tr = 100 Days	Q(normal)	1.61	1.70	1.47	1.87	0.64
P = 0.0009	Q(t)	1.83	1.83	1.65	2.26	0.72
Tr = 3 Years	Q(normal)	1.79	1.83	1.57	2.06	0.69
P = 0.0005	Q(t)	2.01	1.93	1.76	2.33	0.74
Tr = 5.5 Years	Q(normal)	1.83	1.86	1.59	2.11	0.70
P = 0.0001	Q(t)	2.19	2.04	1.91	2.54	0.78
Tr = 27.4 Years	Q(normal)	1.92	1.93	1.65	2.21	0.72

[a]The recurrence intervals listed in this table equal the reciprocal of the predicted probabilities and refer to hot, midweek, summer days. Actual recurrence intervals are four times the amount shown here.

of the year (with climate), especially in the arid western states. In an arid climate, where outdoor water use is a significant component of demand, a large variation in demand occurs between the summer days and the winter days. This increased variation would show up as an increased standard deviation of the data. The equations used previously for estimating flowrates using the t or the normal distribution show that an increase in the standard deviation of the data will result in higher estimates of peak demand at the longer recurrence intervals. This is due to an increase in the slope of the line through the data as the variability of the data increases. Since the objective of this study was to determine peak demands, which occur only during the hot, summer months when outdoor use is at a maximum, the data base was limited to such days. This data base, restricted to the hot days in the summer when the temperature exceeded 85°F and there was no measurable precipitation, should give the proper peaks and the proper variability. A question arises, however, as to the meaning of the recurrence interval. Normally, the recurrence interval is considered to be equal to the reciprocal of the probability (Tr = 1/P). However, long term recurrence intervals should include all of the off peak days as well as the hot, summer days.

Changing the normal recurrence levels (Tr = 1/P) to the actual recurrence intervals required the determination of the average number of days in a year when there was no measurable precipitation and the temperature was greater than 85°F. To determine this average, records from the weather station nearest each water system studied were obtained. The total precipitation from the months of June, July, and August were calculated for each of the ten years from 1968 through 1977. The wettest and the driest three month summer period (June, July, and August) during those 10 years at each station determined the two years used in calculating the average number of days meeting the criteria assumed above. The calculations showed that on the average there were 52 days each year when peak flow could be expected to occur on the Brooklyn Tap water system. There were 63.5 and 72.5 possible peak demand days expected during an average year for the Chesterfield and the South Price systems, respectively.

When the number of possible peak demand days was counted for each water system no accounting for off peak days (Saturday and Sunday) was made. Since it had been determined that the South Price and the Chesterfield systems did not experience peak demands on the weekends. The average number of possible peak demand days was adjusted to 45.4 days for the Chesterfield system (63.5 days x 5/7), and 51.8 days for South Price (72.5 days x 5/7). Because the Brooklyn Tap system experienced peak demands on Saturday but not on Sunday, the average number of peak demand days was adjusted to 44.6 days (52 days x 6/7).

Factors were applied to the original recurrence intervals (Tr = 1/P) to get the actual recurrence intervals. These factors were determined by dividing the number of days in the year (365.25) by the average number of possible peak demand days determined for each system. The factors were 7.05 for South Price (365.25/51.8), 8.05 for Chesterfield (365.25/45.4), and 8.19 for Brooklyn Tap (365.25/44.6). Multiplying the recurrence intervals in Figures 14 and 15 by these factors gives the correct recurrence interval for each system.

A comparison between peak instantaneous demand to be expected at the original recurrence intervals (Tr = 1/P) and at the revised recurrence intervals is shown in Table 11. As the recurrence interval increases, the difference between the two decreases (less than 0.3 gpm/conn at Tr = 27.4 years). The revised recurrence intervals as shown in Figures 16, 17, and 18 are used for subsequent comparisons with the results of other studies.

Impact of the Drought

The effects of the drought on both the Chesterfield and the Brooklyn Tap water systems can be seen by comparing the predicted demand values for 1977 and 1978 in Table 10. In 1977, the drought year, the Chesterfield community was required to comply with temporary water restrictions. The customers were only allowed to water their lawns and gardens after 8:00 p.m. Also, there were severe financial penalties for excessive monthly water use ($10/kgal for any usage over a monthly allotment). The resulting large reduction in monthly water use also clearly reduced instantaneous peaks. In 1978, a very wet year (50 percent more precipitation than an average year) no conservation efforts were made. The difference between the peak demands experienced in 1977 and those which occurred in 1978 is shown in Table 10.

Table 11. A comparison of the predicted flowrates (gpm/conn.) at actual recurrence intervals using the original Brooklyn Tap, 1977 data and using a modified data set.

	Predicted Flowrate at Specified Recurrence Intervals				
Original Tr	50 Days	100 Days	1 Year	5 Years	27.4 Years
Original	1.55	1.61	1.72	1.83	1.92
¼ Original Tr	1.41	1.49	1.61	1.73	1.84
Modified Tr (N x 4)	1.42	1.50	1.62	1.81	1.95

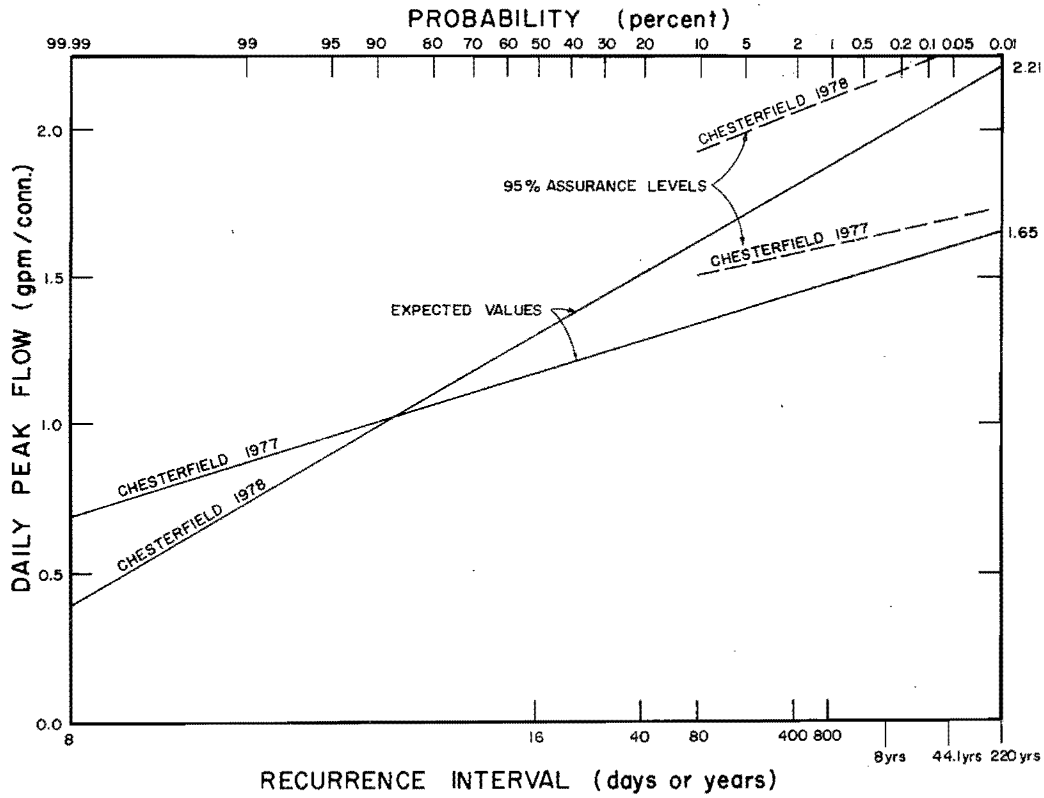

Figure 16. Comparison of daily peak expected values and 95 percent assurance levels for Chesterfield.

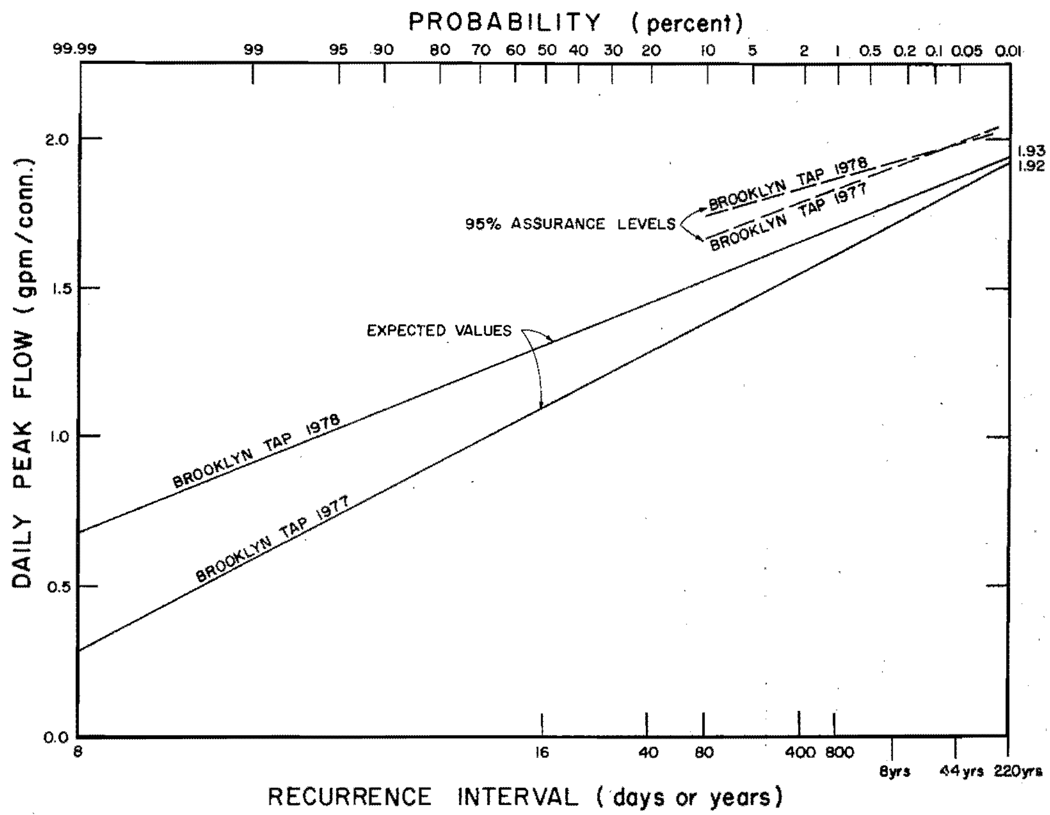

Figure 17. Comparison of daily peak expected values and 95 percent assurance levels for Brooklyn Tap.

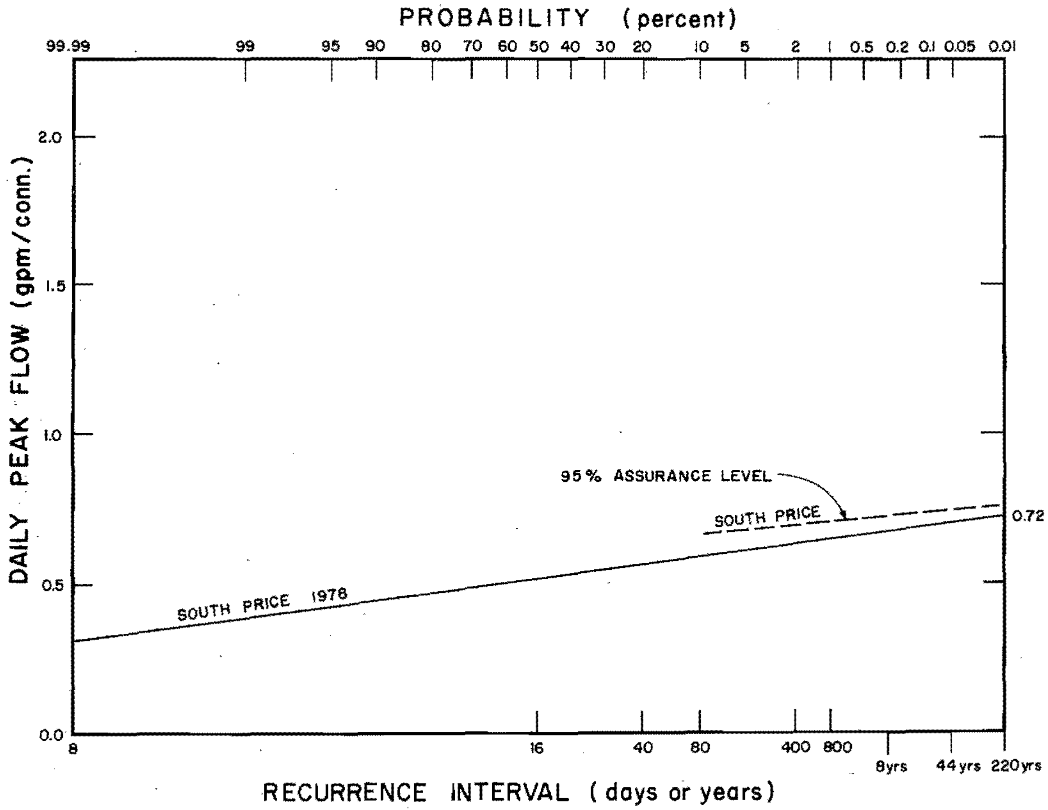

Figure 18. Comparison of daily peak expected values and 95 percent assurance levels for South Price.

The demands during 1977 and 1978 for the Brooklyn Tap water system do not show significant differences because water usage restriction were not imposed upon Brooklyn Tap customers during the drought.

The very dramatic range in weather experienced during 1977 and 1978 proved to be beneficial to this study. The data obtained during the two summers should be representative of essentially the maximum climatological range and therefore the maximum water demand range for these systems.

Risk of Exceedance Analysis

The estimations made in Figures 15 and 16 define the average or expected values of peak flows for hot, summer days at any time probability level. However, in order to define the probability of any level of peak flow occurring during a particular time period, further statistical analysis is necessary. A method used to estimate the probability that a hydrologic event with average probability P will be exceeded exactly K times during a certain time period (Linsley et al., 1975) can be modified for use in this analysis. The equation given is:

$$J = 1 - \left(\frac{N}{K}\right) (1 - P)^{N-K} P^K$$

where

- N = number of days in the specified time interval
- K = number of times the expected value will be exceeded
- P = the average probability of occurrence
- J = the probability that an event with probability P will be exceeded

When risk levels are to be placed on the largest event or, in this case the highest peak demand to occur during a certain time interval, then K = 0 and the equation becomes:

$$J = 1 - (1 - P)^N$$

Figures 16, 17, and 18 show the 95 percent assurance levels drawn above the expected values of peak instantaneous demand. Table 12 gives values of the most probable or the expected peak instantaneous demand and the 95 percent assurance levels for several time intervals. A sample calculation for an interval of 100 days is shown in Appendix C.

Referring to Figure 17 or to Table 12, the Brooklyn Tap system would have a 1

Table 12. Ninety-five percent confidence exceedance levels (gpm/conn.) for unit demand (J = 0.05).

Time Period (N)	Probability (P)	Parameter	Chesterfield		Brooklyn Tap		S. Price
			1977	1978	1977	1978	1978
40 Days	5.12×10^{-3}	95% Limit	1.50	1.92	1.66	1.74	0.65
		Expected	1.33	1.61	1.38	1.52	0.58
200 Days	1.03×10^{-3}	95% Limit	1.56	2.05	1.79	1.83	0.68
		Expected	1.43	1.80	1.55	1.65	0.63
400 Days	5.13×10^{-4}	95% Limit	1.60	2.10	1.83	1.86	0.70
		Expected	1.47	1.87	1.62	1.70	0.64
4 Years	1.40×10^{-4}	95% Limit	1.64	2.18	1.91	1.92	0.72
		Expected	1.53	1.97	1.72	1.78	0.67
12 Years	4.68×10^{-5}	95% Limit	1.67	2.23	1.96	1.95	0.73
		Expected	1.57	2.06	1.79	1.83	0.68
20 Years	2.81×10^{-5}	95% Limit	1.69	2.26	1.98	1.97	0.74
		Expected	1.59	2.10	1.82	1.86	0.69

percent chance (N = 100 = number of hot, summer days) of experiencing a peak flowrate of 1.62 gpm/conn or greater on any given hot, summer weekday. During a 700 day period there is only a 5 percent chance that the peak demand will exceed 1.83 gpm/conn.

Comparison of Instantaneous, Hourly, and Daily Peaks

Peak flow duration analysis

The peak demands estimated from Figure 16, 17, or 18 for a given recurrence interval are based on average flowrates during the interval between meter readings. The data used in this study were taken from master meter readings in intervals of 1, 2, or 5 minutes depending upon the size of the system. The Brooklyn Tap system with 84 connections was read every minute. The data for the South Price and the Chesterfield systems were recorded every 2 and every 5 minutes, respectively. Thus, the recorded demands are actually the average flowrates experienced for those few minutes.

For example, the Chesterfield data for 1978 predicts the peak average demand of 2.06 gpm/connection for a duration of 5 minutes at a recurrence interval of 24 years (P = 8.05/24 X 365). During this 5 minute period, the absolute maximum demand will be something greater than the 2.06 gpm/connection 5 minute average demand.

Peak flowrates for various durations are plotted in Figure 19. The three days with the three highest recorded peak flows for each of the five data sets were used in the calculations for the expected durations. The demand patterns for each of the 15 days are plotted in Appendix D. Values for the average peak demands lasting a given duration are calculated from the hydrographs. Some of these values are listed in Table 13.

Figure 19 shows that the absolute maximum instantaneous demand is unknown. If it were known it would be plotted along the ordinate where the time duration equals zero. In the previous study in Utah (Hughes, 1977), a similar approach to estimating peak flow duration was used. In that report, the data were taken on a continuous recorder. This allowed identification of the absolute peak instantaneous flow. In each case, the demand dropped quickly from the absolute maximum to a lesser demand lasting a duration of 3 or 4 minutes. These duration curves are also plotted in Figure 19. All of the duration curves plotted in Figure 19 tended to level off at a fairly constant flowrate as the time duration increased. The curves

Table 13. Average flowrates (gpm/conn.) lasting various durations on the three days with the highest peak instantaneous demand.

System Name	Number of Connections	Time Duration (Minutes)			
		3	5	15	30
Lapoint	4	3.18	3.10	2.71	2.60
Lapoint	12	2.04	1.98	1.80	1.68
Lapoint	22	1.80	1.71	1.59	1.50
Brooklyn Tap (1977)	84	1.36	1.33	1.29	1.18
Brooklyn Tap (1978)	84	1.51	1.50	1.41	1.38
Chesterfield (1977)	727	-	1.38	1.02	0.96
Chesterfield (1978)	790	-	1.64	1.18	1.05
South Price (1978)	124	0.55	0.49	0.41	0.37

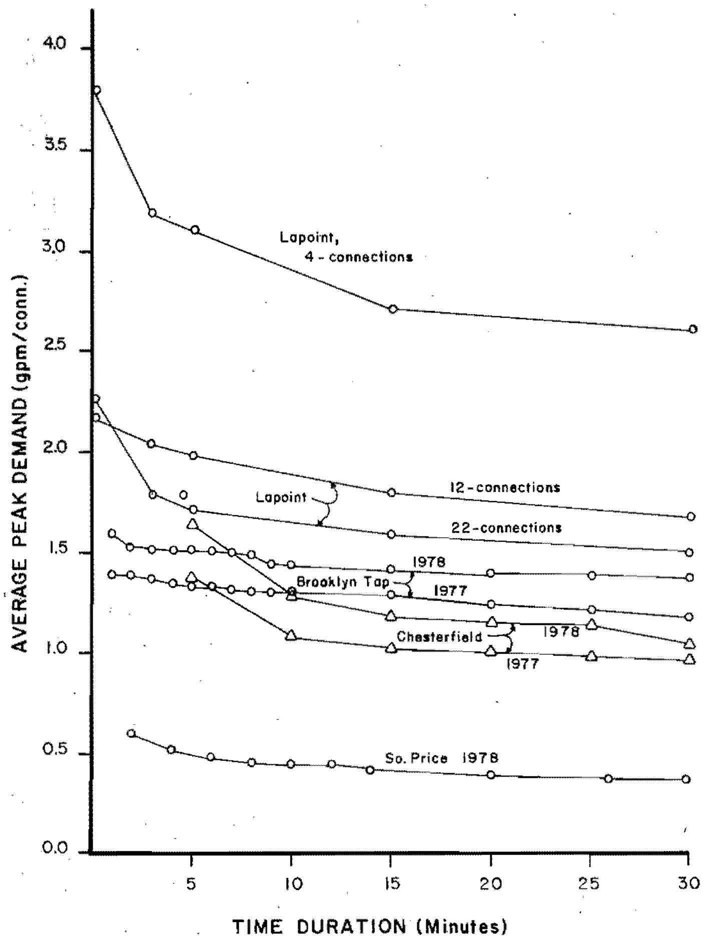

Figure 19. Average peak demand durations.

plotted from the data used in this study do not include the more steeply sloped part of the curve from a time duration equal to zero to a duration of 1, 2, or 5 minutes. However, the number of connections for each of the systems studied in this report were all much larger than the number of connections used in the earlier report (Hughes, 1977). The time duration curves for these new data should be much flatter in the region between 0 and 5 minutes duration than the curves plotted from the Lapoint data. The Lapoint data, taken on water lines serving only 4, 12, or 22 connections, fluctuated much more rapidly than did the data taken from these three systems.

Differences in Peak Instantaneous, Peak Hourly, and Peak Daily Demand

Engineers normally design distribution networks in urban areas to meet peak hourly water demands (Steel, 1960). In an urban area the diversification of water use habits among the many customers tends to reduce the fluctuations in the daily demand hydrograph (Linaweaver et al., 1966). In a more rural area where fewer customers are served by the water system (perhaps < 1000 connections), one would expect to see greater and more frequent variations in the daily demand hydrograph. The instantaneous demand data from Utah systems which have been produced by this study and by Hughes (1977) have confirmed this expectation.

The Lapoint data (4 to 22 connections), which were analyzed in the previous study, showed the greatest fluctuations in the demand. This was expected since the systems were the smallest ones studied (≤ 22 connections). However, the data from Chesterfield used in this study showed large fluctuations in the demand from one 5 minute interval to the next. This amount of deviation (see Appendix D) was not expected since Chesterfield was considered an urban area with between 700 and 800 service connections.

If the demand on a rural water system (less than 1000 connections) fluctuates so rapidly then, perhaps design based on peak hourly flow is not sufficient. If the instantaneous demand exceeds the design capacity of the system (normally peak hourly demand) for more than a few minutes, low or theoretically even negative line pressures may result. Low pressures would cause reduced service to the customers. Negative line pressures may cause contamination of the water supply.

The peak flow duration analysis in the previous section showed that there was very little difference in the highest 30 minute water demand and say the highest 10 minute water demand. Figure 19 shows that only very short duration demands (0 to 5 minutes) were significantly higher than the maximum 30 minute or the peak hour demand. In order to test whether a shorter duration gave a significantly higher demand than the peak hourly demand used in design an analysis of variance was performed.

Many of the equations used by engineers to estimate peak hourly flow are simply some multiple of the peak daily flowrate. Because another objective of this project was to compare actual peak day and peak hour demand to demands estimated by design equations, an analysis of variance was also carried out to determine if peak hourly demands were significantly different from peak daily demands.

In order to get peak day demands from the five data sets it was necessary to have recorded meter readings for successive days. Then the meter readings from the previous day could be subtracted from the next day's reading and recorded 24 hours later to give a typical peak daily demand. Including periods when data were not recorded (days when it rained or Sundays) and then averaging over the period to get several peak days was not done because it had already been decided that those were not peak days and thus, those days should be excluded. With this restriction, the number of days of actual peak daily data which could be extracted from the data varied from 5 days from the Chesterfield 1977 data to 17 days from the Brooklyn Tap 1978 data. The data from South Price were not included in this analysis because the South Price system experienced peaks so low that South Price was not representative of other systems. Reasons for the low peak values will be discussed in the section comparing the results of this study to other research.

The raw data from the days when peak daily demand was available were studied to determine the maximum hourly flowrate, the maximum 10 minute flowrate, and the maximum 5 minute flowrate. An analysis of variance was then performed on each of these four data sets (Brooklyn Tap, 1977 and 1978; Chesterfield, 1977 and 1978) to determine if there were any significant differences between: 1) the peak daily flowrate and the peak hourly flowrate, 2) the maximum 10 minute flowrate and the peak hourly flowrate, and 3) the maximum 5 minute flowrate and the peak hourly flowrate. Where the data permitted (Brooklyn Tap) the comparison was also done between the 1 minute peak flowrate and the peak hourly flowrate.

When samples (data) are selected from different populations (peak daily, peak hourly, peak 10 minute, and peak 5 minute flowrates) which may have different variances ($\sigma_1^2 \neq \sigma_2^2 \neq \sigma_3^2$) then the t-distribution (used for comparing means when there are few data points) is no longer valid. However, Cochran (1964) showed that when the sample size from each of the populations is equal ($n_1 = n_2 = n_3$) then the t-distribution is approximated by:

$$t' = \frac{\bar{y}_1 - \bar{y}_2}{\sqrt{S_2^1/n_1 + S_2^2/n_2}} \qquad n_1 = n_2$$

where:

- t' = the approximated test statistic
- $\bar{y}_1$ = the mean of the sample from population 1
- $\bar{y}_2$ = the mean of the sample from population 2
- S_1 = the standard deviation of the sample from 1
- S_2 = the standard deviation of the sample from 2
- n_1 = the size of sample 1
- n_2 = the size of sample 2

The rejection region for t' is found in standard student-t tables for degrees of freedom, df = n-1 (Ott, 1977).

In all of the analyses the hypothesis to be checked was whether the mean of one population was equal to the mean of the other population. The means and standard deviations for each of the data sets in each of the categories (peak day, peak hour, etc.) were calculated. When comparing two of the categories for significant differences the t' statistic was calculated. When this calculated statistic exceeded the standard students'-t with a given number of degrees of freedom at a certain significance level then, the hypothesis of the two populations having equal means was rejected and a significant difference in the means was statistically shown.

Calculated t' values are shown in Table 14 and Table 15. These tables summarize the comparisons between maximum 10 minute and peak hourly flowrates and maximum five minute and peak hourly flowrates, respectively. Table 16 shows the comparison of maximum 1 minute and peak hourly demand for the Brooklyn Tap data. When there is a significant difference between the means of the two populations being compared the t' statistic exceeded the appropriate student's t. These

Table 14. t' statistics for peak hourly versus 10 minute maximum flowrate.

System Name	Year	n(days)	Significance Level $\alpha=0.1$	Significance Level $\alpha=0.05$
Chesterfield	1977	5	1.44	1.44
Chesterfield	1978	11	2.36	2.36
Brooklyn Tap	1977	9	1.20	1.20
Brooklyn Tap	1978	17	1.55	1.55

Table 15. t' statistics for peak hourly versus 5 minute maximum flowrate.

System Name	Year	n(days)	Significance Level $\alpha=0.1$	Significance Level $\alpha=0.05$
Chesterfield	1977	5	3.00	3.00
Chesterfield	1978	11	3.16	3.16
Brooklyn Tap	1977	9	1.60	1.60
Brooklyn Tap	1978	17	2.03	2.03

Table 16. t' statistics for peak hourly versus 1 minute maximum flowrate.

System Name	Year	n(days)	Significance Level $\alpha=0.1$	Significance Level $\alpha=0.05$
Brooklyn Tap	1977	9	2.31	2.31
Brooklyn Tap	1978	17	3.27	3.27

values are circled. For each data set there was a significant difference between peak daily and peak hourly flowrates even at the 0.005 level of significance. Since this difference was seen for each data set at every level of significance, these t' values were not tabularized.

The analysis of variance showed that although there is very little difference between the 1 hour maximum and the 10 minute maximum flowrate there is a significant difference between the 5 minute peak demand and the maximum hourly flowrate. Also, where 1 minute demand data were available, the difference between the 1 minute demand flowrate and the 1 hour demand was significant at even the lowest (0.05) significance level tested.

The conclusion to be made from the results of the analyses of variance is that design of rural distribution systems (less than 1000 connections) based on peak hourly demands is not sufficient. If the elimination of low or negative pressures is a primary objective of the design then design based on the higher value of instantaneous demand (durations of 5 minutes for example), should be used.

Comparison of Daily and Hourly Peaks to Textbook Recommendations

Factors recommended by several textbooks for estimating peak daily and peak hourly water demand were given in the literature review. The actual values for peak daily water use from the four data sets are listed in Table 17. The estimations of peak daily demand as calculated from the eight water supply textbooks reviewed previously are also listed in Table 17 for comparison.

Table 17 shows that the 1977 drought year peak daily average flows were much lower than the 1978 flows (176 gpd to 294 gpd). Chesterfield's average peak daily demand for 1977 was below all of the textbook estimates. The highest peak day demand recorded that year was lower than all of the estimates except one. However, in 1978, the average value of peak daily demand for the Chesterfield data was within the range of all of the estimations. The highest value recorded (355 gal/pers/day) was higher than or on the high end of the estimated ranges. With the relatively inexpensive water ($0.30/kgal) and the hot, dry climate one would expect this.

The Brooklyn Tap data also show the difference in water consumption between the drought year of 1977 and the wet year (approximately 50 percent above average) in 1978. The average peak daily consumption was within the range of estimates in 1977 but not in 1978. The highest peak day recorded in 1978 was well above all but one of the estimations.

Table 18 lists the estimated peak hourly demands as calculated by the eight water supply textbooks. The actual values are listed for comparison. Since it was shown in the preceding section that the peak 5 minute demand was significantly higher than the peak hourly demand, these values are also listed in Table 18.

Table 18 shows that the Chesterfield peak hourly demand was lower than the range of estimates in 1977 and within the estimates in 1978. The Brooklyn Tap data were on the high side of the estimates during 1977 and well above all but one of the estimates in 1978. Only one of the estimates of peak hourly demand goes high enough to include the 5 minute peak demands.

In defense of the textbook recommendations it can be said that they were reporting typical ranges of peak daily and peak hourly demands as compared to average daily demands. However, in only two of the texts (Clark and Viessman, 1966; Hardenberg and Rodie, 1970) do the authors remind the readers that these ranges are for typical, residential communities and that very large deviations from

Table 17. A comparison of actual peak daily demands to available estimations of peak daily demand.

Authors	Predicted Peak Daily Demand (gal/pers/day)		Actual Peak Daily Demand (gal/pers/day)			
			Chesterfield (Ave = 150 gpd)		Brooklyn Tap (Ave = 125 gpd)	
	Chesterfield	Brooklyn Tap	1977	1978	1977	1978
Babbitt and Doland	225-375	188-313				
Linsley and Franzini	270	225	Ave = 176	Ave = 294	Ave = 241	Ave = 323
Steel	270-300	225-250	High = 192	High = 355	High = 332	High = 399
Walker	-	-				
Twort and Hoather and Law	225-300	188-250				
Clark and Viessman	180-600	150-500				
Fair and Geyer	225	188				
Hardenbergh and Rodie	225	188				

Table 18. A comparison of actual peak hourly and peak 5 minute demands to available estimations of peak hourly demands.

Authors	Predicted Peak Hourly Demand (gal/pers/day)		Actual Daily and 5 Minute Peak Demand (gal/pers/day)			
			Chesterfield		Brooklyn Tap	
	Chesterfield	Brooklyn Tap	1977	1978	1977	1978
Babbitt and Doland	450-600	375-500	Peak Hourly	Peak Hourly	Peak Hourly	Peak Hourly
			Ave = 385	Ave = 441	Ave = 473	Ave = 524
Linsley and Franzini	540	450	High = 434	High = 550	High = 568	High = 671
Steel	405-450	338-375				
Walker	468	390				
Twort, Hoather and Law	<600	<500	Peak 5 Minute		Peak 5 Minute	
			Ave = 479	Ave = 559	Ave = 517	Ave = 568
Clark and Viessman	225-1800	188-1500	High = 541	High = 740	High = 625	High = 720
Fair and Geyer	375	313				
Hardenbergh and Rodie	375-450	313-375				

average demand (7 to 12 times the average) have been recorded. The communities showing the most deviation have been the high economic areas with arid climates and thus, with a large amount of outdoor sprinkling.

In using any of the equations, it must be remembered that the equations were developed from typical residential communities. Water demand is greatly affected by the factors listed in the literature review (climate, leakage, etc.) and thus, experience and knowledge of the characteristics of the population are essential in estimating peak demands of any duration.

Discussion of Results

Peak demands for mobile homes compared to demands for residences

A similarity in peak water demands between mobile homes and other residences was documented by the Brooklyn Tap data. This finding is contrary to the expectation that mobile homes would use less water. One reason for this expectation is that trailers generally have smaller yard areas to irrigate. Secondly, some trailers may have smaller indoor appliances such as washing machines. A third reason is that trailers probably average less occupants per living unit than do households. An important factor in the opposite direction was that although the mobile home parks were master metered, individual trailers were not and therefore individual water users had no reason to conserve water. Unmetered demand is significantly larger than metered (Walker, 1978) and this one factor seemed important enough to increase the instantaneous peak demand of mobile homes to the level of an average residence.

Of the 84 connections within the Brooklyn Tap water system, 50 of these were mobile homes. Water going to the trailer court was metered but individual trailers were not. The mobile home owners pay a flat fee for their water. Originally, it was thought that the water demand from the trailer court would not be representative of the Brooklyn Tap system and so it should be excluded. In order to do this, the meter reader recorded meter readings from the master meter at 1 minute intervals for approximately 30 minutes. He then traveled to the trailer court meter and recorded flows. After 3 or 4 minutes he returned to the master meter where he recorded meter readings for another half-hour. This routine was repeated several times during the 2 to 3 hour daily peak period. This allowed separation of the trailer court demand from the residential portion of the demand. When the data were analyzed separately, using 84 connections and the master meter readings or using 50 connections and the trailer court meter readings, no significant difference was found in the peak instantaneous demand on a per connection basis. The average daily peak demand was found to be 0.97 gpm/trailer and 1.01 gpm/residence. There appeared to be no significant difference between the mobile home and other residential demands.

Comparison of Results with Previous Research

A comparison of the results of this study with those reported in other literature is summarized in Figure 20. The points labeled Utah are the most probable peaks for a 27 year return interval. These values are from this study and from the previous work done by Hughes (1977). Other labeled points are for the maximum recorded event from the following studies: Kansas--Allen and Montgomery counties (Williams, undated); Kansas (Johnson, 1978); Oklahoma #3 (Goodwin, 1973). The function labeled Ginn was produced from the aggregation of individual demand distributions in urban Mississippi into a statistical model.

Included in Figure 20 are the Farmers' Home Administration Ohio minimum and average standards (FmHA, 1976). The FmHA reports that over 5,000 water systems throughout the midwest and eastern U.S. have been designed according to their minimum standard curve. Using the FmHA minimum standard for design in a humid area where the outdoor water demand is small may be adequate. In a more arid climate this minimum design curve is clearly not acceptable. All of the systems studied except for the South Price system and the Kansas systems recorded demands above the FmHA Ohio minimum but below the average standard. The South Price system and the systems studied in Kansas are all well below the FmHA minimum curve. The Kansas studies do not offer any explanation for the low values reported. The price of water in Kansas at the time the studies were conducted was approximately one dollar per thousand gallons of water. This was a high price but not unreasonably so compared to other systems studied. The South Price system had an average price of $1.62/kgal. This is relatively expensive water but, as has been shown the price of water is an important determinant of monthly demand but not daily demand and should be even less important in determining instantaneous demand. South Price also has supplemental ditch water supplied to nearly 100 percent of its service connections. With the availability of supplemental water, one would expect lower water demands upon the culinary system.

Another factor which influences peak instantaneous demands is the nature of the work force in the service area. If a good portion of the working population of a community come home, eat dinner, do the dishes, take showers, etc., at the same time then, the largest demand upon and system will probably be experienced within the few hours after dinner. This is typical of most communities. The South Price area is not a typical community. Most of the working population is employed by the coal industry

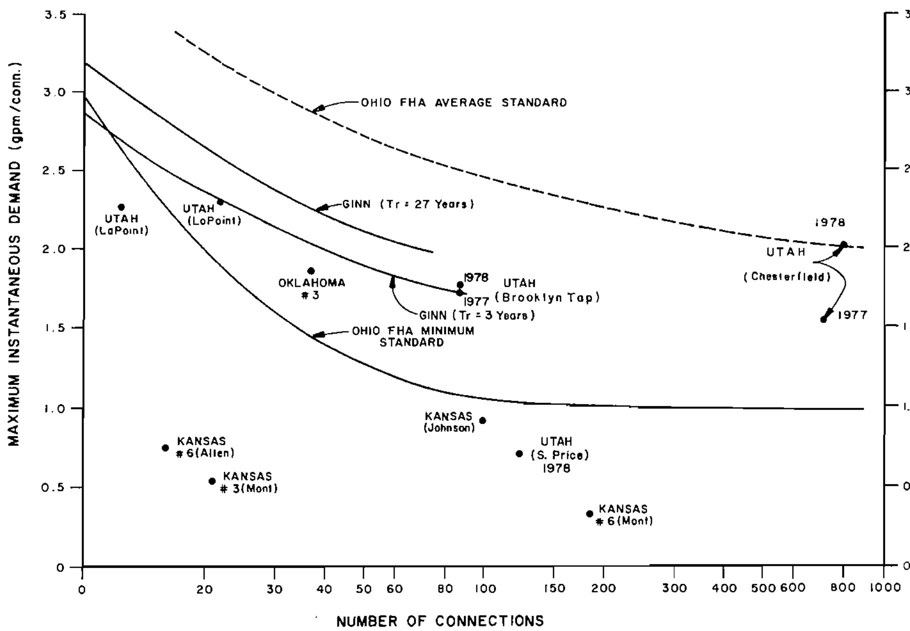

Figure 20. Comparison of measured instantaneous peak flows to FmHA standards.

which operates a few miles north of town. The mining operation continues on a round-the-clock basis. Instead of nearly all of the workers coming home in the evening, the South Price workers are employed in three shifts. This situation reduces the peaks in the daily demand hydrograph and makes the flowrate through the system more uniform. Another reason for the lower peak demands experienced in South Price is the fact that showers are available at the coal mines. Miners always shower before leaving the mine. All of these reasons combine to make the water demand in South Price lower than that in a typical Utah community.

An interesting thing to note about Figure 20 is the close agreement between the statistical model derived by Ginn (Ginn, Corey, and Middlebrooks, 1966) and the data projected by this study. The Ginn function and the Utah data are nearly coincidental throughout their range of service connections. The Utah data and the Ginn function agree despite the fact that two very different approaches were used. Ginn et al. (1966) determined peak flow periods and then read several individual household meters during that peak period. These individual demand patterns were then combined using conditional probability to derive the function shown in Figure 20. The approach taken in this study was to read master meters. In this way, individual flowrates were combined and measured directly.

Derivation of the Expected Demand Function

Figure 21 includes the FmHA minimum and average standard design curves and the Utah State Division of Health standard. It also includes a demand function derived from the Utah rural system data recorded in this report and by Hughes (1977). The Utah expected demand function is for rural water systems with individual household meters. The function was developed to best fit the three data points from the previous report by Hughes and two of the five sets of data from this report (the Brooklyn Tap data). The Chesterfield data were not used in fitting this curve because of a desire not to mix an urban system with otherwise rural data. In order to use a conservatively high estimate of demand, the points used to derive the expected demand function were the 95 percent assurance exceedence levels at a 27 year recurrence interval (P = factor/Tr x 100%) calculated in the section on frequency analysis (see Table 8). The best fit equation here from regression analysis is:

$$Y = \frac{12.68}{X} + 1.80$$

where:

Y = peak demand (gpm/conn) at the 95 percent assurance level for a 27 year recurrence interval

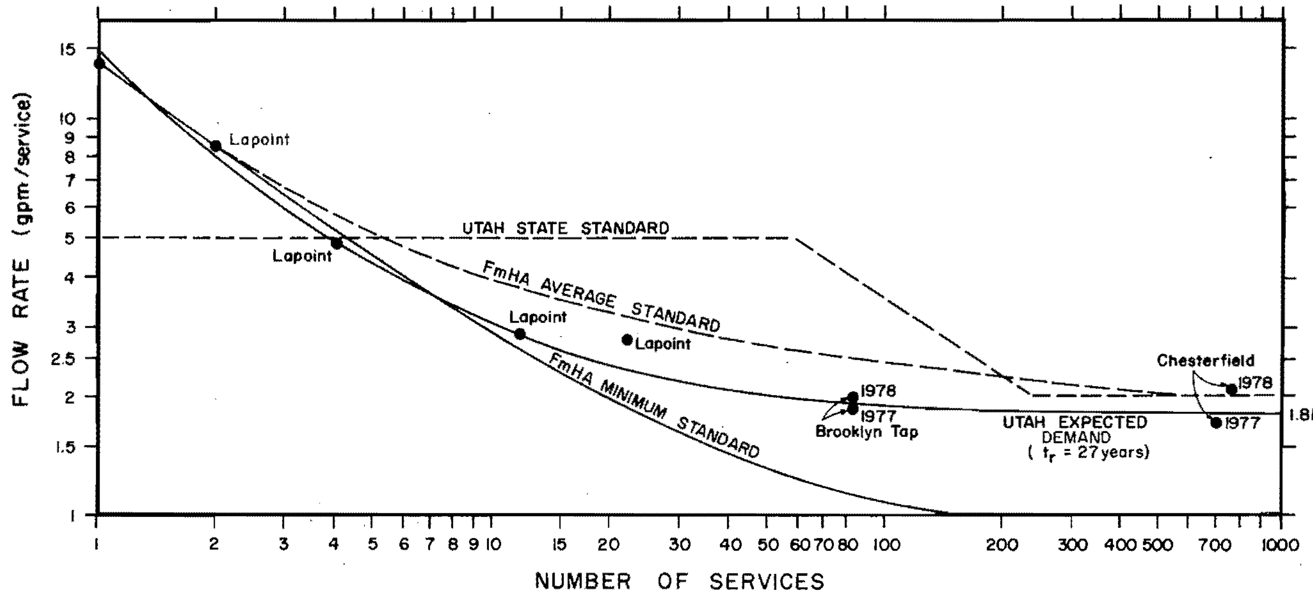

Figure 21. Comparison of Utah expected demand function (27 year recurrence interval) to existing design standards.

X = the number of connections

Returning to the question of whether or not there is a difference between rural and urban water demand, the instantaneous demand function drawn in Figure 21 tends to imply that there is only a slight difference between the two. The Utah demand function fits well through the points from all of the rural systems except South Price (R^2 = 0.966). South Price was excluded from the regression analysis for reasons discussed previously. The Chesterfield system seems to have a slightly higher peak demand than would be predicted from an extension of the rural demand function. One explanation of the higher peak for Chesterfield is that although the residential demands may be no greater than those of rural systems, the industrial demands are, and these show up as higher per connection peaks. The Chesterfield system had 10 industrial water users. It was not possible to separate out this industrial demand on an instantaneous basis and therefore, Chesterfield was not included in the demand function regression analysis.

Figure 21 clearly shows the Utah State design standard to be well above both the expected demand function and the FmHA average standard, especially within a critical range of 4 to 200 service connections. In this range, inflated design standards of this magnitude can cause the cost of a rural water system to increase to where it becomes financially infeasible to the few who are to be served by it.

One reason for the high instantaneous design criteria required by the Utah Division of Health is the possibility of low or negative line pressures when peak periods are experienced by the system. The demand function derived in this study predicts that the flowrates in Figure 21 have a 5 percent chance of occurring once every 27 years for a duration of 5 minutes. During these few minutes negative line pressures can result if the system is limited hydraulically. Negative pressures may cause contamination of the water supply if, for example, a hose end were lying in a puddle or a leak in the pipeline were below the water table. It is more probable however, that low pressures, not negative ones, would simply reduce peak flowrates for those few minutes. If a very short term negative pressure did cause contamination, it would not be any more serious than the contamination caused by the almost yearly pipe ruptures which occur on most systems.

A sample calculation of combined capital investments and operating costs using a hypothetical water distribution system was performed in a previous report (Hughes, 1977). The calculations were performed using the Utah Division of Health requirements, the FmHA average design criteria and a similar design curve derived from the three Lapoint systems. The FmHA minimum design criteria were used as a base calculation. The cost increase above the FmHA minimum caused by using the Lapoint design criteria was 27 percent. The Utah Division of Health requirements would increase the total system cost by 68 percent. The large increase in capital investment and in operating costs could easily make a water project infeasible.

The difference between the Chesterfield demands projected from the 1977 and from the

1978 data should represent essentially all of the variation which can be expected from year to year at that type of system. The lower limit of the projected demand is seen from the 1977 data due to the water use restrictions in effect during the drought. Because 1978 was 50 percent wetter than an average year the utility officials were encouraging water use that year. The demand projected from the 1978 data is probably near the upper limit of the range of expected demands.

Design Recommendations

1. The Utah demand curve (Figure 21) should be used as a design standard for those low density rural systems where cost of a project is a critical issue, and where there is no reason to believe some unusual characteristic such as large landscaped lots exists which would create higher peaks.

2. For rural systems where some factor suggests the possibility of exceptionally high short term peaks, or where the pipe cost is not as critical a higher factor of safety should be achieved by using the FmHA average standard (Figure 21). For example, if the system to be designed includes a high value district with large landscaped lots, the FmHA average standard would be more suitable than the Utah demand function which was derived from systems which were in low to middle value districts. Also, if the size of the lawns to be irrigated is large or if a large percent of the households are to be unmetered, then the FmHA average standard should be used. All of the factors influencing demand that were mentioned in the literature review should be looked at carefully before the design criteria is selected to see if the system to be designed is unusual in some respect.

3. When designing small systems (less than 1000 connections) the textbook equations for estimating peak hourly demand as a multiple of average demand should not be used. Design should rather be based upon either the Utah demand curve or the FmHA average standard depending upon the water use characteristics of the population.

4. Even the high range of textbook factors generally produce peak hour estimates which approximate or are less than actual peak flows in small rural systems.

5. For systems of less than 1000 connections the design should allow for 5 minute peak demands which are higher than peak hour demands. If textbook equations are used or if some estimate of peak hourly flow is available, some adjustment should be made to raise the estimate of hourly demand to the value of the expected 5 minute peak demand. For the systems analyzed in this study that factor varied from 1.08 to 1.24. The high end of the range should be used for middle income neighborhoods where no supplemental irrigation water is available. For lower value districts with secondary water available the low end of the range of factors should be used.

SUMMARY OF DESIGN RECOMMENDATIONS

General Summary

A multiple regression approach was used to develop demand functions for Utah municipal and rural domestic water systems for daily and longer durations. Specifically, peak day, peak month, and average month functions are presented. The independent variables were reduced to either one or two parameters for which data are readily available thereby producing design criteria which are easily usable by consulting engineers. Despite the simple form of the equations, correlation with empirical data from 14 Utah systems was very good with R^2 values from 0.8 to 0.95 and F test ratios much higher than that required for "good" predictive models.

A key to producing the simple demand functions was development of an outdoor use index which characterizes each system in terms of the portion of irrigation demand which is provided by the domestic system. Use of this parameter allowed both rural and urban demands to be represented by a single demand function. The other important demand determinant was price of water. Both expected values (averages) and infrequent peak event demand were presented to allow both average and design level calculations.

Very short term demands (less than one hour) were analyzed by a frequency analysis approach. Both expected values and exceedance levels were presented for any recurrence interval, allowing a designer to select any level of safety factor that is desired. There appears to be a small but significant difference between rural and urban systems in regard to instantaneous demands. The data from this study suggest that the Utah function in Table 21 should be increased by approximately 10 percent for urban design, perhaps more for high value urban districts.

Daily and Monthly Demands

The following demand functions are grouped first as those using price and outdoor use as independent variables and then as functions relating one type of demand to another.

Basic functions

Average demand (thousand gallons per connection):

$D = 4.6 - 5.4\ \ell n(P) + 2.4\ (I)$ (average value)
$D = 5.75 - 6.75\ \ell n(P) + 3\ (I)$ (design)

Average demand (gallons/person/day):

$D = 3.91 - 29.32\ \ell n(P) + 24.64\ (I)$ (average value)
$D = 4.9 - 36.65\ \ell n(P) + 30.75\ (I)$ (design)

Peak month (thousand gallons per connection):

$D = -3.2 - 10.86\ \ell n(P) + 6.74\ (I)$ (average)
$D = -3.84 - 13.0\ \ell n(P) + 8.09\ (I)$ (design)

Peak month (gallons/person/day):

$D = -27.7 - 66.9\ \ell n(P) + 63.64\ (I)$ (average)
$D = -33.2 - 80.3\ \ell n(P) + 76.37\ (I)$ (design)

Peak day (gallons per connection):

$D = 287.4\ (I) - 68$ (average)
$D = 345\ (I) - 82$ (design)

Peak day (gallons/person):

$D = 77.1\ (I) - 17.2$ (average)
$D = 92.5\ (I) - 21$ (design)

Ratios between demands

Peak month as relation to average month (thousand gallons per connection):

$D_{pm} = 2.47\ D_{avg} - 12.31$ (average)
$D_{pm} = 2.96\ D_{avg} - 15$ (design)

Peak month relation to average month (gallon/person/day):

$D_{pm} = 2.43\ D_{avg} - 108.1$ (average)
$D_{pm} = 2.92\ D_{avg} - 130$ (design)

Peak day relation to average month (thousand gallons/conn.):

$D_{pd} = 84.26\ D_{avg} - 206.6$ (average)
$D_{pd} = 101.1\ D_{avg} - 248$ (design)

Peak day relation to average month (gallons/person):

$D_{pd} = 2.5\ D_{avg} - 49.4$ (average)
$D_{pd} = 3.0\ D_{avg} - 59.3$ (design)

Utah systems have peak day demands approximately 2.25 times average demands. It should be noted that this is a higher ratio than that suggested by most literature. The regression equations presented here should used only within the ranges shown in the figures (note minimum values suggested in Figures 3 to 12).

Instantaneous Demands

Very short terms demands (1 to 5 minutes) were measured during this study at 3 Utah systems. The daily peak events varied from 0.5 to 1.6 gallons per minute per connection. Frequency analysis of these data suggest that once in about 30 years, peaks may approach 2 gpm in lines serving 50 or more connections, that 3 gpm peaks may occur in lines serving 10 connections and that 5 gpm levels may be expected only in lines serving 4 to 5 families. These levels are all within the FmHA "average" design standard and that criteria appears to be a reasonable minimum design standard in semi-arid climates such as Utah.

Another significant result of this study is that systems serving less than 1000 connections experience large variations in demand during hourly intervals and that sizing distribution pipelines for hourly peaks is not adequate. Five minute peak flows were as much as 24 percent higher than hourly peaks. Also peak hour levels recommended by most textbooks are lower than hourly peaks measured in the Utah systems.

Another interesting observation was that trailer courts which do not have meters at individual units produced short term peak demands almost identical to those of metered houses.

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APPENDIX A

TYPICAL DAILY DEMAND HYDROGRAPHS

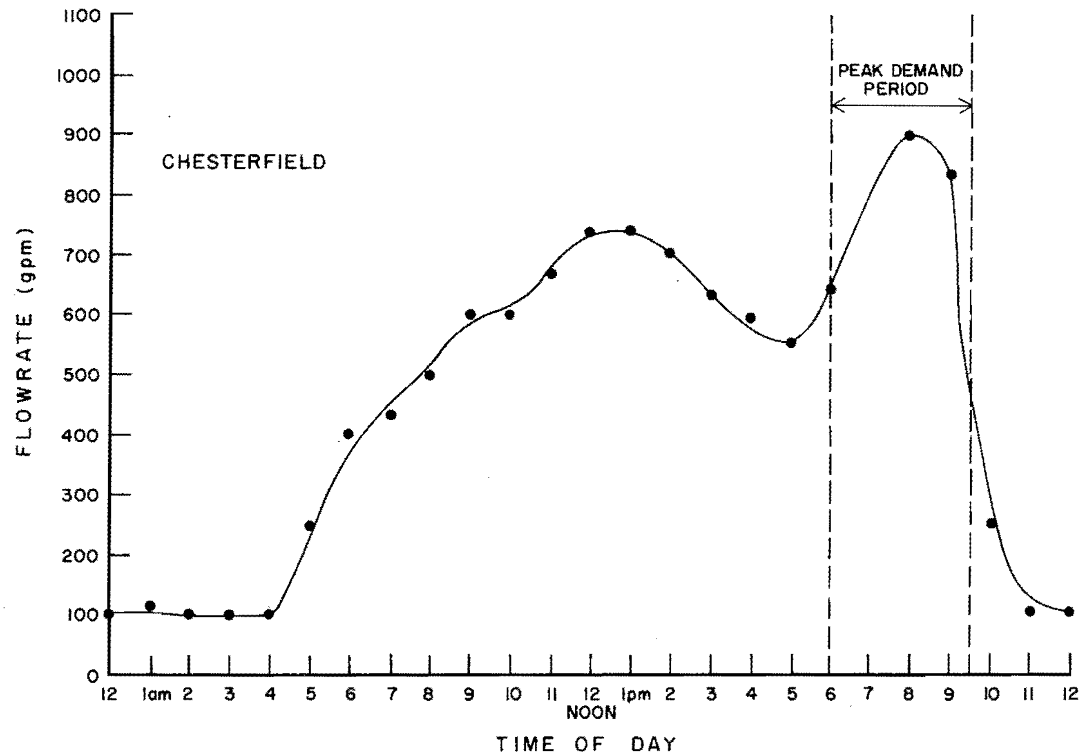

Figure A-1. Typical daily water demand hydrograph for Chesterfield.

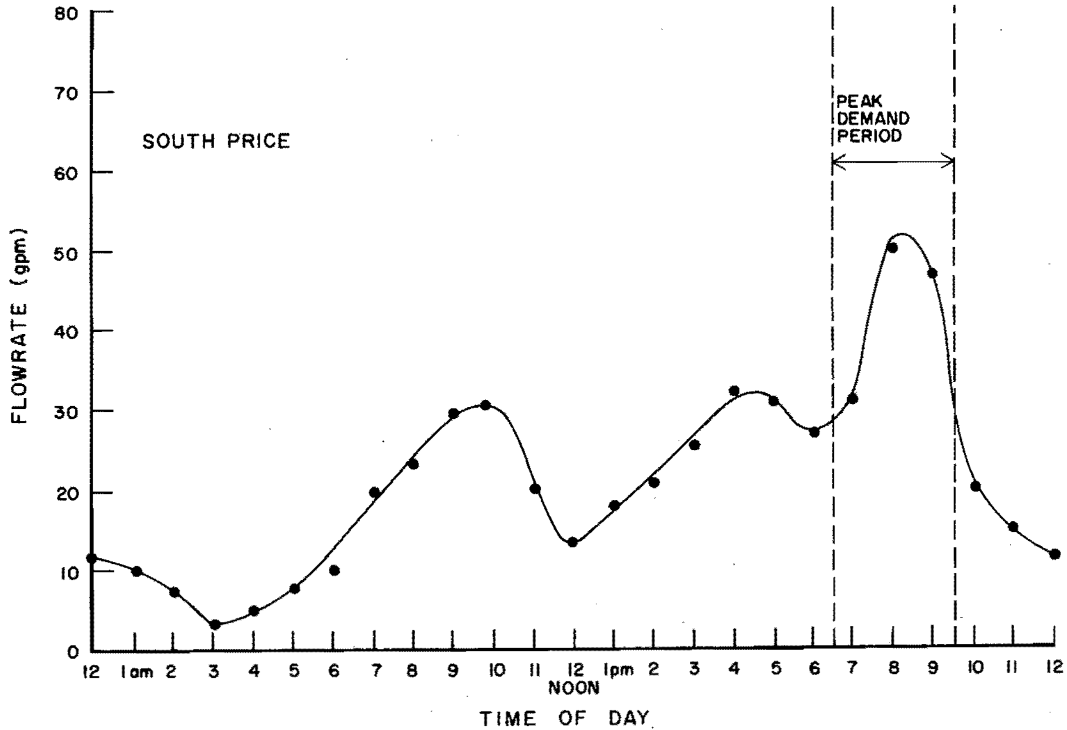

Figure A-2. Typical daily water demand hydrograph for South Price.

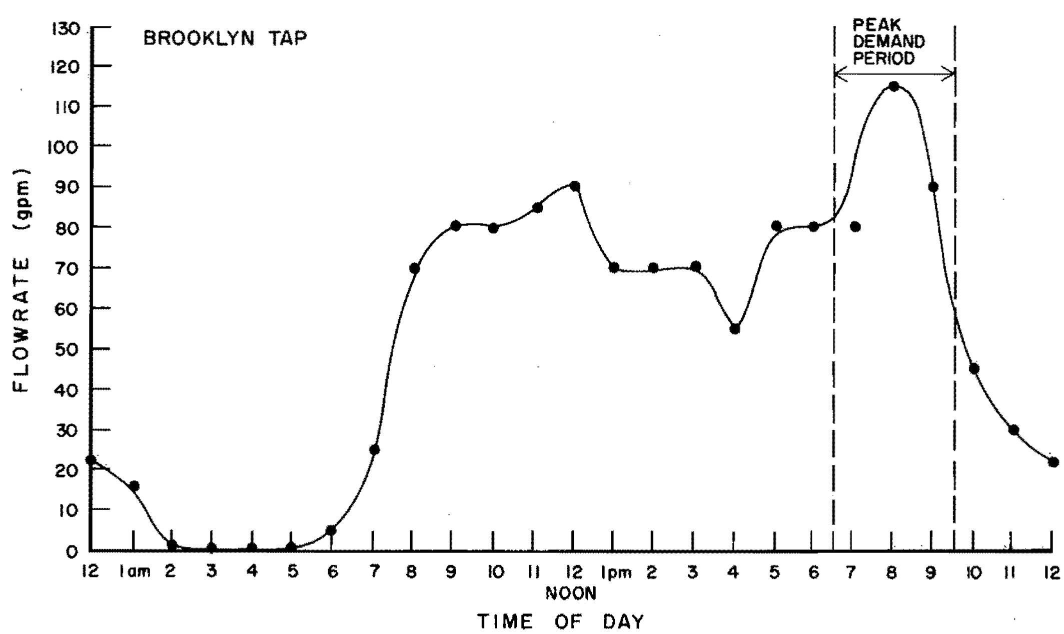

Figure A-3. Typical daily water demand hydrograph for Brooklyn Tap.

APPENDIX B

DAILY MAXIMUM FLOWRATES AND RECURRENCE INTERVALS

Table B-1. Daily maximum flowrates and recurrence intervals, Chesterfield, summer 1977. (727 connections)

Data in Chronological Sequence			Data in Ranked Sequence			
Date	Metered Flow Q (gpm)	Rank M	M	Flow Per Service q_{22} (gpmc)	Recurrence Interval t_r = N+1/M (Days)	Probability $P = 1/t_r$
July 26	954	2	1	1.39	13.00	7.69
27	784	9	2	1.31	6.50	15.38
28	856	7	3	1.27	4.33	23.08
Aug. 10	1010	1	4	1.24	3.25	30.77
11	884	5	5	1.22	2.60	38.46
13	774	10	6	1.18	2.17	46.15
16	924	3	7	1.18	1.86	53.85
22	860	6	8	1.11	1.63	61.54
23	902	4	9	1.08		
Sept. 6	809	8				
7	738	11				
8	685	12				

q = 1.17
s = 0.129
n = 12

Table B-2. Daily maximum flowrates and recurrence intervals, Chesterfield, summer 1978. (790 connections)

Data in Chronological Sequence			Data in Ranked Sequence			
Date	Metered Flow Q (gpm)	Rank M	M	Flow Per Service q_{22} (gpmc)	Recurrence Interval t_r = N+1/M (Days)	Probability $P = 1/t_r$
July 19	1312	2	1	1.70	17.00	5.88
20	1228	4	2	1.66	8.50	11.76
21	904	12	3	1.57	5.67	17.65
23	796	14	4	1.55	4.25	23.53
24	800	13	5	1.50	3.40	29.41
25	1238	3	6	1.47	2.83	35.29
26	1184	5	7	1.30	2.43	41.18
27	1020	8	8	1.29	2.13	47.06
Aug. 4	914	11	9	1.26	1.89	52.94
7	1024	7	10	1.18	1.70	58.82
8	1160	6	11	1.16	1.55	64.71
9	1340	1	12	1.14	1.42	70.59
10	998	9	13	1.01	1.31	76.47
11	936	10	14	1.01	1.21	82.35
14	Rain					
15	Rain					
21	796	15	15	1.01	1.13	88.24
22	778	16	16	0.98	1.06	94.12

$\bar{q}$ = 1.30
s = 0.25
n = 16

Table B-3. Daily maximum flowrates and recurrence intervals, Brooklyn Tap, summer 1977. (84 connections)

Data in Chronological Sequence			Data in Ranked Sequence			
Date	Metered Flow Q (gpm)	Rank M	M	Flow Per Service q_{22} (gpmc)	Recurrence Interval t_r = N+1/M (Days)	Probability $P = 1/t_r$
Aug. 3	114	2	1	1.48	21.00	4.76
8	90	9	2	1.36	10.50	9.52
10	71	16	3	1.35	7.00	14.29
11	111	4	4	1.32	5.25	19.05
12	87	11	5	1.25	4.20	23.81
13	105	5	6	1.25	3.50	28.57
15	72	15	7	1.20	3.00	33.33
16	95	8	8	1.13	2.63	38.10
17	49	20	9	1.07	2.33	42.86
18	82	13	10	1.07	2.10	47.62
19	124	1	11	1.04	1.91	52.38
20	55	18	12	1.01	1.75	57.14
23	55	19	13	0.98	1.62	61.90
24	101	7	14	0.88	1.50	66.67
25	90	10	15	0.86	1.40	71.43
26	74	14	16	0.85	1.31	76.19
27	56	17	17	0.67	1.24	80.95
29	85	12	18	0.65	1.17	85.71
30	113	3	19	0.65	1.11	90.48
31	105	6	20	0.58	1.05	95.24

n = 20
$\bar{q}$ = 1.10
S = 0.220

Table B-4. Daily maximum flowrates and recurrence intervals, Brooklyn Tap, summer 1978. (84 connections)

Data in Chronological Sequence			Data in Ranked Sequence			
Date	Metered Flow Q (gpm)	Rank M	M	Flow Per Service q_{22} (gpmc)	Recurrence Interval t_r = N+1/M (Days)	Probability $P = 1/t_r$
July 11	115	9	1	1.67	32.00	3.13
12	113	11	2	1.60	16.00	6.25
13	128	4	3	1.52	10.67	9.38
14	108	16	4	1.52	8.00	12.50
15	109	15	5	1.50	6.40	15.63
17	85	28	6	1.46	5.33	18.75
19	93	26	7	1.42	4.57	21.88
20	104	22	8	1.38	4.00	25.00
24	101	23	9	1.37	3.56	28.13
25	140	1	10	1.36	3.20	31.25
26	116	8	11	1.35	2.91	34.38
27	85	29	12	1.33	2.67	37.50
28	112	12	13	1.31	2.46	40.63
29	107	18	14	1.31	2.29	43.75
Aug. 2	105	20	15	1.30	2.13	46.88
3	128	3	16	1.29	2.00	50.00
5	110	13	17	1.29	1.88	53.13
8	123	6	18	1.27	1.78	56.25
9	134	2	19	1.26	1.68	59.38
10	119	7	20	1.25	1.60	62.50
11	95	24	21	1.25	1.52	65.63
15	81	30	22	1.24	1.45	68.75
16	108	17	23	1.20	1.39	71.88
17	110	14	24	1.13	1.33	75.00
18	105	21	25	1.12	1.28	78.13
19	94	25	26	1.11	1.23	81.25
21	106	19	27	1.10	1.19	84.38
22	114	10	28	1.01	1.14	87.50
23	79	31	29	1.01	1.10	90.63
25	126	5	30	0.96	1.07	93.75
30	92	27	31	0.94	1.03	96.88

$\bar{q}$ = 1.29
n = 31
S = 0.056

Table B-5. Daily maximum flowrates and recurrence intervals, Price, summer 1978. (124 connections)

Data in Chronological Sequence			Data in Ranked Sequence			
Date	Metered Flow Q (gpm)	Rank M	M	Flow Per Service q_{22} (gpmc)	Recurrence Interval $t_r = N+1/M$ (Days)	Probability $P = 1/t_r$
July 17	60.0	15	1	0.60	21.00	4.76
19	62.0	13	2	0.60	10.50	9.52
20	72.5	3	3	0.58	7.00	14.29
21	67.5	5	4	0.58	5.25	19.05
25	53.5	18	5	0.54	4.20	23.81
26	74.0	1	6	0.54	3.50	28.57
27	57.0	17	7	0.54	3.00	33.33
28	67.5	6	8	0.53	2.63	38.10
31	63.5	11	9	0.53	2.33	42.86
Aug. 2	65.5	9	10	0.52	2.10	47.62
3	72.5	4	11	0.51	1.91	52.38
7	62.5	12	12	0.50	1.75	57.14
8	74.0	2	13	0.50	1.62	61.90
9	67.5	7	14	0.50	1.50	66.67
10	65.0	10	15	0.48	1.40	71.43
11	61.5	14	16	0.47	1.31	76.19
14	51.0	20	17	0.46	1.24	80.95
15	66.1	8	18	0.43	1.17	85.71
16	52.5	19	19	0.42	1.11	90.48
17	58.0	16	20	0.41	1.05	95.24

$\bar{q}$ = 0.51
n = 20
S = 0.056

APPENDIX C

SAMPLE CALCULATION OF 95 PERCENT CONFIDENCE EXCEEDANCE LEVEL

Table C-1. Sample calculation of 95 percent confidence exceedance levels.

Time Interval = 100 days = N

Number of times expected peak will be exceeded = K = 0
This gives us the largest event

J = 0.05 = Probability that the predicted event will be exceeded

$$J = 1 - (1-P)^N$$

$$0.05 = 1 = (1-P)^{100}$$

$$P = 5.128 \times 10^{-4}$$

Looking at Figure 14 for Chesterfield 1978, there is a 1% chance (P = 1) on any summer day that the peak instantaneous flow should reach 1.87 gpm/connection.

To get the 95% confidence exceedance level, go to Figure 14 at:

$$P = 5.128 \times 10^{-4} \times 100\% = 5.128 \times 10^{-2}$$

There is a 95% assurance that a flow of 2.10 gpm/connection would not be exceeded during a 805 day time span

$$T_r = \frac{8.05^*}{P}$$

*8.05 is the recurrence interval factor calculated for the Chesterfield system in the section "Defining the Recurrence Interval."

APPENDIX D

TYPICAL HYDROGRAPHS DURING PEAK DEMAND PERIODS

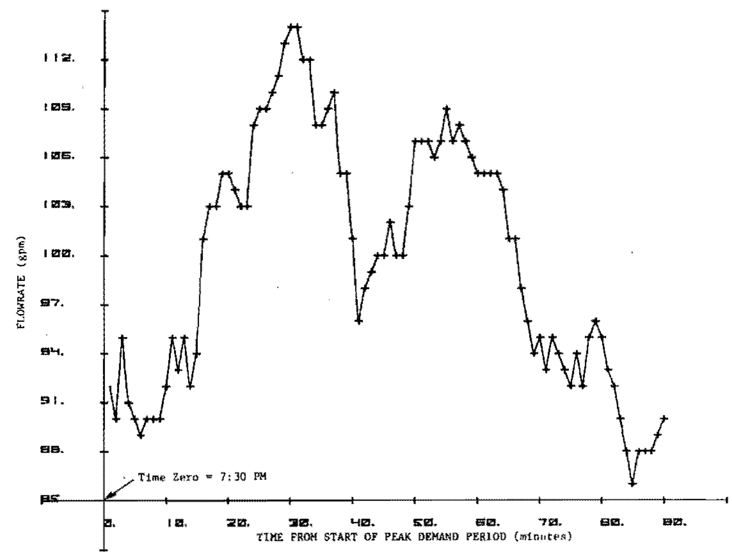

Figure D-1. Daily peak period demand hydrograph for the Brooklyn Tap system, 8-3-77.

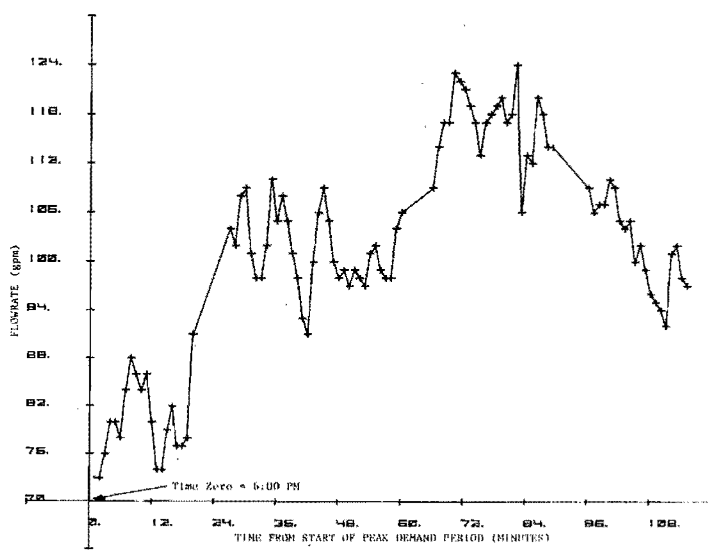

Figure D-2. Daily peak period demand hydrograph for the Brooklyn Tap system, 8-19-77.

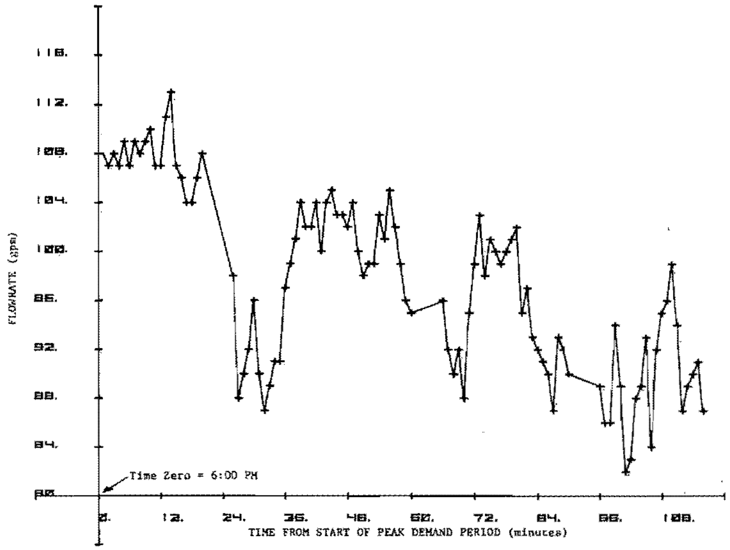

Figure D-3. Daily peak period demand hydrograph for the Brooklyn Tap system, 8-30-77.

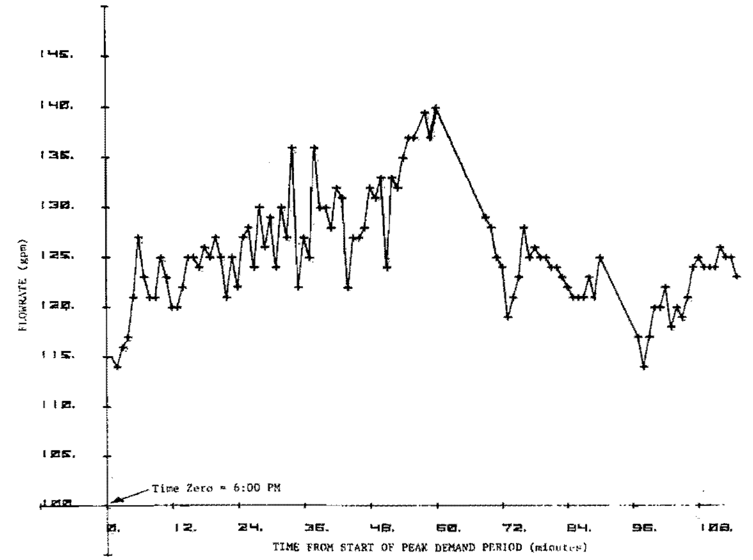

Figure D-4. Daily peak period demand hydrograph for the Brooklyn Tap system, 7-25-78.

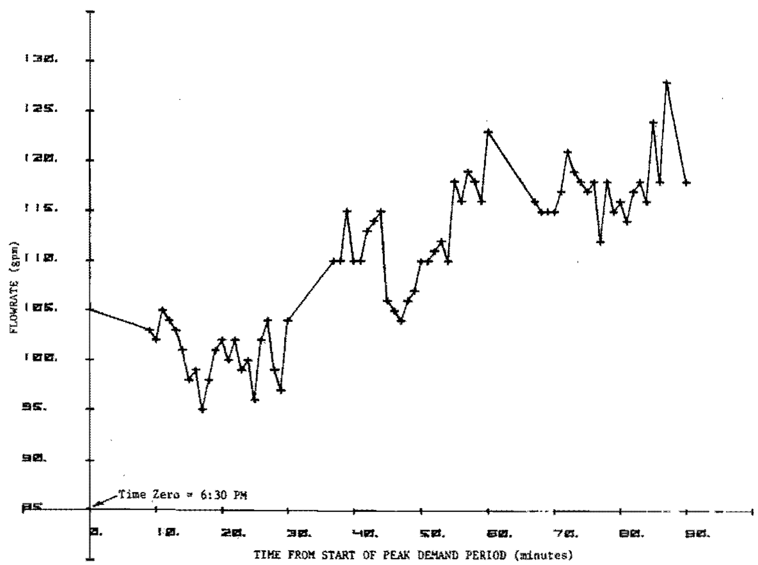

Figure D-5. Daily peak period demand hydrograph for the Brooklyn Tap system, 8-3-78.

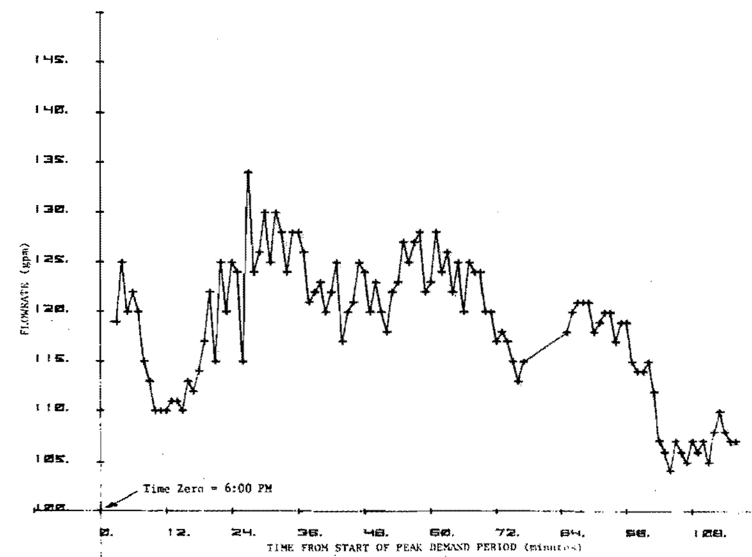

Figure D-6. Daily peak period demand hydrograph for the Brooklyn Tap system, 8-9-78.

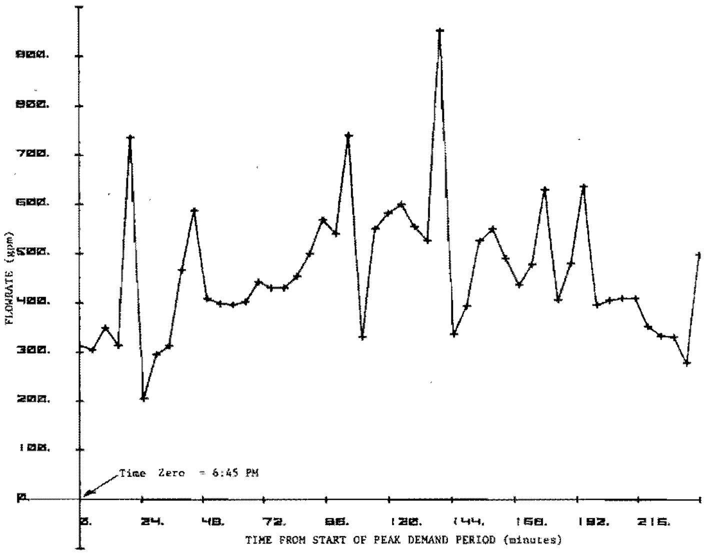

Figure D-7. Daily peak period demand hydrograph for the Chesterfield system, 7-26-77.

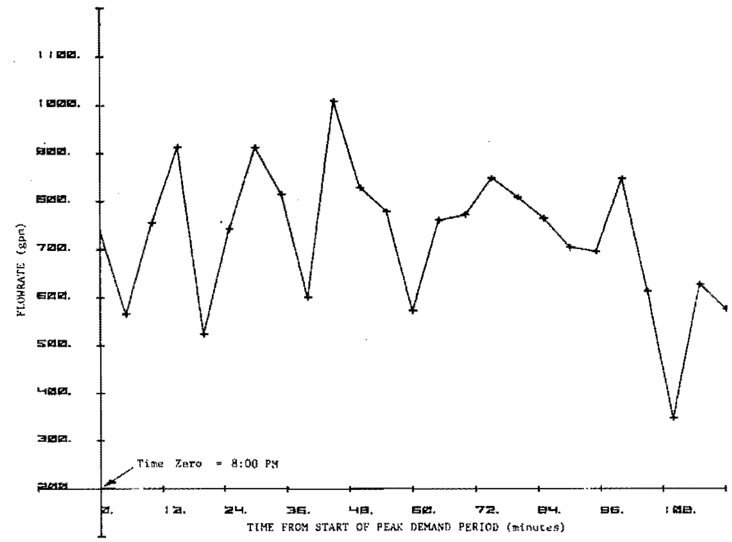

Figure D-8. Daily peak period demand hydrograph for the Chesterfield system, 8-10-77.

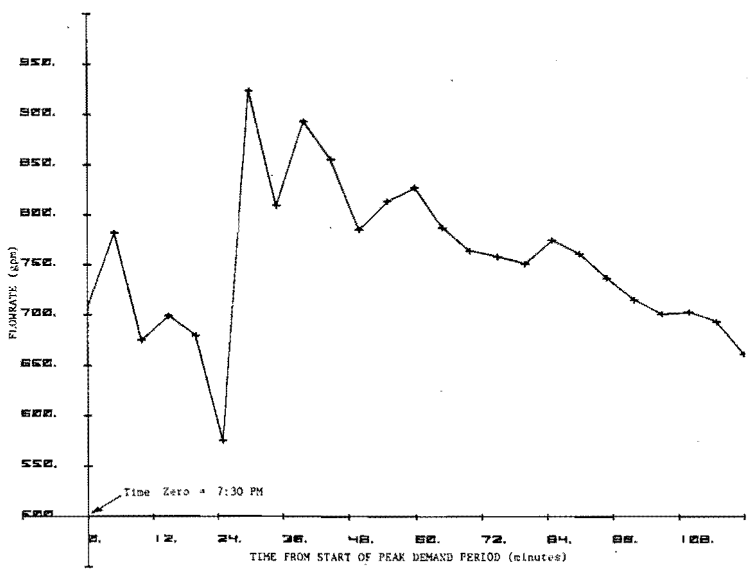

Figure D-9. Daily peak period demand hydrograph for the Chesterfield system, 8-16-77.

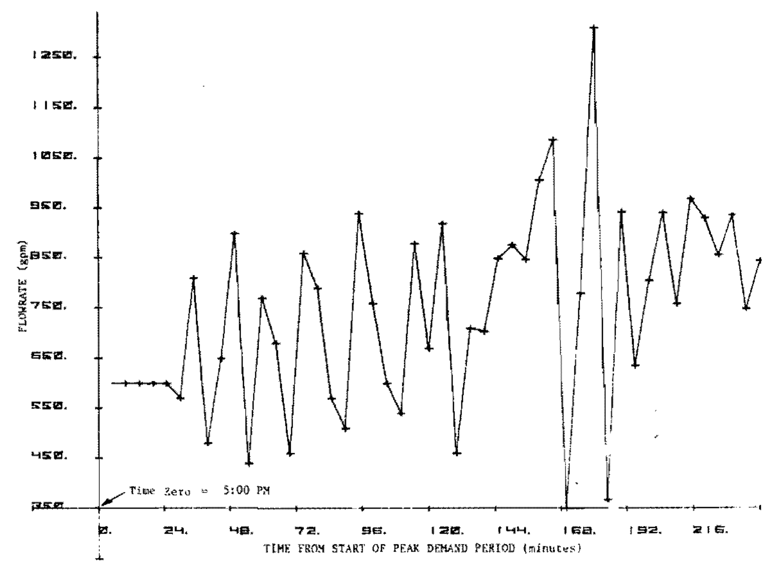

Figure D-10. Daily peak period demand hydrograph for the Chesterfield system, 7-19-78.

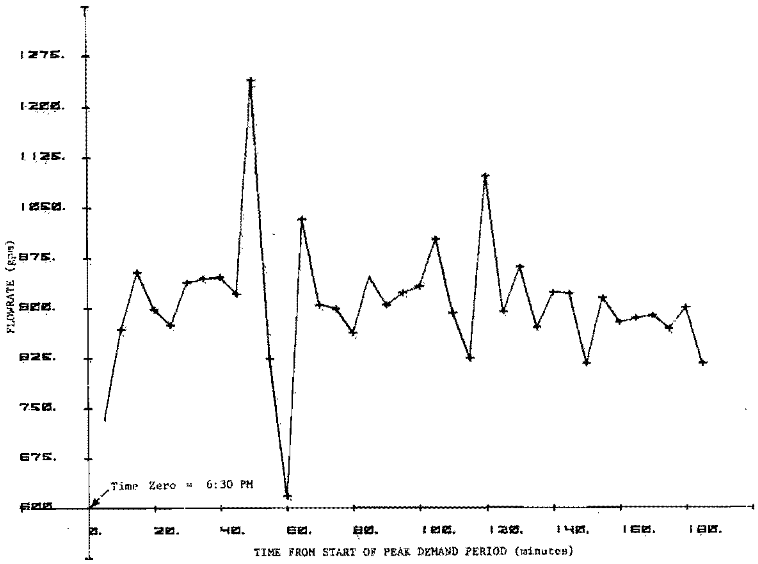

Figure D-11. Daily peak period demand hydrograph for the Chesterfield system, 7-25-78.

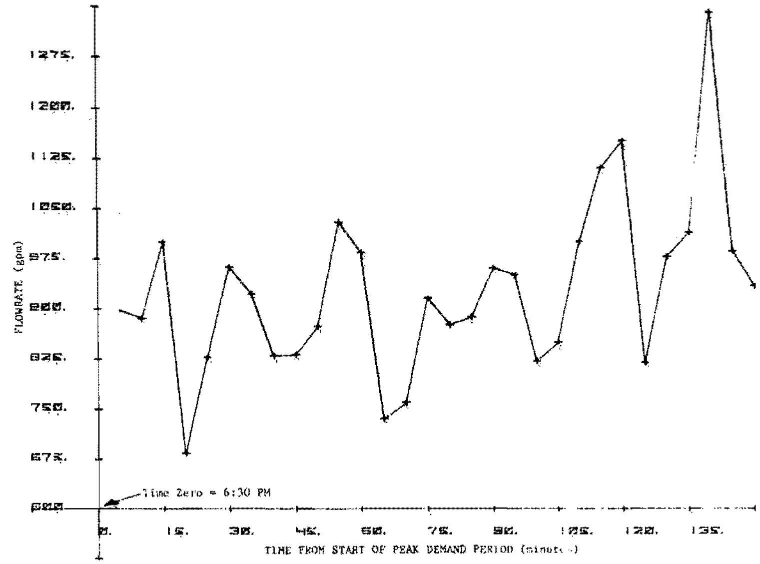

Figure D-12. Daily peak period demand hydrograph for the Chesterfield system, 8-9-78.

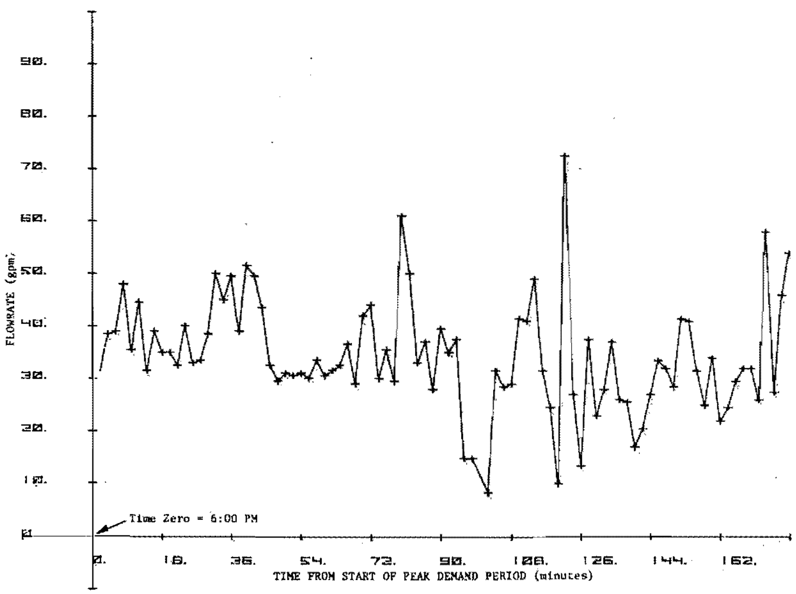

Figure D-13. Daily peak period demand hydrograph for the South Price system, 7-20-78.

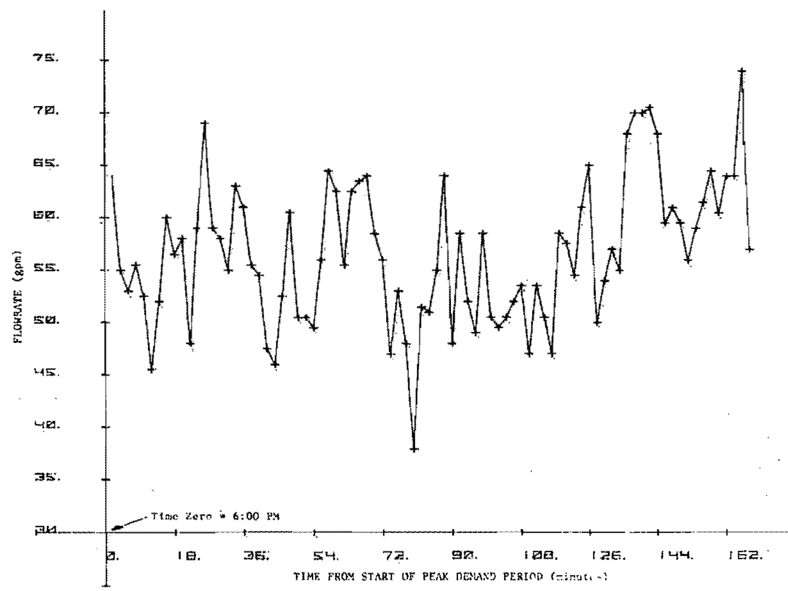

Figure D-14. Daily peak period demand hydrograph for the South Price system, 7-26-78.

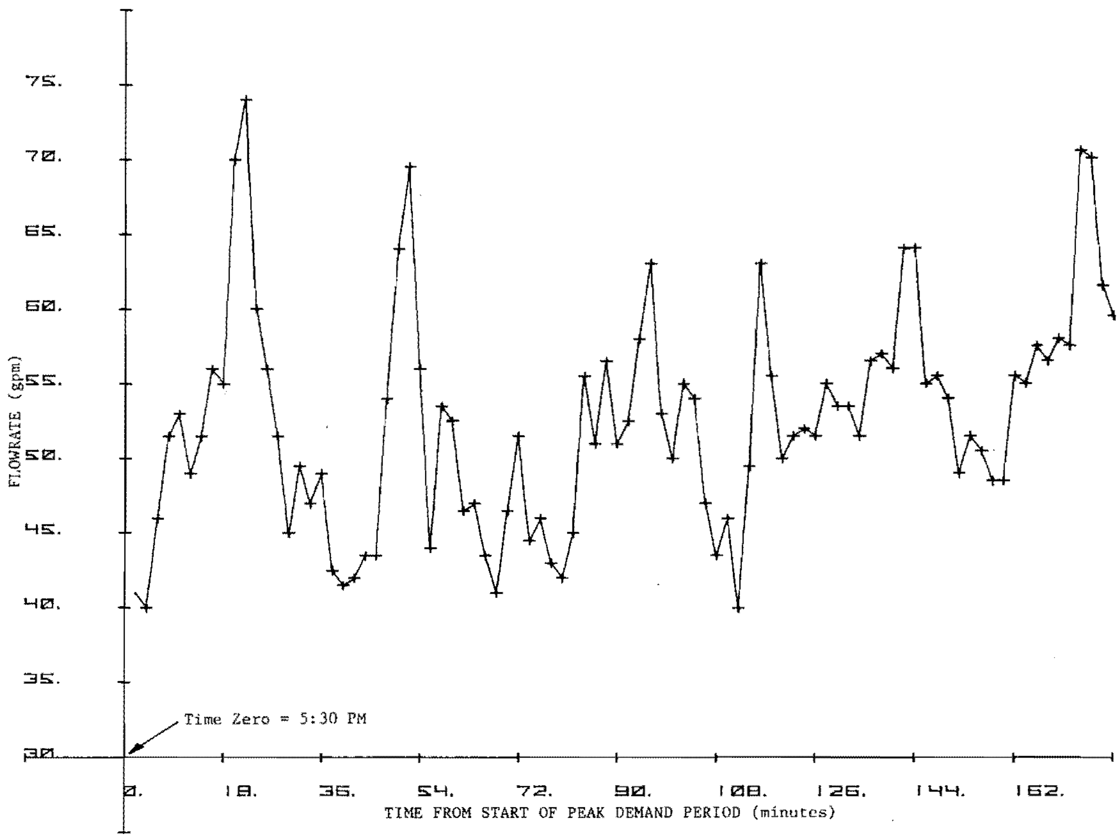

Figure D-15. Daily peak period demand hydrograph for the South Price system. 8-8-78.