CHAPTER 3
REGIONAL WATER SUPPLY PLANNING AND CAPACITY EXPANSION MODELS

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3.1 INTRODUCTION

Water supply planning on a regional scale has attracted more interest in recent years. The major reason for this is that any effect on the quantity and quality of the water resources at a certain location in a region has a bearing on the quantity and quality of this resource at other locations in the region. To meet water supply needs in a given region, it becomes necessary to obtain water from different sources in the region or outside it. This entails not only blending different quality waters but also maintaining the quality of the water distributed within accepted standards, which is an important planning aspect for water supply agencies. Yeh et al. (2000) state that in a multisource, multiquality regional water distribution system, water agencies often find it necessary to impose blending requirements at certain control points in the system in order to secure the desired water quality downstream of the control points.

The desire for efficient water resources management has led to the development of various models for optimal management of water supply systems. Some such models that have been developed in the past consist of nonlinear programming (NLP) models with different sets of objective functions and constraints. These include combinations of nonlinear objective functions and constraints, nonlinear objective functions and a mix of linear and nonlinear constraints, nonlinear objective functions and linear constraints, and so on. Ocanas and Mays (1980, 1981a, b) developed a water reuse planning NLP model for determining the optimum allocation of water and reuse of wastewater on a regional basis for multiperiod planning, which was solved by using the large-scale generalized reduced gradient and successive linear programming methods. Schwartz and Mays (1983) also developed water reuse and wastewater treatment alternative planning models using a dynamic programming approach. Mehrez et al. (1992) developed a model for real-time control and operation of a multisource regional system with multiple customers, which involved both quality and quantity constraints.

A capacity expansion model for water energy systems was developed by Lall and Mays (1981) as a multisystem, multiperiod capacity expansion problem, which was solved using Bender's decomposition with a specialized algorithm. Matsumoto and Mays (1983) also developed a capacity expansion model for large-scale water energy systems as a multisystem, multiperiod capacity expansion problem. Bender's decomposition is applied to the original problem to decompose it into three subproblems: capacity, production, and distribution. Each subproblem was solved using a specialized algorithm (Matsumoto and Mays, 1983). Kim and Mays (1994) formulated the rehabilitation and replacement of water distribution system components as a mixed integer nonlinear programming (MINLP) problem and solved it by decomposing the MINLP problem into a mixed

integer linear programming (MILP) master problem and an NLP subproblem. The NLP subproblem is solved using the generalized reduced gradient (GRG2) (Lasdon and Waren, 1986) computer code, and the MILP master problem is solved using ZOOM, a 0/1 mixed integer programming code.

Herein, two NLP models for regional water supply planning and an MINLP model for capacity expansion of water supply infrastructure are discussed. The NLP models are formulated with nonlinear objective functions to maximize net benefits. The first NLP model is a yearly static model, whereas the second one is a seasonal dynamic model. The revenue from water supplies from different sources, the cost of supplying the water from these sources, and the damage due to poor-quality water being delivered to users are considered in the objective functions of these models. These NLP models are applied to a regional water supply system located along the Rio Grande from Caballo Dam in New Mexico to El Paso County, Texas. The solution obtained for the seasonal model gives the "best" reservoir release policy for four seasons of a year and a maximum net benefit that is slightly higher than that obtained using the static NLP model. Furthermore, compared to the static model, this model generally maintained better-quality water in terms of total dissolved solids (TDS).

The MINLP model for the capacity expansion problem is formulated for water supply conveyance and treatment infrastructure over a long-term planning period. This model incorporates two sets of decision variables: (1) the optimum timing of the capacity expansion of water supply conveyance systems and water treatment plants, and (2) the water allocation policy that maximizes the net benefit. The solution methodology interfaces a simulated annealing (SA) heuristic search algorithm with the GRG2 nonlinear optimization code. The model is applied to El Paso County's water supply system assuming three water supply regions, two potential water sources (surface water and groundwater) for each region, and a planning period of 10 years. The optimum capacity expansion schedules for the water supply infrastructure components for optimum water supply allocations are given.

3.2 MODEL FORMULATIONS

3.2.1 Regional Water Supply Models

A general problem on a regional (watershed) scale is considered in which all the water users in the region are first identified. The general water supply customers considered include municipality, industry, hydropower, irrigation, ranching, recreation, aquaculture, environmental protection, and remediation, whereas general water sources considered include in-stream flow, groundwater pumping, precipitation, interbasin transfer, and return flow or drainage. The various equations used in the models are discussed in the following subsections.

Continuity Equations. A watershed is divided into a total of I reaches where the delineation of a reach is based on whether there is one or more diversions and/or one or more return flows at only one point in the reach (see Fig. 3.1). In each reach, a total of J activities is considered. This approach enables one to model the entire watershed on a reach-by-reach approach.

To develop the continuity equations, which are, in general, mass conservation equations, an arbitrary reach i in Fig. 3.1 is chosen. In Fig. 3.1, all the possible flows associated with reach i are indicated. To derive the first continuity equation, the point of diversion, or return, in reach i is considered. The equation is expressed as

$$\sum_{k=(i-n)}^{i-1} \sum_{j=1}^{J} (Q_{r_{kji}} - Q_{s_{kji}}) + Q_{ds_i} - \sum_{j=1}^{J} Q_{d_{ij}} - Q_{us_{(i+1)}} = 0 \tag{3.1}$$

where $Q_{r_{kji}}$ = return flow from demand point j in reach k to reach i ($k \neq i$)
$Q_{s_{kji}}$ = seepage loss in return flow from demand point j in reach k to reach i ($k \neq i$)
$Q_{d_{ij}}$ = diverted flow from reach i for demand point j, where j represents any kind of demand for water ($i = 1, 2, \ldots, I$; $j = 1, 2, \ldots, J$)

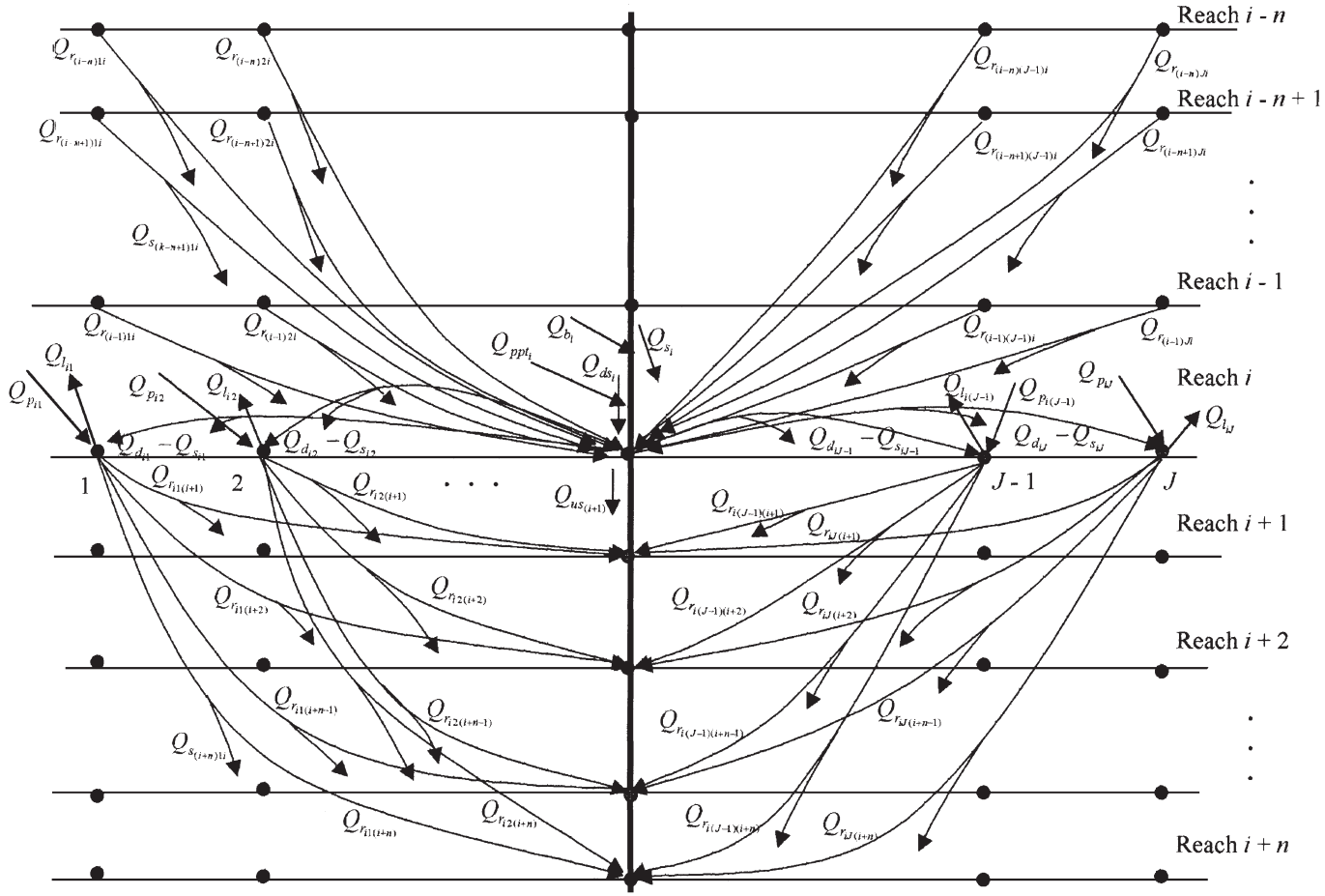

FIGURE 3.1 Schematic representation of the hydraulics of a typical watershed.

Q_{ds_i} = flow at downstream end of reach *i,* that is, in-stream flow into point of diversion in reach *i,* referred to herein as a *node*
Q_{us_i} = flow at upstream end of reach *i*
n = maximum number of reaches a return flow travels before draining into the main stream

The same equation holds true for the entire reaches with boundary conditions for the return flows near the upstream and the downstream ends of the watershed. Also, for each demand point, the following continuity equations can be derived.

$$(Q_{d_{ij}} - Q_{s_{ij}}) + Q_{p_{ij}} - Q_{l_{ij}} - \sum_{k=(i+1)}^{i+n} Q_{r_{ijk}} = 0 \qquad \text{for all } i, j \tag{3.2}$$

where $Q_{s_{ij}}$ = seepage loss in reach *i* on way to demand point *j*
$Q_{p_{ij}}$ = pumped flow from reach *i* for demand point *j*
$Q_{l_{ij}}$ = consumptive use in reach *i* at demand point *j*

Furthermore, within each reach, the following continuity equation is written.

$$Q_{us_i} + Q_{b_i} + Q_{ppt_i} - Q_{ds_i} - Q_{s_i} = 0 \qquad \text{for all } i \tag{3.3}$$

where Q_{b_i} = interbasin transfer to reach *i*
Q_{ppt_i} = flow into reach *i* accountable to net precipitation contribution
Q_{s_i} = in-stream seepage loss in the stream in reach *i*

Water Quality Equations. A typical quality parameter for water distribution on a watershed scale is the amount of TDS in the water supplied. This water quality parameter is taken as an example to develop the equations for water quality management as part of water supply system planning on a watershed scale. Generally, however, multiple water quality parameters may be considered because the quality equations developed will be similar.

The constraints for the point of diversion in reach *i* can be expressed as

$$\sum_{k=(i-n)}^{i=1} \sum_{j=1}^{J} C_{r_{kji}}(Q_{r_{kji}} - Q_{s_{kji}}) + C_{ds_i} Q_{ds_i} - C_{w_i}\left(\sum_{j=1}^{J} Q_{d_{ij}} + Q_{us\,(i+1)}\right) = 0 \tag{3.4}$$

where $C_{r_{kji}}$ = quality parameter in $Q_{r_{kji}}$
C_{ds_i} = quality parameter in Q_{ds_i}
C_{w_i} = weighted quality parameter in $Q_{d_{ij}}$

The flow out of the nodes will have the same quality assuming "perfect" mixing conditions (Yang et al., 1999). Thus, the following equalities also hold true at the diversion point in reach *i.*

$$C_{w_i} = C_{d_{ij}} = C_{us_{(i+1)}} \tag{3.5}$$

Defining $C_{w_{ij}}$ as the weighted TDS concentration at any demand point *j* in reach *i,* the constraints for demand point *j* can be expressed as

$$C_{d_{ij}}(Q_{d_{ij}} - Q_{s_{ij}}) + C_{p_{ij}} Q_{p_{ij}} - C_{w_{ij}}\left(Q_{l_{ij}} + \sum_{k=(i+1)}^{i+n} Q_{r_{ijk}}\right) = 0 \qquad \text{for all } i, j \tag{3.6}$$

Similar equations can be also derived for the in-stream flow in each reach as

$$C_{us_i}Q_{us_i} + C_{b_i}Q_{b_i} + C_{ppt_i}Q_{ppt_i} - C_{ds_i}Q_{ds_i} - \frac{C_{us_i} + C_{ds_i}}{2} Q_{s_i} = 0 \qquad \text{for all } i \tag{3.7}$$

The average TDS concentration is used for the seepage loss in reach i assuming that the change in the concentration in the reach is approximately linear.

Flow Regulation. In Fig. 3.1, the flow system represents the general scenario in which diversions and pumpages in any reach and return flows from any reach to any other reach downstream are possible. However, this may not be the case in practice because, for instance, the number of demand points in one reach may not be equal to those in other reaches. Introducing coefficients with values of 0 or 1 for each of the arcs can regulate these flows, where 0 is used if there is no flow along the arc and 1 is used if there is a flow along the arc. The only exceptions that do not require the 0 or 1 coefficients are the in-stream flows $Q_{u_{si}}$ and $Q_{d_{si}}$, because the coefficient of these flows is always 1. The use of such coefficients has advantages in sensitivity analysis to the optimization model developed, which can be done by turning on and off all uncertain activities.

Let $K_{d_{ij}}$ be the coefficient of flow for $Q_{d_{ij}}$, $K_{p_{ij}}$ be the coefficient of flow for $Q_{p_{ij}}$, and so on. Then, the above continuity and TDS equations are modified by introducing these coefficients.

$$\sum_{k=(i-n)}^{i=1} \sum_{j=1}^{J} (K_{r_{kji}}Q_{r_{kji}} - K_{s_{kji}} + Q_{s_{kji}}) + Q_{ds_i} - \sum_{j=1}^{J} K_{d_{ij}}Q_{d_{ij}} - Q_{us_{(i+1)}} = 0 \text{ for all } i \tag{3.8}$$

$$(K_{d_{ij}}Q_{d_{ij}} - K_{s_{ij}}Q_{s_{ij}}) + K_{p_{ij}}Q_{p_{ij}} - K_{l_{ij}}Q_{l_{ij}} - \sum_{k=(i+1)}^{i+n} K_{r_{ijk}}Q_{r_{ijk}} = 0 \text{ for all } i, j \tag{3.9}$$

$$Q_{us_i} + K_{b_i}Q_{b_i} + K_{ppt_i}Q_{ppt_i} - Q_{ds_i} - K_{s_i}Q_{s_i} = 0 \qquad \text{for all } i \tag{3.10}$$

$$\sum_{k=(i-n)}^{i-1} \sum_{j=1}^{J} C_{r_{kji}}(K_{r_{kji}}Q_{r_{kji}} - K_{s_{kji}}Q_{s_{kji}}) + C_{ds_i}Q_{ds_i} - C_{w_i}\left(\sum_{j=1}^{J} K_{d_{ij}}Q_{d_{ij}} + Q_{us_{(i+1)}}\right) = 0 \qquad \text{for all } i \tag{3.11}$$

$$C_{d_{ij}}(K_{d_{ij}}Q_{d_{ij}} - K_{s_{ij}}Q_{s_{ij}}) + C_{d_{ij}}(K_{p_{ij}}Q_{p_{ij}}) - C_{w_{ij}}(K_{l_{ij}}Q_{l_{ij}} + \sum_{k=(i+1)}^{i+n} K_{r_{ijk}}Q_{r_{ijk}}) = 0 \qquad \text{for all } i, j \tag{3.12}$$

$$C_{us_i}Q_{us_i} + C_{b_i}(K_{b_i}Q_{b_i}) + C_{ppt_i}(K_{ppt_i}Q_{ppt_i}) - C_{ds_i}Q_{ds_i} - \frac{C_{us_i} + C_{ds_i}}{2\,K_{s_i}Q_{s_i}} = 0 \qquad \text{for all } i \tag{3.13}$$

$$C_{w_i} - C_{d_{ij}} = 0 \qquad \text{for all } i, j \tag{3.14}$$

$$C_{w_i} - C_{us_{(i+1)}} = 0 \qquad \text{for all } i \tag{3.15}$$

$$C_{w_{(i-1)}} - C_{ds_i} = 0 \qquad \text{for all } i \tag{3.16}$$

Additional Constraints. Apart from the mass balance constraints, some other physical and resource constraints may be prevalent. The capital expenditure available for pumping the water and

the amount that can be pumped may be limited. Environmental regulations may require that the TDS level at any point be kept below some acceptable level. Such constraints generally can be given as follows.

$$K_{p_{ij}} Q_{p_{ij}} \leq A \tag{3.17}$$

$$C(K_{p_{ij}} Q_{p_{ij}}) \leq B \tag{3.18}$$

$$K_{w_{ij}} C_{w_{ij}} \leq C \tag{3.19}$$

where A = constant
B = constant or a function of amount of pumped water
C = constant or a function of quality parameter in supplied water
$C(K_{p_{ij}} Q_{p_{ij}})$ = cost of any pumping at demand point j in reach i

Objective Function. The objective function can be expressed in terms of the profits from the water resources allocated at each of the demand points, the costs associated with each allocation, and the damages caused due to an undesirable water quality level (such as unacceptable TDS level) at each user point. Defining $P(Q_{d_{ij}}, Q_{p_{ij}})$ as the profit as a function of the allocated supplies (both surface water and groundwater), $C(Q_{d_{ij}}, Q_{p_{ij}})$ as the cost of allocation, and $D(C_{d_{ij}}, C_{p_{ij}}) = D(C_{w_{ij}})$ as the damage, the objective is to maximize the net profit Z given as

$$Z = P(Q_{d_{ij}}, Q_{p_{ij}}) - C(Q_{d_{ij}}, Q_{p_{ij}}) - D(C_{w_{ij}}) \tag{3.20}$$

subject to constraint equations (3.8) to (3.19).

3.2.2 Capacity Expansion Model

Modeling Approach. In this subsection, a water supply system capacity expansion problem is formulated as an MINLP model. Costs for construction, operation, and expansion of the infrastructure are considered as continuous functions of water demands, and the times of expansion are modeled using 0 or 1 integer variables. Thus, for any given unit of the infrastructure, there exist $P - 2$ possible alternatives of expansion of the unit in any P time units of a planning period, such as 20 or 30 years. No expansion during the first or the last time unit in the planning period may be practically feasible, and, thus, we obtain $P - 2 + 1 = P - 1$ possible alternatives for P time units of a planning period. One of the alternatives that doesn't involve any expansion is to initially construct a capacity that is sufficient for the demand during the last time unit of the planning period. Figure 3.2 shows a pictorial representation of such alternatives for any single unit. The objective is therefore to determine the size of the water supply infrastructures and/or when to expand them within the time framework of the planning period at a minimum cost.

The number of possible combinations of construction times grows exponentially with the number of the components to be expanded. In general, for c components that are expandable during any one time unit from a total of p time units, the total possible alternatives will be p^c. For instance, for a certain water supply system with four supply ends, with each end requiring a supply line and a treatment facility, the total number of possible combinations for a planning period of 50 years becomes $(50 - 1)^8 \approx 3.32 \times 10^{13}$. Although a problem with a small number of combinations could be solved by the total enumeration approach, a problem of this size is practically not feasible to solve by this approach. Alternative approaches involving some heuristic search techniques may be employed to obtain a reasonable optimum solution.

For a mathematical formulation, the water supply system is divided into I reaches with J activities (diversion, pumpage, reservoir, etc.) in each reach (see Fig. 3.3). The optimal capacity expansion problem is formulated, in general, as a problem to minimize the sum of costs including construction

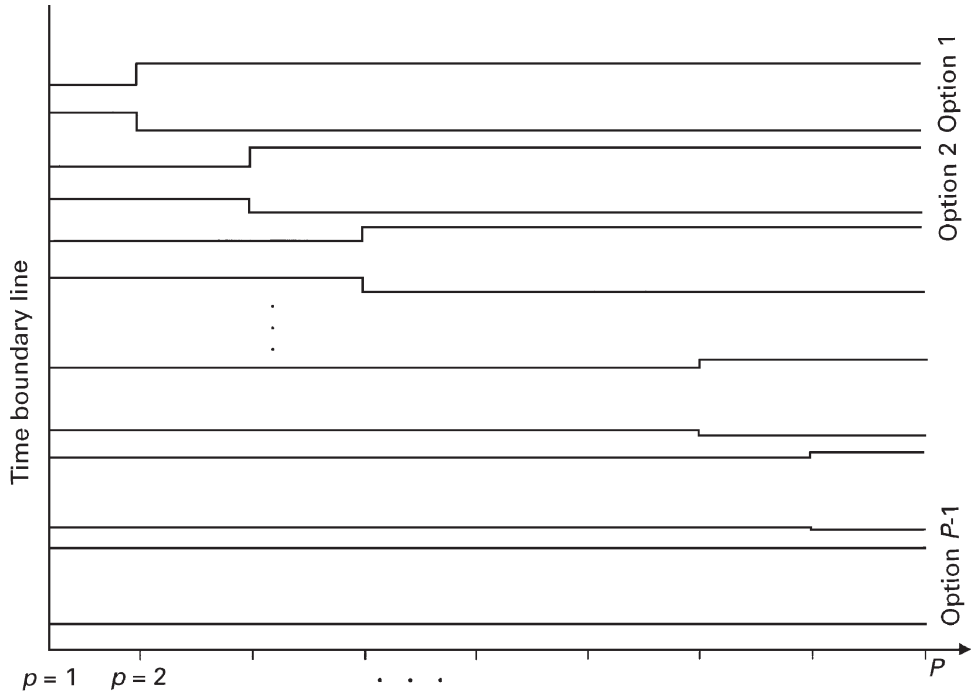

FIGURE 3.2 Different alternative times of capacity expansion of a single water supply unit.

costs, operation costs, and capacity expansion costs subject to constraints on water demand, capacity, mass conservation, and capital availability. The objective function is to minimize the sum of construction, operation, and expansion costs of water conveyance and treatment systems.

Construction Costs. The total construction costs for water supply lines for all activity points j in all reaches i for a supply quantity of Q can be written as

$$\mathrm{CC}_Q = \sum_{i=1}^{I} \sum_{j=1}^{J} \left[\sum_{p=2}^{P-1} X_{ijp} f_c(Q_{ijp}) + X_{ijP} f_c(Q_{ijP}) \right] \tag{3.21}$$

where CC_Q = total cost of construction of supply lines
Q_{ijp} = supply to activity point j in reach i during time p of planning period
$f_c(Q_{ijp})$ = construction cost function for supply line of Q_{ijp}
X_{ijp} = 0 or 1 integer associated with expansion time of capacity of supply line to activity point j in reach i during time p of planning period.

Similarly, the total construction cost for all activity points j in all reaches i can be written as

$$\mathrm{CC}_{\mathrm{CAP}} = \sum_{i=1}^{I} \sum_{j=1}^{J} \left[\sum_{p=2}^{P-1} Y_{ijp} f_c(\mathrm{CAP}_{ijp}) + Y_{ijP} f_c(\mathrm{CAP}_{ijP}) \right] \tag{3.22}$$

where $\mathrm{CC}_{\mathrm{CAP}}$ = cost of construction of treatment facilities
CAP_{ijp} = capacity of treatment facility at activity point j in reach i during time p of planning period

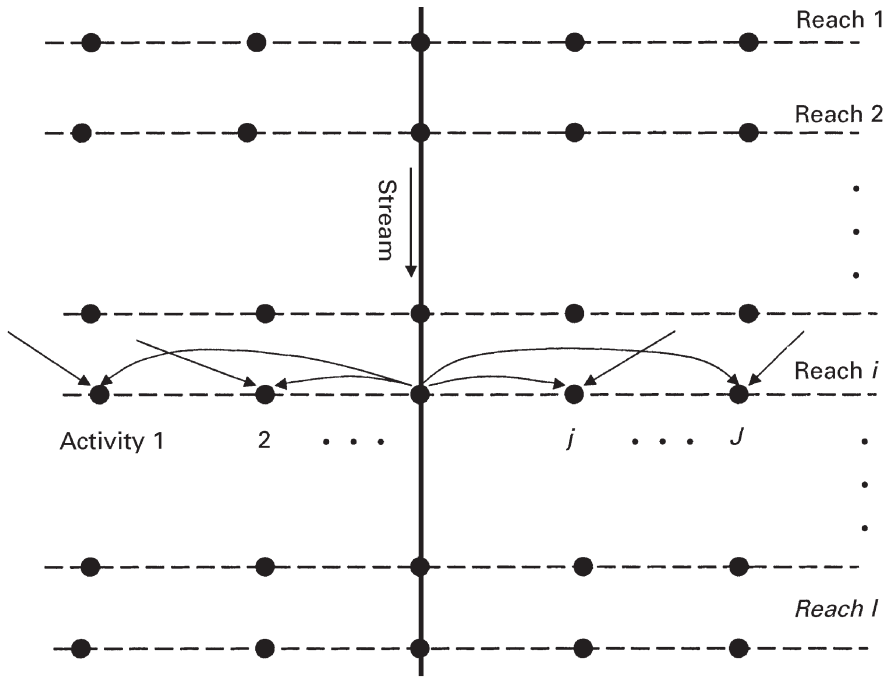

FIGURE 3.3 A water supply system divided into I reaches and J activities with both surface and groundwater supplies.

$f_c(\text{CAP}_{ijp})$ = construction cost for water treatment facility of capacity CAP_{ijp}
Y_{ijp} = 0 or 1 integer associated with expansion time of capacity of water treatment facility at activity point j in reach i during time p of planning period.

In both Eqs. (3.21) and (3.22), the last alternative, which is the "expansion" during time P, is written outside the summation over time to indicate that this case is practically different from the other cases in the sense that there is no capacity expansion involved if this alternative is selected. For such a case, the unit or facility that is constructed at the beginning of the planning period will be sufficient for all the time units in the planning period, that is, no capacity expansion is involved (see Fig. 3.2).

Operation Costs. The total operation cost function can be written for both supply lines and water treatment facilities as

$$\text{OC}_Q = \sum_{i=1}^{I} \sum_{j=1}^{J} \left\{ \sum_{p=2}^{P} X_{ijp} \left[f_o(Q_{ij(p-1)}) \left(\frac{(1+i)^{p-1} - 1}{i\,(1+i)^{p-1}} \right) + f_o(Q_{ijP}) \left(\frac{(1+i)^{P-p} - 1}{i\,(1+i)^{P}} \right) \right] \right\} \tag{3.23}$$

$$\text{OC}_{\text{CAP}} = \sum_{i=1}^{I} \sum_{j=1}^{J} \left\{ \sum_{p=2}^{P} Y_{ijp} \left[f_o(\text{CAP}_{ij(p-1)}) \left(\frac{(1+i)^{p-1} - 1}{i\,(1+i)^{p-1}} \right) + f_o(\text{CAP}_{ijP}) \left(\frac{(1+i)^{P-p} - 1}{i\,(1+i)^{P}} \right) \right] \right\} \tag{3.24}$$

where OC_Q = total cost of operation of supply lines
OC_{CAP} = total cost of operation of treatment facilities
f_o = annual operation function

$$\left(\frac{(1+i)^{p-1}}{i(1+i)^{p-1}}\right) \text{ and } \left(\frac{(1+i)^{P-p}-1}{i(1+i)^{P}}\right)$$

= discount factors to present-worth values

In both Eqs. (3.23) and (3.24), the first term in the sum over time is taken up to time $p - 1$ to avoid any overlap between the operation of the old and the expanded units. In other words, if the expanded unit is ready for time unit p, the operation cost during this time unit should be the operation cost of the expanded unit, not that of the old unit; the operation of the old unit must have ceased at the end of time unit $p - 1$.

Expansion Costs. The total expansion cost function can be written for both supply lines and water treatment facilities, respectively, as

$$EC_Q = \sum_{i=1}^{I} \sum_{j=1}^{J} \sum_{p=2}^{P-1} X_{ijp} \frac{f_e(Q_{ijP} - Q_{ijp})}{(1+i)^p} \tag{3.25}$$

$$EC_{CAP} = \sum_{i=1}^{I} \sum_{j=1}^{J} \sum_{p=2}^{P-1} Y_{ijp} \frac{f_e(CAP_{ijP} - CAP_{ijp})}{(1+i)^p} \tag{3.26}$$

where EC_Q = total cost of expansion of supply lines
EC_{CAP} = total cost of expansion of treatment facilities

$\dfrac{f_e(Q_{ijP} - Q_{ijp})}{(1+i)^p}$ = unit expansion cost discounted to present-worth value

Constraints. The constraints include water demand constraints, capacity constraints, mass conservation constraints, and capital availability constraints. For all i, j, and p, the water demand constraints and the capacity constraints can be given as

$$Q_{ijp} \geq Q_{ijp}^{\min} \tag{3.27}$$

and

$$CAP_{ijp} \geq CAP_{ijp}^{\min} \tag{3.28}$$

where $Q_{ijp}^{\min}$ is the minimum supply required for activity point j in reach i during time p of the planning period and $CAP_{ijp}^{\min}$ is the capacity required for minimum supply requirement at activity point j in reach i during time p of the planning period. The mass conservation constraints can be given as

$$\sum_{j=1}^{J} \sum_{i=1}^{I} Q_{ijp} \leq Q_p^{\text{tot}} \tag{3.29}$$

for all p, where Q_p^{tot} is the total available water supply during time p of the planning period.

Defining TCC, TOC, and TEC as the total capital available for construction, operation, and expansion, respectively, the capital availability constraints can be written for each category as

$$CC_Q + CC_{CAP} \leq TCC \tag{3.30}$$

$$OC_Q + OC_{CAP} \leq TOC \tag{3.31}$$

$$EC_Q + EC_{CAP} \leq TEC \tag{3.32}$$

In Eqs. (3.30) to (3.32), each of the cost terms may be substituted by the right-hand side expressions of Eqs. (3.21) to (3.26).

In addition to the above constraints, each of the integer variables for each activity and in each reach must add to 1 because any unit is expanded once. Thus,

$$\sum_{p=2}^{P} X_{ijp} = 1 \tag{3.33}$$

and

$$\sum_{p=2}^{P} Y_{ijp} = 1 \tag{3.34}$$

for all i and j.

Therefore, the general mathematical formulation of the MINLP capacity expansion problem can be formulated as follows. Minimize:

$$CC_Q + CC_{CAP} + OC_Q + OC_{CAP} + EC_Q + EC_{CAP} \tag{3.35}$$

subject to constraint equations (3.27) to (3.34).

3.3 APPLICATIONS TO THE RIO GRANDE PROJECT AND THE CITY OF EL PASO WATER SUPPLY

3.3.1 Overview of the Rio Grande Project

The Rio Grande project studied stretches from Caballo Dam in New Mexico to the United States–Mexico boundary below El Paso, Texas, on the Rio Grande (see Fig. 3.4). The U.S. Bureau of Reclamation completed the construction of the project in 1916. The primary source of water is the San Juan Mountains of southern Colorado that drain to the Rio Grande, which flows south through New Mexico and Texas to the Gulf of Mexico (Engineering-Science, Inc., 1991). The project's potential water demands include

1. Municipal water supplies for Hatch, Las Cruces, Anthony, and El Paso
2. Irrigation water supplies for the irrigation areas at Percha, Leasburg, Mesilla, and El Paso District No. 1, which consists of two locations
3. Industrial water supplies for Las Cruces and El Paso
4. A 60,000-acre·ft annual water supply to Ciudad Juarez, a city in Mexico, under an international treaty

The competition for the demand of the Rio Grande project water is increasing to the extent where the total water of the project will fall short of the requirements of the different parties, which will have practically few alternative sources. The irrigation districts are currently the primary beneficiaries of the Rio Grande project water. Return flows from drainage from water diverted upstream increase the salinity of the streamflow downstream thereby increasing the damage from irrigation, municipal, and industrial water supplies downstream due to poor water quality. Regulations by the U.S. Environmental Protection Agency (EPA) and local agencies recommend that the TDS level be maintained below 1000 parts per million (ppm).

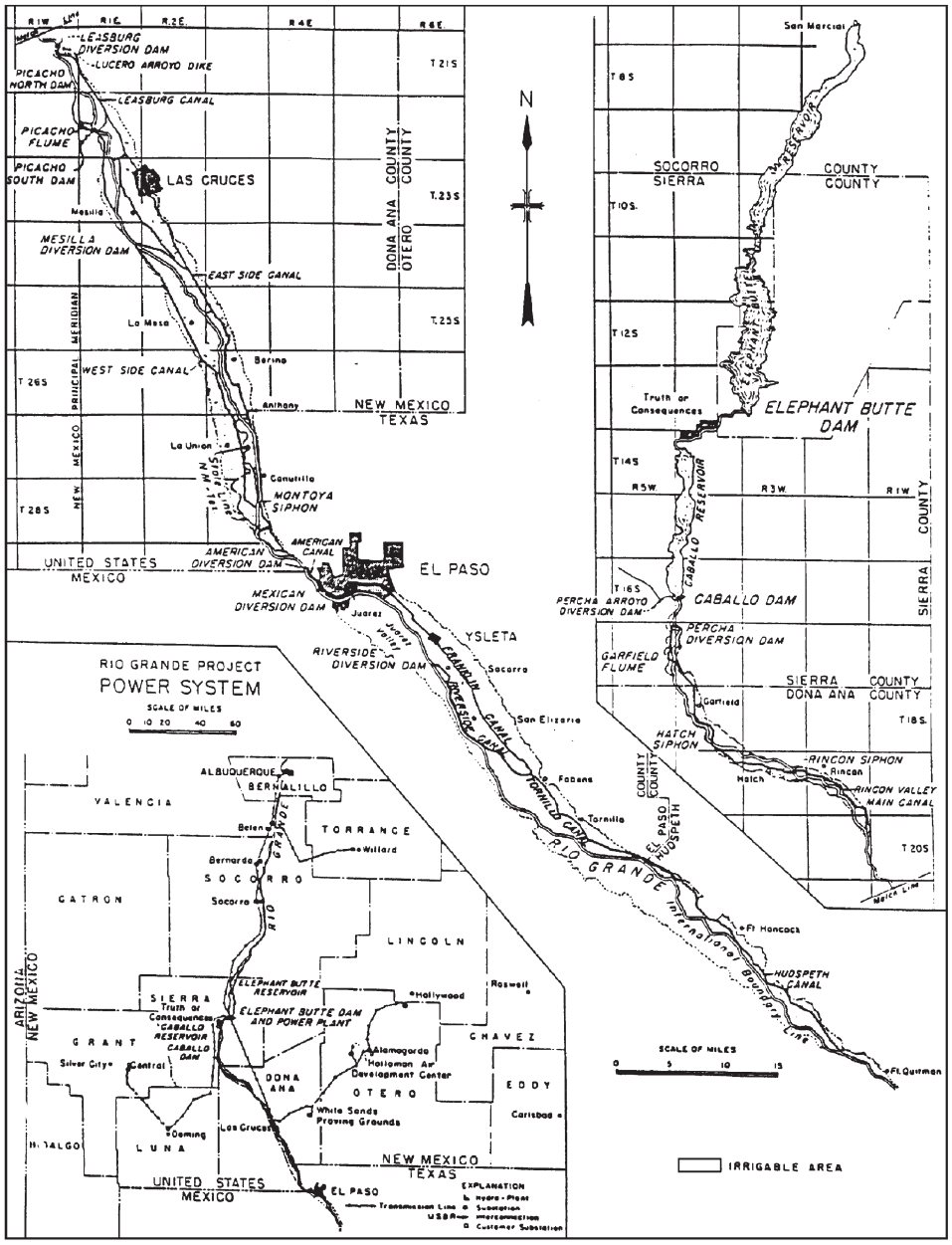

FIGURE 3.4 The Rio Grande project. (*Adapted from USBR project data, 1981.*)

The sources of project water are the Caballo Reservoir, which supplies an annual average of 790,000 acre·ft of water, and groundwater pumping at several locations. The growth of the population of the cities and towns and the maximum limit regulation on the amount of groundwater pumped pose a serious water shortage problem. In addition, the delivery of poor-quality water is also a problem because a portion of the water supply returns to the river, carrying high levels of TDS that affect the water quality of the mainstream flow system downstream. The drainage water first filtrates into the subsurface and then flows to the mainstream with a fairly uniform TDS content approximated at about 1500 ppm.

The five diversion dams for irrigation purposes include Percha, Leasburg, Mesilla, American, and Riverside. The first three diversion dams serve the Elephant Butte Irrigation District (EBID) which consists of the Percha, Leasburg, and Mesilla irrigation areas. The last two diversion dams serve the El Paso County Water Improvement District (EPCWID) No. 1. Percha Diversion Dam is located 2 mi downstream of Caballo Dam and diverts water to the Rincon Valley Main Canal to serve irrigation in the Rincon Valley. Leasburg Diversion Dam is located 62 mi north of El Paso at the head of the Mesilla Valley and diverts water into the Leasburg Canal to serve irrigation in the upper Mesilla Valley. Mesilla Diversion Dam is located 40 mi north of El Paso at the head of the Mesilla Valley and diverts water into the East Side Canal and West Side Canal for irrigation in the lower Mesilla Valley. The American Diversion Dam is located 2 mi northwest of El Paso, immediately above the location where the river becomes the international boundary line between the United States and Mexico. It diverts water into the American Canal for irrigation in the upper portion of the El Paso Valley. The Riverside Diversion Dam is located 15 mi southeast of El Paso and diverts water into the Riverside Canal to serve irrigation in the lower portion of the El Paso Valley.

The current number of estimated households in El Paso is 120,000 whereas that of Hatch, Las Cruces, and Anthony is 1000, 19,000, and 2000, respectively. Hatch, Las Cruces, and El Paso are located in the Rincon, Mesilla, and El Paso valleys, on the opposite sides of the irrigation areas of Percha, Leasburg, and El Paso, respectively. Anthony is located in the Mesilla Valley, below the Mesilla irrigation area.

Several studies have been completed on the Rio Grande project by different agencies and consulting firms, including those by Engineering-Science, Inc. (1991) and jointly by Boyle Engineering Corp. and Parsons Engineering Science, Inc. (1998). The general approach utilized in these studies was the identification and assessment of different management alternatives for the project using available data so that the alternative that resulted in the best management would be selected.

The study by Engineering-Science, Inc. (1991) assessed various opportunities for water storage facilities and the possibilities of additional water supplies by transmountain diversion and by salvage of present losses so that water shortages in the project region would be overcome. The study by Boyle Engineering Corp. and Parsons Engineering Science, Inc. (1998) was aimed at drain mitigation strategies so that acceptable water quality would be maintained for the downstream users in the project area. Based on the results of the simulation study, almost all the selected strategies reduced the problem of quality issues to some extent. However, none of these studies considered the formulation of the problem as a single optimal problem for which the best solution may be obtained.

3.3.2 The City of El Paso Water Supply System

The city of El Paso, Texas, is located along the Rio Grande at the boundary with Mexico (see Fig. 3.4). Most of the population in El Paso County is located in El Paso, which is one of the fastest-growing cities in the nation. The population of El Paso is estimated to become 1.16 million by 2040, whereas that of El Paso County is estimated to become 1.37 million (Boyle Engineering Corporation, 1991), as compared to the 1990 estimates of 529,723 and 752,188, respectively. By 2040, El Paso County's municipal and industrial water requirements are expected to equal 300,000 acre·ft per year, which corresponds to an average consumption rate of 196 gallons per capita per day (gcd). The city is facing a very high water supply shortage because of the apparent increase in demand with time and because its current primary source of groundwater supply is being withdrawn to its potential. It has a limited share of the water of the Rio Grande project. In 1990, the city

owned approximately 2000 acres per year of water rights land from the Rio Grande project (Boyle Engineering Corporation, 1991).

The county's water supply system consists of seven major supply regions: Northwest, Northeast, Central, Lower Valley, East, Fort Bliss, and Hueco. Figure 3.5 shows these seven planning areas, and Table 3.1 gives the population projections for these planning areas for the time from 1980 to 2040, in 10-year intervals. Potential water supply sources and the potential quantity that can be obtained from each of these sources, which include alternative surface water supplies, alternative groundwater supplies, and other alternative sources, have been identified (Boyle Engineering Corporation, 1991). Boyle Engineering Corporation (1991) recommended attempting to overcome future water shortage problems by a more balanced approach of conserving and utilizing both surface water and groundwater supplies to meet the demands.

3.3.3 Static NLP Model Application

The NLP model developed is applied to the Rio Grande project by considering yearly and seasonal time frames. Groundwater and Caballo reservoir's average annual storage of 790,000 acre·ft are considered as the water sources, whereas water requirements for irrigation and municipal purposes are considered as the primary demands. The practical significance of this application can be summarized as follows.

1. All the concerned parties get a fair share of the Rio Grande water.
2. The water requirements under the international treaty are satisfied.
3. The minimum requirements for industrial purposes are satisfied.
4. The net profit from the irrigation and municipal water supplies is maximized.
5. The TDS damage is minimized.

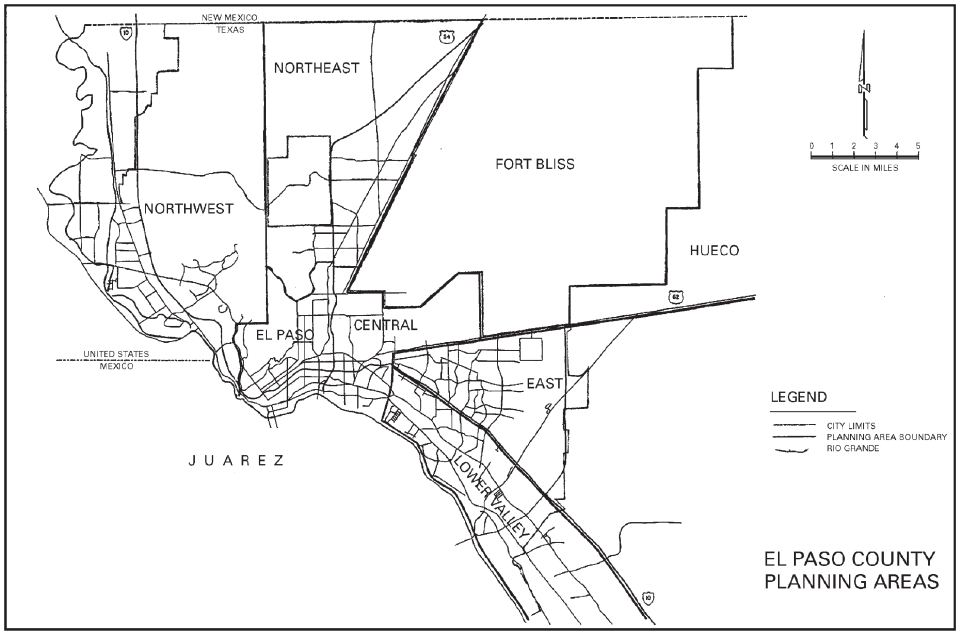

FIGURE 3.5 Major water supply regions in El Paso County. (*Boyle, 1991.*)

TABLE 3.1 Population Projections for the Seven Planning Areas of the City of El Paso

Planning area	Year						
	1980	1990	2000	2010	2020	2030	2040
Northwest	48,938	90,111	135,031	176,800	231,371	280,907	304,634
Northeast	75,398	88,940	106,866	123,696	138,897	154,365	159,162
Central	141,533	140,694	143,184	145,744	145,648	146,184	146,471
Lower Valley	115,807	152,177	192,046	244,025	305,063	370,283	406,870
East	72,023	110,610	141,711	179,014	220,213	267,535	301,026
Fort Bliss	26,200	26,661	26,700	26,700	26,700	26,700	26,700
Hueco			6,650	9,816	13,872	18,731	23,053
Total	479,899	609,193	752,188	905,795	1,081,764	1,264,705	1,367,916

Source: Boyle Engineering Corporation (1991a,b,c).

The potential income from the irrigation water supply is dependent on the location of two major irrigation districts within the project region. These include the EBID and the EPCWID No. 1. The average yearly return from the former irrigation district is estimated to be about $266 per acre of land, which needs a total of 5 acre·ft of water per year. Of this water requirement, 3 acre·ft of water should be obtained from diverted surface water from the Rio Grande and 2 acre·ft of water should be pumped in the vicinity of the irrigation area. For EPCWID No. 1, the average return per acre of land irrigated is estimated to be about $187 per year. All the water for irrigation purposes for this district, also 5 acre·ft per year per acre of land, should be obtained from surface water. Because of its adverse salinity effect on crop growth, groundwater use is not to be opted for in this irrigation district. Table 3.2 gives the incomes from different crops and the weighted average return per acre of irrigated land.

The return from domestic water supply can be derived using the concepts of econometrics. In general, the return is given as

$$R = aQ - \left(\frac{b}{N}\right) Q^2 \tag{3.36}$$

where R = return, $/yr
Q = yearly domestic water supply, acre·ft
N = number of households in city or town
a and b = constants

Table 3.3 gives the values of a and b for different towns and cities in the project area.

The cost of supplying surface water from the river to the towns and cities is estimated at $503.7 per acre·ft per year, whereas the cost of supplying water from the same source to irrigation districts is estimated at $15 per acre·ft per year. The cost of supplying groundwater to the towns and cities is estimated at $325 per acre·ft per year, and the cost of supplying groundwater to the irrigation districts is estimated at $10 per acre·ft per year.

The economic damage that results from supplying saline water to domestic uses and irrigation districts is generally small, but appreciable. The damage results from the reduced lives of household appliances such as dishwashers, water heaters, food waste disposers, water softeners, and evaporative coolers. In the towns and cities that are present in the study area, the damage due to salinity is

TABLE 3.2 Return from Agricultural Activity

Irrigation district	Grains	Forage	Cotton	Chile	Pecans	Average
EBID						
Acreage	4,550	24,240	17,048	17,459	13,767	
Benefit, \$/acre·ft	60	350	126	278	300	266
EPCWID						
Acreage	6,676	11,331	22,404	1,464	5,047	
Benefit, \$/acre·ft	65	319	133	253	187	178

TABLE 3.3 Values of a and b in Eq. (3.36) for Different Cities in the Project Area (1999/2000 Estimates)

S. No.	Town/city name	a value	b value	No. of households	b/N value
1	Hatch	8,948	7,480	1,000	−7.479
2	Las Cruces	21,604	18,050	19,000	−0.950
3	Anthony	8,948	7,480	2,000	−3.739
4	El Paso	8,948	7,480	120,000	−0.062

estimated at 3 cents per household per 1 ppm of TDS per year. Therefore, the TDS damage generally can be expressed as

$$D = 0.03NC(Q) \tag{3.37}$$

where D = damage, \$/yr
N = number of households for which Q is supplied
$C(Q)$ = TDS concentration in Q

On the other hand, the damage to crops because of the salinity of the irrigation water occurs in the form of reduced yield of irrigated crops due to toxic and osmotic effects. In the project area, this is reflected by the reduced income per acre of irrigated land in EPCWID No. 1. The salinity damage to irrigated crops is estimated as the average difference between the income from agriculture in EBID and that in EPCWID's irrigation district, which is presumed to be due to the increase in the salinity in the latter irrigation district. Thus the damage is estimated to be 2.6 cents per the increased TDS over the base value of 450 ppm per acre·ft of water supplied per year.

As shown in Fig. 3.6, the entire project area is subdivided into a total of 17 reaches. In these reaches, the points of interest considered include diversion node, return node, two potential irrigation areas (one on each side), and municipal and industrial points of use. These points of use are currently taken for this project to be the maximum possible points of interest in each reach. Possible return flows to this node or reach from as far as four reaches upstream are considered, which is the maximum distance currently observed to be traveled by the drainage system in the project region (see Fig. 3.7).

At each demand point in a reach, water supply may be obtained from diverted water or from pumpage except for agricultural purposes in reaches 12 and 15 where no pumpage is desirable. Possible return flows from each demand point to the stream as far downstream as four reaches below the reach, in retrospect of the continuity at the node, are considered (see Fig. 3.8). The municipal and industrial pumpages are from deeper groundwater storage, which has a significantly lower TDS level of approximately 250 ppm. However, the pumped waters for irrigation purposes for the Percha, Leasburg, and

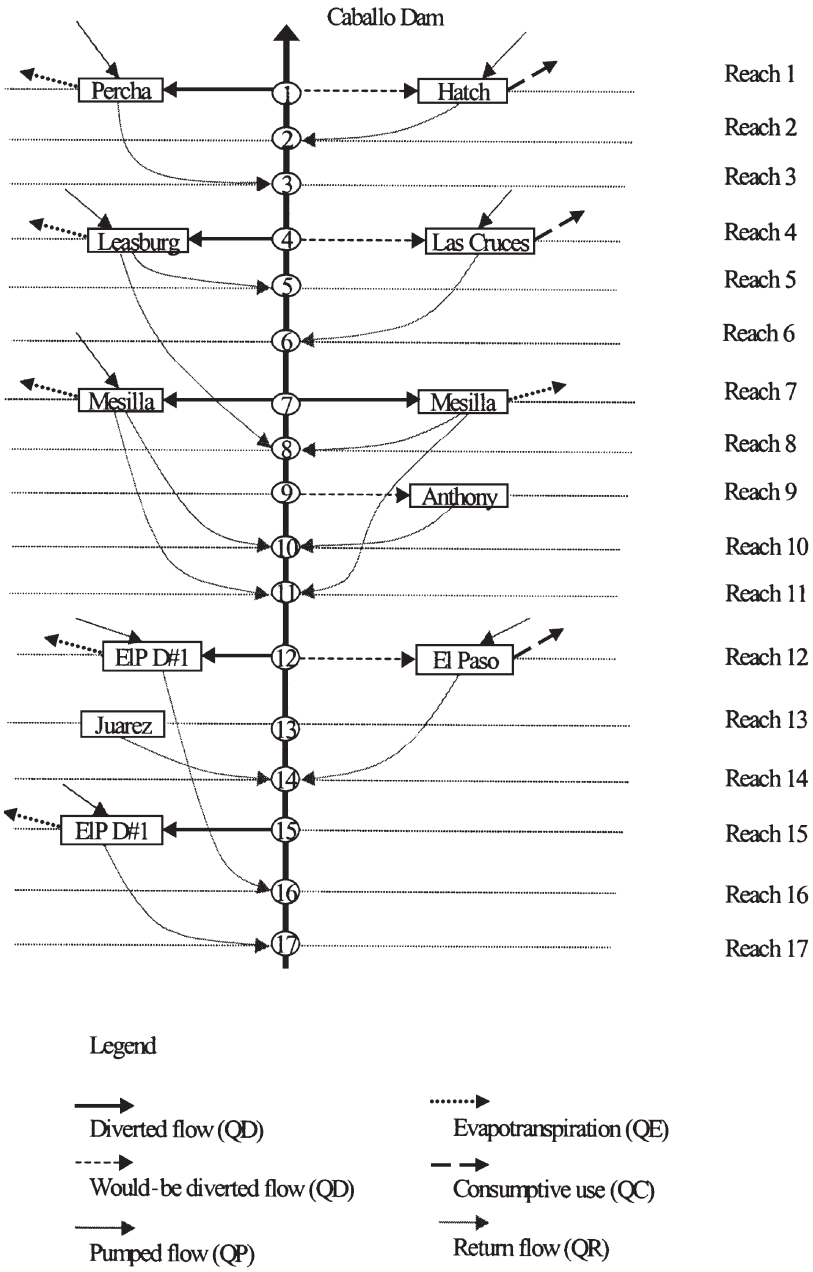

FIGURE 3.6 Schematic diagram of the Rio Grande project system.

Mesilla irrigation areas are from shallow groundwater that has a high TDS level, approximately 1500 ppm, and are assumed to be essentially drawn from the river. Before the diverted water reaches the point of use for irrigation purposes, a significant amount of seepage loss occurs. Although this water is a loss to the immediate purpose for which it is diverted, it will find its way back to the river as a drainage flow or percolate deep into the groundwater storage. For this project, it is assumed that about

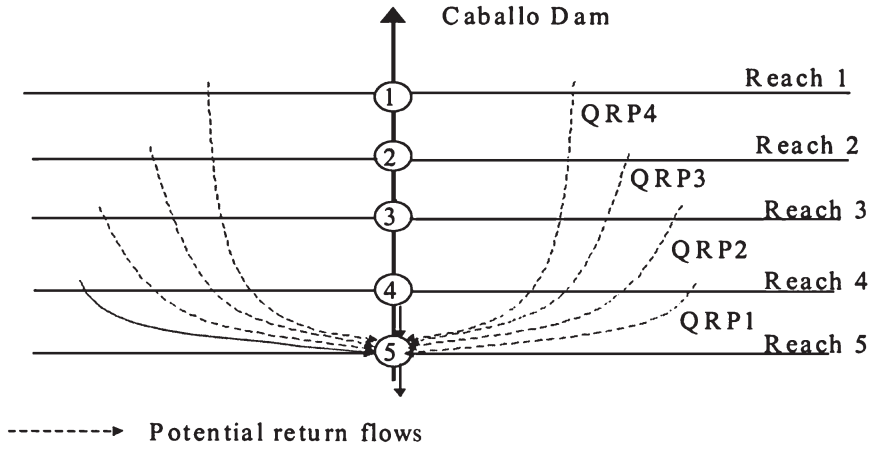

FIGURE 3.7 Schematic diagram showing continuity at a node.

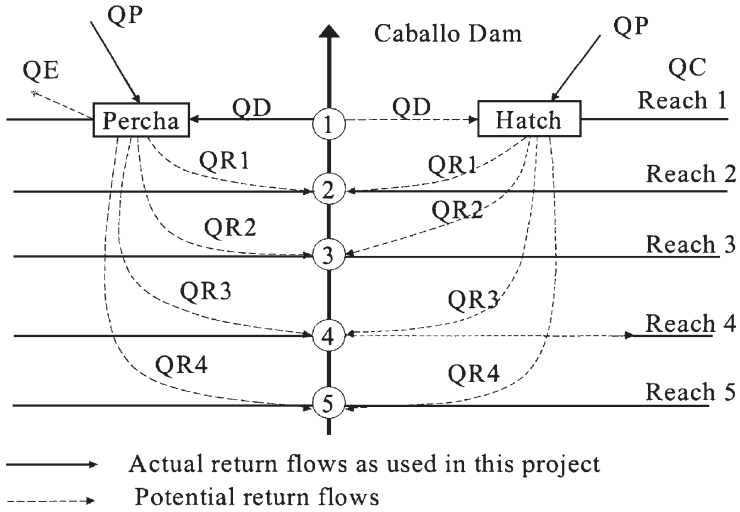

FIGURE 3.8 Schematic diagram showing continuity at a point of use.

40 percent of the water diverted for irrigation purposes will be lost and will become seepage flow, which will eventually drain back to the stream flow. The return flows from the municipalities take place through conduits and thus there is negligible seepage loss.

At each of these demand points, the consumptive use is accounted for by determining the amount of water that is consumed. At the agricultural areas, this constitutes the evapotranspiration, whereas at the municipal points of use, the consumptive losses constitute the consumptive uses for purposes such as drinking and cooking. At the industrial points of use, the consumptive losses constitute the amount that is used in the industrial by-product. To account for these effects, it is assumed that about 50 percent of the water used for irrigation and municipal purposes and about 70 percent of the water used for industrial purposes are "used up." The remaining water from each point of use is assumed to find its way back into the stream flow in the form of drainage.

3.3.4 Seasonal NLP Model Application

The seasonality aspect of the management of the Rio Grande project results from the fact that some of the activities occur only during certain seasons of the year. The irrigation activities occur during the spring, summer, and fall seasons. In addition, the magnitude of the urban water demands depends on the season. For each city and town, this demand is maximum during the summer season and minimum during the winter season. Because of these reasons, consideration of seasonal effects may prove to be more appropriate for this project. Therefore, the planning year is divided into four seasons and the problem is reformulated as a more complex, but more realistic, NLP problem. The seasons considered include December to February as season 1, March to May as season 2, June to August as season 3, and September to November as season 4. The approximate demand proportions by the different activities are given in Table 3.4. This approach surpasses the original problem formulation for a full year that was presented earlier. However, this approach tends to transform the prevailing management practice into a more realistic optimal model.

The general form of the objective function for the urban water demands is essentially the same as the one used in the static model except that the annual urban water demand is taken as the sum of the seasonal demands. For the cities and towns, the modified income (benefit) from the supply of demand is expressed as

$$a[Q(S_1) + Q(S_2) + Q(S_3) + Q(S_4)] - \frac{b}{N}[Q(S_1) + Q(S_2) + Q(S_3) + Q(S_4)]^2 \tag{3.38}$$

where $Q(S_k)$ denotes the demand during season k where $k = 1, 2, 3$, or 4. The other parameters are as defined earlier. It may be noted further that Eq. (3.38) is essentially the same as Eq. (3.36) except that in this case the total yearly demand is substituted by the sum of the four seasonal demands.

For the agricultural activities, it is assumed that the total income will be generated only during the harvesting season of September through November, that is, season 4. The total income taken is the same as that used in the static model, but the water demands for this purpose prevail during the last three seasons of the year to generate this income. Thus, the total income per acre of land is taken as \$266 in the agricultural areas in reaches 1, 4, and 7 and as \$187 in the agricultural areas in reaches 12 and 15. Similarly, the water supply costs and the TDS damages used in the objective function are taken as the sum of the seasonal costs and damages. The costs of supplying surface water and groundwater to the towns and cities are taken as \$503.7 and \$325 per acre·ft, respectively, for the four seasons of the year. These figures were taken in the static model as the annual costs of surface water and groundwater supplies, respectively. The cost of supplying water from the river to the irrigation districts is estimated at \$15 per acre·ft for the last three seasons of the year during which diversions to the irrigation districts prevail. Similarly, the cost of supplying groundwater to the irrigation districts is estimated at \$10 per acre·ft for the two seasons of the year during which pumpage

TABLE 3.4 Approximate Demand Proportions by Different Activities at the Rio Grande Project

Activity	Dec–Feb	Mar–May	Jun–Aug	Sep–Nov
Municipal (fractions)	0.15	0.25	0.35	0.25
Agricultural (unit proportions)				
Diversion	—	0.30	2.20	0.50
Pumpage	—	1.50	0.50	—
Subtotal	—	1.80	2.70	0.50

to the irrigation districts is allowed. Again, these figures were taken in the static model as the annual costs of surface water and groundwater supplies for irrigation, respectively.

The TDS damages are also broken down into seasons so that the seasonality effects are reflected. The seasonal TDS damages that result from both the urban and the irrigation supplies are approximated by taking the same proportionate values as the seasonal demands (see Table 3.4). Thus, the seasonal urban TDS damages per 1 ppm are 0.45, 0.75, 1.05, and 0.75 cents for the first, second, third, and fourth seasons, respectively. Similarly, the estimated irrigation TDS damages per ppm for the last three seasons are 0.936, 1.404, and 0.26 cents, respectively.

In the static model, it was assumed that the total drainage flow occurs during the same year of diversion for all agricultural, municipal, and industrial activities. However, in this model, it is assumed that the total drainage flow from the agricultural activities is spread over two seasons, with about 75 percent of the drainage occurring during the same season as the diversion season and the remaining 25 percent of the drainage occurring during the subsequent season. No diversion or pumping activity for irrigation exists during the first season. However, there is carryover drainage during this season from the diversion during the fourth season of the previous year. This is accounted for by assuming the same 75 percent return flow during the same season and the remaining 25 percent return flow during the first season of the following year. For practical purposes, the quantity of this drainage is taken to be the same as the previous year's drainage during the same season, that is, the first season. Because the drainage from the municipal and industrial activities takes place in conduits, it is assumed to occur during the same season of diversion and pumping.

To write the above distribution of drainage flows over two seasons in equation form, let $Q_{lm(kij)}$ be the return flow from reach *i,* that was diverted for purpose *j,* to *l* reaches downstream from the water diverted during season *m* and returning during season *k.* Then the two general drainage flow distributions over two seasons can be given as

$$Q_{lm\,(kij)} = 0.75\,(Q_{lm\,(kij)} + Q_{l\,(m+1)\,(kij)}) \tag{3.39}$$

$$Q_{l\,(m+1)\,(kij)} = 0.25\,(Q_{lm\,(kij)} + Q_{l\,(m+1)\,(kij)}) \tag{3.40}$$

To better illustrate the notations used here consider, for instance, $Q_{22(S2,R1,A1)}$, which is the drainage flow during season 2 that is returning to two reaches downstream of reach 1 (R1) from the water supplied during season 2 (S2) to agricultural activity 1 (A1). Figure 3.9 shows the general scheme of the seasonal flow system of the Rio Grande project for any given typical season. In general, there is one such scheme for every season.

3.3.5 Capacity Expansion Model Application

Solution Methodology. The MINLP model formulated consists of two sets of variables: discrete time variables and continuous cost variables associated with water demand and treatment, thus making analytical solution of the problem difficult. However, if the discrete binary integer variables are set to specific values, then the problem at that discrete point in time reduces to an NLP problem. The resulting NLP problem can be solved by an appropriate NLP computer program. Nonetheless, such a solution can be obtained only after the values of the integer variables have been fixed.

For small problems, it may be possible to fix the values of the binary discrete time variables and solve the reduced NLP at each of the discrete points, one at a time, using an NLP computer program. In such a case, the best solution will be obtained from these optimal solutions. For a large number of discrete points, in which case it would become very tedious to use this approach, the use of some form of heuristic search algorithm, such as SA, becomes a necessity. One such algorithm may be the use of an automated computer program that is capable of performing the following tasks:

1. Generate the values of the discrete time variables one at a time.
2. Use this information to reduce the MINLP to an NLP.
3. Obtain the optimum solution of the reduced NLP at each of the discrete points.

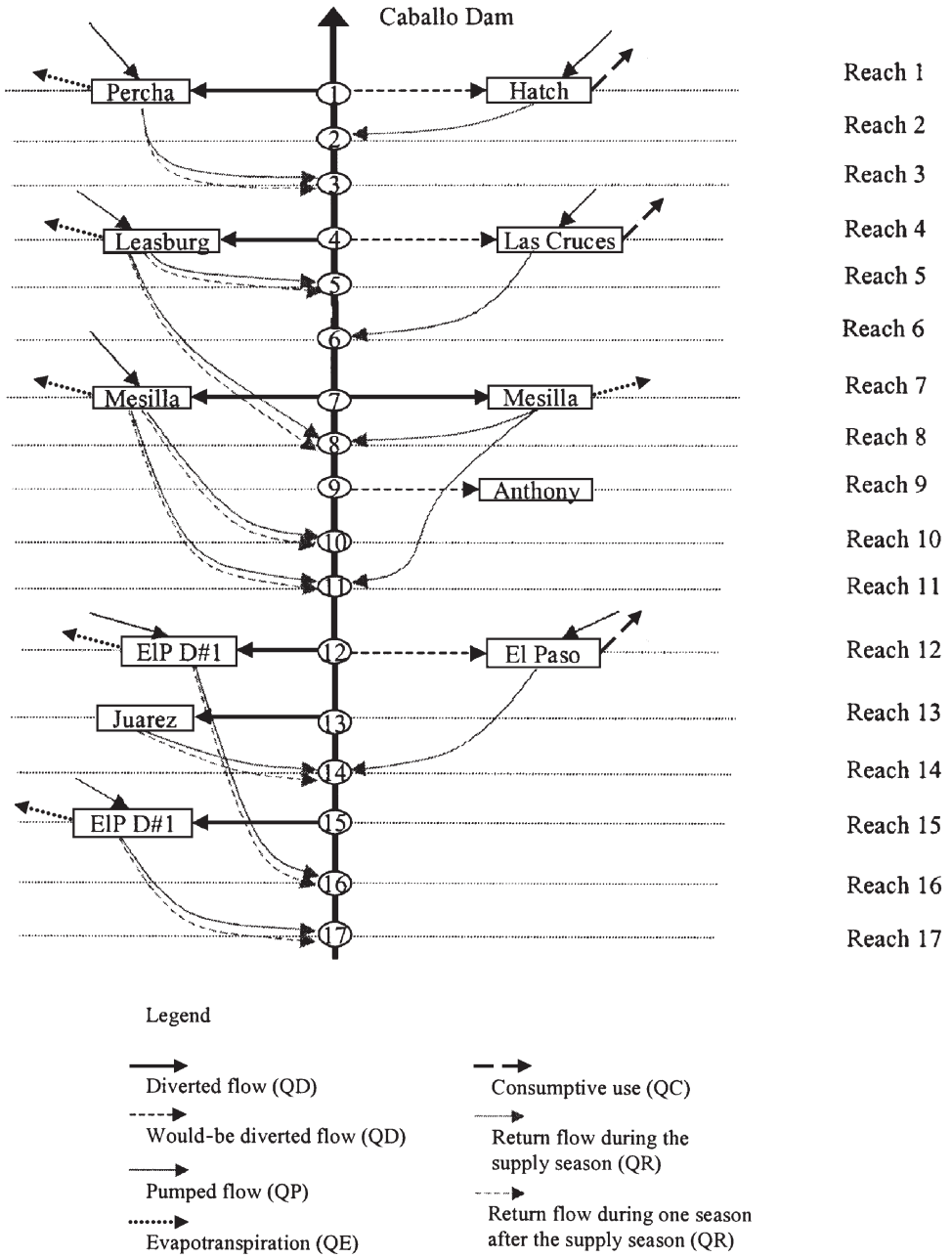

FIGURE 3.9 Schematic diagram of the Rio Grande project system showing the drainage for any given season in which diversion to agriculture exists.

4. Return the best optimum solution among those obtained for all the reduced NLPs.

This solution strategy is pursued in two phases. In the first phase the simulated annealing heuristic search algorithm is implemented to solve a combinatorial problem. In the second phase, the SA code is interfaced with GRG2 to solve the MINLP problem.

The general SA algorithm is given below (Wolsey, 1998, with modifications in the notations).

1. Get an initial solution $\mathbf{X}_o$.
2. Get an initial temperature T and a reduction factor α with $0 < \alpha < 1$.
3. While not yet frozen, do the following:
  3.1 Perform the following loop k times:
    3.1.1 Pick a random neighbor **Y** of **X.**
    3.1.2 Let $\delta = f(\mathbf{Y}) - f(\mathbf{X})$.
    3.1.3 If $\delta \leq 0$, set $\mathbf{X}_{i+1} = \mathbf{Y}$.
    3.1.4 If $\delta > 0$, set $\mathbf{X}_{i+1} = \mathbf{Y}$ with probability $e^{-\delta/T}$; otherwise, set $\mathbf{X}_{i+1} = \mathbf{X}_i$.
  3.2 Set $T \leftarrow \alpha T$ (reduce the temperature).
4. Return the best solution found.

The implementation of the SA heuristic search algorithm requires that a neighborhood and a search strategy be defined first. The starting point with a given set of time units is chosen randomly, and its neighbors are selected. For any given discrete point, the number of neighbors considered is a function of the total number of units to be expanded.

In the course of the neighborhood search, if the current point is at or adjacent to a boundary, that is, where one or more of the time lines are at their limits which include the beginning and the end of the planning period along each time line, then special attention is paid to the search mechanism. This is tackled by the search algorithm in such a way that whether the randomly selected neighbor is at or adjacent to any boundary line is checked before any move. If it is already at the boundary, then no move is allowed, that is, the new neighbor coincides with the center point. If it is already one unit away from the boundary and if the selected neighbor were to move two time units to the boundary, then no two time unit move is allowed; instead, it moves only one time unit to reach the boundary.

The Generalized Reduced Gradient (GRG2). GRG2 solves nonlinear optimization problems with the general form

$$\text{Minimize} \quad f(\mathbf{x}) \tag{3.41a}$$

subject to

$$\mathbf{g}(\mathbf{x}) = \mathbf{0} \tag{3.41b}$$

$$\underline{\mathbf{x}} \leq \mathbf{x} \leq \overline{\mathbf{x}} \tag{3.41c}$$

where $\mathbf{x}$ = n-dimensional column of vector variables
$f(\mathbf{x})$ = objective as a function of $\mathbf{x}$
$\mathbf{g}(\mathbf{x})$ = m constraints
$\underline{\mathbf{x}}$ = lower bound on $\mathbf{x}$
$\overline{\mathbf{x}}$ = upper bound on $\mathbf{x}$

The fundamental idea of the generalized reduced gradient is to express m variables, called basic variables, in terms of $n - m$ variables, called nonbasic variables. The objective function is thus expressed as a function of the nonbasic variables, which are bound between their upper and lower limits. The resulting formulation, called the reduced problem, is solved using the unconstrained NLP solution technique with modifications to account for the bounds for the nonbasic variables. Solving a sequence of reduced problems solves the original NLP problem.

Interfacing the Simulated Annealing Algorithm with GRG2. The SA search algorithm code was interfaced with the GRG2 code to solve the MINLP problem. Figure 3.10 shows the general flow-

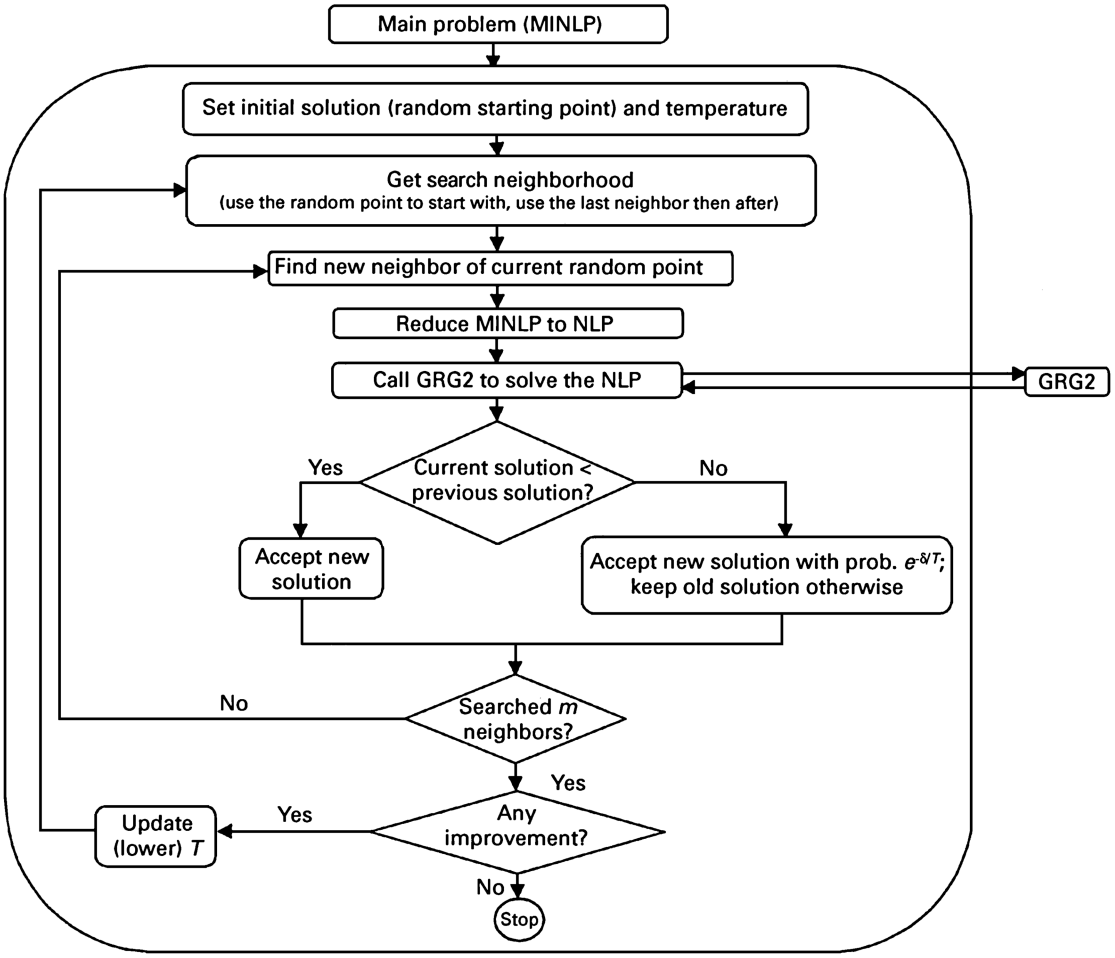

FIGURE 3.10 A flowchart showing the procedures used by the model.

chart of the computer program used to solve the MINLP problem. To use the GRG2 code, a user-supplied GCOMP subroutine was written. The main elements of this subroutine are the definitions of the objective function and the constraint functions, which are given as a list of functions whereby the index of the objective function is specified in the main program. Furthermore, two options exist for the user in selecting the methodology to calculate the gradient. GRG2's default option is to use the forward differencing procedure. Alternatively, partial derivatives of the functions given in the GCOMP subroutine can be determined by the user and another subroutine called PARSH can be provided to the model. For this application, GRG2's default option was used.

General Construction, Operation, and Expansion Cost Equations. In general, many capital facilities are usually constructed with a capacity that will satisfy the requirements over some years to come, instead of a capacity that satisfies immediate requirements. The main reason for this lies in the economies of scale available with a large plant that may be achieved in investment cost or operating cost (Hinomoto, 1974). Chenery (1952) proposed the following power function for an industrial facility.

$$C = \alpha K^{\beta} \tag{3.42}$$

where C = investment cost
K = design capacity
α and β = positive parameters determined by observed data

In this equation, if $K = 1$, then C equals α, which can be defined as the investment cost of a system with a capacity of one unit. Parameter β determines the manner in which investment cost changes capacity. The investment cost increases with capacity at an increasing or decreasing rate depending on whether β is greater than or less than 1, respectively.

The general equation for the construction of a water supply unit, including setup costs K_c, can be expressed as

$$\mathrm{CC}(Q_{ijp}) = K_c + \alpha_c Q_{ijp}^{\beta c} \tag{3.43}$$

where α_c and β_c are constants. Using this general form, the operation cost for a water supply facility during a planning period p is expressed as

$$\mathrm{OC}(Q_{ijp}) = K_o + \alpha_o Q_{ijp}^{\beta_o} \tag{3.44}$$

and for expansion of the water supply facility as

$$\mathrm{EC}(Q_{ijp}) = K_e + \alpha_e (Q_{ij\,(p+1)} - Q_{ijp})^{\beta c} \tag{3.45}$$

where K_c, K_o, K_e α_o, β_o, α_e, and β_e are constants. The construction, operation, and expansion costs of a water treatment facility are expressed, respectively, as

$$\mathrm{CC}(\mathrm{CAP}_{ijp}) = C_c + \theta_c (\mathrm{CAP}_{ijp})^{\phi_c} \tag{3.46}$$

$$\mathrm{OC}(\mathrm{CAP}_{ijp}) = C_o + \theta_o (\mathrm{CAP}_{ijp})^{\phi_o} \tag{3.47}$$

$$\mathrm{EC}(\mathrm{CAP}_{ijp}) = C_e + \theta_e (\mathrm{CAP}_{ij\,(p+1)} - \mathrm{CAP}_{ijp})^{\phi_e} \tag{3.48}$$

where C, θ, and ϕ are constants with the subscripts c, o, and e denoting construction, operation, and expansion, respectively.

The EPA has developed empirical functions for the estimation of the three types of costs for water and wastewater conveyance systems and water and wastewater treatment plants. Clark et al. (2002) developed different cost equations for the construction of water supply distribution system components; for water supply distribution system rehabilitation; and for the construction of pumps, tanks, and reservoirs. The cost functions for water supply distribution system components for different pipe systems include:

1. Base installed pipe costs
2. Trenching and excavation costs
3. Embedment costs
4. Backfill and compaction costs
5. Valve fitting and hydrant costs
6. Dewatering costs
7. Sheeting and shoring costs
8. Horizontal boring costs
9. Pavement removal and replacement costs

10. Utility interference costs

11. Traffic control costs

12. Household service connection costs

The general cost equation for each of these different costs is given as

$$y = a + bx^c + du^e + fxu \tag{3.49}$$

where y = cost of a particular component, \$/ft
x = design parameter (for example, pipe diameter)
u = indicator variable

The variables *a, b, c, d, e,* and *f* are estimated using regression techniques. Table 3.5 gives the values of these parameters for selected pipe types.

The aggregated cost of construction can be determined in an additive manner by considering the construction cost components involved (Clark et al., 2002). Thus, considering the cost of using a ductile iron pipe (base installed, with push-on joints) and the cost of the major construction activities involved, including trenching and excavation, embedment, backfill, and compaction, and valve fitting and hydrant placement, the total aggregated cost can be estimated by

$$y_{\text{total}} = 50.74 + 0.33x^{1.72} + 0.32x^{0.67} + 0.26x^{1.46} - 0.062x^{0.73} + 0.02x^{1.79} + 0.16x \tag{3.50}$$

where y_{total} is the aggregated construction cost (\$/ft) and x is the pipe diameter (in). To use Eq. (3.50) in the model, the diameter is expressed in terms of the flow rate in the pipe because the flow rate, rather than the pipe diameter, is used in the model. Also, the total distance of construction must be determined. Using the equation of continuity and an average design velocity of 5 ft/s, the diameter is expressed in terms of Q as $x = 6.06Q^{0.5}$ from which Eq. (3.51) is obtained.

$$y_{\text{total}} = 50.74 + 7.32Q^{0.860} + 1.07Q^{0.335} + 3.61Q^{0.730} - 0.23Q^{0.365} + 0.50Q^{.895} + 0.97Q^{0.5} \tag{3.51}$$

Therefore, if L_{ij} is the length of construction from the diversion node in reach i to activity point j, the total construction cost $\text{CC}(Q_{ijp})$ for a water supply line in reach i for activity j during planning period p can be given as

$$\text{CC}(Q_{ijp}) = L_{ij}(50.74 + 7.32Q_{ijp}^{0.86} + 1.07Q_{ijp}^{0.335} + 3.61Q_{ijp}^{0.73} - 0.23Q_{ijp}^{0.365} + 0.50Q_{ijp}^{0.895} + 0.97Q_{ijp}^{0.5}) \tag{3.52}$$

Although Eq. (3.52) can be used as an estimate for construction costs, such costs may change with time. In fact, the original equations given in the sum of equations in Eq. (3.50) were derived based on the Producer Price Index (PPI) for 1997. However, such costs can be updated by the use of standard indices. For example, the PPI for 1997 is 358.5, whereas that for 2000 is 388.5, which yields a ratio of 1.08. Therefore, the above costs can be updated to 2000 by multiplying the 1997 costs by 1.08 (Clark et al., 2002).

The costs for pump, reservoir, and tank construction and operation can be given by an equation that is essentially similar to Eq (3.24) as (Clark et al., 2002):

$$y = a + bx^c + du^e + fxu \tag{3.53}$$

where y = construction cost (\$)
x = design variable
u = indicator variable

Parameters *a, b, c, d, e,* and *f* are determined from regression analysis. Table 3.6 gives values of these parameters for some selected activities.

Using the parameters given in Table 3.6, aggregated cost equations for construction, operation, and maintenance of pump stations and reservoirs are determined. Therefore, if a horizontal split case

TABLE 3.5 Parameters for Distribution System Component Cost Equations for Selected Pipe Types

Type of activity	Conditions	Pipe diameter range, in	Indicator variable u value	Parameter values						
				a	b	c	d	e	f	R^2
Ductile iron pipe	Base installed cost, with push-on joints	4–144	50, 52	−44.33	0.33	1.72	2.87	0.74	0.0	0.99
Trenching and excavation	Sandy gravel soil with 1:1 side slope	1. 4–8 2. 8–144	4, 6, 8, 10 ,12 4, 6, 8, 10, 12	−24.18 2.91	0.32 0.0018	0.67 1.90	16.66 0.13	0.38 1.77	0.0 0.0	0.99 0.98
Embedment	Concrete arch	4–144	—	7.11	0.26	1.46	0.0	0.0	0.0	0.99
Backfill and compaction	Sandy native soil with a 1:1 side slope	4–144	4, 6, 8, 10, 12	−0.094	−0.062	0.73	0.18	2.03	0.02	0.99
Valve fitting and hydrant	Medium frequency using ductile iron fitting	12–72	—	9.83	0.02	1.79	0.0	0.0	0.0	0.99
Horizontal boring	—	4–60	—	503.67	1.99	1.52	0.0	0.0	0.0	0.98

Source: Clarke et al. (2002).

TABLE 3.6 Parameters for Costs of Selected Pumps, Tanks, and Reservoirs

Type of activity	Conditions	Independent variable description and range	Indicator variable u values
Horizontal split case pumps	Includes suction discharge piping	Firm pumping capacity (0.5–200 mgd)	100, 200, or 300 ft of total dynamic head
Pump station site work	—	Firm pumping capacity (0.5–200 mgd) for extreme conditions	1 for average conditions and 2
Building installation for horizontal centrifrigal pump	—	Firm pumping capacity (0.5—200 mgd)	2 for a simple slab
Construction of electrical and instrumentation	—	Firm pumping capacity (0.5–200 mgd)	100, 200, or 300 ft of total dynamic head
Expansion for horizontal split case pump	Includes suction and discharge pumping	Additional pumping capacity (0.5–200 mgd)	100, 200, or 300 ft of total dynamic head
Expansion of site work	—	Additional pumping capacity (0.25–33 mgd) conditions	1 for average conditions and 2 for extreme
Expansion of electrical and instrumentation	—	Additional pumping capacity (0.25–12 mgd)	100, 200, or 300 ft of total dynamic head
Annual operation and maintenance for horizontal centrifugal pump installations	—	Firm pumping capacity (0.5–200 mgd)	100, 200 or 300 ft of total dynamic head

Source: Clarke et al. (2002).

TABLE 3.6 Parameters for Costs of Selected Pumps, Tanks, and Reservoirs (*Continued*)

Parameter values						
a	*b*	*c*	*d*	*e*	*f*	R^2
33,246.43	15,615.82	0.99	0.0	—	22.78	0.99
−14,695.02	25,947.34	0.42	0.28	1.00	0.0	0.99
225,500.9	8,629.99	0.84	−116,177.1	0.88	−1,449.77	0.99
24,953.27	40,018.32	0.62	0.072	2.20	33.93	0.91
12,766.78	23,136.77	1.06	0.18	1.81	0.0	0.82
−26,101.60	31,414.44	0.04	159.61	0.0	0.0	0.98
−31,272.00	7,902.90	0.93	9,500.48	0.34	9.92	0.90
10,094.42	1,369.70	0.97	0.08	1.69	135.00	0.99

pump with suction and discharge piping with 200 ft total dynamic head is selected, the aggregated construction cost can be computed as the sum of the base construction cost, the pump station site work, the building cost, and the construction cost for electricity and instrumentation. Considering these costs, we obtain the following equation:

$$CC(Q_{ijp}) = 63{,}506.35 + 23{,}639.68Q_{ijp}^{0.95} + 37{,}931.97Q_{ijp}^{0.87} + 12{,}451.92Q_{ijp}^{0.84} + 52{,}454.49Q_{ijp}^{0.62} + 13{,}062.42Q_{ijp} \quad (3.54)$$

The annual operation and maintenance cost for the selected type of horizontal centrifugal pump installations can also be determined by referring to Table 3.6. For a pump station of firm capacity of 0.5 to 200 mgd and total dynamic head of 200 ft, this cost is obtained as

$$OC(Q_{ijp}) = 10{,}713.62 + 2091.67Q_{ijp}^{0.97} + 41{,}775.18Q_{ijp} \quad (3.55)$$

The expansion cost for the selected type of pump can be determined as the aggregate of the expansion costs of the pump, the site work, the building, and the extension of electrical power and instrumentation. The parameters of these costs are obtained from Table 3.6.

$$EC(Q_{ijp}) = 27{,}151.70 + 36{,}747.74Q_{ijp}^{1.06} + 31{,}967.71Q_{ijp}^{0.04} + 11{,}064.73Q_{ijp}^{1.07} + 11{,}859.66Q_{ijp}^{0.93} - 7935.95Q_{ijp} \quad (3.56)$$

where $EC(Q_{ijp})$ is the total construction cost for a supply line in reach i for activity j during time p of the planning period. Expansion costs for pipelines depend on the approach used in the process, which include pipe bursting, microtunneling, and horizontal directional drilling. For this example, pipe bursting, a method for replacing pipe by bursting from within while simultaneously pulling in a new pipe (Selvakumar et al., 2000), is selected. The liner pipe can be the same size or as much as two pipe sizes larger than the existing pipe (Selvakumar et al., 2000). Boyce and Bried (1998) estimate a range of \$7 to \$9 per inch diameter for installed costs for pipe replacement. Thus, an approximate cost equation for pipe replacement is obtained as

$$EC(Q_{ijp}) = 8(12)\,(6.06Q^{0.5})L_{ij} = 581.76Q_{ijp}^{0.5}L_{ij} \quad (3.57)$$

3.4 MODEL RESULTS

3.4.1 Static Model

The formulation given is prepared as the input code for the GAMS/MINOS solver (Brook et al., 1992), and the results shown in Tables 3.7 and 3.8 are obtained, which give a net objective value of \$434.12 million. A summary of the major breakdown of the costs and benefits of this project in the major water-use districts is given in Table 3.8, whereas Fig. 3.11 gives its pictorial illustration. The results clearly show that the damage from the TDS is more pronounced in the irrigation districts than in the cities and towns. The TDS damages to the cities and towns can be considered practically negligible when compared to the gross income from the cities and towns.

The solution of the problem depends on the initial values of some of the parameters in the model. As a result, different local maximum points were obtained depending on the initial conditions used. The most important parameter among these appeared to be the initial value of the river flow. Using different values of the initial river flow (from 400,000 to 850,000), the result in Table 3.9 was obtained. A comparison of the calculated TDS values at each of the nodes from 1 to 13 with 30-year available data at these nodes is given in Table 3.10 and Fig. 3.12.

The sensitivity analysis to the most important factor in this project, the average annual water supply, shows two important trends (see Fig. 3.13). First, even when there is more than enough

TABLE 3.7 "Best" Optimal Solution Obtained

Objective function value, million $	Water supply to the irrigation districts, acre·ft/yr			Water supply to the cities/towns, acre·ft/yr		
	Area name	Diversion	Pumpage	City/town	Diversion	Pumpage
434.12	Percha	75,000	30,000	Hatch	0	576.682
	Leasburg	100,000	40,000	Las Cruces	0	11,201.686
	Mesilla (right)	75,000	30,000	Anthony	0	1,153.389
	Mesilla (left)	75,000	30,000			
	El Paso (reach 12)	83,333.333	N/A	El Paso	13,028.762	55,000.000
	El Paso (reach 15)	416,666.667	N/A			

Note: N/A = not applicable.

TABLE 3.8 Breakdown of the Benefits and the Damages for the "Best" Optimal Solution Obtained

Benefit/cost type	EBID	EPCWID No. 1	NM cities/towns*	El Paso
Gross benefit, $	17,290,000	11,319,000	130,815,109	321,790,790
Water cost, $	4,225,000	4,500,000	4,202,821	24,437,587
TDS damage, $	4,120,457	3,416,011	150,000	2,036,815
Net benefit, $	8,944,543	3,402,989	126,462,288	295,316,388
Total net benefit, $	434,126,208			

*These include Hatch, Las Cruces, and Anthony.

water in the river, the TDS mixing dynamics make the total damage higher when the surplus flow in the river is low. This is due to the fact that the drainage water from the agricultural areas always carries higher TDS than the water flow in the river. The marginal increase in the objective function versus the annual available flow in the river, shown roughly on the right half of Fig. 3.13, illustrates the dilution effect of the river water on the drainage water. The other half of the figure shows the dilution effects plus the marginal increase due to agricultural activity as a function of the marginal increase in the acreage.

3.4.2 Seasonal Model

The seasonal model formulation was also prepared as a GAMS/MINOS input code, which was solved using the latest release (version 2.50) of GAMS/MINOS software. The optimal solution obtained is given in Table 3.11. The objective function value obtained for this solution is $435.43 million. The optimal seasonal releases from Caballo Reservoir are obtained as 19,188, 128,648, 555,022, and 87,141 acre·ft for the first, second, third, and fourth seasons, respectively. The TDS concentrations observed in the reaches did not show much disparity among the seasons.

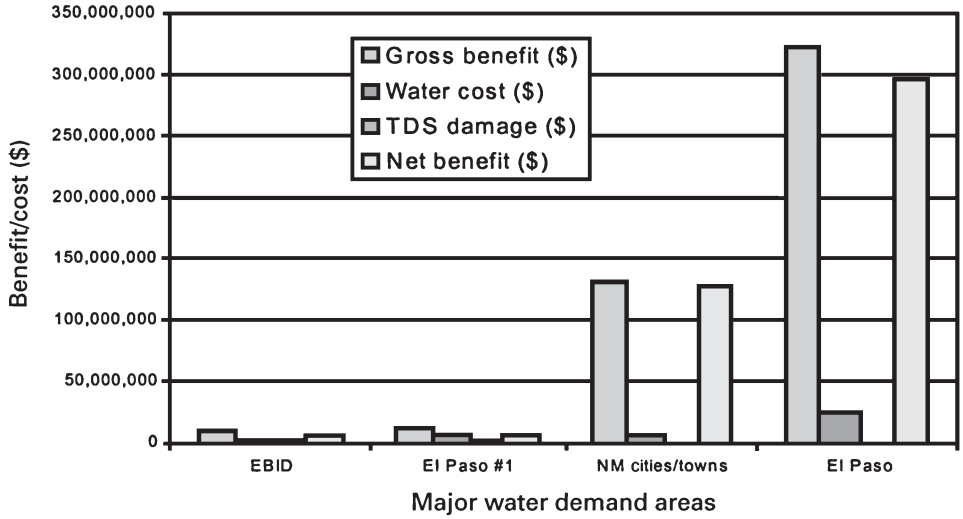

FIGURE 3.11 Comparison of the major benefits and costs for the "best" optimal solution.

TABLE 3.9 Sensitivity Analysis of the Net Benefit Result to Annual Available Flow

Reservoir release, acre·ft/yr	Benefit, million $	Reservoir release, acre·ft/yr	Benefit, million $	Reservoir release, acre·ft/yr	Benefit, million $
400,000	429.78	560,000	431.91	690,000	433.47
410,000	429.91	570,000	432.05	700,000	433.54
430,000	430.17	580,000	432.18	710,000	433.61
450,000	430.44	590,000	432.32	720,000	433.68
460,000	430.57	600,000	432.45	730,000	433.75
470,000	430.71	610,000	432.59	740,000	433.81
480,000	430.84	620,000	432.72	750,000	433.88
490,000	430.97	630,000	432.84	760,000	433.94
500,000	431.11	640,000	432.97	770,000	434.00
510,000	431.24	650,000	433.09	790,000	434.12
520,000	431.38	660,000	433.22	810,000	434.23
540,000	431.64	670,000	433.31	830,000	434.34
550,000	431.78	680,000	433.39	850,000	434.45

3.4.3 Capacity Expansion Model

The seven major water demand areas of El Paso County's water supply system were combined to form a total of three major water demand regions. Herein, the Northwest and Northeast areas are combined to form region 1; the Central, Fort Bliss, and Hueco areas are combined to form region 2; and Lower Valley and the East areas are combined to form region 3. The estimates of the population

TABLE 3.10 Observed 30-Year Average TDS Data and TDS Data Determined Using the Model

Reach	Calculated TDS, ppm	30-year average TDS, ppm
1	450	487
2	450	534
3	494	554
4	544	581
5	544	589
6	636	589
7	631	592
8	631	724
9	787	750
10	787	873
11	809	1037
12	850	1036
13	850	1020

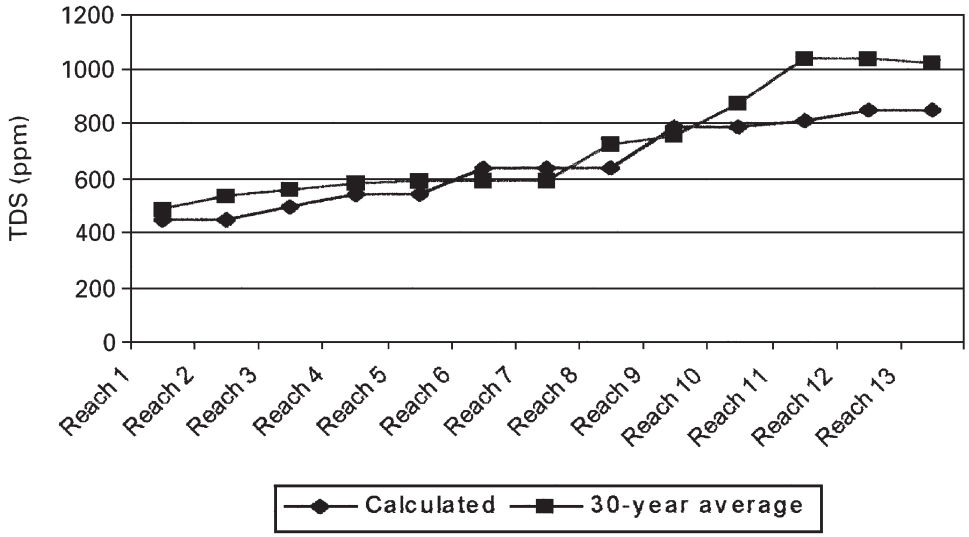

FIGURE 3.12 Graph showing the calculated and 30-year average observed TDS values.

and the households in each region are given in Tables 3.12 and 3.13, respectively. For the purpose of this application, a planning period of 10 years is considered. Therefore, the total number of possible combinations of expansion times equals $(10 - 1)^6 = 531{,}441$. The problem envisages minimizing the total cost over the 10-year planning period.

Using Eqs. (3.51), (3.52), and (3.54) to (3.57) in the problem formulation given by Eq. (3.35), the MINLP problem is solved. The minimum yearly water demands for El Paso for the years 2001 to

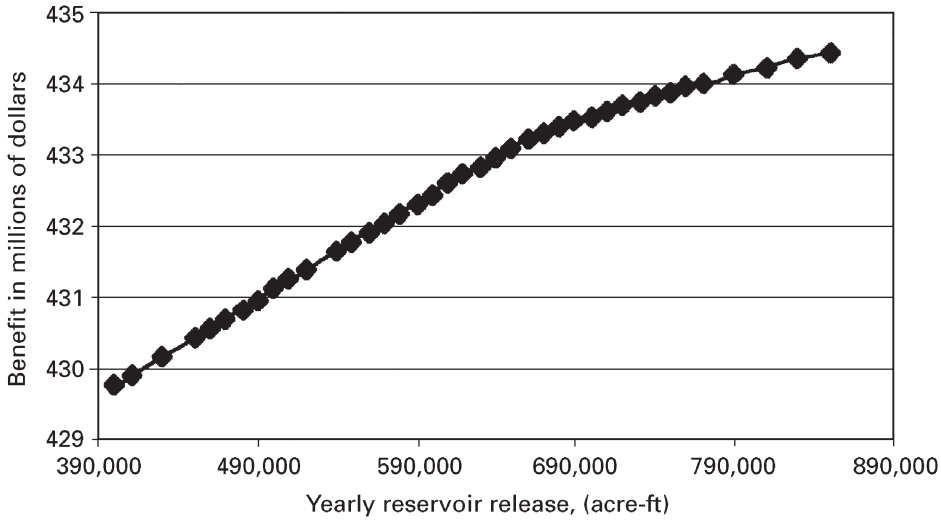

FIGURE 3.13 Net benefit as a function of the annual available flow.

TABLE 3.11 The "Best" Solution Obtained for the Seasonal Model

		Agriculture (left)		Agriculture (right)		Municipal		Industrial	
Reach	Season	Div.	Pump	Div.	Pump	Div.	Pump	Div.	Pump
1	1					0	86		
	2	7,500	22,500			0	143		
	3	54,750	7,500			50	150		
	4	12,750				0	143		
4	1					0	1,680	0	1,625
	2	0	0			0	2,800	0	1,625
	3	0	0			0	3,920	1,045	580
	4	0				0	2,800	0	1,625
7	1								
	2	7,500	22,500	7,500	22,500				
	3	54,750	7,500	54,750	7,500				
	4	12,750		12,750					
9	1					0	172		
	2					0	286		
	3					100	300		
	4					0	286		
12	1					0	10,225	8,513	3,525
	2	8,333				3,292	13,750	12,038	0
	3	60,833				10,109	13,750	12,038	0
	4	14,167				3,292	13,750	12,038	0
15	1								
	2	41,667							
	3	304,167							
	4	70,833							

TABLE 3.12 Estimated Population of Each of the Regions

Year	Region 1	Region 2	Region 3
2001	247,757	177,107	342,685
2002	253,617	177,679	351,613
2003	259,477	178,252	360,542
2004	265,337	178,824	369,470
2005	271,197	179,397	378,398
2006	277,056	179,970	387,326
2007	282,916	180,542	396,254
2008	288,776	181,115	405,183
2009	294,636	181,687	414,111
2010	300,496	182,260	423,039

TABLE 3.13 Estimated Number of Households in Each Region

Year	Region 1	Region 2	Region 3
2001	82,586	59,036	114,228
2002	84,539	59,226	117,204
2003	86,492	59,417	120,181
2004	88,446	59,608	123,157
2005	90,399	59,799	126,133
2006	92,352	59,990	129,109
2007	94,305	60,181	132,085
2008	96,259	60,372	135,061
2009	98,212	60,562	138,037
2010	100,165	60,753	141,013

2010, which are used in the constraints, were determined from a different study by considering supply–benefit analysis. Table 3.14 gives the optimum (minimum) demand by the customers. The maximum yearly groundwater volume that can be pumped per year for each region was limited to 55,000 acre·ft. In the formulation, the length of the pipelines from the diversion points to each of the regions was assumed to be 10 mi each.

The optimal capacity expansion schedule was obtained by solving the above MINLP model. The computed expansion times for the pipelines and the pumping stations to regions 1, 2, and 3 were found to be 9, 8, 9, and 2, 4, 10, respectively. It may be noted that the value 10 for the pumping station to region 3 indicates that there is no expansion necessary and the initial capacity should be built so that it serves during the entire planning period. The total construction, operation, and expansion costs were found to be \$136.20, \$48.04, and \$498.26 million, respectively, with the overall total minimum cost of \$682.50 million. In obtaining the above result, the termination message report by GRG2 was "Termination Criterion Met. Kuhn-Tucker Conditions Satisfied to Within [EPSTOP] at Current Point." This is the best termination message because it ensures that

TABLE 3.14 Minimum Water Supply Requirements for El Paso County

	Region 1		Region 2		Region 3	
Year	Surface water	Groundwater	Surface water	Groundwater	Surface water	Groundwater
2001	84,806	55,000	45,051	55,000	138,677	55,000
2002	88,124	55,000	45,289	55,000	143,222	55,000
2003	91,602	55,000	45,527	55,000	148,969	55,000
2004	94,722	55,000	46,008	55,000	154,017	55,000
2005	97,976	55,000	46,251	55,000	158,240	55,000
2006	101,376	55,000	46,494	55,000	163,764	55,000
2007	104,930	55,000	46,984	55,000	168,394	55,000
2008	108,017	55,000	47,231	55,000	173,224	55,000
2009	111,226	55,000	47,479	55,000	178,268	55,000
2010	114,564	55,000	47,979	55,000	183,540	55,000

the necessary conditions have been satisfied at the current point and thus the current point may be a local optimum point.

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