Cumberland County
Regional Water Supply
Preliminary Engineering Report

Prepared For:

Cumberland County, TN

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by

United States Army Corps of Engineers

Nashville District

and

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Nashville, TN 37211

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EXECUTIVE SUMMARY

This Preliminary Engineering Report (PER) serves the purpose of a reconnaissance or pre-feasibility study. It is a preliminary study of the existing water supply conditions of Cumberland County and a preliminary investigation of water supply alternatives to supplement Cumberland County's existing water supply. The study was requested by Cumberland County.

An assessment of the county water supply needs was followed by data collection and a minimum level of field work with respect to topographic surveys, and soil and geologic investigations. Four possible growth scenarios were assumed which provide growth patterns based on (1) restricted growth, (2) historical growth, (3) a moderate increase to the historical trend, and (4) an unlimited increase to the historical trend. This estimated range of the future demand was used to provide a perspective of the feasibility of several different alternatives to supply additional water to the county. During a Feasibility Study / EIS process, a detailed Needs Assessment would be completed to define a single target water supply need. Table 1 provides a summary of the preliminary needs assessment.

Table 1
Results of Preliminary Needs Assessment

Growth Scenario	Preliminary Predicted Demand in 2050 (MGD)	Additional Water Supply Required in 2050 (MGD)
Limited	7.3	No Need
Historical	10.9	0.9
Median	13.1	3.1
Unlimited	68.3	58.3

[1] - Additional water supply required based on an estimated existing capacity of 10 MGD.

Six water supply alternatives were investigated:

- **Water Conservation**
- **Groundwater**
  - Five well sites located in Fentress and Overton Counties
- **Pipeline to large reservoir**
  - Watts Bar Lake
  - Center Hill Lake
  - Great Falls Lake
- **Storage Impoundments (New)**
  - Caney Fork
  - Meadow Creek (above Monterey Lake)
  - Meadow Creek (below Monterey Lake)
  - Meadow Park Lake (below existing dam)
  - Clear Creek

Storage Impoundments (Improvements to Existing)

Meadow Park Lake
Mayland Lake
Camp Ozone Lake
Tranquilechee Lake

- **Water Harvesting**

  Traditional Method – Stream to constructed off-site impoundment
  From Caney Fork to Meadow Park Lake

- **No Action**

Each alternative considered was sized to its maximum capacity for providing water supply. For those alternatives that require a target yield to be designed, such as a pipeline, 9 million gallons per day (MGD) was used as the target yield. This value was selected based on a previous water supply study the Tennessee Valley Authority (TVA) had performed. The selection of 9 MGD was made prior to the completion of this study's preliminary Needs Analysis so that the alternative analyses could begin in order to meet the study's short schedule. While 9 MGD may or may not be the required yield for the County, using it for all alternatives that required a target yield provided an equitable comparison of each.

Preliminary benefit-cost and financial analyses and environmental screening of the various identified solutions were included in this study. A summary of the preliminary alternative analysis, cost estimate and environmental screening is provided in Table 2.

It is the responsibility of the Cumberland County community and utility districts to refine and expand on the data contained herein through a detailed feasibility study, if so desired. Figure 1 describes the steps toward construction and final operation of a new water supply source(s). Possible options for proceeding into the feasibility phase include:

Option 1 – U.S. Army Corps of Engineers

Work with a Congressional delegation to seek appropriation and authority for the U.S. Army Corps of Engineers (Corps of Engineers) to fully investigate and provide a viable water supply to the Cumberland County region.

Option 2 - Support from Others

Obtain funding from other "outside" sources, such as Rural Utility Service for example. Under this option, the Corps of Engineers could provide planning and design services, as well as construction management on a cost reimbursable basis, if so desired.

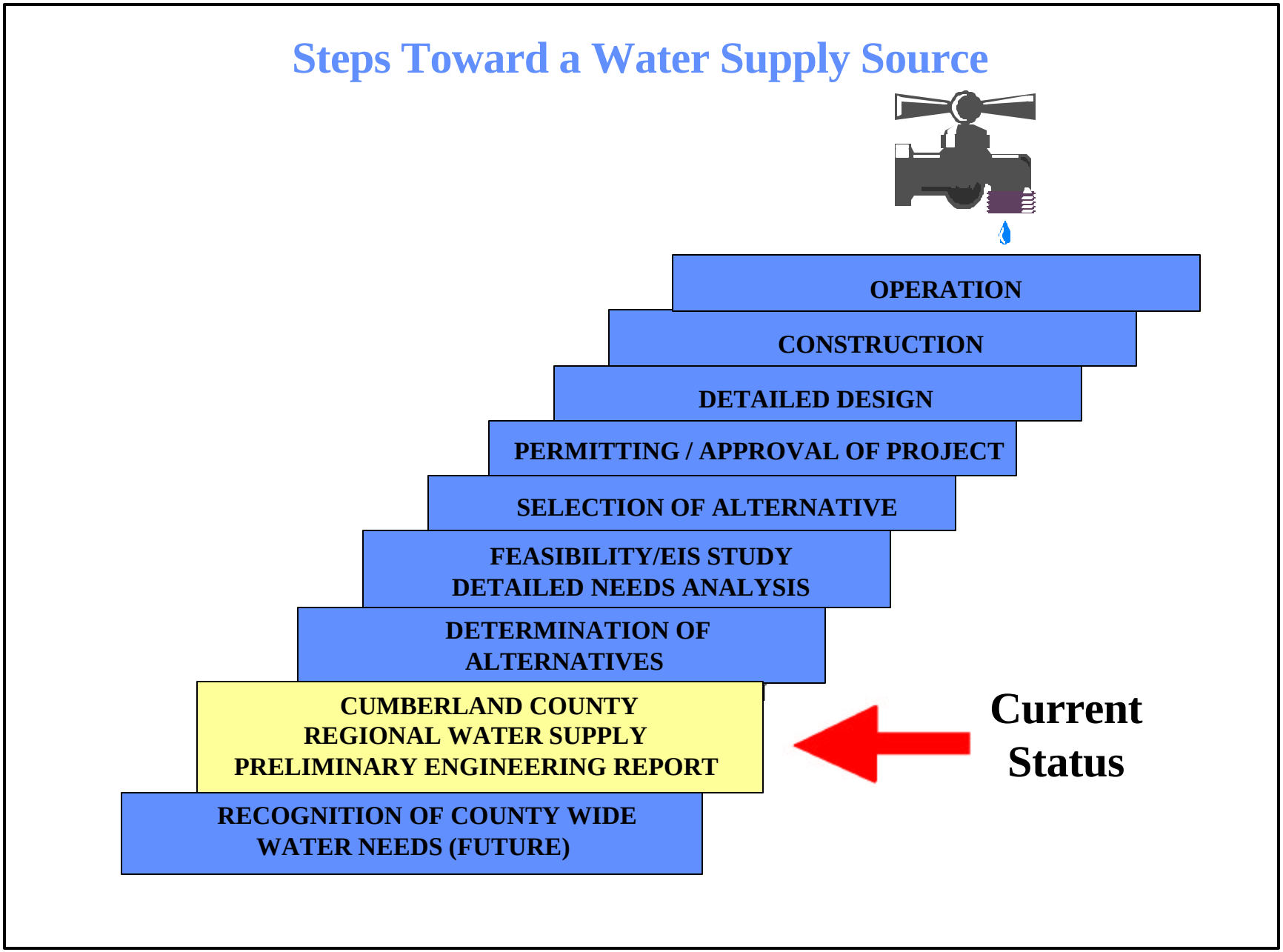

Figure 1 – Steps toward a Water Supply Source(s)

Table 2
Alternative Summary Table

Alternative	Safe Yield (MGD)	Estimated Cost (Million Dollars)	Estimated Cost Per MGD (Million Dollars)	Assessment		Evaluate in Detail in EIS
				Engineering	Permit Actions	
Water Conservation				Positive	Positive	Yes
Groundwater	10.7[1,2]	$51.6	$5.7	Neutral	Positive	Yes
Large Scale Pipeline						
to Center Hill Lake	9.0[2]	$38.4	$4.3	Positive	Positive	Yes
to Dale Hollow Lake	N/A[3]	N/A[3]	N/A[3]	Negative	Positive	No
to Great Falls Lake	9.0[2]	$33.5	$3.7	Positive	Positive	Yes
to Watts Bar Lake	9.0[2]	$27.6	$3.1	Positive	Positive	Yes
Improvements to Existing Reservoirs						
Meadow Park Lake	4.0	N/A[4]	N/A[4]	Negative	Neutral	No
Mayland Lake	2.0	N/A[4]	N/A[4]	Negative	Neutral	No
Camp Ozone Lake	1.0	N/A[4]	N/A[4]	Negative	Neutral	No
Tanquilichee Lake	N/A[5]	N/A[5]	N/A[5]	Negative	Neutral	No
New Impoundments						
Clear Creek	3.0	$28.4	$9.5	Positive	Negative	No
Meadow Creek (above Meadow Creek Lake)	N/A[6]	N/A[6]	N/A[6]	Negative	Negative	No
Meadow Creek (below Meadow Creek Lake)	7.0[7]	$55.7	$7.9	Positive	Negative	No
Meadow Park Lake	N/A[8]	N/A[8]	N/A[8]	Negative	Negative	No
Caney Fork	12.0	$63.5	$5.3	Positive	Negative	No
Water Harvesting						
Traditional Water Harvesting	0.8[9]	$19.1	$23.9	Positive	Positive	Yes
Caney Fork to Meadow Park Lake	8.0[10]	$42.7	$5.3	Positive	Neutral	Yes

Positive Assessment - Based on the results of this preliminary study, the alternative appears to be constructable from an engineering basis or poses minor enough impacts to the environment to make permitting likely.
Neutral Assessment - Based on the results of this preliminary study, the design of the alternative contains unvalidated assumptiopns from an engineering basis or poses impacts to the environment that would make permitting difficult.
Negative Assessment - Based on the results of this preliminary study, the design of the alternative is unfeasible from an engineering basis or poses severe impacts to the environment that would make permitting unlikely.
1 - The design of the groundwater alternative contains several assumptions that will need validation during a feasibility-level study.
2 - Safe yield of 9.0MGD was used as the design yield for this alternative. This is not the recommended, maximum or minimum capacity for this alternative. Refer to Section 1.0.
3 - The pipeline to Dale Hollow Lake was determined to be economically unfeasible due to its longer length that the other routes. A preliminary design and cost estimate were not done for this study. Refer to Section 4.5.
4 - No preliminary cost estimates were prepared for the improved reservoir alternatives due to the fact the alternatives would not provide a significant amount of water supply. Refer to Section 4.7.6.
5 - The safe yield from the raising the height of Tranquilichee Lake was not determined due to the relatively small size and narrowness of the valley the raised reservoir would fill. Refer to Section 4.7.5.
6 - Due to an existing water supply dam downstream (Meadow Creek Dam), a preliminary design and cost estimate were not completed for a new impoundment on Meadow Creek above the existing dam. Refer to Section 4.8.2
7 - The safe yield of 7.0MGD for the new Meadow Creek impoundment below the existing Meadow Creek Dam includes the water supply of the existing reservoir, which provides water to the City of Monterey. Refer to Section 4.8.3.
8 - A new dam downstream of and higher than the existing Meadow Park Lake was removed from consideration due to the watershed's drainage area being to small to support a larger reservoir. Refer to Section 4.8.4.
9 - The safe yield of 0.8MGD for the traditional water harvesting alternative is per detention basin as designed. Modifications to the design could increase or decrease the safe yield, but would also affect the cost estimate. Refer to Section 4.9.2.
10 - The safe yield of 8.0MGD for the water harvesting from the Caney Fork to a raised Meadow Park Lake is provided by the alternative as designed. Modifications to the design could increase or decrease the safe yield, but would also affect th
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Table 2
Alternative Summary Table

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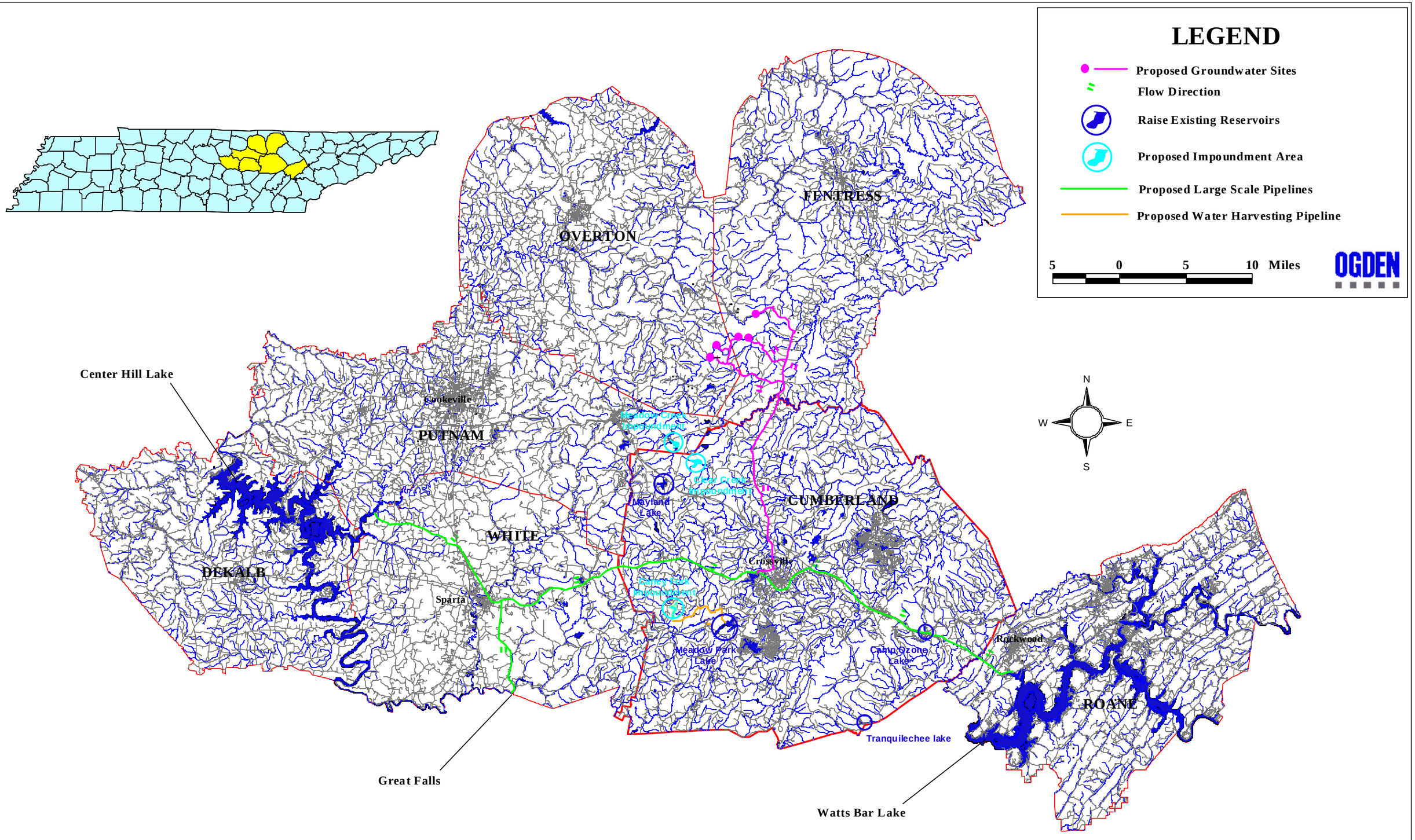

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1.0 GENERAL INFORMATION

This regional water supply study for Cumberland County (see Figure 1-1) is a pioneer project in both the regional methodology and cost share funding. Typically, water supply studies are performed by an individual utility district in search of an adequate water supply source for their respective district needs. For this regional study, the water supply needs for all Cumberland County residents were investigated with the intent of identifying the feasibility of a variety of water supply alternatives.

The project is funded jointly through federal and state assistance. The U.S. Army Corps of Engineers (Corps of Engineers) a participant in the federal program "Planning Assistance to States" matched funds supplied by the State of Tennessee Department of Environment and Conservation (TDEC) to Cumberland County. In addition, the National Parks Service offered to contribute funds.

Recent water supply needs within Cumberland County have been highlighted in *The Nashville Tennessean* and *The Crossville Chronicle*. Articles focused on the current water supply needs in portions of middle and east Tennessee and the troubles specific utility districts are having with obtaining new water supply sources and completing water distribution expansion projects within the respective districts. This regional water supply project is conceived by the Commissioner of the Tennessee Department of Environment and Conservation, Milton H. Hamilton, Jr., to be a model for counties trying to balance rapid economic growth while protecting natural resources.

The first step in this regional water supply study was the preliminary Needs Assessment. Field visits were made to Cumberland County to interview and collect historical water usage data from each of the six utility districts. Four possible growth scenarios were assumed which provide growth patterns based on (1) limited growth, (2) historical growth, (3) a moderate increase to the historical trend (median growth), and (4) an unlimited increase to the historical trend. The intent of this estimated range of the future demand is to provide a perspective of the feasibility of several different alternatives to supply additional water to the County.

The second step was the selection of a variety of water supply alternatives which included data collection and a minimum level of field work with respect to topographic surveys, and soil and

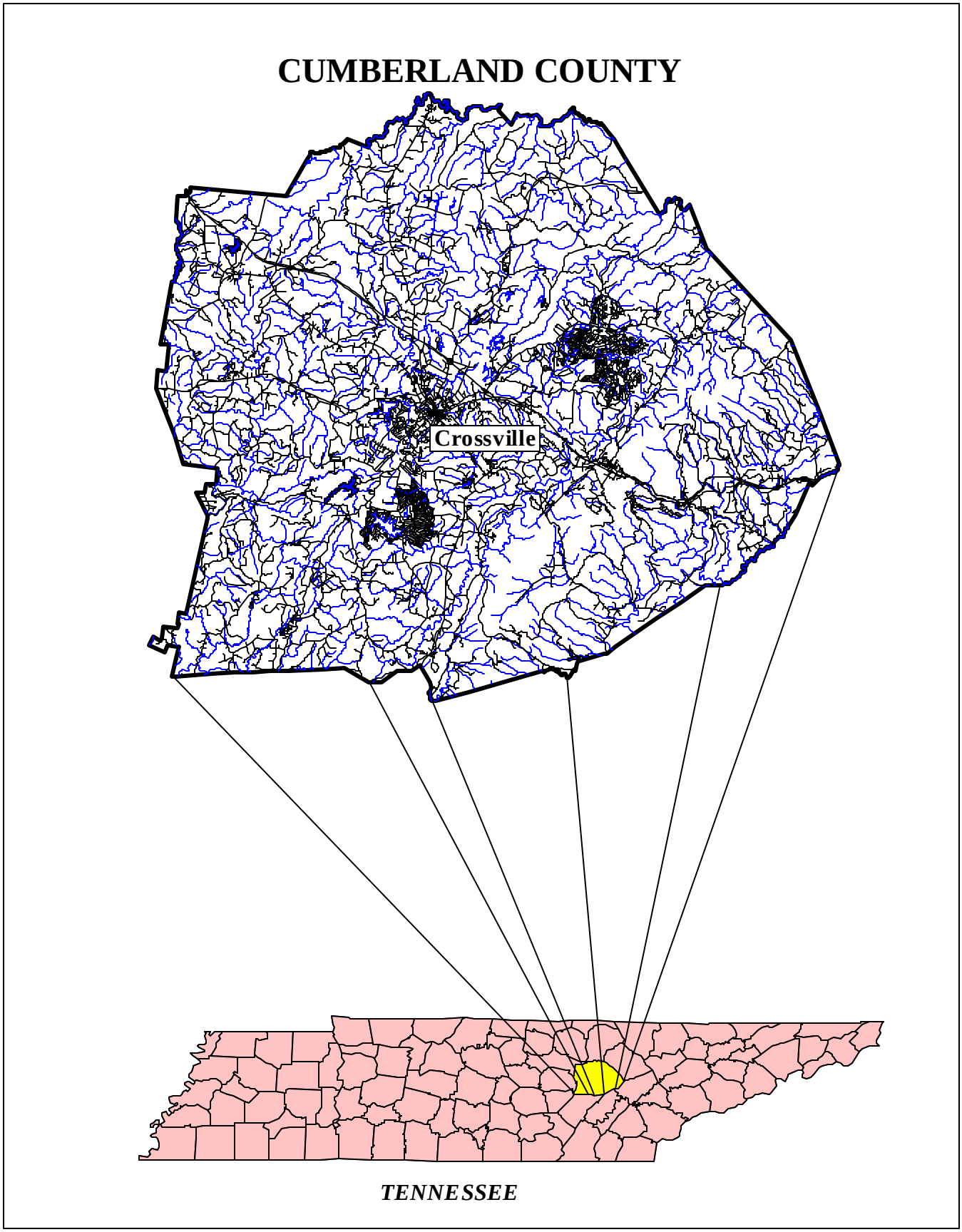


Figure 1-1 Location Map

geologic investigations. Six water supply alternatives were investigated including (1) water conservation; (2) groundwater; (3) large scale pipelines; (4) storage impoundments; (5) water harvesting; and (6) no action. Each alternative considered was sized to its maximum capacity for providing water supply. For those alternatives that require a target yield to be designed, such as a pipeline, 9 million gallons per day (MGD) was used as the target yield. This value was selected based on a previous water supply study the Tennessee Valley Authority (TVA) had performed. The selection of 9 MGD was made prior to the completion of this study's preliminary Needs Analysis so that the alternative analyses could begin in order to meet the study's short schedule. While 9 MGD may or may not be the required yield for the County, using it for all alternatives that required a target yield provided an equitable comparison of each.

On September 29th, 1998, meetings were held with (1) environmental interest groups and (2) local utility districts and their respective engineers to present the results of the Needs Assessment and to present the water supply alternatives that were to be investigated in this study. Comments and input from meeting attendees were encouraged in an effort to include the expressed interest of Cumberland County residents and keep the focus of the study on the goals of the Cumberland County community.

This Preliminary Engineering Report (PER) serves the purpose of a reconnaissance or pre-feasibility study. Included herein is a complete account of the preliminary Needs Assessment and development of the potential growth scenarios. Each water supply alternative is described in detail including locations and details of the engineering modeling processes. The feasibility of each alternative is also discussed. Additionally, a preliminary cost and financial analysis and environmental screening of the various identified water supply sources are presented.

It is the responsibility of the Cumberland County community and utility districts to refine and expand on the data contained herein through a detailed feasibility study, if so desired. To do this, there are two options for the County to choose from: (1) U.S. Army Corps of Engineers (Corps of Engineers) and (2) Support from Others. Option 1, Corps of Engineers, requires Cumberland County to work with a Congressional delegation to seek appropriation and authority for the Corps of Engineers to fully investigate and provide a viable water supply to the Cumberland County region. The Corps of Engineers would then perform a detailed feasibility study and begin the National Environmental Policy Act (NEPA) process. The second option, Support from

Others, involves obtaining funding from other "outside" sources, such as the Rural Utility Service for example. Under this option, the Corps of Engineers could provide planning and design services, as well as construction management on a cost reimbursable basis, if so desired.

The Cumberland County community could also elect to choose an alternative based on this Preliminary Engineering Report and proceed independently with the design and permit process. However, the difficulties likely to be encountered in pursuing this approach, without detailed feasibility planning, to include the provisions of the National Environmental Policy Act (NEPA), are well documented.

Currently, officials from the six utility districts are discussing the formation of a central administrative body for the purpose of managing the regional water supply needs of the Cumberland County region. In the short term, this group could possibly look to the surrounding communities, such as the City of Monterey, for potential connections with existing water supply systems.

This Preliminary Engineering Report is intended to aid the community in planning for the long-term regional water supply needs of the Cumberland County region.

2.0 BACKGROUND INFORMATION

Water use for municipal purposes is increasing in Cumberland County. Municipal water is the publicly-supplied water delivered to residential, industrial, and commercial users of the County including conveyance losses in the distribution system. Residential, industrial, and commercial water taps have increased approximately 5 percent annually since 1990. Projected residential, industrial, and commercial developments in the County suggest that water use is likely to continue to increase. A long term estimate of the water supply needs for the County would help determine the adequacy of existing water supplies to meet future demands.

Water distribution for Cumberland County residents is provided by six utility districts: Catoosa, Crab Orchard, Crossville, Grandview, South Cumberland, and West Cumberland. Catoosa Utility District serves the north and northwest portions of the County. Crab Orchard Utility District serves the eastern portion of the County. Crossville distributes water to the City of Crossville as well as County residents located just outside the city limits. Grandview Utility District, the smallest utility, serves the southeast portion of the County as well as the northern portion of Rhea and Bledsoe Counties. South Cumberland and West Cumberland Utility Districts serve their respective areas of the County (see Figure 2-1).

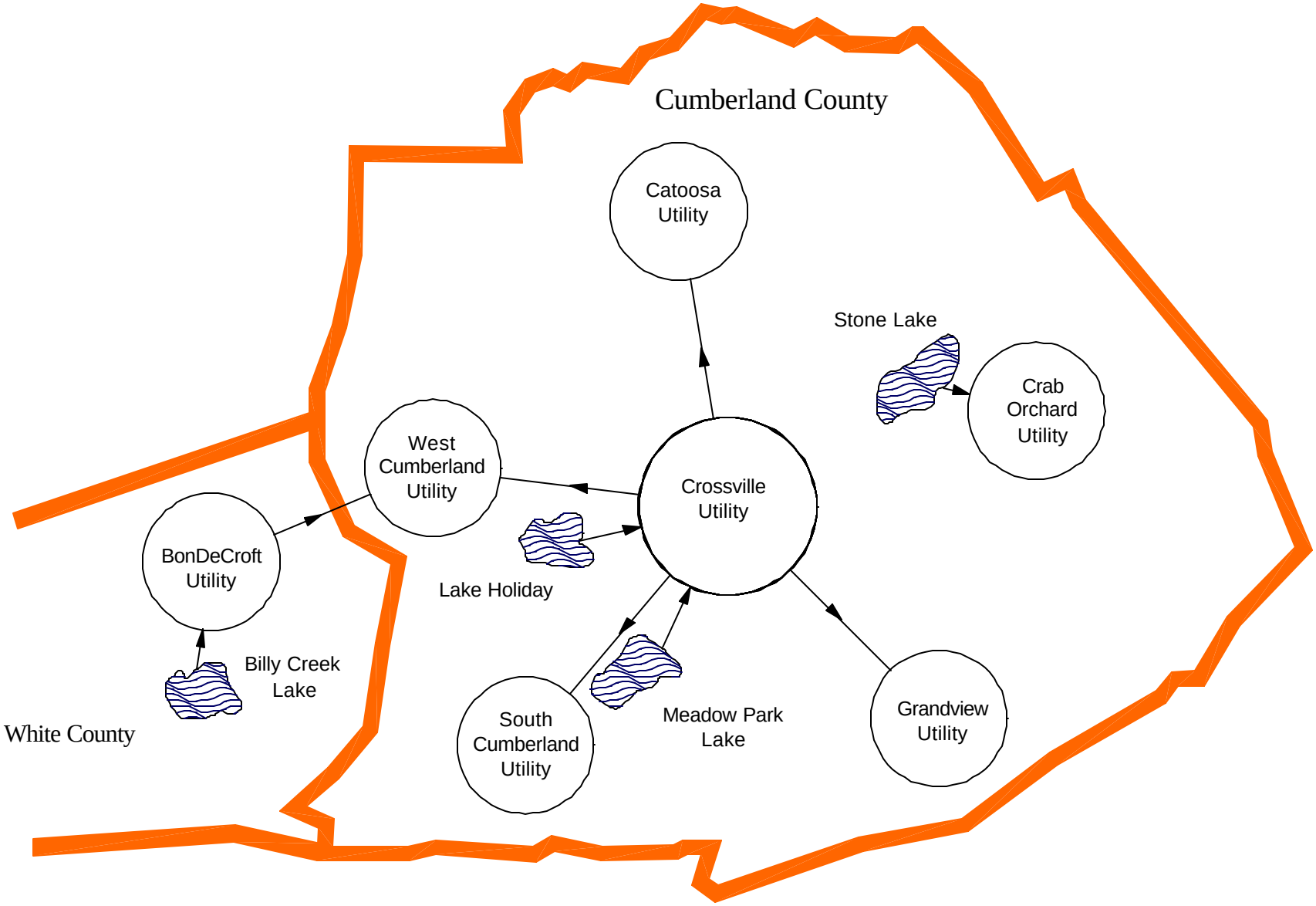
2.1 EXISTING TREATMENT CAPACITY

Meadow Park Lake and Lake Holiday are the two current water supply reservoirs for the City of Crossville. The City operates the two associated treatment plants rated at 1.4 million gallons per day (MGD) and 3.5 MGD, respectively. Treated water is supplied from the City to the Catoosa, Grandview, South Cumberland, and West Cumberland Utility Districts. West Cumberland also purchases water from the Bon de Croft Utility District of White County. Crab Orchard obtains all of its water from Stone Lake, which is fed by Otter Creek. The associated water treatment plant has a design capacity of 2.0 MGD.

2.2 EXISTING SAFE YIELD

Safe yield for this project was defined as the maximum continuous rate water could be drawn from each reservoir during severe drought conditions without lowering the water surface below the sediment storage pool. Preferential drawdown elevations for residential developments or

Figure 2-1 Utility Map



recreational uses were not acknowledged. The existing safe yields of Lake Holiday and Meadow Park Lake were calculated to be 4 MGD and 3 MGD, respectively. Calculations were made using the engineering software HEC1-API (see Section 4.6.5 for details). The existing safe yield for Stone Lake is 3.0 MGD, as given by the Crab Orchard Utility District. This value was not confirmed by similar safe yield modeling. The safe yield of Stone Lake would be confirmed during a Feasibility Study / EIS Process. Table 2-1 provides a summary of the current treatment capacity and safe yield at each of the water supply reservoirs.

Table 2-1
Existing Reservoir Characteristics

Reservoir	Existing Safe Yield	Existing Treatment Capacity
Lake Holiday	4	3.5
Meadow Park Lake	3	1.4
Stone Lake	3	2.0

The results of the safe yield analyses indicate Cumberland County has existing water supply sources totaling 10 MGD. However with the current infrastructure and operation techniques, the County does not have the capabilities to utilize the entire total. The inadequacies in the infrastructure and operating techniques that limit the County's potential to make use of all of its existing water supply are discussed below.

As mentioned in Section 2.1, there is only capacity in the County to treat a total flow of 6.9 MGD. Expansion of the existing treatment plants or construction of a new treatment plant would be necessary to provide water supply greater than 6.9 MGD.

Stone Lake provides 3.0 MGD of the total water supply. The existing infrastructure only allows access to that supply by those taps served by the Crab Orchard Utility. Additional pipelines and pump stations would be required to make the supply at Stone Lake accessible by the entire County.

The City of Crossville does not operate Lake Holiday to the same extent as modeled. Lake Holiday has a large amount of residential development along its banks. Operation of the

reservoir to its maximum water supply capacity would include fluctuation of the pool elevation up to 30 feet, which would expose unattractive scoured banks. This is aesthetically unpleasing to the residents that live along the lake. For this reason, the City of Crossville uses a preferential drawdown level of approximately 5 feet for water supply. Operating the reservoir in this manner reduces its safe yield.

3.0 PRELIMINARY NEEDS ASSESSMENT

3.1 WATER DEMAND PROJECTIONS

Traditionally, the purpose of a Needs Assessment is to estimate a single projected water demand that is used to size water supply alternatives. In this study, the preliminary Needs Assessment and preliminary alternative analysis will be more independent of each other than traditional water supply studies. The purpose of this preliminary Needs Assessment is to provide an estimated range of the future municipal water demand for the Cumberland County region. During a Feasibility Study / EIS process, a detailed Needs Assessment would be completed to define a single target water supply need.

The estimated range of the future demand, included herein, will be used to provide a perspective of the feasibility of several different alternatives to supply additional water to the County. Each alternative considered will be sized to its maximum capacity for providing water supply. For those alternatives that require a target yield to be designed, such as a pipeline, 9 MGD was used as the target yield. This value was selected based on a previous water supply study the TVA had performed. The selection of 9 MGD was made prior to the completion of the Preliminary Needs Analysis so that the alternative analyses could begin in order to meet the study's short schedule. While 9 MGD may or may not be the required yield for the County, using it for all alternatives that required a target yield provided an equitable comparison of each. The preliminary analysis of alternatives based on water conservation and/or no future action will provide a perspective on growth curves lower than those used in this Needs Assessment (see Section 4.3, Water Conservation). Attempting to predict future growth limits of the County and associated water demand by selecting a population and industry saturation point is beyond the scope of this study.

Four growth scenarios were selected which provide growth patterns based on (1) limited growth, (2) historical growth, (3) a moderate increase to the historical trend, and (4) an unlimited increase to the historical trend. Note that actual growth rates may also be less than the limited or historical rates. Estimates of the future municipal water demand were made for a 50 year period (Year 2050) from historical water usage data, socioeconomic characteristics of the resident and non-resident (industry) population, and seasonal variations in economic and climatic conditions of the area. Field visits were made to Cumberland County to interview and collect historical water usage data from each of the six utility districts. The type and quantity of historical data available at each district varied. Table 3-1 summarizes the available data.

Demographic characteristics such as population, housing, business income, industrial mix, and the price of water were obtained from the Upper Cumberland Local Planning Office, U.S. Census Bureau, the Tennessee Economic Development Center, the Tennessee Department of Employment Security, the Tennessee Housing Development Agency, and the aforementioned utility districts of Cumberland County. Climatic data for the area was obtained from the National Climatic Data Center.

Table 3-1
Available Historical Water Usage Data

Utility District	Year	Financial Data		Consumption Data (MGD)	Water Distribution Customers[2]
Catoosa	1978 to 1997	Total Water Revenue	Unit Cost of Water	N/A[1]	Total Number of Customers
Crab Orchard	1974 to 1995	Total Water Revenue	Unit Cost of Water	N/A	Total Number of Customers
	1995 to 1997	N/A	N/A	Monthly Water Consumption	Total Number of Customers
City of Crossville	1984 to 1997	N/A	N/A	Monthly Water Consumption	Total Number of Customers; Customer Classes
Grandview	1997	N/A	N/A	N/A	Total Number of Customers
South Cumberland	1977 to 1997	Total Water Revenue	Unit Cost of Water	N/A	Total Number of Customers
West Cumberland	1981 to 1994	N/A	N/A	Annual Water Consumption	Total Number of Customers
	1995 to 1997	N/A	N/A	Monthly Water Consumption	Total Number of Customers

1 N/A refers to data not available.
2 The term customer is used to define a tap on the water line, not an individual portion of the population.

3.1.1 Limited Growth Scenario

The first method of predicting growth to the water supply system is a limited growth scenario which limits growth in the community to 1.5 percent annually through the Year 2025 and 1.25 percent annually through the Year 2050. This limited percentage growth was selected to allow the community to grow at a gradual pace, slow in the final 25 years, and ease the demand on the current water supply system. This growth scenario would take a concerted effort by the County to curtail the current water consumer growth of approximately 5 percent annually. Cumberland County and the surrounding counties have population increases ranging from 5.6 to 21.1 percent from the Year 1990 to 1996. Those rural communities with populations similar to Cumberland County, between 30,000 and 40,000 residents, and growth rates less than 2 percent include

Obion and Weakley counties in western Tennessee. For comparison, the state of Tennessee has had a growth rate of 9.1 percent during this same time period.

An average daily "per tapita" water use was determined from 1978 to 1997 by dividing the total annual water consumption for the County by 365 days and by the total number of customers for each respective year. Note the term customer applies to a tap on the water line and is not equivalent to population value. Applying this average "per tapita" use to the projected number of customers, the predicted average annual water demand in the Year 2050 is 7.0 MGD for 36,640 customers (see Table 3-2). This predicted demand is less than the combined existing yield available from Lake Holiday, Meadow Park Lake, and Stone Lake of 10 MGD. An effort by Cumberland County residents to restrict growth in the community would not require the County to acquire an additional water supply source(s).

Table 3-2
Limited Growth Scenario

Year	Total	Avg. MGD	Peak MGD	Year	Total	Avg. MGD	Peak MGD
1980	3377	0.67	1.01	2020	24930	4.96	7.44
1990	7575	1.51	2.26	2030	28577	5.69	8.53
2000	18510	3.68	5.53	2040	32357	6.44	9.66
2010	21481	4.28	6.41	**2050**	**36637**	**7.29**	**10.94**

3.1.2 Historical Growth Scenario

Historically, the utility districts have grown due to (1) expansions on the distribution lines to existing Cumberland County residents (well users) and (2) additional taps to the distribution lines for new Cumberland County residents. Future water demand for this historical growth scenario was estimated as the product of projected customers and a projected value of daily "per tapita" water use. For this projection, the total number of customers was a combination of residential, industrial, and commercial users. The total number of customers for each utility district was projected separately using a linear best fit trend (developed from the historical data) for the next 50 year period (Year 2050). This projection, representing the historical steady increase in growth, allowed for the consistent annual growth of each utility as well as the irregular growth spurt of a new line extension to an existing community (existing well users). Again, an average "per tapita" water use, as defined in Section 3.1.1, was applied to the projected number of customers, the predicted average annual water demand in the Year 2050 is 10.4 MGD for 54,750 customers (see Table 3-3).

Table 3-3
Historical Growth Scenario
Linear Best Fit Trend

Year	Number of Customers						Total Customers	Avg. MGD[1]	Peak MGD
	Catoosa	Crab Orchard	West Cumber.	South Cumber.	Grand-view	Crossville			
1970									
1980	860	1413	467	637			3377	0.67	1.01
1990	1668	3330	908	1669			7575	1.51	2.26
1997	**2441**	**4733**	**1443**	**2560**	**443**	**6081**	**17701**	**3.52**	**5.28**
2000	2727	5119	1513	2850	487	6696	19392	3.86	5.79
2010	3768	6974	2060	3996	665	8812	26274	5.23	7.84
2020	4809	8828	2606	5142	910	10927	33222	6.61	9.92
2030	5849	10683	3153	6287	1245	13042	40259	8.01	12.02
2040	6890	12538	3699	7433	1702	15158	47420	9.44	14.16
2050	**7931**	**14392**	**4245**	**8579**	**2328**	**17273**	**54748**	**10.89**	**16.34**

1 Note: Average MGD was calculated from an average "per tapita" usage (see Section 3.1.1).

3.1.3 Median Growth Scenario (Moderate Growth Increase to Historical Trend)

The previous historical method of predicting future water use did not take into account socioeconomic factors such as income, housing stock, industrial mix, and the price of water. Relevant variables such as income measure the water user's ability to pay for water and sewer services, while price influences the amount of water the consumer is willing to pay for. Additionally, daily "per tapita" water use estimates may not accurately represent water use efficiency, and the types of water-using businesses and industries that exist in the County. Incorporation of these additional factors into a water demand projection results in a moderate increase to the historical trend with an estimated average annual demand of 13.1 MGD for the Year 2050. This estimate was derived using a water demand analysis software as discussed in detail as follows.

The Institute for Water Resources-Municipal and Industrial Needs System (IWR-MAIN) is a water demand analysis software which uses econometric water demand models for translating existing demographic, housing, and business statistics into estimates of existing water demands and contributions to sewer flows and uses projections of population, housing, and employment to derive baseline forecasts of water use. Version 6.1 of IWR-MAIN used in this study was prepared by Planning and Management Consultants, LTD., in cooperation with the Corps of Engineers, Institute for Water Resources, Metropolitan Water District of Southern California, Phoenix Water Services Department, and the Illinois Department of Transportation. The user's

manual and system description for IWR-MAIN provides a comprehensive description of the program.

IWR-MAIN forecast predictions are provided for residential, nonresidential, public, and other (unaccounted) sectors. The residential, nonresidential, and public sectors can be further disaggregated into subsectors. Water use is predicted for the winter and summer seasons and can be further disaggregated into indoor and outdoor dimensions of water use for each season.

Average rates of water use for the residential subsectors are estimated using causal water demand models with the following theoretical form:

$$Q = f(I, H, L, T, R, P, B)$$

where:

Q	=	predicted average household water use (gpd)
I	=	median household income
H	=	average household size (persons)
L	=	average housing density (units per acre)
T	=	average maximum daily temperature
R	=	rainfall, total for season
P	=	marginal price of water and wastewater
B	=	fixed charge or rate premium.

The theoretical model of non-residential water use for a given group/category, season, and dimension is:

$$Q = f(GED, L, P, CCD, O) * E$$

where:

Q	=	water use in gallons per day
GED	=	per employee water use in gallons per day
L	=	average labor productivity
P	=	marginal price of water and wastewater
CDD	=	cooling degree days, total for season
O	=	other (user added) variable effecting non-residential water use
E	=	employment.

For the purpose of this study, residential sectors were disaggregated into urban and non-urban subsectors. Non-residential sectors were divided into eight employment groups ranging from construction and manufacturing to retail and public service. Public/unaccounted for conveyance losses were assumed to be 10 percent of the total municipal water use. This coincides with information from the Cumberland County Utilities. While water usage may vary between Winter and Summer months, the marginal price of water was assumed to remain constant. Additionally, IWR-MAIN defaults to a 50/50 percent usage of indoor to outdoor water. This default value was amended to a conservative 75/25 percent usage of indoor to outdoor water.

A beginning or base year of 1990 was selected to coincide with available census data, 1997 was selected as a calibration year because existing water usage was known, and selected future years included 2010, 2025, and 2050. Required input for the water demand forecast included total occupied housing, total employment, average persons per household, and percent conveyance loss for the base year and each future year. For the base year, the total number of occupied housing was replaced with the total number of occupied housing on public water. Several residences within the County rely on well water as the sole water supply. Occupied housing for the future years was made using a linear best fit trend from the historical data of utility customers. This linear projection allowed for the consistent annual growth of utility customers as well as the irregular growth spurt of a new line extension to an existing community (existing well users). Total employment for the future years was assumed to remain a constant percentage of the occupied housing. Average persons per household was also assumed to remain constant at 2.55, as given in the 1990 census. This demographic value is similar to the 1994 and 1996 values of 2.57 and 2.52, respectively, as provided by the Tennessee Business and Economic Research Department.

Additional inputs for the base year included number of employees for each selected non-residential employment group, the number of occupied housing for each selected residential subsector, median income, housing density, the marginal price of water, and climatic data. Housing density, as a variable, is used to simulate irrigation practices in the area. For a rural community, although the housing density is low, residents most likely are not irrigating their entire rural estate. In order to maintain a typical outdoor water usage rate, the housing density was assumed to be 4 houses per acre.

Following an initial forecast run, the predictions made with the theoretical equations were verified against the 1997 calibration year for their accuracy. The IWR-MAIN estimates for the residential-water demand were higher than the observed data, indicating systematic errors for predicting actual water use. The Winter and Summer model constants representing gallons of water per household per day were adjusted to calibrate the model. This preliminary Needs Assessment is within approximately 20 percent of the actual demand values (2.97/3.82). As previously mentioned, this preliminary Needs Assessment would be revisited in detail during a Feasibility Study / EIS process. During the detailed analysis, additional data collection and further calibration of the IWR-MAIN modeling parameters would be conducted to obtain a more accurate historical match.

Due to insufficient data on the existing and historical water usage of the non-residential sectors of the County, the model could not be adequately calibrated in this manner. Therefore, IWR-MAIN estimates present a slightly higher non-residential water usage value. This is acceptable because conservative assumptions made for predicted future employment were based on the projected occupied housing. Limited information available from the Tennessee Department of Employment Security predicts an active annual employment growth rate for Cumberland County through the Year 2005. Assuming this active employment growth rate to continue, the IWR-MAIN estimates are acceptable.

The IWR-MAIN calibrated model was then used to simulate water demand for the forecast years 2010, 2025, and 2050. The results from the model predict an average annual demand of 13.1 MGD for the Year 2050 (see Table 3-4). Note: the IWR-MAIN model includes conveyance losses in the total water usage. The data gathered from the Cumberland County Utility Districts did not include a defined conveyance loss value. Therefore, in order to compare similar water usage predictions, the conveyance losses have not been included in the total IWR-MAIN prediction, but are presented in Table 3-4.

Table 3-4

Median Growth Scenario
(Moderate Increase to Historical Trend)
IWR-MAIN Water Demand Analysis Software

Year	Sector	Average Water Use			Max Day
		Summer (MGD)	Winter (MGD)	Annual (MGD)	(MGD)
1990	Residential	1.60	0.99	1.29	1.94
	Nonresidential	1.38	1.38	1.38	2.07
	Other	0.33	0.26	0.30	0.45
	TOTAL	**2.98**	**2.37**	**2.67**	**4.01**
1997	Residential	2.54	1.35	1.94	2.91
	Nonresidential	1.88	1.88	1.88	2.82
	Other	0.49	0.36	0.43	0.64
	TOTAL	**4.42**	**3.23**	**3.82**	**5.73**
2010	Residential	4.03	2.14	3.08	4.62
	Nonresidential	2.97	2.97	2.97	4.46
	Other	0.78	0.57	0.67	1.01
	TOTAL	**7.00**	**5.11**	**6.05**	**9.08**
2025	Residential	5.97	3.08	4.43	6.65
	Nonresidential	4.27	4.27	4.27	6.41
	Other	1.12	0.82	0.97	1.45
	TOTAL	**10.24**	**7.35**	**8.70**	**13.06**
2050	Residential	8.73	4.64	6.69	10.03
	Nonresidential	6.45	6.45	6.45	9.67
	Other	1.69	1.23	1.46	2.19
	TOTAL	**15.18**	**11.09**	**13.14**	**19.70**

Note: Total does not include conveyance losses (other) in order to compare IWR-MAIN values with available utility district data.

Note: Total does not include conveyance losses (other) in order to compare IWR-MAIN values with available utility district data.

3.1.4 Unlimited Growth Scenario

An unlimited growth projection was also estimated as the product of projected customers and a projected value of daily "per tapita" water use. For this projection, an average annual percentage increase in customers, 5.7%, was determined from 1992 to 1997. This particular time period included historical data from all of the utility districts. This constant percentage increase was then applied to the current customer base and compounded annually through the Year 2050. This projection allowed for the consistent annual growth of each utility as well as incorporating an annual growth in the existing distribution areas. Again, an average "per tapita" water use was

applied. Using this method, the predicted average annual water demand in the Year 2050 is 68.3 MGD for 323,000 customers (see Table 3-5).

Table 3-5
Unlimited Growth Scenario
Constant Customer Percent (5.6) Increase

Year	Total Customers	Avg. MGD[1]	Peak MGD
1997	17701	3.52	5.28
2000	20935	4.16	6.24
2010	36629	7.28	10.93
2020	64087	12.74	19.12
2030	112127	22.30	33.45
2040	196180	39.01	58.52
2050	**343241**	**68.26**	**102.39**

[1]Note Average MGD calculated from an average "per tapita" usage (see Section 3.1.1).

3.2 SUMMARY OF GROWTH SCENARIOS

Three methods of predicting the future municipal water demand for Cumberland County predict an average annual usage greater than the existing capabilities of the water supply system. Figures 3-1 and 3-2 give a visual comparison of the four growth scenario methods. The limited growth scenario is the most conservative water demand in the Year 2050. This scenario assumes the current growth of the existing water supply distribution systems would drop from approximately 5 percent to 1.5 percent. The historical growth scenario also estimates a conservative water demand in the Year 2050. However, this method assumes residential and non-residential sectors of the County would continue to grow at the same rate. The median growth projection using IWR-MAIN estimates a future municipal water demand for the County similar to the historical usage prediction, but takes into account the socioeconomic factors, as well as climatic conditions, influencing future water use. Finally, the unlimited growth scenario is shown for comparative purposes, but grossly over estimates the future water demands in the County for the 50 year period. The population of the County should not be expected to grow at this constant rate without reaching a saturation point. However, this unlimited growth scenario does estimate a rapid growth trend of the County in the short term future. Perhaps, the actual growth rate of the County may behave in a manner similar to a combination of all predicted growth scenarios. Again, a detailed Needs Assessment will be performed during a Feasibility Study / EIS process.

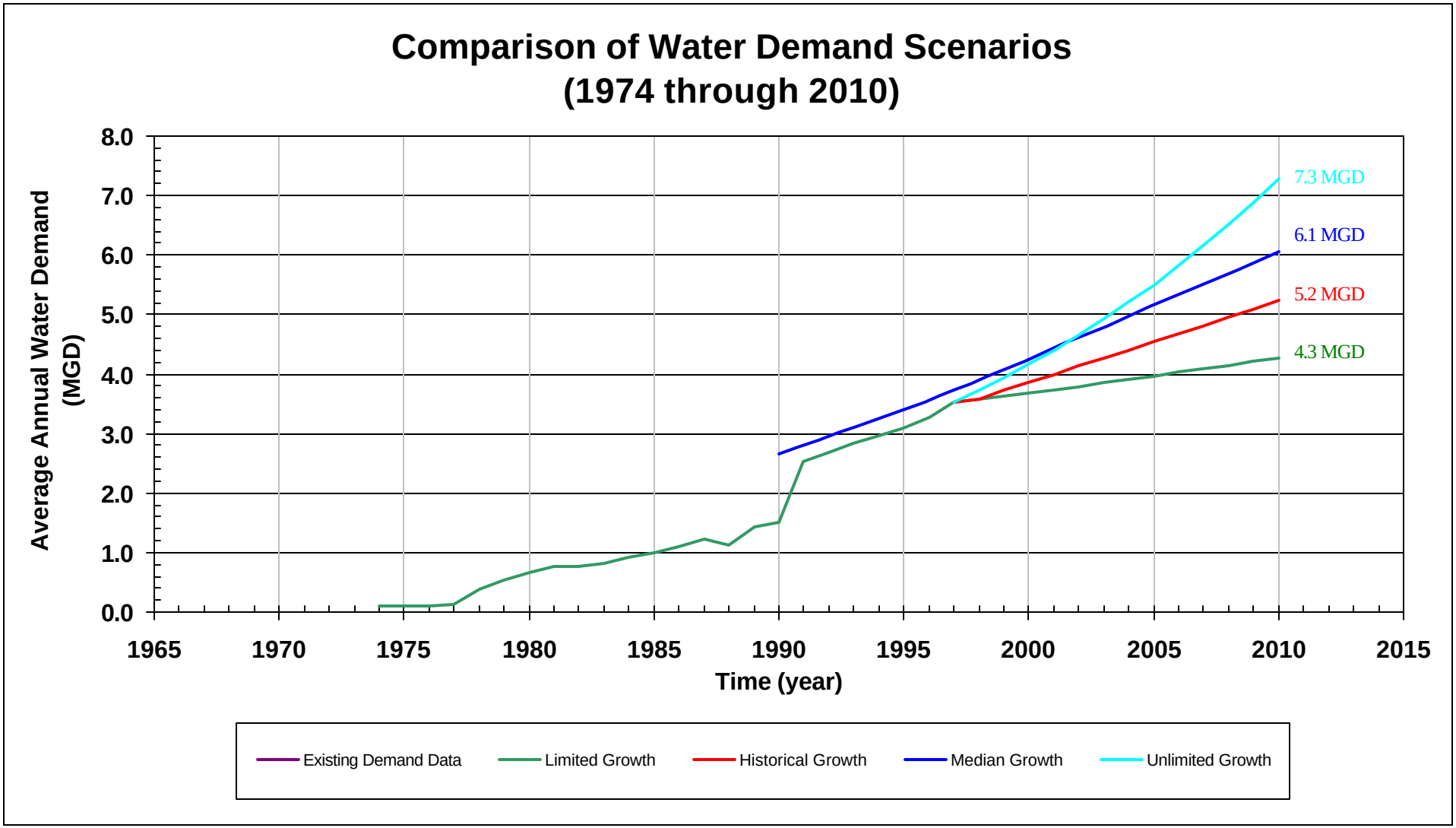


Figure 3-1 - Comparison of Growth Scenarios and Associated Water Demand (1974 – 2010)

Figure 3-2 - Comparison of Growth Scenarios and Associated Water Demand (1974-2050)

4.0 PRELIMINARY ALTERNATIVE ANALYSES

4.1 GENERAL

4.1.1 Objective and Scope

The objective of this study was to develop a list of potential alternatives for additional water supply to Cumberland County. The potential alternatives addressed herein, are in no way intended to be all-inclusive, but rather representative of the broad range of solutions that might be pursued for Cumberland County. More detailed information and analysis of specific alternatives would be required during a follow-on feasibility phase of investigation.

In addition to identifying potential alternatives, the analyses also quantified the amount of water supply each alternative could provide. The scope of this report was to perform the necessary analyses to provide a perspective on the water supply capabilities and financial impacts of each alternative.

4.1.2 Alternatives Considered

Several water supply studies have been performed for Cumberland County in the past. During these studies, different alternatives for providing additional water supply were identified. Those various alternatives have been brought together in this report in order to provide an equitable comparison. Each alternative was analyzed using the same parameter derivation, modeling techniques, assumptions, etc. to ensure the results are at the same level of detail.

The alternatives analyzed for this study include:

- Water Conservation
- Groundwater
- Pipeline to large reservoir
- Storage Impoundments
    - Improvements to Existing Reservoirs
    - New Storage Impoundments
- Water Harvesting
- No Action

4.2 ENGINEERING METHODS

4.2.1 General

The level of detail for this preliminary engineering report was to be adequate enough to quantify the water supply potential and prepare a cost estimate for each alternative. The following design criteria and modeling techniques were used to establish this objective.

4.2.2 Water Supply Yield

The water supply yield throughout the report is presented in terms of million gallons per day (MGD). The yield for each alternative is dependent on severe drought type events.

4.2.3 Historical Drought Identification

Use of drought data is the obvious means by which most water supply alternatives are designed. The idea behind the design of a water supply alternative is to have sufficient water to sustain the worst of drought conditions. Some water supply alternatives are sensitive to short droughts with no rain; while others are sensitive to longer duration droughts with only moderate rain. Therefore, the development of a good historical database of rainfall is essential to the evaluation of water supply alternatives.

For this study, the Crossville Experiment Station (EXP STN) gage was used to identify historical droughts. Rainfall records for the Crossville gage were retrieved from CD-ROM's containing National Weather Service (NWS) data. Drought investigations were conducted by first identifying continuous periods that were particularly dry for 3, 4, 6, 12 and 24 months. A review of the historical record was made and all periods indicating possible drought conditions were identified. From this review, the driest years on record were identified. They are 1922-1927, 1940-1942, 1952-1954 and 1986-1988. Table 4-1 provides a comparison of the average annual rainfall and the total rainfall for the drought years identified at the Crossville EXP STN gage. A tabulation of the monthly rainfall data for these periods is provided in Table 4-2. Note: The months of May 1925 and December 1927 have missing data at this gage.

Table 4-1
Crossville EXP STN Rainfall Gage
Comparison of Annual Average and Drought Year
Rainfall Totals (inches)

Year	Total Rainfall (inches)
Annual Average	56.94
1922	69.75
1923	52.95
1924	46.01
1925	40.81
1926	56.50
1927	54.80
1940	45.83
1941	41.83
1942	66.35
1952	39.97
1953	49.82
1954	58.30
1986	51.73
1987	42.66
1988	54.00

Table 4-2
Crossville EXP STN Rainfall Gage
Droughts During a 85 Year Period of Record, 1912-1997
Monthly Rainfall Totals (inches)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1922	5.32	3.50	9.69	8.16	5.51	7.36	7.10	9.28	1.72	2.85	1.42	7.84
1923	3.81	4.71	6.16	5.17	4.77	4.79	6.10	6.29	2.25	0.77	2.95	5.18
1924	5.23	4.43	3.49	5.66	5.51	2.37	2.88	2.49	5.03	0.06	1.47	7.39
1925	3.49	5.24	1.74	4.28		2.01	3.35	1.35	1.19	10.85	5.23	2.08
1926	2.96	2.61	3.87	1.82	2.46	6.48	3.64	10.77	2.25	2.75	5.80	11.09
1927	4.33	5.12	6.69	5.13	5.61	6.09	9.18	4.12	1.47	0.48	6.58	
1940	2.60	5.65	6.73	2.95	4.30	4.03	3.49	5.93	0.91	1.99	3.40	3.85
1941	2.42	0.62	3.61	6.08	0.91	3.45	6.97	5.03	1.48	3.28	3.97	4.01
1942	3.65	4.87	6.39	0.88	2.60	6.67	9.70	10.33	2.29	2.41	3.51	13.05
1952	7.44	2.88	7.37	2.35	3.21	2.80	1.58	1.80	2.20	0.76	3.33	4.25
1953	5.78	6.08	4.77	5.98	4.54	4.08	6.29	1.82	2.14	0.76	1.71	5.87
1954	11.95	3.75	5.12	5.04	4.65	3.04	1.78	1.70	3.42	4.44	3.89	9.52
1986	1.32	6.43	2.47	1.82	6.36	1.87	3.05	3.10	4.94	6.57	9.06	4.74
1987	4.37	5.16	2.54	3.13	3.85	3.31	3.26	2.70	4.81	0.72	4.07	4.74
1988	5.62	2.80	3.05	4.14	2.42	0.96	7.60	5.41	6.54	2.50	8.09	4.87

4.2.4 Hydrologic Modeling Techniques

Several hydrologic computer models were utilized to size and analyze the capacity of the water supply alternatives. Section A.1 of Appendix A contains a description of each model used, as well as parameter derivations and assumptions used for each model.

4.2.5 Hydraulic Modeling Techniques

The hydraulic calculations in this study were computed with the Corps of Engineers' Program "HEC-RAS, River Analysis System, Version 2.1, October, 1997." Section A.2 of Appendix A contains a description of the HEC-RAS model used, as well as parameter derivations and assumptions used.

4.3 WATER CONSERVATION ALTERNATIVE

4.3.1 Introduction

Water conservation is the act of protecting water from loss or contamination so it can be used for desired purposes. Water conservation measures reduce water consumption, reduce water withdrawal or diversion, reduce water loss or waste, improve water use efficiency, and increase water recycling or reuse. Unfortunately, when water utilities introduce conservation policies to a community, they must depend on thousands of customers to behave in certain ways. In this sense, conservation alone may not be a secure alternative to the expansion of treatment and distribution systems for meeting water needs in the Year 2050. However, water conservation is considered an essential complement to the traditional approach of capital improvements and should be implemented by current Cumberland County residents and promoted by the governing utility districts.

4.3.2 Water Conservation Plan Guidelines

As recommended by the U.S. Department of Interior, Bureau of Reclamation, there are four fundamental water conservation measures which should be included as part of any water conservation plan. These measures include (1) water measurement and accounting, (2) water pricing, (3) information and education, and (4) the assignment of responsibility for conservation activities.

Water Measurement and Accounting System

A water measurement and accounting system should be designed to measure and account for the water conveyed through the utility district's distribution system to water users. Most of unaccounted-for water is lost through leaks in the distribution system. Unaccounted-for water includes leaked water, water used for municipal purposes and fire-fighting, and water unaccounted for by underregistering meters or other unmetered uses. The total amount of unaccounted-for water may vary from 10-15 percent in a well-operated system. This is consistent with the information obtained form Cumberland County utility districts. Leak detection should be a regular activity of every water supply system. Periodic checks should be made on valves, hydrants, and services to locate obvious underground leaks.

Additional improvements to the distribution system include decreasing water pressure in order to diminish the amount of water flowing through open faucets. High pressure is one cause of pipe joint leaks and it generally causes water facilities to wear out more rapidly. High pressure also produces greater losses through existing leaks.

Water Pricing Structure

A water pricing structure should encourage efficiency improvements by water users. A utility district's water pricing structure can provide incentives or disincentives to efficiency improvements by water users. A pricing and billing strategy is based, at least in part, on the quantity of water delivered. It does not imply a utility district would alter it's overall revenue requirements, increase cost to users, or eliminate all present methods of recovering certain fixed costs. Quantity-based charges can be incorporated into various existing pricing structures to provide some degree of economic incentive for efficient users or group of users. Often termed "Conservation-Oriented Pricing," price structures can be changed so that customers pay more for each unit of water after their consumption reaches a certain level. Prices can also be increased during the summer, when water demand increases for lawn watering and other activities.

However, for the utility districts, reducing water usage could also lead to less short-term revenue. Water is marketed like most other products, the more water that utilities sell, the more money they make. In response, State regulatory commissions can adopt policies that reward regulated water utilities for promoting conservation.

Information and Education Program

Education is the key to the success of a conservation plan because it can help users understand why water conservation is needed and how to conserve water. It can also successfully minimize opposition to water conservation programs and can improve water supply planning coordination among community officials. An information and education program should provide users with information about efficient water use and the water conservation services that are available through the utility district or other organizations. There are numerous suggestions for distributing information and promoting water conservation education:

- Developing citizen involvement through public interest groups;
- Establishing speaker's bureaus to make presentations to schools, businesses, and service organizations;
- Newspaper articles, public service announcements, TV news stories;
- Developing and distributing films or slide shows;
- Radio and TV talk shows and interviews;
- Promotion conferences , symposia or seminars;
- Distributing pamphlets brochures, leaflets, and posters promoting conservation;
- Encouraging board of education involvement;
- Bumper stickers, buttons, decals;
- Conducting public demonstrations, displays, and distribution at malls and shopping centers, schools, and booths at fairs; and
- Enclosing water bill inserts and distribution newsletter to non-customer water users such as apartment dwellers.

Education in schools can begin to build a water conservation ethic and environmental awareness among children that would carry on to their adult lives. Additionally, educational material should make direct connection between conservation activities and monetary savings, from the points of view of both consumers and water suppliers. Water conservation education should be an integral part of any system's water supply management program.

Water Conservation Coordinator

An effective water conservation plan identifies the person or persons responsible for development and implementation of the water conservation plan.

Additional Water Conservation Measures

A water conservation plan should identify measures that are applicable to the water management and conservation goals that have been established by the utility district. All potentially applicable water conservation measures are identified, analyzed and evaluated to determine whether it is feasible or practical for the district to implement them, either individually or in various combinations. Suggested additional measures include:

- Residential and governmental audit and incentive programs;
- Commercial and industrial audit and incentive programs;
- Landscape programs;

  A popular conservation landscaping program is the Xeriscape program. Combining the Greek word for dry, "xeros", and the suffix "scape" from landscape, the Xeriscape program is the application of sound horticultural practices in the development of quality landscapes that conserve water and protect the environment. The seven principals of the program are (1) planning and design, (2) soil analysis, (3) appropriate plant selection, (4) drought and heat tolerant turf areas, (5) efficient irrigation, (6) use of mulches, and (7) appropriate maintenance.
- Distribution system audit programs;
- Drought/water shortage contingency plan;
- Plumbing regulations;

  Local governments may adopt ordinances or plumbing codes that require the installation of water conserving devices in all new construction. At present, Tennessee codes do not require low flow fixtures or water conserving devices.
- Fixture replacement programs.

  Installation of water-saving devices can reduce indoor water usage as well as public deliveries and returns to wastewater treatment plants. Water-saving devices include low-

flow toilets (retrofit and new installation), shower devices, faucet devices, and water-saving appliances such as front loading washer.

4.3.3 Existing Water Conservation Programs

Large and small scale water conservation programs implemented across the country seek to reduce water usage both indoors as well as outdoors. The benefits of two such programs are highlighted here.

Georgia Water Wise Council

The Georgia Water Wise Council is a non-profit educational corporation. The creation of the Council was a state wide green industry initiative resulting from the 1988 drought throughout the Southeast. The Council is a partnership of government, education, business and citizen entities with the purpose of promoting conservation education programs for water quality as well as quantity in Georgia.

One of the first goals of the Water Wise Council was to show water utilities and nurseries and landscape contractors how conservation was advantageous. Since that time the Council has become involved in numerous programs some of which include educational programs at public schools, publication of a Xeriscape book, a Xeriscape video, distribution and promotion of the EPA's Water Source Book Series for elementary through high school students, and establishment of an active speakers bureau. The conservation benefit that the Council has produced is the establishment of an educational structure and implementation of programs to help address tomorrow's water supply problems today.

Houston, Texas

The City of Houston was required to develop and implement of a conservation program following a 1993 ruling by the Texas Natural Resource Conservation Commission. The conservation program includes an education program, in-house programs for departments whose budgets are derived through the general fund, a program to require all large contractors to prepare a conservation plan, and conservation planning.

The education program includes a retrofit program, a school education program, a t-shirt design contest and speakers for civic associations, environmental groups, etc. The in-house programs for General Funds departments include irrigation audits at City golf courses, leak detection and repair of City pools and fountains, and tracking and reporting water use by general funds departments. A recommended conservation plan was developed for the city by an independent firm. The recommended plan included such programs as residential water audits, indoor audits on Commercial industries, cooling tower audits, pool/fountain audits, pool/fountain standards, and public education.

Benefits from the recommended plan include capital deferrals such as delaying water purification plant expansions and a reduction in the production of water thus lowering operation and maintenance costs. The recommended plan is expected to reduce water demand in the City of Houston by 21.8 MGD or 7.3 percent of retail water production by the Year 2006.

4.3.4 Benefits of Water Conservation

The monetary benefits of water conservation are clearly noted. With less water usage, fewer chemicals are needed for purification, operating costs are lower for both water and sewage treatment plants, and expansion of these plants to provide adequate capacities is needed less frequently. In addition to lowering costs and improving the reliability of water and wastewater systems, water conservation plans also help meet a community's environmental goals. Environmental benefits include maintaining riparian habitat and restoring fisheries, protecting groundwater supplies from excessive depletion and contamination, improving the quality of wastewater discharges, reducing excessive runoff of urban contaminants, and restoring the natural values and function of wetlands and estuaries. With the list of benefits growing, water conservation plays an invaluable role in the search for and development of new water supplies.

The Cumberland County Utilities estimate public/unaccounted for conveyance losses within the water distribution system to be 10 percent of the total municipal water use. A water conservation program designed solely to repair leaks within the system, could reduce the water demand by this estimated 10 percent and thus lower the projected water need in the Year 2050. Figures 4-1 and 4-2 display the adjusted four predicted growth curves due to a water conservation contribution.

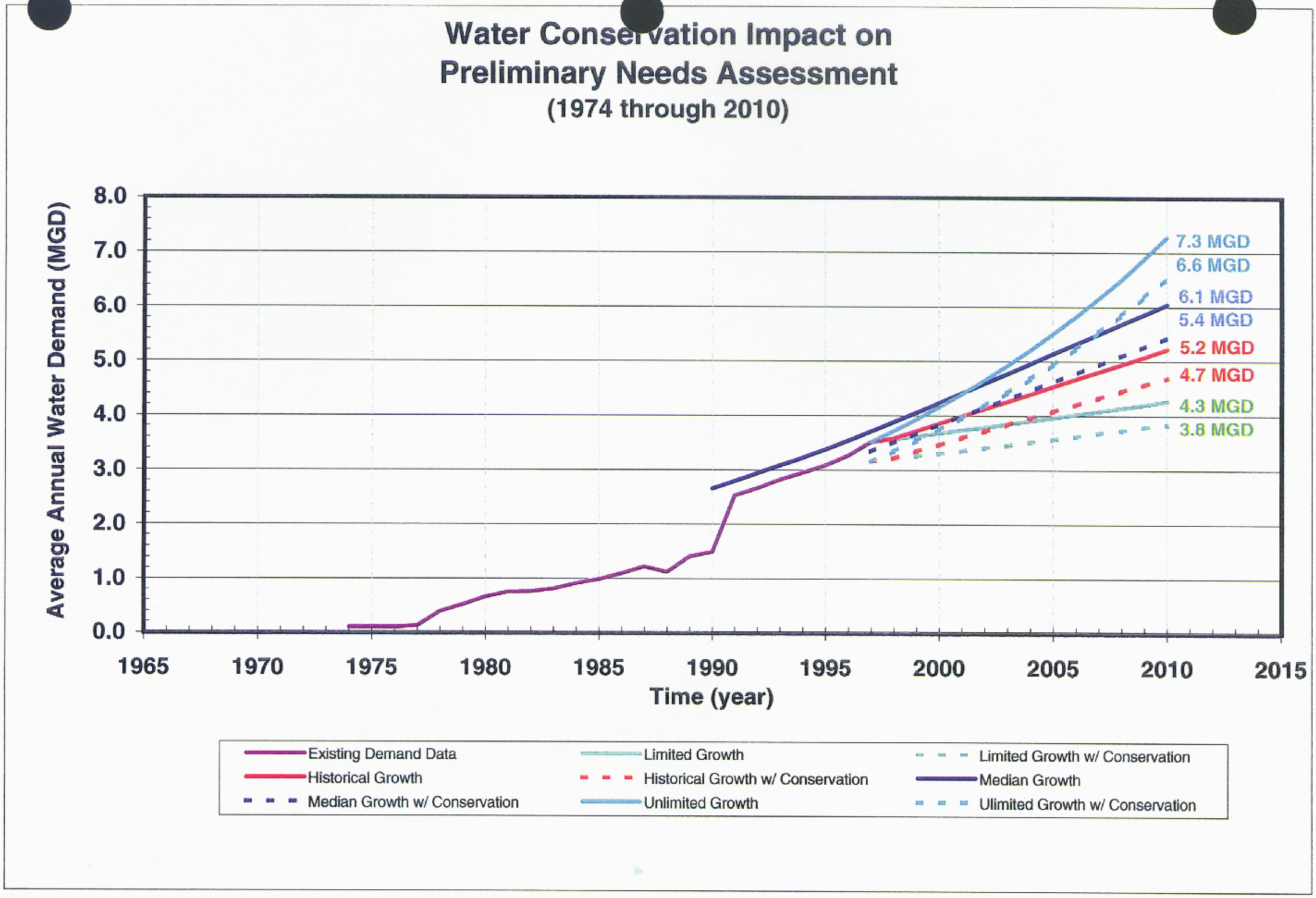


Figure 4-1 Water Conservation Impact on Preliminary Needs Assessment (1974 - 2010)

Figure 4-2 Water Conservation Impact on Preliminary Needs Assessment (1974 - 2050)

4.4 GROUNDWATER SUPPLY ALTERNATIVES

4.4.1 Introduction to Groundwater

Groundwater storage is considered to be more abundant than all artificial and natural surface storage in the United States. However, sources of groundwater storage are much more widely distributed than surface supplies. Strong local concentrations of groundwater are found as a result of the variety of soils, rocks, and geologic structures located beneath the earth surface. The relationship between groundwater and surface storage is one of mutual interdependence. Groundwater reserves sustain the continuing outflow of streams and lakes during prolonged periods, typically following the relatively few runoff-producing rains each year.

4.4.2 Previous USGS Groundwater Study

In 1996, the United States Geologic Service (USGS) conducted a preliminary study to evaluate groundwater as an alternative source for municipal water supply to Cumberland County. The following groundwater alternatives were investigated:

- Groundwater from random household wells
- Groundwater from wells beside a perennial stream
- Groundwater from coal, oil or gas holes
- Groundwater from carbonate rocks at the foot of the Cumberland Plateau

According to the USGS findings, of the alternatives listed above, the most likely one to provide sufficient water supply is groundwater from the toe of the Cumberland Plateau. The results of the USGS investigations can be found in a letter to Mr. Dodd Galbreath dated February 29, 1996 and another letter to the same party dated March 5, 1996 which includes a proposal for further investigation of the western toe of the Cumberland Plateau as an alternative for water supply. Much of the USGS' assumptions about groundwater at the western toe of the Cumberland Plateau are based on a USGS study on the Elkton Aquifer.

Beginning in 1988, the USGS conducted a six year groundwater investigation of resources from New York state to central Alabama entitled the Appalachian Valleys – Piedmont Regional Aquifer-System Analysis (AP/RASA) project (see Figure 4-3). The study focused on the regional analysis of the hydrogeology of carbonate and siliciclastic rocks in the Valley and Ridge Physiographic Province.

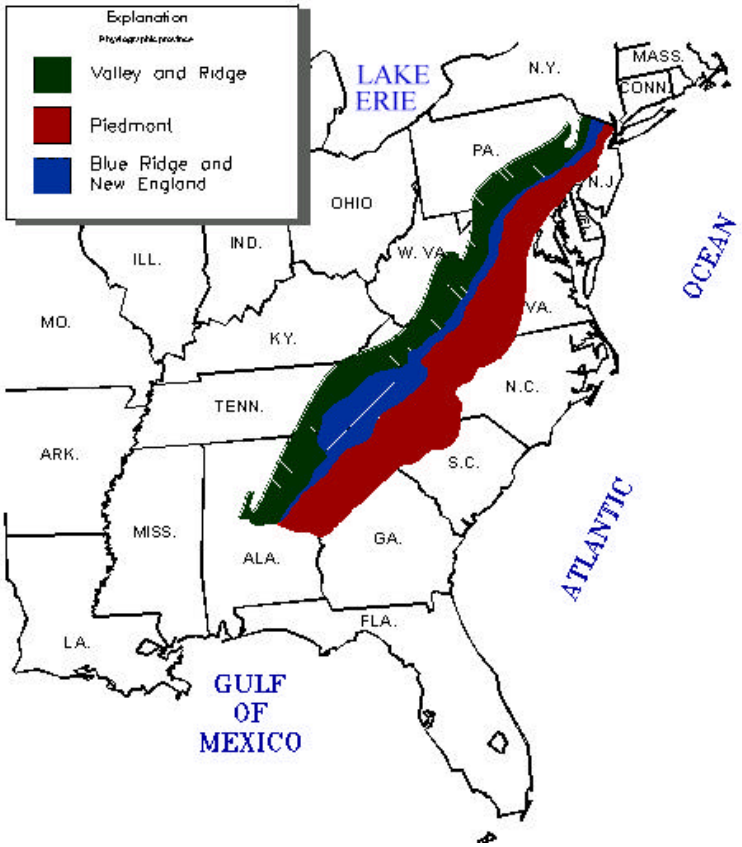


Figure 4-3 USGS Study Area AP/RASA

One of the hydrogeologic terrains identified for its water-yielding properties was the western toe of the Blue Ridge Mountains within Rockingham and Augusta Counties, Virginia. The western toe is an area characterized by an apron of colluvium and alluvium at the toe of the northwestern slope of the Blue Ridge Mountains. The colluvium consists of thin, stony material shed from outcrops of resistant siliciclastic rock. The stony colluvium commonly grades down the slope into dissented alluvial terraces that contain cobble-size gravel, sand and sandy loam. In many discontinuous places, this colluvial-alluvial apron overlies fine-grained residuum and cavernous dolomite bedrock within the southeastern margin of the Valley and Ridge Physiographic Province. In parts of the western toe, the combined thickness of colluvium, alluvium and residuum exceeds several hundred feet. The name "Elkton Aquifer" was given to the cavernous dolomite bedrock that is overlain by the thick residuum, alluvium, and colluvium at the southeastern edge of the Valley and Ridge Physiographic Province (see Figure 4-4). The Elkton Aquifer extends beyond Virginia and West Virginia and exist in parts of 36 counties in the Valley and Ridge Physiographic Province.

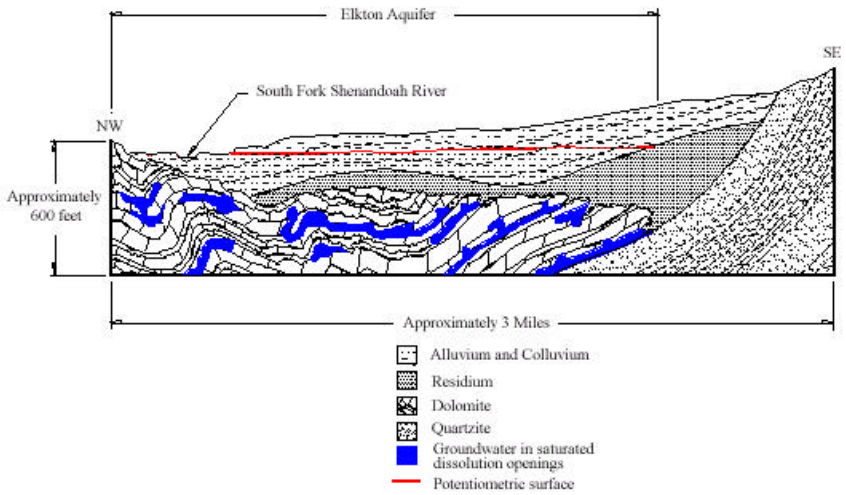


Figure 4-4 Elkton Aquifer

Additionally, a statistical analysis was performed to determine if substantial differences in the water-yielding properties between the Elkton Aquifer and other areas of dolomite could be detected. The average yield for Elkton aquifer wells, 26 gallons per minute (gpm), was more than three times the average yield for other dolomite wells, 7 gpm.

4.4.3 Selection of Potential Groundwater Sites

According to the USGS, geomorphic and lithologic controls similar to those at the toe of the northwestern slope of the Blue Ridge Mountains exist at the foot of the Cumberland Plateau. Carefully located and properly constructed groundwater wells within the western toe of the Cumberland Plateau escarpment is considered to be a viable alternative for supplying Cumberland County with supplemental water. Based on topographic and geologic maps, high-altitude photography, and site visits, the USGS identified five potential sites for groundwater supply along the Cumberland Plateau (see Figure 4-5).

1. Along the canyon of Hurricane Creek from the confluence with Little Hurricane Creek to the confluence with East Fork Obey River;
2. Along East Fork Obey River upstream of the confluence with Hurricane Creek and within Fentress County;

3. Along East Fork Obey River and the lower reaches of Little Indian Creek just upstream of the Fentress-Overton County line;
4. The southeast terminus of East Fork Road out of Hanging Limb off of Route 164 northeast of Monterey; and
5. Along Route 85 and Big Laurel Creek west of the East Fork Obey River bridge.

No detailed groundwater analyses at these sites have been conducted. Testing and verification of the potential of the alternative would require field work, further site selection, test drilling and aquifer testing. This work is beyond the scope of this preliminary engineering study. In order to provide perspective on the groundwater alternative in relationship to the other alternatives, assumptions concerning number of wells that can be supported at each site, groundwater well production and depth to the aquifer(s) were made (refer to Section 4.3.5).

4.4.4 Modeling Process

This design alternative assumes that groundwater is pumped from three wells at each of the five sites at a rate of 500 gpm. Groundwater would be pumped from an assumed drilling depth of 500 feet to the earth surface. From the surface, booster pumps would carry the total flow from each site (1500 gpm) to the Lake Holiday Treatment Plant. The total combined flow from all five sites is 7500 gpm (10.8 MGD). Achieving this amount of flow from groundwater wells could prove to be a difficult task. Another concern is that three wells at each site could cause an overlap in the cones of depression. More certainty in the design would be obtained if the alternative is carried forward to a feasibility-level study.

The route from each of the five identified well sites to the Lake Holiday Treatment Plant was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways (see Figure 4-6).

Site #5 is the northern most site and must cross the East Fork Obey River before following along State Route 85 to U.S. Route 127. Site #1 is the centrally located site and follows along County roadways before joining with U.S. Route 127 and combining with the flow from Site #5 for a total flow of 3000 gpm. Sites #2, #3, and #4 are located to the west and combine at the township of Muddy Pond (4500 gpm) and follow County roadways to State Route 62 to U.S. Route 127.

Total flow is combined (7500 gpm) from all five sites at the junction of State Route 62 and U.S. Route 127. The pipeline then follows U.S. Route 127 into the City of Crossville and U.S. Route 70 to the Treatment Plant at Lake Holiday.

The static head, or vertical distance from the booster pump centerline (earth surface) to the point of free discharge, varies at each site with an average head of over 600 feet. Minimum diameter pipelines were selected based on the associated friction head. All pipelines selected were ductile iron pipes. Table 4-3 gives a summary of all pump and pipe sizes.

Table 4-3
Groundwater Alternative
Pump and Pipeline Design Data

Location	Deep Well Pump		Booster Pump		Pipe Diameter	Pipe Length
	Flow (gpm)	Total Head (ft.)	Flow (gpm)	Total Head (ft.)	(in.)	(ft.)
Site #1	3 @ 500	500	1 @ 1500	630	12	3500
	N/A	N/A	1 @ 1500	250	14	17500
Site #2	3 @ 500	500	1 @ 1500	750	12	7510
Site #3	3 @ 500	500	1 @ 1500	590	12	5180
Site #4	3 @ 500	500	1 @ 1500	615	12	8340
Site #5	3 @ 500	500	1 @ 1500	485	12	2840
	N/A	N/A	1 @ 1500	580	14	31250
Combination #'s1 and 5	N/A	N/A	N/A	N/A	14	9900
Combination #'s 2 and 3	N/A	N/A	N/A	N/A	16	7200
Combination #'s 2,3, and 4	N/A	N/A	1 @ 4500	80	24	29820
Combination #'s 1,2,3,4, 5	N/A	N/A	1 @ 7500	90	30	29100
Along U.S. 127	N/A	N/A	1 @ 7500	110	30	23400
Along U.S. 127	N/A	N/A	1 @ 7500	50	30	44600

The average distance from each of the five groundwater sites to the Lake Holiday Treatment Plant is approximately 26 miles. Because the flow from each of the sites is combined, the total pipeline distance is approximately 40 miles.

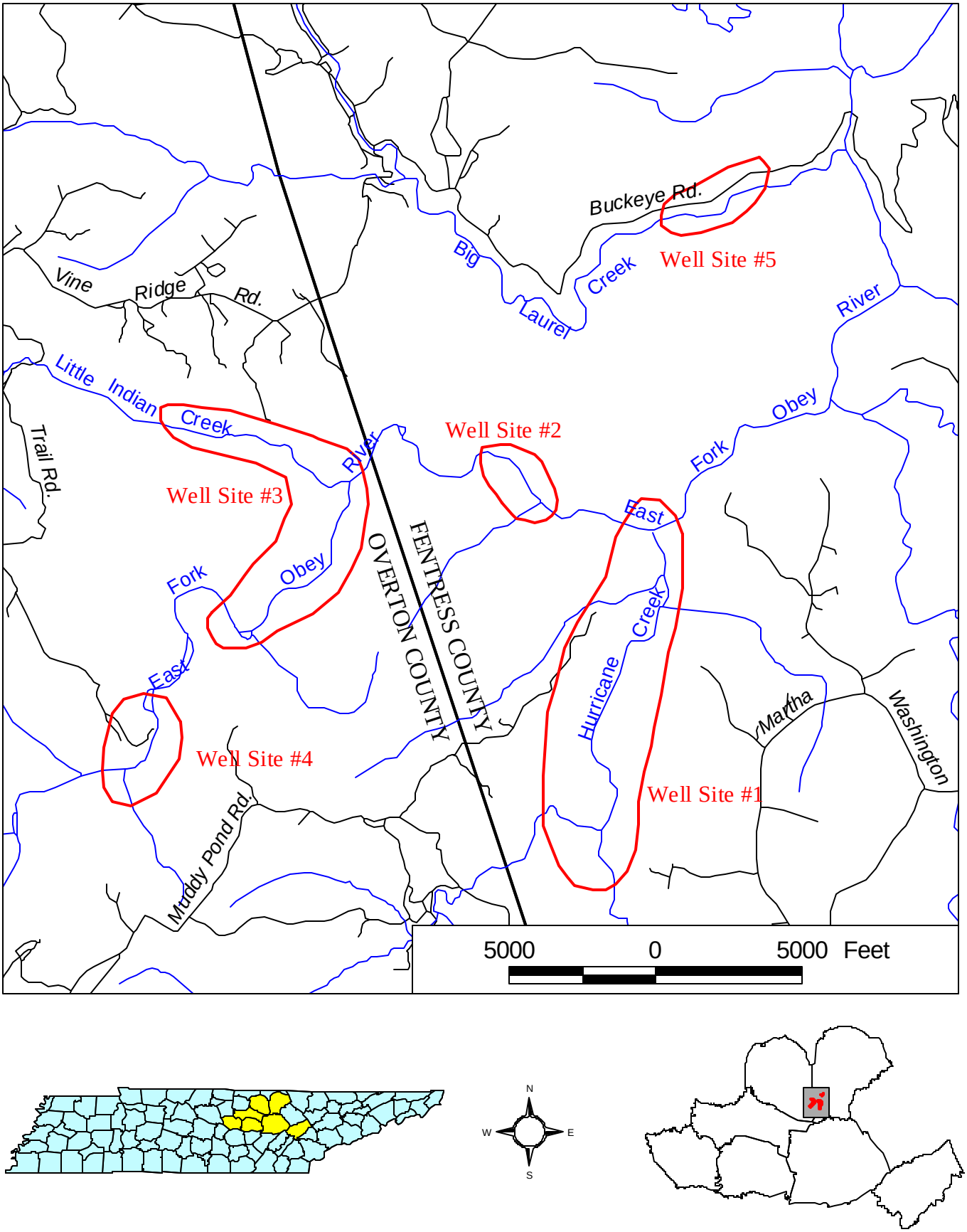


Figure 4-5 Location of Potential Groundwater Sites

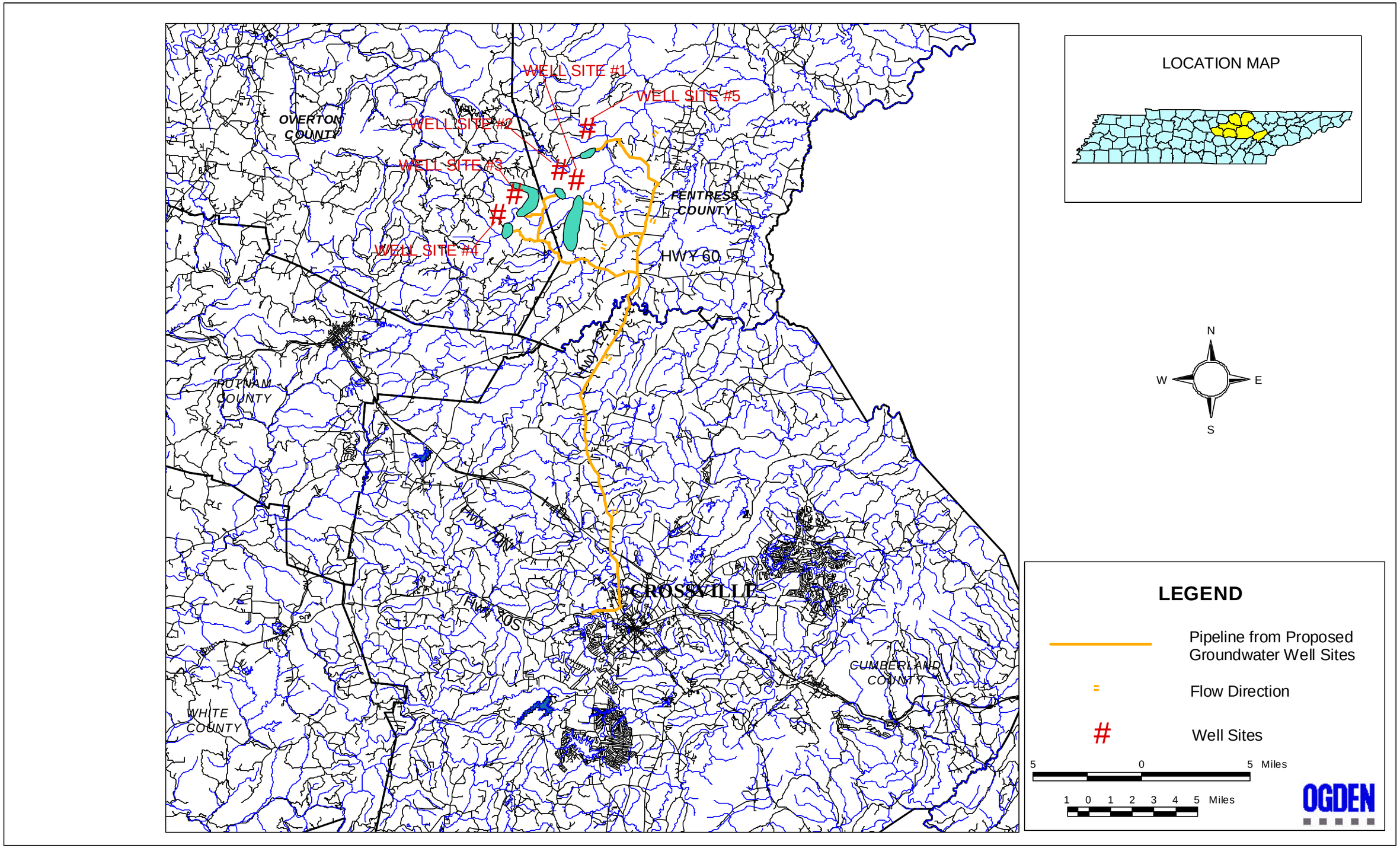


Figure 4-6 Proposed Groundwater Pipeline Route

It should be reiterated that the feasibility of this groundwater alternative has not been fully explored. No test drilling and aquifer testing have been performed at the potential sites. Additionally, the effect of local coal mines on water quality at each of the five groundwater sites is unknown. This questionable data would be researched during an extensive feasibility study of the alternative. Water supply negotiations between Cumberland County and any county where a groundwater well is located would also be required.

4.5 LARGE SCALE PIPELINE

4.5.1 Introduction to Groundwater

This alternative explores the option of transporting raw water from existing reservoirs in the adjacent geographic area to Cumberland County and the Lake Holiday Treatment Plant. The selected sites are Watts Bar Lake in Roane County, Center Hill Lake in Dekalb County, and Great Falls Lake in White County. Water intake from Watts Bar Lake could potentially be treated at the existing Rockwood Treatment Plant before transportation to storage tanks within Cumberland County (see Figure 4.7).

The Tennessee Valley Authority (TVA) initially developed the pipeline alternatives during the previous review of alternatives for the Catoosa Utility District ("Water Supply Development Proposal, Catoosa Utility District and Upper Cumberland Plateau Region, Preliminary Findings Report"). Three pipeline routes were evaluated to supply both Cumberland and southern Fentress County region. As illustrated in Figure 4.7, the pipeline routes followed highway and county roads for the most part with the exception of the last portion to the water source. Existing waterlines follow these same roads for most of the route. Design estimates were determined for two pipe sizes (24" and 30") for each route. Only the 24" pipeline was included in the cost estimate due to the 30" design having a higher estimated cost. Three large water sources considered were Dale Hollow and Center Hill Lakes in the Cumberland River basin and Watts Bar Lake on the Tennessee River. For this Preliminary Engineering Report, the Dale Hollow Lake pipeline was dropped after an initial review since it was obviously more costly than the other two sources and since southern Fentress County was not included in this study. A variation of the Center Hill Lake pipeline was later considered, using Great Falls Lake as the water source. This lake is immediately upstream of Center Hill Lake.

It should be noted that the Corps of Engineers would charge Cumberland County for storage at Center Hill Lake if a water supply intake were constructed there. There is also a strong possibility TVA may charge for lost hydropower at Watts Bar Lake or Great Falls Lake should either of those reservoirs be used for water supply by Cumberland County. Because these charges are an unknown until a more detailed design is completed, they were not included in the cost estimate.

4.5.2 Modeling Process

A target yield was required to size the pumps and pipelines necessary to transport water from each of the raw water intakes to the Lake Holiday Treatment Plant. As discussed in Section 1.0, a target yield of 9 MGD was assumed for the pipeline alternative. Smaller pumps and pipelines would be required if the desired additional water supply for Cumberland County is identified to be less than 9 MGD, conversely larger pumps and pipelines would be needed if a larger yield is desired. This large scale pipeline design was performed by TVA.

The static head varies at each site. Minimum diameter pipelines were selected based on the associated friction head and pipeline cost. Total friction head for each alternative site is approximately 2000 feet, requiring 6 booster pumps along each of the pipelines. All pipelines selected were ductile iron pipes. Table 4-4 gives a summary of all pump and pipe sizes.

Table 4-4
Pipeline Alternative
Pumps and Pipeline Design Data

Location	Booster Pumps		Pipe Diameter	Pipe Length
	Total Flow (MGD)	Total Head (ft.)	(in.)	(ft.)
Watts Bar Lake	9	1810	24	132,000
Center Hill Lake	9	2405	24	188,000
Great Falls Lake	9	2000	24	165,000

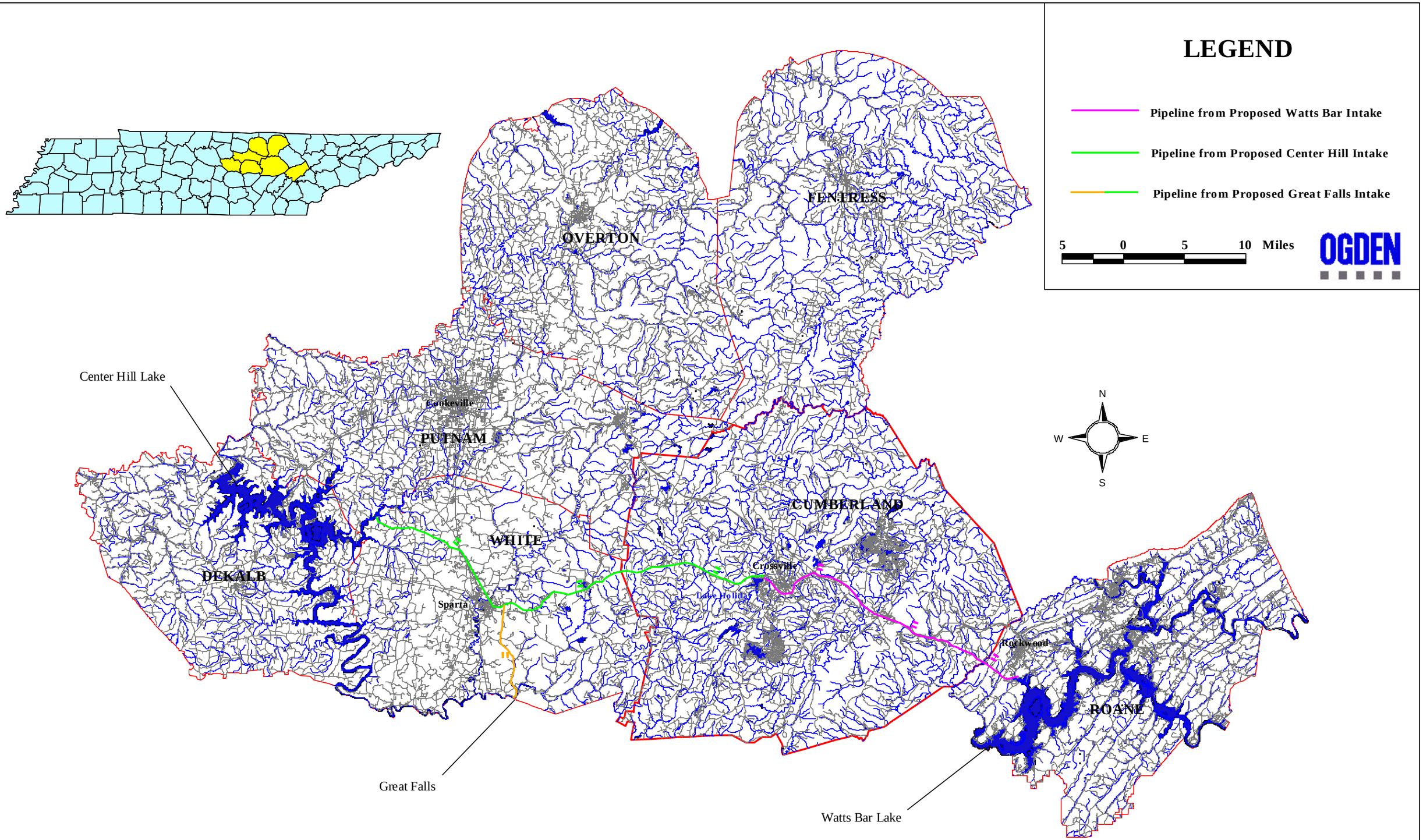


Figure 4-7 Proposed Large Scale Pipeline Alternative

4.5.3 Pipeline Route

Similar to the Groundwater Alternative, the route from each of the three raw water intake sources to the Lake Holiday Treatment Plant was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways.

The Watts Bar Lake pipeline has an intake located near the existing Rockwood water plant. This pipeline has a length of 25 miles and the 24-inch pipe requires six booster pumps. The pool elevation of Watts Bar Lake varies between 733 and 746 feet. Water is pumped up Waldens Ridge (approximately 1440 feet) and follows U.S. Highway 70 through the Cumberland Mountains (1680 feet near Ozone) to the more level plateau lands west of Crab Orchard.

The Great Falls Lake pipeline route would follow U.S. Highway 70 from the base of the Cumberland Plateau east of Sparta, at which point it would follow county roads to an intake on Great Falls Lake near River Hill. Great Falls Lake levels vary between 780 and 805 feet. The pipeline route would follow up Hickory Valley Branch crossing a gap (1280 feet) near Baker Mountain, then climbing up the Cumberland plateau (1800 feet) eventually following U.S. Highway 70 into Crossville. This route was recommended over the Center Hill Lake version because of the higher elevation of Great Falls Lake and the expected improved water quality conditions in Great Falls Lake for water supply purposes. The intake location initially proposed by TVA is in the Falling Water River embayment of Center Hill Lake and this embayment has relatively high levels of algae. No funding was made available from the Corps of Engineers for TVA to provide an equally detailed evaluation of the Great Falls Lake pipeline, compared to the earlier pipeline designs. Based on a cursory review, the route would be shorter and have lower costs than the 36-mile Center Hill Lake route. The Center Hill Lake intake would have to be below the winter pool elevation of 635 feet and the last portion of the pipeline to reach the lake would be much more difficult to construct due to the steep terrain. Water supply negotiations between Cumberland County and TVA or the Corps of Engineers would be required before an intake could be constructed. If Center Hill Lake were the selected reservoir, the Corps of Engineers would charge for the storage being utilized. TVA may require financial offset for lost hydropower at Watts Bar Lake or Great Falls Lake. These additional charges are not included in the cost estimates. The environmental impacts of the pipeline crossings are discussed in Section 6.0.

4.6 STORAGE IMPOUNDMENTS

4.6.1 General

The basis of storage impoundments as water supply alternatives is to capture and store enough water within a reservoir during flood events to sustain the worst of drought conditions. Several depletion processes in addition to the water supply withdrawals would exhaust the volume of water within a water supply reservoir. These include sedimentation, evaporation, leakage and water quality discharges, and must be considered in the design process to accurately quantify the yield a water supply reservoir can provide. Each of these processes and the way they are accounted for in the modeling process is discussed later in this section.

While obtaining the maximum yield is the main objective in designing a water supply reservoir, careful consideration must be given to dam safety. The State of Tennessee passed "The Safe Dams Act of 1973" in May 1973. The Safe Dams Act contains standards to be used in sizing a new dam. Those standards were followed during this study to ensure the dams being designed would meet State standards.

4.6.2 Sediment Allowances

Soil erosion in the watersheds upstream of dams is a constant source of sediment deposition in reservoirs. The Corps has set up monitoring programs to measure the amount of sediment buildup in reservoirs within the Cumberland River Basin. Center Hill Reservoir is one of the Corps reservoirs that has an established sedimentation monitoring program. The results from the Center Hill program have shown the rate at which sediments are deposited in the reservoir is approximately 0.5 acre-feet per square mile per year. The 0.5 rate is an average inflow of sediment over several decades, however, most of the sediment enters a reservoir during major flood events. Since several extreme flood events have occurred within the Center Hill basin over the past two decades, this inflow rate should be considered on the conservative side. Due to the close proximity, similar ground cover, land use and soil type, the sediment deposition rate for the modeled water supply reservoirs in Cumberland County should be equivalent to that of the Center Hill reservoir.

An estimate of the sediment deposition rate is important for two reasons. First, the sediment that enters a water supply reservoir occupies storage area that can no longer be used for water supply.

This may not be important early in the reservoir life. However, as the reservoir gets older, this can become critical, especially after several major floods have occurred. A second reason is that low level outlets such as sluice gates or pump intakes may become covered with silt if located too low in the reservoir.

The depletion in water supply due to anticipated sediment deposition is a small percentage of the total reservoir storage. In contrast, the streambed elevation within the reservoir can change drastically with only moderate rates of sediment inflow. This is due to the small incremental change in storage per foot of depth at the lower elevations of the reservoir. This condition is typical of floodplains with very narrow and steep topographic features. The design sediment deposition rate assumed for this study is 0.5 acre-feet per square mile per year. The design life for the water supply alternatives is 50 years. For the raised impoundment alternatives, sedimentation volumes were calculated from the date each dam was constructed to present, as well as for the fifty-year design life of the raised configuration. Past accumulation of sediment was not determined for the raising of Mayland Lake because its volume was determined by a hydrographic survey that calculated the actual present-day volume. Table 4-5 contains the sediment allowance of each modeled impoundment as well as the portion of each reservoir the allowance would occupy (assuming the sediment would accumulate in the lowest portion of the reservoir). The sediment storage was accounted for in the modeling process by ensuring the reservoir water surface elevation does not drop below the top elevation of the sediment pool (storage area) during the design droughts.

Table 4-5
Sediment Allowances and Sediment Pool Elevations of Impoundment Alternatives

Site	Type of Alternative	Sediment Allowance (ac-ft)	Sediment Pool (feet, NGVD29)
Clear Creek	New impoundment	145.8	1748.0 to 1763.0
Meadow Creek	New impoundment	360.4	1710.0 to 1727.0
Caney Fork	New impoundment	1458.0	1542.0 to 1593.0
Camp Ozone Lake	Raised impoundment	175.1	1665.0 to 1679.0
Mayland Lake	Raised impoundment	64.0	1897.0 to 1898.0
Meadow Park Lake	Raised impoundment	286.0	1784.0 to 1796.0

In reality, the majority of reservoir sedimentation would not deposit at a dam. Most of the inflowing sediment would deposit at the head of a reservoir and form a delta that extends further

and further into the reservoir over time. Modeling this type of deposition is difficult and would require subjective manipulation of the reservoir storage capacity curve. The additional volume gained in the modeling process by allowing the water surface level to drop below the top of the sediment pool is minimal, and would not result in a substantial increase in reservoir yield. Therefore, assuming sediment deposition would occur completely within the lowest part of the reservoir provided a conservative estimate of the reservoir yield.

4.6.3 Evaporation

Evaporation is a substantial factor to be considered in the design of water supply reservoirs. The evaporation referred to here is the evaporation of water from within the reservoir itself. The evaporation from the rest of the watershed is accounted for in the transformation of rainfall to rainfall excess. Evaporation data in general is very limited. It is usually in the form of pan evaporation values at select National Weather Service gages. The closest such gage to the modeled reservoirs is located in Crossville, Tennessee. The values at this gage were compared with other gages in the region to determine if they were accurate. Per this review, it was determined that Crossville's annual pan evaporation rate of 34.55 inches was applicable to the study area. In a feasibility-level study, instead of using the annual evaporation rate, the actual evaporation data for each drought year would be used in the yield analysis.

Widely-published research into pan evaporation rates has concluded that seventy percent of the pan value is equivalent to the evaporation that can be expected in a shallow reservoir. The rate on a deep reservoir would be less. Again, to be conservative, an allowance of seventy percent of the pan evaporation rate, or 24.19 inches per year, was used for the yield analyses. In the rainfall runoff analysis, rainfall on the reservoir itself was also considered. Any rainfall that falls on the reservoir would be transformed directly to storage with no losses. In a typical year, the annual rainfall for this region is greater than the annual evaporation rate, therefore, the rainfall on the reservoir pool would typically offset the storage losses due to evaporation on an annualized type basis (refer to Section 4.2.4, Subbasin Delineation for discussion on accounting for rainfall on the pool.). Evaporation was accounted for in the rainfall runoff modeling process by using a starting reservoir water surface elevation equal to the spillway crest elevation minus the anticipated evaporation from the reservoir over the amount of run time the model reflects, which is typically one year.

4.6.4 Leakage and Water Quality Allowances

A constant outflow of three to seven cubic feet per second (cfs) was provided in all the reservoir routing models except the raised Mayland Lake impoundment alternative. For the raised impoundment alternatives, the outflow was based on the size of the existing outlet works. The existing Mayland Lake dam does not have a low-level outlet. All flow that passes downstream from the dam does so by exiting through the spillway. The raised dam alternative used the same configuration as the existing dam.

The constant outflow was set aside to maintain environmental and water quality aspects. A flow of this magnitude is typically achieved through normal leakage at the dam in addition to direct methods. It is anticipated that a release schedule for water quality flows would be developed in the future for any impoundment alternative carried forward to a feasibility-level study. Once a water quality release schedule is established, the assumptions used to estimate the reservoir yield can be adjusted and a new yield determined. It is also anticipated the water quality releases would require selective withdrawal from the proposed reservoirs. Selective withdrawal is discussed in more detail in Section 4.8.1, Intake Structure and Outlet Works.

The water quality flow was accomplished, from a modeling standpoint, by assuming that an open pipe is located through the dam with an inlet set above the streambed elevation. The pipe was modeled above the streambed elevation to allow for sediment buildup without major impact on water quality releases. For the new impoundment alternatives, the diameter of the pipe was arbitrarily sized to provide a five to seven cfs flow over a normal range of pool elevations. For the raised impoundment alternatives, the diameter of the pipe was sized to provide the same area as the existing low-level outlet of each dam.

4.6.5 Yield Analysis

HEC1-API models are used to simulate rainfall-runoff over a continuous period of time. For this, daily rainfall is applied to the model for the historic drought years (1922-1927, 1942-1944, 1952-1954 and 1986-1988). Prior to running the model for the drought years, a continuous simulation of the reservoir inflow during the year prior to the drought period was made to identify the beginning of the drought. For this, the rainfall-runoff model was started with a reservoir water surface elevation equal to the streambed elevation, thus simulating an empty

reservoir. The last date the reservoir elevation was above the spillway elevation (the last time the reservoir was spilling) was identified. That is the date the depletion of the water supply volume begins due to the drought, and was used as the start date for the yield analysis of each drought.

For the drought years, a starting reservoir water surface elevation equal to the spillway crest elevation minus the anticipated evaporation from the reservoir for the amount of time the model would reflect was used to start the model. The computed pool elevation was checked at each daily computation interval for the simulation periods. If the pumping, leakage and evaporation caused the reservoir water surface to drop to the top of the sediment pool elevation, the daily water supply demand was considered too great and reduced. This process was continued until the maximum yield (water supply volume that can be pumped) was determined.

4.7 RAISING HEIGHT OF EXISTING RESERVOIRS

As previously mentioned, one alternative considered for providing additional water supply to Cumberland County was raising the height of existing dams in the County and using those raised reservoirs for water supply. Raising an existing dam should have less environmental impacts than construction of a new impoundment on a free-flowing stream.

4.7.1 Selection of Existing Reservoirs

A list of reservoirs within Cumberland County was obtained from a database maintained by the State of Tennessee's Division of Water Supply. The list contained 53 reservoirs. The majority of these reservoirs were small in surface area and contributing drainage area. A ratio of expected runoff per square mile of drainage area for Cumberland County was needed to define a minimum drainage area that would be considered for this alternative. The following procedures were used to develop the ratio.

Stream gages on or near the Cumberland Plateau were identified. Precipitation gages in close proximity to the stream gages were also identified. Two sets of precipitation and stream gages in close proximity were found: Collins River stream gage and the Monterey precipitation gage, East Fork of the Obey River stream gage and Jamestown precipitation gage. The close proximity of the stream and precipitation gages was critical to ensure the precipitation records being used

represented the rainfall that fell on the watershed upstream of the stream gage. The use of stream gages on or near the Cumberland Plateau was required to ensure the runoff conditions upstream of the selected gages are similar to those in Cumberland County.

Monthly rainfall data for 1970 to 1996 and 1970 to 1991 were obtained from CD-ROMs containing NWS records for the Monterey and Jamestown gages, respectively. Monthly runoff data were obtained from the USGS publications Water Resources Data for Tennessee for water Years 1970 to 1996 for the Collins River and East Fork of the Obey River gages. The percent of runoff from rainfall for each month of each year was calculated by dividing the monthly runoff total by the total rainfall for the same month. An average of the percent of runoff from rainfall for each month was determined. Any outliers were removed prior to determining the average percent of runoff for each month.

Monthly precipitation records for 1912 to 1997 were obtained from CD-ROMs containing NWS data for the Crossville EXT STN gage. Because of its central position in Cumberland County and its extensive period of record, it was decided the Crossville data would provide an accurate representation of the rainfall data for the County. The Crossville gage data were used to determine average monthly rainfall for each month. The average percent of runoff from rainfall for each month was multiplied by the average rainfall for each month at the Crossville gage to give the estimated monthly runoff in Cumberland County for each month. The monthly estimated runoff values for each month were summed to provide an annual runoff total. The estimated annual runoff was divided by the drainage area of the corresponding stream gage and the results were converted to units of million gallons per day per square mile of drainage area (MGD/mi^2).

These calculations were performed with the runoff data from both stream gages. The calculations for the Collins River stream gage and Monterey precipitation gage resulted in a ratio of 1.34 MGD/mi^2. The calculations for the East Fork of the Obey River stream gage and Jamestown precipitation gage resulted in a ratio of 1.33 MGD/mi^2. It needs to be realized that this ratio does not estimate the yield that can be obtained from an impoundment based upon its drainage area. The ratio simply estimates the maximum amount of runoff that can be obtained in a year from a watershed based upon its drainage area. This yearly amount of runoff is the

maximum yield an impoundment could provide assuming no losses and all runoff could be captured and stored for water supply. Based on the determination that the yearly runoff from a watershed in Cumberland County is 1.34 MGD per square mile of drainage area, all existing reservoirs with drainage areas less than 1.0 mi^2 were not considered for raising since the yield they could provide would be less than 1.34 MGD.

In addition to insufficient drainage area, existing reservoirs were eliminated from consideration if their shoreline was too heavily developed since the houses would have to be purchased. A review of the USGS quadrangle mapping was made to identify reservoirs with heavy shoreline development. After the elimination of reservoirs from consideration for raising based on drainage area and local development, fourteen reservoirs still remained for consideration.

A site visit was conducted to each of the remaining reservoirs. During the site visit nine more reservoirs were eliminated from consideration due to the fact that the dam had been constructed as high as the surrounding geography would permit or due to local development that did not show up on the mapping. The four remaining reservoirs for consideration were: Meadow Park Lake, Mayland Lake, Camp Ozone Lake and Tranquilechee Lake. The following sections discuss the results of the yield analysis for the raised configuration of each of these reservoirs.

4.7.2 Raising Meadow Park Lake

Meadow Park Lake is an existing water supply reservoir located approximately five miles southwest of Crossville, Tennessee. The existing reservoir has approximately 255 acres of surface area at normal pool (elevation 1817.5 feet NGVD29) and a drainage area of 5.19 mi^2. Refer to Figure 4-8.

Data on the current dam configuration were obtained from the State of Tennessee's Safe Dam Inspection reports and a set of plans on proposed modifications to the reservoir. Photo 4-1 is a picture of the existing Meadow Park Lake Dam. Because Meadow Park Lake is one of two reservoirs currently used to provide water for the City of Crossville, a yield analysis on the existing dam configuration was performed to verify the reservoir's current capacity. An HEC1-API model was created to represent the Meadow Park Lake watershed and existing dam

configuration. A yield analysis was performed with the existing condition HEC1-API model and the results indicated the existing dam could provide 3 MGD.

Photo 4-1 – Existing Meadow Park Lake Dam

To identify the maximum yield that could be obtained from raising Meadow Park Lake, the dam configuration was raised as high as the surrounding terrain would permit. The top of the Meadow Park Lake dam can be raised from elevation 1821.5 to 1840.0 feet NGVD29. An HMR52 model was created to determine the Probable Maximum Storm (PMS) for the Meadow Park Lake watershed. An HEC-1 model was then created to utilize the PMS to produce the Probable Maximum Flood (PMF). According to the State of Tennessee's Safe Dam requirements, the raised Meadow Park Lake dam would need to pass the ½PMF event (refer to Table A-3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the raised dam in order to meet the Safe Dam requirements. A 125-foot wide spillway with an invert at elevation 1834.0 feet NGVD29 was required.

The HEC1-API model was modified to represent the raised dam and a yield analysis was performed (refer to Section 4.6.5). The yield analysis indicated the raised dam configuration could provide 4 MGD. The reason for the small increase in yield despite the large increase in

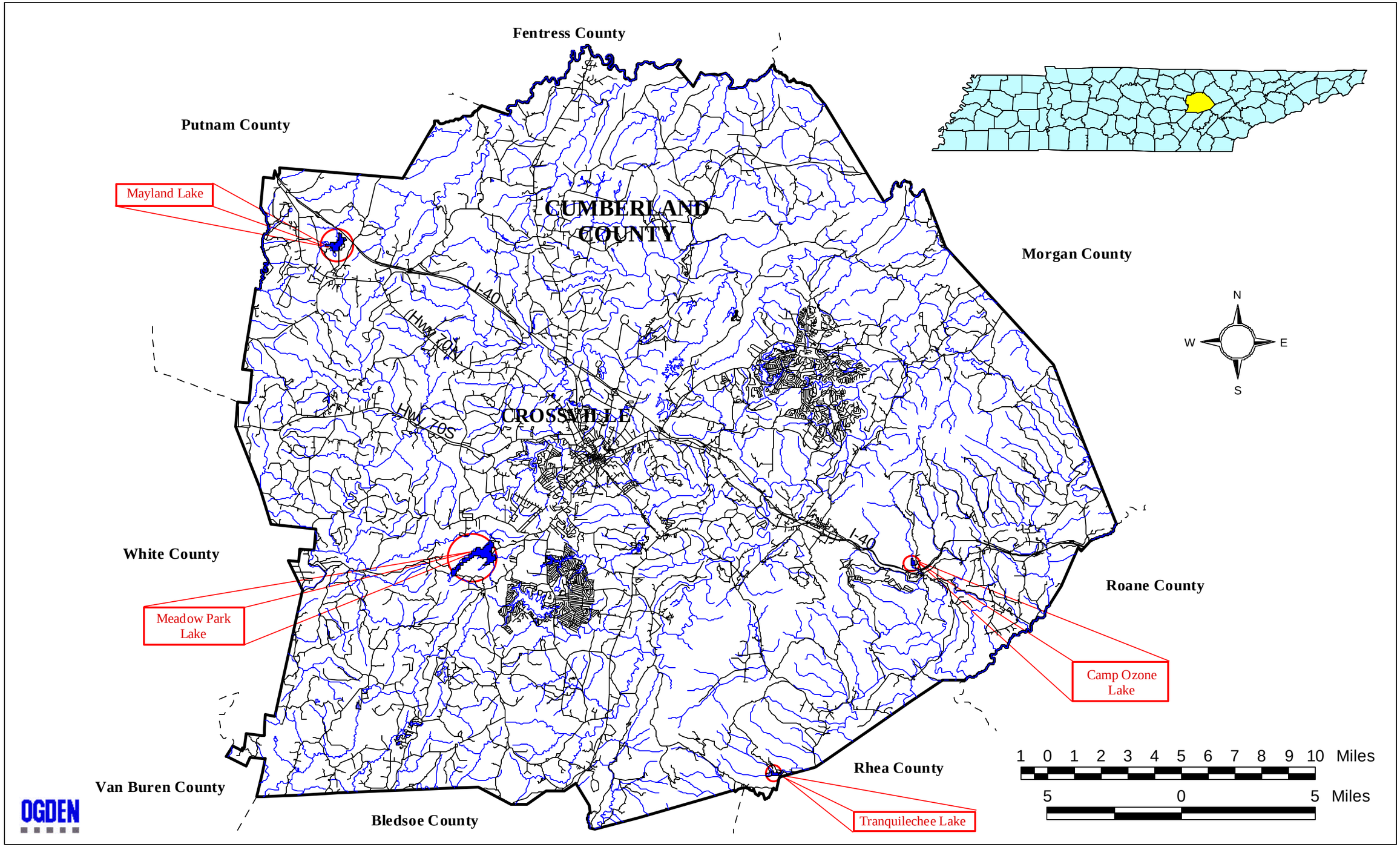


Figure 4-8 Location of Existing Reservoirs for Proposed Raising of Dam Heights.

dam height is the small drainage area of the watershed (5.19 mi^2). The amount of runoff from the watershed is the controlling factor as opposed to the amount of storage provided by the reservoir.

4.7.3 Raising Mayland Lake

Mayland Lake is a recreational lake located just south of Interstate 40 and approximately two and a half miles northwest of Plateau Road. The existing reservoir has approximately 95 acres of surface area at normal pool (elevation 1922.6 feet NGVD29) and a drainage area of 2.76 mi^2. Refer to Figure 4-8.

Data on the current dam configuration was obtained from the State of Tennessee's Safe Dam Inspection reports. Refer to Photo 4-2 for a photograph of the existing Mayland Lake Dam. Because Mayland Lake is a relatively large lake in comparison to others in the county, a yield analysis on the existing dam configuration was performed to define the reservoir's current capacity. An HEC1-API model was created to represent the Mayland Lake watershed and existing dam configuration. A yield analysis was performed with the existing condition HEC1-API model and the results indicated the existing dam could provide 2 MGD.

Photo 4-2 – Existing Mayland Lake Dam

To identify the maximum yield that could be obtained from raising Mayland Lake, the dam configuration was raised as high as the surrounding terrain would permit. The top of the

Mayland Lake Dam can be raised from elevation 1926.4 to 1940.0 feet NGVD29. An HMR52 model was created to determine the PMS for the Mayland Lake watershed. An HEC-1 model was then created to utilize the PMS to produce the PMF. According to the State of Tennessee's Safe Dam Requirements, the raised Mayland Lake Dam would need to pass the ½PMF event (refer to Table A-3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the raised dam in order to meet the Safe Dam requirements. A 150-foot wide spillway with an invert at elevation 1934.5 feet NGVD29 was required.

The HEC1-API model was modified to represent the raised dam, and a yield analysis was performed (refer to Section 4.6.5). The yield analysis indicated the raised dam configuration could provide 2 MGD. The reason there was no increase in yield despite the increase in dam height is the small drainage area of the watershed (2.76 mi^2). The amount of runoff from the watershed is the controlling factor as opposed to the amount of storage provided by the reservoir.

4.7.4 Raising Camp Ozone Lake

Camp Ozone Lake is a recreational lake located just north of Interstate 40 and approximately 3000 feet northwest from the post office in Ozone, Tennessee. The existing reservoir has approximately 13 acres of surface area at normal pool (elevation 1660.0 feet NGVD29) and a drainage area of 3.98 mi^2. Refer to Figure 4-8. Data on the current dam configuration was obtained from the State of Tennessee's Safe Dam Inspection reports. Due to its relatively small size, the yield of the existing dam was not calculated.

To identify the maximum yield that could be obtained from raising Camp Ozone Lake, the dam configuration was raised as high as the surrounding terrain would permit. The top of the Camp Ozone Lake dam can be raised from elevation 1680.0 to 1739.5 feet NGVD29. An HMR52 model was created to determine the PMS for the Camp Ozone Lake watershed. An HEC-1 model was then created to utilize the PMS to produce the PMF. According to the State of Tennessee's Safe Dam requirements, the raised Camp Ozone Lake dam would need to pass the ½PMF event (refer to Table A-3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the raised dam in order to meet the Safe Dam requirements. A 200-foot wide spillway with an invert at elevation 1734.0 feet NGVD29 was required.

An HEC1-API model was created to represent the raised dam, and a yield analysis was performed (refer to Section 4.6.5). The yield analysis indicated the raised dam configuration could provide 1 MGD. The reason for the small yield is the small amount of storage provided by the reservoir. The valley that the proposed reservoir would fill is narrow and does not provide adequate storage.

4.7.5 Raising Tranquilechee Lake

Tranquilechee Lake is a undeveloped residential lake located approximately 4000 feet north of the Cumberland and Bledsoe County line and four and a half miles southwest of Grassy Cove, Tennessee. The existing reservoir has approximately 30 acres of surface area at normal pool (elevation 1452.5 feet NGVD29) and a drainage area of 4.30 mi^2. Refer to Figure 4-8. Due to its relatively small size, the yield of the existing dam was not calculated.

Tranquilechee Lake was eliminated from consideration for raising the dam height to provide water supply after a review of the USGS Grassy Cove Quadrangle Map indicated the valley the raised reservoir would fill is quite narrow. The results of a yield analysis on the raised dam configuration would be similar to the results from the raised Camp Ozone Lake dam discussed above.

4.7.6 Raising Height of Existing Reservoirs - Summary

As the results discussed above indicate, raising the height of existing reservoirs in Cumberland County does not provide a large amount of water supply. The majority of the larger reservoirs in the County that have higher potential for providing a substantial amount of water supply are too heavily developed along their shorelines to raise the dam heights. The dams that can be raised have been constructed too near the headwaters of the streams to have drainage areas large enough to provide sufficient runoff. Due to these facts, no cost estimates were prepared for the raised impoundment alternatives.

4.8 NEW IMPOUNDMENTS

As previously mentioned, construction of new impoundments is an alternative considered for providing additional water supply to Cumberland County. Construction of a new water supply impoundment allows the selection of a site that does not have the same problems the reservoirs

analyzed in the raised impoundment alternative did (insufficient drainage area and/or storage area). The following sections discuss each site considered, the sizing of each dam and results from the yield analysis performed on each site.

4.8.1 Clear Creek

As part of their water supply study for the Catoosa Utility District (provides water to the northwestern portion of Cumberland County), the Tennessee Valley Authority (TVA) designed a water supply dam on Clear Creek. The proposed dam location and spillway orientation are illustrated on Figure 4-9. Clear Creek is a tributary of the Obed River, a portion of which is protected by the Wild and Scenic Rivers Act. Construction of an impoundment on a tributary to the Obed River would pose serious environmental concerns. However, because it had been listed as a potential alternative in a previous study, the Clear Creek dam was included in this study.

To identify the maximum yield that could be obtained from a new impoundment on Clear Creek at the proposed site, the top of dam was set as high as the surrounding terrain would permit. The proposed dam is approximately 72 feet in height, with the top of dam at elevation 1819.5 feet NGVD29. An HMR52 model was created to determine the PMS for the Clear Creek watershed upstream of the proposed site. An HEC-1 model was then created to utilize the PMS to produce the PMF. According to the State of Tennessee's Safe Dam requirements, the Clear Creek Dam would need to pass the ½ PMF event (refer to Table A -3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the dam in order to meet the Safe Dam requirements. A 250-foot wide spillway with an invert at elevation 1814.0 feet NGVD29 was required. The proposed dam design is contained in Table 4-6 below. The reservoir area at normal pool (spillway elevation) would be 155 acres.

Table 4-6
Proposed Clear Creek Dam Configuration

Top of Dam (Elev., feet NGVD29)	Length of Dam (feet)	Spillway Invert (Elev., feet NGVD29)	Spillway Width (feet)
1819.5	500	1814.0	250

An HEC1-API model was created to perform a yield analysis (refer to Section 4.6.5) on the proposed Clear Creek reservoir. The proposed Clear Creek reservoir design would provide a water supply yield of 3 MGD.

Clear Creek Intake Structure and Outlet Works

In order to calculate a cost estimate for the Clear Creek impoundment that are representative of a cost for the construction of the entire alternative (as opposed to just the dam), reconnaissance-level designs of an intake structure and outlet works were required.

The proposed intake structure for the Clear Creek Dam consists of a 6-feet x 6-feet concrete standpipe on the upstream side of the dam that connects to a 4-feet x 4-feet box culvert that extends through the dam. The top of the intake structure would be at Elevation 1814.0 feet NGVD29 (spillway crest elevation). Three sluice gates would be provided for selective withdrawal. Selective withdrawal is provided based on the assumption the reservoir would stratify during portions of the year, and selective withdrawal would be required to obtain the higher quality water from the reservoir. A water quality study would be performed on any of the impoundment alternatives carried forward to a feasibility-level study. The water-quality study would determine if selective withdrawal is required, and, if so, the number of intakes that would be required. A steel walkway would provide access to the intake structure from the dam. The intake tower would connect to the box culvert near the upstream toe of the embankment.

The box culvert serves two purposes. The culvert would serve as a means of diversion of flow during construction. Typically, a seasonal frequency analysis is performed to develop discharges which can be used to size a diversion alternative. Such an analysis is beyond the scope of this study. The size of the diversion culvert for the proposed Clear Creek Dam was computed based on a similar water supply dam being designed for the city of Cullman, Alabama. In the Cullman Water Supply Study, a diversion tunnel was sized based on providing protection up to a 10-year frequency event. The required flow area for the Clear Creek diversion culvert was determined by multiplying the Cullman flow area by the ratio of the Clear Creek to Cullman drainage areas. The 4-feet x 4-feet box culvert was set as the minimum size culvert that would be used; the required flow area from the above mentioned calculation is less than the 16 ft^2 provided. It should be noted these are merely cursory computations, only intended to provide an estimate of

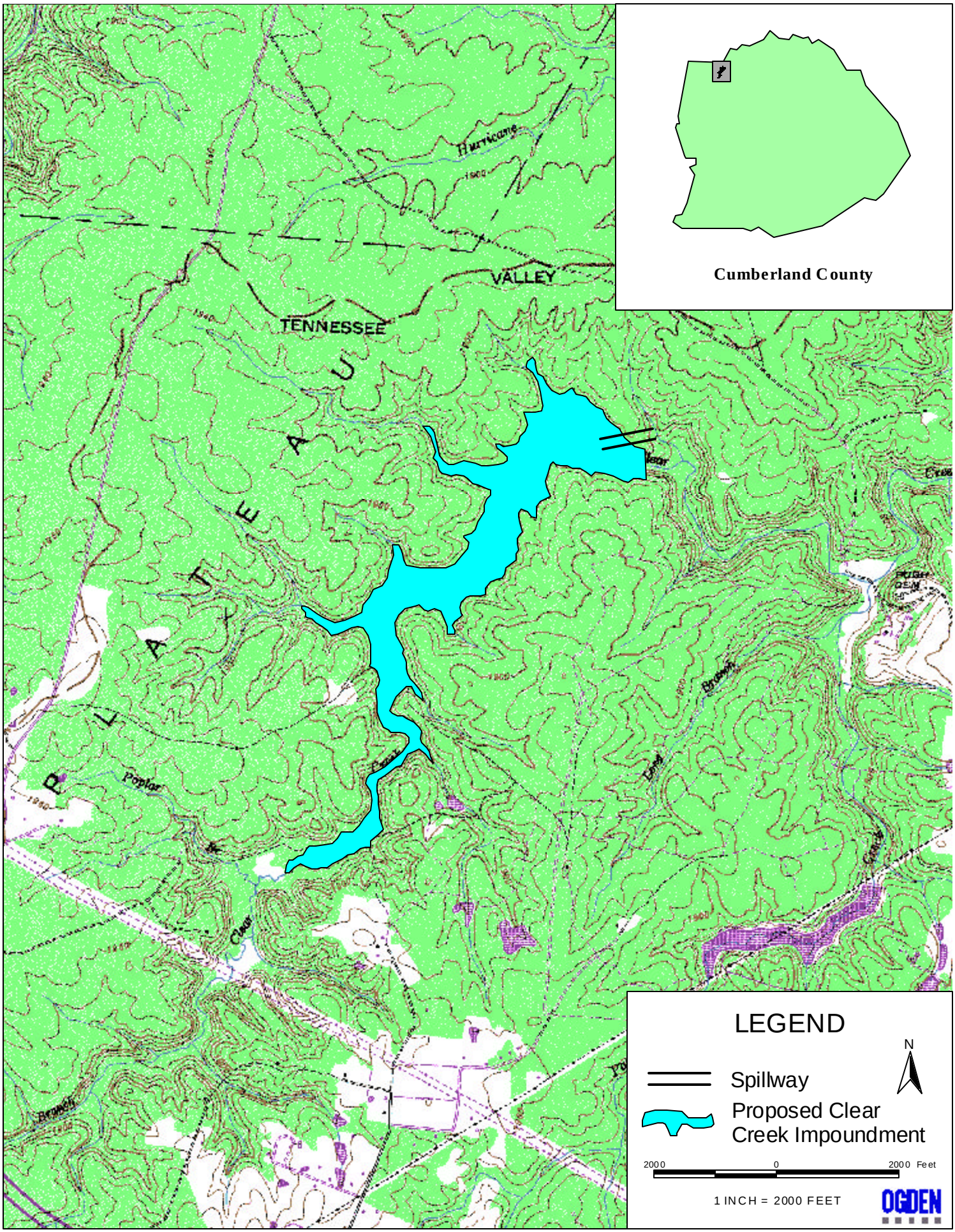


Figure 4-9 Proposed Clear Creek Impoundment

the size of the diversion culvert. A detailed analysis of construction diversion schemes would be analyzed for any of the impoundment alternatives carried forward to a feasibility-level study.

The culvert would also serve as a conduit through the dam for the water quality and water supply releases. As designed, water would be released from the reservoir from one of the three sluice gates and flow from the intake structure through the culvert to the downstream channel. A portion of the flow would continue downstream for water quality while the rest would be pumped to the treatment plant.

Pump Station and Pipeline

For this study it was assumed water supply flows from each reservoir would be pumped to the current water treatment plant located near Lake Holiday (Crossville, Tennessee). To provide for peak demand, which is typically estimated as 150% of the daily yield, the pump station downstream of the proposed Clear Creek Dam and the pipeline to the treatment plant were required to supply 4.5 MGD of raw water.

The proposed pipeline route, illustrated in Figure 4-10, was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways. Additional information on the design of the pump station and pipeline can be found in Section 4.8.6.

4.8.2 Meadow Creek (above Meadow Creek Dam)

Based on a review of the USGS quadrangle maps that cover Cumberland County, a dam built on Meadow Creek at the Cumberland County and Putnam County line appeared to have the potential to provide a substantial amount of water supply. The proposed dam location is illustrated on Figure 4-11. The existing Meadow Creek Dam is not shown on Figure 4-11 because it is not included on the USGS Campbell Junction Quadrangle Map.

Two problems were identified with this proposed dam site. The first problem identified is the Interstate 40 (I-40) crossing over Meadow Creek. The crossing consists of two 10 feet x 10 feet box culverts (according to Tennessee Department of Transportation (TDOT) files). A preliminary dam was designed to provide the maximum yield possible, thus requiring the top of dam to be located as high as the topography of the site would permit. However, based on TDOT

information on the Interstate 40 crossing, the dam would back water approximately 10.8 feet over the top of the culverts at the normal pool elevation. The dam would also impound water over I-40 during the design event (1/2 PMF). A new design would be required with a lower top of dam and lower spillway invert, which would substantially reduce the yield provided.

The second and more critical problem identified is the existence of a dam downstream of the proposed site that was built by the City of Monterey for water supply. As previously mentioned, this reservoir is not shown on the current USGS Campbell Junction Quadrangle Map. Building a dam upstream would capture the majority of the flow that is currently flowing into the existing dam, thus substantially depleting the water supply at the Meadow Creek Dam.

Due to these problems, the construction of a new impoundment on Meadow Creek at the Cumberland County and Putnam County line was not carried forward as an alternative in the study process.

4.8.3 Meadow Creek (below Meadow Creek Dam)

As discussed in Section 4.8.2 above, the City of Monterey has constructed a water supply dam on Meadow Creek. The dam is located immediately south of Highway 62 and approximately a mile north of the Cumberland and Putnam County line (refer to Figure 4-12). The actual dam and reservoir are not shown on Figure 4-12 because they are not included on the USGS Campbell Junction Quadrangle Map. Photo 4-3 provides a photograph of the existing Meadow Creek Dam. During the site visit to the existing structure it appeared a new dam could be built downstream of the current dam at a higher height. Review of the USGS Campbell Junction Quadrangle Map confirmed a new dam built approximately 200 feet downstream of the existing structure could be constructed with a top of dam at elevation 1799.5 feet NGVD29, which is estimated to be 30 feet higher than the existing top of dam.

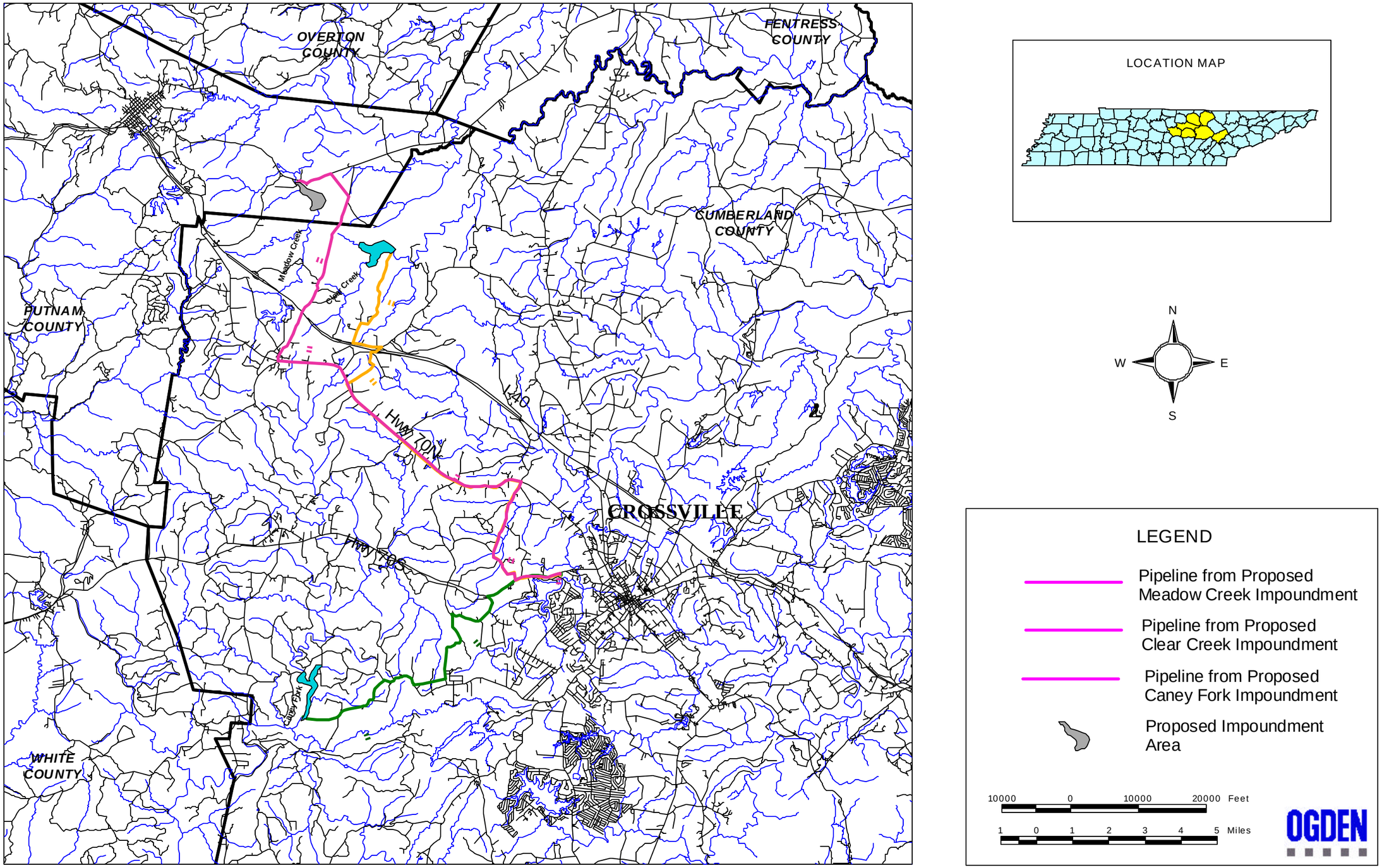


Figure 4-10 Pipelines from Proposed New Impoundments

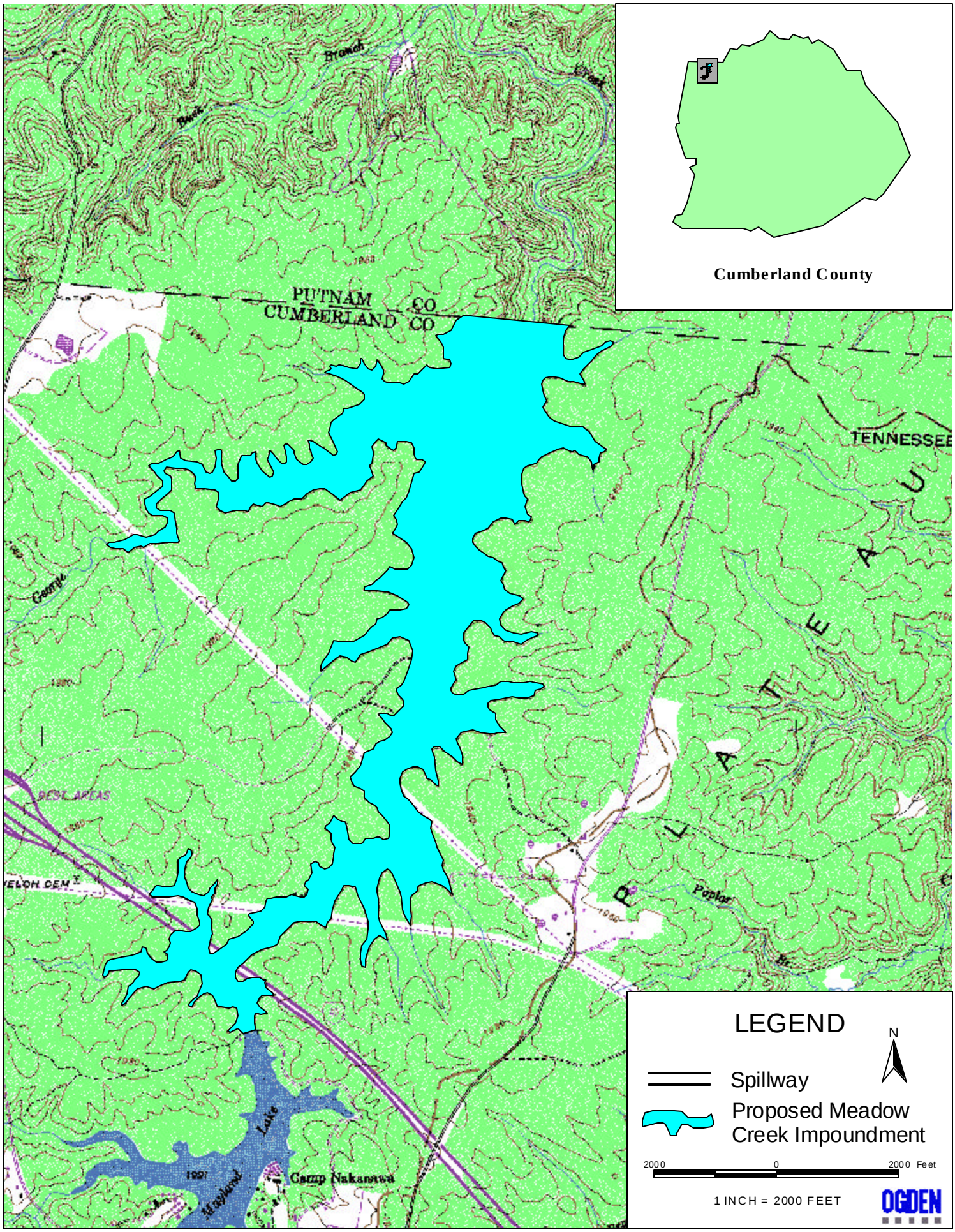


Figure 4-11 Proposed Meadow Creek Impoundment (Above Monterey Dam)

Photo 4-3 – Existing Meadow Creek Dam

The proposed new dam is approximately 90 feet in height. An HMR52 model was created to determine the PMS for the Meadow Creek watershed upstream of the proposed site. An HEC-1 model was then created to utilize the PMS to produce the PMF. According to the State of Tennessee's Safe Dam Requirements, the new Meadow Creek Dam would need to pass the full PMF event (refer to Table A-3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the dam in order to meet the Safe Dam requirements. A 300-foot wide spillway with an invert at elevation 1787.0 feet NGVD29 was required. The proposed dam design is contained in Table 4-7 below. The ponding area at normal pool (spillway elevation) would be 273 acres.

Table 4-7
Proposed Meadow Creek Dam Configuration

Top of Dam (Elev., feet NGVD29)	Length of Dam (feet)	Spillway Invert (Elev., feet NGVD29)	Spillway Width (feet)
1799.5	1500	1787.0	300

An HEC1-API model was created to perform a yield analysis (refer to Section 4.6.5) on the proposed new Meadow Creek Dam. The proposed Meadow Creek dam design would provide a water supply yield of 7 MGD. The yield of 7 MGD includes the yield that is already provided by the existing Meadow Creek Dam, which was not determined during this study. It should be noted that the construction of this dam would require water supply negotiations between the City of Monterey and Cumberland County.

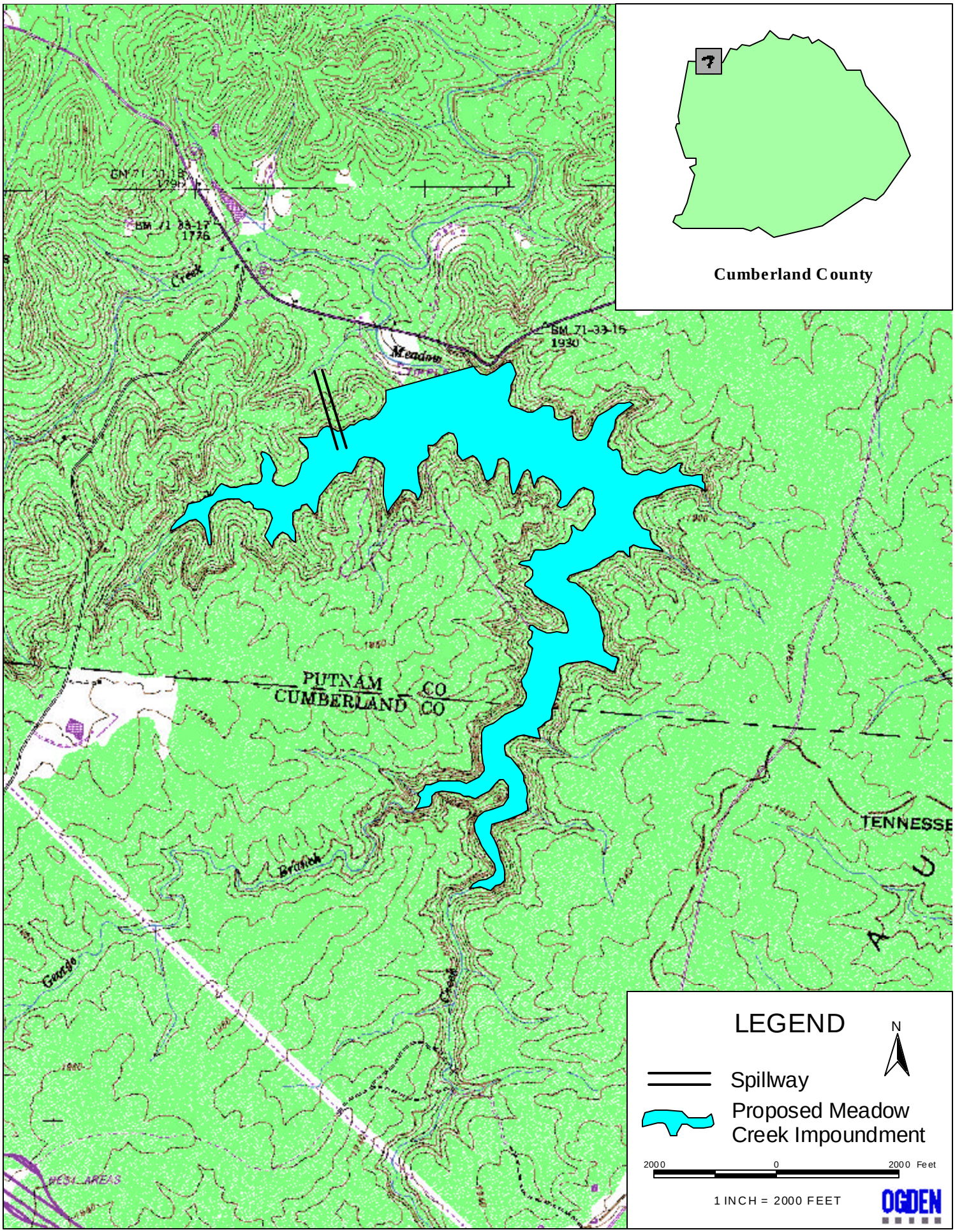


Figure 4-12 Proposed Meadow Creek Impoundment
(Below Monterey Dam)

Intake Structure and Outlet Works

In order to calculate a cost estimate for the new Meadow Creek impoundment that represented a cost for the construction of the entire alternative (as opposed to just the dam), reconnaissance-level designs of an intake structure and outlet works were required.

The proposed intake structure and diversion culvert for the new Meadow Creek dam are the same as those designed for the proposed Clear Creek Dam (refer to Section 4.8.1, Intake Structure and Outlet Works). The top of the 6-feet x 6-feet intake structure would be at Elevation 1787.0 feet NGVD29 (spillway crest elevation). The same assumptions regarding selective withdrawal were also made for the new Meadow Creek dam as those made for the Clear Creek dam. A steel walkway would provide access to the intake structure from the dam. The intake tower would connect to the box culvert near the upstream toe of the embankment.

The 4-feet x 4-feet box culvert would serve two purposes; diversion of flow during construction and a conduit through the dam for the water quality and water supply withdrawals. The size of the diversion culvert for the proposed new Meadow Creek Dam was computed based on the same assumptions and techniques used to size the Clear Creek diversion culvert (refer to Section 4.8.1, Intake Structure and Outlet Works). As previously mentioned, these are merely cursory computations, only intended to provide an estimate of the size of the diversion culvert. A detailed analysis of construction diversion schemes would be analyzed for any of the impoundment alternatives carried forward to a feasibility-level study.

As mentioned, the culvert would also serve as a conduit through the dam for the water quality and water supply releases. As designed, water would be released from the reservoir from one of the three sluice gates and flow from the intake structure through the culvert to the downstream channel. A portion of the flow would continue downstream for water quality while the rest would be pumped to the treatment plant.

Pump Station and Pipeline

For this study it was assumed water supply flows from each reservoir would be pumped to the current water treatment plant located near Lake Holiday (Crossville, Tennessee). To provide for peak demand, which is typically estimated as 150% of the daily yield, the pump station

downstream of the proposed new Meadow Creek Dam and the pipeline to the treatment plant were required to supply 10.5 MGD of raw water.

The proposed pipeline route, illustrated in Figure 4-10, was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways. Additional information on the design of the pump station and pipeline can be found in Section 4.8.6.

4.8.4 Meadow Park Lake

During the site visit to determine the feasibility of raising Meadow Park Lake Dam, it appeared a new dam could be built downstream of the existing structure at a higher height. Review of the USGS Crossville Quadrangle Map confirmed that a new dam built approximately 300 feet downstream of the existing Meadow Park Lake Dam could be constructed with a top of dam at elevation 1859.5 feet NGVD29, which is approximately 20 feet higher than the existing dam could be raised.

Prior to the yield analysis on the raising of Meadow Park Lake Dam, it appeared construction of a new impoundment downstream of the existing dam would be a viable water supply alternative. However, as previously mentioned, the results of the yield analysis on the raising of Meadow Park Lake Dam revealed that the reservoir's watershed is too small to contribute sufficient runoff to support a larger reservoir.

4.8.5 Caney Fork

As part of a water supply study for the City of Crossville, Lamar Dunn & Associates Inc. (LD&A) of Knoxville, Tennessee proposed a water supply dam on the Caney Fork. The proposed site is located a mile and a half east of Clifty, Tennessee and 4000 feet west of Bruce Knob (refer to Figure 4-13). Photo 4-4 provides a photograph of the Caney Fork near the proposed site.

Photo 4-4 – Caney Fork Near Site of the Proposed Caney Fork Impoundment

The proposed dam design is smaller than that contained in the LD&A report because of the U.S. Highway 70 crossings upstream of the proposed site. The dam had to be sized to avoid impounding water over U.S. Highway 70 at the Caney Fork, Beam Creek and Tantrough Creek crossings. Drawings of each of these crossings were obtained from TDOT, and the proposed top of dam was set at an elevation that would not exceed the roadway elevation at these crossings. Photo 4-5 provides a photograph of the U.S. Highway 70 crossing over the Caney Fork.

Photo 4-5 – U.S. Highway 70 Crossing Over the Caney Fork

The proposed dam is approximately 123 feet in height, with the top of dam at Elevation 1665.0 feet NGVD29. An HMR52 model was created to determine the PMS for the Caney Fork watershed upstream of the proposed site. An HEC-1 model was then created to utilize the PMS to produce the PMF. According to the State of Tennessee's Safe Dam requirements, the Caney Fork dam would need to pass the full PMF event (refer to Table A-3 in Appendix A) without overtopping. The HEC-1 model was used to size the spillway of the dam in order to meet the Safe Dam requirements. A 300-foot wide spillway with an invert at Elevation 1635.0 feet NGVD29 was required. The proposed dam design is contained in Table 4-8. The reservoir area at normal pool (spillway elevation) would be 272 acres.

Table 4-8
Proposed Caney Fork Dam Configuration

Top of Dam (Elev., feet NGVD29)	Length of Dam (feet)	Spillway Invert (Elev., feet NGVD29)	Spillway Width (feet)
1665.0	700	1635.0	300

An HEC1-API model was created to perform a yield analysis (refer to Section 4.6.5) on the proposed Caney Fork Dam. The proposed Caney Fork Dam design would provide a water supply yield of 12 MGD.

Intake Structure and Outlet Works

In order to calculate a cost estimate for the Caney Fork impoundment that represented a cost for construction of the entire alternative (as opposed to just the dam), reconnaissance-level designs of an intake structure and outlet works were required.

The proposed intake structure for the Caney Fork Dam consists of a 10-feet x 10-feet concrete standpipe on the upstream side of the dam that connects to an 18-feet x 18-feet box culvert that extends through the dam. The top of the intake structure would be at elevation 1635.0 feet NGVD29 (spillway crest elevation). Based on the water quality assumptions stated in Section 4.8.1, Intake Structure and Outlet Works, three sluice gates would be provided for selective withdrawal. A steel walkway would provide access to the intake structure from the dam. The intake tower would connect to the box culvert near the upstream toe of the embankment.

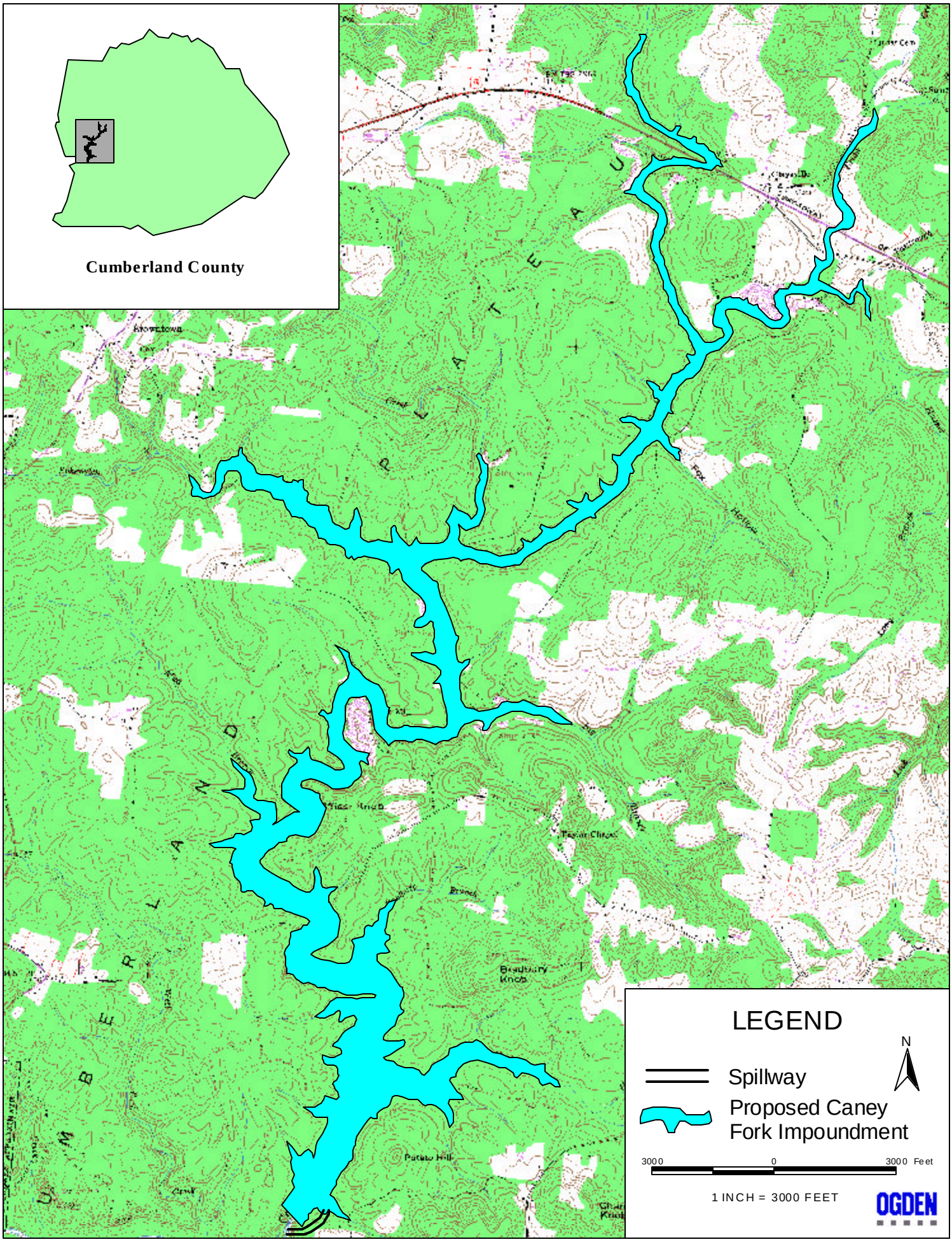


Figure 4-13 Proposed Caney Fork Impoundment

The 18-feet x 18-feet box culvert would serve as a means of diversion of flow during construction. The size of the diversion culvert for the proposed Caney Fork Dam was computed based on the same assumptions and techniques used to size the Clear Creek diversion culvert (refer to Section 4.8.1, Intake Structure and Outlet Works). As previously mentioned, these are merely cursory computations, only intended to provide an estimate of the size of the diversion culvert. A detailed analysis of construction diversion schemes would be analyzed for any of the impoundment alternatives carried forward to a feasibility-level study.

The culvert would also serve as a conduit through the dam for the water quality and water supply releases. As designed, water would be released from the reservoir from one of the three sluice gates and flow from the intake structure through the culvert to the downstream channel. A portion of the flow would continue downstream for water quality while the rest would be pumped to the treatment plant.

Pump Station and Pipeline

For this study it was assumed water supply flows from each reservoir would be pumped to the current water treatment plant located near Lake Holiday (Crossville, Tennessee). To provide for peak demand, which is typically estimated as 150% of the daily yield, the pump station downstream of the proposed Caney Fork Dam and the pipeline to the treatment plant were required to supply 18 MGD of raw water.

The proposed pipeline route, illustrated in Figure 4-10, was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways. Additional information on the design of the pump station and pipeline can be found in Section 4.8.6.

4.8.6 Electrical/Mechanical Designs

General

The raw water pipelines were engineered conservatively to reduce friction loss and ultimately save energy throughout the life of the project. The alignment was chosen to minimize construction costs by utilizing existing routes, i.e., right of way (ROW) along highways to the filtration plant.

Static Conditions

The static head was calculated using low water elevations at the source or flow line of the stream and the nearest high ground along the pipeline or ground profile. The maximum static head would be encountered during start-up or priming of the system. By using siphonic recovery throughout the system, part of the priming head can be negated since the filtration plant is at Elevation 1800.0. These refinements should be considered in subsequent designs.

Dynamic Conditions

The dynamic flow involves velocity head loss and friction loss. The velocity head was minute in most cases and dropped from further consideration. The friction loss was taken from published data using standard schedule 40 steel pipe. By conservatively sizing the pipelines, the water velocity was kept low, which resulted in low friction losses and less power required. Again, future refinements may decrease overall installation and life cycle costs.

Pumping Units

The pumping units would be capable of priming the system and pumping consistently through all changes of the intake and/or discharge conditions. The pumping capacity would be divided between two or three similar pumping units, providing partial capacity in case of unit failure. Sufficient stages would allow the pumping units to prime the piping systems and achieve rated capacity.

Electrical Service

An electric service would be required at each pumping station. The necessary service requirements and possible electric line modifications or extensions would be coordinated with the electrical utility company.

Electric Motors

The pumping units would be driven by electric motors of sufficient horsepower to perform at all conditions. One (1) electric motor at each location should be variable speed, which would provide total capacities between minimum and maximum flows. A power source would be required for the pumping units.

Control Systems

The automatic control system would activate the pumping units in predetermined sequence to deliver the water as needed.

Pipelines

The pipelines would be constructed of schedule 40 steel pipe coated and cathodically-protected along the entire length. Bury of each pipe would be three (3) foot to prevent freezing of the product. Refinements of the design should include siphon assist when available to take advantage of energy recovery. The pipeline alignment generally follows existing ROW along highways, except for approximately two (2) miles at the intake facilities, which is overland requiring new construction. Thrust backing would be provided at all directional changes.

4.8.7 Geology

All of the sites are located on the Cumberland Plateau on Lower Pennsylvanian rocks of the Crab Orchard Mountains (Pco) and Gizzard (Pg) Groups. Listed in order of prevalence, the Crab Orchard Mountains Group consists of conglomerate, sandstone, siltstone, shale and coal. The Gizzard Group consists of shale, siltstone, sandstone and conglomerate.

"The Cumberland Plateau is the name given to the southern portion of the structural province known as the Appalachian Plateaus. It is marked on the east side by a prominent escarpment, but its western margin is more irregular. The average height of these escarpments is 900 feet. Although essentially flat throughout most of its extent, it has some rolling terrain in places an mountains that rise above the general plateau level from eastern Cumberland County northeastward to Claiborne County. The general plateau elevation is approximately 1700 to 1900 feet."

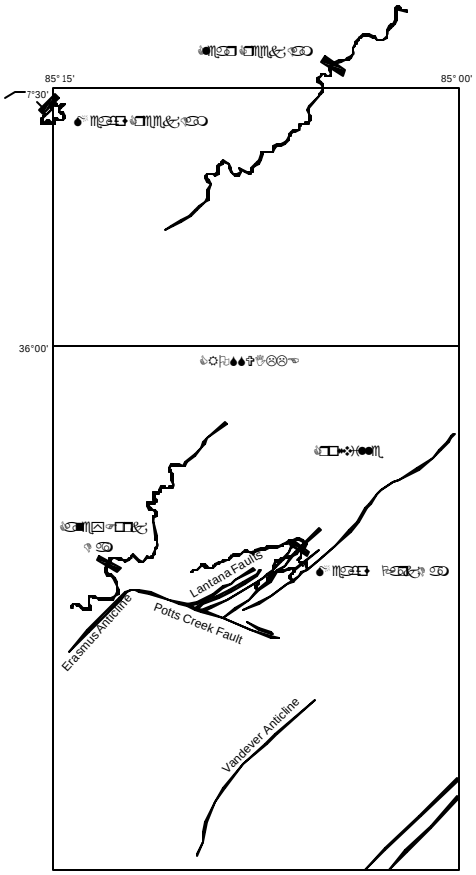
"The same mountain building forces that built the Southern Appalachians and deformed the rocks of the Valley and Ridge formed the structures of the Cumberland Plateau. Rocks along the eastern escarpment of the plateau and for many miles westward along some zones were extensively faulted and folded." (Excerpts from The Geologic History of Tennessee by Robert A. Miller)

"Northwest and north of the Crab Orchard Mountains is a long continuous belt of intense structural deformation. This belt consists chiefly of a series of en-echelon southeastward dipping

thrust faults connected by cross faults and associated anticlines. This belt has been mapped from the Lantana area on the southwest to the edge of the Valley and Ridge province at Elverton and Harriman. The thrust faults in the belt strike northeast and their planes dip southeast. Southeastward dip of the fault planes ranges from less than 10 degrees to about 50 degrees. Thrust faults occur generally in zones of restricted width within which there are several faults. Thrust zones are generally marked by prominent ridges due to dip of resistant sandstone beds." (Excerpts from The Cumberland Plateau Overthrust and Geology of the Crab Orchard Mountains Area, Tennessee by Richard G. Stearns)

These fault related zones include the Crossville fault, Potts Creek fault, Lantana faults, and the Erasmus anticline. They are located near the town of Crossville, Tennessee and the sites of the Meadow Park Dam and the proposed Caney Fork Dam. Relative locations are shown on Figure 4-14.

Figure 4-14 Structural Features and Dam Sites
(Drawing Modified from The Cumberland Plateau Overthrust and Geology of the Crab Orchard Mountains Area.)



Clear Creek Dam

The site of the proposed Clear Creek Dam was not visited. However, it is near other sites that were visited. Based upon geologic maps of the area and observations at other sites it is assumed that bedrock at the Clear Creek Dam site would be sandstone and associated rocks of the Crab Orchard Mountains Group. Foundation treatment requirements should be about the same as for the other sites studied. There are no known fault zones in the immediate area.

Meadow Park Dam

The foundation of the existing Meadow Park Dam is sandstone of the Crab Orchard Mountains Group. Overburden covering bedrock appears to be thin layer of silty-sandy soil. The sandstone is very hard, medium- to thick-bedded, and cross-bedded. The dip of the bedding is highly variable in the area. The Crossville Fault and Lantana Faults occur nearby. Meadow Creek flows along the approximate alignment of one of the Lantana Fault lines and the existing Meadow Park Dam was constructed across this line. There are no apparent foundation problems with the existing dam. Seepage through the foundation and abutments appears to be almost non-existent.

Meadow Creek (below Meadow Creek Dam)

Bedrock in the area is sandstone and associated rocks of the Crab Orchard Mountains Group. The foundation of the existing Monterey Dam is sandstone at the lower levels. Shale and coal also occur in the area and there is some history of coal mining in the vicinity of the dam. A substantial amount of seepage flows from a point on the left bank approximately 800 feet downstream of the dam. Flow estimated to be 50 gallons per minute comes from a point near the contact between overburden and top of rock. This appears to be in about the same area as coal tipple, now removed, which is shown on USGS topographic maps. The reason for the seepage is unknown. It could be related to coal mining activity in the area and should be investigated to determine the implications for new dam construction. There are no known fault zones in the immediate area.

Caney Fork Dam

Bedrock at the site of the proposed Caney Fork Dam is sandstone and related rocks of the Crab Orchard Mountains and Gizzard Groups. The creek bottom is covered with sandstone gravel-

boulders. The dam foundation would probably be sandstone, with shale possibly occurring at the lowest levels along the creek. The dam site is near the Erasmus Anticline and Potts Creek Fault. The effect of these features on dam construction, if any, is unknown.

4.8.8 Geotechnical Design

The sites for the new dams (except for Meadow Creek, which is located adjacent to the Cumberland County line) are located in Cumberland County. Because of budget constraints, no explorations could be performed. Published literature and site visits were the only information available to determine the conceptual designs for the dams and foundations.

As described in the geology section (Section 4.8.7), Cumberland County is located in the Cumberland Plateau and is generally flat through most of its extent with some rolling terrain in places and mountains that rise above the general plateau level in the eastern part. A soil survey (Series 1938, No. 25, issued April 1950) shows that the Hartsell soils cover much of the County. This soil formation is described as a fine sandy loam to a fine sandy clay and generally three feet thick. Sandstone underlies this thin overburden.

Assumptions

All the sites selected for new dams require a spillway to pass the selected design flood. The material excavated for the spillway would be used to construct the dam.

Given the above information the following assumptions were made in determining the type of structure to be constructed in generating a typical design.

1. Excavated material from the spillway would be 80% rock and 20% overburden.
2. Because the excavated material would be predominately rock, a rockfill dam would be constructed.
3. Overburden material from the spillway would be clay and can be used as the impervious core material.
4. Additional clay material (if needed) can be obtained from borrow areas less than a mile away.
5. Additional rock material (if needed) can be obtained by widening the spillway.
5. The overburden under the dam footprint is five feet thick and would be excavated and removed.
6. A concrete culvert would be used to divert the river during construction and maintain low flow after construction is complete.

Typical Design

Given the above assumptions a typical section (see Figure 4-15) for a rockfill dam was developed and can be used at any of the sites investigated. As shown on this section, the width of the clay core at the base (in the deepest section) would be about one half of the height. It would be flanked on both sides by a filter system 10 feet wide. The filter system will consist of a sand filter adjacent to the core and transition filter composed of a sandy gravel between the sand filter and the rock fill. The width of each filter will be five (5) feet. The filters will come from a commercial source, probably in the City of Cookeville. Rockfill from the excavation would be used in the outer shells. Side slopes on both the upstream and downstream sides would be 1V to 2H, and the dam would have a 20-foot top width. A core trench five (5) feet deep (into rock) and 20 feet wide would also be constructed. It should be noted that no explorations or borings have been made to date and that this section could be altered substantially or even make a rockfill dam not feasible.

4.8.9 Foundation

It is assumed that the impervious core trench of each dam would be founded on sandstone. The core trench would extend five feet below top of rock. The foundation would be grouted to a depth below the surface of the rock approximately equal to the reservoir head above the surface of the rock. Initially, a single line grout curtain would be installed beneath the core trench. Primary holes would be 1.5-inches in diameter and drilled on ten foot spacing. Additional grout holes would be added as needed, depending on grout takes in the primary holes. Grout take is estimated to be 0.5 cubic feet of grout per foot of drill hole.

The core trench would be excavated by blasting. The 1H:1V side slopes would be pre-split. Dental excavation, grout and concrete would be used to shape and seal final rock surfaces and make them suitable for the placement of impervious fill and filter material. Spillway rock slopes would be vertical, and would be pre-split prior to production blasting.

Rock reinforcement and protection would be needed primarily in the spillways. One-inch diameter, 10-12 foot long, resin anchored, post-tensioned rock bolts would be used. Shotcrete protection would be used in areas within 50 feet either side of the spillway sill, plus an estimated

10% of the remainder of spillway slopes. Shotcreted slopes would be drained with drain holes lined with slotted PVC pipe and strip drains.

References

Information for Section 4.8.7 and 4.8.9 New Impoundments Foundation was obtained from the following sources:

1. The Cumberland Plateau Overthrust and Geology of the Crab Orchard Mountains Area, Tennessee by Richard G. Stearns; Tennessee Division of Geology Bulletin #60;

2. The Geologic History of Tennessee; Tennessee Division of Geology Bulletin #74;

3. Geologic Map of Tennessee, East-Central Sheet; Tennessee Division of Geology; and

4. Geologic Map of the Crossville Quadrangle, Tennessee.

Figure 4-15 Typical Dam Section

4.9 WATER HARVESTING ALTERNATIVE

The basis of water harvesting is to pump water from a stream during high-flow events and store that water in an off-site detention area for future use. While similar in concept to a water supply reservoir, water harvesting poses less of an environmental impact on the stream because it remains free-flowing.

Two methods of water harvesting were investigated in this study. The first is the "traditional" definition of pumping water from a stream during high flows and storing it in a newly constructed detention area. The second method consisted of pumping water during high-flow events on the Caney Fork and storing it in a raised Meadow Park Lake reservoir.

4.9.1 "Traditional" Water Harvesting

Water harvesting as a water supply alternative is a new concept for Tennessee. The "traditional" water harvesting that has been referred to has for the most part been used for agricultural uses. To achieve a better understanding of the water harvesting concept, a prototype water harvesting system in Huntsville, Alabama was visited.

The system consisted of an 800 gallons per minute (gpm) pump at an intake structure that pumped water to a detention area constructed from a 15 foot tall ring levee. Refer to Photo 4-6 for a photograph of the intake structure and to Photo 4-7 for a photograph of the detention area. The detention area had approximately 13 acres of surface area and a volume of 140 acre-feet (45.6 million gallons) at the top of the pool (4 feet below the top of the levee). The detention area had been lined with a polyethylene liner to ensure no infiltration losses. Permitting for the system allowed pumping from the creek during the "wet" months of November through May. Pumping was also limited to those times when the water surface elevation of the creek was above a certain elevation that had been established during the permitting process. The system was built as part of an Auburn University experiment and is used to irrigate cotton fields. Using its current permitting guidelines, this system could provide a maximum of 0.3 MGD of water supply during non-pumping months of June through October (152 days of no inflow).

Photo 4-6 – Example of intake structure for a traditional water harvesting system

Photo 4-7 – Example of detention area for a traditional water harvesting system

4.9.2 Water Harvesting as a Water Supply Alternative

A larger version of the water harvesting system in Huntsville was designed to determine the yield that could be obtained as well as the cost associated with the system. A detention area

constructed from a 35-feet tall ring levee was designed. The designed detention area had approximately 15 acres of surface area and a volume of approximately 375 acre-feet (123 million gallons) at the top of the pool (4 feet below the top of the levee). The detention area would require an area of land approximately 1000 feet x 1000 feet (23 acres) in size. Because the area typically has a thin overburden and composed of sandy materials, a liner is included in the design to ensure no losses. It was also assumed the same pumping limitations would be imposed on this system as those discussed for the Huntsville prototype. This water harvesting design (volume) could provide a maximum of 0.8 MGD of water supply during the non-pumping months of June through October (152 days of no inflow).

There are several streams in Cumberland County that could support the aforementioned water harvesting system. These include the Caney Fork, Clear Creek, Meadow Creek, Daddys Creek, etc. Due to scope limitations, no specific pump sites were identified for this study. For this reason, a few assumptions had to be made during the sizing of the pump station and pipeline.

The required capacity of the intake pump was determined based on historical modeling of flow in the Caney Fork that is discussed in Section 4.10.3, Modeling Process. The historical modeling of the Caney Fork showed that during the critical drought year there were 20 days the water surface elevation in the Caney Fork was above the minimum elevation for pumping of 1545.3 feet NGVD29 (see Section 4.10.3). Based on the results of that modeling, it was assumed there would be 20 days during the "wet" months of November through May that the water surface elevation in the creek would be high enough to permit pumping. The required capacity of the intake pump was calculated by dividing the volume of water pumped from the system each year (0.8 MGD x 365 days = 292 MG) by 20 days of pumping. The required capacity of the intake pump and pipeline is 14.6 MGD (10,150 gpm).

As was done in the storage impoundment alternatives, it was assumed water supply flows from the water harvesting system would be pumped to the current water treatment plant located near Lake Holiday (Crossville, Tennessee). To provide for peak demand, which is typically estimated as 150% of the daily yield, the pump station and pipeline that would transport water from the detention area to the treatment plant were designed to supply 1.2 MGD of raw water.

Because no specific sites were identified the following assumptions were made in designing the pipelines both to and from the detention area:

Pipeline from the creek to the detention area would:

Have 200 feet of head

Be 500 feet in length

Pipeline from the detention area to the treatment plant would:

Have 200 feet of head

Be 10 miles in length

These assumptions were made based upon a review of the topography along the creeks previously mentioned as potential sites and of the topography between those creeks and the treatment plant. Additional information on the design of the pump station and pipeline can be found in Section 4.8.6.

Levee Design for Water Harvesting

The ring levee system described in Section 4.10.2 would be approximately 1,000 feet by 1,000 feet. A typical section of the levee would have downstream sideslopes of 1V to 2.5H and upstream slopes of 1V to 2H and a top crest width of 10 feet. A cutoff trench 10 feet wide, five feet would also be constructed. If clay can be found within the levee storage area or close by, it would be used to construct the levee. If insufficient impervious material is available, then the overburden material (which is assumed to be a silty sand) can be used to construct the levee, but an impervious liner would have to be installed. Cost estimates were made assuming impervious material can be found. See Figure 4-16 for typical section.

Figure 4-16 Typical Water Harvesting Ring Levee Section

4.9.3 Water Harvesting from the Caney Fork to Meadow Park Lake

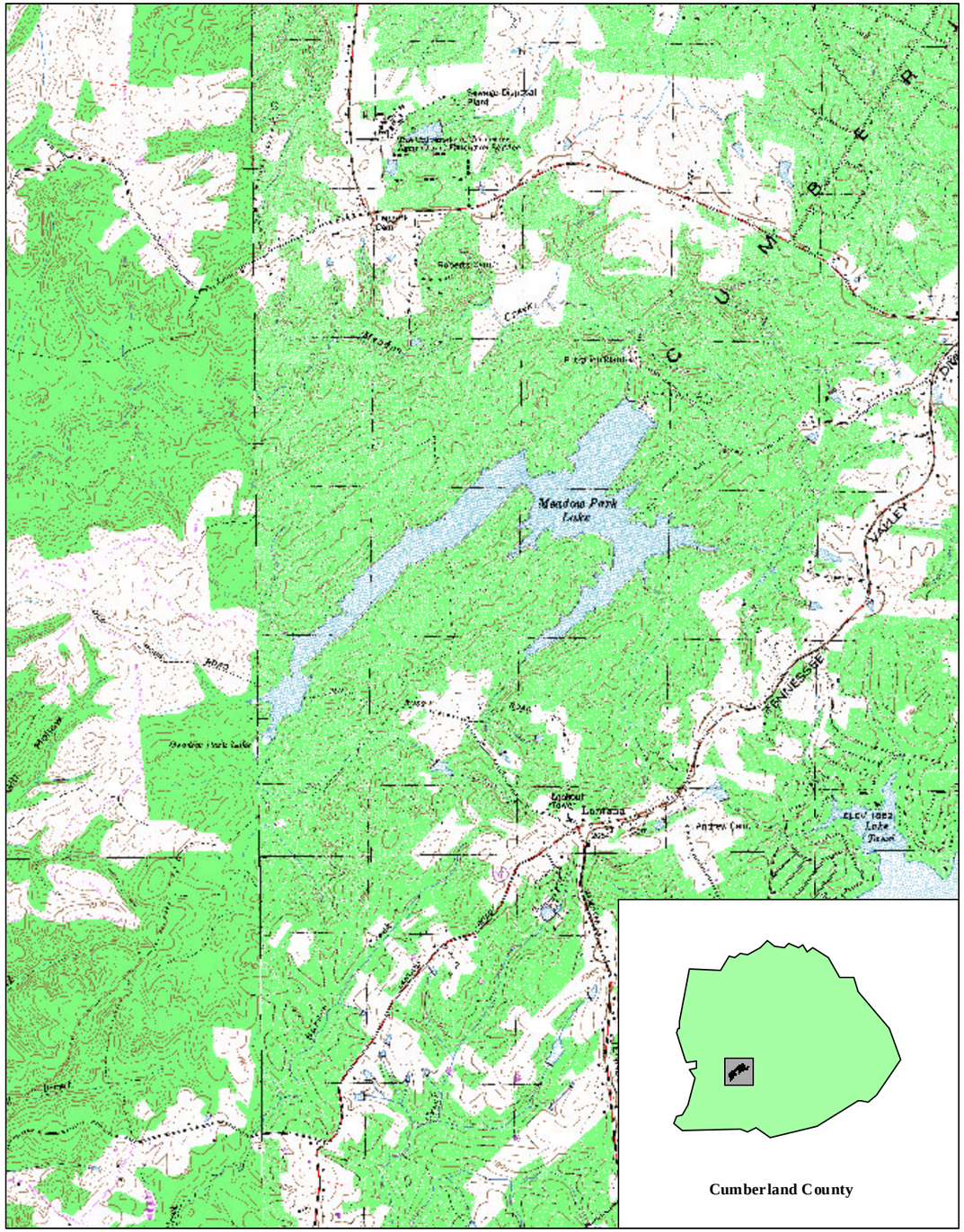
The second method of water harvesting investigated as a source for additional water supply for Cumberland County was pumping water during high-flow events on the Caney Fork and storing it in a raised Meadow Park Lake reservoir. As discussed in Section 4.7.2, the watershed upstream of Meadow Park Lake does not have sufficient drainage area to support a larger dam. However, the topography would allow a larger dam. It was due to these facts that this alternative was identified.

If the existing Meadow Park Lake Dam were raised, a substantial additional storage area could be utilized for water supply. The existing area of the pool is 255 acres at normal pool (spillway elevation). The area of the proposed raised pool area is 610 acres at normal pool. Figure 4-17 provides a comparison of the existing Meadow Park Lake reservoir and the proposed raised Meadow Park Lake reservoir. Pumping water from the Caney Fork into that storage area would offset the fact the Meadow Park Lake watershed is too small to generate enough runoff to fill the reservoir behind the raised dam. This alternative would combine the benefits of both sites: the large drainage area upstream of the Caney Fork site could be utilized without impounding the river and the large storage area upstream of the existing Meadow Park Lake Dam could be utilized for water supply. The proposed site for the intake structure on the Caney Fork is the same as the proposed site for the new impoundment on the Caney Fork (refer to Figure 4-13).

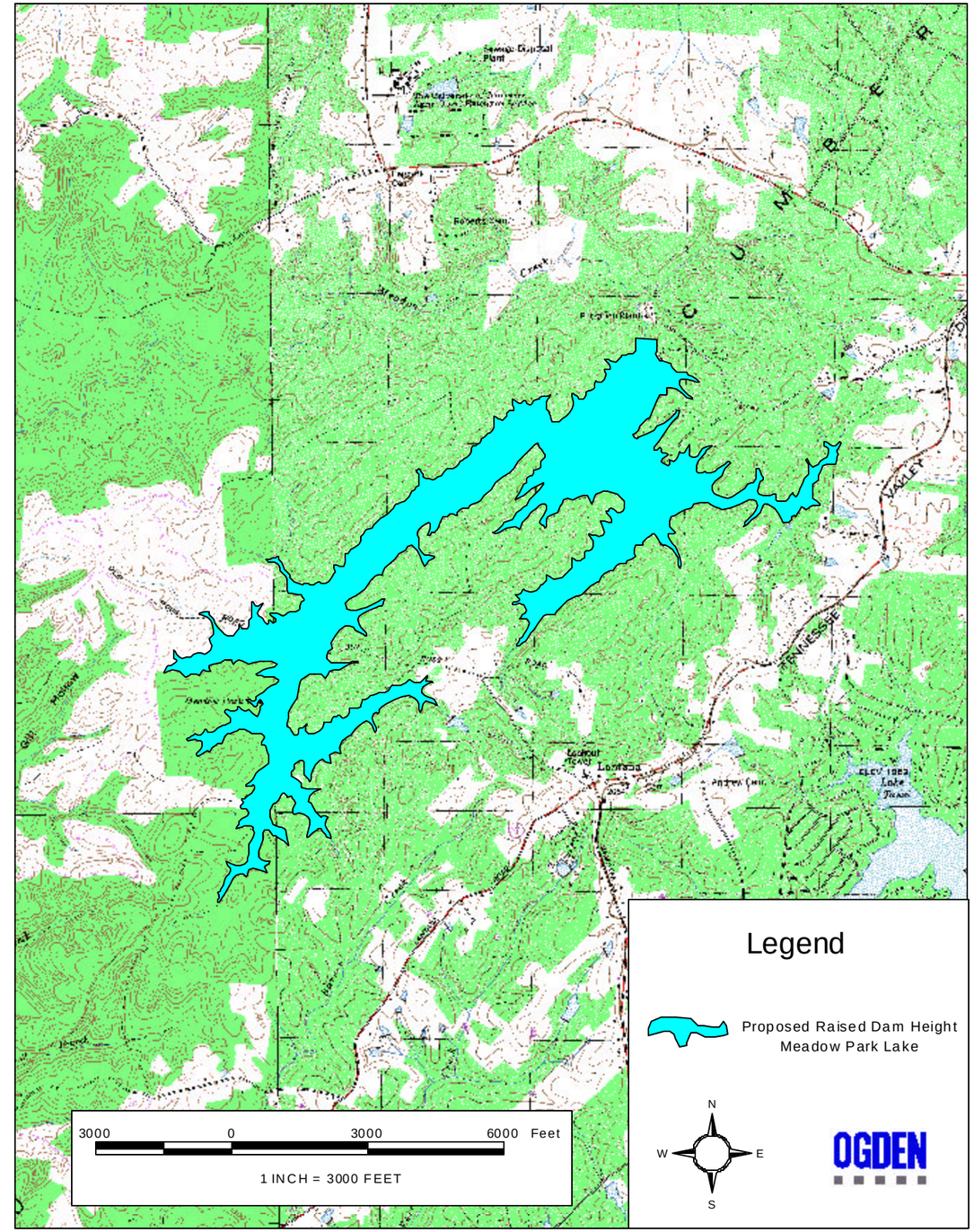
Modeling Process

To model this water harvesting alternative, a target yield had to be assumed. The same yield as that used in the pipeline alternative (refer to Section 4.5.1) was used, 9 MGD.

To determine the total volume of storage required within the Meadow Park Lake, five months of zero inflow from the Caney Fork pump was assumed. Five months of continuous drought conditions is much worse than any of the historical droughts the region has experienced, so this assumption was considered conservative. The storage lost to date from sedimentation as well as the storage that would be lost to sedimentation during the structure's design life of 50 years was also considered in the required volume computations.



Existing Meadow Park Lake



Proposed Raised Dam Height
Meadow Park Lake (Water Harvesting Site)

Figure 4-17 Comparison of Meadow Park Lake Elevations
(Water Harvesting Site)

Based upon these assumptions and considerations, the required total volume of storage is approximately 8575 acre-feet, which corresponds to an elevation of 1829.0 feet NGVD29 within the reservoir. Thus, the raised Meadow Park Lake Dam would need to have a normal pool elevation of 1829.0 feet NGVD29 or greater to provide the additional 9 MGD of water supply.

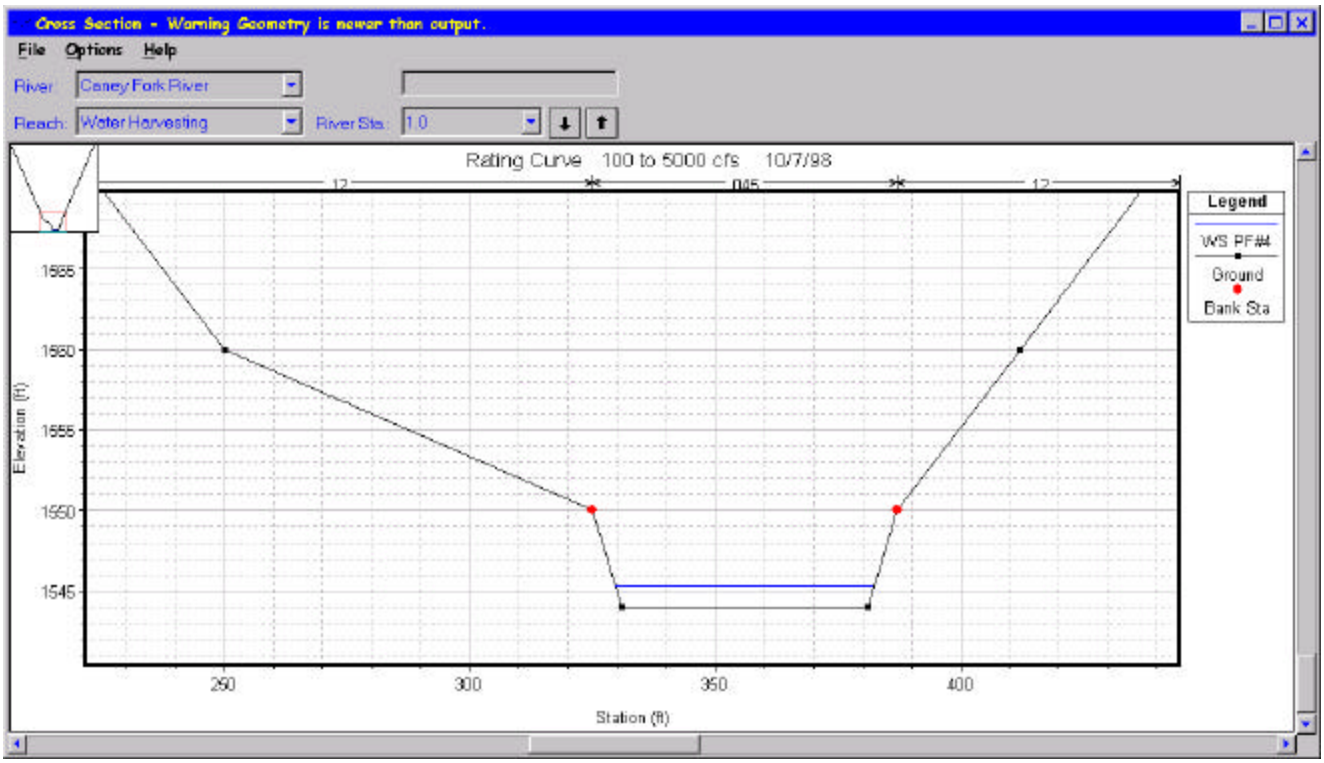
As previously discussed, the existing Meadow Park Lake Dam could be raised to Elevation 1840.0 feet NGVD29. Previous modeling of a raised Meadow Park Lake dam assumed the dam would be raised with earthen material, thus requiring a spillway to pass the freeboard design storm as defined by the "Safe Dams Act of 1973." However, during the work for this water harvesting alternative a second site visit was conducted, during which it was confirmed the existing dam could be raised using roller compacted concrete. This would permit overtopping of the dam during flood events, and eliminate the need for a spillway. Therefore, there is ample room to raise the existing Meadow Park Lake dam to accommodate the storage requirements for this alternative as designed.

The next step in the modeling process was to determine the Caney Fork elevation above which pump withdrawals can be made. Based upon engineering judgment it was decided the elevation corresponding to the 20% duration flow would be used for that elevation. The 20% duration flow is the flow rate in the river that is exceeded only 20% of the time in a year. Historical streamflow data is required to calculate duration discharges, which is not available at the proposed Caney Fork site. However, such data is available further downstream at the Caney Fork stream gage located near Clifty, Tennessee.

The discharge data from the Clifty gage for the Years 1929 to 1949 were retrieved from CD-ROMs containing NWS records. One set of statistical data available on the CD-ROMs for stream gages is duration flows. The 20% duration flow for the Caney Fork at the gage site is 292 cfs. The 20% duration flow at the proposed site of the intake was calculated by multiplying the 292 cfs by the ratio of the drainage areas at the intake and at the gage, 58.3 mi^2 and 111.0 $mi^{2,}$ respectively. The calculated 20% duration flow for the Caney Fork at the proposed site of the intake is 153 cfs. From the HEC-RAS generated rating curve developed for the site (refer to Section A.2 in Appendix A for a discussion on the development of rating curves using HEC-RAS); the elevation that corresponds to a discharge of 153 cfs is 1545.3 feet NGVD29. Therefore, it was assumed pumping would begin when the water surface elevation in the river

exceeded 1545.3 feet NGVD29, and would cease when the water surface elevation dropped below this elevation. Figure 4-18 illustrates the cross-section at the proposed Caney Fork intake site and the 20% duration elevation of 1545.3 feet NGVD29.

Figure 4-18 – Caney Fork cross-section at proposed intake site and 20% duration elevation



The next step was to determine the size of the pump required at the Caney Fork intake. The minimum volume to be pumped in a year was assumed to be equal to the volume pumped from a demand of 9 MGD during the assumed five months of zero inflow. The minimum flow was determined by multiplying 9 MGD by 152 days (five months of zero inflow), which resulted in 1368 MG (4200 acre-feet). The HEC1-API model used for the yield analysis on the proposed new impoundment on the Caney Fork was modified to represent the Caney Fork watershed above the proposed water harvesting alternative intake. The proposed reservoir was replaced with an artificial reservoir that represented the Caney Fork channel upstream of the intake structure. The rating curve developed in HEC-RAS was input using SQ/SE cards. This results in a stage hydrograph at the intake site. A pump that would shut off whenever the water surface elevation fell below 1545.3 feet NGVD29 was also simulated in the HEC1-API model.

The HEC1-API model was used to make continuous simulations of the historical drought years previously mentioned. The volume of water pumped from the river each year was checked at the end of each year's run to ensure it was greater than the minimum 4200 acre-feet. If the volume was less than 4200 acre-feet, the size of the pump was increased. This iterative process was repeated until a pump size was found that could pump the minimum 4200 acre-feet of water during each of the drought periods. The pump size meeting this criterion was 48,000 gpm. During the critical year (1985), there were only 20 days the water surface elevation was above 1545.3, thus only permitting pumping for 20 days. Once the pump size was determined, each of the drought years were run again with the HEC1-API model, and the outflow hydrograph from the pump was written to a DSS database for retrieval in the yield analysis.

The final step was to determine the total yield the raised Meadow Park Lake Reservoir could provide with the additional inflow from the Caney Fork water harvesting. The HEC1-API model used for the raised Meadow Park alternative was modified to represent a raised Meadow Park Lake dam with no spillway (top of dam at elevation 1839.5 feet NGVD29). The model was also modified to include the retrieval of the outflow hydrograph from the Caney Fork pump as inflow to the reservoir. A yield analysis was performed (refer to Section 4.6.5). The proposed water harvesting from the Caney Fork to a raised Meadow Park Lake reservoir would provide a water supply yield of 11 MGD.

Recall the current Meadow Park Lake configuration can provide a yield of 3 MGD, so the water harvesting alternative would provide an additional 8 MGD. The additional yield of 8 MGD is slightly less than the target increase of 9 MGD. Smaller pumps and pipelines would be required if the desired additional water supply for Cumberland County is identified to be less than 8 MGD. Conversely, larger pumps and pipelines would be needed if a larger yield is desired.

The pump station and pipeline at the Caney Fork intake were sized based on a required pumping capacity of 48,000 gpm. As was done in the storage impoundment alternatives, it was assumed water supply flows from the raised Meadow Park Lake would be pumped to the current water treatment plant located near Lake Holiday (Crossville, Tennessee). The pump station and pipeline that would pump the water from the reservoir to the treatment plant were required to supply 150% of the daily yield (3 MGD existing plus 9 MGD additional), or 18 MGD, to provide for peak demand.

The proposed pipeline route illustrated in Figure 4-19 was selected based on minimizing the total distance and optimizing the use of existing roadway right-of-ways. Additional information on the design of the pump stations and pipelines can be found in Section 4.8.6.

Raising Meadow Park Dam – Structural Design

In order to raise Meadow Park Lake by 20 feet, the best alternative is to use a roller compacted concrete dam. This would require that an inspection trench be cut downstream of the existing dam and roller compacted concrete placed to widen the base and build the new dam. The existing dam would be used as the form for the upstream face of the dam with the top 20 feet being formed. A separate spillway would be cut through the hillside. A new valve would be installed in the existing 20 inch diameter pipe at the downstream toe of the dam. The existing valve stems for the water intake would have to be lengthened and moved to the top of the dam.

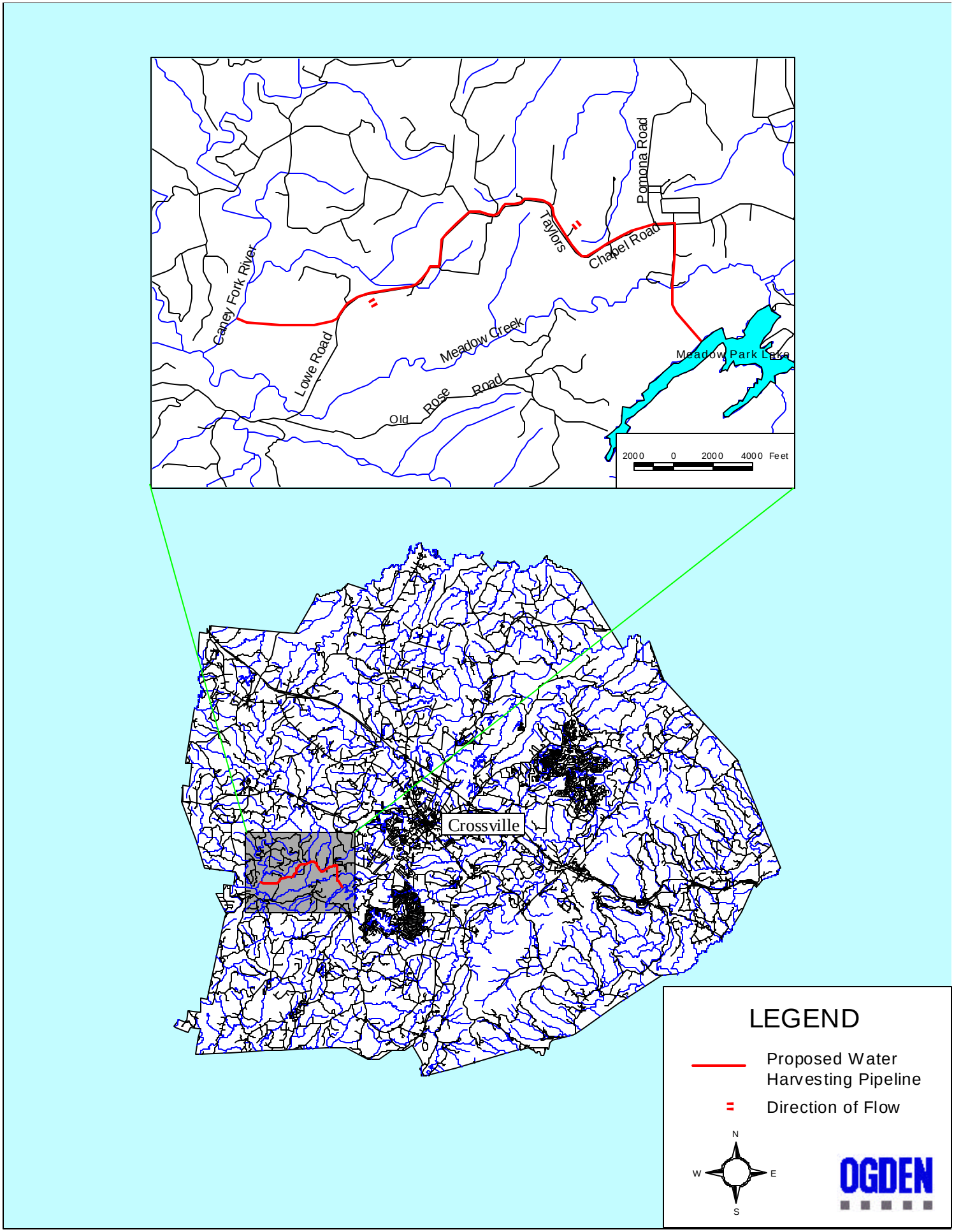


Figure 4-19 Proposed Water Harvesting Pipeline

4.10 NO ACTION ALTERNATIVE

The "No Action" alternative accounts for the possibility that the Cumberland County community may decide not to pursue an additional water supply source(s). As discussed in Section 2.2, the current safe yield available from Lake Holiday, Meadow Park Lake, and Stone Lake is 10 MGD. Table 4-9 tracks the status of Cumberland County's drinking water supply if no action is taken. The values listed in Table 4-11 for "Average Annual Water Use" and "Peak Water Use" follow the median growth scenario for the community which predicts a need of 13 MGD in the Year 2050 (refer to Section 3.1.3).

Table 4-9
No Action Alternative
Status of Drinking Water Supply

Year	Number of Customers	Average Annual Water Use (MGD)	Peak Water Use (MGD)	Status of Drinking Water Supply
1990	7575	3.03	4.54	Available Supply
1997	17701	4.33	6.50	Available Supply
2010	26274	6.86	10.30	Available Supply
2025	36727	9.87	13.1	Available Supply
2050	54748	14.89	22.33	**Inadequate**

In this instance, it is the responsibility of the County residents to either curtail the current residential and commercial growth of the community or acknowledge the need for an additional water supply source. The importance of water to the well being of all members of the community is obvious from meeting drinking, cooking, and sanitary needs to lawn watering, swimming pool maintenance, and fire fighting.

It is important to note that tracking of the current water supply against average annual water use from a different growth scenario could result in an inadequate status either earlier or later than that illustrated in Table 4-9, depending on the growth scenario used. Should the actual growth rate of the community be less than the projected moderate growth rate discussed above, no action could be a viable alternative. In the short term, Cumberland County residents and utility districts could look to the surrounding communities for potential connections with neighboring water supply systems. Possible connections include Spring City to serve the Grandview utility district, the City of Monterey to serve the West Cumberland utility district, Fentress County to serve the Catoosa utility district, and the City of Rockwood to serve the Crab Orchard utility district in addition to the current support provided to the Grandview utility district. This information

regarding potential interconnections with neighboring utility districts was provided by the Rural Development Branch of the Tennessee State Office of the U.S. Department of Agriculture. Reportedly, these interconnections could be easily made with the existing Cumberland County distribution lines and expansion projects in neighboring utility districts that are planned for the near future.

5.0 PRELIMINARY COST ANALYSIS

5.1 INTRODUCTION

The preliminary cost estimates were prepared in accordance with the requirements of EI01D010, "Engineering Instructions." The cost account numbers are in accordance with the Civil Works Breakdown Structure (CWBS). The price level is October 1998. The Cumberland County Regional Water Supply Study provides preliminary estimates for three new impoundments, three pipelines, groundwater wells and associated pipelines, and three water harvesting alternatives. The preliminary cost estimates generated for this study received primary input from Nashville District's Engineering-Planning Division, OGDEN, and TVA. Unit costs were based on historical cost data and adjusted to meet the needs of current site conditions.

5.2 COST ITEM DESCRIPTION

Each preliminary cost estimate consists of six cost items associated with design and construction of each alternative. A description of each cost item: lands and damages; relocations; cultural resources; planning, engineering, and design; construction management; and contingencies is provided. Tables 5-1 through 5-4 present the preliminary cost totals for the groundwater, pipeline, impoundment(s), and water harvesting alternatives. No cost is associated with the no action and water conservation alternatives. These alternatives do not require any cost items associated with design and construction and comparisons would not be applicable. A water conservation program can be as simple as pamphlets included in a water bill or complex as upgrading plumbing equipment. Selection of water conservation as a water supply alternative would require the Cumberland County community to determine the extent of the conservation program.

5.2.1 Lands and Damages

Due to time and money constraints, there was no input from the Nashville District's Real Estate Division. Land and damages costs for this preliminary engineering report are thus not included as a lump sum. The cost for the purchase of right-of-way may include some or all of the following: (1) cost of the private property; (2) damages incurred because of construction within the right-of-way; (3) additional cost for sloping construction within the urban area; and (4) damages to improvements along the right-of-way. Improvements along the right-of-way in an urban area include signs, housing, or fencing, as an example.

5.2.2 Relocations

Relocation costs are those costs associated with the relocation, abandonment, vacation, or alteration of existing highways, roads, railroads, utilities, cemeteries, municipal facilities and structures which involve the acquisition of an interest in real estate. The relocation of facilities is a function involving procurement, legal, real estate, and engineering principles.

The procedure for acquisition of real estate interests through relocation consists of determination of what facilities would be affected, determination of legal obligation for relocation, determination of compensable interests, and the proposal of a substitute facility, with the determination of its adequacy and reasonableness of cost. The relocation costs herein are cursory and were determined in an effort to give the cost of all alternatives an equal comparison base. Note extensive field review of the project features may reveal additional detailed relocation costs.

Groundwater Alternative

Relocations costs associated with the groundwater pipeline route were assumed to be three percent of the pipeline cost for the placement of the pipeline.

Large Scale Pipeline Alternative

Relocations costs associated with the Watts Bar pipeline route were assumed to be three percent of the pipeline cost, due to the placement of the pipeline through an urban area. Relocations costs associated with the Center Hill and Great Falls pipeline routes were assumed to be 1.5 percent of the pipeline cost, due to the placement of the pipelines through a rural area.

Clear Creek Dam and Reservoir

Two transmission lines cross the proposed lake site. The lines are a 500 kilovolt (kv) line from Roane County to Hartsville and a 161 kv line from West Cookeville to Crossville. According to TVA transmission line engineering, there are adequate wire clearances. The transmission lines are located 100 feet above the proposed water surface elevation of 1820 feet.

Meadow Park Lake Dam and Reservoir (Water Harvesting)

A small roadway (boat ramp) located on the right bank of the existing dam site provides access to a marina. Should the marina be impacted by the proposed dam site, the roadway would no longer provide adequate loading area for the boat ramp. However, relocation of the roadway is a real estate cost not a relocation cost.

Caney Fork Dam and Reservoir

The Caney Fork Dam site would require the relocation of a county road with the construction of a 600 linear foot bridge. The bridge would require 1500 linear feet of approach roadway. An estimated cost for the bridge and approach roadway construction is $2,670,000.

5.2.3 Cultural Resources

The costs for this feature includes all costs associated with identifying and preserving cultural resources. Example of costs included in this feature would be recovery and removal of artifacts, relocation structures, construction of fences to protect cultural resources, etc. The costs in this feature are estimated at one percent of construction costs.

5.2.4 Planning, Engineering and Design

The costs for this feature includes all planning, engineering and design cost to produce a feasibility report and final plans and specs. Also included in this feature are engineering costs estimated to be incurred during construction. The costs in this feature are estimated at eighteen percent of construction costs.

5.2.5 Construction Management

The costs for this feature include all costs associated with the construction management of this project. These costs are estimated to be seven percent of construction costs.

5.2.6 Contingencies

Contingencies were developed and applied where areas of uncertainty exist. A contingency of 35% was used for all alternatives.

5.2.7 Cost Summary

Cost estimating calculations were made using the cost estimating software MCACES, Micro-Computer Aided Cost Engineering System. This is the cost estimating software used for all Corps of Engineers projects. The output and cost summary is included in Appendix B.

5.2.8 Project Specific Cost Items

Each alternative requires cost items specific to that project. The groundwater and large scale pipeline alternatives include costs for pumping stations, numerous booster pumps of varying size, pipeline excavation, and pipeline installation. Stream and/or roadway crossings require additional construction costs. Details regarding specific pump and pipeline sizes for the groundwater and large scale pipeline alternatives are included in Sections 4.3.5 and 4.5.1, respectively.

The impoundment alternatives require construction costs for diversion of streams, excavation from borrow pits and the dam footprint, foundation preparation, and dam and spillway material costs. Pumping plants and pipelines are also required at each reservoir to transport water from the new impoundment to the existing filtration plant at Lake Holiday. Details regarding specific pump and pipeline design data are included in Section 4.8.6.

Table 5-1 Groundwater Alternative

Western Toe Phenomenon

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$884,000.00	$309,400.00	$1,193,400.00
13	Pumping Station	$29,469,200.00	$10,314,200.00	$39,783,400.00
18	Cultural Resources Preservation @1%	$300,000.00	$105,000.00	$405,000.00
30	Planning, Engineering & Design (E&D)	$5,464,000.00	$1,912,400.00	$7,376,400.00
31	Construction Management (S&I) @7%	$2,124,000.00	$743,400.00	$2,867,400.00

Total Estimated **Preliminary Cost** $51,625,600.00

Table 5-2 Pipeline Alternative

Watts Bar (24" Pipe)

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$475,000.00	$166,300.00	$641,300.00
13	Pumping Plant	$15,760,800.00	$5,516,300.00	$21,277,100.00
18	Cultural Resources Preservation @1%	$162,000.00	$56,700.00	$218,700.00
30	Planning, Engineering & Design (E&D) 18%	$2,922,000.00	$1,022,700.00	$3,944,700.00
31	Construction Management (S&I) @7%	$1,137,000.00	$398,000.00	$1,535,000.00

Total Estimated Preliminary Cost $27,616,700.00

Great Falls (24" Pipe)

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$291,000.00	$101,900.00	$392,900.00
13	Pumping Plant	$19,404,700.00	$6,791,600.00	$26,196,300.00
18	Cultural Resources Preservation @1%	$197,000.00	$69,000.00	$266,000.00
30	Planning, Engineering & Design (E&D) 18%	$3,545,000.00	$1,240,800.00	$4,785,800.00
31	Construction Management (S&I) @7%	$1,378,000.00	$482,300.00	$1,860,300.00

*Note – Cost for withdrawal of water from a Corps of Engineers lake has not been determined at this time.

Total Estimated Preliminary Cost $33,50,100.00

Center Hill (24" Pipe)

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$334,000.00	$116,900.00	$450,900.00
13	Pumping Plant	$22,267,300.00	$7,793,600.00	$30,060,900.00
18	Cultural Resources Preservation @1%	$226,000.00	$79,100.00	$305,100.00
30	Planning, Engineering & Design (E&D) 18%	$4,068,000.00	$1,423,800.00	$5,491,800.00
31	Construction Management (S&I) @7%	$1,582,000.00	$553,700.00	$2,135,700.00

*Note – Cost for withdrawal of water from a Corps of Engineers lake has not been determined at this time.

Total Estimated Preliminary Cost $38,444,400.00

Table 5-3 Impoundments

Clear Creek

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$0.00	$0.00	$0.00
03	Reservoirs	$368,300.00	$128,900.00	$497,200.00
04	Dams	$5,601,000.00	$1,960,400.00	$7,561,400.00
13	Pumping Station	$10,728,900.00	$3,755,100.00	$14,484,000.00
18	Cultural Resources Preservation @1%	$167,000.00	$58,500.00	$225,400.00
30	Planning, Engineering & Design (E&D) 18%	$3,005,000.00	$1,051,800.00	$4,056,800.00
31	Construction Management (S&I) @7%	$1,169,000.00	$409,200.00	$1,578,200.00
	Total Estimated Preliminary Cost			$28,403,000.00

Caney Fork

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$2,670,000.00	$934,500.00	$3,604,500.00
03	Reservoirs	$646,300.00	$226,200.00	$872,600.00
04	Dams	$21,717,600.00	$7,601,200.00	$29,318,700.00
13	Pumping Station	$12,284,100.00	$4,299,400.00	$16,583,600.00
18	Cultural Resources Preservation @1%	$373,000.00	$130,600.00	$503,600.00
30	Planning, Engineering & Design (E&D) 18%	$6,717,000.00	$2,351,000.00	$9,068,000.00
31	Construction Management (S&I) @7%	$2,612,000.00	$914,200.00	$3,526,200.00
	Total Estimated Preliminary Cost			$63,477,100.00

Meadow Creek (below Monterey Dam)

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$500,000.00	$175,000.00	$675,000.00
03	Reservoirs	$475,300.00	$166,300.00	$641,600.00
04	Dams	$15,363,400.00	$5,377,200.00	$20,740,600.00
13	Pumping Station	$16,424,000.00	$5,748,400.00	$22,172,400.00
18	Cultural Resources Preservation @1%	$328,000.00	$114,800.00	$442,800.00
30	Planning, Engineering & Design (E&D) 18%	$5,899,000.00	$2,064,700.00	$7,963,700.00
31	Construction Management (S&I) @7%	$2,293,000.00	$802,600.00	$3,095,600.00
	Total Estimated Preliminary Cost			$55,731,500.00

Table 5-4 Water Harvesting Alternative

New Meadow Park Dam / Caney Fork Intake

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$423,000.00	$148,100.00	$571,100.00
03	Reservoirs	$696,200.00	$243,700.00	$939,900.00
04	Dams	$5,857,300.00	$2,050,100.00	$7,907,400.00
13	Pumping Station	$21,649,000.00	$7,577,200.00	$29,226,200.00
18	Cultural Resources Preservation @1%	$286,000.00	$100,100.00	$386,100.00
30	Planning, Engineering & Design (E&D) 18%	$5,152,000.00	$1,803,200.00	$6,955,200.00
31	Construction Management (S&I) @7%	$2,004,000.00	$701,400.00	$2,705,400.00
	Total Estimated Preliminary Cost			$48,691,300.00

Raise Meadow Park Dam/ Caney Fork Intake

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$730,000.00	$255,500.00	$985,500.00
04	Dams	$2,700,500.00	$945,200.00	$3,645,700.00
13	Pumping Station	$21,649,000.00	$7,577,200.00	$29,226,200.00
18	Cultural Resources Preservation @1%	$250,000.00	$87,500.00	$337,500.00
30	Planning, Engineering & Design (E&D) 18%	$4,515,000.00	$1,580,300.00	$6,095,300.00
31	Construction Management (S&I) @7%	$1,755,000.00	$614,300.00	$2,369,300.00
	Total Estimated Preliminary Cost			$42,659,500.00

Small Off-site Detentions (price per “typical” system)

WBS	Description	Contract	Contingency	Total Cost
01	Lands & Damages	N/A	N/A	N/A
02	Relocations	$320,000.00	$112,000.00	$432,000.00
03	Reservoirs	$5,661,900.00	$1,981,600.00	$7,643,500.00
13	Pumping Station	$5,245,800.00	$1,836,000.00	$7,081,800.00
18	Cultural Resources Preservation @1%	$112,000.00	$39,200.00	$151,200.00
30	Planning, Engineering & Design (E&D) 18%	$2,020,000.00	$707,000.00	$2,727,000.00
31	Construction Management (S&I) @7%	$786,000.00	$275,100.00	$1,061,100.00
	Total Estimated Preliminary Cost			$19,096,600.00

6.0 ENVIRONMENTAL SCREENING

6.1 INTRODUCTION

The purpose of this Preliminary Engineering Report is to evaluate the Technical Feasibility of various water supply alternatives available for the County; not to select or recommend an alternative. To support this effort, an environmental screening was performed for each alternative to discuss potential environmental issues that would likely need to be addressed for a particular alternative. Issues identified in this section may not be all encompassing of issues that might come up at a later date through the National Environmental Policy Act (NEPA) process and corresponding public scoping. A general discussion of regulatory drivers is outlined along with how the applicable regulatory program would likely apply to each alternative.

The Tennessee Valley Authority (TVA) produced a report for the Catoosa Utility District that provided a discussion of "Permitting Viability" for a water supply impoundment on Clear Creek. This impoundment alternative is included in this Preliminary Engineering Report. The TVA discussion is still applicable for the alternatives included in this report and is repeated where appropriate.

6.2 REGULATORY DRIVERS

Discharges of dredged or fill material into all waters of the U.S. requires a Department of the Army (DA) permit pursuant to Section 404 of the Clean Water Act (CWA) from the Corps of Engineers. A DA permit pursuant to Section 10 of the Rivers and Harbors Act permit is required from Corps of Engineers for obstructions or alteration of streams designated as "navigable waters of the U.S." The Tennessee Department of Environment and Conservation (TDEC) must issue water quality certification under Section 401 of the CWA before the Corps of Engineers can approve a permit action. Under the Corps of Engineers regulatory program, permits can be issued either individually or under a Nationwide Permit (NWP). Individual Permits require NEPA documentation which can be either an Environmental Assessment and Finding of No Significant Impact (EA/FONSI) or an Environmental Impact Statement and Record of Decision (EIS/ROD). Depending on the scope of the project and its impacts, an EIS is required if impacts of the proposal are significant. One major component of the NEPA process is a Public and Agency Scoping step that is required to develop a list of major issues to be evaluated in the EA

or EIS. NEPA review has already been performed for activities eligible to be covered under a NWP.

As part of the Section 404 permit issuance, the Section 404(b)(1) Guidelines require an evaluation of a project that would result in placement of fill into waters of the U.S. Part of this evaluation is a determination of whether less environmentally damaging alternatives exist that would avoid or minimize the impact. For example, construction of a new impoundment would be permitted only if other practicable alternatives did not exist. An alternative is practicable if it is capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes.

If a project is proposed by the Corps of Engineers, a DA permit would not be issued, however, the Corps must provide an equivalent 404 permit evaluation. This evaluation must be included in the NEPA document for the project. If another Federal agency proposes (or funds) the project, that agency would be responsible for compliance with NEPA. Assuming a DA permit is required, it is likely the Corps of Engineers would become a cooperating agency on the NEPA documentation. If no Federal agency is a proponent or funds the action, NEPA applies only to the proposed action that requires a DA Permit.

Approval under Section 26a of the TVA Act is required for any structures constructed in waters within the Tennessee River Basin. All alternatives would also require an NPDES permit from TDEC for discharges of stormwater during construction, since it is probable that more than 5 acres would be disturbed. Any DA permit action or NEPA Document would require consultation with the U.S. Fish and Wildlife Service under Section 7 of the Endangered Species Act if the project may affect federally listed threatened or endangered species.

6.3 ENVIRONMENTAL ISSUES

6.3.1 Pipeline From Existing Impoundment

Three pipeline options were initially considered to supply 9 MGD from an existing water source to the Crossville Lake Holiday Water Treatment Plant (WTP). Pipelines to Watts Bar Lake on the Tennessee River and two sites in the Cumberland River Basin; Great Falls Lake (or Center Hill Lake) and Dale Hollow Lake were initially considered. Dale Hollow Lake was removed

from further consideration, as it obviously would be less feasible than the other two pipeline options due to the length of pipeline required.

The route of the pipeline would determine specific impacts that would be encountered. The pipeline route selected generally follows highway routes that have existing waterline easements following most of the route. By following the highway routes, environmental impacts should be minimized. Localized impacts that are encountered during subsequent phases might be avoided by varying the pipeline routing. For example, a mussel bed or cultural resource site could be avoided by routing the pipeline alignment around such a site.

A DA permit would be required for any stream crossings and any intake structure for a pipeline. If the project has federal involvement, impacts to all resources will have to be evaluated. This would require a Cultural Resources Survey as well as some level of inventory of various environmental resources.

Watts Bar Lake Pipeline

An approximately 25-mile long pipeline to Watts Bar Lake was evaluated with an intake in the vicinity of the Rockwood WTP. The route would generally follow Highway 70 from Crossville, except for the portion descending Waldens Ridge into Rockwood. Existing waterlines are present for all but the last portion of this route from Piney Creek at the Cumberland-Roane County line to the base of Waldens Ridge. Major stream crossings would include the Obed River, Daddys, Renfroe, Fall, Mammys, and Piney Creeks. There would likely be some special aquatic habitat considerations for crossing Daddys Creek since it is designated as critical habitat for spotfin chub, a federally listed endangered species. The existing waterline crossing of Daddys Creek is an aerial crossing. There appears to be public sentiment against using Watts Bar Lake as a raw water source due to perceived contamination problems. This concern does not appear to be warranted since the problems are more related to contaminated sediment than water quality and many other water systems currently use the lake as a water source.

Great Falls Lake Pipeline (Center Hill Lake)

A pipeline from either Great Falls Lake or Center Hill Lake would have similar environmental concerns as those discussed for the Watts Bar pipeline. The Great Falls Lake pipeline route would follow Highway 70 from the base of the Cumberland Plateau east of Sparta, at which

point it would follow county roads to an intake on Great Falls Lake near River Hill. This route would have fewer major stream crossings than the Watts Bar route and would have a somewhat higher potential for terrestrial impacts due to the longer length. Streams to be crossed are the Caney Fork and several smaller tributaries. Most of the route would be over pasture or residential lands. The stream crossings appear to qualify for coverage under the NWP for Utility Line Crossings. A cultural resources survey would likely be required.

6.3.2 Groundwater

Environmental issues associated with a groundwater system would include construction of well(s) and transmission line. Impacts from the transmission line would be similar in scope to the pipeline options. Another issue is potential impacts on flow in adjacent streams due to groundwater withdrawals. The latter cannot be determined with current information and would depend on the location of the wells and how much groundwater withdrawals would impact recharge to surface streams.

6.3.3 New Impoundments

Environmental impacts from new impoundments would be much more substantial than those of pipelines. The construction of an impoundment would convert pool and riffle habitat to lacustrine (lake) habitat. Pool and riffle areas are considered Special Aquatic Sites under the 404(b)(1) Guidelines. Flow modifications below a dam would be another substantial issue. Each impoundment and transmission pipeline to the Lake Holiday WTP would have similar impacts associated with construction of a water intake and transmission pipeline. Four new or modified reservoir sites were considered in this study. Since all new impoundments involve discharges of fill material into waters of the U.S., each new impoundment requires a DA permit review.

<u>Clear Creek Impoundment</u>

The Catoosa Utility District's previously proposed impoundment at Mile 43.2 of Clear Creek is included in this study. Potential environmental impacts associated with this project are discussed in detail in the TVA report. One additional concern with this site involves the implications of the Designated Wild and Scenic River Sections of Clear Creek and the Obed River. No impoundment can be permitted on Clear Creek without a technical determination that the project would not "diminish" designation values or "invade" the designated area. Additional

environmental issues would be similar in nature to the other impoundment sites: conversion of stream habitat to lacustrine habitat and terrestrial and cultural resources in the proposed pool area. Clear Creek downstream of Interstate 40 is listed as critical habitat for the Spotfin Chub, a federally listed threatened species. This proposed impoundment alternative was dropped after the negative responses from the public hearing and letters from TWRA and EPA expressing concerns about the project. If the project were proposed again, approval from TVA under Section 26a would be required, in addition to the DA Section 404 permit and TDEC water quality certification. It is anticipated that an EIS would be required.

Meadow Park Lake (Caney Fork Tributary)

Two alternatives were evaluated that affect the existing Meadow Park Lake: 1) construct a new dam 300 feet downstream of the existing dam or, 2) raise the existing dam to impound 20 additional feet of water. Environmental impacts that are likely to occur with either of these modifications to Meadow Park Lake are not as substantial as constructing a new reservoir since only small headwater streams would be impounded. The tailwater of the existing dam is highly degraded by the reduced stream flow below the existing lake and the loss of this 300 foot stream reach would be much less substantial than loss of a free-flowing stream reach. The additional impounded area is owned by the City of Crossville and could harbor terrestrial T&E species and cultural resource sites, although the potential of this is probably much less in comparison to the other impoundment sites. The watershed above Meadow Park Lake is predominately forested. This impoundment is also being considered in conjunction with the water harvesting alternative discussed later. A DA Section 404 permit review would be required.

Meadow Creek (Tributary of the East Fork Obey River)

A new impoundment was evaluated immediately below the existing Meadow Creek Lake on Meadow Creek (upstream of State Route 62). The existing dam was recently constructed by the Monterey Water System for an additional water source. Environmental issues with this project would be the conversion of sections of Meadow Creek from pool and riffle area to lacustrine habitat by enlarging the pool area. Impacts to the downstream sections of Meadow Creek would have to be considered even though it is already highly degraded from abandoned coal mine discharges. Meadow Creek above the existing lake appears to be of high quality. The gorge area above the lake is relatively pristine and forested. A cultural resource survey would likely be

required for the additional inundated area pool areas. A DA Section 404 permit review would be required.

Caney Fork

A potential dam site was evaluated on the upper Caney Fork. The design team visited the dam site, which is located in forested gorge area. The proposed impoundment would convert about 7.2 miles of pool and riffle habitat to lacustrine habitat. The pool area is estimated to be 272 acres at the spillway elevation (elevation 1635 feet). Portions of the proposed pool area have been previously disturbed by strip mining or sandstone quarries. This project would have substantial environmental impacts and would likely require an EIS if pursued by a Federal agency. If it were a non-Federal project, an Individual Section 404 and Section 10 DA Permit would be required. An EIS would likely be required as part of the DA permit issuance. Construction of an impoundment on the Caney Fork appears to be difficult to permit since it will likely fail the 404(b)(1) guidelines due to significant degradation and the presence of other less impacting alternatives. However, this determination cannot be made without information obtained by going through a public interest review (NEPA process).

The dam site is approximately 3.7 miles upstream of the 4000-acre Bridgestone/Firestone Conservation Area (a.k.a. Scott's Gulf) that was recently deeded to the State of Tennessee. The Conservation Area contains about twelve miles of the Caney Fork, and any upstream impoundment would have to consider impacts to the downstream river uses and quality.

There would be a high potential for threatened and endangered (T&E) species to be present within the proposed impoundment's pool area due to the pristine nature of the stream and lands. Based on records of the US Fish and Wildlife Service, the proposed pool area does contain occurrences of Cumberland Rosemary, listed as a Federally threatened plant. Three other federally listed T&E species that are likely to occur are the Cumberland Pigtail (mussel), Bluemask Darter (fish), and Virginia Spiraea (plant). No known surveys have been previously performed in this area since the area has not been developed. Likewise, cultural resources would be another issue to be considered since the pool area could contain cultural resource sites. A survey of significant natural and cultural resources would be required as part of an EIS.

6.3.4 Water Harvesting

This alternative consists of installing a water intake on the Caney Fork to supply water to an enlarged Meadow Park Lake or to a new upland impoundment site. Water withdrawal rates and timing, along with construction of an intake structure/pumping station and transmission pipeline, would be the primary environmental issues to be evaluated. In theory, water would be withdrawn during high flow events when impacts to aquatic systems would be reduced. When river flow drops below a certain level, water withdrawals would be curtailed. The minimum flow level would be set to avoid substantial impacts to the aquatic resources in the stream. Impacts associated with this project include flow reduction below the intake location, pipeline impacts of transmission line from the intake to the storage impoundment, and impacts from construction of the storage impoundment. Terrestrial and cultural resource impacts would be more critical for the transmission line and the upland storage impoundment. The impacts of flow reduction would be less substantial compared to constructing a new reservoir; however, water withdrawal impacts could be substantial enough to require an EIS. As discussed in the section on the new impoundment on the Caney Fork, impacts to the Bridgestone/Firestone Conservation Area would be considered. A DA Section 404 and Section 10 Permit would be required for the intake, including Water Quality certification from TDEC.

6.4 ENVIRONMENTAL IMPACT SUMMARY

Each alternative discussed in this report would have both construction and operation impacts. The proposed new impoundments appear to have the most potential for substantial impacts due to being located within very high quality, pristine stream sections and undeveloped gorge areas. The potential for impacts to threatened and endangered species is higher for the reservoir alternatives. The construction of an impoundment would eliminate habitat for these T&E species. Based on known habitat preferences, several T&E species could potentially occur in the proposed pool areas, especially on the Caney Fork. No formal review of known T&E species records with the TWRA, USFWS or TDEC (Division of Natural Heritage) has been performed to date. A key issue with constructing a new impoundment would be compliance with the Section 404(b)(1) Guidelines. Alternatives that do not involve impounding free-flowing streams appear to have the potential to supply the same or greater quantity of water supply with fewer environmental impacts. However, this formal determination cannot be made without going through a full public interest review under NEPA. The uncertainties with determining the projected water demand are another issue that is expected to surface during the NEPA process.

Each of these alternatives should be thoroughly evaluated before selecting a preferred option. Since the total additional water supply may not be needed until some time in the future, flexibility and expandability of each alternative should be a key consideration.

7.0 SUMMARY

This Preliminary Engineering Report (PER) serves the purpose of a reconnaissance or pre-feasibility study. It is a preliminary study of the existing water supply conditions of Cumberland County.

A preliminary Needs Assessment of the county water supply needs was followed by data collection and a minimum level of field work with respect to topographic surveys, streamflow measurements, and soil and geologic investigations. Six water supply alternatives were investigated: (1) Water Conservation; (2) Groundwater; (3) Pipeline to large reservoir; (4) Storage Impoundments which includes new impoundments and improvements to existing reservoirs; (5) Water Harvesting; and (6) No Action. A summary of the alternative analyses follows.

Water Conservation

Water conservation measures reduce water consumption, reduce water loss or waste, and improve water use efficiency. Unfortunately, conservation policies depend on thousands of customers behaving in certain ways. In this sense, conservation alone may not be a secure alternative to the expansion of treatment and distribution systems for meeting water needs in the Year 2050. However, water conservation is considered an essential complement to the traditional approach of capital improvements and should not be overlooked

Groundwater

A viable groundwater alternative was investigated because geomorphic and lithologic controls similar to those in Erwin, TN exist at the foot of the Cumberland Plateau. This area is located within a feasible piping distance from Cumberland County. Based on topographic and geologic maps, high-altitude photography, and site visits, the USGS identified five potential sites for groundwater supply along the Cumberland Plateau. The groundwater wells were designed with the assumption that groundwater could be pumped from three wells at each site at a rate of 500 gallons per minute (gpm) (approximately 0.75 million gallons per day (MGD). The total yield would therefore be approximately 11.25 MGD. The pipeline route from each of the five identified well sites was designed to flow to the Lake Holiday Treatment Plant in Crossville, TN. It should be noted that the feasibility of a viable groundwater alternative has not been fully

explored. No test drilling and aquifer testing have been performed on the potential ground-water area. Additionally, the effect of local coal mines on water quality at each of the five groundwater sites is unknown. This questionable data would be researched during an extensive feasibility study of the alternative.

Pipelines

This alternative explores the option of transporting raw water from existing reservoirs in adjacent geographic areas to Cumberland County and the Lake Holiday Treatment Plant. The selected sites are Watts Bar Lake in Roane County, Center Hill Lake in Dekalb County, and Great Falls Lake in White County. The Tennessee Valley Authority (TVA) initially developed the pipeline alternatives during a previous water supply study for the Catoosa Utility District. A target yield of 9 MGD was used to size the pumps and pipelines necessary to transport water form each of the raw water intakes to the Lake Holiday Treatment Plant.

Raising of Existing Dams

The fourth alternative discussed is to raise the height of existing dams in the County and use those raised reservoirs for water supply. Raising an existing dam would have less environmental impacts than construction of a new impoundment on a free-flowing stream. Reservoir selection began with a list of reservoirs within Cumberland County obtained from a database maintained by the State of Tennessee's Division of Water Supply. The list contained 53 reservoirs. Through a systematic elimination process, four reservoirs were identified for consideration Meadow Park Lake, Mayland Lake, Camp Ozone Lake and Tranquilechee Lake.

Meadow Park

Meadow Park Lake is an existing water supply reservoir located approximately five miles southwest of Crossville, Tennessee. The top of the Meadow Park Lake dam can be raised from elevation 1821.5 to 1840.0 feet NGVD29, the maximum height the surrounding terrain would permit. A yield analysis was performed with the engineering software HEC1-API and the results indicated the existing dam could provide 3 MGD. The yield analysis of the raised dam configuration indicated the enlarged reservoir could provide 4 MGD. The reason for the small increase in yield despite the increase in dam height is the small drainage area of the watershed

(5.19 mi^2). The amount of runoff from the watershed is the controlling factor as opposed to the amount of storage provided by the reservoir.

Mayland

Mayland Lake is a recreational lake located just south of Interstate 40 and approximately two and a half miles northwest of Plateau Road. A yield analysis was performed with an existing condition HEC1-API model and the results indicated the existing dam could provide 2 MGD. The top of the Mayland Lake dam can be raised from elevation 1926.4 to 1940.0 feet NGVD29, the maximum height the surrounding terrain would permit. The yield analysis indicated the raised dam configuration could provide 2 MGD. The reason there was no increase in yield despite the increase in dam height is the small drainage area of the watershed (2.76 mi^2). The amount of runoff from the watershed is the controlling factor as opposed to the amount of storage provided by the reservoir.

Camp Ozone

Camp Ozone Lake is a recreational lake located just north of Interstate 40 and approximately 3000 feet northwest from the post office in Ozone, Tennessee. The top of the Camp Ozone Lake dam can be raised from elevation 1680.0 to 1739.5 feet NGVD29, the maximum height the surrounding terrain would permit. The yield analysis indicated the raised dam configuration could provide 1 MGD. The reason for the small yield is the small amount of storage provided by the reservoir. The valley that the proposed reservoir would fill is narrow and does not provide adequate storage.

Tranquilechee

Tranquilechee Lake is an undeveloped residential lake located approximately 4000 feet north of the Cumberland and Bledsoe County line and four and a half miles southwest of Grassy Cove, Tennessee. Tranquilechee Lake was eliminated from consideration for raising the dam height to provide water supply after a review of the USGS Grassy Cove Quadrangle Map indicated the valley which the raised reservoir would fill is quite narrow.

New Impoundments

The construction of new impoundments is another alternative considered for providing additional water supply to Cumberland County.

Clear Creek

Clear Creek is a tributary of the Obed River, a portion of which is protected by the Wild and Scenic Rivers Act. Construction of an impoundment on a tributary to the Obed River would pose serious environmental concerns. However, because it had been listed as a potential alternative in a previous study for the Catoosa Utility District, the Clear Creek dam was included in this study. The proposed dam is approximately 72 feet in height, with the top of dam at elevation 1819.5 feet, the maximum height the surrounding terrain would permit. A yield analysis performed on the proposed Clear Creek reservoir calculated a potential water supply yield of 3 MGD.

Meadow Creek – Above Monterey Dam

Based on a review of the USGS quadrangle maps that cover Cumberland County, a dam built on Meadow Creek at the Cumberland County and Putnam County line appeared to have the potential to provide a substantial amount of water supply. Two problems were identified with this proposed dam site. The first problem identified is the Interstate 40 (I-40) crossing over Meadow Creek. The second and more critical problem identified is the existence of a dam downstream of the proposed site that was built by the City of Monterey for water supply. Building a dam upstream of the existing dam would capture the majority of the flow that is currently flowing into the existing dam, thus substantially depleting the water supply at the Monterey Dam. Due to these problems, the construction of a new impoundment on Meadow Creek at the Cumberland County and Putnam County line was not carried forward as an alternative in the study process

Meadow Creek – Below Monterey Dam

The existing Monterey Dam is located immediately south of Highway 62 and approximately a mile north of the Cumberland and Putnam County line. During the site visit to the existing structure it appeared a new dam could be built downstream of the current dam at a higher height. Review of the USGS Campbell Junction quadrangle quad confirmed a new dam built

approximately 200 feet downstream of the existing structure could be constructed with a top of dam at elevation 1799.5 feet NGVD29, which is estimated to be 30 feet higher than the existing top of dam. The proposed new dam is approximately 90 feet in height. The proposed Meadow Creek dam design would provide a water supply yield of 7 MGD. The yield of 7 MGD includes the yield that is already provided by the existing Monterey Dam, which was not determined during this study. It should be noted that the construction of this dam would require water supply negotiations between the City of Monterey and Cumberland County.

Meadow Park Lake

As previously mentioned, Meadow Park Lake is an existing water supply reservoir located approximately five miles southwest of Crossville, Tennessee. A new dam was proposed downstream of the existing structure at a higher height, approximately 300 feet downstream of the existing Meadow Park Lake Dam with a top of dam at elevation 1859.5 feet NGVD29, which is approximately 20 feet higher than the existing dam could be raised. However, as also previously mentioned, the results of the yield analysis on the raising of Meadow Park Lake Dam revealed that the reservoir's watershed is too small to contribute sufficient runoff to support a larger reservoir.

Caney Fork

The proposed impoundment site is located a mile and a half east of Clifty, Tennessee and 4000 feet west of Bruce Knob. The proposed dam was sized to avoid impounding water over U.S. Highway 70 at the Caney Fork, Beam Creek and Tantrough Creek crossings. The proposed dam is approximately 123 feet in height, with the top of dam at elevation 1665.0 feet NGVD29. The proposed Caney Fork dam design would provide a water supply yield of 12 MGD. Construction of an impoundment on the Caney Fork would pose serious environmental concerns.

Water Harvesting

The basis of water harvesting is to pump water from a stream during high-flow events and store that water in an off-site detention area for future use. While similar in concept to a water supply reservoir, water harvesting poses less of an environmental impact on the stream because it remains free-flowing. Water harvesting impacts on the source stream will be gradual as future

needs arise. Water is removed in gradually increasing increments as opposed to constructing a new impoundment based on a predicted need.

Two methods of water harvesting were investigated in this study. The first is the "traditional" definition of pumping water from a stream during high flows and storing it in a newly constructed detention area. The second method consisted of pumping water during high-flow events on the Caney Fork and storing it in a raised Meadow Park Lake Dam

Traditional Method

The typical detention area would require an area of land approximately 1000 feet x 1000 feet (23 acres) in size. There are several streams in Cumberland County that could support the aforementioned water harvesting system. These include the Caney Fork, Clear Creek, Meadow Creek, Daddys Creek, etc. Due to scope limitations, no specific pump sites were identified for this study. The designed water harvesting system could provide a water supply yield of 0.8 MGD, per detention area.

Caney Fork to Meadow Park Lake

The second method of water harvesting investigated was pumping water during high-flow events on the Caney Fork and storing it in a raised Meadow Park Lake reservoir. If the existing Meadow Park Lake Dam were raised, a substantial additional storage area could be utilized for water supply. This alternative would combine the benefits of both sites: the large drainage area upstream of the Caney Fork site could be utilized without impounding the river and the large storage area upstream of the existing Meadow Park Lake Dam could be utilized for water supply. As designed, harvesting water from the Caney Fork to a raised Meadow Park Lake Reservoir could increase the yield of Meadow Park Lake from 3 MGD (existing yield) to 11 MGD.

No Action

The "No Action" alternative accounts for the possibility that the Cumberland County community may decide not to pursue an additional water supply source(s). Following a moderate growth trend, the community would have an inadequate supply of water by the Year 2025, experiencing a water shortage in 25 years. In this instance, it is the responsibility of the County residents to either curtail the current residential and commercial growth of the community or acknowledge

the need for an additional water supply source. The actual growth rate of the community may be less than the projected moderate growth rate. Thus, no action would be a potential alternative. The actual growth rate of the community may also be greater than the projected moderate growth rate. In the short term, Cumberland County residents and utility districts could look to the surrounding communities for potential connections with neighboring water supply systems. Possible connections include Spring City to serve the Grandview Utility District, the City of Monterey to serve the West Cumberland Utility District, Fentress County to serve the Catoosa Utility District, and the City of Rockwood to serve the Crab Orchard Utility District in addition to the current support provided by the Grandview Utility.

APPENDIX A

HYDROLOGIC MODELING TECHNIQUES
HYDRAULIC MODELING TECHNIQUES
AREA CAPACITY CURVES
and
MODEL CALIBRATION CURVE

A.1 Hydrologic Modeling Techniques

Several hydrologic computer models were utilized to size and analyze the capacity of the water supply alternatives. The following paragraphs provide a brief description of each model used, as well as parameter derivations and assumptions used for each model. More specific details of how each model was used for the different water supply alternatives are provided later in this report.

Rainfall-Runoff Modeling

The Corps' computer program, HEC-1 "Flood Hydrograph Package," was used to size and analyze the effectiveness of the storage impoundments and water harvesting alternatives. HEC-1 produces discharge hydrographs that represent the flow in a stream or inflow into a reservoir. These discharge hydrographs are a result of applying rainfall excess, or runoff, to a unit hydrograph. The unit hydrographs were developed by synthetic methods and calibrated to historic events by simulating observed events. Rainfall excess, for this type modeling effort, is essentially that portion of rainfall that can be observed in a stream as either baseflow or surface runoff. A substantial portion of the volume of rainfall is either absorbed into the ground or vegetation. A modified version of HEC-1 (HEC1-API) was used to predict these transformations and provide the tools necessary to complete basic hydrologic designs for the water supply alternatives. This section outlines the procedures and assumptions used to develop the HEC-1 models.

Subbasin Delineation

The first step in a rainfall-runoff modeling process is to delineate and measure the drainage area of the subbasins within the watershed being modeled. Table A-1 provides specific information on the subbasin delineations for each of the alternative analyses that utilized an HEC-1 model.

Table A-1
Subbasin Delineations for HEC-1 Models

Site	Type of Alternative	Number of Subbasins	Size Range (sq. mi.)	Total Drainage Area (sq. mi.)
Clear Creek	New impoundment	1	5.55	5.55
Meadow Creek	New impoundment	2	2.62 to 7.32	10.6
Caney Fork	New impoundment	8	3.16 to 11.55	58.29
Camp Ozone Lake	Raised impoundment	1	3.98	3.98
Mayland Lake	Raised impoundment	1	2.76	2.76
Meadow Park Lake	Raised impoundment	1	5.2	5.2
Caney Fork	Water Harvesting	8	3.16 to 11.55	58.29

As shown in Table A-1, the alternatives located in the headwaters of a stream required only one subbasin. The alternatives located lower in the stream's watershed, thus having a larger drainage area, were divided into several subbasins. The subbasin divisions were located at points of substantial changes of flow, such as at the confluence of a tributary, or at reservoirs the flow travels through.

For the impoundment alternatives an additional smaller subbasin was separated out to allow for direct runoff of rain on the pool area. All subbasin boundaries were determined from USGS 7.5-minute quadrangle topographic maps at a scale of 1" = 2000' and a contour interval of 20 feet. The drainage areas of the subbasins were determined by using a digital planimeter.

Estimation of Modeling Parameters

A unit hydrograph was used to distribute the runoff for the subbasins with respect to time. Each of the subbasins listed in Table A-1 was represented by a Clark synthetic unit hydrograph. This type unit hydrograph is based on average basin time of concentration, a storage or basin shape coefficient (R), and a time-area curve. The Clark method produces better control than other hydrograph methods in defining the shape and the volume expected from a subbasin.

The definition of shape and volume for any hydrograph is usually very important; however, for the impoundment analyses only accurate volume definition is critical. This is due mainly to two factors. First, all the inflows from the watershed are expected to be either captured in the reservoir or delayed by the reservoir for a sufficient amount of time to negate the need to accurately define the shape of the hydrograph. Second, the available rainfall data found for the

historical drought periods are recorded as daily values. The times of concentration for the basins delineated are much less than 24 hours. Therefore, the peaking of these hydrographs due to intense rainfall bursts cannot be accurately simulated with this type data.

Times of Concentration

The time of concentration (T_c) for each subbasin was determined using methods described in the Soil Conservation Service's (SCS) Technical Release 55, "Urban Hydrology for Small Watersheds." Time of concentration is based on overland, shallow concentrated and channel flow along the longest flow path. Table A-2 lists the range of times of concentration for the modeled subbasins.

Table A-2
Times of Concentration for Subbasins

Site	Type of Alternative	Number of Subbasins	T_c Range (hours)
Clear Creek	New impoundment	1	2.23
Meadow Creek	New impoundment	2	2.62 to 7.32
Caney Fork	New impoundment	8	1.54 to 4.39
Camp Ozone Lake	Raised impoundment	1	3.96
Mayland Lake	Raised impoundment	1	2.62
Meadow Park Lake	Raised impoundment	1	4.71
Caney Fork	Water Harvesting	8	1.54 to 4.39

Unit Hydrograph

The Clark R coefficient is usually referred to as a storage coefficient. This is because R affects the amount the hydrograph is attenuated with respect to time. Because this coefficient directly affects the shape of the outflow hydrograph, it can also be used to account for the effects of the basin shape. The basin shape is typically accounted for by a time area curve, however, such a curve is very cumbersome to develop. Therefore, the use of the R coefficient is faster and more direct. The coefficient R can be estimated using the relationship $R/(T_c + R)$. By setting this expression equal to a value between 0.1 and 0.7, a sharply peaked to an attenuated unit hydrograph can be obtained. Selection of this value was based on engineering judgment of the expected shape of each subbasin's outflow hydrograph and calibration to the average daily flow data at the Crooked Creek gage near Jamestown, TN (closest stream gage with adequate period of record for calibration). A Clark R coefficient determined by setting the expression equal to

0.3 to 0.6 was selected for all the subbasins modeled. In addition to the calibration of the Clark R coefficient, a cursory sensitivity analysis was performed to verify the R coefficients used. In using R to account for basin shape, the default time area curve option was selected within the HEC-1 program. The time-area curve was not used to account for the basin shape, and therefore was held constant in the modeling process.

API Continuous Losses

The Antecedent Precipitation Index (API) method was used to determine rainfall loss rates for the continuous simulations. This method was adopted by the Nashville District to model continuous events on the Cumberland River and its tributaries. To distinguish between continuous and single event simulations, the models utilizing the API methodology for performing continuous simulations will be referred to as HEC1-API models. The models used to perform a single event simulation will be referred to as HEC-1 models. The API method of transforming rainfall to runoff is empirically based. Therefore, the matching of observed occurrences is accomplished by using numerical techniques. The actual rainfall loss rate is based on the week of the year and the antecedent rainfall. The week of the year accounts for several physical processes such as temperature, evaporation rates, vegetation and hours of sunlight. The calibration of this model and its use in this study is discussed at the end of this section.

Reservoir Routing

The Modified Puls routing technique within HEC-1 was used to perform all reservoir routings. This is a simple level pool storage routing technique based on the principle of conservation of mass and the relationship that change in reservoir storage, for a given time period, is equal to average inflow minus average outflow. The reservoir storage data used in the HEC-1 models were obtained from several sources. For the new impoundments the storage data were determined from USGS 7.5-minute quadrangle topographic maps at a scale of 1" = 2000' and a contour interval of 20 feet.

Determining the storage data for existing reservoirs that were being considered for raising as a water supply alternative proved to be more difficult. No as-built plans containing area-capacity curves could be found for any of the reservoirs. Archived USGS quadrangle topographic maps were found which were created prior to the impoundment of Meadow Park Lake and Camp

Ozone Lake. The 1912 edition of the 15-minute Crossville Quadrangle Map with a contour interval of 20 feet was used to develop the storage data for the Meadow Park Lake HEC-1 model. The 1949 edition of the 7.5-minute Ozone Quadrangle Map with a contour interval of 20 feet was used to obtain the storage data for the Camp Ozone Lake HEC-1 model. The only archived mapping that could be found for the Mayland Lake area had a contour interval of 100 feet, which is too large to develop an accurate area-capacity curve. To determine the volume of Mayland Lake a hydrographic survey was conducted. A boat equipped with GPS hydrographic equipment was used to map Mayland Lake below the water surface. The Hydro Volume module of the Hydro Processing software was utilized to calculate the volume of the reservoir at each whole foot interval. This data was input into the Mayland Lake HEC-1 model. The area-capacity curves for each model are illustrated in Figures A-1 through A-6 of this Appendix.

Non-Reservoir Routing

The Muskingum-Cunge routing technique within HEC-1 was used to perform the non-reservoir routings for the HEC-1 models. This is a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. A representative eight-point channel cross-section is used for each routing reach. The eight point channels used were developed from the digital USGS 20-foot contour interval quadrangle mapping and site visits. In addition to the eight point channel, the Muskingum-Cunge technique also utilizes the roughness coefficients (Manning's n-values) for the channel and overbanks, reach lengths and slope in the routing calculations.

Storm Development

Several different types of storm events were used in the analyses of the water supply alternatives. The spillway widths of the impoundments were determined based on the one-half and full Probable Maximum Flood (PMF). The water supply production capacity (yield) of the alternatives was based on drought studies, which utilize historical rainfall. In the following paragraphs there is a general discussion of each of the storm events.

Probable Maximum Flood (PMF)

In May 1973, "The Safe Dams Act of 1973" was passed by the State of Tennessee, and amended in October 1995. The Safe Dams Act contains standards for new and existing reservoirs, which

include the magnitude of the freeboard design storm to be used in sizing the dam. The magnitude of the required freeboard design storm is dependent on the size and downstream hazard potential of a dam. A dam with a height of 20 to 40 feet is classified as "Small," a height of 41 to 100 feet is classified as "Intermediate," and a height greater than 100 feet is classified as "Large." Two downstream hazard potential categories apply to the impoundments analyzed for this study, "Significant" and "High." A hazard potential classification of "Significant" indicates a dam located where failure may damage downstream private or public property, but such damage would be relatively minor, and chances of loss of human life would be possible but remote. A hazard potential classification of "High" indicates a dam where failure would probably result in any of the following: loss of human life; excessive economic loss due to damage of downstream properties; excessive economic loss, public hazard, or public inconvenience die to loss of impoundment and/or damage to roads or any public or private utilities. Table A-3 lists the Tennessee Safe Dam classifications of each impoundment and the required freeboard design storm.

Table A-3
Tennessee Safe Dam Classifications
of Impoundment Alternatives

Site	Type of Alternative	Size Classification	Hazard Potential Category	Freeboard Design Storm
Clear Creek	New impoundment	Intermediate	Significant	1/2 PMF
Meadow Creek	New impoundment	Intermediate	High	PMF
Caney Fork	New impoundment	Large	Significant	PMF
Camp Ozone Lake	Raised impoundment	Intermediate	Significant	1/2 PMF
Mayland Lake	Raised impoundment	Intermediate	Significant	1/2 PMF
Meadow Park Lake	Raised impoundment	Intermediate	Significant	1/2 PMF

It should be noted for those sites that require only the ½ PMF for the freeboard design storm, State criteria states the freeboard design storm would have to be upgraded to a full PMF if the hazard potential classification of a site changes due to downstream development.

The Probable Maximum Storm (PMS) is the precipitation event that produces the PMF. This storm represents the most severe combination of flood producing rainfall patterns possible for a given area. The PMF is selected as the design flood for projects that allow no degree of risk and an unusually high degree of protection. This condition usually exists for high head dams or where a threat to life and/or extreme damage could occur downstream should the dam ever fail.

The selection of the design storm is a key link between a rainfall-runoff model and the determination of the size of the actual dam. The PMS used in the designs was determined using methods provided in National Weather Service publications Hydrometeorolgical Report Number 51 ("HMR-51") and "HMR-52." The average rainfall for the watersheds upstream of the impoundments was determined based on centering the PMS over the watershed. The resulting hourly rainfall values were then input into the HEC-1 model to determine the PMF and ½ PMF discharges. The PMS hourly rainfall values are shown in Table A-4.

HEC-1's multi-plan capability was used to determine the 1/2 PMF. The "JR" card was used to ratio the PMS rainfall values by fifty percent. The model was used with these reduced hourly rainfall values to determine the 1/2 PMF discharges.

Table A-4
Probable Maximum Storm Precipitation
Hourly Values

Depth in inches for 1-hour increments of PMS											
0.063	0.063	0.063	0.063	0.063	0.063	0.077	0.077	0.077	0.077	0.077	0.077
0.099	0.099	0.099	0.099	0.099	0.099	0.140	0.140	0.140	0.140	0.140	0.140
0.208	0.218	0.230	0.245	0.263	0.283	0.608	0.746	0.876	1.000	1.117	1.228
1.606	2.448	4.190	15.782	3.326	2.126	0.500	0.434	0.381	0.341	0.313	0.298
0.177	0.177	0.177	0.177	0.177	0.177	0.116	0.116	0.116	0.116	0.116	0.116
0.087	0.087	0.087	0.087	0.087	0.087	0.069	0.069	0.069	0.069	0.069	0.069
Total Rainfall = 43.735 inches during a 3-day period											

Historical Rainfall Information

Historical precipitation data for the modeled watersheds and surrounding areas were collected from various precipitation gages. The precipitation gages used in this study are in the vicinity of the watersheds not more than fifty miles away. A total of thirteen gages were selected and collectively used to determine total continuous rainfall amounts and patterns for the Years 1912 to 1997. The gages and their respective years of record available for use in this study are listed in Table A-5.

Rainfall records for all of the gages listed in Table A-5 were retrieved from CD-ROM's containing NWS records. All of the precipitation gages used for determining daily basin average precipitation values in this study had daily precipitation value totals as the minimum time period available.

Table A-5
Available Precipitation Gage Information
Within Fifty Miles of The
Modeled Watersheds

Gage Name	Type	Dates in Operation
Allardt	Precipitation	1928 – 1997
Cookeville	Precipitation	1951 – 1997
Crossvile EXP STN	Precipitation	1912 – 1997
Crossville FAA	Precipitation	1954 – 1997
Crossville	Precipitation	1949 – 1954
Decatur	Precipitation	1927 – 1956
Fall Creek	Precipitation	1949 – 1970
Jamestown	Precipitation	1951 – 1997
Livingston	Precipitation	1948 – 1988
Monterey	Precipitation	1948 – 1997
Pikeville	Precipitation	1962 – 1997
Rockwood	Precipitation	1962 – 1997
Sparta	Precipitation	1948 – 1997

DSS Database

All of the historical precipitation gage data records were entered into the Corps' Data Storage System (DSS) database system. The data were entered in regular time series format. The use of this database system allows direct input and output from many Corps' models such as HEC-1 and PRECIP. The modified version of HEC-1 used for this study (HEC1-API) requires the use of a DSS database system.

Basin Average Rainfall

The Corps' computer program PRECIP was used to develop basin average rainfall for the watersheds modeled. PRECIP computes area-average hyetographs from observed precipitation gage data. Like HEC-1, the program is designed for use with a DSS database. Rainfall at the centroid of each watershed is computed based on a weighted average of nearby rain gages. The gages are weighted based on the least distance squared from the watershed centroid. The daily basin average precipitation values for the 85-year period of record were computed by PRECIP and written into the DSS database. The computed values were then read directly into the HEC-1 and HEC1-API models that transform them into inflow to the water supply alternatives.

Rainfall-Runoff Model Calibration

Observed stream gage data must be available to perform accurate calibration of an HEC1-API model. In Cumberland County there are no stream gages with a similar drainage area size as

those modeled for water supply alternatives and an adequate period of record to calibrate the API parameters. Therefore, data from the Crooked Creek gage near Jamestown, Tennessee was used. The Crooked Creek gage has 3.62 square miles of contributing drainage area above the gage site and was in operation from 1977-1981.

To calibrate the API parameters a simple HEC1-API model was setup to represent the Crooked Creek watershed above the Crooked Creek gage. Precipitation from the Allardt, Tennessee precipitation gage (located less than 3 miles from the Crooked Creek stream gage) was applied to the watershed, and the API parameters were varied until the calculated discharges matched the observed discharge data from the Crooked Creek gage. Plots of the final Crooked Creek model calibration runs are shown in Figures A-7 through A-11.

As illustrated in Figures A-7 through A-11, the computed hydrographs from the Crooked Creek HEC1-API model match the timing and peaks of smaller events but tend to be lower than the observed hydrographs during large events. The HEC1-API model is not able to reproduce the high peaks because of the use of daily rainfall values. As previously mentioned, all of the precipitation gages used in this study had daily precipitation value totals as the minimum time period available. The use of daily rainfall values in a rainfall runoff model cannot accurately simulate the peaking of larger events caused by intense rainfall bursts.

Because the capacities of water supply alternatives are determined from drought events, accurate simulation of the amount of runoff expected from a watershed (volume of the hydrograph) during drought conditions is more critical than matching the peaks of the hydrographs during large events. For this reason, the API parameters in the Crooked Creek model were not only adjusted to best match the historical hydrographs at the gage, but also to match the historical volume of runoff measured at the gage. Table A-6 lists the observed and computed runoff amounts from the Crooked Creek gage and HEC1-API model. Note, the gage was missing runoff data for the years 1978 and 1981, making only the years 1977, 1979 and 1980 available.

Table A-6
Calibration of HEC1-API Model
Observed and Computed Runoff Amounts

Year	Observed Runoff (inches)	Computed Runoff (inches)
1977	25.60	25.63
1979	33.30	32.74
1980	17.80	16.48

The calibrated API parameters from the Crooked Creek model were then used in the various HEC1-API models for determining the capacity of the water supply alternatives. Due to the close proximity of the Crooked Creek and modeled watersheds (less than 40 miles), and similarities in soil type, range of temperatures, seasonal variation in temperatures and vegetation types, the API parameters should be similar.

A.2 Hydraulic Modeling Techniques

The hydraulic calculations in this study were computed with the Corps of Engineers' program "HEC-RAS, River Analysis System, version 2.1, October 1997." HEC-RAS is typically used to develop water surface profiles for a stream or system of streams for various frequency or historical flow events. HEC-RAS can also be used to develop a rating curve (a plot of water surface elevation versus discharge) for a given cross-section. This latter use is how the HEC-RAS program was utilized in this study.

The water harvesting alternative requires modeling that can calculate the water surface elevation in a creek at each time interval of the modeling process. This can be accomplished using the calibrated HEC1-API model previously discussed if a rating curve at the site is provided. To accomplish this, an HEC-RAS model was created which included a cross-section at the site as well as two cross-sections downstream from the site. The two downstream sections were included to provide mathematical stability to the calculations. The data used to develop the cross-sections for the HEC-RAS models were the USGS 20-foot contour interval quadrangle maps and field information from site visits. The HEC-RAS models were run with discharges ranging from 10 to 50000 cubic feet per second (cfs). Each discharge and corresponding water surface elevation was input into the HEC1-API model.

APPENDIX B

MCACES

(Micro-Computer Aided Cost Engineering System)

COST ESTIMATING OUTPUT