ATTACHMENT 1
SUMMARY OF SIZING CRITERIA

Information presented below was extracted from the "*Draft Project Extension Report-Development of Miner Flat Dam and Canyon Day Irrigation Project*", Feb. 2007, and is provided to assist with background. Much of the discussion relates to alternatives that were evaluated and are no longer relevant. Except as noted in the paragraph below, the information only assists with an historical understanding and should only be useful in future designs at the discretion of the consultant selected for design of the water treatment plant and pipelines to Cibecue.

THE DEMAND OF 11,568 GPM IS THE ONLY SIZING CRITERIA THAT IS NOT REVIEWABLE BY THE CONSULTANT. MOST INFORMATION IN THE EXCERPT BELOW IS FOR ASSISTANCE ONLY. THE CONSULTANT WILL BE REQUIRED TO DEVELOP DESIGNS BASED ON INDEPENDENT ANALYSES.

Project facilities were sized for year 2030 based on population and maximum daily water use. The design population of 35,907 in year 2030 was used for the communities of the Greater Whiteriver Area, Carrizo and Cibecue or 93% of the Reservation population.

Average day demand in 2030 for the combined project communities of 6,786,000 gallons. Design for maximum day increased the demand to 15,270,000 gallons applying a peaking factor of 2.25. When converted to flow rate, the maximum day demand is equivalent to 11,568 gallons per minute (gpm) based on 22 hours of operation at the water treatment plant or source of supply.

The following describes the facilities necessary to serve the Greater Whiteriver, Carrizo and Cibecue areas based on water supply from the north fork of the White River.

<u>North Fork White River Diversion</u>

The Indian Health Service (IHS) has developed preliminary designs for a diversion facility on the North Fork White River to serve a new water treatment facility for the Greater Whiteriver Area. This diversion facility would be located downstream from Diamond Creek near 51st Street to connect to a proposed water treatment plant (WTP). The diversion facility would consist of an intake system and raw water pump station to supply the WTP.

The diversion works for the project proposed here would be the same as the diversion that would exist at the initiation of this project.[1] Therefore, there would be no additional cost if future diversions are taken from the same location on the North Fork White River. Due to the

[1] It is now considerable likely that a new diversion structure will be needed because the actual development of the IHS water treatment used all available lands in the contiguous site. The consultant will be required to site the new water treatment plant and then determine the usefulness, if any, of the existing diversion dam. The consultant will likely locate and design a new diversion dam to serve the new water treatment plant.

additional demand for the domestic system, construction of a new raw water pump station or expansion of the existing would be necessary.

Water Treatment Plant

The IHS project contemplated the construction of a water treatment plant with associated raw water transmission main settling basins near the diversion site on the North Fork White River below Diamond Creek. The project would provide treatment of 2 million gallons per day (MGD) with room to expand to a potential 4 MGD. Two package treatment plants would be placed in parallel and would be enclosed in a pre-engineered building. Water would then be conventionally treated using polymer injection, flocculation, sedimentation, filtration and chlorination. The project would serve the Greater Whiteriver area.

The IHS water treatment facility would be supplemented when the project proposed here is implemented, and the costs of the expanded water treatment facility beyond the 2 MGD level are considered here. Micro filtration, media filtration or conventional water treatment are proposed for the project. Pilot studies would be conducted to the extent necessary to make final determinations of the most cost effective water treatment plant. The WTP would be sized to meet project demands for the Greater White River area and the communities of Carrizo and Cibecue in year 2030 as presented in Chapter 3. The future demand of 11,568 gpm would be offset by the continuation of existing sources of supply totaling 2,520 gpm. Therefore, an additional capacity of 9,048 gpm would be required from the water treatment plant proposed here.

Table 4-1 summarizes selected water quality characteristics of the White River near Fort Apache as measured by the U.S. Geological Survey gaging station 9-4910 at considerable distance downstream from the diversion point. Upstream water quality should be superior to samples taken at the gaging station. The characteristics selected are those most relevant to treatment processes designed to bring surface water into compliance with present and future drinking water regulations and to produce a highly aesthetic finished product for the users in the public water system. The streamflow and water quality data points were based on a minimum of 36 common measurements (turbidity) and as many as 45 measurements for other constituents with streamflow ranging from 26 to 1,660 cfs. All measurements were taken between 1976 and 1979.

Some of the water quality constituents are reasonably well correlated with streamflow (R^2 =0.635 to 0.7354, TDS, hardness and sulfate concentration), and others are poorly correlated (R^2 = 0.02 and 0.067, total nitrogen and turbidity, respectively). Some characteristics vary directly with increasing flow (total iron, total organic carbon (TOC), turbidity and total arsenic). Others vary inversely with streamflow (hardness, TDS and sulfate).

Table 4-1 provides predicted values of each constituent for streamflows ranging from 10 to 500 cfs. All predictions fall within acceptable ranges with the exception of hardness at low flows (315 mg/l at 10 cfs) and a maximum observed total arsenic level of 11 µg/l. The latter is an outlier and may represent an error in measurement or analysis. It also represents “total” arsenic, which includes dissolved arsenic and arsenic carried in suspension with sediments. Normally dissolved arsenic is much lower in value than total arsenic. Because sediment is removed in drinking water treatment processes, dissolved arsenic is expected to fall well within ranges of acceptability.

Table 4-1 discloses that finished water quality will be highly satisfactory from both a health and aesthetic perspective. The low concentration of TDS and sulfate make the raw water exceptional from the standpoint of taste and odor. Water quality normally degrades from upstream to downstream in a natural surface water system. The location from which the measurements were taken is downstream from the community of White River, the Canyon Day Farm and the regional wastewater facility. The latter were not in operation, however, during the period of measurement. Water quality data at upstream locations on the White River or North Fork White River is expected to be more applicable than data presented in Table 4-1.

TABLE 4-1
TABLE 10.1.1.2.1
SUMMARY OF SELECTED WATER QUALITY CHARACTERISTICS
WHITE RIVER NEAR FORT APACHE

						Predicted Concentration					
			Regression			Flow cfs					Maximum
Constituent	Units	Standard	a	b	R^2	10	25	50	100	500	Observed
Secondary											
Hardness	mg/l	250	2.8571	(0.3592)	0.714	315	226	177	138	77	230
Total Iron	mg/l	0.3	1.2783	0.6605	0.310	0.1	0.2	0.3	0.4	1.2	10.0
TDS	mg/l	500	2.8847	-0.30594	0.635	379	286	232	187	115	294
TOC	mg/l	--	(0.1138)	0.23915	0.155	1.3	1.7	2.0	2.3	3.4	20.0
Turbidity	JTU	--	0.1683	0.34201	0.067	3	4	6	7	12	260
Suspended Sed	mg/l	--	(0.7530)	0.86528	0.819	1	3	5	9	38	217
Primary											
Total Nitrogen	mg/l	10	-0.24377	-0.14224	0.018	0.4	0.4	0.3	0.3	0.2	4.7
Sulfate	mg/l	400	2.5880	(0.4917)	0.735	125	80	57	40	18	87
Total Arsenic	μg/l	10	0.0731	0.0359	0.004	1.29	1.33	1.36	1.40	1.48	11.00

Form of Regression $y_i = 10^{\wedge\, K}$

Where y_i = water quality value for constituent "i" in units for that constituent
$K = (a+b(\log_{10}(x_i)))$
x_i = Streamflow, cfs
a and b are coefficients given above

When alkalinity is low (no known measurements of alkalinity were available), TOC removal of 25% and 40%, respectively, is proposed by EPA. Large surface water systems (greater than 10,000 persons) would be required to sample at the plant on a monthly basis for TOC and alkalinity. Conventional filtration treatment systems must monitor (1) source water TOC prior to any treatment and (2) treated TOC at the same time in paired samples.[2] Removal of TOC at the levels proposed by EPA may not be feasible for many public water systems. In the event a public water system cannot provide the necessary percentage TOC removal, jar test

[2] Federal Register, May 10, 2000, *National Primary Drinking Water Regulations: Ground Water Rules; Proposed Rules*, Vol. 65, No. 91, p. 69422, *et seq*, Environmental Protection Agency.

procedures are proposed by EPA for determining the point at which addition of alum or an equivalent dose of a ferric coagulant has reached a point of diminishing returns and further removal is infeasible.[3] Jar testing for TOC removal is proposed for this project in final design.

EPA initially disallowed pre-disinfection credit in order to maximize removal of organic precursors prior to the addition of disinfectant However, based on comments from public water systems, the proposed rule does not impose constraints on the practice of pre-disinfection as proposed at the water treatment plant. Credits will be applicable for pre-disinfection.

Suspended sediments, an indicator of turbidity, will also be carried by raw water diverted from the White River. Removal of suspended sediments (turbidity) will remove most arsenic, as discussed above, and some TOC. Suspended sediments averaged 36 mg/l and ranged from 3 mg/l (41 cfs) to 217 mg/l (1,660 cfs) for a limited number (11) samples collected in the late 1970s.

<u>Processes</u>

White River raw water, as described in the previous section, can be treated satisfactorily by several treatment methods to meet federal safe drinking water criteria. These alternatives will be investigated in more detailed design-level studies outside the scope of this document, and a selection will be made based on costs and the ability to produce a high quality dependable finished water supply.

Water treatment at the White River plant will involve the removal, including filtration, of suspended particles from the raw water and disinfection of the filtered water to remove microorganisms. The following processes are potentially available within the proposed treatment plant, subject to requirements to produce a finished product meeting federal safe drinking water standards and public opinion respecting matters such as fluoridation and methods of disinfection:

- potassium permanganate oxidation;
- powdered activated carbon absorption;
- alum (or ferric chloride) and cation coagulation;
- flocculation;
- sedimentation;
- gravity filtration;
- pH modification;
- corrosion inhibitors;
- disinfection (chlorimination with consideration of ozone for partial disinfection);
- fluoridation.

While direct filtration operates without treatment processes involving sediment removal before filtration, this alternative was eliminated from consideration on the basis that suspended sediments in relatively high concentrations are expected during runoff periods. On the other hand, some treatment processes can be bypassed and lower operating costs will result during

[3] Ibid., p. 69413

some periods of the year when raw water quality does not require all the processes associated with sediment removal before filtration and direct treatment can be effective.

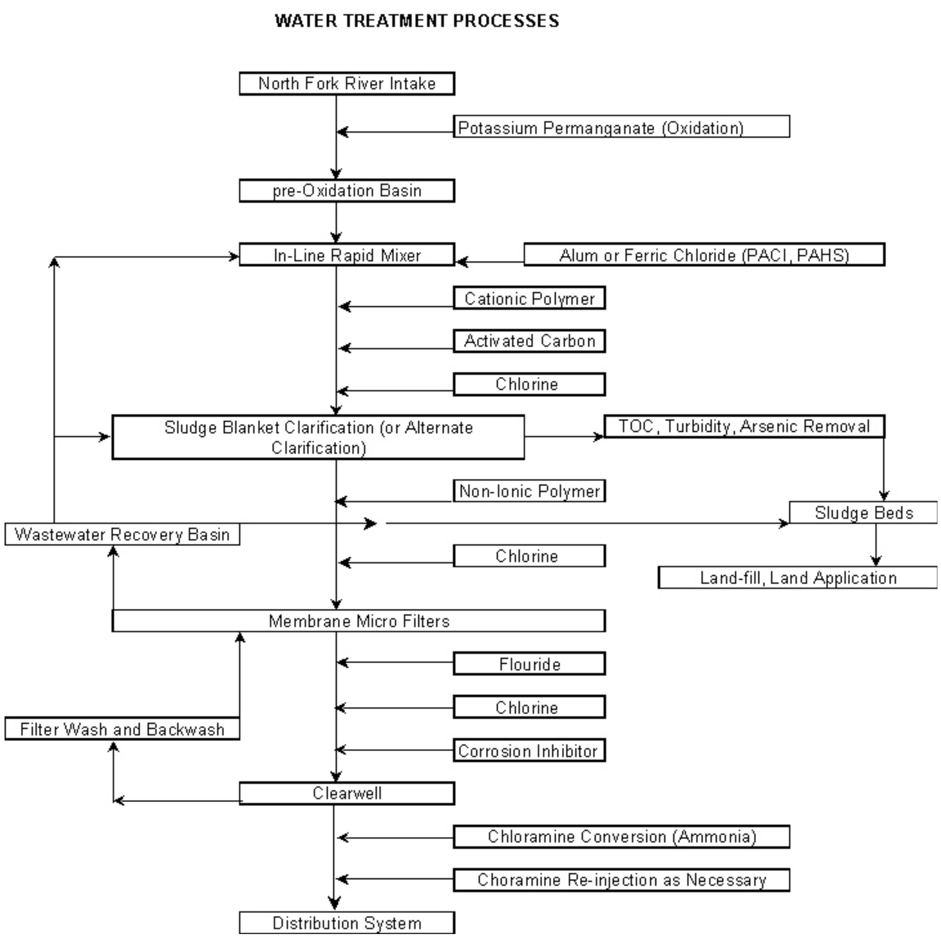
The White River treatment plant can provide a product to a future nano-filtration, reverse osmosis or other comparable process to remove contaminants that are not known to have an impact on human health at levels currently regulated.

Figure 4-1 summarizes the general process of treating water delivered from the raw water intake on the White River to the finished water in the clear well before entry to the distribution system.

Pre-Oxidation

Potassium permanganate would be added (as necessary) as the initial chemical to promote oxidation and minimize taste and odors. This would be accomplished with the delivery of raw water to a pre-oxidation basin followed by an in-line (or other similar type of) rapid mixer with controls to prohibit backflow of chemicals. Depending on final site conditions, the raw water

FIGURE 4-1



pipeline from the intake may be used as the "pre-oxidation basin" if an adequate contact time (15 to 30 minutes) can be achieved prior to the water treatment plant rapid mixer.

Mixing, Coagulation and Flocculation

Mixing, as referred to above, is a process to uniformly disperse chemicals added for coagulation through the raw water taken at the intake. Coagulation is the addition of chemicals that destabilizes the forces among particles that keep them apart and promotes their attachment to one another for removal as the treatment process progresses. These particles may be silts, clays and organic matter that remain suspended in the source water. Enhanced coagulation will be designed to remove organic material to comply with the disinfectant byproducts rules. This will be accomplished by increasing chemical dosage and/or pH adjustment. Ferric chloride is the preferred coagulant by other surface water treatment plants in the region as a means of achieving arsenic removal. The most common coagulant, absent the presence of arsenic, is alum (aluminum sulfate). Flocculation is the process that settles suspended particles and follows the addition of coagulation chemicals. In a conventional water treatment plant, flocculation occurs in sedimentation basins prior to the clarification process. Agents that can aid the flocculation process include cationic or anionic polymers, activated silica and bentonite. The rapid mixing, coagulation and flocculation processes may be combined in proprietary devices, such as a Superpulsator™. Pilot studies will be undertaken to determine the whether separate facilities for rapid mixing, coagulation and flocculation consistent with a conventional water treatment plant will be utilized or whether these processes will be combined in a proprietary clarifier. Alum or ferric chloride would be added to the rapid mixer for coagulation. Ferric chloride will be used if needed to enhance arsenic removal. Alum will be used if arsenic can be successfully removed with turbidity. Polyaluminum chloride (PACL) and partially neutralized alum-polyaluminum hydroxy sulfate (PAHS) are alternative coagulants. Selection of a final coagulant will be based on effectiveness of turbidity reduction, arsenic removal, organics removal, impact on disinfection byproduct reduction, sludge production, pH and corrosion impacts, ease of handling and storage, and costs.

Clarification

Clarification will reduce the remaining suspended sediments, including organics, after the coagulation and flocculation processes, or combined with these processes, before filtration. Alternatives for clarification include membrane filtration and media filtration. Membrane filtration may include microfilters or nano filters. The latter will remove particle sizes that are 1,000 times smaller than the particle sizes removed by microfilters. This level of removal is not considered necessary for this project.

Before entering the clarifier, cationic and non-ionic polymers, activated carbon and the first stage of chlorine injection for disinfection will be provided as necessary. The principal difference in the water treatment process discussed here and a conventional treatment process is the substitution of sludge blanket clarification (or another alternative clarification system) for conventional flocculation/sedimentation. The clarifier will remove suspended organic carbon (a precursor to formation of disinfectant byproducts), turbidity and suspended arsenic. These contaminants will be delivered to sludge beds and thereafter to landfill or land application, depending on compliance requirements for the final concentrations of constituents that are produced.

Preliminary cost estimates indicate that a pulsed blanket clarifier may be more cost effective than conventional flocculation/sedimentation. Detailed sizing based on recommendations from manufacturers and a review of other facilities treating similar waters should be performed before this clarifier system is selected. Pilot testing may be warranted since this process does not work well with all types of waters and contaminants. In addition to the pulsed blanket clarifier, other types of alternative flocculation/sedimentation systems should be evaluated, including:

- Solids contact clarification.
- Conventional (not pulsed) sludge blanket clarification.
- Contact clarification.
- Ballasted clarification.

It is not contemplated at present that arsenic in the waste sludge will be of sufficient concentration to cause concern with any disposal method. Emphasized is the fact that arsenic removal is part of the planning process, but removal of turbidity is expected to remove arsenic to the point that the remaining dissolved concentration will be well below a 10 µg/l level.

Filtration

From the clarifier, water will be delivered to gravity micro (membrane) or media filters. Conceptual value engineering of the water treatment plant determined that conventional gravity media filters would be less costly than membrane filters, but both alternatives will be re-examined in final design of the water treatment plant. Before water is delivered to the filters, additional injection of chlorine for disinfection, polymers and corrosion inhibitors is proposed. Beyond the filters, fluoride is proposed for injection, depending on public acceptance, as a beneficial dental treatment. Additional chlorine and conversion to chloramines through addition of ammonia is proposed to finish the treatment of water before and after the clearwell. Part of the finished water delivered to the clear well will be used to wash the surface and backwash the filters. The wash water will then be delivered to a recovery basin and thereafter to sludge drying beds or returned to the front of the treatment process at the in-line rapid mixer or to the clarifier, depending on quality of the wash water. This latter phase in the process will be an operational decision based on conditions that will vary throughout the seasons and the year.

Disinfectants and Disinfectant Byproducts

Alternatives for disinfectants include chlorine, chlorine dioxide, chloramines, ozone, ultraviolet light and combinations thereof. Because residual levels of disinfectant are required in the finished water, any use of ozone or ultraviolet light must be followed by chlorine or chloramines to complete the disinfection process and provide a residual. Ultraviolet light was not considered here. Some consideration may be given to ozone, which is gaining in popularity in combination with chloramines (a secondary disinfectant). This combination generally produces better taste than chlorination. Ozone is particularly effective in achieving log 3 (99.9%) removal or inactivation of *Giardia Lambia* cysts and log 4 (99.99%) removal or inactivation of viruses.[4]

Chloramines are formed from the reaction of chlorine and ammonia in the following steps:

(1)	$Cl_2 + H_2O$	→	$HOCL + H^+ + Cl^-$	
(2)	$NH_3 + HOCl$	→	$NH_2Cl + H_2O$	monochloramine
	$NH_2Cl + HOCl$	→	$NHCl_2 + H_2O$	dichloramine
	$NHCl_2 + HOCl$	→	$NCl_3 + H_2O$	nitrogen trichloramine

The competing reactions in the second step are dependent on pH, the chlorine: ammonia nitrogen (Cl2:N) ratio, temperature and contact time.[5] Monochloramine is the preferred form due to its disinfectant properties and minimal taste and odor.

Chloramine residuals may be maintained for as many as 21 days[6] or significantly longer than chlorine residuals. Thus, chloramines are of considerable interest in regional water projects of the nature here with long distances between the points of initial disinfection and end-users. The number of re-injection points to maintain residual concentrations of disinfectant can be minimized. Chloramines form very few disinfection byproducts and are superior to chlorine in maintaining low levels of total trihalomethanes (TTHMs) and haloacetic acids (HHAs). Trihalomethane reductions of 40% to 80% are reported when chlorination was replaced with chlorimination. Haloacetic acids may not be as effectively controlled by chloramines.[7] Contact time for chloramines is significantly greater than with chlorine.

Disadvantages of chloramines include requirements to remove chloramines before use in kidney dialysis. This will require attention in the project area where diabetes is prevalent. Chloramine will bind to iron in the red blood cells during the dialysis process.[8] Treatment centers can remove chloramines ahead of the dialysis process. Although not considered as aggressive as chlorine, chloramine contributes to bladder and other cancer risks.

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[4] US Bureau of Reclamation, January 2000, *Red River Valley Water Needs Assessment, Phase II, Appraisal Of Alternatives to Meet Projected Shortages*, Dakotas Area Office, p 4-1.

[5] EPA, April 1999, *EPA Guidance Manual, Alternative Disinfectants and Oxidants*, p. 6-1, *et seq*.

[6] Bureau of Reclamation, April 30, 2001, *Value Engineering, Fort Peck Assiniboine and Sioux Water Supply System, Dry Prairie Rural Water System, Final Report*, p. 53

[7] AWWA RF, August 1999, *How Chloramines Improve Water Quality*, Research Application: Research in Use, p. 2

[8] Ibid.

Nitrification is a risk, particularly in warmer waters. Ammonia from chloramine is converted to nitrite and then to nitrate. This can deplete the chloramine residual and increase bacterial production. Chloramines can also lead to accelerated corrosion and degradation of gaskets and some metals in distribution systems. Temperature, pH, ammonia concentration, organic compounds, detention time and the time that water may stand in dead-end lines or other parts on the distribution system are among the factors that require attention with use of chloramines.[9]

WTP Alternative Capacities

Additional alternatives were developed for supply of water for communities that would be at the distal end of the rural water system, most notably, the community of Cibecue. Alternative treatment supplies to the community of Cibecue include a conventional filtration or microfiltration WTP with Cibecue Creek/Salt Creek Reservoir (Chapter 5) serving as the source, and a reverse osmosis groundwater treatment facility with the Redwall formation serving as the source.

Costs estimates for treatment facilities were made by comparison of costs of other similar treatment plants in the Inter-Mountain region, use of the U.S. Bureau of Reclamation's Water Treatment Estimation Routine (WATER),[10] and recent quotes from manufacturer's and suppliers of water treatment equipment. Cost estimates were developed for the following alternatives:

1. 12.3 MGD Conventional Filtration WTP North Fork White River
2. 12.3 MGD Microfiltration WTP North Fork White River
3. 10.2 MGD Conventional Filtration WTP North Fork White River
4. 10.2 MGD Microfiltration WTP North Fork White River
5. 2.1 MGD Conventional Filtration WTP Cibecue Creek/Salt Wash Reservoir
6. 2.1 MGD Microfiltration WTP Cibecue Creek/Salt Wash Reservoir
7. 2.1 MGD Reverse Osmosis WTP Redwall Formation

Interpolating between the developed cost estimates allowed for a comparison of costs to supply the Demand Scenarios described below.

Distribution System

The distribution system for the domestic water system as analyzed under this extension report was to specifically serve the area outside of Whiteriver. The WTP would be connected to the Diamond Creek Tanks to serve the community of White River. The principal areas served

[9] Ibid.
[10] Bureau of Reclamation, August 1999, *Water Treatment Estimation Routine (WATER) User Manual*, Water Desalination Research and Development Program Report number 43, Lower Colorado Regional Office, Boulder City, Nevada.

outside of Whiteriver by the distribution system are Fort Apache, Canyon Day, Cedar Creek, Carrizo and Cibecue. A pipeline connection already exists between Whiteriver and Cedar Creek. However, cost estimates were based on the construction of a new pipeline between the communities. A small diameter pipeline may prove adequate in final design in the event that the existing 6-inch pipeline is determined to be suitable for a portion of the supply.

Five different distribution system alternatives were modeled and cost estimates prepared. The five alternatives were based on a progressive model to determine the incremental costs to serve each of the five areas outside of Whiteriver. The alternatives analyzed and their corresponding maximum day demands were as follows:

Demand Scenario	Demand, gpm
#1 – Distribution to White River	None, tie into existing system
#2 – Distribution to Fort Apache	1,820
#3 – Distribution to Canyon Day	2,956 (1,136 + 1,820)
#4 – Distribution to Cedar Creek	3,209 (253+1,136+1,820)
#5 – Distribution to Carrizo	3,346 (137+253+1,136+1,820)
#6 – Distribution to Cibecue	4,919 (1,573+137+253+1,136+1,820)[11]

Each of the alternatives was analyzed to develop the pipe sizes, pressure ratings, pumping requirements and storage requirements for each scenario. Pipelines were generally considered to consist of AWWA rated PVC pipe. Pump stations, tanks, and pressure-reducing stations are discussed in the next section. Standard appurtenances such as isolation valves, air release/vacuum valves, blowoff hydrants, and other items necessary for construction were included in the costs estimates. The cost estimates for the pipeline only address connecting the major communities as discussed above. Branch lines may eventually be developed between the communities to serve new housing project along the main transmission pipeline in the rural areas. The cost estimates for the pipeline do not address any upgrade or improvement of the main transmission system within the existing public water system necessary to accommodate increased demands and flow rates for the future. Because all land crossed by the pipeline between Whiteriver and Cibecue is held in trust by the United States for the White Mountain Apache Tribe, virtually no lands in private or individual ownership would be crossed, and it was assumed that no cost of easements would be incurred.

Modeling results for the five alternatives are discussed below.

Pump Stations, Pressure Reducing Facilities and Tanks

The terrain between Whiteriver and Cibecue is undulating and would require pump stations to overcome static head and friction losses in the pipeline. Most of the pumping

[11] This is the only demand scenario currently under consideration and is consistent with alternative 5.

requirements between White River and Cibecue would be to cross Cibecue Ridge between Carrizo and Cibecue and to overcome friction losses in the pipeline. Pump stations were estimated based on a package pump station construction that would be delivered to the site and installed on the pipeline. Booster stations would pump between tanks, with the first reservoir in the series providing suction pressure to the booster station, and the second reservoir serving as the discharge point.

Alternatives 1 thru 3 serving the communities of Fort Apache, Canyon Day, and Cedar Creek require no booster station, only the High Service Pump Station at the WTP. Alternative No. 4 with the distribution system serving Carrizo only requires one booster station. Alternative No. 5 serving Cibecue requires four booster stations, with one station at approximately the same location as the Carrizo booster station, and three additional booster stations to cross Cibecue Ridge.

Pressure reducing stations were used to limit the pressure in distribution to a maximum of 200 pounds per square inch (psi), the maximum allowable working pressure for the majority of AWWA C900 rate PVC pipe. Alternatively, pressure reducing stations could be used to limit pressure to a higher pressure, but this would require higher class pipe such as steel or ductile iron. Pressure reducing facilities on the transmission pipeline were modeled in Alternative 1, 2 and 5. A pressure reducing facility was used on Alternatives 1 and 2 to keep the pressure rating of the transmission pipeline below 165 psi. Pressure reducing facilities for Alternative 5 were modeled to limit the pressure in the transmission pipeline on the downstream side of Cibecue Ridge to no greater than 200 psi. The crest elevation of Cibecue Ridge is approximately 6,200 feet msl while the community of Cibecue is at elevation 5,100 feet msl. Intermediate demands at Fort Apache, Canyon Day, Cedar Creek, and Carrizo for Alternatives 3, 4, and 5 were assumed to have pressure reducing facilities on the central meter taps for those demand points.

Water storage tanks would be provided between pump stations for the distribution system. Tanks were typically located at the highest point between pump stations to provide water in distribution both upstream and downstream under non-pumping conditions. The purpose of the storage tanks is to provide a water source at the suction side of the next pump station in distribution, as well as provide an uninterrupted supply of water during peak use periods, power failure, or loss of a system component. The total capacity of water storage tanks was determined to be the volume of flow between the Greater Whiteriver and Cedar Creek public water supply systems (1,753 gpm) discharging over a 24-hour day. This volume equated to 2,524,000 gallons. It was assumed that the storage tanks would be equally sized between the total number of tanks required between the WTP and Cibecue, including storage at the WTP and Cibecue itself. Alternative No. 5 indicates that four storage tanks along the distribution system between the WTP and Cibecue are necessary along with clearwell storage at the WTP and elevated storage at Cibecue. Six equally sized tanks would equate to approximately 420,000 gallons of storage per reservoir.

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Alternative No. 5 – Distribution to Cibecue

Hydraulic analysis for Alternative No. 5 indicates that Cibecue Ridge is the main control feature of this Alternative. Reservoir and pump station locations between Carrizo and the WTP are essentially as detailed under Alternative No. 4. However, to cross Cibecue Ridge, three additional pump stations, two additional tanks, and three transmission pipeline pressure reducing facilities are necessary to cross the topographic high. Reservoir No. 1 location and size between Canyon Day and Amos Wash remain the same. Similarly, the location of Pump Station No. 1 just northeast of Cedar Creek is relatively the same, only moving slightly farther downstream. Pump Station No. 1 would be sized at the maximum day demand of Carrizo and Cibecue (1,710 gpm) at a total dynamic head of 300 feet. Reservoir No. 2 between Carrizo and Cibecue would still be located on the ridge between the communities but would be sized at 420,000 gallons.

Downstream of Reservoir No. 2, progressive pump stations would be installed to provide the required head to cross Cibecue Ridge while maintaining the maximum pressure in the pipeline of 200 psi. This 200 psi limitation allows for the use of PVC throughout the system. Alternatively, high pressure pipe such as steel or ductile iron could be used in conjunction with high pressure-rated pump stations to limit the number of booster stations. However, preliminary analyses indicate that lower costs would be associated with additional pump stations and PVC pipe. The progressive pump stations (Nos. 3 thru 5) would each be sized at the maximum day demand of Cibecue at 1,573 gpm at approximately 460 feet of TDH. Tanks are provided between pump stations to provide suction pressure and a discharge point for the pump stations. Matched pump stations utilizing variable frequency drives (VFDs) could be used in place of the tanks, however, it was assumed that a less complex operational scenario would be more desirable. Pumping to the tanks does not require that exact flow matching as with sequential booster stations and their associated operational difficulties and would minimize transient pressure potentials. Reservoir Nos. 3 and 4 between Pump Station Nos. 2 and 3 and 3 and 4 would be sized at 30,000 gallons to provide only for minimum cycle times on the pumps. These 30,000 gallons ground level storage tanks are not too costly and would provide for sufficient pump operation capability. Reservoir No. 5 would be sized at 420,000 gallons to provide the necessary storage for supply to Cibecue.

Downstream of Reservoir No. 5, three pressure-reducing valve stations would be installed to maintain the pressure in the transmission pipeline below 200 psi to allow for the use of Class 200 or less C900 PVC pipe. Alternatively, high pressure pipe such as steel or ductile iron could be used without pressure reducing stations. However, preliminary analyses indicate that lower costs would be associated with pressure reducing stations and PVC pipe.

An elevated storage reservoir would be constructed in Cibecue to provide the minimum pressure requirements for distribution throughout the community. This elevated reservoir would be sized at 420,000 gallons with a minimum head height of 6,105 feet msl.