EXECUTIVE SUMMARY

PURPOSE OF THE STUDY

This report presents the results of a Wastewater Facilities Plan authorized by agreement between the City of Baker City, Oregon, and Anderson·Perry & Associates, Inc., dated April 6, 1998. The Plan was subsequently placed on hold in 2000 and re-activated in 2004 for final completion. The purpose of this planning document is to provide the City with planning guidelines to meet the needs for the wastewater facility over the 20-year planning period. The Wastewater Facility Plan addresses the wastewater collection system, the wastewater treatment plant, and outfall system. Planning guidelines consider existing conditions, current regulations, and projected design criteria.

PROJECT SCOPE

The following describes the project scope:

Study Area Evaluation. A study area evaluation will be performed in order to develop design criteria for both the water and wastewater plans. The evaluation shall include defining service area and boundaries, City-wide and growth area population projections, present and future flows and loadings, regulatory standards, and service goals. Growth area design criteria will include three estimates: low, medium, and high projections. The study area evaluation will also include an overview environmental description.

Collection System.

1. The existing collection system will be evaluated and deficiencies sited using the adopted design criteria and results of an I/I evaluation. The I/I evaluation will include the City's TV inspection work and selected basin area flow monitoring performed by the Engineer. The majority of the collection system evaluation work will be on the north, east, and west perimeter trunk mains, the lift station, and the southeast sewer mains where the majority of infiltration and inflow occurs. The flow monitoring would occur in areas the City has identified as having I/I problems based on TV inspection work. Additionally, evaluation of needs in growth areas and the impact on existing trunk mains will be performed including the trunk main to the treatment plant.

2. Based upon the evaluation, collection system improvement alternatives for sewer main replacement, sewer main additions, trunk sewer mains, and lift station replacement and additions will be developed. The alternatives will include description of alternatives, cost estimates, and pros and cons of alternatives.

Treatment Plant and Outfall.

1. The treatment plant and outfall will be evaluated and deficiencies cited using the City-wide design criteria. The evaluation will include National Pollutant Discharge Elimination System (NPDES) Permit review and coordination with DEQ regarding the development of future treatment, discharge, and monitoring design criteria. Performance evaluation of major unit processes including headworks, treatment, storage, discharge, and monitoring system will be conducted.

2. Based upon the evaluation, unit process improvement alternatives for meeting current and future needs will be developed including description of alternatives, cost estimates, and pros and cons of alternatives.

Recommendation. Based upon review with the City, recommended system improvements will be identified for collection, treatment, and discharge. Included with these recommendations will be a prioritization of needs, cost estimates, and an initial environmental overview.

Aerial Mapping. The Engineer will assist the City in selecting an aerial mapping firm to perform the mapping work, provide assistance in developing the overall mapping scope, perform aerial mapping ground control as determined by the mapping firm, and assist the City in selecting computer and drafting equipment for using the digital mapping information.

WASTEWATER SYSTEM HISTORY

The existing collection system gravity feeds to the wastewater treatment plant except for a small lift station at H Street and 8th Drive. The collection system contains pipe sizes ranging from 6-inch to 36-inch in diameter. Most of the collection system piping within the city center is terra cotta (clay) pipe and is estimated to be approximately 95 years old. The majority of collection system piping outside the city center is concrete. Other pipe types are transite and PVC.

During the sewage collection and treatment facility improvements in 1963, many of the trunk sewer lines were upgraded and/or constructed. These trunk sewer lines included the main 36-inch trunk sewer to the wastewater treatment plant, the interceptor along Hughes Lane and 17th Street, and other miscellaneous interceptor lines. Since that time, most of the collection system work has been additions in previously unserviced areas, replacement of failing lines, and maintenance.

The existing wastewater treatment and disposal facility was constructed in 1963, replacing a mechanical plant that was located just south of Hughes Lane. The 1963 project included a lift station, a 70-acre primary facultative lagoon, three 9-acre facultative polishing lagoons, a chlorine control building, and a 24-inch concrete pipe outfall to the Powder River. The outfall pipe also served as the chlorine contact chamber. In 1994, a new chlorine contact chamber and dechlorination facility was constructed in order to meet the City's NPDES Permit requirements. This work included a concrete serpentine contact chamber just east of the final polishing lagoon, a dechlorination building and equipment, emergency power, a new water supply well, and other related equipment needs.

More recently, in 1999, emergency improvements were performed at the lift station. The aging controls and switch gear that operate the duplex pumping system was failing and immediate improvements were necessary. A new level control system was installed and minor improvements to the switch gear were performed. During the same period, a new emergency generator with a manual transfer switch was installed in order to provide reliable backup power.

Also, during the development of this study, hybrid aeration units were installed within the facultative lagoons. The aeration units were installed to remove sludge by biological treatment. The process takes several years, but has been found to be very effective. The aeration units also maintain a slow mixing of the ponds that enhances treatment of the wastewater within the lagoons.

POPULATION PROJECTIONS

One of the main guidelines for developing a planning study is to project population growth for the planning period. An increase in population translates directly to additional demand on infrastructure. Normally, a single population projection is developed for the planning period utilizing historical trends. For most communities, long-term trends remain fairly consistent. Therefore, evaluating a community's needs based upon a single projection can be appropriate. On the other hand, communities can experience a surge in growth not anticipated by past trends which can tax the infrastructure beyond the anticipated needs, or a major commercial/industrial user may desire to locate within the community, having the same effect.

Based upon these possibilities, the City has chosen to develop a low, medium, and high population projection in order to evaluate system needs. In order to develop a low, medium, and high population projection, both population curves and expected percent build-out are viewed. The population curves project the rate of growth. The expected percent build-out gauges the ability to meet the growth rate and provides an approximate development date. With this data, a probability of occurrence can be concluded for the 20-year design period.

Figure E-1 shows the various population curves generated for developing the low, medium, and high population projections. The figure shows the historical trend, 1975 and 1979 CH2M Hill projections, minimum, low average, high average, and maximum growth trends. Figure E-1 was generated using existing population data from 1970 to 1998. The 1998 population was 10,160. From 1990 to 1998 there was a steady trend of growth from 9,140 to 10,160. After the 2000 census, the population estimates were reduced significantly. The 2000 census showed a population estimate of 9,860 and the 2003 population estimate is 9,840. Whether these current estimates are accurate is questionable. For the purpose of this study, the historical trend from 1970 though 1998 will be used. This will provide slightly conservative design criteria if, in fact, the 2000 census data are accurate.

Figure E-1 also shows estimates of percent build-out of available lands within the community. These estimates of build-out were developed utilizing the aerial mapping prepared in conjunction with the study. The lands within the UGB were separated into three zones as shown on Figure E-2. These areas are designated as the north, west, and east service areas. By overlaying the City's current zoning map, the total residential, commercial, and industrial lands available within each service area were estimated. These areas were delineated based on the evaluation of undeveloped property or property with few structural or significant improvements. Figure E-3 delineates the estimated buildable lands, and Table E-1 provides the acreage calculated for each service area.

The percent build-out population was estimated using the corresponding build-out in residential zones. Commercial and industrial build-out was developed for later use in estimating wastewater flows.

Utilizing Figure E-1 and Table E-1, the three population projections for the design period can be made. The design life for a wastewater collection system is typically greater than 40 years, while planning efforts for treatment facilities usually address a 20-year period. Because of this difference in planning goals, projections for 6, 20, and 40 percent build-out are used to develop the design criteria for the wastewater treatment plant. The respective population projections are 10,850, 12,338, and 14,516. The 20, 40, and 70 percent build-out is used to estimate long-term effects on the collection system. The respective population projections are 12,338, 14,516, and 17,783.

DESIGN FLOWS AND LOADS

Two sets of design criteria are needed to evaluate the performance of the wastewater facilities. One design criteria, which is flow based, will be used for the collection system. The other design criteria, which is both flow and loading based, will be used to evaluate the wastewater treatment plant.

Collection System Design Criteria. The design criteria to be used for evaluating the performance of the existing collection system is composed of the existing wastewater flows plus the anticipated future build-out flows.

Tables E-2 through E-5 show the projected collection system flows based on the 6, 20, 40, and 70 percent build-out of available lands, respectively.

Wastewater Treatment Plant Design Criteria. The design criteria for the wastewater plant include both influent flow and loading criteria as well as specific regulatory requirements. The flow and loading criteria are based upon the historical influent flows and loadings and the future low, medium, and high population projections. Using the 1993 through 1998 Discharge Monitoring Report (DMR) data, the existing average day flow is 1.600 million gallons per day (MGD) (157 gpcd). The maximum monthly average day flow is 2.229 MGD (219 gpcd). From the flow monitoring data, the peak hour is 3.120

MGD. The corresponding biochemical oxygen demand (BOD) and total suspended solids (TSS) mass loading are shown on Table E-6. Also shown on Table E-6 is the 2002 and 2003 summary DMR data for Average Day and Maximum Month Average Day. The 2002 and 2003 DMR values are below the 1998 DMR data. It is believed this occurred because of climatological conditions. For the purpose of this study, the 1998 data will be used for the design criteria. Using the 1998 influent data and incorporating the population projections, Table E-6 shows the projected flows and loadings for evaluating the treatment plant unit processes.

Specific regulatory requirements are shown in Table E-7. This table shows the current NPDES Permit limits. The wastewater treatment plant must be able to achieve the level of treatment shown. Beyond these current treatment requirements, the City will also be facing additional requirements regarding discharge to the Powder River.

Once TMDL's have been established for the Powder River, the City will have more direction from DEQ as to what the requirements will be. Most likely, it will include limited discharge to the Powder River during low river flows and during some periods of summertime flows. The 45:1 dilution ratio, or some derivative thereof, for discharge to the Powder River will most likely come into effect. Other restrictions that may apply to discharge to the Powder River are ammonia limits and river-related temperature limits.

COLLECTION SYSTEM

General. The City of Baker City's wastewater collection system consists of approximately 61 miles (322,400 feet) of concrete, transite, PVC, and terra cotta (clay) piping, with sizes ranging from 6 inches to 36 inches in diameter. It currently serves the majority of the developed area within the City's UGB. Wastewater within the system flows primarily by gravity. It is collected from service laterals and transported by branch, main, and one of four trunk lines, to empty into an interceptor line. This interceptor transports these flows to the City's wastewater treatment facility located north of the City. The City's collections system has one wastewater lift station located near the intersection of H Street and 8th Drive. The collection system also crosses beneath the Powder River at Hughes Lane, at F Street, and at Nevada Avenue. The crossings at F Street and Nevada Avenue employ inverted siphons to transport wastewater beneath the river.

The oldest portions of the wastewater collection system are located in the "Old Town" area near the center of the City, with some pipe installations recorded as early as 1905. Other portions of the collection system have pipe ages that range from 75 years to the present. As is typical with development, the older system piping is generally located toward the center of the City, while the newer pipe installations are located toward the outer portions, where new growth and development typically occur.

The majority of the City's collection system piping lies within public rights-of-way such as streets and alleys, and easements have been obtained where the collection system piping is located on private property. House service laterals lie partially on private property and connect to the City's collection system through manufactured tees and wyes installed either at the time of original construction or through field-constructed taps that have been added over the years.

The City's existing wastewater collection system is divided into eight basins. A basin refers to an area of collection system piping that is interconnected and drains to a common point within the system. The basin designations are taken from the City's wastewater collection system base map, where they are used to locate and identify manholes and pipe sections. In this Plan, they are used for the preparation of flow monitoring, field work maintenance, and presentation purposes. Figure E-4 shows the general basin boundaries as they lie within the City's UGB. Table E-8 summarizes the approximate number of manholes, pipe material type, size, and lineal feet of pipe for each basin.

A nationally-used publication that lists the policies regarding municipal wastewater systems recommends eight inches as the minimum diameter for gravity wastewater collection pipes. Smaller piping is acceptable for smaller areas. As Table E-8 indicates, approximately 91 percent of the City's total collection system piping is eight inches or larger in size.

Infiltration and Inflow. During historical periods of high groundwater, the City has consistently received increased flows at their treatment plant as a result of infiltration and inflow (I/I). An I/I evaluation was performed in order to estimate the amount of I/I, locate the areas where I/I occurs, and determine methods to reduce I/I. Each basin shown on Figure E-4 was evaluated for I/I. Table E-9 shows the amount of I/I within each basin and the total I/I contribution within the collection system. The total I/I contribution during the high groundwater period was estimated to be 1.395 MGD. Using the influent flow monitoring at the wastewater treatment plant gives an I/I value of 1.149 MGD as seen on Table E-10. Therefore, the I/I testing within the collection system corresponded fairly well with estimated I/I obtained at the wastewater plant.

During the same period of I/I evaluation, City staff performed extensive TV inspection of lines to locate defects that would tend to create I/I. Table E-11 shows the results of the TV inspection work.

I/I Removal Strategies. The overall strategy in effectively removing I/I from a collection system is to identify the areas within the system where I/I contributions are concentrated, then rehabilitate those areas having the largest I/I sources contributing to the smallest sections of piping. Given the nature of the I/I identified and the possibility of the presence of unidentified sources in each of the rehabilitation areas, a two-step approach is proposed. The steps are defined as Phase 1 and Phase 2. Phase 1 refers to replacement of the defects associated with the identified I/I sources. Phase 2 refers to an overall treatment of the sections of pipe where those sources were found and may consist of CIPP or insertion techniques. If Phase 1 rehabilitation measures do not reduce the quantity of I/I sufficiently, Phase 2 rehabilitation measures may be employed. The characteristics of most rehabilitation areas suggest that Phase 1 rehabilitation measures should be sufficient. However, Area DIII has a large ratio of I/I contributions to identified pipe defects, which suggests that Phase 2 measures may be required.

The typical pattern of the rehabilitation measures should consist of several steps. The first of these is to measure the I/I of the individual areas prior to construction. The sections of pipe and manholes should subsequently be cleaned and televison inspected. The television inspection should locate previously identified sources as well as any new I/I sources. The defects should then be repaired by replacing service connections, sections of the pipe, manhole grouting, etc. Where replacement of service laterals are called for, the construction crews should begin excavation at the property line. When the service is uncovered at that point, it should be opened to determine if the I/I is located on private property or within City right-of way. If the I/I is determined to be in the City right-of-way, replacement of the service lateral continues. However, if it is determined that the I/I is located on private property, the City should investigate the possible source and notify the property owner of his/her responsibility to correct the problem. As mentioned earlier, an ordinance should be in place prior to starting the rehabilitation projects and might consist of a monthly surcharge to the owner until the defect is corrected.

As repairs in an area are completed, the line should be televison inspected to determine if the existing I/I sources were corrected, and the flows should be measured again to evaluate the degree of I/I removal. It should be expected that a 100 percent removal is not possible. The removal of those visible sources may reduce flows to a point where previously unidentified sources then become visible. The selection of an acceptable degree of removal will aid in judging the effectiveness of the rehabilitation measures. It is proposed that a typical removal of at least 60 percent of I/I contributions for any area is acceptable. Higher removal percentages may be utilized where techniques and conditions allow. If, after Phase 1 rehabilitation measures are performed and the percentage removal is not acceptable, Phase 2 measures may be warranted.

I/I Removal Costs. The majority of the City's I/I problems appears to be located in sections of the interceptor line located in Basin A and in areas within the upper reaches of Basins C, D, and E. Several minor sources were discovered in Basins B, F, and G, but did not appear to contribute significant amounts of I/I to the collection system as a whole. The results of the I/I evaluation were used to establish selected rehabilitation areas within the collection system. Table E-11, Figure E-5, and the large fold-out map located in the back of this Plan identify these areas by pipe section and manhole. The type of defect associated with I/I contributions is also identified and used to estimate the costs associated with rehabilitation. The feasibility of I/I removal from the collection system is controlled primarily by the cost of rehabilitation, versus the cost of treatment and disposal of the wastewater. The total project cost of treatment and disposal improvements is estimated at approximately $5.318 million, based on a maximum month average daily flow of 2.707 MGD. This breaks down to an estimated cost of $1.96/gallon/day in treatment capacity. Determining the costs for each rehabilitation area will also help prioritize removal efforts. The following are descriptions of the proposed rehabilitation areas and their associated costs:

Area A. Area A consists of the collection system piping between the wastewater treatment plant and Manhole A9. It has an estimated total I/I contribution of 0.445 MGD with unidentified I/I sources. It is located in the general area of Chico Street/Imnaha Road north of Pocahontas Road. The CIPP rehabilitation method was selected based on the assumption that the I/I contributions were from leaking pipe joints throughout the area and has a total estimated cost for repair of $1,210,600. By assuming a 75 percent removal of the 0.455 MGD total I/I contribution, the expected quantity of I/I removed is approximately 0.334 MGD. This converts to an estimated removal cost of $3.62/gallon/day. This value is considerably higher than the treatment cost of $1.96/gallon/day. By this analysis the rehabilitation of this area would not prove to be cost effective.

However, since no identifiable sources of I/I were discovered during the study period of this plan, a more detailed investigation of this area may identify individual I/I contributions or a reduction in the total length of pipe rehabilitation needed, thus reducing construction costs. If any of these situations developed, rehabilitation of the area may become feasible.

Area DI. Area DI consists of the collection system upstream of Manhole D155 in Basin D. It has an estimated total I/I contribution of 0.066 MGD with 28 visible I/I sources identified. It is located in the general area south of Grace Street and east of Elm Street. The total estimated cost for repair of the 28 identified defects is $37,200. Assuming a 60 percent removal of the 0.066 MGD total I/I contribution through Phase 1 rehabilitation measures, the expected quantity of I/I removed is approximately 0.040 MGD. This converts to an estimated removal cost of $ 0.93/gallon/day. When compared to the treatment cost of $1.96/gallon/day, rehabilitation of this area appears to be very cost effective.

Area DII. Area DII consists of the collection system between Manholes D117 and D155 in Basin D. It has an estimated total I/I contribution of 0.066 MGD with 49 visible I/I sources identified. It is located in the general area between the Powder River and Elm Street, south of Bridge Street. The total estimated cost for repair of the 49 identified defects is $58,000. Assuming a 60 percent removal of the 0.066 MGD total I/I contribution through Phase 1 rehabilitation measures, the expected quantity of I/I removed is approximately 0.040 MGD. This converts to an estimated removal cost of $1.45/gallon/day. When compared to the treatment cost of $1.96/gallon/day, rehabilitation of this area appears to be cost effective.

Area DIII. Area DIII consists of the collection system between Manholes D46 and D84, primarily in the pipe sections between Manholes D58 to D60 and D80 to D205. It has an estimated total I/I contribution of 0.184 MGD with 14 visible I/I sources identified. It is located in the general area between Campbell Street and Auburn Avenue, and between East and Ash Streets. The total estimated cost for repair of the 14 identified defects is $100,600. Assuming the same 60 percent removal of the 0.184 MGD total I/I contribution through Phase 1 rehabilitation measures, the quantity of I/I removed is approximately 0.110 MGD, converting to an estimated removal cost of $ 0.91/gallon/day. However as stated earlier, given the large quantity of I/I flow and the small number of identified defects for this area, the assumption of a 60 percent removal value appears questionable. Prior to performing I/I repairs in this area, a more detailed evaluation of the I/I contributions should be conducted in an attempt to identify additional sources. Should any additional I/I sources be located, the estimated cost will need to be adjusted to include those sources.

Should no additional sources be located and if the I/I reduction from Phase 1 repairs is less than acceptable, Phase 2 may then be implemented. Phase 2 will require the City to contract outside services in order to perform CIPP liner rehabilitation measures listed at an estimated cost of $238,900. When the costs from Phase 1 and 2 are added together and if the 60 percent removal of I/I is achieved, the removal cost becomes $2.95/gallon/day. When compared to the treatment cost of $1.96/gallon/day, rehabilitation of this area appears to be more costly than treatment plant improvement costs.

Area EI. Area EI is located in the collection system upstream of Manholes E78 and E79. It has an estimated total I/I contribution of 0.061 MGD with 23 visible I/I sources identified. It is located in the general areas between Campbell and Broadway Streets, between Oak and Ash Streets; and Birch Street, between Washington Avenue and Auburn Avenue. The estimated cost for repair of the 23 identified defects is $95,200. Assuming a 60 percent removal of the 0.061 MGD total I/I contribution through Phase 1 rehabilitation measures, the expected quantity of I/I removed is approximately 0.037 MGD. This converts to an estimated removal cost of $2.57/gallon/day. Comparing this to the treatment cost of $1.96/gallon/day, rehabilitation of this area does not appear to be cost effective.

Area EII. Area EII consists of the collection system from Manholes A99 to E78, and E79, primarily upstream of Manhole E50. It has an estimated total I/I contribution of 0.034 MGD with 25 visible I/I sources identified. It is located in the general area east of Birch Street, between Campbell Street and Washington Avenue. The 25 identified defects have a total estimated cost for repair of $109,300. With a total estimated I/I contribution of 0.034 MGD, a 60 percent removal of I/I through Phase 1 rehabilitation measures may remove approximately 0.020 MGD of I/I contributions in the area. This converts to an estimated removal cost of $5.47/gallon/day. This value is well above the treatment cost of $1.96/gallon/day, making rehabilitation of this area appear unfeasible. If the City should decide not to include rehabilitation of the entire area as part of their I/I removal program, some consideration should be given to repairing the manholes only. Repair of I/I sources in manholes have a higher percentage of removal with minimal disruption to the surrounding area and can be performed at a minimal cost.

Area 'MH'. Area 'MH' refers to various manholes identified as having I/I but not associated with other rehabilitation areas. Rehabilitation of manholes tends to have a higher degree of I/I removal than that of most pipe methods, since the defects are more easily accessed and inspected. These currently consist of Manholes A7, A10, A18, A55, B66, G63, G65.1, G66, and G68. Area 'MH' has an estimated total I/I contribution of 0.173 MGD with 10 visible I/I sources identified. Manholes are located in Basins A, B and G. The total estimated cost for repair of the 10 identified defects is $21,800. Assuming an 80 percent removal of I/I through rehabilitation measures, the expected quantity of I/I removed is approximately 0.138 MGD. This converts to an estimated removal cost of $0.16/gallon/day. When compared to the treatment cost of $1.96/gallon/day, rehabilitation of the manholes described in area 'MH' appears to be extremely cost effective.

By comparing the cost/gallon/day associated with each rehabilitation area, the areas can be ranked in order of their increasing cost/gallon/day. Ranking the associated areas this way, it can be seen that Area 'MH' has the lowest cost/gallon/day, at $0.16. It is followed by Areas DI and DII, with total cost/gallon/day values of $0.93 and $1.45, respectively. Area EI follows with a value of $2.57/gallon/day. Prioritizing rehabilitation this way allows the City to treat the areas with the greatest benefit to cost ratio first. The Areas of DIII and A will require further evaluation to determine the feasibility of their repairs. Areas EI and EII appear to be the only areas that were not cost effective to rehabilitate at this time.

While other small I/I sources undoubtedly exist, determining their locations and quantities as a part of this Plan was considered not to be cost effective for either the evaluation or rehabilitation of those sources. These sites will likely be uncovered in time through regular maintenance and inspection of the collection system and can be addressed individually at that time.

Capacity Evaluation. The capacity of a collection system is controlled by several characteristics. The pipe diameter, minimum pipe slope, and internal roughness determine the capacity of any section of collection piping. Within the system these pipe characteristics generally change at manholes. The large fold-out map of the City's wastewater collection system located inside the back cover of this Plan identifies the locations of the manholes used.

Table E-12 shows the estimates of the current collection system capacity based on 2/3 full and full pipe conditions and provides a summary of the peak flows determined in the 20, 40, and 70 percent build-out projections. Pipe capacities were estimated using Manning's Equation for open channel flow, assuming a pipe roughness coefficient of 0.015 and minimum pipe slopes. Minimum pipe slopes refer to the flattest slope found within the described pipe section and were obtained from collection system as-built plans and survey information supplied by City personnel. Where as-built and survey information were absent, the DEQ's minimum specified pipe slopes were utilized to estimate capacity. These DEQ pipe slopes are intended to provide a minimum flow velocity of 2 feet per second when flowing at full pipe conditions. Evaluating collection systems at a 2/3 full condition helps to ensure sufficient capacity remains in the system to account for temporary blockages and sediment deposits.

Comparing the projected peak flow values to the estimated 2/3 full pipe capacities shown in Table E-15, show the current collection system piping can handle the projected peak flows at all locations except one. The peak flow for the 70 percent build-out projection between Manholes F1 and B21 in Basin B is 0.204 MGD greater than the estimated 2/3 full pipe capacity. While this section still has the sufficient overall capacity to carry projected flows, it may become a future maintenance concern. While removal of the proposed I/I contributions from the collection system will reduce treatment and disposal costs, it will not reduce flows sufficiently to correct this capacity problem in Basin B.

Terra Cotta Pipe. As mentioned in earlier sections of this chapter, portions of the collection system piping are constructed of terra cotta pipe. The City has an estimated combined total of 39,000 feet of terra cotta pipe in Basins C and G, which constitutes approximately 12 percent of the total quantity of collection system piping. The City's terra cotta pipe is constructed in 4-foot sections, the joints sealed with lime-based grout. Over time, the grout deteriorates, eventually allowing roots and groundwater to penetrate the joints. Since sections of the terra cotta pipe in Basin C are believed to have been constructed as early as 1905, some degree of deterioration of the pipe joint grout can be expected. Leaking pipe joints are believed to be one of the major contributors to the widespread I/I found in the upper reaches of Basin C.

Considering that some of the terra cotta pipe in Basin C may be nearly 100 years old, sections of the pipe are expected to have structural defects and be in generally poor condition. Typical structural defects that may be present in older terra cotta pipes consist of partial collapse, structural cracking, holes, cracked or open joints, root intrusion, corrosion, etc. In areas where the structural integrity of the pipe is compromised, soil loads may either fully or partially collapse the pipe, leading to partial or complete blockage of wastewater flows. The potential for pipe collapse is increased in areas of high groundwater where high rates of groundwater infiltration can erode the pipe bedding material.

As the collection system continues to age, the structural integrity of the terra cotta pipe is expected to continue to deteriorate. As such, the City should consider the rehabilitation/replacement of the terra cotta pipe in Basins C and G. The following steps should be incorporated into a rehabilitation/replacement program:

> **Identify Structural Deficiencies.** At present, the actual condition of the terra cotta pipe is uncertain. The City should begin television inspection of the areas containing terra cotta pipe and catalogue deficiencies when found. Rehabilitation/replacement work should begin as soon as sufficient defects are found and not wait for completion of the television inspection of the entire basin.
>
> **Evaluate the Severity of the Defect.** When deficiencies are located, the City should evaluate them and assign a rating factor based on the potential for pipe failure. Rating factors may consist of the following:

Potential for Pipe Failure	Rating Factor
Collapse or imminent collapse	5
Collapse likely in foreseeable future	4
Collapse unlikely in near future, deterioration likely	3
Minimal collapse risk, potential further deterioration	2
Etc.	1

Prioritize Rehabilitation/Replacement Work. The rehabilitation/ replacement work should be prioritized based on the severity of the defect as determined from the evaluation. Sections with the highest rating factor should be addressed first.

Determine the Required Rehabilitation/Replacement Technology. Given the nature and degree of deterioration, the City should select the most viable method for rehabilitation or repair. Four feasible rehabilitation/replacement technologies were presented in the section on Collection System I/I: Excavation and Replacement, CIPP Lining, Insertion (Slip-Lining), and Fold and Form Lining.

For sections where the pipe has collapsed or where collapse is imminent, excavation and replacement may be the only acceptable option. Where the deficiencies are less severe, the most cost-effective method that repairs the problem should be employed.

The cost of rehabilitation/replacement of the terra cotta pipe in Basins C and G is difficult to determine since the quantity, severity, and location of deficiencies are unknown at this time. In an effort to provide the City an estimate of the costs, the rehabilitation/ replacement of a typical section using the four technologies was considered. The typical section between manholes in Basin C consists of approximately 330 feet of 8-inch pipe, nine feet deep, and has an average of ten service laterals. Tables E-13, E-14, E-15, and E-16 present the cost estimate associated with the four repair technologies.

The rehabilitation/replacement of the terra cotta pipe will most likely occur over an extended period of time (30 to 50 years) and should be considered as a long-term maintenance program item. Taking the 39,000 feet of terra cotta pipe and using the CIPP lining cost per foot of $137, approximately $178,000 of rehabilitation work per year for 30 years would be needed to repair all of the lines.

System Maintenance and Repair. The location and identification of I/I sources, structural deficiencies, and areas of decreased capacity will be a continual process. Therefore it is important that the City develop a system of inspection, maintenance, and repair. The City should initially clean and television inspect their collection system to define problem areas, develop a meaningful rating system to prioritize areas needing repairs or replacement, and correct the highest priority areas on an annual basis as funds permit.

Once a basin is eventually rehabilitated through pipe replacement or repairs, a regular collection system cleaning program should be maintained to ensure the collection system lines continue to operate adequately.

H Street Lift Station. The H Street Lift Station has been a continual maintenance problem for years and needs to be replaced with a new facility. A new submersible duplex pumping system with a new wet well is proposed. The estimated cost of the new lift station is $189,000.

WASTEWATER TREATMENT AND DISPOSAL

Background. The Baker City Wastewater Treatment Facilities consist of a headworks, a 69.7-acre primary facultative lagoon, three 9-acre facultative polishing lagoons, chlorination/ dechlorination facilities, and an outfall into the Powder River (see Figure E-6). The wastewater flows through these facilities in the order listed above and as outlined in Figure E-7. Figure E-7 also shows the hydraulic profile of the treatment process from the collection system to disposal into the river. These facilities were constructed in 1964 with the chlorine contact basin and dechlorination facilities being completed in 1994. Other modifications were made to the system at other times in order to improve the treatment processes, such as the aeration system installation for sludge removal.

The entire system can be categorized into four parts consisting of headworks (lift station), primary and secondary treatment, disinfection, and disposal. Each one of these parts will be evaluated separately as they pertain to the entire system. The standards used for the evaluation of the treatment system are defined as design criteria. The evaluation of the system is highly dependent on the design criteria as chosen by the City and outlined by the regulatory agencies. The design criteria indicate the amount of waste to be treated and the level of treatment these wastes must receive.

Headworks Deficiencies. Due to the age of the existing system and the limited design capacity, there are several noted deficiencies. These are outlined as follows:

- Septage that is dumped into the influent channel located upstream of the wet well is not screened. The septage typically contains rocks, bottles, cans, and other materials that damage the impellers of the lift station pumps. The large slug of septage included in the influent stream also gives false sample data if the sampling happens to occur during septage receiving.

- The headworks does not include grinding or screening facilities. Large objects plug the pumps from time to time.

- The existing pumps are nameplate rated at 2,500 gpm, but a recent pump test indicated they are only producing a maximum of 1,750 gpm when operated on high speed with a high wet well level.

- The existing pump capacities of 1,750 gpm and the wet well design are too small for the current peak flow of 2,167 gpm (3.120 MGD) being experienced by the plant, and the expected future peak flows of 2,300 gpm (3.313 MGD) to 3,000 gpm (4.340 MGD) will tax the system beyond its current capacity. The smaller wet well size and pump problems have caused the pumps to cycle on and off at a rate that is stressing them beyond their normal design capacity. Both pumps are required to operate at the current peak flows of 2,167 gpm, leaving no redundancy for periods of down time or maintenance. The DEQ requires systems be designed with a minimum of one redundant pump to allow the system to operate at peak capacity with any one pump out of service.

- There are no isolation or check valves on the discharge piping of the pumps for pump maintenance. The entire line from the pump to the flume has to be drained into the wet well with the remaining undrained water being deposited into the dry well in order to perform maintenance on the pumps.

- The existing flume is located downstream of the lift station pumps. Therefore, the record flow is not representative of the actual flow rate to the headworks. This affects the operator's ability to determine the peak hour flows, historical flow trends, and excessive inflow from I/I. It also limits the operator's ability to optimize the pump operation through set point adjustments based on flows into the wet well. The piping configuration to the Parshall flume also creates turbulent water that reduces the accuracy of the flowmeter.

- The proximity of the electrical and controls equipment to the wet well has caused accelerated corrosion in the equipment. The equipment is currently located over the wet well and is separated by an access hatch and concrete slab. This separation is apparently insufficient. Any future electrical and instrumentation equipment will tend to have components that are smaller and more sensitive to harsh environmental conditions and may fail at a quicker rate.

- The current standby power generator is controlled by a manual transfer switch. This requires an operator to respond to a power failure alarm and manually start the generator. If the operator does not respond in a timely manner, the influent flows will flood the incoming pipeline and cause solids to deposit in the pipeline.

- The engine driven pump is worn out and needs to be removed from the system. A standby power generator has been installed by the City to provide emergency power to the lift station.

- The concrete and masonry portions of the building are in excellent condition, but the doors, windows, and other metal components are in need of replacing. The corrosive environment has caused deterioration of these metal components.

- The ventilation system is rusting away. The ventilation system is important in maintaining a work space that is free of toxic and acidic gases. Hydrogen sulfide and methane are the main gaseous products from municipal sewage. Both are extremely hazardous when inhaled, but hydrogen sulfide also produces sulfuric acid in a moist environment. This acid will corrode any metal over a period of time. In addition, the dry well is defined as a confined space. With a properly operating ventilation system, the City may determine the space to be a Non-Permit Confined Space area. The City should review this item as part of their regular safety program.

- The current alarm system generates a common alarm for the entire headworks and sends it back to the chlorine building via underground wire where the dialer is activated. This provides adequate alarm, but problems can occur in the signal between the headworks and the chlorine building. In addition, the alarm signal does not indicate the type of failure existing at the headworks.

- The portable samplers currently used are not refrigerated. A 24-hour composite sample is required, and the sample needs to be kept cool to maintain its integrity. The samplers are currently packed with ice to keep them cool during periods of warm weather. Based on a review of DMRs sampling, the data appear to be very irregular. This is probably due to the sample point within the Parshall flume. The quality of samples is highly dependent on the homogeneousness of influent pumped from the wet well.

- The water supply well for the headworks has been deemed unacceptable by the Department of Human Services - Drinking Water Program (DWP) and will need to be abandoned. Apparently, the well was not constructed according to acceptable standards for potable water supply.

Headworks Improvement. The following describes the improvements proposed to alleviate the deficiencies at the headworks:

1. Construct a manual septage receiving facility including storage, manual bar screen, and piping. The estimated cost is $36,500.

2. Install an in-channel grinder upstream of the wet well that will grind all solids into particles small enough to be passed through the pumps into the ponds without plugging. Install a bypass system for equipment maintenance. The estimated improvement cost is approximately $144,300.

3. Replace the existing pumps with new VFD pumps that produce flows of 3,000 gpm each. The estimated cost of improvements is $151,000.

4. Perform improvements to the existing Parshall flume to improve performance. The estimated cost of improvements is $9,900.

5. Provide a new electrical control building. The estimated cost of improvements is $65,300.

6. Refurbish the existing lift station building at an estimated cost of $10,000.

7. Provide refurbished samplers in enclosures at the headworks and pond outfall at an estimated cost of $44,000.

The total estimated cost of improvements for the headworks is $461,000.

Wastewater Treatment Deficiencies. The lagoons have noted deficiencies related to the capacity of the system and operational inefficiencies. These deficiencies are outlined as follows:

- The current BOD mass load into the lagoon is 2,819 lbs/day based upon the maximum month average day. The recommended loading capacity is 2,900 lbs/day based upon 30 lbs/acre/day for the lagoon system. The existing loading rate to the facultative lagoon system is, therefore, within design tolerances, yet there have been times effluent BOD concentrations have approached design limits and exceeded limits in one instance. The apparent reason for this ineffective treatment is the short circuiting that takes place in the primary lagoon and the existing sludge depths in the primary lagoon that reduce the amount of treatment volume. Referring to Figure E-6, influent discharges into Pond A and short circuits directly to Pond B. A large portion of Pond A is ineffective treatment area due to the location of the inlet and outlet. The inlet is located at the southwest corner of the primary lagoon while the outlet is located in the northwest corner of the lagoon. Without proper mixing, the wastewater tends to flow directly from the inlet to the outlet without circulating through the entire lagoon for treatment. This short circuiting reduces the actual detention time of the flows being treated, and thus the treatment capacity of the lagoon. This short circuiting has also caused increased sludge deposits in the southeast corner of the lagoon. This is due to a lack of flow distribution through the entire lagoon.

- Sludge depths within Pond A are from approximately 3 feet at the inlet to an average of 1 foot within the pond. This sludge has been accumulating over the last 40 years and, along the west side of the pond, sludge depths are in the 2-foot range. This amount of sludge reduces the treatment volume of the pond. If sludge depths were reduced and short circuiting eliminated, treatment levels would improve greatly. Though improving the circulation of influent and reducing sludge in Pond A will greatly improve treatment, future BOD loadings could go beyond the limits of the existing treatment capacity. The 20-year design average day BOD loading is 2,127 to 2,845 lbs, which translates to 22 to 29 lbs per acre. The 20-year design maximum month average day BOD loading is 3,015 to 4,030 lbs, which translates to 31 to 42 lbs per acre. From this data, it appears that as influent loading approaches the high growth rate, the existing lagoon system will not be able to sufficiently treat the wastewater.

- Another permit requirement that will limit the capacity of the existing lagoon system is the Allowable Mass Loading to the Powder River. The current permit allows 750 lbs/day on a monthly average to be discharged to the Powder River. Based on a 45 mg/L treatment level, future mass loads will be 893 to 1,195 lbs/day for the maximum month average day flow and 639 to 855 lbs/day for the annual average day flow. This shows future mass loadings to the Powder River could exceed permit limits. It is doubtful the permit could be changed to increase mass loadings to the Powder River; therefore, either treatment levels would have to increase or discharge to another outfall would be required.

- The current lagoon detention time at maximum water surface elevations is about 98 days for annual average day flow and 70 days for maximum month average day flow. Recommended detention times for facultative lagoons is a minimum of 80 to 120 days. The permitted average design flow of 2.0 MGD produces a detention time of approximately 80 days for the existing lagoons with a depth of 5 feet. This data indicates the existing lagoons produce an adequate detention time for the current flows, but the detention time for the future flows will need to be increased by an increase in the storage volume of the facultative system.

- The lagoon outlet structures have fixed elevations at which water can be drawn. This limits the ability of the operator to optimize system performance by selecting the elevation at which the water can be drawn out of the lagoon.

- The existing lagoons freeze over in the wintertime causing the treatment process to turn anaerobic due to a lack of oxygen. The lack of oxygen also causes an odor problem during the spring thaw. This anaerobic condition reduces the treatment capacity of the lagoons. In addition, during certain times of the year when the algae growth blooms, problems are caused with the disinfection system, TSS, and pH. A large amount of algae in the effluent causes high TSS and also requires large amounts of chlorine in order to meet the disinfection requirements. This can cause the TSS and disinfection parameters to be out of compliance. The algae can also contribute to a higher pH in the effluent. The elevated pH can be caused when the algae has consumed the readily available carbon source (carbon dioxide in the water) needed for photosynthesis. It then turns to consuming other sources of carbon, of which carbonate is used. As the carbonate is consumed, the buffering capacity of the effluent is reduced and the pH increases. Therefore, the pH, ammonia, BOD, and TSS at the lagoon discharge are higher than allowable for discharge. This causes a need to store the effluent in the lagoons, which reduces the limited storage volume available. It should be noted that fluctuations in TSS and pH levels are typical with lagoon treatment. Most lagoon systems provide sufficient storage to allow for these fluctuations or provide a separate means of discharge.

Disposal System Deficiencies. The following describes deficiencies with the existing disposal system.

- Figures E-8 and E-9 show the allowable discharge based on a 45:1 dilution ratio with the Powder River flows, the existing treatment plant flows, and the excess effluent. The only month of the year that would approach full discharge into the river based on the 45:1 dilution ratio is May. The Powder River flows shown in the figure are taken from the average monthly flow in 1988, the lowest river flow in the past 20 years. Figure E-10 shows the future treatment plant flows and excess effluent with respect to each projected design flow. Based on this data, the disposal system for the wastewater treatment facilities would have to be modified to comply with current regulations. Another restriction to river discharge that must be considered is the total mass load allowed into the river. OAR 340-41 indicated that any discharges occurring after June 1996 will not be allowed an increase in mass load. This means the level of treatment must increase with an increase in flows, or the increased flows must be disposed of in another manner. An evaluation of the restrictions on discharge due to dilution ratio and the restrictions due to the mass load increase must occur concurrently. With the current treatment system, the mass load will allow only 2.0 MGD to be discharged into the river, but the current historical flows in the Powder River would allow much more than 2.0 MGD at times and much less than 2.0 MGD at other times considering the 45:1 dilution ratio (see Figure E-9).

- The existing permit has allowable pH limits for the effluent when discharging to the Powder River. The pH must be within 6.0 to 9.0 when discharging to the Powder River. This is not always achievable without chemical addition. Figure E-11 shows the upper pH limit is approached each year from March through May during spring turnover and high algae growth periods. The only real way to combat this problem is to provide chemical addition, additional storage, or some other type of discharge that is not pH limiting.

- The existing outfall line to the Powder River is a 24-inch concrete pipeline that was installed in 1964. The approximate hydraulic capacity of the pipeline is 1.65 to 4.56 MGD depending upon Powder River flows. The condition of the pipeline is not known and the pipeline should be TV inspected to ensure the pipe is still structurally sound.

Wastewater Treatment and Disposal Improvements. Three overall wastewater treatment and disposal options were evaluated. The options were:

- Minor facultative lagoon improvements with reuse to constructed wetlands and discharge to the Powder River. Estimated cost of improvements is $4.395 million.

- Aerated lagoon treatment with tertiary slow sand filtration and discharge to the Powder River. Estimated cost of improvements is $7.249 million.

- Facultative lagoon improvements with river discharge and irrigation reuse. Estimated cost of improvements is $7.997 million.

The constructed wetland option has the least capital cost and least long-term operation and maintenance cost. Therefore, it is the preferred option.

Discharge of effluent to a wetland would require the construction of a primary wetland for treatment, a gravity pipeline to Baldock Slough, and construction of a secondary wetland at Baldock Slough. Refer to Figure E-12. Discharge to the Powder River would continue at a 45:1 ratio with the remaining effluent discharging to the wetland. The primary wetland would be lined, while the secondary wetland at Baldock Slough would be unlined. The primary wetland would be sized for three days detention time, at an approximate 6-inch depth, for the annual average day flow. This criterion would require approximately 40 acres of lined wetland. The property owned by the City just west of the existing lagoon could be used for the wetland site.

The secondary wetland would be sized to maintain a maximum depth of 2 feet considering inflow, precipitation, evaporation, and seepage. The primary wetland would be sized using the high design criteria of 2.279 MGD and the secondary wetland would be staged in phases. Phase 1 would consider current flows, Phase 2 would consider the additional flows with the medium design criteria, and Phase 3 would consider the high design criteria. Considering this criteria, 120 acres of unlined wetland would be required for Phase 1 , 144 acres for Phase 2, and 174 acres for Phase 3. Ducks Unlimited would be responsible for constructing the secondary wetland.

The pipeline from the treatment plant to Baldock Slough would be sized for the high design criteria maximum monthly average day and gravity flow. It is assumed the pipeline would begin near the existing river discharge point and parallel the river proceeding north for most of the way (refer to Figure E-12). At the section line 28/21, the pipeline would veer east and then parallel the highway to the Highway 203 interchange, cross Chandler Lane, and terminate at an irrigation drainage structure that crosses underneath the highway. The pipe diameter size to maintain gravity flow would be 18-inch diameter at an average slope of about 0.0015 ft/ft for a 3.185 MGD flow rate.

This alternative would be similar to the City of La Grande's wetland project constructed to utilize treated wastewater effluent. This would require partnering between the City and Ducks Unlimited to develop the project on the privately owned land that is currently being restored into a wetland by Ducks Unlimited. Presently, Ducks Unlimited has secured an easement on the property to restore wetlands on the site. Approval would also need to be obtained from the DEQ to implement the wetland concept developed with the La Grande project.

Disinfection System Deficiencies. The disinfection system has had several recent upgrades due to the changes in the discharge permits. For this reason, most of the system is currently operationally sound, but there are some major concerns over the new regulations requiring a risk management plan and the safety issues concerning the storage and use of the one-ton chlorine cylinders. The historic chlorine usage under normal conditions is about 15 pounds of chlorine per million gallons. The historic sulfur dioxide usage has been fairly constant at about 5 pounds per million gallons. The only flows requiring dechlorination are the flows being sent to the river. For this reason, the existing dechlorination facilities and usages should be adequate.

Disinfection contact time is achieved in the serpentine concrete contact basin constructed in 1994. This basin has a volume ranging from 83,400 to 112,500 gallons. Depending on water level, this provides 60 to 80 minutes of contact time for the existing permit flow of 2 MGD. Using the average day design criteria and minimum volume of 83,400 gallons, future contact time will range from 79 minutes to 59 minutes. Historically, the maximum flow to the river was 2.825 MGD recorded June 4, 2002, with a contact volume of 112,500. The resulting contact time was 57 minutes. DMR records also show the disinfection resulted in an *E.coli* count of less than 1. From this data, the contact basin seems to be functioning properly and will be adequate for the future. The only exception would be if land application is employed and higher flow rates generated. For this case, some of the pipeline to the irrigation site would possibly have to be used as contact basin to achieve the necessary contact time.

Disinfection System Improvements. Since the preferred treatment and disposal option is constructed wetlands, gas chlorination will continue to be the most likely means of disinfection. Following are the proposed improvements to the disinfection system:

> Upgrade the existing chlorine system by installing a scrubber and ventilation system. This would reduce the risk associated with widespread chlorine exposure in the event of a leak or spill and bring the existing system into compliance with existing unmandated regulations. The work would include the remodeling of the existing building to provide acceptable containment for released gas and the installation of a ventilation system and scrubber to clean up any released gas. Estimated capital cost of this alternative is $462,000 with an estimated annual operation and maintenance cost of $9,000.

SUMMARY OF WASTEWATER PLANT IMPROVEMENTS

Headworks - $461,000

- Septage receiving facility
- Grinder facility
- New two-stage pumping system
- Improvements to existing Parshall flume
- New electrical control building
- Refurbish existing headworks building
- Refrigerated automatic samplers within enclosures

- **Treatment and Wetland Reuse- $4.395 Million**

  - Continue to reduce sludges within primary lagoon with hybrid aerators
  - Modify influent piping to prevent short-circuiting
  - Modify transfer structures to provide more flexibility
  - Develop 40-acre lined wetland
  - Piping to Baldock Slough for discharge

- **Disinfection - $462,000**

  - Upgrade the gas chlorination system by installing a scrubber and ventilation system
  - Remodel existing building to provide acceptable containment and equipment space

Estimated Project Cost for Constructed Wetlands - $5.318 Million

IMPLEMENTATION

Implementation of the improvements require three factors to be addressed: the anticipated schedule for improvements, the methods used to construct improvements, and the funding sources for improvements.

Project Schedule. The overall philosophy for the project schedule is to perform improvements incrementally over time and not necessarily to perform all improvement at one time. Regulatory requirements over time may affect this philosophy as well as construction cost trends. Another philosophy is to view improvements within a 5-year incremental plan. The City of Baker City has been using this philosophy and it has been a good means of meeting the City's goals. Based on these philosophies, the following table provides a project schedule for improvements:

Area	Year
Collection System Improvements:	
DI (D155-D165A)	1-5
DII (D119-D154)	6-10
DIII (D58-D205)	11-15
EI (E67-E108)	6-10
EII (E50-E66)	6-10
A (A0.1-A9) Further Investigation	1-5
Terra Cotta Pipe Replacement	11-55
Manhole Rehab/Replacement	1-5
H Street Lift Station	6-10
Headworks	
Septage Receiving Facility	1-5 ✓
Grinder Facility	1-5 ✓
New Lift Station Pumps	1-5 ✓
Parshall Flume	1-5 ✓
Electrical Control Building	1-5 ✓
Refurbish Existing Building	1-5 ✓
Refrigerated Automatic Samplers	1-5 ✓
Screening Facility	11-15
Treatment and Discharge	
Continued Sludge Removal	1-5 ✓
Modify Influent Piping	1-5 ✓
Modify Transfer Structures	6-10
Develop 40-Acre Wetland	6-10
Construct Piping to Baldock Slough	1-10
Disinfection	
Upgrade Gas Chlorination System	6-10

Construction Methods. There are two construction methods to be employed in order to complete the improvements. One method is to use City staff and the other is to have a private contractor construct improvements. Most pipeline work as well as small structures can be constructed by City staff at a cost savings over utilizing a contractor to perform the work. The following provides the project construction methods to complete the work.

Collection System Improvements

- City Forces

Headworks

- Contractor

Treatment and Discharge

- Continued Sludge Removal
  - Use Existing Aeration System
- Modify Influent Piping
  - City Forces
- Modify Transfer Structures
  - Contractor
- Develop 40-acre Wetland
  - Contractor
- Piping to Baldock Slough
  - City Forces
- Secondary Wetlands at Baldock Slough
  - Ducks Unlimited

PRESENT BUDGET

The present budget allocates funds for several different tasks within the sewer department. The major areas of fund allocation are personnel services, material and services, capital outlay, debt service, contingency, and reserve fund. In the past, the City has organized a committee to assist in determining approximate allocations for each of these areas. In 2001, a rate committee presented to the council the following recommendations for the sewer department budget:

	Rate Committee 2001/2002 Budget	Audited 2001/2002 Budget
Personnel Services	$228,000	$183,033
Materials and Services	218,000	213,874
Capital Outlay	180,000	182,173
Reserve Fund	75,000	0
Debt Service	0	7,209
Contingency	25,000	0
Total Budget	**$726,000**	**$586,289**
Residential Sewer Rate	$11.42	$7.80
Revenue Received		$503,427

After the recommendations were received and further reviewed by the budget committee during the budget process, some budget allocation changes were made. The final audited 2001/2002 sewer fund is shown on the above table. The main reason for the difference in the budgets was that the recommended rate increase needed to fund the budget was instituted over several years. Therefore, the budget was modified to meet anticipated revenues. The following table shows subsequent budget years and associated sewer rates:

	Audit 2002/2003	Audit 2003/2004	Adopted 2004/2005
Personnel Services	$261,480	$285,378	$290,000
Materials and Services	215,117	182,927	236,420
Franchise Fee	29,695	39,228	36,500
PERS Increase	12,235	25,240	15,000
Insurance Increase	13,204	21,184	0
Subtotal Material and Services	**$270,251**	**$268,579**	**$287,920**
Capital Outlay	228,363	68,639	202,451
Reserve Fund[1]	0	0	0
Debt Service	78,557	78,557	78,557
Contingency	0	0	0
Total Budget	**$917,208**	**$701,153**	**$858,928**

	Audit 2002/2003	Audit 2003/2004	Adopted 2004/2005
Residential Rate	10.05	12.30	12.30
Revenue Received	631,749	784,558	855,000[2]

[1] Though the City does not have an identified reserve fund line item, they have been saving money within the sewer fund as indicated by the beginning fund balance. The beginning fund balance is anticipated to be approximately $460,000 for 2005/2006.

[2] $125,000 revenue is anticipated from the truck stop for sewer improvements. Revenue from rates is anticipated to be $730,000.

Using the above sewer fund budget data, an overall Sewer Department budget for evaluating revenue availability for funding improvements is as follows:

Personnel Services	$290,000
Material and Services	292,000
Capital Outlay	145,000
Reserve Fund	0
Debt Service	78,600
Contingency	75,000
Total Budget	**$880,600**
Residential Rate	$12.30
Anticipated Revenue	$782,000
Anticipated Main Line Charge	28,000
Anticipated Service Fees	12,000
Incidental Sales	25,000
Interest	6,500
Rent of Property	2,300
From Beginning Fund Balance	24,800
Total Revenue	**$880,600**

FUNDING IMPROVEMENTS

The population of Baker City limits the available funding options for performing the improvements. Most of the funding will need to come from sewer rate revenues. Some grant funds may be obtained by Ducks Unlimited for constructing the primary wetland and pipeline to Baldock Slough, but these grants will be limited to approximately \$250,000 to \$500,000. Table E-17 shows the allocation of cost over time using the project schedule and the incremental sewer rate that would be needed to meet revenue requirements. It should be noted that Table E-17 does not consider any rate of inflation. Rate of inflation will be discussed later in this section.

For the first 5-year period, the improvements would require a rate increase of approximately \$2.06 per EDU based on 5,300 EDUs if funds were not available within the current budget. Of this rate increase, \$0.33 per EDU would be for City staff-constructed improvements and \$1.73 per EDU for contractor- constructed improvements. The City staff work would be constructed over time, while the contractor-constructed improvements would be constructed in one year with project funds secured by a loan.

Using the overall Sewer Department budget shown in this section, it appears there are sufficient funds to construct the first 5-year improvements as long as inflation adjustments are considered each year. The first five years will require, on average, \$20,800 per year for City staff-constructed work. The contractor-constructed work will require \$110,000 per year for five years for loan repayment. Total funds per year needed for the first five years would be \$130,800.

The second 5-year period would be the time period when the majority of wastewater treatment plant construction would take place. For the work performed by City staff, approximately \$7.73 per EDU rate increase would be needed. For contracted services, between \$2.65 to \$3.35 per EDU would be needed to construct associated improvements that would be dependent upon grant fund contributions. The City staff work would be constructed over time and the contractor work would be constructed over an approximate 1-1/2 to 2-year period. Funds for the contractor work would be secured by loan funds and possibly some grant funds. The total rate requirement for this period of work would range from 10.38 to 11.08 per EDU.

The third 5-year period would start the replacement of terra cotta pipe within the collection system as well as other collection system work. This work would be performed by City forces and outside speciality services as needed. Approximately \$1.73 per EDU would be needed for this work. Additionally, final sludge removal and screening improvements would occur within this period and would probably be performed by a contractor. This work would require approximately \$1.42 per EDU. The total rate requirement for this period of work would be \$3.15 per EDU.

The fourth 5-year period would continue collection system work. Subsequent periods would concentrate on continued replacement of the terra cotta pipe.

OTHER FUNDING CONSIDERATIONS

Another budgeting consideration is long-term inflation rates. Areas affected by inflation are personnel, material and services, and capital outlay. If inflation is not considered, especially when projects are performed over a long period of years, available revenue may not meet the budget requirements. At least a 2 percent inflation rate should be assessed to rates each year due to inflation. Considering the existing $12.30 sewer rate, the yearly inflation would increase rates by approximately $0.25 per year.