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1. Introduction

1.1 Project Background

The western areas of Comal County and the Canyon Lake area have been growing at a substantial rate over the last few years. A predominance of these areas have relied on groundwater supplies mainly from the Trinity Group Aquifer while some have developed surface supplies as a supplemental source. Groundwater levels have been falling in this aquifer due to pumpage and in recent years, thus greater emphasis has been placed on conserving groundwater supplies.

The Guadalupe-Blanco River Authority (GBRA) has continually supported the development of a surface water supply system for these areas. But, the initial cost of such a system has historically prevented realization of such a system.

Due to increased awareness of water needs, drought conditions, and overall regional shortages of groundwater resources, interest in this project has increased. In addition, studies conducted during the Trans-Texas Water Program addressed these areas. The Letter of Intent (LOI) identified two objectives to reduce costs:

1) provide a water treatment plant of a size large enough to reach economies of scale, and
2) provide a delivery system operating at a base load such that the majority of the constructed facilities would be in use as soon as the project was put into service.

In this regard, the GBRA formulated a plan in which the west Comal County water demands can be met as part of a larger delivery system. The larger system will provide water to the major water purveyors in the Bexar and Kendall County areas while also serving west Comal County water needs. The original concept for the plan was described in the Western Comal County Water Supply Study prepared by HDR Engineering, January 1997.

The overall project, now known as the Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties, consists of two preliminary engineering contracts:

- Contract A - Raw Water Intake, Pumping, Transmission, and Water Treatment Plant
- Contract B - Treated Water Delivery System

GBRA retained Malcolm Pirnie, Inc. in December 1999 for professional services related to Contract A, and retained HDR Engineering, Inc. for professional services related to Contract B.

This report advances the concepts described in the above referenced study, delineates alternatives for the Contract A facilities, presents estimated costs, and makes recommendations. Evaluations have been conducted upon lake intake, raw water pumping and transmission, source water quality, finished water quality treatment goals, initial and ultimate plant treatment capacity, and viable treatment technologies. A treatment proposal for the planned Regional Water Supply Project Water Treatment Plant (WTP) is also presented.

Guadalupe-Blanco Water Authority

The GBRA is a political subdivision of the State of Texas, created in 1935 by an act of the Texas Legislature. The GBRA was established to develop, conserve, and protect the water resources of the Guadalupe River Basin. The ten counties within the district include Kendall, Comal, Hays, Caldwell, Guadalupe, Gonzales, DeWitt, Victoria, Calhoun, and Refugio.

GBRA is organized into eleven divisions. These include:

- General
- Rural Utilities
- Water Resources
- Port Lavaca Water Treatment Plant
- Calhoun County Rural Water Supply
- Victoria Regional Reclamation
- Luling Water Treatment Plant
- Guadalupe Valley Hydroelectric
- Canyon Hydroelectric
- Lockhart Wastewater Reclamation
- Coleto Creek

Project Participants

Participating in-district treated water customers include City of Boerne, City of Fair Oaks Ranch and Comal Independent School District and several others. Out-of district customers include San Antonio River Authority, Bexar Metropolitan Water District, and San Antonio Water System.

Estimated quantities of water for the above participants can be found in the Basis of Design Report for Project B, prepared by HDR Engineering, Inc. Delivered water quality is especially of concern to the participants. Most have historically delivered disinfected groundwater pumped from the Edwards and surrounding aquifers. The desire of the participants is that the treated surface water posses no discernable taste and/or odor to facilitate blending with the existing water supplies.

1.2 Project Scope

Project A consists of the following major scope areas:

<u>Phase 1A – Basis of Design Report Services</u>

Malcolm Pirnie will provide preliminary engineering services including project management, regulatory coordination, lake intake, raw water pipeline and plant site evaluation, water treatment technology selection, facilities plan, and optional services. Major areas of work in Phase 1A are:

- **Regulatory Coordination with TNRCC and USACE**
- **Lake Intake, Raw Water Pipeline, and Plant Site Evaluation**
  Task 300 Lake Intake Options Identification and Evaluations
  Task 301 Raw Water Pipeline Route Identification
  Task 302 Treatment Plant Site Identification and Evaluations
- **Preliminary Site Geologic Assessment**
- **Raw Water Delivery System Hydraulics**
- **Water Treatment Technology Selection**
  Existing Raw Water Data Review and Characterization
  Supplemental Raw Water Data and Laboratory Testing
  Analyze Blending Results and Delivered Water Quality Goals
  Finished Water Data Collection and Characterization
  Development of Blending Scenarios
  Bench-Scale Testing for Evaluation of Finished Water Compatibility
  Establish Delivered Water Quality Objectives
  Treatment Technology Options Identification and Evaluations
- **Residuals Management Technology Options Identification and Evaluations**
- **Operations Planning and O&M Definition**
- **Facilities Plan**
  Plant Site Layout Alternatives
  Plant Structures and Support Facilities
  IT/SCADA System Plan
  Cost Opinions
  Basis of Design Report

<u>Phase 1B – Surveying and Right of Way</u>

The objective of this work phase is to establish the surveying needs for Project A.

- **Lake Intake, Raw Water Pipeline, and Plant Site**
  Establish Surveying Needs

Phase 1C – Permitting

The objective of this work phase is to obtain the necessary environmental permits to construct a raw water intake, raw water pipeline, and water treatment facility.

- **Support for Design Report**
  Pre-application Coordination with USACE, TNRCC, TPWD, TGLO
  Environmental Screening for the Proposed Project
- **Field Investigations**
  Jurisdictional Waters/Wetland Field Investigations
  Baseline Environmental Inventory and Threatened/Endangered Species Investigations
  Preliminary Examination of the Raw Water Pipeline Route and Plant Site for Archaeological and Cultural Resources
- **Environmental Assessment and Archaeological Report**
  Environmental Assessment
  Archaeological Report
- **Permit Applications**
  USACE 404 Permit
  Federal Lands Easement Request
  TNRCC Application
  TPWD and TGLO Application
  Submittal and Processing of the Permit Applications

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Lake Intake Options

Technical Memorandum

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2. Lake Intake Options Identification and Evaluations

2.1 Introduction

Raw water for the project may be diverted from Canyon Reservoir or the Guadalupe River downstream from the reservoir in accordance with the terms of the GBRA's water rights permit issued by the Texas Natural Resource Conservation Commission. Prior studies by HDR Engineering, Inc. recommended diversion from Canyon Reservoir due to its closer proximity to the potential customers as well as the higher elevation of the water in the reservoir, which reduces the pumping costs.

Canyon Reservoir and the surrounding property below elevation 948 MSL are owned fee-simple by the U.S. Army Corps of Engineers (COE). Accordingly, any raw water intake on Canyon Reservoir will require an easement from the COE. Meetings and discussions conducted as part of the current study have identified issues that the COE is concerned about. These include public safety, recreational impacts, aesthetic impacts, and impacts on adjacent or nearby parks that may be caused by any structures in the lake. These concerns will need to be addressed through the COE approval and easement acquisition process.

There are several options for the location and configuration of the raw water intake. This section describes the alternatives for the location and type of intakes that are feasible for the project.

2.2 Review of Intake Locations

Prior engineering studies recommended that the raw water intake be located where there was deep water near the shoreline of the lake. Deep water is required to ensure a reliable water supply since yield studies show that the water level could decline by as much as 100 feet during an extended drought and full utilization of the yield of the lake. Suitable deep-water locations are limited to those shoreline areas close to the original river channel. The deep-water location in closest proximity to the proposed water treatment plant is in the area of Comal Park on the south shore of the Canyon Reservoir. Other deep-water locations are available on the south shore, but they would require the construction of a substantially longer pipeline to convey raw water to the treatment plant.

The preferred intake and pipeline location is shown on Figure 2.1 and is identified as Alternative 1. It would require locating the raw water intake off the point of land that juts into the lake from Comal Park where the peninsula is immediately adjacent to the original river channel and installing a pipeline through the park. During the initial on-site meeting with the COE on January 28, 2000, the COE personnel in attendance stated a preference that all facilities be located outside of the boundaries of the park. They also mentioned that the pipeline alignment under consideration was the main access road to the park; although, it is not

Corps property. Two alternative locations for the intake, labeled as Alternatives 2 and 3 on Figure 2.1, were identified in consultation with the COE.

The intake site for Alternative 2 is located on a peninsula on the south shore of the lake and due west of the emergency spillway. This location provides the necessary access to deep water. Alternative 2 would require the construction of an additional 13,000 feet of pipeline through a developing area, resulting in substantially higher construction and operating costs. No further study of this alternative location is recommended.

The intake site for Alternative 3 is located on the south shore immediately south of the emergency spillway. While the site provides access to deep water, it is in an inaccessible location from the shore because of the steep slopes. It would also require the construction of an additional 11,000 feet of pipeline. The location next to the emergency spillway is unsuitable for a floating structure since it could be damaged by floating debris or strong currents in the event the emergency spillway ever engaged. No further study of this alternative location has been completed at this time.

After the meeting with the Corps, a variation to Alternative 1 was considered. It is possible to gain reasonable access to deep water immediately west of the Comal Park boundary. The original river channel is located approximately 480 feet from shore at this location. Alternatives 1A and 1B depict two possible routes for the raw water pipeline that avoid the park entirely as well as address the concern about disrupting access to the park. Both routes are within private rights-of-way and begin at the intake location adjacent to the west boundary of the park and follow the boundary line to the existing county road right-of-way that leads to the park. After traversing to the southwest approximately 4,000 feet along the existing road, the two routes diverge. Alternative 1A follows existing county roadway alignments back to FM 2673 while Alternative 1B follows a more cross-country route. HDR will provide a more complete assessment of the pipeline route alternatives under the Contract B Design Report for the treated water delivery system.

Construction of an intake downstream from the reservoir was also considered as Alternative 4. Such an intake must be constructed where there is adequate water depth during all streamflow conditions. If adequate depth is not available at an existing site, then construction of a channel dam would be required to create a pool of water in the river. A site upstream from Horseshoe Falls, approximately 1.6 river miles below the dam was considered. Assuming the depth is adequate at this location, the need for a COE easement could be avoided (refer to Section 2.3.5 for a discussion of constructing an intake at this location).

2.3 Alternative Intake Configurations

Key criteria for selection of an intake design include:

- Cost – both operating and maintenance costs.
- Function – ability to withdraw high quality water from the lake regardless of the lake's level.
- Reliability and Maintainability - not subject to frequent outages yet readily maintainable when equipment breakdowns occur; this includes long-term reliability and minimizing long-term maintenance needs that are inherent to the type of structure selected.
- Expandability - the ultimate capacity of the treatment and delivery system is greater than the capacity initially planned for installation
- Aesthetics – compatibility with recreational and vacation use; this includes visual and acoustic impacts.
- Safety – the intake should not create a public safety hazard either through placement, lack of lighting or signing, or "attractive nuisance" qualities, nor should it cause safety problems for the operational staff.
- Acceptability by COE – the Corps will ultimately permit whatever intake is selected, so acceptability to the COE is critical.

From a design and operations standpoint, the three most suitable intake designs for the topography and water level conditions at Canyon Reservoir appear to be a floating intake, tower intake, or on-shore shaft intake. Another, less desirable possibility is a submerged intake. Each of these options is described below. The option of a river intake downstream of the dam is also presented.

2.3.1 Floating

A floating intake consists of several components. The substructure is typically constructed of hot-dip galvanized steel box-truss frame elements that are bolted together and rest on flotation cells. The flotation cells are typically polyethylene-encapsulated polystyrene floats with flanges. Stainless steel screws attach the flanges to the box-truss frame elements, and the cells can be removed and replaced should they become old or damaged. A deck of grating or thin concrete panels typically rests on top of the frames. An alternative substructure would use modular barge components such as "FlexiFloats" as manufactured by Robishaw Engineering to form a stable, rigid platform for installation of the pumps.

The superstructure is supported by the substructure and includes the walls and roof of the enclosure. The walls can be architecturally designed for appearance and sound attenuation. A solid roof is recommended to protect the pumps from sunlight and rain. The superstructure should also support a monorail hoist to permit removal of the pumps for maintenance or repair.

Vertical turbine pumps are suspended into the water to the appropriate depth for the water quality desired. Typically, this is between 15 and 25 feet. The pumps' discharge header is connected to shore by two or more flexible rubber hoses that are custom-designed and fabricated for the operating pressures to be encountered. The hoses are typically built with flanged steel nipples embedded in the hose, which is reinforced with multiple plies of Kevlar or polyester cord and a coil of high-strength steel wire.

Power to the intake platform is provided by flexible, water-resistant cables of the type typically used for mining equipment. Similar cables are used for low voltage control applications. The motor control center and SCADA controls would be located onshore.

Floating intakes are usually anchored with stainless steel mooring cables secured to underwater anchor points. The opposite end of the mooring cable is secured to a winch on the intake platform, which allows adjustment of the tension as the lake rises and falls. The mooring lines typically radiate 200-600 feet from the intake platform, which reduces the frequency of winch adjustment.

Advantages of floating intakes include their economy, low profile relative to tower intakes, and the ability to remove pumps for repair relatively easily since the vertical turbines would have short columns and can be hoisted in a single lift with a monorail hoist. They also have the advantage of withdrawing water near the surface of the lake, regardless of water level. Disadvantages include the need for adjustment of the mooring system as the lake level changes, the need for access by boat for routine inspection and maintenance, and the long-term need for maintenance or replacement of the flotation system.

A detailed concept was prepared for the floating intake since it was originally conceived as the preferred alternative. The conceptual plan is shown in Figure 2.2 and the site plan for the location adjacent to Comal Park is shown in Figure 2.3. The floating intake would be located offshore above the original channel of the Guadalupe River. Three pumps would provide the initial capacity of 10 mgd, and space would be left for the addition of a fourth pump. Each pump would be equipped with pump control and isolation valves as well as an air relief valve. A pressure relief and surge anticipator valve would be provided along the discharge header for surge relief.

Two rubber hoses would connect the discharge header of the intake to rigid pipelines at the shore. The pipes would be supported by concrete piers, and they would extend up the slope to the top of the hill. Space would be left for a third hose and steel pipe for future expansion of the intake.

Figure 2.4 shows the conceptual plan for the on-shore facilities at the top of the hill. A common header pipe would join the three steel lines from the lake, and the header would discharge through a flow meter vault. A small electrical and controls building would be provided to house the motor starters, SCADA equipment, and controls.

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The construction cost estimate for a floating intake at the proposed location is detailed in Table 2-1. The total estimated construction cost is approximately $3,000,000 which includes all of the piping, valves, and equipment from the intake to the end of the discharge header where the raw water line begins (refer to Contract B Report, Section 7).

Table 2-1: Construction Cost Estimate for Floating Raw Water Intake & Pump Station

Item	*Estimated Cost*
Floating Intake Structure	$750,000
Pumps & Piping at Intake	$427,000
Submerged Hoses	$175,000
Onshore Valves and Piping	$241,000
Control Building	$23,000
Electrical	$230,000
SCADA/Instrumentation	$30,000
Sitework	$48,000
Other Items	$306,000
Subtotal	**$2,230,000**
Contingency (15%)	$334,500
Subtotal	**$2,564,000**
Mobilization (5%)	$128,000
Contractor's OH&P (12%)	$308,000
Total	**$3,000,000**

2.3.2 Tower

A common type of fixed intake is the tower intake with line-shaft driven vertical turbine pumps suspended from an elevated deck or platform into the water. The platform must be elevated above the maximum pool elevation to ensure that the motors and other electrical gear are not subject to flooding. In the case of Canyon Reservoir, the platform would be located at 948 MSL, 39 feet above the normal pool elevation of 909 MSL. Such intakes tend to be expensive in deep water since they require extensive underwater construction of their support structure, which is typically cast-in-place concrete. Their height also offers an obtrusive visual presence, particularly during low water periods. An expensive element of a tower intake is the provisions required to withdraw water from varying levels.

Another, more expensive approach, is to construct a box tower with multiple gates. The reinforced concrete box provides a wet-well for the pumps and has multiple gates. The problem with this alternative is forming and placing the concrete for this type of structure underwater. Drilled shafts are more readily constructed in the water than formed structures.

The pump platform of a tower intake could be enclosed with walls or left open. Fencing of the platform would not be required if walls were omitted because the height prevents unauthorized entry; although, handrails or a parapet wall would be required. Access to the platform is usually via a bridge; otherwise access by boat is required. The 450-foot distance from shore of the proposed intake location near Comal Park would make the installation of a bridge relatively expensive. An access bridge is usually preferred since it allows better access for regular inspection and maintenance, and it also permits easy access by a crane for removal of the pumps. A bridge crane can optionally be provided for equipment handling but is mandatory if an access bridge is not installed.

The raw water pipeline and power conduits are typically suspended underneath the access bridge where one is provided; otherwise, submerged cables and pipelines are required. The submerged cables would be as described previously for a floating intake. The pipelines would probably be restrained-joint ductile iron pipes laid directly on the lakebed to the shore.

Advantages to a tower intake include a permanent, fixed structure with low maintenance requirements and, if a bridge is provided, easy access for routine inspection and maintenance. Disadvantages include cost, aesthetic impact, and the difficulty associated with removing the long pump columns for repair.

A conceptual plan for a tower intake on Canyon Reservoir adjacent to Comal Park is presented by Figures 2.5 and 2.6. The tower is shown as an elevated platform supported on reinforced concrete piers. The platform would be surrounded by precast concrete panels for noise attenuation and visual aesthetics. The panels would also serve as a windbreak during inclement weather. The roof would be open to allow equipment manipulation by a crane parked on the access bridge, but a bridge crane is also shown inside the structure.

The plan shows three pumps to be installed with space for a fourth, future pump. The pumps would be deep vertical turbines suspended in suction cans. The cans would have multiple inlets with knife gate valves for selecting the depth of withdrawal. Each suction can would be equipped with three inlet gates at different elevations.

The overall length of the access bridge would be approximately 520 feet, and the bridge would slope gradually from the hilltop above the intake site to the platform as shown on Figure 2.5. Fencing at the shore would be installed to deter trespassing on the bridge. The bridge would be constructed with precast concrete beams and a concrete deck similar to a highway bridge.

The controls and electrical gear would be installed in a separate building on shore. This would be similar in size to the building shown in Figure 2.4 for the floating intake.

The construction cost estimate for a tower intake at the proposed location is detailed in Table 2-2. The total estimated construction cost is approximately $6,210,000 which includes all of the piping, valves, and equipment from the tower intake to the end of the access bridge where the raw water line begins (refer to Section 7).

Table 2-2: Construction Cost Estimate for Tower Intake & Pump Station

Item	*Estimated Cost*
Intake Platform Structure	$1,719,000
Access Bridge	$520,000
Miscellaneous Metals	$100,000
Pumps & Suction Cans	$1,050,000
Bridge Crane	$70,000
Pipe & Valves	$406,000
Painting	$55,000
Sitework	$122,000
Control Building	$29,000
Electrical	$300,000
SCADA/Instrumentation	$30,000
Other Items	$214,000
Subtotal	**$4,615,000**
Contingency (15%)	$692,000
Subtotal	**$5,308,000**
Mobilization (5%)	$265,000
Contractor's OH&P (12%)	$637,000
Total	**$6,210,000**

2.3.3 On-Shore Shaft

An on-shore shaft intake is similar to a tower intake with respect to the pumps – line-shaft driven vertical turbines - and the elevation of the operating floor – a minimum of 948 MSL. The primary difference is that the pumps are suspended into a well shaft on shore instead of suction cans from a platform in the lake. The well shaft would be excavated to a depth approximately twenty feet below the lowest intake level, a liner installed, and hydraulic conduits from the lake would be constructed. The liner would probably be cast in place concrete; although, precast panels are possible depending on the final shaft size and shape. The latter would be grouted in place after placement.

The hydraulic conduit could either be some sort of slot or trench, or a series of lined, horizontal shafts at varying depths. The former would probably require a bulkhead at the shaft with a series of gates at varying depths for selective water withdrawal. It probably would not extend to the full depth of the original river channel at approximately elevation 800 MSL. Instead, it would only extend down to elevation 850 MSL to avoid a large quantity of expensive underwater rock excavation. Withdrawing water at extremely low lake levels below 850 is projected to have only a 3 percent probability of occurrence for future years under full firm yield conditions and the record draught of 1947-1956. A temporary pumping arrangement could be provided to pump water into the intake shaft at those times.

The alternative to a slot or trench is the use of horizontal conduits. These could be constructed by directional drilling of small pilot bores and back-reaming to the desired diameter. A coated steel liner pipe would be pulled through each bore and grouted in place. Grates or screens would be placed over the lake-side ends of the pipes to prevent entry by divers and large debris, and gates inside the shaft would control the depth of withdrawal. This approach would probably be more economical to construct than the trench system if full-depth withdrawal is required without the need for temporary pumping facilities during low-water events.

A shaft intake could be constructed at the top of the hill adjacent to the lake, but an alternative would be to construct it at the lowest elevation possible just outside the limits of the COE property at elevation 948. This would require benching the hillside and constructing an access road to the site, but it would reduce the shaft's total depth by approximately 70 feet.

Advantages to an on-shore shaft intake include a permanent, fixed structure with low maintenance requirements and easy access for routine inspection and maintenance. It also eliminates the presence of any structure or improvements in the lake and has the least exposure to the public. Disadvantages include cost and the difficulty associated with removing the long pump columns for repair. There are also uncertainties regarding the suitability of geologic conditions for excavating the shaft and constructing the horizontal bores below the water level in the lake. These uncertainties can be defined with a thorough geotechnical review.

A conceptual plan for a shaft intake adjacent to Comal Park is shown in Figures 2.7 and 2.8. The plan shows a shaft located near the top of the slope above the lake. The shaft is shown to be approximately 230 feet deep (bottom at elevation 780+/-), which would permit withdrawal at all lake levels through three horizontal intake pipes. The shaft would be excavated by conventional means. Primary support of the shaft excavation might be limited to the use of rock bolts and/or steel mesh and shotcrete in the weaker formations, but more extensive support, such as liner plate, could be required in some areas. Secondary (permanent) support may not be required if the rock is competent and a continuous mesh/shotcrete lining is applied during excavation. For estimating purposes, a two-foot thick cast-in-place concrete lining has been assumed.

Installation of the intake pipes can be achieved by directional drilling after the shaft is constructed. Pilot bores would be drilled from a location behind the shaft opposite the lake. The pilot bores would be drilled to intersect the shaft, and sleeves would be placed through the shaft to permit drilling to continue into the lake. Once the pilot bores enter the lake, reaming tools would be used to back-ream the bores from the lake to the shaft. Three reams in increasing size would be required to achieve a 48-IN bore diameter that would be sufficient for installation of a 36-IN coated steel liner pipe. The feasibility of this construction method depends on the ability of the reamed bore hole to be self supporting. This is necessary for the liner pipe to be pulled into place. Collapse of the hole could prevent proper placement of the liner.

Excavation of a deep shaft adjacent to the lake involves a risk of significant groundwater infiltration from the lake through fractures in the rock. One means or reducing the potential for groundwater infiltration involves drilling a series of bore holes circumferentially around the shaft location and twenty feet deeper than the shaft and water testing each hole for exfiltration, which would indicate the presence of fractures. Boreholes that fail the water test would be grouted to refusal to seal the fractures. This method would not preclude the need for dewatering the excavation, but it could minimize the volume of water that infiltrates the shaft. The cost estimate for drilling and water testing the bore holes is approximately $200,000. Grouting may cost as much as an additional $550,000, depending on the integrity of the holes and degree of fracturing. A total estimated cost of $477,000 was used for the purposes of this report.

The construction cost estimate for a shaft intake at the proposed location is detailed in Table 2-3. The total estimated construction cost is approximately $8,479,000 which includes all of the piping, valves, and equipment to deliver raw water into the raw water transmission pipeline (refer to Section 7, Contract B Report).

It must be stressed that construction of a shaft intake appears to be feasible, but an extensive geotechnical investigation will be required to assess the unknowns regarding groundwater, competence of the rock for excavation, and ability of the horizontal bores to be self supporting. These factors may have a significant influence on the cost of the intake. The costs presented above are conservative, and substantial cost savings may be achieved upon completion of a more detailed investigation.

Table 2-3: Construction Cost Estimate for Shaft Intake & Pump Station

Item	Estimated Cost
Grout Curtain	$477,000
Wet-Well Shaft Excavation	$1,507,000
Cast-in-Place Wet-Well Liner	$642,000
Directional Drilling & Reaming	$1,918,000
Suction Liner Pipes	$242,000
Pumps	$600,000
Bridge Crane	$70,000
Pipe & Valves	$191,000
Painting	$10,000
Sitework	$52,000
Pump Building	$205,000
Electrical	$230,000
SCADA/Instrumentation	$30,000
Other Items	$127,000
Subtotal	**$6,302,000**
Contingency (15%)	$945,000
Subtotal	**$7,247,000**
Mobilization (5%)	$362,000
Contractor's OH&P (12%)	$870,000
Total	**$8,479,000**

2.3.4 Submerged

Many different types of submerged intakes are possible. The key advantage to a submerged intake is that there are very few facilities above water. The principal disadvantage is that obtaining water from multiple depths is very difficult, and the pumps must be submerged in the lake. There are a number of designs that have been used for submerged intakes, but the most feasible designs employ vertical turbine pumps driven with submersible motors. For the current project, vertical turbines are the only submersible pumps capable of developing the required discharge head.

The designs commonly used for submerged intakes require inclining the pumps or laying them horizontally. Vertical turbines are designed for vertical installations in wells or on platforms, and the pump and motor bearings are not designed to carry the weight of the rotating elements when loaded laterally. Another problem is maintaining the integrity of the shaft seals on the submersible motors. The motors are typically filled with a dielectric oil, and the seals must keep the water out and oil in. Laying a motor on its side causes more wear on one side of the seal since the bearings allow a very slight play in the shaft. Some pumps employ pressurized oil systems to ensure that water does not enter the motor, but the seals are still subject to premature failure.

Central Texas Water Supply Corporation operates three different submerged intakes on Stillhouse Hollow Reservoir near Killeen, Texas. The on-shore facilities are shown in Figure 2-9. The original intake, a ramp-type intake, has inclined pumps that are slid down ramps made from curved plate. Removal and installation of the pumps is difficult to accomplish and requires a crane. Also, varying the intake depth is difficult with this type of system. The second intake CTWSC constructed was a hinged intake. It also uses inclined pumps, but they are secured to a frame that is hinged at the top and uses ballast tanks to raise and lower the pumps in the water. Removal of the pumps is straightforward using a special work barge, but the intake operates at a fixed depth. Since both of CTWSC's original intakes are relatively shallow and can not be extended to deeper water, CTWSC recently built a new submerged intake with submersible turbine pumps mounted horizontally on a skid lowered to the lakebed. The skid is connected to shore with flexible piping. Each pump has guide pins on the pump housing and a slide coupling on the discharge flange to permit removal by hoisting the pump from a barge, but a diver must first attach the lifting cable. Currently, one pump is out of service due to sediment deposition or settling of the skid into the mud. The second pump experienced a seal failure on the motor and is awaiting repair. This intake configuration also does not provide the ability for selective withdrawal.

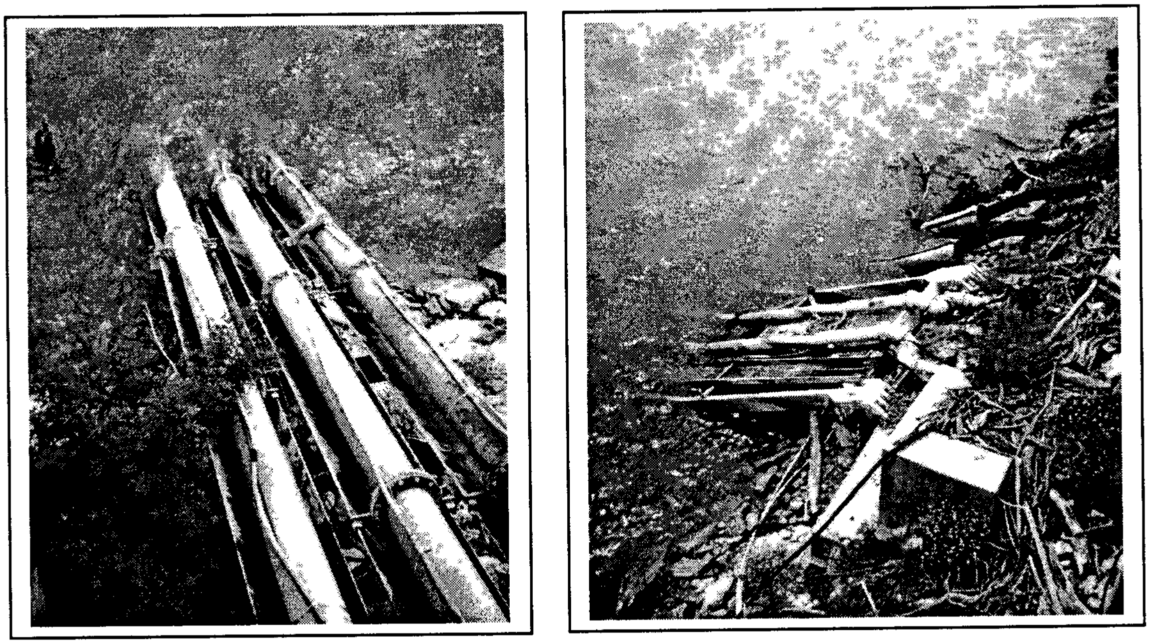
Further evaluation of a submerged intake is not recommended since no installation option will readily provide for maintenance and repair, and providing for selective withdrawal is highly problematic.

2.3.5 Downstream River Intake

A suitable location for a river intake appears to be near the existing gauging station approximately 1.6 river miles below the dam. Constructing an intake at this location would preclude the need for obtaining an easement from the Corps of Engineers, but the static head on the raw water pumps would be increased by approximately 130 feet. Additionally, the length of the raw water pipeline would be increased by approximately 24,000 feet, which would also cause an increase in the friction head on the pumps. The result would be an increase in the pumping cost by approximately $120,000 annually for a 10 mgd delivery rate.

A river intake downstream from the Canyon Reservoir dam would be similar to a shaft intake on the lake. The pumps are suspended into a well shaft on shore that would be excavated to the desired depth, a liner installed, and a hydraulic conduit from the river would be constructed. The liner would probably be cast in place concrete in this instance.

Figure 2-9: Central Texas Water Supply Corporation Intakes on Stillhouse Hollow Reservoir



The left photo shows CTWSC's ramp intake. The right photo shows the hinged intake with the white, painted frame. The two hoses right of the hinge frame are from the submerged skid intake located on the lake bed.

The hydraulic conduit for a river intake would be a horizontal pipe installed by boring or jacking. The conduit would withdraw water from the river through a slotted screen, which would be installed in a cove excavated from the bank of the river and lined with rip-rap. Excavation and work on the intake screen and pipe would be facilitated by the construction of a rock and earthen cofferdam. Steel sheet piping is probably impractical with the shallow depth to rock at the bed of the river.

The advantages to a river intake include a permanent, fixed structure with low maintenance requirements and easy access for routine inspection and maintenance. The disadvantages include the increased pumping cost over the life of the intake, river recreation impacts, and water quality concerns since the water released at the dam is withdrawn from the lower level of the reservoir at its deepest location. While a COE easement may not be required for a downstream intake, a COE Section 404 permit will be required.

A conceptual plan for a river intake downstream from the dam is shown in Figures 2.10 and 2.11. The construction cost estimate for a river intake is detailed in Table 2-4. The total estimated construction cost is approximately $7,690,000 which includes all of the piping, valves, and equipment to deliver raw water into the raw water transmission pipeline (refer to Section 7). Approximately

$3.9 million (including contingencies and contractor mark-ups) of this amount is associated with construction of the additional 24,000 feet of pipeline required for the remote location downstream of the dam. The additional right-of-way cost for the longer pipeline is not included in this estimate and would add another $200,000 to the overall project cost. Also, the present worth of the additional pumping cost should also be considered. This would add the equivalent of approximately $1.5 million to the capital cost, yielding a total present worth for of approximately $9.4 million for this alternative.

Table 2-4: Construction Cost Estimate for River Intake & Pump Station

Item	*Estimated Cost*
Wet-Well	$240,000
Pumps	$600,000
Pipe & Valves	$476,000
Additional 30" Raw Water	$2,880,000
Sitework (includes wet-well excavation)	$730,000
Pump Building	$205,000
Electrical	$280,000
SCADA/Instrumentation	$30,000
Other Items	$275,000
Subtotal	**$5,715,000**
Contingency (15%)	$857,000
Subtotal	**$6,573,000**
Mobilization (5%)	329,000
Contractor's OH&P (12%)	$789,000
Total	**$7,690,000**

2.4 Conclusions

A floating intake structure is recommended for this project. A floating intake is the most economical to construct, is very feasible, and has been demonstrated to be functional elsewhere. It also requires a relatively small structure in the lake compared to a tower intake, the next most economical alternative, and will have substantially less impact on water recreation and aesthetics. The tower and shaft alternatives offer improved access and maintainability over a floating intake, and the shaft intake results in the least impact on the lake itself; however, both the tower and shaft intakes are relatively expensive to construct and require the use of pumps with long columns, which are more prone to failure than pumps with shorter columns due to the increased number of bearings required to stabilize the lineshaft. The tower intake will have the greatest impact on the lake environment and recreation due to its height, the long access bridge, and the number of piers required to support the bridge and operating platform. The downstream alternative is the most expensive in present worth terms and results in substantially increased operating costs. Overall, a floating intake appears to best balance the need for economy while producing a minimal impact on the lake.

It is important to note that the proposed location and configuration for the intake is subject to the approval of the COE, and changes to the final recommendations may be imposed as a result of the COE review process. Any recommendations should be considered as preliminary until such time that an easement is obtained from the COE.

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Raw Water Hydraulics and Pipeline Route

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3. Raw Water Hydraulics and Pipeline Route

3.1 Introduction

The raw water delivery system will convey water from the intake to the Startz Hill Reservoir, from which it will flow by gravity to the treatment plant. As described in Section 2, the preferred location for the raw water intake is immediately west of Comal Park on the south shore of Canyon Lake. The Startz Hill Reservoir will be located near the crest of Startz Hill.

Two potential areas for the location of the treatment plant are described in Section 4.

3.2 Identification of Pipeline Route

The routes for the raw water and treated water pipelines are being investigated by HDR pursuant to Contract B, and additional detail can be found in the Contract B Design Report. The primary corridor currently under consideration is shown in Figure 3-1. The remainder of the pipeline alignment is shown on Figure 4-1. The route begins at the intake location and extends along the western property line of the park to an existing county right-of-way. The pipeline route then follows several county roadways to the intersection of FM 2673 and FM 3159, from which it parallels FM 3159 to the proposed reservoir at Startz Hill. Note that the route shown is a corridor, and the final alignment on a particular side of each roadway has not been established. In general, the alignment will be within a separate easement outside of existing road rights-of-way except in the area north of FM2673 where existing rights-of-way may be used if existing development precludes acquisition of a separate easement.

3.3 Preliminary Hydraulic Analysis

Preliminary hydraulic analysis of the proposed raw water piping was performed for a base project demand of 10 mgd and potential future expanded project capacity of 15 mgd. Average supply demands were developed based on available information provided from project customers during participant meetings held throughout late 1999 and early 2000, as detailed in the Contract B Design Report. The distribution of additional future project capacity was weighted to in-district project participants and is also detailed in the Contract B Design Report.

The hydraulic grade line (HGL) was computed by setting the desired HGL elevation at the outlet of the pipeline and summing the computed head loss for each segment of the pipeline progressing back to Canyon Lake. Determination of the desired HGL elevation at the outlet was made based on the design criteria that the minimum working pressure be at least 10 psi for the entire pipeline. The minimum working pressure occurs near the crest of Startz Hill and the proposed storage reservoir.

The Hazen-Williams equation was used to compute the head loss expected within the raw water system. A Hazen-Williams friction coefficient value of 130 was applied to the preliminary design of the pumps. This selected value has been developed through HDR's experience with design and operation of pumping and piping systems. Past experience has found this value to be representative of actual operating conditions for similar systems.

Determination of pipe pressure class throughout the raw water pipeline was accomplished by subtracting the elevation of the centerline of the pipe from the HGL and converting the result to a pressure. Based on the calculated working pressure, a pressure class was assigned to each segment of pipe (with minimum pressure class of 100 psi). Table 3-1 is a summary of approximate lengths of pipe required within each pressure class. It should be noted that these values are approximate and will be refined by detailed design once a specific route is selected. In order to plan for an expandable system, it is necessary to design the pipe based on the 15 mgd phased expansion.

Table 3-1: Summary of Approximate Lengths – Raw Water System[1]

	30-inch Diameter Pipe	
Pressure Class (psi)	***Base Project, 10 MGD***	***Expanded Project, 15 MGD***
250	8,400	10,500
200	17,300	15,200
100	90	90

[1] *Offshore intake piping upstream of on-shore metering facilities is not included.*

3.3.1 Varying Flow Rates and Lake Levels

The required raw water supply flow rates will vary in response to the system demand. The minimum demands within the system were estimated to accommodate the design for the full range of expected conditions. Minimum demands were estimated from available information provided from project customers during the participant meetings and represent the condition where the system capacity is minimized due to inability of project customers to take available contract supplies. Where information was not available, the minimum daily demand was estimated as:

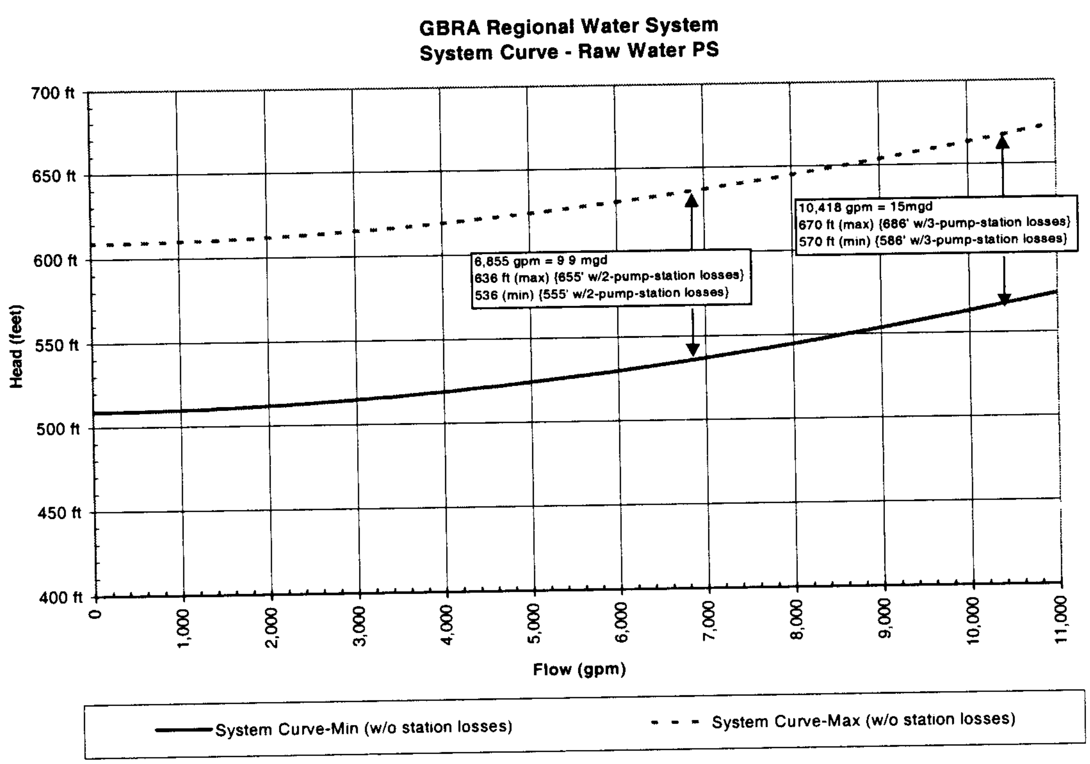
- half of the average daily demand where peaking needs are met from available on-site storage, and
- one-quarter the average daily demand where peaking needs are to be met by the system.

The minimum daily demands estimated for initial operation of the system are detailed in the Contract B Design Report. Hydraulic evaluation considered both the contractual maximum supply demand as well as the estimated minimum daily demand for the full range of expected flows.

In addition to demand fluctuations in flow rate, the raw water system must consider the variations in the lake's elevation. The Canyon Lake normal pool elevation is at 909 MSL. The reservoir is subject to significant level fluctuations, where the firm yield elevation under extended drought conditions is approximately 809 MSL, 100 feet below the normal pool elevation.

The system head curve was developed based on the piping head losses calculated in addition to estimated individual pump and discharge piping losses (station losses) at the intake pumping station. Static head was computed by subtracting the water surface elevation at the intake from the water surface elevation at the Startz Hill tank. The static head was added to the piping and station losses (friction and minor) to determine the total head requirement. Static head values associated with the safe yield and normal pool elevation at Canyon Lake, corresponding to the minimum and maximum pool elevations at which pumping will occur, were used to create the system curves presented. The system head curve for the raw water pump station was developed as shown below in Figure 3-2.

Figure 3-2: System Head Curve - Raw Water Pump Station



3.4 Pump and Pipeline Sizing

The pumps and piping required for the raw water system are sized on the basis that three pumps would be provided for the 10 mgd (6950 gpm) design condition.

Two of the pumps would operate at any one time to provide the required flow with the remaining pump as standby.

The flow would be conveyed through a single raw water pipeline, having the capacity to allow for additional expansion of the system in the future. The 30-inch diameter pipeline size minimizes headloss through the pipeline without violating minimum velocity criteria of 2.0 feet per second.

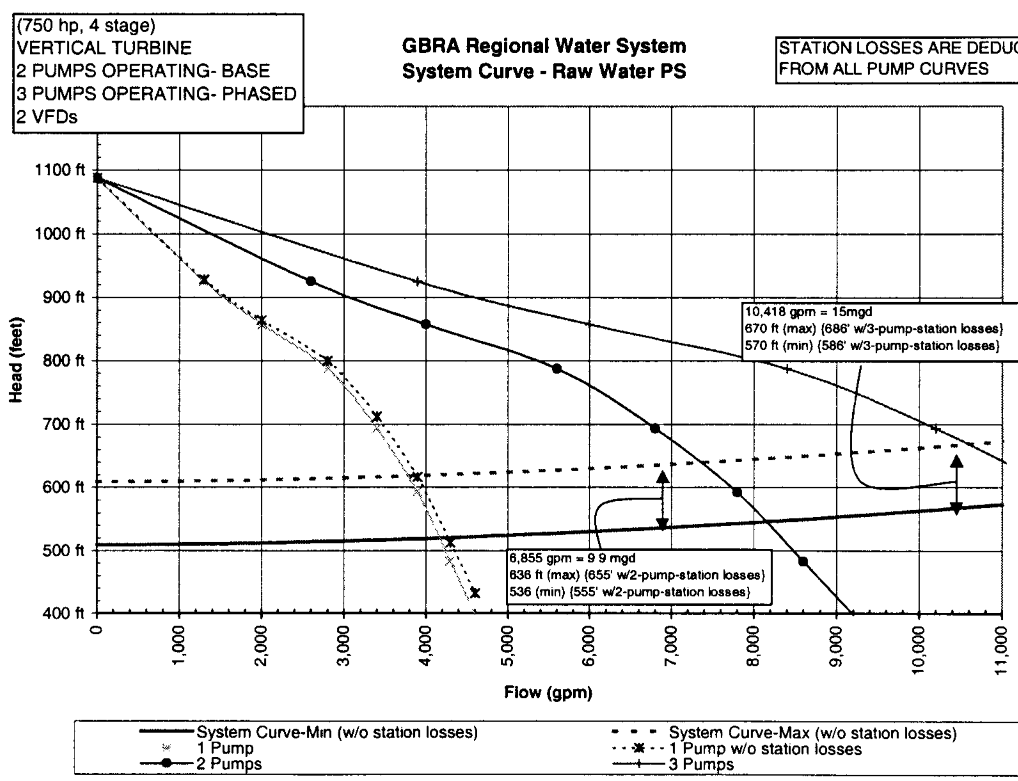
The pumping design conditions for the raw water system is tabulated below in Table 3-2.

Table 3-2: Pumping Design Conditions – Raw Water System

	Base Project	***Future Expansion***
Number of Pumps Operating	***2***	***3***
Total Flow (mgd)	10	15
Each Pump Flow (gpm)	3,430	3,473
Minimum Head (ft)	555	586
Maximum Head (ft)	655	686

Vertical turbine pumps were selected to provide the required suction lift from the lake. The selected pump curves are shown with the system curve in Figure 3-3.

Figure 3-3: Pump Curves- Raw Water Pump Station



Pumps operating at a bowl efficiency of 84% and speed of 1770 rpm require 750 horsepower motors. Addition of a third operating pump provides the required head of 686 feet at an expanded capacity of 15 mgd. These pumps represent vertical turbine pumps available from one manufacturer and are used for illustrative purposes only. Actual pumps will be selected during the final design phase. Intake grates designed in accordance with Hydraulic Institute standards should be provided to prevent the pumps from ingesting large solids.

3.5 Startz Hill Reservoir and Operation of Pumps

The raw water pumps will discharge to the Startz Hill Reservoir, which will then supply water to the treatment plant by gravity. There are several reasons for this approach. First, the reservoir will serve as a head tank, providing water to the treatment plant under a relatively constant pressure. A flow control valve at the treatment plant will modulate to provide a constant flow even though the water level in the reservoir (pressure) will vary. The head and discharge of the raw water pumps will vary significantly in response to the fluctuating water level of Canyon Lake. Raw water pump hydraulics are simplified with the reservoir. Without the reservoir, pump variable speed drives or other flow throttling systems would be required. Another related advantage is that the reservoir will significantly reduce hydraulic transients from the starting and stopping of the raw pumps from being transmitted to the plant. A hydraulic transient analysis will be performed on the portion of the pipeline between the tank and plant.

An alternative to placing a reservoir on Startz Hill is to use variable frequency drives (VFDs) for the raw water pumps. Using VFDs would allow the pumping rate to be modulated as required, and it would avoid the more frequent start-stop operation associated with constant speed pumps. This would also reduce hydraulic transients and achieve a uniform pumping rate and pressure at the plant, but the capital cost would be significantly greater. VFDs for 750-horsepower medium-voltage motors are estimated to have an installed cost of $200-250,000 each. At least two would be required with a total installed cost of up to $500,000. Additionally, such VFDs are very large – approximately 15 to 18 feet in length – and would require a substantially larger control building than currently envisioned, adding another $30-50,000 of capital cost. When compared to the estimated cost of $181,000 for the reservoir, VFDs are not recommended.

Basic operation of the raw water pumping and piping system should involve adequate sequencing of pump start/stops, using the Startz Hill reservoir for control. The pumps would be set to start sequentially based on water level in the reservoir. All pumps would stop once the storage reservoir is full, and pump starts would be rotated so the standby pump on the prior cycle became the lead pump on the new cycle.

The raw water storage reservoir proposed near the location of Startz Hill requires adequate sizing for intended system operations. The reservoir should be sized for the ultimate capacity of the system for future cost savings. If the raw water pumps are sequentially started and stopped to maintain a water level range in the reservoir, then the incremental volume of the reservoir must be adequate to prevent short-cycling of the pumps, which is a condition that causes heat build-up in the motors and can lead to premature failure. Assuming a minimum time of 30 minutes between starts for any individual pump, with three pumps cycling and one standby for an ultimate system capacity of 15 mgd (10,417 gpm), the incremental reservoir volume between starts should be not less than approximately 26,000 gallons. The various dimension options available for diameter and height of a storage reservoir required to provide three incremental reservoir volumes in addition to a minimum reservoir depth of 10 feet were evaluated. A storage reservoir 30 feet in diameter designed for a total depth of 25 feet is recommended, for a total storage capacity of 132,200 gal.

The 30-minute cycle time is an absolute minimum based on starting and stopping one pump as the incremental volume is filled and withdrawn. The minimum time would only occur when the incremental withdrawal rate by the plant is equal to one-half of the incremental pumping rate of the pump being considered. The actual starting scenario would be based on using the entire working volume of the reservoir and alternating the pumps to extend the cycle time. Under these conditions, the cycle time would be lengthened in excess of one hour under most pumping scenarios.

The closed-top reservoir should be constructed either with welded steel or bolted steel with a fused glass coating. The latter offer the advantage of being maintenance free since the inert glass coating is fused to blasted steel surfaces and will not peel, delaminate, or degrade over time.

3.6 Management of Hydraulic Transients

Transient pressures are caused by sudden changes in the velocity of the water column being conveyed by the pipeline. Causes of such sudden changes include electrical power loss at the pump station, valve movements, controlled pump shutdown, pump failure, pump start-up, air venting from the lines, failure of flow or pressure regulators, or pipe rupture. Transients create widely varying changes in pressure, or surges, in the pipeline and could cause damage to the pipeline system via weakened pipe material, dislodged joints, ruptured pipe, or damage to pumping equipment.

Pipeline systems must be protected from the effects of transient flows by relief of the surge pressures to within acceptable pipeline pressure ratings. This relief can be accomplished with the use of surge control equipment. Design of surge control equipment requires analysis of the specific unsteady flow conditions expected to occur within the system. Modeling of these transient pressures will be performed once a final pipeline route is selected. It is anticipated that the surge control equipment will consist of pump control valves on the discharge of each pump as well as pressure relief and surge anticipator valves on the pump discharge header, and combination air valves along the pipeline alignment.

3.7 Alternatives For Future Increases in System Capacity

The alternatives for future increases in system capacity were investigated throughout the design development, as discussed previously. Future expansion of the system from the base project from 10 to 15 mgd would require addition of a fourth raw water pump at the intake to achieve the increase in head and capacity. The 30-inch diameter raw water piping should be designed according to the higher-pressure class ratings as listed previously to accommodate the increased pressures.

3.8 Cost Evaluation

The construction cost estimate for the proposed raw water piping is detailed in Table 3-3. The total estimated construction cost is $4,093,000, which includes the raw water system from the end of the intake pump station discharge header to the water treatment plant and includes the proposed Startz Hill Reservoir. Relocation of the intake to one of the alternate locations shown in Figure 2-1 would increase the cost of the raw water system by $2-3 million due to the additional length of piping required. It would also result in higher O&M costs due to the energy cost of overcoming the higher friction losses of a longer pipeline.

Table 3-3: Construction Cost Estimate for Raw Water System Piping and Storage

Item	***Estimated Cost***
Raw Water Piping	2,722,000
Waterline Markers	1,000
Corrosion Monitoring Station	6,000
Other Items	4,000
Site Work	444,000
Shop Drawings, O&M Manuals, Testing, Start-up	62,000
Subtotal	3,239,000
Contingency (5%)	162,000
Subtotal	3,401,000
Mobilization/Bonds/Insurance (5%)	170,000
Contractor's OH&P (12%)	408,000
Piping Subtotal	3,979,000

Raw Water Storage Reservoir	187,000
Contingency (10%)	28,000
Subtotal	215,000
Mobilization (2%)*	4,000
Contractor's OH&P (12%)	26,000

Raw Water Storage Subtotal	245,000

Total Construction Cost	$ 4,224,000

*No bonds/insurance included for reservoir. 2% used for reservoir.

3.9 Conclusions

The raw water delivery system will convey water from the intake to the Startz Hill Reservoir, from which it will flow by gravity to the treatment plant. The preferred location for the raw water intake is immediately west of Comal Park on the south shore of Canyon Lake, as described in Section 2. The Startz Hill Reservoir will be located near the crest of Startz Hill.

The raw water pipeline routes currently under consideration are shown in Figure 3-1 and 4-1. Preliminary hydraulic analysis of the proposed raw water piping was performed for a base project demand of 10 mgd (11,200 acft/yr) and potential future expanded project capacity of 15 mgd (16,800 acft/yr). The range of expected supply demands were developed based on available information provided from project customers during participant meetings held throughout late 1999 and early 2000 and engineering judgement.

3.9.1 Pump, Piping and Storage Sizing

The pumps for the raw water system are sized on the basis that three pumps would be provided for the 10 mgd (3400 gpm) design condition. Two of the pumps would operate at any one time to provide the required flow with the remaining pump as standby. Vertical turbine pumps were selected to provide the required suction lift from the lake. Addition of a fourth pump operating provides the required head increase at an expanded capacity of 15 mgd.

The raw water piping is sized assuming all flow would be conveyed through a single 30-inch diameter raw water pipeline. This sizing allows for expansion to the 15 mgd condition.

The raw water storage reservoir proposed near the location of Startz Hill is sized for 30-minute minimum pump cycle time. A storage reservoir 30 feet in diameter designed for a total depth of 25 feet is recommended, for a total storage capacity of 132,200 gal.

3.9.2 Operation Schemes

Control of the raw water pumps will be by the water levels in the Startz Hill Reservoir by sequencing the pumps according to loss/gain of incremental reservoir volume.

Surge control/relief equipment will be provided for management of hydraulic transients that may occur within the system. Modeling of these transient pressures will be performed once a pipeline route is selected.

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Water Treatment Plant Siting

Technical Memorandum

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4. Water Treatment Plant Siting

4.1 Introduction

The possible locations for the water treatment plant are dependent on the preferred hydraulic conditions for the raw water and treated water delivery systems. HDR Engineering, under Contract B, determined that the most feasible treated water delivery system needed to include a pump station located at the water treatment plant (WTP). An alternative delivery system did not require a pump station at the plant, but locating a pump station at the plant appeared to offer substantial economic benefits. Another benefit includes the ability to locate the WTP at a lower elevation, permitting the ability to gravity feed from a reservoir on Startz Hill to the plant as described in Section 3.

4.2 Preferred Elevation Range for WTP

The preferred elevation range for the WTP is determined by considering reasonable inlet pressures desired at the WTP. Membrane processes require higher inlet pressures than conventional processes. Minimum pressure options for 35 psi and 50 psi have been evaluated since 35 psi is considered the minimum operating pressure for low-pressure membranes, and 50 PSI provides additional pressure for future process modifications. These minimum pressures would be substantially less if a conventional type of process is selected for the WTP. The most feasible conventional process alternatives require up to 15 psi inlet pressures.

The required elevation of the WTP is determined by evaluation of the hydraulic grade line (HGL) downstream of the Startz Hill reservoir. The HGL elevation at the WTP is determined from the following equation:

HGL (ft)	=	reservoir static head (ft) (free water surface)	-	Friction Losses (ft) (raw water pipe)

A minimum reservoir depth of 10 feet is assumed. The limiting ground surface elevation at the WTP is calculated as follows:

Limiting Ground Surface ELEV (ft)	=	HGL (ft)	-	WTP Inlet Pressure (Min. Req'd, ft)	-	Distance CL Pipe to Ground (ft)

The elevation of the WTP must be at or below the limiting ground elevation.

Two potential plant site locations were identified downstream of the proposed Startz Hill raw water reservoir based on the limiting ground elevations determined for each site. These sites have been evaluated further to assess other factors pertinent to preferred site selection.

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4.3 Review of Two Potential Sites

The two potential plant site locations are identified as Plant Area A and Plant Area B in Figure 4.1. The limiting ground surface elevations determined for each plant site are indicated with reference to the corresponding minimum WTP inlet pressure for a membrane treatment process.

The limiting ground elevation for each potential plant site location corresponding to the required minimum outlet pressures of 15 psi, 35 psi, and 50 psi are tabulated in Table 4-1.

Table 4-1: Limiting Ground Elevations in Feet above Mean Sea Level (MSL)

Potential Location	Raw Water Piping Minimum Outlet Pressure		
	50 psi	35 psi	15 psi
Plant Area A	1260	1295	1341
Plant Area B	1244	1279	1325

4.3.1 Size

The size of the tract for the WTP location must accommodate the necessary requirements of the plant, ensuring that sufficient area is available for all proposed plant facilities, which is approximately 20 acres. Appropriate selection of tract size impacts the capital and operational costs of the project as well as the feasibility for future capacity expansion of the system.

Both Plant Area A and Plant Area B will accommodate the area that would be needed by the plant. Depending on the desired minimum outlet pressure of the raw water line, the available area that would be available for the plant is within contours lower than the limiting ground surface elevation.

4.3.2 Topography

Each potential site should be evaluated for availability to utilize the existing physical features and site topography as this may effect the resulting capital cost of the project or may lead to limitations on the future expansion capacity.

Both potential plant site locations, Plant Area A and Plant Area B, are within mildly depressed areas. Dry Comal Creek channel flows are conveyed through the Plant Area A and West Fork Creek and its tributaries run through Plant Area B. A plant site near these drainages would need to ensure flooding is avoided. Both areas provide development areas with relatively mild slopes, requiring minimal grading. Some topographic relief is desirable to permit gravity flow from the treatment process to an above-ground clearwell reservoir. Both areas appear to contain sites that meet the topographic criteria.

4.3.3 Access

Site accessibility directly impacts the capital cost of the project and requires consideration for plant site location selection.

Cranes Mill Road passes through the eastern portion of Plant Area A, both within the 50 psi and 35 psi minimum inlet pressure contours. Cranes Mill Road is a light-duty county road that is not designed for regular use by heavy trucks. Major access to the road is via FM 3159 to the north, a minimum of 2,300 feet from the outside boundary of the 50 psi minimum contour. In addition to the on-site roads required, selection of Plant Area A may require construction and/or rehabilitation of the Cranes Mill Road to allow for the construction equipment and chemical delivery trucks that will be required for construction and operation of the plant.

Plant Area B is bordered by major access roads with FM 3159 to the north and Highway 46 to the west. In addition, FM 311 passes through the western portion of the site. Site access by large construction and maintenance vehicles is not a problem, requiring only construction of on-site access roads.

4.3.4 Utility Service Availability

The proximity of each potential site to existing electrical service impacts both the capital cost of the project as well as the reliability of the plant. Dual service from two separate utilities would provide increased reliability without requiring an on-site generator.

The electric service providers along the proposed pipeline route shown on Figure 4-1 were determined from phone discussions with Pedernales Electric Cooperative, Inc. (PEC). PEC is the provider for areas east of the FM 3159 and Cranes Mill Road intersection. The area west of the intersection to the FM 3159 and Highway 46 crossing is serviced by PEC to the north and New Braunfels Utilities (NBU) to the south. As a result, both Plant Area A and Plant Area B are bordered by PEC and NBU service areas and dual service may be possible.

4.4 Other Considerations

Other selection criteria of importance that should be considered include:

- Avoidance of significant environmental and archaeological features
- minimizing location of WTP off-set from proposed pipeline corridor,
- land ownership and tract configuration evaluation,
- geotechnical conditions,
- site visibility, and
- adjacent land ownership.

4.5 Conclusions

Potential locations for the WTP are dependent on hydraulic conditions of the delivery system. Previous work has determined that a pump station located at the WTP is most feasible for the treated water delivery system. Location of the

WTP at a lower elevation permits the ability to gravity feed from a reservoir on Startz Hill to the plant.

The preferred elevation range for the WTP is based on required raw water pipeline inlet pressures at the WTP. Options for 35 psi and 50 psi minimum pressures were evaluated and presented for the membrane treatment processes currently in consideration for the plant. The inlet pressures could be lower if more conventional-type processes are selected for the WTP.

Two potential plant site locations (Plant Area A and Plant Area B) were identified downstream of the proposed Startz Hill raw water reservoir. Both locations will accommodate the area that would be needed by the plant.

Depending on the actual site under consideration, accessibility may be better for Plant Area B since the area is bordered by existing major roads (FM 3159, Highway 46, and FM 311) designed to accommodate heavy loads. Access to Plant Area A is primarily along Cranes Mill Road, a light-duty county road.

Plant Area A offers the advantage of being closer to Startz Hill, which will allow the plant to be at a slightly higher elevation. Also, this location would reduce the length of a treated water pipeline to serve areas north of the plant in the event service is provided to that area.

Proximity to electrical service appears adequate for both potential plant site locations. Plant Areas A and B are both bordered by PEC and NBU electric service areas and dual service may be possible.

Based on evaluation of the advantages and disadvantages presented for each potential plant site location, a plant site within either area is acceptable. The final site selected should be within the elevation range required and as close to existing paved, state highways and the pipeline corridor as reasonable.

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Geologic Assessment

Technical Memorandum

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5. Geologic Assessment

5.1 Introduction

A preliminary geologic assessment of the proposed water treatment plant site is required to help determine whether poor soil conditions and features exist that would make construction difficult and expensive. In addition to environmental and site features such as slope of land, geotechnical data, are critical site selection criteria.

The geologic assessment of the water treatment plant site has not been completed. This assessment will be performed after submission of the final basis of design report. An initial field trip to evaluate potential plant sites is scheduled for early June 2000. After a plant site is selected, the geotechnical firm (Raba Kistner Consultants Inc.) will drill three to five borings, collect soil samples and perform limited laboratory testing to preliminary assess subsurface conditions. Raba will excavate a test pit to assess subsurface water conditions and allow for long term monitoring, if required. Following the investigation, a letter report will be prepared indicating site soil properties, preliminary foundation design information, and construction considerations. A description of anticipated subsurface conditions which may have an adverse effect on the proposed project will be included. This letter report will be an independent supplement to the basis of design report. During the final design stage of the project, additional geotechnical investigations will be required.

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Regulatory Strategy

Technical Memorandum

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6. Regulatory Strategy

6.1 Introduction

The purpose of this section is to review pending drinking water regulations and to develop a regulatory strategy for GBRA for the new 10 mgd WTP and transmission main and the existing distribution system. The discussion that follows provides an overview of the Disinfectant / Disinfection By-Products (D/DBP) Rule, the Interim Enhanced Surface Water Treatment Rule (IESWTR), and Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). In addition to the regulatory overview, the discussion includes an assessment of the potential impacts of current and pending regulations on the treatment of Canyon Lake water and its distribution to consumers when blended with other waters in the participants' distribution systems.

Applicable rules and regulations for GBRA include:

Current and Effective:

- Total Coliform Rule
- Lead and Copper Rule
- Surface Water Treatment Rule (SWTR)
- TNRCC, Texas Annotated Codes Chapter 290
- USEPA Primary and Secondary Drinking Water Standards

Promulgated:

- Stage 1 D/DBP Rule
- Interim Enhanced Surface Water Treatment Rule (IESWTR)

Future Rules:

- Stage 2 D/DBP Rule
- Long Term 2 Enhanced Surface Water Treatment Rule (LT2 ESWTR)
- Filter Backwash Recycle Rule (FBRR) (as part of the Long Term Rules)
- Groundwater Disinfection Rule (GWDR)

Section Organization:

This section is organized under the following discussion topics:

Section	Section Title	Contents
6.2	Disinfectant/Disinfection By-Products (D/DBP) Rule	Discusses Stage 1 DBP MCLs, MRDLs, TOC removal requirements, and potential Stage 2 DBP MCLs
6.3	Interim Enhanced Surface Water Treatment Rule (IESWTR)	Presents disinfection and turbidity removal requirements
6.4	Long Term 2 ESWTR	Presents proposed disinfection

		requirements including removal/ inactivation of *Cryptosporidium*; Introduces potential requirements of the FBRR for filter backwash recycle and filter-to-waste operations
6.5	Impacts of Current and Pending Regulations on the 10 mgd WTP	Summarizes the impacts of these regulations on the new WTP expansion; highlights potential future impacts and a regulatory strategy for compliance
6.6	Impacts of Current and Pending Regulations on the distribution system	Summarizes the impacts of these regulations on the GBRA distribution system; highlights potential future impacts and a regulatory strategy for compliance

6.2 Disinfection/Disinfection By-Products Rule

6.2.1 Background

DBPs are formed when natural organic matter (NOM) or inorganic compounds such as bromide react with oxidants added in a water treatment system. Certain DBPs are recognized for their potential to cause adverse health effects.

The primary objective of the Disinfectant/Disinfection By-Products (D/DBP) Rule is to reduce exposure of drinking water users to these DBPs, while other regulations (e.g., ESWTR and Total Coliform Rule) assure adequate disinfection. The D/DBP Rule is to be proposed in two stages. Stage 1 includes requirements for Maximum Residual Disinfectant Levels (MRDLs), Maximum Contaminant Levels (MCLs) for trihalomethanes (TTHMs), the sum of five haloacetic acids (HAA5), bromate and chlorite, and DBP precursor, or total organic carbon (TOC) removal. The implementation schedule for Stage 1 is shown in Table 6-1.

Table 6-1: Stage 1 D/DBP Rule Implementation Schedule

Final (date of promulgation)	December 16, 1998
Effective – Systems Serving > 10,000	December 16, 2001
Effective – Systems Serving < 10,000	December 16, 2003

* Utilities can negotiate more time with States if changes are significant.