BEFORE THE FLORIDA PUBLIC SERVICE COMMISSION

DOCKET NO. 100104-WU

WATER MANAGEMENT SERVICES, INC.

IN RE: APPLICATION FOR INCREASE IN WATER RATES IN FRANKLIN COUNTY BY WATER MANAGEMENT SERVICES, INC.

REBUTTAL TESTIMONY & EXHIBITS OF:

MICHAEL SCIBELLI

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WATER MANAGEMENT SERVICES, INC.

Q. **Please state your name, profession and address.**

A. My name is Mike Scibelli. I am a Project Director and Associate Vice President with Post, Buckley, Schuh and Jernigan, Inc. (PBS&J). My business address is 2639 North Monroe Street, Building C, Tallahassee, Florida 32303.

Q. **Please summarize your educational background and professional experience.**

A. I am a graduate of Vanderbilt University with a Bachelor of Engineering degree in Environmental and Water Resource Engineering and Mathematics (1983). I have also earned a Master of Science in Planning from The Florida State University with an emphasis in Growth Management (1991). I have practiced civil and environmental engineering in Florida since 1983 and became a Professional Engineer in the State of Florida in 1988. In that time, I have worked on municipal and industrial facilities related to water and wastewater treatment. I have performed and managed the planning, design, permitting, and construction administration of water wells, storage tanks, water treatment facilities, wastewater treatment facilities, booster pumping stations, sanitary lift stations, odor controls systems, potable transmission and distribution systems, wastewater collection systems, grease treatment facilities, septage treatment facilities, and sludge treatment facilities. Clients include the City of Tallahassee, Leon County, City of Webster, City of Umatilla, City of Vernon, City of Fort White, Hillsborough County, City of Tampa, City of St. Petersburg Beach, Wakulla County, City of Sopchoppy, City of Mascotte, Town of Branford, Sarasota County, Pinellas County, Hernando County, City of Tarpon Springs, U.S. Department of Energy, City of Coleman, City of Umatilla, City of Pinellas Park, City of Carrabelle, Horseshoe Beach Water Authority, City of Trenton, City of Hampton, Town of Greenwood, Town of Lake Placid, The St. Joe Company, Lykes Brothers, Winn Dixie, Klondike, Hunter Jersey Farms, SuperBrand Diary, Corrections Corporation of America, Florida State University, N-Viro, and Pasco County. A summary of my experience and education is attached hereto as Exhibit (MS-1) ___.

Q. Are you a registered engineer in the State of Florida?

A. Yes. Florida P.E. No. 40238.

Q. Are you a member of any professional or technical societies and associations?

A. Yes, I am a member of several, including Water Environment Federation, Florida Water Environment Association, and Project Management Institute.

Q. On whose behalf are you presenting testimony?

A. I am presenting testimony and appearing on behalf of the applicant, Water Management Services, Inc. (WMSI).

Q. Have you testified previously in this docket?

A. No.

Q. What is the purpose of your testimony?

A. The purpose of my testimony is to sponsor the PBS&J evaluation as an exhibit and to respond to portions of the direct testimony presented by Office of Public Counsel (OPC) witnesses Andrew Woodcock and Donna Ramas. The PBS&J evaluation was provided to OPC in response to a discovery request and is referred to in the direct testimony of OPC witness Woodcock.

Q. Are you sponsoring any exhibits?

A. Yes, I am sponsoring three exhibits. Exhibit (MS-1) ___ is a summary of my education and experience. Exhibit (MS-2) ___ is PBS&J's evaluation of WMSI's water system, dated April 2010. Exhibit (MS-3) ___ is an addendum to our evaluation.

Q. Can you elaborate on the PBS&J evaluation of WMSI's water system?

A. Yes. The evaluation included a review of operation to assess the overall condition of the system, and identify needed capital improvements to ensure long-term viability and reliability of the system to provide water to the residents of St. George Island. The evaluation of the water system included a detailed review and assessment of the following major system components: (i) Raw Water Transmission Main; (ii) Capacity Assessment to identify the limiting capacity component in the system; (iii) Source Water Supply evaluation to assess adequacy vulnerability and weakness; (iv) Assessment of Water Plant Process, overall condition and review of current operation; (v) Structural observations of the water plant with determination of a need to perform repairs/remediation or replacement; (vi) Review electrical systems and controls at the water facility and each individual well to determine adequacy and general condition; and (vii) Water Distribution Operation, maintenance and review water quality parameters.

Q. Did PBS&J have recommendations for WMSI based on the evaluation of WMSI's water system?

A. Yes. PBS&J's detailed recommendations are contained in Exhibits (MS-2) ___ and (MS-3) ___.

Q. Are you aware of the increasing trend of WMSI's customers drilling shallow wells on St. George Island?

A. Yes.

Q. Do you have any concerns with this trend? If so, explain why.

A. Yes. The use of shallow wells increases the potential for cross contamination of the public water supply within the distribution system. This potential would occur if a homeowner were to connect the plumbing from a shallow well to the plumbing associated with the central water system. This connection happens either on purpose or by accident. This is a health concern, as the water from shallow wells would not meet disinfection criteria for a public water system. The water may also not meet other water quality criteria. In addition, most of the structures on the island utilize septic tanks and drain fields for sewage disposal. There is a potential that shallow wells could draw partially treated sewage from the ground water and contaminate the public supply with pathogens resulting in water customers becoming sick. To my knowledge, there is no mechanism for mapping or tracking the existing locations of septic tanks, drain fields, or existing shallow wells so it is likely that the minimum separation between a well and a septic system could be violated unknowingly. The current cross contamination plan would likely need to be modified to include the requirement for all connections to the public system to include a pressure reducing backflow device as approved by the Florida Department of Environmental Protection (FDEP). Currently, FDEP only allows one type of device which is above ground and includes two check valves and a relief valve. These backflow preventers are expensive, easily damaged by vehicles and mowers, and require regular certification and maintenance. FDEP rules are currently under review and are expected to change in the future regarding backflow preventers and other types of backflow preventers may be allowed but additional cross contamination prevention controls will likely be required by the FDEP in the future.

Q. Do you have additional concerns with the increasing trend to utilize shallow wells?

A. Yes, there are two. First, there will be increasing cost pressures on WMSI to continually monitor for potential cross connects. Second, the displacement of gallons previously sold by WMSI will likely result in inadequate revenues to cover costs.

Q. Have your read the corrected direct testimony of Office of Public Counsel witness Andrew T. Woodcock in this docket?

A. Yes.

Q. At page 9 of his direct testimony, Mr. Woodcock states that the capital improvements recommended by PBS&J would "replace aging assets, improve the quality of service to the customers, or improve the safety and reliability conditions of the utility system." What is your response to this statement?

A. I agree with Mr. Woodcock's assessment.

Q. At pages 9 through 11 of his direct testimony, Mr. Woodcock compares Alternatives 2 and 3 regarding the construction of a new ground storage tank (GST) and on page 11 states that "customers would be equally served by installing a new tank on the existing GST site with a cost savings of $191,492." What is your response to Mr. Woodcock's statement on page 11 and his comparison of the two alternatives for the GST?

A. In general, I agree with Mr. Woodcock that, all things being equal, the customers would be equally served by having a new tank built on either a new site or on the existing GST site; however, after reviewing our comparison of Alternatives 2 and 3, it is apparent that these alternatives are not an "apples to apples" comparison. We have revised our alternative analysis for Alternatives 2 and 3 to provide a more accurate comparison between the two alternatives and have included that as an addendum to our report as Exhibit (MS-3) ___.

The main problem with utilizing the existing tank location for the new tank is risk, which is often hard to reflect in terms of estimated cost. In order to use the existing location, the old tank would need to be taken out of service during the demolition and construction of the new tank. This would require the use of temporary piping and pumping facilities. Use of such facilities are problematic from a constructability standpoint for several reasons including: (i) lack of available space to locate temporary tanks and pumps, an increase in the complexity of the system which inherently reduces the overall system reliability; (ii) lack of redundancy in the system which could lead to extended outages of supply of water; and (iii) discovery of unforeseen circumstances during construction which could lead to extending the time required for temporary facilities thereby increasing the associated costs. It is my opinion that given the reduction in land costs experienced over the past year and given the uncertainty with the actual cost of using temporary facilities and the related risks, the actual cost difference between Alternatives 2 and 3 is insignificant and therefore I still recommend building the tank on a vacant adjacent site.

Please refer to our report addendum (Exhibit (MS-3) ___) for a revised, detailed listing of costs for Alternatives 2 and 3.

Q. Does PBS&J provide on-going engineering services to WMSI? If so, what is the scope of those services?

A. Yes. In addition to preparing the Water System Evaluation and providing services related to this rate request, PBS&J has been providing consulting services on an as needed basis to both the general manager and operations staff. Some recent examples include providing advice on meter sizes for various users and on painting specifications. WMSI has requested that PBS&J assist in other types of non capital services, including review of proposed FDEP rules associated with cross contamination management plans; the new Franklin County ordinance associated with construction on St. George Island; an analysis of fire flows and pressures in the plantation area and also towards the State Park, assistance with a Northwest Florida Water Management District (NWFWMD) permit renewal; analysis of the control system for the raw water wells; assistance with a permit condition associated with rotation of well pumping; a review of chlorine residuals throughout the system and recommendations for improvement; assistance with leak detection; analysis of the integrity of the distribution system; assistance with general compliance of all federal, state, and local rules and regulations; raw water supply analysis; oversight of current raw water transmission main painting and maintenance; oversight of elevated tank maintenance; assistance with Franklin County ordinances and related interpretations and implementations; planning and evaluation of system expansion alternatives; development of standards related to meter sizes and other relevant standards for normal operations; implementation of a geographic information system (GIS); advice on operations procedures and standards; advice on system security; advice and evaluation of use of smart meters; regular system inspections; availability to assist with interface with public agencies; and other services which require professional engineering assistance.

To date, funding has been limited and we have been providing consultation on a very limited basis and we have not been able to engage in many of the above services due to a lack of available funds.

Q. PBS&J just completed an extensive evaluation of WMSI's water system. In light of that, why does WMSI need on-going engineering services?

A. It is our experience that WMSI would benefit from having a multi-discipline professional engineering firm with expertise in potable water on retainer to provide expertise and assistance that is needed above that of what a licensed operator could normally be expected to provide. It is not unusual for utility companies to have an engineering staff to assist with normal operations. Given the size of WMSI, a full time staff person would not be required, but having a firm such as PBS&J or another similar firm would be a benefit to the end users in terms of reliability and quality.

Q. **Have you read the direct testimony of Office of Public Counsel witness Donna Ramas in this docket?**

A. Yes.

Q. **At pages 21, 22 and 23, Ms. Ramas proposes that the annual retainer of $48,000 to PBS&J be disallowed because such engineering services costs are not historically consistent and because "future engineering services would likely be of a capital nature and something that would be recorded as an expense on the Company's books." Do you agree with her assessment? If not, why?**

A. No, I do not agree with the assessment of Ms. Ramas. I believe that having a consultant under contract or on retainer is both prudent and necessary. There are valid reasons why most public utilities have either an engineering staff or an engineering consultant or both as it is necessary in order to provide safe and reliable service to their customers.

We were asked by WMSI to provide an estimate of the lowest retainer that would allow PBS&J to be able to provide on call services and that is how we came up with the $48,000. This number is based on $4,000 per month which roughly equates to 32 hours of consultation per month. While we expect that some months may be higher and some may be lower, this is a minimum estimate to provide an adequate level of engineering support to WMSI based on what we know about the system and their operations.

We disagree with Ms. Ramas' suggestion that $5,500 per year is an adequate budget. This roughly equates to less than four hours per month, on average, of engineering support. We would not be able to provide support for this budget. We have reviewed the memo sent to me from Gene Brown dated August 24, 2010, regarding engineering services and we generally agree that these services are needed by WMSI. Quite frankly, we would need to prioritize those items and come up with a plan to accomplish the most important items in descending order for the proposed budget of $4,000 per month or $48,000 per year. We would need to cut those services substantially or terminate our services to WMSI if the proposed budget was not available.

Q. At pages 42 and 43 of her direct testimony, Ms. Ramas proposes that the wastewater certificate application amortization cost not be approved because it had nothing to do with the provision of water service and is only an attempt to expand WMSI's services. Do you agree that recovery of this amortization cost should be denied?

No, I do not agree. At the time we did the work associated with a central wastewater system, the commercial district of St. George Island was experiencing numerous septic tank failures from commercial facilities and restaurants. The County Health Department was issuing violations and actually restricting the capacity of the establishments. On a site visit, my staff went to lunch and visually witnessed an overflowing septic tank at a local restaurant resulting in sewage running over ground. There was also an increase in the number of water quality notices in the gulf and in the bay on St. George Island which were resulting in warnings regarding swimming in the salt water. As St. George Island is primarily a vacation and tourist destination, the requirement to post swimming warnings in rental properties had the potential to adversely affect property values. Franklin County was discussing the issue at County Commission meetings and contemplating action. I personally made a presentation to the Franklin County Commission at one of these meetings to present the findings of the feasibility study in question. The residents and business owners of the island and the customers of the water system are essentially the same population. They were at risk from untreated sewage exposure due to overflowing tanks, from partially treated sewage exposure through use of shallow wells, and potentially subject to additional regulation and expense from possible action by the County Commission. Given this specific scenario and the possibility of change, it makes logical sense that the existing water service provider would investigate the feasibility and costs associated with providing a central sewer system to the center core (commercial district) of the island and to the entire island to their existing customers as there would be inherent efficiencies with the same utility provider supplying both potable water and sanitary sewer service. I believe that WMSI would be able to provide the least expensive sewer rates for a central sanitary system because of inherent efficiencies. These efficiencies include common billing, common administration and potential for cross trained operators and maintenance personnel reducing the total number of employees for both utilities. The analysis provided by WMSI was utilized by the local citizens, the local business owners and the County Commission. While all parties did not agree on the outcome to date, the information was useful to all parties involved. It is my opinion that the water ratepayers benefited from the analysis.

Q. **Was the pursuit of a wastewater certificate application designed to benefit water customers in any other way?**

A. Yes. In addition to the environmental and efficiency benefits, the wastewater certificate would have allowed existing commercial customers to stay on the water system and to potentially expand. It would also have enabled new commercial customers to be added to the existing water system. This would benefit existing water customers by maintaining and even expanding the base upon which fixed costs are recovered. This would have a direct economic benefit for WMSI's existing water customers.

Q. Does that conclude your rebuttal testimony?

A. Yes, it does.

Michael A. Scibelli, PE

Associate Vice President/Project Director

PBS&J

Education

M.S., Planning, Urban and Regional, Florida State University, 1991

B.E., Environmental & Water Resources Engineering & Mathematics, Vanderbilt University, 1983

Registrations/Licenses

Professional Engineer
Florida 40238, 1988

Mr. Scibelli is a senior civil engineer and urban planner experienced in project planning, permitting, design, and management of private development and public works projects, and master planning studies and reports. Mr. Scibelli is also experienced in the construction administration of related projects. He has 27 years of engineering experience in Florida.

Wastewater

Project manager for Killearn Lakes Low Pressure Sewer System serving over 1,300 units in Leon county, Florida.

Project manager for City of Tallahassee Master Reuse Plan.

Project manager for City of Webster Master Sanitary Plan. Includes planning of a centralized sanitary sewer system including preparation and administration of a $1.6 billion Florida Department of Environmental Protection grant.

Project manager for City of Umatilla Master Sanitary Plan.

Project manager for City of Vernon Wastewater Treatment Plant (WWTP) expansion.

Design engineer for Falkenburg Road Transmission Facilities in Hillsborough County including a 15.5-mgd pump station with odor control and 18 through 42-inch gravity sewers.

Conducted the evaluation and design for the master sewer plan for Wakulla County. The project included the implementation of a sanitary sewer to protect the oyster population in the adjacent Ochlockonee Bay.

Feasibility study for implementation of sanitary sewers in Mascotte, Florida.

Project manager for Town of Branford Wastewater Treatment Plant expansion.

Project manager for the contract administration and resident inspection of the Henry Street Pump Station, Hillsborough County (3.6-mgd)

Responsible for design and production of construction documents for Henry Street Pump Station and Manhattan Avenue Force Main (16-inch). U.S.Environmental Protection Agency Grant Project.

Evaluated waste and pre-designed a treatment facility for dairy wastes at Hunter Jersey Farms in North Carolina.

Assisted with the design of Toytown Leachate Treatment Plant in Pinellas County.

Responsible for process evaluation at Bee Ridge Septage Treatment Plant in Sarasota. The project included evaluation of a septage and grease treatment facility with an influent BOD of 6000 mg/1. Developed a report for implementation of grit removal, chlorination modifications, and procedures for increased operator efficiency.

Project engineer for the expansion of the Apollo Beach WWTP in Hillsborough County. The project included conversion to submerged aerators and the addition of a sand filter system

Managed the design of the Tampa Suburban Pump Station in Hillsborough County.

Master sanitary plan and system design for the Gateway Centre Industrial Park in Pinellas Park for Braewood Development, Inc.

Design and inspection of Timber Pines interim treatment plant in Hernando County.

Designed and constructed an odor control system for headworks of the Tarpon Springs WWTP.

Designed force main for Buncess Pass Bridge crossing for Pinellas County.
Designed portion of effluent main system for Tarpon Springs connecting Howard Park to reuse system.

Sludge

Special consultant to City of Tallahassee in charge of a full-scale pilot testing of FKC Class A residuals treatment process.
Project manager for Fort Meade regional N-Viro Facility (950 wet tons/day).
Design engineer and construction manager for the redesign of the Tarpon Springs Residuals Handling Facilities.
Served as project manager for Pinellas County Sludge Management Plan.
Served as project engineer for Pasco County Sludge and Septage Management Plan.
Served as project manager for preparation of a market study for dried sludge in Florida.
Project engineer for analysis and preliminary layout of an indirect steam drying process to dry sludge in Pinellas County
Project manager for the pilot testing of centrifuge implementation at South Cross Bayou Plant in Pinellas County.
Project manager for general consulting services to BioGestor, Inc., a grease elimination system.

Industrial Pre-Treatment

Designed and constructed a dairy waste pre-treatment facility for SuperBrand Dairy in Plant City with a daily flow of 250,000-gpd.
Designed and constructed a food waste pre-treatment system for Winn Dixie Deep South Products in Altamonte Springs. System also included a separate grease degradation system capable of treating 10,000 gallons of grease per week.
Designed and built a stormwater treatment system for Coastal Casson utilizing a combination of filters and constructed wetlands.
Consultant for Lykes Brothers meat packing plant in Plant City.

Water

Engineer of record for construction of Coleman Public Water System including water plant, distribution system, and well.
Project manager for Fort White Public water system including treatment plant, transmission and water distribution mains, well field, water ordinance, and rate structure.
Project manager for City of Umatilla Master Water Plan.
Conducted design on potable water transmission and distribution system for City of Tampa Water Department.
Design engineer for over 10,000 feet of water main in City of Vernon.
Performed irrigation modeling and system design for Gateway Centre in Pinellas Park.
Evaluated water supply problems and designed a booster pump station for the Land's End development on Sunset Beach.
Performed layout and design for a 5.0-mg storage tank in Hillsborough County.
Design engineer for water plant improvements at two water plants in City of Mascotte.

Contract manager for emergency improvements to Horseshoe Beach Water Treatment Plant.

Stormwater

Designed and permitted numerous drainage projects in Pinellas County including municipal, commercial, and residential sites.
Conceptualized an innovative approach to polish treated leachate and detain surface water run-off from a landfill site utilizing an artificial wetland in Charlotte County.
Designed a pump station to supply cooling towers and to supply irrigation demand at Bridgeway Acres Landfill in Pinellas County.
Designed a stormwater pump station at the Tillman Ridge Landfill in St. Johns County.

Community Development Projects

Served as project manager for the following Community Development Block Grant projects:

City of Sopchoppy
Town of Branford
Town of Fort White
City of Trenton
City of Hampton
Town of Inglis
Town of Greenwood
Town of Astatula
City of Coleman
City of Lake Butler
Town of Lake Placid
City of Sebastian
City of Vernon
City of Mascotte
City of Newberry

Development

Project coordinator for the design and construction of SouthWood, a 3,300-acre master planned mixed-use development by Arvida in southeast Tallahassee, Florida.
Project manager for the Shared Resource Center for the state of Florida Department of Management Services in Tallahassee, Florida.
Project manager for Twin Oaks Apartments in Tallahassee, Florida.
Preparation of Whetstone Apartment Complex construction drawings; Roger Broderick, Inc., Pinellas Park.
Preparation of Longbranch Apartment Complex construction drawings; Roger Broderick, Inc., Pinellas Park.
Preparation of Courtesy Lincoln-Mercury Dealership construction drawings; The Petit Co., Hillsborough County.
Commercial development for Larson's Topsoil, Pinellas County; Larry Larson.
Design of Oakhurst Run Subdivision, Seminole, Florida
Commercial Expansion of Peps SeaGrill 4th Street, St. Petersburg.

Project manager for Skybolt manufacturing facility at Leesburg Airport.
Project manager for Fire Station, Inglis, Florida.
Project manager for community center, Inglis, Florida.
Project engineer for Roger's Mobile Home Park in Pinellas Park.

Miscellaneous

Assisted the City of Tallahassee, Utilities Department with analysis of biosolids treatment and marketing alternatives.
Developed initial plan and project outline for Pinellas Park Equestrian Trails.
Provided expert witness testimony for Hy-Com Development regarding wastewater treatment facility.
Project manager for City of Vernon Sportsplex.
City Engineer for City of Vernon and City of Branford.

Publications

1996 Disinfection Survey, Water Environment Federation, 1997 National Conference, Chicago, Illinois.

1993 Disinfection Survey, Water Environment Federation, 1994 National Conference, Chicago, Illinois.

Re-Use of Residuals, Florida Pollution Control Association, 1992 Specialty Conference, Tampa, Florida.

Innovation Site Plans for Affordable Housing, Tampa Bay Regional Planning Council, 1985, Workshop, Tampa, Florida.

Professional Affiliations

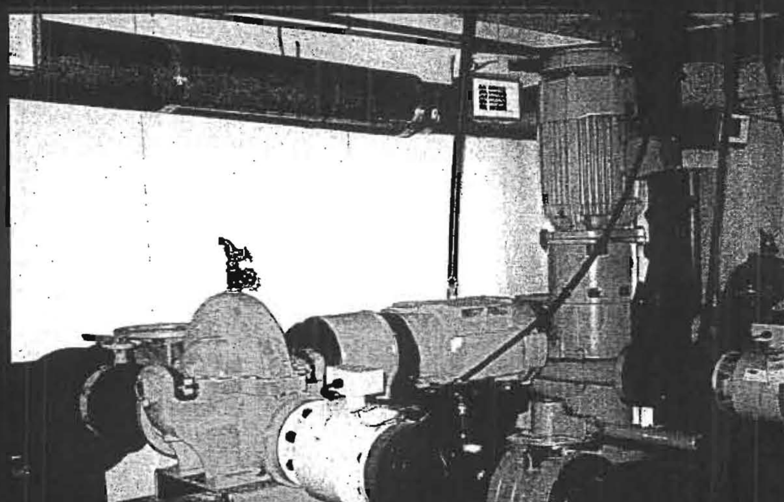
Water Environment Federation (WEF)

EXHIBIT (MS-2) ______ IS THE PBS&J
ST. GEORGE ISLAND WATER SYSTEM EVALUATION
FINAL REPORT

St. George Island Water System Evaluation

Final Report

Docket No. 100104-WU
Michael Scibelli, Exhibit MS-2



Water Management Services Inc.

PBS&J

ST. GEORGE ISLAND

WATER SYSTEM EVALUATION

FINAL REPORT

WATER MANAGEMENT SERVICES, INC.

PBSJ
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BLDG C
TALLAHASSEE, FL 32303

ST. GEORGE ISLAND

WATER SYSTEM EVALUATION

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EXECUTIVE SUMMARY

Water Management Services Incorporated (WMSI) is a private water utility serving as the sole source drinking water provider for St. George Island since 1974. The water system provides service to approximately 1800 customers. The water supply consists of four (4) water wells located on the mainland in the town of Eastpoint, Florida. Water is transported from the mainland wells to St. George Island via a 12-inch pipeline. Water is processed on St. George Island by aerating and chlorinating prior to pumping into the distribution system.

PBS&J prepared an evaluation of the water system and its operation to assess the overall system, and identify needed priority capital improvements to ensure long-term viability and reliability of the system. Our evaluation of the water system included a detailed review and assessment of the following major system components:

- Raw Water Transmission Main.
- Capacity Assessment to identify the limiting capacity component in the system.
- Source Water Supply evaluation to assess adequacy vulnerability and weakness.
- Assessment of Water Plant Process, overall condition and review of current operation.
- Structural evaluation of the water plant with determination of a need to perform repairs/remediation or replacement.
- Review electrical systems and controls at the water facility and each individual well to determine adequacy and general condition.
- Water Distribution Operation, maintenance and review water quality parameters.

Technical memorandums (TM's) were developed with discussion of findings including recommendations for improvements. Estimated probable costs for the recommended improvement are included in the various TM's. Priority Water System improvements are described in the following paragraphs. PBS&J's recommendations established a priority Capital Improvements Project list (CIP) valued near $2,200,000. Below is a summary of the priority CIP improvements.

Raw Water Transmission Improvements: The 12-inch raw water pipeline is susceptible to catastrophic failure as portions of the pipeline are unprotected within the bay as a result of wave erosion. Should large debris or boaters rupture the exposed pipeline, all potable water supplies to St. George Island would be compromised. PBS&J recommends relocation of approximately 2300 linear feet of pipeline to the west side of the bridge and within FDOT right-of-way on St. George Island for an estimated construction cost of $156,156.00.

The recommended relocation of a segment of the raw water line to the west side of the bridge will provide the following benefits:

- Improved system reliability with the new pipeline.
- Improved accessibility, to perform service and repairs.
- Relocation to the west side of the bridge provides the transmission main with an increased level of protection against storm and wave erosion.

Capacity Assessment:

WMSI is experiencing declining water sales in combination with decline of active customer connections. On January 4, 2010, the Northwest Florida Management District adopted new rules eliminating any permitting requirements for new wells on St. George Island. As a result of new well construction, WMSI anticipates a further reduction of potable water sales with subsequent decline in water revenues. With the current economic down turn, housing sales and tourism on St. George Island is down, resulting in further reduction in water revenues.

The Capacity Assessment utilized an ERC based approach to identify limiting capacity components within the water system. The estimated ERC value under current conditions is 1403.

Our Capacity Assessment examined regressive growth conditions, assumed growth conditions at 3% per year without supplemental wells and assumed 3% growth conditions including installation of shallow irrigation wells. Under the regressive ERC condition, the capacities of water system components are adequate for the next five years. However, adequate does not imply a lack of need to incorporate system improvements. Should growth return to the island with/or without shallow wells, the limiting capacity components are prioritized below:

- Distribution system -1873 ERC's
- Permitted aquifer withdrawal -2361 ERC's
- Treatment plant capacity-2742 ERC's

Because of the economic recession in collaboration with the ability to install shallow wells for irrigation the following is recommended,

1) Implement a priority CIP program to improve operation and maintenance of the current water system to improve reliability. Program should concentrate on source water improvements, treatment plant, structures, electrical systems and the distribution system.
2) Update the Capacity Assessment in 5 years to better define future needs and improvements.
3) Maximize operational performance to lower operational costs by implementing improved water distribution chlorination and flushing procedures described in TM 7- Water Distribution System.

Source Water Supply Evaluation:

In TM 3, Source Water Supply Evaluation, PBS&J examined well water quality data provided by WMSI which revealed the current source water quality from the Floridian Aquifer is good. WMSI is operating at 58% of the annual average daily flow (ADD) permitted. Previous studies referenced in this report, identified wells 1 & 2 as susceptible or vulnerable to underground contamination. Monitoring of water quality at wells1&2 will identify, if/or when, a fifth well would need to be installed. At this time a new well is not required and is not included in the CIP plan.

In addition to water quality the source water supply evaluation, we examined the conditions of the well houses, documented security and building improvements suggested at these well houses, identified instrumentation and flow metering improvements and examined the emergency power generators. We recommend the priority CIP improvements include repairs to well 3 generator and installation of a new generator at well 4. SCADA controls for the well houses are presented later..

Water Plant Process and Structural Evaluations:

TM 4 examined the water plant process. WMSI water treatment plant consists of tray aerators, ground storage tank, chlorine systems and high service pumps. The plant has a permitted capacity of 0.714 million gallons per day, and a maximum single day capacity of 1.240 mgd. The process includes addition of chlorine at the tray aerators to assist in the removal of hydrogen sulfide and other gases from the well water. A separate chlorination system is utilized for disinfection. The process components of the water plant and its operation are consistent with similar facilities processing groundwater. Even though one tray aerator was installed in 1987 and its condition is fair, the unit is near the end of its useful life assuming a typical equipment life of 20- 25 years. The second tray aerator installed in 2001 is in good condition.

TM 5 includes a structural evaluation of the exiting water plant process buildings. Of major concern is the condition of the existing ground storage tank constructed in 1975. There is visible evidence of leakage at precast panel joints in the sidewalls, cracking of perimeter structural beams and deterioration of the hollow core panels comprising the roof. The water tightness of the roofing material installed over the hollow core roof panels is also questionable. Our structural evaluation identified exposed reinforcing wire, and corrosion both of which are significant factors relating to service life of the structure, Our structural evaluation did not include a review of the interior of the tank, and for the purposes of our report, we have referenced an independent review of the interior performed by Brice Nist, P.E. in 2009 concluding the interior top portions of the walls and entire interior roof surface showed significant signs of concrete corrosion and indications of long term roof seepage of rainwater into the ground storage tank. PBS&J's evaluation of the ground storage tank in conjunction with the independent structural review re-affirms the ground water tank condition is structurally deteriorating and as a result the remaining service life is questionable. Hidden items may exist that are not visible and these items may affect the ability to cost effectively rehabilitate the tank. Without a more indebt review of the structural integrity using core samples and structural analysis of the strength lost due to corrosion, we cannot eliminate the possibility of a catastrophic failure.

With the information at hand, PBS&J evaluated options to either rehabilitate the ground storage tank or construct a new tank and as a result of the evaluation we recommend construction of a new Crom Style 325,000 gallon storage tank with new high service pumping equipment/electrical gear located on the roof of the tank to protect this equipment from flooding during a storm surge or 100 year flood. The tray aerators will be located on top of the Crom tank. We anticipate relocating one of the existing aerators (installed in 2001) and installing a new aerator of similar size to the existing.

We arrived at the conclusion to install a new tank recognizing the deteriorating condition of the ground storage tank and the potential for unforeseen issues and costs associated related to its rehabilitation, and considering the cost to move the existing high service pumps and electrical gear out of the flood zone, with all of the associated MOPO (maintenance of plant operation during construction) issues related to maintaining water service during tank rehabilitation and equipment relocation, we recommend the construction of a new ground storage tank with new pumping, one new aerator and relocation of a second aerator and electrical gear. With construction of a new tank on adjacent property, the MOPO issues relating to continued water service are minimized; pumping equipment and electrical gear is protected above the flood elevations and new tank construction reduces risk typically associated with rehabilitation projects.

Four empty lots at the intersection of West Pine Avenue and 2nd Street West (adjacent to the existing ground storage current location) have been identified as the location for the new ground storage tank. The estimated cost of the property including closing cost is $450,000.

Components of the water plant include the following:

- Multiple tray aerators mounted atop GST
- Crom style concrete with a capacity of 325,000 gallon providing a 50-year operational life and constructed to current standards.
- Controlled access/ egress to improve security at the GST.
- To improve operations flexibility, the GST will include dual chambers design to allow cleaning of one chamber and concurrent operation of the second chamber providing water to St. George Island.
- Clearwell baffling to improve disinfection efficiency..
- New VFD vertical turbine high service pumps will be installed on top of GST and above storm surge elevation to improve overall reliability of operation.
- Upgrading of the chlorination system to provide up to 200 pounds/day feed capabilities for disinfection.
- Provide automatic chlorine pacing by rate of flow to optimize chemical usage and minimize the potential for production of Disinfection By-Products.
- Relocation of a generator with fuel containment

Facilities Electrical Assessment:

The water treatment facility houses a master programmable logic controller (PLC) that controls the operation of the high service pumps and the four remote well sites and monitors the tank levels at both GST and the elevated tank. The controller is vulnerable to failure and requires replacement as a priority CIP project. Additional priority electrical improvements include the installation of new SCADA and RTU's at all wells, repairs to the Generator at well No. 3 and the replacement of the generator at well No. 4. The estimated construction cost for the priority improvements is $337,700.

Replacement of the master control panels and upgrading the SCADA will provide the following benefits:

- Eliminate out dated and substandard equipment
- All well houses will be fitted with similar equipment and controls
- Generator repairs and replace will improve the reliability of operations
- The operators to remotely change control parameters

Water Distribution System Evaluation:

TM-7 includes a review of the water distribution system and its operation. Several aspects regarding operation were reviewed to evaluate Disinfection Residual Maintenance, Disinfectant By-Products, Lead and Copper Levels, System Flushing Water Loss and various maintenance programs associated with valves and hydrants. From our evaluation we identified priority CIP improvements which include the installation of a chlorine chart recorder on the high service distribution line, the incorporation of an in-line chlorine analyzer or probe device to continuously measure the chlorine residual of water entering the distribution system and the purchase of at least one portable leak detection device to assist in locating distribution system leaks for repairs thereby reducing the volume of unaccounted for water.

The chlorine residual at opposite ends of the island and at dead ends in the distribution system is difficult to maintain. The State park has very low flow demands, and to achieve a residual, flushing the distribution system is required on a regular basis to bring freshly chlorinated water to the park. This flushing procedure, when employed, results in providing the required result but at a cost of wasting finished water, as well as electricity from high service pumps and chlorine. It is very likely the investment and maintenance of a booster disinfection system would reduce flushing requirements by rechlorinating near the end of the system. A more detailed evaluation will be needed to identity the exact location(s) of chlorine booster pump station(s) and to determine the cost effectiveness of using a chlorine booster station versus flushing. The incorporation of 2-inch diameter continuous blow-offs at dead ends may help to maintain better water quality at distal ends of the distribution system, but this practice may result in using large qualities of water. A more effective means is looping dead-ends to improve circulation in the water distribution system. Because the process of flushing is currently working, we have not identified the installation of a chlorine booster station(s) as a priority issue for funding in a CIP.

To improve the flushing program, we recommend WMSI incorporate into their operation, an enhanced flushing program using unidirectional flushing techniques. A unidirectional flushing program isolates areas of the system, and enables progressive flushing in a single direction at a high velocity to remove biofilm and corrosion products while improving overall water quality.

Using the 2009 data included in the, WMSI Annual Report for water pumped, flushed and purchases, indicates a water loss of 9.17 %, with flushing volumes not included in the calculation.

We suggest revising Section 26 of WMSI Water Tariff, identifying maximum flow and maximum pressure loss for water service meters. By doing such this will provide WMSI justification for meter replacement to an appropriate size, if flow and pressure values in the Tariff are exceeded. This change in the Tariff will improve the service meter accuracy and revenues.

Lead, copper and disinfection by-products in the finished water are within prescribed limits at this time.

Synopsis

PBS&J's evaluation of the St. George Island Water System identified several potential Capital Improvement Projects. We narrowed the list of potential projects down to a priority list that we believe, will maximize the return on investment by improving the overall reliability, functionality and cost effectives of processing and selling water to the customers of St. George Island.

These priority projects have an estimated construction cost of $2,202,481. The following table identifies the priority projects and PBS&J's final budgetary cost breakdown to construct.

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St. George Island Water System Improvements		
PRIORITY ITEM	DESCRITPION	ESTIMATED COST
Raw water transmission line	12- inch water main, pvc	$70,000
	12 -inch gate valves	$6,800
	well point dewatering	$10,000
	tie-in connections	$10,000
	Mobilization, site work, permitting	$14,380
	Contractors bond, insurance	$1,936
	Contractors OH&P	$9,680
	Contingency	$19,360
	Engineering	$14,000
	total	$156,156
Plant Improvements	Chlorine system manifolding	$500
	Repalce cylinder mounted chlorinators	$2,500
	Clearwell baffling	$15,000
	Chlorine diffuser	$4,000
	High service pumps	$100,000
	Generator relocation	$7,500
	Generator fuel containment	$3,000
	Pumping and plant controls	$93,500
	Ground storage tank installation	$389,000
	Ground storge tank	$326,000
	Engineering	$61,500
	Mobilization, site work, permitting	$24,525
	Electrical	$61,500
	Contingency	$12,300
	Yard piping	$61,500
	Contractors bond, insurance	$12,300
	Contactors OH&P	$61,500
	subtotal	$1,236,125
	GST Property and closing costs	$450,000
	total	$1,686,125
Electrical System Replacement/Rehabiltation	SCADA/RTU contols for wells 1-4	$252,000
	Well 3 genereator repairs	$21,700
	Well 4 new generator	$64,000
	total	$337,700
Distribution System	Chlorine chart recorder	$7,000
	Chlorine probe	$7,500
	Portable leak detection equipment	$8,000
	total	$22,500
	Grand total	**$2,202,481**

END

PBSJ

Technical Memorandum 1
Raw Water Transmission Main Corrective Action

To: Mr. Gene Brown, WMS

From: Desi Maldonado | Project: Water Management Services,

CC: Mike Scibelli

Date: October 29,2009
Finalized : June 1, 2010 | Job Number:100010111

1.0 Executive Summary

Water Management Service (WMS), is a private water utility which has served potable water to St. George Island since 1974 under a franchise from the Florida public service commission. The water system serves approximately 1,800 service connections, including single family homes, multifamily units, and hotel/motel units, commercial structures, and public authority accounts. Since there are year-round residents, the water system on St. George Island is vital. Should a catastrophic event occur to the raw water transmission pipeline, there are no alternative means in place for providing the island with water supply other than by emergency bulk transfer tanker, and a limited number of shallow wells.

The entire water supply source for Water Management Services, Inc. consists of four (4) water wells, all of which are located on the mainland in the town of Eastpoint, Florida and interconnected prior to leaving the mainland. Water is transferred to St. George via a common 12-inch diameter ductile iron water main hung from the St. George Island Bridge. When the bridge lands, on St. George Island, the transmission main descends underground running eastward, towards the bay. Once at the bay, the pipeline is fitted with a 12-inch gate valve (partial exposed) and serves as the primary isolation valve, before running north within the bay totally unprotected. The transmission line running through the bay is vulnerable to failure, because of exposure to floating debris or boaters. <u>A failure of the transmission line will result in total interruption of service for the water system.</u>

Work was completed on the 12-inch transmission line in 2000. At the time of construction and at FDOT specific direction and not per construction plans, the transmission line was installed as far east as possible within the right- of -way to accommodate bridge construction on St. George Island. Since construction, the right -of -way has eroded leaving the water transmission main exposed. Recently, emergency repairs to support the pipeline in the bay had to be performed.

This Technical Memorandum (TM) developed four alternatives for mitigating transmission line vulnerability to failure. Of the alternatives evaluated, PBS&J recommends relocation of the transmission line to the west side of the bridge, and within the right -of -way, providing the transmission main with additional protection from waves and coastal erosion thus improving overall system reliability for an estimated total cost of $156,156.

2.0 Assessment of Current Conditions

The 12-inch water transmission main attached to the St. George Island Bridge was placed into service in 2004. **Figures 2.1** and **2.2** illustrates the transmission pipeline as it transitions from the bridge at St. George Island. A site inspection of water transmission pipeline was performed by PBS&J on August 27, 2009. The investigation was entirely visual, to evaluate the overall condition, configuration, and susceptibility to damage.

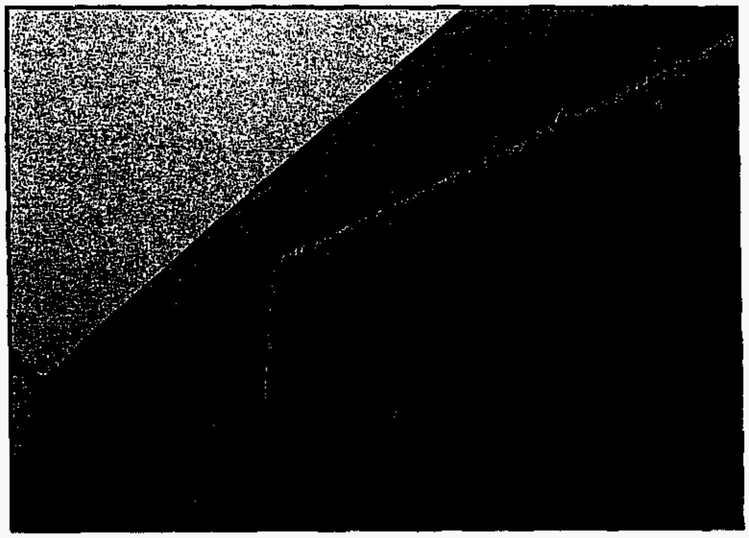

FIGURE 2.1 - Transmission pipeline on bridge at St. George Island

FIGURE 2.2 - Transmission main near access roadway

Table 2.1 provides assessment information of the pipeline from the site investigation.

TABLE 2.1
Raw water transmission main on St. George Island

Item or Description	Condition	Comments
Isolation valve condition	12-inch gate valve bonnet exposed to salt-water environment. Significant signs of corrosion	Priority item to correct
Site accessibility to isolation valve(s)	12-inch gate valve located within a field of rip rap	Accessibility to operate the valve is difficult. Valve should be accessible and exercised on at least once per year on a regular basis.
Exposed C-900 PVC	Pipeline section exposed in the bay	C-900 PVC pipe does not contain ultraviolet light resistance compounds. PVC becomes brittle and may fail with extended exposure to UV light.
Flooded raw water main	Water supply within the transmission main could become contaminated with sea water in the event of a transmission main pressure drop	Priority item to correct to reduce the sanitary risk to customers
Transmission main protection in the bay	Segment of pipe in bay is visible with zero protection against boaters or debris creating a vulnerability hazard to the water system.	Priority item to correct to maintain reliable service
Pipeline protection at bridge	Pipe protection at transition from bridge to underground	Rip Rap material and size is acceptable
Pipeline coatings	Evidence of corrosion	Regular repainting of pipe is needed for control corrosion
Raw water interconnection to another system	Does not exist and represents a significant deficiency	If intermittent failures of the raw water transmission main begin to occur, consideration for a parallel raw water main should be a consideration
Emergency repairs	Not aware that a contract exists to repair large valves or pipeline in event of emergency condition.	Priority item to correct

Figure 2.3 illustrates the current routing of the raw water transmission main as it enters onto St. George Island.

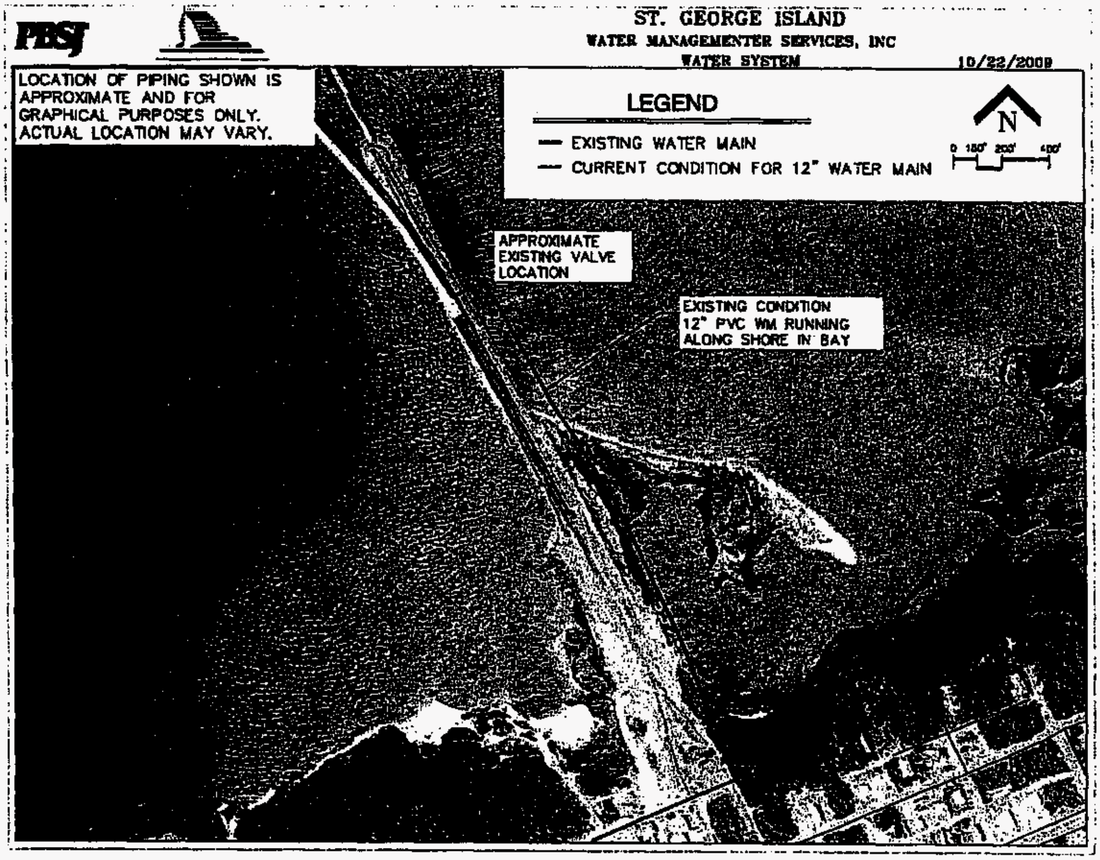

Figure 2.3 – Current Configuration

3.0 Raw Water Transmission Main Corrective Action Alternatives

The following corrective action alternatives have been developed. Alternatives include:

- Alternative 1: Relocate Water Main Within FDOT Right of Way
- Alternative 2: Add Rip Rap Around Existing Water Transmission Main
- Alternative 3: Extend pipeline along bridge retaining wall
- Alternative 4: Relocate Water Main Within FDOT Right of Way of west side of bridge

4.0 Evaluation of Alternatives

The following is discussion and evaluation of the alternatives.

Alternative 1: Relocate Water Main within FDOT Right of Way on East side of Bridge

This alternative requires the water transmission main and gate valve (for isolation purposes) be relocated onto the Florida Department of Transportation (FDOT) Right-Of-Way (ROW) alongside the east access road paralleling the bridge, with subsequent abandonment of the transmission line running east and south in the bay. Moving the water main from the bay to an underground location away from the shoreline protects the main and addresses several concerns noted in **Table 2.1.** The relocation of the transmission main inside the FDOT right of way provides a location with easy access for pipeline repairs and provides the WMS staff the necessary accessibility to service and exercise the isolation gate valve on a regular basis. This alternative would incur construction labor and materials costs to install segments of new 12-inch PVC water main with 12-inch gate valve. In addition, design services will be required to survey, design and permit the new transmission pipeline. Right-of-way and maintenance of traffic permits would likely be required prior to any construction activities within FDOT's ROW and alongside their roadway. Refer to **Figure 4.1** for schematic of Alternative 1.

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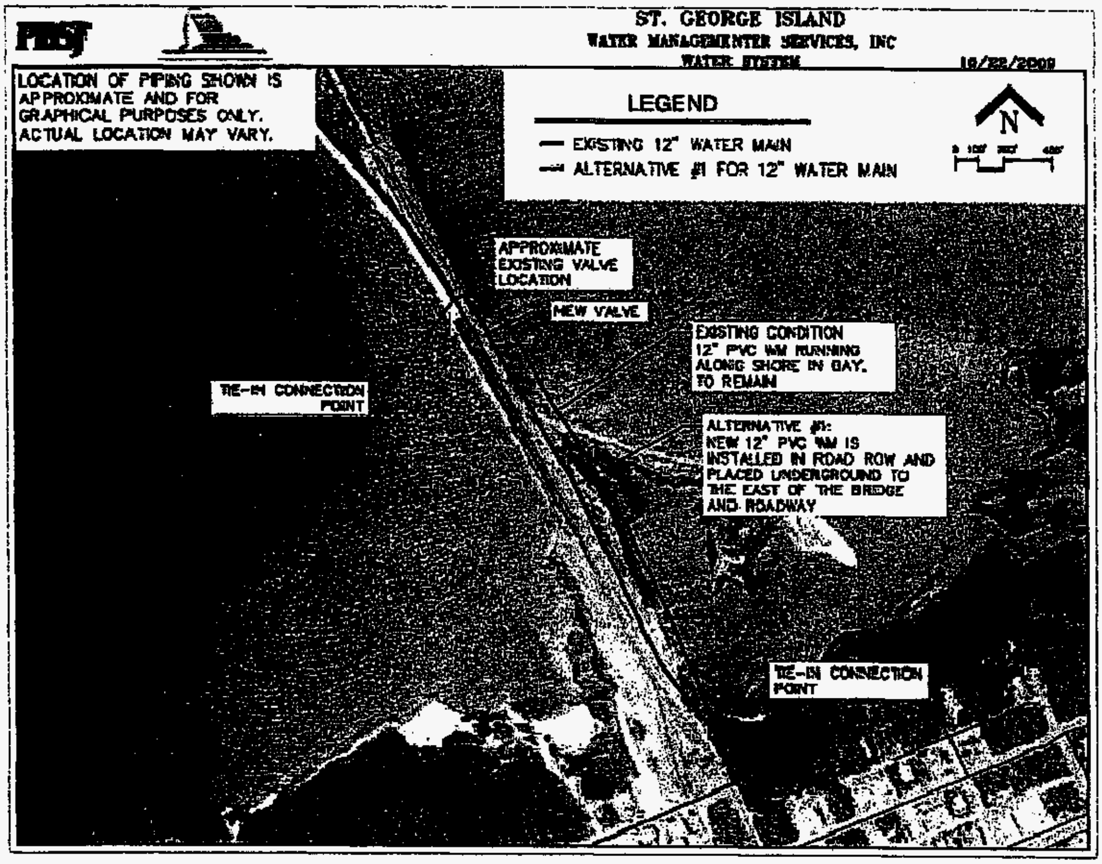

Figure 4.1 - Alternative 1

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Alternative 2: Install Rip Rap to Protect Existing Water Main

Alternative 2 assumes the PVC transmission pipe and routing will be maintained along the bay. To provide pipe protection, the C-900 PVC pipe currently within the bay would be surrounded by hand placed riprap revetment on the bayside. We have assumed additional pipe bedding material consisting of gravel bedding will be required under the C-900 PVC piping to insure the water transmission pipeline will not collapse due to the weight of the rip -rap when placed against the pipe. This option would offer increased protection of the PVC water main from boats and debris in the bay and would likely have a higher cost of construction versus relocating the water main. This alternative would not address the difficulties in accessing the transmission water main or isolation gate valve for maintenance or servicing purposes. As this work is within the bay, we believe permitting will be required from the Florida Department of Environmental Protection (FDEP) or other agencies allowing placement of a riprap revetment along the shoreline. If permitting is required, a survey may also be required. See **Figure 4.2** presents a schematic of Alternative 2.

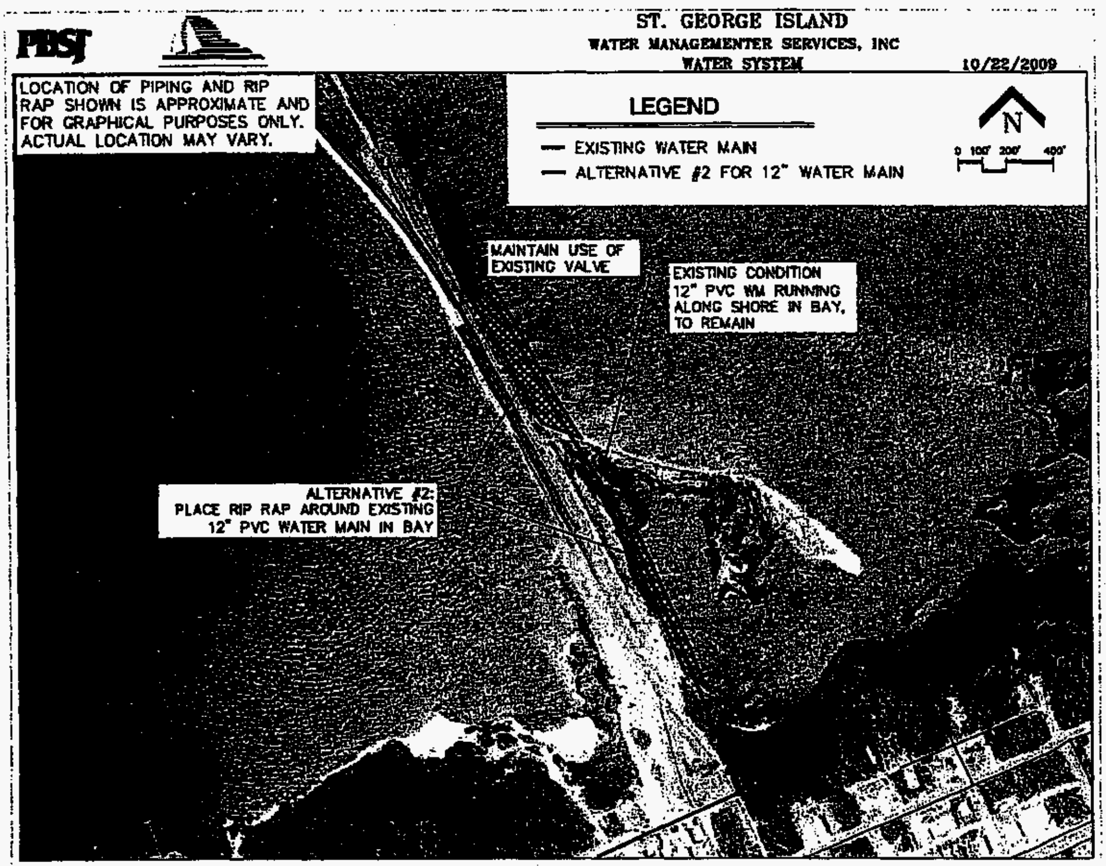

FIGURE 4.2 - Alternative 2

Alternative 3: Extend pipeline along bridge retaining wall

This alternative proposes installation of new 12-inch ductile iron pipeline with mounting of the transmission main from the bridge retaining wall and subsequent abandonment of the transmission line running east and south in the bay. Moving the water main above ground and away from the shoreline will address several concerns presented in **Table 2.1**. This alternative will incur construction labor and materials costs to install segments of new 12-inch ductile iron water main including the design services required to survey, design and permit the new transmission pipeline. Right-of-way and maintenance of traffic permits would likely be required prior to any construction activities within FDOT's ROW and alongside their roadway. Refer to **Figure 4.3** for presentation of Alternative 3. Additional piping will be exposed to the weather and additional operations and maintenance costs will be incurred should this alternative be selected.

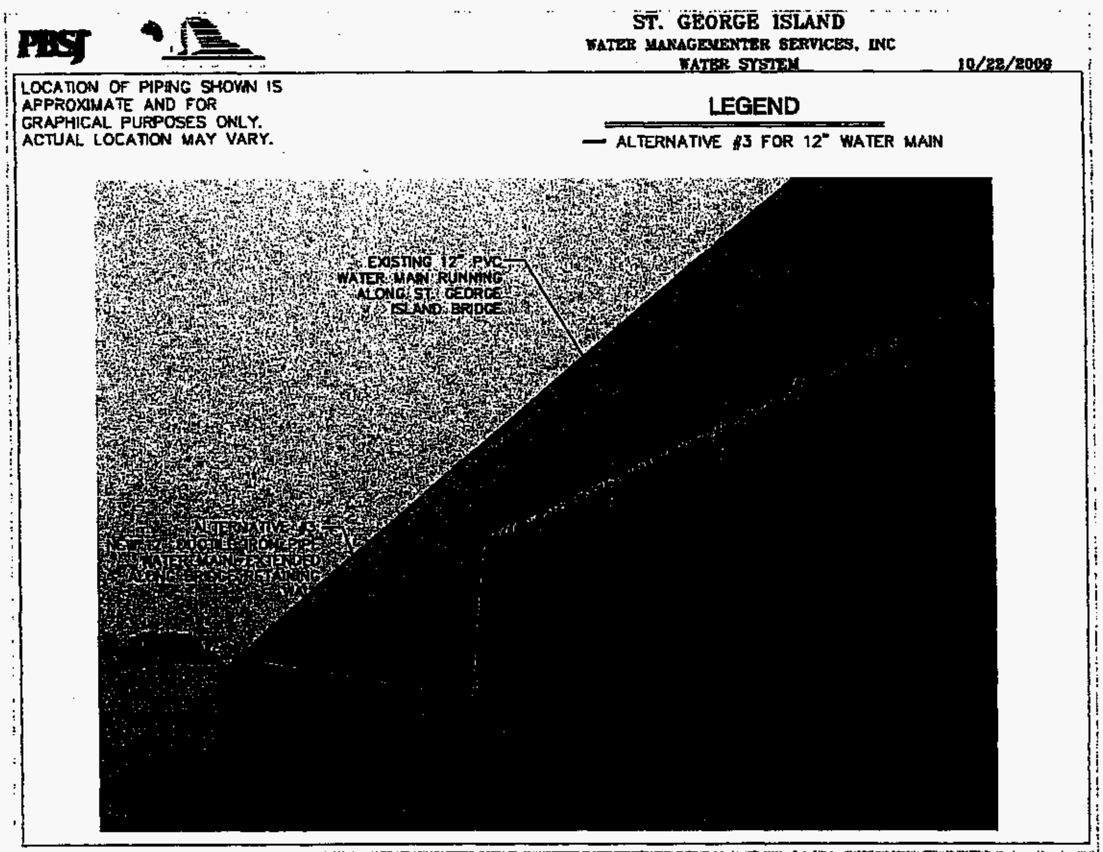

FIGURE 4.3 - Alternative 3

Alternative 4: Relocate Water Main within FDOT Right of Way on West side of Bridge

This alternative requires the water transmission main and gate valve (for isolation purposes) be relocated onto the Florida Department of Transportation (FDOT) Right-Of-Way (ROW) alongside the access road paralleling the bridge on the west side. The existing transmission line near the bay shall remain intact and isolated from use. Moving the water main underground and away from the shoreline protects the main and addresses several concerns noted in **Table 2.1**, and being on the west side of the bridge adds additional protection from erosion due to storms and waves approaching from coastal side of the island. The relocation of the transmission main inside the FDOT right of way provides a location with easy access for repairs and allows WMS staff the ability to service and exercise isolation gate valve on a regular basis. This alternative would incur construction labor and materials costs to install segments of new 12-inch PVC water main with 12-inch gate valve. In addition, design services will be required to survey, design and permit the new transmission pipeline. Right-of-way and maintenance of traffic permits would likely be required prior to any construction activities within FDOT's ROW and alongside their roadway. Refer to **Figure 4.4** for schematic of Alternative 4.

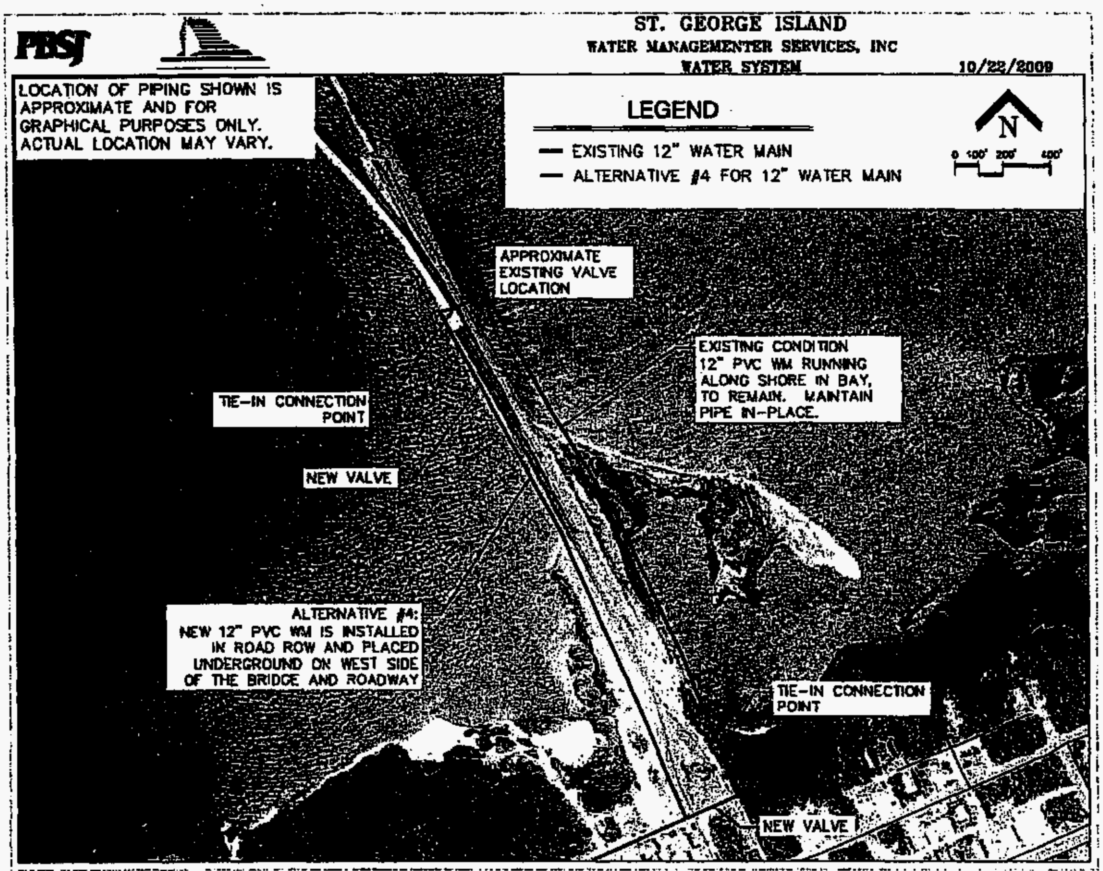

Figure 4.4 - Alternative 4

Table 4.1 Advantages and Disadvantages of Alternatives

Advantages and Disadvantages of Alternatives		
Alternative	**Advantages**	**Disadvantages**
1: Relocate Water Main within FDOT Right of Way	▪ Water main is protected ▪ Accessibility is improved to valves and pipelines for servicing ▪ Transmission main no longer flooded and reduces sanitary risk	▪ Abandon existing infrastructure with a cost to provide this alternative ▪ Construction may require installation of well points for dewatering
2: Install Rip Rap to Protect Existing Water Main	▪ Water main is protected	▪ Pipe and Valve remain inaccessible ▪ Pipeline remains flooded ▪ Existing PVC Pipe has become brittle due to years of exposure. Weight of rip rap could cause failure
3: Extend Pipeline along Bridge Retaining Wall	▪ Pipeline is protected from flooding and reduces sanitary risk ▪ Direct mounting of pipe eliminates need for construction well points ▪ Enhanced accessibility for servicing pipe	▪ Painting of the pipeline will be required to control corrosion. ▪ Abandon existing infrastructure with a cost to provide this alternative
4: Relocate Water Main within FDOT Right of Way on West side of Bridge	▪ Accessibility is improved to valves and pipelines for servicing ▪ Transmission main no longer flooded ▪ Added protection from waves and coastal erosion being on west side of bridge	▪ Abandon existing infrastructure with a cost to provide this alternative ▪ Construction may require installation of well points for dewatering

5.0 Economic Analysis of Alternatives

Economic analyses were performed for the alternatives. Estimates of probable construction cost were calculated for each alternative. The construction costs presented for the alternatives, shown in **Table 5.1**, are in 2009 dollars. All costs include overhead, profit, mobilization and contingency costs. **Table 5.2** provides a detailed breakdown of the estimated probable cost for the project including engineering.

Table 5.1 Alternatives Construction Cost Estimates

	Alternative 1: Relocate Water Main within FDOT Right of Way east side of bridge	**Alternative 2: Install riprap to protect the existing**	**Alternative 3: Extend Pipeline along Bridge Retaining Wall**	**Alternative 4: Relocate Water Main within FDOT ROW west side of bridge**
Total Construction Cost	$148,556	$234,033	$157,876	$156,156
Rank by Cost	1	4	3	2

Alternative 1 will require installation of approximately 2,300 linear feet of 12-inch C-900 PVC pipe paralleling the existing east access road. This alternative assumes the new PVC pipeline will have a tie-in connection near the bridge and at the opposite end by connecting to the existing water transmission pipeline supplying water to the water treatment plant. The estimated cost for construction includes the labor and material for trenching the entire segment of pipe. Well point dewatering may be required to allow for proper pipe installation. New 12-inch gate valve(s) and valve boxes have been included in the construction cost estimate, as it may be required to isolate the new segment of piping from the existing length of piping. Salvaging the existing 12-inch gate valve for reinstallation was not considered, as a method to control capital expenditure for this alternative, as the operational condition of the valve is questionable. To control costs, we recommend abandoning the existing transmission line in place. If the existing 12-inch PVC piping is removed, it may require a FDEP dredge and fill permit. To avoid this requirement, the existing piping would be cut off, capped, and then abandoned in place. Careful coordination during construction will be required with final tie-in procedures. To accomplish the final tie–in with minimum water system down time will require two construction crews working, simultaneously to accomplish all of the work. We estimate the tie-ins will take four hours to complete. Pressure testing of large segments of the new line along and chlorine swabbing of the pipeline would be performed in advance of final connections. Pressure testing of final connections will be self -evident when flow to the pipeline is re-established. The cost for removing the 12-inch PVC piping has not been included in our detailed estimate. Alternative 1 will require additional costs for permitting from FDOT as any work done in FDOT ROW requires a permit. Permit requirements may include a signed and sealed survey, as well as possibly plans signed and sealed by a professional engineer. They will likely require as-built plans signed and sealed by a professional engineer. Since the work may be done near the base of the bridge's landing on St. George Island, an FDOT inspector may also be required as part of the permit conditions. Part of the limits of construction may obstruct traffic, which will likely require maintenance of traffic (MOT) plan.

The construction cost estimate for Alternative 2 is approximately $213,000 to construct a small revetment of large stone rip- rap and covering the area where the 12-inch PVC water main is currently located. Additional costs have been included to install bedding stone under the pipeline to stabilize and prevent collapse due to additional loading resulting from the riprap addition. Alternative 2 will require, at minimum, a dredge and fill permit from FDEP, and possibly a coastal armoring permit for the revetment construction for the riprap placement along the shore. Since there are no inhabited structures nearby, this may not be necessary.

Alternative 3 is similar to Alternative 1; however, ductile iron piping and support attachments will be used to extend the transmission main above ground until the end of the bridge retaining wall is reached. At this point, the ductile iron pipe will transition underground and connect to the existing water transmission pipeline supplying water to the water treatment plant. To control costs, we recommend abandoning the existing transmission line in place. If the existing 12-inch PVC piping is removed, it may require a FDEP dredge and fill permit. As with alternative 1, careful coordination during construction will be required with final tie-in procedures. To accomplish the final tie--in will require two construction crews working, simultaneously to accomplish all of the work in a four hour time period. Pressure testing of large segments of exposed piping along and chlorine swabbing inside the pipeline would be performed in advance of final connections. Pressure testing of final connections will be self -evident when flow to the pipeline is re-established. The cost for removing the 12-inch PVC piping has not been included in our detailed estimate. Alternative 3 will require additional costs for permitting from FDOT as any work done in FDOT ROW requires a permit. Permit requirements may include a signed and sealed survey, as well as possibly plans signed and sealed by a professional engineer. They will likely require as-built plans signed and sealed by a professional engineer. Since the work may be done at the base of the bridge's landing on St. George Island, an FDOT inspector will be required as part of the permit conditions. Part of the limits of construction may obstruct traffic, which will likely require maintenance of traffic (MOT) plan.

Alternative 4 is very similar to alternative 1 with the exception of maintaining the existing PVC transmission line in place for use as a redundant line. Alternative 4 provides the benefit of additional pipeline protect resulting from its location relative to the causeway. In the event of a storm from the northeast, alternative 4 alignment provides a greater level of protection from catastrophic weather events, with a higher increase in capital expenditure than alternative 1.

Table 5.2 presents total project cost estimates for each alternative.

Table 5.2 Estimates of Probable Project Cost

DESCRIPTION	ALTERNATIVE 1: New Water Main within FDOT Right of Way on East side of Roadway	ALTERNATIVE 2: Install Rip Rap to Protect Existing Water Main	ALTERNATIVE 3: Extend Pipeline Along Bridge Retaining Wall on East side of Bridge	ALTERNATIVE 4: New Water Main within FDOT Right of Way on West side of Roadway
12" Water Main (PVC)	$65,000.00	$0.00	$33,500.00	$70,000.00
12" Gate Valves	$6,800.00	$0.00	$6,800.00	$6,800.00
12" Water Main (Ductile Iron and Fittings)	$0.00	$0.00	$38,000.00	$0.00
Rip Rap and Bedding Stone	$0.00	$144,900.00	$0.00	$0.00
Well Point Dewatering	$10,000.00	$0.00	$0.00	$10,000.00
Structural Supports	$0.00	$0.00	$9,500.00	$0.00
Tie-in Connections	$10,000.00	$0.00	$10,000.00	$10,000.00
Subtotal	**$91,800.00**	**$144,900.00**	**$97,800.00**	**$96,800.00**
Mobilization (10%)	$9,180.00	$14,490.00	$9,780.00	$9,680.00
Site Work	$500.00	$800.00	$500.00	$500.00
Contingency (20%)	$18,360.00	$28,980.00	$19,560.00	$19,360.00
Contractor's Bond and Insurance (2%)	$1,836.00	$2,898.00	$1,956.00	$1,936.00
Contractor's Overhead and Profit (10%)	$9,180.00	$14,490.00	$9,780.00	$9,680.00
Permitting	$4,200.00	$6,200.00	$4,200.00	$4,200.00
Engineering (10%)	$13,500.00	$21,275.00	$14,300.00	$14,000.00
Estimated Project Total	**$148,556.00**	**$234,033.00**	**$157,876.00**	**$156,156.00**

6.0 Recommend Alternative

Based upon review of the economic analysis and review of the advantages and disadvantages for each of the alternatives, we recommended Alternative 4, relocating the water transmission main within the FDOT right-of-way and to the west side of the bridge. This alternative provides the following benefits over the others.

- The alternative when implemented will provide improved access to the pipeline for emergency repairs and/or servicing of the transmission line and valves.
- The alternative mitigates the pipeline flooding and the potential for subsequent water contamination resulting from a pressure loss.
- Provides added pipe protection during a significant weather event.
- Offers use the PVC pipeline line to the east of the bridge as a redundant transmission line, if needed.

END

PBSJ

Technical Memorandum 2
Capacity Assessment

To: Mr. Gene Brown, WMS	
From: David Gauker	Project: Water Management Services, Inc
CC: Mike Scibelli	
Date: February 16, 2010 Final: June 23,2010	Job Number: 100010111

1.0 Executive Summary

WMSI is experiencing declining water sales in combination with a decline of active customer connections over the past several years. Beginning in 2007, WMSI identified several illegal wells in the Plantation, used for irrigation. The use of the illegal wells has contributed to lower water sales. On January 4, 2010, the Northwest Florida Water Management District adopted new rules eliminating permitting requirements for wells on the St. George Island. As a result of this ruling, WMSI anticipates an increase the number of shallow irrigation wells, resulting in a further reduction in potable water sales.

The Capacity Assessment projects a continuation of declining water sales over the next 5+ years. The anticipated declining water sales will likely be a result of the combined effect of continued customer decline, increased use of shallow wells and more importantly, a result of the current economic recession.

Capacity assessments using ERC based approach identified areas of limiting capacity in the distribution system, raw water transmission system supply and treatment plant.

With the current regressive ERC condition, the capacity of the water system critical components is adequate. However, adequacy does not imply a lack of need to make system improvements. In subsequent TM's, PBS&J has identified several priority items that require maintenance or replacement to maintain the reliability of the water system. Should growth return the distribution system appears to be the limiting factor in the future, followed by Floridian aquifer capacity and ultimately the capacity of the water treatment plant.

Because of the economic recession in collaboration with the ability to install shallow wells for irrigation, we recommend WMSI update this Capacity Assessment in 5 years. The future assessment would be able to quantify the impact of the recession and shallow wells to better define future needs and improvements.

Below are recommendations for consideration:

1) Implement a maintenance type, Capital Improvement Program, to address the Operation and Maintenance of the current water system. A total of $2,200,000 in capital investments will carry the utility for the next five years. The CIP program should include items prioritized in subsequent TM's on source water, treatment plant, structures, electrical systems and water and distribution. The funding for the CIP program must come thru a combination of rate increases and consideration for implementing impact or

surcharge fees on customers electing to utilize shallow irrigation wells for irrigation. The rate increases and fees need to be sufficient to meet the annual revenue requirements to enable to WMSI to remaining financially solvent.

2) Update this Capacity Assessment in 5 years to evaluate the need for future expansion, planning and permitting to better define the future needs and improvements.

3) Under the current regressive growth periods, WMSI needs to be aggressive in identifying and disconnecting illegal connections, addressing metering deficiencies, all in an effort to increase water sales and revenue.

4) Maximize operational performance in the distribution system. By carefully examining the flushing procedures, duration and a more careful evaluation of flushing needs, WMSI may lower the volume of water lost. Every gallon of water flushed, results in lost revenue.

5) Maximize performance at the treatment facility to lower the cost of producing water.

6) Reduce all non- essential expenses to improve the financial performance.

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2.0 Purpose

This technical memorandum (TM) will serve to update the 1995 Water Management Services Capacity Analysis performed by Les Brown, P.E. for the St. George Island Water System. This analysis will identify current conditions; identify issues relating to future capacity and operation of the system as a whole.

Sources of information used in the preparation of this TM include:

- Annual Report of Water Management Services for the year ending December 31, 2009
- GeoTrans, Inc. Technical Memorandum on Numerical Modeling of Potential Groundwater Development on St. George Island, September 30, 2009.
- 1995 Capacity Assessment for St. George, Island, May 8, 1995
- Interviews with Water Management Services personnel and staff by PBS&J.

3.0 Definitions

The capacity of a water system is based upon its ability to obtain, treat, and deliver water to its users. Factors, which dictate the capacity/requirements of the St. George Island Water System, are:

- **Consumption Demands -**

The consumption demands are the various quantities of water that the consumers have typically required or used at different times of the day or year. The quantities are expressed as follows:

- **Annual Average Day Flow (ADF) or Demand -**

The total quantity of water used during a year divided by 365. For typical single families, 100 gallons per day per person or 350 gallons per day per single-family residence is typical. For this TM the definition of an Equivalent Residential Connection or ERC - is 350 gpd.

- **Maximum Day Demand –**

The total water used on the day of highest usage. This is typically 150% of the annual average day consumption. This is the quantity of water that a system's supply (well field and treatment plant) must be capable of providing.

- **Peak Demand –**

The average water used in terms of gallons per minute during the peak hour. Typically, this is equivalent to 350% of the Average Day Demand.

- **Storage Demand –**

DEP requires that 50% of the Maximum Day Demand Volume be provided by any, or by a combination of: Elevated Storage Volume, Ground Storage Volume with Emergency Power, or wells with emergency power pumping directly into the distribution system

- **Floridian Aquifer Capacity -**

The quantity of water which can be taken from the Floridian aquifer without creating adverse affects to either the aquifer, such as salt water intrusion; or to existing users, such as lowering the water table below their wells or well pumps. The Northwest Florida Water Management District is the agency, which establishes and permits the quantity to be withdrawn. The aquifer capacity is expressed in terms relating to the maximum day demand.

- **System Withdrawal Capacity -**

The quantity of water, which can be physically pumped from the Floridian aquifer. This is the 24-hour quantity of water, which the pumps can deliver to the water plant with the largest pump out of service. Its capacity is rated in terms of maximum day demand.

- **Raw Water Transmission Capacity -**

The quantity of water, which can be transferred to St. George Island. Its capacity will be rated in terms of maximum day demand, as it must be able to deliver all the water needed for the maximum day.

- **Treatment Capacity -**

The quantity of water that can be treated in 24 hours. The capacity is rated in terms of maximum day capacity.

- **Distribution Capacity -**

The quantity of water in gallons per minute, which can be delivered throughout the system without allowing the pressure to drop below 20 psi anywhere in the system. It is rated in terms of gallons/minute and relates to the peak day and fire flow demands.

- **Fire Flow and Franklin County Requirements -**

The State of Florida Department of Environmental Protection does not require a system to provide "fire flow". The department does, by reference, incorporate the "RECOMMENDED STANDARDS FOR WATER WORKS (or the 10 States Standards). These standards do not require a system to provide fire protection/flows either. The standards indicate, "When fire protection is to be provided, the system design should be such that fire flows and facilities are in accordance with the requirements of the state Insurance Services Office". The State of Florida ISO office does not require fire protection. The Department of Environmental Protection does require that a fire hydrant be installed on no less than a six (6) inch line if provided.

Franklin County passed an ordinance in 1989, which requires that all new subdivisions provide fire hydrants and that they be within 500 feet of any structure and no more than 1,000 feet apart. Subdivisions which were in existence prior to 1989 are exempt from this ordinance. Any new subdivisions (an area of land 5 acres or less subdivided into three or more lots) would have to have fire hydrants.

WMSI at the request of the Fire Department will install private fire hydrants for a labor and material fee. WMSI maintains the hydrants at this time and does not charge for water usage from the hydrants. Currently, the Water Tariff filed with the Florida Public Service Commission does not include providing public fire protection within WMSI service area.

4.0 Current and Future Conditions

WMSI has one water treatment facility that has a permitted capacity of 0.714 mgd as an annual average and a maximum single day capacity of 1.240 mgd. The combined monthly withdrawal is permitted at 32.7 mg for or approximately 1.10 mgd per day. **Table 1** identifies the current connections and anticipated future connections.

Table 1 – Current and Anticipated Future Connections

Active service connections	1799**
Inactive service connections	178
Vacant buildable lots	1,209*
Projected total connections	3,186

* Vacant buildable lots are not anticipated to have multiple ERCs. This figure accounts for the 200 lots that are not buildable as a result of use for septic tank drain field effluent.
** As of February 2010

Figure 1 presents a three-year historical trend for water sales indicating a downward trend in water consumption.

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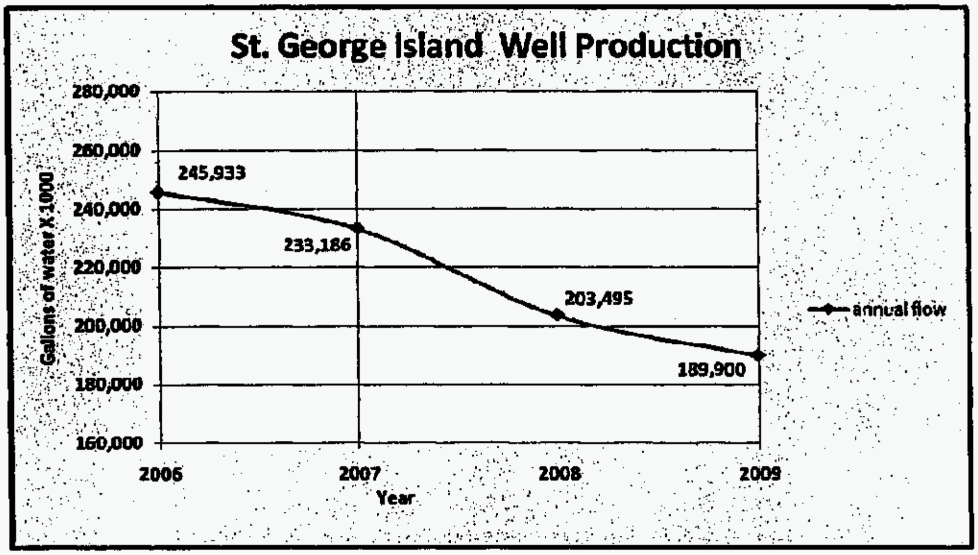

Figure 1- Historical Trends

4.1 Consumption Demands

The customers served by are primarily residential; however, commercial customers are served via restaurants, hotels, State Park or other type of commercial business. The consumption demand for the commercial customers has a higher demand than residential.

Currently there are 1799 customers' connections, not ERC's. The WMSI 2009 Annual Report, page W-13, estimates the total number of meter equalivalents of 2024 (1804 ERC) or an additional 220 equivalent residential connections.

a) <u>Calculation of current ERC's based on average daily flow data</u>

For the year, ending December 31, 2009 a total of 151,136,000 gallons of water was sold to customers or an average of 414,071 gallons per day (gpd). Using a value of 350 gpd per ERC, the total number of ERC's is estimated as follows:

$$= 151{,}136\ \textit{gals sold} \div [365\ \ \textit{days/year} \times 350\ \textit{gal}\backslash/\textit{ERC}]$$

Current ERC bases on flow = 1183

WMSI has the obligation to serve 220 ERC's as indicated above, in addition to the calculated value above for a total 1403 ERC's. Below is the calculation.

$$Current\ total\ ERCs = [ERC's\ based\ on\ flow + obligated\ ERC's\ ]$$

$$Current\ total\ ERCs = [\ 1183 + 220]$$

$$Current\ total\ ERCs = 1403$$

b) Calculation of current consumption demands using ERC's

i. $average\ day\ demand, gpd\ = [1403\ total\ ERC\ 's\ x\ 350\ gpd/ERC]$

$$average\ day\ demand, gpd\ = 491{,}050$$

ii. $maximum\ day\ demand, gpd\ = [1403\ total\ ERC's\ x\ 350 \frac{gpd}{ERC} x\ 1.50\ peak\ factor]$

$$maximum\ day\ demand, gpd\ = 736{,}575$$

iii. $storage\ demand, gpd\ = [736{,}575\ gpd\ maximum\ day\ demand\ x\ 50\%\ ]$

$$storage\ demand, gpd\ = 368{,}287$$

iv. $peak\ hourly\ , gpm\ = [491{,}050\ average\ day\ \ x\ \frac{1\ day}{1440\ minutes}\ x\ 3.50\ \ peak\ factor]$

$$peak\ hourly\ , gpm = 1193$$

c) Calculation of current consumption demands as a percentage of total permitted maximum day operating capacity

$$demand\ , as\ percentage\ of\ total\ permitted\ maximum\ day$$
$$= [(736{,}575\ gpd\ /1{,}240{,}000\ \max\ gpd)]$$

$$demand\ as\ , percentage\ of\ total\ permitted\ maximum\ day\ \ = 59.4\%$$

- **Implementation of Shallow Wells and Impacts to Potable Water Demand in the Future**

GeoTrans, Inc. performed numerical modeling analysis of potential groundwater development on St. George Island for the Northwest Florida Water Management District. The GeoTrans, Inc Technical Memorandum is included in **Appendix 1.** The purpose of the modeling was to assess the potential for development of local water supplies on St. George Island for the purpose of irrigation to supplement the potable water supply. The concept modeled is to allow the use of shallow wells for irrigation. These shallow wells would pull water from the surficial aquifer to reduce water withdrawal from Floridian aquifer. A technical memorandum was prepared by GeoTrans, Inc. and issued in September, 2009.The analysis identified the possibility of successfully developing irrigation wells on the island. Under the model, under the current level of development on the island, would yield approximately 200,000 gallons of irrigation water per day (or 73 million gallon annually). In the future, with full island build out, the average projected pumping from the surficial aquifer would yield nearly 390,000 gallons of irrigation water per day or (142 mg annually).

The volume of water the surficial aquifer could provide, with the implementation of drilling shallow wells on the island <u>will certainly influence the volume of potable water sales and subsequent revenue for the WMSI</u>.

The following calculations of future demand have been prepared <u>without</u> the assumption the shallow irrigation wells will be installed. In later discussion, the impact of shallow well production is considered.

d) <u>Calculation of estimated future ERC's based on information in **Table 1**</u>

$$future\ ERC's = [total\ projected\ connections\ as\ ERC's + current\ obligated\ ERC's\ ]$$

$$estimated\ future\ ERC's = [3186 + 220]$$

$$estimated\ future\ total\ ERC's = 3406$$

e) <u>Calculation of future consumption based on estimated ERC's w/o irrigation wells</u>

i. $average\ day\ demand, gpd\ = [3406\ total\ ERC\ 's\ x\ 350\ gpd/ERC]$

$$average\ day\ demand, gpd\ = 1{,}192{,}100$$

ii. $maximum\ day\ demand, gpd\ = [3406\ total\ ERC's\ x\ 350 \frac{gpd}{ERC} x\ 1.50\ peak\ factor]$

$$maximum\ day\ demand, gpd\ = 1{,}788{,}150$$

iii. $storage\ demand, gpd = [1{,}788{,}150\ gpd\ maximum\ day\ demand\ x\ 50\%]$

$$storage\ demand, gpd = 894{,}075$$

4.2 Aquifer Capacity

WMS currently operates under an Individual Water Use Permit for consumptive water use/supply to St. George Island. The Water Use Permit was most recently approved by the Governing Board of the Northwest Florida Water Management District, (the District) on June 22, 2006. This permit expires on July 1, 2011. Source water is from the Floridian Aquifer for public supply. The permit authorizes WMS to make a combined average annual withdrawal of 714,000 gallons of water per day (GPD), a maximum combined withdrawal of 1,240,000 GPD, and a combined monthly withdrawal of 32,700,000 gallons. The **Table 2** sets forth individual withdrawals for each well facility; however, the total combined amount of water withdrawn by all well facilities cannot exceed the amounts previously stated.

Table 2 - Well Information and Location

Well No.	Withdrawal Point ID No.	Location	GPD (maximum)	Year Constructed	Well Diameter (inches)	Total Depth (feet)
1	WMS #1/AAA5300	130 Creamer St.	360,000	1975	8	263
2	WMS #2/AAA5299	9 Adams St.	360,000	1985	8	300
3	WMS #3/AAA5297	99 Island Dr.	720,000	1993	12	311
4	WMS #4/AAD9754	203 Patty Lane	720,000* – 1,080,000	2000	12	329

- (Information obtained from Individual Water Use Permit and Valuation Report and O&M Manual, provided by WMS)
- *720,000 GPD is the maximum permitted daily flow rate; Well No. 4 can reportedly produce up to 750 gpm.

Additionally, WMS is required by the District to limit the combined withdrawal amounts from wells No. 1, 2 and 3 to no more than 50 percent of its total annual withdrawal. Also, WMS shall not withdraw at a rate of more than 250 gallons per minute (gpm) from either well No. 1, 2, nor withdraw at a rate of more than 500 gpm from either well No. 3 or No. 4. The requirements of the permit, water quality reports, water usage, billing, efficiency and water conservation are required to be regularly submitted to the District.

Careful interpretation of this data results in the conclusion that the District desires for WMS to withdraw 50% of their water from Well No. 4. The decision to do this was made in the late 90's and the well was placed into operation in 2000. The reason for this decision was the District's concern that excessive drawdown in peninsula of Eastpoint. Wells No. 1-3 are in close proximity to wells being used by the Eastpoint Water System. The District's concern was that excessive drawdown in this area could create a landward migration of the saltwater/freshwater interface which exists within the St. George Sound. Well No. 4 was developed further north of the northernmost existing supply well. This well was developed so that pumping could be reduced from wells located along the southern peninsula and to ease the concentrated demand on the aquifer at that location. (Groundwater Monitoring Plan, 1996, Jim Stidham & Associates, Inc.)

For the year, ending December 31, 2009 Floridian aquifer pumping data is included in **Table 3.**

Table 3 – 2009 Aquifer Withdrawal	
Annual total gallons	189,900,000
Average day , gallons	520,274
Maximum month, gallons	24,812,000
Maximum day, gallons	1,029,000

a) Calculation of current aquifer withdrawal as ERC's:

i. $ERC's\ at\ average\ day\ aqufier\ withdrawl\ = [520{,}274\ gpd\ /(350 \frac{gpd}{ERC})]$

$$average\ day\ ERC's\ = 1486$$

ii. $ERC's\ at\ maximum\ day\ = [1{,}029{,}000\ gpd\ /(350\ gpd\ per\ ERC x\ 1.5\ peak\ factor)]$

$$maximum\ day\ ERC's\ = 1960$$

b) Calculation of permitted aquifer withdrawal as ERC's:

i. $ERC's\ at\ average\ day\ aquifier\ withdrawl\ = [714{,}000\,gpd\ /(350 \frac{gpd}{ERC})]$

$$average\ day\ ERC's\ = 2040$$

ii. $ERC's\ at\ maximum\ \ day\ = [1{,}240{,}000\,gpd\ /(350\ gpd\ per\ ERCx\ 1.5\ peak\ factor)]$

$$maximum\ day\ ERC's\ = 2361$$

4.3 Raw Water Transmission Line

In TM 3 of this report, well field capacity is identified as 1750 gpm with all well pumps in service. The 1750 gpm flow rate exceeds the permitted rate of withdrawal. With the single largest well out of service, the pumping rate is estimated to be 1000 gpm. A single 12-inch ductile iron raw water line connects the well fields on the mainland to the water treatment plant. The line was placed into service in early 2004. Using the 1000 gpm flow rate, the estimated pressure at the well pump discharge is 61 psi considerably lower than the 100-psi value used in the previous capacity assessment. A review of recent records indicates the line can transport 927 gpm (peak day, May '06) of water from the wells to the treatment plant. Using 100 psi as the basis of maximum pressure discharge pressure at the wells, **Table 4** indicates the line should be able to handle 1575 gpm or 2,270,000 gallons per day. Based upon this information, the ERC's capacity of the line is estimated as follows:

a) Calculation of transmission line as ERC:

$ERC's\ at\ maximum\ \ day\ = [\ 2{,}270{,}000\,gpd\ /(350\ gpd\ per\ ERCx\ 1.5\ peak\ factor)]$

$$maximum\ day\ ERC's\ = 4365$$

Table – 4
Transmission Main Estimated Headloss

Flow Rate, gpm	Velocity, fps	Nominal Pipe Diameter, inches	Length of pipeline with Fittings, feet	Total Head Loss, feet	Discharge Pressure at Pump, psi
250	0.71	12.00	30492.00	77.22	33.58
500	1.42	12.00	30492.00	90.83	39.49
868	2.46	12.00	30492.00	124.24	54.02
1000	2.84	12.00	30492.00	139.89	60.82
1200	3.40	12.00	30492.00	167.12	72.66
1575	4.47	12.00	30492.00	229.31	99.70
1750	4.96	12.00	30492.00	263.16	114.42

4.4 Treatment Plant Capacity

The water treatment plant provides for the removal of hydrogen sulfide and adds chlorine for disinfection. The plant's aerators are capable of handling raw water at a rate of 800 to1000 gallons per minute. Using 1000 gallon per minute, the plant ERC is estimated as follows:

a) <u>Calculation of treatment plant as ERC:</u>

i. $ERC's = [(1000\,gpm\,X\,1440\,min/day)/(350\;gpd\;per\;ERCx\;1.5\;peak\;factor)]$

$$maximum\;day\;ERC's = 2742$$

4.5 Distribution Capacity

The high service pumping system, which delivers water to the distribution system via the use of a pressure demand system, composed of all pumps operates on variable speed. The jockey pump is out of service at the time of our site inspection and when in service , the pump is typically not used. A computer calculated hydraulic analysis of the entire system was completed in 1995 by Les Thomas P.E.. Updating the Kentucky Pipe distribution analysis is outside the scope of this TM. For the purposes of this TM, the results of the analysis are included.

The analysis of the distribution system indicated the system could deliver peak flow rates of 1,225 gpm while maintaining system pressures above the FDEP requirement. Specifically the pressure at the Bob Sikes Cut was calculated to be 57 psi and 51 psi at the state park.

The analysis also shows that the water system can deliver up to 1,570 gpm with a pressure at the Park of 24 psi and 31 psi at the Bob Sike's Cut. This equates to delivering the peak hour flow rate for 1,873 ERCs.

In the future a finished water booster pump station(s) may be required to increase pressures as the demand increases at the extreme ends of the island. The exact sizing and location of the booster station will need to be determined by using a detailed hydraulic analysis of the distribution system.

4.6. Fire Flow

A computer calculated hydraulic analysis of the entire system was completed in 1995 by Les Thomas P.E. Updating the fire flow distribution analysis is outside the scope of this TM. For the purposes of this TM, the results of the fire flow analysis performed by Les Thomas are included herein.

The analysis was performed with the system demand set at 1/2 its calculated maximum day demand. To maximize the conditions a 500-gpm fire demand was placed at the Bob Sike's Cut. The system delivered the demands while maintaining a pressure of 34.7 psi at the point of fire flow (85 psi at the Park).

An analysis was also performed with a 500-gpm fire demand at the state park. This analysis showed that the pressure at the Park would be 27 psi, (86 psi at Sike's). An analysis was also performed with a 350-gpm fire demand at the park. This showed that the system could deliver 350- gpm with a residual pressure of 22.6 psi.

An analysis was also performed with a 1,000-gpm fire in the commercial area. The system delivered the flow with pressures at the Sikes (Government) Cut of 81 psi and 79 psi at the Park.

The analysis of the distribution system was performed to determine the modifications necessary to upgrade the distribution system to meet Franklin County Regulations. That analysis indicated that approximately 100 fire hydrants would be needed, to comply with the requirement for having a fire hydrant within 500 feet of every building under public fire protection. To implement public fire protection it was determined that approximately 36,400 feet of 6" water main needed to be constructed comply with regulations for minimum line size for hydrants, as well as hydraulic requirements to deliver 500-gpm. The system has the pumping and storage capacities to meet these requirements at this time.

As of the date of this TM, these modifications have not been implemented as WMSI has implemented a private fire protection plan requiring the customer to request through the fire department a hydrant to service their property during a fire. The property owner pays for the cost for the hydrant and the installation.

H. Summary of System Capacities or regulated Limitations:

Table 5 presents a summary of the ERC's with identification of the limiting factor.

Table 5
Summary of System Capabilities

Item	Estimated ERC'S	Rank of Limiting Factors for Future
Permitted aquifer capacity, max day	2361	2
Current aquifer withdrawal, max day	1960	NA
Raw water transmission pipeline	4365	4
Treatment plant capacity, based on aerator	2742	3
Distribution capacity, based on hydraulic analysis at peak hour	1873	1
Current, Max day consumption demand	1403	NA
Future, Max day consumption demand	3406	NA

The distribution system appears to the limiting factor for growth in the future followed by the permitted aquifer capacity.

I. Future situation and impact of shallow irrigation wells

➢ **Regressive customer base future situation**

Using recorded well production data (which shows a significant decline in recent years), we extrapolated this information forward assuming the same trending slope to estimate the ERC's that may be served by WMSI in the near future. **Figure 2** illustrates the anticipated decline in ERC's, Should the customer decline rate continue; the ERC's will drop dramatically and as a result will have a impact on revenues for the water system.

In the future, to complicate a declining customer base, it appears WMSI water customers will have the ability to install shallow wells for irrigation purposes. Obviously, the installation and use of shallow wells pumping water from the surficial aquifer will reduce the volume of groundwater pumped from the Floridian aquifer. The concept that everyone who is a current customer and all future customers of WMSI will undertake the installation of a shallow well for the purposes of irrigation water appears unrealistic. As a result of this unrealistic expectation, and for the purposes of this TM, we developed two options which we believe seem more realistic to allow an objective review of how shallow wells may impact WMSI water production. Two options are presented below.

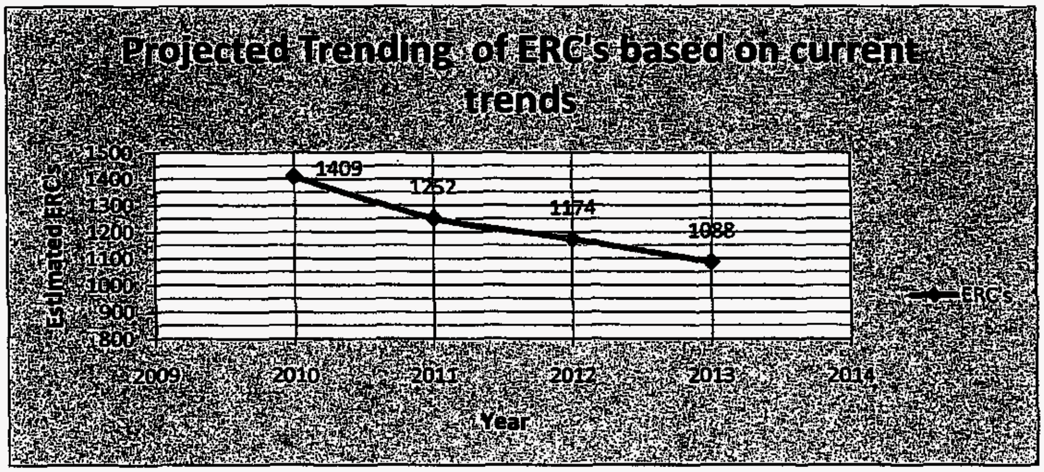

Figure 2

Shallow irrigation well options considered

Option 1- Fifty percent of the current customers will install shallow wells and once the wells are installed, the use will be limited to 50 percent of the time or 183 days per year. For the purposes of volume per day, this TM assumes each irrigation well will produce 132 gpd of water. The 132 gpd is a value taken from the GeoTrans, Inc. Technical Memorandum referenced earlier.

Option 2- Seventy-five percent of the current customers will install shallow wells and once the wells are installed, the irrigation well use will be limited to 50 percent of the time or 183 days of the year. As with Option 1, the production volume from each well is 132 gpd.

The two shallow irrigation well options are displayed in **Figure 3** with the anticipated gallon per month identified. Assuming the pattern of declining ERC's continues, the total volume of water from the shallow wells is also anticipated to decline.

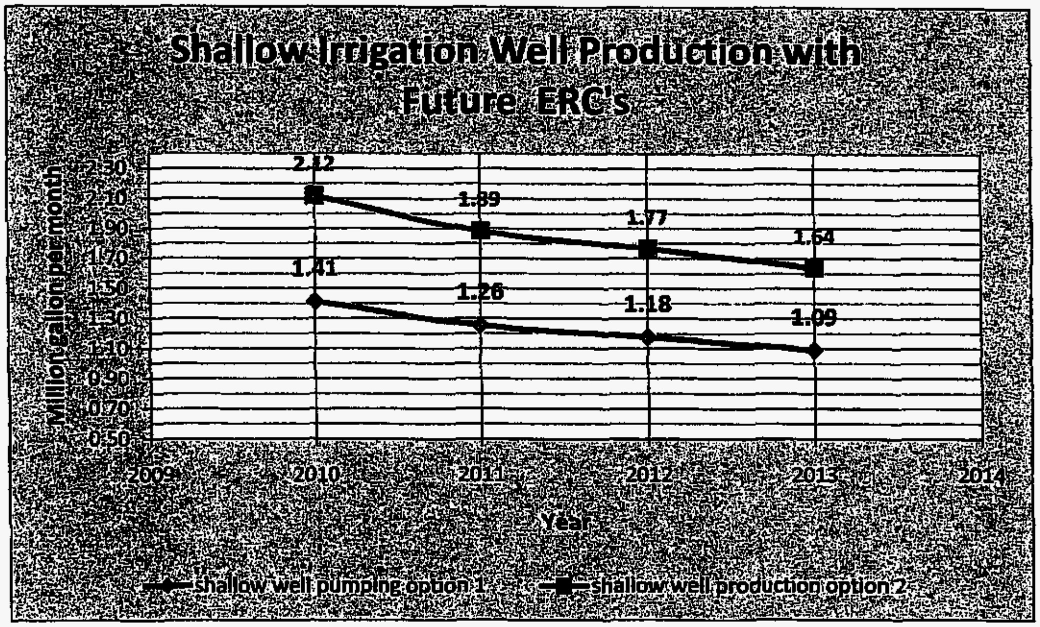

Figure 3

Using shallow well Options 1 and Option 2 presented above and assuming a declining customer base (ERC'S) , an estimate of the impact the shallow wells will have on mainland well production was developed. **Figure 4** presents a graphic display of the analysis.

Figure 4 illustrates:

- Without considering either shallow well option, the anticipated continued declining in ERC's will result in the mainland well production drop of approximately 41,000,000 gallons a year over the next four years.
- With declining ERC's and Option 1- the anticipated mainland well production will drop from 179,999,750 gallons per year to 125,887,040 gallons per year or approximately 54,000,000 million gallons.
- With declining ERC's and Option 2- the anticipated mainland well production will drop from 179,999,750 gallons per year to 119,334,560 gallons per year or approximately 60,000,000 million gallons.

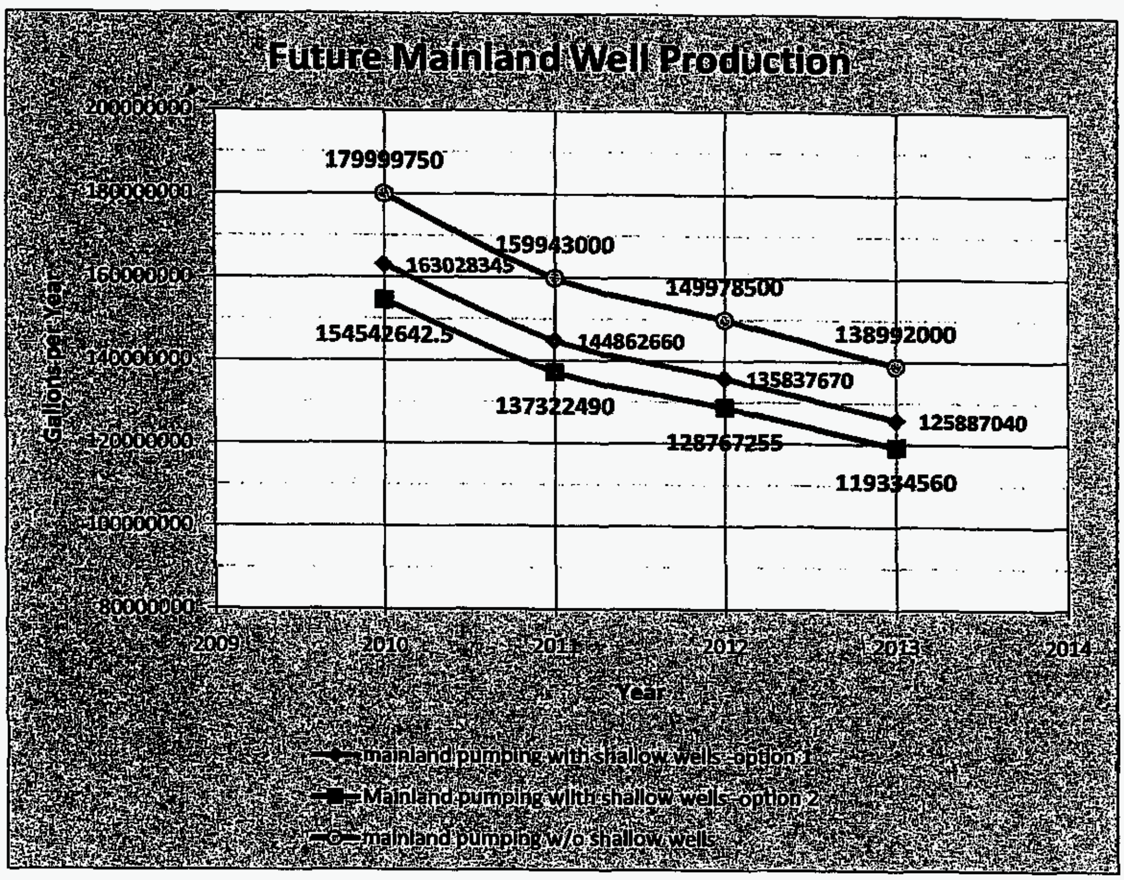

Figure 4

Progressive customer base future situation

Even though the customer base is declining and economic environment is unfavorable at this time for growth, we decided to evaluate the future impacts on the water system should growth occur. We developed a scenario using 3 percent growth for the future. In addition, we incorporated the previous two options for shallow irrigation well installation to examine the impacts to WSMI potable water production.

Figure 5 displays anticipated future growth ERCs with and without shallow irrigation well options 1 and 2. **Figure 6** displays the estimated water production demands.

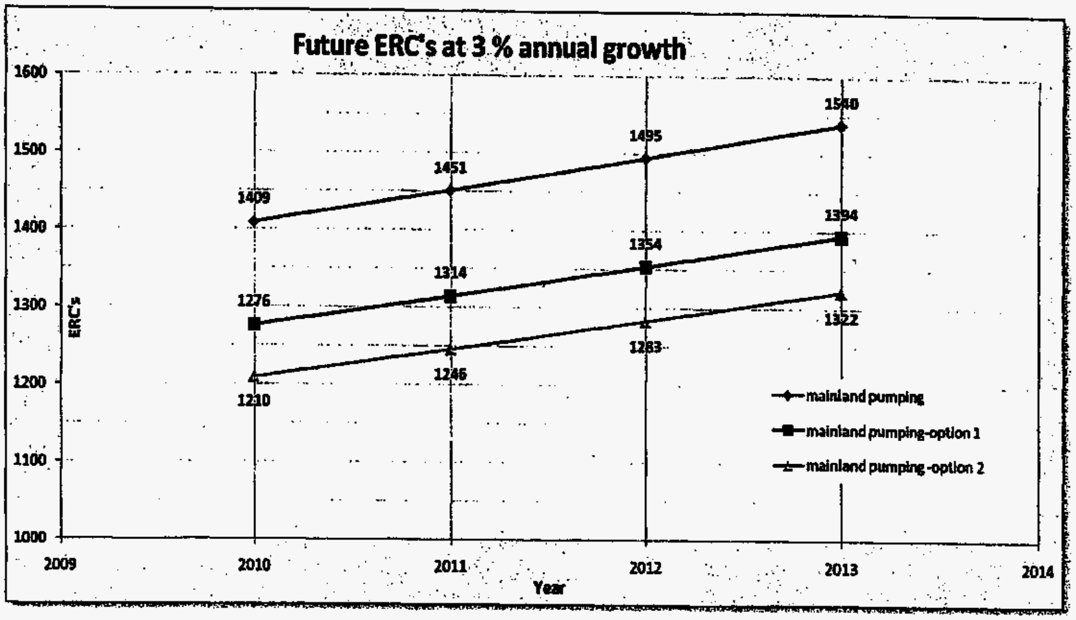

Figure 5

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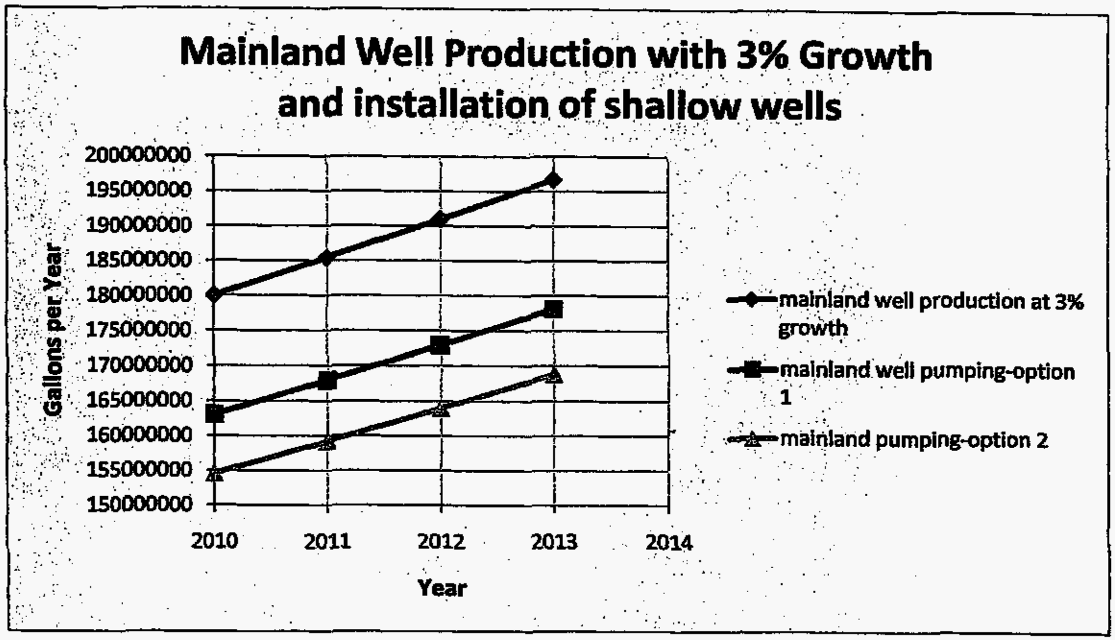

5.0 Future Situation Recommendations

With the history of declining water sales and declining active connections in combination with the current economic recession, it appears that growth is questionable in the immediate future. Capacity assessments are prepared to identify weaknesses in the distribution, supply and treatment capacities. The results of our investigation into capacity limitations indicate that with regressive customer base, sufficient distribution, aquifer and treatment capacity exist for several years. Below are recommendations for consideration:

1) Implement a maintenance type, Capital Improvement Program, to address the Operation and Maintenance of the current system. The CIP program should include items prioritized in subsequent TM's in this report on source water, treatment plant, structures, electrical systems and water and distribution. The funding for the CIP program must come thru a combination of rate increases and consideration for implementing impact fees to customers that elect to utilize shallow irrigation wells for irrigation. The rate increases and impact fees need to be sufficient to meet the annual revenue requirements to enable to WMSI to remaining financially solvent.

2) Update this Capacity Assessment in 5 years to evaluate the need for future expansion, planning and permitting to better define the future needs and improvements.

3) Under the current regressive growth periods, WMSI needs to be aggressive in identifying and disconnecting illegal connections, addressing metering deficiencies all in an effort to increase water sales and revenue.

4) Maximize operational performance in the distribution system. By carefully examining the flushing procedures and methods, WMSI can lower the volume of water lost. Every gallon of water flushed is results in lost revenue.

5) Maximize performance at the treatment facility to lower the cost of producing water.

6) Reduce all non- essential expenses to improve the financial performance.

END

APPENDIX 1

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TECHNICAL MEMORANDUM

From:	Peter F. Andersen and Lisa M. Grogin, GeoTrans Inc.
To:	Chris Richards, Northwest Florida Water Management District
Date:	September 30, 2009
Subject:	Numerical Modeling Analysis of Potential Groundwater Development on St George Island, Florida
Project:	Northwest Florida Water Management District, St. George Island

Professional Engineer:
Peter F. Andersen
License No. 62133
Date: September 30, 2009

SEAL

PETER F. ANDERSEN
LICENSE
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PROFESSIONAL ENGINEER

Introduction

This technical memorandum briefly describes groundwater flow modeling of the surficial aquifer, on St George Island, Franklin County, Florida. The purpose of the modeling is to assess the potential for development of local water supplies on the island for irrigation purposes to supplement the potable water supply, all of which is transported to the island from mainland Franklin County. Data from a literature review and a 72 hr aquifer performance test (APT) were used to formulate a conceptual model of the hydrogeologic system underlying the island. A MODFLOW-based numerical model was constructed based on the conceptual model and various assumptions regarding uncertain hydrologic factors. The model was calibrated to available data, including data from the APT. Predictions were made regarding the likely effects of withdrawals of an estimated quantity of water from small, areally distributed landscape irrigation wells. An estimate of the potential for saltwater intrusion from this additional pumping was also made using an analytical model.

This memorandum is organized as follows. The hydrogeologic conceptual model (HCM), including a review of the literature and a summary of the APT is presented first. The HCM forms the basis for the numerical flow modeling, which is presented in the next major section. Construction, calibration, and predictions are described in that section. The results of the flow model are used to estimate the potential for saltwater upconing, which is described next. Conclusions are recommendations are then provided.

Hydrogeologic Conceptual Model

St. George Island is a barrier island located in the Gulf of Mexico south of Apalachicola, Florida. The island measures approximately 30 miles in length by 0.3 miles in width (Corbett et al., 2000). Figure 1 shows the location of the study area, which encompasses the central part of the island, from Government Cut (the channel that divides the island into Little St George Island on the western side) approximately to the Bryant Patton (Hwy 300) Bridge that connects the island to the mainland. Although the developed part of the island extends further east than the Bryant Patton Bridge, the specified study area is considered representative of the developed part of the island. The portion of St George Island that is included in the study area is about 3 mi^2, which represents approximately 33% of the island. Figure 2 shows a conceptual hydrogeologic cross section of the study area. The section is relatively shallow, as the vertical area of interest is primarily the freshwater portion, which extends only to a depth of approximately 40 ft. Data sources for the hydrogeology of the island are sparse. However, two independent studies have contributed to the conceptual understanding of the hydrogeology of the island: 1) a series of studies on barrier island hydrogeology conducted by University of South Florida that involved field work on St George Island, and 2) a drilling program and APT funded by NWFWMD and conducted by Stidham and Associates (2009). These and other studies are reviewed below.

Corbett, et al (2000) conducted a field study on St George Island with the objective of determining the quantity of submarine discharge of fresh groundwater. Tracer tests were conducted at multiple sites using sulfur hexafluoride and flourescein dye. Estimates of hydraulic conductivity from this test ranged between 9.8 and 590 ft/d, with an average value of 118 ft/d. Two methods, hydraulic gradient and water balance, were used to estimate flux to Apalachicola Bay. The results of the analyses using the two methods were comparable.

A series of papers by Schneider and Kruse of the University of South Florida analyze the shape of freshwater lenses on barrier islands and use St George and Dog Islands as examples. Schneider and Kruse (2001) cite hydrologic distinctions between Dog and St George Islands. They used geophysical techniques to locate the saltwater interface and then applied the SEAWAT (Guo and Langevin, 1998) model to explain the non-symmetrical shape of the freshwater lens. Schneider and Kruse (2001) attribute the shape to spatial variations in recharge, where recharge is highest in the central part of island on the gulf side. The authors surmise that the recent imbalance of additional water from septic on St George Island may influence the hydrologic system significantly. A follow-up paper Schneider and Kruse (2003) is similar, initially focusing on the differences in freshwater lens shape between silicalstic and carbonate aquifers. They continue to discuss the relation of recharge variation on the shape of the freshwater lens and note that recharge variation is related to slope, vegetation, and terrain. The final paper in the series, Schneider and Kruse (2005), expands further on their prior work. Important conclusions are: 1) the seasonal variability of the lens in insignificant, 2) artificial recharge (from septic) may represent 7-25% of natural recharge, 3) development has thinned the central part of the lens, 4) the freshwater lens is not in a state of equilibrium and is eroding, 5) short-term (intra-year) variations of boundary conditions do not change the shape or position of the freshwater lens.

Dulaiova and Bennett (2007) is primarily a study of flushing rates of Apalachicola Bay and does not have much significance to the current study. However, data from a series of seepage meters is presented that provide an estimate of flow rates from barrier islands as contributing factors to the flushing of the bay. The seepage study is located on the bay side of St George Island.

Li, et al (2008) attempt to determine if submarine groundwater discharge is primarily related to tidal recirculation or land based hydraulic gradients. A 2-dimensional SEAWAT (Langevin et al, 2003) model is used for sensitivity analyses of various parameters and boundary conditions. The study concludes that the fresh or new component of SGD is 4 to 50%. Simulations of spring and summer conditions gave ratios of 9 and 15%.

An APT was conducted by Stidham and Associates (2009) on behalf of the NWFWMD. The APT took place at Leisure Lane and Kumquat Court, centrally located on the island in the Plantation subdivision. The APT involved pumping 65 gpm of water from a well screened between a depth of 12 and 22 ft. Water level response was noted in two monitoring wells close to the pumping well (20 ft) and one more distant well (70 ft). The APT pumped water from storage and water levels attained equilibrium, after approximately 1 day. Stidham and Associates (2009) used curve-matching techniques to derive values for transmissivity, delayed yield factor, specific yield, and storage coefficient. The curve matching was somewhat problematic in that a consistent set of values could not be obtained for the wells close to the production well (OB-2 and OB-3) and the more distant well (OB-1). Stidham and Associates (2009) contemplated that heterogeneity could be the cause of the inconsistency. Values obtained from the test are shown in Table 1.

Table 1. Parameter values obtained from APT.

Parameter	Average value	Minimum value	Maximum value	Notes
Transmissivity (ft^2/d)	3881	2089 (OB-1)	7029 (OB-3)	Average of 5, excluding half distance analysis
Specific Yield	0.10	0.03 (OB-3)	0.13 (OB-1)	Average of 5, excluding half distance analysis
Storage Coefficient	0.0034	0.0007 (OB-1)	0.005 (OB-1 and -2)	Average of 5, excluding half distance analysis

The references described above each contain data that may be used to formulate the conceptual model of groundwater flow on St George Island. These data are summarized in Table 2.

Table 2. Hydrogeologic data that forms the HCM of St George Island.

Parameter	Value	Notes	Reference
Structural			
Topography	-3 to 22 ft	Lidar map	NWFWMD
SAS bottom elevation	-39 to -49 ft		Schneider and Kruse (2001)
Holocene thickness	16 to 33 ft		Schneider and Kruse (2001)
	25 to 30 ft		Corbett, et al (2000)
	26 ft	Used in calculations	Corbett, et al (2000)
	22 ft		Stidham and Associates (2009)
Pleistocene thickness	16 to 20 ft		Schneider and Kruse (2001)
Saltwater lens thickness	16 to 98 ft		Schneider and Kruse (2001)
Groundwater Flow			
Horizontal Hydraulic conductivity	16 ft/d		Schneider and Kruse (2001)
	10 to 43 ft/d		
	157 ft/d		Schneider and Kruse (2005)
	66 to 591 ft/d	Computed from tidal	Corbett, et al (2000)

Parameter	Value	Notes	Reference
		fluctuations	
	14.8 to 246 ft/d	Dye tracer study at SP site	Corbett, et al (2000)
	9 to 23 ft/d	Dye tracer study at JA site	Corbett, et al (2000)
	95 to 161 ft/d	APT conversion from Transmissivity	Stidham and Associates (2009)
Vertical hydraulic conductivity	1.6 ft/d	10:1 anisotropy	Schneider and Kruse (2001)
	0.95 to 1.61 ft/d	100:1 anisotropy	Stidham and Associates (2009)
Effective porosity	0.25		Schneider and Kruse (2001)
	0.35		Schneider and Kruse (2005)
	0.3 to 0.35	Gravimetric measurements	Corbett, et al (2000)
Specific Storage	0.0066 / ft		Li, et al (2008) from model calibration
Boundaries			
Precipitation	40.9 in/yr	1993-2000 average	Schneider and Kruse (2001)
Net recharge	-5 to 14 in/yr	Maximum = 1/3 precipitation	Schneider and Kruse (2001)
	-4 to 18 in/yr		Li, et al (2008)
Evapotranspiration	80% of rainfall	NWFWMD from AFSIRS	Corbett, et al (2000)
	50-90% of rainfall	Used in analysis	Corbett, et al (2000)
Septic inflow (from imported water)	2.8 in/yr	For developed part of island	Schneider and Kruse (2001)
Submarine discharge rates	2 to15 m^3/s	Bay side of SGI	Dulaiova and Burnett (2007)
	1 to 9 x 10^6 m^3/yr	Bay side of SGI	Corbett, et al (2000)
Saltwater transport			
Dispersivity	3.3 and 0.3 ft		Schneider and Kruse (2005)
	3.3 and 0.3 ft	James Island simulations	Li, et al (2008)
	0.33 to 1.6 ft	Based on 220 ft scale tracer test	Corbett, et al (2000)
Molecular diffusion	9.3 x 10^{-4} ft^2/d	James Island simulations	Li, et al (2008)

The data compiled above was used to formulate two separate water budgets, one for the entire island and another for the modeled area. The water budget for the entire island (Table 3) relies primarily on the work by Corbett, et al (2000) and uses a reasonable value of submarine discharge to balance the water budget. This water balance represents a current condition.

Table 3. Water budget for St George Island.

Component	Quantity (ft^3/y)	Quantity (mgd)	Percentage	Notes
Inflow				
Precipitation	5.30 x10^8	10.86	97%	24.8 in/yr x area of island
Injection (septic)	1.77 x 10^7	0.36	3%	1997-1998 imported water
Total	5.48 x10^8	11.21	100%	
Outflow				
Evapotranspiration	4.24 x10^8	8.69	78%	80% of precipitation
Submarine discharge	1.24 x10^8	2.52	22%	Difference to obtain balance
Pumping	0	0	0%	
Runoff	0	0	0%	
Downward leakage	0	0	0%	
Total	5.48 x10^8	11.21	100%	

The second water balance (Table 4) is specific to the modeled area and provides high and low bounds on the estimates of precipitation and injection.

Table 4. Water budget for the modeled area.

Component	High		Low		Notes
	Quantity (mgd)	Percentage	Quantity (mgd)	Percentage	
Inflow					
Precipitation	17.77	99%	8.00	99%	Hi = 55 in/yr Appalachicola; Lo = 24.8 in/yr St George Island causeway
Injection (septic)	0.12	1%	0.10	1%	=333 gpd/unit x 486 units x factor (0.75 hi, 0.6 low)
Total	17.89	100%	8.10	100%	
Outflow					
Evapotranspiration	14.22	79%	6.40	79%	80% of precipitation
Submarine discharge	3.67	21%	1.70	21%	Difference to obtain balance
Pumping	0	0	0		
Runoff	0	0	0		
Downward leakage	0	0	0		
Total	17.89	100%	8.10	100%	

These water balances may be used to conceptualize the hydrogeologic system, as a guide to the relative impact of specific development strategies, or as reasonableness checks for the groundwater model that is described below.

Flow Model Analysis

MODFLOW-2000 (Harbaugh et al., 2000) was selected as the groundwater flow simulation software. This public-domain product of the U.S. Geological Survey is very widely used and accepted among groundwater modeling professionals. It is appropriate for the multi-layer porous media constant density groundwater flow simulation.

The Groundwater Vistas (Environmental Simulations Inc, 2007) graphical user interface was used as a shell to MODFLOW. The GV calibration routine was used as a tool to assist in the calibration.

Model Construction

The model domain, with the finite difference grid overlain, is shown in Figure 3. The horizontal grid of rectangular cells has variable spacing with a maximum spacing of 200 ft x 100 ft and a minimum spacing, in the area of the APT production well, of 20 ft x 20 ft (Figure 3). There are 187 columns and 62 rows covering a 35,030 ft by 5,390 ft area extending from Sikes Cut to just west of the Bryant Patton (Hwy 300) bridge. The southwest corner of the grid is located at 1823070 ft East, 222980 ft West in 1983 State Plane Florida North coordinates. The grid is tilted at an angle of 30 counterclockwise to align the rows of the model with the general direction of the shoreline. Note that the remaining figures in the report align the rectangle of the study area with the rectangle of the report page; directions normally inferred to be north-south by page orientation do not follow this convention in the figures presented herein.

The model is divided into 3 layers that correspond to those of the conceptual model shown in Figure 2. These layers were selected to represent specific hydrogeologic layers and are uniform in thickness of 8, 4, and 12 ft. Layer 1 is modeled as an unconfined layer and all other layers are modeled as confined layers.

The boundary conditions for the model include no-flow, specified head, and specified flux, as shown in Figure 3. No-flow boundaries are placed along the two boundaries running perpendicular to the island. These boundaries are placed for convenience at the edge of the area of interest. Hydrologically, they represent flow lines that are envisioned to run cross island at these locations. A no-flow boundary is also placed at an elevation of -20 ft. This boundary represents the approximate location of the depth of aquifer materials. Specified head boundaries are placed along the remaining two boundaries that parallel the coast of the island. Hydrologically these boundaries represent discharge to the ocean. Initially these boundaries were assigned a head of 0 ft. However during the initial model simulations it was found that this simplification did not adequately describe the flow dynamics near the saltwater-freshwater interface; too much water exited the system by flowing horizontally to the boundary. As an alternative, relative freshwater heads were assigned along this boundary—starting at 0 ft at sea level and increasing linearly by a factor of 0.025 ft/ft with depth. This boundary specification approximates the complex flow directions near the freshwater/saltwater interface and maintains the freshwater bubble beneath the island.

A specified flux of 26.3 in/yr is assigned (using the MODFLOW recharge package) along the top face of the model in layer 1 to represent areal recharge to groundwater (after the processes of interception, runoff, and surface evaporation have been subtracted from precipitation). This inflow is balanced somewhat by a head dependent boundary (MODFLOW evapotranspiration package) also in layer 1. A maximum groundwater ET rate of 31 in/yr and extinction depth of 6 ft are used.

The well used in the APT model calibration simulation is represented using a specified flux boundary (well package in MODFLOW). The well is screened in layer 3 and pumps at a rate of 65.2 gpm. For the predictive simulations, a series of specified flux cells (MODFLOW well package) are used to simulate potential net irrigation water use for developed parcels. This representation is more fully described in the section on predictive model simulations.

The modeling presented here uses uniform parameter values within specific hydrogeologic layers. Insufficient spatial data exist to identify specific zones or trends of hydraulic conductivity variation. This approach ignores likely local heterogeneity of aquifer materials in favor of a simple model that can more easily be used to perform model tests.

The pre-conditioned conjugate gradient (PCG) solver of MODFLOW-2000 was used to solve the groundwater flow equations. Closure criteria were 0.001 ft for head and 1 ft^3/d for the residual. A maximum of 500 outer iterations and 10 inner iterations were allowed to achieve convergence. The relaxation and acceleration parameters were each assigned values of 1.0.

Convergence was achieved for all simulations presented in this memo, and model mass balance errors for all simulations were much less than 1%. These facts indicate that the model provides a reasonable solution to the groundwater flow equation.

Model Calibration

Calibration consists of adjusting model parameters and boundary conditions within reasonable ranges such that the model results, referred to as calibration targets, match observed conditions. Two data sets were used for model calibration: 1) regional island water levels, and 2) drawdown measurements from the APT. These calibrations are described below.

<u>Island water levels</u>

This calibration is intended to match the expected distribution of water levels in the study area on the island. This distribution is essentially theoretical in nature, with high water levels in the central part of the island decreasing to 0 at the shore. This expected head distribution provides bounded calibration targets in the sense that modeled water levels should be less than land surface at all cells in the model. Static water levels at the APT site (Figure 1) are used as the primary calibration targets. These wells only provide water levels at one location on the island and these water levels are subject to some uncertainty due to a fair amount of responsiveness to precipitation events. Low water levels are used as steady state calibration targets following the assumption that the spikes in water level are driven by short-duration precipitation events and are not representative of long-term water levels. Values of calibration targets at two of the three wells at the APT site are shown in Table 5.

Table 5. Comparison of modeled and observed hydraulic heads.

Well	Observed head (ft)	Modeled head (ft)	Residual (observed – modeled) ft
OB-2	1.14	1.10	0.04
OB-3	1.35	1.11	0.24

In practice, the model calibration involved iteration between the two data sets (island water levels and APT drawdown), although the model calibrations are described separately in this memorandum to maintain clarity. A prototype island water level calibration was performed to provide initial conditions for the APT drawdown model calibration. Since the island water level calibration involved adjustment of both recharge and hydraulic conductivity, a non-unique solution is likely for the prototype model. The APT drawdown calibration only involves adjustment of hydraulic conductivity. Therefore, the APT provides an improved value of hydraulic conductivity that can plugged back into the island water level calibration to refine the recharge value. Calibrated water levels for the island water level calibration are shown for the targets in Table 5 and across the model domain in Figure 4.

APT drawdown

As described above, the model calibration was performed iteratively, with the APT drawdown calibration providing the most reliable values of hydraulic conductivity. The APT drawdown calibration was initially performed as a steady state simulation using the drawdown at the end of the test as a calibration target. The final calibration of the APT involved a transient model used to simulate variable time steps.

Table 6 shows the calibration targets and modeled drawdowns for the APT drawdown calibration.

Table 6. Comparison of modeled and observed drawdowns.

Well	Observed drawdown (ft)	Modeled drawdown (ft)	Residual (observed – modeled) ft
OB-1	2.05	1.40	0.65
OB-2	2.49	2.61	-0.12
OB-3	1.13	1.04	0.09

Table 7 shows the parameter values for the calibrated model, based on both calibrations. These values generally match the expected range. However, these values should be viewed as best estimates, with some range of possible values due to the simplicity of the model and lack of calibration targets. The transmissivity of the entire aquifer is 2,740 ft^2/d. The calibrated recharge rate is 26.3 in/yr.

Table 7. Calibrated hydraulic parameters.

Parameter	Horizontal Hydraulic conductivity (ft/d)	Vertical Hydraulic Conductivity (ft/d)	Specific Storage (1/ft)	Specific Yield
Layer 1	100	10		0.2
Layer 2	10	0.44	1e-5	
Layer 3	158	14	1e-5	

Predictions

The impact of pumping groundwater from the island for the purpose of irrigation was investigated by running the model using assumed conditions for four primary variables: pumping distribution, pumping quantities, recharge, and duration of stress (pumping and/or recharge). Table 8 summarizes the predictive simulations that were made.

Table 8. Summary of predictive simulations.

Evaluation	Pumping distribution	Pumping Quantity	Recharge rate	Duration
Average pumping conditions under current level of development	Developed parcels	ADR	Average	Steady State
Average pumping conditions under full buildout	Full buildout	ADR	Average	Steady State
Climatic variability	none	none	25% reduction	Steady State
Short-term maximum stress	Full Buildout	MMR	0	30 days (average pumping conditions under full buildout as initial condition)

Average pumping conditions for current level of development

This simulation addresses the primary concern of allowing additional pumping on the island. The model is set up by specifying a net average day (ADR) withdrawal of 197 gpd per parcel from 1,022 parcels distributed across the island as shown in Figure 5a. This quantity of pumping was derived by NWFWMD based on pumping 660 gpd per parcel (assumes ¼ acre irrigated per parcel) and subtracting historical OSDS recharge (233 gpd) and 35% return flow for irrigation (230 gpd). The distribution is based on the number of public water supply interconnections relative to the total number of buildable parcels on the island. This quantity, 51%, was used to randomly select parcels from all buildable parcels that were greater than one quarter acre in size. A net pumping of 0.2 mgd was therefore assigned to 1,022 parcels in the model area. Drawdown from the calibrated steady state model is shown in Figure 6. Maximum water level declines are 0.17 ft.

Average pumping conditions under full buildout

This simulation is similar to the previous simulation, except the net average day withdrawal of 197 gpd per parcel is applied to all buildable parcels within the model area (Figure 5b). A total of 0.39 mgd is therefore applied to 1,992 parcels. Drawdown from the calibrated steady state model is shown in Figure 7. Maximum water level decline is 0.32 ft.

Climatic variability

This simulation is intended to evaluate the sensitivity of water levels to lowered precipitation that could result from periodic droughts or climate change. Of interest are the water levels at which a new equilibrium is attained. The calibrated steady state model (without pumping) was run with a 25% reduction in areal recharge rate to steady state. The results of this simulation are shown in Figure 8. Maximum water level decline from the steady state calibrated model is 0.40 ft at the center of the island.

Short-term maximum stress

This simulation is made to evaluate the impact of a short-term drought that is accompanied by maximum monthly net pumping (MMR) of 707 gpd per irrigated parcel. In this case, the results of the average pumping conditions under full buildout simulation are used as initial conditions and the model is run for 30 days without recharge and the 3.6-fold increase in pumping. As a transient model, a specific yield of 0.2 in layer 1 and specific storage of 1×10^{-5} ft^{-1} in layers 2 and 3 is assigned. Figure 10 shows the drawdown from the calibrated steady state model (no pumping) to the end of the 30 day maximum stress period. The maximum drawdown is 0.38 ft. Comparison to the average pumping conditions (full buildout) drawdown of 0.32 ft indicates that the 30-day drought combined with the 3.6 fold increase in pumping results in an additional 0.06 ft of drawdown.

Water balance from predictive simulations

Water balances were developed for each of the four predictive simulations. These water balances are compared to the water balance for the calibrated model, which has no pumping, in Table 9. Note that the pumping is a small (5 and 10% for the average pumping under the current level of development and full buildout, respectively) component of the recharge. A common feature of the water balances is that the pumping is made up primarily by reductions in outflow to specified heads and secondarily by reductions in evapotranspiration (ET capture). The 30-day simulation accounts for the temporary elimination of recharge and increase in pumping (648,235 ft^3/d) by drawing from storage (225,342 ft^3/d), decrease in specified head outflow (186,865 ft^3/d), increase in specified head inflow (191,986 ft^3/d), and decrease in evapotranspiration (44,040 ft^3/d).

Table 9. Water balances (ft^3/d) for the groundwater flow model.

	Calibrated Model	Average pumping conditions under current level of development	Average pumping conditions under full buildout	Climatic variability	30 days (average pumping conditions under full buildout as initial condition
INFLOW					
Recharge	514,085	514,085	514,085	389,784	0.0
Specified head	21,658	22,492	23,427	40,550	215,413
Storage	0	0	0	0	225,342
Total	535,743	536,576	537,512	430,334	440,756
OUTFLOW					
Evapotranspiration	262,393	258,849	255,560	253,248	211,520
Specified head	273,376	251,160	230,218	177,089	43,353
Wells	0	26,563	51,732	0	185,882
Storage	0	0	0	0	0
Total	535,726	536,571	537,511	430,337	440,754
% DIFFERENCE	0.0	0.0	0.0	0.0	0.0

Saltwater Upconing Analysis

The model described above assesses drawdown due to additional pumping at the parcels. Drawdown is one constraint that may limit water production or it may be a surrogate for another impact, that of saltwater upconing. Saltwater upconing is a concern on St George Island because the relatively low water level elevation implies a shallow depth of the saltwater interface (between 40 and 80 ft for 1 and 2 ft of water table elevation, according to the Ghyben-Herzberg principle). The additional pumping will lower the water table elevation and hence raise the elevation of the saltwater interface. The localized effect of the pumping and contamination of individual wells is of most concern because the water level is lowered the most close to the well.

An analysis method developed by Motz and Acar (2007) was used to approximate the critical pumping rate at which the saltwater interface would rise into the well and contaminate it. Like most analytical equations, many simplifying assumptions are made, and these must be considered when judging the applicability of the method and the results obtained. The following is a description taken from Motz and Acar (2007) describing the method:

In an aquifer overlain by a leaky confining unit, the steady-state drawdown in a piezometer at a depth of penetration z and a distance r from a steadily discharging well that is screened between the penetration depths d and ℓ is given by (Hantush 1964).

$$s = \frac{Q}{4\pi K_r b}\left[2K_0\left(\frac{r}{B}\right) + f_s\right] \qquad (1)$$

where s = drawdown; Q = pumping rate of the well; K_r = horizontal hydraulic conductivity of the aquifer; b = thickness of aquifer; K_0 = modified Bessel function of the second kind, zero order; r = radial distance from the well; $1/B = (K'/b'T)^{1/2}$; K'/b' = leakance of the overlying confining unit; T = transmissivity of the aquifer = $K_r b$; and f_s = partial penetration correction factor:

$$f_s = \frac{4b}{\pi(\ell - d)}\sum_{n=1}^{\infty}\frac{1}{n}\left[\sin\left(\frac{n\pi\ell}{b}\right) - \sin\left(\frac{n\pi d}{b}\right)\right]\cos\left(\frac{n\pi z}{b}\right)K_0\left[\left(\frac{K_z}{K_r}\right)^{1/2}\left(\frac{n\pi r}{b}\right)\right] \qquad (2)$$

where ℓ = distance from top of aquifer to bottom of well screen; d = distance from top of aquifer to top of well screen; n = summation index; and K_z = vertical hydraulic conductivity of the aquifer.

Based on Equations 1 and 2, the analytical, sharp-interface solution that describes the upconing of saltwater beneath a well pumping freshwater from an aquifer overlain by a leaky confining unit is (Motz 1992):

$$Q_c = \frac{2\pi(0.3)T(b-\ell)}{\delta\left[K_0\left(\frac{r_w}{B}\right) + \frac{f_i}{2}\right]} \qquad (3)$$

where: Q_c = critical pumping rate; $\delta = [(\gamma_f/(\gamma_s - \gamma_f)]$; γ_f = specific weight of fresh water; γ_s = specific weight of salt water; r_w = radius of well; and f_i = partial penetration correction factor for drawdown along the saltwater-freshwater interface at z = b:

$$f_i = \frac{4b}{\pi(\ell - d)}\sum_{n=1}^{\infty}\frac{(-1)^n}{n}\left[\sin\left(\frac{n\pi\ell}{b}\right) - \sin\left(\frac{n\pi d}{b}\right)\right]K_0\left[\left(\frac{K_z}{K_r}\right)^{1/2}\left(\frac{n\pi r}{b}\right)\right] \qquad (4)$$

The upconing solution in Equations 3 and 4 is based on Muskat's (1946) approach, in which it is assumed that the rise in the saltwater-freshwater interface is small, the interface acts as a streamline, and no flow occurs in the saltwater beneath the interface. Thus, in this solution, the aquifer thickness b is the distance from the top of the aquifer to the saltwater-freshwater interface. The critical rise of the interface is assumed to occur when the interface has risen to a height equal to 0.3 times the distance between the initial location of the interface and the bottom of the pumped well (Motz 1992).

The most critical assumption in this method is that the pumped zone is a leaky aquifer in which water is obtained from above. The St George Island conceptual model can be described as such, although the amount of confinement and supply from above are probably limited. To account for the data and applicability concerns regarding this method, a range of parameters and configurations were run to obtain critical flows, as shown in Table 10. To construct Table 10, the range in parameters was determined as:

Transmissivity = 2000 to 4000 ft^2/d

Leakance = 0.11 to 0.55 /d

Distance from top of aquifer (top of layer 3) to saltwater interface = 20 ft;

Distance from top of aquifer (top of layer 3) to top of well screen = 0 ft; and

Distance from top of aquifer (top of layer 3) to bottom of well screen = 10 ft.

Transmissivity and leakance were then varied to obtain a range in critical pumping.

Table 10. Critical pumping rates (gpd) for individual wells (saltwater upconing analysis).

Parameter	T = 2000 ft^2/d	T = 4000 ft^2/d
L = 0.11	2,510	4,470
L = 0.55	3,520	6,010

The critical pumping rate is more representative of the total pumping (not accounting for return flow) than the net withdrawal rate and hence should be compared to the average day pumping (660 gpd) and maximum month pumping (1,440 gpd) for quarter acre parcels. This analysis suggests that there is a limitation on how much water should reasonably be allowed per well. Note that the critical pumping rate is just slightly more sensitive to doubling the transmissivity than the leakance. Also note that the presence of a semi-confining bed immediately below the pumped zone and approximately at the location of the saltwater interface is not considered in this analysis. The effect of the semi-confining bed would be to retard upward movement of saltwater and hence allow higher critical pumping rates than suggested by this analysis.

Summary and Recommendations

The modeling presented in this technical memorandum provides an approximation to the drawdown that would take place if irrigation withdrawals were permitted using quantities estimated by NWFWMD. A review of the prior studies, which included a series of analyses regarding submarine discharge at barrier islands, was conducted and used in formulating an HCM. In addition, a site-specific APT was reviewed and incorporated into the HCM. A numerical groundwater flow model was constructed based on the HCM and calibrated to water levels at a limited number of locations and to the response to the APT. The APT provided data on hydraulic conductivities and these values were used to approximate an average recharge rate. Following calibration, the model was configured to evaluate several predictive scenarios.

The predictive scenarios that were conducted with the model were primarily related to the effect of irrigation withdrawal on the island. For average projected pumping under the current level of development, a total of 1,022 parcels with a net withdrawal of 0.2 mgd were modeled. This withdrawal gave a maximum steady state drawdown of 0.17 ft. For average pumping under full buildout conditions, a total of 1,992 parcels with a net withdrawal of 0.39 mgd were modeled. This withdrawal resulted in a maximum steady state drawdown of 0.32 ft. A steady state simulation with a reduction in recharge of 25% indicated that water levels on the island would decline by a maximum of 0.40 ft. Sensitivity to recharge was observed by others (Li et al, 2008 and Schneider and Kruse, 2005). A transient simulation using the average full buildout pumping as an initial condition and 30 days without recharge and maximum monthly pumping resulted in a total maximum drawdown of 0.38 ft.

An analytical evaluation of the potential for saltwater upconing showed that the critical pumping rate at which upconing would occur ranges between 2,510 and 6,010 gpd per well. Note that these quantities should be subject to a factor of safety, do not consider interference effects from other wells, and are based on simplifying assumptions that are inherent to the analytical solution.

The current modeling analysis was limited by the data. Additional data, including several water level recorders and water quality monitoring, would allow development of a transient model and the ability to understand the dynamics of flow on the island. Another useful activity would be to integrate useful aspects of the prior models by Schneider and Kruse (2005) and Li, et al (2008) into a single model. The work of these researchers focused on different aspects of the hydrogeology and used unique calibration targets in the form of the position of the saltwater interface, as determined from a geophysical investigation. The information from these studies could be added to the current model to develop a SEAWAT model that would be capable of evaluating variable density saltwater transport.

References

Corbett, Dillon, and Burnett, (2000) Tracing groundwater flow on a barrier island in the northeast Gulf of Mexico. *Estuarine, Coastal and Shelf Science.*

Dulaiova and Burnett (2007) Evaluation of the flushing rates of Appalachicola Bay, Florida via natural geochemical tracers. *Marine Science.*

Environmental Simulations, Inc (2007). User's manual for Groundwater Vistas

Guo, W. and Langevin, C. (1998) SEAWAT, version 2.04.

Harbaugh, A.W., Banta, E.R., Hill, M.C., and McDonald, M.G., 2000, MODFLOW-2000, the U.S. Geological Survey modular ground-water model—User guide to modularization concepts and the Ground-Water Flow Process: U.S. Geological Survey Open-File Report 00-92, 121 p.

Langevin, C.D., Shoemaker, W.B., and Guo, Weixing, 2003. MODFLOW-2000, the U.S. Geological Survey Modular Ground-Water Model–Documentation of the SEAWAT-2000 Version with the Variable-Density Flow Process (VDF) and the Integrated MT3DMS Transport Process (IMT): U.S. Geological Survey Open-File Report 03-426, 43 p.

Li, Hu, Burnett, and Santos (2008) Modeling Submarine Groundwater Discharge (SGD) Driven by Tidal Pumping in a Heterogeneous Coastal Aquifer. *Ground Water*

Schneider and Kruse (2001) Characterization of freshwater lenses for construction of groundwater flow models on two sandy barrier islands, Florida USA. *First International Conference on Saltwater Intrusion in Coastal Aquifers.*

Schneider and Kruse (2003) A comparison of controls on freshwater lens morphology of small carbonate and siliclastic islands: examples from barrier islands in Florida, USA *Journal of Hydrology*

Schneider and Kruse (2005) Assessing selected natural and anthropogenic impacts on freshwater lens morphology on small barrier islands: Dog Island and St George Island, Florida USA. *Hydrogeology Journal*

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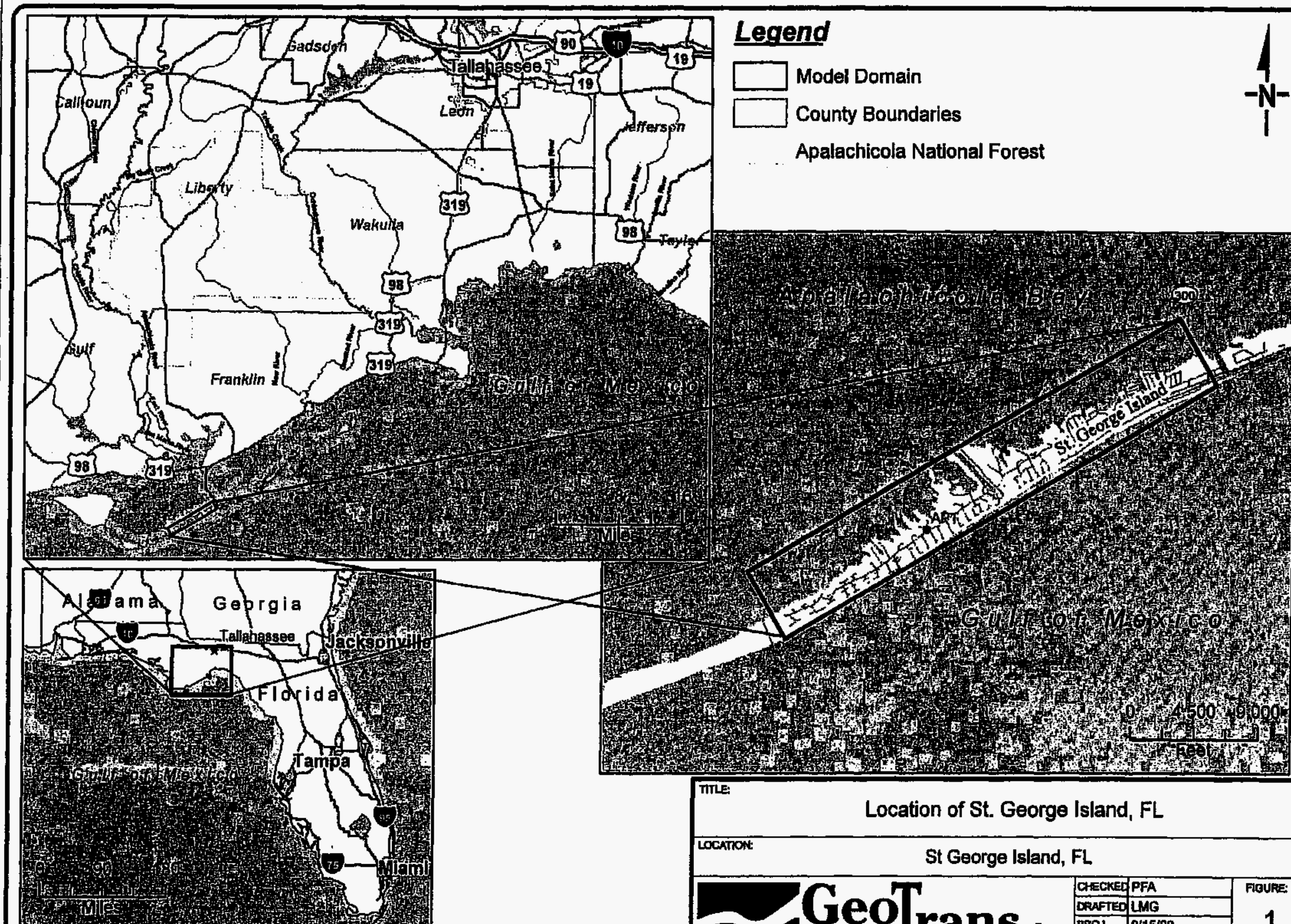

TITLE: Location of St. George Island, FL

LOCATION: St George Island, FL

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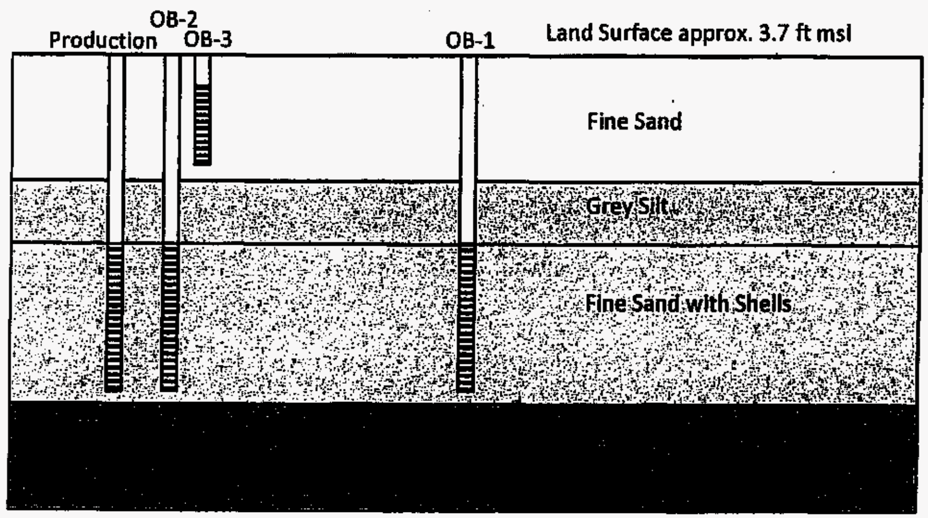

TITLE:
Conceptual Cross Section of St George Island

LOCATION:
St George Island, FL

GeoTrans, Inc.

CHECKED:	PFA	FIGURE:
DRAFTED:	LMG	2
PROJECT:	117-1305022	
DATE:	9/18/09X	

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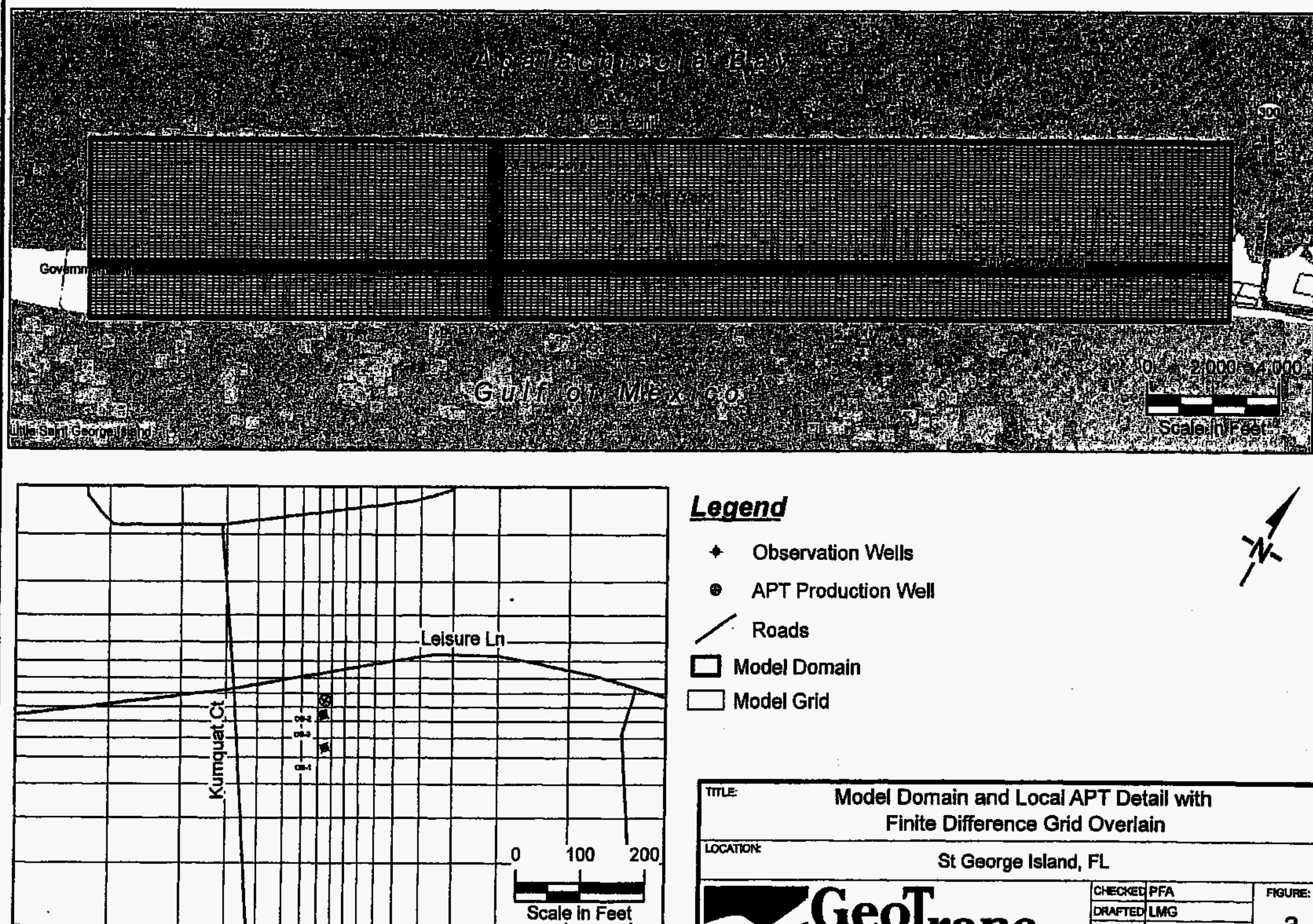

TITLE: Model Domain and Local APT Detail with Finite Difference Grid Overlain

LOCATION: St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	3
PROJ	9/15/09	
DATE	117-1305022	

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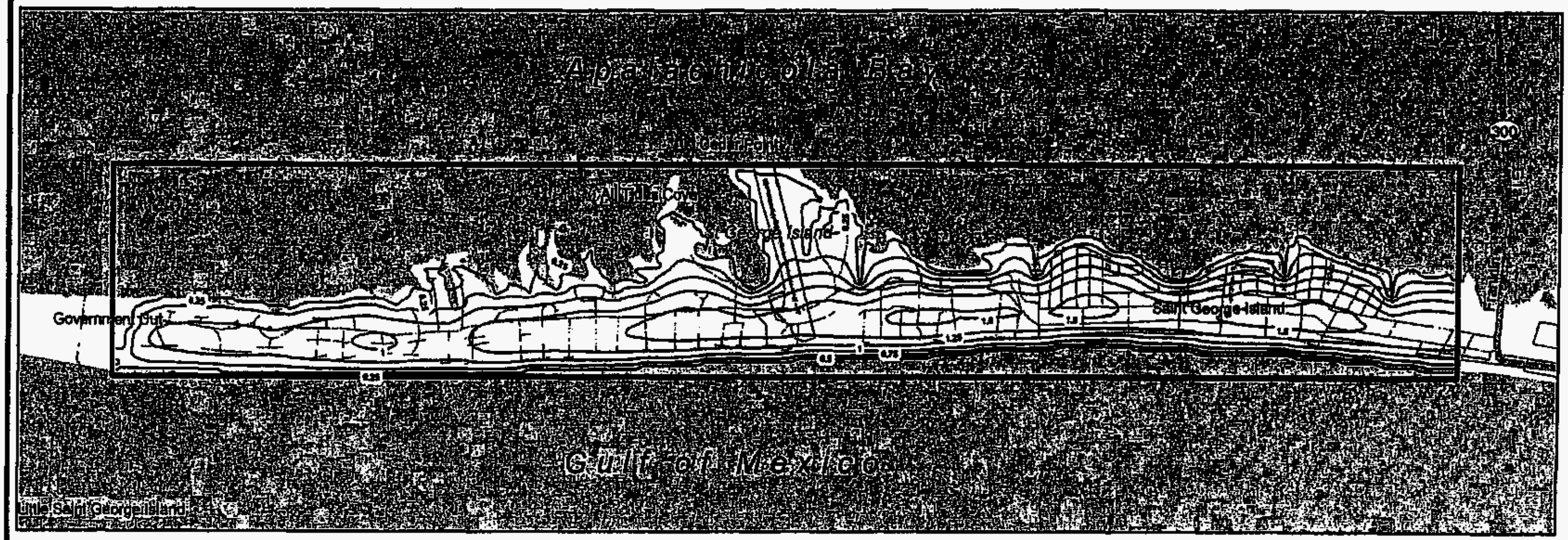

N

0 2,000 4,000

Scale in Feet

TITLE:
Modeled Steady State Hydraulic Head (0.25 ft interval) in Layer 1

LOCATION:
St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	4
PROJ	9/15/09	
DATE	117-1305022	

Docket No. 100104-WU
Water System Eval. Final Report
Exhibit MS-2, Page 000063 of 000237

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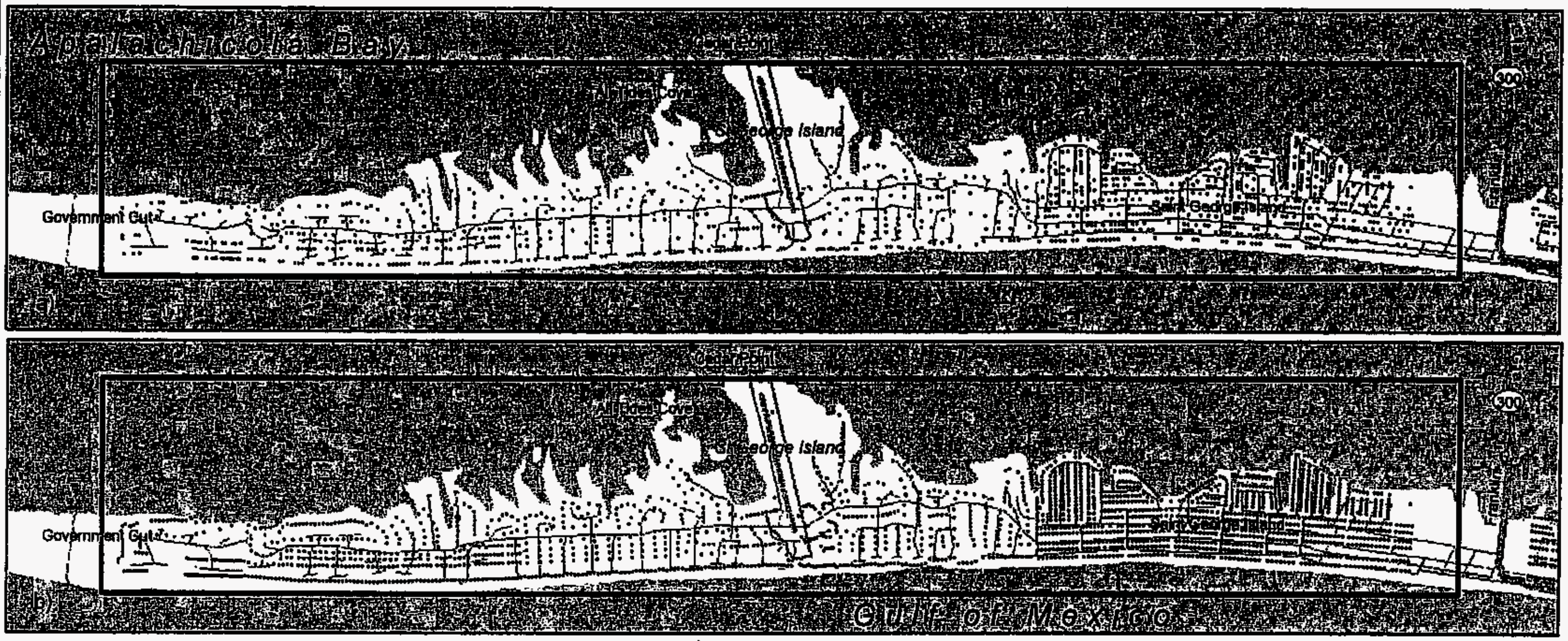

0 2,000 4,000
Scale in Feet

TITLE: Location of Simulated Wells Associated with Current Level of Development (a) and Full Buildout (b) Property Parcels

LOCATION: St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	5
PROJ	9/16/09	
DATE	117-1305022	

Docket No. 100104-WU
Water System Eval. Final Report
Exhibit MS-2, Page 00064 of 000237

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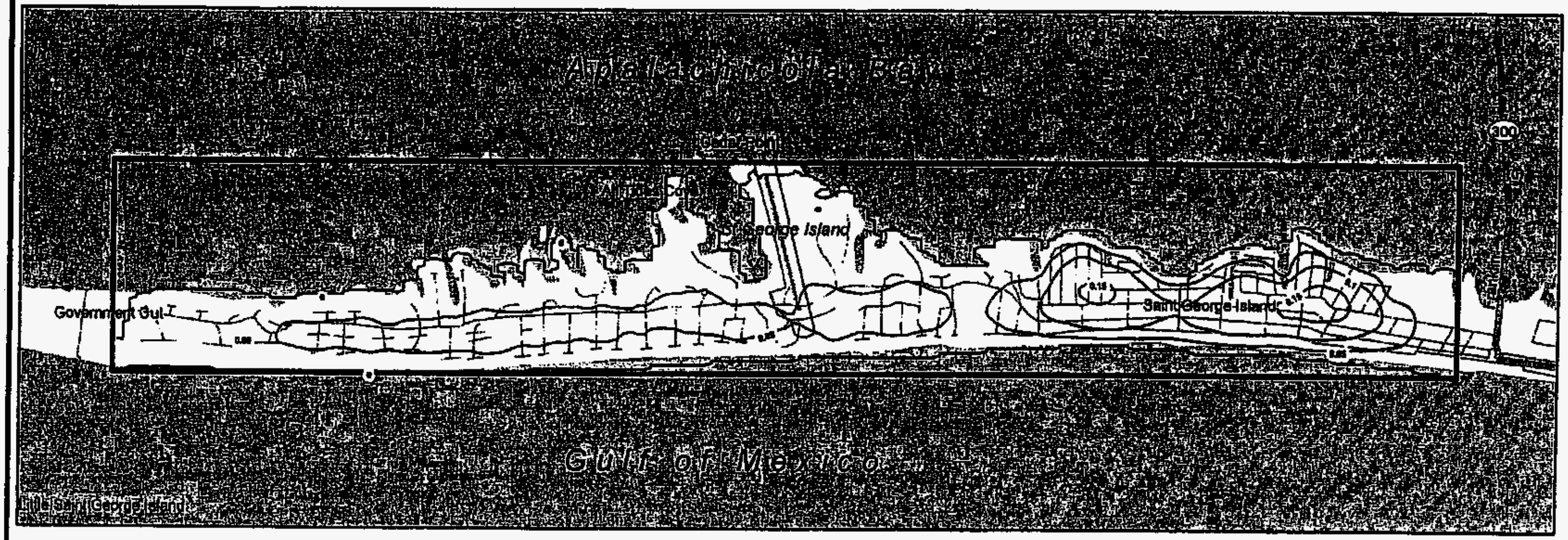

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Scale in Feet

TITLE:	Modeled Drawdown Contours (0.05 ft interval) for Steady-State ADR Pumping for Current Level of Development
LOCATION:	St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	6
PROJ	9/15/09	
DATE	117-1305022	

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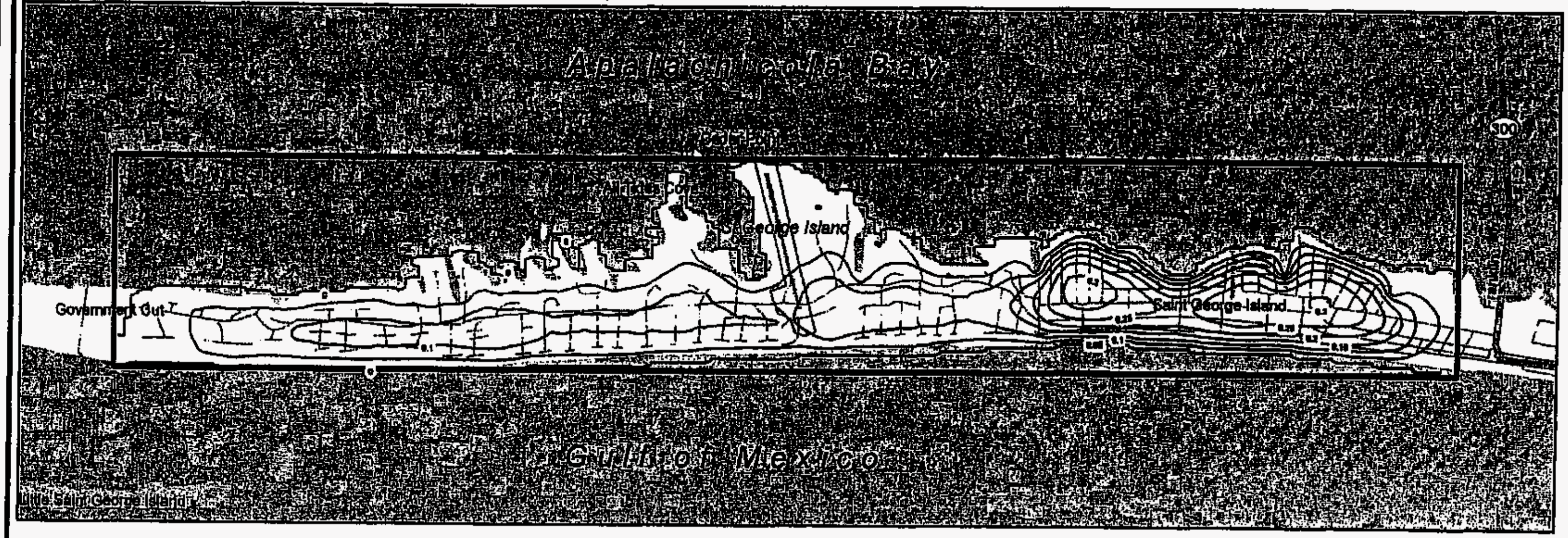

0 2,000 4,000

Scale in Feet

TITLE: Modeled Drawdown Contours (0.05 ft interval) for Steady-State ADR Pumping for the Full Buildout

LOCATION: St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:	7
DRAFTED	LMG		
PROJ	9/15/09		
DATE	117-1305022		

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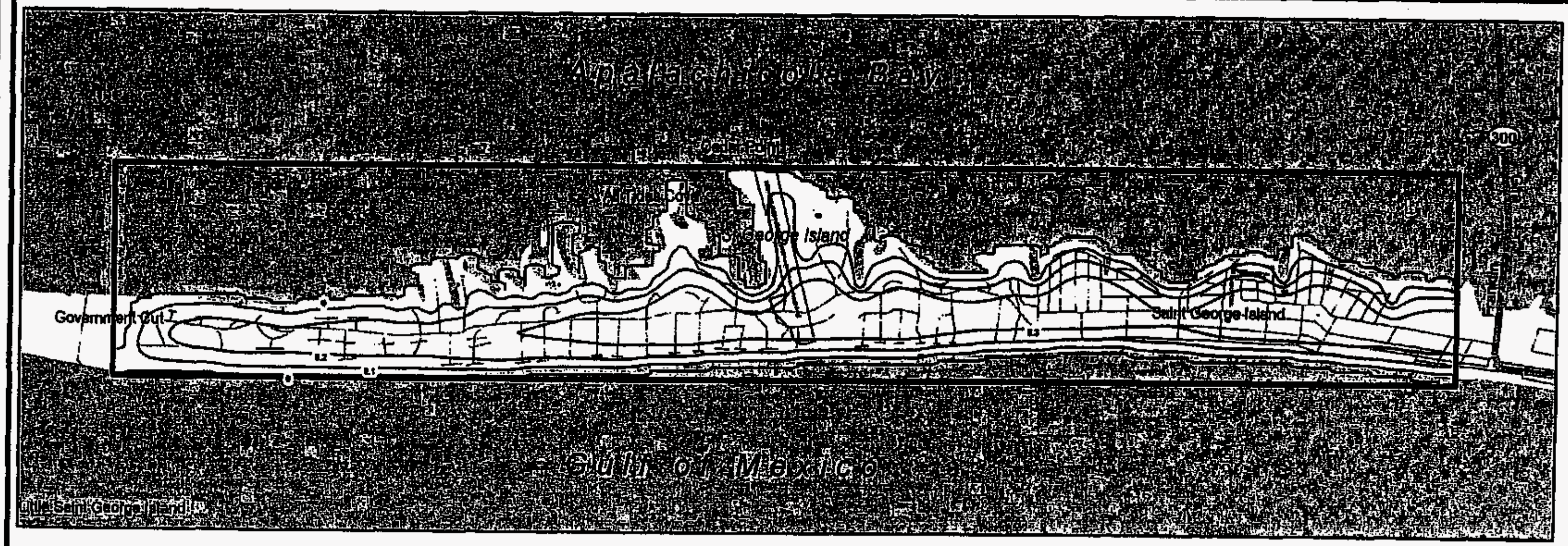

0 2,000 4,000

Scale in Feet

TITLE: Modeled Water Level Decline Contours (0.1 ft interval) for Steady-State 25% Recharge Reduction (No Pumping)

LOCATION: St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	8
PROJ	9/15/09	
DATE	117-1305022	

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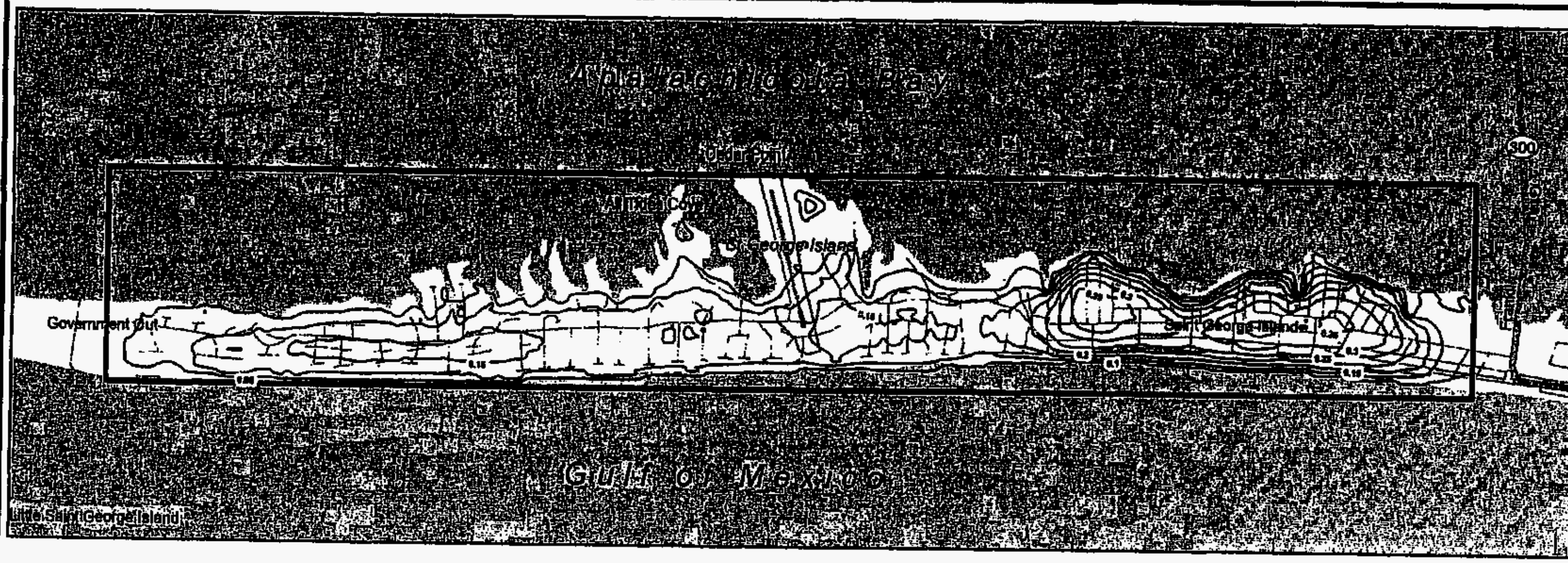


0 2,000 4,000

Scale in Feet

TITLE: Modeled Drawdown Contours (0.05 ft Interval) for Steady State, Full Buildout ADR with Short-Term Maximum Stress (MMR and No Recharge)

LOCATION: St George Island, FL

GeoTrans, Inc. A TETRA TECH COMPANY

CHECKED	PFA	FIGURE:
DRAFTED	LMG	9
PROJ	9/15/09	
DATE	117-1305022	

Docket No. 100104-WU
Water System Eval. Final Report
Exhibit MS-2, Page 000068 of 000237

PBSJ

Technical Memorandum 3
Source Water Supply Evaluation

To:	Gene Brown		
From:	Mike Scibelli	Project:	Water Management Services, Inc
CC:			
Date:	Nov 4, 2009 Revised June 25, 2010	Job Number:	100010111

1.0 Background

Water Management Services, Inc. (WMS) supplies water to the residential and commercial water customers of St. George Island. Source water is provided by four (4) wells located on the mainland in Eastpoint, Florida. Each well discharge is interconnected on the mainland prior to joining to a common raw water main, which crosses on the St. George Island Bridge. The raw water main continues to the Water Treatment Facility (WFT) located on St. George Island where the water is aerated, chlorinated and then pumped into the water distribution system. WMS operates under a franchise issued and regulated by the Florida Public Service Commission. WMS is responsible for the production of potable water on St. George Island, including assuring the quality of the drinking water for it's consumers that meets the State of Florida's drinking water standards.

The purpose of this Technical Memorandum (TM) is to identify weaknesses at the source water wells, their adequacy, vulnerability and summarize information obtained during a field investigation of the four (4) well sites, and develop recommendations for capital improvement projects.

2.0 St. George Islands Drinking Water Source(s)

WMS currently operates under an Individual Water Use Permit for consumptive water use/supply to St. George Island. The Water Use Permit was approved by the Governing Board of the Northwest Florida Water Management District, (the District) on June 22, 2006. This permit expires on July 1, 2011. Source water is from the Floridian Aquifer. The permit authorizes WMS to make a combined average annual withdrawal of 714,000 gallons of water per day (GPD), a maximum combined withdrawal of 1,240,000 GPD, and a combined monthly withdrawal of 32,700,000 gallons. **Table 1** sets forth individual withdrawals for each well facility; however, the total combined amount of water withdrawn by all well facilities cannot exceed the amounts previously stated.

Table 1 Individual Well Information								
No.	Withdrawal Point ID No.	Location	GPD (maximum)	Year Constructed	Well Dia (in)	Total Depth (feet)	Casing Depth (feet)	Motor (hp)
1	WMS #1/AAA5300	130 Creamer St.	360,000	1975	8	263	170	50
2	WMS #2/AAA5299	9 Adams St.	360,000	1985	8	300	190	50
3	WMS #3/AAA5297	99 Island Dr.	720,000	1993	12	311	185	50
4	WMS #4/AAD9754	203 Patty Lane	720,000*–1,080,000	2000	12	329	190	75

(Information obtained from Individual Water Use Permit and Valuation Report and O&M Manual, provided by WMS)
*720,000 GPD is the maximum permitted daily flow rate; Well No. 4 can reportedly produce up to 750 gpm.

Additionally, WMS is required by the District to limit the combined withdrawal amounts from wells No. 1, 2 and 3 to no more than 50 percent of its total annual withdrawal. Also, WMS shall not withdraw at a rate of more than 250 gallons per minute (gpm) from either well No. 1, 2, nor withdraw at a rate of more than 500 gpm from either well No. 3 or No. 4. The requirements of the permit, water quality reports, water usage, billing, efficiency and water conservation are required to be regularly submitted to the District.

Careful interpretation of this data results in the conclusion that the District desires for WMS to withdraw 50% of their water from Well No. 4. The decision to do this was made in the late 90's and the well was placed into operation in 2000. The reason for this decision was the District's concern that excessive drawdown in peninsula of Eastpoint. Wells No. 1-3 are in close proximity to wells being used by the Eastpoint Water System. The District's concern was that excessive drawdown in this area could create a landward migration of the saltwater/freshwater interface which exists within the St. George Sound. Well No. 4 was developed further north of the northernmost existing supply well. This well was developed so that pumping could be reduced from wells located along the southern peninsula and to ease the concentrated demand on the aquifer at that location. *(Groundwater Monitoring Plan, 1996, Jim Stidham & Associates, Inc.)*

Source Water Vulnerability

WMS is required to document and provide water quality information to the District and to its consumers. As stated previously, the wells withdraw water from the Floridian Aquifer; therefore, the quality of water is good. Because of the good quality of water, the only treatment methods currently being used by WMS are aeration and gaseous chlorine injection. The 2008 Annual Drinking Water Quality Report by WMS reports that, "In 2008 the Department of Environmental Protection performed a Source Water Assessment on our system. The assessment was conducted to provide information about any potential sources of contamination near our wells. There are three potential sources of contamination identified for this system with a high susceptibility level." The Florida Department of Environmental Protection's (FDEP) website identifies these sources as three (3) petroleum storage tanks. Two (2) of these tanks have the potential to affected well No. 1 and one (1) has the potential to affected Well No. 2. FDEP lists these tanks as a Moderate level of concern.

3.0 Well Locations and Flood Plain Information

Well Configurations

Wells No. 1 and No. 2 have submersible pump/motors and Wells No. 3 and No. 4 have vertical turbine type pumps, where the motor is located at grade and the pump is coupled to the motor via a long, extended shaft to the submerged pump. Each of these well sites generally consists of wood framed pump house in a fenced area. Wellhead, pump, motor, controls and electrical equipment are housed in these pump houses. The size of each well house and fenced area varies depending on the particular location. Wells No. 3 and 4 have standby emergency power provided by emergency generators. Well No. 4 is the only well with an operating emergency generator for standby power supply.

Well Flooding Potential

FEMA flood zone maps were reviewed to determine the vulnerability of the wells to potential flooding. According to the FEMA flood zone maps (see appendix for maps) and well elevations provide to PBS&J by WMS (Edwin G. Brown & Associates, Inc. survey data), it appears that none of the wells are in danger of flooding based on this information. However, Wells No. 2 and No. 4 are within the FEMA 100-year flood zone. Well No. 3 is close to estimated high water elevations, also. Therefore, consideration could be given to a recommendation to raise critical equipment outside the finished floor of the well house. FEMA flood maps do not list a flooding elevation at Well No. 4, but based on the survey data it would appear the finished floor elevation (FFE) to be sufficiently high enough not to be in danger. (FFE +22')

4.0 System Capacity and Current Demands
Well Production Summary

Well production data for the first half of year 2009 is presented in **Figure 1**.

The well production data indicates that wells No. 1 and No. 2 are averaging approximately 82, 000 GPD, respectively. Well No. 3 appears to have the largest daily output at approximately 236,000 GPD average and Well No. 4 is utilized to make-up the difference, with an average in 2009 of approximately 174,000 GPD.

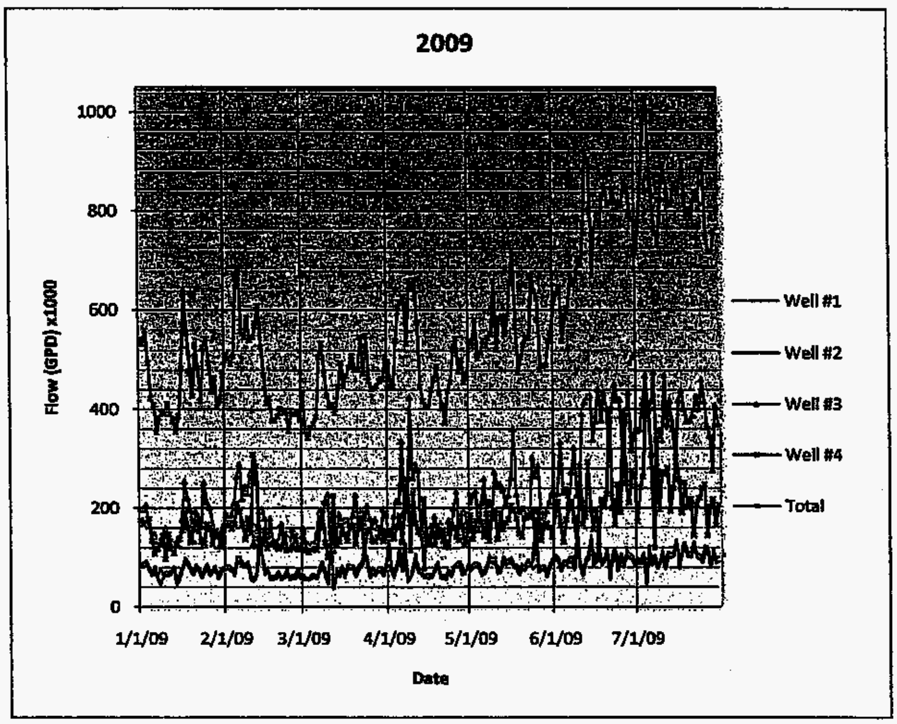


Figure 1

Current Demands

One of the most important requirements for any water system is the ability to meet the water quantity demands of the customers at all times. These demands for water vary daily and vary depending on the time of year. Typically, the demand for water is greatest in the summer and lowest in the winter. The capacity of the system must be equal to the meet current demands over the last several years. **Table 2** provides information on the current demands of the system.

Table 2 Current Water System Daily Demands	
Connections	1805*
Average Daily Demand (GPD)	414,071
Annual Average Capacity (GPD)	714,000
Maximum Monthly Demand (Gallons)	23,622,000
Total Water Sold in 2009 (Gallons)	151,136,000
Calculated Daily Demand per connection (GPD)	229
* represents the number of year end customer connections	

Currently the system is operating at 58 % of the annual average daily flow (ADD) permitted. Production water data of ADD and maximum daily demand (MDD) was used to review the daily demand patterns from 2006 to July 2009. The data presented in **Figure 2** shows a general decline in daily system demands over the period; therefore, the adequacy of supply (comparison of demand versus production) indicates the well supply capacity is sufficient.

Wells No. 1 and No. 2 are susceptible or vulnerable to underground contamination. Continued monitoring of water quality will establish if or when, a fifth well is required, should either well No. 1 or Well No. 2 become contaminated.

Well No. 4 is the only well with operational standby power currently and can supply the ADD to the system in an emergency.

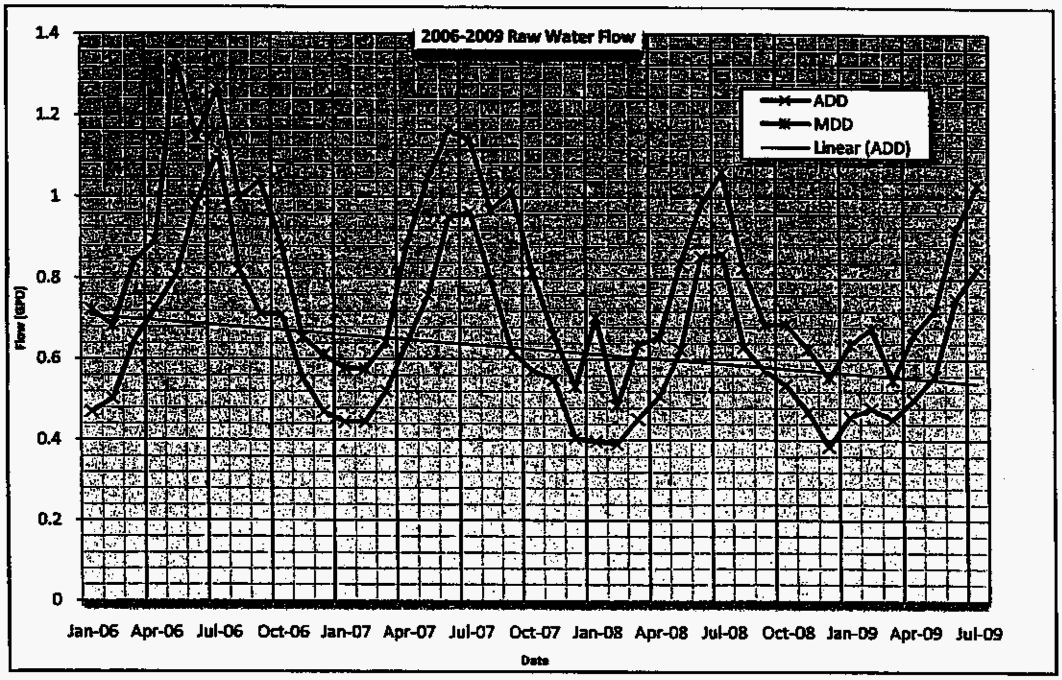


Figure 2

Well Field Capacity

Well No. 1 or No. 2 in combination with Well No. 3 or 4 will meet the maximum daily demand. Although, maximum daily production from Well No. 4 is equivalent to 500 gpm, the well has reported capacity is 750 gpm. The total reported well field capacity is stated to be 1,750 gpm and 1,000 gpm with the largest well out of service, which results in a stated reliable firm pumping capacity of 1,000 gpm or 1,440,000 gallons per day. These available combinations result in redundancy in pumping capacity. However, WMS must still meet the District guidelines previously stated that requires Well No. 4 to produce 50% of the average demand. Actual individual and combined pumping capacities were not field verified during the site investigation.

5.0 Water Quality

5.1 Information in **Table 3** was obtained from WMS from their 2008 Annual Drinking Water Quality Report. The information shown is only a portion of the information in the report. This information is specific to the source water quality. A review of the data indicates water quality contaminants are low with no violations detected.

Table 3
2009 TEST RESULTS

Contaminant & Unit of Measure	Dates of sampling (mo./yr)	MCL Violation Y/N	Level Detected	Range of Results	MCLG	MCL	Likely Source of Contamination
Radiological Contaminants							
Alpha emitters (pCi/l)	Sep-08	No	2.6	N/A	0	15	Erosion of natural deposits
Radium 226+228 or combined radium (pCi/l)	Sep-08	No	1.1	N/A	0	5	Erosion of natural deposits
Inorganic Contaminants							
Fluoride (ppm)	Sep-08	No	0.4	N/A	4	4.0	Erosion of natural deposits; discharge from fertilizer and aluminum factories. Water additive which promotes strong teeth when at optimum levels between 0.7 & 1.3 ppm
Sodium (ppm)	Sep-08	No	12.0	N/A	N/A	160	Salt water intrusion, leaching from soil
Barium	Sep-08	No	0.016	N/A	2	2	Discharge of drilling wastes; discharge from metal refineries; erosion of natural deposits

MCL – Maximum Contaminant Level: The highest level of a contaminant that is allowed in drinking water.
MCLG – Maximum Contaminant Level Goal: The level of a contaminant in drinking water below which there is no known or expected risk to health.

6.0 Assessment of Current Conditions

In TM 6, PBS&J prepared an evaluation of the electrical systems for the wells and water plant. The following describes our assessment of other constituents at the wells. PBS&J performed a site assessment of each well site to evaluate security, facility maintenance, electrical and instrumentation systems, piping and other miscellaneous items associated with the production wells. The following photos were taken during the site investigations at each well site. The wells are listed in the order in which they were visited. They include text-identifying areas of concern that were noted.

Well No. 3

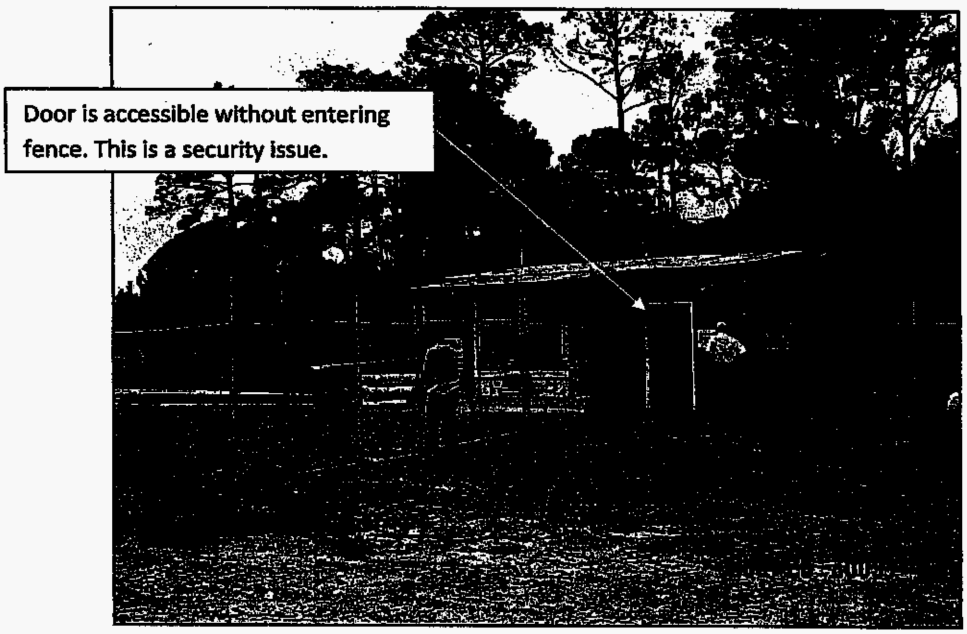


Figure 3-Well No.3

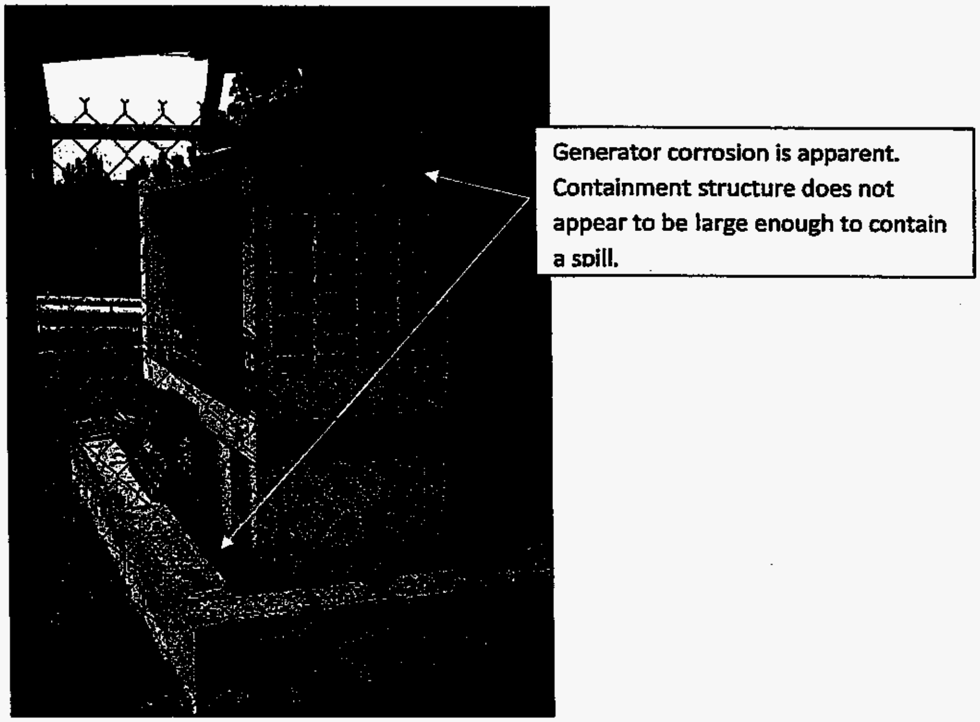


Figure 4-Standby Generator

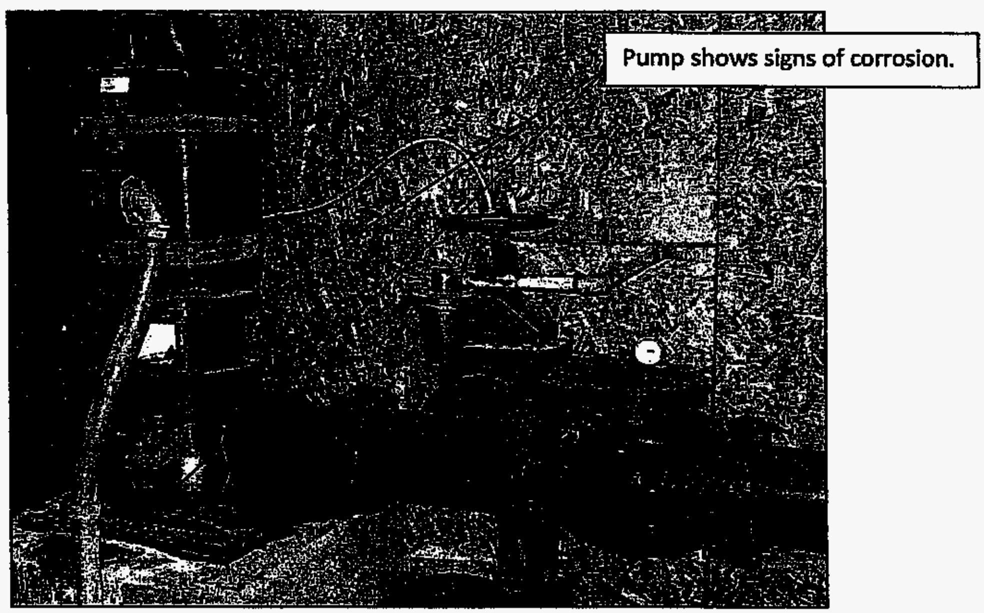


Figure 5-Well Pump & piping

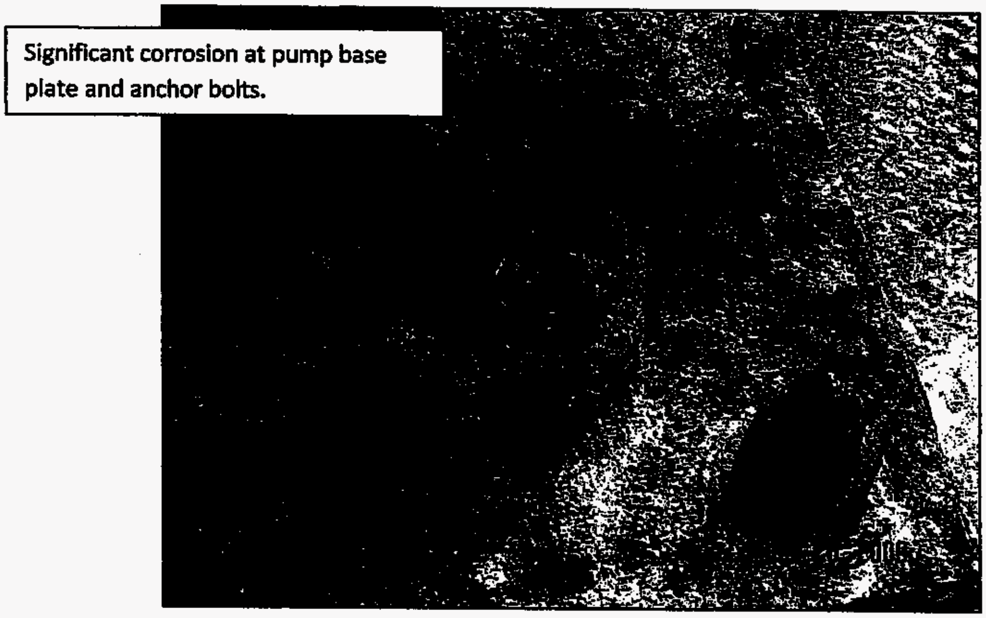


Figure 6-Pump base

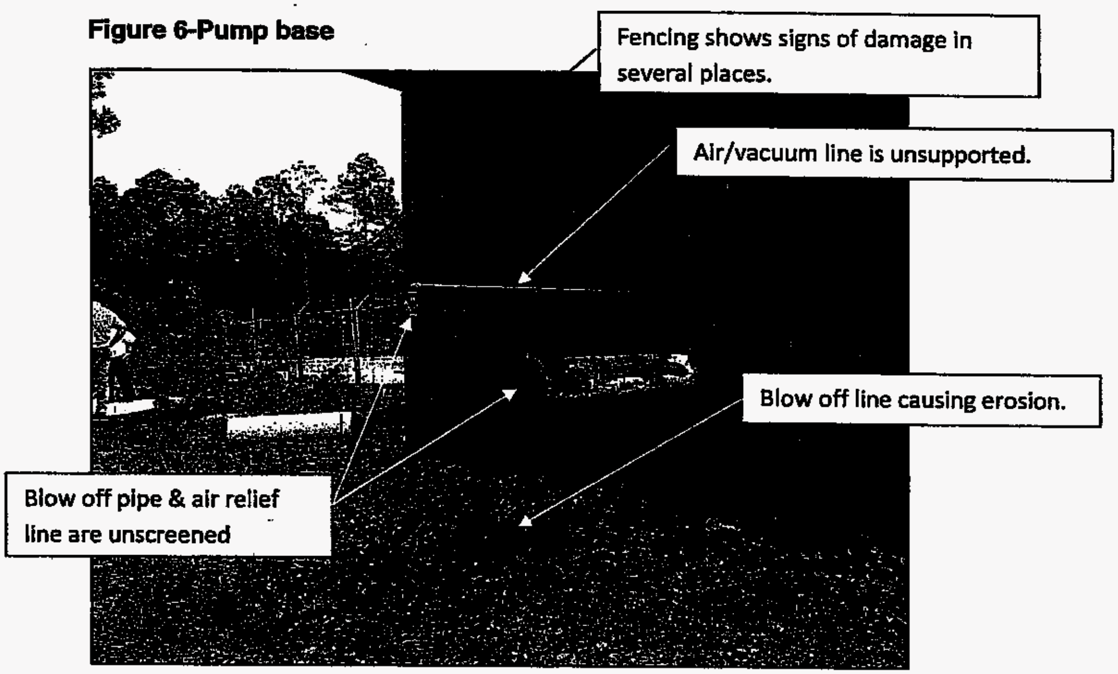


Figure 7-Pump blow off line

Figure 8-Pump station fencing

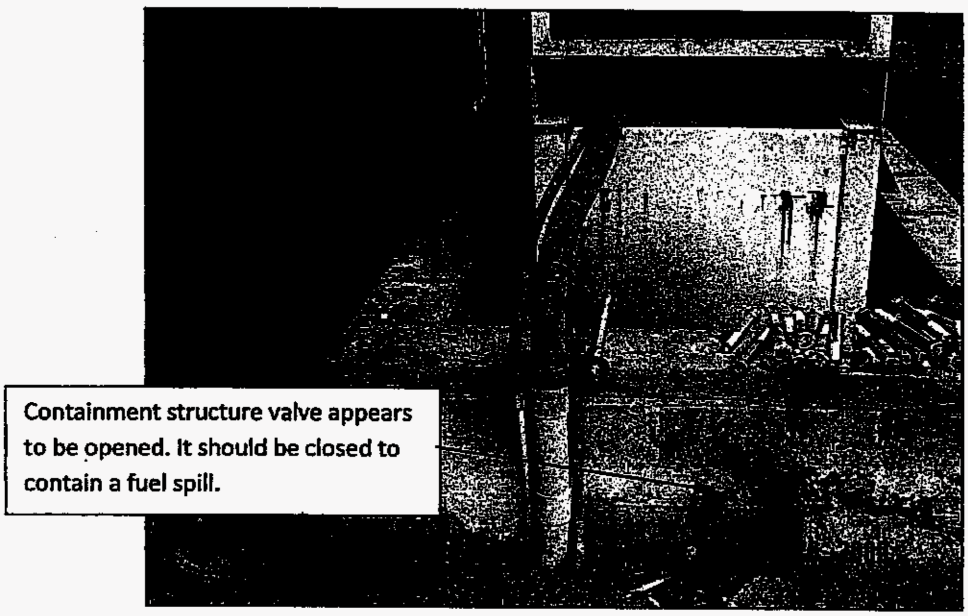


Figure 9-Containment structure

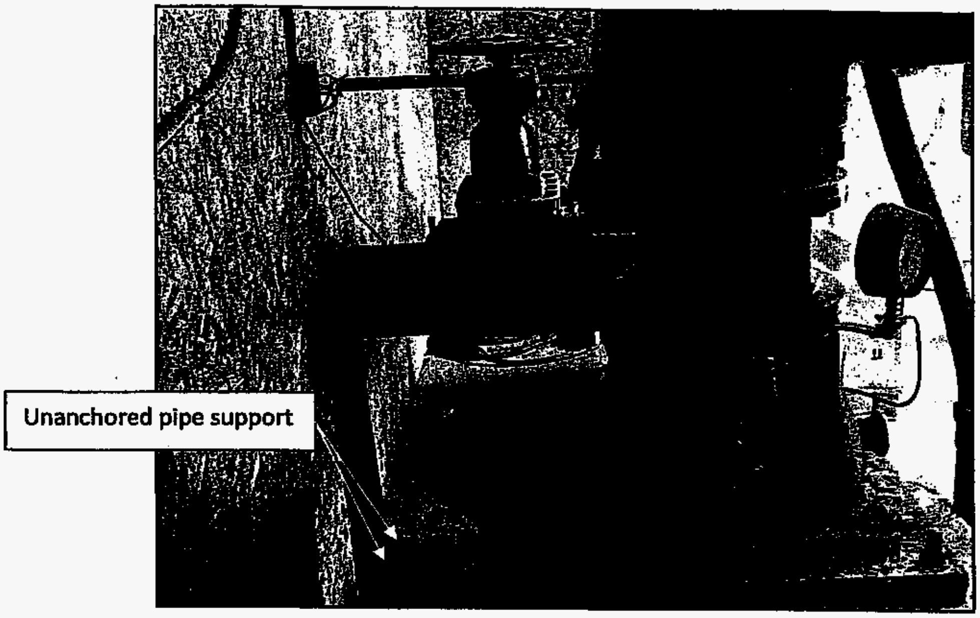


Figure 10-Pump blow off piping

6.1 Well No. 1

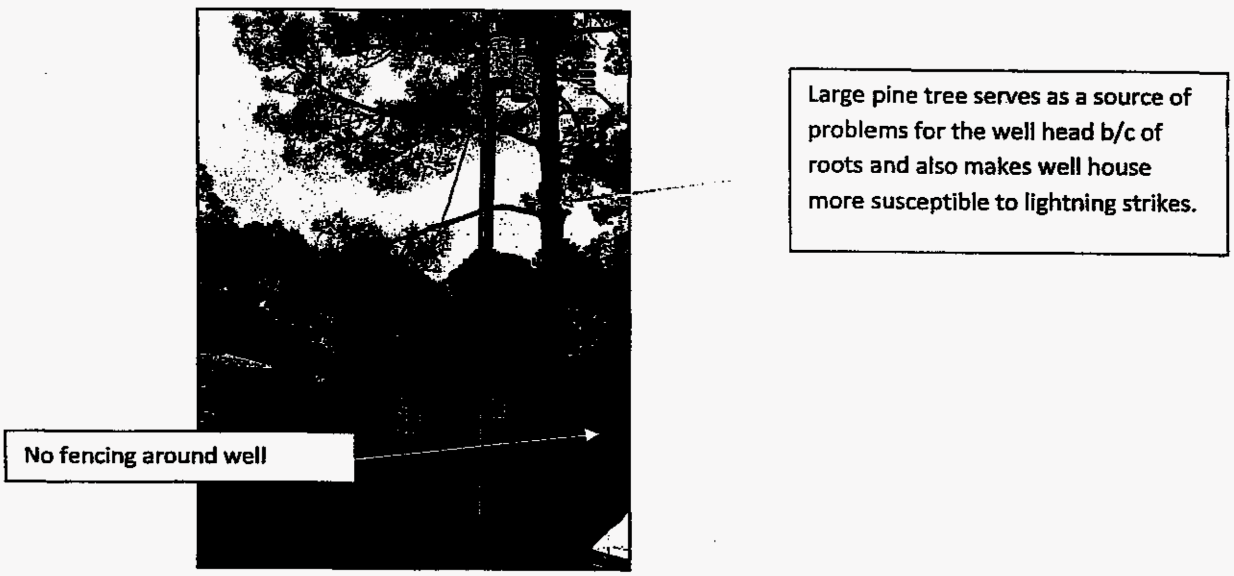


Figure 11-Well No.1

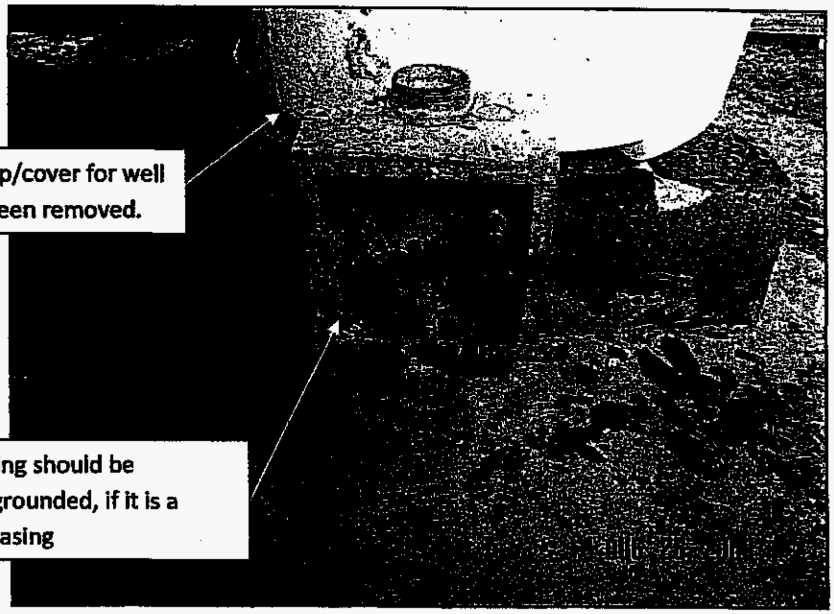


Figure 12-Well head (pump removed)

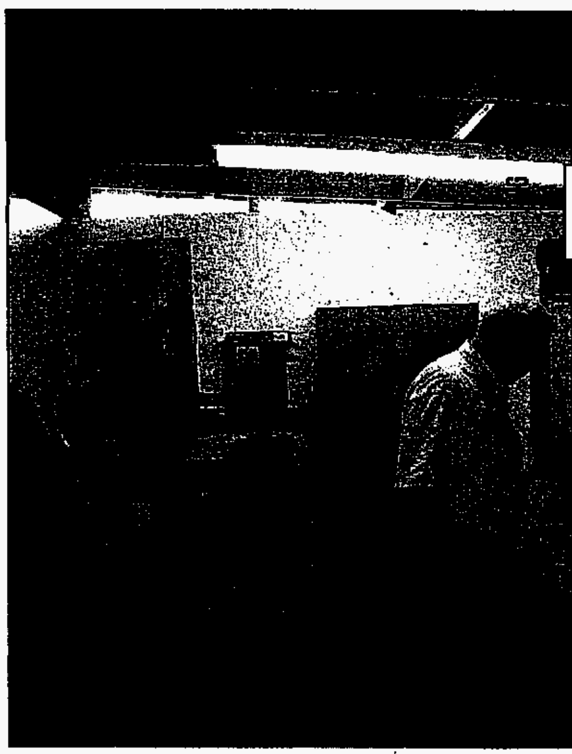


Figure 13-Well house interior

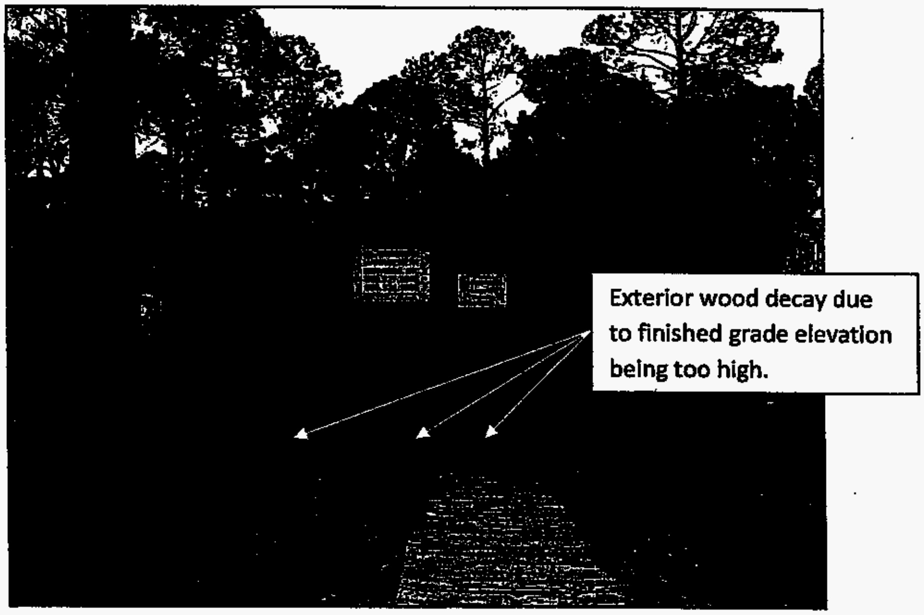


Figure 14-Well house exterior

6.2 Well No. 2

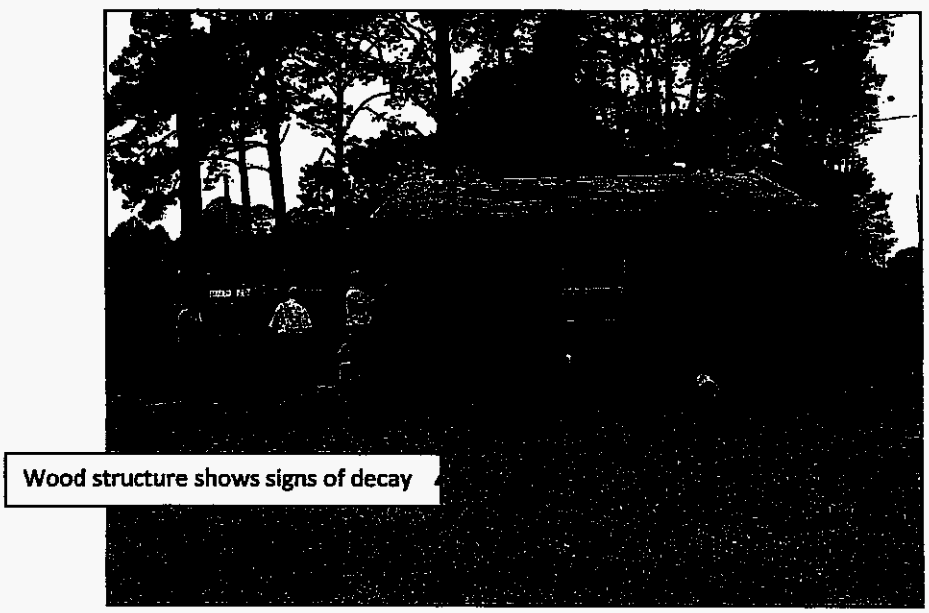


Figure 15-Well No. 2 exterior

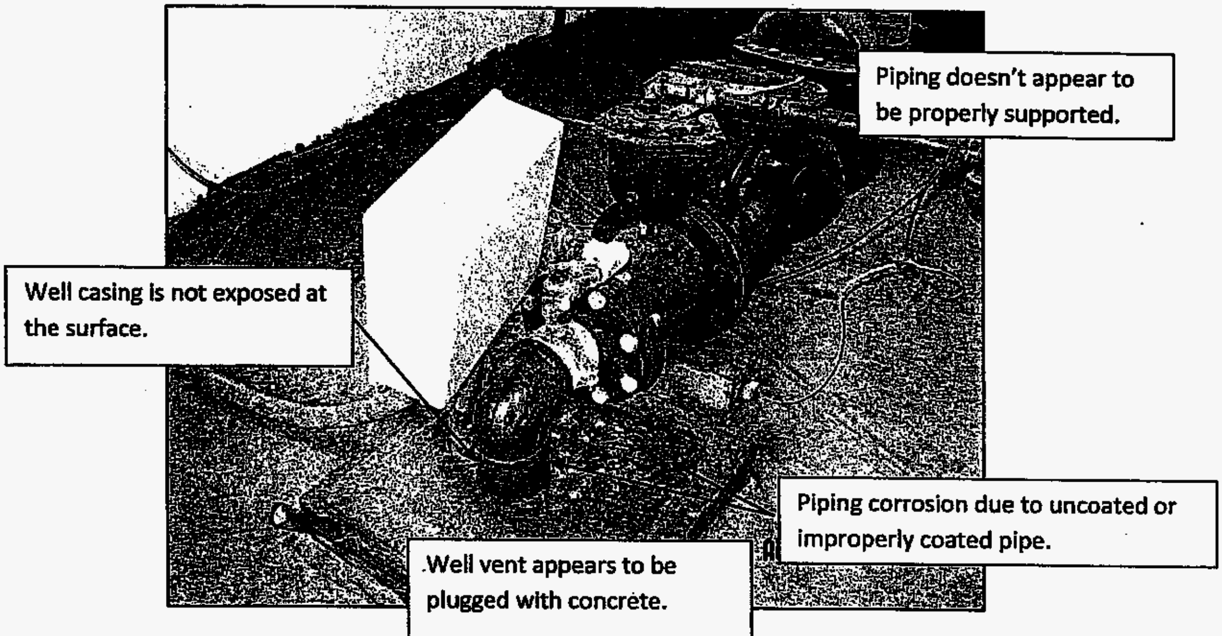


Figure 16-Well head

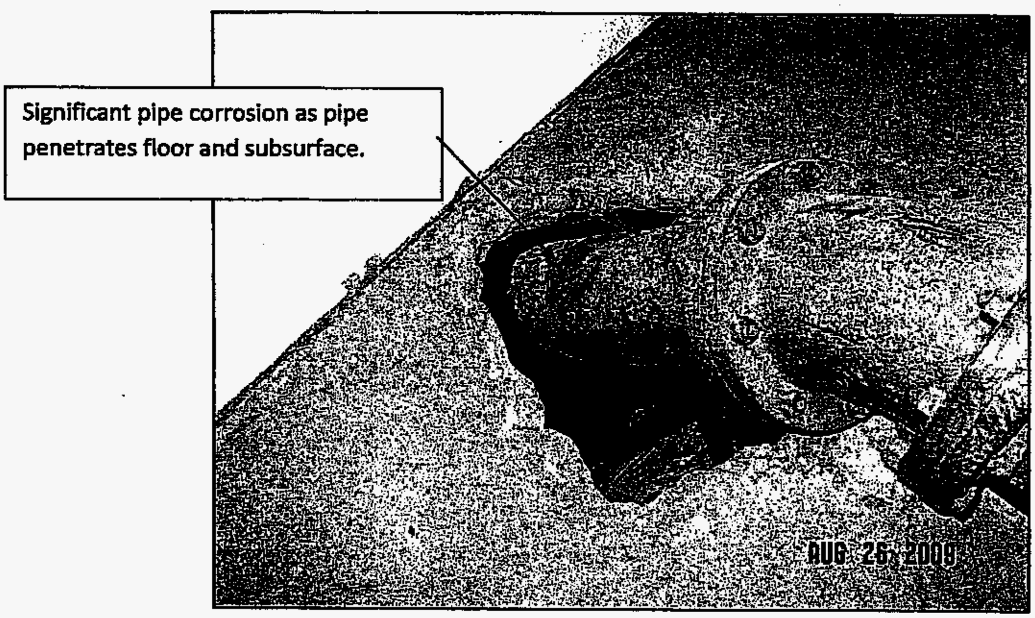


Figure 17-Well pump discharge pipe

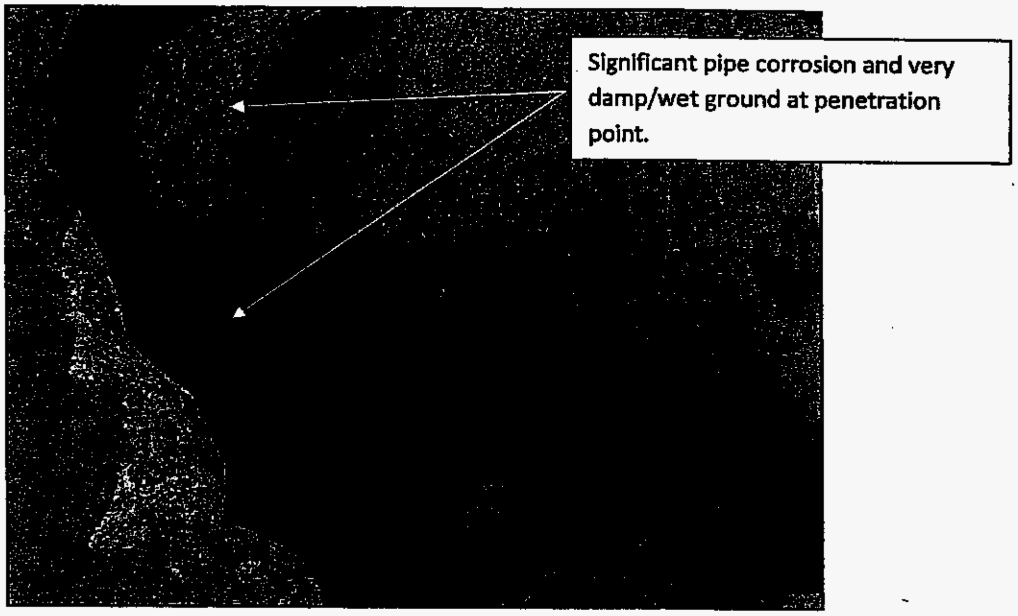


Figure 18-Well pump discharge pipe close-up

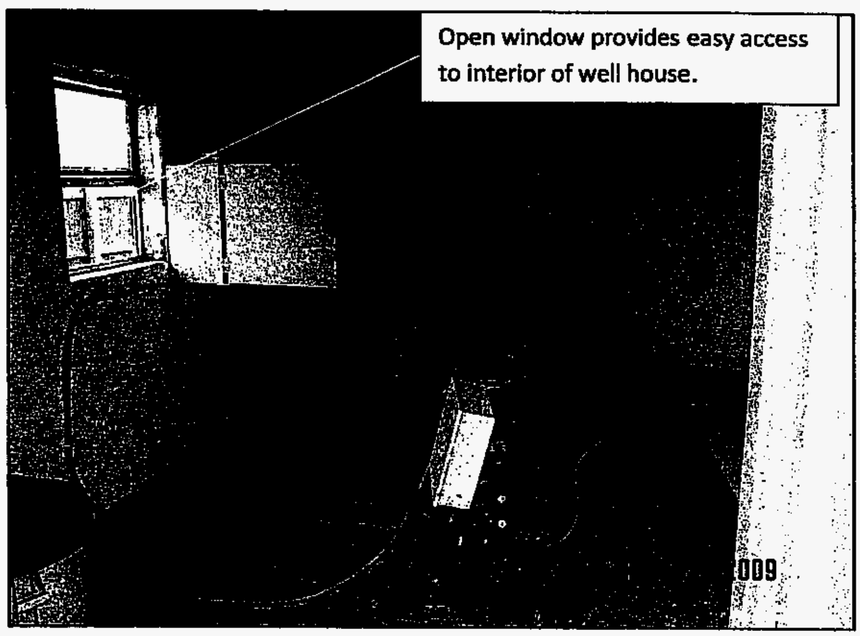


Figure 19-Well house interior

6.3 Well No. 4

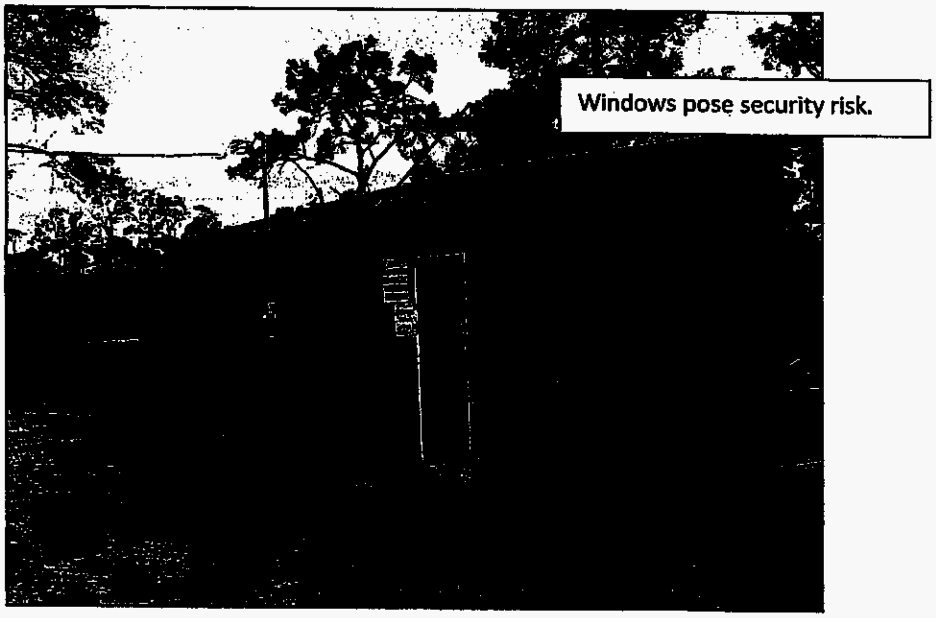


Figure 20-Well house exterior

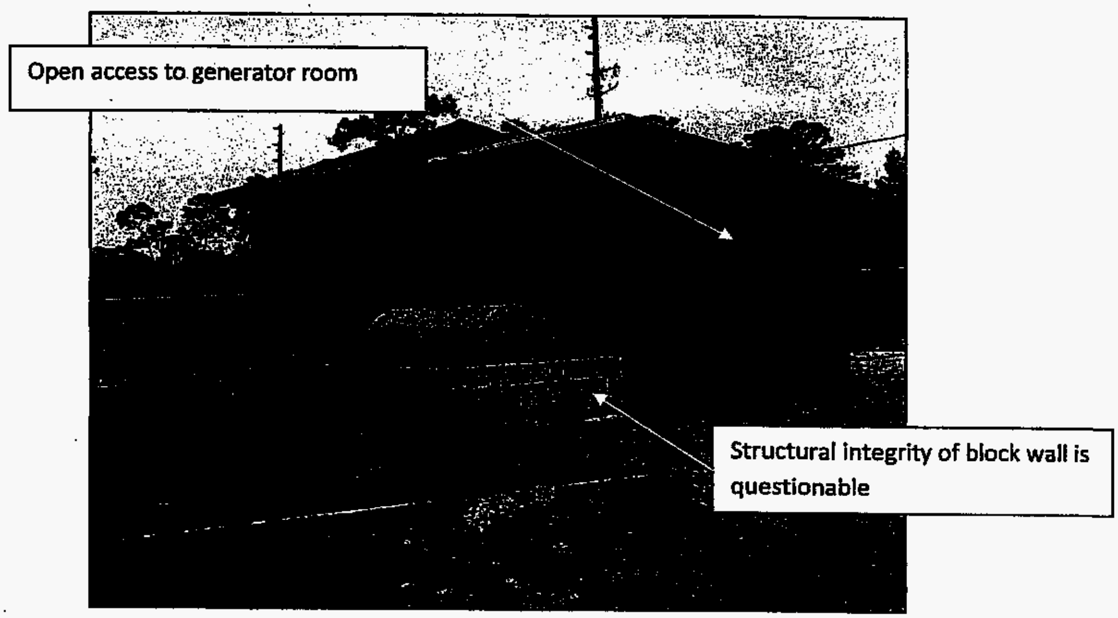


Figure 21-Well house exterior (backside)

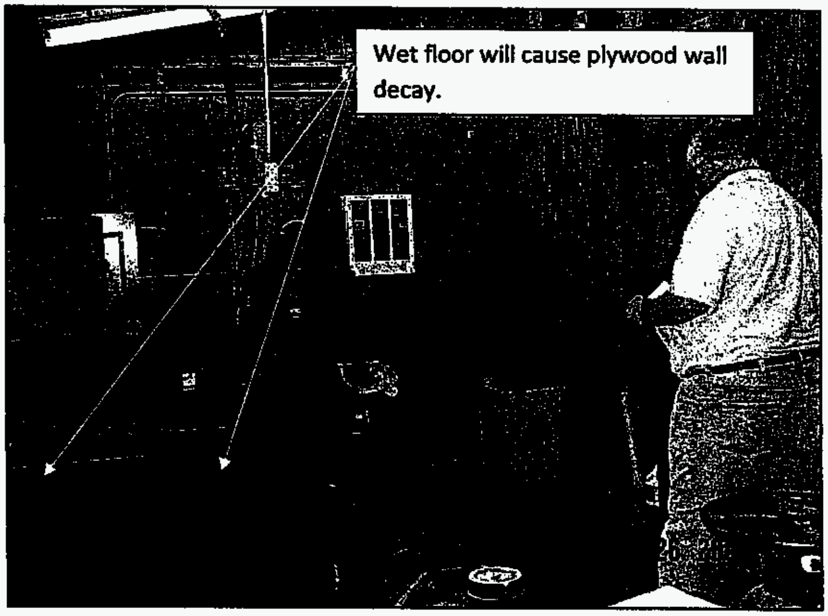


Figure 22-Well pump & piping

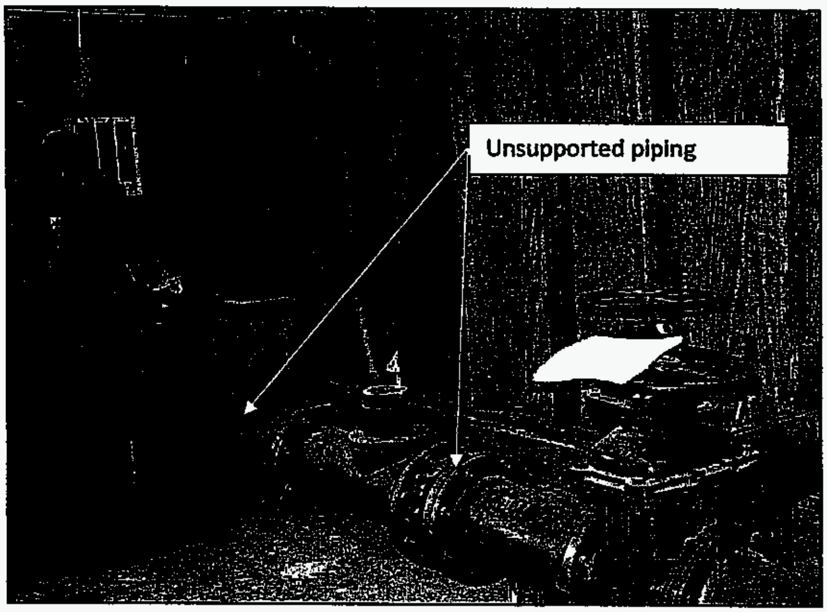


Figure 23-Discharge piping

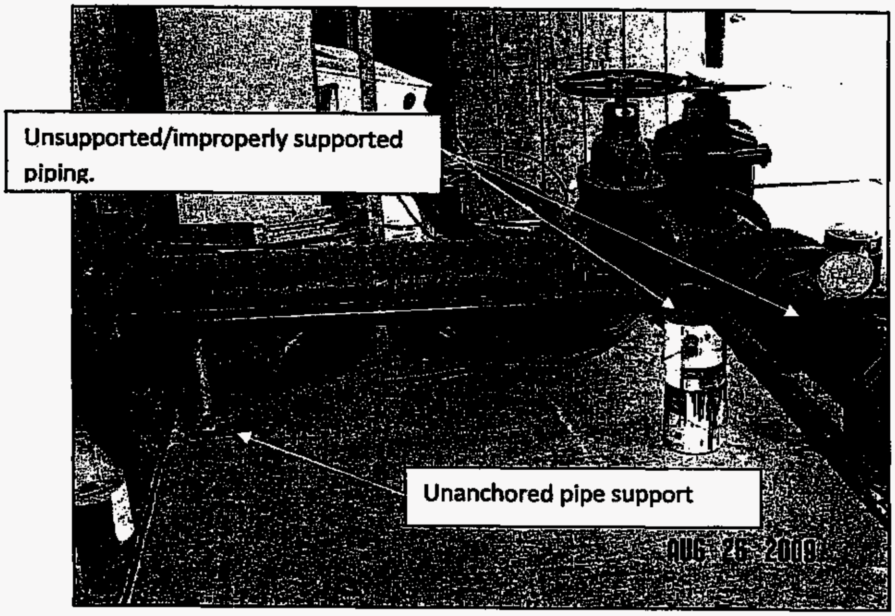


Figure 24-Blowoff piping

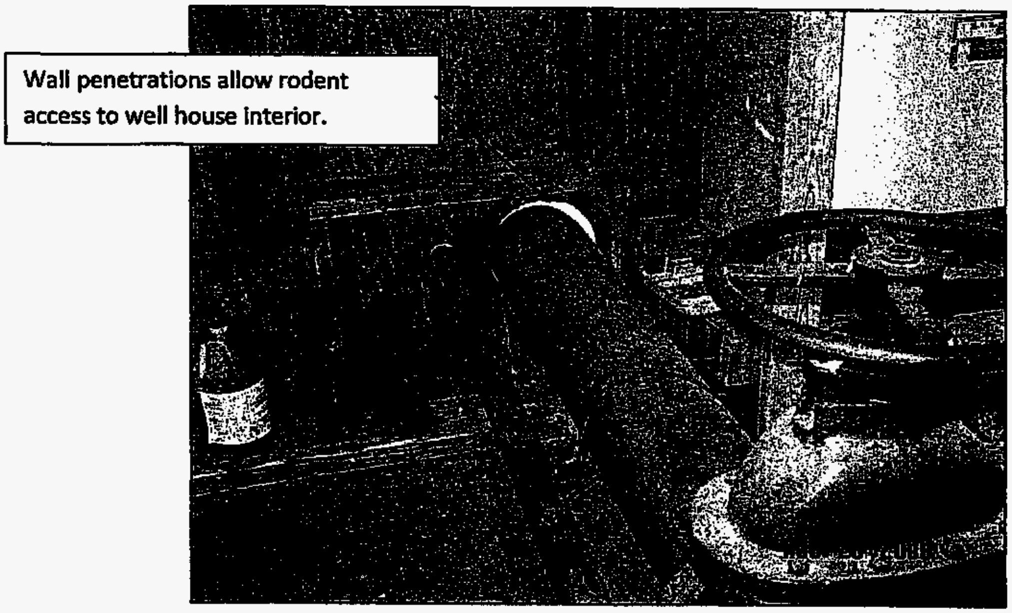


Figure 25-Blowoff piping

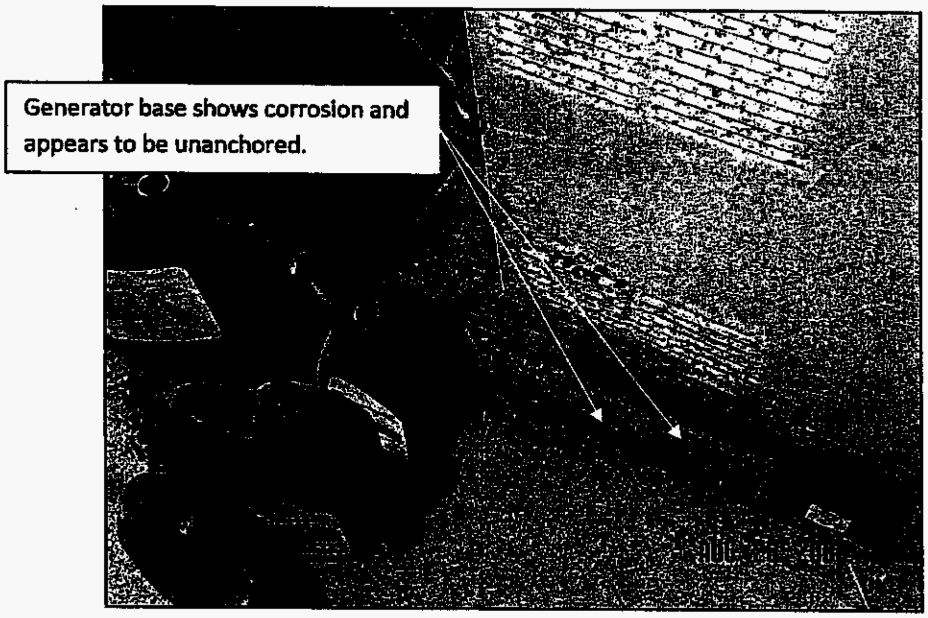


Figure 26-Generator base

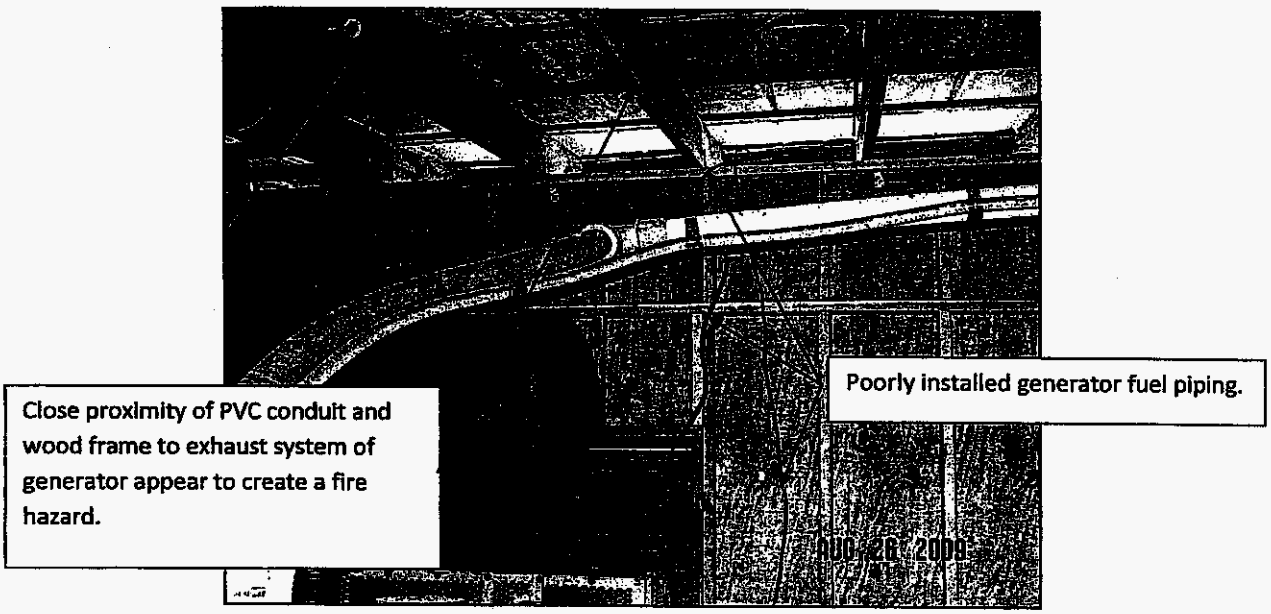


Figure 27-Generator room

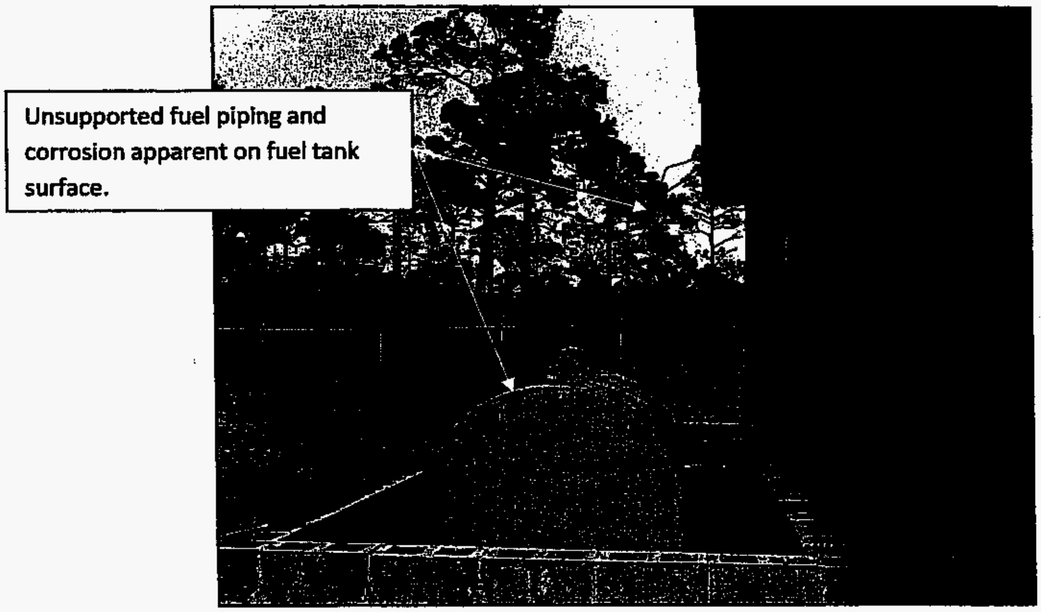


Figure 28-Diesel fuel tank

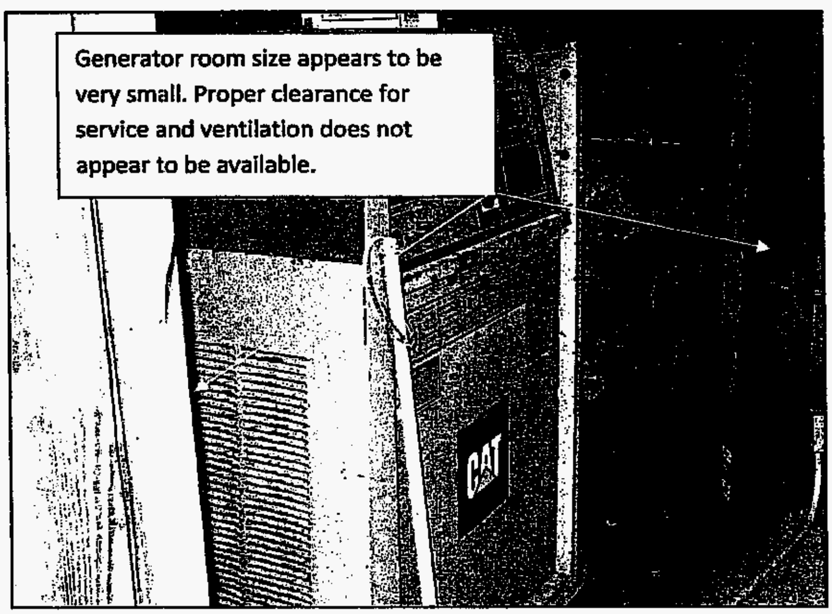


Figure 29-Standby generator

6.4 Well Inspection Check Lists

The following table was developed prior to site investigations at each well site. **Table 4** lists items of particular interest during these visits.

TABLE 4		
WELL NO. 1 INSPECTION CHECK LIST		
Item	**Yes/No/Other**	**Comments & Observations**
Is sanitary seal in place?	Yes	
Does casing extend 18" above slab?	No	
Does well have vent?	Yes	
Does well vent have a gooseneck & screen?	Yes	
General condition of piping & valving	Good	
Electrical System general condition	Fair	
Lighting protection installed?	No	
Is well maintained?	Yes	Replace steel door
Can well pump be removed easily?	Yes	
Identify visual indication of contamination	No	
Are pesticides used in aquifer recharge area?		
What is the well capacity?		
What is the pump type & motor horsepower?	Submersible	
Is a well screen in place?	Yes	
Sample tap and locations?	Yes	
Chemical addition at well? If so, what chemical?	No	
Is a well log available & maintained?	Yes	
Is a wellhead protection plan in place?		
Is well casing PVC?	No	
Is well monitored for fecal & chemical compounds? If so, identify schedule		
Does the well have a meter? If so, what type?	Yes, mechanical	
Is well meter calibrated on regular basis?	No	
Can the well flood?	No	

Has the well ever flooded?	No	
Type of well site security in place?	Fence	Need protective screen over one window.
WELL NO. 2 INSPECTION CHECK LIST		
Item	**Yes/No/Other**	**Comments & Observations**
Is sanitary seal in place?	Other	
Does casing extend 18" above slab?	No	
Does well have vent?	No	
Does well vent have a gooseneck & screen?	?	
General condition of piping & valving	Fair	
Electrical System general condition		
Lighting protection installed?	No	
Is well maintained?	Yes	Replace steel door
Can well pump be removed easily?	Yes	
Identify visual indication of contamination	No	
Are pesticides used in aquifer recharge area?	?	
What is the well capacity?		
What is the pump type & motor horsepower?	Submersible	
Is a well screen in place?	?	
Sample tap and locations?	Yes	
Chemical addition at well? If so, what chemical?	No	
Is a well log available & maintained?	Yes	
Is a wellhead protection plan in place?		
Is well casing PVC?	No	
Is well monitored for fecal & chemical compounds? If so, identify schedule	?	
Does the well have a meter? If so, what type?	Yes, mechanical	
Is well meter calibrated on regular basis?	No	
Can the well flood?	No	
Has the well ever	?	Maybe, signs of flooding

flooded?		
Type of well site security in place?	Y	
WELL NO. 3 INSPECTION CHECK LIST		
Item	**Yes/No/Other**	**Comments & Observations**
Is sanitary seal in place?	Yes	
Does casing extend 18" above slab?	Yes	
Does well have vent?	Yes	
Does well vent have a gooseneck & screen?	Yes	
General condition of piping & valving	Good	
Electrical System general condition	Good to Fair	Showing age; dead generator set w/ fuel tank rusting
Lighting protection installed?	No	
Is well maintained?	Yes	Replace steel door
Can well pump be removed easily?	Yes	
Identify visual indication of contamination	No	
Are pesticides used in aquifer recharge area?		
What is the well capacity?		
What is the pump type & motor horsepower?	50hp vertical turbine	Peerless
Is a well screen in place?	?	
Sample tap and locations?	Yes	Pump Head
Chemical addition at well? If so, what chemical?	No	
Is a well log available & maintained?	Yes	
Is a wellhead protection plan in place?	?	
Is well casing PVC?	?	
Is well monitored for fecal & chemical compounds? If so, identify schedule		
Does the well have a meter? If so, what type?	Yes, mechanical	
Is well meter calibrated on regular basis?	Not needed	
Can the well flood?	No	
Has the well ever	No	

flooded?		
Type of well site security in place?	Bldg	
WELL NO. 4 INSPECTION CHECK LIST		
Item	**Yes/No/Other**	**Comments & Observations**
Is sanitary seal in place?	N/A	
Does casing extend 18" above slab?	No	
Does well have vent?	Yes	
Does well vent have a gooseneck & screen?	N/A	
General condition of piping & valving	Good	
Electrical System general condition		
Lighting protection installed?		
Is well maintained?	Yes	Replace steel door
Can well pump be removed easily?	Yes	
Identify visual indication of contamination	No	
Are pesticides used in aquifer recharge area?		
What is the well capacity?		
What is the pump type & motor horsepower?	75hp vertical turbine	Peerless
Is a well screen in place?	?	
Sample tap and locations?	Yes	Pump Head
Chemical addition at well? If so, what chemical?	No	
Is a well log available & maintained?	Yes	
Is a wellhead protection plan in place?		
Is well casing PVC?	No	
Is well monitored for fecal & chemical compounds? If so, identify schedule		
Does the well have a meter? If so, what type?	Yes	
Is well meter calibrated on regular basis?	N/A	
Can the well flood?	No	
Has the well ever flooded?	No	

Type of well site security in place?	Yes	

6.5 Summary of Well House Inspections

The following paragraphs summarize PBS&J inspections and identify areas where improvements should be considered.

Security Issues

- The condition of the equipment and facility at each well site varies It is important that well facilities are secure from possible sources of damage and contamination. Security at facilities is provided by intact fencing, secure doors, and secure windows/openings People as wells as animals can pose a threat to well sites. PBS&J recommends the security improvements should be incorporated as part of the annual operations budget. The security recommendations are presented below in **Table 5.**

Table 5
Well house Security Recommendations

Item	Description
Well No. 1, 2, 3 & 4	Replace wood doors with steel doors
Well No. 3	Move fence to enclose entire well house
Well No. 3	Repair fencing
Well No. 1	Install security fence
Well No. 1	Close well house openings
Well No. 2 & 4	Add security bars at all windows
Well No. 4	Louver opening requires security enclosure

Facility Maintenance

- Protection of the facilities also comes from the proper installation and maintenance of existing pumps, piping, generators and building facilities should be a priority item. Wood frame buildings are inherently difficult structures to maintain in the water and wastewater industry due to the damp and sometimes wet conditions. All buildings show signs of damage due to interior and exterior conditions. Piping, pumps, generators, fuel tanks and all miscellaneous metals in and around water facilities like these require frequent painting to prevent corrosion. All well facilities show signs of corrosion to equipment. Corrosion, if left unchecked or undetected, will cause premature failure to piping, supports and frame members. This damage can lead to piping leaks and other equipment damage, which poses a risk to continued operations at these well facilities.

- Mechanical piping is not properly supported at the well sites. Piping, when initially installed, may appear to be capable of easily supporting itself, fittings, valves and meters; however, over time gaskets and joints can leak and even crack due to fatigue of supporting heavy items. Pipe supports should be used to properly support runs of pipe, fittings, valves and meters. This reduces the risk of leak and pipe failure.

Table 6 presents well house suggested maintenance items for incorporating into the annual operations budget.

Table 6 Well House Facility Maintenance Suggestions	
Priority	**Description**
Well No. 1, 2, 3 & 4	Add pipe supports
Well No. 1, 2, 3 & 4	Painting
Well No. 1, 2, 3 & 4	Screens for all exposed pipe openings

Flow Metering / Instrumentation

- Proper flow meter installation is an issue at each of the well sites. Typically, flow meters require fully opened gate valves, fittings and other obstructions causing flow disturbances to be a minimum of five pipe diameters upstream and one pipe diameter downstream from the meter and as much as ten pipe diameters upstream and two pipe diameters downstream of the meter. During site investigations it was determined the well flow meter installations do not meet the criteria presented above. The meter selection is not in question here, but simply the meter installation is likely affecting the overall meter accuracy. WMSI provided Flow Meter Calibration Reports by Barrett Supply Inc. indicating the meters are regularly calibrated. The latest report we received indicated the meters are reading between -1% to +4% accuracy. We would anticipate with a properly installed meter, the accuracy to be +/- 2%. Assuming 2% accuracy as our baseline accuracy and comparing to the upper end current accuracy range of 4%, it appears that meter accuracy could be improved by re-piping of the well houses to provide the necessary up and downstream piping criteria. This re-piping would likely require extending piping outside the limits of the well houses. Alternatively, relocation of each flow meter to a new location, meeting the up and downstream piping requirements, outside the well house and installed in a meter box seems to be a more cost effective approach.

- The Individual Water Use Permit requires monitoring of the water level in each production well. This water level data is required to be reported each month, to the District by the Permittee. Our investigation could not identify an instrument at any of the well sites that is being used to gather static water level data from the wells. Typical well designs incorporate a level transducer to continuously monitor water level in the well. Installation of a level transducer at each well site would be a helpful improvement for monitoring and logging historical data.

Table 7 presents well house suggested instrumentation items for incorporating into the annual operations budget.

Table 7
Well House Facility Instrumentation

Item	Description
Well No. 1, 2, 3 & 4	Add Well Level monitor
Well No. 1, 2, 3 & 4	Re-install meters and add transducer head for digital signals to WTF

Well Emergency Power

- Well sites with diesel fuel storage tanks are of particular concern, because these generator fuel tanks store tens and sometimes hundreds of gallons of diesel fuel. If a fuel tank were to rupture in such a close proximity of the well, the fuel could easily contaminate the well site. The diesel fuel tanks were visibly in very poor condition due to rust and corrosion. A new storage tanks should be considered to replace existing. The containment structures were also questionable as to their ability to fully contain a spill. These structures must be capable of holding 115% of the diesel fuel tanks full capacity and also must be structurally sound enough to support the weight of diesel fuel as it pools inside the containment structure. Typically, poured-in-place concrete walls are used for these structures, sometimes the interior of the walls is coated to provide additional protection. It is questionable that unfilled concrete masonry walls would be strong enough to support a pool of diesel fuel and that the walls would not allow leakage in a short period of time. Also, double wall fuel tanks are typically used today.

Standby power is provided at Wells No. 3 and No. 4; however, the generator at No. 3 is reportedly inoperable. Therefore, standby power is only available at a single well source. The generator at Well No. 3 should be replaced to provide redundancy in source water pumping capacity.

Well Head

- Finally, well heads should be visibly extended above finished grade. Design recommendations vary for how far above finished grade the casing need to be; however, at Wells No. 1 and 2, the casing is not visibly extended above grade. This is of concern due to the possibility of contamination. Today it is more and more common that wells are much higher than finished grade in coastal areas. This is due to the possibility of wave action flooding. FDEP regulations currently address the need to raise well heads above flood elevations.

7.0 Current Florida Department of Environmental Protection Well Standards

7.1 The following are a few excerpts from current FDEP standards for water well construction that should be considered, if improvements are to be made at any or all of the well sites:

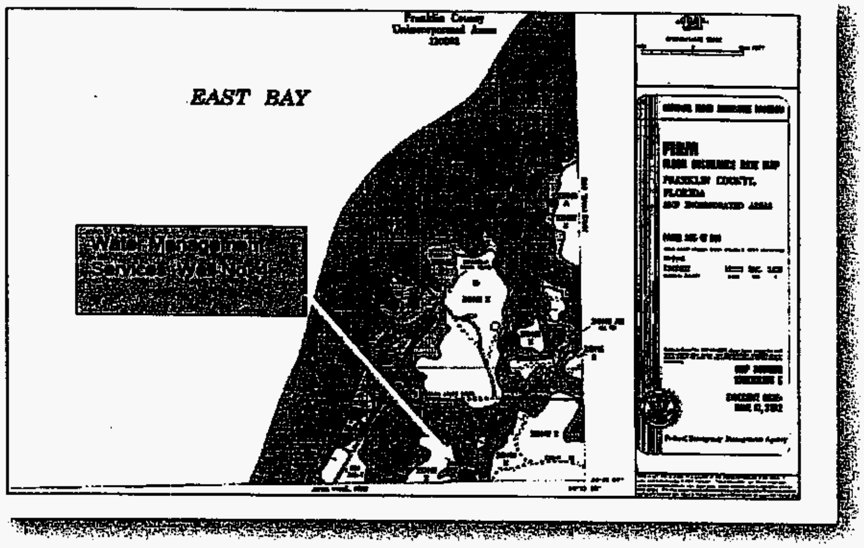
- 62-532: "Well casing shall project at least 12 inches above the 100 year flood elevation and 100 year wave action elevation."
- 62-555.320: "(4) Flood Protection. Community water systems (CWSs) shall be designed and constructed so that structures, and electrical or mechanical equipment, used to treat, pump, or store drinking water…are protected from physical damage by the 100-year flood and, in coastal areas subject to flooding by wave action, from physical damage by the 100-year wave action. Additionally, CWSs shall be designed and constructed so that the aforementioned structures and equipment remain fully operational and accessible during the 25-year flood and, in coastal areas subject to flooding by wave action, the 25-year wave action……"

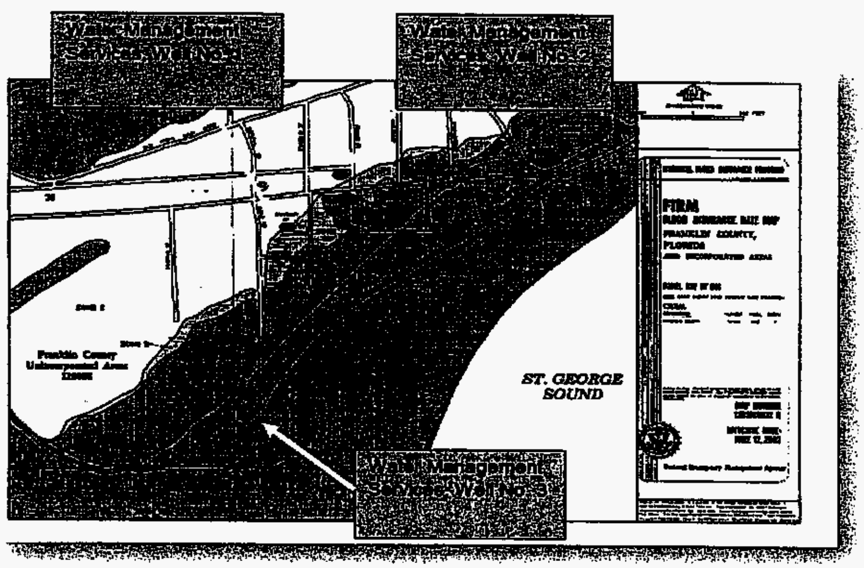
- 62-555.320: "(5) Security. Drinking water treatment or pumping facilities shall be enclosed by fences with lockable access gates, housed in lockable buildings or enclosures, or otherwise protected to prevent tampering, vandalism, and sabotage.
- 62-555.320: "(8b) 1. New or altered discharge piping shall be designed and constructed in accordance with Section 3.2.7.3 in Recommended Standards for Water Works…."
- 62-555.320: "(14a) By no later than Dec. 31, 2005, each CWS serving, or designed to serve, 350 or more persons or 150 or more service connections shall provide standby power for operation of that portion of the system's water source, rate at least equal to the average daily water demand for the system."
- AWWA 4.7.10.3:" Height of casing aboveground. Unless otherwise specified by the purchaser, the casing shall extend not less than 24" above the final ground-level elevation and not less than 24" above the 100-year flood level of record, whichever is higher."

END

APPENDIX 1

FEMA FLOOD MAPS


EAST BAY
FIRM


ST. GEORGE SOUND
FIRM
Water Management Services Well No. 3

PBSJ

Technical Memorandum 4
Existing Water Plant Process Evaluation

To: Mr. Gene Brown, WMS

From: Desi Maldonado — Project: Water Management Services, Inc

CC: Mike Scibelli

Date: January 25, 2010
Revised June 26, 2010 — Job Number:100010111

1.0 Purpose

PBS&J reviewed information, records and interviewed staff during the evaluation process to assess the overall condition, operation and maintenance of the existing water plant processes including aeration and chlorination. The purpose of this Technical Memorandum is to review the water treatment process, outline our findings and recommendations.

2.0 Background

WMSI has one water treatment facility that has a permitted capacity of 0.714 mgd as an annual average and a maximum single day capacity of 1.240 mgd. The combined monthly withdrawal is permitted at 32.7mg for or approximately 1.10 mgd per day. The water plant including ground storage tank, administration office and elevated storage tank are adjacent to each other.

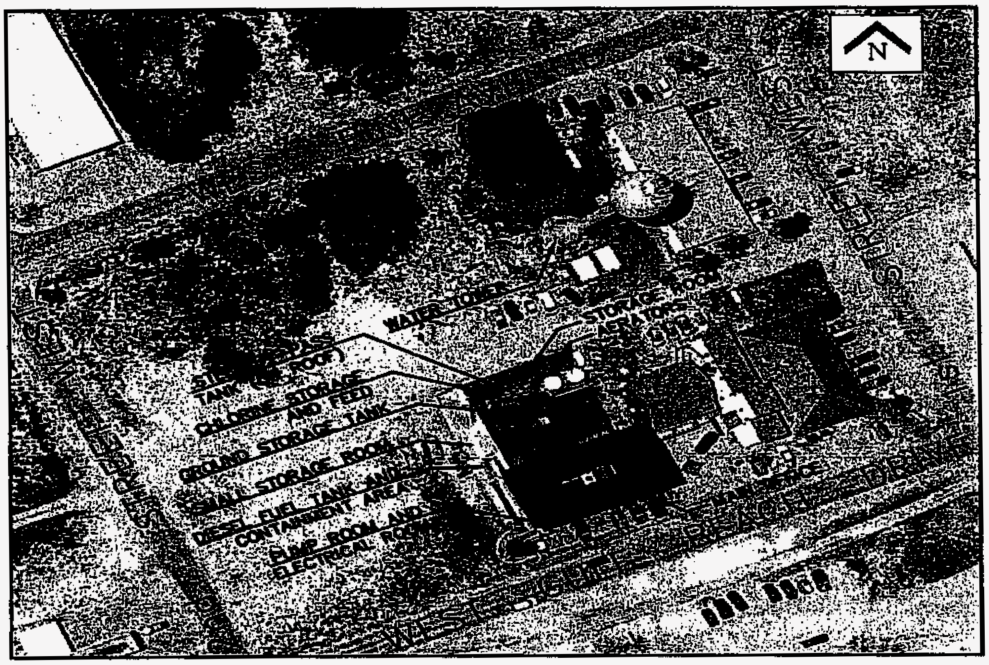


FIGURE 1 St. George Island Water Management Services Treatment Facility

The treatment process consists of tray aeration to remove gases such as hydrogen sulfide, methane and carbon dioxide from well water. Chlorination is used to disinfect the water prior to pumping into the distribution system.

TM 5 discusses PBS&J's structural evaluation of the existing water plant ground storage tank. The analysis included in TM 5 finds the ground storage tank is structurally deteriorating and is likely near the end of its useful service life resulting in either rehabilitation or replacement. Construction of the existing ground storage tank was in 1975 (information provided by WMSI).

3.0 Review of Water Plant Operation

The following summarizes PBS&J's review of water plant operation and management.

3.1 Tray Aeration

Figure 2 illustrates the tray aeration system. One aerator unit was installed in 1987 and a second unit was installed in 2001. Both units are typically in service. The condition of both tray aerator units is in good to fair condition and operable. Paint is peeling from the exterior of the aerators, and stair access up to each aerator is missing handrails.

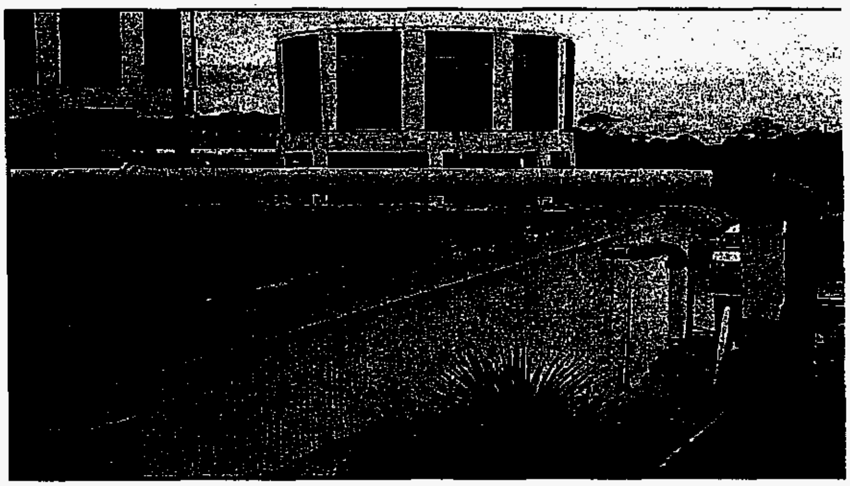


Figure 2- Tray aeration system

We recommend continued use of a tray aeration system in the water plant process.

3.2 Chlorination System

There are two chlorine systems. One system provides chlorine solution to each tray aerator and the second system is used to feed chlorine into the high service pump discharge manifold. The chlorine added at the tray aerators assists in the reduction of hydrogen sulfide and other gases. Chlorine solution fed to the aerators is routed from the chlorine room up to and across the top of the ground storage tank with branch lines and hand modulated valves and riser piping connecting to each aerator. Both of the chlorination systems are located in a room adjacent to the high service pumps. These chlorination systems are vacuum operated, solution type comprised of the following:

- Dual chlorine scale for 150 pound chlorine cylinders dedicated to the primary disinfection
- Dual chlorine scale for 150 pound chlorine cylinder dedicated to chlorine application to tray aerators
- Cylinder mounted vacuum regulators, 100 pound per day. Ecometrics Series 4000
- Auto switchover for primary disinfection chlorinators
- Ejectors
- Chlorine booster pumps, piping and valves
- Chlorine storage room with ventilation fan
- Chlorination room with ventilation fan and leak detector/ audio alarm
- Solution piping with valves

Figure 3- dual chlorine scale, vacuum regulators and ejector for the main disinfection system

Photo 4- dual chlorine scale, vacuum regulators for the chlorine feed to the tray aerators

3.2.1 Disinfection Chlorine Cylinders Review

Chlorine is delivered in 150 pound chlorine cylinders to the water plant. Chlorine Institute states the maximum dependable rate of continuous discharge from a single 150 pound cylinder is 1.5 lbs of chlorine /day/°F. Using this information and assumptions, we estimated the number of active chlorine cylinders required for service to provide chlorine for primary disinfection under various flow and ambient temperatures. Results of our evaluation indicate that WMSI's current operation with one active cylinder on line is adequate. Assuming the chlorine dose remains the same, as the flows increase additional cylinders will be needed. Refer to **Table 1**.

Table 1
Estimate of Chlorine Usage for Primary Disinfection

Flow rate, mgd	Estimated pounds of chlorine /day	Cylinders on line at 40°F	Cylinders on line at 90°F
0.400 current average day	45	1	1
0.714 permitted average day	80	2	1
1.240 permitted peak day	140	3	2

Table 1 Assumptions:

1. *Chlorine dose: 13.5 mg/l (from staff) to achieve 3 mg/l free chlorine residual*
2. *Ambient temperature: 40°F as average daily low temperature, 90°F as the high.*
3. *Without air circulation around the cylinder*

Providing additional cylinders can be accomplished by connecting the gas, take-offs from each cylinder into a common manifold so that chlorine from multiple cylinders can feed a common chlorinator.

3.2.2 Disinfection Chlorinator Capacity

The disinfection chlorination system must be able to deliver enough chlorine to maintain a free chlorine residual of 2 mg/l at maximum flow under maximum chlorine demand. Current capacity of each chlorinator has a capacity of 100 pounds per day. When peak day flows reach 0.85 mgd or above, the chlorinators will need to be replaced with larger capacity units capable of feeding 200 pounds per day.

3.2.3 Chlorination_Control

Operation of both chlorine systems is manual, by hand adjustment of chlorine rotameters to increase or decrease the quantity of chlorine fed. This is an inefficient method of controlling chlorine. Manual adjustment creates the opportunity to "over or under feed" chlorine. Because the chlorine demand may change with flow, or water quality or combination of flow and water quality may change, automatic controls are typically utilized for primary disinfection chlorination system. Currently, if chlorine demands change and the operations staff is unavailable to make the appropriate adjustments, improper feeding of chlorine may occur. Over feeding of chlorine during the disinfection will increase operational costs with excessive chlorine delivered to the distribution system. As an example, at the design flow rate of 0.714 mgd, for every 1 mg/l of chlorine that is over fed, adds approximately $450 per year in operational costs (assumes $0.20/ pound for chlorine). This extra chlorine will likely increase the production of disinfection byproducts in the distribution system. Conversely, feeding too little chlorine will compromise the disinfection process and lower the germicidal effectiveness.

Recommendation: At a minimum, installation of automatic chlorine pacing by flow is recommended to improve the disinfection process. The best method of feeding chlorine for disinfection is the use of a compound loop controller to pace chlorine feed on flows and desired free chlorine residual. This method uses the information obtained from an on-line chlorine analyzer. All instrumentation signals, flow pacing, chlorine residual feedback from the analyzer and use of a residual set point control from the operations staff are input into set point controller. The controller sends out only one analog output signal that issued to control the rate at which chlorine is fed.

3.2.4 Existing chlorine injection points and CT values

Currently, chlorine solution is injected into the high service pump discharge manifold .Because the groundwater is not under the direct influence of surface water, the disinfection removal rate required is Log 2 (99 percent) or Log 3 (99.9 percent) removal of viruses. The effectiveness of chlorine and its ability to disinfect is directly proportional to the concentration of chlorine and the time the chlorine is in contact with the organism or CT time. CT is measured in milligrams-minutes per liter as follows for virus removal:

(Disinfectant free residual, in mg/l) or C x (Contact time in minutes) or T = CT, in mg-min/l

Both pH and temperature of water have an impact on the CT time required as displayed in **Table 2.**

Table 2
CT values for inactivation of viruses

Temperature		Log Inactivation			
		2.0		3.0	
		pH		pH	
(°C)	(°F)	6-9	10	6-9	10
0.5	32.9	6	45	9	66
5	41	4	30	6	44
10	50	3	22	4	33
15	59	2	15	3	22
20	68	1	11	2	16
25	77	1	7	1	11

Source: Guidance Manual for Compliance with filtration and Disinfection Requirements for Public Works using surface water sources, EPA 1990

Using **Table 2** information, we compared the St. George Island finished water at a pH of 7.0 with a water temperature of approximately 80°F. The results indicate for both a 2.0 and a 3.0 log inactivation of viruses, **a minimal CT of 1mg/l-min is required at all times.**

Using information from the field visit and information obtained from the WMSI staff, PBS&J calculated the CT value to determine if adequate chemical disinfection was being provided before the first customer. The calculation of CT used the following information:

1) Chlorine free residual: 1,2 and 3 mg/l
2) Pipe size : 12-inch ductile iron, pressure class 350 (IP diameter equals12.64 inches)
3) Distance to first customer: 75 feet
   Flow rates as shown in **Table 3**
4) First customer is adjacent to water plant on West Gulf Beach Drive

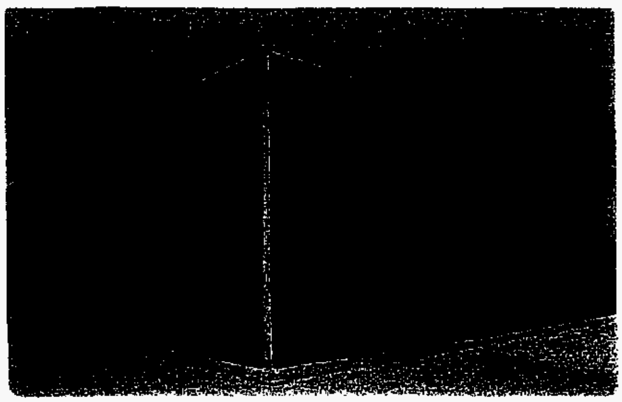
Table 3
Estimated CT values for finished water

Flow rate, mgd	Contact time, minutes	CT, mg/l-min Free Chlorine residual, mg/l		
		1.0	2.0	3.0
0.400 current average day	1.76	1.76	3.52	5.28
0.714 permitted average day	0.99	0.99*	1.97	2.96
1.240 permitted peak day	0.57	0.57*	1.13	1.70

*** Below the minimum CT**

Table 3 indicates that with the exception of two conditions, the CT value of 1.0 can be met based upon our assumptions and calculations. When flows exceed 0.714 mgd, the free chlorine residual will need to be increased to approximately 2.0 mg/l to meet the CT values. If the existing ground storage tank remains in service, we recommend including the ability to add chlorine directly to the inlet of the ground storage tank. By adding chlorine in the ground storage tank for primary disinfection, this will provide operations flexibility while increasing the contact time to the first customer.

To accomplish the addition of chlorine solution to the ground storage tank would require small chlorine solution piping modifications, and the addition of a floor mounted chlorine diffuser. We also recommend the addition of NSF certified baffling to create a serpentine flow path thereby reducing short-circuiting of flows to insure the detention time is being achieved. Below is a typical baffle system.



Environetics Inc. Baffle System

3.3 Chlorination System Recommendations

Priority capital improvements for the chlorination systems include:

1. Install common manifolding on the primary disinfection chlorine system to increase the number of chlorine cylinders in service.
2. Upgrade the disinfection chlorinator capacity to 200 pounds per day.
3. Install automatic chlorine pacing of primary disinfection by flow to improve the disinfection process and lower operational costs.
4. Install chlorine diffuser system, piping and valves to provide operations reliability and increase the contact time to the first customer.
5. Install baffling to the ground storage tank to prevent short-circuiting thus insuring adequate detention time to achieve proper levels of disinfection. .

4.0 High Service Pumping

Variable frequency drives are used to control three high service pumps at the plant. A forth pump or Jockey pump was out of service at the time of our inspection. The variable speed high service pumps are horizontal split case design and operate to maintain a set point pressure in the system of approximately 85 psi. The electrical system's (including the standby generator) is susceptible to a storm surge and/or wave action flooding. PBS&J surveyed the finished floor of the existing building and compared the results to the 100-year storm. **Table 4** has information on the current elevations relating to storm, high service room and top of the existing ground storage tank.

Table 4
Water Plant Elevation Information

item	elevation
100 year storm or wave action elevation	17.0 NGVD29
Finished floor of the high service pump room	10.59 NGVD29 DATUM
Top of GST	21.59+/-

Because the high service pumps and their associated electrical gear is subject to flooding we recommend positioning the pumps and electrical gear at a higher elevation to maintain reliable water service should a flood occur. <u>Relocation of pumps and equipment is a priority item that should be included in the CIP.</u>

If the pumping equipment is moved to the top of the existing GST, reconfiguration of the pump suction piping to meet current Hydraulic Institute Standards to prevent intake vortexing and pump cavitation will be required.

The feasibility of moving the high service pumps and electrical gear to the top of the existing GST requires a coordinated maintenance of plant operations (MOPO) plan that presents several challenges including:

- Structural modifications to the GST may be required to support the pumping equipment and to accommodate these changes the GST may have to be out of service for an extended period.
- Temporary electrical service will be required
- Temporary or temporary GST facilities may be required during the construction to provide sufficient water for fire protection. The cost for temporary storage will increase the overall project costs.
- Relocation of the existing pumping equipment will likely require temporary pumping equipment and controls to be installed. The costs of this temporary pumping will increase the project construction cost and complexity.

5.0 Water Plant Recommendations and Construction Estimate

Recognizing the deteriorating condition of the ground storage tank and the unforeseen issues and costs associated that may be related to its rehabilitation, and considering the cost to move the existing high service pumps and electrical gear out of the flood zone, with all of the associated MOPO issues related to maintaining water service during tank rehabilitation and equipment relocation, PBS&J recommends the construction of a new ground storage tank with new pumping and electrical gear.

Table 5 provides a detailed breakdown of priority items recommended for the water plant improvement with estimated construction costs.

Table 5 **Water Plant Improvements** **Estimate of Construction**	
Chlorine system cylinder manifolding	$500
Replace cylinder mounted chlorinators	$2,500
Clearwell baffling	$15,000
Chlorine diffuser	$4,000
New high service pumps	$100,000
Relocate generator	$7,500
Generator fuel containment	$3,000
New pump/plant control panel	$93,500
Ground storage tank installation	$389,000
Subtotal	$615,000
Engineering @10 %	$61,500
Mobilization, site work, yard piping, electrical, contractor bond and OH&P @38%	$233,625
Total	**$910,125**

END

PBSJ

Technical Memorandum 5
Evaluation of Water Plant Structures

To: Mr. Gene Brown, WMS

From: Desi Maldonado | Project: Water Management Services, Inc

CC: Mike Scibelli

Date: December 18, 2009/ Revised June 27,2010 | Job Number:1000101

1.0 Executive Summary

PBS&J conducted an inspection of major process structures at the St. George Island Water Management Services water treatment complex for the purposes of evaluating their condition. We have significant concerns relating to the integrity of the GST structure including, but not limited to the following:

- Post-tensioning strands in the precast 8" hollow core roof panels are exposed and corroding. Corrosion of the steel reduces the structural integrity of these panels.
- Hydrogen sulfide and chlorine odor is present inside the main office indicating a crack is present in the GST common wall shared with the office. Cracking can affect the structural integrity by allowing chlorinated water come in contact with structural steel.
- Random intermittent cracking of the GST exterior perimeter concrete beam running along the top of the ground storage tank may be evidence of a change in loading and bearing conditions or degradation of materials.
- Several GST joints on the main walls between tilt-up concrete wall panels are leaking.
- Hidden structural defects may exist which could impact the service life of the tank.
- Water tightness of the roof is questionable.

After touring and inspecting each structure, it is clear the ground storage tank is in need of replacement or remediation. Our team developed and evaluated four ground storage tank (GST) options along with costs to either replace or repair the tank. These Options include:

Replacement or Remediation Options

1. Construct new 500,000 gallon, dual chamber GST, conversion of existing GST to workshop and abandon elevated storage tank.
2. Construction of new 325,000 gallon dual chamber GST, abandon existing GST and continue use of elevated storage tank and relocate one aerator.
3. Demolish GST and construction of new tank at same location and maintain use of elevated storage tank
4. Refurbish GST and maintain use of elevated storage tank.

Cost Estimates and associated ranking of Options are presented in **Table 1**.**The cost estimates presented below where used to evaluate the options prior to the development of any priority CIP estimate preparation.**

Table 1 Ground Storage Tank Options		
Ground Storage Tank Options	Estimated construction costs	Value Ranking
Option 1	$2,028,990.00*	2
Option 2	$1,706,330.00*	1
Option 3	$708,187.00	3
Option 4	$311,684.00	4
* includes the cost of high service pumping equipment		

Each option was value ranked based upon a weighted percentage assigned. The value factors and weighted percentages are presented below. The ranking results are presented in **Table 2.**

- Cost- 25%
- Reliability- 30%
- Operational Flexibility- 30%
- Water Quality- 15%

Table 2 Options Ranking					
Item	Weighted %	Option 1	Option 2	Option 3	Option 4
Cost*	25	1	2	3	4
Reliability*	30	3	3	2	1
Flexibility*	30	3	3	2	1
Water Quality*	15	3	3	2	1
Weighted score*		2.5	2.75	2.25	1.75
Rank**		2	1	3	4
*Weighted score legend :1-Least Desirable to 4- Most Desirable,** Rank score legend: 1-Most Desirable to 4- Least Desirable					

Recommended Option

PBS&J recommends Option 2, construction of a new 325,000 gallon GST. This option provides the best overall ranking of the options investigated. The recommended GST tank design would consist of the following features:

1. New pre-stressed concrete tank , flat roof design with interior columns
2. Internal hydrostatic wall separating the tank into dual chambers
3. Draft aerators mounted on top of tank and vents
4. Interconnection gates to allow isolation of chambers and pump chambers
5. Access hatches, level indicators and exterior stairway to the top of GST
6. Three manways
7. Concrete slab
8. Vertical turbine pump mounting on the top of tank (above 100 year storm or wave action elevation of 17.00 to provide reliable water pumping during a storm event)

With construction of the new 325,000 gallon, use of the elevated storage tank will continue and provide additional water storage for fire protection. PBS&J recommends the current diesel fuel containment wall be demolished and rebuilt to provide adequate storage volume with proper structural integrity.

2.0 Background

On March 30, 2009, Brice R. Nist, P.E., conducted an inspection of the existing ground storage tank. In the final inspection report, Mr. Nist found that the ground storage tank (GST) was in poor condition and recommended the construction of a new tank. The report recommended the tank could be rehabilitated and requested Crom Engineering and Construction Services (CECS) prepare an evaluation and an estimate for performing repairs. CECS performed its evaluation and estimated the rehabilitation of the GST would cost approximately $124,000.We believe this cost estimate to <u>rehabilitate the GST is too low</u> and does consider all related factors such as temporary water storage systems, temporary piping, maintaining of operations of the water treatment process. This issue is discussed in more detail later.

The scope of rehabilitation recommended in the Nist Report included:

- Remove and replace the build-up roofing system
- Remove two hollow-core roof panels and replace with new
- Clean out all joints between roof panels and refill with flexible joint sealant
- Inject wall leaks at the closure points of tilt-up panels
- Fabricate and replace the hatch curb with a new fiberglass cover
- Sandblast and paint tank roof and 2 feet down the interior wall
- Chip and patch miscellaneous spalled concrete on the tank walls and roof

The service life of the GST after the proposed rehabilitation is unknown. Typical service life of concrete structures is 50 years. Using 50 years as the target, the GST is approaching 70% of its service life. We cannot confirm if the remediation will extend the service life of the GST beyond 50 years.

3.0 Method of Investigation and Elevation Information

PBS&J conducted a site inspection of the entire facility on August 26 and August 27, 2009. The facility operators gave engineers from PBS&J a tour of the facility with access to each building for the investigation. The investigation was a visual non-destructive inspection.

Each structure was inspected for visible defects such as cracks, erosion, damage, corrosion, exposed reinforcing steel, and stains. **Appendix A** includes aerial mapping of the site. The defects were photographed and included with comments in the summary of observations in **Appendix B**. **Appendix C** is the site key plan with pictures referring to the location from which they were taken. **Appendix D** is a full archive of all photos taken during the inspection with the date and timestamp of the pictures.

PBS&J surveyed the finished floor of the existing building and compared the results to the 100-year storm. Below is information on the current conditions.

100- Year storm or wave action, elevation: 17.0 NGVD29

Finished floor of the high service pump room: 10.59 NGVD29 DATUM

Top of GST: 21.59 +/- (assumes roof is 11.00 feet above finished floor of pump room)

The high service pumping station and associated electrical gear is subject to flooding and we recommend relocating the pumps and electrical gear to a higher elevation to maintain service during a flood in accordance with Ten States Standards. The pumping equipment should be located a minimum of three feet above the 100-year flood to protect the equipment. The GST elevation appears to be acceptable to prevent flooding via water entry through the roof hatches or vents.

4.0 Description of Structures

The St. George Island Water Management Services water treatment facility complex is comprised of several small buildings and structures (see **Figure 4.1**). The largest structure is the ground storage tank (GST), which has a capacity of approximately 292,000 gallons, and is approximately 64'x64'x11' (clear well interior dimensions). The tank was built in 1975 and the construction is 8" thick tilt-up concrete wall panels and 8" concrete hollow core roof panels. There is an interior baffle wall running east-west at full-height that supports the hollow core panels at the middle of the GST. Water is fed into the tank by first passing through two aerators located on the roof of the GST. Located on the roof of the GST is a temporary 20,000 gallon stainless steel bypass tank (approx 6'Wx15'Lx4'H) used to bypass the GST during periods of GST cleaning. On the north side of the GST are two rooms that share a wall with the GST. These rooms have concrete masonry unit (CMU) walls and traditional common framed wood roof with asphalt shingles. The room to the west is the chlorine storage and feed room. The room to the east is used for storing spare parts used for maintenance of the system. To the west of the GST is a small (approx 6'Wx15'Lx10'H) wooden storage room and a diesel tank with a short (approx 3'H) CMU fuel containment wall. To the south of the GST are the high service pump room, electrical room, and main office. To the north east of the GST is a 110 feet tall water tower used for additional storage (approx 150,000 gallons).

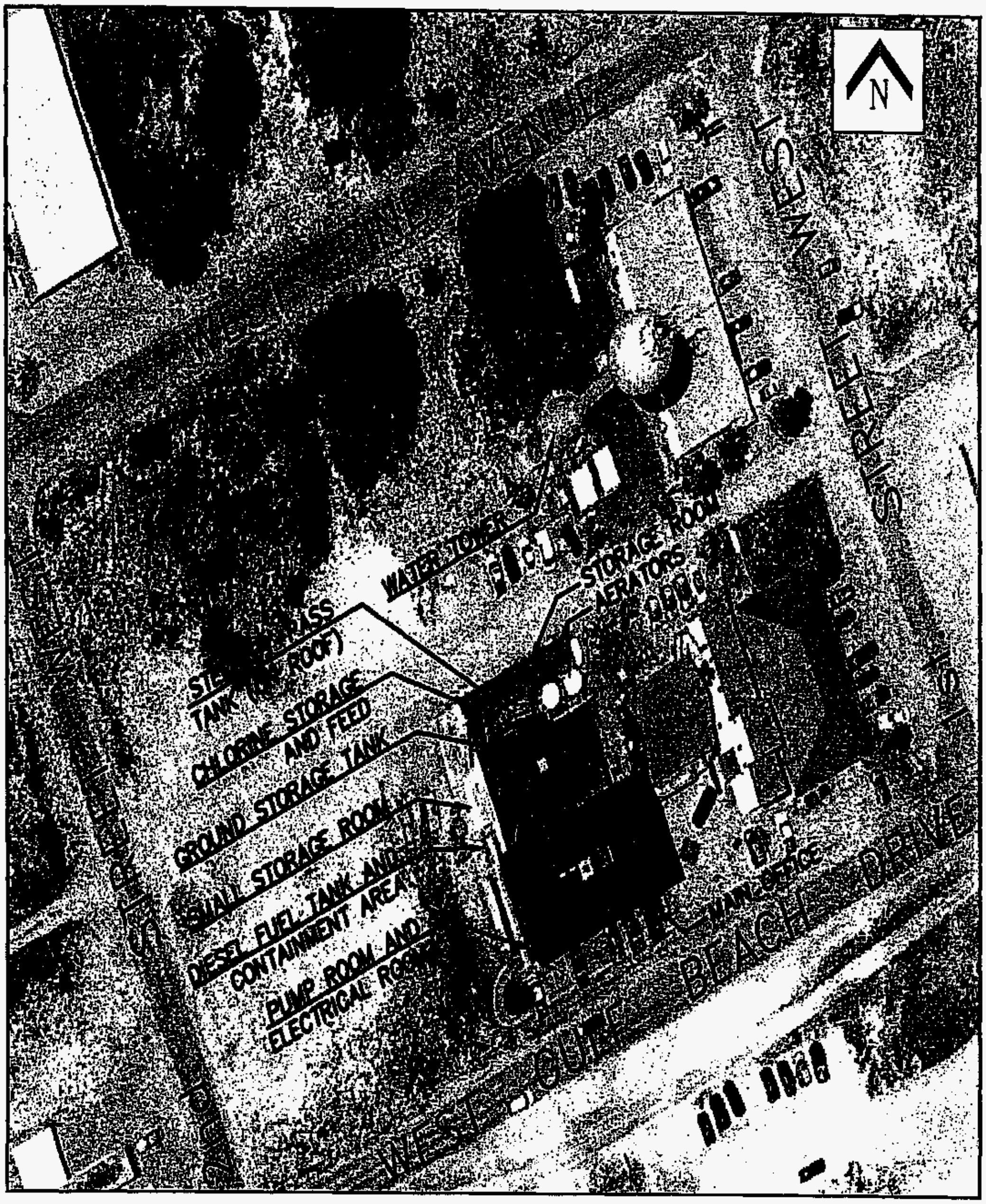


FIGURE 4.1 St. George Island Water Management Services Treatment Facility

5.0 Summary of Observations

Ground Storage Tank

- During the inspection of the exterior of the GST showed degradation of the 8-inch concrete hollow core roof panels around the access hatch on the roof and at the joints between panels (See **Figure 5.2**). When the roof hatch was opened, there were very strong odors of both hydrogen sulfide and chlorine. The hollow core panels' surface exposed to the inside of the water tank show signs of degradation (See **Figure 5.2**). Coarse aggregates in the concrete are exposed on face of these panels. It is not possible to determine the thickness of hollow core concrete or to confirm the condition of the tensioning strands without taking core samples of the roof and wall panels. The access ladder also showed signs of corrosion and chemical precipitation.

FIGURE 5.1 Joints between Hollow Core roof panels showing longitudinal cracking and exposed tensioning strands (Red arrows). Spalling is occurring at edge of concrete beam on surface bearing against roof panel (Blue arrow).

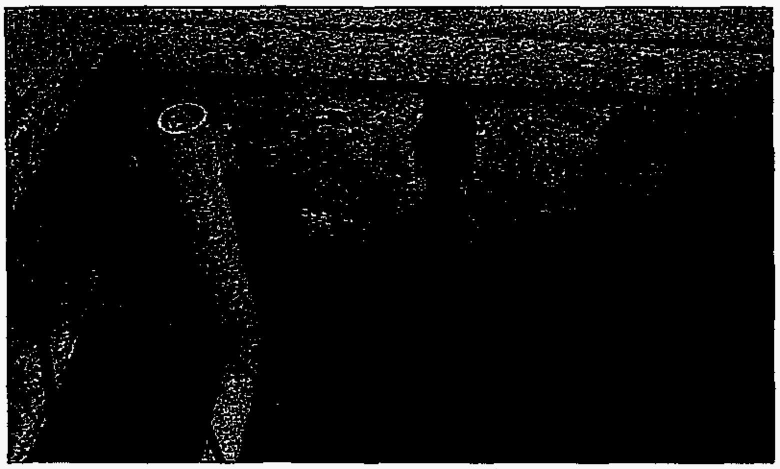


FIGURE 5.2 Degradation of concrete hollow core roof panels with rust and discoloration present on bottom (Red arrows). Chemical precipitation of metallic salts also present on ladder (Blue arrow).

- The joints of the GST on the main walls between tilt-up concrete wall panels shown signs of leaking and dark discoloration around many of the vertical joints. The cause of discoloration is unconfirmed, but is likely from mold and/or rust (See **Figure 5.3**).

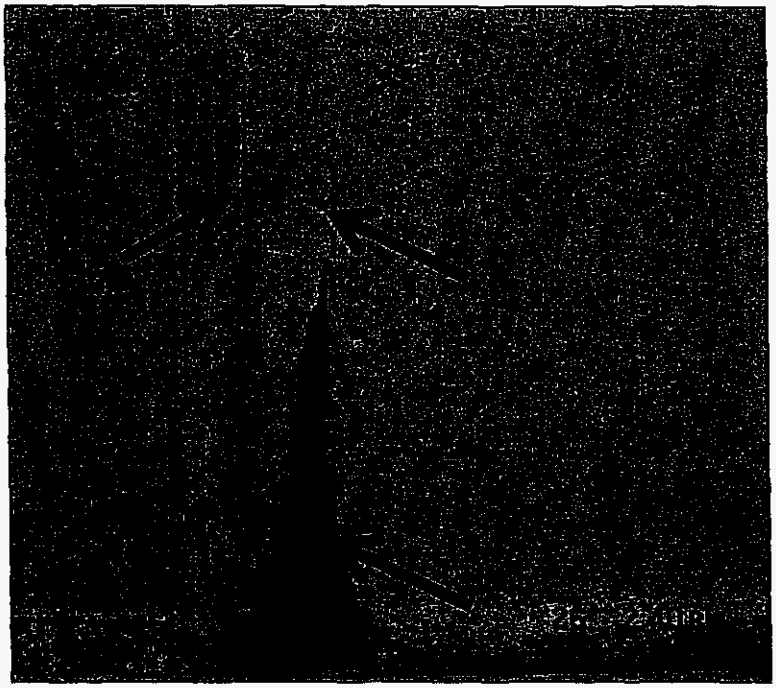


FIGURE 5.3 Cracks occurring at joint between tilt-up concrete wall panels (Red arrows). Probable cause of discoloration is from mold and/or rust (Blue arrow).

- The very strong smell of hydrogen sulfide and chlorine was also present inside the main office. This indicates a crack is present in the common wall above the maximum water surface allowing hazardous hydrogen sulfide and chlorine vapor to enter the work area.
- There are post-tensioning strands are exposed at the ends of the 8" hollow core roof panels and show indications of rust and corrosion of the strands (See **Figure 5.1**).

- The team observed random intermittent cracking and spalling along the edge of the exterior perimeter concrete beam running along the top of the ground storage tank (See **Figure 5.4**). The cause of the cracking and spalling is unknown, but may be evidence of a change in loading and bearing conditions or simple degradation of materials.

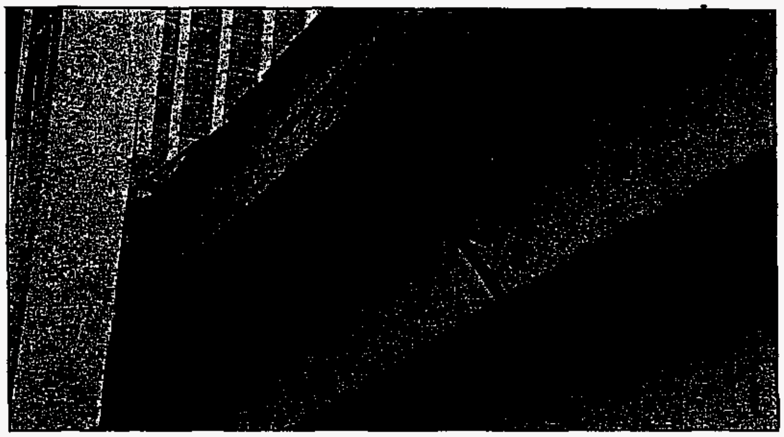


FIGURE 5.4 Spalling along the edge of the perimeter concrete beam where 8" hollow core roof panels bear down on the perimeter beams (Red arrow).

- Chemical precipitation of metallic salts are forming on the exterior of the concrete perimeter beams and wall panels inside the storage and maintenance room on the north side of the ground storage tank (See **Figure 5.5**).

FIGURE 5.5 View of bottom/end of hollow core roof panel for ground storage tank inside storage room on north side of ground storage tank. Edges along roof panel have spalling exposing tensioning strands and rust (Red arrows). There is a presence of chemical precipitation of metallic salts (Blue arrows).

- The recently painted exterior of the GST structure obscured the conditions and hindered the evaluation of the condition of concrete in many locations. The paint masked visually clues and issues such as possible rust, corrosion, or chemical precipitation that may have been present on the surface of the concrete.

Ground Storage Tank Roof

- The roof drainage does not appear adequate, as is a low slope roof with very little pitch. This allows for pooling and puddling that can compromise the tank via the hatch. The roof access system is lacking proper safety considerations such as handrails, and more importantly a lock to keep unauthorized and untrained people out of a confined space. The roof maintenance does not appear to be adequate and this may result in breeches in the tanks integrity via the roof. It is not known if the roof is watertight.

FIGURE 5.6 Panoramic view of the roof facing south. The roof is generally flat and puddling was observed.

High Service Pump Room

- The wall panels of the high service pump room appeared to be in fair condition. Spalling was observed on edge of concrete perimeter beam (See **Figure 5.7**).

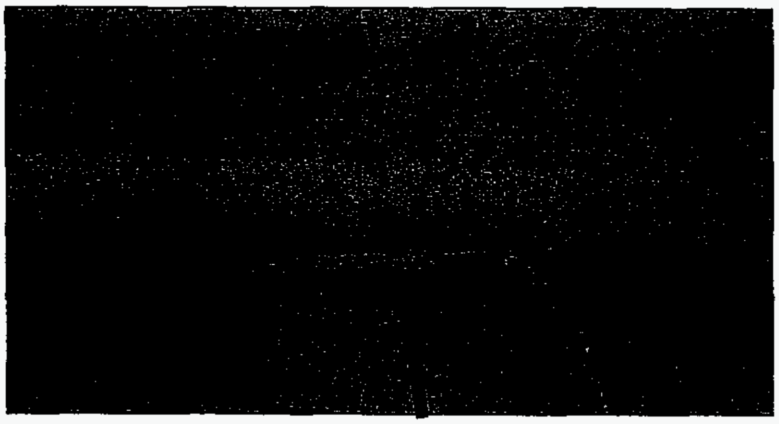


FIGURE 5.7 Spalling and cracking along the edge of the perimeter concrete beam and concrete wall where 8" hollow core roof panels bear down on the perimeter beams (Red Arrow).

Main Office

- The structural elements were not visible, as the office has faux wood paneling covering the common wall shared with the GST. A strong odor of hydrogen sulfide was present in the building. The odor was strong enough to be nauseating, indicating a level of at least 0.01 ppm. The odor level indicates a potential safety hazard for exposure to hydrogen sulfide. There was also evidence of water leaking into the office from the ground storage tank, which shares a wall with the office.

- Water leakage from tank indicates a serious reliability issue. Since the area was not fully visible during the inspection, the extent of the leak and cracking is unknown.

Diesel Fuel Tank and Containment Area

- The diesel tank shows signs of rust and the exterior protective paint is starting to peel and flake off. It is recommended that the tank be mechanically cleaned, the surface prepared to at least a SSPC-SP2 or SSPC-SP3 level and re-coated with protective paint.

- The containment wall for the diesel fuel tank did not have any steel reinforcing and does not provide adequate volume for the size of the diesel tank (see **Figure 5.8**).

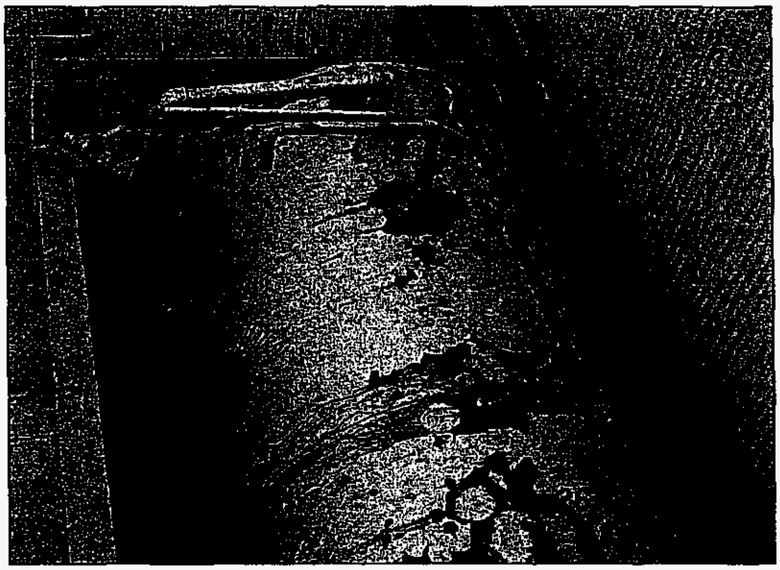


FIGURE 5.8 Diesel Fuel Tank and containment area

- During inspection of the fuel tank containment area, the containment wall fell due to a heavy rain from the previous night (see **Figure 5.9**). The new containment area must comply with local and state codes for fuel containment. Failure to comply with state codes risks potential site contamination in the event of a tank leak, resulting in an extremely costly clean up and possible post clean-up site-monitoring requirement per FDEP. It is also recommended that the new containment area be covered to prevent rainfall from entering containment structure.

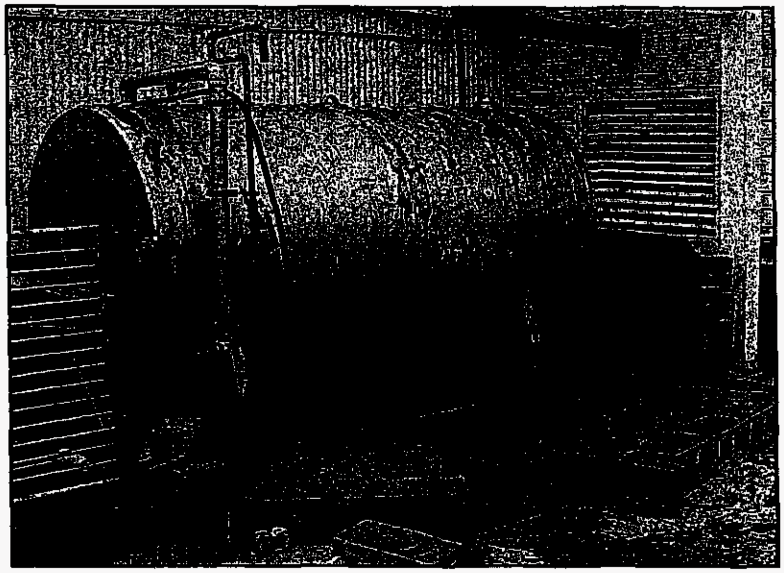


FIGURE 5.9 Diesel Fuel Tank and containment area after containment wall failure

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<u>Elevated Water Storage Tank</u>

- Based on the limited access to the lower level of the tank, visual inspection of the interior and minimal exterior inspection appears the elevated storage tank is in good condition. Minor rust is present on piping and valves, but no significant rust was observed on tank structure itself (See **Figure 5.10**).

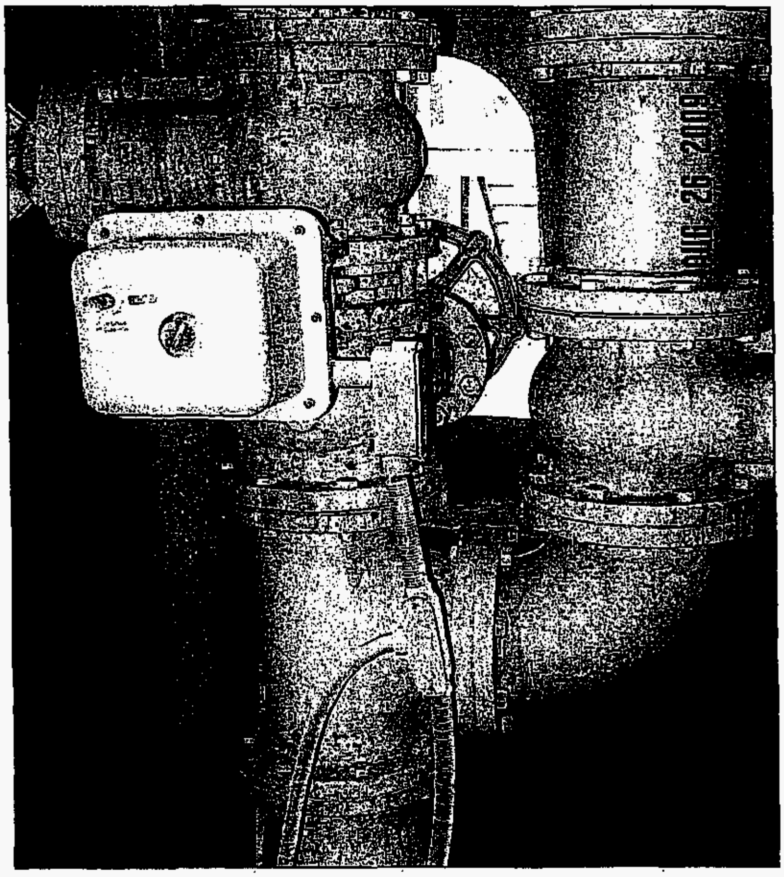


FIGURE 5.10 Minor rust on steel piping and tank structure observed.

6.0 Cost Estimation of Replacement /Rehabilitation Options

Estimates of project costs for each of the four replacement/rehabilitation options were developed to evaluate and compare each option. Option 1 assumes the elevated storage will be taken out of service. The construction of options 1 and 2 can be performed with the existing facility in operation. With the completion of construction, the existing ground storage tank can be taken out of service. **Table 3** provides estimates of probable construction

Table 3
Estimates of Probable Construction Costs*

St. George Island , Florida Estimates of probable construction costs for GST options					
DESCRIPTION		ALTERNATIVE 1: New GST on Lots behind WTP and refurb existing GST into a new workshop, abandon EST	ALTERNATIVE 2: New GST on Lots behind WTP, abandon existing GST and maintain EST	ALTERNATIVE 3: Construct new GST in current location, no new workshop and maintain EST	ALTERNATIVE 4: Remediation of existing GST, no new workshop and maintain EST
Demolition of existing GST (assume $10/sf)				$40,960.00	
Refurbish existing GST into a new workshop($40/sf)		$169,000.00			
Construction of new 500 KG GST with dual wetwell		$744,000.00			
Construction of new 325 KG GST with dual wetwell			$715,000.00		
Construction of new 325 KG GST with common wetwell				$270,000.00	
Remediation of existing GST (Crom Estimate April 17, '09)plus contingency)					$124,740.00
New aerators		$56,000.00	$28,000.00	$56,000.00	
New Vertical turbine high service pumps; roof mounted		$100,000.00	$100,000.00		
New chlorine room (Approx $30/sq ft)				$3,000.00	$3,000.00
Relocate generator and fuel storage facilities		$7,500.00	$7,500.00		
New containment structure for diesel fuel		$5,000.00	$5,000.00	$5,000.00	$5,000.00
Temporary Operations During Construction					
	temporary pumping ($25K/month)			$50,000.00	$25,000.00
	temporary chemical facility			$2,000.00	$2,000.00
	yard piping modifications(3% of gst cost)			$8,100.00	$3,742.20
	relocate aerators		$5,000.00	$5,000.00	$5,000.00
	new pumping chamber			$15,000.00	$15,000.00
	miscellaneous			$30,000.00	$30,000.00
Subtotal without property		$1,081,500.00	$860,500.00	$485,060.00	$213,482.20
Cost of four (4) lots with closing cost		$450,000.00	$450,000.00		
Mobilization/Demobilization (1%)		$10,815.00	$8,605.00	$4,850.60	$2,134.82
Site Work (2 %)		$21,630.00	$17,210.00	$9,701.20	$4,269.64
Contingency (20%)		$216,300.00	$172,100.00	$97,012.00	$42,696.44
Contractor's Bond and Insurance (2%)		$21,630.00	$17,210.00	$9,701.20	$4,269.64
Contractor's Overhead and Profit (10%)		$108,150.00	$86,050.00	$48,506.00	$21,348.22
Permitting (1%)		$10,815.00	$8,605.00	$4,850.60	$2,134.82
Engineering (10%)		$108,150.00	$86,050.00	$48,506.00	$21,348.22
Estimated Project Total		$2,028,990.00	$1,706,330.00	$708,187.60	$311,684.01

<u>* The cost estimates presented above where used to evaluate the options prior to the development of any priority CIP estimates. The values included in the table should not be confused with our final project cost estimate.</u>

Estimated construction duration for Option 1 and 2 is estimated to be 6-months. The construction of the new ground storage tank will take approximately 2 months. Long lead items such as new vertical turbine pumps and motors set the critical path time for completion of construction.

<u>Advantages of Options 1 and 2</u>

- Flexibility -Dual storage chambers allows cleaning or maintenance while maintaining operation
- Reliability- All pumping equipment above storm water elevation
- Eliminates elevated storage tank maintenance and operation-Option 1 only
- High quality, low maintenance option, with superb water tightness

Option 3 and 4 will require construction of a temporary piping, tankage and pumping systems to maintain operation of the water system. **Figure 6** illustrates the anticipated components of the temporary water treatment system.

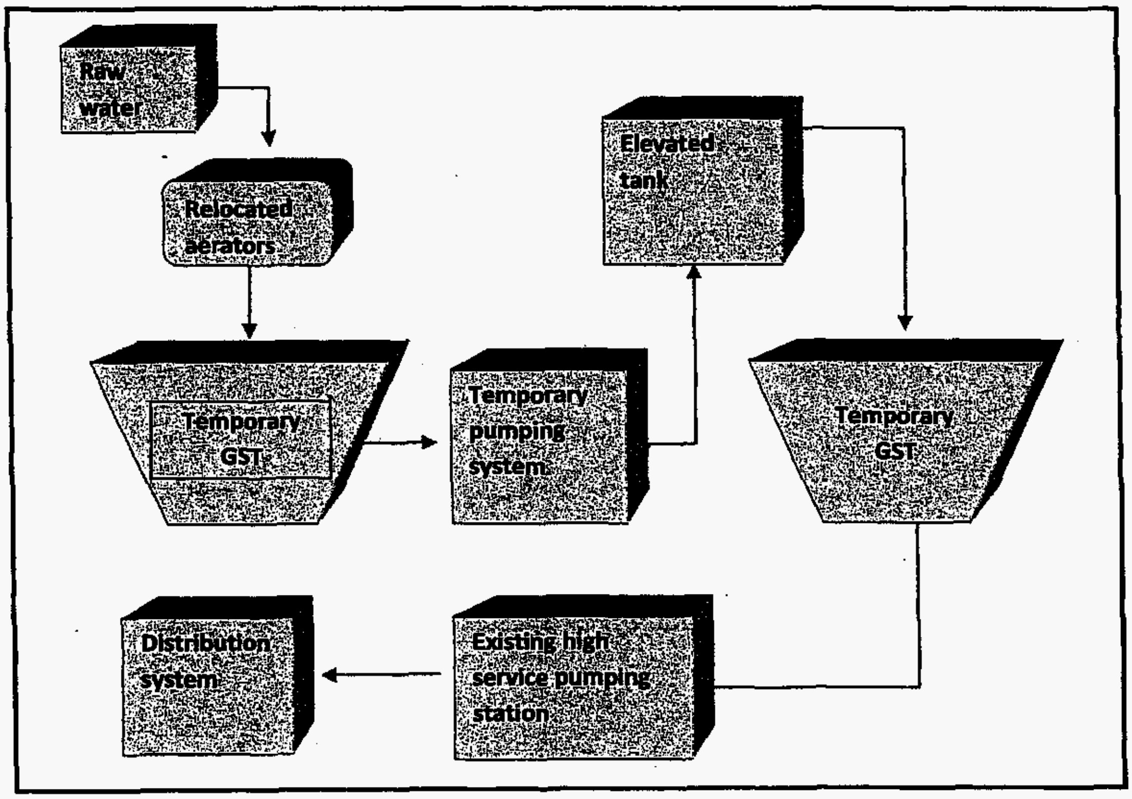


Figure 6
Temporary Water Treatment System

Estimated construction duration for Option 3 is estimated to be 2 months and 1 month for option 4. With either option, 3 or 4 the use of the existing high service pumps will be maintained. However, the pump station may flood as noted earlier. Construction of Option 3 provides the following advantages.

Advantages of Options 3

- Medium cost
- High quality, low maintenance option, with superb water tightness
- Reuse of existing property and high service pumping equipment
- Relative short construction period

Advantages of Options 4

- Lowest cost
- Reuse of existing property and high service pumping equipment
- Short construction period

7.0 Conclusions and Recommendations

Our inspection of the existing GST has confirmed the findings from the previous inspection conducted by Brice Nist, P.E. The Ground Storage Tank is in need of either remediation or replacement. The GST's tilt-up wall panel and hollow core roof system are not the typical or preferred method of construction for ground water storage tanks. Given the condition of the wall joints and roof panels, it appears the GST is approaching the end of its service life. It may be possible for the GST to be remediated, but this will require a means to bypass the GST during construction or remediation activities. The possibility and extent of remediation for the GST would be determined from the results from core samples of the wall and roof panels, as well as an extensive inspection inside the ground storage tank itself. We have developed a probable cost for refurbishment based on several significant assumptions.

Based on the inspection of the structures in the St. George Island Water Management Services water treatment facility complex, it is recommended that a new ground storage tank be constructed to replace the existing ground storage tank. A new GST will have a known level of quality and reliability since it will be constructed to current standards. The new GST will reduce the vulnerability to structural failure to a level that is consistent with current construction methods and should be designed for a service life of 50 years. The new tank will have proper life safety considerations such as controlled access, and egress methods and lifeline anchorages. The new construction will also remedy the issues of noxious gasses outside the tank (hydrogen sulfide & chlorine) as well as assure the sanitary condition inside the tank. A new ground storage tank will likely improve overall water quality, system efficiency, overall service life, and will allow the current GST to remain in service until the new ground storage tank is completed and placed into service. The new GST will serve as a location for high service pumps and electrical gear to insure continued and reliable operation during flooding events.

The GST might be able to be refurbished and provide some level of use with a different purpose. The re-tasked structure may be able to be used as storage or a workshop. The challenges of this, however, will be to establish some understanding of the current structures components and condition. With this information established, then the suitability in the new function can be determined. Several windows and doors will need to be added as well as ventilation, lighting and electrical systems. A more reliable solution would be to raze the building and put up a simple pre-engineered metal storage building that is site specific. These are relatively inexpensive and have a known level of serviceability and strength. **Figure 7** is a preliminary plan view of pre-stressed concrete storage tank with aerators and pumps located on the roof.

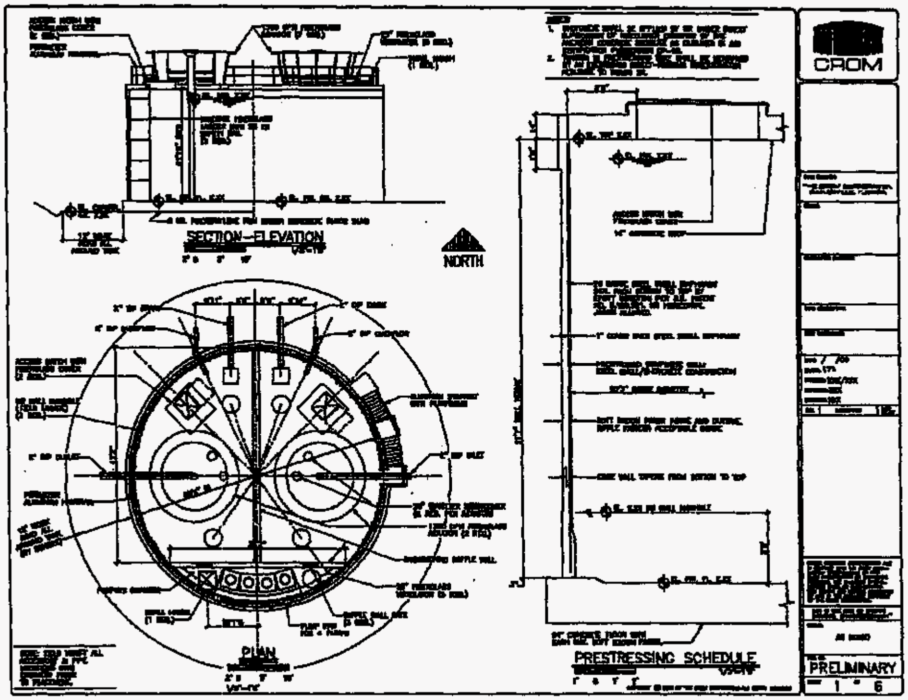


Figure 7

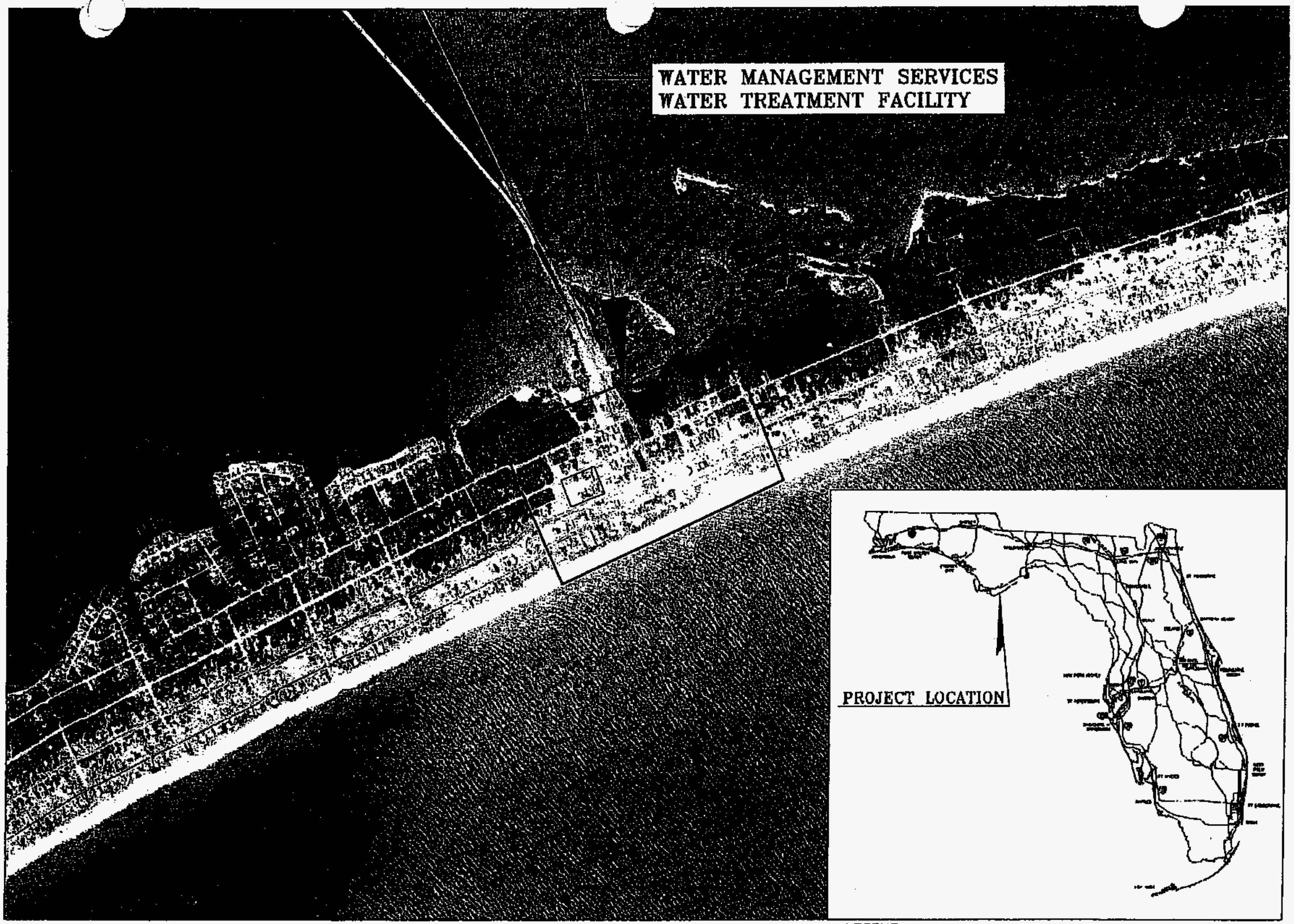
Plan View of New GST

The diesel fuel containment wall appears to be inadequate both in size and structural design. The current containment wall is simply a 3' tall CMU wall without any reinforcing. Based on the size of the diesel fuel tank on-site (approx 2,500 gallons), there did not appear to be adequate volume to contain all the diesel fuel contained in the tank. The containment wall will need to be re-built and designed to have enough volume for 125% the diesel fuel tank's capacity (excluding volume of empty tank), as well as withstand the force of being filled with diesel fuel (52 PSF).

END

APPENDIX A

Aerial and Vicinity Maps



LEGEND

ST. GEORGE ISLAND WATER TREATMENT FACILITY

N

0 400' 800' 1,600'

ST. GEORGE ISLAND

WATER MANAGEMENT SERVICES WATER TREATMENT FACILITY

PROJECT LOCATION MAP

ST. GEORGE ISLAND

WATER MANAGEMENT SERVICES WATER TREATMENT FACILITY
PROJECT LOCATION MAP

LEGEND

EXISTING WATER MANAGEMENT SERVICES WATER SYSTEM

N

0 20' 40' 80'

APPENDIX B

Photographs with commentary

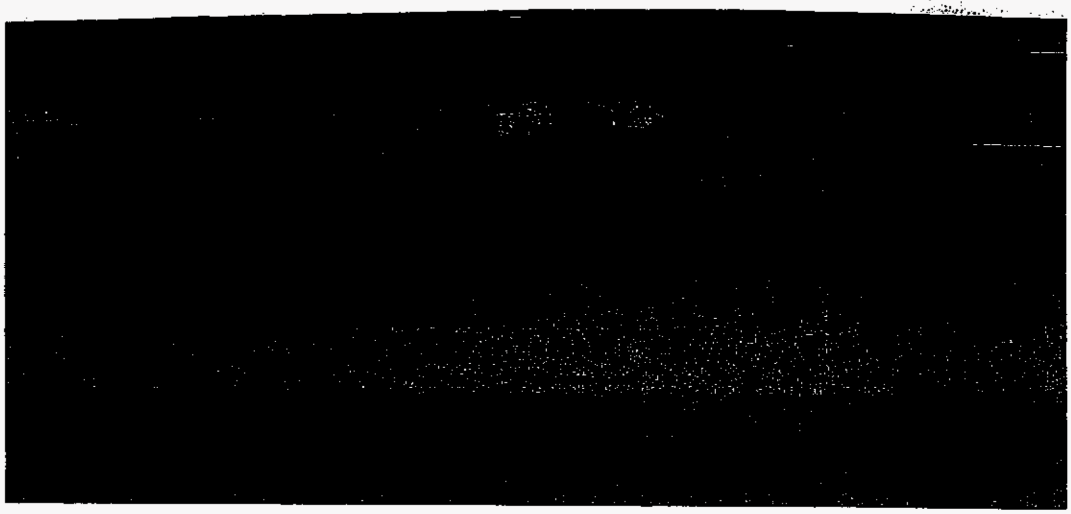


FIGURE B.1 Joint between Hollow Core roof panels showing longitudinal cracking and exposed tensioning strands (Red arrows). Spalling is occurring at edge of concrete beam on surface bearing against roof panel (Blue arrow).

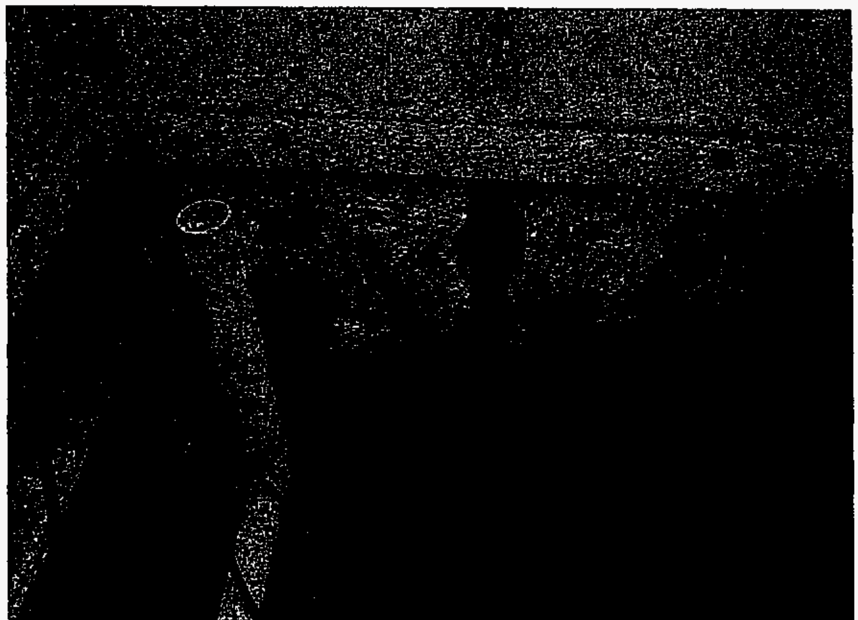


FIGURE B.2 Degradation of concrete hollow core roof panels with rust and discoloration present on bottom (Red arrows). Chemical precipitation of metallic salts also present on ladder (Blue arrow).

FIGURE B.3 View of bottom of hollow core roof panel inside ground storage tank. Rust and discoloration present on concrete (Red arrows).

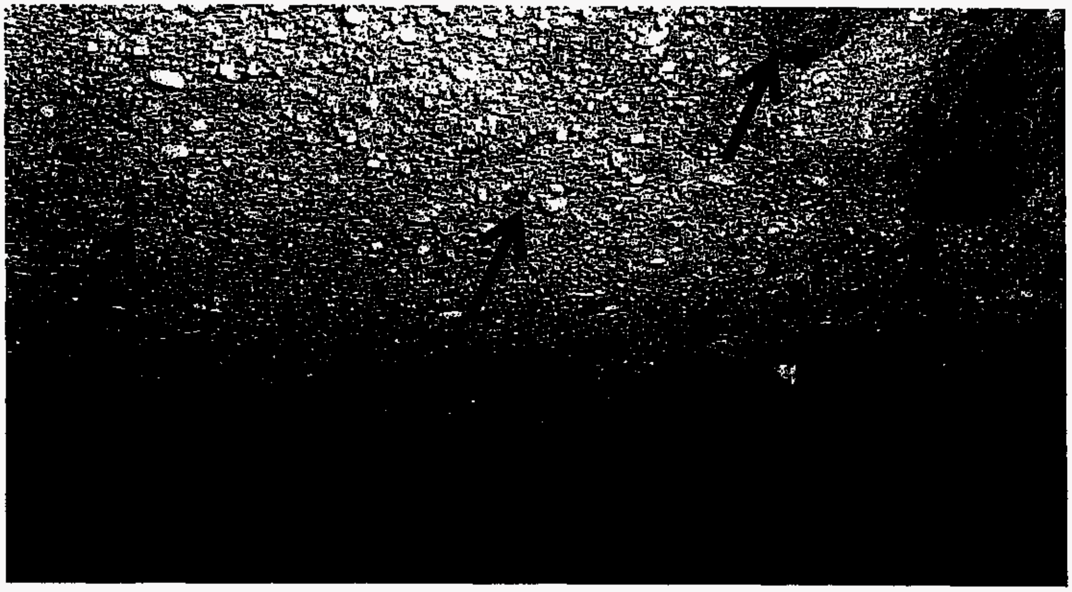


FIGURE B.4 View of bottom of hollow core roof panel inside ground storage tank. Rust and discoloration present on concrete (Red arrows). Surface of concrete panel has become very coarse and exposed coarse aggregate from concrete mix (Blue arrows).

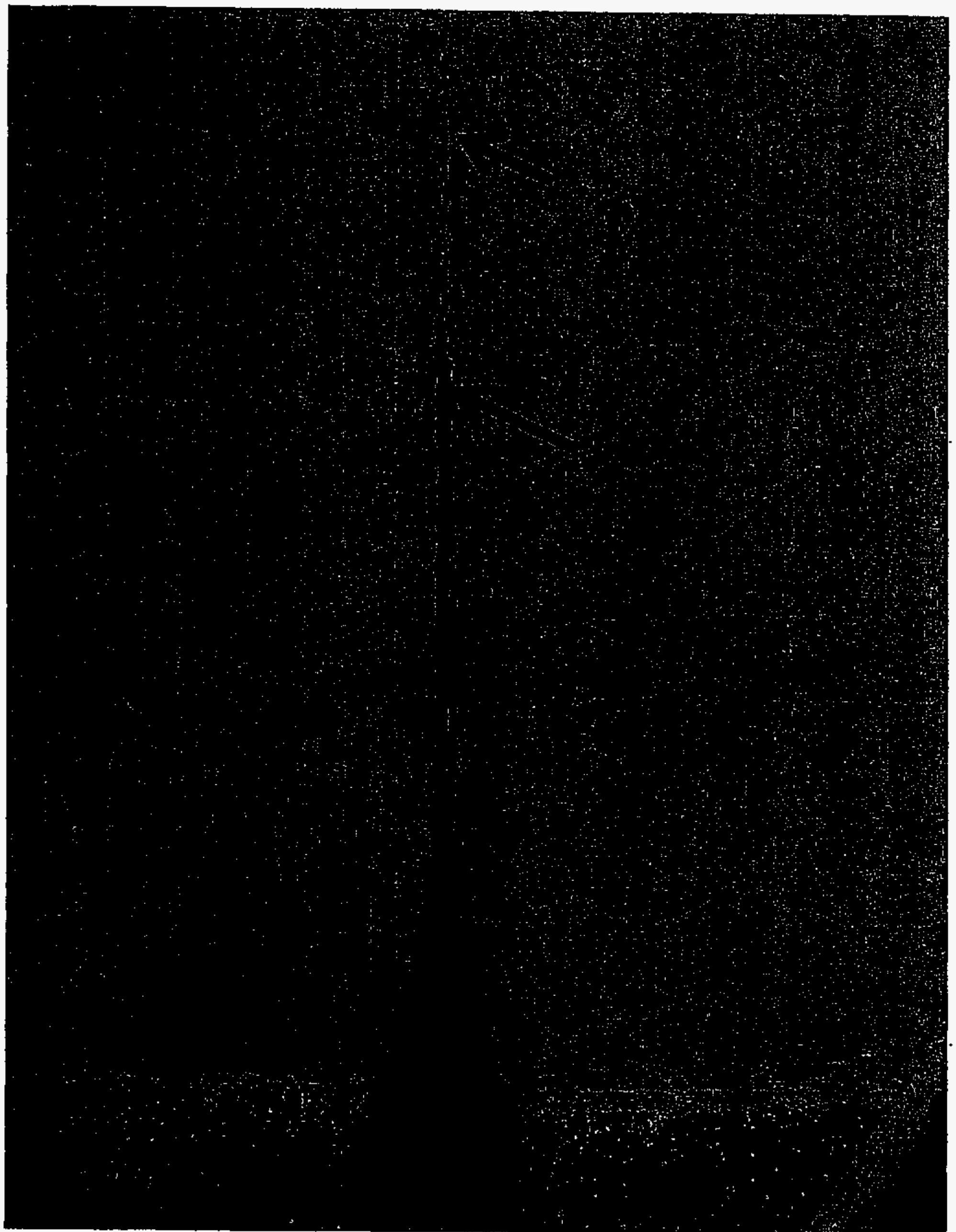


FIGURE B.5 Cracks occurring at joint between tilt-up concrete wall panels (Red arrow). Probable cause of discoloration is from mold and/or rust (Blue arrow).

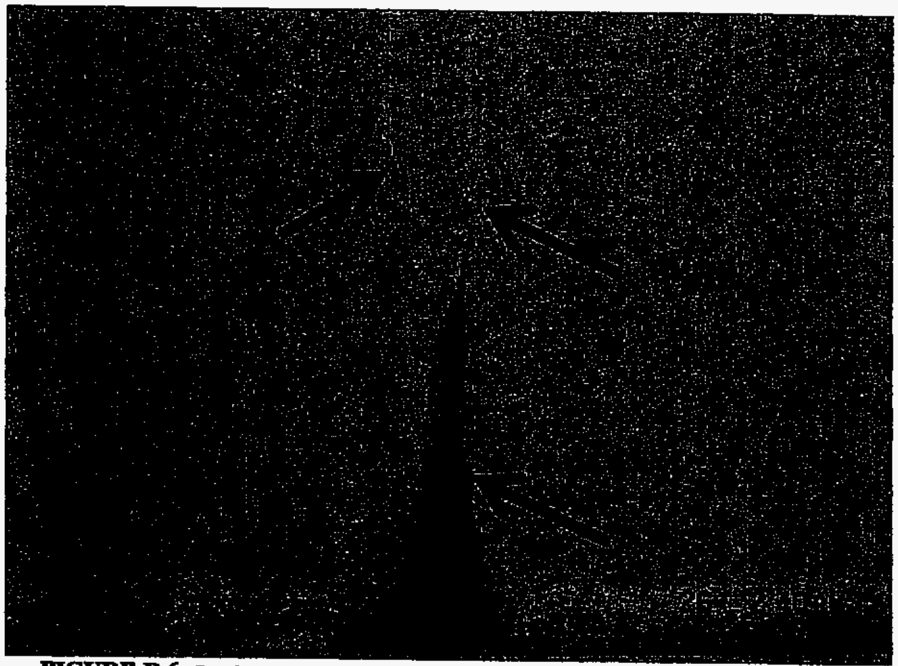


FIGURE B.6 Cracks occurring at joint between tilt-up concrete wall panels (Red arrows). Probable cause of discoloration is from mold and/or rust (Blue arrow).

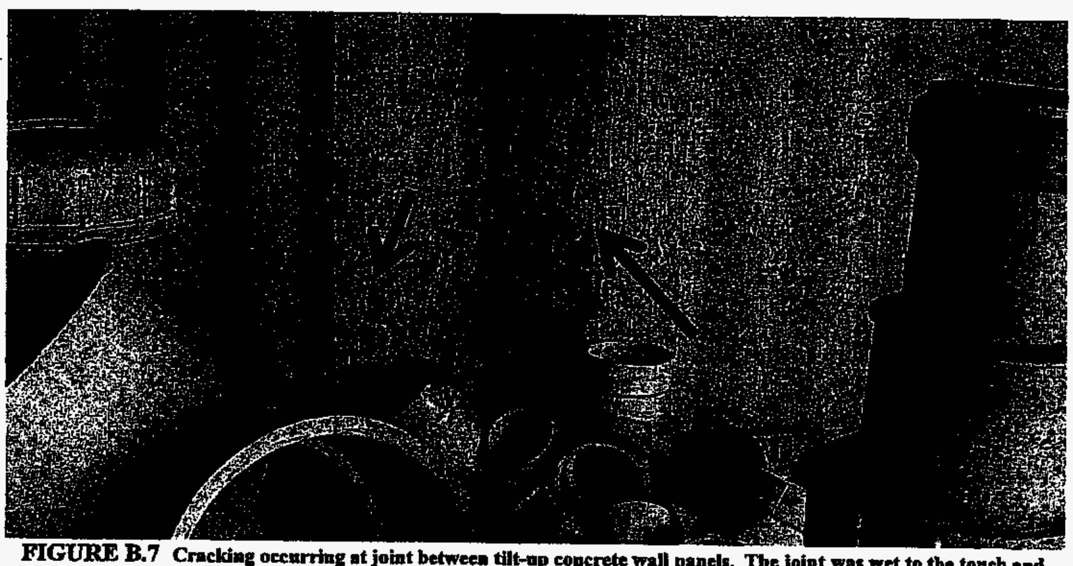


FIGURE B.7 Cracking occurring at joint between tilt-up concrete wall panels. The joint was wet to the touch and showed signs of having been grouted or patched in the past (Red arrow). A diagonal crack in the concrete panel to the left of the joint was also visible (Blue arrow).

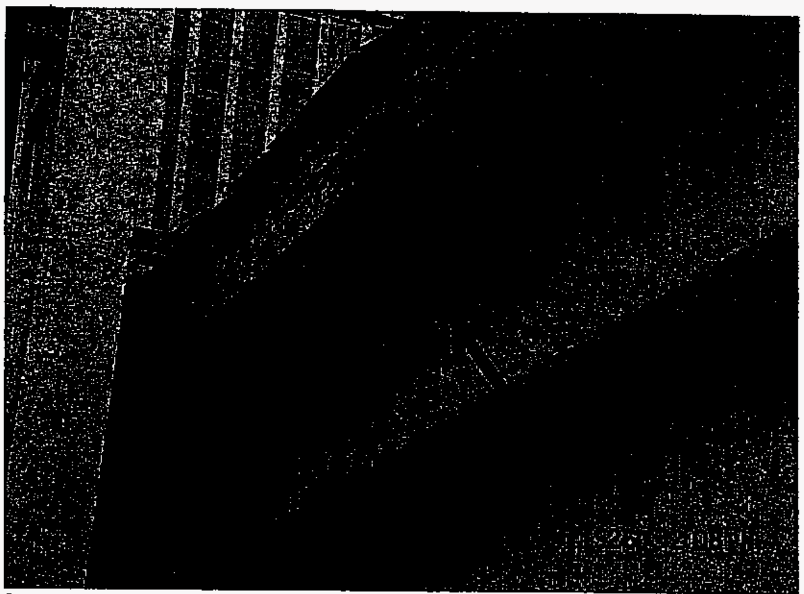


FIGURE B.8 Spalling along the edge of the perimeter concrete beam where 8" hollow core roof panels bear down on the perimeter beams (Red arrow).

FIGURE B.9 View of bottom/end of hollow core roof panel for ground storage tank inside storage room on north side of ground storage tank. Edges along roof panel have spalling exposing tensioning strands and rust (Red arrow). There is a presence of chemical precipitation of metallic salts (Blue arrow).

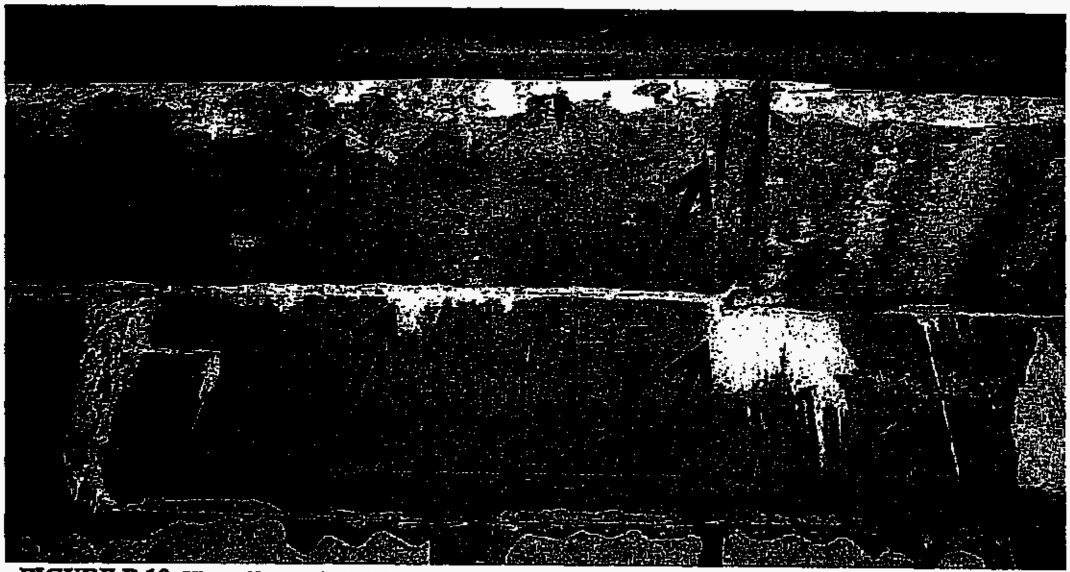


FIGURE B.10 View of bottom/end of hollow core roof panel for ground storage tank inside storage room on north side of ground storage tank. Edges along roof panel have spalling exposing tensioning strands and rust (Red arrows). There is a presence of chemical precipitation of metallic salts (Red arrows).

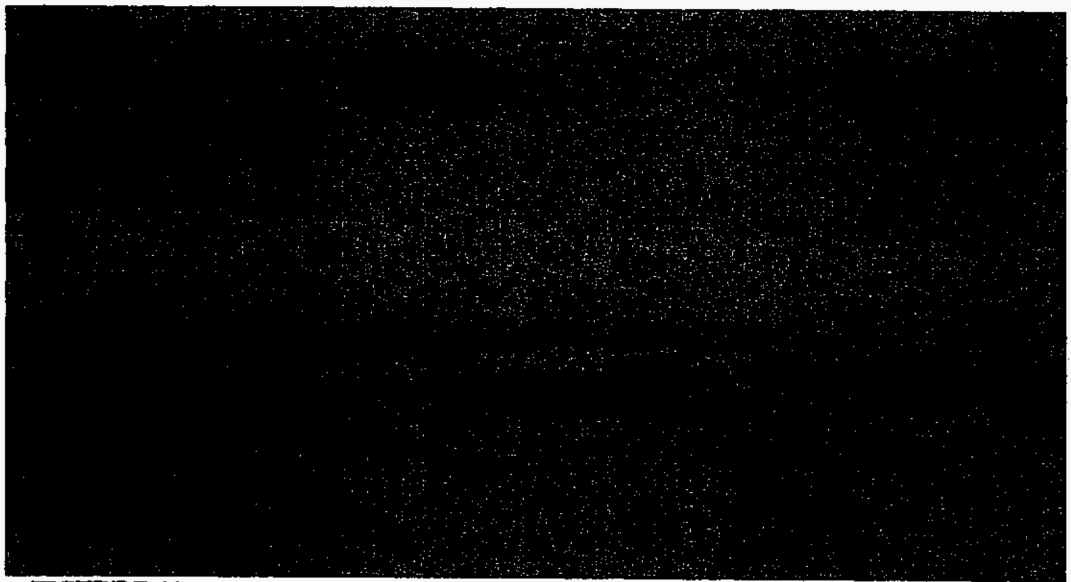


FIGURE B.11 Spalling and cracking along the edge of the perimeter concrete beam and concrete wall where 8" hollow core roof panels bear down on the perimeter beams (Red arrows).

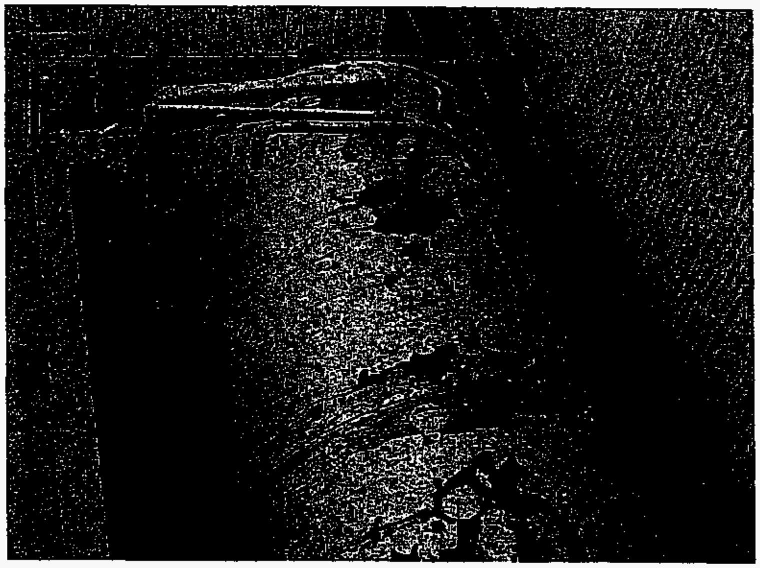


FIGURE B.12 Diesel Fuel Tank and containment area

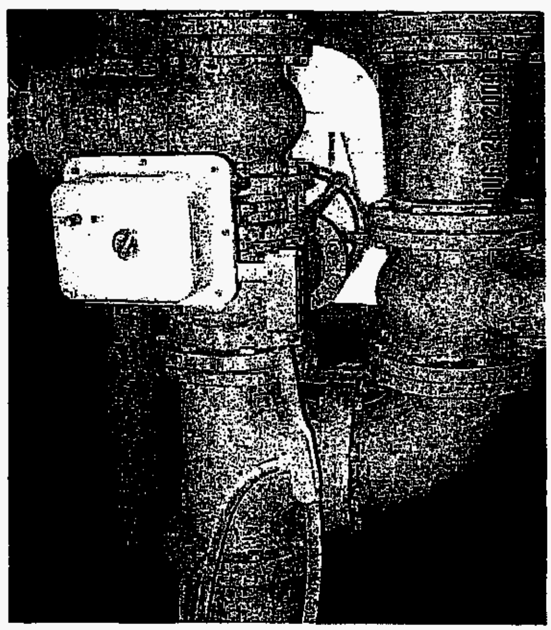
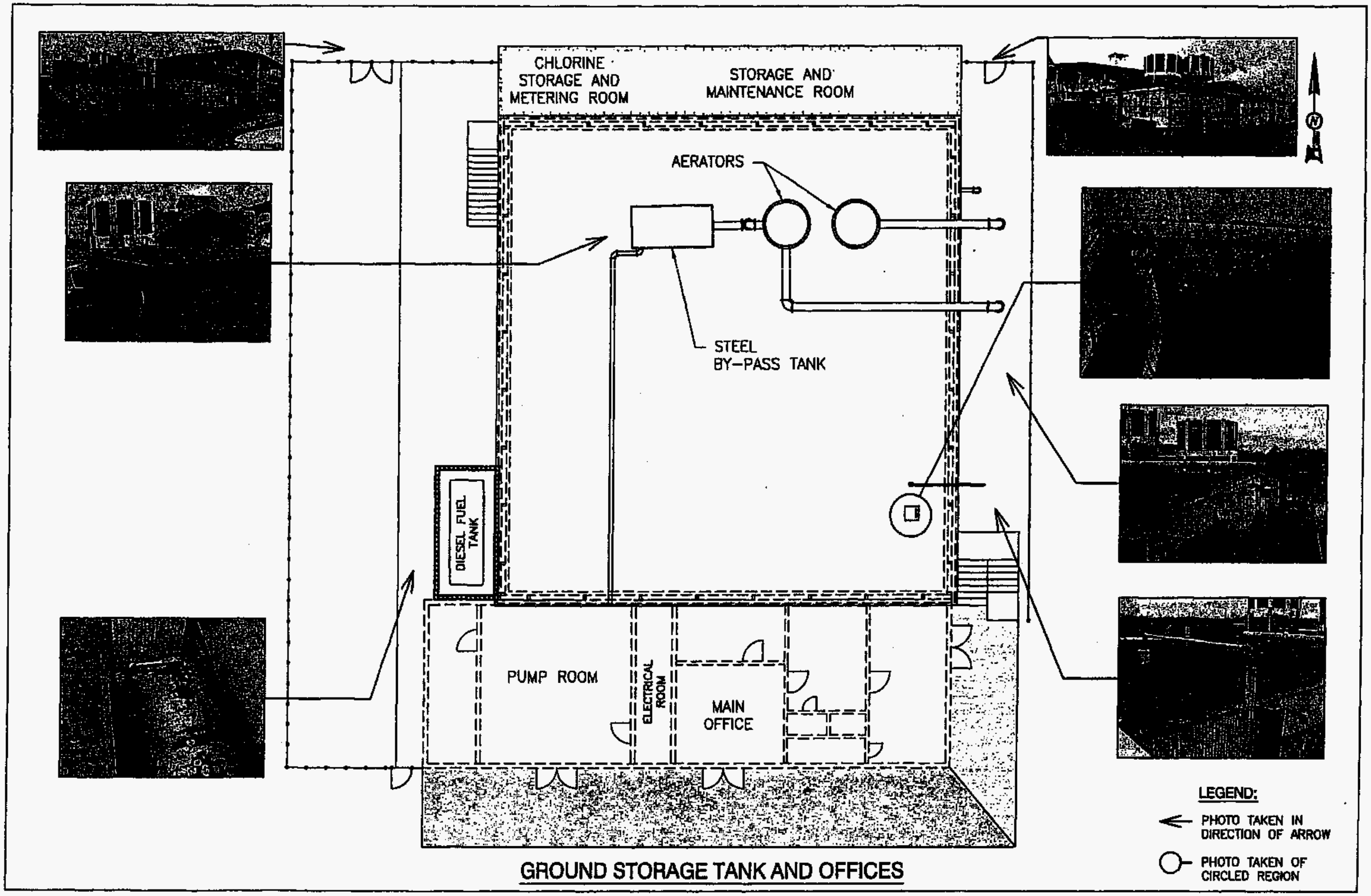


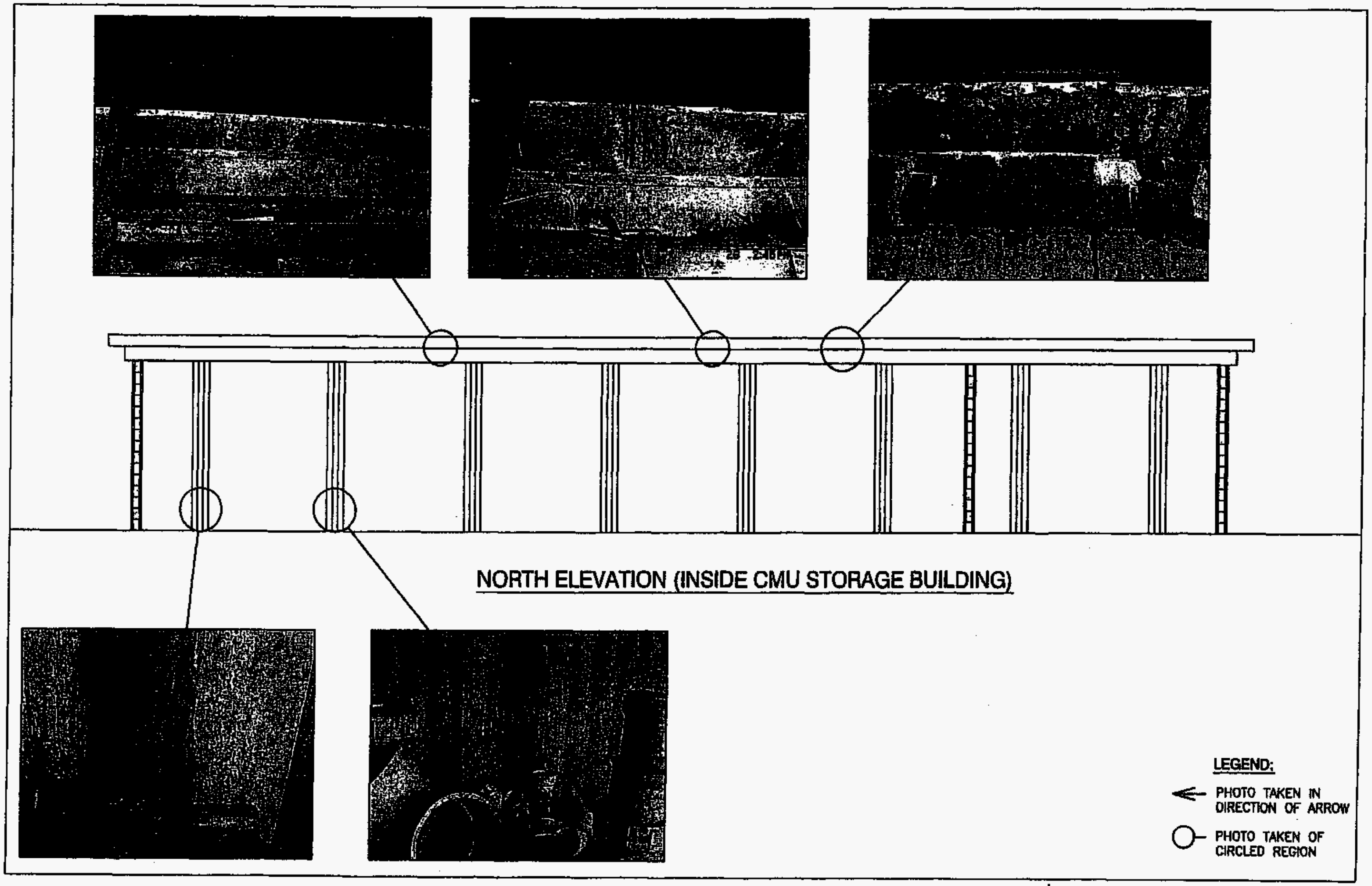
FIGURE B.13 Minor rust on steel piping and tank structure were observed.

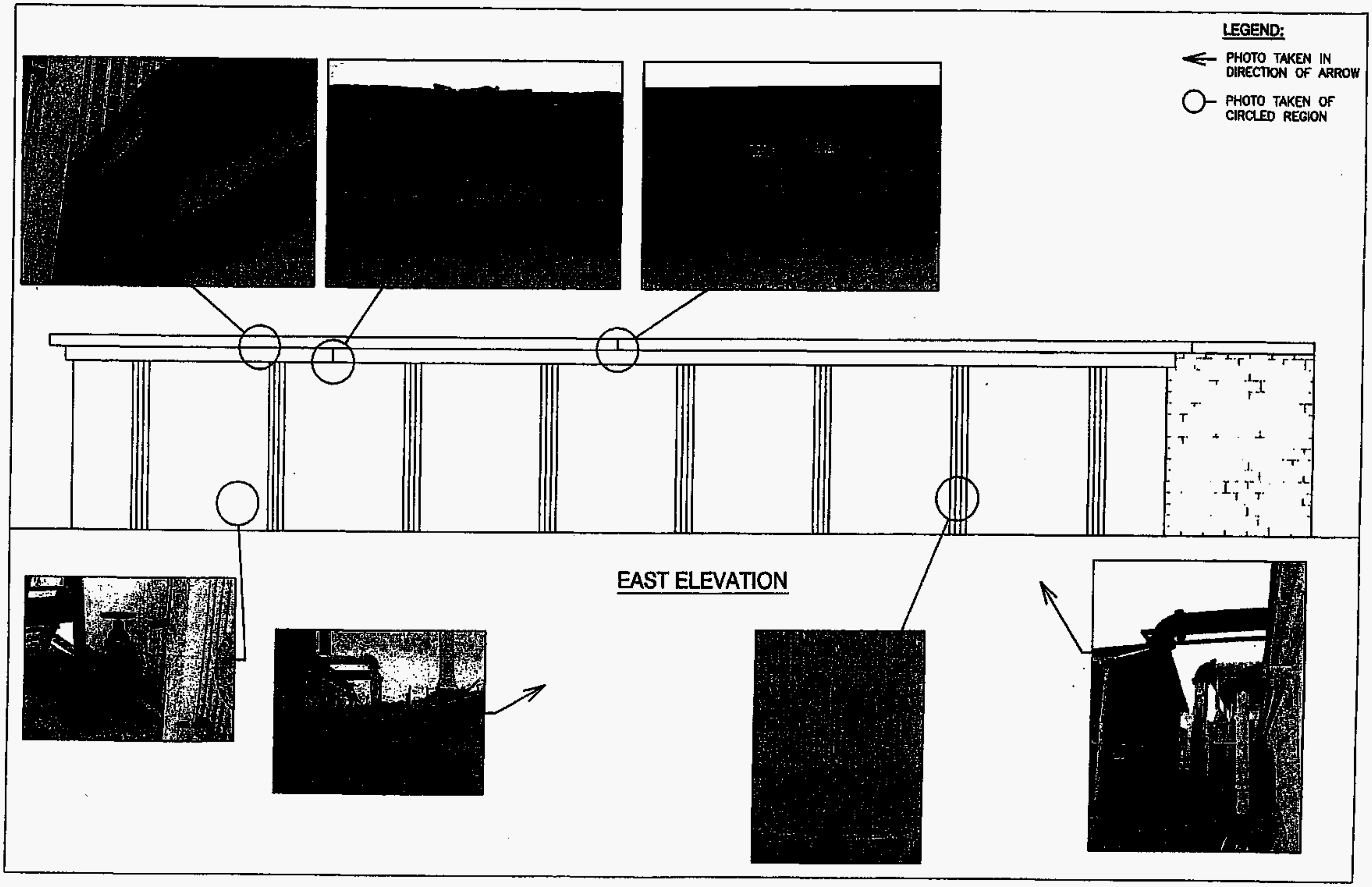
APPENDIX C

Keyplan of Ground Storage Tank and Office and Elevations with pictures



GROUND STORAGE TANK AND OFFICES


NORTH ELEVATION (INSIDE CMU STORAGE BUILDING)
LEGEND:
PHOTO TAKEN IN DIRECTION OF ARROW
PHOTO TAKEN OF CIRCLED REGION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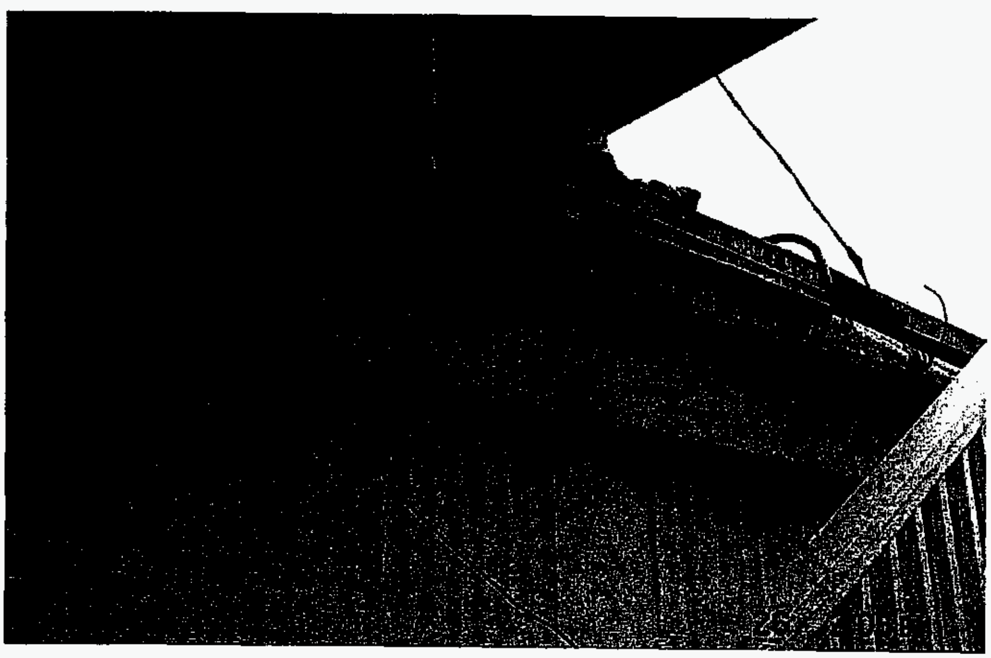
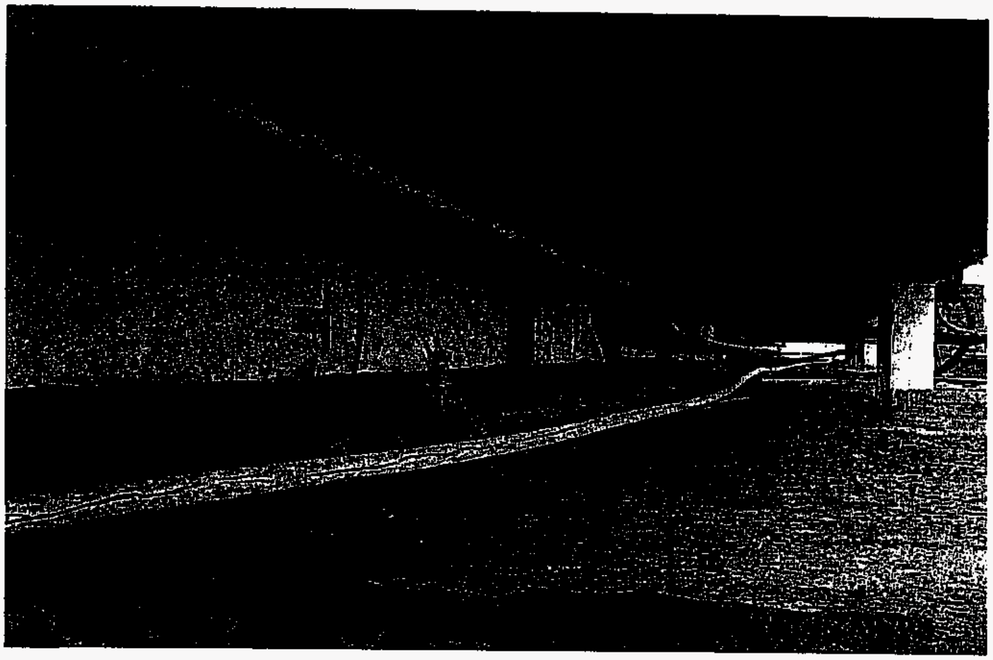
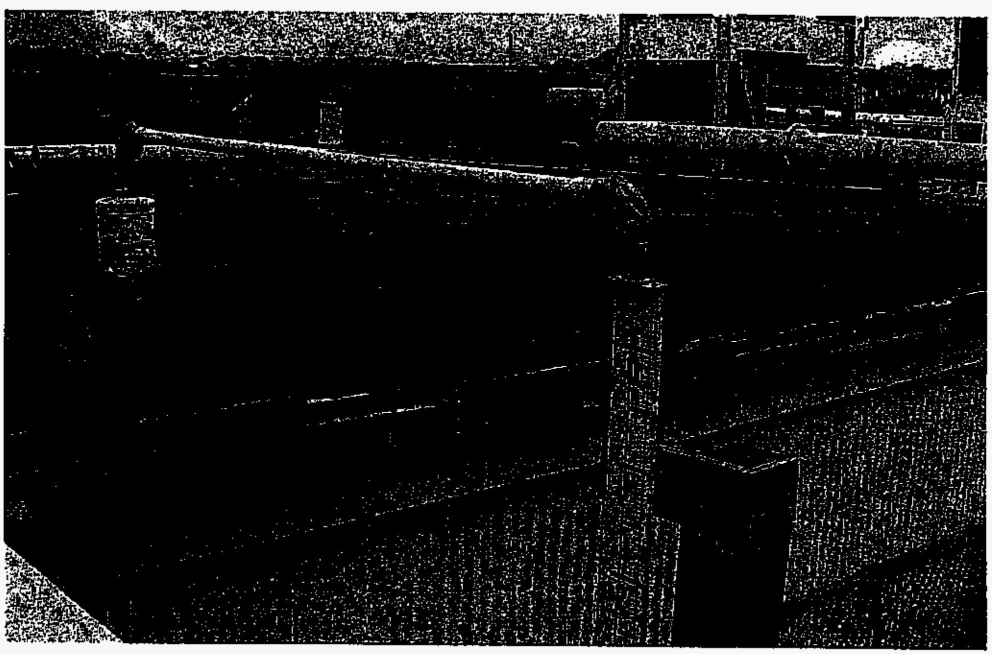
EAST ELEVATION

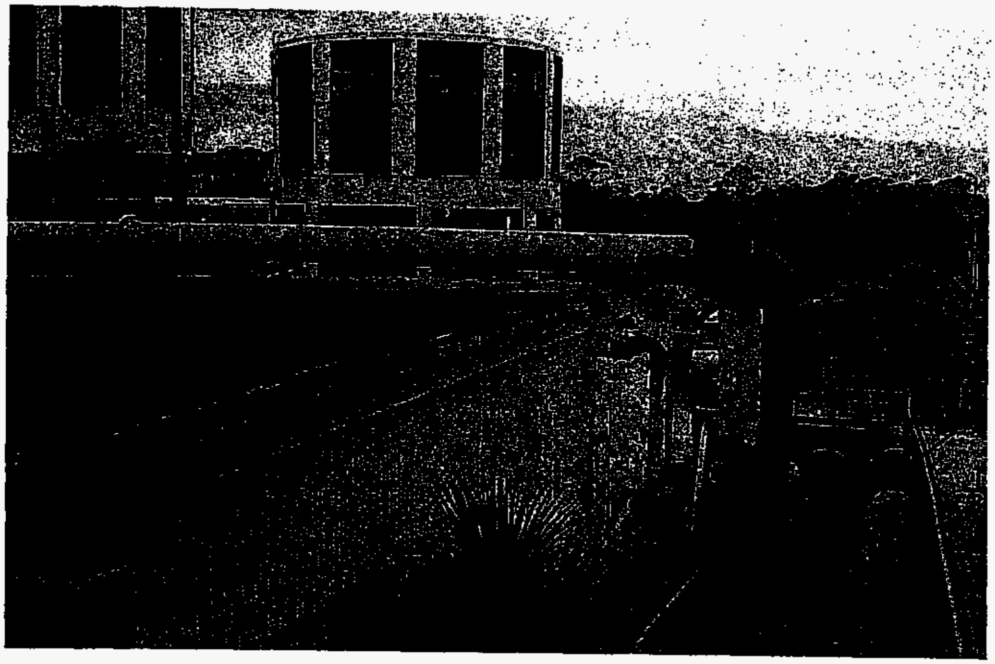
APPENDIX D

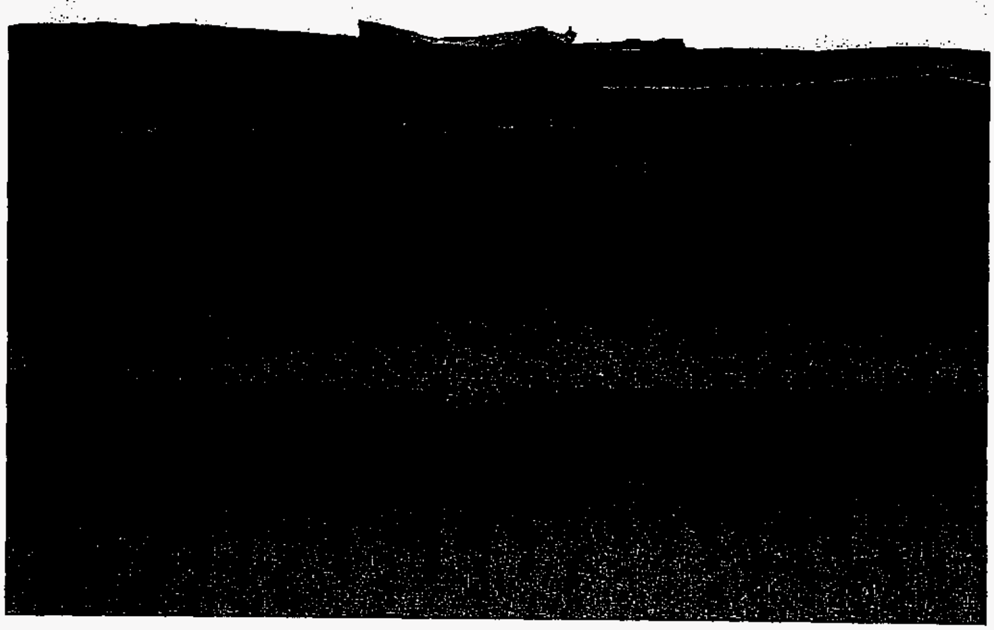
All photographs taken during site inspection

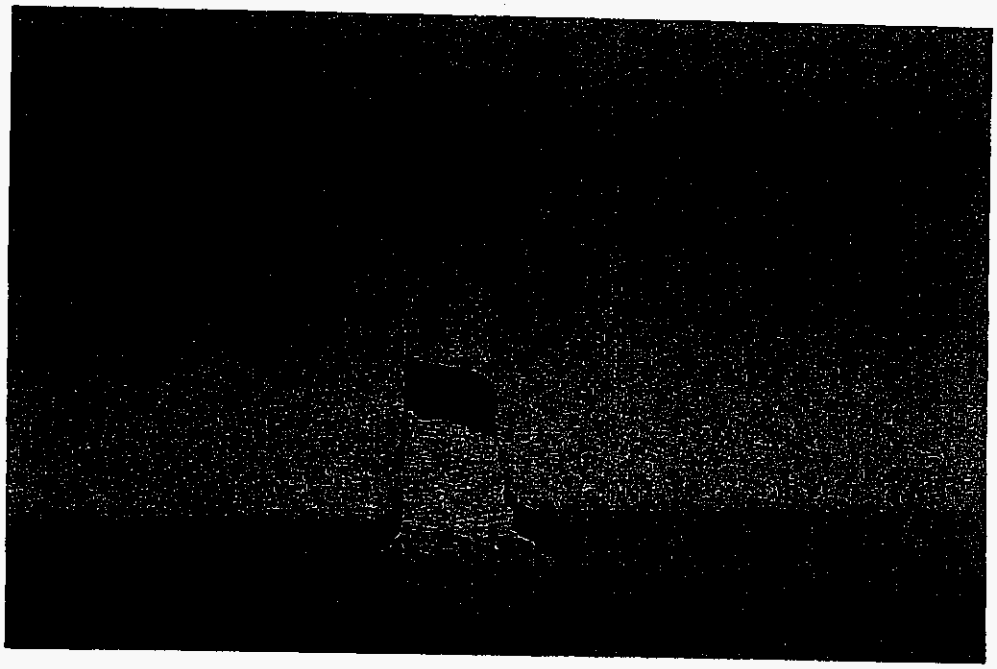


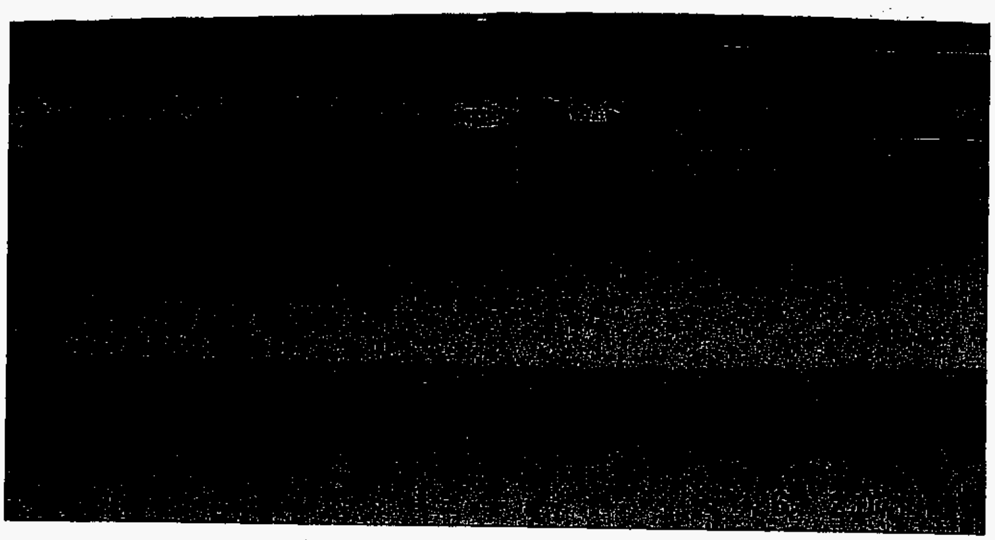


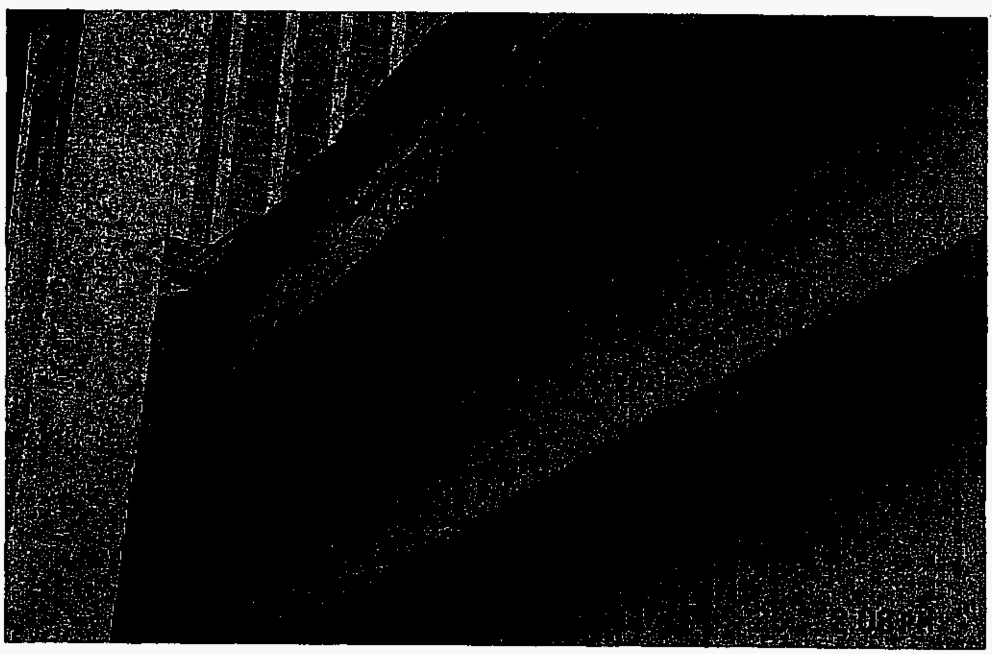


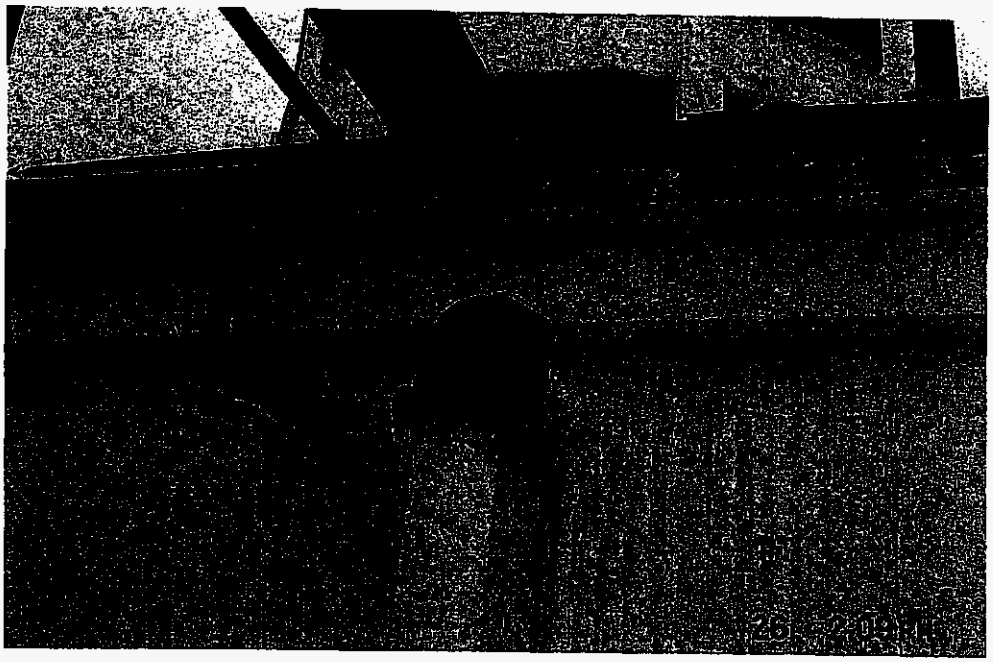


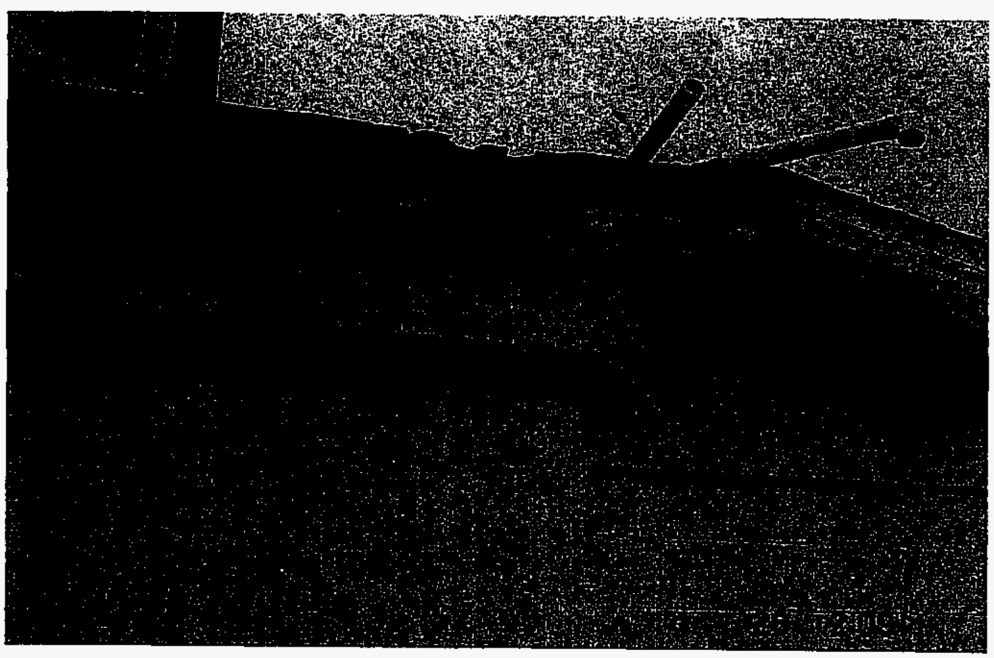


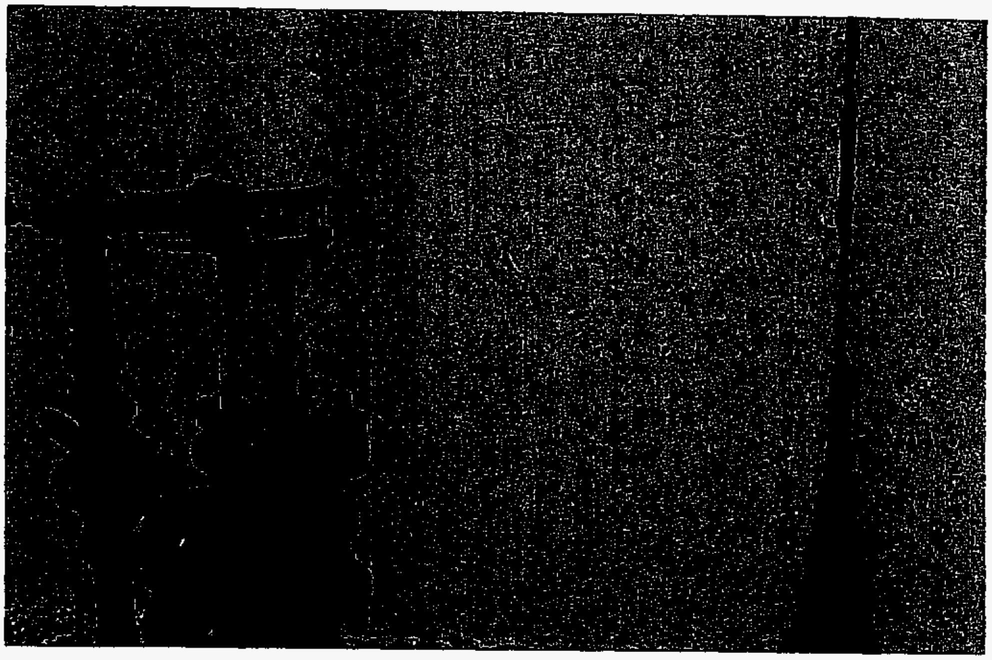


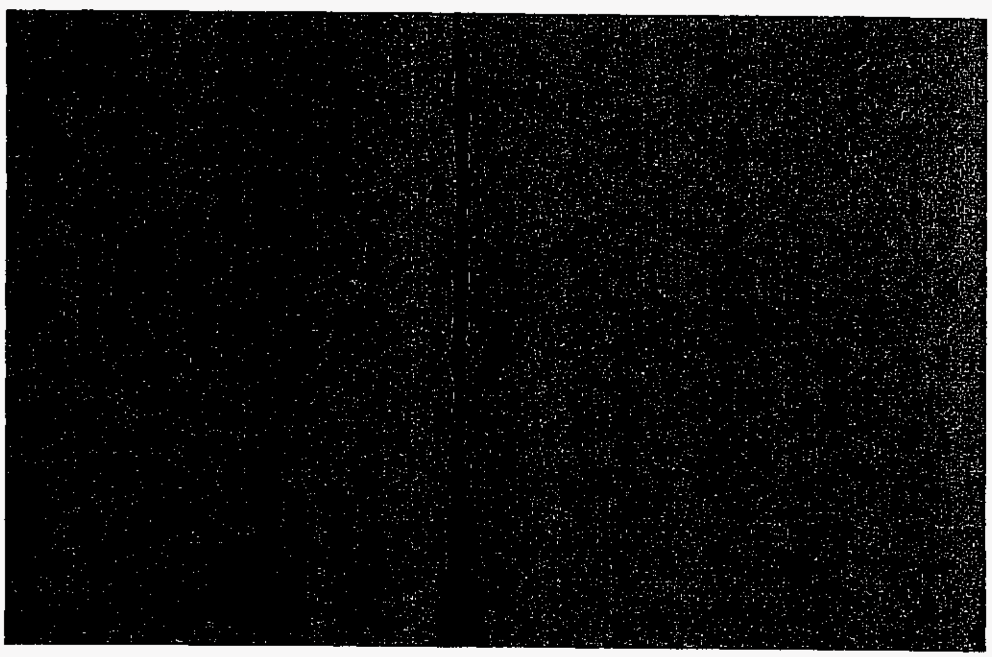


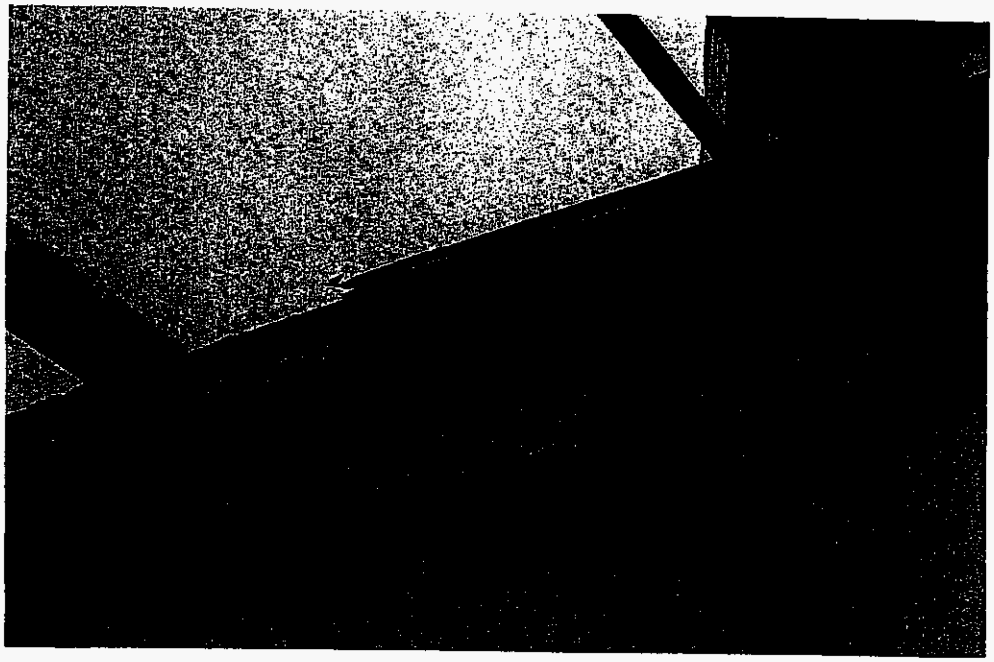


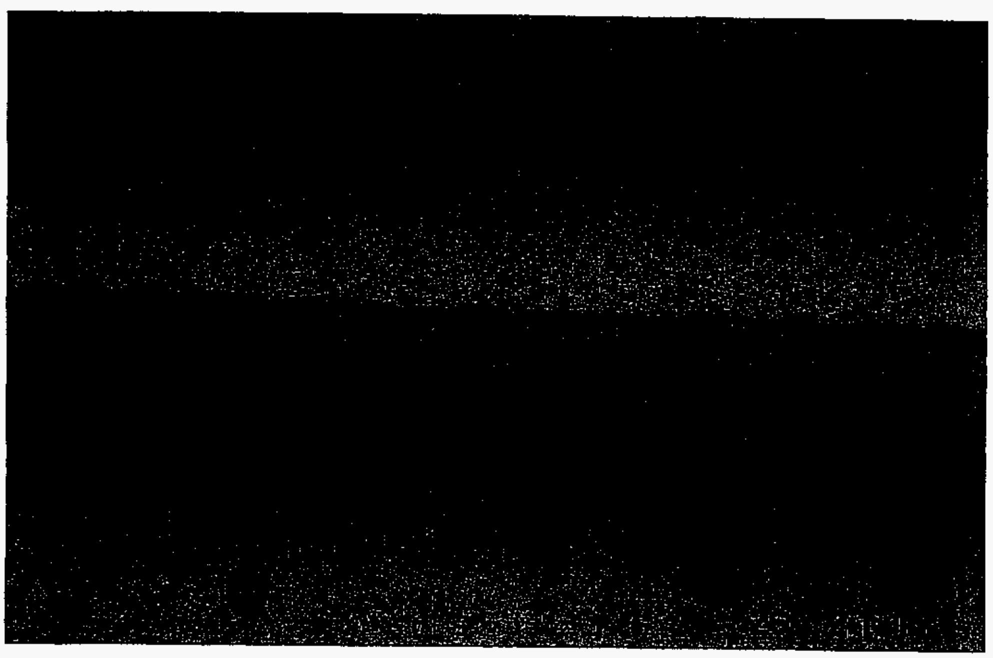


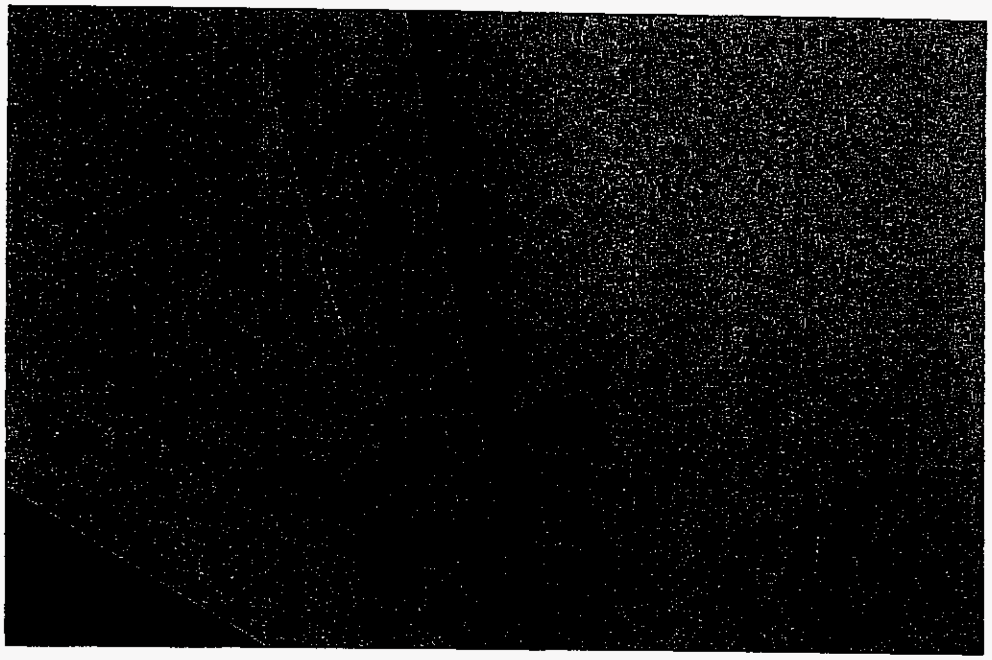


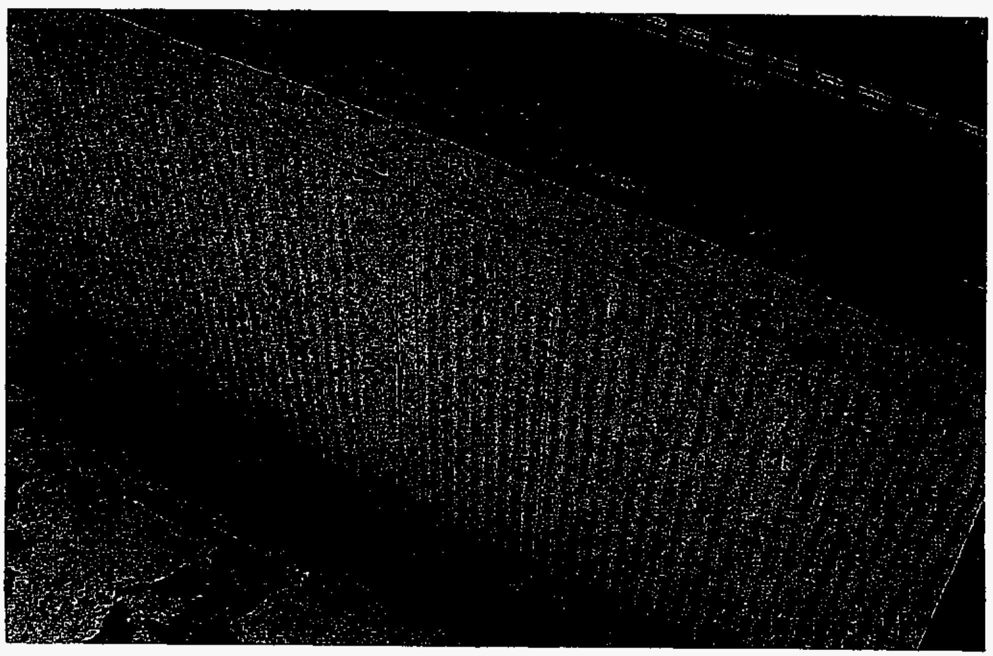


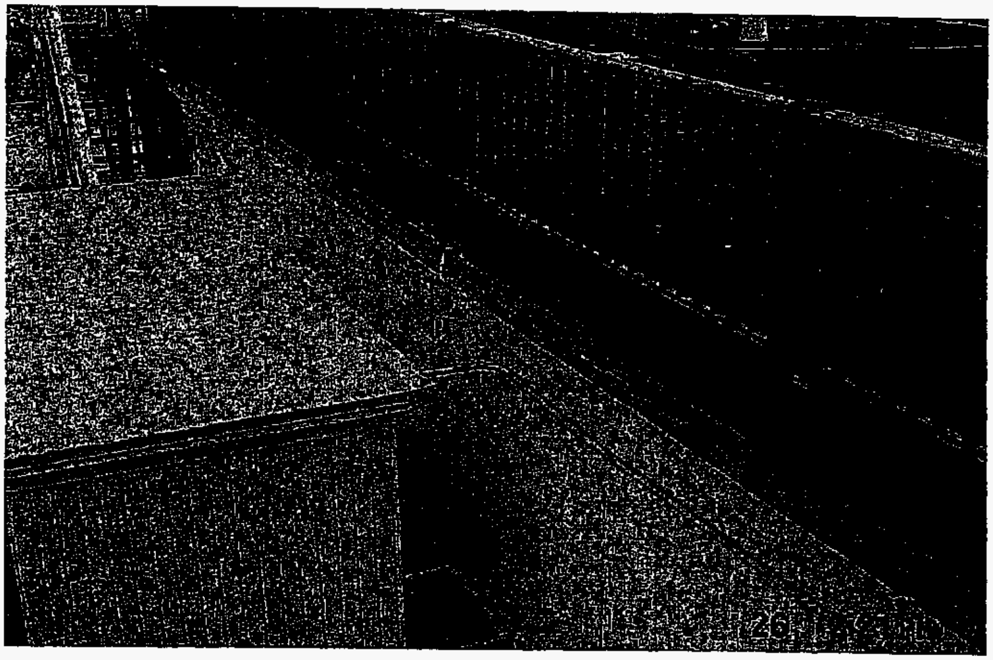


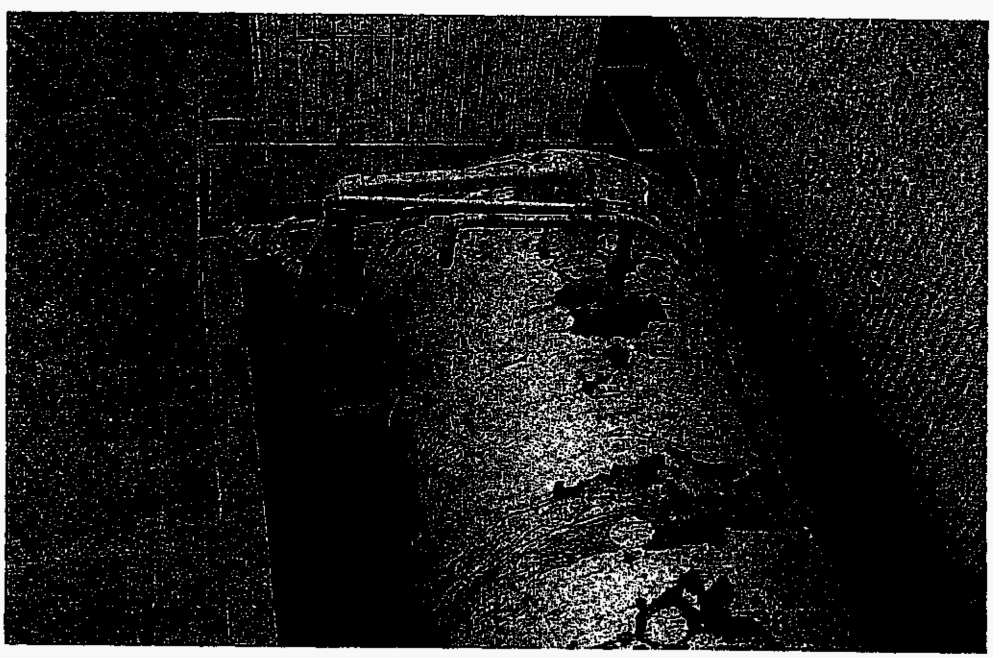


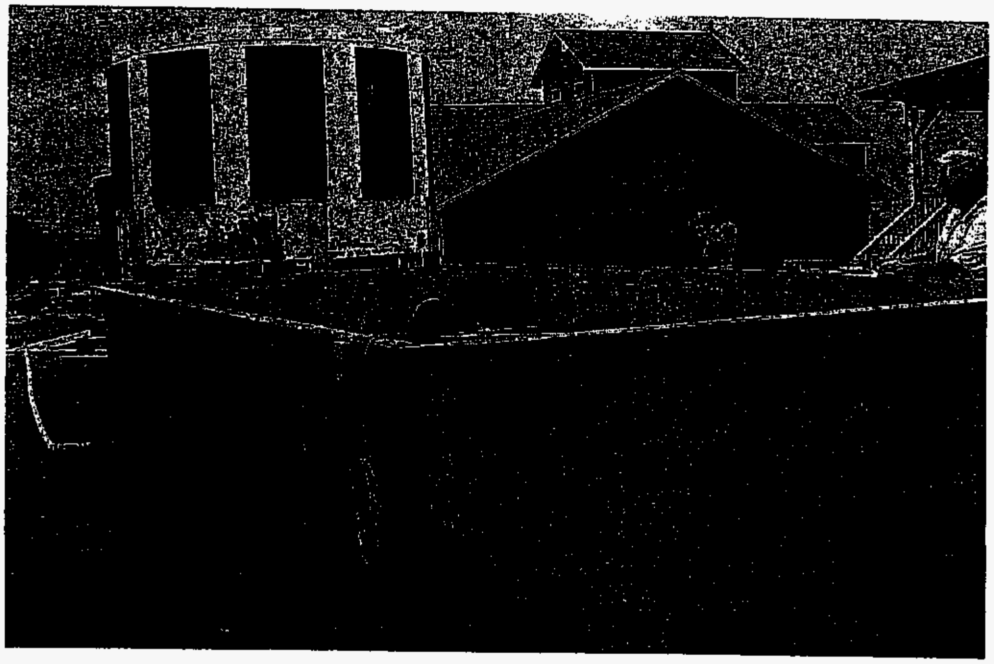


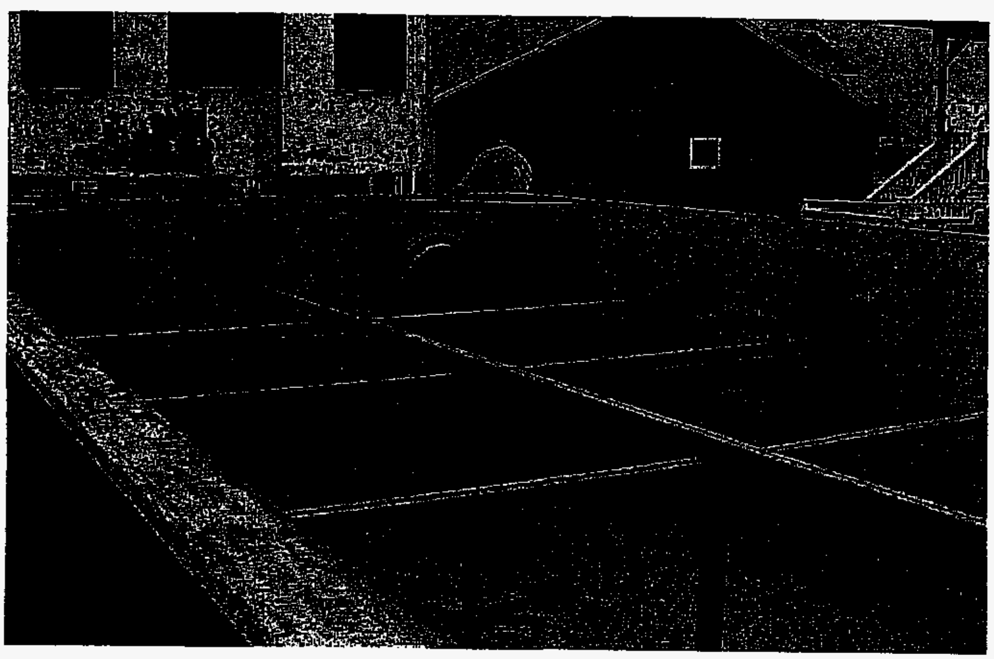


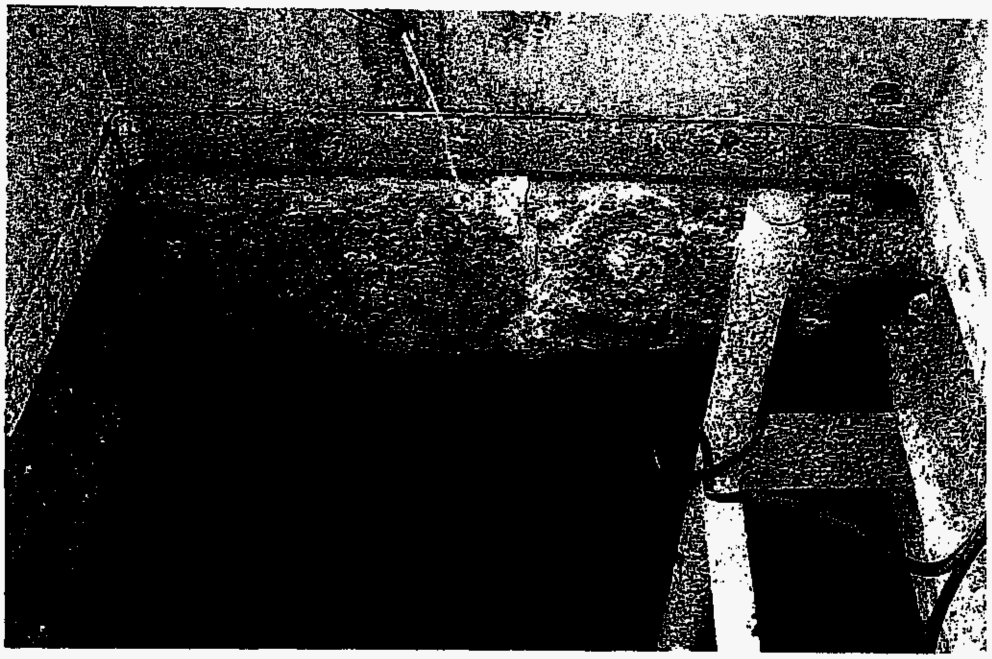


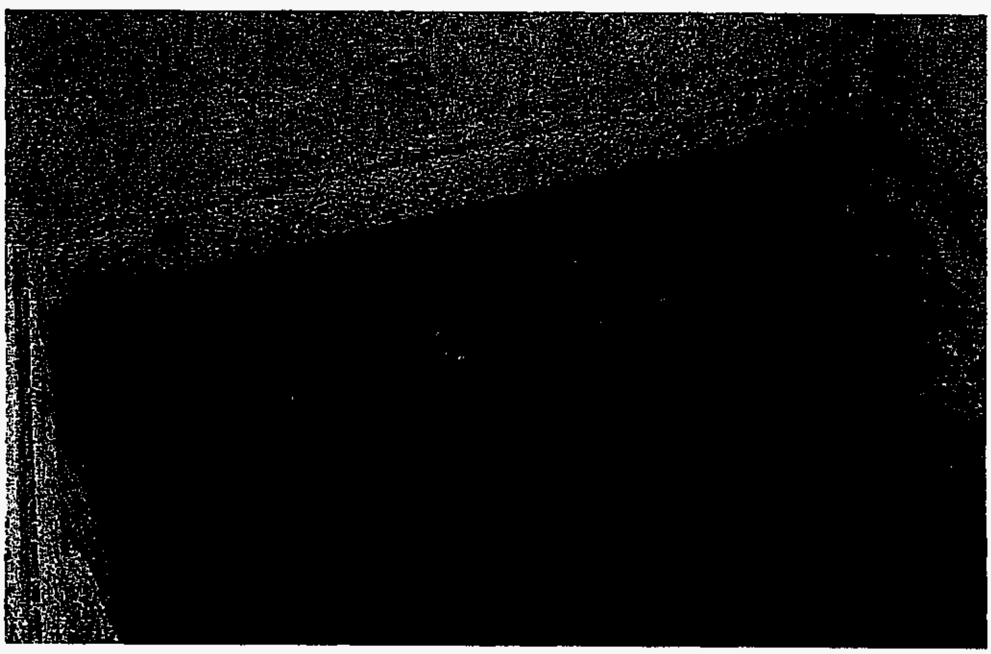


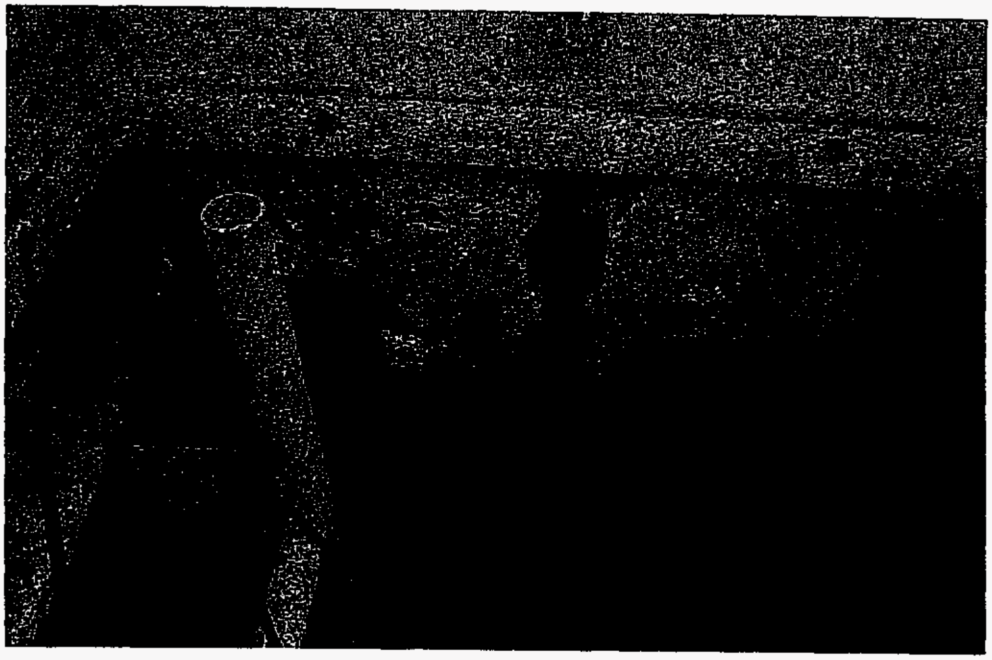


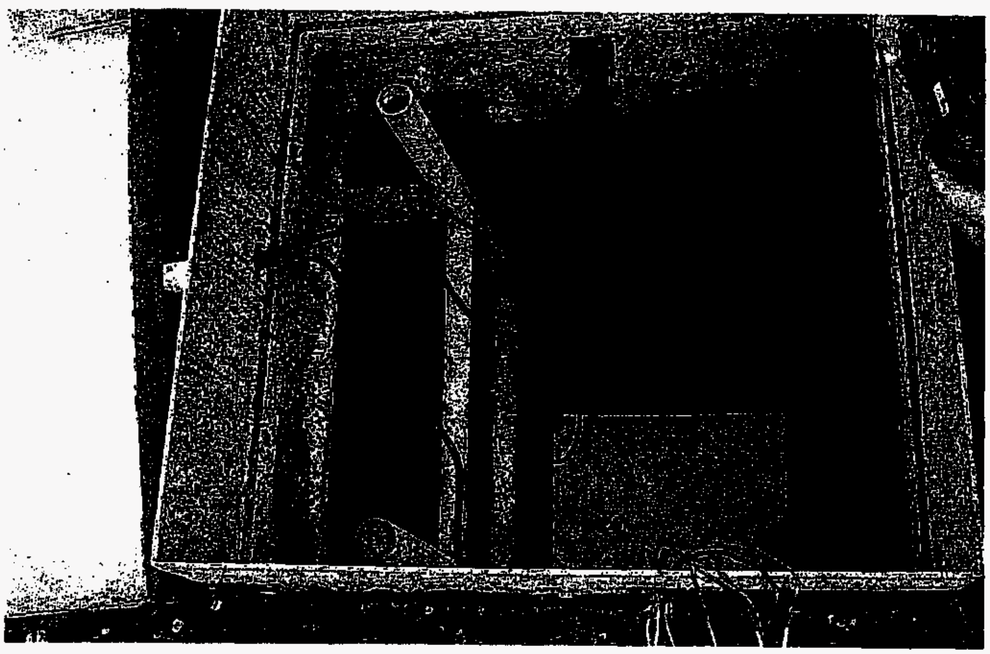


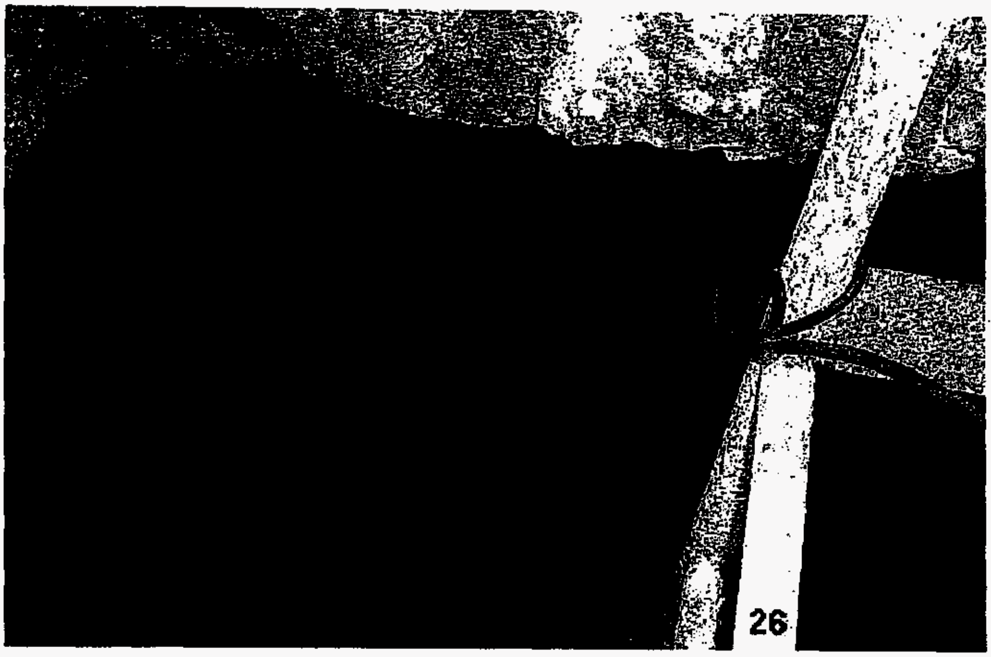








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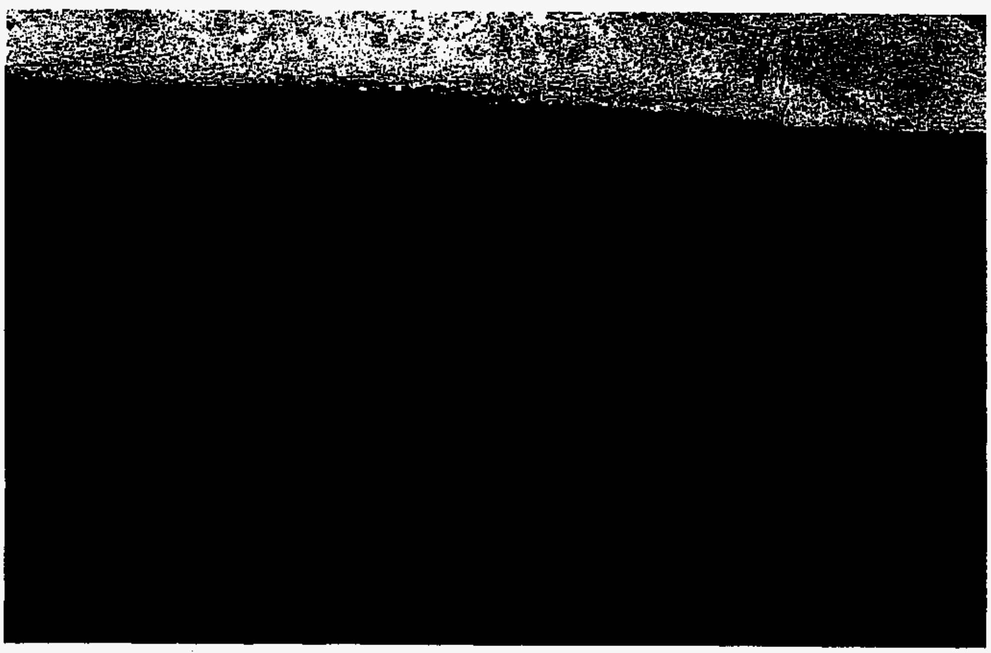

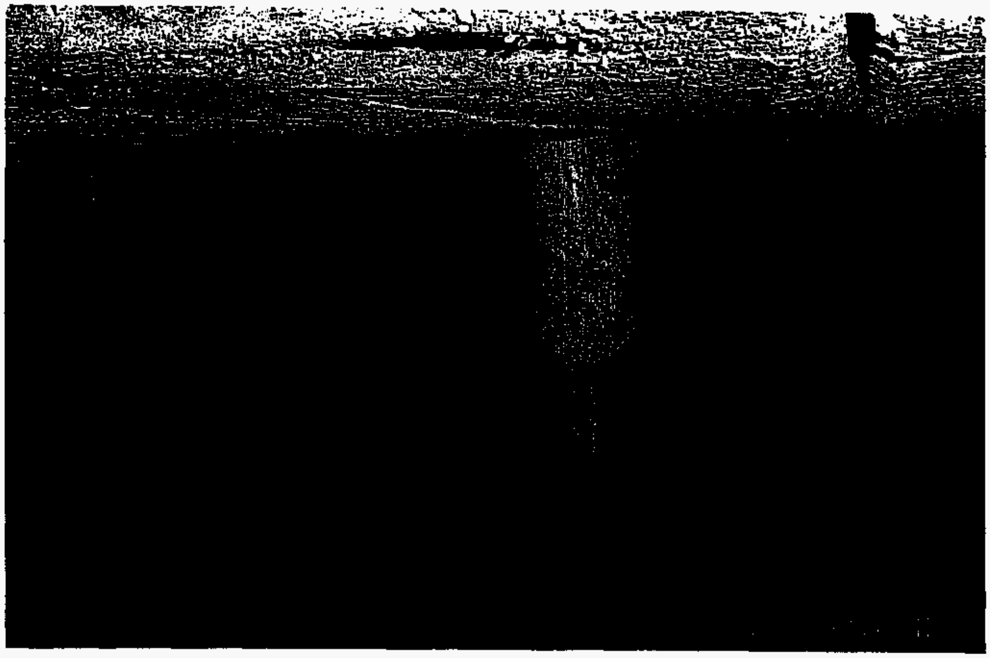

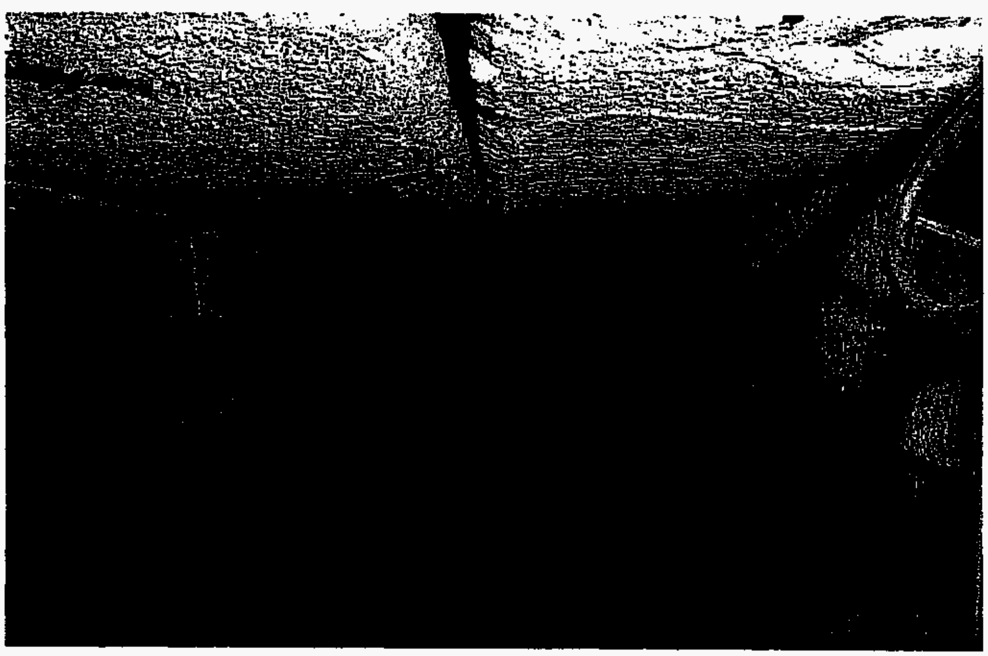

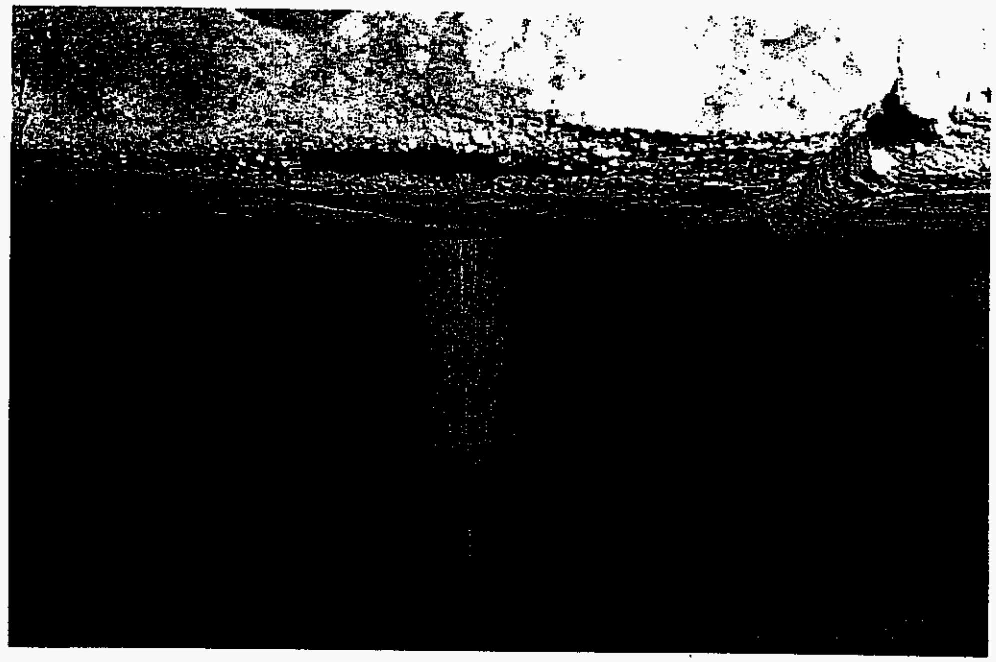

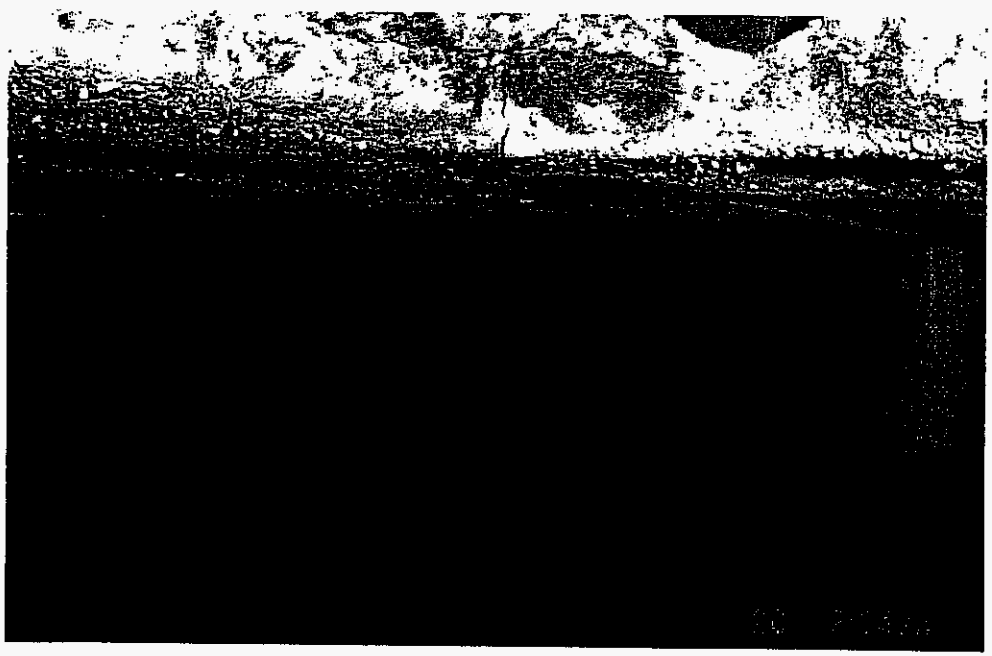

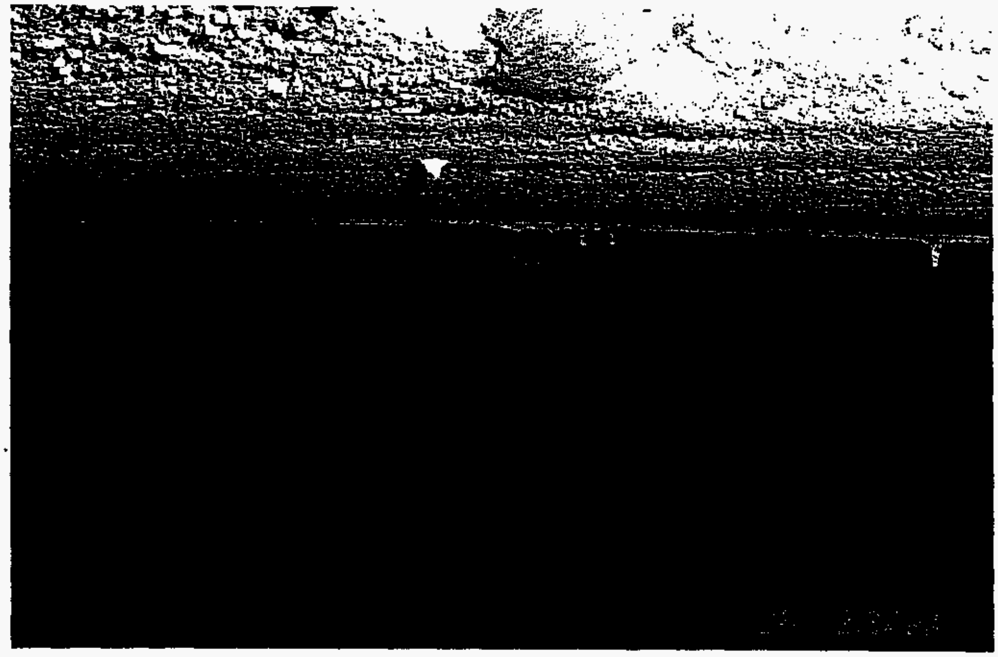

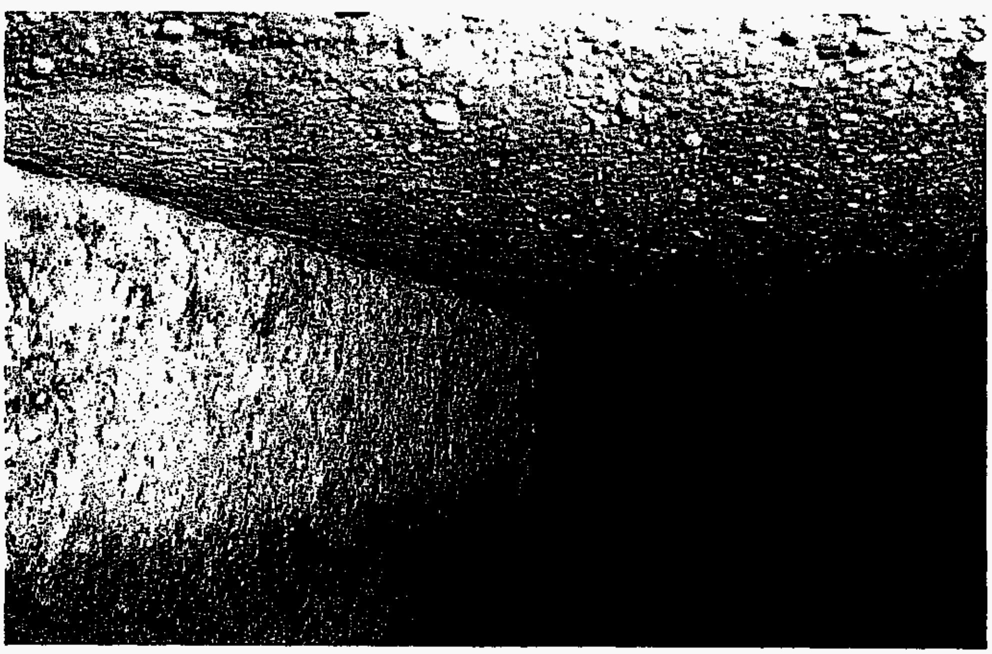

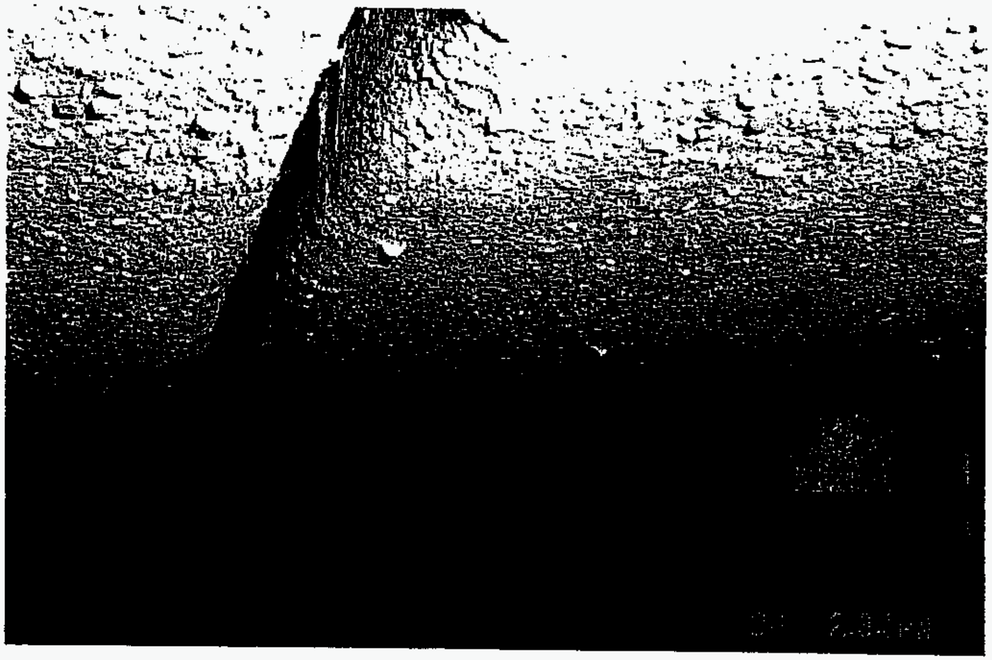

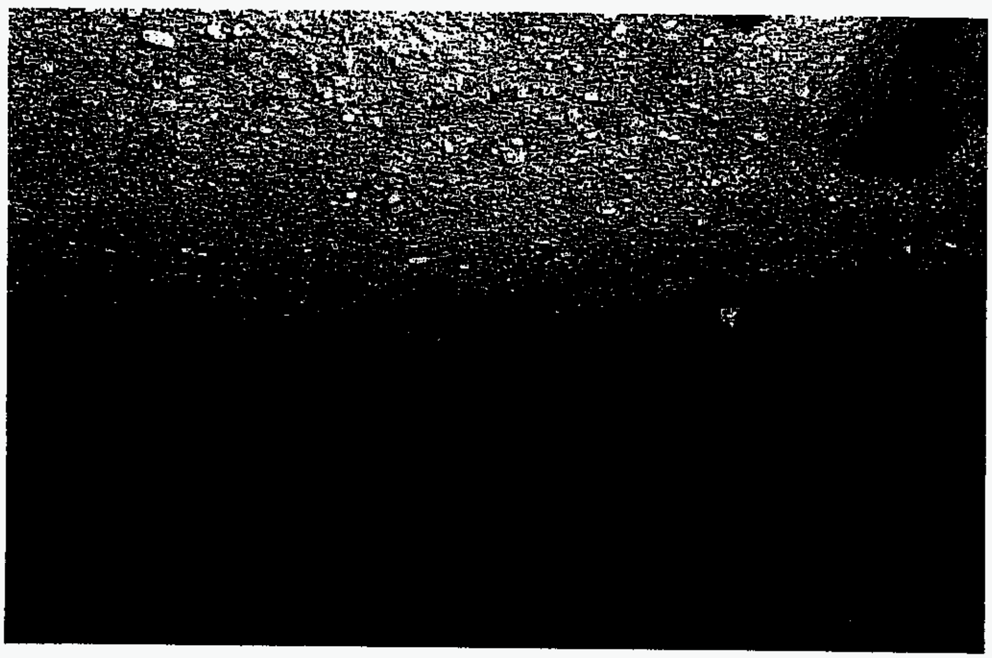

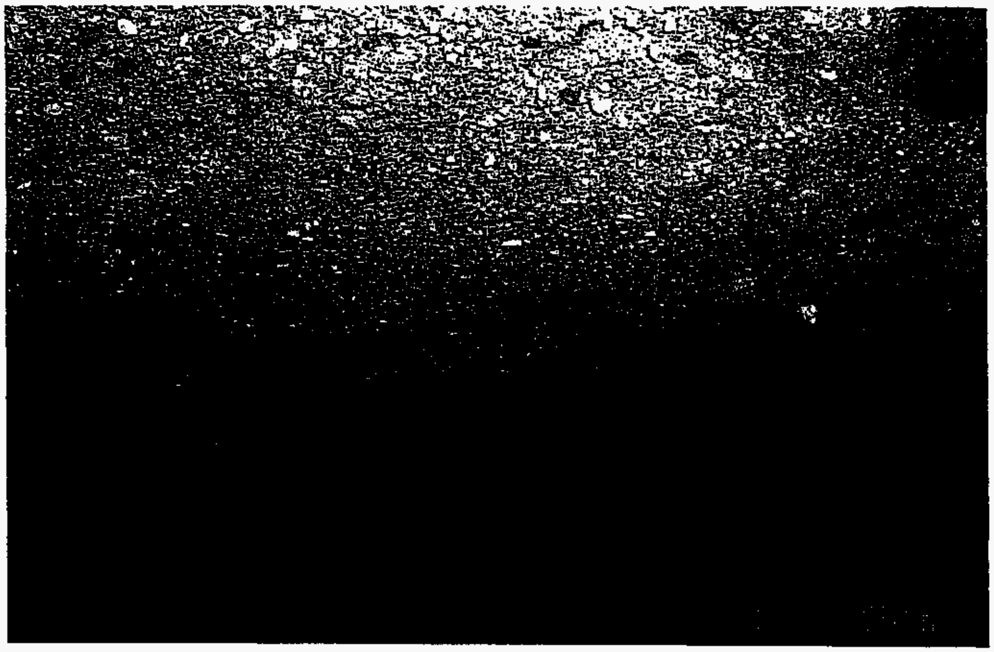

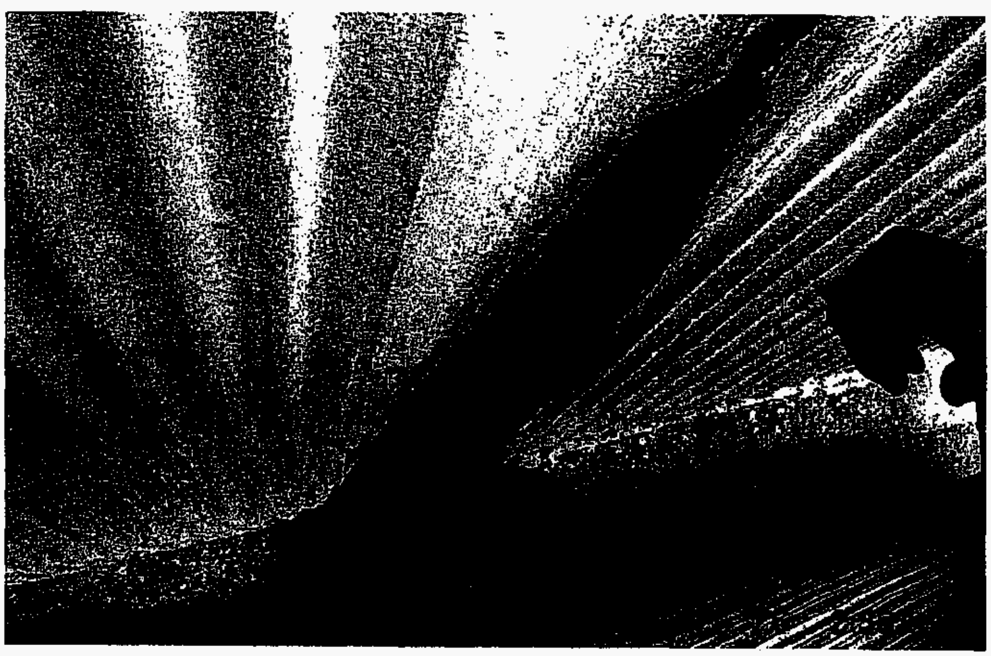

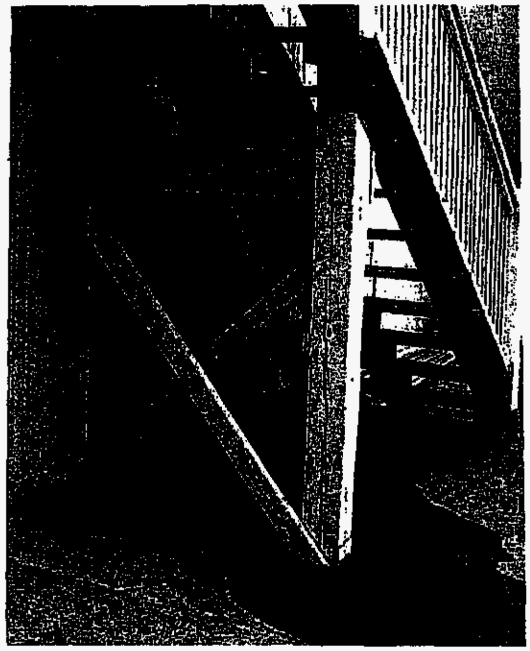

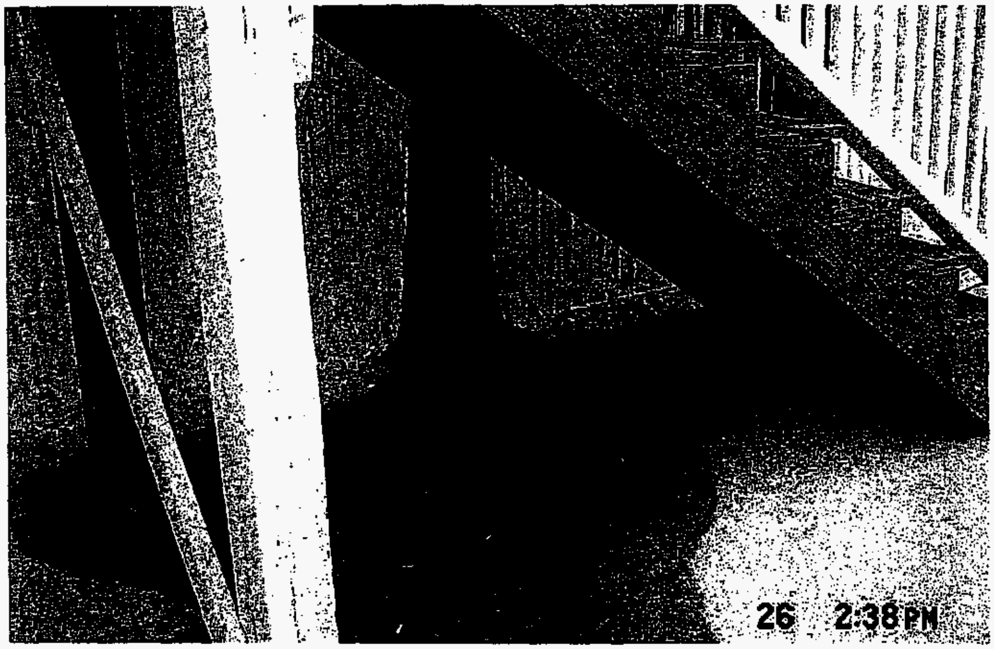
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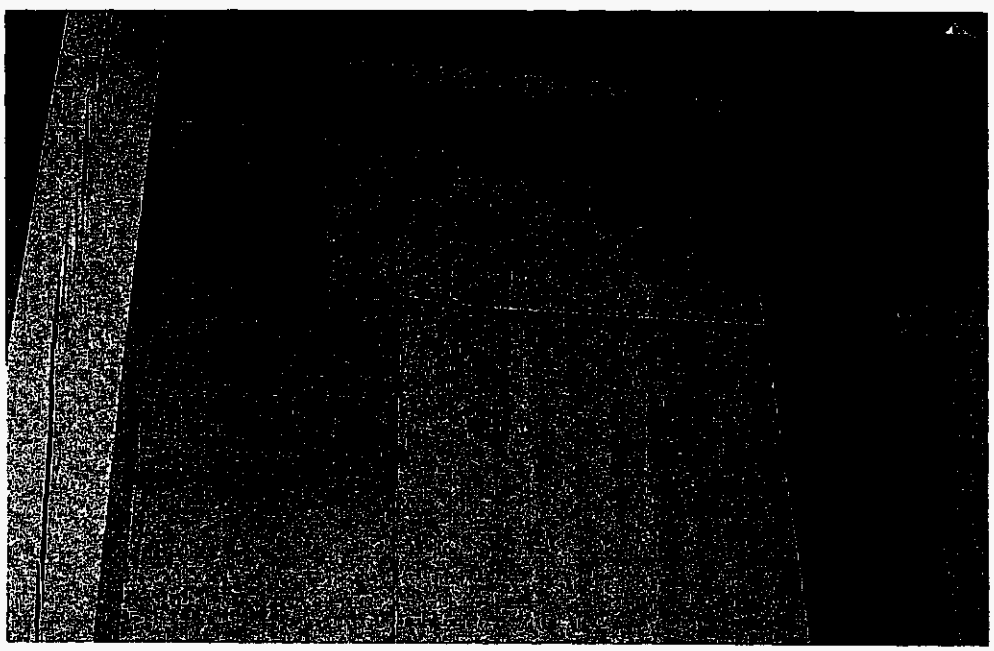

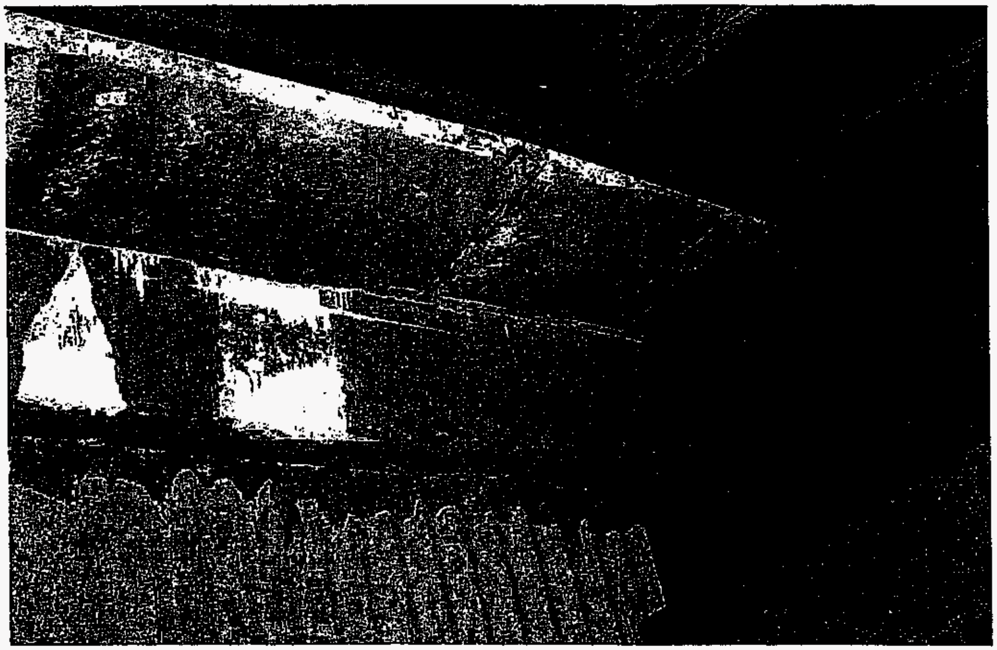

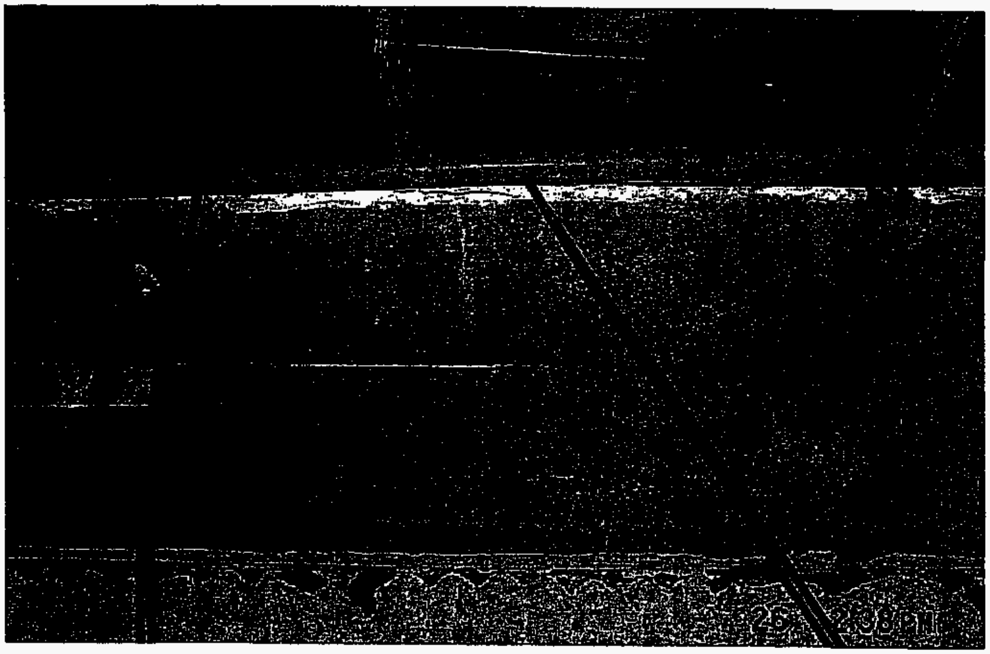

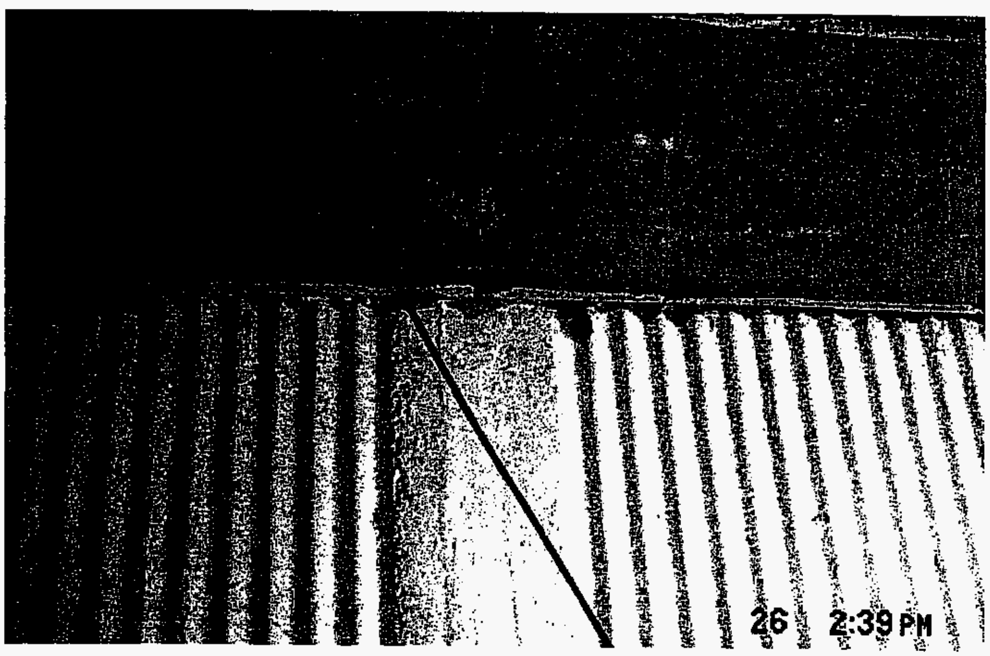
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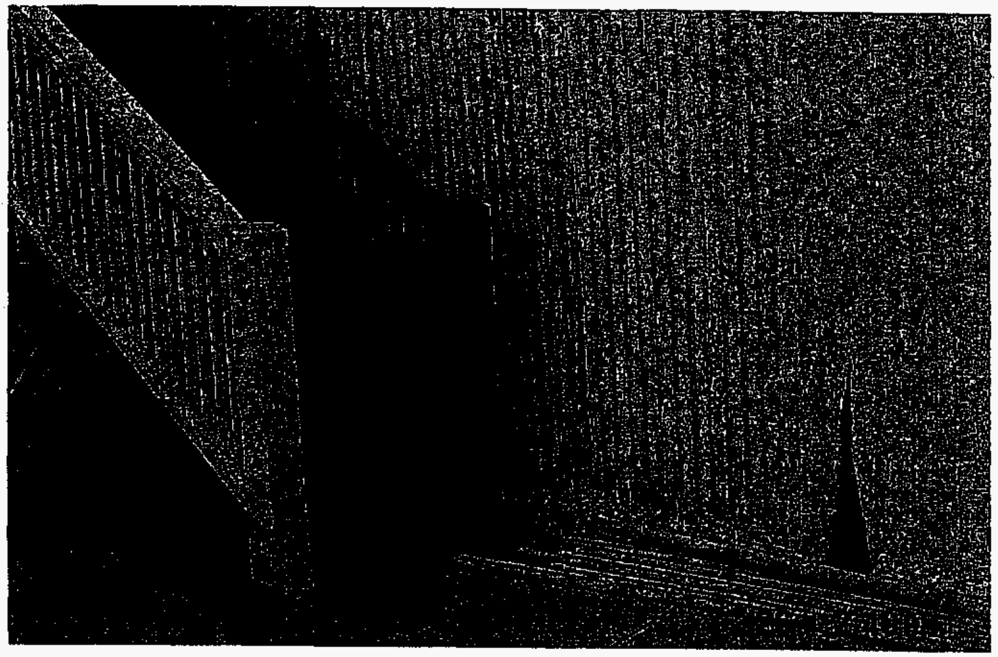

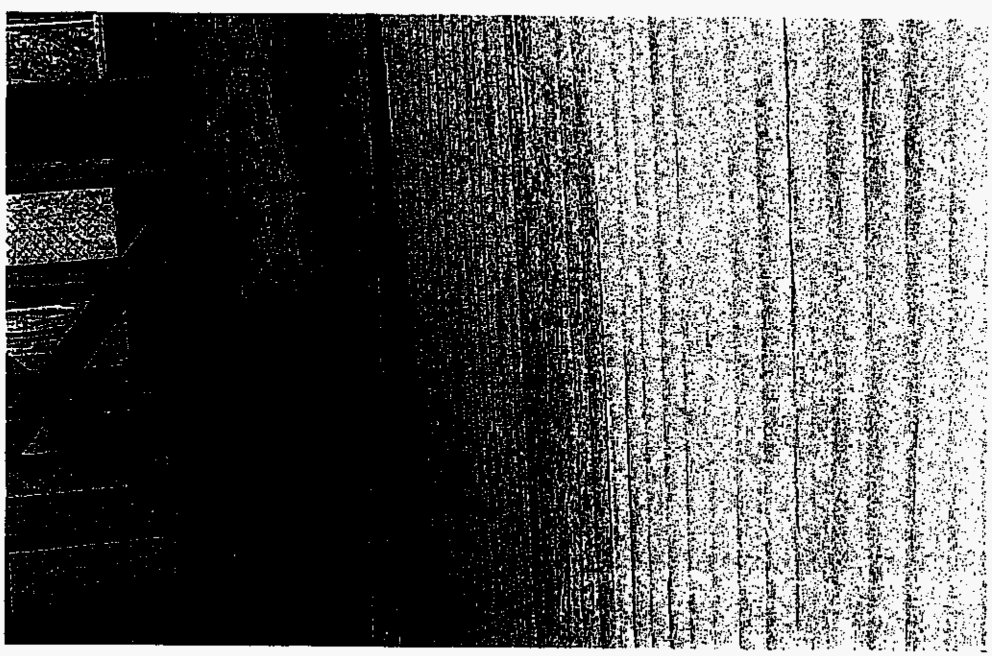

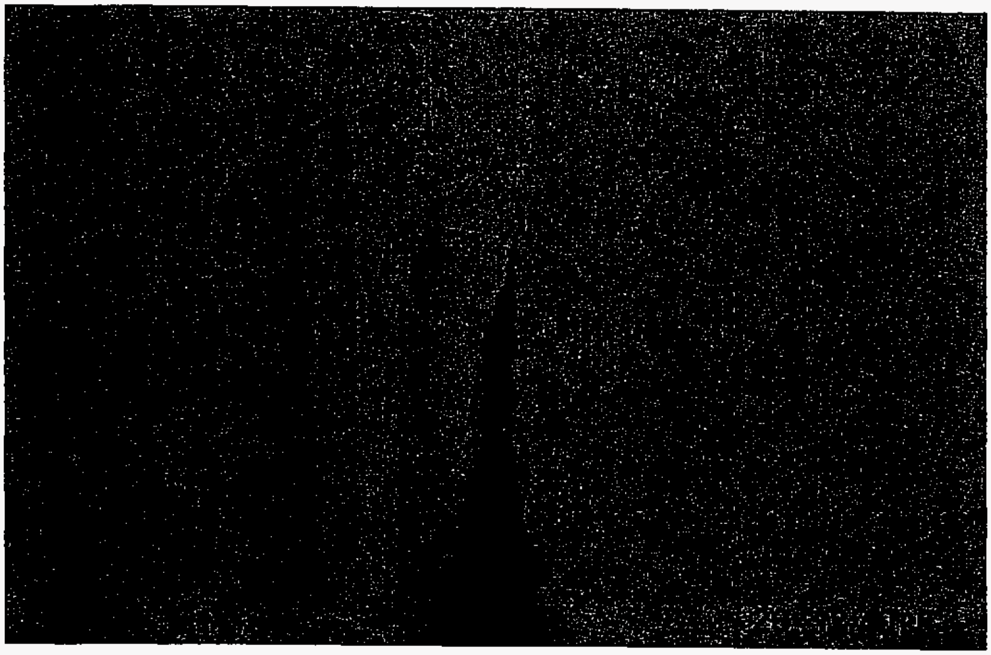

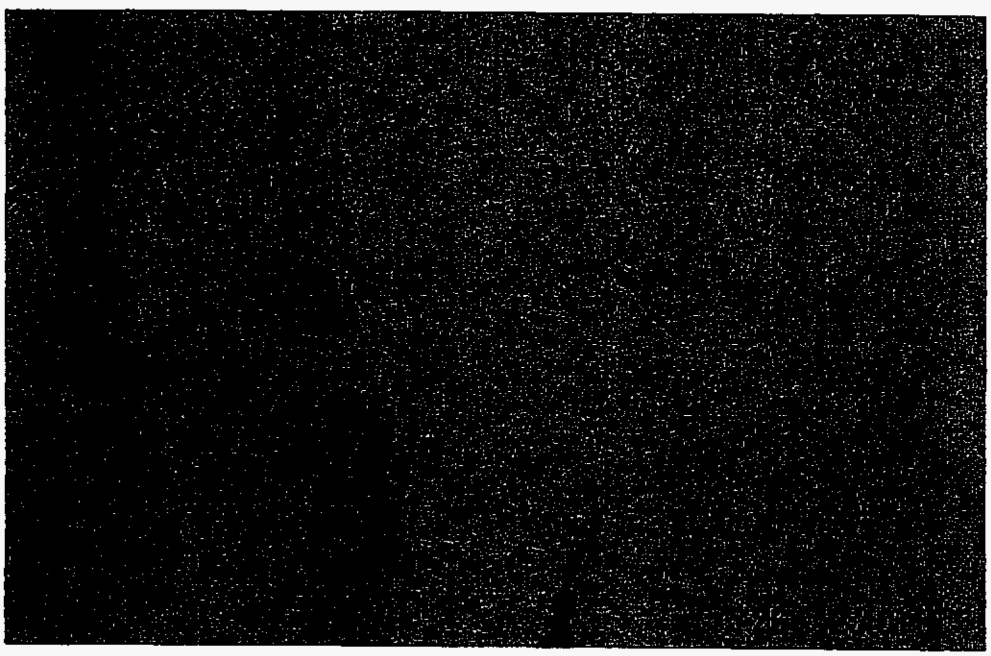

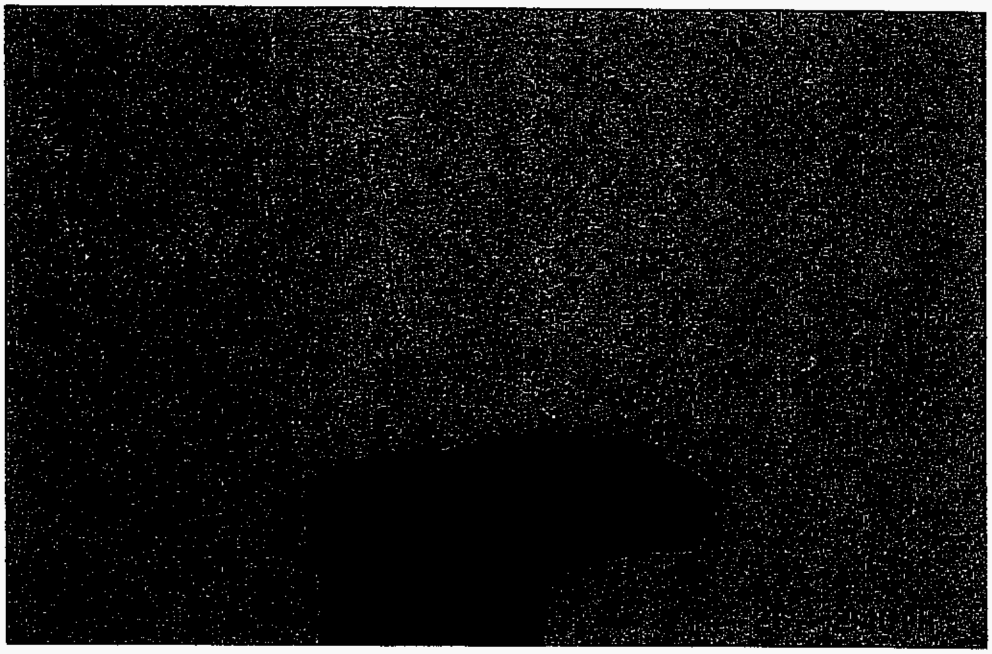

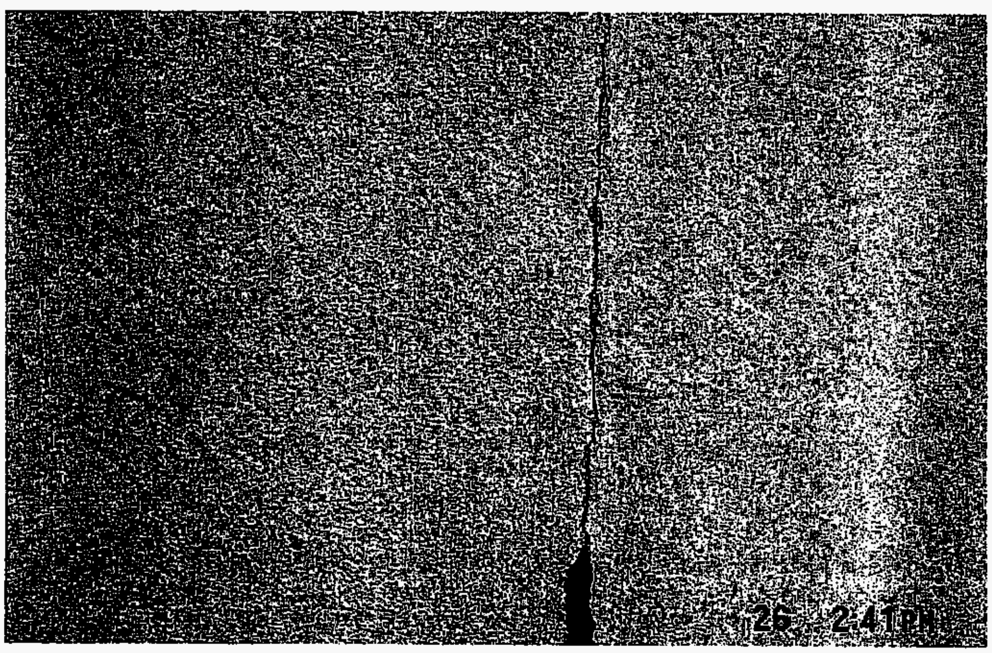
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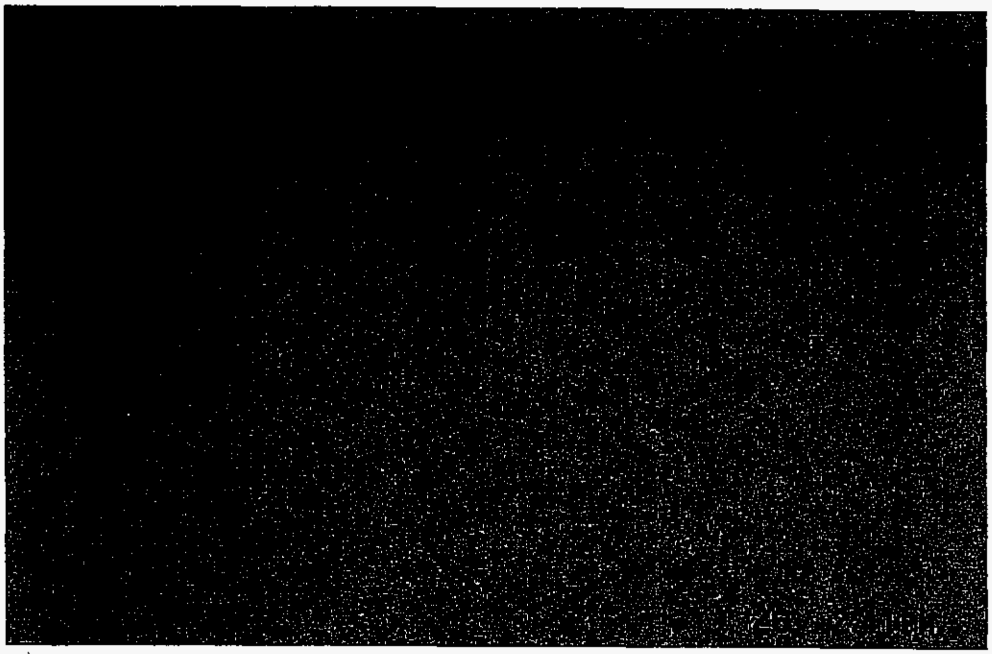

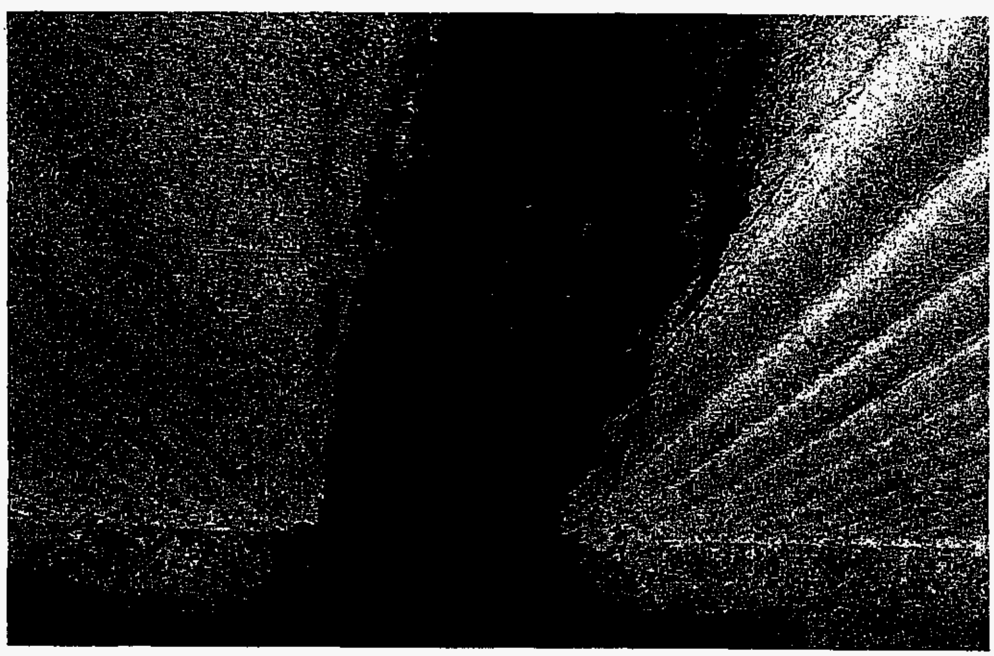

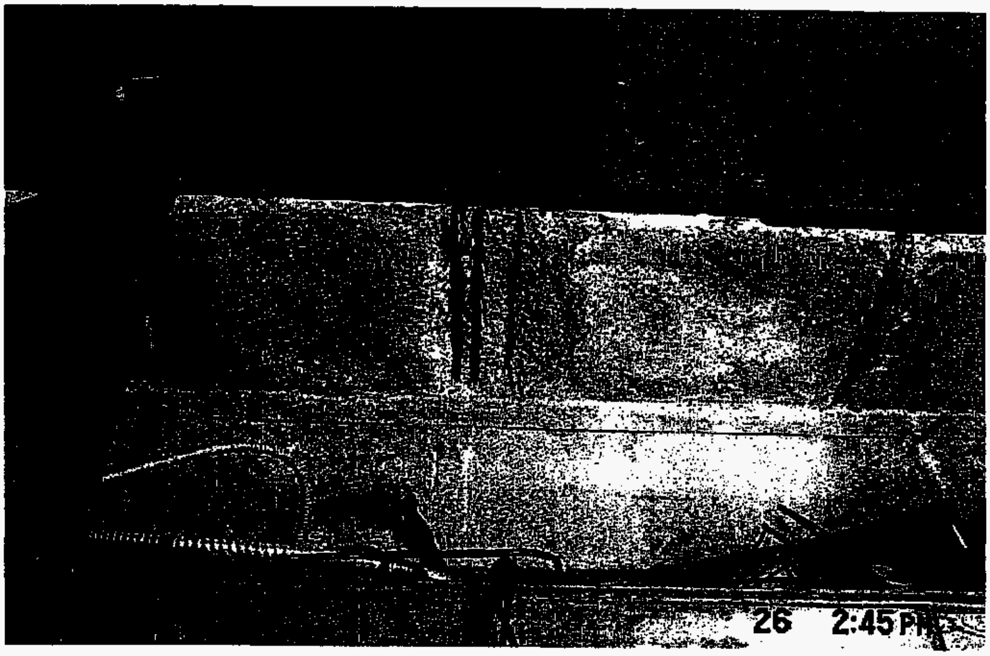
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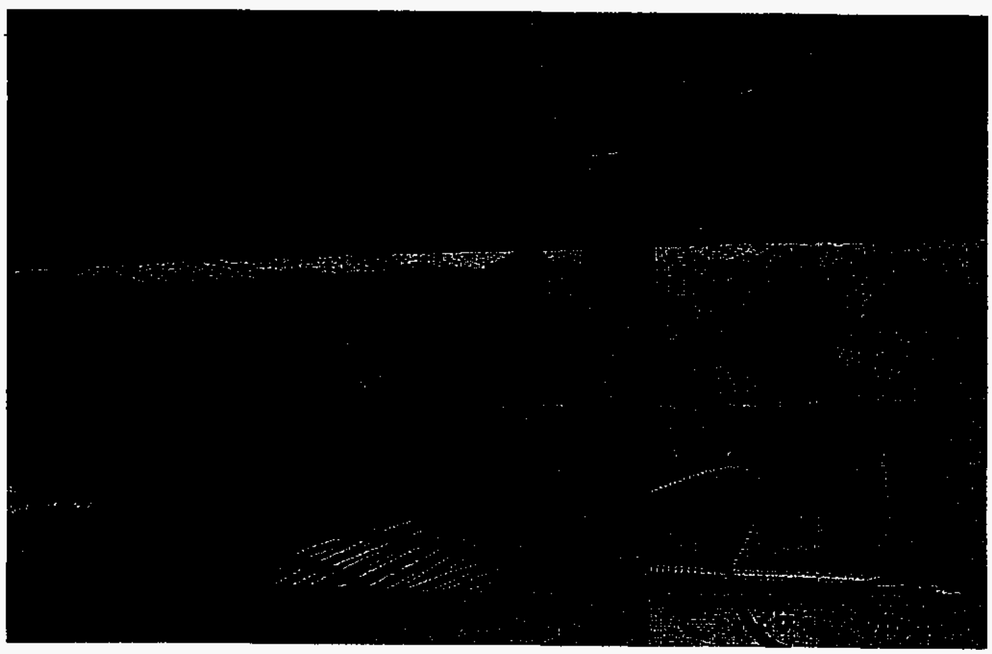

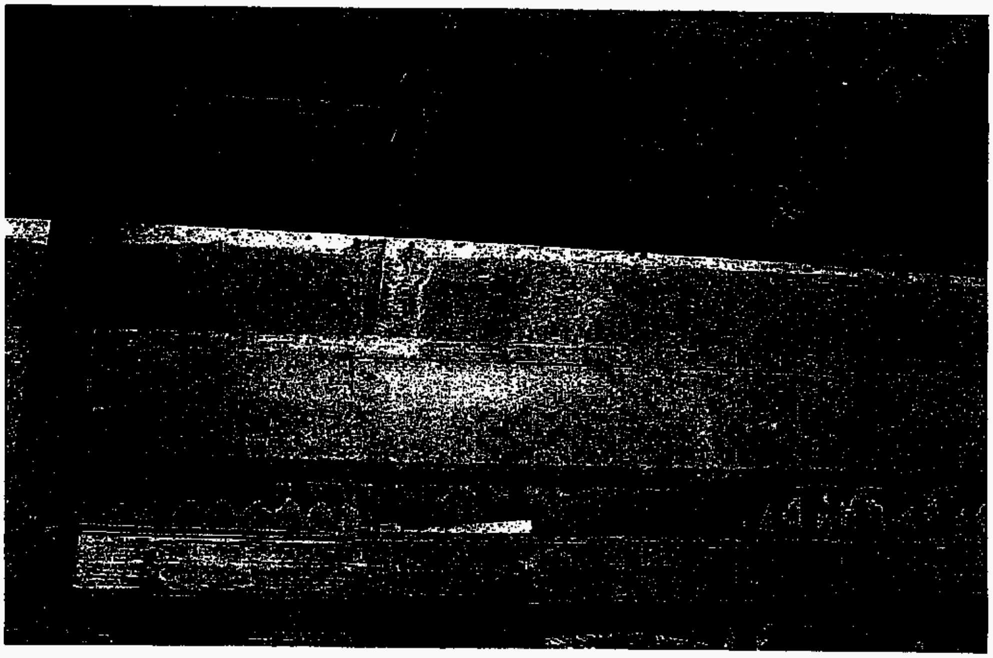

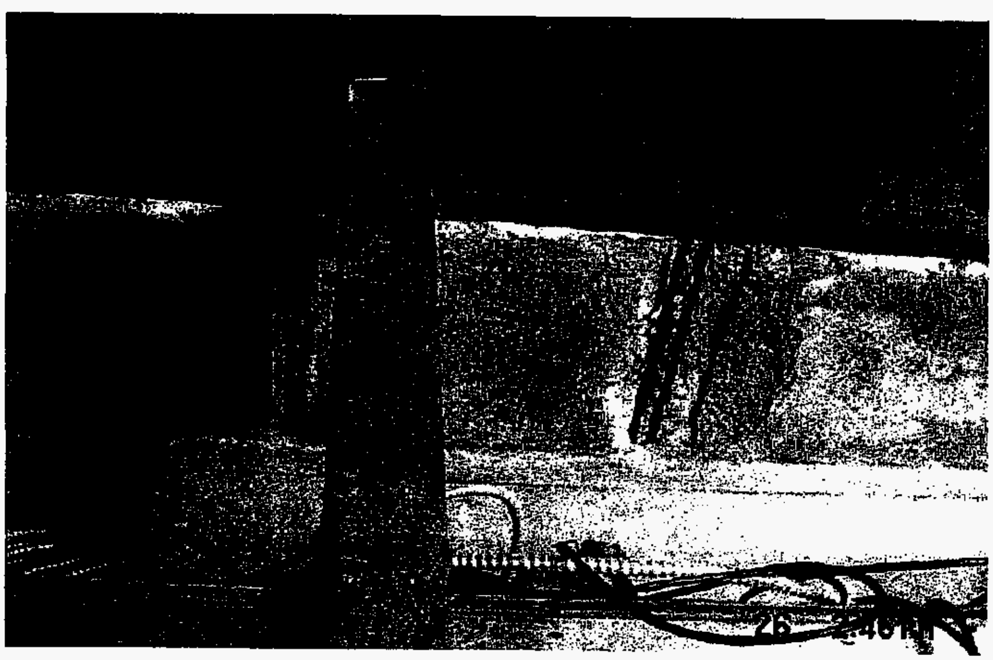

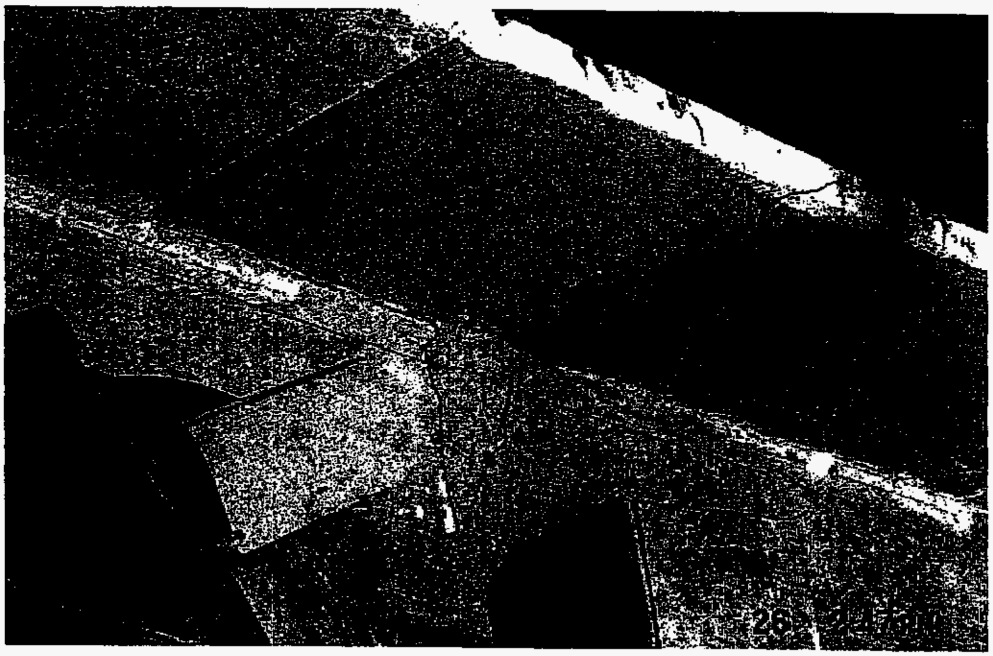

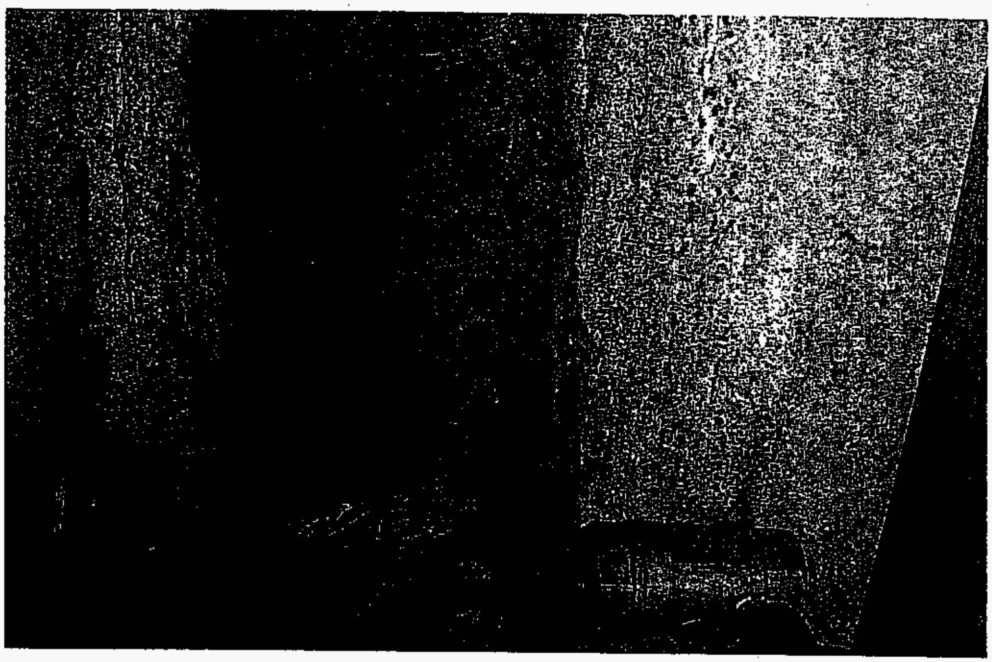

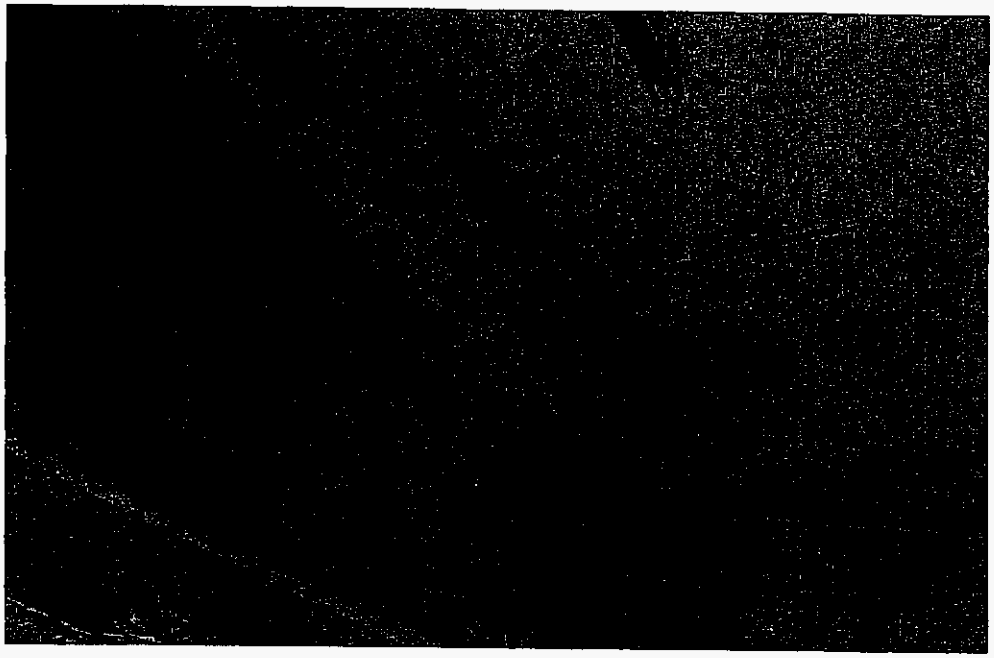

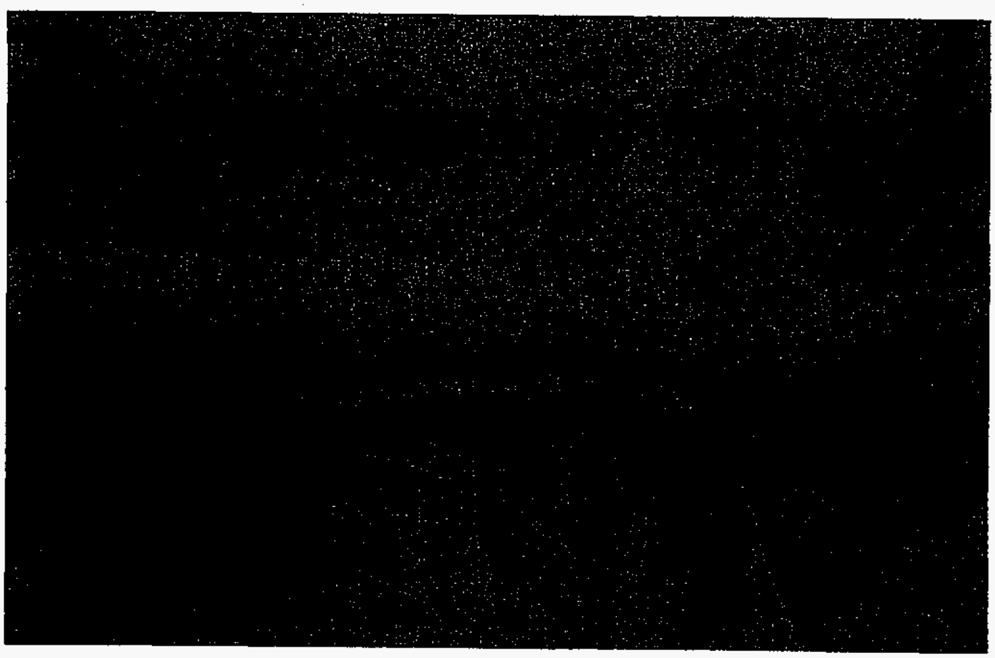

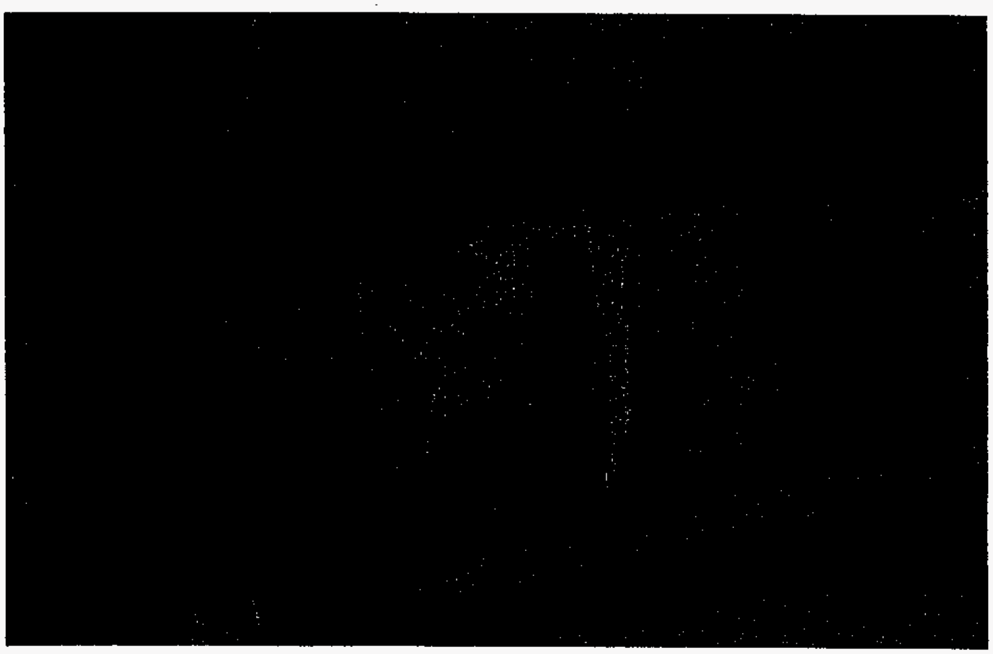

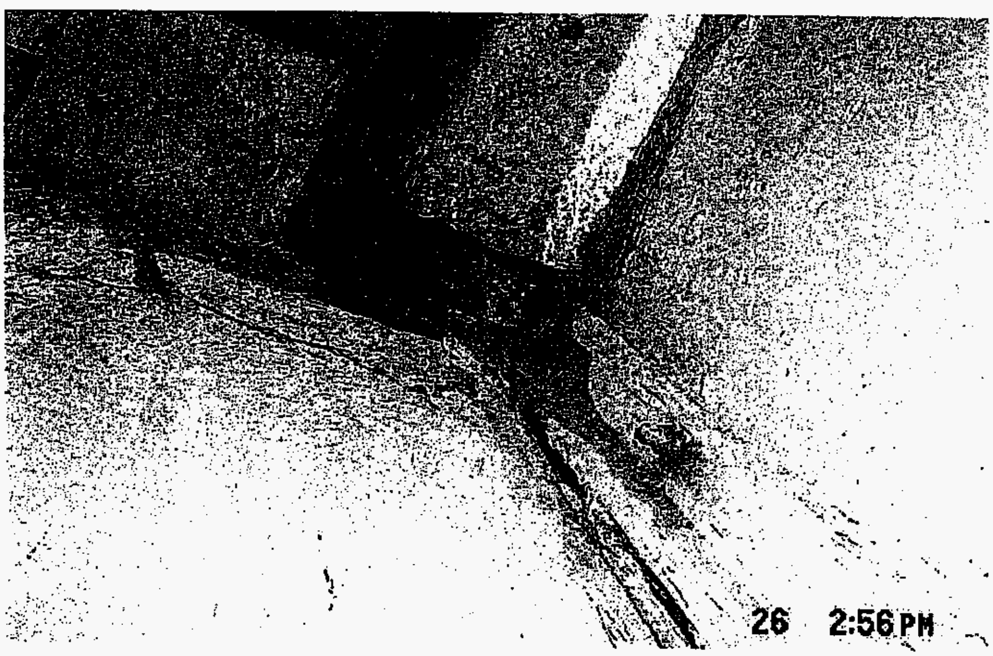
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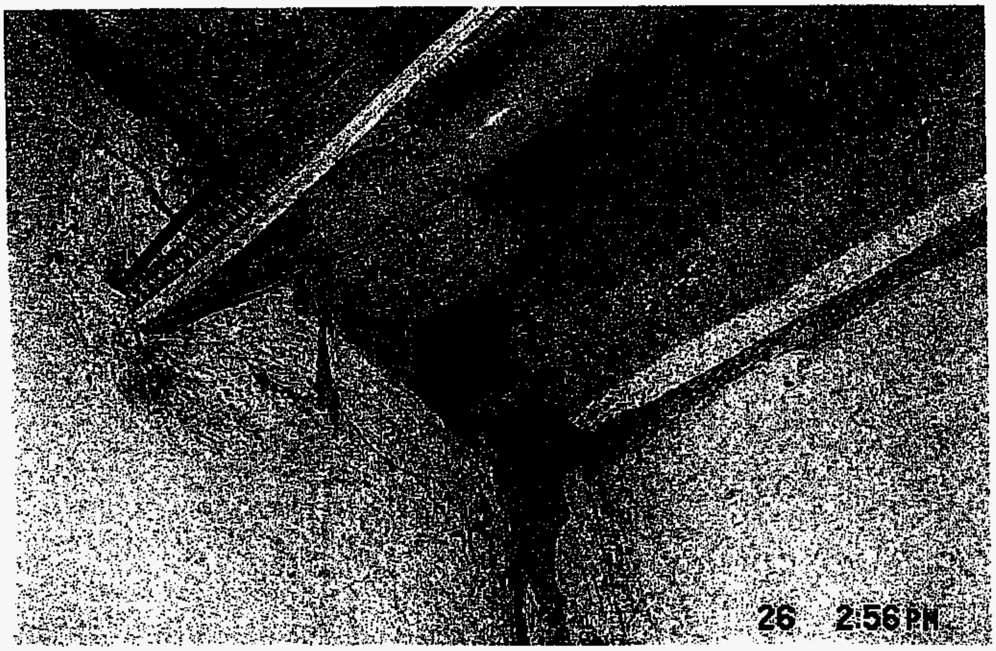
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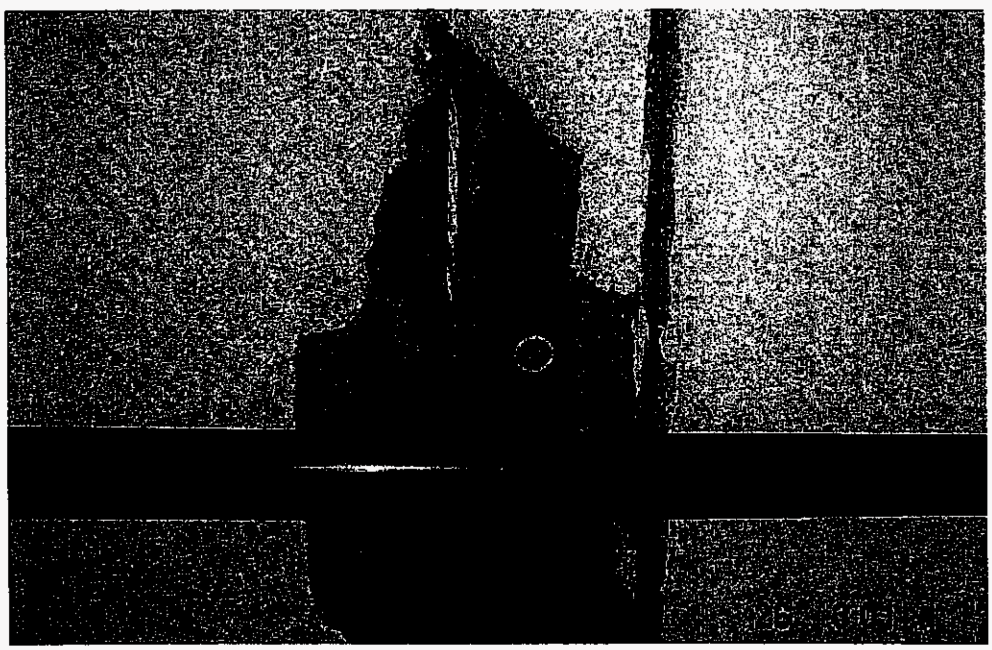
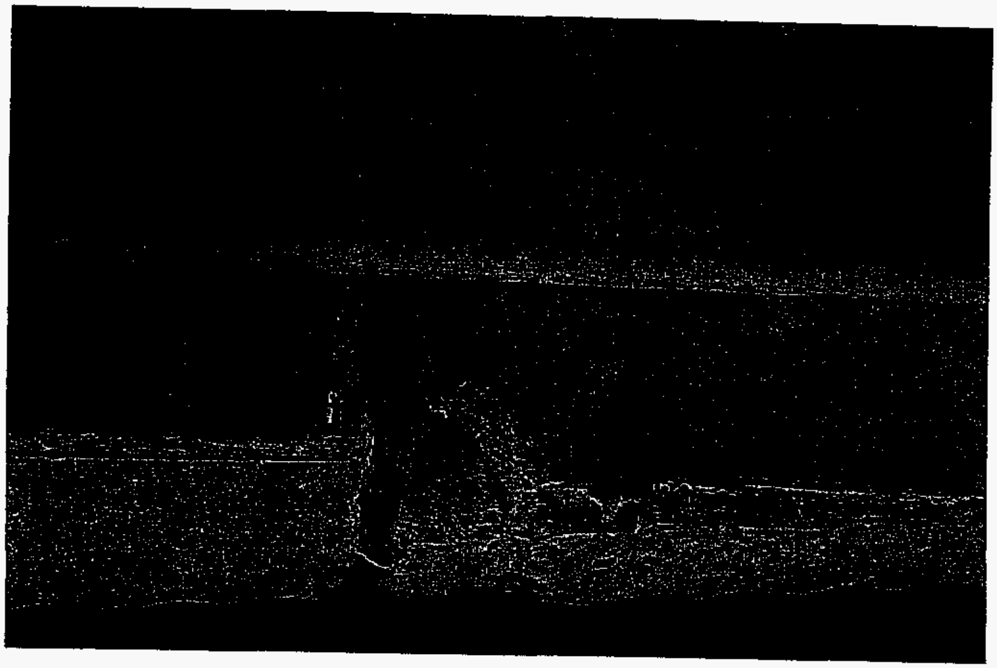
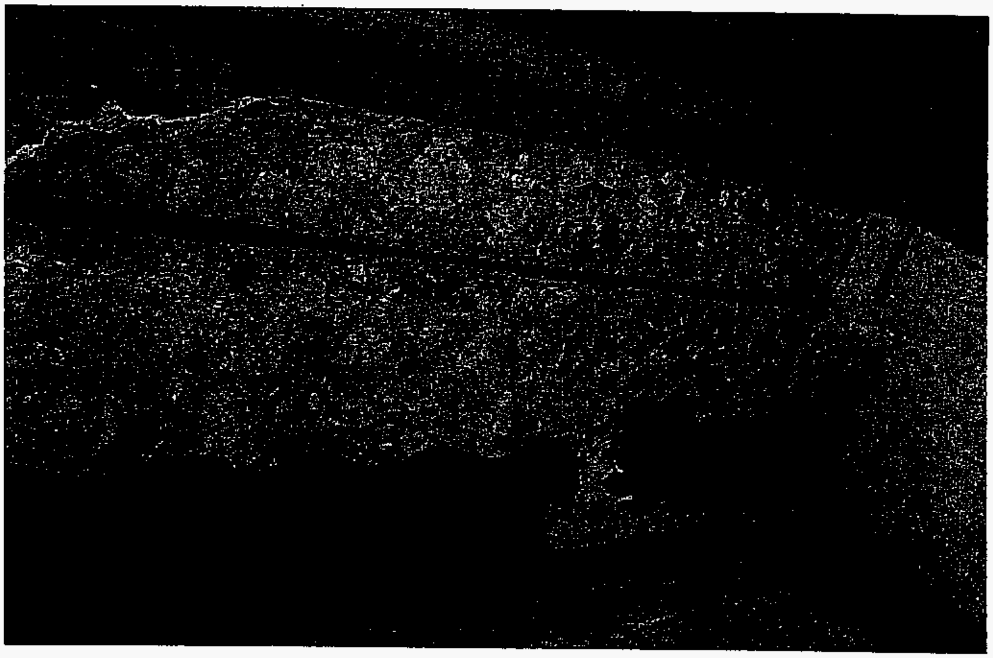
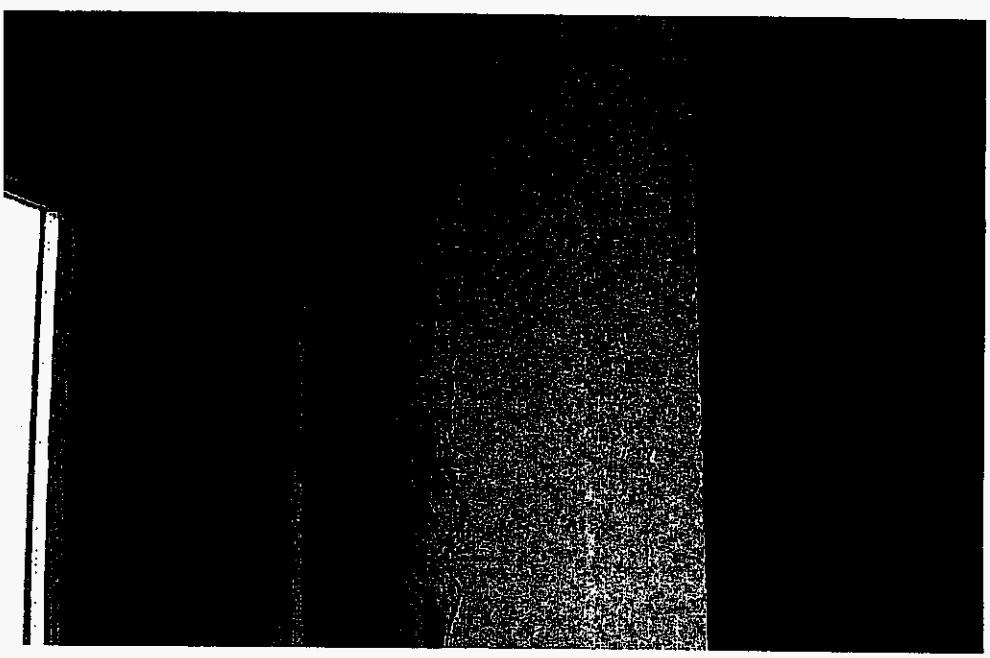








PBSJ

Technical Memorandum 6
Facilities Electrical Assessment

To:	Mr. Gene Brown, WMS		
From:	Brad Dickerson	Project:	Water Management Services, Inc.
CC:	David Gauker		
Date:	Nov 24, 2009 Revised June, 2010	Job Number:	100010111

1.0 Purpose

The purpose of this Technical Memorandum (TM) is to summarize field investigations and data collected on existing electrical and controls systems for Water Management Services, Inc. facilities at St. George, Island. The TM will describe the condition of electrical equipment, its adequacy and recommendations for improvement. The intent is to make the Owner aware of potential problems, equipment that is substandard, equipment that has reached the end of its lifecycle, visible code violations and then make recommendations to mitigate these issues.

2.0 Description of Existing Electrical

2.1 Water Treatment Facility

Electric power service is brought to the plant via pole mounted transformers provided by the local power company (Progress Energy). The local distribution voltage is dropped down to a 3-phase, 480/277 volt system using these transformers. The service feeders leave these transformers, continue down the power pole and go underground to a 400 amp meter base and 400 amp main service disconnect on the southwest exterior side of the facility. The local power company is typically responsible for the power feeder up to the line side of the meter base; from the load side downstream typically belongs to the Owner. Power cables leave the main breaker and then feed power into the electrical room adjacent to the high service pump room. The major electrical equipment inside the electrical room is the following:

- 400 amp main breaker
- 400 amp automatic transfer switch
- 480/277 volt distribution panel
- Individually mounted variable frequency drives
- 1-phase, 50 kVA step down transformer
- 42 circuit, 240/120 panel board

The main breaker inside the electrical room provides a means to disconnect normal power, while inside the electrical room, from the power provider source. The automatic transfer switch "switches" power from normal to emergency power, if the normal power source fails. Normal power is provided by the local power provider, emergency power is provided by an onsite, standby generator. (Based on information within the Valuation Report provided by WMSI, this is a 230 kW Caterpillar generator set, manufactured in 2003.)

Following the transfer switch is a distribution panel. This panel is used to distribute electrical feeders to 6 major loads at the water treatment facility using 6 breakers.

- 100 amp breaker - Pump 1 variable frequency drive
- 100 amp breaker - Pump 2 variable frequency drive
- 100 amp breaker - Pump 3 variable frequency drive
- 20 amp breaker - Pump 4 (out of service starter)
- 15 amp breaker - Chlorine Pump 1 starter
- 15 amp breaker - Chlorine Pump 2 starter
- 150 amp breaker - 50 kVA transformer

Variable frequency drives are used to control the three high service pumps at the plant. Traditional across-the-line motor starters are used to control the smaller chlorine pumps. Motor feeders leave each of these variable frequency drives and motor starters to feed power to the pumps in the high service pump room. The 50 kVA transformer is used to step-down the 480/277 volt 3-phase system to 240/120 volt 1-phase system. This voltage is used and supplied to 240 and 120 volt equipment throughout the water treatment facility, including lights, receptacles, HVAC equipment, controls, water heater etc.

The electrical system's (including the standby generator) current elevation at the WTF is susceptible to a storm surge and/or wave action. Raising the electrical equipment and standby generator above the storm surge elevation is vital.

2.2 Water Wells

There are four operational water wells and their electrical systems are very similar with the exception of standby power. Therefore, the following is a general description of all four of these systems with major differences noted as applicable.

Power is brought to each of the four wells via overhead service drop from local power company provided, pole mounted transformers. Each well house has an over head mast, weatherhead, meter base and service disconnect switch. Wells 2, 3 and 4 have an exterior and interior service disconnect; however, Well 1 only has an interior service disconnect; no exterior. These are all 480 volt, 3-phase, 200 amp electrical services.

Well 3 also has an onsite generator and automatic transfer switch. However, staff reports that the Kohler generator set is not operational. Well 4 has an operational generator and automatic transfer switch. Wells 1 and 2 do not have generators for standby power.

Power feeders are brought to each of the four well pump motor combination motor starters. These starters are housed in custom built enclosures, which house the motor circuit breaker, starter and relay controls. Wells 1, 2, and 4 are using solid state soft start starters. Well 3's method of starting was not recorded. Each starter has a Hand-Off-Automatic switch (HOA). This switch typically would be placed in the Auto position for Master PLC remote control. In the Off position the pump will not run. In the Hand position the pump can be run local-manually.

These are the major electrical components at each well house. There are minor components like small step down transformers for lights and receptacles. Some of the well houses have surge arrestors also.

3.0 Description of Existing Instrumentation and Controls

3.1 Water Treatment Facility and Wells

The Water Treatment Facility houses a Master Programmable Logic Controller (PLC). This PLC controls the high service pumps locally at the plant, the four remote well sites and monitors tank levels at the plant (one ground storage tank (GST) and one elevated storage tank). The exterior face of the panel has run/fail indicator lamps, hand-off-automatic switches and tank level displays for operator monitoring and control. Hand-off-automatic switches must be in the auto position for Mater PLC control. Pumps can be taken out of service or manually run by using the H-O-A switches at the Mater PLC.

Locally, the PLC monitors distribution system pressure and using the high service pumps it tries to maintain a range of system pressure using the three operational high service pumps. Pressure is monitored by a small pressure transducer in the high service pump room. A typical pump control scheme is used to maintain pressure. As pressure falls due to system demand a set point calls the lead pump to run, if the pressure continues to fall and the lead pump is running at 100% speed, the lag pump is called to run and so on, until all available pumps are running at 100% speed. As system pressure rises the pumps begin to slow down and finally shut down on high pressure. The high service pumps have VFDs, which allow a ramping control of the flow rate. Pump's speed is changed using the VFDs. This in turn increases or decreases flow rate. Flow rate is monitored using magnetic flow meters on the discharge of each of the high service pumps. This flow rate is monitored and used for control by the PLC. Flow rate is also indicated at each of the flow meter's transmitters mounted next to the PLC controller.

The Master PLC also monitors the GST level and calls well pumps 1-4 to run based on the GST level. The hand-off-automatic switches at the Mater PLC and remotely at each well must be in the automatic position for Mater PLC control to work. Communications with each well is via telephone modem. The Master PLC communicates via the phone line with a small PLC at each well house. These small, local PLCs call the well pump to run or stop at each well house when given a signal from the Master PLC. The front of the Master PLC control panel indicates well pump running, fail and communication failure. It is not clear what type level control device is used inside the GST; however, a float switch was visible from the exterior hatch. Therefore, it is possible that a single or multiple floats are being used to monitor level and control the well pumps.

4.0 Site Evaluation

The intent of this TM is to discuss the existing conditions of the electrical distribution system, controls and instrumentation in order to make the Owner aware of potential problems, code violations, etc. The following items were observations during the visual investigation. This TM is meant to point out major issues; however, it certainly is not comprehensive of all potential problems that may exist at the Water Treatment Facility and Well Houses. Many times major problems and code violations are hidden from a simple visual inspection.

4.1 Water Treatment Facility

The condition of the existing electrical system and controls at the Water Treatment Facility is a major concern. It is obvious that over several years many, many changes have taken place at the WTF and these changes have included electrical additions. These additions have been made with disregard to standard and typical industrial electrical installation methods. The result of this type of work has created many code violations and electrical systems that cannot be properly maintained. Furthermore, these systems are unreliable and hazardous to operations staff. Operators mentioned during the site evaluation that the controls are problematic and because they are proprietary; therefore, they are not easily serviced.

<u>Master Control Panel</u>

- The condition of the Master Control Panel is poor, as such, the entire water treatment and associated wells are vulnerable to failure resulting in total loss of service to the customers. As shown in **Figure 4.1**, wiring is not neatly trained or supported. There are many devices hanging in the air with no support. Receptacles cover the bottom of the panel with no covers or protection. The door cannot be closed because of cabling between the panel and external modems. A technician could not work on any device in this panel without having to push wiring aside and critical components, like phone connections to well houses are simply hanging in mid-air without support. Equipment in this panel is poorly installed and is a substandard quality for a typical application in this industry. According to WTF staff there are no schematics or record drawing information available for this control panel. Lack of control schematics is an issue that needs to be addressed.

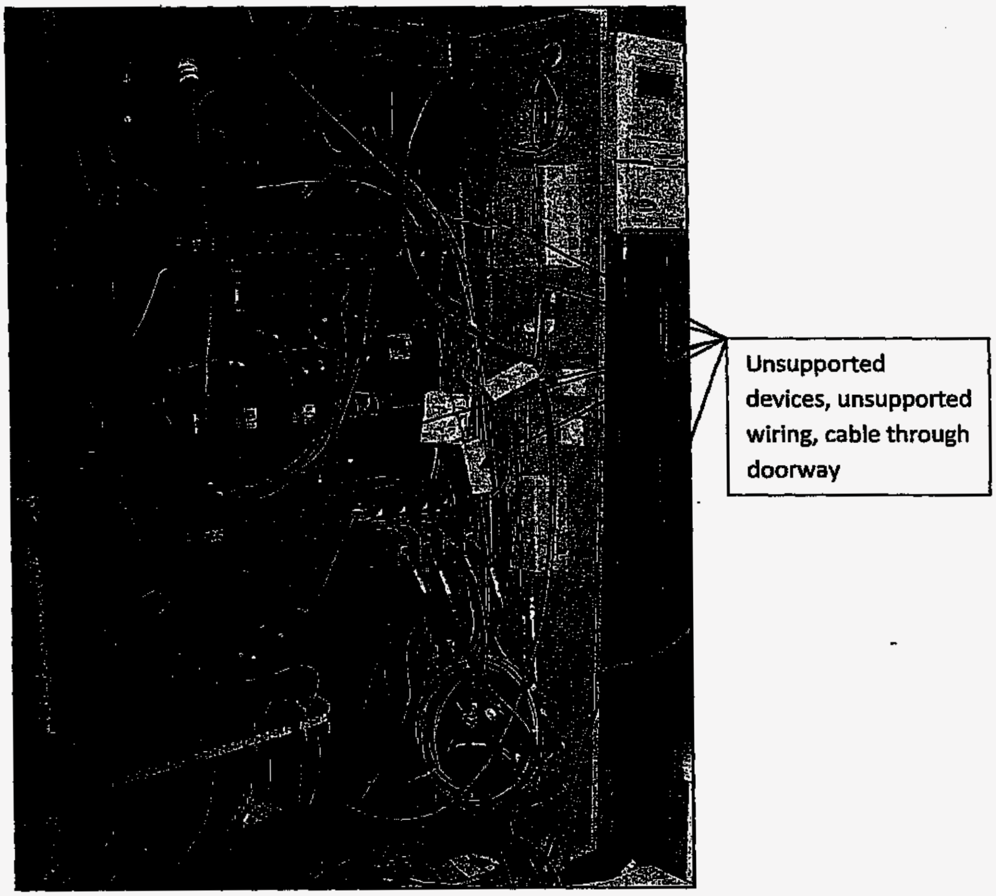


Figure 4.1 - WTF Control Panel Interior

WTF Electrical Room

- The electrical room's condition is very poor and multiple code violations are present inside the electrical room. Safety is a major concern. All issues relate to personal safety, fire hazards and code violations. The following is a short list of visible problems (refer to **Figure 4.2**):
  - The National Electrical Code (NEC) requires clear working space in front of every electrical enclosure. A minimum of 36" is needed in front of every enclosure. During our investigation, the electrical room was being used as a storage area. There were a lot of boxes and other materials in front of electrical equipment. This does not meet NEC.
  - Wireways are not properly closed and secure.
  - Wire is exposed everywhere with no protection from damage.
  - Wiring and conduit are hanging lose from the ceiling with insufficient or no support.
  - Live wires appear to be wrapped around ceiling grids to support the wire.
  - Old abandoned equipment and enclosures were not removed prior to new equipment being installed. These old enclosures appear now to be used for wireways.
  - Multiple taps and connections are visible in the ceiling. These connections have been made in free space; no enclosure or protection for the wire and the tap.

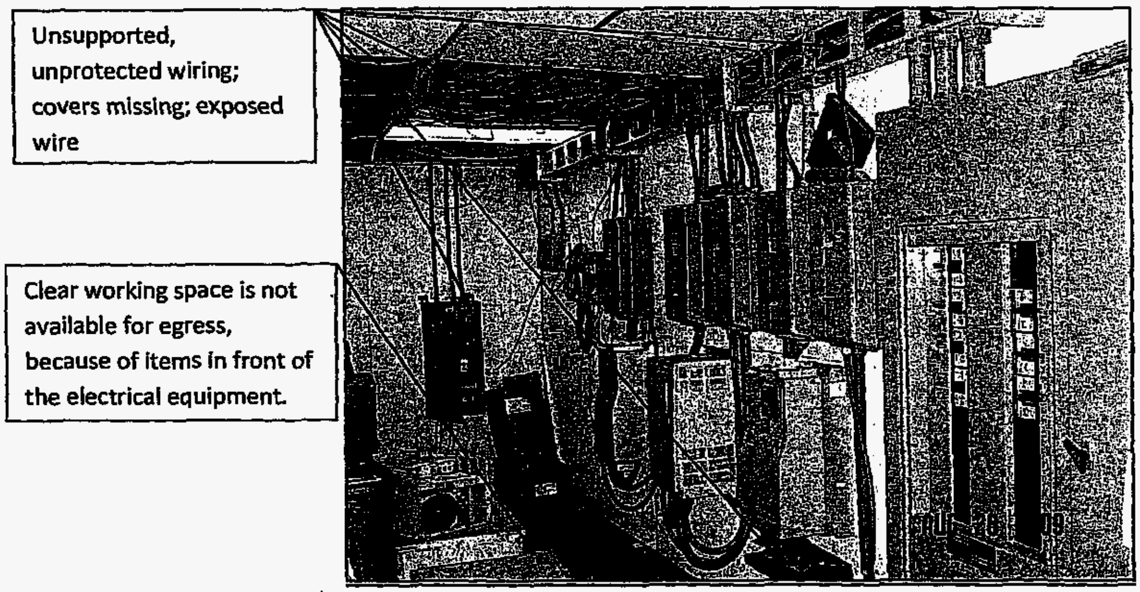


Figure 4.2 - WTF Electrical Room

Multiple cable splices and multiple cables hanging, unsupported and unprotected.

Figure 4.3-Electrical Room Conduits at ceiling

NM type-residential cable is being used to feed many circuits in the facility; wire is wrapped around ceiling tile grid; PVC conduit partially run and unsupported

Figure 4.4-Wiring in electrical room above panel board

Enclosure cover is not properly installed. This changes the equipments actual withstand rating.

Flammable materials should not be stored on transformers.

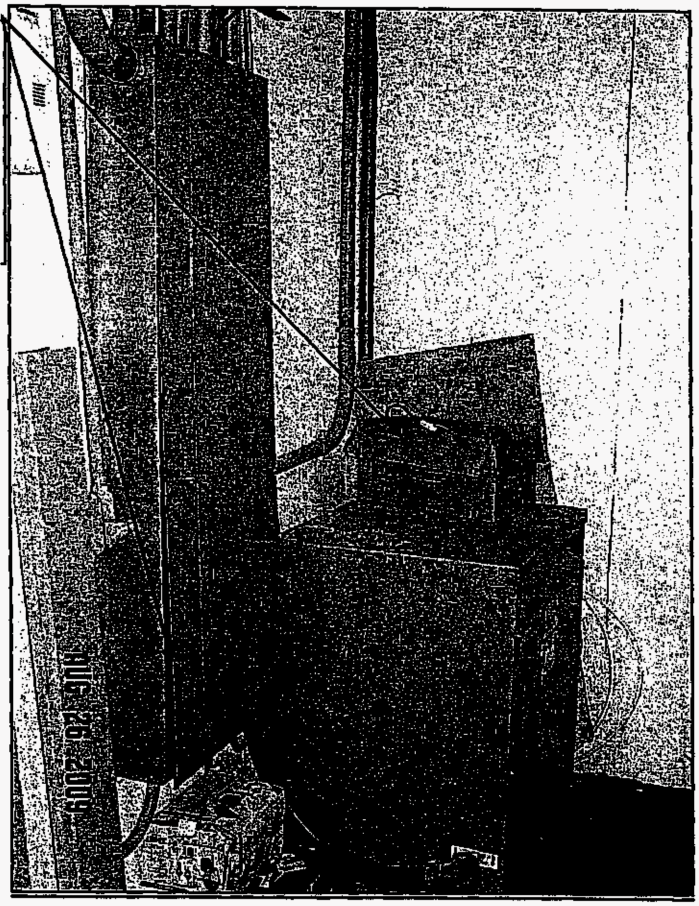


Figure 4.5-Electrical Room panelboard and 50kVA transformer

WTF General Electrical Issues:

- Below are photos of several reoccurring issues at the WTF. Many locations have bad conduit connections and improper electrical materials being used.

Bad conduit connection; exposing wire

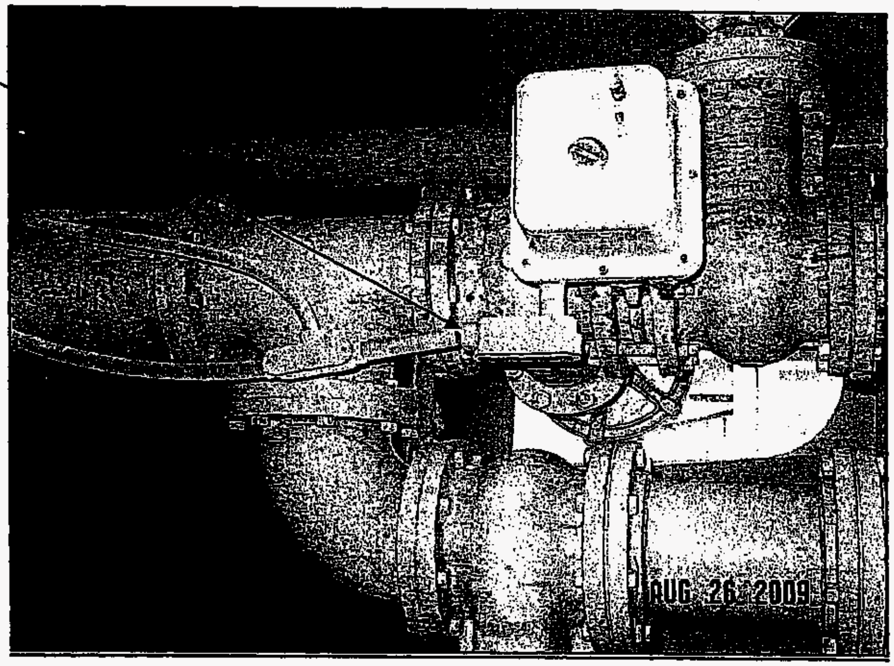


Figure 4.6-Elevated tank control valve

Bad conduit connection, exposed wire and exposed terminations.

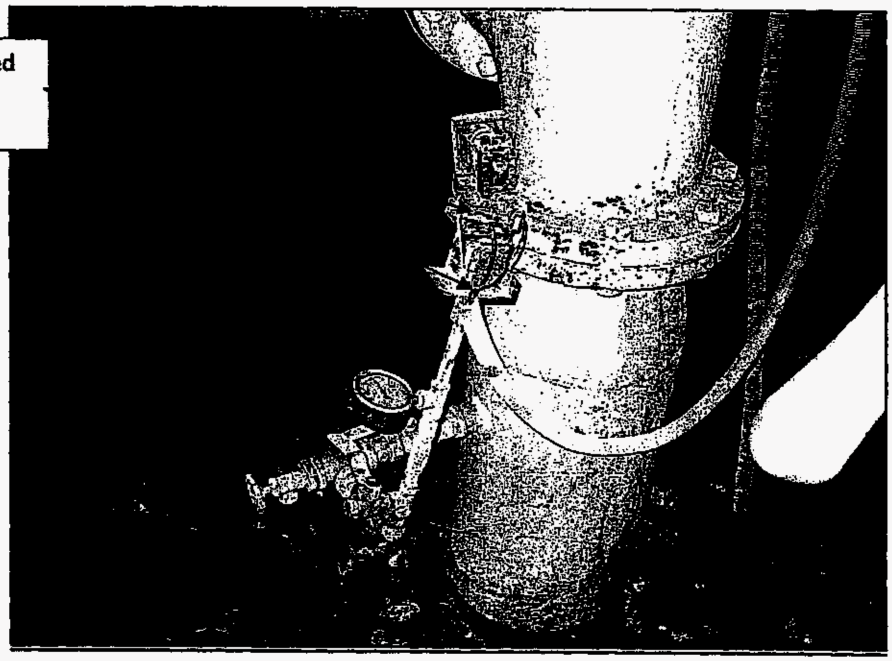


Figure 4.7-Elevated tank pressure switch

.dential NM wire being used
exterior circuit; not listed for
this use and not protected

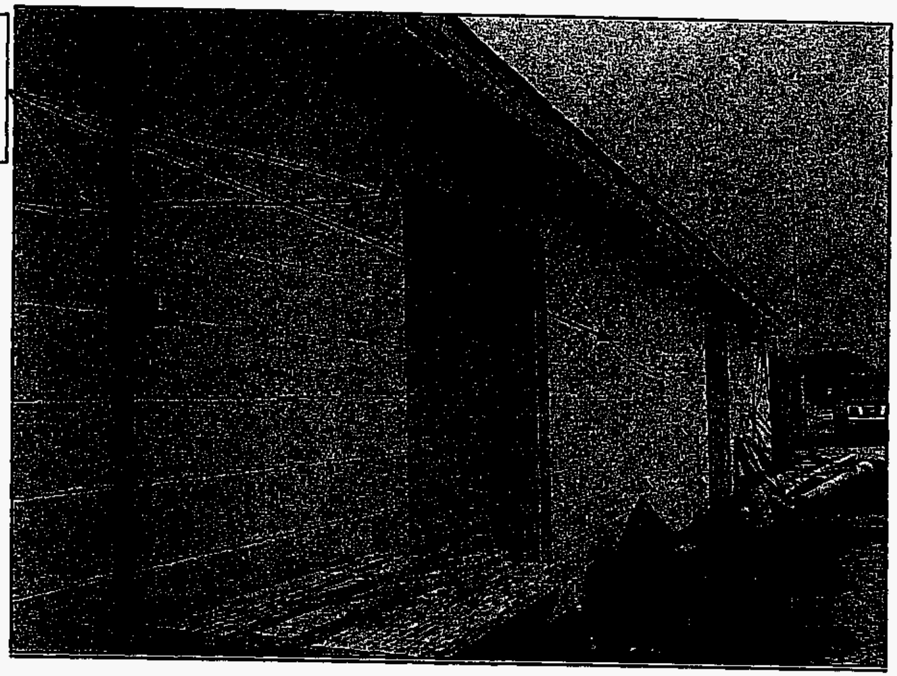


Figure 4.8-Northside of WTF

Exposed, abandoned wire;
float switches do not have
proper supports

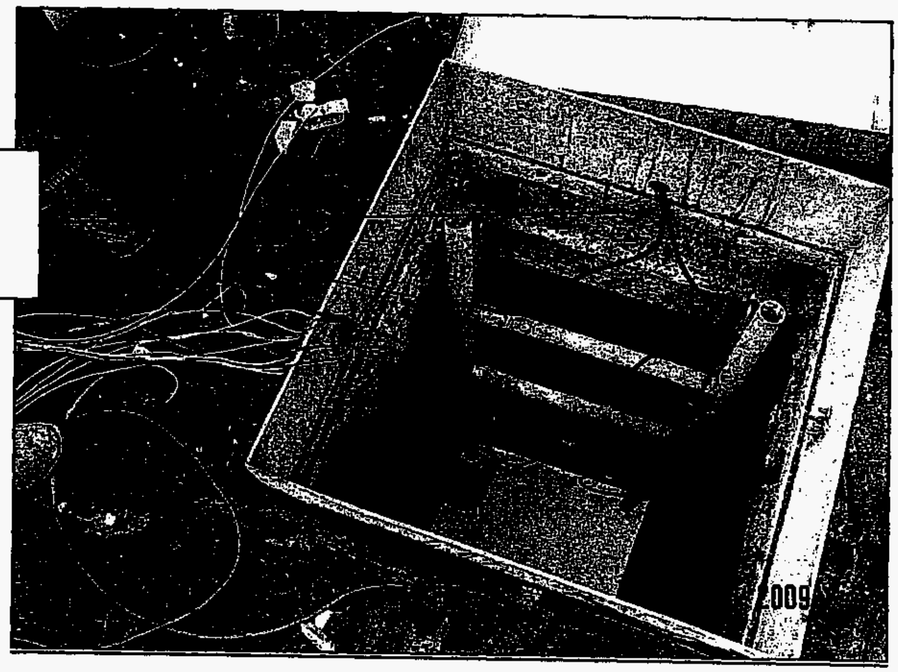


Figure 4.9-GST access batch

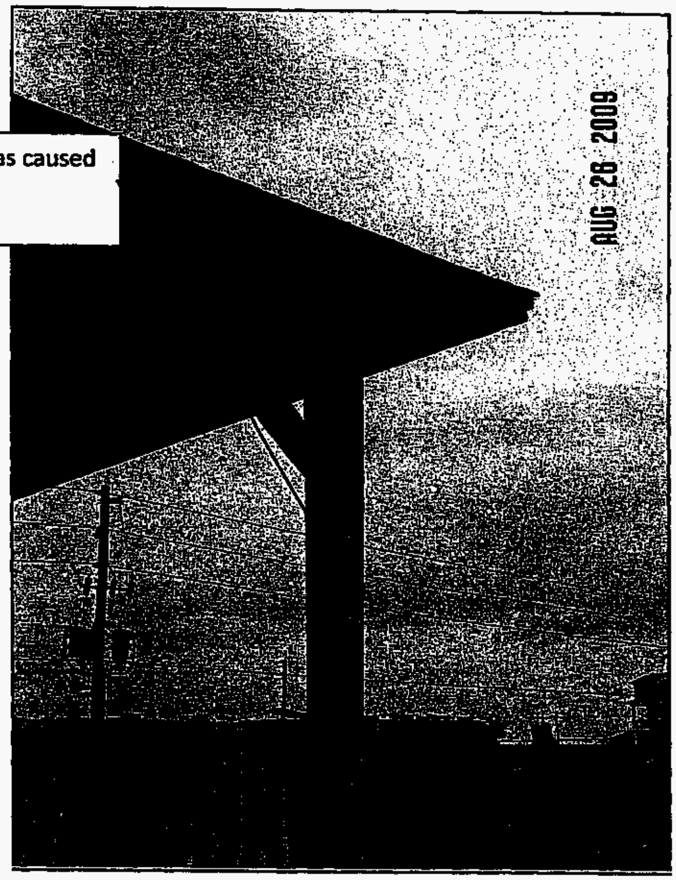


Figure 4.10-Northwest corner WTF

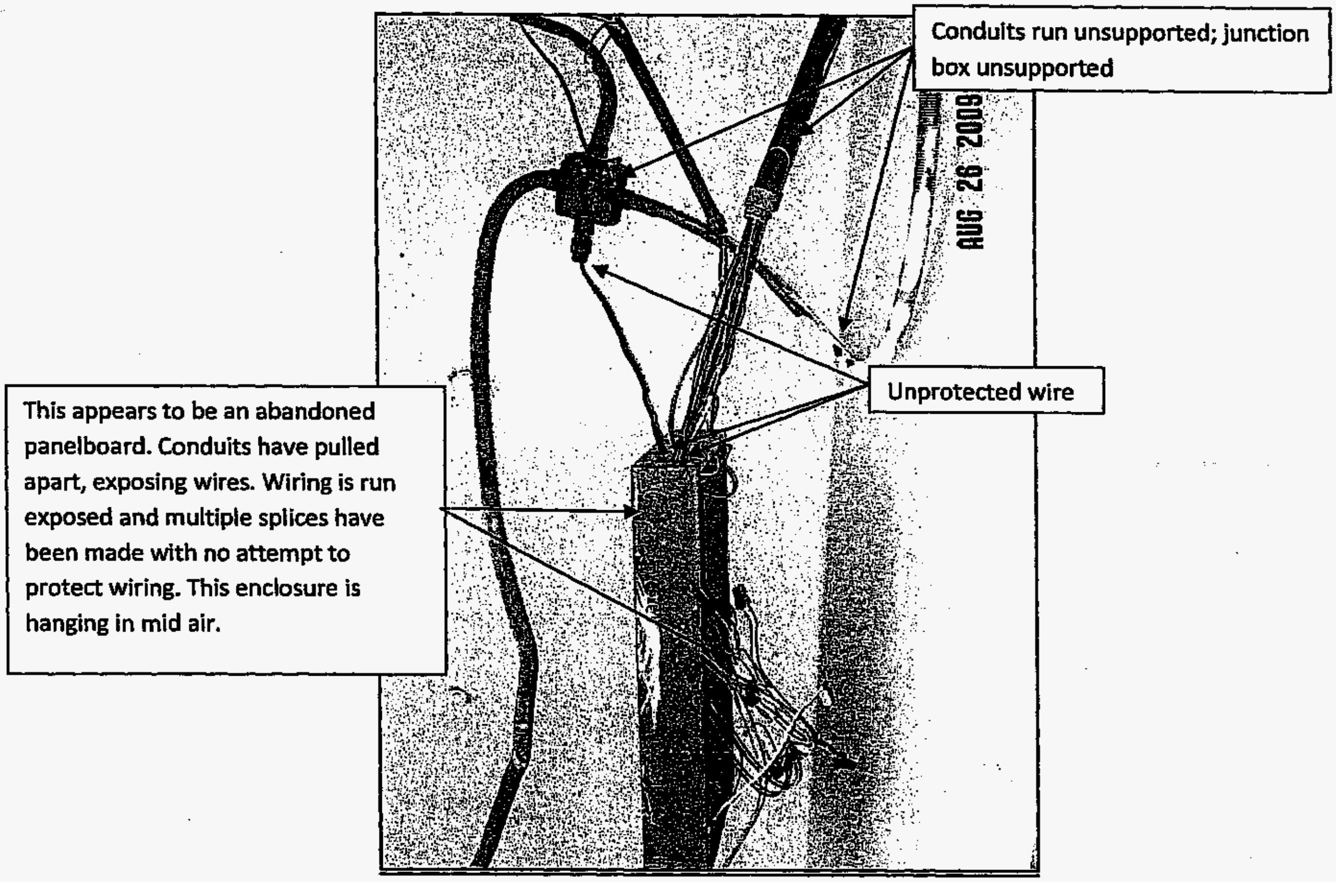


Figure 4.11-WTF Pump Room

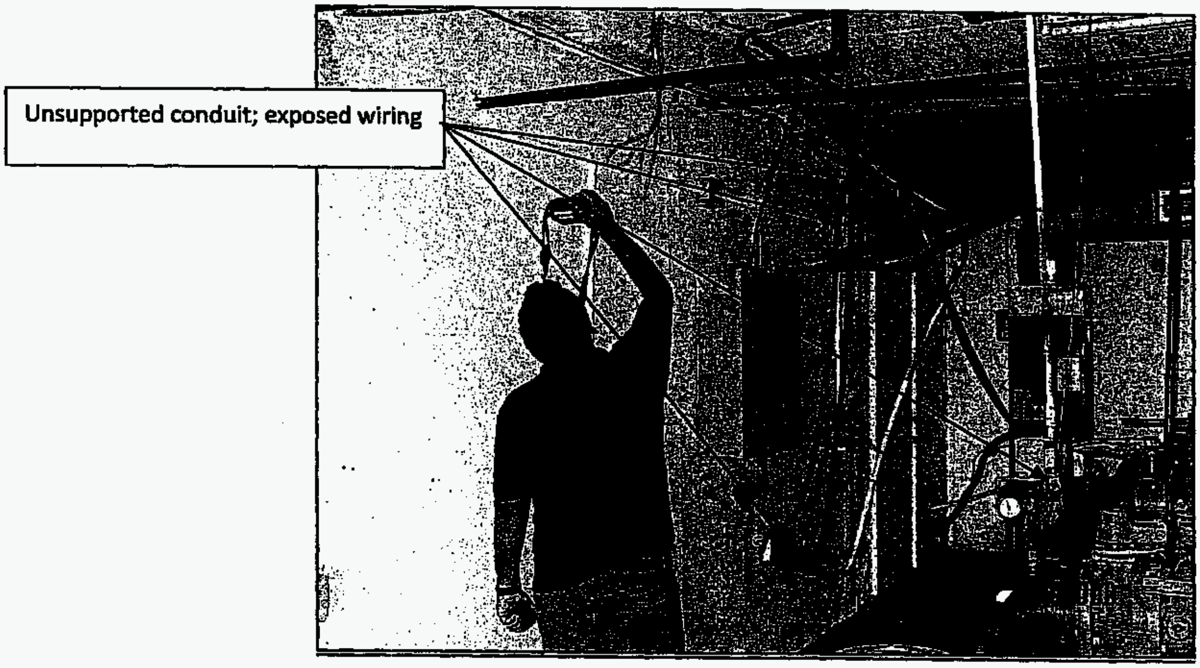


Figure 4.12-WTF Pump Room west wall

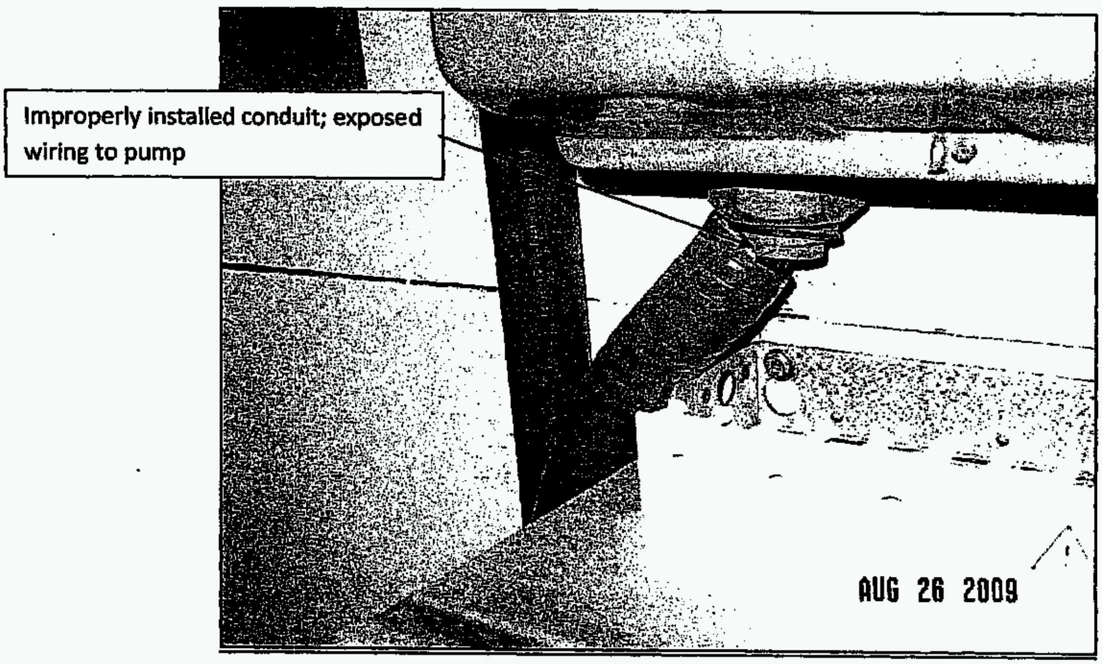


Figure 4.13-WTF Electrical Room high service pump starter

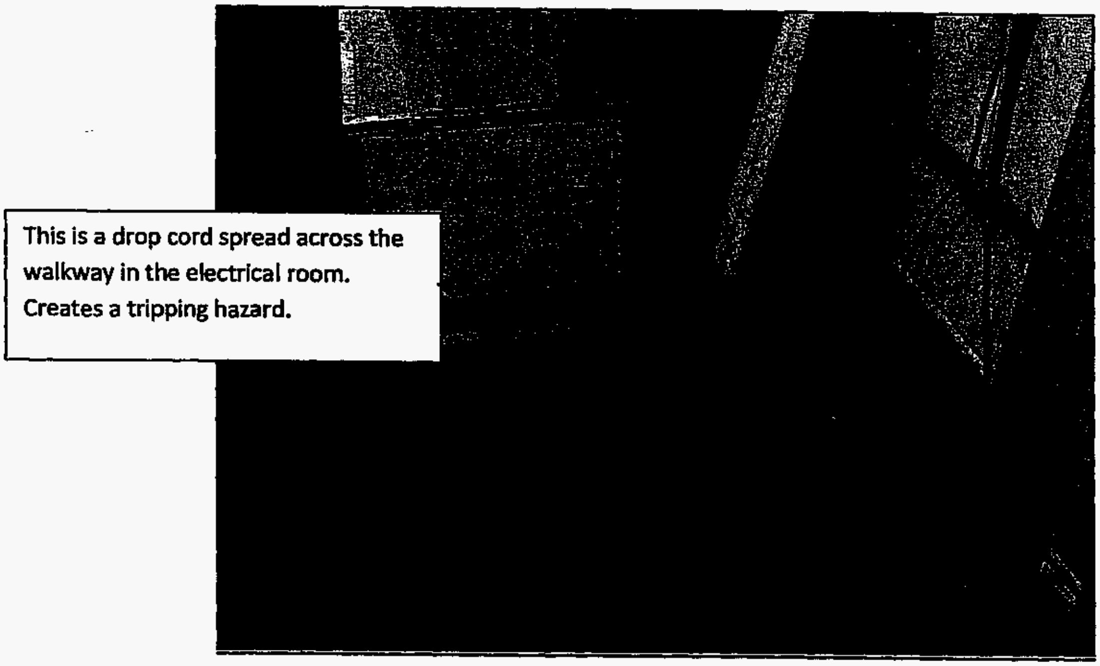


Figure 4.14-Electrical Room

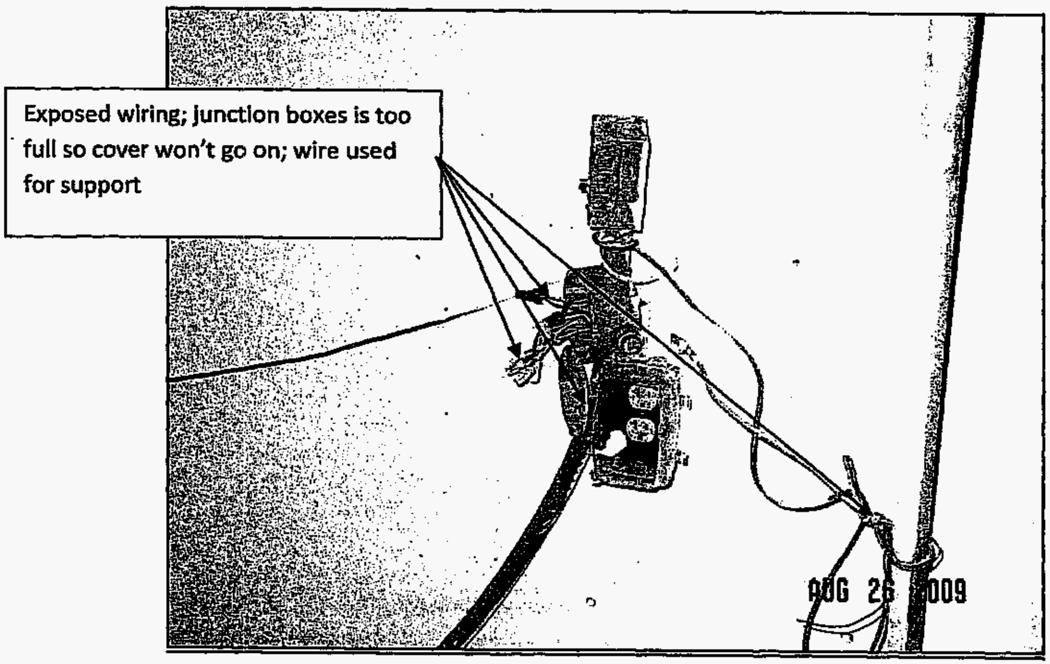


Figure 4.15-High service pump room

Circuits are not all labeled. There should be a circuit card in the pocket with all circuit numbers and labels.

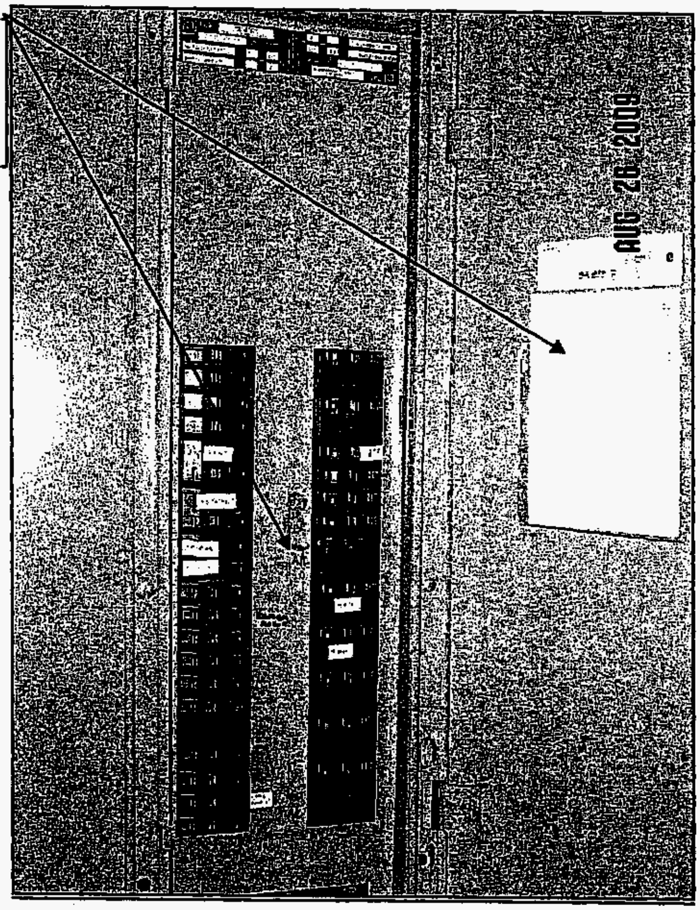


Figure 4.16-Electrical room panelboard

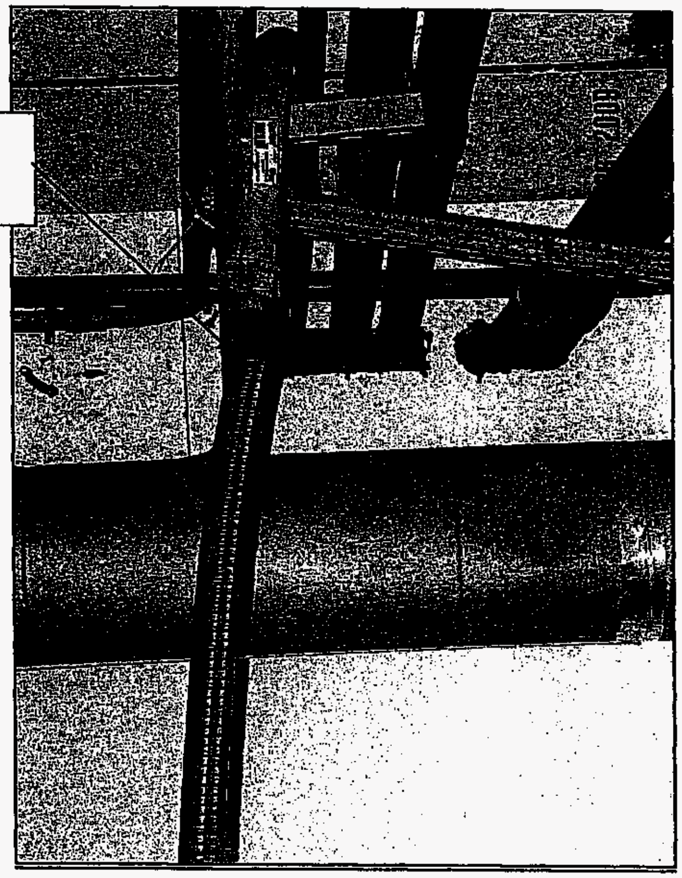
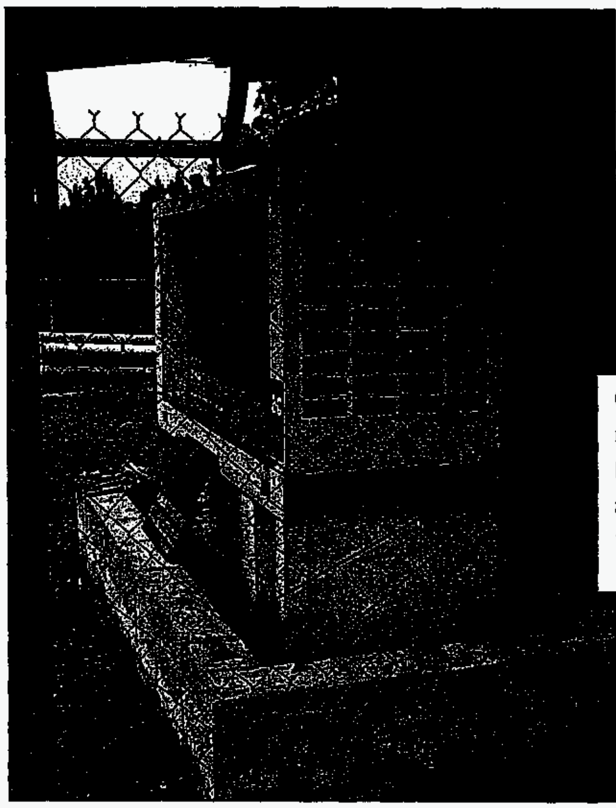


Figure 4.17-High service pump room

4.2 Well No. 3

Electrical components at Well Site 3 show signs of wear and there are instances of improper installation. The photos below show some observations of issues, most of which are minor:

Figure 4.18-Well No.3



Generator containment has caused major corrosion to the diesel tank. Fuel containment structure does not appear to be sufficient size to contain 115% fuel capacity of diesel tank.

Figure 4.19-Generator

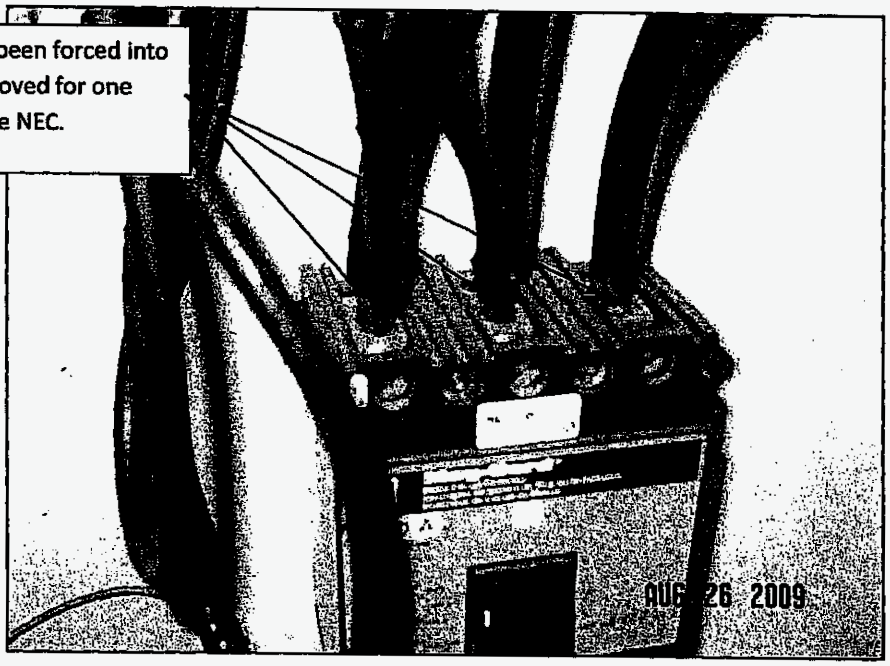


Figure 4.20-Pump Control Panel

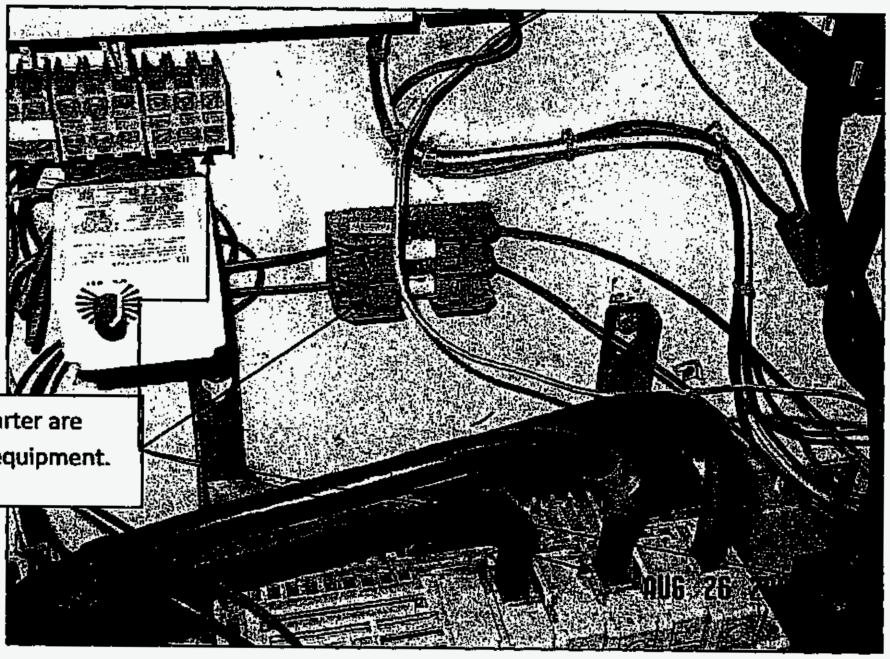


Figure 4.21-Pump Motor Starter

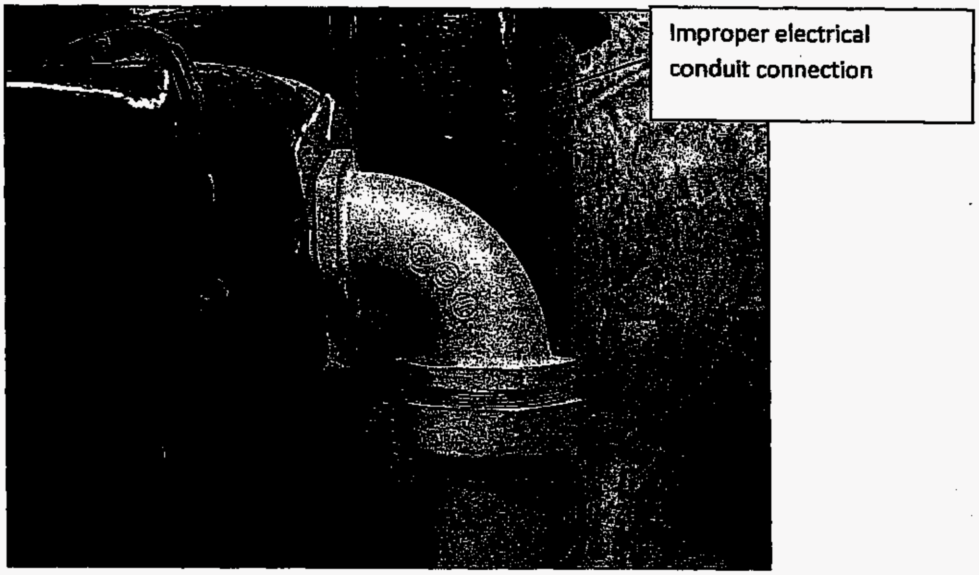


Figure 4.22-Pump Connection box

4.3 Well No. 1

Well No. 1's electrical equipment shows signs of significant deterioration and wear due to age.

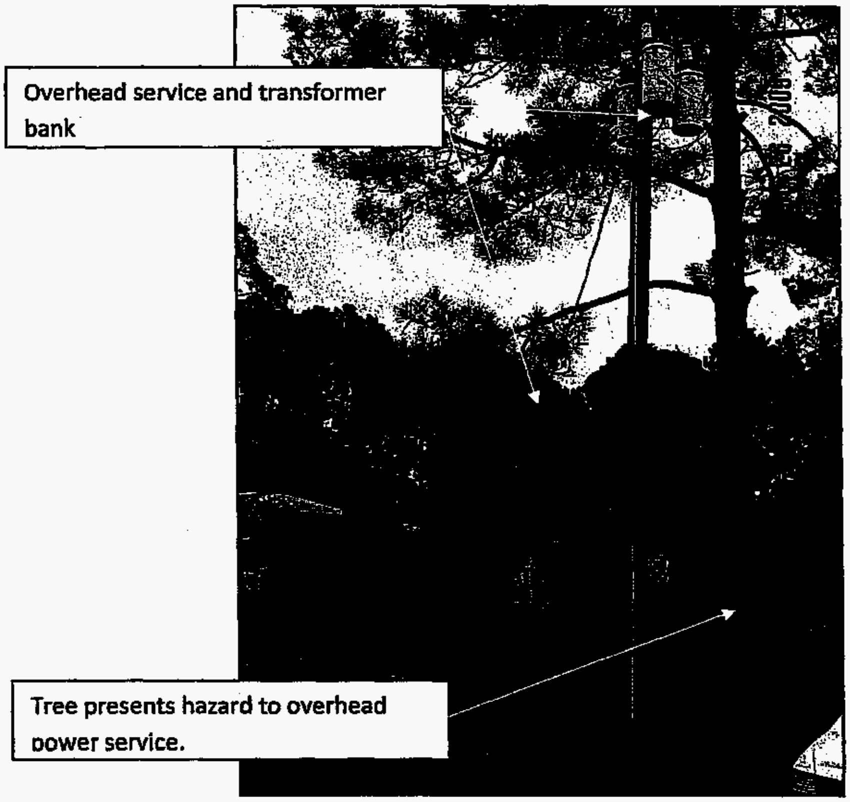


Figure 4.23-Well No.1

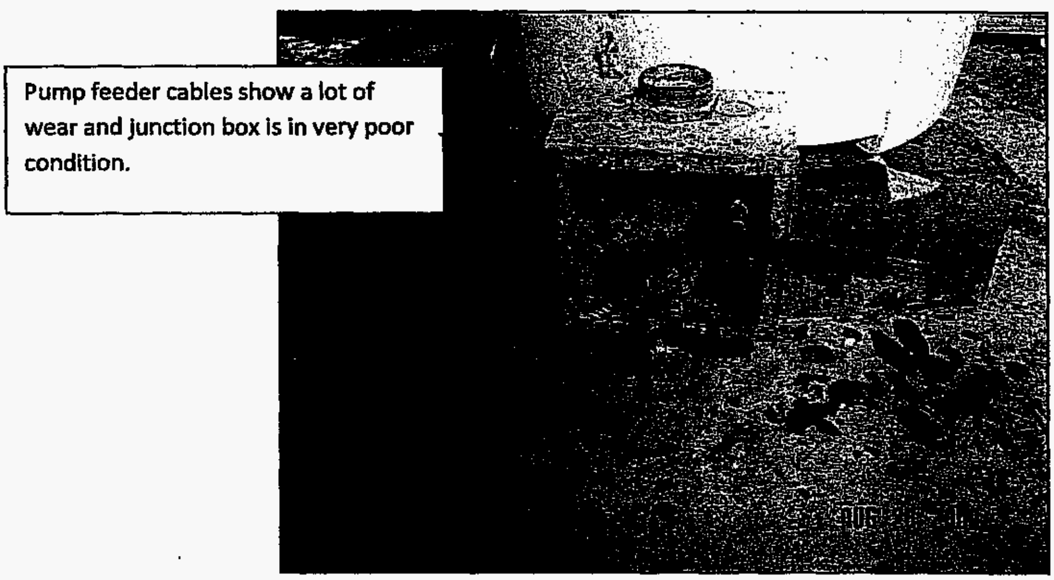


Figure 4.24-Power cable junction box at well head

4.4 Well No. 2

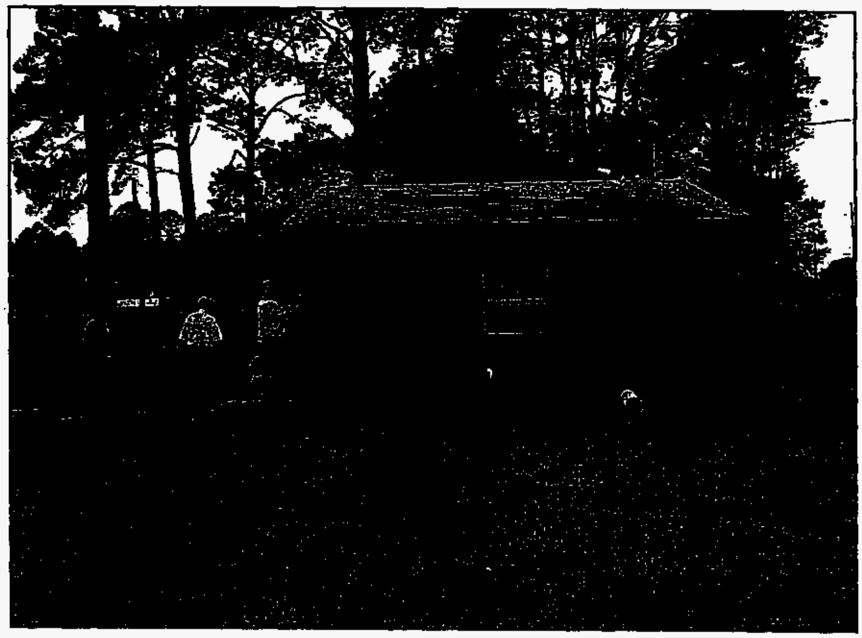


Figure 4.25-Well No.2

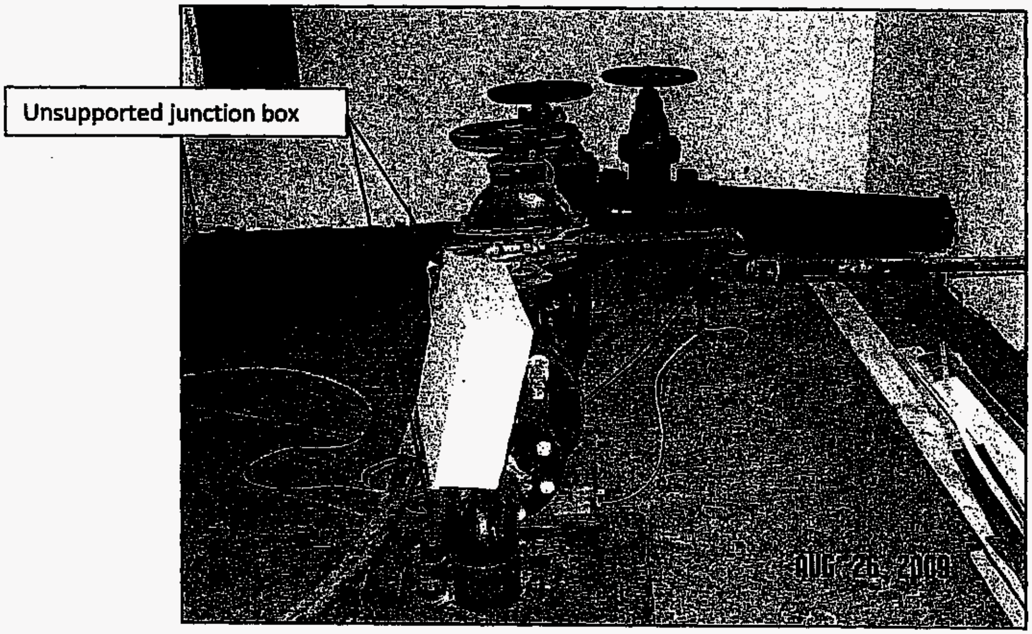


Figure 4.26-Well head & junction box

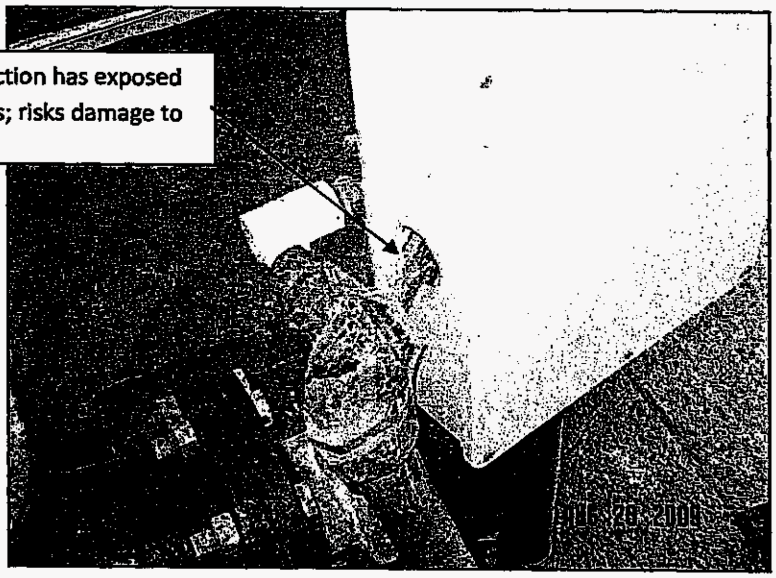


Figure 4.27-Well head junction box

Figure 4.28-Well pump motor starter enclosure

Figure 4.29-Well house interior

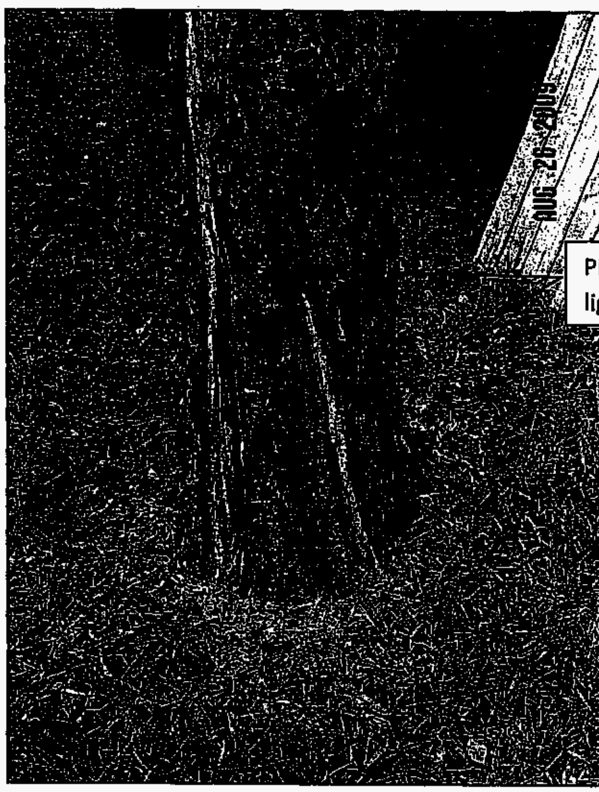


Figure 4.30-Pump house lighting rod

4.5 Well No. 4

Figure 4.31-Well No.4

Open junction box, exposed wire, improper flex couplings

Figure 4.32-Well house interior

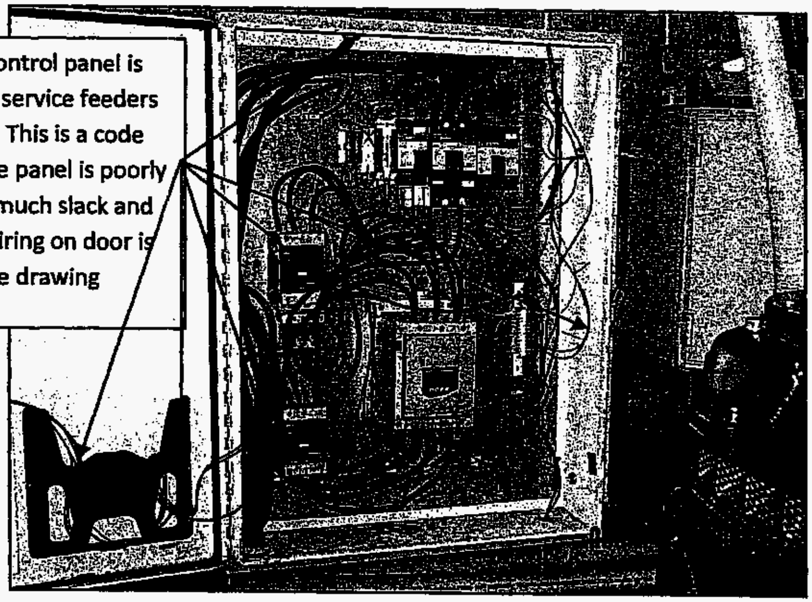


Figure 4.33-Well pump motor starter

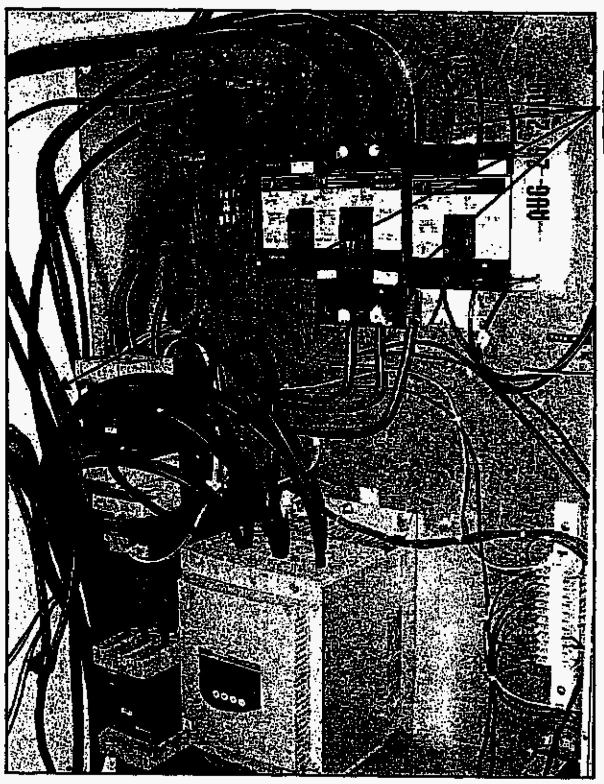


Figure 4.34-Well pump motor starter

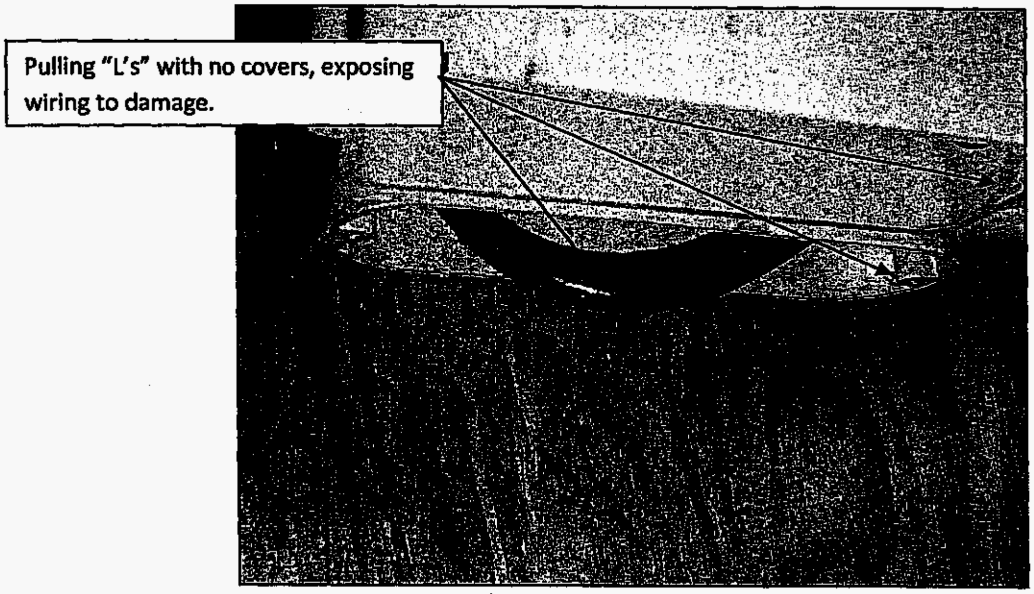


Figure 4.35-Conduits under ATS

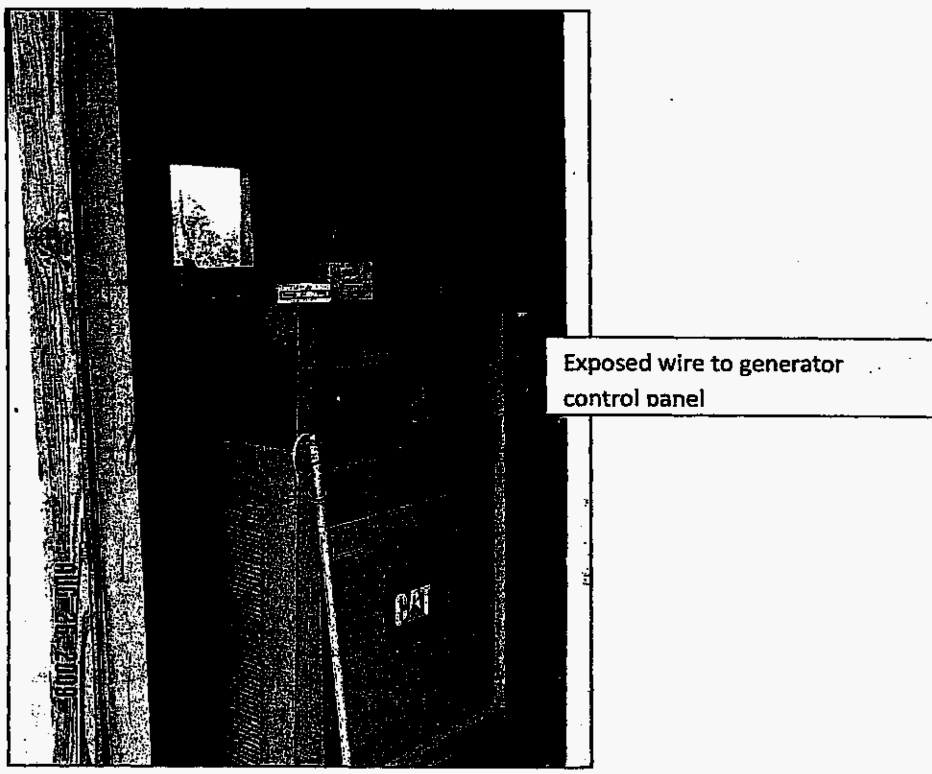


Figure 4.36-Standby generator

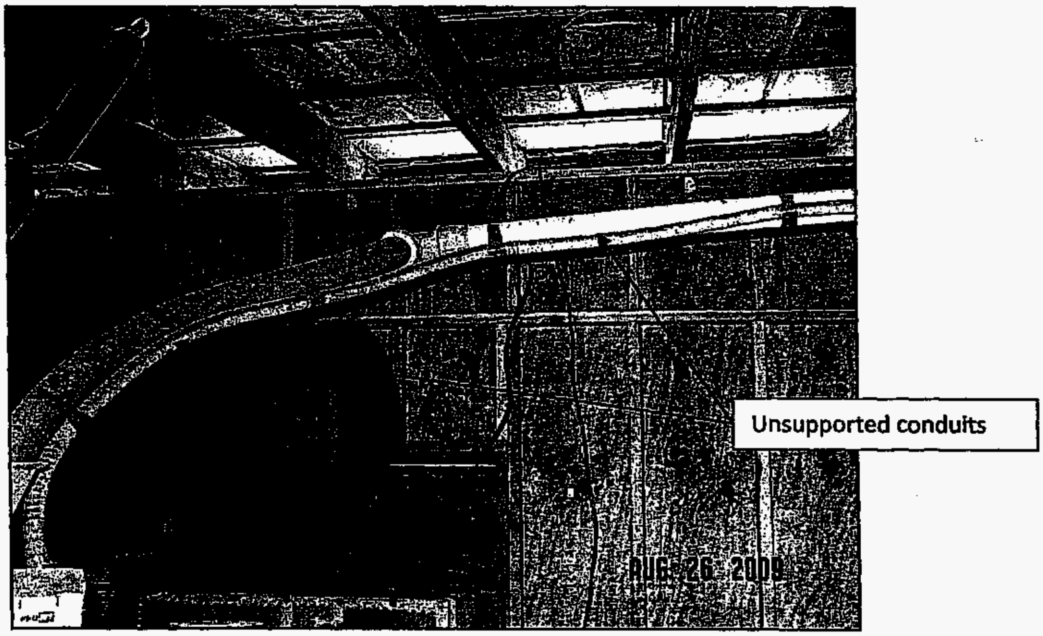


Figure 4.37-Generator power conduit feeds

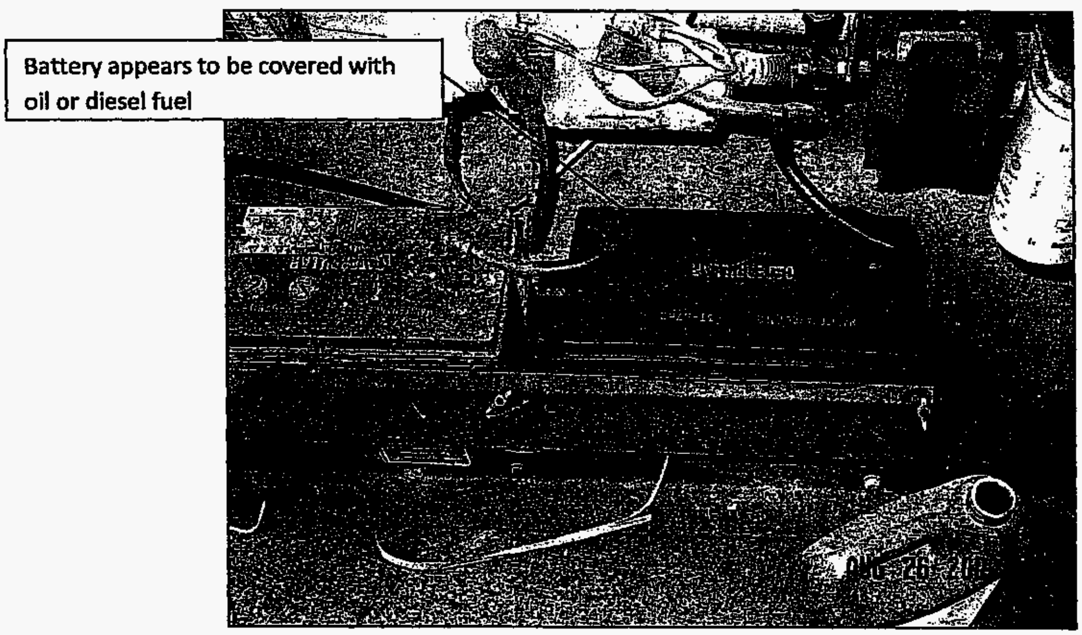


Figure 4.38-Generator batteries

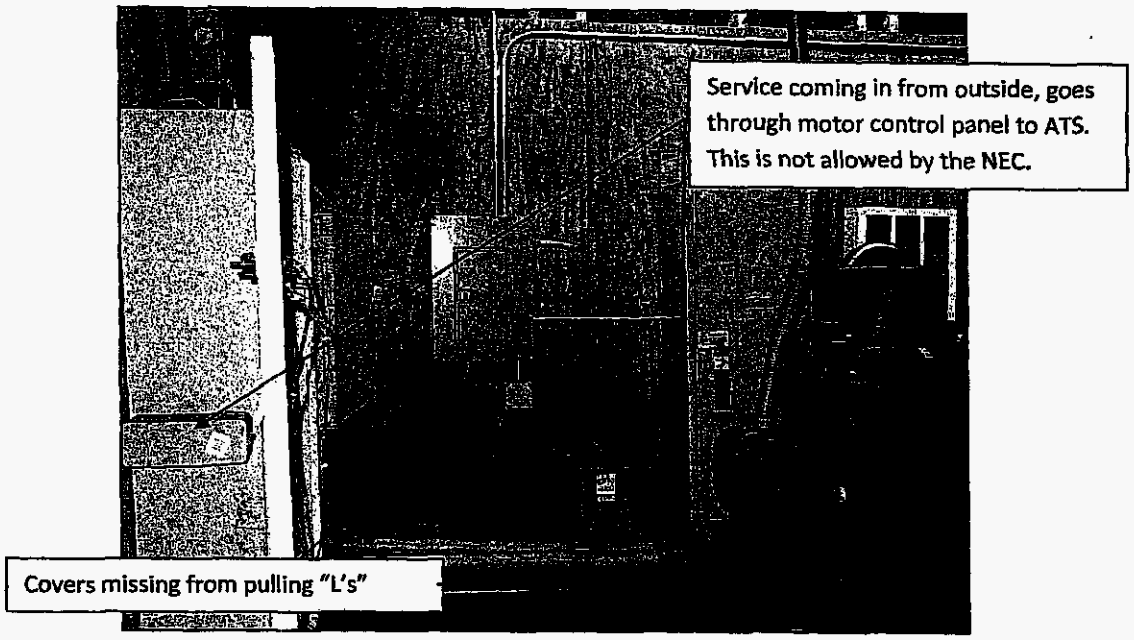


Figure 4.39-Well No. 4 electrical equipment

5.0 Condition Assessment

In general the condition of electrical, instrumentation and controls at all Water Management Service facilities is extremely poor and causes concern for the safety of operations staff and any person that must come in close contact with these systems. Also, there is certainly concern regarding the systems reliability to continue operating correctly without critical failure of these systems, which could result in the loss of water production in the area. In regards to instrumentation and controls, the control panels at the wells are in fair condition; however, the Master PLC control panel at the plant is in very poor condition. The system and its serviceability is of major concern. Operators note that they cannot make any changes to pump operations and the controls are not easily serviced. Industrial Control panels should be designed and constructed by a UL 508 recognized panel shop. UL 508 is the standard for industrial control equipment.

5.1 The National Electrical Code (NEC)

Electrical systems are required at a minimum to be installed in accordance with the NEC. "The purpose of the NEC is to provide practical safeguarding of persons and property from hazards arising from the use of electricity." (NFPA70, 2008) There are certainly many code violations easily identified in all facilities. Codes violations in some instances may seem minor; however, if left uncorrected can place the Owner in a vulnerable position in regards to liability. Code violations should be seriously considered and corrected in short order. Electrical Contractors working on facilities should be licensed electrical contractors. Licensing requires them to provide installations that meet and should many times exceed the requirements of the NEC. The following NEC sections relate to our review and photos presented in this TM:

- 110.12 Mechanical Execution of Work. Electrical equipment shall be installed in a neat and workmanlike manner.
- 110.13 Mounting and Cooling of Equipment. (A) Mounting. Electrical equipment shall be firmly secured to the surface on which it is mounted.
- 110.26 Spaces About Electrical Equipment. (F) Dedicated Equipment Space. All switchboards, panelboards, distribution boards, and motor control centers shall be located in dedicated spaces and protected from damage.
- 300.4 Protection Against Physical Damage. Where subject to physical damage, conductors shall be protected.
- 300.6 Protection Against Corrosion and Deterioration. Raceways, cable trays, cablebus, auxiliary gutters, cable armor, boxes, cable sheathing, cabinets, elbows, couplings, fittings, supports, and support hardware shall be of materials suitable for the environment in which they are to be installed.
- 300.11 Securing and Supporting. (A) Secured in Place. Raceways, cable assemblies, boxes, cabinets, and fittings shall be securely fastened in place.
- 352.30 Securing and Supporting. (A) Securely Fastened. PVC conduit shall be securely fastened within 900 mm (3 ft) of each outlet box, junction box…etc.
- 352.30 Supports. (B) PVC conduit shall be supported as required in Table 352.30.

5.2 Assessment Tables

The following assessment tables were developed to help quickly review the assessment of electrical equipment and devices at these facilities:

TABLE 5.1 WATER TREATMENT FACILITY-ELECTRICAL		
Item or Description	**Condition**	**Comments**
Electrical Service	Fair	Most of the electrical service is concealed; not visible
Automatic transfer switch	Fair	
480 volt panelboard	Good	
High Service Pump Motor Starters	Poor	Motor starters are not properly installed; enclosures are needed
50kVA transformer	Fair	
240/120 volt panelboard	Fair	
Master PLC Control Panel	Poor	Panel is not serviceable and is hazardous because of exposed electrical devices. This problem needs corrective action immediately.

Instruments	Poor	Instruments are of poor quality and improperly installed. This problem needs corrective action immediately.
General conduit installation	Poor	Conduit installations do not meet NEC requirements
General wiring installation	Poor	Wiring methods do not meet NEC requirements
Generator	n/a	The generator was not inspected during the site investigation.

TABLE 5.2
Well No. 3 - ELECTRICAL

Item or Description	Condition	Comments
Electrical Service	Fair	
Automatic transfer switch	Old	
Well pump motor starter	Poor	Control panel construction is very poor; Code violations exist. Replace immediately.
Step down transformer	Fair	
240/120 volt panelboard	Old	
PLC Control Panel	Fair	
General conduit installation	Poor	Conduit installations do not meet NEC requirements. Repair required.
General wiring installation	Poor	Wiring methods do not meet NEC requirements. Repair required.
Generator	Old	The generator has not been maintained; shows signs of severe corrosion. Replace/refurbish

TABLE 5.3
Well No. 1 - ELECTRICAL

Item or Description	Condition	Comments
Electrical Service	Fair	
Well pump motor starter	n/a	Not inspected
Step down transformer	Fair	
240/120 volt panelboard	Fair	
PLC Control Panel	Fair	
General conduit installation	Fair	
General wiring installation	Poor	Wiring methods do not meet NEC requirements. Correct deficiencies.

TABLE 5.4 Well No. 2 ELECTRICAL		
Item or Description	Condition	Comments
Electrical Service	Old	Update needed.
Well pump motor starter	Poor	Control panel is very poorly constructed and in poor condition; motor circuit protection is needed. Replacement needed.
Step down transformer	Fair	
240/120 volt panelboard	Old	Update needed.
PLC Control Panel	Fair	
General conduit installation	Poor	Conduit installations do not meet NEC requirements. Correct immediately.
General wiring installation	Poor	Wiring methods do not meet NEC requirements. Correct immediately.

TABLE 5.5 Well No. 4 ELECTRICAL		
Item or Description	Condition	Comments
Electrical Service	Poor	Service routing through pump control panel is not acceptable
Automatic transfer switch	Fair	
Well pump motor starter	Poor	Control panel construction is very poor; Code violations exist. Correct immediately.
Step down transformer	Fair	
240/120 volt panelboard	Fair	
PLC Control Panel	Fair	
General conduit installation	Poor	Conduit installations do not meet NEC requirements. Correct immediately.
General wiring installation	Poor	Wiring methods do not meet NEC requirements. Correct immediately.
Generator	Poor	The generator shows signs of severe corrosion. Replace or refurbish.

6.0 Recommended Improvements

The following is a brief and general discussion of recommended improvements for these facilities. As was stated earlier, it is impossible to see all issues from a quick visual tour, which was conducted here. It is clear, however, that all facilities are in need of updated electrical equipment and proper installation of electrical equipment, instruments and controls to provide for safe and code compliant facilities. **Table 6** includes the priority electrical CIP projects items.

6.1 Water Treatment Facility

Generally, the electrical equipment being used at the WTF is in fair condition. The problem with the existing system is that it is poorly or incorrectly installed and maintained. The installation of conduit and wire is extremely poor and code violations are visible everywhere around the plant. Wiring of any type is used in any location. There are many methods of installation that do not meet the NEC that create unsafe working environments and conditions. Therefore, our recommendation would be to completely remove all existing conduit and wiring and replace it with properly sized and properly installed conduit/wire systems. This includes the removal of old, abandoned wire and conduit systems seen all around the plant. It also would include control and instrumentation wiring systems. As was mentioned earlier, the entire system is susceptible to flood and/or wave action as it is installed at the current elevation. Therefore, it is our recommendation to consider replacing the entire system and designing a new system above the storm surge elevation.

The control and instrumentation systems at the WTF are in very poor condition. The Master PLC control panel is old, outdated and unserviceable. Code violations also existing in regards to the safety of the enclosure and exposed wiring. Our recommendation would to remove this system completely and replace it with a current system design to meet the NEC and UL for industrial control panels. Instruments around the plant are poorly or completely installed incorrectly. All instruments should be replaced with new up to date instruments that will work in coordinate with the new plant control panel. These new systems should also be designed to be above storm surge and instruments should be closely reviewed and specified for their location and elevation in regards to surge levels.

6.2 Wells No. 1 – 4

All the well houses contain various electrical equipment and the condition of this equipment and its installation varies; however, all these systems have issues and are showing signs of age and wear due to the conditions the systems are installed in. Therefore, we recommend that all new electrical equipment and electrical systems be installed at each of the well sites. There is little equipment that would be worth saving at these facilities. However, we have been given information that new VFDs have been purchased for each of the well sites. It is possible that these drives could be reused. Totally new electrical systems would give the Owner the advantage of the fact that all these facilities would be built and house the same equipment. It would also be our recommendation that the pump telemetry/PLC panels be replaced by a more up to date system, which would be much more reliable. Furthermore, our recommendation would include standby power at each well site.

6.3 Instrumentation and Controls Systems

During our investigation operations staff at the plant made it very clear that there are a multitude of problems with the existing Master PLC control panel, Remote PLC control panels and the communications systems that are currently being used. This system is old and is not serviceable. Currently in the water/wastewater industry radio supervisory control and data acquisition systems (SCADA) are widely used and accepted. These systems are time tested and have been successfully used for decades throughout the industry. Our recommendation would be that a radio path study is conducted and a SCADA system is designed and installed to control operations at the WTF and to control and monitor all well sites.

This system would typically include the following:

- Master Remote Terminal Unit (RTU) at the WTF – This RTU would contain a PLC to control and monitor plant operations and to communicate with each of the well site's remote RTUs.
- Master radio, antenna and tower at the WTF
- Operator Interface Terminal (OIT) with human machine interface software
- Remote RTU at each well site – These RTUs would contain a PLC to control and monitor well operations, alarms and to communicate with the Master RTU at the WTF.
- RTU radio, antenna and tower

This system could monitor and control all points of interest at the WTF to record data and develop reports for the operations staff. The OIT would provide the operators with the ability to remotely monitor the wells and change control parameters. This type of system can also be remotely monitored from a portable laptop computer. This would especially be useful since the WTF is not manned 24/7. Additional software can be used that would dial cell phones to report alarms to operators also. These systems are very versatile and many options are readily available. This system would not be proprietary. Standard industrial PLCs, software and control panel components should be used. Documentation of the system and its components is also critical for maintenance purposes. A system like this requires detailed plans and specifications to be written by a professional engineer. These documents can then be used to bid the project out to properly qualified contractors.

Table 6 Replace/Rehabilitate Electrical Equipment Priorities		
Priority	**Description**	**Estimated Cost**
Well No. 1, 2, 3 & 4	New SCADA/RTU controls	$252,000
Well No. 3	Generator Repairs	$35,700
Well No. 4	New Generator	$64,000
	Total	$337,700*

*Includes engineering costs

END

PBSJ

Technical Memorandum 7
Water Distribution System Evaluation

To: Mr. Gene Brown, WMSI

From: David Gauker

Project: Water Management Services, Inc

CC: Mike Scibelli

Date: December 18, 2009
Revised June, 2010

Job Number:100010111

1.0 Purpose

PBS&J reviewed information, records and interviewed staff to evaluate the overall condition, operation and maintenance of the water distribution system. The purpose of this Technical Memorandum (TM) is to review the existing distribution system, outline our findings and prepare recommendations along with associated costs. The review of distribution operation includes:

- Review of Pressure Monitoring
- Flow Recording and Monitoring
- Disinfection Residual Maintenance
- Disinfectant By-Products levels
- Lead and Copper levels
- Flushing Program
- Backflow Prevention
- Water Loss
- Valve Maintenance
- Hydrant Maintenance Program
- External Corrosion Monitoring

2.0 Background

The St. George Island distribution system consists of approximately 59 miles of pipe consisting of sizes from 1 to 12-inch in diameter. At one end of the island is a state park and at the opposite end are expensive estate properties. For the purposes of our evaluation, we have utilized the flow records included in the Annual Report of Water Management Services, submitted to the State of Florida, for the year ending December 31, 2009. Information in **Table 1** presents water supply and distribution water balance.

Table 1 St. George Island Water Balance 2009			
Water pumped from wells	Water utilized for line flushing and fire fighting	Water pumped from the water treatment facility	Water sales to customers
189.900 M Gallon	23.514 M Gallon	*166.386 M Gallon	151.136 M Gallon
* Does not include recycled flows to/ from the elevated storage tank.			

The distribution system consists of the following components:

- High Service Pumps and Flow Meters
- Elevated Storage Tank
- Distribution Piping and Valves
- Service Meters
- Fire Hydrants

Distribution system operation

Water is stored in the ground storage tank prior to entering the distribution system. The distribution system operates under pressure provided by variable speed high service pumps. Flow from the high service pumps enters the distribution-piping network with a controlled volume of water diverted to the elevated tank located near the water plant. An electric actuated valve positioned on the elevated tank fill line, serves to throttle the volume of water entering the elevated storage tank. The elevated tank level is not controlled automatically. The position of the electric inlet valve is manually set by the operations staff to control the fill rate. The elevated tank is no longer an effective means of providing sufficient hydraulic grade to pressure to the system ; as both growth at opposite ends of the St. George Island and permitted vertical building height exceed the capabilities of the tank. The elevated tank's primary purpose is to provide additional water storage volume for fire fighting and a back-up for tank cleaning, etc. A six-inch drain valve is adjusted allowing water to drain from the elevated tank and return to the ground storage tank for subsequent recycling on a continual basis. Approximately 30% of the elevated tank volume is recycled with all recycled flow metered as it returns to the ground storage tank.

Typically, a tank that is continually "topped off" will have stagnant water in it, especially in summer and this may cause taste, odor or bacteriological issues. If possible, the entire tank contents should be re-circulated once every 24-hours.

Chlorine residual and water quality is maintained by flushing the distribution system through hydrants and dead-end flushing. The water system plant operators monitor finished water pH and chlorine residual in the distribution system. Other distribution laboratory analysis is performed by an outside laboratory with samples provided by WMSI staff.

Internal corrosion control using chemicals to reduce the potential for scale formation and to control water hardness, color, taste, and odor in the distribution system does not exist at this time.

3.0 Review of System Operation and Management

The following summarizes PBS&J's review of system operation and management.

System Pressure Monitoring

WMSI monitors the distribution system pressure from the water plant to maintain adequate operating pressure in the system above 20 psi. The system is monitored using a pressure sensor at the high service pump station. To maintain adequate pressure in the distribution system requires approximately 80 to 85 psi pressure at the high service pump discharge. Combination of high service pumps operating at various speeds operate to maintain the system pressure. If the number of pumps required to maintain pressure continues to increase, and pressure cannot be maintained at the high service pump station, this alerts the plant operating staff of a distribution system line break. This method of monitoring is typically and appears to be working satisfactorily.

Flow Recording and Monitoring

Flow meters are located at the discharge of each high service pump. The location of the meters relative to the high service pump discharge is not in conformance with manufacturer's recommendations thus inaccuracies with the total flow pumped may occur. Each flow meter is fitted with a digital indicator located in the control room. Flows are totalized daily from each meter to determine the total flow pumped. PBS&J recommends installing a digital flow chart recorder. The chart recorder will provide a continuous display of the pumped flows to the distribution system and the recordings provide historically data. The chart recorder would take the digital signals from each flow meter, totalize the flow, and simultaneously display the totalized flow.

Recommended capital improvements:

1. Priority CIP item: Purchase and installation of high service flow chart recorder.

Elevated Storage Tank

The elevated storage tank is a 150,000-gallon capacity, pedestal configuration. The pedestal tank appears to be in good condition. Currently, WMSI has a maintenance program for periodic cleaning, inspection and refurbishment of the elevated tank. Cleaning with internal inspection is recommended at least every five years. On an annual basis, visual inspection should be performed to assess and repair environmental damage and verify the integrity of vents and screens and an assessment of the painting and coatings of the tank.

Recommended capital improvements: None, at the time of this report.

Disinfectant Residual Maintenance

Currently, disinfectant residual maintenance consists of adding chlorine to the water prior to pumping into the distribution system. The amount of chlorine added is based upon flow and desired chlorine dose using a compound loop control method. This is very common and satisfactory method of pacing the proper amount of disinfectant. Water samples taken at the plant and in the distribution system are used to determine the chlorine residual with the overall objective of maintaining a detectable disinfectant residual in the distribution system at all times. A working chlorine residual analyzer and chart recorder is needed to supplement the current configuration of chlorination equipment.

The chlorine residual at opposite ends of the island and at dead ends in the distribution system is difficult to maintain. The State park has very low flow demands, and to achieve a residual, flushing the distribution system is required on a regular basis to bring freshly chlorinated water to the park. This flushing procedure, when employed, results in providing the required result but at a cost of wasting finished water, as well as electricity from high service pumps and chlorine. It is very likely the investment and maintenance of a booster disinfection system would reduce flushing requirements by rechlorinating near the end of the system. The incorporation of 2-inch diameter continuous blow-offs at dead ends may help to restore water quality and help restore disinfectant levels, but this practice may result in using large qualities of water. A more effective means is looping dead-ends to improve circulation in the water distribution system and reduce flushing.

Recommended capital improvements:

1. Priority CIP item: Purchase and installation of a chlorine analyzer and chlorine chart recorder.

2. Suggestions on rechlorination systems and automatic flushers are presented later in this document.

Disinfectant By-Products Monitoring

FDEP mandates utilities monitor the presence of disinfection by-products at various locations in the distribution system.

- **Types of disinfection byproducts**

  When chlorine is used as a disinfectant, hundreds of disinfection byproducts may form. The composition of the water determines which types of disinfection byproducts will form. The Total Organic Carbon (TOC) content indicates the level of disinfection predecessors and the concentration of disinfection byproducts that will eventually be formed. Disinfection byproducts may be volatile and hydrophobic. There are also non-volatile, hydrophilic disinfection byproducts, which include chlorinated and non-chlorinated aromatic and aliphatic substances. The disinfection by-products of primary concern for WMS will be trihalomethanes and halogenic acids. Refer to **Table 2** for a summary of disinfectant by-products.

Trihalomethanes (CHX_3) substances are formed during chlorine disinfection and disinfection by chlorinated disinfectants. Trihalomethanes can be divided up into trichloromethane (chloroform, $CHCl_3$). Trihalomethanes are suspected to damage the liver, kidneys and central nervous system. They are considered carcinogenic.

Halogenic acetic acids (HAA) are an important type of chlorinated disinfection byproducts. HAA are non-volatile compounds. HAA can, occasionally, be found in the water in higher concentrations than trihalomethanes (THM). This is determined by the pH value of the water. When the pH value is lower, more HAA are formed and when the pH value is higher, more THM are formed. The composition of naturally present organic matter (NOM) also determines the amount of THM or HAA that is formed.

Table 2

Summary of Disinfectant By-products (DBP's)

Regulated Contaminants/Disinfectants					
Regulated Contaminants	**MCL (mg/L)**	**MCLG (mg/L)**	**Regulated Disinfectants**	**MRDL* (mg/L)**	**MRDLG* (mg/L)**
Total Trihalomethanes (TTHM)	0.080		Chlorine	4.0 as Cl_2	4
Chloroform Bromodichloromethane Dibromochloromethane Bromoform		- zero 0.06 zero			
Five Haloacetic Acids (HAA5)	0.060		Chloramines	4.0 as Cl_2	4
Monochloroacetic acid Dichloroacetic acid Trichloroacetic acid Bromoacetic acid Dibromoacetic acid		- zero 0.3 - -	Chlorine dioxide	0.8	0.8
Bromate (plants that use ozone)	0.010	zero	*Stage 1 DBPR includes maximum residual disinfectant levels (MRDLs) and maximum residual disinfectant level goals (MRDLGs) which are similar to MCLs and MCLGs, but for disinfectants.		
Chlorite (plants that use chlorine dioxide)	1.0	0.8			

Halogenic acetic acids are suspected to increase the risk of cancer.

The formation of DBP's is dependent upon the following:

1. Type of disinfectant, dose and residual concentration
2. Concentration and characteristics of precursors
3. Water temperature
4. Water Chemistry (pH, organic nitrogen, iron and manganese)
5. Contact time and mixing for disinfectant

The recycling of water from the elevated storage tank increases the "age of water" in the distribution system and this additional age contributes to the production of DBP's. For example, during the month of May 2009, a total of 15.8 million gallons of water was sold. Assuming the 50,000 gallon per day recycle rate, this represents 1.5 million gallons recycled or 9.4 % of all water sold. This 9.4 % represents an "unidentified increase" in water age. When recycled water that has previously been disinfected is re-chlorinated, the disinfectant level is increased, creating the possibility to produce additional disinfection by products.

The longer the contact times between disinfectant and the precursors, the greater the amount of DBP production. THM's and HAA's once formed, are chemically stable as long as sufficient residual exists and as a consequence high concentrations of DBPs can accumulate in water with old age.

Figure 1 illustrates the concentrations TTHM's and chlorine residual in the distribution system over the past four years. The chlorine residual and HAA concentration have increased approximately 20% from year 2007 to 2008. All values, except the 2005 THM concentration are below the maximum containment levels allowed.

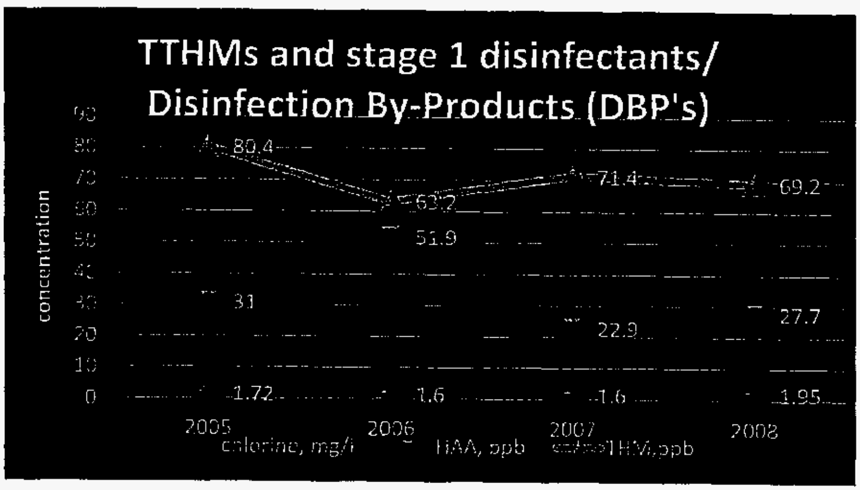


Figure 1

Lead and Copper Levels

Figure 2 provides summary information on tap water lead and copper concentrations from 2005 through 2008. The lead level in the tap water has increased by 50% in a single year. Treatment techniques to reduce levels must be initiated when lead levels equal to or greater than 15 ppb are measured. Treatment options include addition of corrosion control agents, lead service line replacement and public education. If the lead and copper levels are exceeded, If WMSI incurs a violation, a public education program must be developed an issued to customers within sixty days. Currently the lead concentration is trending is upward and should be carefully monitored. If the levels reach 15ppb, notification will need to be issued to customers identifying lead in the drinking water is an issue. We recommend being proactive in addressing the lead concentration. A good program of corrosion control will optimize the disinfection process and retard the dissolution of metals such as lead. With the limited information available, we cannot identify a specific cause for the spike in lead concentration over a single year period of time. The action level for lead is when tap water reaches 15 ppb and for copper the action level is1.3 ppm

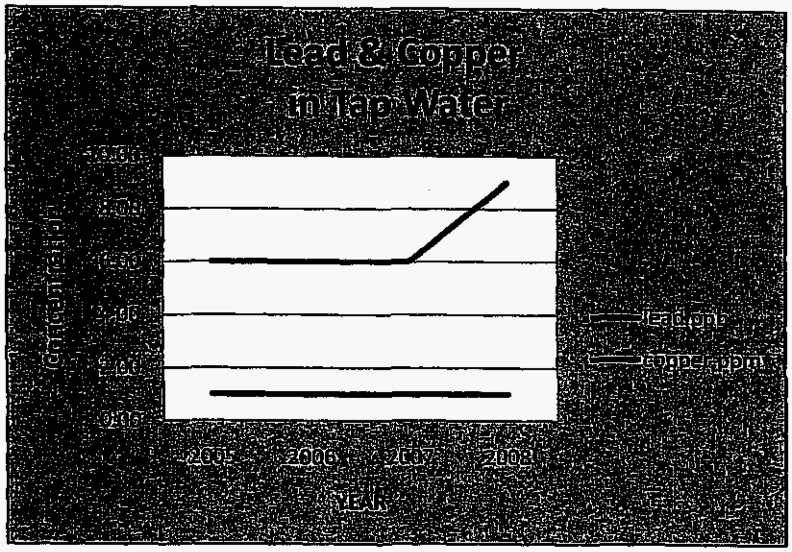


Figure 2

Flushing Program

WMSI has a flushing program in operation to flush the system to maintain water quality. A copy of dead end flushing program is included in **Appendix One.** A review of 2009 water use and sales data indicates a total of 23.514 million gallons of water was used to flush the distribution system.

Using the Water Management Services rate Schedule GS with a gallonage charge of $4.72 per thousand, the water lost to flushing results in lost potential revenue of $ 110,900. With the need to flush the system on a regular basis, not all of the $110,900 revenue could be claimed, but it appears that an excessive amount of flushing may be occurring.

The incorporation of chlorine analyzers in the distribution system to monitor and control rechlorination may be an alternative to spot flushing to help maintain chlorine residuals. Reliable analyzers are economical with only monthly maintenance required. Chlorine analyzers located ,for example near the State Park, could be used to monitor chlorine levels in the water automatically and when the chlorine levels fall below desired values, provide a control signal to turn on a rechlorination system. The use of a rechlorination system to raise the chlorine residual may increase the levels of THM's and HAA's. However, rechlorination can be a very useful tool in decreasing the DBP's by reducing the concentration of disinfectant needed in the finished water leaving the water plant.

Suggested improvements:

1. Develop an enhanced flushing program that incorporates unidirectional flushing, and spot flushing to maintain water quality. Every distribution system is different and development an appropriate flushing program is recommended. A unidirectional flushing program consists of isolating areas of the system, and flushing in a single direction at a high velocity to remove biofilm and corrosion products. Spot flushing is usually implemented by opening hydrants with little planning until the disinfect level is reached.

2. At dead end locations, the installation of portable automatic flushers can save labor. These devices are connected to the hydrant 2-1/2 inch nozzle. Flushers can operate several times a day. The devices are programmable and battery operated. A typical flusher with dechlorinating device, security device and locking mechanism can cost as little as $2,300. We recommend the incorporation of these devices in the flushing program. The exact quantity will need to be identified, but for a small system, three flushers may be a suitable number and moved as needed within the system. The purchase of these devices could be incorporated into the annual operating budget in lieu of purchasing through a CIP program.

Backflow Prevention

The water system must have a comprehensive cross connection and back-flow prevention plan to meet either FDEP standards. WMS has a cross connection program in place with testing performed annually. Various testers are utilized. The system includes 604 backflow prevention devices.

Suggested improvements:

1. None, at the time of this report.

Water Loss

Drinking water providers need to manage the valuable water resource to provide their customers water, at a reasonable price while maintaining reliable operations. A strong water loss control program is essential to meeting these obligations. A water loss control program, when properly implemented is intended to achieve the following goals:

- Provide safe and reliable drinking water while reducing the potential for contamination resulting from the lack of system integrity.
- Optimize revenues
- Limit unnecessary or wasteful water use
- Optimize supply while minimizing system disruptions

Water loss is a combination of "real" and "paper" losses. A real loss is identified as a physical loss of water from the distribution system. A paper loss includes meter inaccuracy, billing errors, unauthorized use, and/or accounting errors. Using the 2009, Water Management System Annual Report data for pumping and purchased water, the calculated water loss is 9.17 %, with flushing volumes not included.

A reduction in "real losses" reduces operational costs including power, chemicals and treatment costs. If the losses can be reduced through the implementation of a water loss control program, the water saved can be used to expand the service area without a significant increase in developing new supply sources, thus stretching the existing supply to meet the new demands.

Paper losses can be reduced by improving the metering system and billing systems. The use of meters that are improperly sized or old meters (typically older than 15 years) can result in under registration of flow. A properly implemented plan for replacement of meters can increase water revenue and reduce water loss. Because the WMSI system includes over a thousand service meters, it is not practical to inspect and test every meter each year. PBS&J suggests testing all meters sized 2-inch and larger every year and periodically testing residential meters. The testing of meters for the largest water users will confirm the meter accuracy and identify if mechanical repairs are needed. By periodically testing the residential meters, the test data can be used to develop a historical database on meter accuracy as the meters age and this information can be utilized to develop a meter replacement program.

PBS&J preformed a preliminary water audit to trace the flow of water from the water plant to the customer. The water audit process is designed to identify the consumption, losses and prioritize water loss control program initiatives.

Water Loss Auditing

Using the 2009 WMSI Annual Report data in combination with American Water Works Associations (AWWA) water the loss control committee, audit software, PBS&J performed a water loss audit to identify areas to reduce water losses and increase revenues. A copy of the audit is included in **Appendix Two**.

Results of the water audit are presented in **Table 3**.

Table 3
Water Audit
Water Distribution System Operational Efficiency

Performance Indicator	Value
Non revenue water as a percent by volume of water supplied	21.2 %*
Apparent loss (inaccurate meter) per service connection per day	3.74 gallons
Estimated real losses per mile of water main	594 gallons per mile per day

*includes water used for flushing and firefighting

This information indicates controlling water loss and reducing the percentage of non-revenue water should be a high priority for WMSI. In general, the water audit results indicate the areas needing improvement include:

1. Improvements to data collection and metering.
2. Development and implementation of a metering testing program.
3. Develop an infrastructure-monitoring plan for leakage.
4. Improve business plan to improve data collection; for example, replacement of existing well metering devices with new. Replacement or recalibration of finished water metering devices.
5. Establish target setting for water loss reduction in apparent losses.

Results of the audit include recommendations on water loss control planning and areas that need improvement. The results of the audit are summarized below in **Table 4.**

Table 4
Water Loss Audit
Priority Areas

Priority areas of attention	Comment
Master metering error adjustment	Errors in metering at the wells are in question. PBS&J addressed this in TM3.
Volumes of water	Errors in finished water metering. PBS&J addresses this in this TM.
Customer metering inaccuracies	WMS has flagged this area as an issue internally. The water tariff should to be updated to incorporate metering inaccuracies.

Service Meter Inaccuracies

American Water Works Association, AWWA Standard C700-02, discusses the general design and characteristics of water service meter behavior. **Table 5** provides detailed information on the recommended minimum test flow (0.25gpm) and normal test flow (1-20 gpm). At the normal test flow, the meter registers between 98.5% and 101.5% of actual water flow. At the minimum test flow, however, the meter registers between 95% and 101%.

In Section 26 of WMSI Water Tariff, there are accommodations in 26.0 to adjust customer water bills for meter error and fast service meters, as well as meter accuracy requirements. We recommend that both the maximum operating flow and the maximum allowable pressure loss be incorporated into metering requirements in the Water Tariff. With identification of flows and pressure loss, if these values are exceeded, WMSI would have the justification for meter replacement.

Suggested Metering Improvements:

1. Suggested (not a CIP priority item)-Once the tariff has been modified, we recommend the incorporation of a replacement plan for service meters that do not meet the standard. An allowance of $600 per meter (Small 5/8-inch meters) be included in an annual capital improvements plan. The service meter replacement plan should be implemented over the next several years.

Table 5
Characteristics of displacement type meters*

Meter size Inches	Safe maximum operating capacity, gpm	Maximum pressure loss at safe maximum capacity, psi	Recommended Maximum rate for Continuous operations, gpm	Minimum test flow, gpm	Normal test flow limits, gpm
½	15	15	7.5	1/4	1-15
1/2x3/4	15	15	7.5	1/4	1-15
5/8	20	15	10	1/4	1-20
5/8x3/4	20	15	10	1/4	1-20
¾	30	15	15	½	2-30
1	50	15	25	¾	3-50
1-1/2	100	15	50	1-1/2	5-100
2	160	15	80	2	8-160

- **Information taken from AWWA C-700-02 cold water meters displacement type, bronze main case, Table1.**

Service Meter Replacement Program

The average service meter life is less than 25 years. Lifetime is related to factors such as water quality, system pressure and most importantly volume of water metered. PBS&J recommends replacing a select number of aging service meters on an annual basis and funded through the annual capital improvements budget. Our experience in other areas of the county indicates a meters useful lifetime is closer to 15 years. The best method to determine if service meters need to be replaced is to initiate a meter-testing program, to periodically test, approximately 50 meters every 5 years to establish if the service meter is accurate and compare how these meters perform against AWWA standards or the service meter warranty.

WMSI may also consider the incorporation of new metering technologies, such as advanced metering systems, that include electronic equipment to read meters remotely. Remote read metering can reduce labor costs to read the meters. Either new electronic metering heads or new meters can be installed when initiation of a remote read system.

Valve Maintenance Program

WMSI has in place a valve maintenance program with an accompanying maintenance log sheet for each individual valve in the system. The valves are located using GPS. Valves should be exercised at least once per year.

Program appears to be satisfactory and working. Below are few suggestions for consideration to improve your program.

1. Identify the date of manufacture and the manufacture name
2. Categorize valves with poor service records so that replacements can be incorporated into the CIP.
3. Identify valves that are complex and hard to maintain and consider replacing with valves of simpler design.
4. Set goals to replace non- functional valves on an annual basis.

Hydrant Maintenance Program

WMSI has a fire hydrant and maintenance policy in place for installation and maintenance. The program has a list of every fire hydrant location. The program appears to be satisfactory and includes annual exercising.

We suggest adding information items to your inspection checklist:

Dry barrel

- On traffic models check for breakaway device for damage
- Remove outlet cap and use listening device to detect for leakage.
- Check flanges for leaks when full open
- Tag hydrant if in operable and notify fire department. Schedule hydrant for repairs.

Wet barrel

- Clean cap and thread nozzles, lubricate if necessary
- Exercise both the hydrant and the isolation valve

Hydrant flow testing

- Recommend 10 percent of the hydrants be flow testing every year to determine that water and pressure are available for firefighting. A record of each test should be cataloged to provide a historical trending of changes to the distribution over time.
- Hydrant testing should be performed in accordance with AWWA Manual of Practice 17.

As with the valve program, we suggest that a more detailed list of information be complied with each hydrant manufacturer identified and contacts for spare parts be developed. With the current program, the individual homeowner purchases the hydrant for installation. Once installed WMSI services and maintains the hydrants. The lifetime of a hydrant is approximately 50 years. With this said, at some point in the future it is unclear how WMSI will implement a replacement program. Consideration for standardizing around a single manufacturer and single type (wet or dry barrel) can help with reducing the quantity and type of spare parts needed.

External Corrosion Monitoring

WMSI needs to incorporate an external corrosion-monitoring program. The plan would incorporate a standardize method for mapping and recording the location of line breaks and the details associated with the break including, conditions of failed pipe, the pipe material, pipe size, and other conditions. This information may be of value when evaluating an improvements program.

Recommended capital improvements:

1. Priority CIP item- Purchase a listening device to detect flow leakage in the distribution system. This is an essential tool in maintaining and detecting systems leaks.

END

APPENDIX ONE

DEAD END FLUSHING PROGRAM

WATER MANAGEMENT SERVICES, INC.
139 W. GULF BEACH DR.
ST. GEORGE ISLAND, FL 32328
(850) 927-2648 PHONE
(850) 927-3395 FAX

Pursuant to FAC Rule 62-555.350(2) Water Management Services Inc. adopts the following Dead End Flushing Program:

Dead-end lines must be flushed on a routine basis to replace water that is stagnant due to water demand being low relative to size of the water main. As St. George Island is a barrier island and population varies due to the season, we shall institute the following policy:

1. Dead end lines more than 6 inches in diameter shall be flushed quarterly.
2. Dead end lines less than 6 inches in diameter shall be flushed when there is a complaint by a customer.
3. When possible use a meter to determine amount of water being flushed.
4. Upon completion of flushing, a chlorine residual is taken to insure that the chlorine residual is within the regulated requirements.
5. If the chlorine residual is less than regulated flushing is continued the minimum requirement is reached.
6. Field personnel should make note of any location needing flushing more often and take steps to insure a good quality of water for our customers.

DEAD END LIST

6 INCH

1. BAY SIDE SUBDIVISION-WET END
2. PELICAN COURT
3. DOVE LANE
4. GULF VIEW WAY
5. RESORT VILLAGE LANE
6. PARK LANE
7. BAYBERRY LANE
8. ACACIA-WEST END
9. SANDY LANE-EAST END
10. TRACT 34

2 INCH

1. SCHOONER LANDING-WEST END
2. COQUINA DRIVE-EAST END
3. EGRET POINT ROAD
4. SEAGULL WAY
5. BLUE HERON TRAIL
6. TURPINTINE TRAIL-EAST END
7. TURPINTINE TRAIL-WEST END
8. SAND DOLLAR TRAIL-WEST END
9. SEMINOLE LANE
10. SMUGGLERS COVE ROAD
11. INDIAN HARBOR ROAD
12. HARBOR LIGHT LANE
13. BAY VIEW DRIVE
14. LARK LANE
15. KINGFISHER ROAD
16. KINGFISHER COURT
17. IBIS WAY
18. GANNET TRAIL
19. FORSYTHIA TRAIL
20. PELICAN LANE
21. EVODIA COURT

22. CURLEW WAY-EAST END
23. CURLEW WAY-WEST END
24. BITTERN COURT
25. AVOCET LANE
26. DOLPHIN COURT-EAST END
27. DOLPHIN COURT-WEST END
28. WHELK COURT-WEST
29. PELICAN COURT-EAST
30. REED COURT-WEST
31. CORAL COURT-EAST
32. SUZIE COURT-WEST
33. DENISE COURT-EAST
34. FORSYTHIA COURT
35. FORSYTHIA TRAIL
36. SEASAIDE DRIVE-WEST
37. SEASIDE DRIVE-EAST
38. WINDY PASS
39. CANOPY LANE
40. ELM COURT
41. CAMELIA COURT-EAST
42. 12TH ST. WEST
43. AKEL STREET
44. 7TH AND WEST BAYSHORE
45. PALMER STREET
46. BUCK STREET
47. PATTON STREET
48. HOWELL STREET
49. WING STREET
50. GANDER STREET
51. EAST PINE BETWEEN 5TH AND 6TH STREET
52. EAST PINE BETWEEN 6TH AND 7TH STREET
53. TRACT 1
54. TRACT 3
55. TRACT 6
56. TRACT 8
57. TRACT 9
58. TRACT 10
59. TRACT 11
60. TRACTS 13-14
61. TRACTS 15-18

62. TRACT 34
63. TRACTS 35-36
64. TRACTS 38-39
65. TRACT 40
66. TRACT 41
67. TRACT 42
68. TRACTS 43-44
69. TRACTS 46-47
70. TRACT 48
71. TRACT 49
72. TRACT 50
73. STATE PARK-EAST END
74. STATE PARK-CAMPGROUND

APPENDIX TWO

WATER LOSS AUDIT

AWWA WLCC Free Water Audit Software: Reporting Worksheet

WAS v3.0

Back to Instructions

? Click to access definition

Water Audit Report for: St. George Island, Florida
Reporting Year: 2008

Please enter data in the white cells below. Where available, metered values should be used; if metered values are unavailable please estimate a value. Indicate your confidence in the accuracy of the input data by grading each component (1-10) using the drop-down list to the left of the input cell. Hover the mouse over the cell to obtain a description of the grades

All volumes to be entered as: MILLION GALLONS (US) PER YEAR

WATER SUPPLIED << Enter grading in column 'E'

Volume from own sources:	7	203.499	Million gallons (US)/yr (MG/Yr)
Master meter error adjustment (enter positive value):	2	3.000	under-registered MG/Yr
Water imported:	n/a	0.000	MG/Yr
Water exported:	n/a	0.000	MG/Yr
WATER SUPPLIED:		205.499	MG/Yr

AUTHORIZED CONSUMPTION

Click here: ? for help using option buttons below

Billed metered:	8	165.379	MG/Yr
Billed unmetered:	n/a	0.000	MG/Yr
Unbilled metered:	5	5.000	MG/Yr
Unbilled unmetered:	9	23.697	MG/Yr
AUTHORIZED CONSUMPTION:		194.076	MG/Yr

Pcnt: Value: 23.697

Use buttons to select percentage of water supplied OR value

WATER LOSSES (Water Supplied - Authorized Consumption) 11.423 MG/Yr

Apparent Losses

Unauthorized consumption:		0.514	MG/Yr
Customer metering inaccuracies:	3	2.000	MG/Yr
Systematic data handling errors:	6	1.000	MG/Yr
Apparent Losses:		3.514	

Pcnt: Value:

Default option selected for unauthorized consumption - a grading of 5 is applied but not displayed

Value: 2.000

Choose this option to enter a percentage of billed metered consumption. This is NOT a default value

Real Losses

Real Losses = Water Losses - Apparent Losses:	7.909	MG/Yr
WATER LOSSES:	11.423	MG/Yr

PBSJ
2639 N Monroe St
Bldg C
Tallahassee, FL 32303

Phone (850) 575-1800
Fax (850) 575-1099
www.pbsj.com

NON-REVENUE WATER

NON-REVENUE WATER:	?		40.120	MG/Yr

= Total Water Loss + Unbilled Metered + Unbilled Unmetered

SYSTEM DATA

Length of mains:	?	8	58.3	miles	
Number of active AND inactive service connections:	?	8	1,813		
Connection density:			31	conn./mile main	
Average length of customer service line:	?	10	10.0	ft	(pipe length between curbstop and customer meter or property boundary)
Average operating pressure:	?	3	80.0	psi	

COST DATA

Total annual cost of operating water system:	?	8		$/Year
Customer retail unit cost (applied to Apparent Losses):	?	8		
Variable production cost (applied to Real Losses):	?	[illegible]		$/Million gallons

PERFORMANCE INDICATORS

Financial Indicators

Non-revenue water as percent by volume of Water Supplied:	15.5%	
Non-revenue water as percent by cost of operating system:		
Annual cost of Apparent Losses:		
Annual cost of Real Losses:		

Operational Efficiency Indicators

Apparent Losses per service connection per day:	5.30	gallons/connection/day
Real Losses per service connection per day*:	N/A	gallons/connection/day
Real Losses per length of main per day*:	368.34	gallons/mile/day
Real Losses per service connection per day per psi pressure:		gallons/connection/day/psi
? Unavoidable Annual Real Losses (UARL):	16.01	million gallons/year
? Infrastructure Leakage Index (ILI) [Real Losses/UARL]:	0.49	

* only the most applicable of these two indicators will be calculated

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PBSJ

ST. GEORGE ISLAND WATER SYSTEM EVALUATION ADDENDUM TM-5

To: Gene Brown

From: Mike Scibelli, P.E.

CC:

Date: September 12, 2010

Re: Water Management Services Incorporated
Addendum Re-evaluating Alternatives 2 and 3

Purpose

Based upon a recommendation included in the Direct Testimony of Mr. Andrew T. Woodcock, before the Florida Public Service Commission, regarding the Application for Increase in Water System Rates in Franklin County by Water Management Services, Inc., PBS&J has prepared a standalone Addendum to Revise Technical Memorandum 5 (TM-5).

Mr. Woodcock testimony indicates he is of the opinion a new ground storage tank can be constructed on the site of the existing tank with a cost savings of $191,492 with the same features and operational flexibility as included in PBS&J's recommended Alternative 2.

The purpose of this Addendum is to re-evaluate costs, risk factors and real estate land costs relative to PBS&J's Alternatives No. 2 and No.3 regarding the construction of a new ground storage tank.

Description of Alternatives

PBS&J developed four alternatives to address either replacement or remediation of the existing ground water storage tank. Each alternative was value ranked in terms of flexibility, reliability, water quality and cost. Of the four alternatives, Alternative 2 and Alternative 3 each included the construction of a new ground storage tank using a Crom Style Prestressed Composite Tank. The original estimated cost for Alternatives 2 and 3 are presented below:

Alternative 2-Construct new 325,000 gallon, dual chamber ground storage tank on 4 lots adjacent to the existing water plant and maintaining the use of the elevated storage tank -$1,706,330

Alternative 3 -Construct a new 325,000 gallon, in the location of the existing ground storage tank- $708,187

PBS&J agrees with Mr. Woodcock evaluation that Alternatives 2 and 3 are not functionally identical. The key differences include:

- Alternative 2 includes new high service pumps located on the roof of the new ground storage tank to enable continued operation in the event of a flood
- Alternative 2 includes relocation of the emergency generator to operate the high service pumps.
- Alternative 2 has a significantly higher cost as a result of hydrostatic walls separating the tank into two independent chambers allowing cleaning and maintenance of the tank without interruption of service. The tank also hydrostatic pumping chamber and reinforced roof to support the pumping equipment.

We have revised our alternative analysis for alternatives 2 & 3 to provide a more accurate comparison between the two alternatives and developed revised opinions of costs and with description of changes to clarify our recommendation.

Re- evaluation of Alternative 2

Alternative 2 includes the construction of a dual chamber tank with hydrostatic wall, a 25,000 gallon pumping chamber with 4-0 feet deep sump and hydrostatic wall separating the pump chamber from the main tank, interconnecting slide gates, new tray aerators and specially designed top for mounting high service pumps. The original alternative included $450,000 to purchase adjacent real estate that will serve as the location for the new ground storage tank. Since the development of the original cost estimate we have identified the following:

- **Cost of property and closing costs**

  As a result of the economic downturn the cost to acquire the adjacent property to the water plant has dropped from $450,000 to $300,000, a value provided by our client.

- **Aerator credit**

  The original estimate ($715,000) for the ground storage tank included cost to replace two tray aerators. The intent of Alternative 2 is to relocate one of the existing aerators from the water plant and re-install the aerator on the new ground storage tank. In the PBS&J detailed estimate developed in TM-5, we inadvertently added the cost of another new aerator in the amount of $28,000 and failed to reduce the of the original estimate to accommodate the relocation of one existing aerator resulting in an over estimated cost of $56,000.

Figure 1 presents a comparison between the costs originally estimated in the left hand column and the revised costs presented in the right hand column. The result is a revised estimated construction cost for Alternative 2 of $1,474,570 resulting in an estimated cost reduction of $231,760.

Figure 1
Alternative 2 Revised Costs

DESCRIPTION		ALTERNATIVE 2: New GST on Lots behind WTP, abandon existing GST and maintain EST	REVISED ALTERNATIVE 2: New GST on Lots behind WTP, abandon existing GST and maintain EST
Demolition of existing GST (assume $10/sf)			
Refurbish existing GST into a new workshop($40/sf)			
Construction of new 500 KG GST with dual wetwell			
Construction of new 325 KG GST with dual wetwell		$715,000.00	$687,000.00
Construction of new 325 KG GST with common wetwell			
Remediation of existing GST (Crom Estimate April 17, '09)plus contingency)			
New aerators (included in the cost of gst)		$28,000.00	
New Vertical turbine high service pumps; roof mounted		$100,000.00	$100,000.00
New chlorine room (Approx $30/sq ft)			
Relocate generator and fuel storage facilities		$7,500.00	$7,500.00
New containment structure for diesel fuel		$5,000.00	$5,000.00
Temporary Operations During Construction			
	temporary pumping ($25K/month)		
	temporary chemical facility		
	yard piping modifications(3% of gst cost)		
	relocate aerators	$5,000.00	$5,000.00
	new pumping chamber		
	miscellaneous		
Subtotal without property		**$860,500.00**	**$804,500.00**
Cost of four (4) lots with closing cost		$450,000.00	$300,000.00
Mobilization/Demobilization (1%)		$8,605.00	$8,045.00
Site Work (2 %)		$17,210.00	$16,090.00
Contingency (20%)		$172,100.00	$160,900.00
Contractor's Bond and Insurance (2%)		$17,210.00	$16,090.00
Contractor's Overhead and Profit (10%)		$86,050.00	$80,450.00
Permitting (1%)		$8,605.00	$8,045.00
Engineering (10%)		$86,050.00	$80,450.00
Estimated Project Total		**$1,706,330.00**	**$1,474,570.00**

Re- evaluation of Alternative 3

We have revised Alternative 3 to make the functionality of this option similar to Alternative 2 so that an "apples to apples" cost comparison can be performed. Our revised cost for Alternative 3 includes the cost a new concrete slab at the site of the existing ground storage tank and the cost of the slab is included in the $687,000 line item for the new ground storage tank. **Figure 2** presents the revised Alternative 3 costs. The comparison between the alternatives indicates a cost differential of $ 64,000, with the construction of the tank at the location of the old ground storage tank projected as the lower cost.

Figure 2
Alternative 3 Revised Costs

DESCRIPTION		REVISED ALTERNATIVE 2: New GST on Lots behind WTP, abandon existing GST and maintain EST	REVISED ALTERNATIVE 3: Construct new GST in current location and maintain EST
Demolition of existing GST (assume $10/sf)			$40,960.00
Refurbish existing GST into a new workshop($40/sf)			
Construction of new 500 KG GST with dual wetwell			
Construction of new 325 KG GST with dual wetwell		$687,000.00	$687,000.00
Construction of new 325 KG GST with common wetwell			
Remediation of existing GST (Crom Estimate April 17, '09)plus contingency)			
New aerators (included in the cost of gst)			
New Vertical turbine high service pumps; roof mounted		$100,000.00	$100,000.00
New chlorine room (Approx $30/sq ft)			$3,000.00
Relocate generator and fuel storage facilities		$7,500.00	$7,500.00
New containment structure for diesel fuel		$5,000.00	$5,000.00
Temporary Operations During Construction			
	temporary pumping ($25K/month)		$50,000.00
	temporary chemical facility		$2,000.00
	yard piping modifications(3% of gst cost)		$20,610.00
	relocate aerators	$5,000.00	$5,000.00
	new pumping chamber		$15,000.00
	miscellaneous		$30,000.00
Subtotal without property		**$804,500.00**	**$966,070.00**
Cost of four (4) lots with closing cost		$300,000.00	
Mobilization/Demobilization (1%)		$8,045.00	$9,660.70
Site Work (2 %)		$16,090.00	$19,321.40
Contingency (20%)		$160,900.00	$193,214.00
Contractor's Bond and Insurance (2%)		$16,090.00	$19,321.40
Contractor's Overhead and Profit (10%)		$80,450.00	$96,607.00
Permitting (1%)		$8,045.00	$9,660.70
Engineering (10%)		$80,450.00	$96,607.00
Estimated Project Total		**$1,474,570.00**	**$1,410,462.20**

In order to construct a circular prestressed composite ground storage tank, requires the installation of a temporary perimeter road surrounding the tank. This roadway is utilized by equipment during the construction of the tank. Typically, the road requirement is 10 to 12 feet in width. For the alternative 3, the 325,000 gallon tank will have an outside diameter of 61 feet, and will require a minimum construction diameter of 81feet. The 81 foot diameter requirement will be problematic, when using the existing site, as the existing tank is approximately 64 foot square with very little room available on three sides and the forth side is shared with the WMS offices. To accommodate the space constraint, a modification to the ground storage tank diameter will have to be made to reduce the diameter and increase its height to over 21 feet to provide the 325,000 capacity. We have obtained preliminary pricing from a tank vendor for a smaller diameter tank and the price was estimated at $694,000, a value close to that used in our revised estimate. Even with the smaller diameter tank, construction conflicts and resultant cost increases are likely when the construction site is constrained.

The major issue with utilizing the existing tank location for the construction of the new ground storage tank is **risk**, which is often hard to reflect in terms of estimated cost. In order to use the existing location, the old tank would need to be taken out of service during the demolition and construction of the new tank. This would require the use of temporary piping and pumping facilities. Use of such facilities is problematic from a constructability standpoint for several reasons including:

- Lack of available space to locate temporary tanks and pumps,
- Space constraints during construction may add to the cost of the project,
- An increase in the complexity of the system which inherently reduces the overall system reliability,
- Lack of redundancy in the system which could lead to extended outages of supply of water,
- Most importantly, the discovery of unforeseen circumstances during construction which could lead to extending the time required for temporary facilities thereby increasing the associated costs.

Summary

It is PBS&J opinion that given the reduction in land costs experienced over the past year and given the uncertainty with the actual cost of using temporary facilities and the related risks, the actual cost differences between alternatives 2 & 3 are insignificant and therefore PBS&J continues to recommend building the new storage tank on a vacant adjacent site.

End