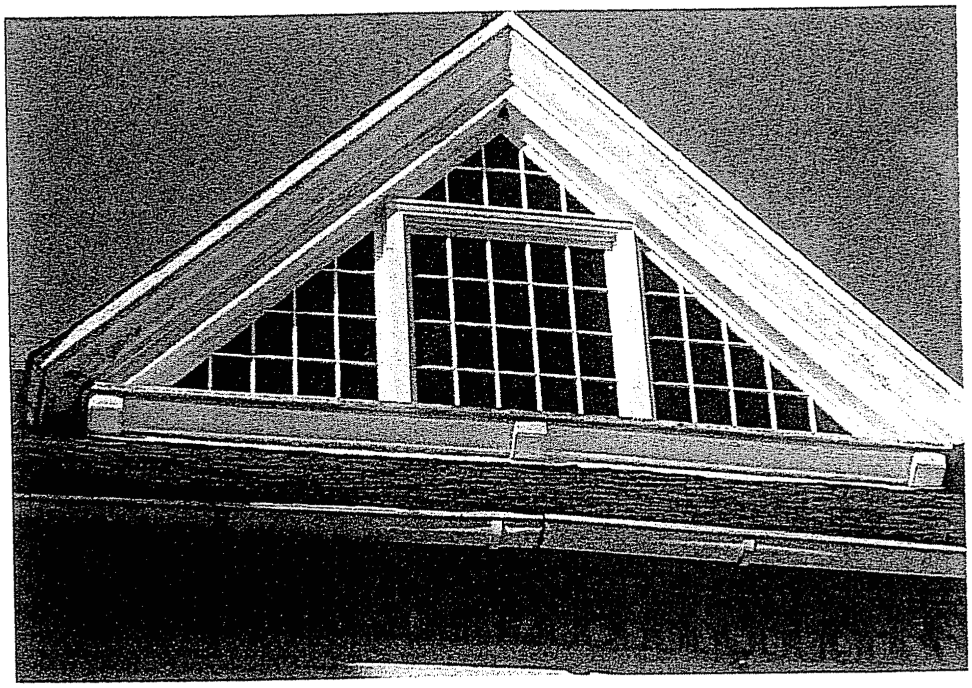

Photograph 9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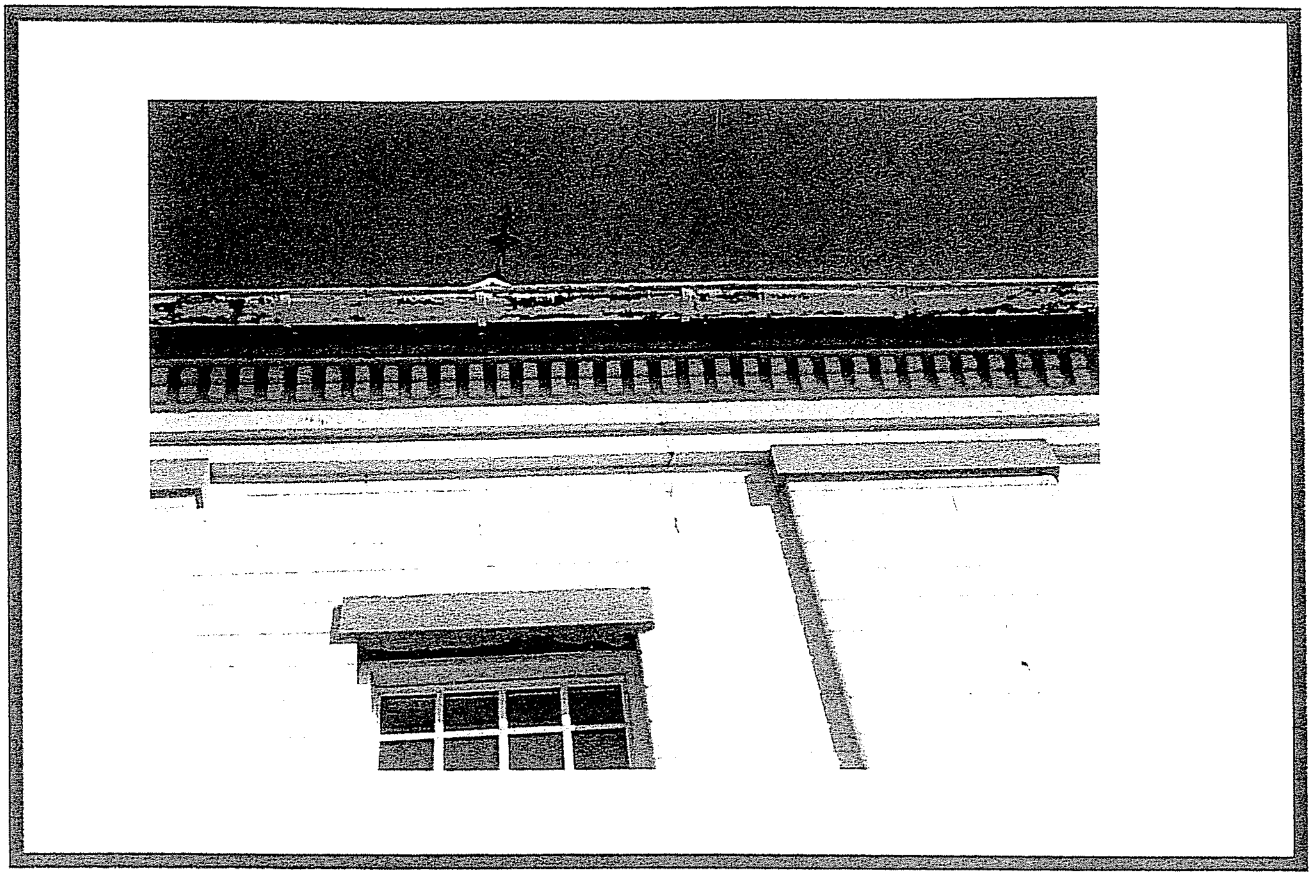

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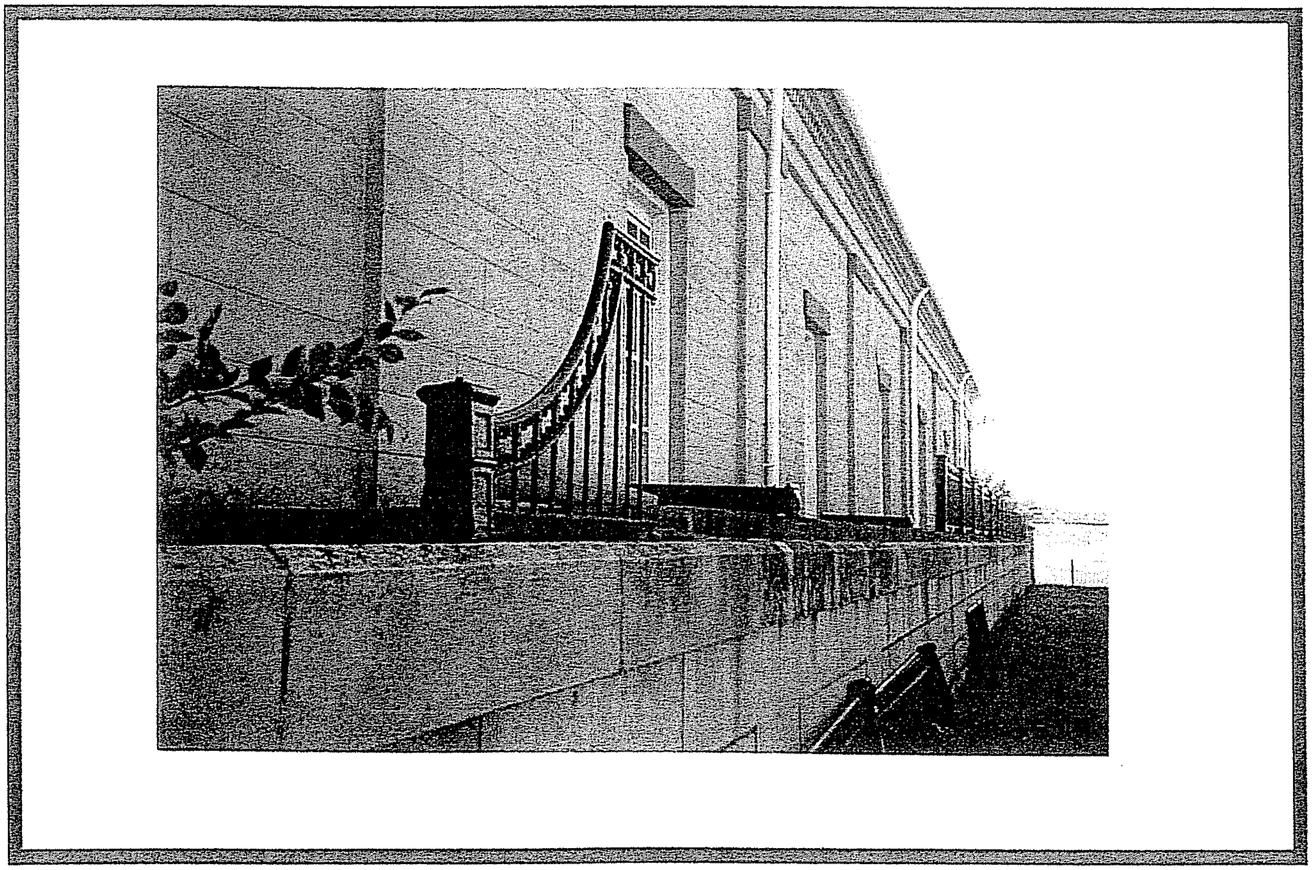

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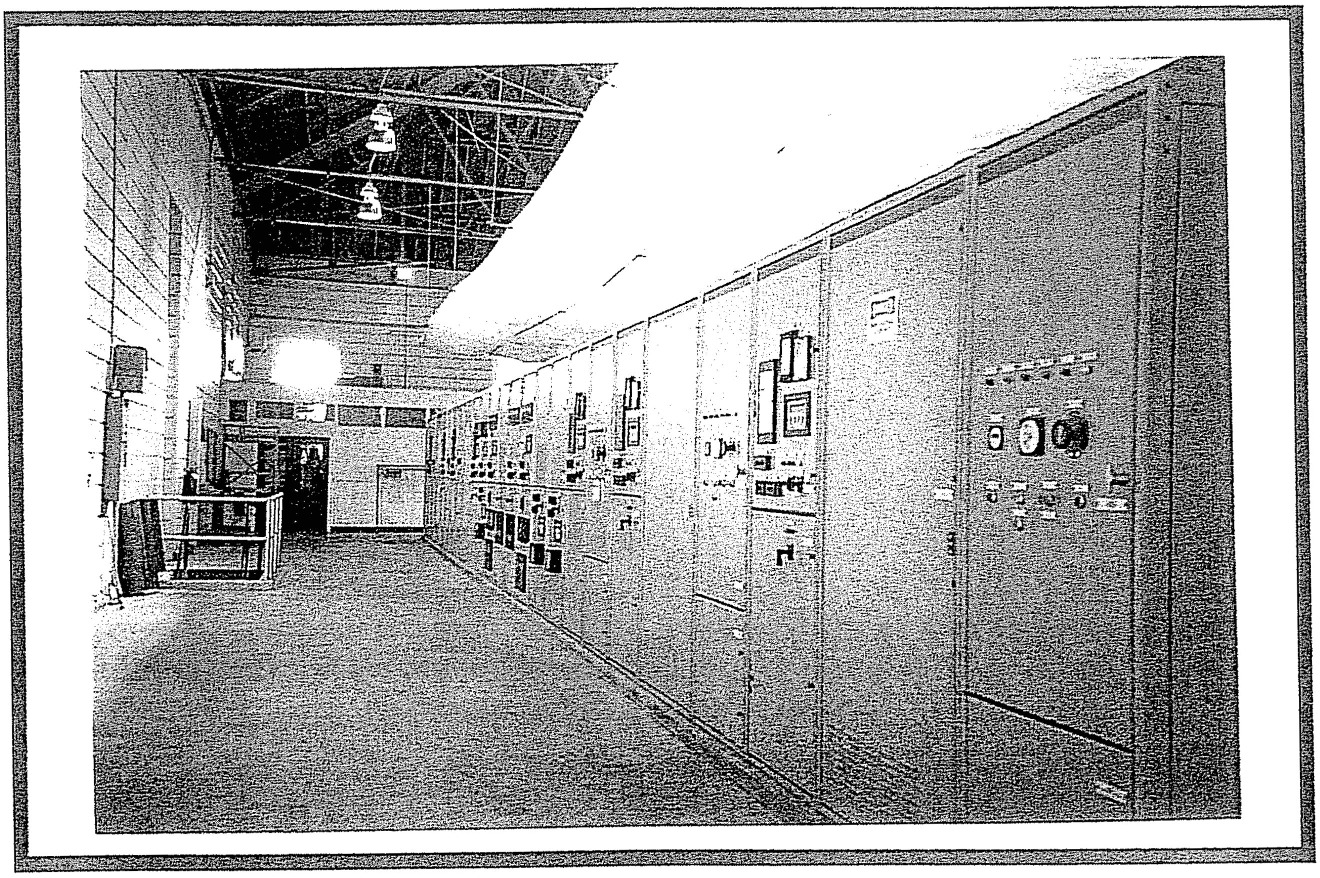

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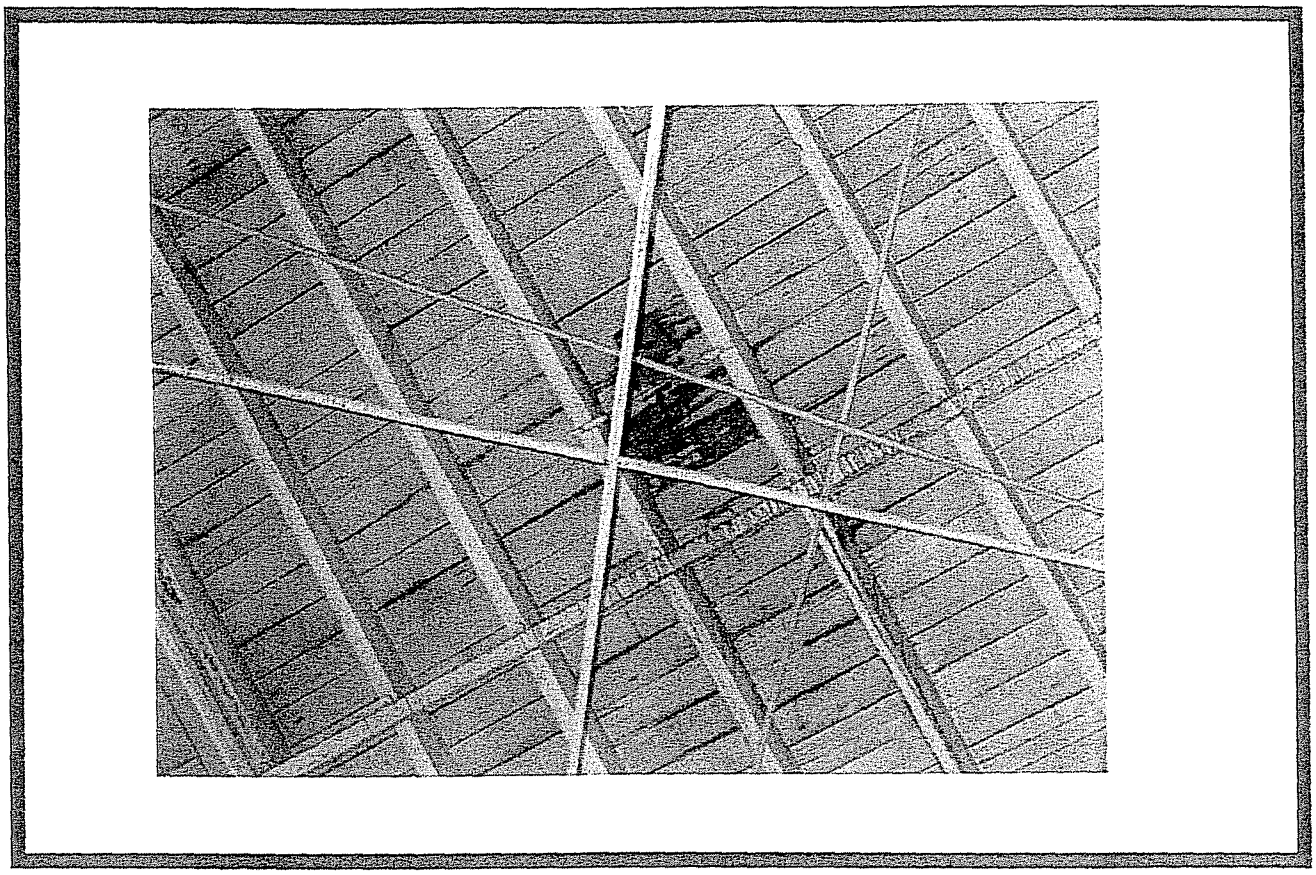

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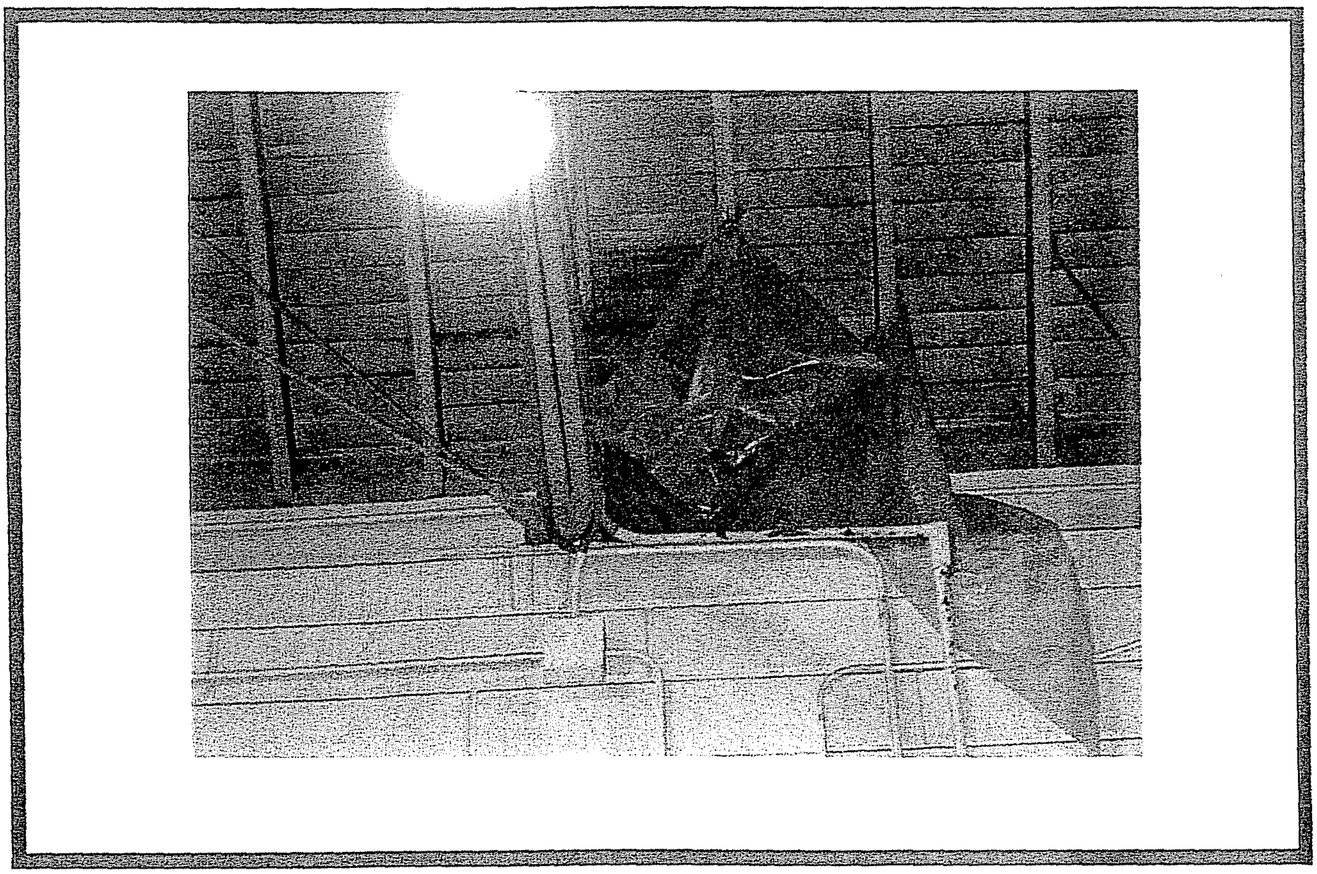

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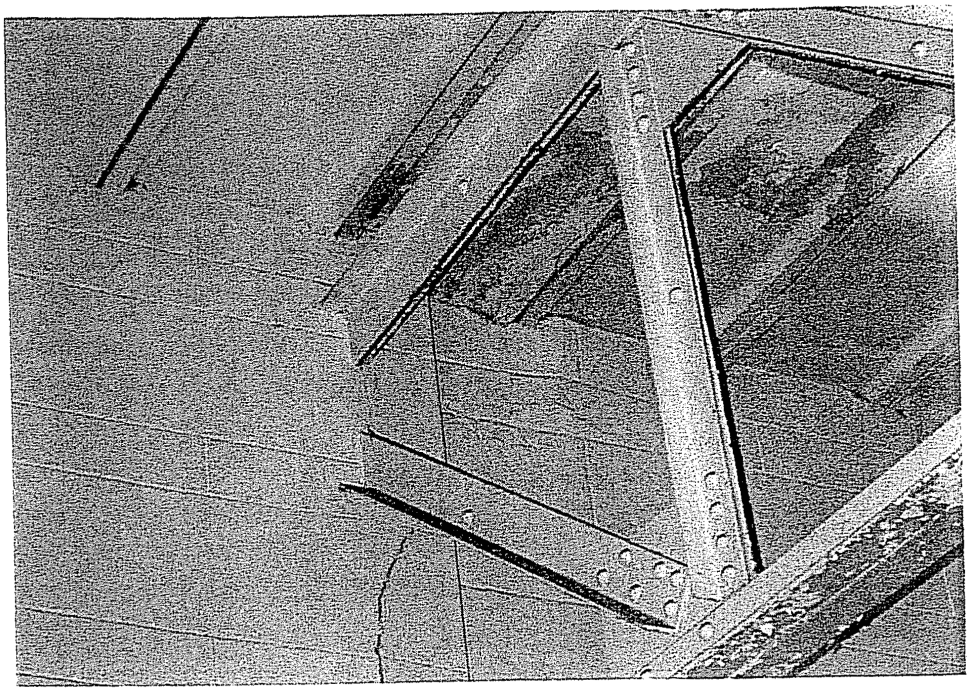

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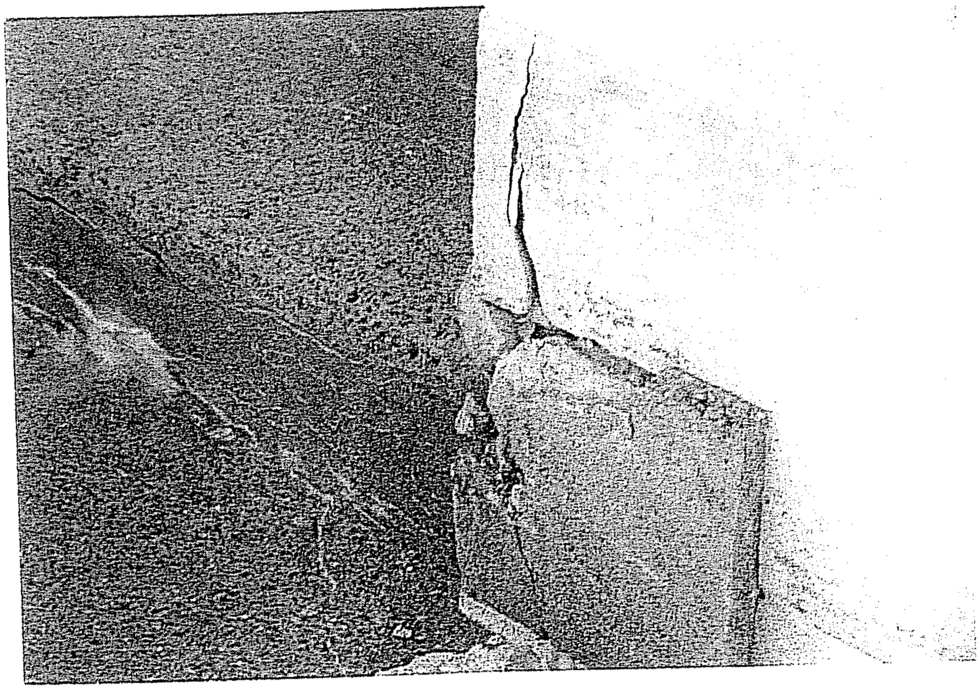

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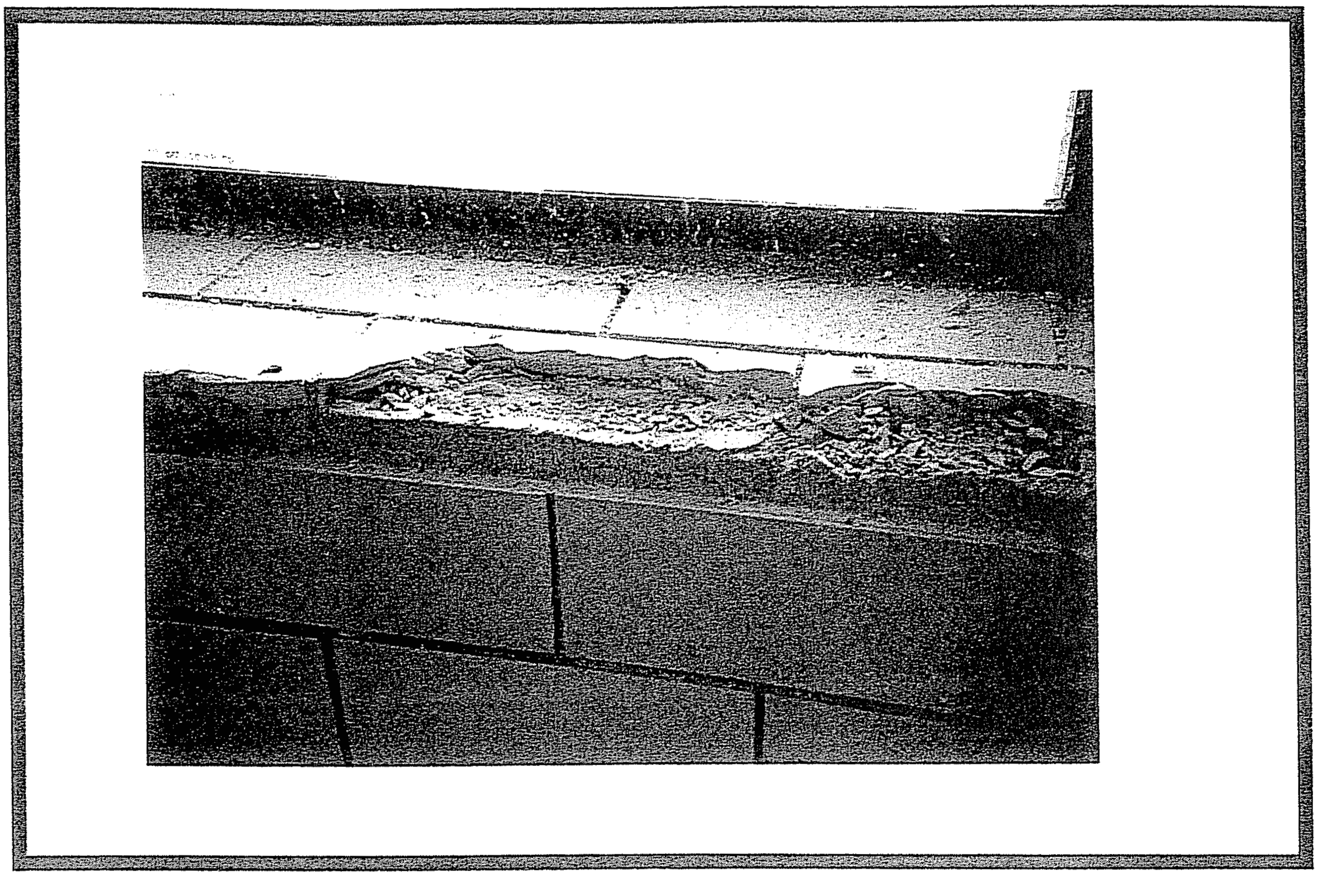

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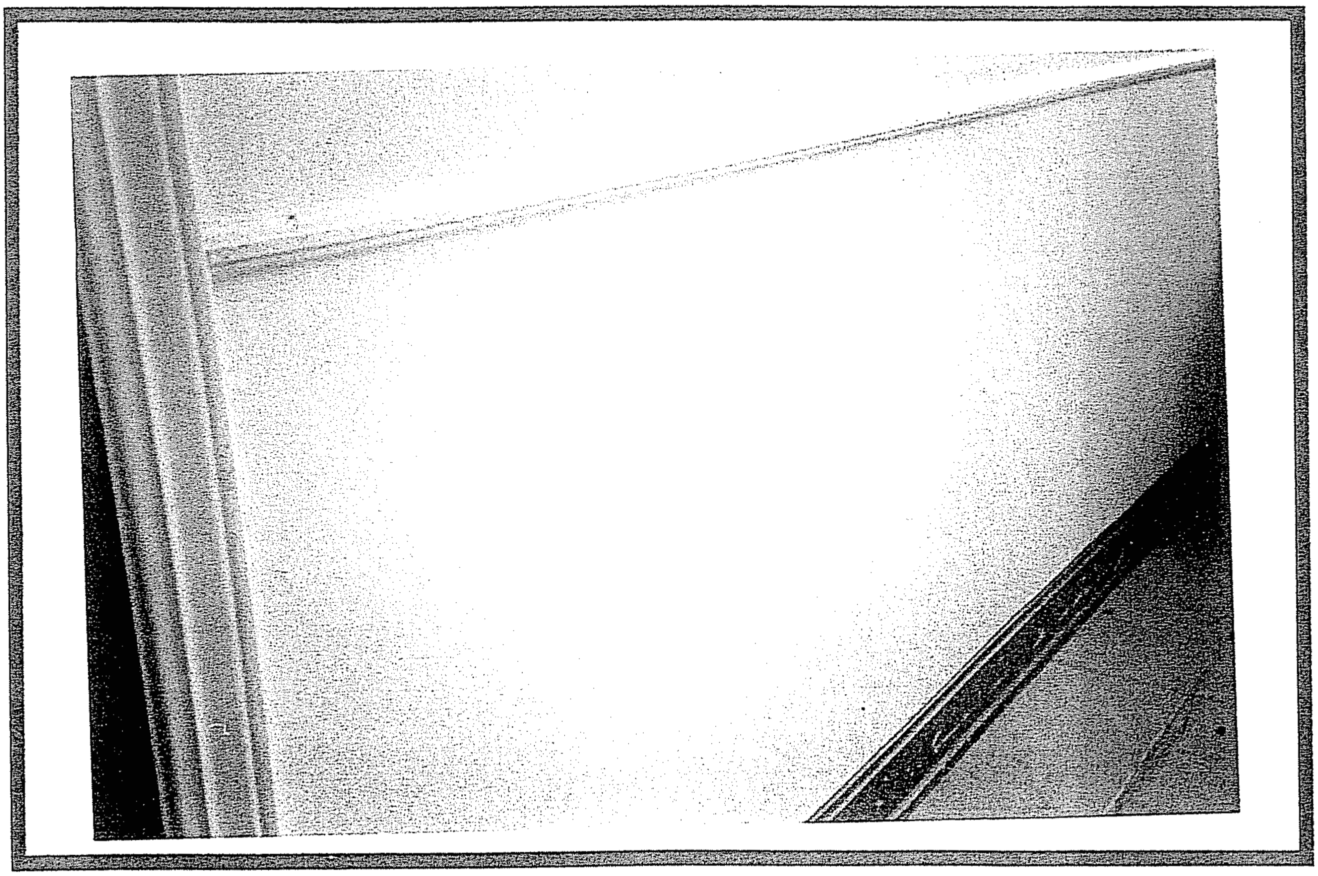

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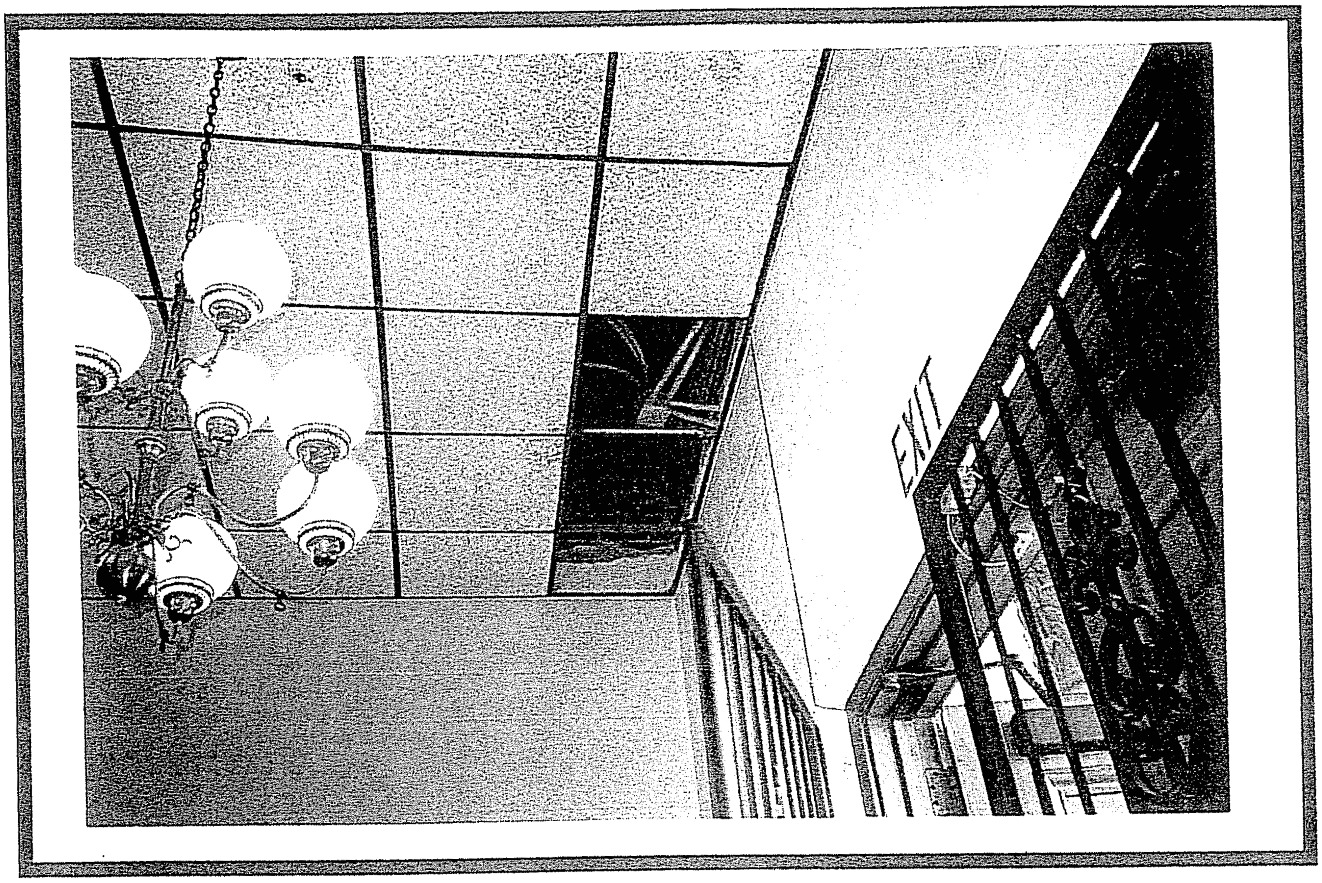

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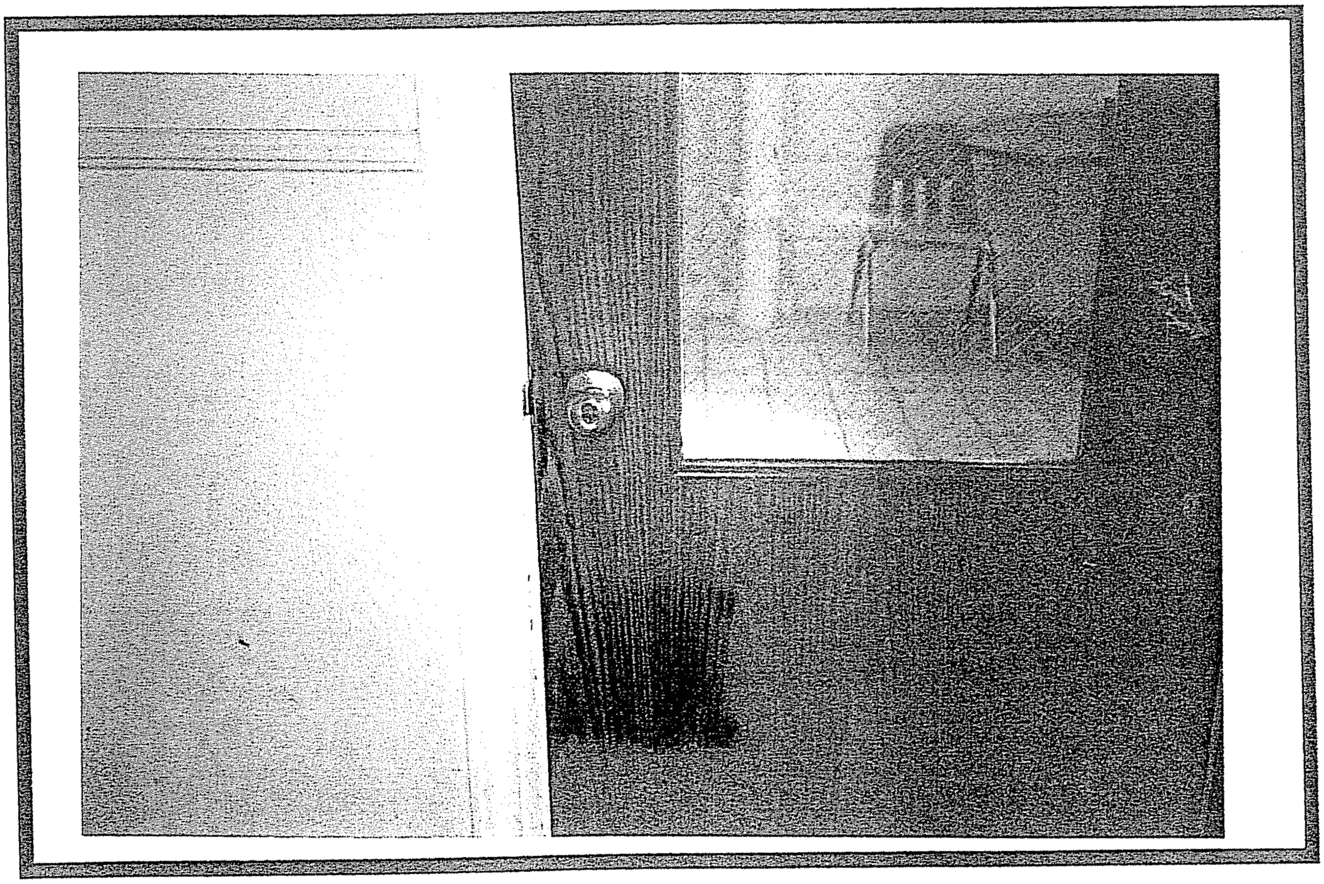

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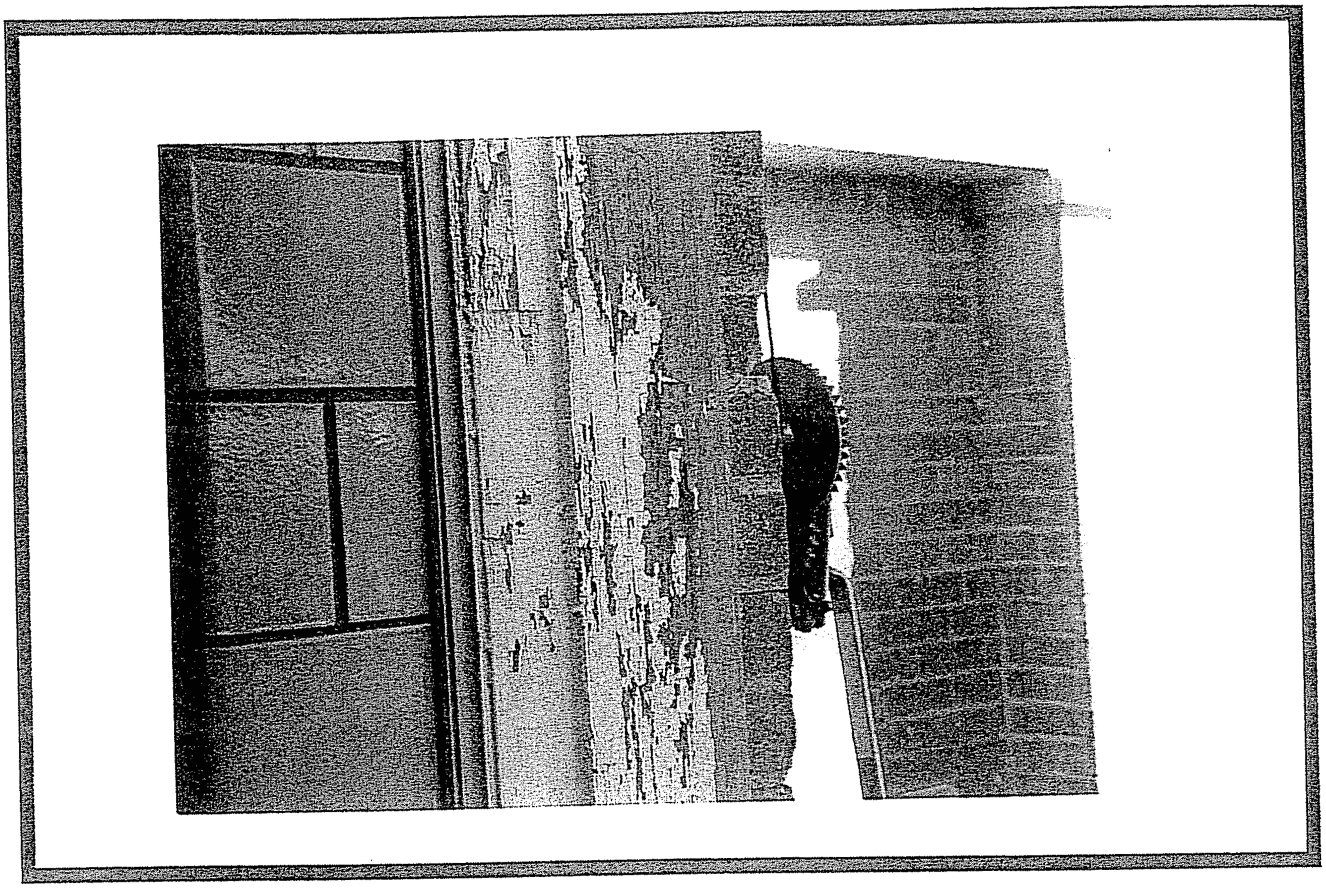


Photograph 25

Photograph 26

B.E. PAYNE FACILITY

(PHOTOGRAPH NOT DISPLAYED)

B.E. Payne Facility

Pavement and Site Condition- (Poor)

- The concrete apron at the loading dock is cracked and is failing. Due to heavy equipment traffic in this area, it is recommended that the existing concrete be removed. The sub-grade should be re-compacted and new concrete installed
  See Photo Number 27

- The concrete walks in the front of the building have failed and present extreme trip hazards. The walks must be replaced. We suggest a sub-grade investigation occur prior to replacing the new concrete walks.
  See Photo Number 28

- Cut out and patch asphalt at the Lift Station loading door.

- Repair trip hazard at the front door of the Lift Station.

- Cut out and repair the drive to the loading dock (300-ft.)
  See Photo Number 29

- Cut out and repair main parking lot (5 locations were noted, 700 sq. ft. each).

- Rails along wall and steps are breaking out of the concrete.

- The rails around the exterior water basins are failing. The concrete landings were stressed either by drilling the post or insufficient concrete coverage around the sleeves. The corrective work will involve the edge forming and the placement of high strength grout.

Exterior Wall-Condition- (Fair to Good)

- Caulk seams in the aluminum fascia.

- Point and patch masonry at corners.

- The exterior wall above the lower entrance has rotated and will require a structural analysis to determine the most appropriate corrective action.
  See Photo Number 30.

- Seal around pipe penetrations.

Roof-Condition (Poor to Fair)

- We suggest you enter into a roof management plan.

- If roof replacement occurs, Insignia recommends the replacement roof be an EPDM membrane as discussed earlier.

- The downspouts are damaged at the ground level and are not functioning correctly.

- The gutters need cleaning.

Glazing Units-Condition (Fair)
- Mowers in the rear have hit the vertical mullions. These mullions should be repaired or replaced to avoid water infiltration.

Structural Condition (Fair to Good)
- The rotation of the exterior wall described above in the exterior wall section must also be considered as a structural concern.
  See Photo Number 30

- The lower level slab on grade has contracted away from the exterior wall. This could indicate either a uniform shrinkage of the slab or an external rotation of the foundation under the wall. This condition should be observed to determine if the opening continues to increase. If the gap is stabilized the crack can be filled with a urethane caulk on top of butyl rod packing. Should the crack continue to open, sub-surface investigation will be necessary.

- The below grade front wall is cracked at the pipe penetrations. Again this cracking and separation of the concrete may indicate foundation roll. However, if the cracking and separation does not increase, repairs may be made to the concrete wall.

Building Interiors (Good)
- Several exit signs are not functioning on the lower level.

- The weather stripping has failed around the exit doors.

- Several water stains were observed on the interior ceiling tiles.

- The terrazzo floor in the main building is cracked. The sections should be cut out and replaced.

Mechanical Systems
- Time did not permit review of the mechanical systems in the facility. However, the rooftop units should be inspected to determine the full extent of deferred maintenance.

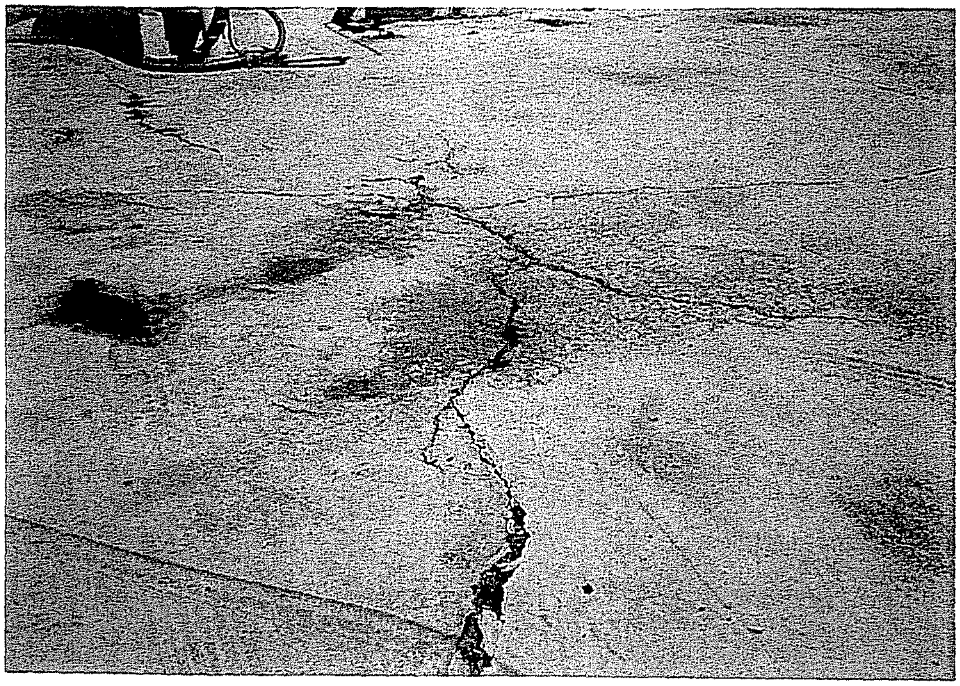
Plumbing System-Condition (Good)
- No deficiencies were noted.

Electrical Systems (Good)
- The electrical system appears to be adequate. This report does not address future needs.

Life Safety-Condition (Poor to Fair)
- Flammable liquids were observed in a standard storage cabinet. This material should be stored in a fireproof cabinet.

- There was no breathing apparatus in either the ammonia or chlorine feed rooms. Code requires an easily obtainable escape method. The ammonia and chlorine are under a vacuum or negative pressure condition. However, this system could fail should the pressure be accidentally altered. The device is very inexpensive and the risk of human life out weights the small price for additional safety precautions. We recommended a breathing apparatus in and outside the room. The outer device will assist in a rescue attempt.

- We did not observe any Material Safety Data Sheets or required labeling.



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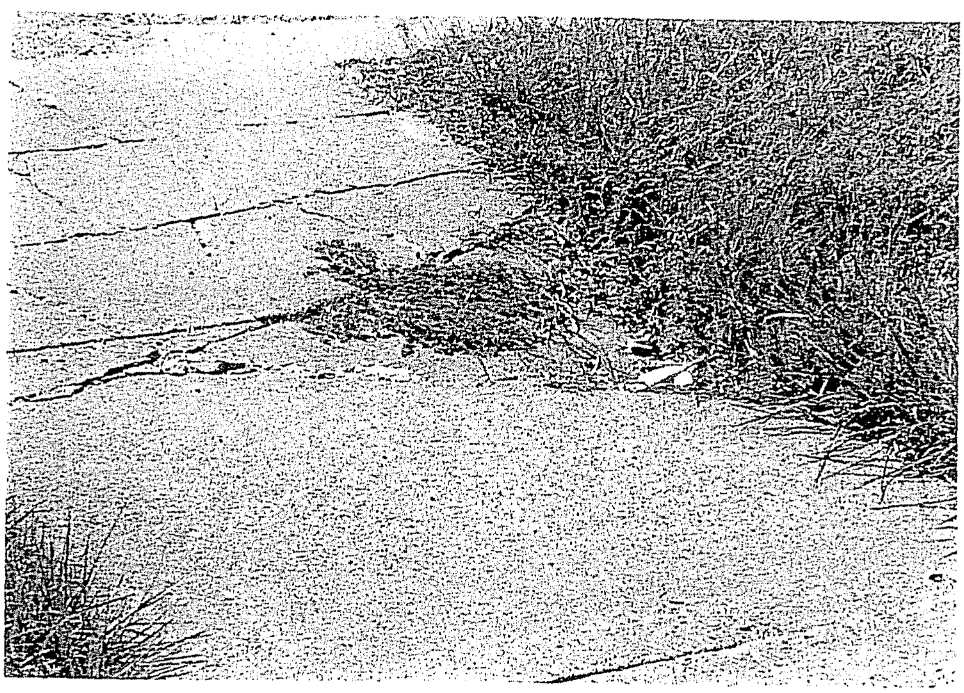
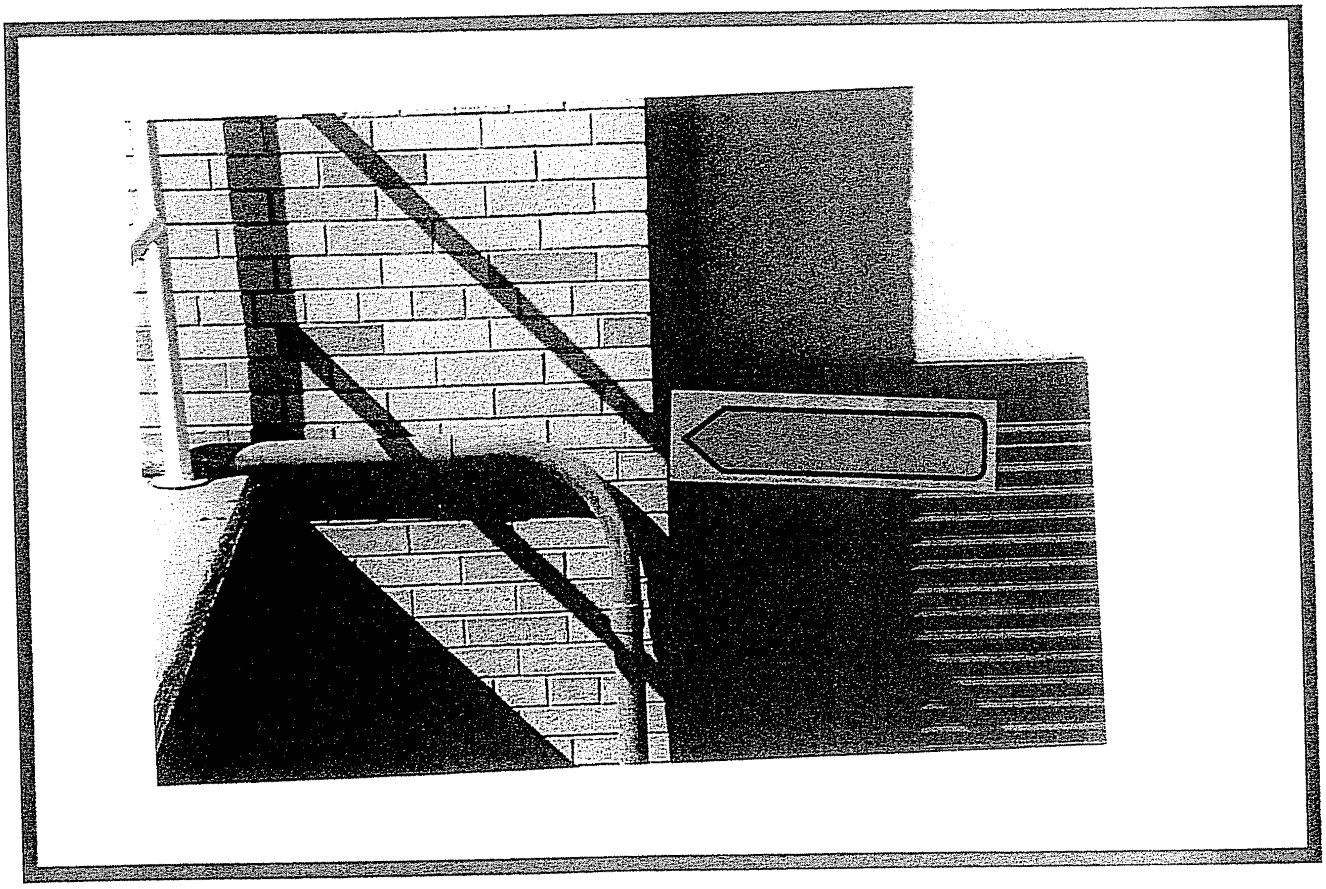


Photograph 28

Photograph 29



Photograph 30

MIDDLETOWN FACILITY

Middletown Distribution Facility

Pavement and Site-Condition- (Good)

- Install pipe guards along the south and north walls to protect the buildings metal panels.
  See Photo Number 31

- Cut out and replace concrete sidewalk where differential settlement has occurred.

- Remove dead trees around material storage building.

- Repair fence around material storage building.

- Repair fence leading to back road to prevent unauthorized dumping.

Material Storage Building-Condition- (Poor)

- Repair soffits and fascia.
  See Photo Number 32

- Repair left corner where building has been hit by trucks.
  See Photo Number 33

- Install pipe guards at corners.

- Install protective angles on corners and across the top of the truck opening.

- Install height restriction pipe to alert dump truck drivers during dumping.
  See Photo Number 34

Exterior Walls (Good)

- Post guards should protect the metal panels. Once the protection is in place the damaged panels should be repaired or replaced.
  See Photo Number 31

- The aluminum soffits are damaged and should be repaired to prevent water infiltration.

- The truck wash should have a containment curb to prevent oil and gasoline from being washed into the storm sewer system.

Roofing System-Condition-Did Not Determine

- The roofing system should be placed under a roof management plan.

- The gutters should be cleaned out.

Glazing Units-Condition- (Good)

- The windows system appears in good condition; however, re-caulking should be scheduled for next year.

Structural Condition-Condition- (Good)

- No visual deficiencies were observed.

Building Interiors (Good)

- The personnel door leading from the rear parking area to the garage needs repair.

Mechanical System

- Due to the size of this facility and time constraints, we did not observe this component.

Electrical

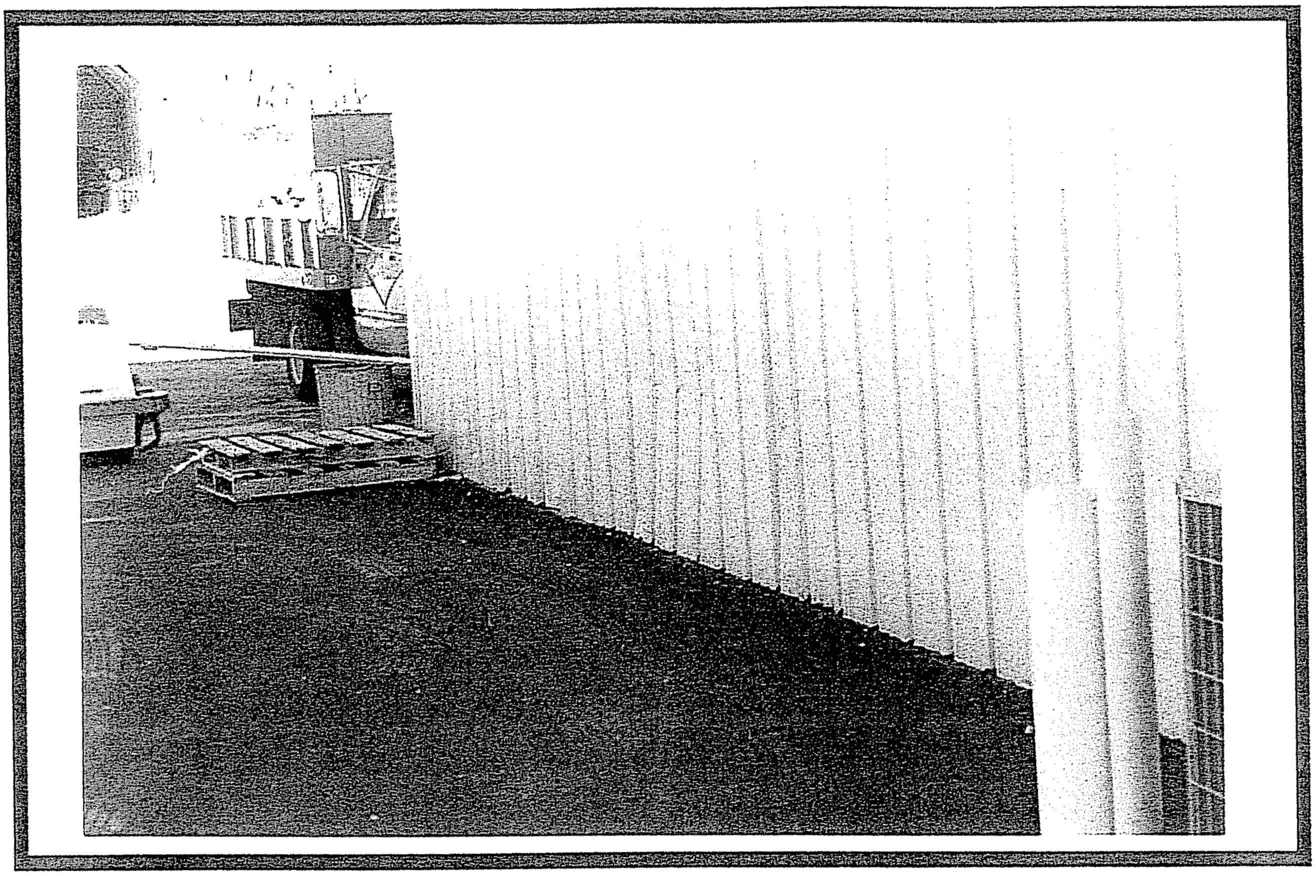
- Only a cursory review was performed, no deficiencies were observed.

Life Safety

- Flammable liquids including gasoline were not stored in a fireproof cabinet.

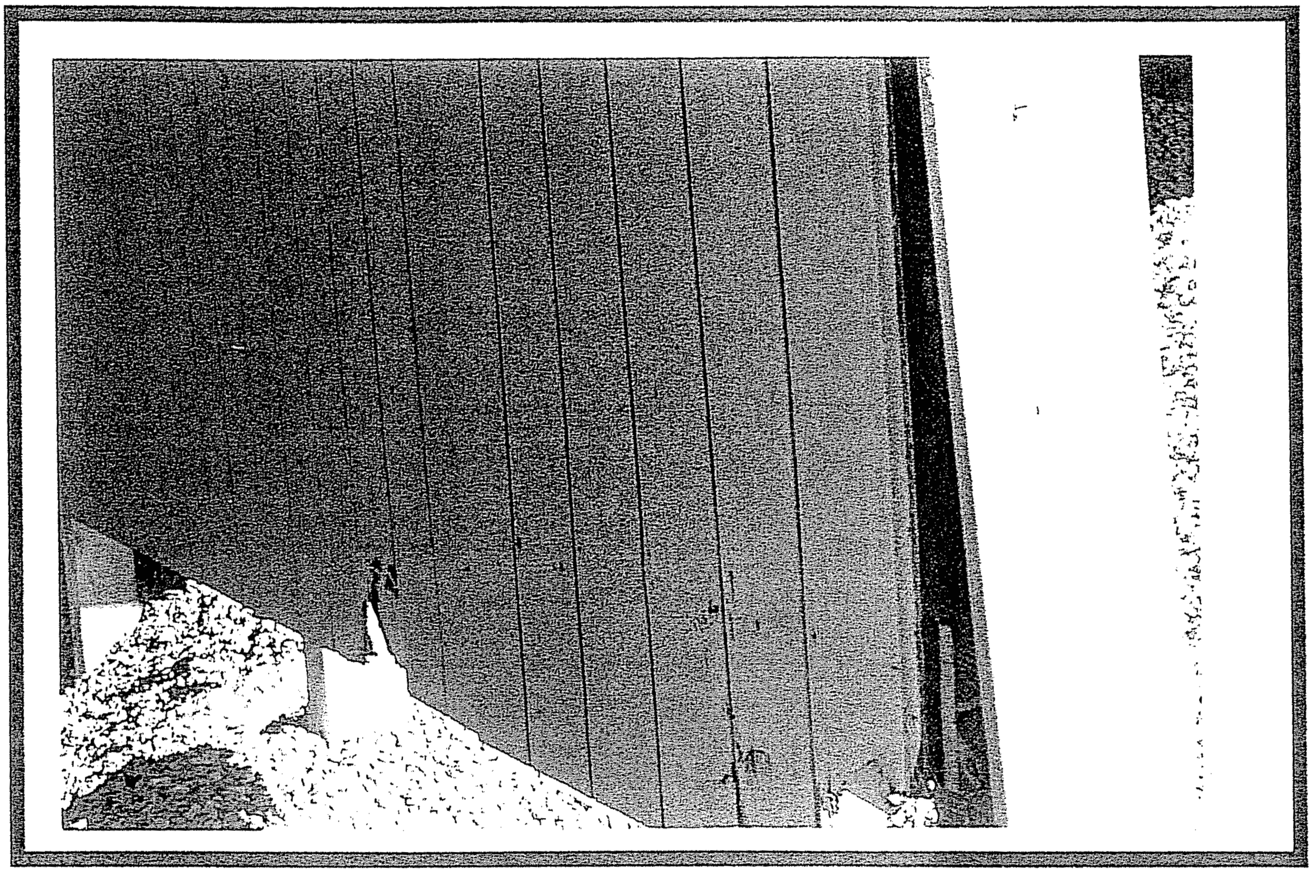
ADA Compliance (Poor to Fair)

- Wrap drain pipes in restrooms.
- Lower towel dispensers in restrooms.
- Install grab bars in handicap stall.
- Change out faucets to lever type.
- Change swing of door and hardware to the front parking lot.
- Dedicate parking for disabled.



Photograph 31

Photograph 32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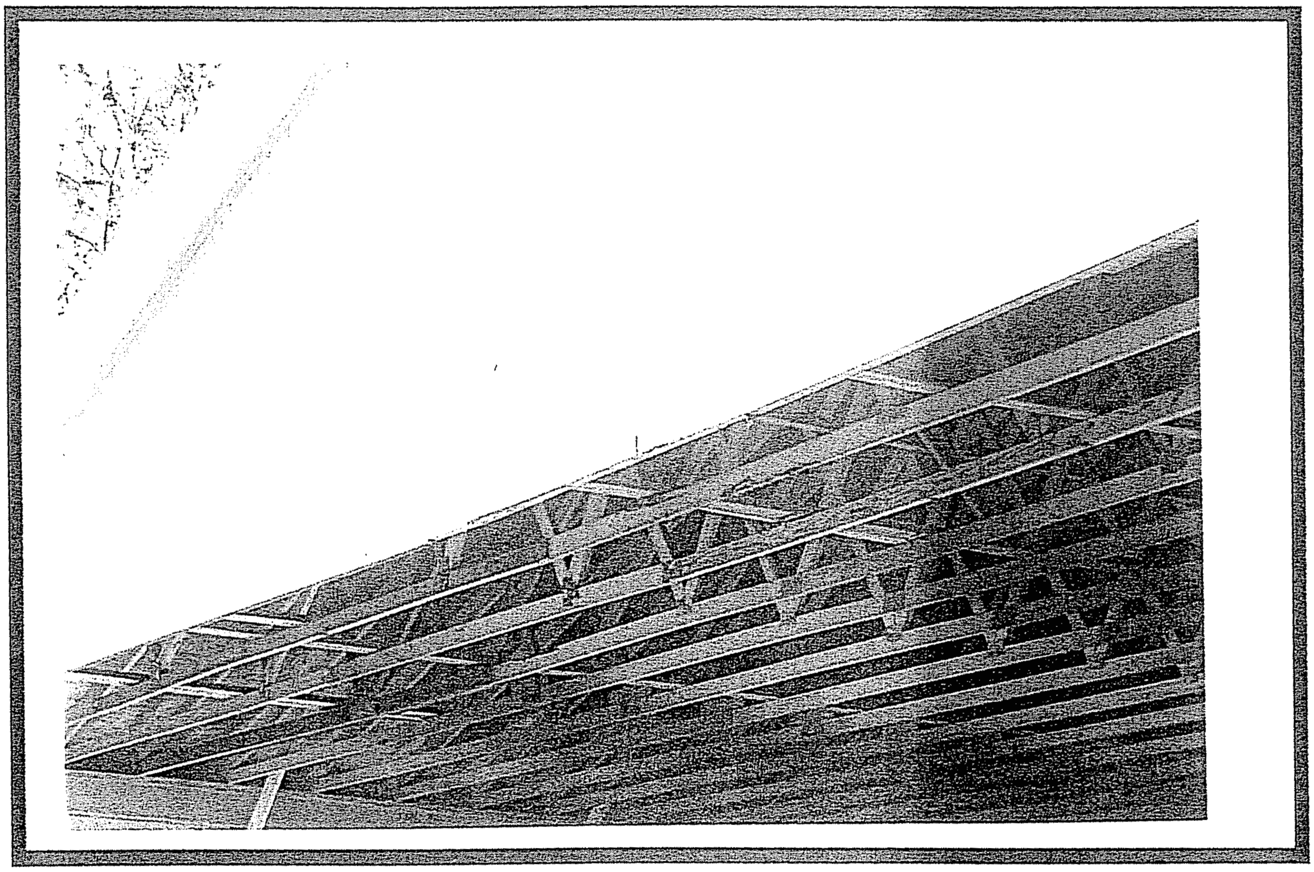
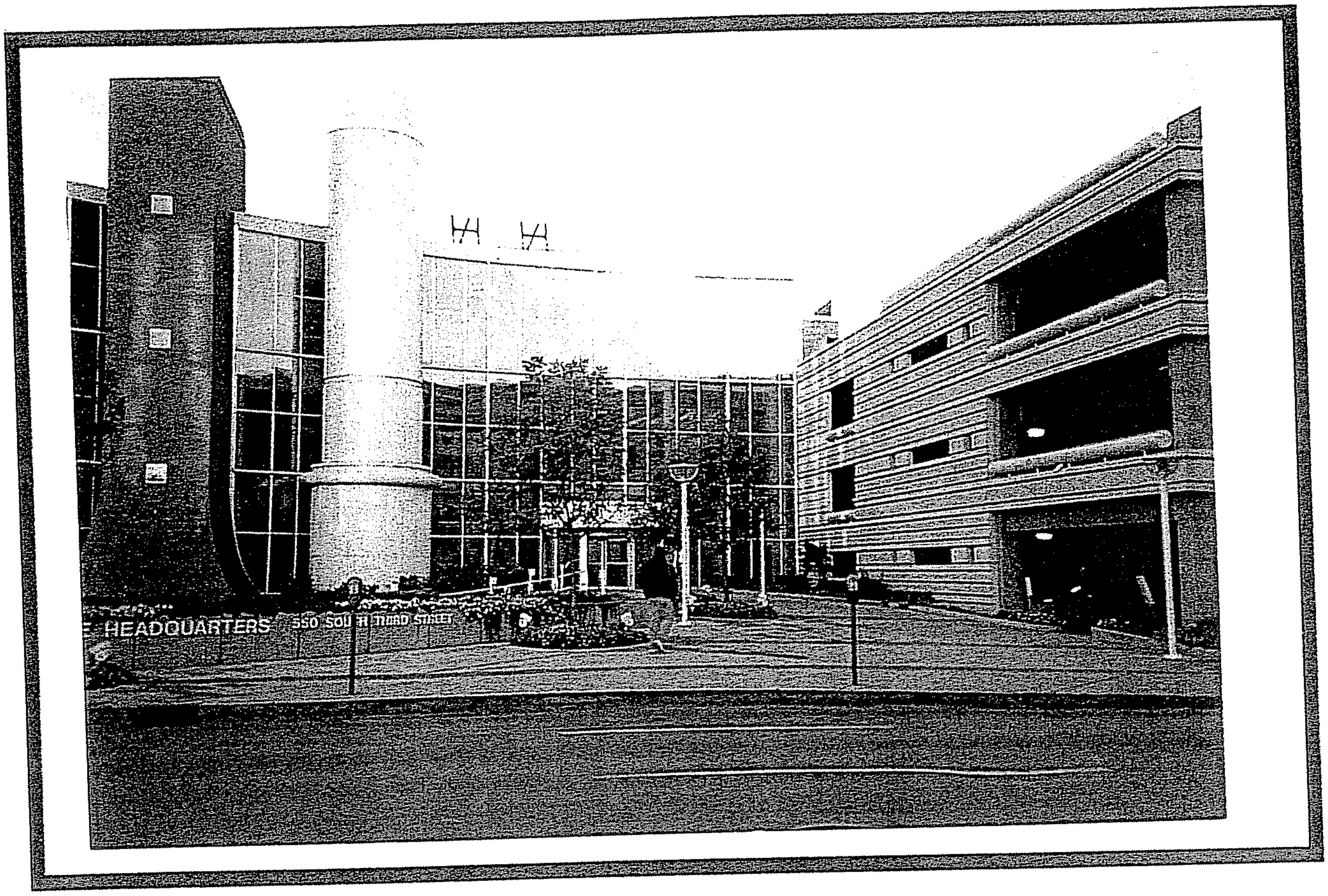

Photograph 34

Photographs 35 & 36 have been omitted



550 SOUTH THIRD OFFICE

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550 South Third Street Facility

The condition of all components of this facility is (Good to Excellent)

Pavement and Site

- The Parking Garage is experiencing water infiltration in a beam on the first level. The cause may be the light fixtures on the upper exposed deck. The conduit from this light fixture may actually run through the beam as a path to the electrical panel. The light fixture should be inspected and sealed.
  See Photo Number 36

- Some irrigation heads are too close to the exterior walls and the walks.

- A wind gauge should be installed on the fountain to prevent wetting pedestrians.
  See Photo Number 37

- The Handrails in the parking garage are oxidizing. They should be primed with a rust inhibitor primer and re-painted.
  See Photo Number 38

- A cover is recommended over the parking garage steps.

- Skate boarders are damaging the rails in the front of the building. We recommend stainless steel ball bearings installed to prevent this activity.

- The striping in the parking deck is fading and will not be visible much longer. We suggest the deck be re-striped in black, as the black traffic paint contains more pigment for longer wear.

Exterior Walls

- The concrete has spalled and exposed the reinforcing steel. This condition exists in several locations on the upper floors. The reinforcing steel must be protected, or additional spalling will occur and metal depletion will begin.
  See Photo Number 39

- Holes are knocked into the concrete block parapet wall on the roof. These holes should be repaired.
  See Photo Number 40

Roof

- Additional protection should be installed to protect the roof base sheet from window washing equipment.
  See Photo Number 41

- Cabling for the window washer stage is wrapped around corners without protection. This will cause a leak.
  See Photo Number 42

- The skylight flashing consists of turn-up base sheet. This condition will fail due to expansion and contraction. A termination bar should be applied to the wall under the skylight and the base sheet counter flashed.

- Bubbles have begun to develop along the base sheet at the skylight. This should fall within the roof warranty.

- There are several bad details in the flashing, especially at the turndowns where the metal is not continuous. If this condition is not leaking, it will.
  See Photo Number 43

- Depending on the warranty on the roofing system, the roof may be a good candidate for a management plan

Glazing Units
- There is evidence of previous leaks around some of the window units. However, as the building is still in warranty the contractor is repairing them as they are discovered. We recommend someone determine why the failures occurred to prevent the reoccurrence after the warranty period.

Structural
- The front retaining foundation wall is leaking. The infiltration is occurring at the wall and foundation intersection as well as mid span of the wall. An invasive discovery effort is required to both determine the cause and effect the repairs.
  See Photo Number 45

Building Interiors
- See Photo Numbers 46, 47,48,49,& 50

- Several previous window leaks have resulted in stained drywall and ceiling tiles.
  See Photo Numbers 51, & 52

- Outside corner protection strips should be installed to protect the walls.
  See Photo Number 53

- The drywall tape has failed at a previous window leak.
  See Photo Number 54

- The drywall was not terminated correctly at several window headers.
  See Photo Number 55

Mechanical Systems
- The mechanical system is new and in warranty. The only concern is the amount of outside air introduced into the system. The intake on the roof does not appear to be large enough to provide the 20cfms recommended by ASHRE. An analysis should be performed to ensure adequate IAQ.

Plumbing

- No deficiencies observed.

Electrical

- The electrical system distribution is very good, as 75 KVA transformers are installed on each floor. This will provide over 8 watts per sq. ft. of 120/208-office use power.

- The lightning protection electro-bar is not continuous and will not provide adequate protection. The interruption in the bar will disrupt the grounding of the system.
  See Photo Number 44

ADA Compliance

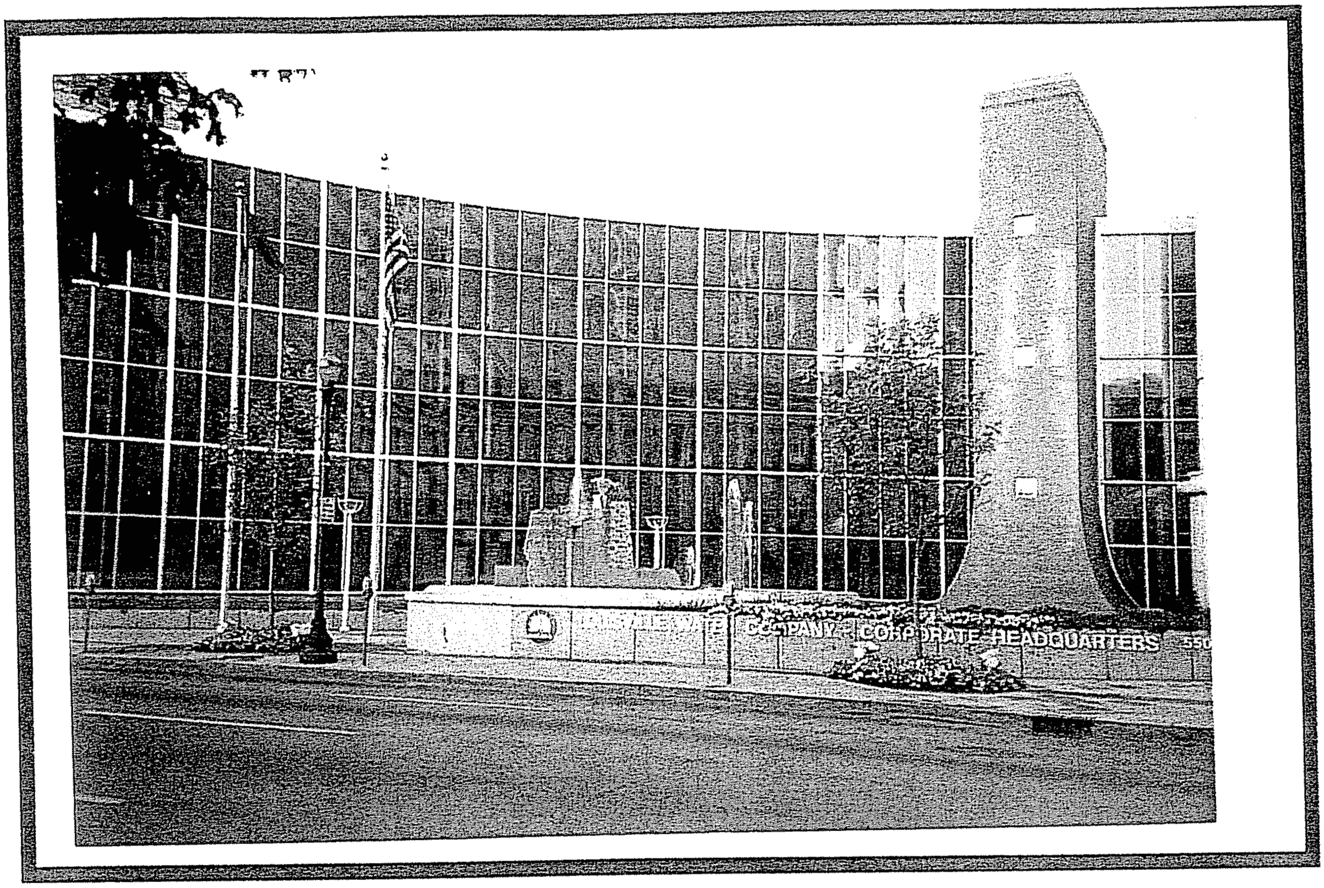
- The building is ADA compliant in most cases. However, the telephone in the elevators is not a hands free. This does not comply.
  See Photo Number 56

- The assembly room in the basement does not have horns and strobes.

Life Safety

- As noted in previous facilities lock-out/tag-out, and hazardous communications programs are not effectively being used.

- The fire escape stairs terminate into a dead end in the basement. A one way traffic gate must be installed on the first floor to prevent the continuation into the basement.



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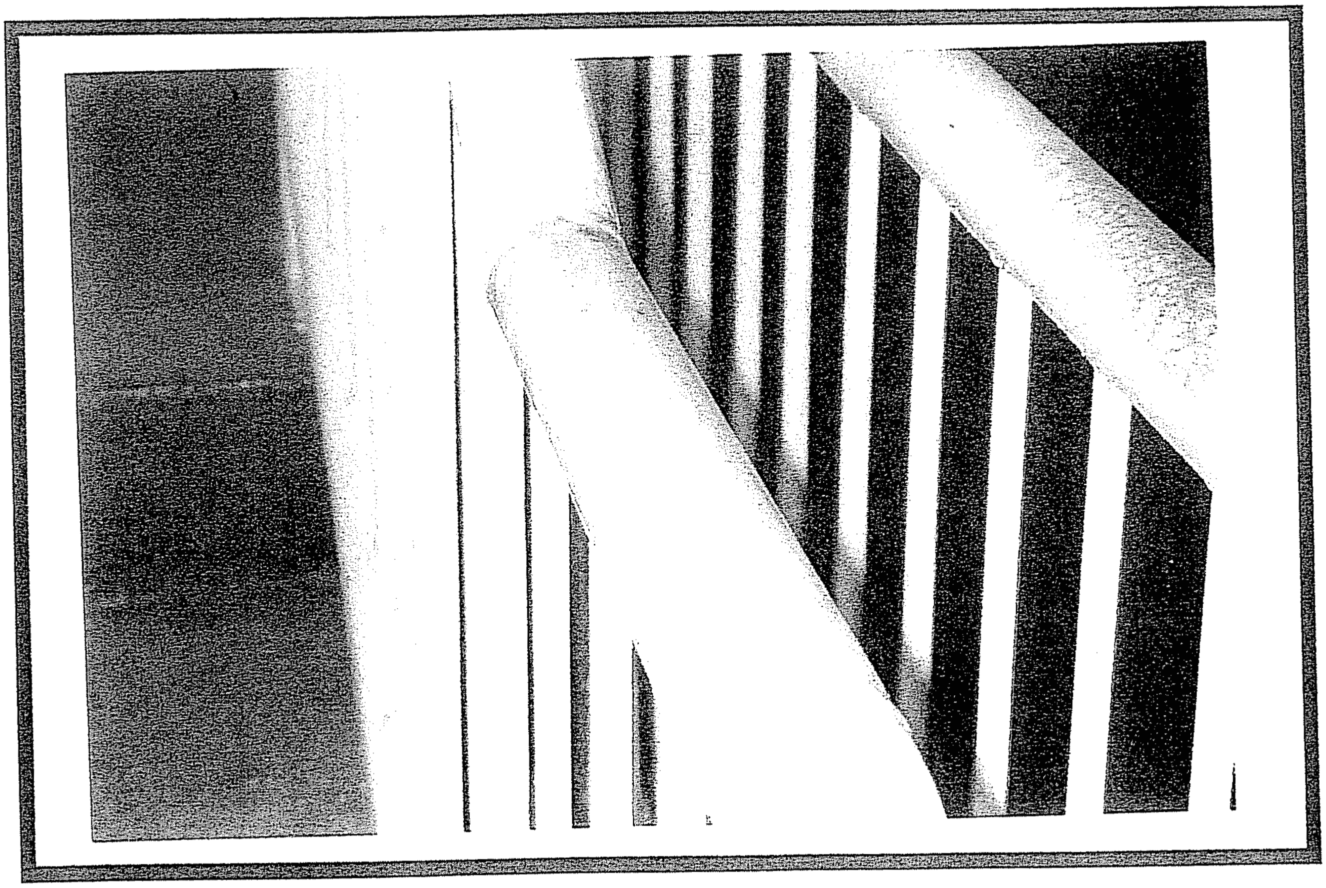

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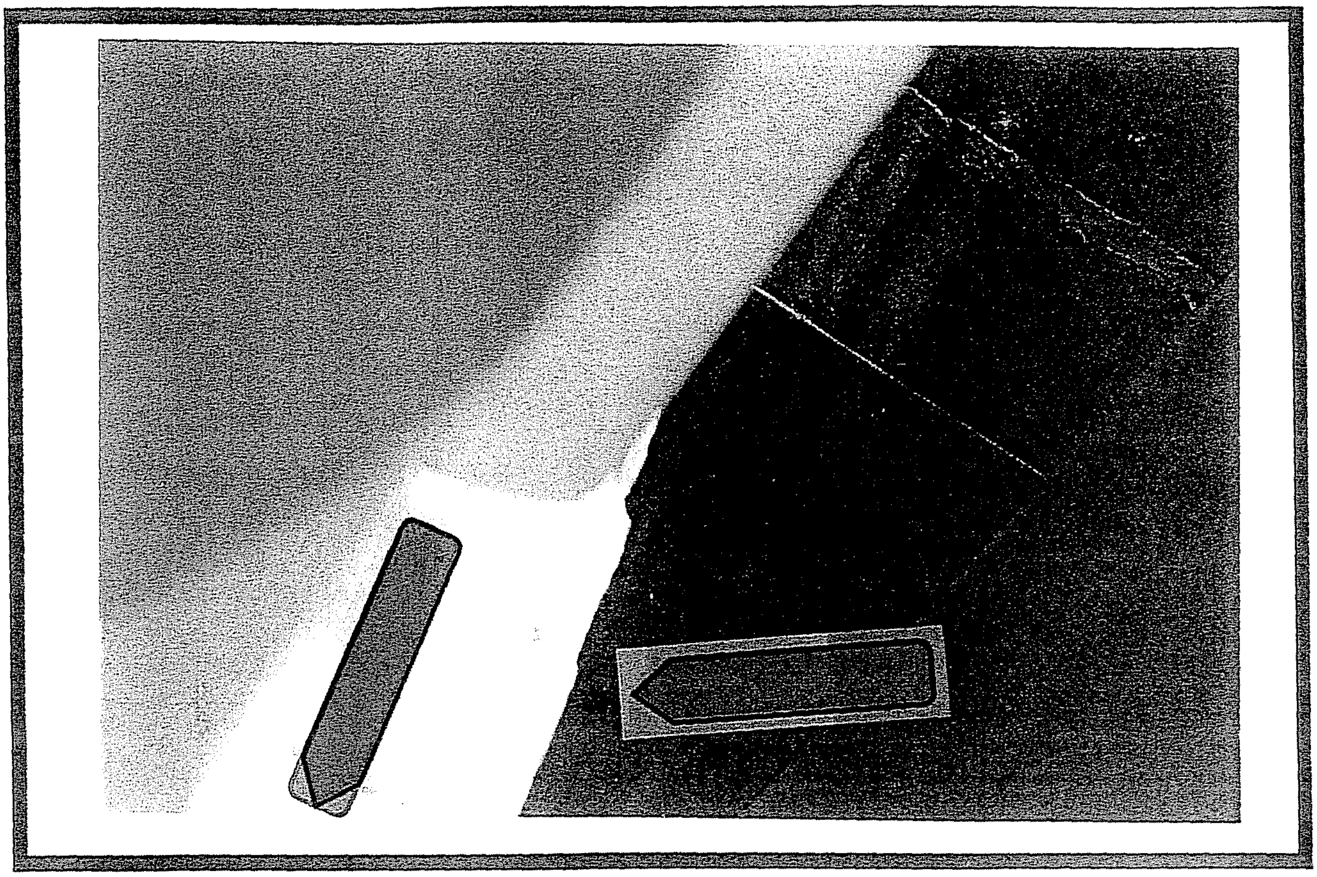

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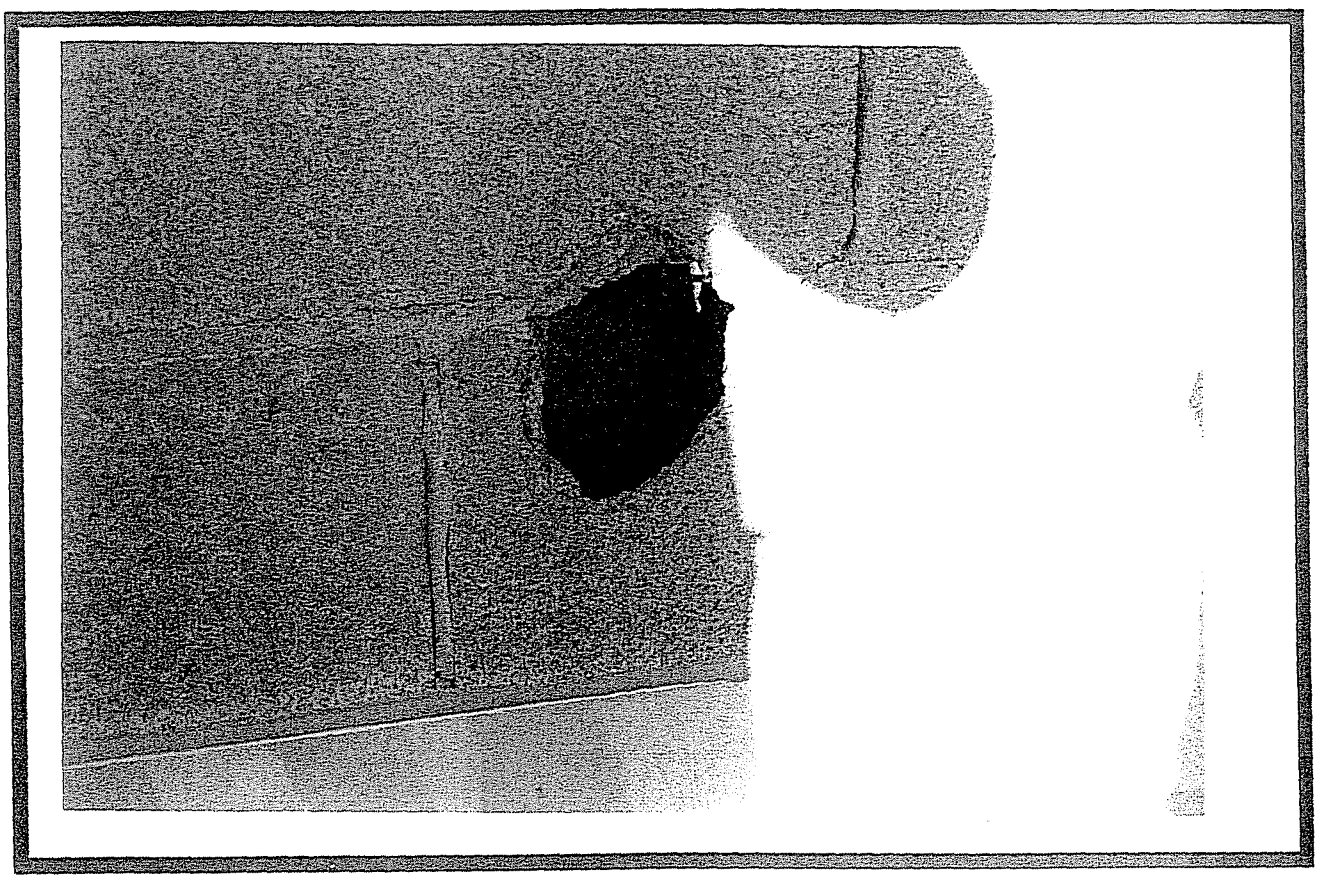

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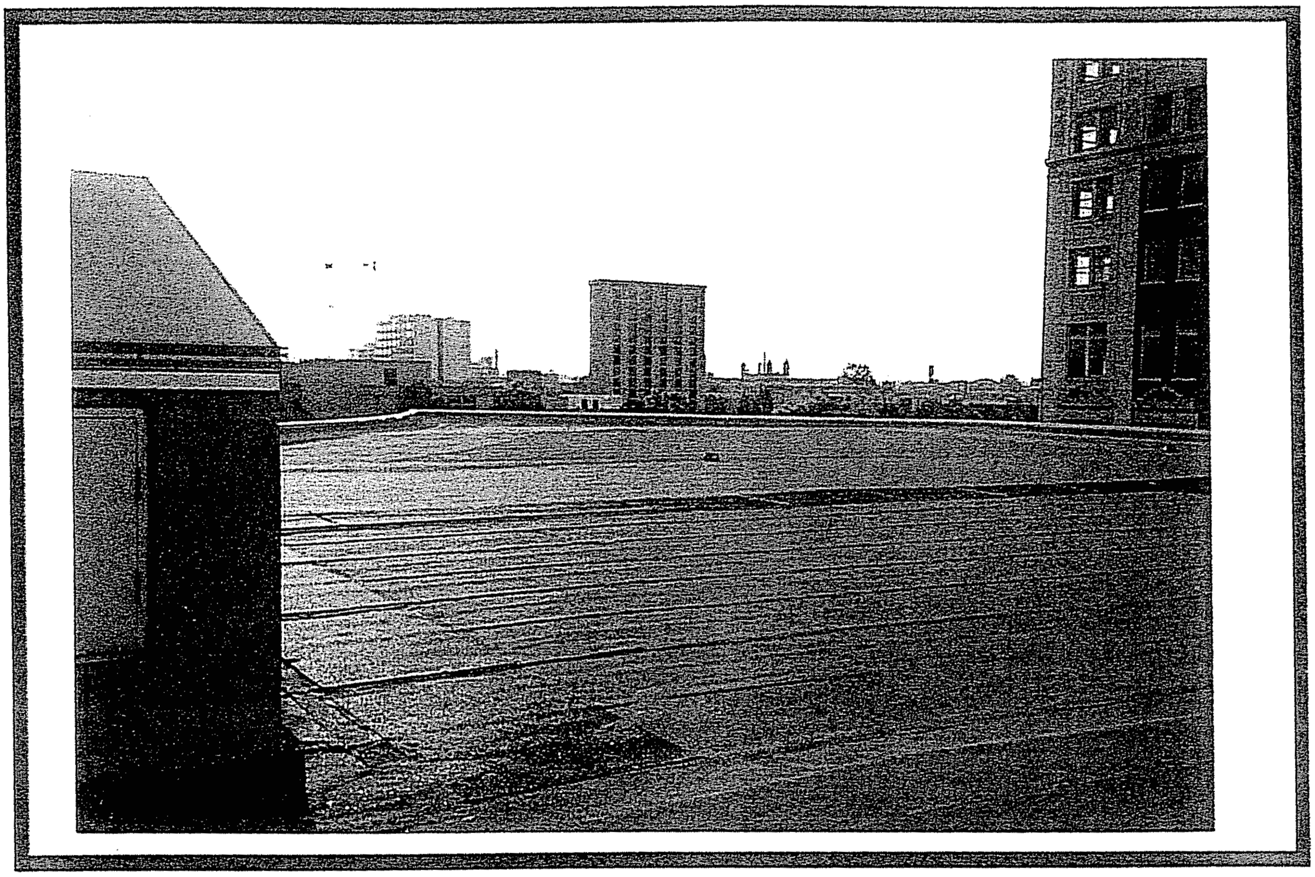

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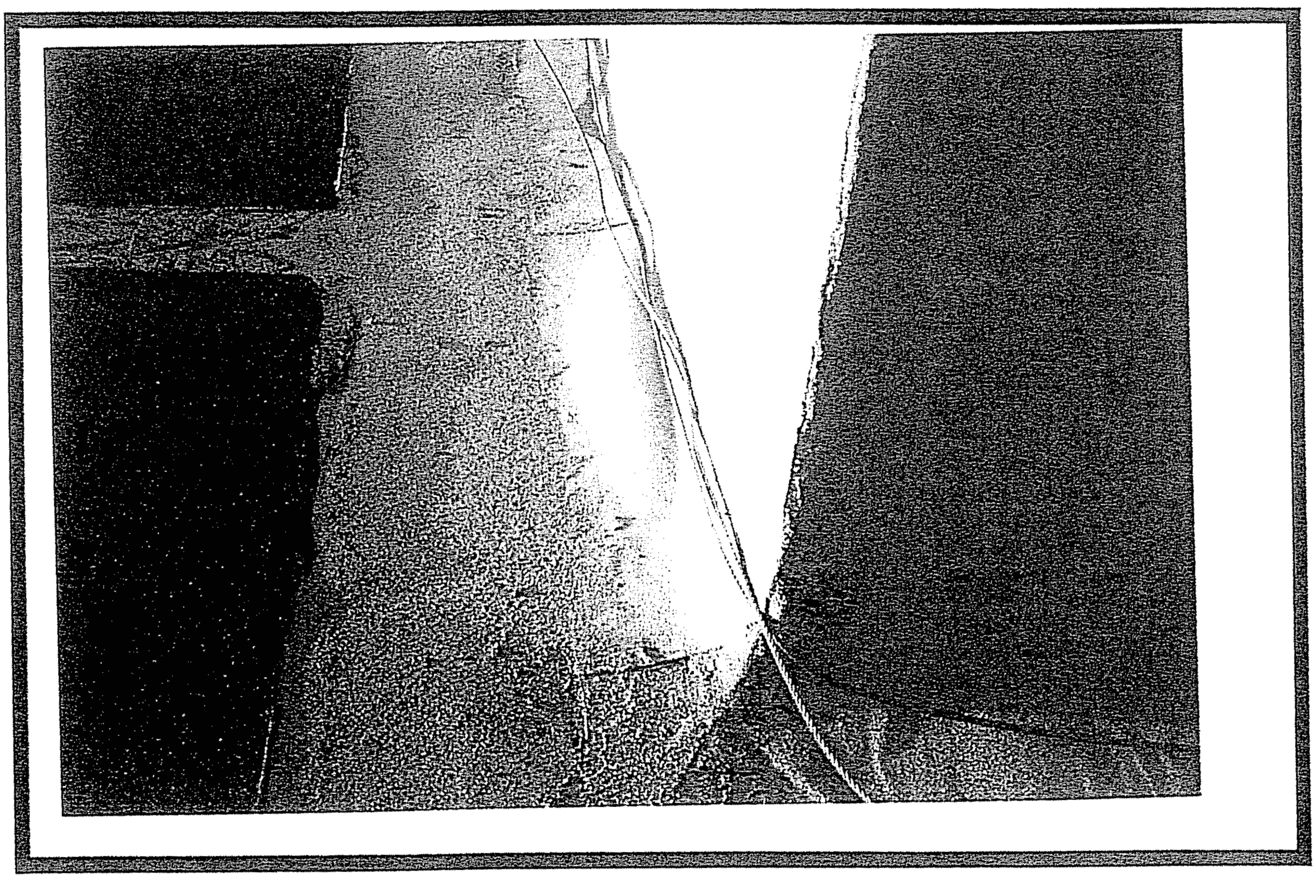

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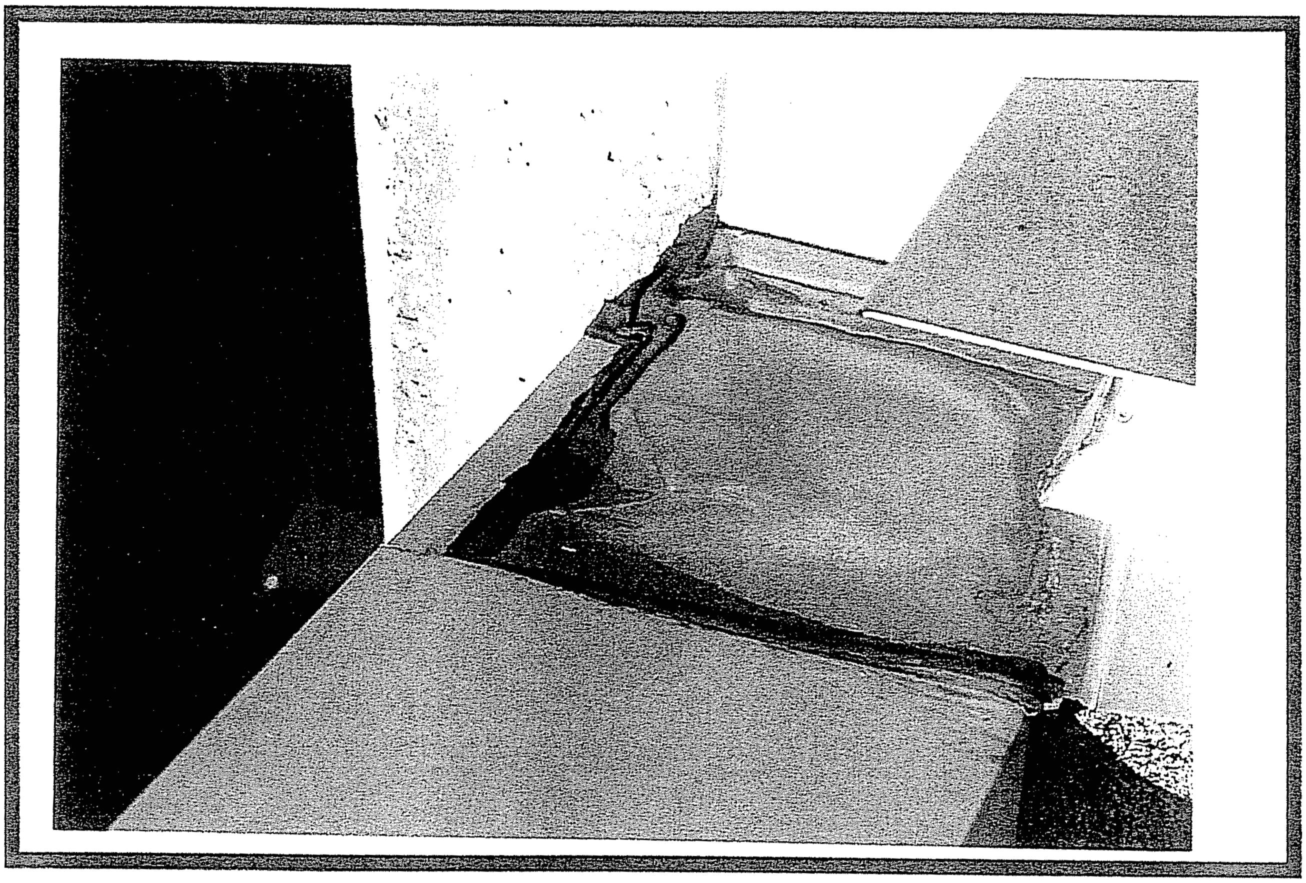

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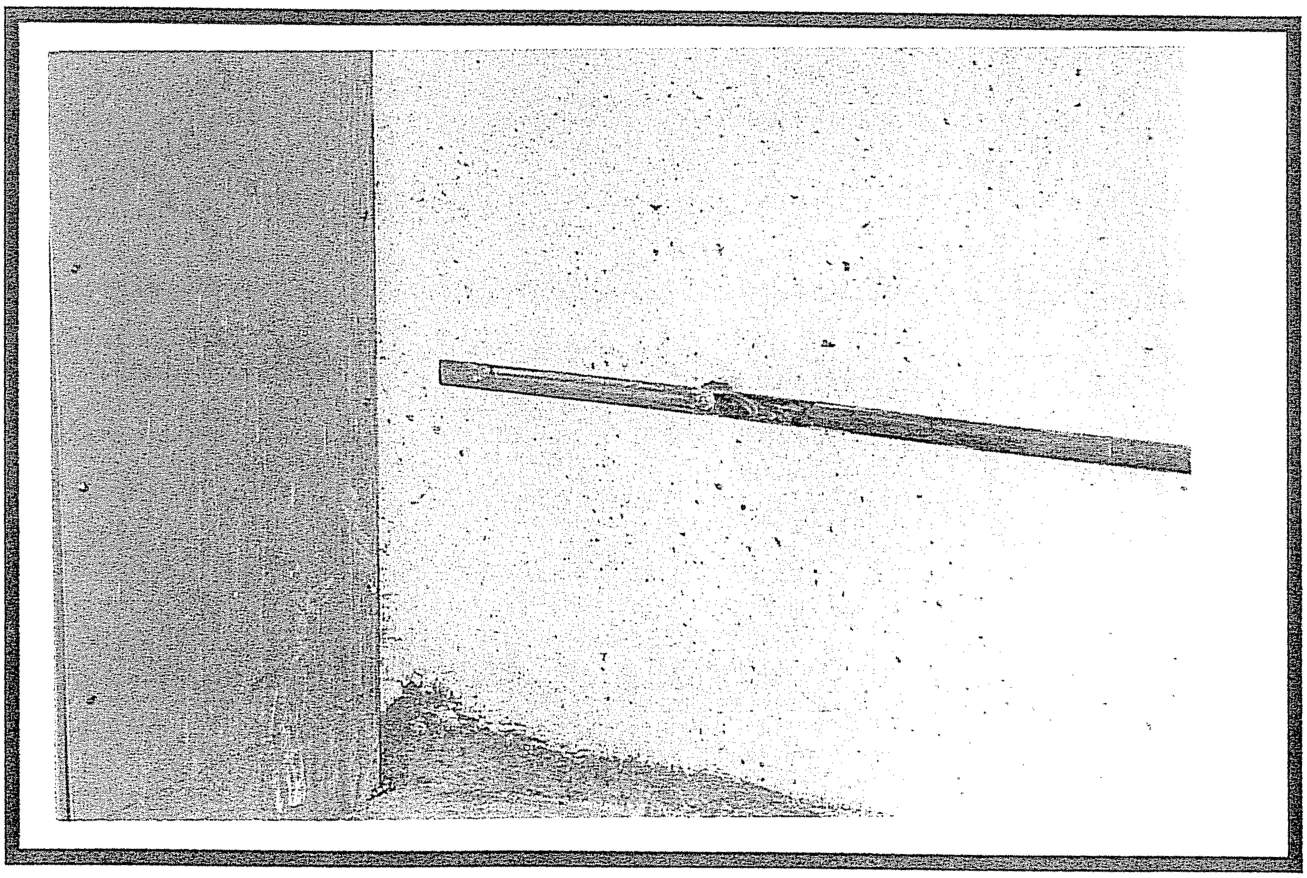

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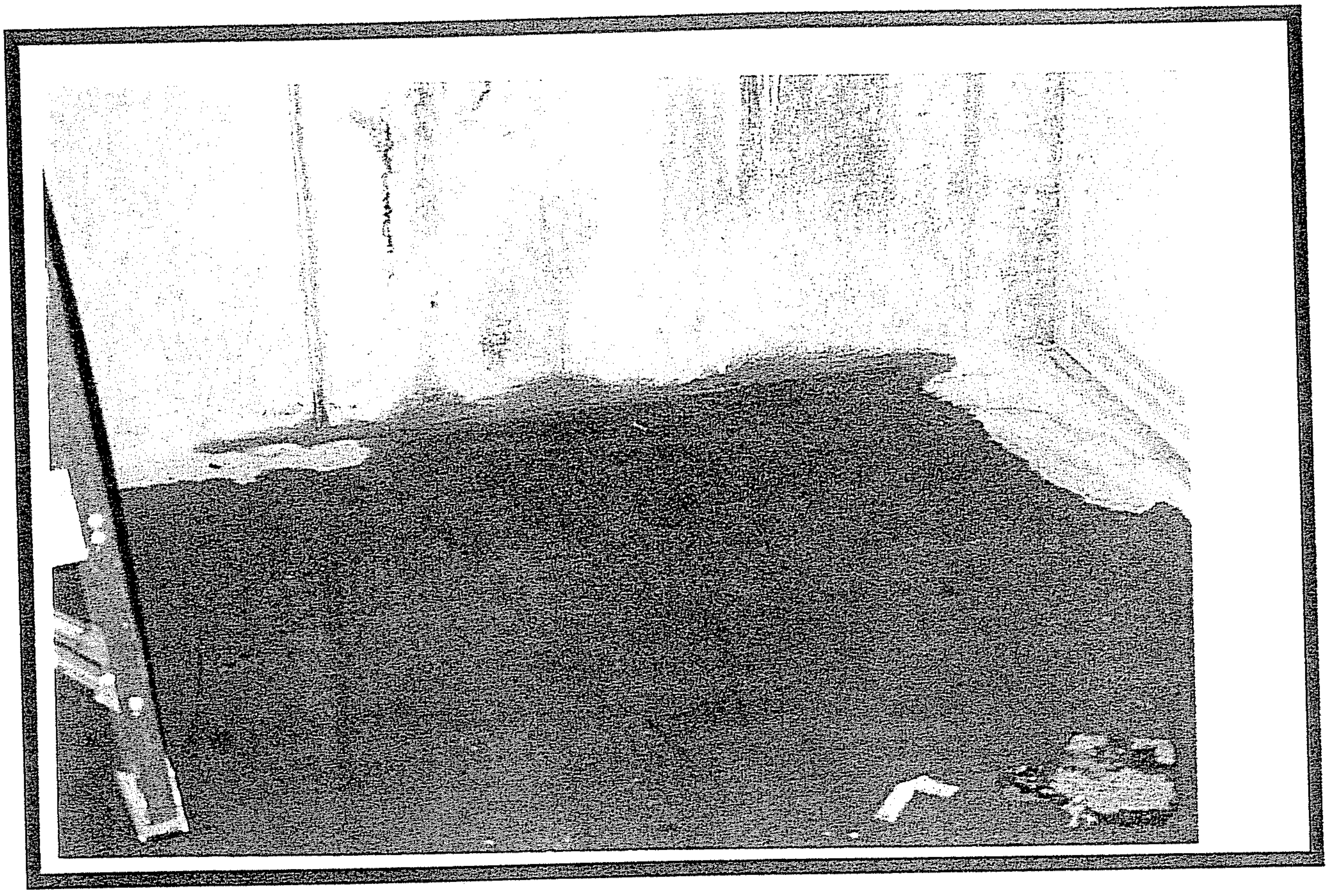
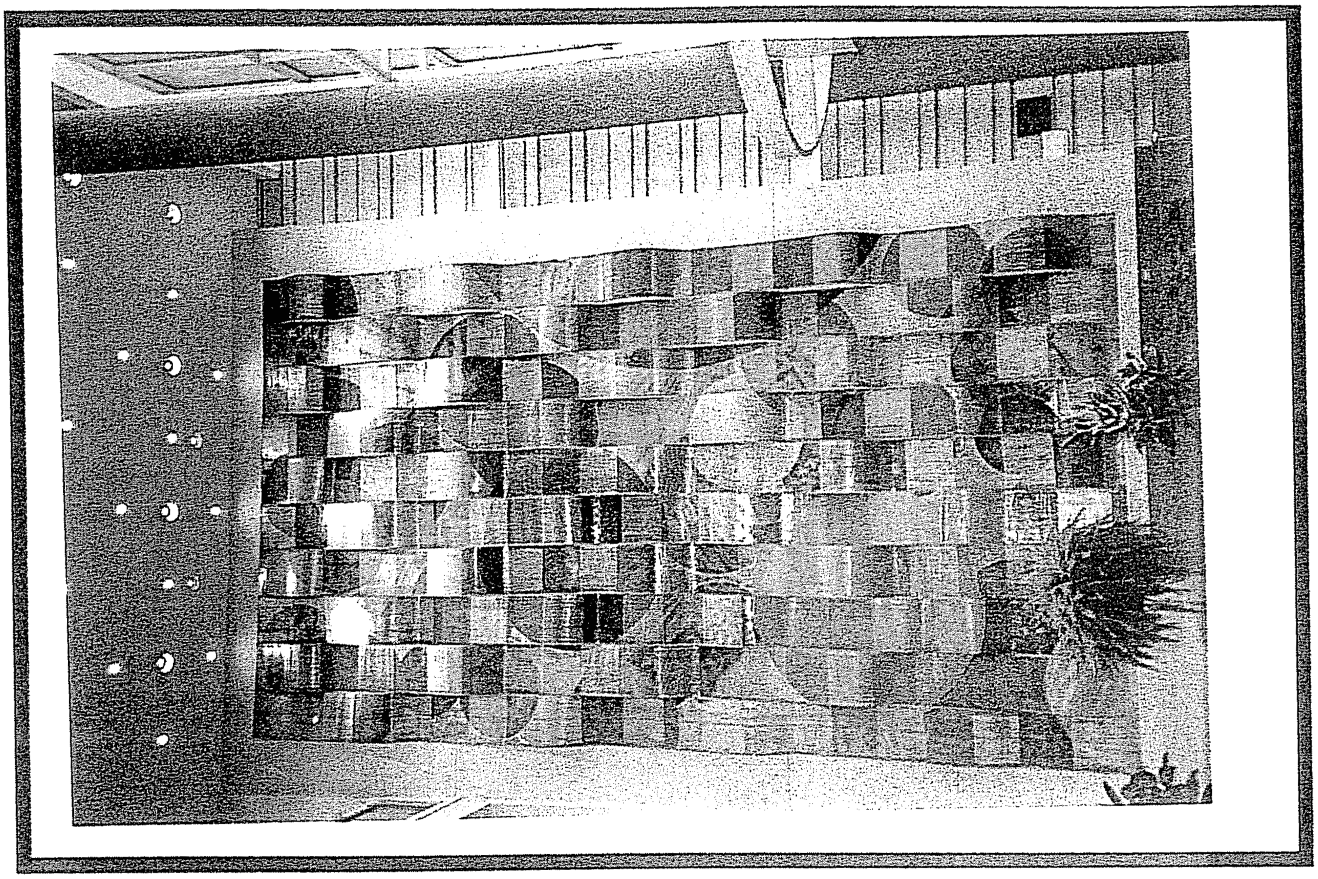


Photograph 45

Photograph 46



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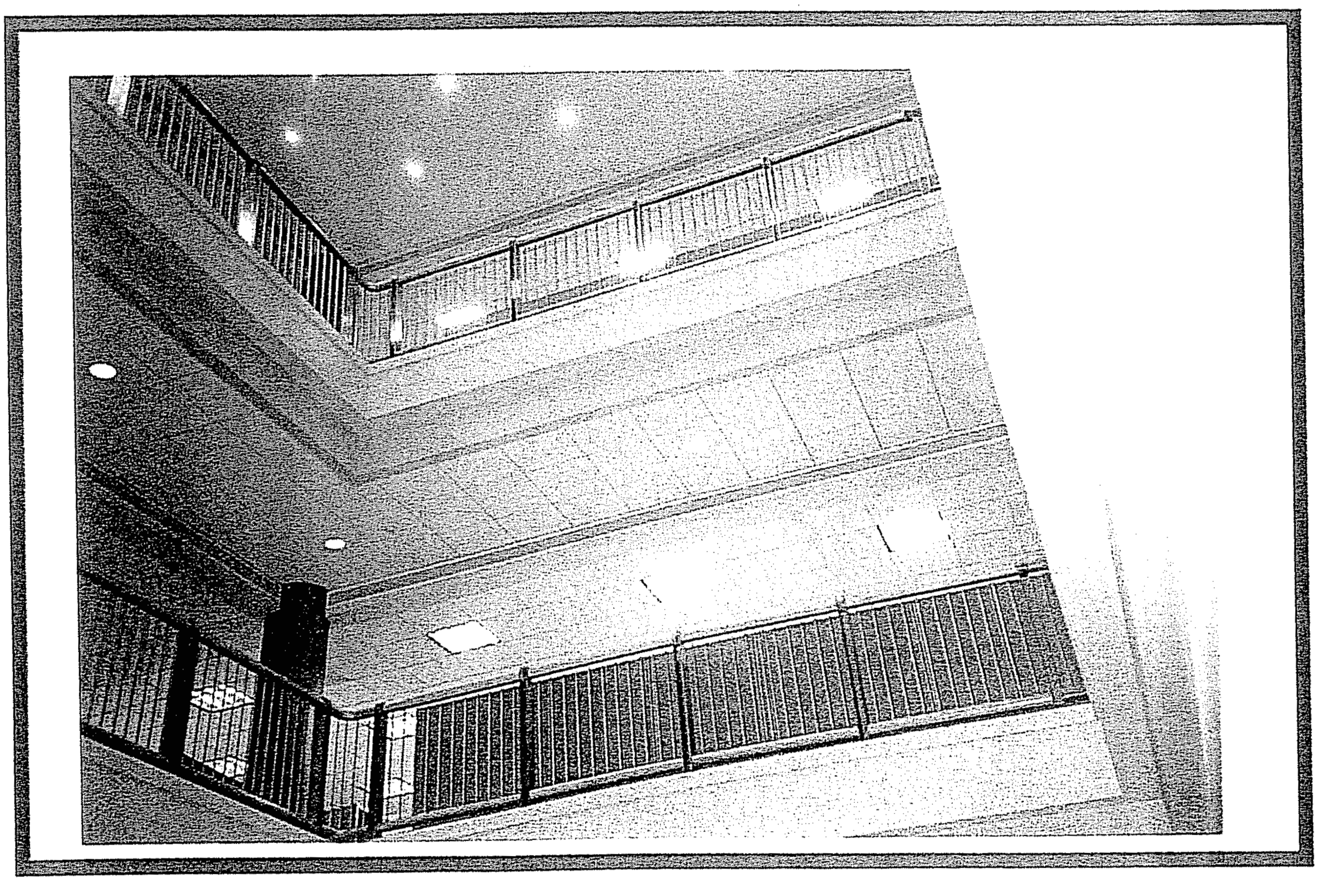

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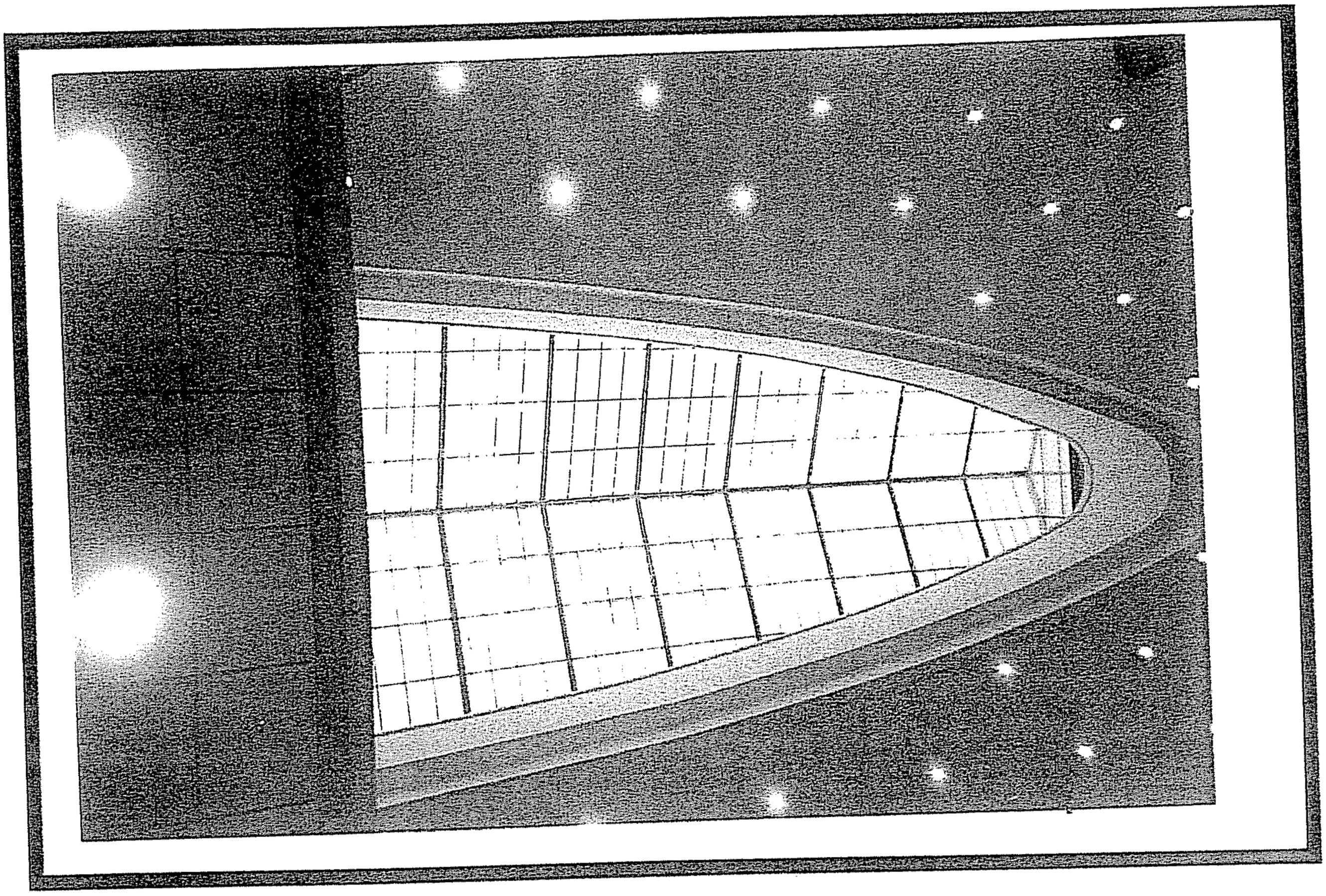

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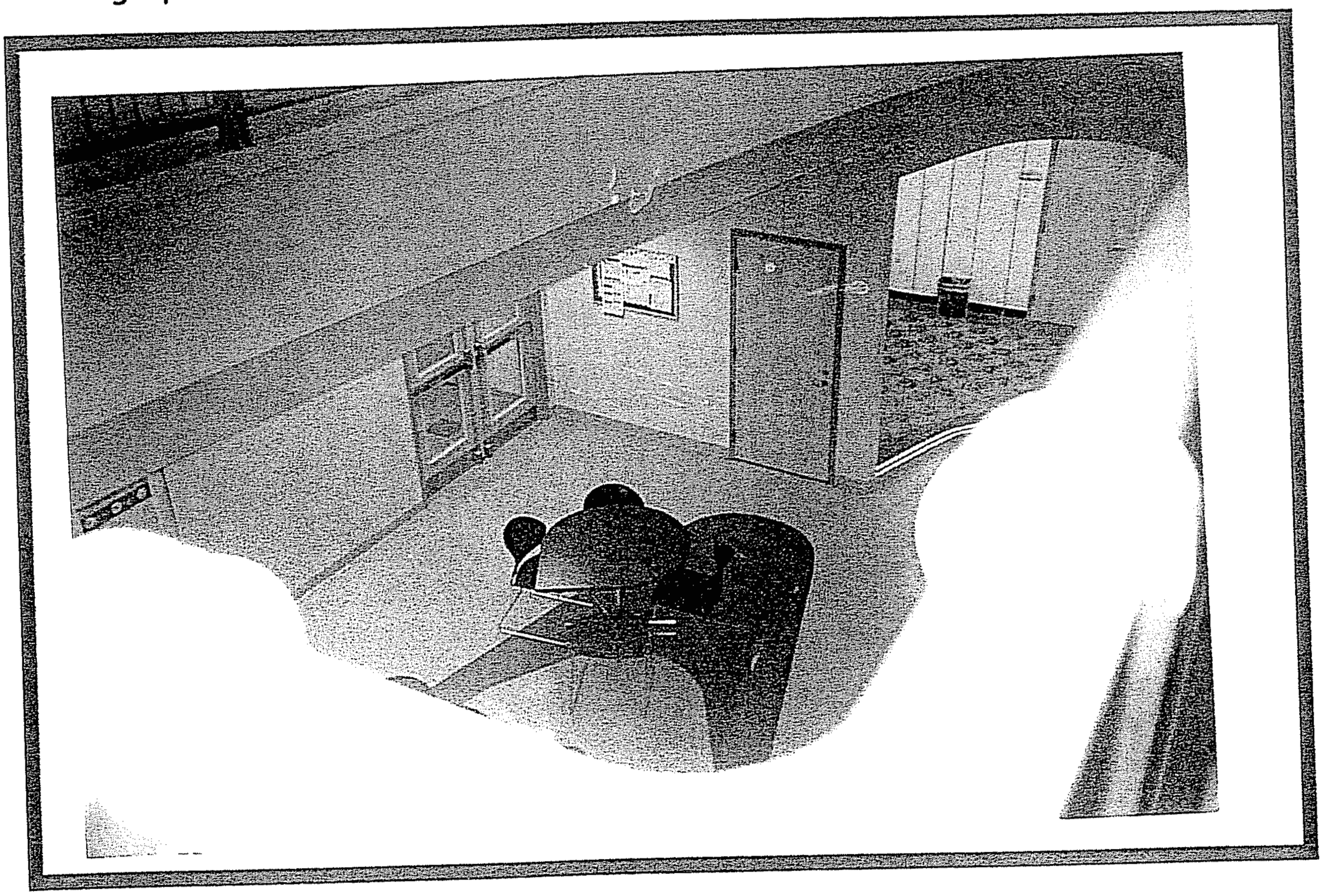

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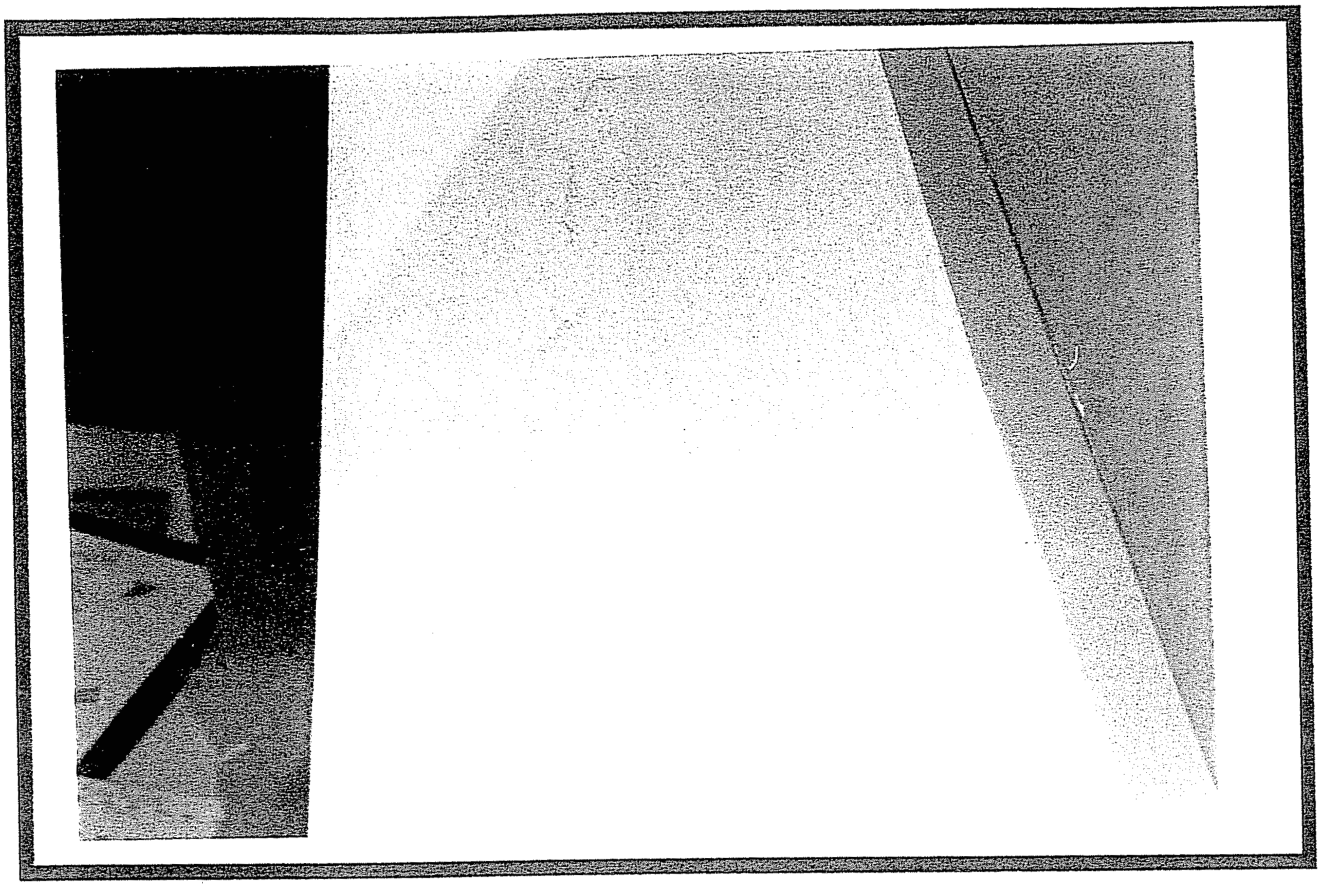

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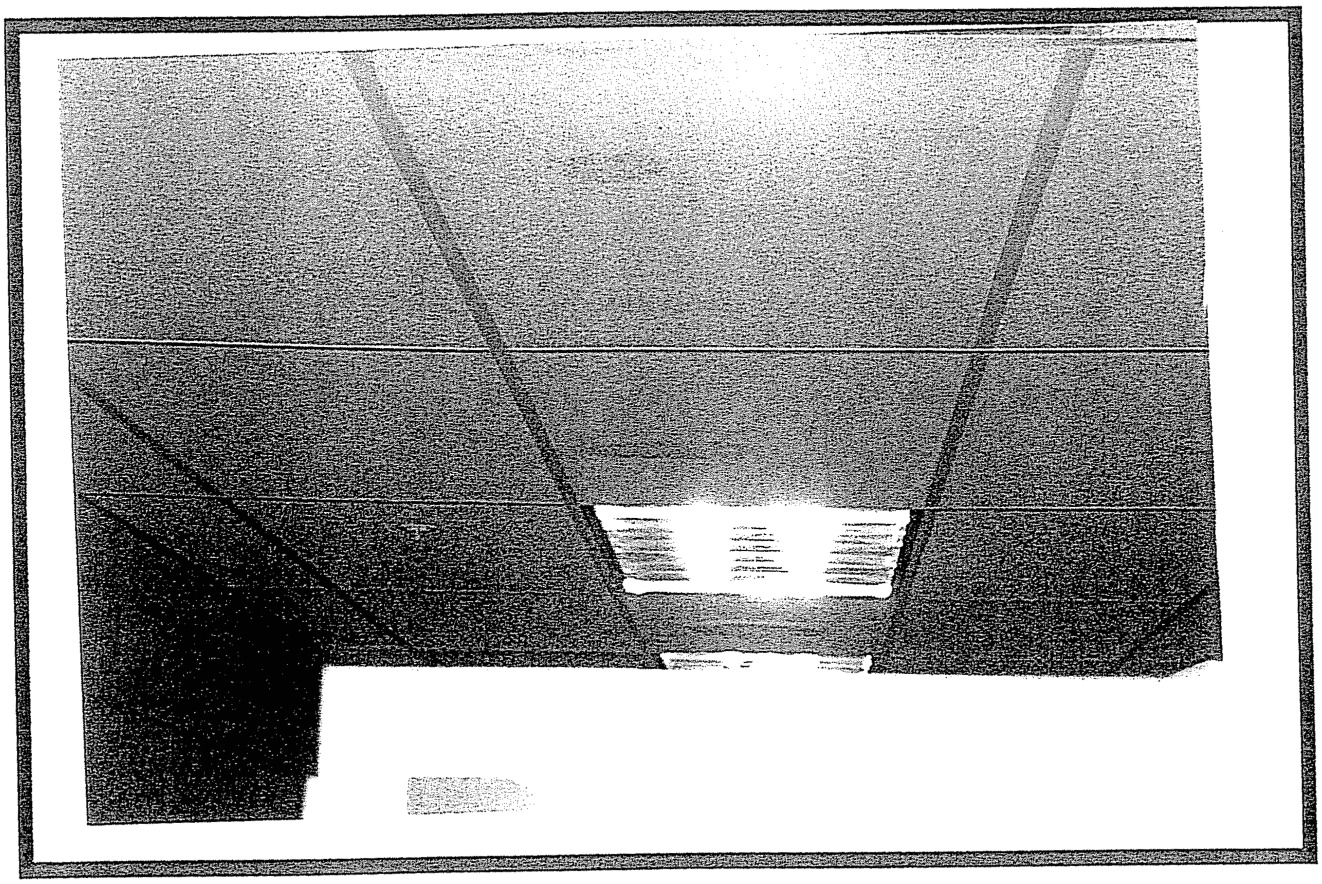

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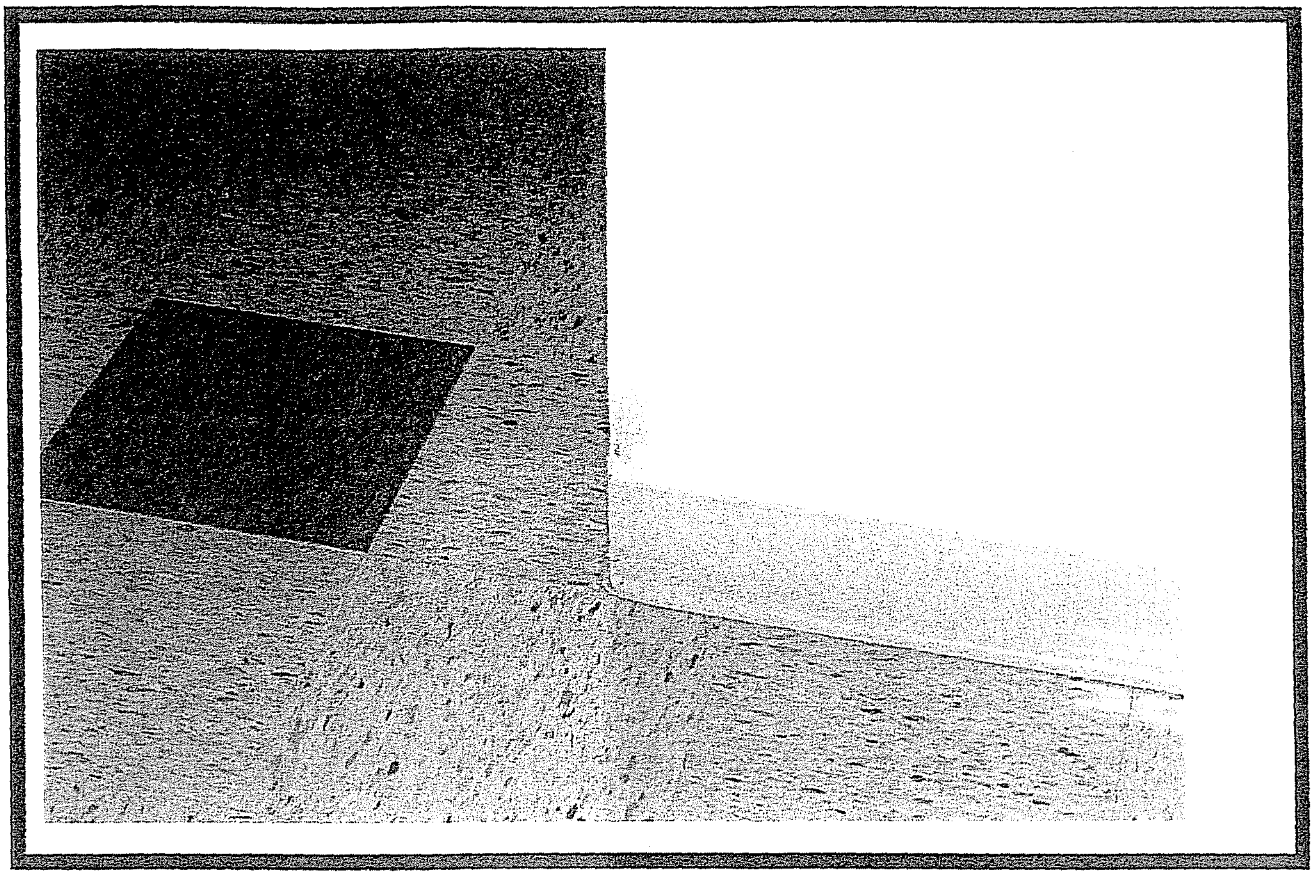

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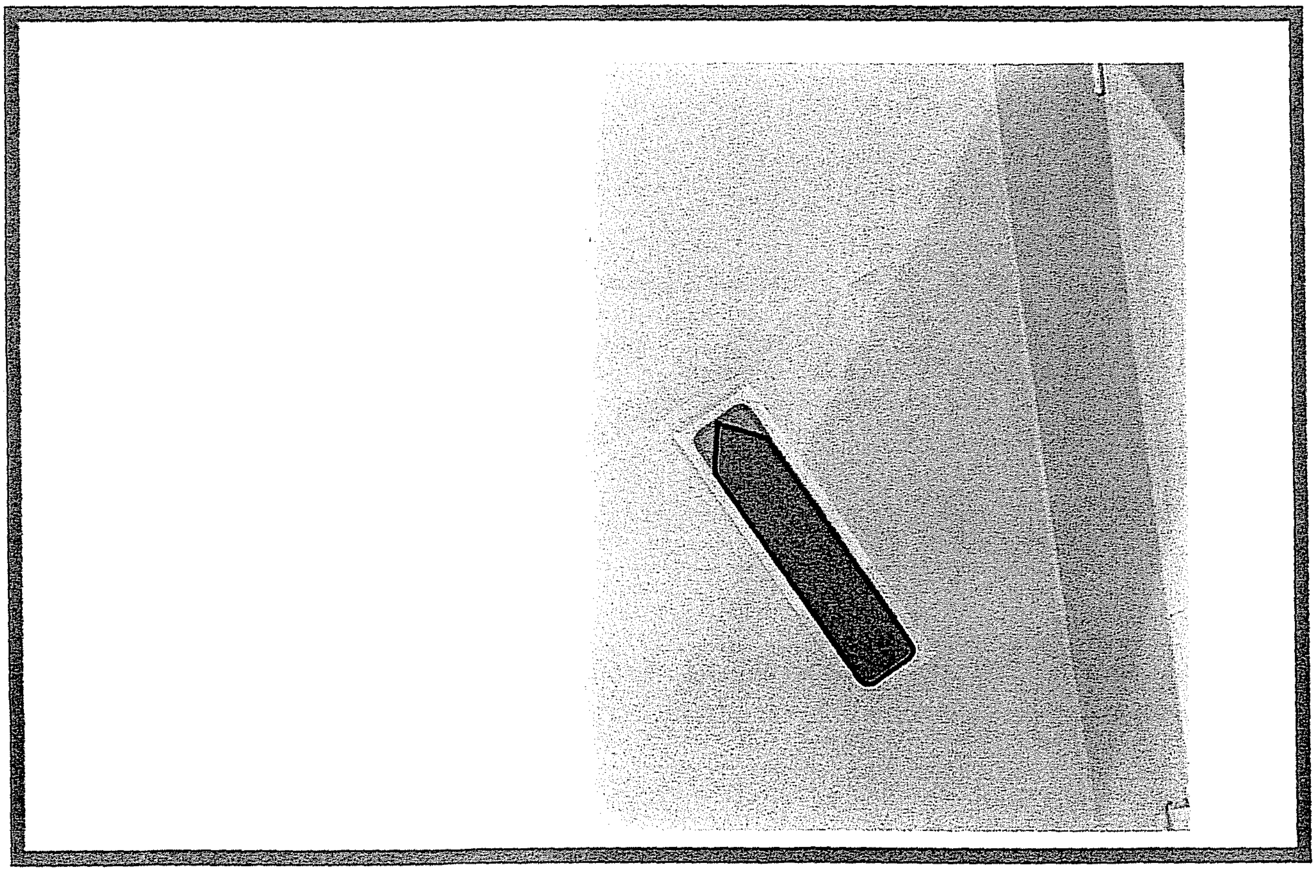

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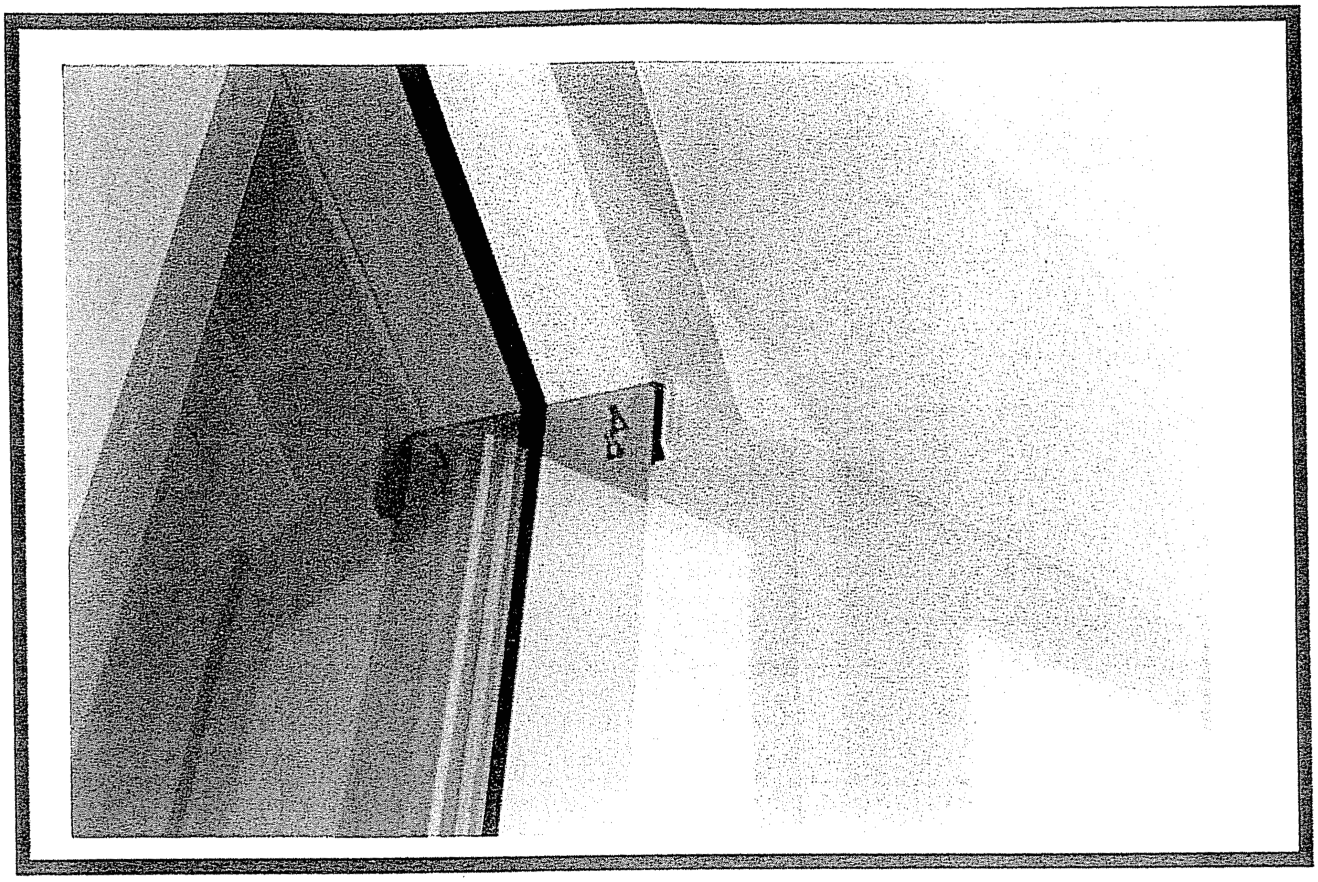
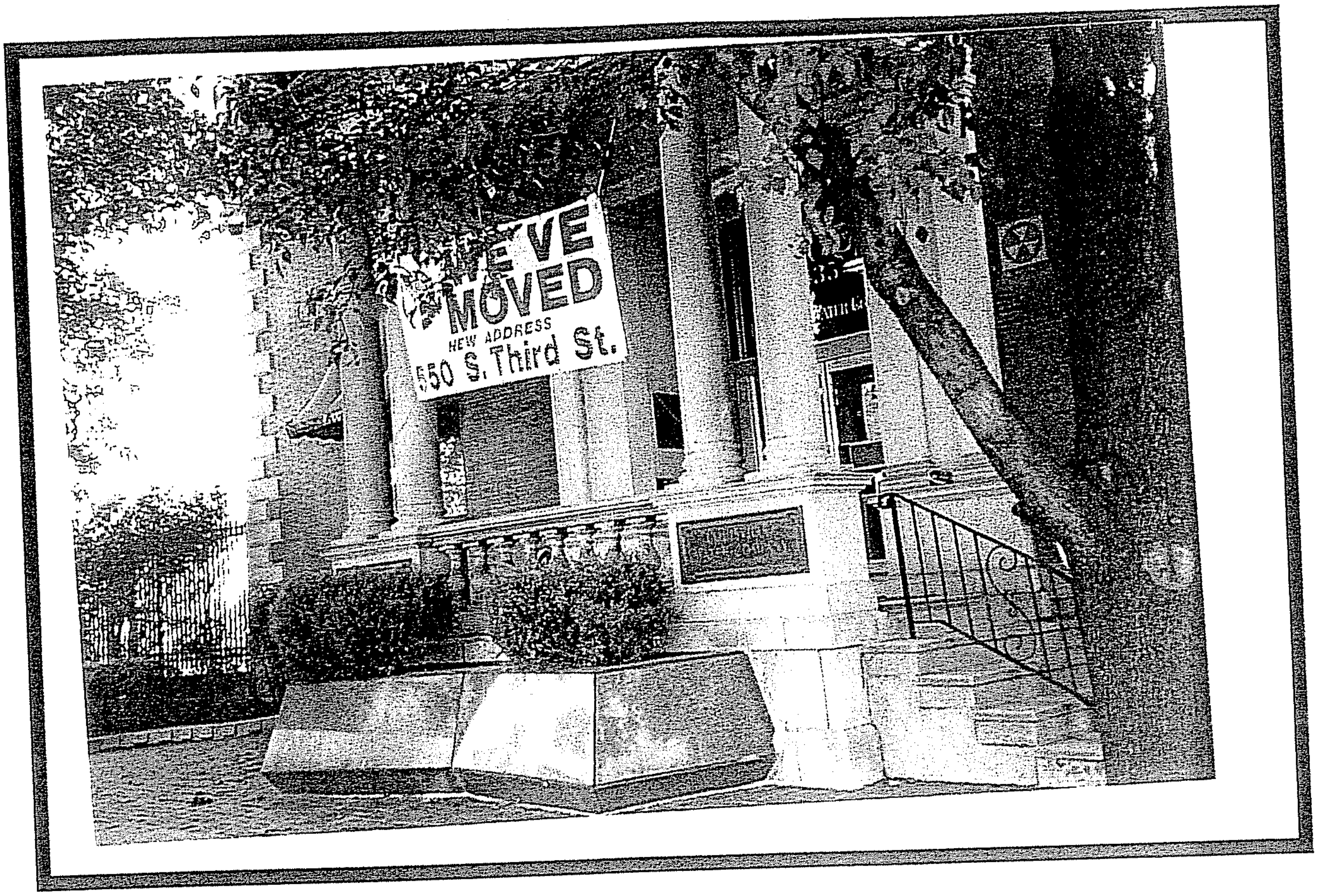


Photograph 55

Photograph 56

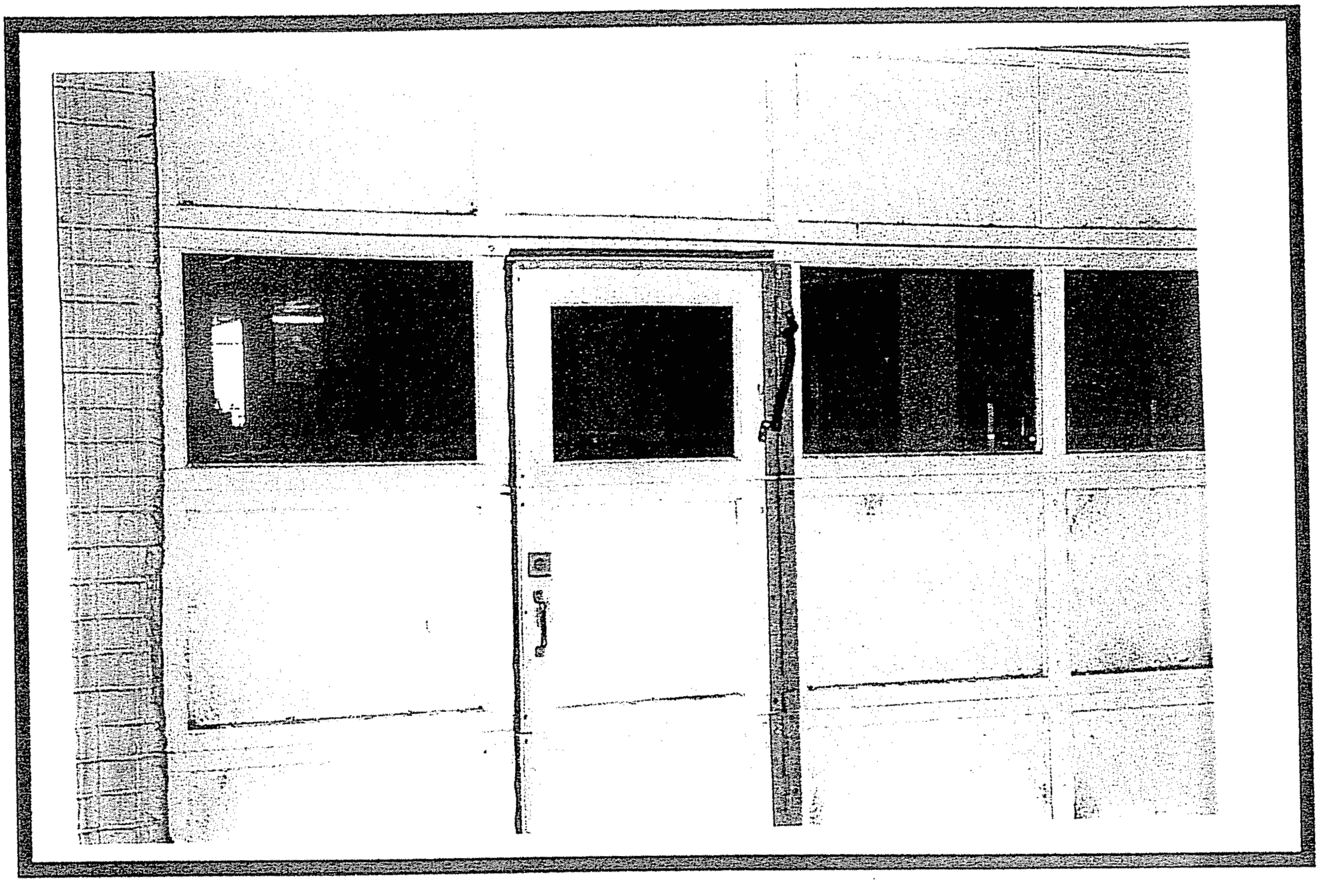


435 SOUTH THIRD FACILITY

435 South Third Street Facility

This is a vacant facility. The purpose of the survey was to determine the action required securing the buildings and preventing further decay of the structure.

- All electrical except that required to run the heating system should be cut off at the main breaker.
- The roof top units not in use should be covered with plastic to prevent water infiltration.
- All sprinkler pipes should be wrapped in electric tape.
- All water, except the sprinkler system, should be shut down and the lines drained.
- The heating system should be set back to 50 degrees.
- The roof should be placed under a roof management plan. The current leaks should be repaired to avoid decay of the roof and flooring structure.
- Install iron gates on the doors and windows.
  See Photo Numbers 57, 58, 59, & 60
- Install razor wire on top of fence both front, rear and side parking lots.
- The exterior gothic fascia is in poor condition. If it is the intent to save this structure in its current style, immediate restoration efforts must occur.
  See Photo Numbers 61, 62, 63, & 64



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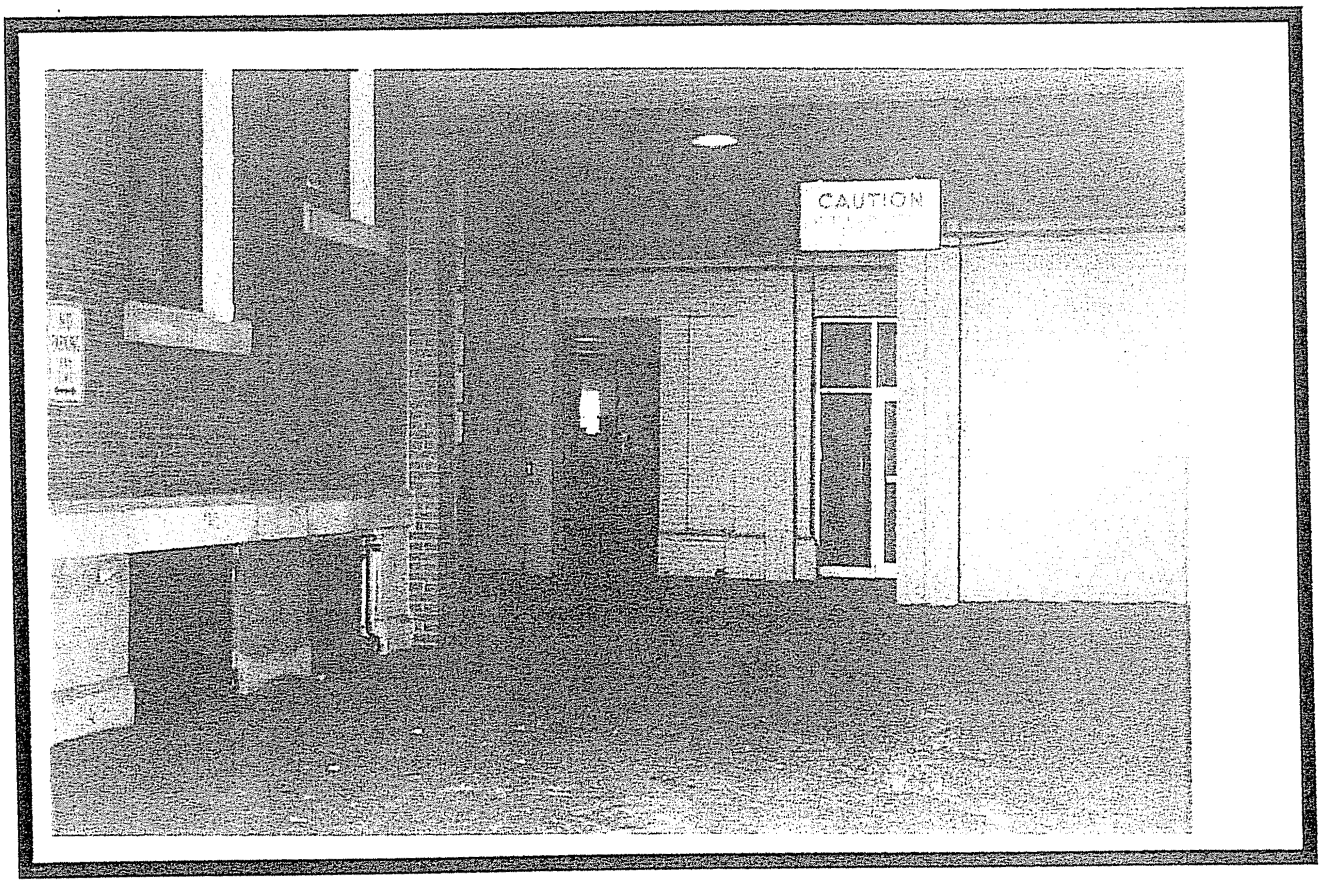
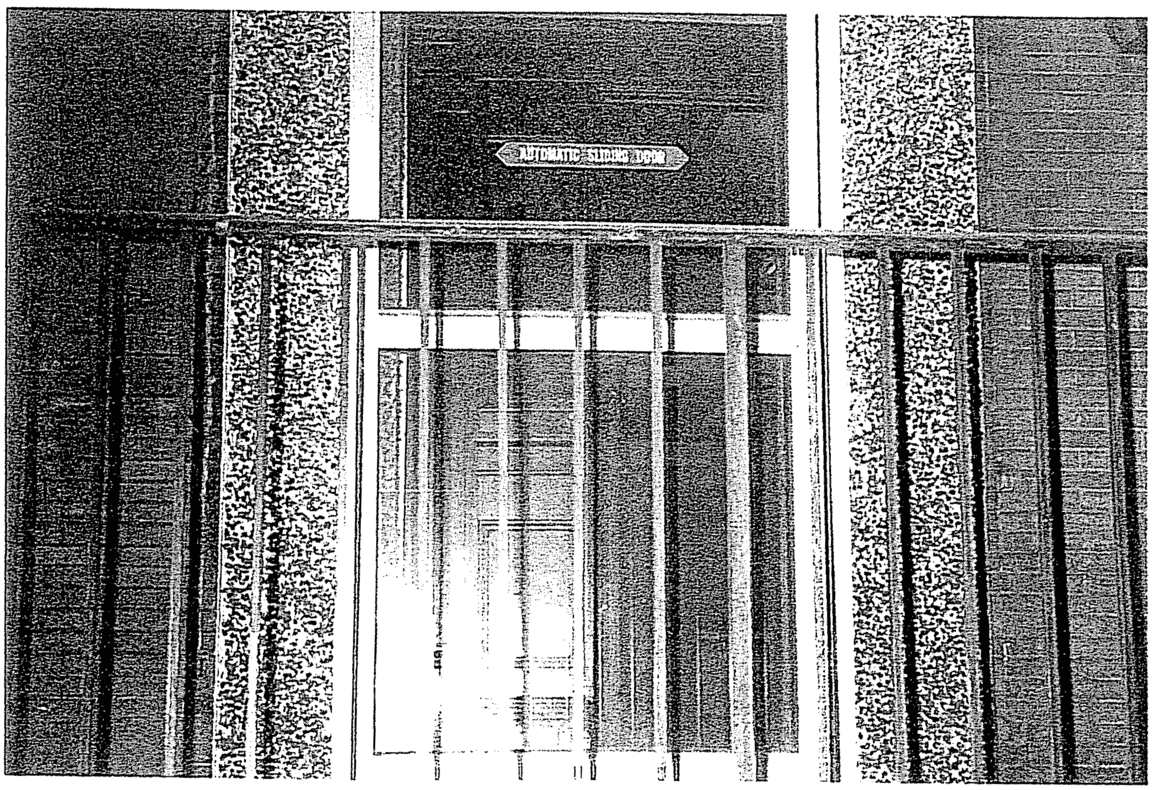


Photograph 58

Photograph 59



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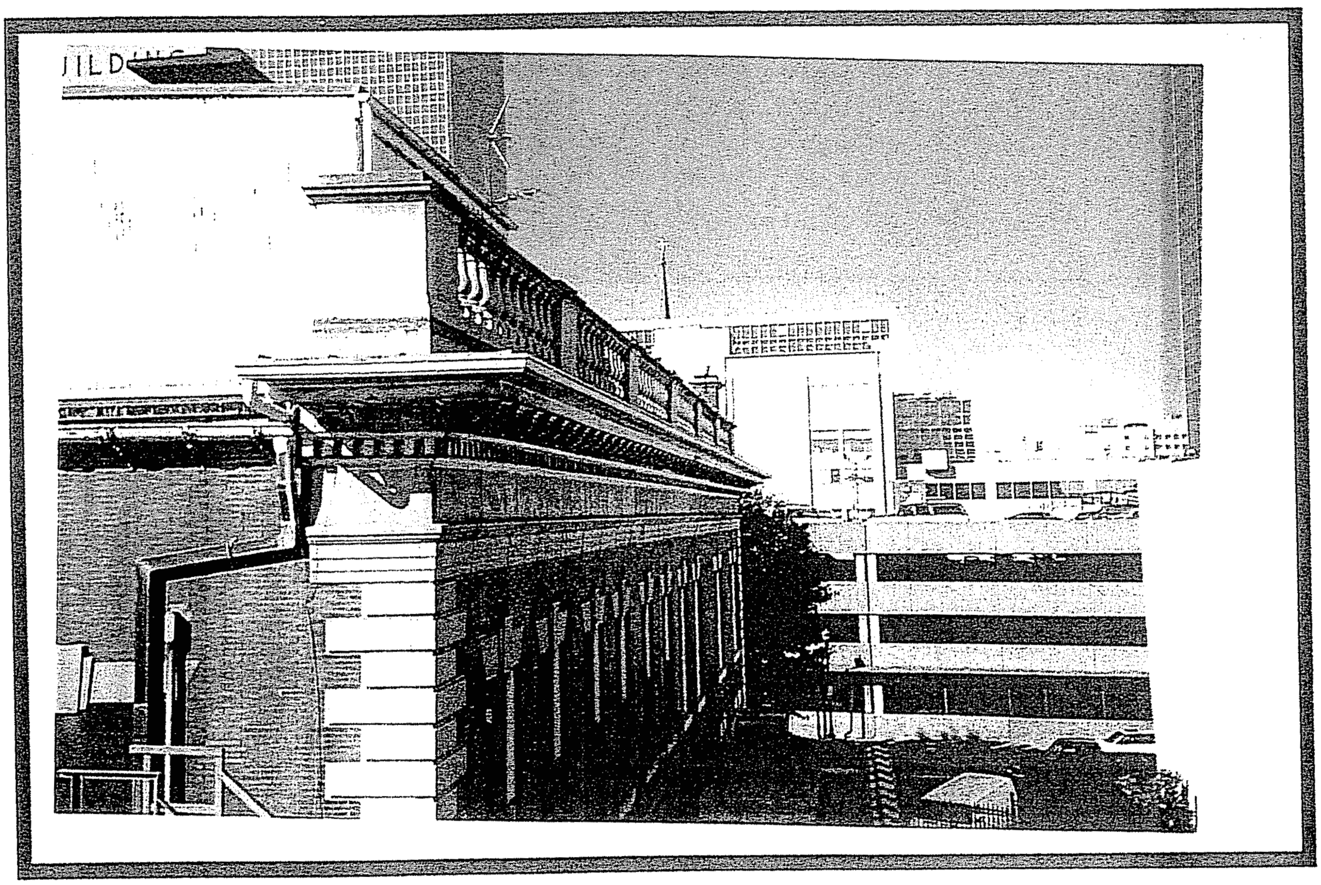
Photograph 61



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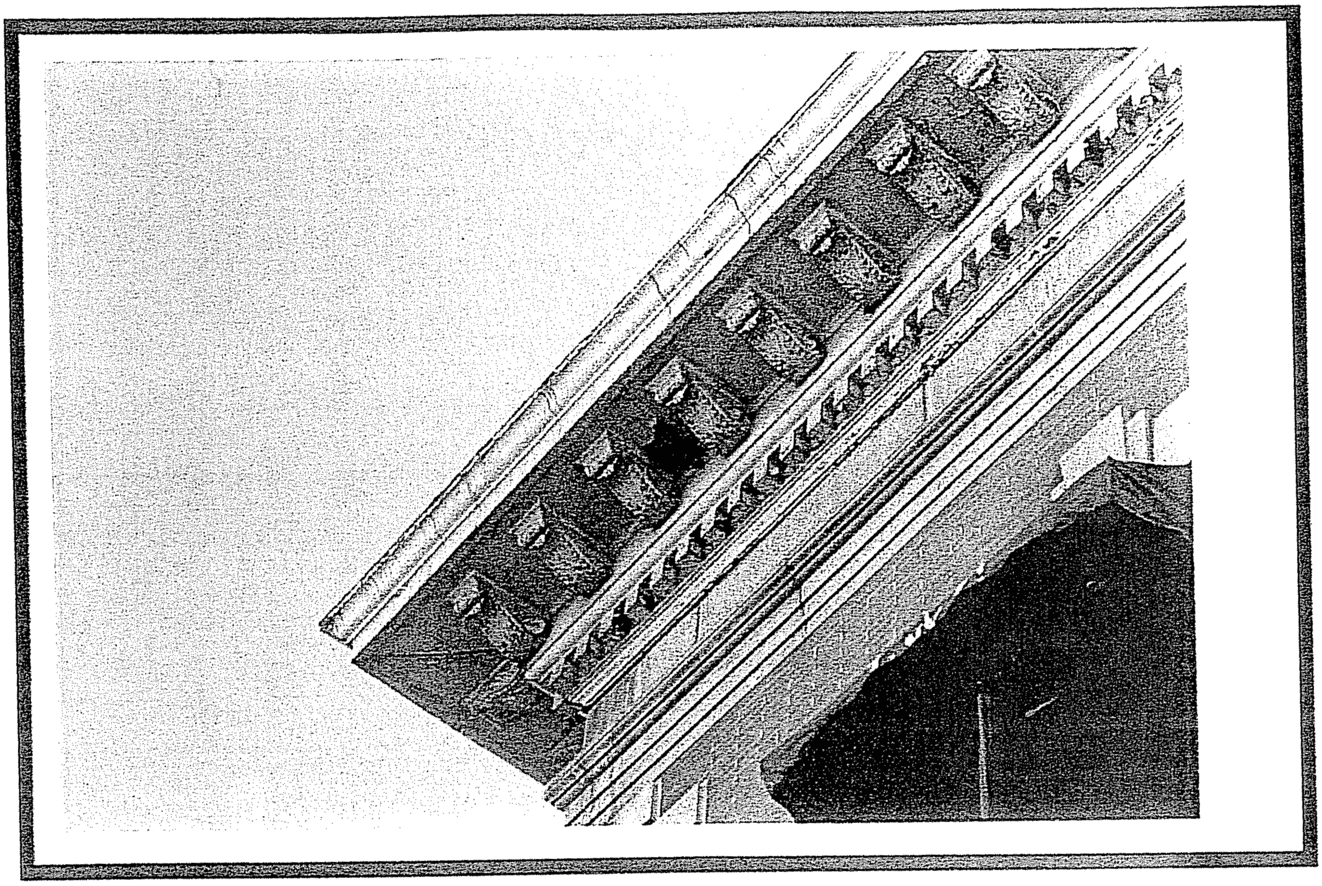

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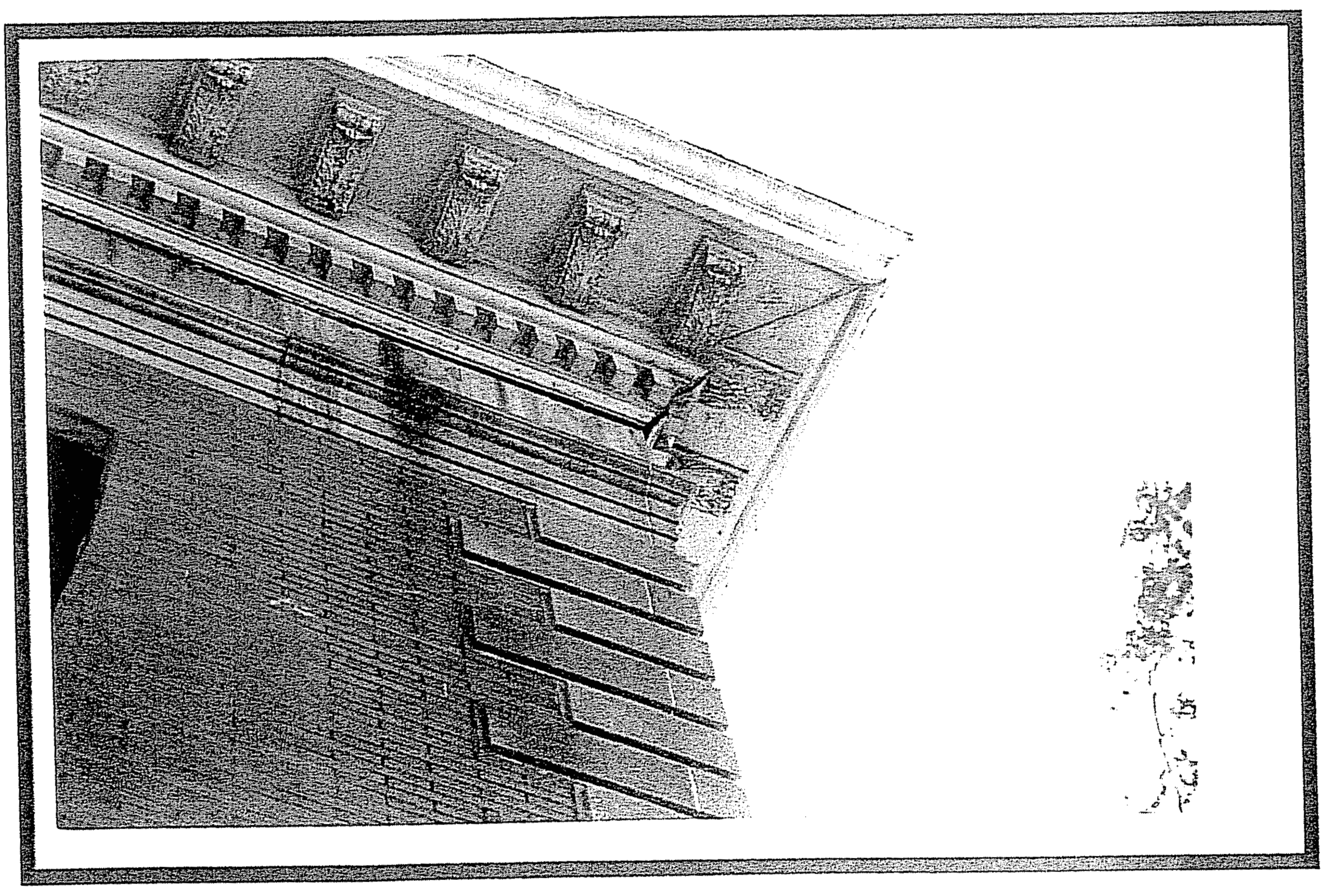


Photograph 64

ALLMOND AVENUE FACILITY

Allmond Avenue Distribution Facility

Pavement and Site Condition (Poor to Fair)

- The concrete paving is broken and depressed in many areas. The existing concrete should be cut out; the sub-grade re-compacted and new 5,000 p.s.i. concrete placed.
  See Photo Numbers 65 & 66

- The existing pavement joints are not filled. The joints should be cleaned and filled with silicone based concrete joint filler.
  See Photo Number 67

- The traffic arrows and directional traffic controls are faded. Suggest replacement with black traffic paint. The black traffic paint contains additional pigmentation and will last longer.

Exterior Walls Condition (Fair to Good)

- The cover the dock has been hit by trucks and is damaged. The edge should be repaired or covered with an aluminum channel.
  See Photo Numbers 68 &69

- The wood truck bumpers are damaged and no longer provide protection for the dock. The wood bumpers should be replaced with rubberized dock bumpers to absorb the shock from trucks hitting the dock area.

Roof Condition (Poor to Fair)

- These roofs should be placed under a roof management program. The exception is the roof over Engineering; this roof must be replaced. We suggest an EPDM, 60 mil membrane, fully adhered.
  See Photo Numbers 70, 71, & 72

- The counter flashing is not secured at the top metal break. This detail will not survive high wind and will allow water to infiltrate under the flashing.
  See Photo Number 73

- Attempts have been made to caulk the top of the wall flashing. However, the material used is not appropriate for this use. The material must be cleaned off, and a silicone based sealer applied.
  See Photo Number 74

- The base sheet is turned up to the coping without a termination bar. This condition will crack and produce a leak.
  See Photo Number 75

Glazing Units Condition (Poor)

- The caulking in the metal to masonry joints have failed and producing leaks. Due to the age of the caulk, the caulk must be removed, the joints cleaned and re-caulked.

Structural Condition (Fair)

- An interior load-bearing wall is rotating outward at the top. A structural analysis must be performed to determine the cause. However, the wall should be supported back to the slab with clip angles as a safety precaution. The space between the wall and the slab should be filled with backer rod and grouted.
  See Photo Number 76 & 77

- The ladder to the roof (upper section) has pulled away from the wall. It should be reattached ASAP.

- The steel in the storage building has oxidized to the point of failure. This building is no longer safe. Immediate repair and or shoring is required.

Building Interiors

- The office interiors are dated.

- Active roof leaks continue to stain ceilings.
  See Photo Numbers 78, & 79

- Carpet and Paint should be scheduled within the next two (2) years.

Mechanical

- The HVAC system utilizes roof top air-cooled roof top units, above ceiling units, and window units. An analysis of the complete system should be performed to recommend the most efficient solution to the comfort level problems the workers are currently experiencing. At least one of the roof top units utilizes a CFC refrigerant and should be replaced within the next two years.
  See Photo Numbers 80, 81, & 82

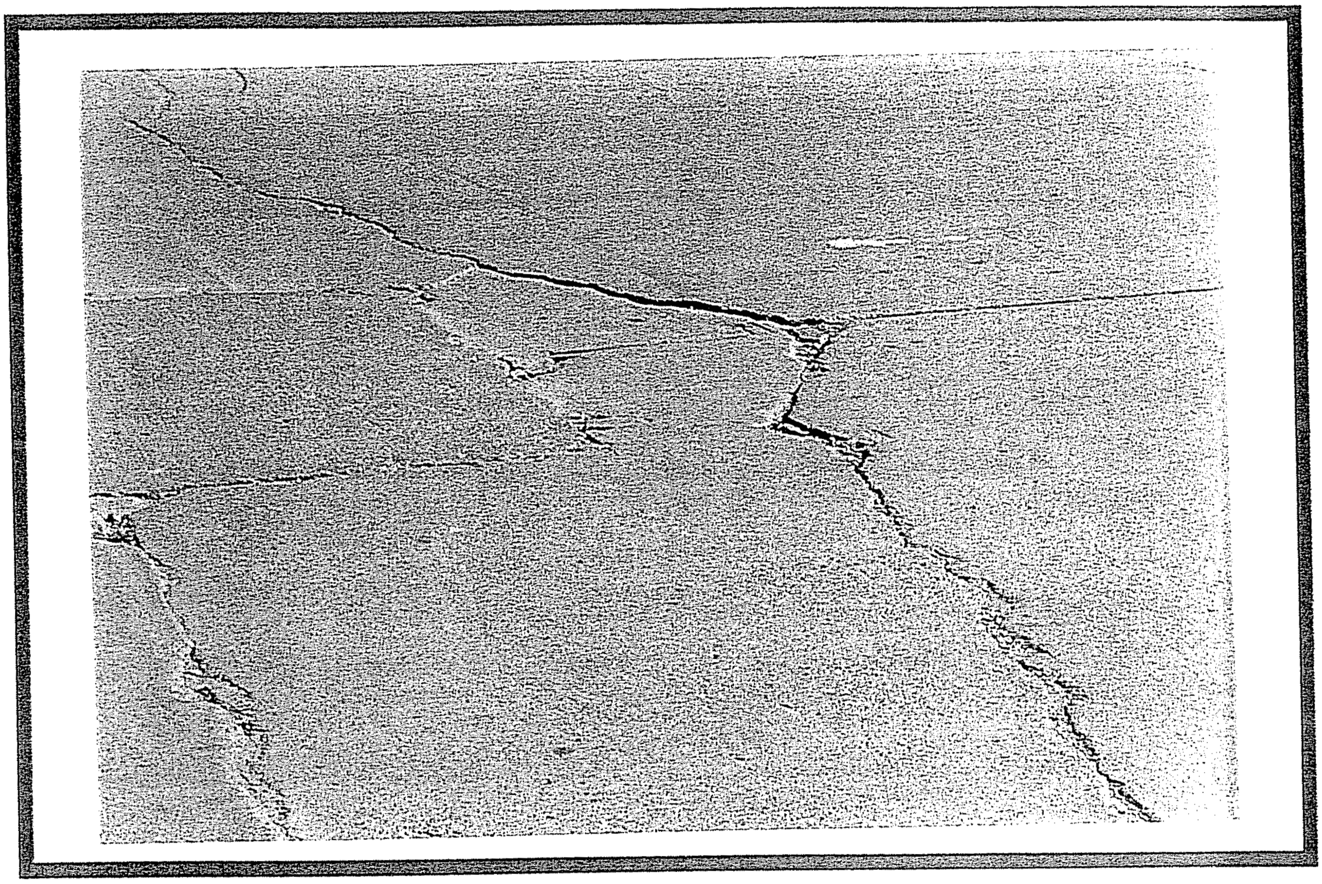
Electrical

- The electrical appears to be adequate for the current needs. However several open junction boxes and panels with open covers was observed. Additional discovery will be required to determine the extent of code and distribution concerns.

ADA

- The restrooms observed in this facility are not compliant.

- The narrow hall at the entrance to some restroom will have to be removed as will all lockers and benches. We suggest a full ADA study be completed with the focus on installing uni-sex compliant restrooms.
  See Photo Numbers 83, 84, 85, & 86



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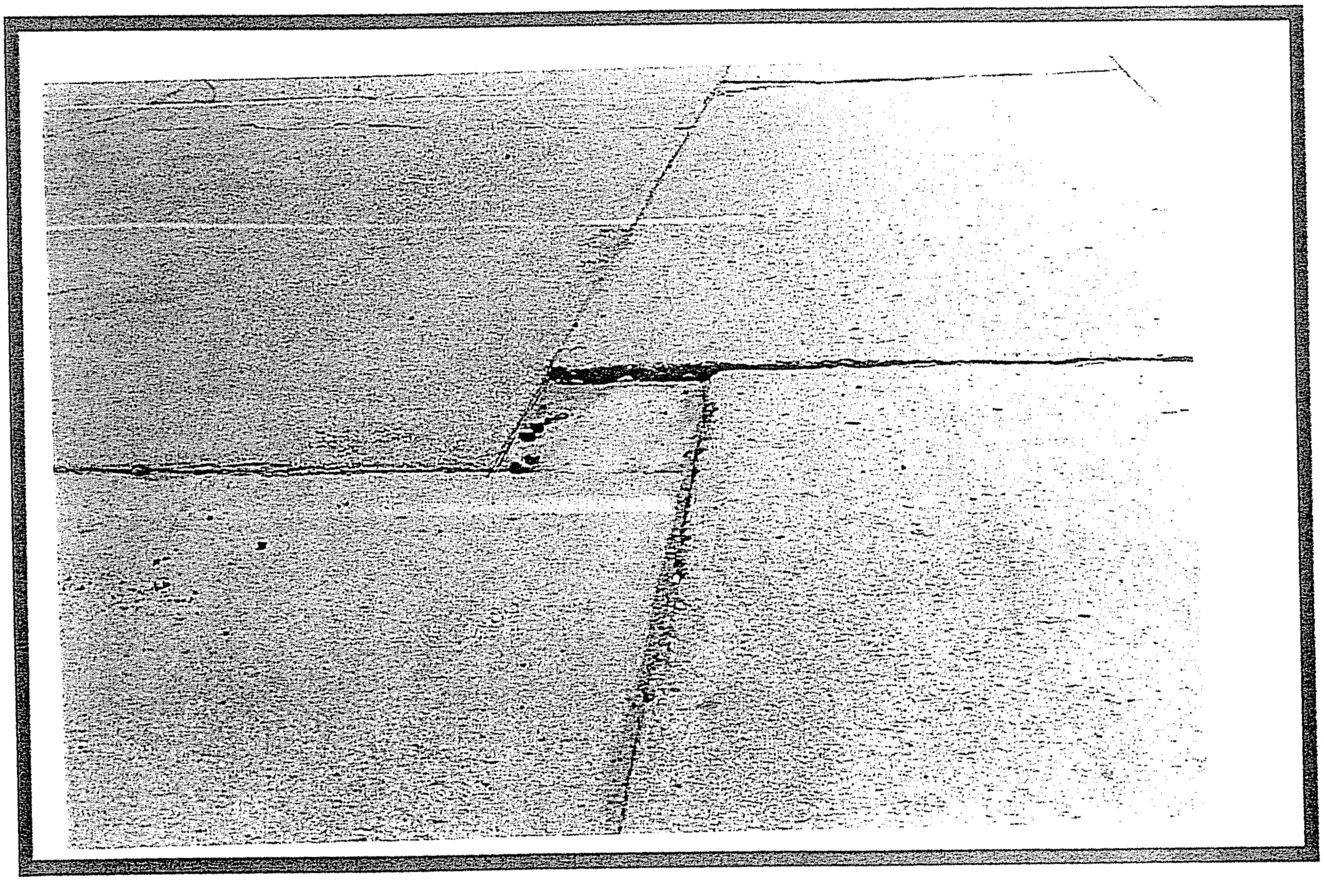

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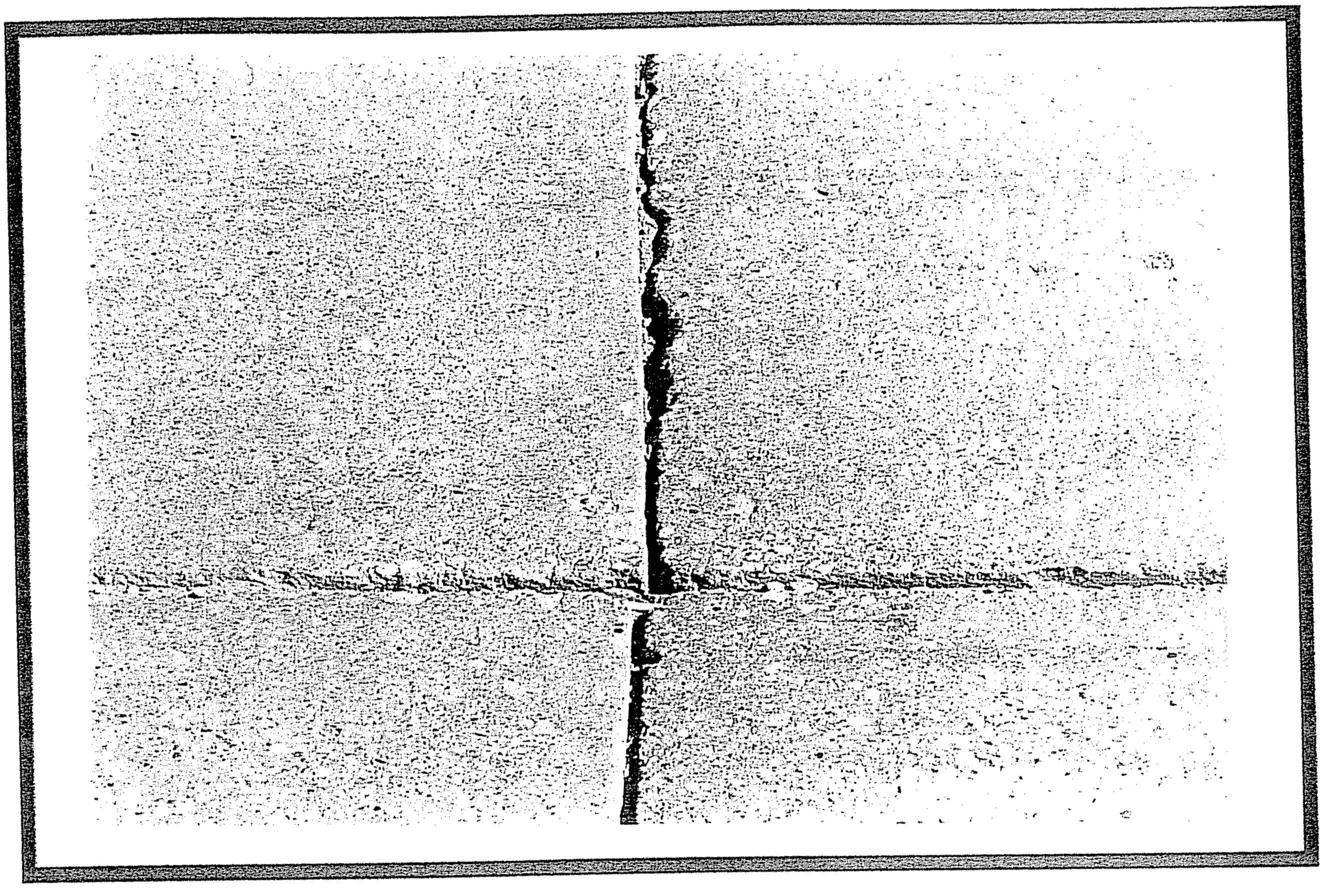

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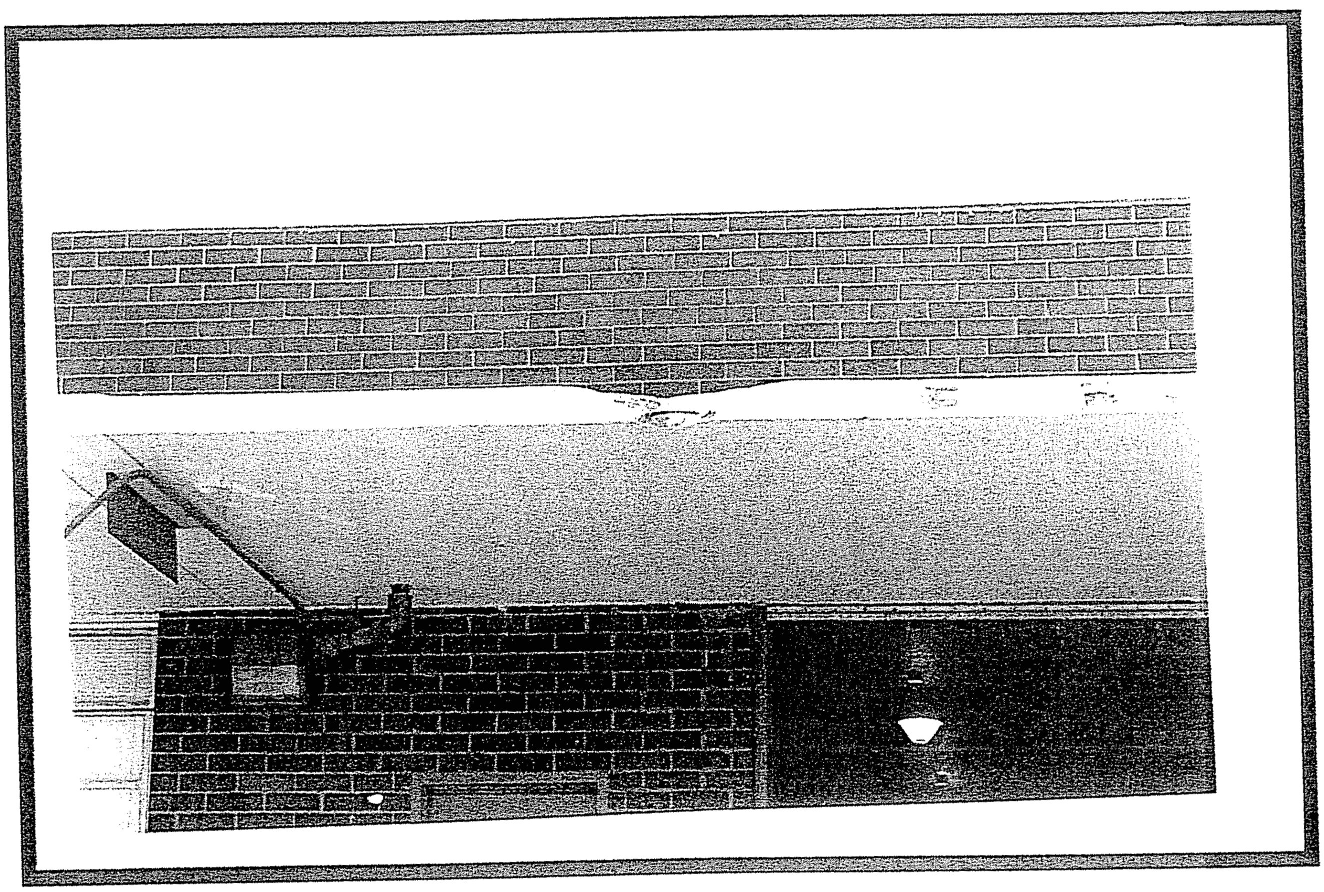

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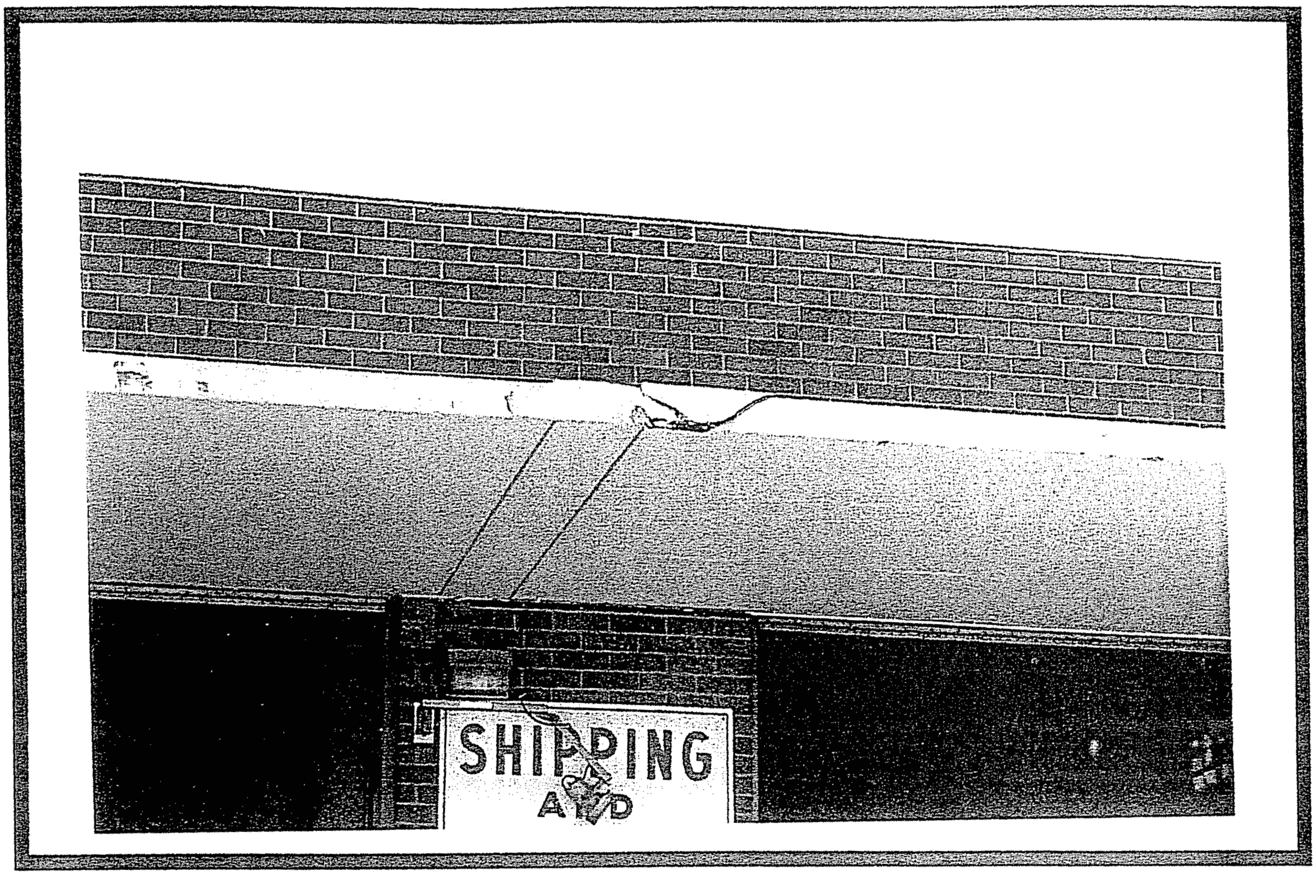

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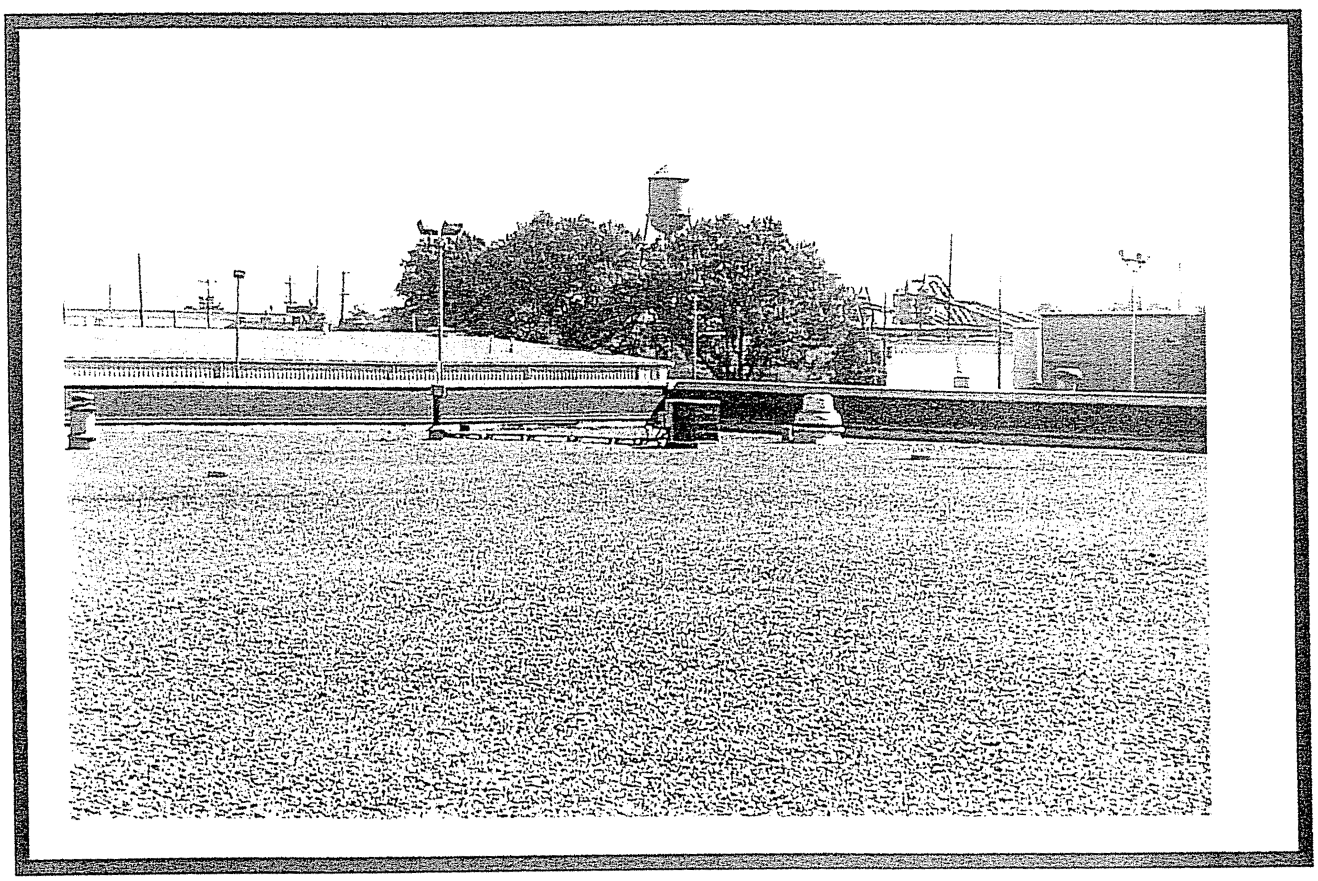

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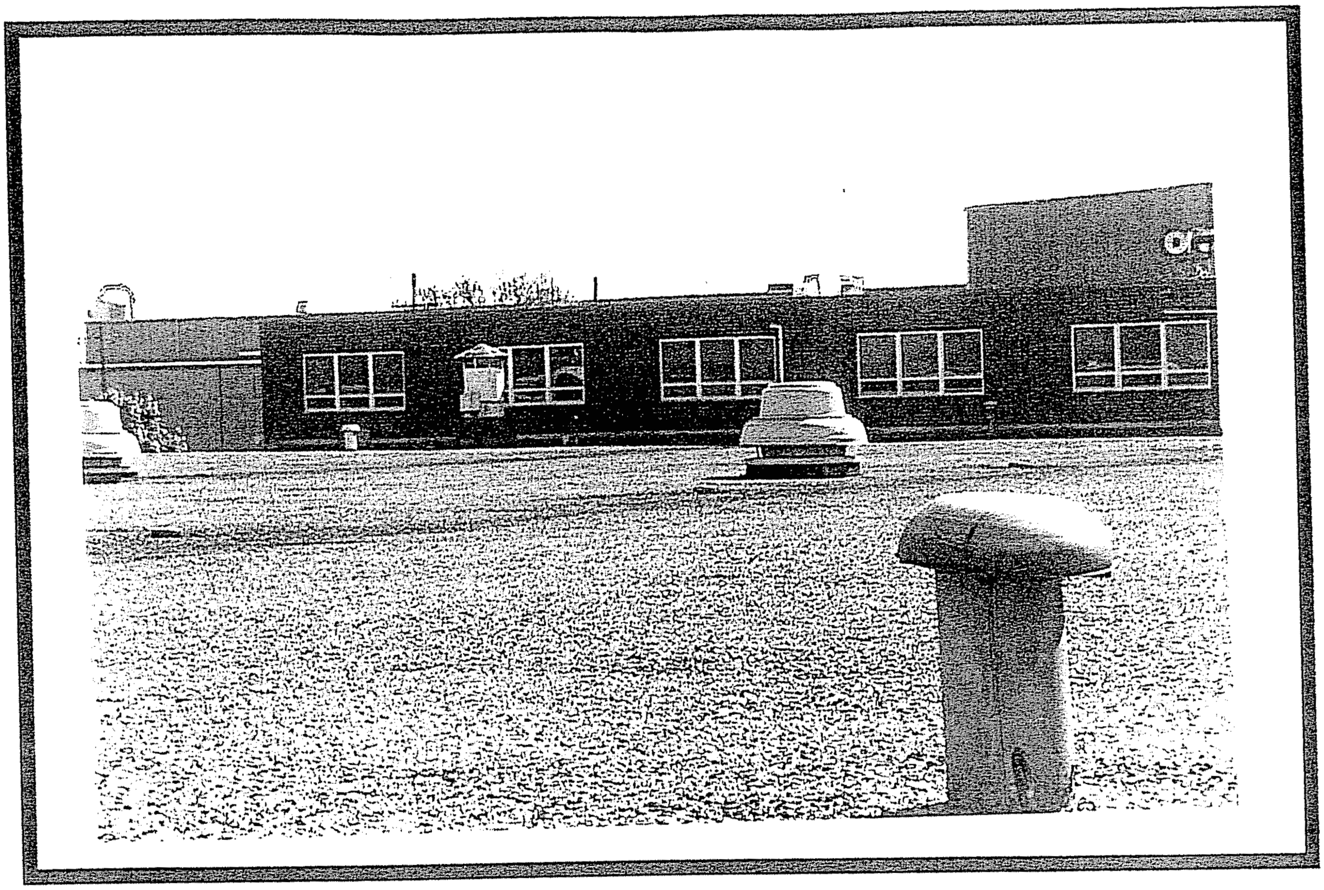

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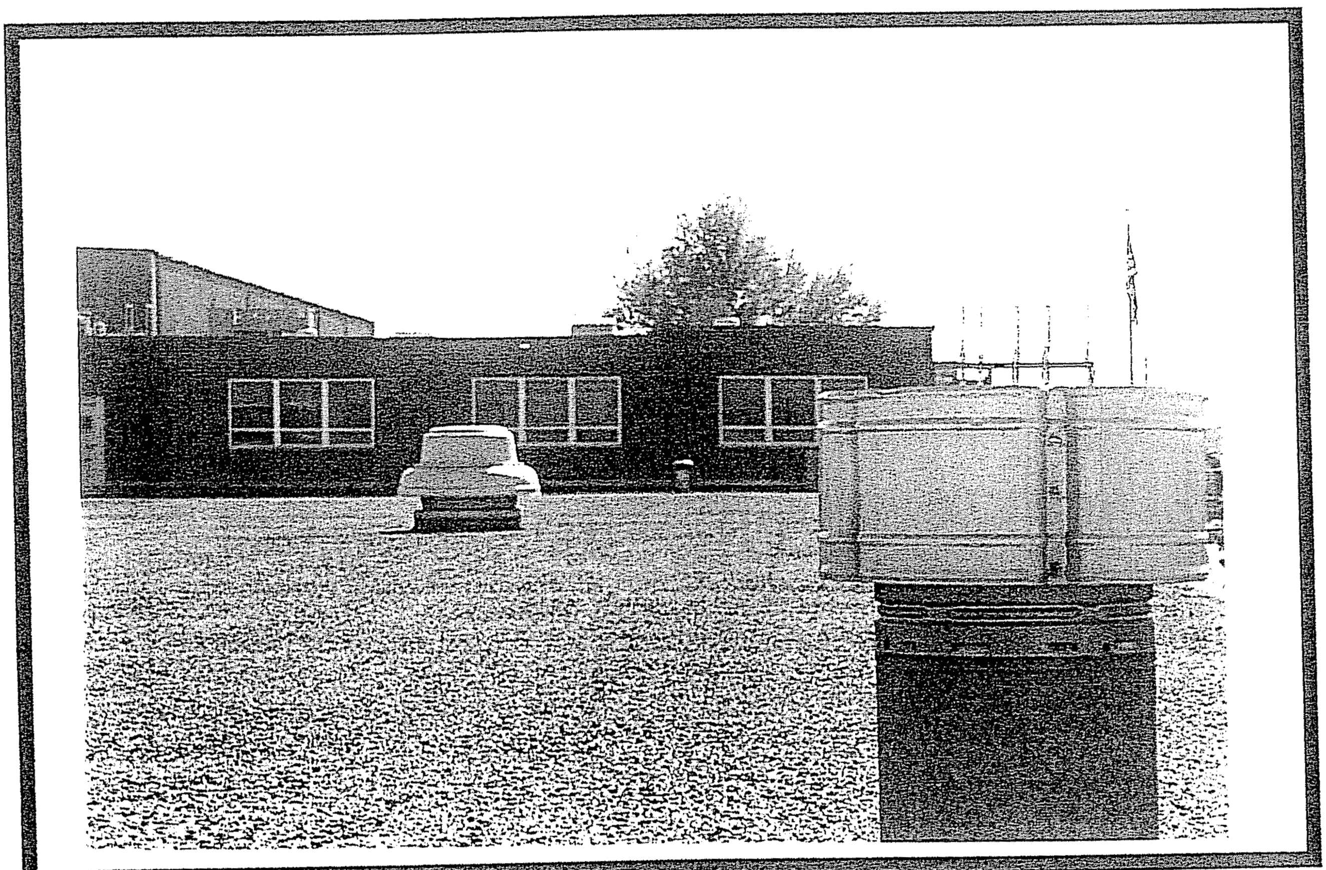

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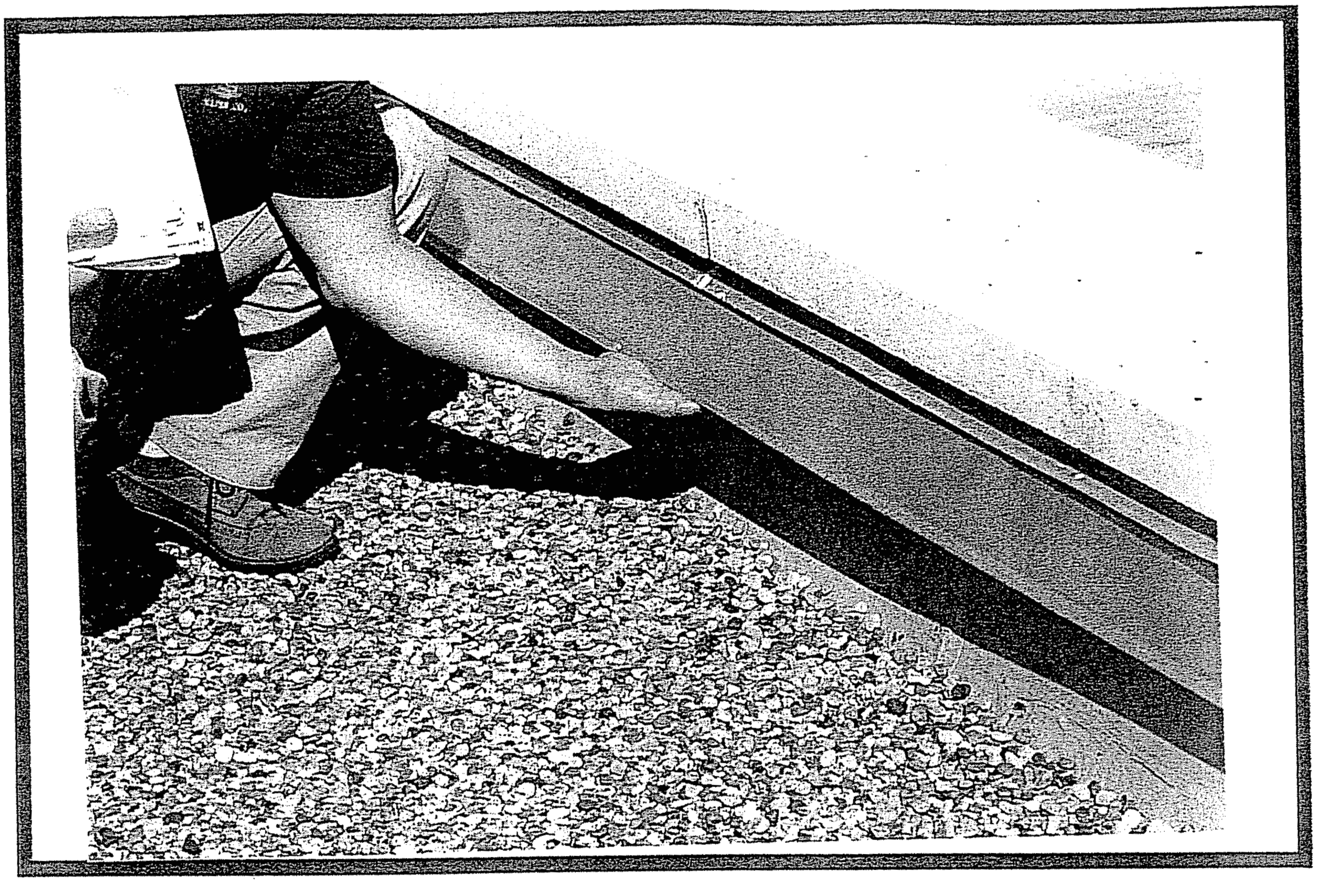

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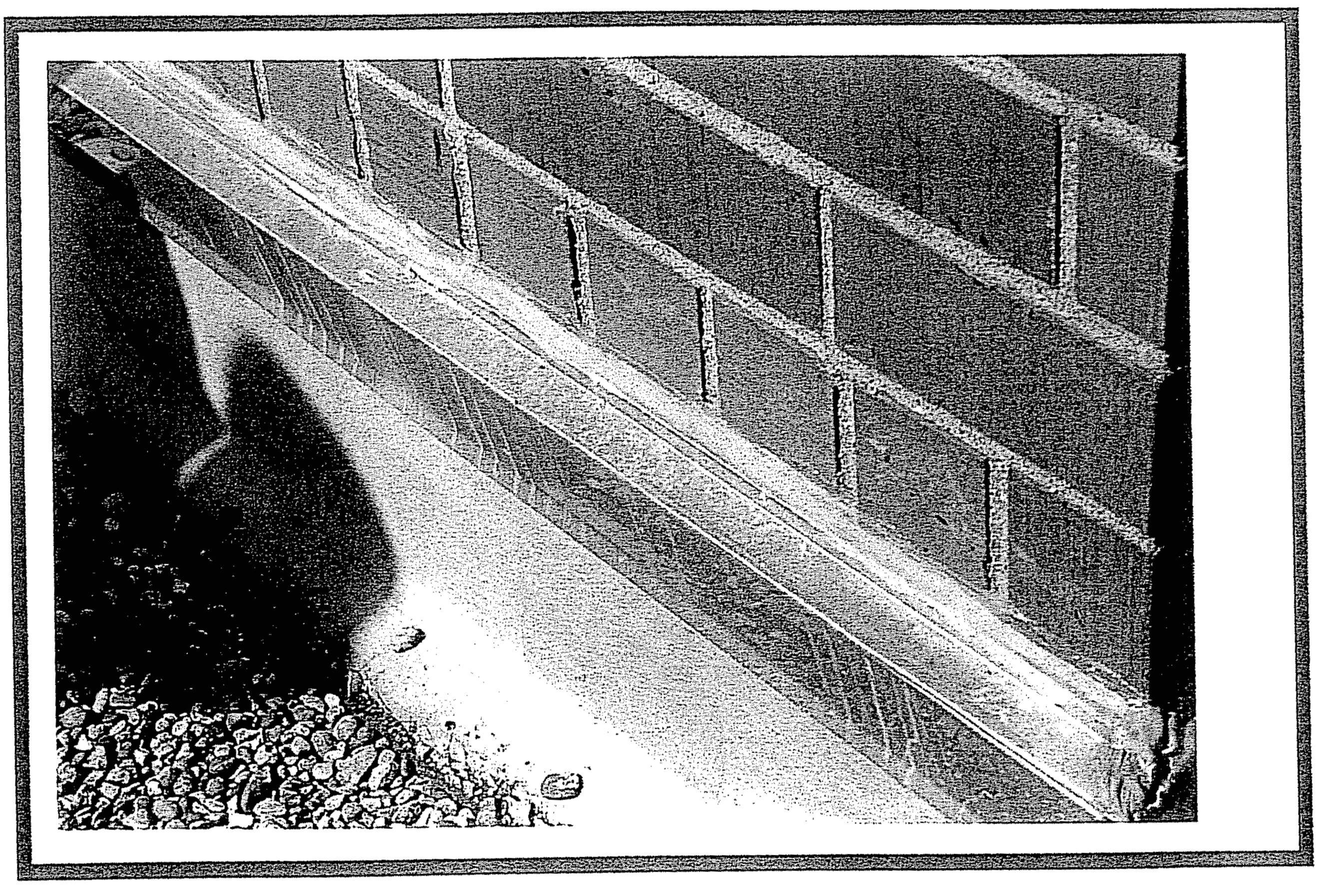

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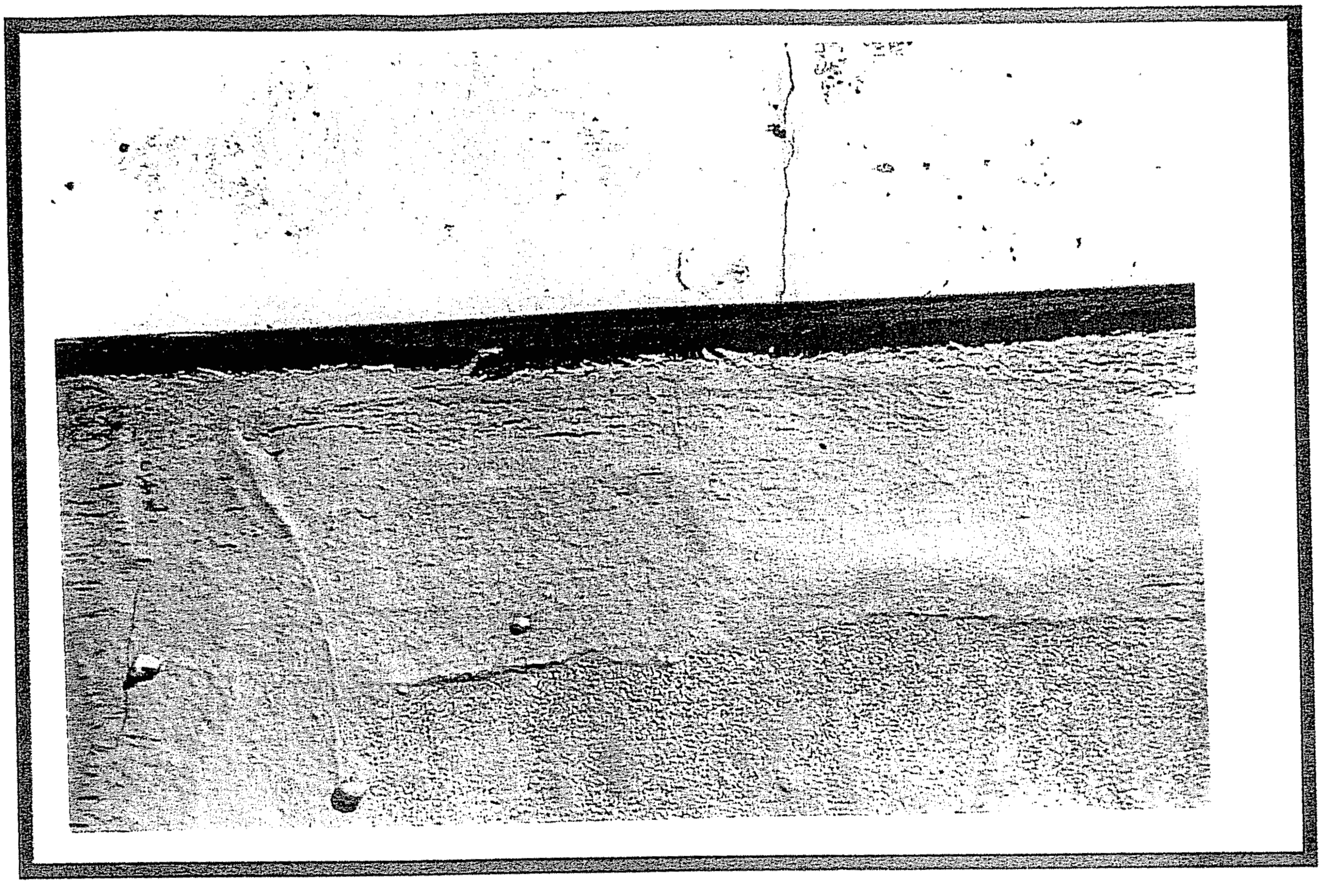

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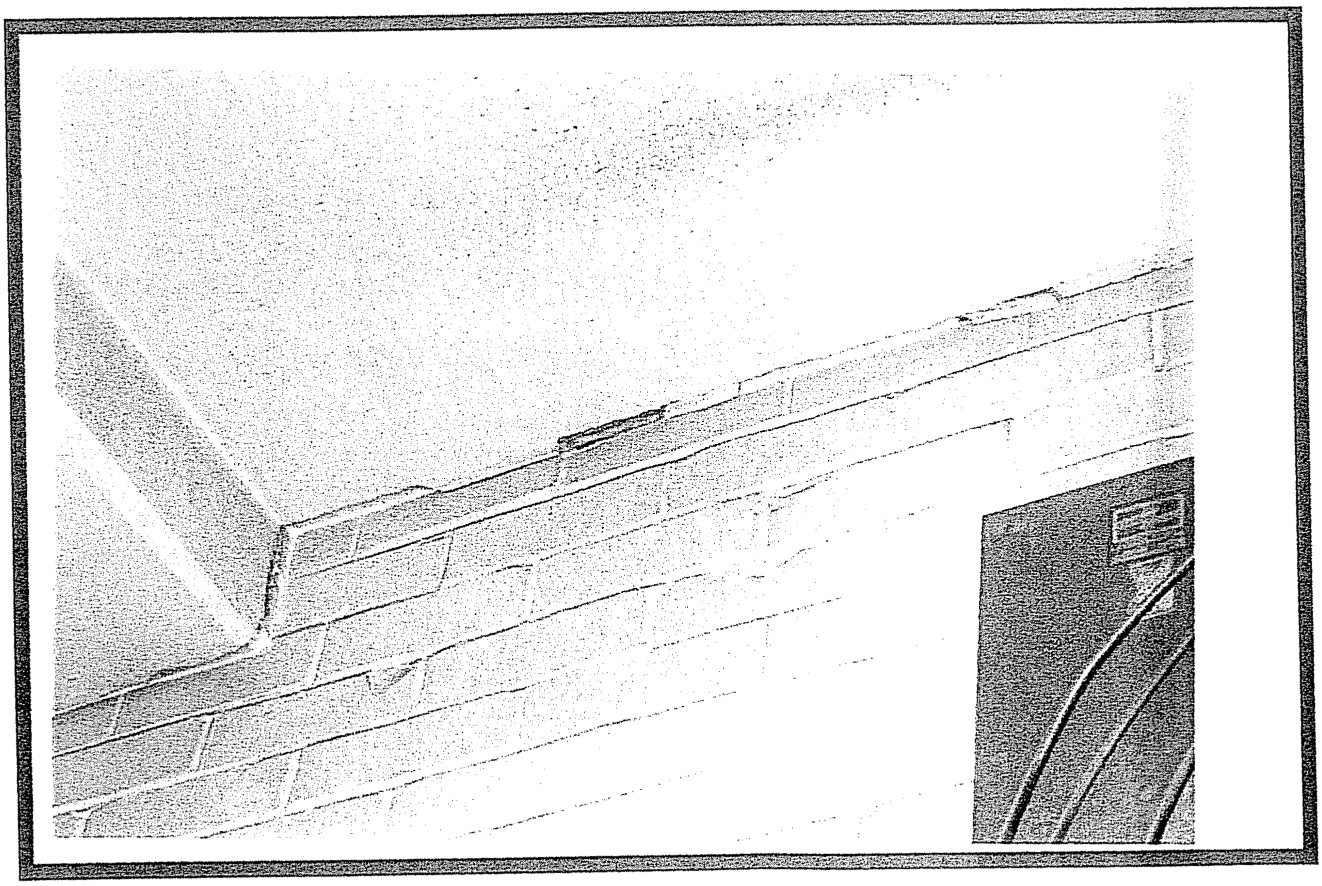

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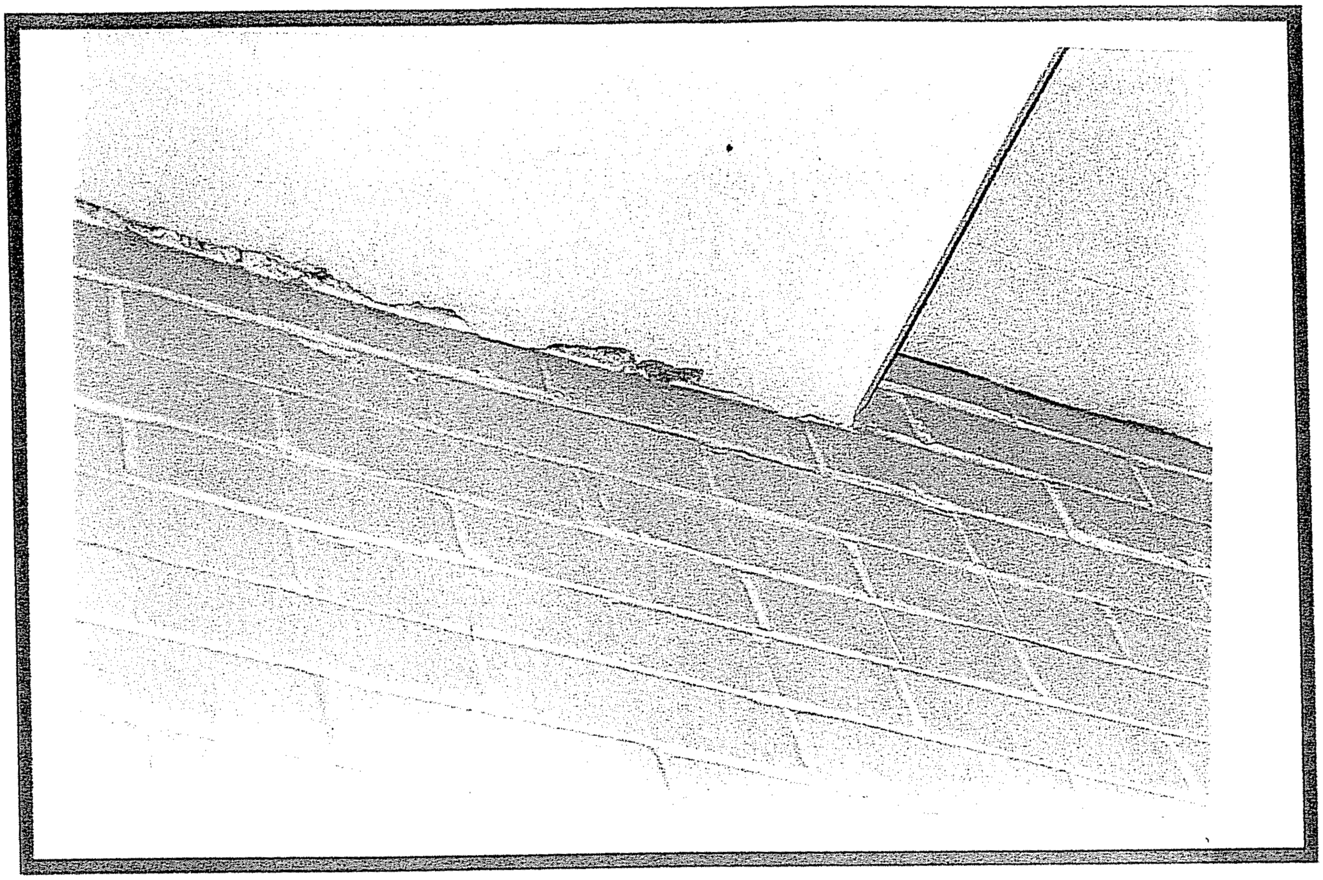

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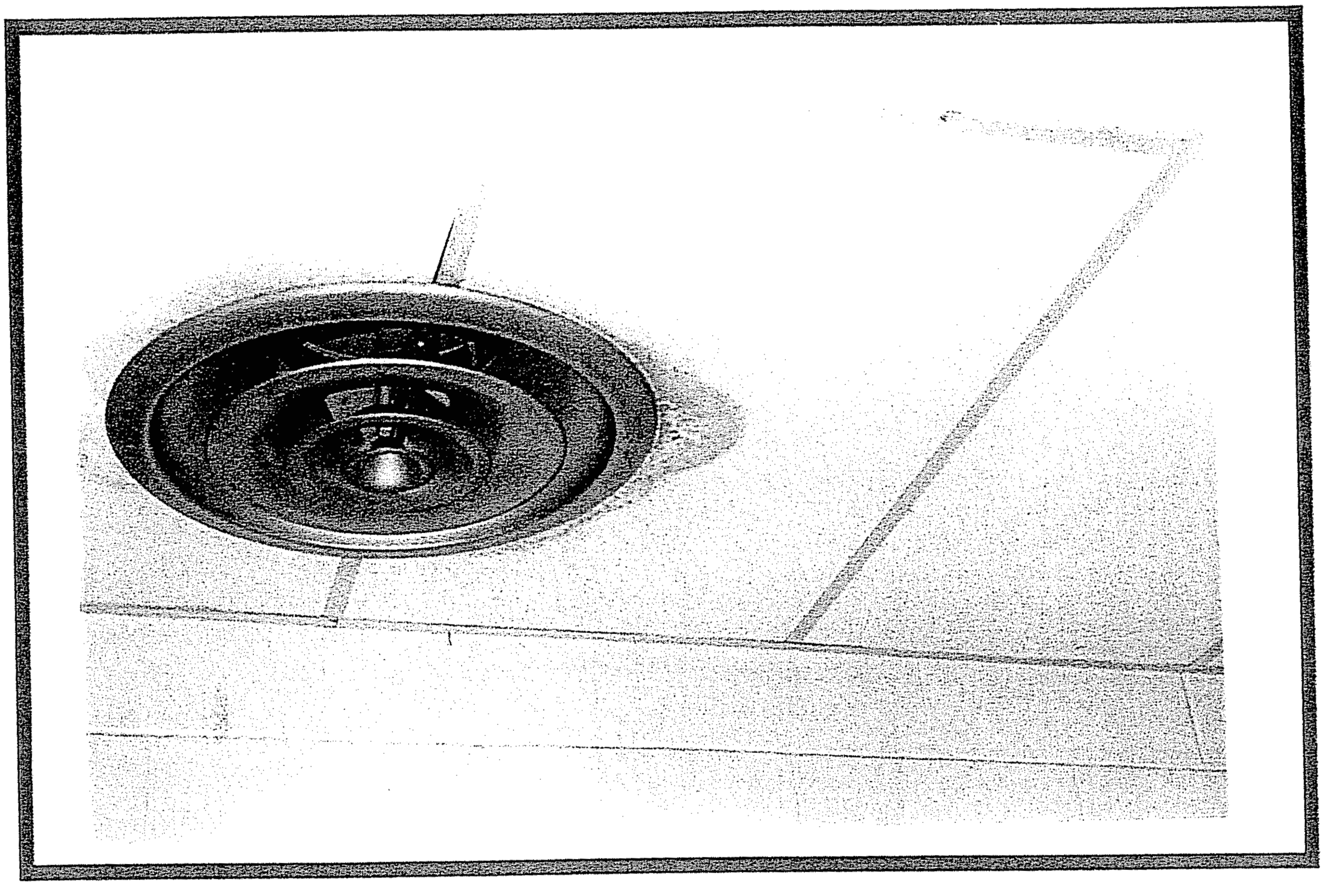

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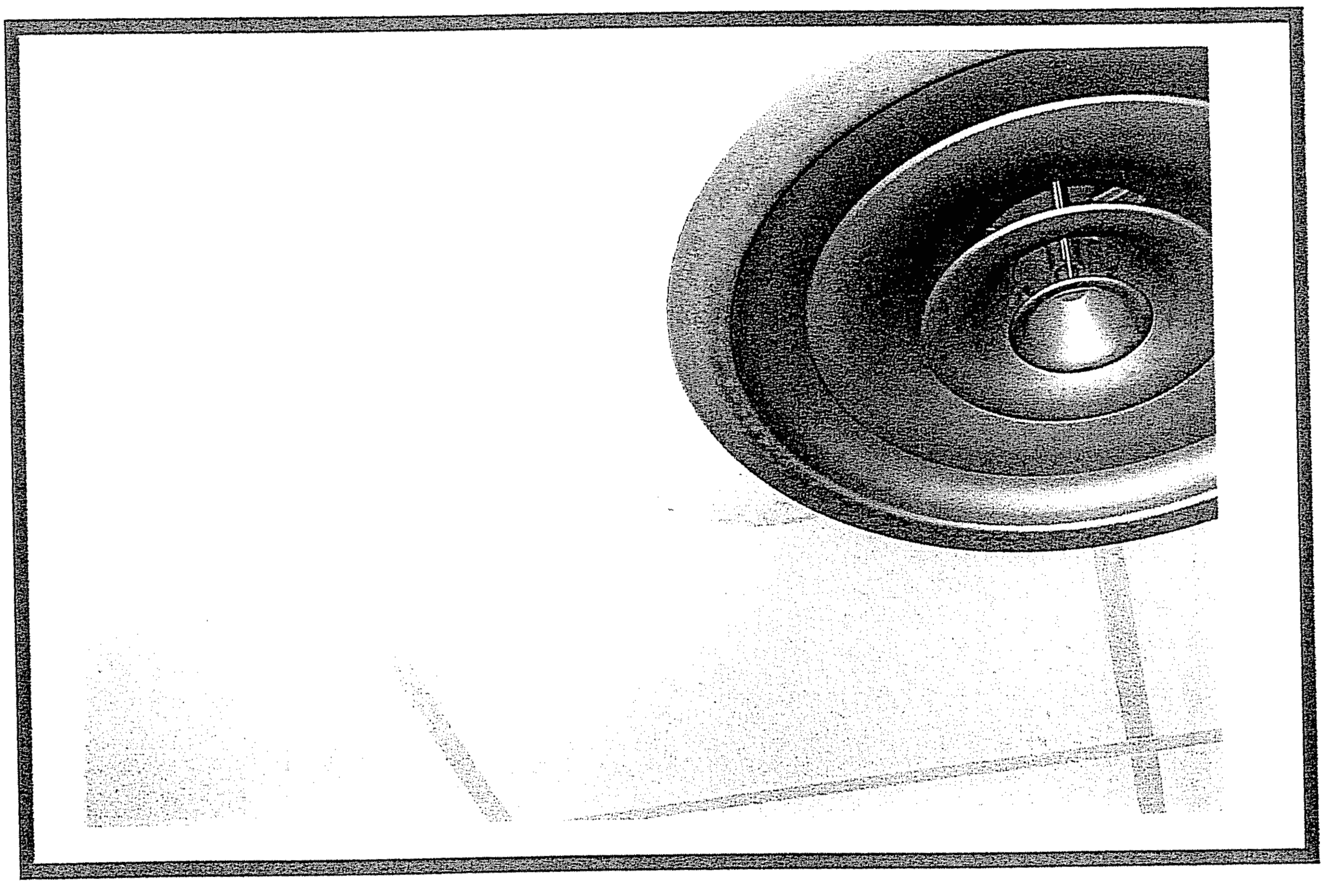

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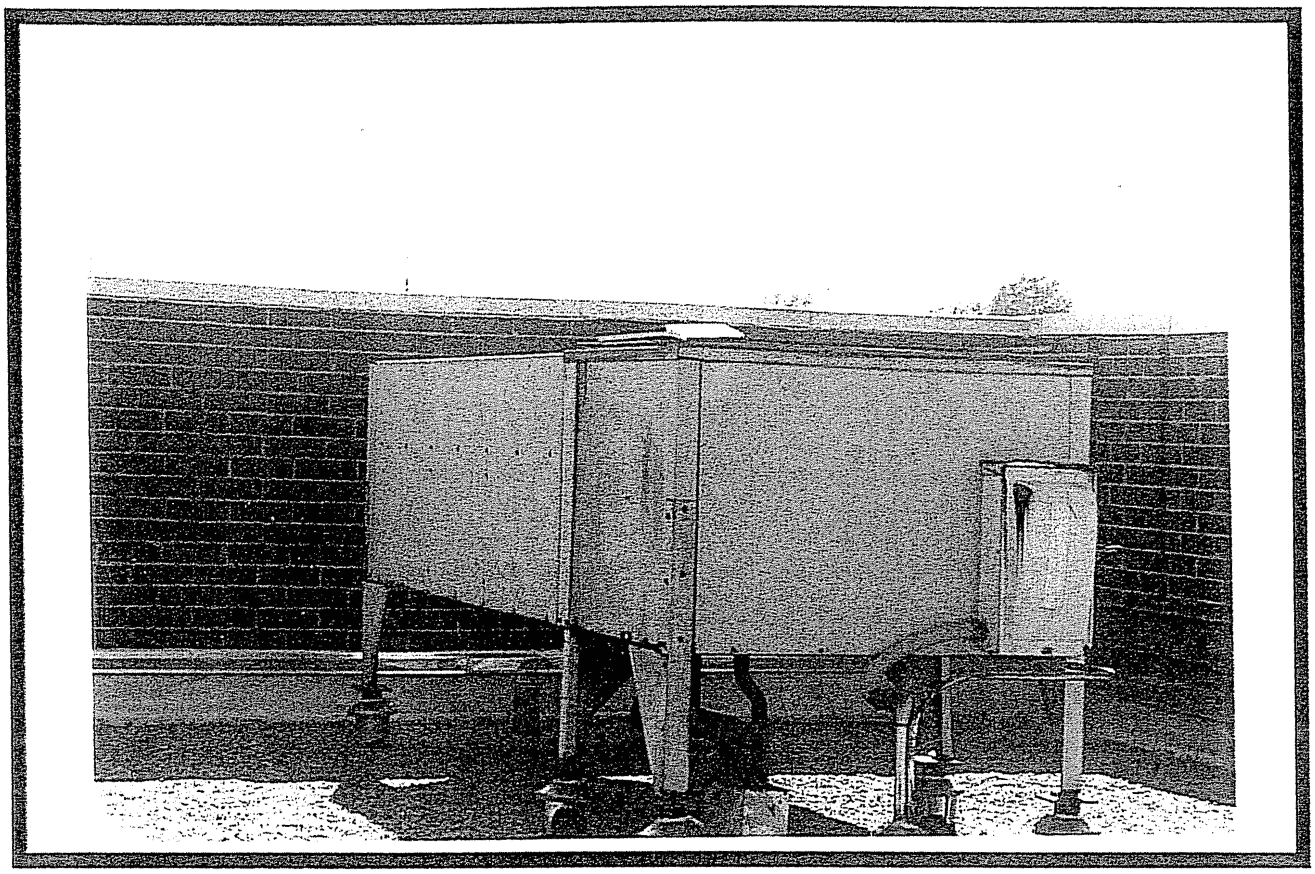

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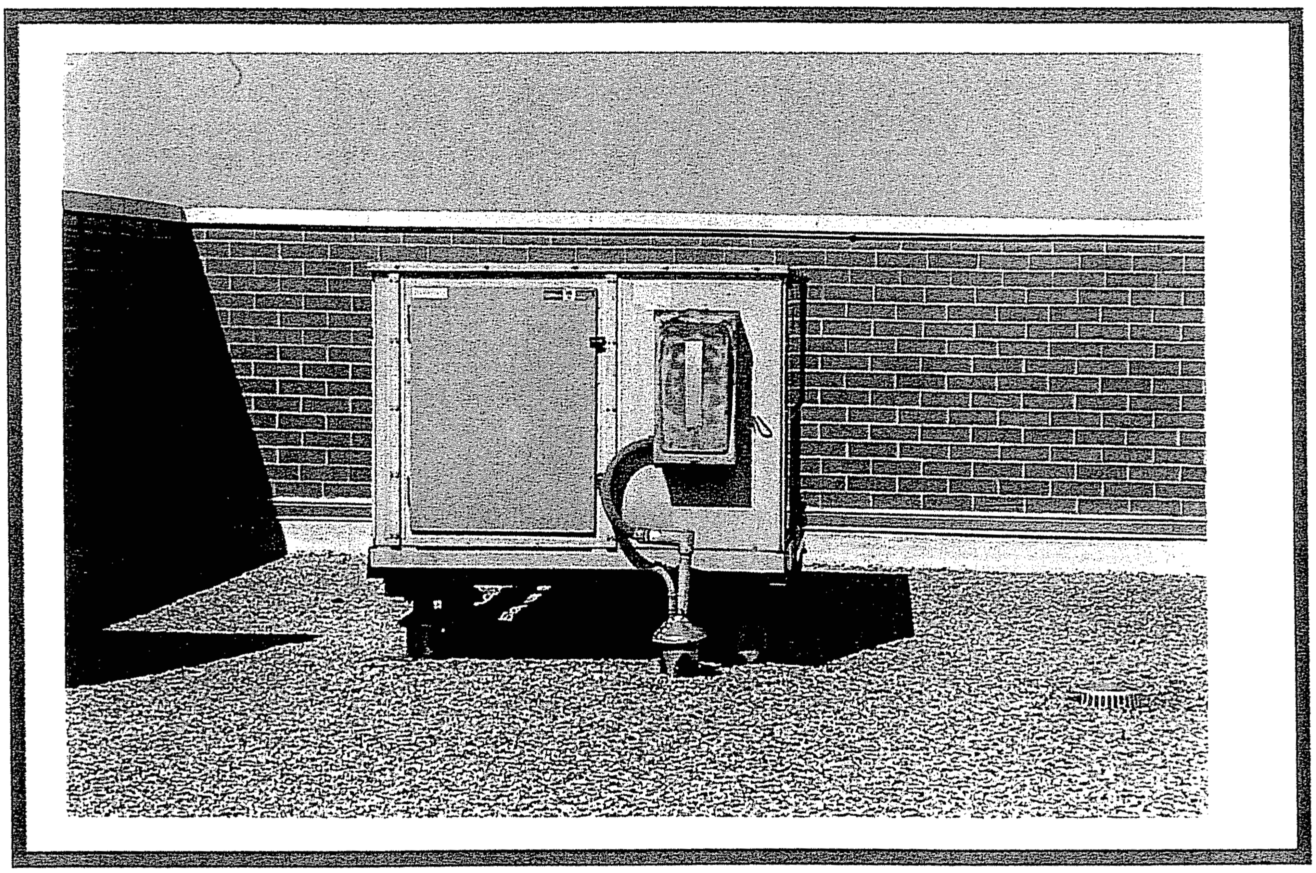

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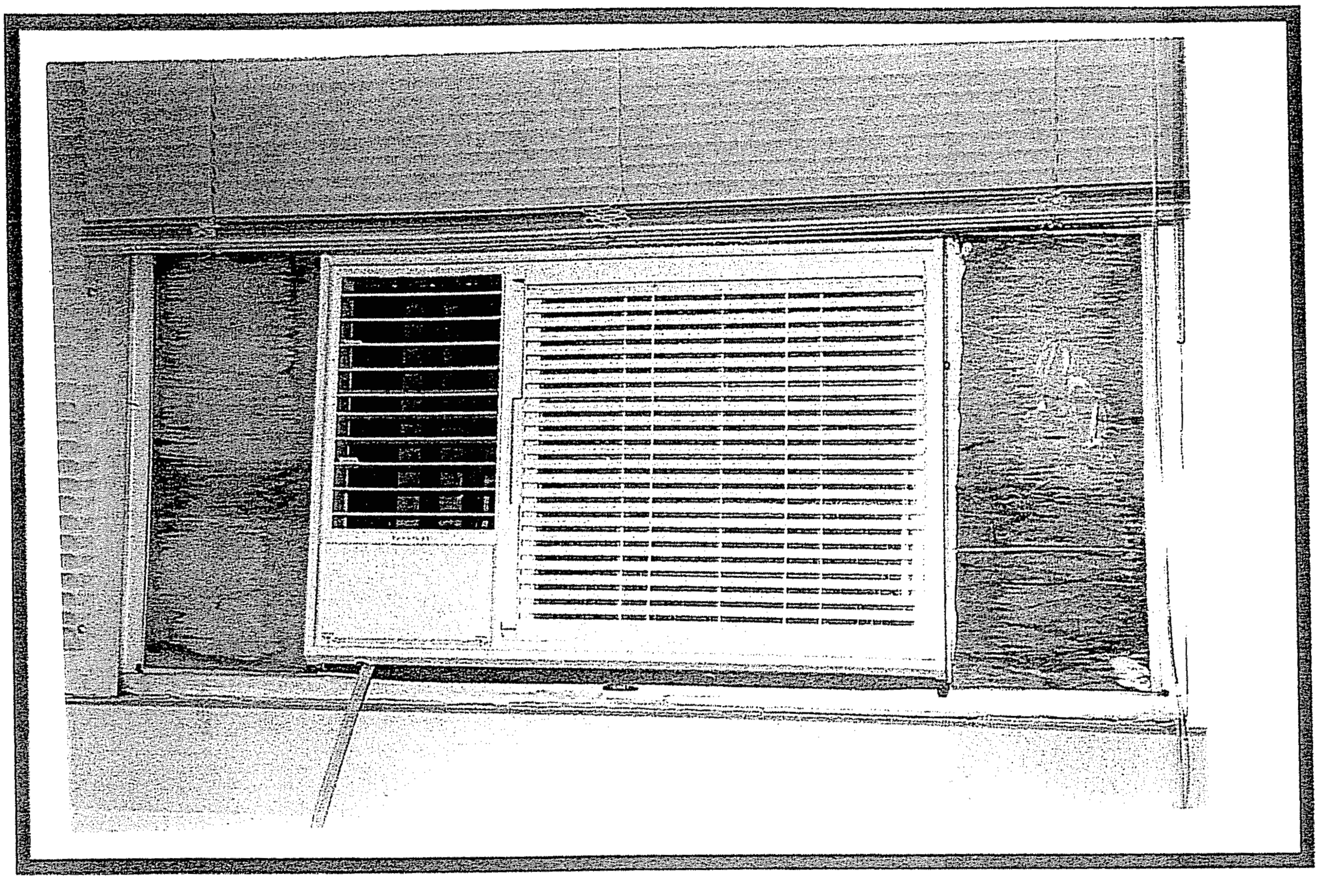

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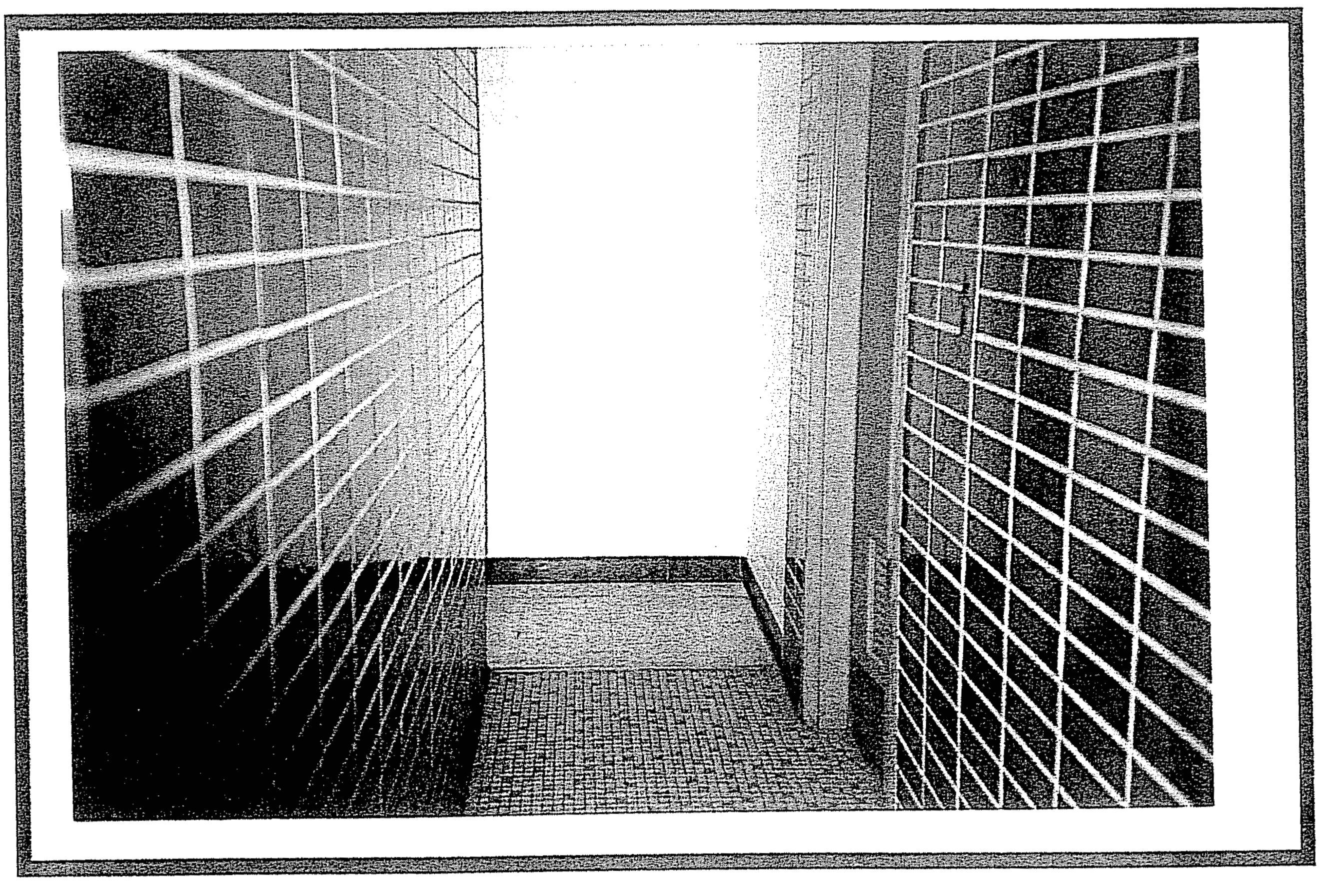

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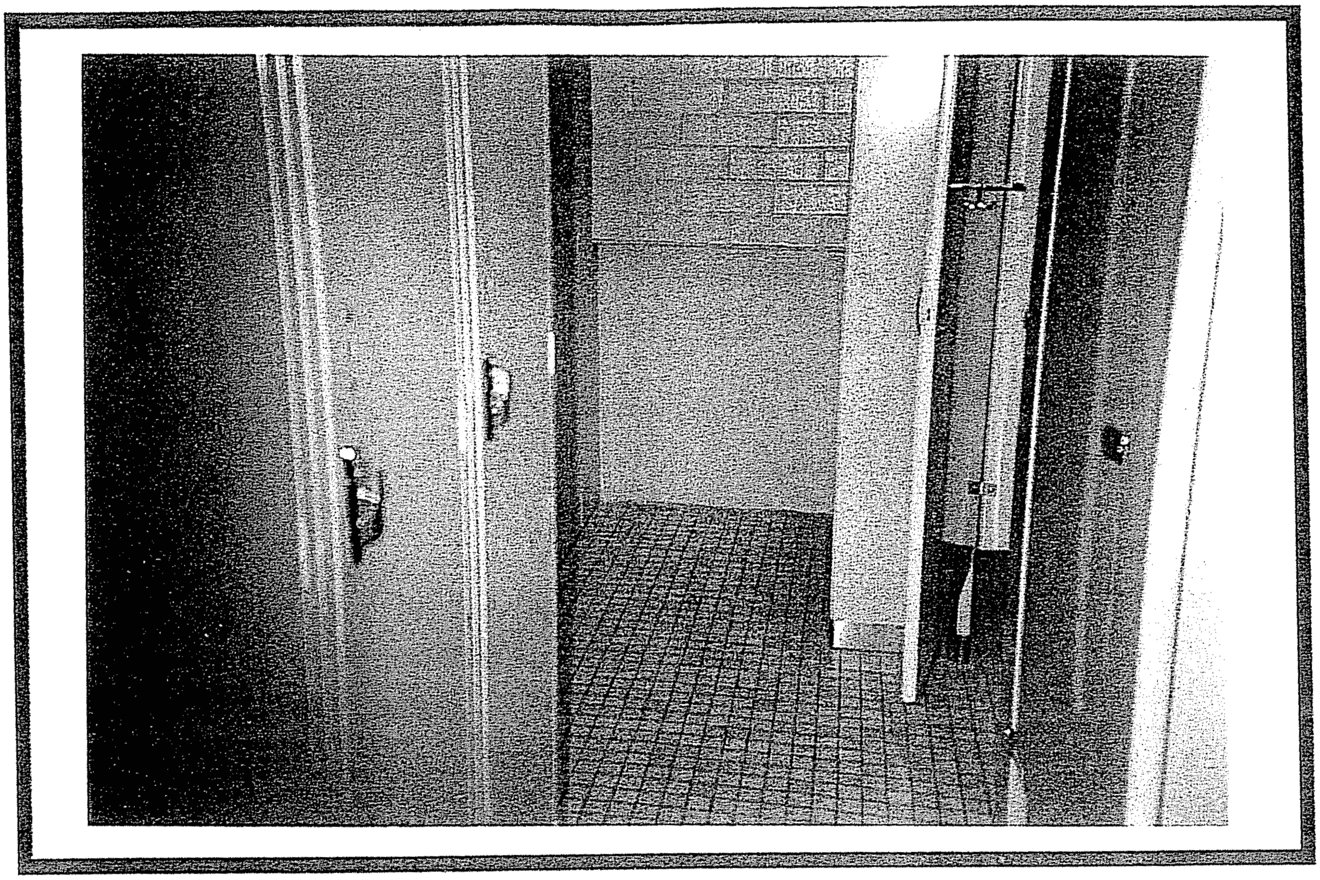

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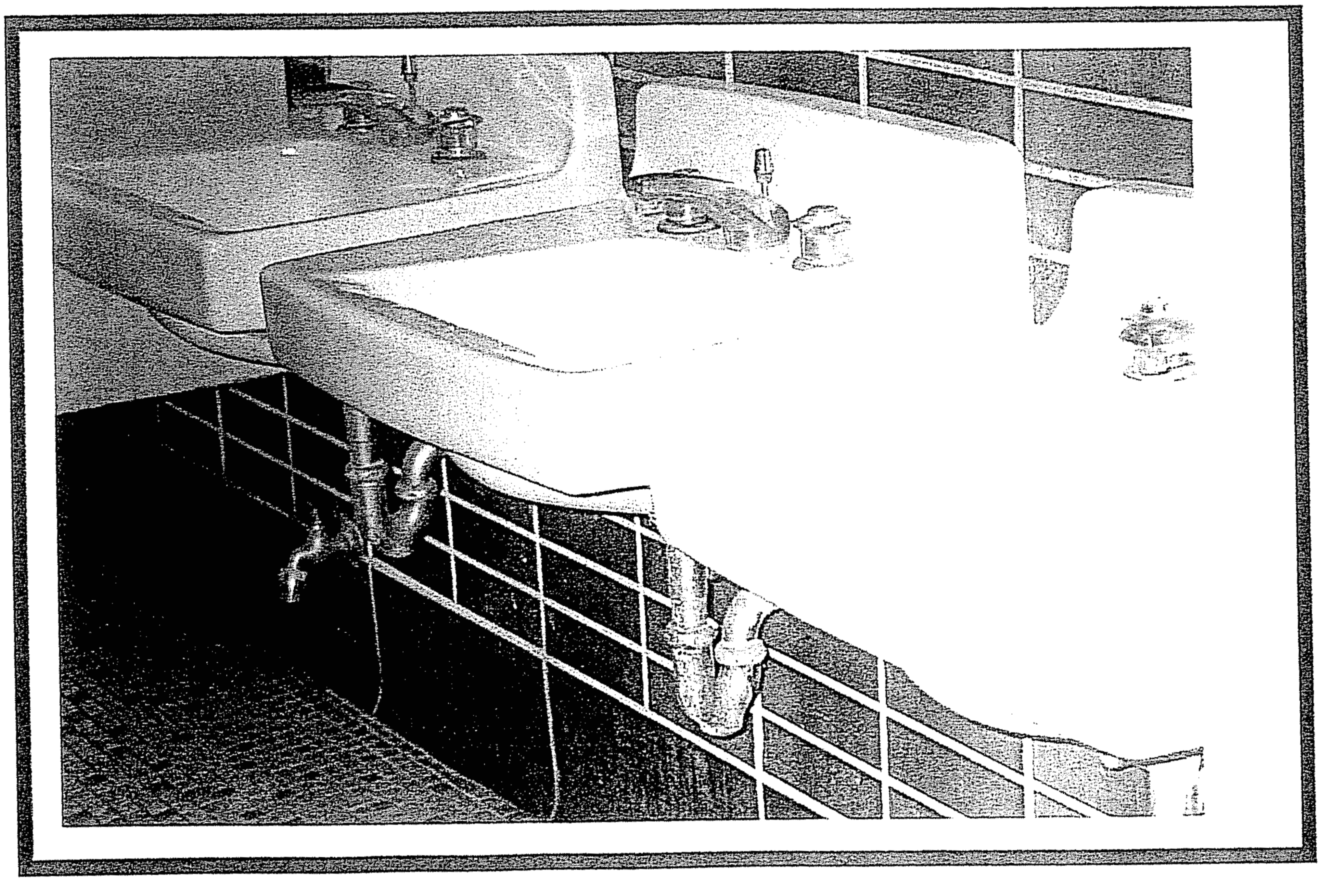

Photograph 85

Photograph 86

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

TOTAL FACILITY COST SUMMARY AND SPECIAL COSTS FOR ALL FACILITIES:

SPECIAL COSTS FOR ALL FACILITIES:

	Immediate	Short Term	Long Term
Painting Maintenance Agreement (Annual Cost)	$135,000		
HVAC Maintenance Agreement (Annual Cost)	$50,000		
Elevator Consultant (One Time Cost)	$12,000		
Automated Work Order and Preventive Maintenance Program	$35,000		
Roof Management Program (Up-Front Cost)	$35,000		
Overhead Door Maintenance Agreement (Annual)	$25,000		
Outsourced Safety Training Program (Annual)	$8,000		

TOTAL FACILITY COST SUMMARY:

IMMEDIATE	SHORT TERM	LONG TERM
$1,480,480	$2,296,000	$147,000

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

CRESCENT HILL FACILITY

	Immediate	Short Term	Long Term
Pavement Repairs see photos no. 3 and 4	$45,000	$100,000	
Point and Patch Exterior Walls see photo no. 5,6,7 & 8	$40,000	$75,000	
Pressure Grout Cracks see photo no. 5,6,7 & 8	$10,000	$20,000	
Caulk Windows see photo no.10	$35,000		
Repair to Soffits and Fascia see photo no. 11	$100,000		
Paint Exterior Wood Trim	$25,000		
Repair Wrought Iron Handrails	$4,500		
Slate Roof Repairs Immediate cost to stop leaks. Short term is restoration	$10,000	$40,000	
Built-Roof Replacement Immediate = 50% of 163,000sq.ft. @ $4.00 per sq.ft.	$326,000	$326,000	
Gutter Repair and Replacement	$8,000		$11,000
Additional Roof Drains	$3,500	$7,500	
Window Replacement	$40,000		
Repair Load Bearing Wall @ Steel Girder	$6,000	$9,000	
Clean and Paint Steel Framing immediate steel over filters	$150,000		$60,000
Clean and Paint Piping immediate= piping over filters	$40,000	$20,000	
Interior Office Renovations 57,000 sq. ft @ $12		$684,000	
Chiller Replacement replace CFC refrigerant chiller	$85,000		
Roof Top A/C unit Replacement Immediate= replacement of malfunctioning units	$40,000	$80,000	
Install New Sewer Lines	$20,000		
Lighting Retrofit		$40,000	
Dock Enclosures /'Separators		$6,000	
Variable Speed Drives		$60,000	
HVAC / Mechanical Analysis	$12,000		
Install Uni-Sex Restrooms Immediate = two restrooms @ $20,000		$40,000	$40,000
Develop ADA Compliant Plan code required a plan on file	$6,000		
FACILITY TOTALS:	**$1,006,000**	**$1,507,500**	**$111,000**

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

B.E. PAYNE FACILITY

	Immediate	Short Term	Long Term
Replace Concrete @ Dock see photo no.27	$4,000		
Replace Concrete Walks see photo no.28	$3,000		
Repair Asphalt @ loading door	$2,500		
Repair Drive To Loading Dock	$15,000		
Repair Asphalt @ Main Parking Lot	$8,000		
Repair Rails at Wall and Steps	$3,500		
Caulk Seams In the Aluminum Fascia		$2,500	
Point and Patch Masonry		$16,000	
Structural Analysis of Wall Rotation		$2,500	
Roof Replacement			
Filter bldg.- 30,000sq.ft @ $4.00.	$120,000		
Office 20,000 sq. ft. @ $4.00		$80,000	
Repair/Replace downspouts	$1,500		
Repair window mullions	$3,000		
Re-Caulk Windows	$8,000		
Caulk Slab to Wall Joint	$1,500		
Repair Terrazzo Floor	$5,000		
FACILITY TOTALS:	**$175,000**	**$101,000**	**$0**

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

MIDDLETOWN FACILITY

	Immediate	Short Term	Long Term
Install Guard Post	$2,500		
Repair to Side Walks	$5,000		
Remove Dead Trees	$3,000		
Repair Fences		$5,500	
Repairs to Material Storage Bldg. see photo no. 32, 33 34	$3,500	$5,500	
Clean and Paint Water Tank		$6,000	
Repair Exterior Wall Panels and Soffit	$3,000		
Install Containment Curb at Truck Wash	$2,500		
Roof Management System immediate repairs =	$2,500		
Re-Caulk Windows		$3,000	
ADA Improvements convert existing restroom, provide access		$12,000	
FACILITY TOTALS:	**$22,000**	**$32,000**	**$0**

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

550 SOUTH THIRD STREET

	Immediate	Short Term	Long Term
Seal Parking Garage Wall Light Fixtures	$500	(Should fall under Warranty)	
Move Irrigation Heads	$1,000	(Should fall under Warranty)	
Install Wind Gauge	$2,000		
Clean and Re-Paving Railing	$3,500	(Should fall under Warranty)	
Install Cover on Garage Steps	$15,000		
Install Ball Bearings on Rails	$600		
Re-Strip Parking Garage	$4,000	(Should fall under Warranty)	
Exterior Wall Repairs	$6,000	(Should fall under Warranty)	
Additional Roof Protection	$900		
Install Termination Bar on wall at the Skylight	$750	(Should fall under Warranty)	
Repair Bubbles in Turned-Up Base Flashing	$500	(Should fall under Warranty)	
Repair Flashing	$2,500	(Should fall under Warranty)	
Extend Lightning Protection	$600	(Should Fall under Warranty)	
Repair water damaged drywall	$3,500	(Should Fall under Warranty)	
Install outside Corners	$900		
Correct termination of drywall at the window headers	$1,500	(Should fall under Warranty)	
FACILITY TOTALS:	$43,750	$0	$0
Waterproof Foundation Wall	**$5,000--$200,000**	**(Should fall under Warranty)**	

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

435 SOUTH THIRD STREET FACILITY

	Immediate	Short Term	Long Term
Turn off non-required electrical	$500		
	$1,000		
Install elec. Tape on Sprinkler Pipe	$600		
Turn off water and drain pipes	$700		
Roof Management Program 20,000 @ $.15	$3,000		
Replacement 20,000 @ 6.00		$120,000	
Install Iron Gates and Bars	$17,000		
Install Razor Wire	$900		
Repairs to Fascia	$100,000		
FACILITY TOTALS:	**$123,700**	**$120,000**	**$0**

LOUISVILLE WATER COMPANY
FACILITY SURVEY
ESTIMATED COST/DEFERRED MAINTENANCE

ALLMOND AVENUE FACILITY

	Immediate	Short Term	Long Term
Concrete and Joint Repairs see photos no. 65, 66 Immediate = repairs to asphalt Short term= drives and new topping	$25,000	$125,000	
Traffic Marking	$2,500		
Repairs to Dock Cover see photo no. 68, 69		$6,000	
Install Truck Bumpers	$4,000		
Roof Management Plan 57,000 @ .15	$8,550		
Roof Replacement 30,000 sq. ft. @ 6.00	$180		
Caulk Windows	$7,000		
Correct Rotating Wall tie wall to slab, grout crack	$7,000		
Repair Roof Ladder	$300		
Replace storage bldg. Repairs exceed value of bldg.	$30,000		
Renovate Offices 30,000 sq.ft. @ $12		$360,000	
HVAC Modernization Immediate = 30,000 sq.ft.X .$85 Short term 50,000 sq.ft. X .$.65	$25,500	$32,500	
ADA Improvements unisex restroom at $12,000 each		$12,000	$36,000
FACILITY TOTALS:	**$110,030**	**$535,500**	**$36,000**

Drainage and Solids Management Improvements

Crescent Hill
Water Treatment Plant

STATE OF KENTUCKY
JERRY L. ANDERSON
20790
LICENSED
PROFESSIONAL ENGINEER

CH2MHILL
Quest Engineers
July 2000

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Appendixes

SECTION ES

Executive Summary

At 180 mgd firm capacity, the Crescent Hill Water Treatment Plant (CHWTP) is the primary source of treated water for the Louisville Water Company (LWC). The second source of treated water comes from the BE Payne Water Treatment Plant (BEPWTP) with a firm capacity of 45 mgd. LWC retained CH2M HILL in association with Quest Engineers to evaluate the present drainage and solids handling system for CHWTP. This report presents the findings of the evaluation and proposed recommendations for improvements.

The present drainage and solids handling facilities were inspected and inventoried, and process wastewater streams were sampled and analyzed to characterize CHWTP wastewater. Regulations pertaining to drinking water standards and potential impacts of wastewater disposal were reviewed and summarized. Drainage and solids handling operations were evaluated to identify potential areas needing improvement or opportunities to reduce wastewater volume. LWC practices were compared to five similar water treatment plants owned and operated by other utilities. The current method of solids handling and disposal was compared on a capital and annual cost basis to two alternative methods. Conclusions and recommendations were prepared and are presented below.

Overview of Recommendations

The following conclusions and recommendations are concisely presented in the sequence of general need to implement. Some recommendations are dependent on others being implemented first. These are presented in more detail in Section 8.

1. **Develop plan for lagoon operation, filling sequence, and cleaning.** Three alternative long-term disposal methods and their life cycle costs were investigated for CHWTP process wastewater—1) continuation of piping wastewater to the BEP lagoons with mechanical removal of dewatered solids and land disposal; 2) disposal to the MSD system; and 3) construction of mechanical thickening and dewatering system near CHWTP with land disposal of solids. The present method of lagooning was by far the most cost-effective from a capital and annual cost basis and should be continued. However, many maintenance activities will soon be needed to keep this system in good functioning order. One of the major maintenance activities will be to clean the BEPWTP lagoons. Overall, the lagoons are about three-fourths filled, and at the current rate of solids production all lagoons will be filled in less than 8 years if none are cleaned. Lagoon No. 2 is the most completely filled lagoon and should be cleaned first, which will require an estimated cost of $3.6 million for excavation and disposal. A land disposal (monofill) site should be investigated as the most cost-effective means of disposing of water treatment residuals.

2. **Conduct a geotechnical investigation of Lagoon No. 1 and 4** to determine if they are leaking, as reported, and complete berm repairs if needed. It would be prudent to make

needed repairs as soon as possible while the lagoons are out of service and before they are needed.

3. **Modify BEPWTP backwash drain piping** to allow discharge from Lagoon No. 3 to Lagoons No. 1 and 2, and modify the collector box drain to divert water from Lagoon No. 2 to Lagoon No. 1. The estimated cost of modifying the 42-inch backwash drainage system is $265,000. A cost estimate for the collector box drainage improvements was not prepared because the requirements are not well defined; however, a gravity drainpipe to Lagoon No. 1 would be a nominal cost, if pumping was not required.

4. **Build new backwash water holding tank and pump station.** Presently, filter backwash water must be routed through the backwash hold tank to avoid an overflow into the MSD combined sewer system. This places the plant in a vulnerable condition should maintenance problems occur in the backwash wastewater drainage system. A volume of 0.8 MG, equal to the existing tank is recommended to operate in parallel with existing tank to provide flexibility in operations and to be adequately sized for use beyond 2010. The new tank would be interconnected with the backwash drainpipe presently flowing into the existing tank. A new solids-handling pump station would be connected to both tanks so that either tank could be removed from service without interrupting filter backwashing.

5. **Develop and use the Zorn Avenue Lagoon.** Based on a hydraulic computer model investigation, the 24-inch drainage pipeline in its present state will be unable to transport to the lagoons the projected filter backwash wastewater plus the basin sludge flows. The model indicates that an overflow at the manhole connected to the coagulation sludge line in Reservoir Avenue and at the weir boxes in the Coagulation Control Houses would result. The predicted overflow condition at the weir boxes could be reduced or eliminated by improving the carrying capacity of the 24-inch pipeline. Construction of the Zorn Avenue lagoon would be the first step to inspecting or making improvements to the 24-inch pipeline. Once the lagoon is completed and placed in service, the 24-inch pipeline could be removed from service for inspections and maintenance. The estimated cost to construct this lagoon is $500,000.

6. **Check air release valves on 24-inch pipeline to BEP lagoons.** Air may be trapped at high points in the pipeline. Replace valves and bleed off trapped air if inoperative valves are found.

7. **Inspect sludge and drain pipelines using a TV camera.** Solids deposition is suspected to be the cause of the poor carrying capacity of the drainage system. To determine the extent of solids deposition and cleaning requirements, all drain and sludge lines should be inspected by TV, starting with the 24-inch pipeline to BEP Lagoons. The coagulation sludge and backwash wastewater could be converted to the drainage pipeline while inspections are conducted.

8. **Improve pipeline capacity by jet cleaning or pigging.**

9. **Install flow rate metering** for each of the 14 basin sludge flows at their weir boxes. Sludge flows out of identical basins appears to be highly variable because flow rate measurements are difficult to take. An ultrasonic flowmeter could be installed and programmed for each type of weir used and would provide a direct readout. The pump

discharge flow rate out of the backwash holding tank should be measured with a magnetic flowmeter on the 12- and 20-inch lines. Alternatively, a single pipeline leaving the pump station could be metered before it splits into the 12- and 20-inch force mains. The backwash holding tank level should also be monitored.

10. **Establish target basin sludge flows** of 0.25 and 0.35 mgd, each, for the North and South Coagulation Basins, respectively, and 0.1 mgd, each, for the softening basins. This will be an initial target value. These values should be further reduced, if possible to do without clogging sludge lines. Adding flow rate measurement will facilitate testing for optimum sludge flow rates. If softening is resumed in any of the basins the target sludge flow rate should be increased to 0.3 mgd per basin based on anticipated solids loading.

11. **Clean and remove vegetation from the lagoons.** The vegetation restricts visibility and ability to determine condition of the lagoons. When the lagoons are to be cleaned the vegetation will hamper the cleaning process. The resulting vegetative debris intermixed with the residuals may not be compatible with the intended use of the solids or the disposal site. The trees attract beavers, which can result in more maintenance to clean out debris from the outlet structures. As a result, all lagoons should be kept clean of vegetation. Once cleaned the vegetation may be discouraged from returning by maintaining a foot or so of water above the bottom or accumulated solids in the bottom. The underdrain pipe that discharges into the outlet structure may need to be plugged or valved off to maintain a certain level of water.

12. **Improve 24-inch pipeline reliability along River Road** by installing a double-barrel configuration at problem areas such as Harrods Creek or other creek crossings. Valved connections with pig launchers would be installed at each end of the two barrels and either barrel could be inspected or cleaned while the other was kept in service. The estimated cost to install a parallel pipeline at a 150-foot-long creek crossing using jack-and-bore installation with 30-inch casing, fittings, four isolation valves, two pig launchers and appurtenances is $235,000 per crossing.

13. **Remove cover of existing backwash holding tank.** Because the tank used to be a clear well, it is covered and below grade. The cover hampers maintenance and cleaning and the pumps are also difficult to access. The walls would be extended above grade with handrails on top to facilitate operation and maintenance of the tank.

14. **Exercise valves and paint metalwork in the lagoons at BEPWTP.**

15. **Conduct total suspended solids (TSS) sampling of the softening basin sludge flows** on a regular schedule so that a relationship between sludge flow rates, lime dosage, and TSS concentrations can be established. This relationship will allow solids production to be more accurately projected.

16. **Replace sludge butterfly valves** with V-port plug valve or L Series knife gate valve with a V-port that is designed for flow control of slurries, both manufactured by DeZurik. The existing valves do not provide good control of flow rates. Consider automating flushing of the sludge lines to reduce overall volume wasted.

17. **Construct new higher manhole on LWC property** connected to the coagulation sludge pipeline and seal off existing manhole that overflows. A higher manhole will permit a higher driving head in the coagulation sludge pipeline to increase its carrying capacity.

18. **Establish filtered water turbidity goals and investigate feasibility of filter-to-waste.** The turbidity limits for individual filters established by the Interim Enhanced Surface Water Treatment Rule (IESWTR) require filters by December 2001 to become sufficiently ripened to produce filtered water initially below 1.0 NTU, and below 0.5 NTU after 4 hours of operation. Presently, filters are reportedly producing water at less than 0.25 NTU immediately after backwash, which will comply with the rule. If turbidity levels are lowered further in future regulations, or if voluntary strategies, such as the Partnership for Safe Water Program, are adopted resulting in lower target turbidity levels, the feasibility to provide filter-to-waste or some type of filter media conditioning should be investigated. Establish LWC goals for filtered water turbidity and determine what filter renovations and operating modifications are required to meet the goals. Conduct turbidity profiling for all filters to determine duration of filter media ripening after backwashing and whether or not FTW could improve filtered water quality. If FTW appears to be feasible, consider a pumped FTW system that would keep FTW water isolated from other process wastewater so that FTW could be recycled.

19. **Investigate the feasibility of improving filter backwash effectiveness** by increasing rate and shortening duration.

20. **Investigate the feasibility of longer filter runs,** if turbidity breakthrough and excessive head losses do not occur.

21. **Investigate the feasibility of higher filtration rates,** again if turbidity breakthrough and excessive head losses do not occur.

22. **Consider resuming the practice of recycling backwash water** to the raw water reservoir to significantly reduce process wastewater flows. Although many utilities are now avoiding this practice because of the threat of *Giardia* and *Cryptosporidium* cysts being recycled to the plant influent in greater concentration, recycle to the raw water reservoirs in CHWTP appears to be permissible under the proposed FBR Rule. Before considering this practice again, however, filter backwash water should be sampled for *Giardia* and *Cryptosporidium* and compared to occurrence in the raw water.

23. **Provide basin transfer pumps** to significantly reduce the amount of water to be drained when emptying basins. One pump could be installed and shared for each of the three basin groups to withdraw the top two-thirds of the basin water and discharge to the basin influent flume. Alternatively, a portable pump could be used.

24. **Develop a plan for cleaning the raw water reservoir** at regular, more frequent intervals than done in the past (e.g. no less often than every five years) so that the cleaning projects will be simpler and quicker. When time to clean again, use manual wash down to drainage to avoid damaging the plastic liner. Other methods of cleaning could be used if certain precautions to protect the plastic liner are followed.

25. **Consider a 3-inch concrete protective layer** in the bottom of the reservoir after the next scheduled cleaning. If manual cleaning is still considered too labor intensive and time consuming even if the reservoir basins are cleaned more frequently, a concrete overlay should be installed to protect the liner from more invasive methods of cleaning. The concrete overlay will avoid expensive liner repair costs and prolong the liner's life. This protection may be useful for either dredging or mechanical cleaning. Assuming a bottom surface area of about 700,000 square feet in both basins, total cost would be $1,400,000.

SECTION 1

Introduction

At 180 mgd firm capacity, the Crescent Hill Water Treatment Plant (CHWTP) is the primary source of treated water for the Louisville Water Company (LWC). The second source of treated water comes from the BE Payne Water Treatment Plant (BEPWTP) with a firm capacity of 45 mgd. LWC retained CH2M HILL in association with Quest Engineers to evaluate the present drainage and solids handling system for CHWTP. This report presents the findings of the evaluation and proposed recommendations for improvements.

The project objectives consist of the following activities:

- Inventory existing drainage and solids handling systems
- Profile individual waste streams by water quality characteristics
- Provide an update on current and proposed residual management regulations
- Evaluate several issues pertaining to process wastewater operations
- Recommend a method of raw water reservoir cleaning
- Compare LWC's solids handling operations to industry standards
- Investigate alternative process wastewater disposal options, and
- Prepare a conceptual design report

The report is organized by groups of activities as follows:

Section 2 describes the waste streams generated from CHWTP including existing and projected flow rates and water quality characteristics

Section 3 describes the existing drainage and solids handling facilities and their life expectancy.

Section 4 addresses several aspects of the drainage and solids handling operations, including flow rate monitoring recommendations, impacts of backwash and sludge handling operations, opportunities for reducing process wastewater, assessment of lagoons and the drainage system to the lagoons, and raw water reservoir cleaning.

Section 5 presents the results of a survey of solids handling practices in other utilities.

Section 6 provides an update on several regulations that could impact solids handling operations.

Section 7 investigates alternative process wastewater disposal options and compares their costs.

Section 8 presents the conclusions and recommendations.

SECTION 2

WTP Flow Rates and Wastewater Stream Profiles

WTP Flow Rates

Wastewater streams are dependent on plant production rates. Table 2-1 shows the current plant firm capacities (maximum design flow rates), minimum flow rates, and projected ten-year average flows for both plants. For the next 10 years, the CHWTP is estimated to produce an average of 114 mgd. The 10-year average flow was assumed to coincide with the projected flow rate for the midpoint of the 10-year period, or 2005. Table 2-2 presents the historical water production rates at both plants from 1976 to 1999. After a period of declining production, the most recent 10 years of flow records show steady growth. Using multiple regression for the last 10 years of flow records, production rates for both plants are projected for the next 10 years in Table 2-2. Through 2005 these projections track well with the 1995 Facilities Plan; however, the growth projected by the facilities plan flattens out at this point while Table 2-2 shows continued growth. Figure 2-1 presents the last 10 years of data and the projected production rates through Year 2010, assuming linear growth.

TABLE 2-1
WTP Capacities and Design Flow Rates

Flow Rate Condition	BE Payne	Crescent Hill	Total
Maximum Design Flow Rate, mgd	60	240	300
Firm Flow Rate, mgd	45	180	225
Minimum Flow Rate, mgd	16	90	106
Projected 10-year average flow rate, mgd	40	114	154

Process Wastewater Flow Streams

Several sources of process wastewater are generated in CHWTP. The projected water production rates were reviewed to compute what future wastewater flows will occur in CHWTP. The historical plant production rates were reviewed and future production was projected. A list of the process wastewater facilities is provided in Table 2-3.

Presedimentation

Heavy solids settle out by gravity in the two raw water reservoirs, which have a total volume of 106 MG. When the reservoirs are drained and cleaned there is an enormous amount of water that first must be disposed, then the solids have to be removed. The North Reservoir has been recently lined to prevent leakage. The South Reservoir has been dewatered and cleaned and will soon be relined as well. The reservoir drainage system connects to a 36-inch pipe in Reservoir Avenue, roughly 11,000 feet long, traveling down

Figure 2-1
Historical WTP Flows and Projections

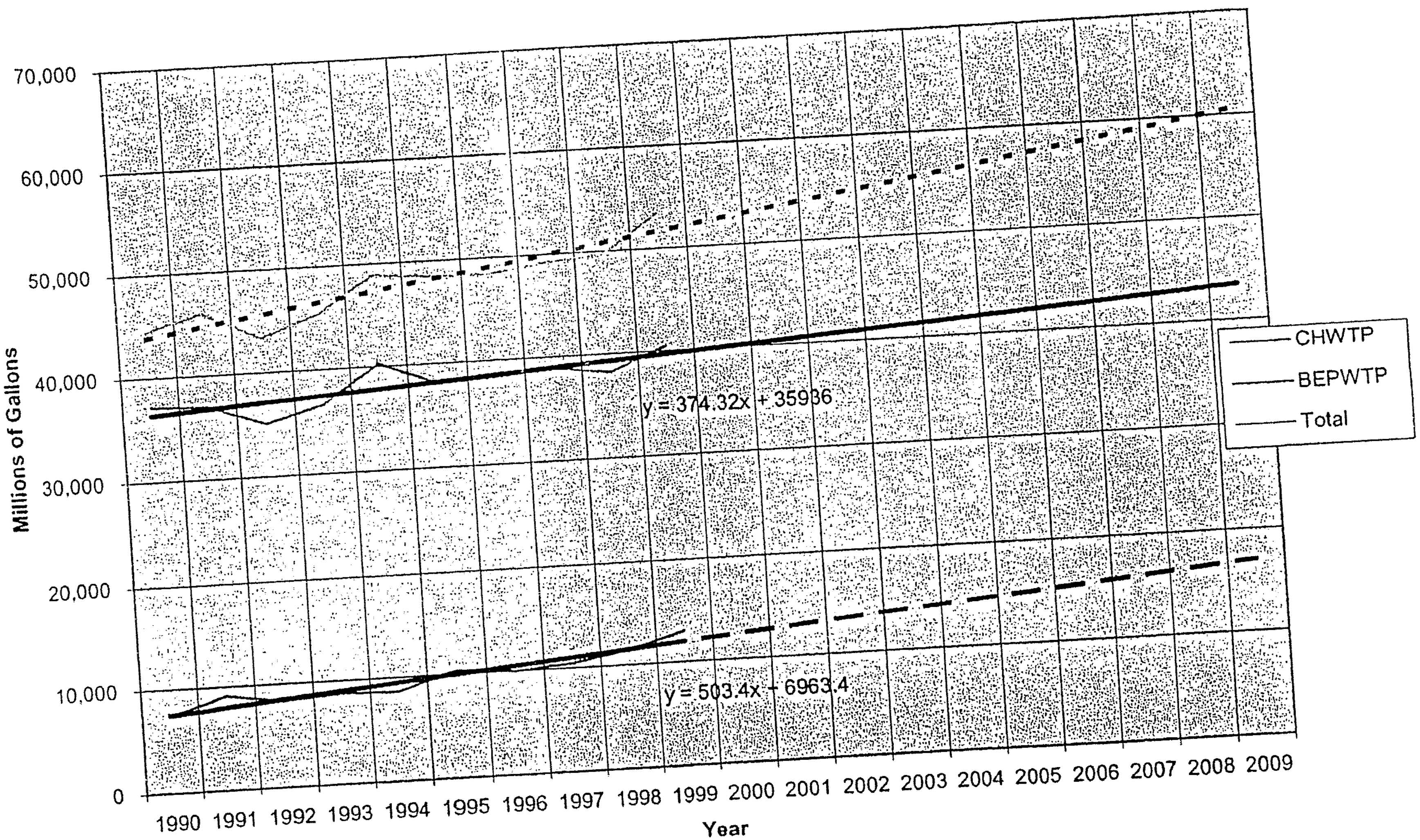


TABLE 2-2
Total Water Produced

Year	CHWTP		BEPWTP		Total		% BEP
	MG/year	MGD	MG/year	MGD	MG/year	MGD	
1976	46,043	126	0		46,043	126	0.0%
1977	49,066	134	886	2.4	49,952	137	1.8%
1978	43,009	118	4,174	11.4	47,183	129	8.8%
1979	41,063	113	4,760	13.0	45,823	126	10.4%
1980	41,389	113	5,510	15.1	46,899	128	11.7%
1981	37,202	102	6,661	18.2	43,863	120	15.2%
1982	37,271	102	8,181	22.4	45,453	125	18.0%
1983	37,635	103	8,162	22.4	45,797	125	17.8%
1984	37,742	103	7,777	21.3	45,518	125	17.1%
1985	37,156	102	7,596	20.8	44,752	123	17.0%
1986	39,516	108	4,950	13.6	44,466	122	11.1%
1987	38,273	105	6,877	18.8	45,150	124	15.2%
1988	38,106	104	7,461	20.4	45,567	125	16.4%
1989	36,213	99	6,798	18.6	43,012	118	15.8%
1990	37,082	102	7,374	20.2	44,456	122	16.6%
1991	36,905	101	9,093	24.9	45,998	126	19.8%
1992	35,058	96	8,310	22.8	43,368	119	19.2%
1993	36,616	100	8,730	23.9	45,346	124	19.3%
1994	40,248	110	8,625	23.6	48,872	134	17.6%
1995	38,109	104	10,342	28.3	48,451	133	21.3%
1996	38,396	105	9,987	27.4	48,383	133	20.6%
1997	38,842	106	10,434	28.6	49,276	135	21.2%
1998	38,258	105	11,429	31.3	49,686	136	23.0%
1999	40,435	111	12,997	35.6	53,431	146	24.3%
2000	39,679	109	11,997	32.9	51,677	142	23.2%
2001	40,054	110	12,501	34.2	52,554	144	23.8%
2002	40,428	111	13,004	35.6	53,432	146	24.3%
2003	40,802	112	13,508	37.0	54,310	149	24.9%
2004	41,176	113	14,011	38.4	55,187	151	25.4%
2005	41,551	114	14,514	39.8	56,065	154	25.9%
2006	41,925	115	15,018	41.1	56,943	156	26.4%
2007	42,299	116	15,521	42.5	57,821	158	26.8%
2008	42,674	117	16,025	43.9	58,698	161	27.3%
2009	43,048	118	16,528	45.3	59,576	163	27.7%
2010	43,422	119	17,031	46.7	60,454	166	28.2%

Flow Projection Equations:

CHWTP: $= 374.32x + 35936$

BEPWTP: $= 503.4x + 6963.4$

Projected data

Note: 1999 was a drought year

TABLE 2-3
Process Wastewater Disposal Facilities

Facility	Qty	Size/Diameter/Length	Comments
Pipelines			
Cast iron pipe reservoir drain	1	36 inches; 11,000 feet long	From reservoir basins to the future Zorn Ave. lagoon site and outfall
North Coagulation Basin sludge collection pipelines	4	16 inches	Discharge from basins to sludge draw-off weir tanks
South Coagulation Basin sludge collection pipelines	4	20 inches	Discharge from basins to sludge draw-off weir tanks
Coagulation sludge draw-off tank drains	4	12 inches	Discharge from weir tanks to 16-inch coagulation basin sludge pipeline
Concrete pipe coagulation basin sludge pipeline	1	16 inches; 1,015 feet long	Connects North and South Coagulation Basins to drain line
Coagulation sludge pipeline	1	30 inches; 8,100 feet long	Connects to the 24-inch BEP Lagoon pipeline in Zorn Ave.
Backwash wastewater drain	1	60 inches reducing to 42 inches	Discharges to backwash water holding tank
Softening basin sludge collection pipelines	6	8 inches	Discharge to sludge draw-off weir tanks
Softening basin sludge draw-off	1	12 inches	Discharges into wash water holding tank
Backwash Water Holding Basin force mains	2	12 and 20 inches; 2,540 feet long in parallel	Terminate and connect into 30-inch wash water drain in Reservoir Ave.
Backwash Water Holding Tank gravity drain	1	30 inches; 8,100 feet long	In parallel with coagulation basin and reservoir drainage pipelines
BE Payne Lagoon pipeline	1	24 inches, 35,450 feet long	Begins at Zorn Ave. and terminates at Lagoon No. 1.
Filter Backwash Water			
Backwash Water Holding Tank	1	0.8 MG	Collects filter backwash water and softening basin sludge
Backwash Water Holding Tank Pumps	3	2 mgd quick-disconnect solids handling pumps	Other pumps in tank are not in use.
Lagoons			
BE Payne Lagoons	4	Combined area: 44 acres	See Table 3-4 for additional data
Zorn Lagoon (future)	1	Approximate area is 3.5 acres	Site needs to be developed
Metropolitan Sewer District			
MSD direct connection to Backwash Holding Tank	1	42 inches, gradually increasing to 127 inches	Only used in emergencies. Discharges to combined sewer system.
Raw water reservoir auxiliary drain (planned)	1	30-inch outlet	Design is underway to provide a raw water reservoir drainage system of up to 24 mgd capacity.

Zorn Avenue to a future lagoon site on the Zorn Avenue Pump Station site and continuing to a permitted outfall to the Ohio River. At this point through different valving configurations, drainage water can be diverted to a 35,450-foot, 24-inch pipeline that transports sludge by gravity to four lagoons behind BEPWTP, or to the outfall into the Ohio River adjacent to the Zorn Avenue Pump Station. The drainage systems for the coagulation basins also connect to the same 36-inch pipeline in Reservoir Avenue.

Coagulation

There are four North and four South Coagulation Basins. The basins within each of the two groups are equally sized, but the South Group is 39 percent larger than the North Group in basin volume and surface area. Each basin is equipped with mechanical sludge scrapers that move sludge to a hopper at the center of the basin. The north basins each have a 16-inch sludge drain line that discharges into a V-notch weir box so that flow can be observed and monitored. The south basins each have a 20-inch sludge drain line that discharges into a V-notch weir box. Each weir box has a butterfly valve used for manually controlling sludge flow rate. The valves do not consistently control flow rates. They must be frequently operated to flush the lines to prevent clogging, which often causes inconsistent sludge flows.

Sludge flow rates are not precisely measured at the weirs but are visually observed in an attempt to balance flow rates among all basins. After weir overflow the sludge is collected into a 8,100-foot, 30-inch pipeline parallel to the 36-inch drain line that also terminates at Zorn Avenue and River Road. At the termination point it also connects to the 24-inch pipeline routed to the lagoons.

Softening

There are six equal-size softening basins on the south side of CHWTP. Sludge handling is similar to the coagulation basins in that there are mechanical sludge scrapers that move sludge to a central hopper and sludge drains continuously into weir boxes. Softening has not been practiced for several years. Only small dosages of lime are currently added for pH adjustment. As a result, the solids loading from the softening basins is small compared to the coagulation basins.

Coagulation and Softening Sludge Flows

In 1980 coagulation and softening sludge flow rates were monitored and recorded. The results are shown in Table 2-4. An average of 0.21 mgd for the coagulation basins and 0.23 mgd for the softening basins was recorded in 1980. These flow rates amounted to 2.7 percent of total water production that year for coagulation and softening sludge flow rates combined.

The sludge flow rates of all basins were measured on two occasions in April 2000 to compare with the 1980 data. Table 2-5 shows the measurements taken and computes an average waste sludge flow rate for all basins that were in service. The table also shows the flow rates that were assumed in the past as a result of old operating data. The actual measurements from 1980 and April 2000 are similar, but less than the flow rates assumed in the past. For example, Table 2-6 summarizes the basin characteristics and flow rate comparisons. Comparing April 2000 to 1980, the coagulation sludge flow rate has increased and the softening sludge flow rate has decreased, making the overall wastewater flow rate about the same as a percent of total water produced (2.7 versus 2.86 percent, respectively) in both years. Sludge flow rates for both years were considerably less than the assumed flow rates, which were 4.55 percent of total water produced.

The ideal or target flow rates for basin sludge flows should be as low as possible without allowing the sludge drain lines to clog.

TABLE 2-4
Basin Sludge Flows for 1980

Month	Total Water Treated, MG	Coagulated Sludge, MG	Softening Sludge, MG
January	3,482	46.9	46.5
February	3,344	43.8	43.5
March	3,707	46.9	46.5
April	3,375	48.1	37.6
May	3,551	52.5	34.0
June	3,668	51.8	30.0
July	3,876	53.6	45.5
August	3,787	53.6	40.5
September	3,365	50.8	45.0
October	3,119	49.9	46.3
November	2,962	51.8	45.0
December	2,949	53.6	46.5
Total	41,185	603.3	506.9
Average	3,432	50.3	42.2
Rate, mgd	112.84	1.65	1.39
Max month	3,876		
Max/Avg	1.13		
Avg. Rate per Basin, mgd		0.21	0.23
Percent of Total Treated		1.46%	1.23%

TABLE 2-5
Field-Measured Basin Sludge Flow Rates

Location	Weir No.	V-notch Angle, deg.	Weir Sludge Flows, mgd			
			Assumed	04/04/2000	04/13/2000	Average
North Coag. Basin	1	60	0.400	0.105	0.239	0.172
North Coag. Basin	2	60	0.400	0.216	0.239	0.228
North Coag. Basin	3	60	0.400	0.338	0.586	0.462
North Coag. Basin	4	60	0.400	0.148	0.338	0.243
		Average	**0.400**	**0.202**	**0.351**	**0.276**
		Subtotal	**1.60**	**0.81**	**1.40**	**1.11**
South Coag. Basin	5	60	0.400	0.288	0.518	0.403
South Coag. Basin	6	60	0.400	0.288	0.166	0.227
South Coag. Basin	7	60	0.400	NS	NS	NS
South Coag. Basin	8	60	0.400	0.393	0.343	0.368
		Average	**0.400**	**0.323**	**0.342**	**0.333**
		Subtotal	**1.60**	**0.97**	**1.03**	**1.00**
Softening Basin	1	90	0.300	0.181	0.181	0.181
Softening Basin	2	90	0.300	0.075	0.104	0.089
Softening Basin	3	90	0.300	0.104	0.181	0.143
Softening Basin	4	90	0.300	NS	NS	NS
Softening Basin	5	90	0.300	NS	NS	NS
Softening Basin	6	90	0.300	0.050	0.104	0.077
		Average	**0.300**	**0.103**	**0.143**	**0.123**
		Subtotal	**1.800**	**0.410**	**0.570**	**0.490**
TOTAL			5.00	2.19	3.00	2.59

NS = Not in service

TABLE 2-6
Basin Sludge Flow Rate Summary

Source	North Coag. Basins	South Coag. Basins	Softening Basins	Total
Quantity	4	4	6	14
Surface area, sq. feet	20,736	28,900	22,500	
Basin volume, each, gallons	2,327,000	3,243,000	2,693,000	
Process treatment capacity, mgd	25.0	35.0	40.0	240
Hydraulic capacity, each, mgd[a]	18.8	26.2	30.0	
Total hydraulic capacity, mgd[b]	75.2	104.8	180.0	180.0
Assumed sludge flow rate per basin, mgd	0.40	0.40	0.30	
Total assumed sludge flow rate per basin group, mgd	1.6	1.6	1.8	5.0
Percent of assumed sludge flow/current avg. production rate	**3.48%**	**2.50%**	**1.64%**	**4.55%**
Average field-measured sludge flow rate per basin, mgd[c]	0.28	0.33	0.12	
Field-measured sludge flow rate per basin group, mgd[c]	1.11	1.33	0.74	3.17
Percent of field-measured sludge flow/total production[c]	**2.39%**	**2.06%**	**0.66%**	**2.86%**
Sludge flow rates per basin for 1980, mgd	0.21	0.21	0.23	
Sludge flow rates per basin group for 1980, mgd	0.83	0.83	1.39	3.04
Percent of recorded sludge flow for 1980/total production[ce]	**1.75%**	**1.26%**	**1.23%**	**2.70%**

Notes:

[a]Coagulation basin hydraulic loading rate, gpm/sf	0.6295
[b]Plant design flow rate, mgd	180
[c]Sludge flows field-measured twice in April 2000	
[d]Average WTP production rate assumed for April 2000, mgd	111
[e]Average WTP production rate in 1980, mgd	113
[f]Current average WTP production rate, mgd	110

Filter Backwash Wastewater

CHWTP has 33 rapid sand filters configured in three groups (see Table 2-7). Filter backwash water is discharged into an 800,000-gallon wash water holding tank that is below ground. Sludge from the softening basins also flows by gravity into the wash water holding tank. The combined wastewater is pumped out of the tank by three 2-mgd, quick-disconnect, submersible pumps. Three additional sludge pumps are inside the holding tank but are not used. The pumped wastewater is discharged into parallel 2,540-foot, 12- and 20-inch force mains that tie into a second 30-inch gravity main in Reservoir Avenue, although connection to other pipelines is possible through different valving configurations. Similar to the 36-inch basin drain and 30-inch coagulation sludge pipelines, the 30-inch backwash wastewater pipeline terminates at Zorn Avenue and River Road and connects to the 24-inch pipeline routed to the lagoons.

TABLE 2-7
Rapid Sand Filters

Group	No. of Units	Capacity, mgd	Target Run Time, hours	Backwash Volume, gal
North	12	3	72	192,650
South	6	6	96	385,300
East	15	6	120	385,300

The characteristics of the filters are summarized in Table 2-8. For projecting future backwashing wastewater volume, the water production rates per filter per backwash cycle were computed in the table. The current target duration of each group of filters was taken into consideration and the volume of wastewater generated is shown as a percent of total water filtered. To evaluate wash water holding tank and conveyance facilities, a maximum flow condition was estimated based on a 2-week maximum flow event that would be expected to occur in the summer time. The 2-week maximum flow peaking factor was assumed to be 1.25, based on the maximum month-to-average demand peaking ratio being between 1.1 and 1.2. Based on the 2-week maximum flow rate, a selection of filters from each group was assumed to be in operation and the number of filters to be backwashed was determined. Backwash volume was calculated from these assumptions.

Others

There are other process wastewater flow streams, such as sample streams, basin wash downs, and flushing, but these are insignificant volumes in regard to disposal and are not addressed in this report.

Water Quality Analyses

A description of each waste stream indicating water quality characteristics was prepared. One round of samples as shown in Table 2-9 were collected on April 4, 2000, with the assistance of LWC staff. Samples were analyzed for the Kentucky Pollutant Discharge Elimination System (KPDES) discharge permit monitoring parameters of total suspended solids, pH, and total chlorine residual by a local laboratory service, EnviroData Group. Biochemical oxygen demand (BOD) was analyzed on selected samples. The results are summarized in Table 2-9.

TABLE 2-8
Filter Backwash Wastewater Flows
Crescent Hill WTP

Item	Filter Group			Total
	South	East	North	
Present Characteristics				
Quantity of filters	6	15	12	33
Area per filter, sq. feet	2,100	2,100	1,050	5,283
Design flow per filter, mgd	8.89	8.89	4.45	
Filtration rate at design flow, gpm/sf	2.94	2.94	2.94	
Total group design capacity, mgd	53	133	53	240
Normal flow per filter, mgd	6	6	3	
Filtration rate at normal flow, gpm/sf	1.98	1.98	1.98	
Typical backwash volume, gal	385,300	385,300	192,650	
Typical run time, hours	96	120	72	
Mid-Point Year 2005				
Flow rate at mid-point year (2005)[a]	25.3	63.3	25.4	114
Typical No. of filters in operation	4.2	10.6	8.5	23
Avg. No. of backwashes/day	1.06	2.11	2.82	6.0
No. of backwashes/day--rounded	1	2	3	6
Avg. backwash volume, gal/day	407,000	813,000	543,000	1,763,000
Avg.backwash/tot. production vol.	1.61%	1.28%	2.14%	1.55%
Maximum Flows Year 2010				
Avg. flow rate at design year (2010)[a]	26.4	66.1	26.5	119
2-week maximum flow rate	33.0	82.6	33.1	148.8
2-week max. No. of filters	5.5	13.8	11.0	30.3
2-week max. backwashes/day--avg.	1.4	2.8	3.7	7.8
2-week max. backwashes/day--rounded	1	3	4	8
2-week max. backwash volume/day	385,300	1,155,900	770,600	2,311,800

Assumptions:

[a]Individual filter groups were prorated from total design capacities; total flow was projected from last 10 years of record

[b]Two week max flow/avg. flow 1.25

TABLE 2-9
Laboratory Results for Physical and BOD Analyses

Sample Location	pH	Total Chlorine Residual, mg/L	Total Suspended Solids, mg/L	Turbidity, NTU	BOD_5, mg/L
South Coag. Influent	7.2	< 0.020	29		
South Coag. Weir No.8	7.1	< 0.020	51		
South Coag. Weir No. 5	7.0	< 0.020	332		
South Coag. Weir No. 6	7.1	< 0.020	49		
South Coag. Weir Composite	7.2	< 0.020	52		< 1
North Coag. Influent	7.3	< 0.020	25		
North Coag. Weir No. 1	7.3	< 0.020	636		
North Coag. Weir No. 2	7.3	< 0.020	120		
North Coag. Weir No. 3	7.3	< 0.020	333		
North Coag. Weir No. 4	7.3	< 0.020	88		
North Coag. Weir Composite	7.3	< 0.020	752		2
Softening Influent 1-4	7.6	2.3	< 3		
Softening Influent 5-6	7.3	2.7	3		
Softening Weir No. 1	8.1	2.2	84		
Softening Weir No. 2	8.9	2.6	11		
Softening Weir No. 3	8.7	2.5	12		
Softening Weir No. 6	7.8	2.3	4		
Softening Weir Composite	8.4	2.6	17		< 1
South Filter Influent	8.2	1.2	< 3	1.0	
East Filter Influent	8.2	2.2	9.0	1.0	
North Filter Influent	8.2	2.1	< 3	1.5	
Filter Influent Composite	8.2	3.2	< 3	1.0	< 1
Wash Water Holding Tank	8.1	1.9	441		< 1
Lagoon No. 2 Influent	7.8	0.5	358	60	< 1

Notes:
BOD_5 = 5-day biochemical oxygen demand
Sampling sites can influence the results; and do not necessarily represent steady-state conditions
Lime distribution to the six softening basins tends to be uneven, which causes sludge concentrations to be uneven

Although the KPDES permits have not been renewed for 2000, the most recent permits for discharges from the lagoons at BEPWTP and the Zorn Avenue Pump Station regulate:

- Flow (monitoring only)
- Total Suspended Solids (TSS) (30 mg/L monthly average or 50 mg/L daily average)
- Total Residual Chlorine (monitoring only)
- pH (not less than 6.0 or more than 9.0 standard units)
- Floating solids or visible foam (none other than in trace amounts)

Based on these permit discharge limitations, the water quality characteristics presented in Table 2-9 comply with permit requirements, except for TSS that settle out to a great extent in the lagoon. TSS and turbidity are monitored monthly at the influent and effluent streams of the active lagoon in BEPWTP. Table 2-10 shows a summary of the monthly monitoring for the last several months. Although influent TSS and turbidity are highly variable and often quite high, the lagoon effluent TSS has always been below the daily maximum limit of 50 mg/L and typically well below the monthly average limitation of 30 mg/L. Turbidity monitoring is not required but it provides useful relationships for computing future residuals production further described below.

TABLE 2-10
Lagoon TSS and Turbidity

		Lagoon Influent			Lagoon Effluent		
Date	Number	Turbidity	Suspended Solids	SS/T	Turbidity	Suspended Solids	SS/T
Jan 1999	4	4,672	5,229	1.12	4.21	9	2.14
23-Feb-1999	4	204	303	1.49	2.04	8	3.92
23-Mar-1999	4	714	992	1.39	2.56	7	2.73
20-Apr-1999	4	510	683	1.34	3.49	1	0.29
18-May-1999	4	194	241	1.24	2.26	6	2.65
22-Jun-1999	4	109	182	1.67	1.21	3	2.48
21-Jul-1999	4	429	709	1.65	0.79	13	16.46
19-Aug-1999	4	98.0	346	3.53	0.83	15	18.07
21-Sep-1999	2	94.5	401	4.24	11.20	4	0.36
19-Oct-1999	2	129	223	1.73	11.60	7	0.60
17-Nov-1999	2	94.5	143	1.51	12.20	24	1.97
16-Dec-1999	2	2,386	4,040	1.69	4.91	11	2.24
19-Jan-2000	2	263	526	2.0	5.0	6	1.2
22-Feb-2000	2	15,000	8,972	0.60	10.70	44	4.11
21-March-2000	2	441	632	1.43	5.30	4	0.75
19-April-2000	2	350	757	2.16	3.97	6	1.50
23-May-2000	2	462	651	1.41	3.79	9	2.37
21-June-2000	2	1,647	1,964	1.19	14.1	6	0.43
Average		**1,544**	**1,500**	**0.97**	**5.6**	**10**	**1.82**

Thirteen priority pollutant metals were analyzed on a sample from Lagoon No. 2 influent. The results are shown in Table 2-11. Although the KPDES permits do not regulate or require monitoring of these metals, this analysis indicates that at the time of sampling none of these metals were present in unusual concentrations. If the wastewater streams continue reflecting these low levels, the accumulation of metals in the lagoon sludge will be considered insignificant as previously reported in the *Washwater Lagoon Investigation and Monitoring Program*, August 1992.

TABLE 2-11
Metals Analyses in Lagoon No. 2

Parameter	Reporting Limit, mg/L	Concentration, mg/L
Total Silver	0.01	Below Reporting Limit
Total Arsenic	0.05	Below Reporting Limit
Total Barium	0.01	0.08
Total Beryllium	0.002	Below Reporting Limit
Total Cadmium	0.005	Below Reporting Limit
Total Chromium	0.01	0.01
Total Copper	0.005	0.010
Total Iron	0.05	20
Total Mercury(Cold Vapor)	0.0002	Below Reporting Limit
Total Nickel	0.02	Below Reporting Limit
Total Lead	0.05	Below Reporting Limit
Total Selenium	0.05	Below Reporting Limit
Total Zinc	0.05	Below Reporting Limit

Residuals Characteristics

Residuals production is a function of the TSS and certain chemicals fed. For example, the amount of coagulation residuals can be calculated from raw water TSS and coagulant feed concentration. For ferric chloride about 2.9 pounds of ferric hydroxide sludge is generated for every 1 pound of coagulant applied. Raw water TSS produces sludge at a one-to-one ratio. Because WTP records monitor raw water turbidity rather than TSS, a relationship between TSS and turbidity is needed to determine TSS in the raw water. Based on the historical data shown in Table 2-10 a relationship of 1 Nephelometric Turbidity Unit (NTU) was assumed to equal 2.0 mg/L of TSS. This was approximated by ignoring the very high and low values in the table, which occur less frequently.

Residuals also accumulate as a result of lime addition. The lime softening process generates tremendous amounts of calcium carbonate sludge. In recent years LWC only applied lime at very low dosages to adjust pH of finished water. The amount of sludge generated from this lime addition is more difficult to predict. Table 2-12 shows that the average concentration of TSS in the lime sludge of CHWTP is 28 mg/L. This value is based only on a single day of grab samples and is subject to significant variance from day to day as lime dosages and sludge flow rates are adjusted by operators. The TSS concentration at the sludge flows measured on the sampling day represent 1.8 percent of the total lime solids fed. It would be prudent to conduct TSS sampling of the softening basin sludge flows on a regular schedule so that a relationship between sludge flow rates, lime dosage, and TSS concentrations can be established. This relationship will allow solids production to be more accurately projected.

TABLE 2-12
TSS in Softening Basin Sludge

Sample Location	Total Suspended Solids, mg/L
Softening Weir No. 1	84
Softening Weir No. 2	11
Softening Weir No. 3	12
Softening Weir No. 6	4
Average	**28**

Table 2-13 shows the operating records and estimated residuals produced for the last 6 years at both CHWTP and BEPWTP. The data were averaged and normalized to the plant production rates so that the concentration of residuals based on full water production could be estimated.

Assuming a solids concentration of 50 percent in the lagoons, this amounts to about an average of 1.5 million cubic feet of solids that have accumulated annually in the last 6 years at both plants combined.

Projected Wastewater Flows and Residuals

Table 2-14 provides target wastewater flows from the coagulation and softening basins and combines these with projected filter backwash wastewater flow rates for 2010. Because of increased plant production rates that are projected for both plants, the residuals flow rate and accumulation in the lagoons will increase. The target flow rates were established by reviewing sludge flow rates measured in April. Ideally, the flow rates will be as low as possible without causing excessive clogging. For the North and South Coagulation Basins the suggested sludge flow rates are 0.25 and 0.35 mgd per basin, respectively. For the softening basins the suggested sludge flow rate is 0.10 mgd per basin due to the low solids loading when only pH adjustment without softening is practiced.

Hardness in the raw water of BEPWTP increased significantly last year due to the startup of the Riverbank Infiltration (RBI) well. To counteract the increased hardness, partial softening to remove about 30 mg/L of hardness was instigated at BEPWTP in August 1999. As a result, the amount of sludge generated at BEPWTP this year has increased substantially over recent years.

Wastewater flow rates are projected to average 4.8 mgd with a maximum of 5.3 mgd during the 2-week maximum plant production rates of the summer, based on current treatment practices. Short-term basin drainage would add to these flow rates.

Current treatment practices include partial softening at BEPWTP and no softening at CHWTP. Based on these flow rates and current treatment practices, residuals from both plants are projected over the next 10 years to accrue at 60 million pounds per year, and at 1.9 million cubic feet volume per year, assuming the sludge consolidates in the lagoons to 50 percent solids concentration.

TABLE 2-13

Historical Flow Rate, Turbidity, and Chemical Usage and Residuals Estimate

Year	Flow Rate (mgd)	Avg Raw Turbidity (NTU)[1]	Lime Fed		Ferric Chloride Fed		Estimated Sludge Produced, mg/L				Est. Annual Sludge Produced[5]	
			lb.	mg/L	lb.	mg/L	Turbidity[2]	Lime[3]	Fe(OH)3[4]	Total	lb/year	cf/year
Crescent Hill WTP												
1994	110	32.3	3,548,648	10.6	3,074,443	9.2	64.6	0.2	26.6	91.3	30,657,000	983,000
1995	104	18.4	3,501,878	11.0	2,560,232	8.1	36.8	0.2	23.4	60.3	19,177,000	615,000
1996	105	48.8	4,039,105	12.6	3,602,441	11.2	97.6	0.2	32.6	130.4	41,766,000	1,339,000
1997	106	50.2	3,236,992	10.0	3,564,005	11.0	100.4	0.2	31.9	132.5	42,911,000	1,375,000
1998	105	53.0	4,138,866	13.0	4,451,877	14.0	106.0	0.2	40.5	146.7	46,799,000	1,500,000
1999	111	29.6	4,016,187	11.9	3,139,023	9.3	59.2	0.2	27.0	86.4	29,132,000	934,000
Average	**107**	**38.7**	**3,746,946**	**11.5**	**3,398,670**	**10.5**	**77.4**	**0.2**	**30.3**	**107.9**	**35,074,000**	**1,124,000**
BE Payne WTP												
1994	23.6	32.3	1,301,335	18.1	805,453	11.2	64.6	0.3	32.5	97.4	7,004,000	224,000
1995	28.3	18.4	679,478	7.9	1,045,927	12.1	36.8	0.1	35.2	72.1	6,218,000	199,000
1996	27.4	48.8	1,144,102	13.7	1,609,232	19.3	97.6	0.2	56.0	153.9	12,815,000	411,000
1997	28.6	50.2	1,056,782	12.1	1,272,110	14.6	100.4	0.2	42.4	143.0	12,443,000	399,000
1998	31.3	53.0	1,228,560	12.9	1,715,081	18.0	106.0	0.2	52.2	158.4	15,097,000	484,000
1999	35.6	29.6	1,304,634	12.0	1,469,171	13.6	59.2	0.2	39.3	98.7	10,698,000	343,000
Average	**29.1**	**38.7**	**1,119,149**	**12.8**	**1,319,496**	**14.8**	**77.4**	**0.2**	**42.9**	**120.6**	**10,713,000**	**343,000**
TOTAL	**136**									**110.6**	**45,787,000**	**1,467,000**

Notes:

[1]Raw water turbidities for both plants are averaged together

[2]Average TSS/turbidity unit (mg/L/NTU) 2.0

[3]Assumed percent of lime solids retained for pH adjustment 1.6%

[4]Lbs sludge per pound of ferric hydroxide applied 2.9

[5]Assumed percent dryness of lagooned solids 50.0%

TABLE 2-14
Projected Wastewater Flows and Residuals

Waste Flow Stream	Recommended Target Flow Rate, mgd	Maximum Flow Rate, mgd	Daily Solids Accumulation, lb[b]	Avg. Conc. in Waste Stream, mg/L	Annual Solids Accumulation, lb/year[bc]	Annual Solids Accumulation, cf/year[bc]
N. Coagulation Basins	1.0	1.0	42,700	5,100	15,586,000	
S. Coagulation Basins	1.4	1.4	59,600	5,100	21,754,000	
Softening Basins with pH adjustment	0.6	0.6	177	35	64,000	
Softening Basins with softening	1.0	1.0	56,900	11,400	20,769,000	
Maximum filter backwash[a]		2.3				
Average filter backwash[b]	1.8		2,100	143	767,000	
Subtotal, (CHWTP with pH adjustment)	**4.8**	5.3	**104,600**	**10,400**	**38,171,000**	**1,223,000**
Percent of CHWTP Production Rate	4.2%	3.7%				
Subtotal, (CHWTP with softening)	**5.2**	5.7	**161,300**	**21,700**	**58,876,000**	**1,887,000**
Percent of CHWTP Production Rate	4.5%	4.0%				
Subtotal, BEPWTP (all wastewater streams)			59,900		21,864,000	701,000
TOTAL, Both WTPs with pH adjustment at CHWTP					**60,035,000**	**1,924,000**
TOTAL, Both WTPs with softening at CHWTP					**80,740,000**	**2,588,000**

Assumptions:

N. Coag Basin sludge flow rate, mgd	0.25	
S. Coag Basin sludge flow rate, mgd	0.35	
Softening Basin sludge flow rate, mgd	0.10	pH adjustment only
Softening Basin sludge flow rate, mgd	0.30	1 treatment train for softening
N. Coag Basin flow split	41.8%	
S. Coag Basin flow split	58.2%	
TSS/turbidity	1.5	
Average filter influent turbidity, NTU	1.5	
Average lagoon sludge solids content	50.0%	
Sludge produced/hardness removed	2.0	
Hardness removed, mg/L[d]	30	

Notes:

[a]Based on a 2-week maximum flow condition

[b]Annual average for CHWTP, Year 2005, mgd — 114

[c]Annual average for BEPWTP, Year 2005, mgd — 39.8

[d]Hardness assumed to be removed in 2000 and beyond

If softening is resumed at CHWTP or if the rate of softening is increased at BEPWTP, the residuals production could substantially increase. Assuming partial softening at CHWTP might be practiced in the future by operating two of the six basins in a single-stage softening process, Table 2-14 shows the increased solids production that would result. In addition, the suggested target sludge flow rate would be increased to 0.30 mgd per basin for the basins where softening is practiced.

SECTION 3

Existing Facilities Assessment

The existing facilities for drainage and solids handling consist of the filter backwash wastewater holding tank and pumps, several gravity and pumped pipelines, and the lagoons. An assessment of these facilities is presented below.

Filter Backwash Wastewater Holding Tank and Pumps

The backwash holding basin used to be a clear well until the 1960s and is below grade. It is 90 feet in diameter and has a sloped bottom to facilitate drainage. The low water level for pumping is an elevation of about 528 feet and the high water elevation is 545 feet, with an effective volume of 800,000 gallons.

There is one vertical turbine pump (inactive) and three quick-disconnect submersibles that are difficult to retrieve with a crane and maintain. Each submersible is about 2 mgd and all three are never used simultaneously because of possible discharge pipeline overload. The tank has performed well for several years as a backwash holding basin, but there is no reason why the basin must remain closed at the top since it no longer contains potable water, unless the cover provides structural integrity. None of the pumps could be inspected due to inaccessibility. The three working submersibles were installed in 1992 and appear to be in good condition; however, their present capacity compared to original capacity is unknown.

A significant amount of leaves and hunks of calcium carbonate enter the drainage system through the softening basins and much of this debris settles in the holding tank. During an inspection on March 8, 2000, the access hatches were opened and a significant amount of solids up to a depth of 11 feet were observed on the southeast side of the tank. These solids are difficult to remove because the tank is below grade and it cannot be removed from service for an extended period of time. If the tank is bypassed, the softening basin sludge and filter backwash would be diverted to the Louisville and Jefferson County Metropolitan Sewer District (MSD) sewer system, which is a combined sewer system (CSO) that can discharge to Beargrass Creek during wet weather conditions.

Based on previous cleaning records the present day cost to clean the tank is about $25,000. This cost could be substantially reduced if the tank was opened and cleaned every year so that a significant amount of solids do not accumulate.

There are hydraulic constraints with backwashing. Filter backwashing has to be carefully scheduled to avoid overflowing the backwash holding tank. When water production increases, filter backwashing frequency also must increase. The backwash holding tank overflow is diverted to the MSD sewer system, and if used, a combined sewer overflow could result.

A spreadsheet model of the backwash holding tank was prepared to determine holding tank volume and pumping rate requirements for 2010. For the model, a 24-hour day is divided into 96 15-minute segments. Backwash volumes are input into the spreadsheet at the earliest time possible without exceeding the holding tank volume. Softening basin sludge is also input at a constant rate in 15-minute increments. Pumped water is removed at one of three rates: 2, 4 , or 5 mgd to simulate the three existing submersible pumps. A running total volume of wastewater accumulated in the holding tank is computed. Four scenarios were investigated (see Table 3-1).

TABLE 3-1
Backwash Holding Tank Modeling Scenarios and Results

Scenario	No. of South Filters	No. of East Filters	No. of North Filters	Results
2-week maximum flow rate + sludge flow at 1.0 mgd. (Table A-1)	1	3	4	Entire 0.8 MG tank is required and must be kept clean. Maximum pumping rate is 4 mgd. Pumping rate can be reduced to 2 mgd at 2145 hours until 0600 hours the next morning. Last backwash occurs at 1800 hours.
2-week maximum flow rate + basin drainage + sludge flow at 1.0 mgd. (Table A-2)	1	3	4	Entire 0.8 MG tank is required and must be kept clean. Maximum pumping rate is 5 mgd. Pumping rate can be reduced to 4 mgd at 1100 hours until 0600 hours the next morning. Softening basin drainage is stretched out over 72 hours, minimum. Last backwash occurs at 2045 hours.
Average flow rate for new basin sizing + sludge flow at 1.8 mgd. (Table A-3)	1	2	3	A minimum new basin volume is 0.5 MG if the existing basin is temporarily removed from service. Maximum pumping rate is 4 mgd. Pumping rate can be reduced to 2 mgd at 2415 hours until 0600 hours the next morning. Last backwash is 1915 hours.
Average flow rate for new basin sizing + sludge flow at 1.0 mgd. (Table A-4)	1	2	3	A minimum new basin volume is 0.4 MG if the existing basin is temporarily removed from service. Maximum pumping rate is 4 mgd. Pumping rate can be reduced to 2 mgd at 1445 hours until 0600 hours the next morning. Last backwash is at 2015 hours.

Note: Backwashing is presently scheduled to occur between 0600 and 2200 hours to avoid high system pressures.

The first two scenarios determine if the existing 0.8 MG holding tank is large enough to handle future backwashing requirements in 2010. Both scenarios assume that softening basin sludge flows would be no more than 1.0 mgd for partial softening. The second scenario is the same as the first except that softening basin drainage was added to the tank loading. The tank is large enough if the following conditions are met:

- The tank will be kept clean to utilize its full capacity; cleaning this tank in the past cost approximately $25,000 in today's dollars because debris had accumulated for several years and the covered tank required cost-intensive manual labor in a confined space
- Softening basin sludge flows will be maintained at 1.0 mgd for all basins combined
- Pumping rates will be maintained as indicated in Table 3-1
- Softening basin drainage will be stretched out 72 hours or more
- Backwashing will be carefully sequenced to prevent tank overflow and to schedule all filter backwashes within the 16-hour daily window

The backwash holding basin is a critical vulnerability to plant operations. If it becomes inoperable for any reason, including being removed from service for maintenance, a discharge to the MSD system would occur. As a result, an evaluation was made to determine minimum size requirements for a new holding tank. The third and fourth scenarios investigate the new tank sizing that would be required if the existing tank was taken off-line during an average day of filter backwashing in 2010. The third scenario conservatively uses the original assumed sludge flow rate out of the softening basins, totaling 1.8 mgd. The fourth scenario is the same as the third except the sludge flow rate is reduced to 1.0 mgd as used for the first two scenarios. The following results were observed:

- The new holding tank volume must be sized for at least 0.4 MG with the reduced sludge flow rate of 1.0 mgd, or 0.5 MG with the original assumed sludge flow rate of 1.8 mgd. This volume does not include an allowance for solids retainage.
- For either scenario the peak pumping rate is 4 mgd and the pumping rate can be reduced to 2 mgd after the day's backwashing is completed.

The four spreadsheet model scenarios are shown in Tables A-1 through A-4 of Appendix A.

Pipelines

Drainage and sludge flow pipelines were described in Section 2. A brief assessment of their condition follows.

Raw Water Reservoir Drainage Pipeline

The north and south basins of the reservoir both connect via a 20-inch drain pipe to the 36-inch pipeline in Reservoir Avenue. At the vicinity of Zorn Avenue and River Road the 36-inch pipeline terminates and can be valved to:

- connect to the 24-inch pipeline to the BEPWTP lagoons,
- discharge to the future Zorn Avenue lagoon through a 20-inch pipeline with effluent discharge to the Ohio River, or
- bypass the lagoon and discharge directly to the river through a 20-inch pipeline.

Because each basin is about 50 MG, the drainage flow rate to empty either side of the reservoir is substantial, even if drainage is spread out over 10 days. Because the 24-inch pipeline is near its full carrying capacity with routine sludge and backwash flows, reservoir drainage must be diverted to the river at the Zorn Avenue Pump Station.

In the past, the reservoirs were once cleaned with a hydraulic dredge. The dredge wastewater discharge was connected to a 12-inch pipeline that connected to the 36-inch drain pipeline. As a result of the dredging operation the 36-inch pipeline partially clogged and had to be cleaned. Segments of this pipeline and its interconnection to the 24-inch pipeline in River Road may still have significant debris inside.

An emergency drainage system for the reservoirs to connect to the storm water system is in the planning stage. Its capacity will be in the range of 18 to 24 mgd during dry weather conditions, but it cannot be used for routine reservoir cleaning and solids disposal.

Coagulation Sludge Pipeline

Sludge from the South Coagulation Basins is transported in a 20-inch pipeline north to the where two 12-inch pipelines collecting North Coagulation Basins sludge tee into it. The 20-inch pipeline then connects to a 30-inch coagulation sludge pipeline in Reservoir Avenue. The 8,100-foot-long pipeline terminates with a connection to the 24-inch pipeline in Zorn Avenue at I-71, but can also discharge into the Zorn Avenue lagoon. The sludge flow rates from the basins historically range from about 0.2 to 0.4 mgd per basin, resulting in a total flow rate of 1.6 to 3.2 mgd in the 30-inch pipeline. The resulting velocity at these flow rates ranges from 0.5 to 1 feet per second (fps), which is too slow to prevent deposition of solids.

Wastewater Holding Basin Force Mains and Backwash Waste Pipeline

The 12- and 20-inch force mains are both cast iron and reportedly interconnected at the pump discharge and at their termination in Reservoir Avenue. The interconnections at the pump discharge should be verified because schematics of these pipelines prepared in 1983 do not show an interconnection and the hydraulic characteristics would be more favorable with one. Each force main can be valved to flow to different termination points. The 12-inch main is connected and valved to the 30-inch coagulation sludge line and the 30-inch backwash waste pipe. The 20-inch main is connected only to the 30-inch backwash pipe, but additional valving in the 30-inch pipe can divert the backwash water to the raw water reservoirs for backwash recycling. At the time this study began, the 12-inch force main was reportedly valved to connect to the 30-inch coagulation sludge drain. Because of the hydraulic loading already on the coagulation sludge drain, it is recommended the 12-inch main be valved to connect only to the 30-inch backwash waste pipeline.

The 12- and 20-inch parallel force mains normally transport between 2 and 4 mgd, if the pumps operate close to their original rated capacities. The velocities that correspond to this range of flow rates are 1.2 to 2.4 feet per second (fps), which may not be adequate to keep heavy solids in suspension. Pumping rates through these force mains should be measured to determine if force main velocities and pump operation are appropriate.

The 30-inch backwash waste pipeline, about 8,100 feet long, terminates with a connection to the 24-inch pipeline in Zorn Avenue at I-71, but can also discharge into the Zorn Avenue lagoon. At the same 2- to 4-mgd flow rates as passing through the 12- and 20-inch force mains, the resulting velocity ranges from 0.6 to 1.3 fps, which is too slow to prevent deposition of solids.

24-Inch Pipeline to the BEPWTP Lagoons

The 24-inch concrete pressure pipeline, constructed in 1971 and 35,450 feet in length, begins with interconnections to the two 30-inch and one 36-inch drain and sludge pipelines at Reservoir Avenue and River Road and terminates with discharge connections into the four lagoons behind BEPWTP.

Because this pipeline is smaller in diameter than the pipelines it connects to and it carries backwash and sludge flows combined, its interior condition and carrying capacity are significant. Combined flow rates would typically fall into a range of 3.6 to 7.2 mgd. Approximately 0.9 mgd additional flow could be added if a coagulation or softening basin is drained over a 72-hour period. Overall flow range for the 24-inch pipeline then would be 3.6

to 8.1 mgd, and the corresponding velocity would be 1.8 to 4 fps. At this velocity range solids deposition should be prevented; however, there is evidence of capacity shortfalls due to solids deposition or some other cause.

In 1991, Pitometer Associates tested two major segments of the 24-inch pipeline. The test report concluded that C-factors were 132 between Zorn Avenue and Boxhill Lane and 113 between Boxhill Lane and Harrods Creek. A C-factor value of 100 for concrete pipe denotes a pipe lining in average condition. Concrete pipe with a C-factor of 130 would be in excellent condition with a fairly smooth lining.

In December 1999 Pitometer Associates again tested the 24-inch pipeline and found segments with C-factors ranging from 12 to 100. Pitometer Associates conducted air scouring in the pipeline to help resuspend deposits and flush out the pipeline. After the air scour operation, C-factors were measured in March 2000 and found to improve as shown in Table 3-2.

TABLE 3-2
Measured C-Factors

24-inch Pipeline Segment	Flow on 3/91, mgd	C-factor on 3/91	Flow on 12/99, mgd	C-factor on 12/99	Flow on 3/00, mgd	C-Factor on 3/00 [a]
Zorn to Box Hill	9.4	132	4.6	50	6.8	77
Box Hill to Harrods Creek	8.5	113	4.6	100	6.8	142
Harrods Creek Crossing	—	—	4.6	12	6.8	76

[a] The March 2000 measurements are being rechecked for accuracy due to the higher than expected C-factors between Box Hill and Harrods Creek for 30-year-old concrete pipe.

An EPANET hydraulic computer model of the drainage system was prepared and several operating scenarios were investigated. A schematic of the model is depicted in Figure 3-1. (The 36-inch basin and reservoir drain line is not included in the model or schematic.) Three key scenarios were investigated. A target sludge flow rate per basin of 0.25 mgd for the North group and 0.35 mgd for the South group was established (discussed later) and used for all three scenarios. All three scenarios used the factory pump performance curves for the backwash holding tank to simulate the projected backwashing rates expected to occur in 2010. In Scenario 1 one pump is on-line, and in Scenarios 2 and 3 two pumps are on-line. In Scenarios 1 and 2, pipe friction factors were set equal to 100, except for segments where Pitometer Associates measured a lower value. In Scenario 3 those segments with C-factors below 100 were set equal to 100, assuming a cleaning program could restore high C-factors.

Table 3-3 summarizes the results of three key scenarios. A single backwash holding tank pump, rated at 2 mgd, operates at 3.47 mgd in Scenario 1. Two pumps operating together increase total flow rate to 4.72 mgd in Scenario 2, and 5.27 mgd in Scenario 3 when the pipe friction factor is improved. The significant decrease in pump performance is a result of increased system head when the second pump is on-line. This response from the pumps indicates that smaller size pumps with steeper performance curves may be appropriate and provide better flow range in a single and dual pump operation.

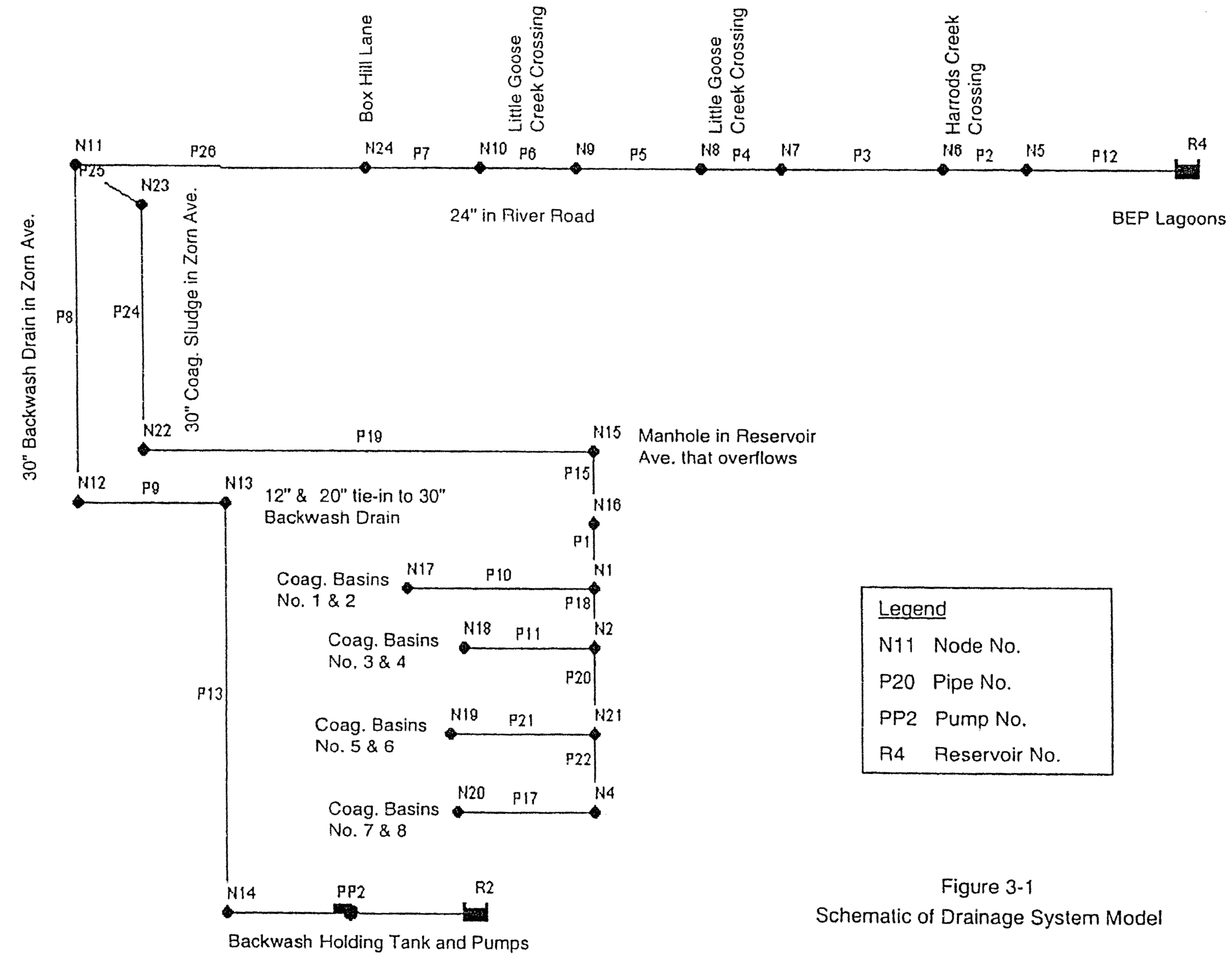


Figure 3-1
Schematic of Drainage System Model

TABLE
Hydraulic Modeling Summaries

Results	Scenario 1	Scenario 2	Scenario 3
EPANET Filename	2010 basecondition-1 pump.net	2010 basecondition-2 pumps.net	2010 improved friction-2 pumps.net
No. of BW holding tank pumps	1	2	2
Pumped flow rate, mgd	3.47	4.72	5.27
Flow rate to lagoons, mgd [a]	5.87	7.12	7.67
HGL @ Node 11 (Zorn Ave. at River Rd.), ft	533.5	570.9	560.1
HGL @ Node 15 (manhole on Reservoir Ave.), ft [b]	534.6	572.1	561.3
HGL @ Nodes 17 & 18 (N. Coag Basins' weir box), ft [c]	534.8	572.2	561.4
HGL @ Nodes 19 & 20 (S. Coag Basins' weir box), ft [d]	535.0	572.4	561.6
Conclusions			
Pumped flow rate sufficient for design year? [e]	No	No	Yes
Overflow at Node 15 manhole?	No	Yes	Yes
Overflow at coag. basin weir boxes?	No	Yes	No

Notes:
[a] Assumes sludge flows from N. & S. Coag. Basins are 0.25 mgd and 0.35 mgd per basin, respectively
[b] Grade at manhole is 557.4 feet
[c] Weir box elev is 562.5 feet
[d] Weir box elev is 562.7 feet
[e] Assumes that 5 mgd is required based on total sludge flow rate being set to 1.0 mgd for partial softening

The operation of backwash holding tank pumps greatly influences the coagulation basin sludge drainage even though the two systems are not interconnected until Zorn Avenue at River Road. For example, the coagulation sludge pipeline is connected to a manhole in Reservoir Avenue that will overflow if backpressure (hydraulic grade line—HGL) exceeds the top elevation of the manhole, which is 557.4 feet. This manhole prevents the sludge weir boxes from backing up and overflowing into the coagulation control buildings. This manhole is depicted by Node 15 in the model. In Scenario 2 the HGL of Node 15, and Nodes 17-20 which depict the coagulation basin weir boxes, are much higher than the manhole elevation and even the weir boxes, signifying that overflows would occur at all locations. If the C-factors are improved as simulated in Scenario 3, the overflow condition would be eliminated at the weir boxes but the manhole still have an overflow condition. The overflow condition can be corrected by raising the manhole.

These overflow conditions are brought about by the significant variation of head loss in the 24-inch pipeline due to flow rates in the range of 5 to 7 mgd. The degree of head loss in the model is highly sensitive to the assumed friction factor. According to the model, a flow rate in the range of 7 mgd though the 24-inch is not attainable with the assumed friction factors. If Pitometer Associates' recent report is verified and still indicates a measured flow rate of 6.8 mgd after air scouring, the flow rate should be reinvestigated by field-measuring flow rates again so that the true pipeline capacity can be accurately computed.

Appendix B contains the EPANET node and pipe data printouts for the three scenarios described in Table 3-3.

Pitometer Associates also reported that some air release valves were found to be inoperative. If valves are stuck closed, trapped air will accumulate and significantly reduce carrying capacity of the pipeline.

Because the C-factor at the creek crossing is lower than along the other pipeline segments, there is probably significant deposition at low points in the pipeline. The pipeline should be taken out of service for a TV inspection of selected segments to determine the extent of deposition and cleaning required. If low points in the pipeline, such as creek crossings, have considerably more deposition than the rest of the pipeline, a double-barrel configuration with valved connections on both sides of the creek crossings should be considered. That way, each barrel segment could be cleaned by jetting or pigging while the other barrel segment remained in service. The next section describes how the 24-inch pipeline could be temporarily removed from service without disrupting CHWTP operations.

Lagoons

Zorn Avenue Pump Station

Because the 24-inch pipeline to the BEPWTP lagoons operates continuously, the pipeline cannot be removed from service for inspection or maintenance. An operating lagoon that could settle out suspended solids and discharge clear water to the river at the Zorn Avenue Pump Station would allow the 24-inch pipeline to be temporarily removed from service for short-term maintenance projects.

A site in front of the river pump station about 3.5 acres in size is reserved for a lagoon. A pipe that would serve this site and currently discharges to the river already possesses a KPDES discharge permit. This pipe has previously been used to discharge drainage from the raw water reservoir; however, the lagoon site was used and the drainage was directly discharged to the river without treatment since the drainage was good quality water.

The 30- and 36-inch sludge and drain pipelines can be valved to discharge to this lagoon site instead of into the 24-inch pipeline; however, site work is required to develop the lagoon with viable holding capacity. A topographic survey should be considered to determine the amount of earthwork that would be required and the potential volume of a lagoon on this site. A new effluent structure would also be required to release only the best quality supernatant for discharge to the river to comply with the 30 mg/L TSS discharge limitation.

BE Payne WTP Lagoons

A summary of the lagoon characteristics is presented in Table 3-4. This is an extension of the information first provided in the *Washwater Lagoon Investigation and Monitoring Program* report (August 1992). To update the estimated sludge volume of each lagoon and make an assessment of the lagoons' condition, the four lagoons were inspected in April 2000. The findings on individual lagoons, presented below, are based on the conditions at the time of inspection.

TABLE 3-4
Lagoon Volume Available

Lagoon No.	Effective Surface Area, acres[1]	Total Lagoon Volume, cf[1]	Estimated Sludge Volume, cf[2]	Available Capacity, cf[4]	Available Capacity, %	Estimated Life Remaining, Yr[3]
1	19.6	10,497,000	6,600,000	2,847,000	27.1%	1.5
2	17.7	9,476,000	7,300,000	1,228,000	13.0%	0.6
3	16.3	10,610,000	7,200,000	2,349,000	22.1%	1.2
4	30.2	23,753,000	12,900,000	8,478,000	35.7%	4.4
Total	83.8	54,336,000	34,000,000	14,902,000	27.4%	7.7

[1] Lagoon sizing information excerpted from *Washwater Lagoon Investigation and Monitoring Program*, August 1992

[2] Sludge volumes were visually estimated without measurements

[3] Only 90 percent of total lagoon volume is assumed available for sludge storage

[4] Assumes current (1999) rate of solids production at 50 percent moisture content

Lagoon No. 1

This lagoon has been out of service for 3 to 4 years and is almost completely covered in vegetation. According to LWC personnel, there is reportedly a leak in the berm on the west side of the lagoon when the lagoon is in operation. Erosion of the berm was not evident. Both the inlet and outlet structures appear to be in good condition structurally. The outlet structure handrails are rusty. The inside of the structure is free from debris. The sluice gate valves are difficult to operate because of corrosion. Operators at the plant stated that all the sluice gates needed at least two people to exercise them. The estimated solids volume is approximately 6,603,000 cubic feet, and the estimated available capacity is about 37 percent.

Lagoon No. 2

At present, Lagoon No. 2 is the active lagoon, receiving sludge from both CHWTP and BEPWTP. The lagoon has been in continuous operation for over a year. Generally, a lagoon will stay in operation for approximately a year, then the sludge will be directed towards another lagoon. Vegetation completely covers the berm surrounding the lagoon and was present on about 30-40 percent of the lagoon bottom. Erosion is not evident. The inlet and outlet structures appear to be structurally sound. Handrail and grating are in good condition. Both the sluice gates and the inside of the outlet structure are filled with debris. Apparently, in an attempt to keep the water levels to a maximum, the majority of the debris was placed in the sluice gates by beavers. As evidence to this, beaver 'dams' are present in shallow areas and along the berms of both Lagoons No. 2 and 3. The estimated solids volume is approximately 7,278,000 cubic feet, and the available capacity is about 23 percent.

Lagoon No. 3

This lagoon is the only lagoon that receives filter backwash from the BEPWTP. The conditions of Lagoon No. 3 are similar to those of Lagoon No. 2. Vegetation covers the berm surrounding the lagoon and 25-35 percent of the lagoon bottom. Erosion is not evident. The inlet and outlet structures were in good condition. The handrail and grating show little rust. Beavers show their presence by the amount of debris lodged into the sluice gate and located inside the outlet structure. The estimated solids volume is approximately 7,162,000 cubic feet, and the available capacity is about 33 percent.

Lagoon No. 4

Lagoon No. 4 is not in use. This lagoon is the largest of the four lagoons. Vegetation covers about 5-10 percent of the lagoon bottom and 30-40 percent of the berm. The berm appears to be lined with rocks on the northern side and dirt on all other sides. Erosion of the berm is evident, mostly near the inlet and outlet structures. When the lagoon is in operation leakage reportedly occurs in the north berm along Mayfair Road. Both the inlet and outlet structures appear to be in good condition and free of debris. The handrails and grating show little rust. The estimated solids volume is approximately 12,945,000, and the available capacity is about 46 percent.

Assessment

All four lagoons have substantial vegetation, which grows freely where there is no standing water, in the lagoons and on the berms. Some of the vegetation consists of trees more than 20 feet high with trunk diameters of 3 inches, or more. The vegetation restricts visibility and ability to determine condition of the lagoons. When the lagoons are to be cleaned the vegetation will hamper the cleaning process. The resulting vegetative debris intermixed with the residuals may not be compatible with the intended use of the solids or the disposal site. The trees attract beavers, which can result in more maintenance to clean out debris from the outlet structures.

To determine available capacity in the lagoons the volume up to the inlet pipe invert elevation was computed. Only 90 percent of this total volume is assumed to be available for sludge storage because of the need to settle and remove solids before discharging to the river. Based on the estimated available capacity of the lagoons shown in Table 3-4 and the rate of residuals accumulation projected in Table 2-14 with no softening at CHWTP, approximately 7.7 years of service are estimated to remain for all four lagoons to fill, if none of the lagoons are cleaned.

SECTION 4

Solids Handling Practices in Other Communities

The purpose of this section is to summarize a survey conducted to compare the solids handling operations of LWC to industry standards (see Table 4-1). The utilities surveyed were Northern Kentucky Water Service District (Ft. Thomas WTP); Cincinnati Water Works (Richard Miller WTP); EA_2 Systems (Environmental Management Company/American Water Works Service Company/ Anglican) in Evansville, IN; Pittsburgh Water and Sewer Authority; and American Water Works Company—WV (Huntington).

Northern Kentucky Water Service District

The source for the 44-mgd Ft. Thomas plant is the Ohio River. The average and maximum day production rates are 23 mgd and 42 mgd, respectively. The plant contains a backwash holding basin that dumps into a thickener. The decant is pumped to the raw water reservoirs for recycling. The thickened sludge from the backwash holding basin along with the sludge from the coagulation basin is pumped to belt filter presses. The pressed sludge is hauled to a landfill at $100 per trip plus $14.40 a ton. The backwash water volume is approximately 0.28 mgd. The total wastewater volume is approximately 3 percent of the total volume. The annual solids handling budget is $155,000. The annual operation and maintenance (O & M) budget is $3.1 million.

Cincinnati Water Works

The Ohio River is also the source for the Richard Miller WTP. The plant's capacity is 220 mgd. The average and maximum day production rates are 110 mgd and 200 mgd, respectively. The plant is permitted to discharge sludge from the coagulation basins into the river without treatment. The sludge volume is approximately 2.97 mgd. The filter backwash is recycled. The filter backwash water volume is approximately 1.4 mgd. The solids handling budget is $78,000. The annual O & M budget is $11.2 million.

EA_2 Systems

This Evansville company's water source is also the Ohio River. The plant's capacity is 60 mgd. The average and maximum day production rates are 32.5 mgd and 40.5 mgd, respectively. EA_2 is permitted to discharge sludge into the river without treatment. All wastewater, including filter backwash, is discharged into the river. The filter backwash water volume is 0.37 mgd. The total wastewater volume is 0.5 mgd. Because this company is a private entity, they would not disclose any operational budgets.

TABLE 4

Summary of Solids Handling Operations

Question	Lousiville Water Company	Cincinnati Water Works	Environmental Management Company (Evansville)	Northern KY Water Service District	Pittsburg Water & Sewer Authority	American Water Works Company--WV (Huntington)
Contact Name	Phil Scott	Debra Metz-Supervisor	Neil Sutherland	Bill Wulfeck	Raymond Wisloski	Bret Morgan
Phone #	(502) 569-3600	(513) 624-5658	(812) 428-0568	(859) 441-0482	(412) 782-7552	304-340-2035
Source	Ohio River	Ohio River	Ohio River	Ohio River	Allegheny River	Ohio River
Plant name	Crescent Hill WTP	Richard Miller WTP	Evansville WTP	Fort Thomas WTP	Pittsburg WTP	Huntington WTP
Plant capacity, mgd	180	220	60	44	120	24
Avg. day production, mgd (1999)	111	110	32.5	23	65	13
Max. day production, mgd (1999)	156	200	40.5	42	100	19
Coagulant(s) used and is softening performed	Ferric sulfate; no softening	Ferric chloride; no softening	Alum; no softening	Ferric sulfate or Clarion 5100; no softening	Ferric chloride; no softening	Ferric sulfate and cationic polymer; no softening
Method of wastewater disposal	Discharged to lagoons and supernatant returned to river	Returned to river	Returned to river	Sludge dewatered by filter belt press, then transported to landfill	Clarifier sludge discharged to sewer; sedimentation basins cleaned every 10-15 yrs (disposed offsite)	Sludge piped to two lagoons; eventually dried and transported to landfill
Backwash wastewater rate, mgd	1.8	1.395	0.37	0.28	1.0	0.32
Is filter recycle practiced?	No	Yes	Put back in river	ıcycle top of backwash ta	yes	Decant recycled
Total wastewater rate, mgd	4.8	2.97	0.5	0.7	1.3	0.4
Backwash water/total production rate	1.6%	1.3%	1.1%	1.2%	1.5%	2.5%
Wastewater/total production rate	4.3%	2.7%	1.5%	3.0%	2.0%	3.1%
Annual solids handling budget[a]	$55,000	$78,000	NA	$155,000	$0	$150,000
Annual operating budget	$8,300,000	$11,200,000	NA	$3,100,000	$6,000,000	$600,000
Solids handling budget/total budget	0.7%	0.7%	NA	5.0%	0.0%	25.0%

Notes:

NA=not available

[a]The annual solids handling budget for LWC is planned for future maintenance activities

Pittsburgh Water and Sewer Authority

The source water for this municipality is the Allegheny River. The WTP's capacity is 120 mgd. The average and maximum day production rates are 65 mgd and 100 mgd, respectively. Sludge from the coagulation basins is discharged directly into the sanitary sewer system. An independent authority owns the sewers. Because of an unspecified political relationship between the two authorities, the fee for dumping approximately 0.3 mgd of coagulation sludge is waived. Secondary treatment is provided through clarifier basins totaling 120 million gallons of storage. The retention time for these basins is approximately 24 hours. The sludge from these basins is removed every 10 to 15 years. The sludge is hauled to a landfill. Ten years ago, the cost for sludge removal and disposal was near $3 million. The volume of filter backwash is approximately 1 mgd. Filter backwash is recycled. Excluding the costs of the sediment basin sludge removal, there is no budget for annual solids handling. The annual O & M budget excluding payroll is $6 million.

American Water Works Company—West Virginia (Huntington)

The Ohio River is the source for the 24-mgd Huntington WTP. The average and maximum day production rates are 13 mgd and 19 mgd, respectively. The plant contains a backwash-thickening basin. The decant from the basin is discharged into the river without treatment. The sludge from the basin is pumped into two nearby lagoons. Sludge from the primary clarifiers is gravity fed to these same lagoons. The combined size of these two lagoons is approximate to a football field. Each lagoon is completely drained and cleaned once a year. The sludge is moved to drying beds with a backhoe. Once dried, the sludge is hauled to a landfill. The volume of filter backwash is approximately 0.32 mgd. The total volume of wastewater is approximately 0.4 mgd. The annual solids handling budget is $150,000 and the annual operating budget is $600,000.

Summary

The findings presented in Table 4-1 are used to compare LWC operations with these utilities. As a percentage of total water production, backwash water usage falls within the middle of the range reported by the other utilities. Total wastewater flow including basin sludge flows, however, are higher for LWC than for the other utilities. Operating budgets for solids handling vary so widely among the surveyed utilities that it is difficult to make any comparisons or draw any conclusions; however, the planned solids handling budget for LWC, as a percent of total operating budget, is equivalent to what is presently budgeted by Cincinnati Water Works.

SECTION 5

Residuals Management Regulations Review

Both federal and Kentucky regulations for drinking water and wastewater have jurisdiction for handling and disposing of WTP residuals. The drinking water regulations fall under the general heading of the Safe Drinking Water Act (SDWA) and the wastewater regulations fall under the Clean Water Act (CWA). The wastewater regulations apply because of the two river discharge permits held by LWC for continuous discharge of lagoon supernatant and for intermittent discharge of raw water reservoir contents to the Ohio River. The Commonwealth of Kentucky has primacy to administer both the SDWA and the CWA on behalf of the USEPA. Water and wastewater administration is handled by the Division of Water of the Kentucky Department for Environmental Protection. Disposal of solids from the lagoons is regulated by the Division of Waste Management.

Residuals handling has been a significant focus of the drinking water regulatory process in recent years as a result of cryptosporidiosis outbreaks in several communities that were linked to public drinking water supplies. In the massive Milwaukee outbreak, one likely source of the *cryptosporidium* cysts was believed to be recycled residuals streams at the WTP. As a result, federal regulations are pending that will, for the first time, govern the processing of residuals streams within the treatment process rather than the finished water leaving the plants.

Drinking Water Regulations

Interim Enhanced Surface Water Treatment Rule (IESWTR)

The IESWTR was promulgated in November 1998 and became effective in February 1999. The primary purposes of the IESWTR are (1) to improve control of microbial pathogens in drinking water, particularly for the protozoan Cryptosporidium, and (2) to guard against significant increases in microbial risk that might otherwise occur when systems implement the Stage 1 Disinfectants/Disinfection Byproducts Rule. Major components of the IESWTR include the following provisions:

1. **Maximum Contaminant Level Goal** (MCLG) of zero is established for the protozoan genus *Cryptosporidium.*
2. ***Cryptosporidium*** **Removal.** Surface water systems serving 10,000 or more people, that are required to filter under the SWTR, must achieve at least 2-log removal of Cryptosporidium. Systems that use conventional or direct filtration meet this requirement if they comply with strengthened turbidity performance standards for combined filter effluent (described below) and the current requirements under the SWTR (e.g., meet design and operating conditions as specified by the State). Systems that use slow sand filtration or diatomaceous earth meet the 2-log removal requirement if they are in compliance with existing turbidity performance standards under the SWTR (less than or equal to 1 NTU in at least 95 percent of measurements taken each month or, for slow sand, alternative criteria as approved by the State; and a maximum of 5 NTU).

3. **Turbidity in Combined Filtered Water.** For all surface water or GWUDI systems that use conventional treatment or direct filtration, serve 10,000 or more people, and are required to filter: (1) The turbidity level of a system's combined filtered water at each plant must be less than or equal to 0.3 NTU in at least 95 percent of the measurements taken each month, and (2) the turbidity level of a system's combined filtered water at each plant must at no time exceed 1 NTU. For both the maximum and the 95th percentile requirements, compliance is determined based on measurements of the combined filter effluent at four-hour intervals.

4. **Individual Filter Requirements.** All surface water or GWUDI systems that use conventional or direct filtration, serve 10,000 or more people, and are required to filter must conduct continuous monitoring of turbidity for each individual filter and must provide an exceptions report to the State on a monthly basis. Exceptions reporting must include the following: (1) Any individual filter with a turbidity level greater than 1.0 NTU based on two consecutive measurements fifteen minutes apart; and (2) any individual filter with a turbidity level greater than 0.5 NTU at the end of the first 4 hours of filter operation based on two consecutive measurements fifteen minutes apart. A filter profile (which is a graphical representation of an individual filter performance) must be produced within seven days of the exceedance if no obvious reason for the abnormal filter performance can be identified. If an individual filter has turbidity levels greater than 1.0 NTU based on two consecutive measurements fifteen minutes apart at any time in each of three consecutive months, the system must make an exceptions report and conduct a self-assessment of the filter. If an individual filter has turbidity levels greater than 2.0 NTU based on two consecutive measurements fifteen minutes apart at any time in each of two consecutive months, the system must make an exception report and arrange for the conduct of a Comprehensive Performance Evaluation (CPE) by the State or a third party approved by the State. State Authority.

The turbidity limits for individual filters require filters by December 2001 to become sufficiently ripened to initially produce filtered water below 1.0 NTU, and below 0.5 NTU after 4 hours of operation. Presently, LWC filters are reportedly producing water at less than 0.25 NTU immediately after backwash and will comply with the rule.

The proposed Stage 2 Disinfection/Disinfection Byproducts Rule (D/DBPR) and Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) are scheduled to be published for public comment in February, 2001. EPA intends to meet the statutory deadlines to finalize these regulations by May, 2002. The proposed rules may contain more stringent performance requirements for turbidity levels, particularly after backwashing. If turbidity levels are lowered further in future regulations, or if voluntary strategies, such as the Partnership for Safe Water Program, are adopted resulting in lower target turbidity levels, the feasibility to provide filter-to-waste or some type of filter media conditioning should be investigated.

Filter Backwash Rule

In April 2000, USEPA issued the proposed Long-Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) and Filter Backwash Recycling (FBR) Rule. After receiving public comments, the FBR Rule is to be promulgated in August 2000.

While the LT1ESWTR is targeted toward public water systems serving less than 10,000, the Filter Backwash Rule (FBR) has a potential impact on LWC treatment facilities. USEPA has decided that the rule should address not just filter backwash recycle but other plant recycle flows as well. Other the major elements of the proposed rule include:

- All systems using surface water or ground water under the direct influence of surface water would be required to recycle flows prior to the rapid mix unit, if recycling is practiced. Waivers from this requirement would be available from State primacy agencies for unique treatment conditions including plants that are designed to recycle to other locations to maintain optimal treatment performance, and plants that are designed to employ recycle flows as an intrinsic component of their operations.
- Direct filtration plants using surface water or ground water under the direct influence of surface water would be required to report to the State Primacy agency whether flow equalization or treatment is provided for recycle flow prior to its return to the treatment process. Information on any equalization and treatment provided would be passed on to the State. The State would use the information to determine which plants need to change their current recycle practice to provide additional public health protection.
- All plants using surface water or ground water under the direct influence of surface water would be required to complete a self-assessment to determine the impact of recycle on plant operations, if the following two conditions occur:
  - Twenty or fewer filters are used during the highest production month
  - Spent filter backwash or thickener supernatant are directly recycled to the treatment process without providing recycle flow equalization, treatment, or another form of hydraulic detention such as a lagoon

The self-assessment would be reported to the State primacy agency for review and a determination of whether changes to recycle practices are needed.

In summary, the pending drinking water regulations will place certain requirements on systems that recycle backwash water back to the treatment plant. Because LWC does not practice backwash water recycle at either plant, the FBR would not apply. Even if LWC resumes filter backwash recycle to the raw water reservoir at CHWTP, it appears that recycle provisions of the FBR, as now proposed, would be met at CHWTP without any process modifications.

Wastewater Regulations

As noted previously the LWC holds two KPDES permits issued by the Kentucky Division of Water for discharge of water from the lagoons at the BEPWTP and from the raw water reservoir at the Zorn Pumping Station. Both permits have TSS limits of 30 mg/L for monthly average concentration and 50 mg/L for maximum day concentration. There are no known changes pending in federal or state regulations that would necessitate a significant change in the discharge permit conditions. The Kentucky Division of Water verified the expectation that the permit conditions can be expected to remain unchanged.

Solids Disposal Regulations

The USEPA has delegated regulation over water and wastewater treatment plant residuals to the Kentucky Department for Environmental Protection. The Department has assigned regulation of such residuals to the Division of Waste Management. Within the Kentucky Administrative Requirements, (KAR) water plant residuals are governed by the provisions of Title 401, Chapter 45, Special Waste. The term "special waste" generally applies to wastes with the potential to contain hazardous materials or other constituents at hazardous concentrations. The primary constituents of concern with water treatment residuals are heavy metals.

The Division and KAR provide for four methods of disposing of water plant residuals:

- Landfilling in a mixed waste landfill
- Landfilling in a monofill
- Land application as a soil amendment
- Use as a structural fill material

The following paragraphs describe the regulatory requirements for disposal of the residuals by the various methods.

Landfilling

If water treatment solids are disposed of at a commercial landfill, the permit for the facility is held by the landfill owner and no permit is required for the water utility. Standards for the characteristics of the material are to be determined by the landfill owner and the utility. The Division of Waste Management considers water plant residuals to be of high quality potentially suitable for use as daily cover in the landfill. Such use may provide a benefit to the landfill owner and result in reduction or elimination of tipping fees. Use of the residuals as daily cover necessitates obtaining a special permit by the landfill permit holder.

Landfilling by Monofill

Two avenues exist for monofilling of the residuals. One avenue is to decant and cover a lagoon in place thereby creating a monofill and the other is to remove the solids from a lagoon and deposit them in another impoundment to be covered and closed. Monofill disposal of special waste is covered in 401KAR45:010 and the landfill design is governed by general landfill provisions of 401KAR 30:031. For a lagoon to be converted to a monofill the lagoon bottom would need to be in compliance with the landfill liner requirements for a special waste. However, liners are not required in all cases and the water solids may qualify for disposal in an unlined landfill. Other provisions of the regulations relate to groundwater leachate, surface runoff, gas migration, and odors. Requirements for systems to monitor and/or control these potential environmental hazards depend on the characteristics of the special waste material.

Land Application

The KAR uses the term *land farming* for the application of solids to agricultural land as a soil amendment. This practice is governed in 401KAR45:100, which differentiates between Type A and Type B land farms. The concentration of heavy metals in the special waste is the determining factor between Types A and B, with Type B considered the higher quality material. The following parameters in Table 5-1 define the difference between Type A and B solids.

TABLE 5-1
Concentrations of Heavy Metals in Type A and Type B Solids

Constituent	Type A	Type B
Cadmium	Greater than 10 mg/kg	Less than or equal to 10 mg/kg
Copper	Greater than 450 mg/kg	Less than or equal to 450 mg/kg
Lead	Greater than 250 mg/kg	Less than or equal to 250 mg/kg
Nickel	Greater than 50 mg/kg	Less than or equal to 50 mg/kg
Zinc	Greater than 900 mg/kg	Less than or equal to 900 mg/kg

Analyses of the LWC sludge in Lagoons 2 and 4 at BEPWTP in the *Washwater Lagoon Investigation and Monitoring Program* report indicate, at the time of sampling, levels of each of the metals to be within the Type B limits for all parameters, although one of the four samples showed a nickel concentration of 45 mg/kg, which is near the limit. A Type B land farm site is limited to application of 250,000 gallons or 250 dry tons of solids applied per calendar year, although a variance to this provision can be obtained. Type B facilities are exempt from requirements for publishing a public notice, posting of financial assurance, monitoring of groundwater, and post closure care. Certain site management practices apply to land farms, such as setbacks from structures or specific land uses, prohibition of application to permeable or steeply sloped land, and other provisions.

Structural Fill

The use of water residuals for structural fill is termed beneficial reuse by the Division of Waste Management and is governed by the least restrictive site management requirements. Use as fill is covered in 401KAR45:070 and by Division of Solid Waste policy. Each beneficial reuse location must be permitted, and general distribution is prohibited. Among the policy provisions is a requirement that the residuals be mixed with soil so that the structural fill mixture is not comprised of more than 50 percent special waste.

SECTION 6

Operations

Impacts Of Sludge Handling and Backwash Operations

In Section 2, Table 2-5 shows that sludge flows from the coagulation and softening basins were 2.7 percent of total plant production in 1980, and more recently 2.86 percent of plant production based on measurements made on two different days in April 2000. Filter backwash wastewater averages 1.55 percent of total plant production based on targeted filtration rates and run times for each filter group. This brings total wastewater flow to 4.4 percent of total water production. For surface WTPs, typical wastewater usage is probably less than 4 percent of total water production. In fact, all five WTPs surveyed in Section 4 reported wastewater flows less than or equal to 3.1 percent of total water production. By comparison to other WTPs, wastewater flows in CHWTP may be higher than what is typical in other similar facilities.

Practices that may be causing above-average wastewater flow rates are:

- High sludge flow rates to prevent sludge pipeline clogging
- Long filter backwashes because the available backwashing rate is less than optimum
- Shorter filter run times in some filter groups than may be necessary to prevent particle breakthrough
- Low rates of filtration that may result in less filtered water production per cycle than attainable if optimized

Opportunities for Reducing Process Wastewater

Establish target sludge flow rates as low as possible to minimize clogging and regularly monitor flow rates to avoid excessive flow. Initial target sludge flow rates are shown in Table 2-13. These values are based on 0.25 mgd in the South Coagulation Basins, 0.35 mgd in the North Basins, and 0.10 mgd in the softening basins for pH adjustment only and 0.30 mgd for softening. These target flow rates, or even lower values, have been measured in one or more basins of each corresponding basin group. If clogging is held to a minimum, the target values should be set even lower for each basin group.

Consider automated flushing of the basin sludge lines. If routine basin flushing was automated to occur by time-of-day with electrically actuated valves, the frequency of flushing and flushing duration could be optimized to reduce the overall volume, flushing could be scheduled in proper sequence to avoid coinciding with peak backwashing flow rates, and operator time for this activity would be reduced.

Improve backwash supply system to provide higher filter backwashing rates, which will more effectively clean media and possibly reduce overall backwash volume by reducing backwash duration. Maximum backwash rates are reportedly 13 gpm per square foot due to hydraulic limitations in the backwash supply. For adequate media expansion and cleaning, the backwash rate may need to be increased by up to 50 percent during warm water conditions. To reduce backwash volume at this higher backwash rate, the duration would have to be shortened by more than one-third.

Consider longer filter runs if turbidity breakthrough and excessive head losses can be prevented. The opportunity for longer filter runs in each filter group must be carefully investigated to insure that filtered water quality is not compromised. Some filter groups may need to be renovated before longer filter runs are considered.

Consider higher filtration rates for the south and east filter groups and operate fewer filters. The south and east filter groups should be tested for higher filtration rates to ascertain if filter runs can be maintained as long with higher filtration rates, again without compromising water quality.

Consider recycling filter backwash water. Recycling backwash water was discontinued several years ago due to the potential threat of recycling protozoan cysts. The Filter Backwash Recycle Rule, now proposed and scheduled to be finalized in August 2000, will address this concern by limiting the practice of recycling. Based on the rule as now proposed, filter backwash recycling will be permitted as long as the wastewater stream is returned to the rapid mix or further upstream. Because filter backwash water can be returned to the raw water reservoir upstream of the rapid mix, LWC would comply with the rule as presently proposed. The raw water reservoir also provides an additional flow equalization benefit. If recycling of filter backwash water was resumed, the wastewater flow rate to the lagoons would be reduced by about one-third. Before considering this practice some safeguards should be considered and are discussed in Section 8.

Transfer clear water from basins to be drained to other basins. Basin drainage, although not a significant volume of water compared to annual wastewater volumes, adds an appreciable load to the 24-inch lagoon pipeline when a basin is being drained. To reduce this loading, each group of coagulation and softening basins could be equipped with a transfer pump to withdraw approximately the top two-thirds of the clearest water from the basin to be drained and discharge the water to an adjacent basin or to a basin influent flume. A single transfer pump could be used for each basin group by manifolding a suction and discharge header to each basin with valves to isolate the basin to be drained. A similar arrangement could be used for the raw water reservoir.

Flow Rate Monitoring

Drainage, basin sludge withdrawal, and filter backwashing disposal are difficult to adjust and optimize because the flow rates of these wastewater streams are not monitored. The following flow rate monitoring points are recommended.

Coagulation and Softening Basin Sludge Flows

The basin sludge drains are equipped with butterfly valves at the weir boxes. To prevent clogging in the pipes the valves are typically opened twice per week to flush the sludge pipes at high velocities. Because these valves are frequently operated, the sludge flow rates often vary as shown in Table 2-4, which shows field measured flow rates for two different days in April 2000. As a result, a positive means of flow rate measurement would be desirable to insure that the valves are correctly positioned to normal targeted flow rates.

Flow rate measurement over the weirs could be as simple as installing a staff gauge in the weir channel for the operators to read and providing a weir head-discharge table to determine flow rates. The disadvantage of a staff gauge may be their difficulty to read from the operating floor where the valve is located. Another means of computing flow rate would be to install an ultrasonic meter at each weir, similar to Milltronics OCM III. The device has a display and is pre-programmed for measurement and direct computation of weir flow rate, and could be located near each valve. Installed cost is about $2,000 per unit and there are a total of 14 weirs to be measured.

The butterfly valves often are unable to fully seat because of solids and do not provide good flow rate control. Either one of two different types of replacement valves would provide good service in this application. Both are made by DeZurik. The first option is a V-port plug valve, which is available up through 20-inch size, designed specifically for flow control service with 90-degree-turn action. The second option, which would be less expensive, is an L Series knife gate with a V-ported opening designed specifically for throttling control of thick slurries. When the butterfly valves need to be replaced, switching to one of these valves may provide a more favorable performance.

Backwash Holding Tank

The flow rate in the pumped discharge from this tank is unknown. In the summer when filter backwashing is more frequent, backwashing rates must be carefully sequenced to utilize the backwash holding tank to the extent possible without overflowing. Backwash holding tank pumping affects the capacity of the coagulation basin sludge flow as well. Therefore, measuring the pumped flow rate would help balance the use of the holding tank and coagulation basin sludge flows. Magnetic flowmeters would be the best device to measure flow rate of this wastewater with a high solids content. A budget level cost estimate (without installation) would be $15,000 for the 20-inch and $7,500 for the 12-inch sizes. A vault to contain the flow meters for both the 12- and 20-inch force mains could be constructed downstream of the pumps.

Drainage

Drainage from the raw water reservoirs, coagulation and softening basins, and other process basins occurs too infrequently to justify providing flow rate monitoring.

Lagoons

The vegetation in the lagoon restricts the ability to observe their operation and condition and leads to additional maintenance problems, as previously mentioned. Plans should be made to clean vegetation from all lagoons. Once cleaned, vegetation could be discouraged by maintaining a foot or more of water above the solids in the lagoon. Maintaining a layer of water on top of the solids may not be possible unless the underdrain system that discharges to the outlet structure is valved closed or plugged.

Regular preventive maintenance activities such as exercising valves and painting metalwork should be scheduled.

Wastewater is discharged to each lagoon in a rotation. As a result, the lagoons are filling fairly evenly as shown in Table 3-4. Based on the estimated values of residuals accumulation and percent filled listed in the table, Lagoons 1, 2, and 3 each have less than 2 years of capacity remaining. By diverting all wastewater to one of these three lagoons until it is filled, the lagoon can then be removed from service for cleaning before other lagoons become filled. This pattern of staggered lagoon cleaning will allow each lagoon to be removed from service for 2 or 3 years. This will provide lots of flexibility for an outside contractor to clean the lagoon, and if a generous amount of time is made available for the work, more competitive bidding will result. Once the first lagoon is filled and removed from service, all wastewater flow should be discharged to a second lagoon until it becomes filled, and then the cleaning process can be repeated. Lagoon No. 2 is nearly full and should be removed from service first. After a resting period of 6 to 12 months to further dewater the solids, the lagoon can be cleaned by mechanical excavation.

Because of restrictions in plant piping, backwash water from BEPWTP can only be discharged to Lagoon No. 3 through a 42-inch pipeline, so this lagoon cannot be removed from service. Similarly, wash water from the traveling screens, and two sump pumps, four grit pumps, pump seals, and roof leaders discharge to a collector box behind the BEPWTP Raw Water Pump Station. Currently, the collector box water discharges by open channel flow to Lagoon No. 2. Alternate discharge points should be provided for the 42-inch backwash pipeline to divert backwash discharge from Lagoon No. 3 to either Lagoon No. 1 or 2. Additionally, an alternate drainpipe should be installed to divert the collector box water to Lagoon No. 1, so that Lagoon No. 2 can be removed from service.

Raw Water Reservoir Cleaning

In the past, the raw water reservoirs have been cleaned by mechanical excavation, hydraulic dredging, and manual hose wash down to the drains. Most recently, both basins of the reservoir were mechanically cleaned. A plastic liner to restrict leakage was installed on the north basin and installation is presently underway on the south basin.

Mechanical excavation is labor intensive and requires hauling to a disposal site. If mechanical excavation is used as the means to clean the reservoirs, special precautions will be needed to protect the plastic liner. The liner warranty has several restrictions on activities in contact with the liner. In fact, the warranty may be compromised if any vehicles are operated on top of the liner, even if planks or other protective surfaces are used. Warranty limitations must be investigated before any maintenance activity is planned. An advantage of mechanical excavation is that vehicle access ramps to the inside were constructed for installation of the liner and will remain for future use.

Hydraulic dredging is a simpler and quicker procedure but there are two disadvantages: 1) use of a cutterhead may damage the new plastic liner on the bottom; and 2) the dredged solids are discharged into the 36-inch drain pipeline and the flow through this pipeline is insufficient to maintain minimum velocity to keep heavy solids suspended. A cage attachment is available from most dredge manufacturers and if used properly can almost eliminate the chance of liner damage by the cutterhead. An example of one manufacturer's cutterhead protection system is provided in Appendix C.

Another means to protect the liner from the cutterheads, or from mechanical excavating equipment, would be to install a concrete overlay on top of the liner in the bottom of the reservoir. This would be an expensive procedure, probably costing as much as $700,000 per reservoir basin for a 3-inch-thick, reinforced concrete layer on top of the liner, but it would help insure that the liner would not get damaged from any activity and would prolong its life. This option should be considered after the south basin liner is installed but before it is placed back into service.

The last time dredging was used, a considerable amount of solids deposited in the pipeline because the flow rate was too small to maintain flushing velocities. This material had to be removed manually by a slow, laborious process. To maintain a flushing velocity of at least 2 fps, a flow rate of 9.2 mgd (6,400 gpm) would be required in the 36-inch-diameter drainage pipeline. A dredge sized for this reservoir would typically require a flow rate of about 1,000 to 2,500 gpm to operate efficiently, so additional drainage water would be needed to supplement the dredging water to attain flushing velocity in the 36-inch drain. Additional inflow to the basin would also be required just to maintain a minimum water level for dredging. At this flow rate either reservoir basin would be drained in about 5 days excluding any contribution of makeup water that would be used for maintaining water level. However, the 24-inch pipeline to the lagoons would be not capable of handling this flow rate. A new lagoon at the Zorn Avenue Pump Station may also be too small to handle this flow rate while adequately settling out suspended solids. An alternative method for disposing of dredging water would be to connect to the coagulation sludge pipeline. Because it is only 30 inches in diameter and would already have a base flow of about 2.4 mgd, flushing velocity would be attained with a much smaller flow rate and a new lagoon at Zorn Avenue or the 24-inch pipeline to the BEPWTP lagoons may be capable of handling the flow.

The third method of reservoir cleaning is manual wash down with high-pressure hoses, using the basin drains as done in the past. Although labor intensive, this method poses the least risk to the liners and would help ensure that solids in the drainage system would be sufficiently diluted to prevent deposition in the drainage pipelines.

A summary of the reservoir cleaning methods is presented in Table 6-1. The plastic liner has a design life of 20 years. Because of the warranty restrictions associated with the liner, a concrete overlay is recommended for either of the first two methods of cleaning. Once this type of protection is provided, the liner life is likely to be extended to more than 20 years. In addition to avoiding expensive repairs, these benefits may justify part or all of the cost of a concrete overlay.

Regardless what decision is chosen for the method of cleaning, the frequency of cleaning should be significantly increased (that is, no less often than every five years), so that fewer solids will have to be removed. As a result, the cleaning projects will be simpler and quicker. Because the first two methods are high risk for damaging the liner, the first basin cleaning should be by manual wash down and drainage of the north basin no later than 2004. The feasibility of continuing this procedure can be reviewed at that time. If manual wash down and drainage is deemed to be too labor intensive or time consuming, the concrete overlay could be installed in the north basin and the following year in the south basin. After the concrete overlay is installed, competitive bids for both mechanical excavation and dredging could be received to determine the better method of cleaning.

TABLE 6-1
Raw Water Reservoir Cleaning Methods

Method Description	Advantages	Disadvantages
1. Mechanical excavation and haul away.	Does not risk overloading the 24-inch pipeline to the lagoons. Does not risk clogging the drain pipelines. Vehicle access ramps already installed.	Excavating equipment could potentially damage the liner. Cleaning would probably take longer and create truck traffic.
2. Dredging with a protective cage on cutterhead and a means to provide flushing velocities in the pipeline handling dredge discharge.	Simpler procedure and will be completed more quickly.	Dredging equipment could potentially damage the liner. Drain pipeline may get clogged with dredged materials.
3. Manual wash down cleaning with high-pressure hoses.	Proven and simple procedure requiring no equipment; method is least likely of all to damage liner.	Labor intensive and time consuming.
OPTION: Protective concrete overlay installed on reservoir bottom.	Protects liner for either dredging or mechanical excavation. Prolongs life of liner.	Cost.

SECTION 7

Alternative Disposal Options

Three alternative disposal options were investigated for life-cycle costs:

1. Continue lagoon disposal with recommended improvements and periodic cleaning of lagoons and dispose to a monofill or arrange for agricultural land use
2. Discharge process wastewater to the MSD
3. Construct thickening and dewatering facilities on the CHWTP site and dispose dewatered residuals to a landfill or arrange for agricultural land use

For Alternatives 1 and 3, ultimate disposal would mean transport to a landfill, monofill, or land application for agricultural purposes. Landfill disposal typically costs between $60 to $100 per ton, including hauling. Agricultural land application typically costs between $60 and $80 per ton and monofill disposal could be as low as $10 per ton if site conditions are favorable. Because of the more favorable cost and expected availability of land for this purpose, land disposal by monofill was assumed for Alternatives 1 and 3.

All three alternatives would require a new backwash holding tank in parallel with the existing tank to permit the existing tank to be removed from service for cleaning and maintenance. The estimated cost of the tank is $1,000,000 and backup data for the estimate is included in Appendix E. All alternatives would need flow monitoring facilities, too, which aid in plant operation. Disposal to MSD would require additional flow monitoring for billing purposes.

Alternative No. 1 continues the present operation but, in addition to the common capital improvements, lagoon cleaning and upkeep, residuals removal and land application, and pipeline maintenance have been added as annual costs. Lagoons would be filled and taken out of service for drying, and residuals would be excavated and hauled away to a monofill or agricultural land application site. Annual costs including amortized capital costs are about $777,000.

Disposal to the MSD system (Alternative No. 2) would conceptually use the existing 42-inch sewer on Grinstead Drive for disposal. Although this alternative would eliminate further handling and disposal of residuals by the LWC, operating costs, shown in Table 7-1, would be very expensive for the estimated level of solids and volume of wastewater projected to occur. Disposal costs would have three components: flat administrative fee, volume charge, and water quality charge. Table 7-1 shows the charges that would occur for industrial customers based on projected wastewater flow rates and solids loadings. MSD customers have the freedom to choose one of two methods for rate determination, based on cost advantage. For the CHWTP, Method 2 yields the lower annual cost of $4.817 million. Where process wastewater could actually be discharged, and approval to do so, would need to be verified with MSD, particularly since the Grinstead Drive line is a combined sewer. However, it appears that even if no capital improvements were required (except for the

TABLE 7-1
MSD Disposal Costs

Waste Flow Stream	Regular Industrial Volume Charge/Year[a]	Discount Industrial Volume Charge/Year[b]	Full Quality Charge/Year, ($/mg/L)[c]	Quality Charge/ Year, -270 mg/L ($/mg/L)[d]	Total Annual Cost, Method 1[e]	Total Annual Cost, Method 2[e]
	A	B	C	D	A + D + ADMIN	B + C + ADMIN
N. Coagulation Basins	$686,000	$354,000	$1,272,900	$1,205,500	$1,892,000	$1,627,000
S. Coagulation Basins	$961,000	$496,000	$1,782,100	$1,687,700	$2,649,000	$2,278,000
Softening Basins	$412,000	$212,000	$5,300	$0	$412,000	$217,000
Average filter backwash[b]	$1,210,000	$624,000	$62,800	$0	$1,210,000	$687,000
Total	**$3,269,000**	**$1,686,000**	**$3,123,100**	**$2,893,200**	**$6,163,000**	**$4,809,000**

Notes:

[a] Regular Industrial Volume Rate, $/1,000 gal	$1.88
[b] Discount Industrial Volume Rate, $/1,000 gal	$0.97
[c] Quality Charge for all SS, $/mg/L per 1,000 gal	$0.0006838
[d] Discounted Quality Charge for SS >270 mg/L, $/mg/L per 1,000 gal	$0.0006838
[e] Includes administrative services charges, $/month	$39.90

backwash holding tank and flow monitoring facilities) to discharge wastewater to MSD at Grinstead Drive, this would be the highest cost alternative with annual costs of $4.9 million.

Alternative No. 3 would provide gravity thickeners and gravity belt presses in a new building for dewatering residuals, ideally near the CHWTP site. Coagulation sludge would be transported to the thickener. Thickened sludge would then be pumped to the belt presses for dewatering. Before dewatering, the sludge would be conditioned with polymer. After dewatering, residuals would be hauled to an agricultural land application site. Land area for a dewatering system close to CHWTP may be challenging to acquire. Four thickeners approximately 75 feet in diameter would be required. The belt press building would require less space than the thickeners but would need to be configured for continuous truck traffic to haul off an average of 50 dry tons per day of residuals. Annual costs including amortized capital costs are about $2.6 million.

Cost comparisons for all three alternatives are shown in Table 7-2. Several assumptions, listed at the bottom of the table, were used to prepare this cost comparison. Alternative No. 1, the present lagoon system, is by far the most cost-effective option in consideration of both annual and total life cycle costs, and disposal using the existing facilities should continue. However, many maintenance activities will soon be needed to keep this system in good functioning order. These activities are described in the next section.

If the 24-inch pipeline cannot be rehabilitated to restore flow rates to an adequate capacity for future wastewater flows, a combination of Alternatives No. 1 and 2 may be possible as an interim solution while the pipeline is repaired or even replaced. For example, because the filter backwash and softening basins have much lighter solids content than the coagulation basin sludge, the MSD disposal of this wastewater would be considerably less than the composite average of all wastewater from CHWTP, so the unit cost ($/1,000 gallons) would be much less. As a result, the MSD system could be cost-effectively used for only a portion of the wastewater disposal.

TABLE 7-2
Cost Comparison of Residuals Disposal Options for Crescent Hill WTP

Description	Option 1 Pipeline and Lagoon Dewatering	Option 2 Disposal to MSD Sewers	Option 3 Mechanical Dewatering
Annual Costs			
Payment to MSD	$0	$4,809,000	$0
Lagoon brush cleaning[b]	$50,000	$0	$0
Lagoon solids excavation[cd]	$136,000	$0	$0
Pipeline maintenance[efg]	$40,000	$0	$0
Mechanical dewatering O&M[h]	$0	$0	$1,146,000
Dry land disposal[i]	$382,000	$0	$382,000
Subtotal, annual costs	**$608,000**	**$4,809,000**	**$1,528,000**
Capital Costs			
Washwater holding tank & pumps	$1,000,000	$1,000,000	$1,000,000
Flow monitoring facilities	$75,000	$100,000	$75,000
Gravity thickeners[jk]	$0	$0	$3,900,000
Dewatering equipment and building[l]	$0	$0	$5,200,000
Subtotal, capital costs	**$1,075,000**	**$1,100,000**	**$10,175,000**
Amortized capital costs[m]	**$101,000**	**$104,000**	**$960,000**
TOTAL	**$709,000**	**$4,913,000**	**$2,488,000**

Notes and assumptions:

[a]Total projected dry solids at CHWTP, tons/year	19,100
Total projected volume of wet solids at CHWTP, cy/year	45,300
[b]Assume one lagoon brush-cleaned per year	$50,000
[c]Lagoon solids onsite excavation, $/cy	$3
[d]Lagoon solids content	50.0%
(Lagoon solids density, tons/cy)	0.84
[e]Cleaning truck use, $/hour	$150
[f]Pipeline cleaning crew, $/hour	$100
[g]Pipeline cleaning labor, hours/year	160
[h]Mechanical dewatering O&M cost, $/ton	$60
[i]Land disposal including 15-mile haul, $/ton	$10
[j]Gravity thickener loading rate, lb/sf	6
[k]Thickener cost, $/sf	$226
[l]Capital cost for dewatering equipment, $/dry ton/day	$100,000
[m]Capital recovery factor for 20 years and 7%	0.0944

SECTION 8

Conclusions and Recommendations

Findings and Conclusions

1. Flow projections for the 10-year study period indicate an average finished water production rate of 40 and 114 mgd for BEPWTP and CHWTP, respectively.

2. Sludge flow rates assumed (not measured or targeted) for coagulation basins are 400,000 gpd, each (3.2 mgd total), although the four south basins are 40 percent larger than the four north basins. The assumed sludge flow rate for the six softening basins is 300,000 gpd, each (1.8 mgd total). Therefore, total assumed sludge flow for all 14 basins is 5 mgd. Actual sludge flow rates appear to be less than the assumed values and sometimes significantly less. If the assumed values were used in operations, a total of 2.8 percent of the treated water would be drawn off in sludge flow at the plant's design capacity of 180 mgd. At lower capacities the percentage drawn off would increase since sludge draw-off rates remain fairly constant. For example, at a 115-mgd plant production rate, the assumed sludge flows represent 4.4 percent of total production.

3. Basin sludge flows for 1980 when specific records were kept reportedly averaged 210,000 gpd, each, for the coagulation basins and 230,000 gpd, each, for the softening basins totaling 3.04 mgd for all basins, which is 60 percent of the assumed values. The average sludge flow rates based on two field measurements made in April 2000 were 276,000 gpd, each, for the North Coagulation Basins, 333,000 gpd, each, for the South Coagulation Basins, and 123,000 gpd, each, for the softening basins, totaling 2.32 mgd for all basins, which is 46 percent of the assumed values. Clearly, the assumed values are too high to reflect present operating conditions or target values.

4. Filters are backwashed at a very low flow rate of 13 gpm/square foot. The plant staff believe that the media is not being adequately cleaned as could be accomplished with higher backwash rates, but further investigation by LWC is planned. Higher backwash rates at shorter durations may be more effective and use less water.

5. Based on normal filtration protocol for filter cycle time and backwashing, the north filter group has a lower net production rate than the east and south groups, yet even the North Group with the highest estimated ratio of 2.1 percent for filter backwash divided by total filtered water per cycle does not use an excessive amount of backwash by industry standards. Normally, gravity rapid sand filters would be expected to use less than 3 percent backwash and those that use less than 2 percent would be considered very good. The North Group produces the lowest unit filter run volume of 8,500 gallons, based on targeted filtration rates and run times, and yet this exceeds the industry standard that would expect a minimum of only 5,000 gallons. Regardless of these favorable indicators, filter run times may be shorter than necessary to prevent particle breakthrough in one or more filter groups, and filtration rates may be lower than the optimal rates needed to maximize water production.

6. The turbidity limits for individual filters established by the IESWTR require filters by December 2001 to become sufficiently ripened to produce filtered water initially below 1.0 NTU, and below 0.5 NTU after 4 hours of operation. Presently, LWC filters are reportedly producing water at less than 0.25 NTU immediately after backwash, which will comply with the rule. If turbidity levels are lowered further in future regulations, or if voluntary strategies, such as the Partnership for Safe Water Program, are adopted resulting in lower target turbidity levels, the feasibility to provide filter-to-waste or some type of filter media conditioning should be investigated.

7. Most surface water treatment plants that do not practice filter backwash recycling would typically generate less than 4 percent process wastewater. In the solids handling survey reported in Section 4, all five utilities produce total process wastewater ranging from 1.5 to 3.1 percent of total water production, whereas CHWTP averaged 4.4 percent based on field measurements taken April 2000. For filter backwashing alone the usage ranges from 1.2 to 2.5 percent for the five utilities, and CHWTP averages about 1.6 percent, if an equal mix of all filter groups are used and standard protocol for filtration rates and run times are followed. Therefore, LWC falls in the mid-range of what is occurring with filter backwash water at the other surveyed utilities. However, total process wastewater flow at 4.4 percent in CHWTP last April appears to be somewhat higher than the industry average and considerably higher than the five utilities surveyed, due to the sludge flows. Target values to reduce current sludge flows should be established but would be difficult to adhere to without regularly monitoring flow rates after flushing.

8. For the 2-week maximum production period of 2010, a finished water production rate of 149 mgd is projected. During this period one filter from the South Group, three filters from the East Group, and four filters from the North Group would need to be backwashed in a 24-hour period if present day protocol is followed.

9. The wash water holding tank has an effective volume of 800,000 gallons. In the southeast portion of the tank solids have accumulated up to about 11 feet of depth. The overall water depth available is about 19 feet (elevation 526 to 545 feet). Because the tank used to be a clear well, it is covered and below grade. The cover hampers maintenance and cleaning and the pumps are also difficult to access. There is no reason to use a covered tank for backwash wastewater, although the cover may provide structural benefit.

10. Based on the projected backwashing requirements for 2010, the holding tank volume (allowing for no solids deposition) is just adequate to handle backwash, softening basin sludge flows of 1.0 mgd, and one basin drainage stretched out over 72 hours, which is estimated to be an additional 0.9 mgd average rate for a 2.7-MG basin volume. However, the backwashing schedule must be precisely sequenced to optimize holding tank volume for equalization. This projected condition does not allow any storage space for solids deposition and places the plant in a vulnerable condition should maintenance problems occur in the backwash wastewater drainage system.

11. The wash water holding tank will require pumping rates of 5 mgd during peak conditions of 2010 reduced to 4 mgd after the last backwash of the day has been completed.

12. If a new backwash holding tank was provided in parallel with the existing one to improve reliability it could be sized to handle average backwashing conditions projected for a design year of 2010. For this application the tank's minimum volume requirement would be 0.5 MG, if the presently assumed softening basin sludge flows of 1.8 mgd were handled by the tank and the existing backwash tank was out of service for maintenance. The volume requirement would be reduced to 0.4 MG if the softening basin sludge flows were reduced to 1.0 mgd, which is an attainable target flow rate if partial softening in two of the six basins is practiced. These volumes are the minimum requirement and offer little flexibility in backwash scheduling.

13. The present factory performance curves of the backwash holding tank pumps were input in a plant drainage computer model. Flow rate for one pump operating was 3.47 mgd at a very low head. This is not a favorable operating condition for the pump because it is far outside its intended operating range. However, when two pumps operate, the model indicates that the combined flow rate increases to 4.72 mgd (2.36 mgd per pump) and the pumps would then be in a favorable operating range. This is caused by the significant increase in head loss in the 24-inch pipeline as a result of the total flow rate (with coagulation sludge flow) increasing from 5.9 to 7.1 mgd. Flow rate and performance of the pumps are unknown because there is presently no flow rate monitoring.

14. Based on the computer model results investigated, the drainage system will be unable to transport to the lagoons the projected filter backwash wastewater plus the basin sludge flows, even reduced to suggested target values, if the friction factors computed by Pitometer Associates in their March 27, 2000, report are correct. (Friction C-factors reported to be higher than 100 were set equal to 100.) The model indicates that an overflow at the manhole connected to the coagulation sludge line in Reservoir Avenue and at the weir boxes in the Coagulation Control Houses would result. The predicted overflow condition at the weir boxes, but not the manhole, could be eliminated if the friction C-factor for all drain piping including the 24-inch pipeline to the lagoons was increased to 100, which is a reasonable value to assume for old pipe. Eliminating the solids deposition may be the only improvement needed to attain C-factors of 100, or better.

15. Samples were collected and laboratory analyses were performed on several wastewater streams in CHWTP and at Lagoon No. 2. All pH results were within the 6 to 9 S.U. range stipulated in previous KPDES discharge permits. Chlorine residual ranged from 1.2 to 3.2 mg/L in the wastewater processes but was only 0.5 mg/L entering Lagoon No. 2. That residual concentration would quickly dissipate before reaching the outlet structure and discharging to the river. TSS was highly variable among all the sampling locations ranging from less than 3 mg/L for filter influent to 752 mg/L for coagulation basin sludge. BOD_5 was 2 mg/L, or less, in the six analyses performed. Thirteen priority pollutant metals were analyzed in a sample of Lagoon No. 2 influent and all but barium, chromium, copper, and iron were below reportable limits. Of the metals above reportable limits, none were present in unusual concentrations and would be considered insignificant as previously reported in *Washwater Lagoon Investigation and Monitoring Program* (August 1992).

16. Overall, the BEPWTP lagoons are about three-fourths filled. Both plants are projected to produce residuals at an average rate of approximately 60 million pounds per year at for the next 10 years. At an assumed dryness concentration of 50 percent, the residuals volume would be 1.9 million cubic feet per year at CHWTP and BEPWTP, combined. At this rate all lagoons will be filled in less than 8 years if none are cleaned.

17. Velocities in the 30- and 36-inch sludge and drain pipelines are probably too slow to prevent deposition of heavy solids. Even at low flow rates, the velocity in the 24-inch pipeline along River Road may be too slow to prevent deposition, especially at low points. Air release valves on the 24-inch pipeline were recently observed to be inoperative and trapped air in this pipeline could greatly affect carrying capacity.

18. Three alternative disposal methods and their life cycle costs were investigated. In addition to the present pipeline and lagoon disposal option, discharge to the MSD sewers and construction of onsite (or nearby) thickening and dewatering facilities for agricultural land application. The present method was by far the most cost-effective from a capital and annual cost basis and should be continued. However, many maintenance activities will soon be needed to keep this system in good functioning order.

Recommendations

The following recommendations are presented in the general sequence of need to implement. Some recommendations are dependent on others being implemented first.

1. **Develop plan for lagoon operation, filling sequence, and cleaning.** Focus on filling a single lagoon at a time until full so it can be removed from service, cleaned and made ready to return to service. A land disposal site (monofill) should be investigated as the most cost-effective means of disposing of water treatment residuals. By filling and cleaning each lagoon one at a time, the lagoon cleaning requirements will be spread out more evenly and more time will be available for cleaning, whether done in-house or by outside contractor. Lagoon No. 2 is the most completely filled lagoon and should be cleaned first. Once the lagoon is filled to 90 percent of its gross capacity, which will occur very soon, 8,500,000 cubic feet, or 315,000 cubic yards, of sludge will need to be removed. After a lagoon is removed from service it should be allowed to rest about 6 months to a year to dewater as much as possible before solids are excavated. At an assumed $3 per cubic yard mechanical excavation cost and $10 per ton land disposal cost, the overall sludge removal cost would be about $3.6 million for the entire Lagoon No. 2. The land disposal cost assumes that nearby favorable property (within 15 miles of the lagoon) can be obtained and that special liners, drainage systems, or other environmental protective measures or site embellishments (such as fencing, access roads, stormwater diversion, etc.) would not be required. After a potential site is located, the need for these measures and embellishments would have to be investigated. If improvements are required the cost per cubic yard could be substantially higher.

2. **Conduct a geotechnical investigation of Lagoon No. 1 and 4** to determine if they are leaking and complete berm repairs if needed. It would be prudent to make needed

repairs as soon as possible while the lagoons are out of service and before they are needed.

3. **Modify BEPWTP backwash drain piping** to allow discharge to Lagoons No. 1 and 2, and modify the collector box drain to divert water from Lagoon No. 2 to Lagoon No. 1. The estimated cost of modifying the 42-inch backwash drainage system is $265,000 (see Appendix E), assuming the use of 1,000 feet of 42-inch pipe, two new inlet splash pads and other erosion control measures for Lagoons No. 1 and 2, and three sluice gates. A cost estimate for the collector box drainage improvements was not prepared because the requirements are not well defined; however, a gravity drainpipe to Lagoon No. 1 would be a nominal cost, if pumping was not required.
4. **Build new wash water holding tank and pump station.** A volume of 0.5 MG is all that is needed to handle average filter backwashing conditions in 2010, but a volume of 0.8 MG, equal to the existing tank is recommended for additional flexibility in operations and to be adequately sized beyond 2010. The new tank would be interconnected with the backwash drainpipe presently flowing into the existing tank. A new dry well for vertical centrifugal solids handling pumps would be connected to both tanks so that either tank could be removed from service without interrupting filter backwashing. A screening and grit chamber could be installed in front of both holding tanks to capture leaves, heavy lime solids, and other debris that is not desired in the drainage system. The debris collected in the chamber could be loaded into a dumpster or dump truck for offsite disposal, similar to screenings at a wastewater treatment plant.
5. **Develop and use the Zorn Lagoon.** This lagoon could be used any time the 24-inch pipeline is out of service for inspections and maintenance. The site should be surveyed and to construct a lagoon as large as possible on the site. An outlet structure similar to the BEPWTP lagoons should be provided to minimize the solids carryover to the river, since TSS for this discharge is regulated by the NPDES permit. The existing bypass pipe that connects to the river outlet should remain in service in case it is needed. The estimated cost to construct this lagoon with piping connections, fencing, a short driveway, telemetry interconnections to the Zorn Avenue Pump Station for water level and effluent flow rate measurements, and concrete effluent structure is $500,000 (see Appendix E). The earthwork was assumed to have a balanced cut and fill requirement for a lagoon approximately 10 feet deep.
6. **Check air release valves on 24-inch pipeline to BEP lagoons.** Replace valves and bleed off trapped air if inoperative valves are found.
7. **Inspect sludge and drain pipelines using a TV camera.** To determine the extent of solids deposition, all drain and sludge lines should be inspected by TV. The coagulation sludge and backwash wastewater would have to be converted to the drainage pipeline while inspections are conducted.
8. **Improve pipeline capacity by jet cleaning or pigging.**
9. **Install flow rate metering** for each of the 14 basin sludge flows at their weir boxes. An ultrasonic unit as manufactured by Milltronics can be programmed for each type of weir used and will provide a direct readout. The pump discharge flow rate out of the backwash holding tank should be measured with a magnetic flowmeter on the 12- and

20-inch lines. Alternatively, a single pipeline leaving the pump station could be metered before it splits into the 12- and 20-inch force mains. The backwash holding tank level should also be monitored.

10. **Establish target basin sludge flows** of 0.25 and 0.35 mgd, each, for the North and South Coagulation Basins, respectively, and 0.1 mgd, each, for the softening basins. This will be an initial target value. These values should be further reduced, if possible to do without clogging sludge lines. Adding flow rate measurement will facilitate testing for optimum sludge flow rates. If softening is resumed in any of the basins the target sludge flow rate should be increased to 0.3 mgd per basin based on anticipated solids loading.

11. **Clean and remove vegetation from the lagoons.** Once cleaned the vegetation may be discouraged from returning by maintaining a foot or so of water above the bottom or accumulated solids in the bottom. The underdrain pipe that discharges into the outlet structure may need to be plugged or valved off to maintain a certain level of water.

12. **Improve 24-inch pipeline reliability along River Road** by installing a double-barrel configuration at problem areas such as Harrods Creek or other creek crossings. These double-barreled creek crossings at problem areas would not be required if TV inspections show that solids deposition is not occurring in these areas or if pigging of the entire pipeline can be performed in a short enough time period that can be tolerated by the CHWTP drainage system. Valved connections with pig launchers would be installed at each end of the two barrels and either barrel could be inspected or cleaned while the other was kept in service. The estimated cost to install a parallel pipeline at a 150-foot-long creek crossing using jack-and-bore installation with 30-inch casing, fittings, four isolation valves, two pig launchers and appurtenances is $235,000 per crossing (see Appendix E). A long-term solution would be to provide a parallel pipeline from Zorn Avenue to the BEPWTP lagoons.

13. **Remove cover of existing backwash holding tank** and extend walls above grade with handrails on top to facilitate operation and maintenance of the tank.

14. **Exercise valves and paint metalwork in the lagoons at BEPWTP.**

15. **Conduct TSS sampling of the softening basin sludge flows** on a regular schedule so that a relationship between sludge flow rates, lime dosage, and TSS concentrations can be established. This relationship will allow solids production to be more accurately projected.

16. **Replace sludge butterfly valves** with V-port plug valve or L Series knife gate valve with a V-port that is designed for flow control of slurries, both manufactured by DeZurik. Consider automating flushing of the sludge lines to reduce overall volume wasted.

17. **Construct new manhole on LWC property** connected to the coagulation sludge pipeline and seal off existing manhole that overflows. Set manhole top elevation at approximately 562 feet to provide extra pressure head to the pipeline, but still protect the weir boxes in the Coagulation Control Houses from overflowing.

18. **Establish filtered water turbidity goals and investigate feasibility of filter-to-waste.** Establish LWC goals for filtered water turbidity and determine what filter renovations and operating modifications are required to meet the goals. Conduct turbidity profiling

for all filters to determine duration of filter media ripening after backwashing and whether or not FTW could improve filtered water quality. If FTW appears to be feasible, consider a pumped FTW system that would keep FTW water isolated from other process wastewater so that FTW could be recycled.

19. **Improve filter backwash effectiveness** by increasing rate and shortening duration.
20. **Investigate the feasibility of longer filter runs,** if turbidity breakthrough and excessive head losses do not occur.
21. **Investigate the feasibility of higher filtration rates,** again if turbidity breakthrough and excessive head losses do not occur.
22. **Consider resuming the practice of recycling backwash water** to the raw water reservoir to significantly reduce process wastewater flows. Although many utilities are now avoiding this practice because of the threat of *Giardia* and *Cryptosporidium* cysts being recycled to the plant influent in greater concentration, recycle to the raw water reservoirs in CHWTP appears to be permissible under the proposed FBR Rule. Before considering this practice again, however, filter backwash water should be sampled for *Giardia* and *Cryptosporidium* and compared to occurrence in the raw water.
23. **Provide basin transfer pumps** to significantly reduce the amount of water to be drained when emptying basins. One pump could be installed and shared for each of the three basin groups with manifolded suction pipes that withdraw the top two-thirds of the basin water and discharge to the basin influent flume. Alternatively, a portable pump could be used, provided that the suction piping is properly installed to minimize uptake of solids on the basin bottom.
24. **Develop a plan for cleaning the raw water reservoir** at regular, more frequent intervals than done in the past (e.g. no less often than every five years) so that the cleaning projects will be simpler and quicker. When time to clean again, use manual wash down to drainage to avoid damaging the plastic liner. If this method is still considered too labor intensive and time consuming even if the reservoir basins are cleaned more frequently, a concrete overlay should be installed to protect the liner from more invasive methods of cleaning. The concrete overlay will avoid expensive liner repair costs and prolong the liner's life. When time to again clean the basins after the overlay is installed, obtaining contractor bids for both mechanical excavation and hydraulic dredging will promote competition between the two methods to procure better prices for cleaning. If dredging is to be considered, consider diverting dredging wastewater to the 30-inch coagulation sludge pipeline rather than the 36-inch drain line so that flushing velocities will be more readily attained. Also consider draining the lagoon while dredging to maintain high velocities and flushing in the pipeline.
25. **Consider a 3-inch concrete protective layer** in the bottom of the reservoir after the next scheduled cleaning. This protection may be useful for either dredging or mechanical cleaning. Assuming a 3-inch thick concrete layer with reinforcing steel, an allowance of $2 per square foot would cover the cost of this protective layer. Assuming a bottom surface area of about 700,000 square feet in both basins, total cost would be $1,400,000. If dredging is considered without the protective layer, the dredge should be equipped

with a protective cage around the cutterhead. See Appendix C for a manufacturer's catalog cut sheet of a cutterhead with this attachment.

Appendix A

Backwash Holding Tank Modeling Results

TABLE A-1
Wastewater Holding Basin Pumping Requirements--2-Week Maximum Flow in Year 2010
Condition: One South, Three East, and Four North Filter Group Backwashes; four Softening Basins at 0.1 mgd each, two at 0.3 mgd

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
		Backwashing begins				
1	6	South #1	0.385	0.010	0.042	0.354
2				0.010	0.042	0.323
3				0.010	0.042	0.292
4				0.010	0.042	0.260
5	7	East #1	0.385	0.010	0.042	0.614
6				0.010	0.042	0.583
7				0.010	0.042	0.552
8				0.010	0.042	0.521
9	8	North #1	0.193	0.010	0.042	0.682
10				0.010	0.042	0.651
11				0.010	0.042	0.620
12				0.010	0.042	0.588
13	9			0.010	0.042	0.557
14				0.010	0.042	0.526
15				0.010	0.042	0.495
16				0.010	0.042	0.463
17	10			0.010	0.042	0.432
18		East #2	0.385	0.010	0.042	0.786
19				0.010	0.042	0.755
20				0.010	0.042	0.724
21	11			0.010	0.042	0.692
22				0.010	0.042	0.661
23				0.010	0.042	0.630
24		North #2	0.193	0.010	0.042	0.791
25	12			0.010	0.042	0.760
26				0.010	0.042	0.729
27				0.010	0.042	0.697
28				0.010	0.042	0.666
29	13			0.010	0.042	0.635
30				0.010	0.042	0.604
31				0.010	0.042	0.572
32				0.010	0.042	0.541
33	14			0.010	0.042	0.510
34				0.010	0.042	0.479
35				0.010	0.042	0.447
36				0.010	0.042	0.416
37	15	East #3	0.385	0.010	0.042	0.770
38				0.010	0.042	0.739
39				0.010	0.042	0.708
40				0.010	0.042	0.677
41	16			0.010	0.042	0.645
42				0.010	0.042	0.614
43		North #3	0.193	0.010	0.042	0.775
44				0.010	0.042	0.744

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
45	17			0.010	0.042	0.713
46				0.010	0.042	0.682
47				0.010	0.042	0.650
48				0.010	0.042	0.619
49	18	North #4	0.193	0.010	0.042	0.781
50				0.010	0.042	0.749
51				0.010	0.042	0.718
52				0.010	0.042	0.687
53	19			0.010	0.042	0.656
54				0.010	0.042	0.624
55				0.010	0.042	0.593
56				0.010	0.042	0.562
57	20			0.010	0.042	0.531
58				0.010	0.042	0.499
59				0.010	0.042	0.468
60				0.010	0.042	0.437
61	21			0.010	0.042	0.406
62				0.010	0.042	0.374
63				0.010	0.042	0.343
64				0.010	0.021	0.333
65	22			0.010	0.021	0.322
66				0.010	0.021	0.312
67				0.010	0.021	0.301
68		Backwashing ends		0.010	0.021	0.291
69	23			0.010	0.021	0.281
70				0.010	0.021	0.270
71				0.010	0.021	0.260
72				0.010	0.021	0.249
73	24			0.010	0.021	0.239
74				0.010	0.021	0.228
75				0.010	0.021	0.218
76				0.010	0.021	0.208
77	1			0.010	0.021	0.197
78				0.010	0.021	0.187
79				0.010	0.021	0.176
80				0.010	0.021	0.166
81	2			0.010	0.021	0.156
82				0.010	0.021	0.145
83				0.010	0.021	0.135
84				0.010	0.021	0.124
85	3			0.010	0.021	0.114
86				0.010	0.021	0.103
87				0.010	0.021	0.093
88				0.010	0.021	0.083
89	4			0.010	0.021	0.072
90				0.010	0.021	0.062
91				0.010	0.021	0.051
92				0.010	0.021	0.041
93	5			0.010	0.021	0.031

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
94				0.010	0.021	0.020
95				0.010	0.021	0.010
96				0.010	0.021	-0.001
TOTALS			2.312	1.000	3.313	

Volumes in 15-minute increments:			Rates:	
1 Pump	0.02083	MG	2.000	mgd
2 Pumps	0.04167	MG	4.000	mgd
3 Pumps	0.05208	MG	5.000	mgd
Softening Basin Sludge	0.01042	MG	1.000	mgd

Wash Water Holding Basin

HWL	545	feet
LWL	528	feet
Diameter	90	feet
Effective Volume	809,000	gallons

Filter Backwashing Volumes, MG

South Group	0.385
East Group	0.385
North Group	0.193

Conclusions:

1. Entire 0.8 MG basin is required; it must be kept clean
2. Maximum pumping rate is 4 mgd
3. Last backwash is at 1800 hours
4. Pumping rate can be reduced to 2 mgd at 2145 hours until 0600 hours the next morning

TABLE A-2

Wastewater Holding Basin Pumping Requirements--2-Week Maximum Flow in Year 2010

Condition: One South, Three East, and Four North Filter Group Backwashes; four Softening Basins at 0.1 mgd each, two at 0.3 mgd

Softening Basin Drainage in 72 hours

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Softening Basin Drainage, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
		Backwashing begins					
1	6	South #1	0.385	0.010	0.009	0.052	0.353
2				0.010	0.009	0.052	0.321
3		East #1	0.385	0.010	0.009	0.052	0.674
4				0.010	0.009	0.052	0.641
5	7			0.010	0.009	0.052	0.609
6				0.010	0.009	0.052	0.577
7				0.010	0.009	0.052	0.545
8		North #1	0.193	0.010	0.009	0.052	0.705
9	8			0.010	0.009	0.052	0.673
10				0.010	0.009	0.052	0.640
11				0.010	0.009	0.052	0.608
12				0.010	0.009	0.052	0.576
13	9			0.010	0.009	0.052	0.543
14				0.010	0.009	0.052	0.511
15				0.010	0.009	0.052	0.479
16				0.010	0.009	0.052	0.447
17	10	East #2	0.385	0.010	0.009	0.052	0.800
18				0.010	0.009	0.052	0.767
19				0.010	0.009	0.052	0.735
20				0.010	0.009	0.052	0.703
21	11			0.010	0.009	0.042	0.681
22				0.010	0.009	0.042	0.659
23				0.010	0.009	0.042	0.637
24				0.010	0.009	0.042	0.615
25	12	North #2	0.193	0.010	0.009	0.042	0.786
26				0.010	0.009	0.042	0.764
27				0.010	0.009	0.042	0.742
28				0.010	0.009	0.042	0.720
29	13			0.010	0.009	0.042	0.698
30				0.010	0.009	0.042	0.677
31				0.010	0.009	0.042	0.655
32				0.010	0.009	0.042	0.633
33	14			0.010	0.009	0.042	0.611
34				0.010	0.009	0.042	0.589
35				0.010	0.009	0.042	0.567
36				0.010	0.009	0.042	0.545
37	15			0.010	0.009	0.042	0.523
38				0.010	0.009	0.042	0.502
39				0.010	0.009	0.042	0.480
40				0.010	0.009	0.042	0.458
41	16			0.010	0.009	0.042	0.436
42				0.010	0.009	0.042	0.414
43		East #3	0.385	0.010	0.009	0.042	0.778
44				0.010	0.009	0.042	0.756

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Softening Basin Drainage, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
45	17			0.010	0.009	0.042	0.734
46				0.010	0.009	0.042	0.712
47				0.010	0.009	0.042	0.690
48				0.010	0.009	0.042	0.668
49	18			0.010	0.009	0.042	0.646
50				0.010	0.009	0.042	0.624
51		North #3	0.193	0.010	0.009	0.042	0.795
52				0.010	0.009	0.042	0.773
53	19			0.010	0.009	0.042	0.751
54				0.010	0.009	0.042	0.730
55				0.010	0.009	0.042	0.708
56				0.010	0.009	0.042	0.686
57	20			0.010	0.009	0.042	0.664
58				0.010	0.009	0.042	0.642
59				0.010	0.009	0.042	0.620
60		North #4	0.193	0.010	0.009	0.042	0.791
61	21			0.010	0.009	0.042	0.769
62				0.010	0.009	0.042	0.747
63				0.010	0.009	0.042	0.725
64				0.010	0.009	0.042	0.703
65	22			0.010	0.009	0.042	0.682
66				0.010	0.009	0.042	0.660
67				0.010	0.009	0.042	0.638
68		End of Normal Backwashing		0.010	0.009	0.042	0.616
69	23			0.010	0.009	0.042	0.594
70				0.010	0.009	0.042	0.572
71				0.010	0.009	0.042	0.550
72				0.010	0.009	0.042	0.528
73	24			0.010	0.009	0.042	0.507
74				0.010	0.009	0.042	0.485
75				0.010	0.009	0.042	0.463
76				0.010	0.009	0.042	0.441
77	1			0.010	0.009	0.042	0.419
78				0.010	0.009	0.042	0.397
79				0.010	0.009	0.042	0.375
80				0.010	0.009	0.042	0.353
81	2			0.010	0.009	0.042	0.332
82				0.010	0.009	0.042	0.310
83				0.010	0.009	0.042	0.288
84				0.010	0.009	0.042	0.266
85	3			0.010	0.009	0.042	0.244
86				0.010	0.009	0.042	0.222
87				0.010	0.009	0.042	0.200
88				0.010	0.009	0.042	0.178
89	4			0.010	0.009	0.042	0.157
90				0.010	0.009	0.042	0.135
91				0.010	0.009	0.042	0.113
92				0.010	0.009	0.042	0.091
93	5			0.010	0.009	0.042	0.069
94				0.010	0.009	0.042	0.047
95				0.010	0.009	0.042	0.025
96				0.010	0.009	0.042	0.003
TOTALS			2.312	1.000	0.900	4.208	

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Softening Basin Drainage, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG

Volumes in 15-minute increments:			**Rates:**	
1 Pump	0.02083	MG	2.000	mgd
2 Pumps	0.04167	MG	4.000	mgd
3 Pumps	0.05208	MG	5.000	mgd
Softening Basin Sludge	0.01042	MG	1.000	mgd
Softening Basin Drainage	0.00938	MG	0.900	mgd

Wash Water Holding Basin

HWL	545	feet
LWL	528	feet
Diameter	90	feet
Effective Volume	809,000	gallons

Filter Backwashing Volumes, MG

South Group	0.385
East Group	0.385
North Group	0.193

Conclusions:

1. Entire 0.8 MG basin is required; it must be kept clean
2. Maximum pumping rate is 5 mgd
3. Last backwash is at 2045 hours
4. Pumping rate can be reduced to 4 mgd at 1100 hours until 0600 hours the next morning
5. Softening basin drainage is stretched out over 72 hours

TABLE A-3
Wastewater Holding Basin Size Requirements for Average Conditions--New Basin Sizing
Condition: One South, Two East, and Three North Filter Group Backwashes; six Softening Basins at 0.3 mgd each

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
		Backwashing begins				
1	6	South #1	0.39	0.019	0.042	0.36
2				0.019	0.042	0.34
3				0.019	0.042	0.32
4				0.019	0.042	0.29
5	7			0.019	0.042	0.27
6				0.019	0.042	0.25
7				0.019	0.042	0.22
8				0.019	0.042	0.20
9	8			0.019	0.042	0.18
10				0.019	0.042	0.16
11				0.019	0.042	0.13
12		East #1	0.39	0.019	0.042	0.50
13	9			0.019	0.042	0.47
14				0.019	0.042	0.45
15				0.019	0.042	0.43
16				0.019	0.042	0.40
17	10			0.019	0.042	0.38
18				0.019	0.042	0.36
19				0.019	0.042	0.34
20		North #1	0.19	0.019	0.042	0.50
21	11			0.019	0.042	0.48
22				0.019	0.042	0.46
23				0.019	0.042	0.44
24				0.019	0.042	0.41
25	12			0.019	0.042	0.39
26				0.019	0.042	0.37
27				0.019	0.042	0.34
28				0.019	0.042	0.32
29	13			0.019	0.042	0.30
30				0.019	0.042	0.28
31				0.019	0.042	0.25
32				0.019	0.042	0.23
33	14			0.019	0.042	0.21
34				0.019	0.042	0.18
35				0.019	0.042	0.16
36				0.019	0.042	0.14
37	15	East #2	0.385	0.019	0.042	0.50
38				0.019	0.042	0.48
39				0.019	0.042	0.45
40				0.019	0.042	0.43
41	16			0.019	0.042	0.41
42				0.019	0.042	0.39
43				0.019	0.042	0.36
44				0.019	0.042	0.34

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
45	17			0.019	0.042	0.32
46				0.019	0.042	0.29
47		North #2	0.19	0.019	0.042	0.46
48				0.019	0.042	0.44
49	18			0.019	0.042	0.42
50				0.019	0.042	0.40
51				0.019	0.042	0.37
52				0.019	0.042	0.35
53	19			0.019	0.042	0.33
54		North #3	0.19	0.019	0.042	0.50
55				0.019	0.042	0.47
56				0.019	0.042	0.45
57	20			0.019	0.042	0.43
58				0.019	0.042	0.40
59				0.019	0.042	0.38
60				0.019	0.042	0.36
61	21			0.019	0.042	0.34
62				0.019	0.042	0.31
63				0.019	0.042	0.29
64				0.019	0.042	0.27
65	22			0.019	0.042	0.24
66				0.019	0.042	0.22
67				0.019	0.042	0.20
68		Backwashing ends		0.019	0.042	0.18
69	23			0.019	0.042	0.15
70				0.019	0.042	0.13
71				0.019	0.042	0.11
72				0.019	0.042	0.08
73	24			0.019	0.042	0.06
74				0.019	0.021	0.06
75				0.019	0.021	0.06
76				0.019	0.021	0.05
77	1			0.019	0.021	0.05
78				0.019	0.021	0.05
79				0.019	0.021	0.05
80				0.019	0.021	0.05
81	2			0.019	0.021	0.04
82				0.019	0.021	0.04
83				0.019	0.021	0.04
84				0.019	0.021	0.04
85	3			0.019	0.021	0.04
86				0.019	0.021	0.03
87				0.019	0.021	0.03
88				0.019	0.021	0.03
89	4			0.019	0.021	0.03
90				0.019	0.021	0.03
91				0.019	0.021	0.02
92				0.019	0.021	0.02
93	5			0.019	0.021	0.02

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
94				0.019	0.021	0.02
95				0.019	0.021	0.02
96				0.019	0.021	0.01
TOTALS			1.734	1.800	3.521	

Volumes in 15-minute increments:			Rates:	
1 Pump	0.02083	MG	2.0	mgd
2 Pumps	0.04167	MG	4.0	mgd
3 Pumps	0.05208	MG	5.0	mgd
Softening Basin Sludge	0.01875	MG	1.8	mgd

Wash Water Holding Basin

Minimum size 0.50 MG

Filter Backwashing Volumes, MG

South Group	0.385
East Group	0.385
North Group	0.193

Conclusions:

1. Provide a new basin with minimum volume of 0.50 MG
2. Maximum pumping rate is 4 mgd
3. Last backwash is at 1915 hours
4. Pumping rate can be reduced to 2 mgd at 2415 hours until 0600 hours the next morning

TABLE A-4
Wastewater Holding Basin Size Requirements for Average Conditions--New Basin Sizing
Condition: One South, Two East, and Three North Filter Group Backwashes; four Softening Basins at 0.1 mgd each, two at 0.3 mgd

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
		Backwashing begins				
1	6	South #1	0.39	0.010	0.042	0.35
2				0.010	0.042	0.32
3				0.010	0.042	0.29
4				0.010	0.042	0.26
5	7			0.010	0.042	0.23
6				0.010	0.042	0.20
7				0.010	0.042	0.17
8				0.010	0.042	0.14
9	8			0.010	0.042	0.10
10				0.010	0.042	0.07
11				0.010	0.042	0.04
12		East #1	0.39	0.010	0.042	0.40
13	9			0.010	0.042	0.36
14				0.010	0.042	0.33
15				0.010	0.042	0.30
16				0.010	0.042	0.27
17	10			0.010	0.042	0.24
18				0.010	0.042	0.21
19				0.010	0.042	0.18
20		North #1	0.19	0.010	0.042	0.34
21	11			0.010	0.042	0.31
22				0.010	0.042	0.28
23				0.010	0.042	0.24
24				0.010	0.042	0.21
25	12			0.010	0.042	0.18
26				0.010	0.042	0.15
27				0.010	0.042	0.12
28				0.010	0.042	0.09
29	13			0.010	0.042	0.06
30				0.010	0.042	0.03
31		East #2	0.39	0.010	0.042	0.38
32				0.010	0.042	0.35
33	14			0.010	0.042	0.32
34				0.010	0.042	0.29
35				0.010	0.042	0.25
36				0.010	0.021	0.24
37	15			0.010	0.021	0.23
38				0.010	0.021	0.22
39				0.010	0.021	0.21
40		North #2	0.19	0.010	0.021	0.40
41	16			0.010	0.021	0.38
42				0.010	0.021	0.37
43				0.010	0.021	0.36
44				0.010	0.021	0.35
45	17			0.010	0.021	0.34
46				0.010	0.021	0.33
47				0.010	0.021	0.32

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
48				0.010	0.021	0.31
49	18			0.010	0.021	0.30
50				0.010	0.021	0.29
51				0.010	0.021	0.28
52				0.010	0.021	0.27
53	19			0.010	0.021	0.26
54				0.010	0.021	0.25
55				0.010	0.021	0.24
56				0.010	0.021	0.23
57	20			0.010	0.021	0.22
58		North #3	0.19	0.010	0.021	0.40
59				0.010	0.021	0.39
60				0.010	0.021	0.38
61	21			0.010	0.021	0.37
62				0.010	0.021	0.36
63				0.010	0.021	0.35
64				0.010	0.021	0.34
65	22			0.010	0.021	0.33
66				0.010	0.021	0.32
67				0.010	0.021	0.31
68		Backwashing ends		0.010	0.021	0.30
69	23			0.010	0.021	0.29
70				0.010	0.021	0.28
71				0.010	0.021	0.27
72				0.010	0.021	0.25
73	24			0.010	0.021	0.24
74				0.010	0.021	0.23
75				0.010	0.021	0.22
76				0.010	0.021	0.21
77	1			0.010	0.021	0.20
78				0.010	0.021	0.19
79				0.010	0.021	0.18
80				0.010	0.021	0.17
81	2			0.010	0.021	0.16
82				0.010	0.021	0.15
83				0.010	0.021	0.14
84				0.010	0.021	0.13
85	3			0.010	0.021	0.12
86				0.010	0.021	0.11
87				0.010	0.021	0.10
88				0.010	0.021	0.09
89	4			0.010	0.021	0.08
90				0.010	0.021	0.07
91				0.010	0.021	0.06
92				0.010	0.021	0.05
93	5			0.010	0.021	0.04
94				0.010	0.021	0.03
95				0.010	0.021	0.02
96				0.010	0.021	0.00
TOTALS			1.734	1.000	2.729	

Volumes in 15-minute increments:

Rates:

15-Min Time Increment	Hour of Day	Filter Group	Backwash Volume In, MG	Softening Basin Sludge In, MG	Wastewater Flow Pumped Out, MG	Accumulated Volume, MG
1 Pump	0.02083	MG	2.0	mgd		
2 Pumps	0.04167	MG	4.0	mgd		
3 Pumps	0.05208	MG	5.0	mgd		
Softening Basin Sludge	0.01042	MG	1.00	mgd		

Wash Water Holding Basin

Minimum size 0.40 MG

Filter Backwashing Volumes, MG

South Group	0.385
East Group	0.385
North Group	0.193

Conclusions:

1. Provide a new basin with minimum volume of 0.40 MG
2. Maximum pumping rate is 4 mgd
3. Last backwash is at 2245 hours
4. Pumping rate can be reduced to 2 mgd at 1445 hours until 0600 hours the next morning

Appendix B

EPANET Model Input/Output Results

	Elevation ft	Demand MGD	Total Head ft	Pressure psi
Junction N5	424	0.00	462.32	16.60
Junction N6	437	0.00	462.96	11.25
Junction N7	438	0.00	468.14	13.06
Junction N8	438	0.00	468.34	13.14
Junction N9	440	0.00	472.18	13.94
Junction N10	440	0.00	473.71	14.61
Junction N11	442	0.00	533.50	39.65
Junction N12	556	0.00	535.49	-8.89
Junction N13	556	0.00	535.51	-8.88
Junction N14	549	0.00	538.46	-4.57
Junction N15	557.4	0.00	534.64	-9.86
Junction N16	556	0.00	534.65	-9.25
Junction N1	556	0.00	534.69	-9.23
Junction N2	556	0.00	534.72	-9.22
Junction N4	556	0.00	534.97	-9.11
Junction N17	562.5	-0.50	534.75	-12.02
Junction N18	562.5	-0.50	534.77	-12.02
Junction N19	562.7	-0.70	535.00	-12.00
Junction N20	562.7	-0.70	535.00	-12.00
Junction N21	556	0.00	534.96	-9.11
Junction N22	556	0.00	534.51	-9.31
Junction N23	442	0.00	533.51	39.65
Junction N24	451	0.00	487.73	15.91
Reservoir R2	530	-3.47	530.00	0.00
Reservoir R4	446	5.87	446.00	0.00

Pipes for 2010basecondition-1 pump.net

	Length ft	Diameter in	Roughness	Flow MGD	Velocity fps	Headloss ft/Kft	Friction Factor
Pipe P2	200	24	76	-5.87	2.90	3.19	0.049
Pipe P3	2700	24	100	-5.87	2.90	1.92	0.029
Pipe P4	100	24	100	-5.87	2.90	1.92	0.029
Pipe P5	2000	24	100	-5.87	2.90	1.92	0.029
Pipe P6	800	24	100	-5.87	2.90	1.92	0.029
Pipe P7	7300	24	100	-5.87	2.90	1.92	0.029
Pipe P8	8100	30	100	-3.47	1.10	0.25	0.033
Pipe P9	100	30	100	-3.47	1.10	0.24	0.033
Pipe P12	8500	24	100	5.87	2.90	1.92	0.029
Pipe P13	2540	21.8	100	3.47	2.08	1.16	0.031
Pipe P15	100	30	100	-2.40	0.76	0.12	0.035
Pipe P1	50	20	100	-2.40	1.71	0.89	0.033
Pipe P10	100	12	100	-0.50	0.99	0.59	0.039
Pipe P11	75	12	100	-0.50	0.99	0.59	0.039
Pipe P17	30	12	100	-0.70	1.38	1.10	0.037
Pipe P18	50	20	100	-1.90	1.35	0.58	0.034
Pipe P20	250	16	100	-1.40	1.55	0.97	0.035
Pipe P21	30	12	100	-0.70	1.38	1.10	0.037
Pipe P22	20	16	100	-0.70	0.78	0.27	0.039
Pipe P24	8100	30	100	2.40	0.76	0.12	0.035
Pipe P25	20	24	100	2.40	1.18	0.37	0.034
Pipe P26	14700	24	77	-5.87	2.90	3.11	0.048
Pipe P19	1000	30	100	2.40	0.76	0.12	0.035
Pump PP1	#N/A	#N/A	#N/A	3.47	0.00	8.46	0.000
Pump PP2	#N/A	#N/A	#N/A	0.00	0.00	0.00	0.000

Nodes for 2010basecondition-2 pumps.net

	Elevation ft	Demand MGD	Total Head ft	Pressure psi
Junction N5	424	0.00	469.30	19.63
Junction N6	437	0.00	470.21	14.39
Junction N7	438	0.00	477.62	17.17
Junction N8	438	0.00	477.89	17.28
Junction N9	440	0.00	483.37	18.79
Junction N10	440	0.00	485.57	19.74
Junction N11	442	0.00	570.94	55.87
Junction N12	556	0.00	574.44	7.99
Junction N13	556	0.00	574.49	8.01
Junction N14	549	0.00	579.69	13.30
Junction N15	557.4	0.00	572.07	6.36
Junction N16	556	0.00	572.08	6.97
Junction N1	556	0.00	572.13	6.99
Junction N2	556	0.00	572.16	7.00
Junction N4	556	0.00	572.41	7.11
Junction N17	562.5	-0.50	572.19	4.20
Junction N18	562.5	-0.50	572.20	4.20
Junction N19	562.7	-0.70	572.43	4.22
Junction N20	562.7	-0.70	572.44	4.22
Junction N21	556	0.00	572.40	7.11
Junction N22	556	0.00	571.95	6.91
Junction N23	442	0.00	570.95	55.87
Junction N24	451	0.00	505.58	23.65
Reservoir R2	530	-4.72	530.00	0.00
Reservoir R4	446	7.12	446.00	0.00

Pipes for 2010basecondition-2 pumps.net

	Length ft	Diameter in	Roughness	Flow MGD	Velocity fps	Headloss ft/Kft	Friction Factor
Pipe P2	200	24	76	-7.12	3.51	4.56	0.048
Pipe P3	2700	24	100	-7.12	3.51	2.74	0.029
Pipe P4	100	24	100	-7.12	3.51	2.74	0.029
Pipe P5	2000	24	100	-7.12	3.51	2.74	0.029
Pipe P6	800	24	100	-7.12	3.51	2.74	0.029
Pipe P7	7300	24	100	-7.12	3.51	2.74	0.029
Pipe P8	8100	30	100	-4.72	1.49	0.43	0.031
Pipe P9	100	30	100	-4.72	1.49	0.43	0.031
Pipe P12	8500	24	100	7.12	3.51	2.74	0.029
Pipe P13	2540	21.8	100	4.72	2.82	2.05	0.030
Pipe P15	100	30	100	-2.40	0.76	0.12	0.035
Pipe P1	50	20	100	-2.40	1.71	0.89	0.033
Pipe P10	100	12	100	-0.50	0.99	0.59	0.039
Pipe P11	75	12	100	-0.50	0.99	0.59	0.039
Pipe P17	30	12	100	-0.70	1.38	1.10	0.037
Pipe P18	50	20	100	-1.90	1.35	0.58	0.034
Pipe P20	250	16	100	-1.40	1.55	0.97	0.035
Pipe P21	30	12	100	-0.70	1.38	1.10	0.037
Pipe P22	20	16	100	-0.70	0.78	0.27	0.039
Pipe P24	8100	30	100	2.40	0.76	0.12	0.035
Pipe P25	20	24	100	2.40	1.18	0.37	0.034
Pipe P26	14700	24	77	-7.12	3.51	4.45	0.046
Pipe P19	1000	30	100	2.40	0.76	0.12	0.035
Pump PP1	#N/A	#N/A	#N/A	2.36	0.00	49.69	0.000
Pump PP2	#N/A	#N/A	#N/A	2.36	0.00	49.69	0.000

Nodes for 2010 improved friction condition-2 pumps.net

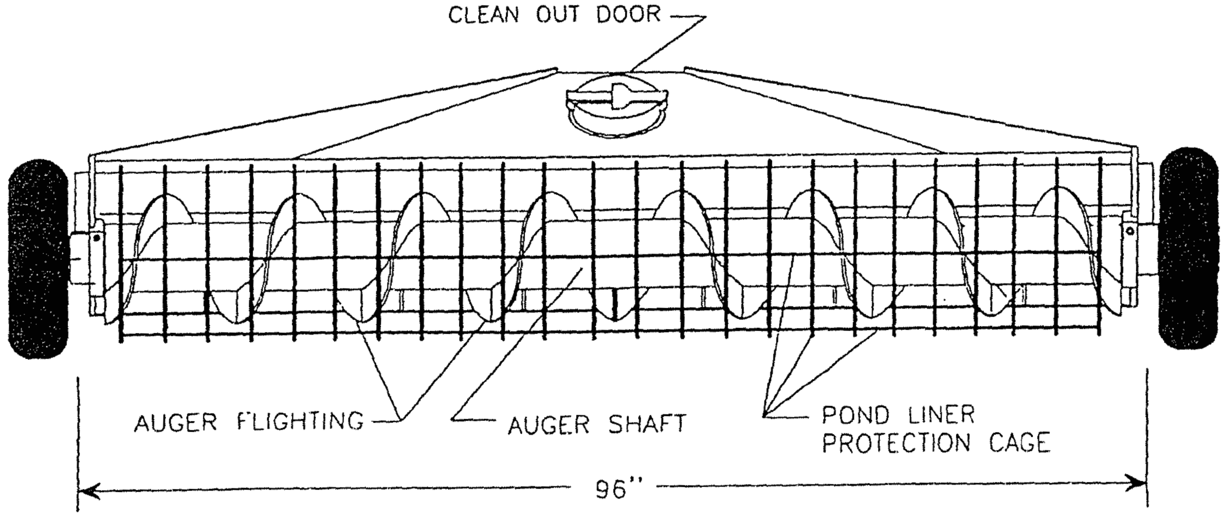
	Elevation ft	Demand MGD	Total Head ft	Pressure psi
Junction N5	424	0.00	472.73	21.11
Junction N6	437	0.00	473.36	15.75
Junction N7	438	0.00	481.85	19.00
Junction N8	438	0.00	482.16	19.13
Junction N9	440	0.00	488.45	20.99
Junction N10	440	0.00	490.96	22.08
Junction N11	442	0.00	560.14	51.19
Junction N12	556	0.00	564.43	3.65
Junction N13	556	0.00	564.48	3.68
Junction N14	549	0.00	570.85	9.47
Junction N15	557.4	0.00	561.27	1.68
Junction N16	556	0.00	561.28	2.29
Junction N1	556	0.00	561.33	2.31
Junction N2	556	0.00	561.36	2.32
Junction N4	556	0.00	561.61	2.43
Junction N17	562.5	-0.50	561.39	-0.48
Junction N18	562.5	-0.50	561.40	-0.48
Junction N19	562.7	-0.70	561.63	-0.46
Junction N20	562.7	-0.70	561.64	-0.46
Junction N21	556	0.00	561.60	2.43
Junction N22	556	0.00	561.15	2.23
Junction N23	442	0.00	560.15	51.19
Junction N24	451	0.00	513.92	27.26
Reservoir R2	530	-5.27	530.00	0.00
Reservoir R4	446	7.67	446.00	0.00

Pipes for 2010 improved friction condition-2 pumps.net

	Length ft	Diameter in	Roughness	Flow MGD	Velocity fps	Headloss ft/Kft	Friction Factor
Pipe P2	200	24	100	-7.67	3.78	3.14	0.028
Pipe P3	2700	24	100	-7.67	3.78	3.14	0.028
Pipe P4	100	24	100	-7.67	3.78	3.14	0.028
Pipe P5	2000	24	100	-7.67	3.78	3.14	0.028
Pipe P6	800	24	100	-7.67	3.78	3.14	0.028
Pipe P7	7300	24	100	-7.67	3.78	3.14	0.028
Pipe P8	8100	30	100	-5.27	1.66	0.53	0.031
Pipe P9	100	30	100	-5.27	1.66	0.53	0.031
Pipe P12	8500	24	100	7.67	3.78	3.14	0.028
Pipe P13	2540	21.8	100	5.27	3.15	2.51	0.030
Pipe P15	100	30	100	-2.40	0.76	0.12	0.035
Pipe P1	50	20	100	-2.40	1.71	0.89	0.033
Pipe P10	100	12	100	-0.50	0.99	0.59	0.039
Pipe P11	75	12	100	-0.50	0.99	0.59	0.039
Pipe P17	30	12	100	-0.70	1.38	1.10	0.037
Pipe P18	50	20	100	-1.90	1.35	0.58	0.034
Pipe P20	250	16	100	-1.40	1.55	0.97	0.035
Pipe P21	30	12	100	-0.70	1.38	1.10	0.037
Pipe P22	20	16	100	-0.70	0.78	0.27	0.038
Pipe P24	8100	30	100	2.40	0.76	0.12	0.035
Pipe P25	20	24	100	2.40	1.18	0.37	0.034
Pipe P26	14700	24	100	-7.67	3.78	3.14	0.028
Pipe P19	1000	30	100	2.40	0.76	0.12	0.035
Pump PP1	#N/A	#N/A	#N/A	2.63	0.00	40.85	0.000
Pump PP2	#N/A	#N/A	#N/A	2.63	0.00	40.85	0.000

Appendix C
Dredge Cutterhead Protection System

ADD AN UNDERWATER ROTOTILLER TO YOUR PUMP



FRONT VIEW

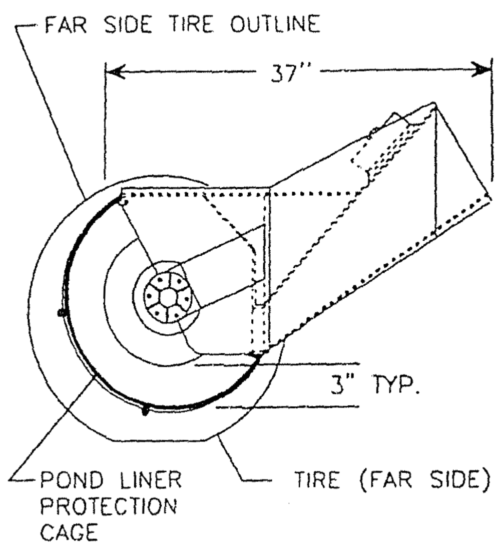
* STANDARD FEATURES

DIGGING TORQUE	10,000 INCH POUNDS
CONSTRUCTION MATERIALS	1/8" & 1/4" THICK A36 MILD CARBON STEEL
ROTATION SPEED	65 RPM (VARIABLE)
FLIGHTING	12" DIAMETER SECTIONS FULL PITCH 3/16" THICK
AUGER SHAFT	6" DIA. A36 STEEL
CLEANOUT DOOR	KWIK LATCH POP LATCH
DRIVE MECHANISM	DUAL HIGH TORQUE HYDRAULIC MOTOR

* OPTIONAL FEATURES

CONSTRUCTION MATERIALS	STAINLESS STEEL AND ABRASION RESISTANT STEEL
DIGGING TORQUE	HIGHER DIGGING TORQUE
ROTATION SPEED	VARIABLE ROTATION SPEEDS
CUTTERHEAD WIDTH	UP TO 12 FOOT WIDE CUTTERHEADS AVAILABLE
PUMP SUCTION	ALTERNATE SUCTION ARRANGEMENTS AND POSITIONS AVAILABLE

* THESE FEATURES MAY CHANGE WITHOUT NOTICE.



SIDE VIEW

NEARSIDE TIRE NOT SHOWN
FOR CLARITY

Hydraulic Power Requirements

The Crisafulli 96" cutterhead requires ten gallons per minute at 2200 pounds *per square inch* of hydraulic fluid. The hydraulic fluid should be filtered to 25 micron and not exceed 180° Fahrenheit inlet temperature. The minimum oil viscosity is 50 SUS.

REV. (2) BY CTT 12/06/94

SRS
Crisafulli Sludge Removal Systen

DREDGE CUTTERHEAD
8 FOOT - MODEL CD9600W
WITH POND LINER PROTECTION SYSTEM

Dwn By: DRT	Ckd.:	Date:3/19/91	Dwg.#: CPC-91198

Appendix D
Dredging Manufacturers and Contractors

APPENDIX D

Dredging Manufacturers and Contractors

Manufacturers

SRS Crisafulli, Inc.
406-365-3393
Contact: Carl K. Richards

Ellicott International
Mud Cat Division
410-545-0261
Contact: Don McCaig

Contractors

AquaDredge
914-273-3179
Contact: Charlie Pound

Commonwealth Disposal
717-545-4235
Contact: Art Davis

Heartland Pump Rental and Sales
618-985-5110
Contact: John Payne

Appendix E

Backup Data for Cost Estimates

06

ESTIMATE

PROJECT : LOUIS LE WATER CO. CRESENT HILL WTP
FACILITY : WASH WATER HOLDING TANK
FILE NAME: J:\CONSTRUC\CMS\LOUISVIL\WASHWATERTANK0.XLS

ESTIMATOR: D JONES
PROJ. MANAGER:
PROJ. NO.: 157683.DR.OU
ESTIMATE NO. : 200031
REV. NO. : 0
DATE: 6/2/2000

MARK-UPS:

	MATL	LABOR	EQUIP.	INSTL or S/C
OVERHEAD =	10.00%	10.00%	10.00%	10.00%
PROFIT =	5.00%	5.00%	5.00%	5.00%
MOB/BOND/INS. =	5.00%	5.00%	5.00%	5.00%
CONTINGENCY =	20.00%	20.00%	20.00%	20.00%

NO.	DESCRIPTION	QTY	UNIT	MATERIALS UNIT $	MATERIALS AMOUNT	LABOR MH	LABOR RATE	LABOR AMOUNT	CONST. EQUIP. UNIT $	CONST. EQUIP. AMOUNT	INSTL or S/C UNIT $	INSTL or S/C AMOUNT	TOTAL	RESOURCE
	WASH WATER HOLDING TANK													
	EXCAVATION AND BACKFILL													
	STRUCTURAL EXCAVATION	10015	CY								$8.00	$80,119	$80,119	CH2M HILL COST DATA
	STRUCTURAL BACKFILL AND COMPACTION	4478	CY								$10.00	$44,778	$44,778	CH2M HILL COST DATA
	8" OF #57 STONE UNDER CONCRETE SOG	154.9	CY	$19.00	$2,943		$5.00	$774					$3,717	CH2M HILL COST DATA
											$115.00	$34,730	$34,730	CH2M HILL COST DATA
	DEWATERING													
	DEWATERING WELL POINTS	302	LF								$220.00	$127,131	$127,131	CH2M HILL COST DATA
	CONCRETE													
	2.5' THICK WET WELL CONCRETE BASE SLAB 79' X 79' X 2.5'	577.87	CY								$403.00	$280,477	$280,477	CH2M HILL COST DATA
	2.5'THICK X 25' HIGH THICK CONCRETE WET WELL WALLS	696.0	CY								$220.00	$7,130	$7,130	CH2M HILL COST DATA
	DRY WELL													
	2.5' THICK X 25' HIGH DRY WELLL CONCRETE BASE SLAB	32.4074	CY											
	1.5' X 25' HIGH THICK CONCRETE WALLS	55.56	CY											
											$40.00	$12,800	$12,800	CH2M HILL COST DATA
											$24.00	$4,800	$4,800	CH2M HILL COST DATA
	METALS													
	ALUMINUM HANDRAIL	320	LF											
	ALUMINUM GRATING INCLUDING SUPPORTSHANDRAIL	200	SF										$18,975	CH2M HILL COST DATA
	MECHANICAL													
	1400 GPM PUMP AT 60' TDH	3	EA	$5,500.00	$16,500		$825.00	$2,475			$16,128.75	$16,129	$16,129	CH2M HILL COST DATA
	PIPING AND VALVES	1	LS								$30,000.00	$30,000	$30,000	CH2M HILL COST DATA
											$14,000.00	$14,000	$14,000	CH2M HILL COST DATA
	ELECTRICAL AND I & C													
	ELECTRICAL	1	LS											
	I&C	1	LS								$8,000.00	$8,000	$8,000	CH2M HILL COST DATA
	FINISHES													
	FINISHES	1	LS											
A	SUBTOTAL				$19,443			$3,249				$660,094	$682,785	
B	OVERHEAD & PROFIT	(A*ohd)+((A+ohd)*p)			$3,014			$504				$102,315	$105,832	
C	MOB / BOND / INSUR.	(% of A)			$972			$162				$33,005	$34,139	
D	CONTINGENCY	(% of A)			$3,889			$650				$132,019	$136,557	
E	TOTAL ESTIMATED CONSTRUCTION COST				$27,317			$4,565				$927,432	$859,300	

CH2MHILL

UNIT PRICE SUMMARY REPORT No. 1 Ver 2.0

PROJECT:	Zone Ave Lagoon & Backwash Improvements Est		ESTIMATOR:	Pete Bredehoeft/DFB
CLIENT:	Louisville Water Company		PROJECT No.:	157683.DR.00
LOCATION:	Louisville, KY		EST DATE:	June 5,2000 (CCI #6,238.10)
DESIGN STAGE:	Planning Level, Order-of-Magnitude		EST. NO:	PRB2K-015
PROJECT MGR:	Jerry Anderson/DAY		REVISION:	Rev 1 07-13-200

BID ITEMS (Unit Price Contract)	QUANTITY	UNIT of MEAS	UNIT PRICE	TOTAL
- Zone Ave Lagoon & Backwash Improvements Est	1.00	LS	$978,445.20	$978,450
+ 01 Zorn Ave. Lagoon	2.00	AC	$241,150.85	$482,300
+ 02 Backwash Piping	1000.00	LNFT	$152.37	$152,370
+ 03 Inlet/Splash Pad	3.00	EACH	$37,034.73	$111,100
+ 04 Pig Launcher	2.00	EACH	$82,380.75	$164,760
+ 05 24" Dia Jack-and-Bore	150.00	LNFT	$452.71	$67,910

PARALLEL 24-INCH CRIVC CROSSINGS 164,760
67,910
$ 232,670

BE PAYNE 42" BACKWASH PIPING IMPROVEMENTS
$152,370
111,100
$ 263,470

CH2MHILL

ESTIMATE MATRIX SUMMARY

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

#	FACILITIES %'s	01000 GENERAL	02000 SITEWORK	03000 CONCRETE	04000 MASONRY	05000 METALS	06000 WOOD	07000 MOISTURE	08000 DOORS	09000 FINISHES	10000 SPECIALS	11000 EQUIP	12000 FURNISH	13000 I &C	14000 CONVEY	15000 MECH	16000 ELECT	TOTAL
01	Zorn Ave. Lagoon		$349,875	$30,706								$34,216		$50,936		$2,017	$14,553	$482,302
02	Backwash Piping		$44,848													$107,523		$152,372
03	Inlet/Splash Pad		$4,424	$28,106								$62,446				$16,128		$111,104
04	Pig Launcher		$7,628	$2,328												$154,805		$164,762
05	24" Dia Jack-and-Bore		$67,906															$67,906
	TOTAL		$474,681	$61,141	$0	$0	$0	$0	$0	$0	$0	$96,661	$0	$50,936	$0	$280,474	$14,553	$978,445
	PERCENT OF TOTAL	0.00%	48.51%	6.25%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	9.88%	0.00%	5.21%	0.00%	28.67%	1.49%	

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon SITEWORK									
023154104420 Cut Notes: *Assumed Simple calculation of Total Area * 5' Depth.*	Unit Costs----> 16,133.33 CY		B10B 24.46	0.022 355	0.54 $8,682	1.48 $23,863		2.02 $32,545	2.94 $47,363
023154104420 Fill Notes: *Assumed Simple calculation of Total Area * 5' Depth.*	Unit Costs----> 16,133.33 CY		B10B 24.46	0.022 355	0.54 $8,682	1.48 $23,863		2.02 $32,515	2.94 $47,363
023153200300 Compaction	Unit Costs----> 16,133.33 CY		B10G 24.46	0.009 145	0.22 $3,552	0.43 $6,868		0.65 $10,420	0.94 $15,164
023104400010 Fine grade and Shape Area Notes: *Assumed total lagoon area.*	Unit Costs----> 9,680.00 SY		B11L 21.70	0.030 290	0.65 $6,303	0.97 $9,365		1.62 $15,668	2.36 $22,802
029205001100 Seeding, hydro or air seeding for lg areas, W/ wood fiber mulch added Notes: *Assumed Total lagoon area.*	Unit Costs----> 9,680.00 SY	0.25 $2,420	B81 24.30	0.006 58	0.15 $1,411	0.16 $1,518		0.55 $5,350	0.80 $7,785
028205300660 Fence, vinyl covered, 9 Ga wire, 6' high	Unit Costs----> 1,700.00 LF	8.50 $14,450	B1 14.10	0.254 431	3.58 $6,083	2.47 $4,206		14.55 $24,740	21.18 $36,003
028205285070 Fence, double swing gates, 6' high, 20' opening Notes: *Nature Center Access*	Unit Costs----> 1.00 OPNG	1200.00 $1,200	B80 20.57	40.000 40	822.72 $823	688.06 $688		2710.78 $2,711	$3,945

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon SITEWORK									
027202150100 Base, prepare & roll sub-base, large areas over 2500 SY Notes: *Assumed 100' Long Gravel Dive x 12' Wide*	Unit Costs----> 6,133.33 SY		B32 25.84	0.009 55	0.23 $1,426	0.44 $2,668		0.67 $4,095	0.97 $5,959
027202000100 Base course, large areas, crushed 3/4" stone, compacted to 6" deep Notes: *Assumed 100' Long Gravel Dive x 12' Wide*	Unit Costs----> 6,133.33 SY	5.95 $36,493	B36B 23.96	0.013 80	0.31 $1,911	0.64 $3,937		6.90 $42,341	10.05 $61,619
027200000010 Dewatering Allowance	Unit Costs----> 1.00 LPSM						50000.00 $50,000	50000.00 $50,000	$72,765
027200000020 Landscape Buffer Allowance	Unit Costs----> 1.00 LPSM						15000.00 $15,000	15000.00 $15,000	$21,830
027200000030 Drainage Structure Excavation/Backfill Allowance	Unit Costs----> 1.00 LPSM						2500.00 $2,500	2500.00 $2,500	$3,638
027000000040 Demo Existing Drainage Outlet Structure	Unit Costs----> 1.00 LPSM						2500.00 $2,500	2500.00 $2,500	$3,638
Subtotal Markups using CH-MK		$54,563 $24,843			$38,873 $17,699	$76,978 $35,048	$70,000 $31,871	$240,415	$109,461
TOTAL 02000 SITEWORK 2.00 AC Division Unit Cost-->		$79,406 $39,703.01		1,810	$56,572 $28,285.94	$112,026 $56,013.23	$101,871 $50,935.50		$349,875 $174,937.67

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
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LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon CONCRETE									
	Unit Costs--->	80.44	CONC03	3.556	65.09	19.34		164.88	239.95
033100201010018 Concrete Slab on Grade, Cast-in-Place, 4,000psi, 18" Thick	14.22 CUYD	$1,144	18.31	51	$926	$275		$2,345	$3,412
	Unit Costs--->	142.22	CONC04	5.555	104.04	28.19		274.48	399.44
033100202030012 Concrete Walls, Straigt, Cast-in-Place, 4,000psi, 12" Thick Notes: *Assumed 10' wide x 10' wdie x 10' High Concrete Drainage Structure, open on top.*	21.33 CUYD	$3,034	18.73	118	$2,219	$601		$5,855	$8,520
	Unit Costs---->						4500.00	4500.00	
033100000025 Concrete Valve Vault Box for Flowmeter, Allowance	1.00 LPSM						$4,500	$4,500	$6,549
	Unit Costs---->						2800.00	2800.00	4074.84
033100000050 Concrete Headwalls for Outlets, 24" Dia Pipe Allowance	3.00 EACH						$8,400	$8,400	$12,225
Subtotal		$4,178			$3,145	$876	$12,900	$21,099	
Markups using CH-MK		$1,902			$1,432	$399	$5,873		$9,606
TOTAL 03000 CONCRETE	2.00 AC	$6,080		169	$4,577	$1,275	$18,773		$30,706
Division Unit Cost-->		*$3,040.17*			*$2,288.34*	*$637.61*	*$9,386.69*		*$15,352.80*

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
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LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon EQUIPMENT									
11000000010 24" X 24" Aluminum Sluice Gate w/Manual Operator Notes: *Assumed 3 Gates are required, Assumed 36" dia outlet.*	Unit Costs----> 3.00 EACH		SKWK 37.09	32.000 96	1187.00 $3,561	250.00 $750	5900.00 $17,700	7337.00 $22,011	10677.53 $32,033
11000000020 Misc Equipment Allowance	Unit Costs----> 1.00 LPSM						1500.00 $1,500	1500.00 $1,500	$2,183
Subtotal					$3,561	$750	$19,200	$23,511	
Markups using CH-MK					$1,621	$341	$8,742		$10,705
TOTAL 11000 EQUIPMENT	2.00 AC			96	$5,182	$1,091	$27,942		$34,216
Division Unit Cost-->					*$2,591.16*	*$545.74*	*$13,970.88*		*$17,107.78*

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon I & C									
13000000010 Flowmeter Allowance, Assumed 24" Dia	Unit Costs----> 1.00 LPSM						24000.00 $24,000	24000.00 $24,000	$34,927
13000000015 Level Transmitter Allowance	Unit Costs----> 1.00 LPSM						3500.00 $3,500	3500.00 $3,500	$5,094
13000000020 Radio Tranmitting Unit Modification Allowance	Unit Costs----> 1.00 LPSM						5000.00 $5,000	5000.00 $5,000	$7,277

CH2MHILL

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PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon I & C	Unit Costs----> 1.00 LPSM						2500.00 $2,500	2500.00 $2,500	$3,638
13000000025 Control Planet Allowance								$35,000	$15,936
Subtotal Markups using CH-MK							$35,000 $15,936		$50,936 $25,467.75
TOTAL 13000 I & C 2.00 AC Division Unit Cost-->							$50,936 $25,467.75		

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon MECHANICAL	Unit Costs----> 50.00 LF	18.20 $910	B14	15.60	0.480 24	7.49 $374	2.03 $102	27.72 $1,386	40.33 $2,017
025307302040 Piping, drainage & sewage. RCP. class 3. no gaskets, 24" dia Notes: *Inlet Piping Assumed 100'*								$1,386	$631
Subtotal Markups using CH-MK		$910 $414				$374 $170	$102 $46		$2,017 $1,008.37
TOTAL 15000 MECHANICAL 2.00 AC Division Unit Cost-->		$1,324 $662.16			24	$545 $272.35	$148 $73.86		

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PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
01 Zorn Ave. Lagoon ELECTRICAL	Unit Costs---->						10000.00 $10,000	10000.00 $10,000	$14,553
16000000010 Electrical Allowance for Flowmeter and Level Transmitter	1.00 LPSM						$10,000 $4,553	$10,000	$4,553
Subtotal Markups using CH-MK							$14,553 $7,276.50		$14,553 $7,276.50
TOTAL 16000 ELECTRICAL	2.00 AC Division Unit Cost-->								

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PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
02 Backwash Piping SITEWORK									
023159001000042 Trench Excavation and Backfill, Cost per LF, 42" Dia	Unit Costs----> 1,000.00 LNFT		B12BA 33.59	0.133 133	4.46 $4,455	14.05 $14,047		18.50 $18,502	26.93 $26,926
020000000010 Excavation & Backfill Allowance	Unit Costs----> 3.00 EACH						1200.00 $3,600	1200.00 $3,600	1746.36 $5,239
026302001210 Catch basins or manholes, concrete, precast, 6' I.D., 8' deep	Unit Costs----> 3.00 EA	1175.00 $3,525	B6 17.09	24.000 72	410.13 $1,230	203.00 $609		1788.13 $5,364	2602.27 $7,807
026302001500 CB or manholes, conc, slab tops, precast, 8" thick, 6' dia	Unit Costs----> 3.00 EA	315.00 $945	B6 17.09	3.429 10	58.60 $176	29.00 $87		402.60 $1,208	585.90 $1,758
026302002200 Catch bsns or manholes, frs and covs, hvy traffic, 36" diam, 1150 lb.	Unit Costs----> 3.00 EA	510.00 $1,530	B6 17.09	8.000 24	136.71 $410	67.67 $203		714.38 $2,143	1039.63 $3,119
Subtotal		$6,000			$6,271	$14,946	$3,600	$30,817	$14,031
Markups using CH-MK		$2,732			$2,855	$6,805	$1,639		
TOTAL 02000 SITEWORK 1,000.00 LNFT	Division Unit Cost-->	$8,732 $8.73		239	$9,126 $9.13	$21,751 $21.75	$5,239 $5.24		$44,848 $44.85

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
02 Backwash Piping MECHANICAL									
025307302070 Piping, drainage & sewage, RCP, class 3, no gaskets, 42" dia	Unit Costs---->	47.50	B13B	0.778	15.16	11.23		73.88	107.52
	1,000.00 LF	$47,500	19.48	778	$15,156	$11,228		$73,884	$107,523
Subtotal		$47,500			$15,156	$11,228		$73,884	
Markups using CH-MK		$21,627			$6,900	$5,112			$33,639
TOTAL 15000 MECHANICAL 1,000.00 LNFT		$69,127		778	$22,056	$16,340			$107,523
Division Unit Cost-->		$69.13			$22.06	$16.34			$107.52

CH2MHILL

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PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
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ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
03 Inlet/Splash Pad SITEWORK									
023703000100 Rip-rap, random, 12" thick	Unit Costs----> 88.89 CY	17.05 $1,516	B12G 33.59	0.258 23	8.67 $770	8.48 $754		34.20 $3,040	49.76 $4,424
Subtotal		$1,516			$770	$754		$3,040	
Markups using CH-MK		$690			$351	$343			$1,384
TOTAL 02000 SITEWORK	3.00 EACH	$2,206		23	$1,121	$1,097			$4,424
Division Unit Cost-->		$735.20			$373.73	$365.59			$1,474.52
03 Inlet/Splash Pad CONCRETE									
033100201010012 Concrete Slab on Grade, Cast-in-Place, 4,000psi, 12" Thick	Unit Costs----> 32.67 CUYD	82.40 $2,692	CONC01 18.88	3.724 122	70.31 $2,297	25.24 $824		177.94 $5,813	258.95 $8,460
033100000010 Concrete Headwall Allowance for Sluice Gate, 42" Dia Pipe	Unit Costs----> 3.00 EACH						4500.00 $13,500	4500.00 $13,500	6548.85 $19,647
Subtotal		$2,692			$2,297	$824	$13,500	$19,313	
Markups using CH-MK		$1,226			$1,046	$375	$6,147		$8,793
TOTAL 03000 CONCRETE	3.00 EACH	$3,917		122	$3,343	$1,200	$19,647		$28,106
Division Unit Cost-->		$1,305.76			$1,114.26	$399.94	$6,548.85		$9,368.81

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CH2MHILL

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LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
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ESTIMATOR: Pete Bredehoeft/DFB
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EST. DATE: June 5,2000 (CCI #6,238.10)
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DESCRIPTION	QTY UNIT	MATERIALS	CREW	RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
03 Inlet/Splash Pad EQUIPMENT Unit Costs---->	3.00 EACH									
11000000015 42" Diia" Aluminum Sluice Gate w/Manual Operator			SKWK	37.09	54.000 162	2003.06 $6,009	300.00 $900	12000.00 $36,000	14303.06 $42,909	20815.24 $62,446
Subtotal						$6,009	$900	$36,000	$42,909	
Markups using CH-MK						$2,736	$410	$16,391		$19,537
TOTAL 11000 EQUIPMENT Division Unit Cost-->	3.00 EACH				162	$8,745 $2,915.05	$1,310 $436.59	$52,391 $17,463.60		$62,446 $20,815.24
03 Inlet/Splash Pad MECHANICAL Unit Costs---->	150.00 LF	47.50 $7,125	B13B	19.48	0.778 117	15.16 $2,273	11.23 $1,684		73.88 $11,083	107.52 $16,128
025307302070 Piping, drainage & sewage, RCP, class 3, no gaskets, 42" dia										
Subtotal		$7,125				$2,273	$1,684		$11,083	
Markups using CH-MK		$3,244				$1,035	$767			$5,046
TOTAL 15000 MECHANICAL Division Unit Cost-->	3.00 EACH	$10,369 $3,456.34			117	$3,308 $1,102.81	$2,451 $817.02			$16,128 $5,376.17

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
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LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
REVISION: Rev 1 07-13-200

DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
04 Pig Launcher SITEWORK									
02000000010 Excavation Allowance At Pig Launchers	Unit Costs----> 1.00 LPSM						3500.00 $3,500	3500.00 $3,500	$5,094
023159001000024 Trench Excavation and Backfill, Cost per LF, 24" Dia	Unit Costs----> 150.00 LNFT		B12BA 33.59	0.083 12	2.79 $419	8.81 $1,322		11.61 $1,741	16.89 $2,534
Subtotal					$419	$1,322	$3,500	$5,241	
Markups using CH-MK					$191	$602	$1,594		$2,386
TOTAL 02000 SITEWORK 2.00 EACH				12	$610	$1,924	$5,094		$7,628
Division Unit Cost-->					$305.07	$962.03	$2,546.78		$3,813.87
04 Pig Launcher CONCRETE									
03300000010 Concrete Pad Allowance for Pig Launcher	Unit Costs----> 2.00 EACH						800.00 $1,600	800.00 $1,600	1164.24 $2,328
Subtotal							$1,600	$1,600	
Markups using CH-MK							$728		$728
TOTAL 03000 CONCRETE 2.00 EACH							$2,328		$2,328
Division Unit Cost-->							$1,164.24		$1,164.24

Report Date: 07/13/2000 15:26:26

CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
04 Pig Launcher MECHANICAL									
15000000010 Cut-in Allowance for new Wye's	Unit Costs----> 2.00 EACH	50.00 $100	SKWK 37.09	15.000 30	556.41 $1,113	200.00 $400		806.41 $1,613	1173.56 $2,347
150600110105024 Cement Lined Ductile Iron Pipe, Push-On Joint, Pressure Class 350, 24" dia	Unit Costs----> 150.00 LNFT	38.51 $5,777	PIPE02 32.88	0.150 23	4.93 $740	0.80 $120		44.24 $6,636	64.39 $9,658
150600120203024 Ductile Iron Pipe, Restrained Joint Adder per LNFT, Fast-Grip Gasket, 24" dia	Unit Costs----> 150.00 LNFT	23.25 $3,488	PIPE02 32.88	0.280 42	9.21 $1,381	1.49 $224		33.95 $5,092	49.41 $7,411
150600130102024 CLDI Filling, Push-On Joint, C110-SR, Wye, 24" Dia	Unit Costs----> 4.00 EACH	2400.00 $9,600	PIPE02 32.88	11.250 45	369.88 $1,480	60.02 $240		2829.90 $11,320	4118.36 $16,473
150600130102024 CLDI Filling, Push-On Joint, C110-SR, 45 Deg Elbow, 24" Dia	Unit Costs----> 6.00 EACH	2400.00 $14,400	PIPE02 32.88	11.250 68	369.88 $2,219	60.02 $360		2829.90 $16,979	4118.36 $24,710
151000210101024 Valves, Historical Costs, Gate Valve V100, Flanged, 24" Dia	Unit Costs----> 4.00 EACH	11150.00 $44,600	PIPE06 33.85	26.980 108	913.17 $3,653	143.98 $576		12207.15 $48,829	17765.07 $71,060
150600110301284 Cement Lined Ductile Iron Pipe, Class 53, Flg x Flg,24" dia, 4' Spool Piece	Unit Costs----> 4.00 EACH	818.83 $3,275	PIPE06 33.85	13.631 55	461.36 $1,845	72.74 $291		1352.93 $5,412	1968.92 $7,876
0150441502011 24" SCH .5 150# RFWN FLANGE A-181 FS SHOP BW	Unit Costs----> 4.00 EA						791.59 $3,166	791.59 $3,166	1152.00 $4,608

CH2MHILL

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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
04 Pig Launcher MECHANICAL									
0150440801044 30" X 24" SCH .5 CONCENTRIC REDUCERS A-234 FS SHOP BW	Unit Costs---->** 4.00 EA						1275.18 $5,101	1275.18 $5,101	1855.77 $7,423
0150440301032 30" SCH .5 A-53 CS PIPE SPOOL, INSIDE BATTERY LMTS OF A PROCESS AREA	Unit Costs---->** 4.00 LF	118.75 $475	MECH1 32.00	1.558 6	49.84 $199			168.59 $674	245.34 $981
0150441202006 3" X 12" LONG SCH 80 SHAPED A-53 CS SHOP NIPPLE REINF 90_ TO HEADER	Unit Costs---->** 2.00 EA						154.30 $309	154.30 $309	224.55 $449
151101601520 Valves, bronze, ball, 150 psi, threaded, 3" diameter	Unit Costs---->** 2.00 EA	210.00 $420	PLUM 33.18	1.000 2	33.18 $66			243.18 $486	353.90 $708
0150441202006 3" X 4" LONG SCH 80 SHAPED A-53 CS SHOP NIPPLE REINF 90_ TO HEADER	Unit Costs---->** 2.00 EA						85.00 $170	85.00 $170	123.70 $247
150000000200 3" Dia Quick Connect Coupling and Cap Allowance	Unit Costs---->** 2 00 EACH	85.00 $170	SKWK 37.09	1.000 2	37.09 $74	2.50 $5		124.59 $249	181.32 $363
151101601450 Valves, bronze, ball, 150 psi, threaded, 1/2" size	Unit Costs---->** 2.00 EA	6.40 $13	PLUM 33.18	0.364 1	12.08 $24			18.48 $37	26.89 $54

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CH2MHILL

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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
04 Pig Launcher MECHANICAL									
Unit Costs--->	2.00 EACH						150.00 $300	150.00 $300	218.30 $437
151000000450 Pressure Guage Allowance		$82,317 $37,479			$12,794 $5,825	$2,216 $1,009	$9,046 $4,118	$106,373	$48,432
Subtotal Markups using CH-MK		$119,796 $59,898.05		380	$18,620 $9,309.84	$3,225 $1,612.66	$13,164 $6,582.09		$154,805 $77,402.64
TOTAL 15000 MECHANICAL	2.00 EACH								
Division Unit Cost-->									

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CH2MHILL

CH2MHILL ESTIMATE DETAIL REPORT No.1 Ver 2.0

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
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LOCATION: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
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EST. DATE: June 5,2000 (CCI #6,238.10)
EST. No.: PRB2K-015
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DESCRIPTION	QTY UNIT	MATERIALS	CREW RATE	MH	LABOR	EQUIPMENT	INSTL S/C	TOTAL DIRECT	TOTAL W/MRKUPS
05 24" Dia Jack-and-Bore SITEWORK									
02000000010 Pit Excavation Allowance	Unit Costs----> 2.00 EACH						2500.00 $5,000	2500.00 $5,000	3638.25 $7,277
02000000020 Dewatering Allowance	Unit Costs----> 1.00 LPSM						3500.00 $3,500	3500.00 $3,500	$5,094
024453000200 Horizontal boring, roadwork, 1/2" thick wall, 36" diameter casing, 24" Pipe	Unit Costs----> 150.00 LF	75.00 $11,250	B42 22.02	4.000 600	88.08 $13,213	71.33 $10,699		234.41 $35,161	341.13 $51,170
024453001100 Horizontal boring, prepare jacking pits, incl mobil & demob, min	Unit Costs----> 1.00 EA						3000.00 $3,000	3000.00 $3,000	$4,366
Subtotal		$11,250			$13,213	$10,699	$11,500	$46,661	
Markups using CH-MK		$5,122			$6,016	$4,871	$5,236		$21,245
TOTAL 02000 SITEWORK		$16,372		600	$19,228	$15,570	$16,736		$67,906
150.00 LNFT Division Unit Cost-->		$109.15			$128.19	$103.80	$111.57		$452.71

CH2MHILL

CH2MHill MARKUPS REPORT

PROJECT: Zone Ave Lagoon & Backwash Improvements Est
CLIENT: Louisville Water Company
PROJECT NO: Louisville, KY
DESIGN STAGE: Planning Level, Order-of-Magnitude
PROJECT MGR: Jerry Anderson/DAY

ESTIMATOR: Pete Bredehoeft/DFB
PROJECT No.: 157683.DR.00
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MARKUPS SETS USED

MARKUP RESOURCE	DESCRIPTION	MARKUP COMPONENT ITEM	PERCENT	TO-MAT'L	TO-LABOR	TO-EQUIP	TO-INSTALL S/C
		Success PWS Branch assigned to: 01 Zorn Ave. Lagoon					
CH-MK	CH2M HILL Standard Markup Set						
		1. Overhead	10.00%	Yes	Yes	Yes	Yes
		2. Profit	5.00%	Yes	Yes	Yes	Yes
		3. Mob/Bond/Insurance	5.00%	Yes	Yes	Yes	Yes
		4. Contingency	20.00%	Yes	Yes	Yes	Yes

Report Supplement:

Drainage and Solids Management Improvements

Crescent Hill Water Treatment Plant

Prepared for

LOUISVILLE WATER COMPANY

April 20, 2001

CH2MHILL
and
Quest

Introduction

In July 2000 a report entitled *Drainage and Solids Management Improvements, Crescent Hill Water Treatment Plant* was prepared for the Louisville Water Company (LWC) by CH2M HILL in association with Quest Engineers. In December 2000, CH2M HILL was authorized by the LWC to prepare this supplement to the report, which addresses the following topics:

- NPDES Discharge Permit Investigation
- Booster Pump Station on North Side of Crescent Hill WTP
- Stormwater Management on South Sidė Crescent Hill WTP
- Measure BE Payne Lagoon Solids

NPDES Discharge Permit Investigation

The Kentucky Division of Water (KDOW) Kentucky Pollutant Discharge Elimination System (KPDES) Branch was contacted to investigate alternatives to the current discharge permits at Zorn Avenue and BE Payne. Topics that were discussed included:

- Use and scope of general KPDES permits for water treatment plants
- Exceptions to the general permits
- Less stringent discharge limitations at Zorn Avenue for intermittent and/or unusual discharge activities, such as pipeline maintenance and reservoir draining
- Seasonal variances to allow less stringent discharge limitations during high river discharge conditions

Mr. Ronnie Thompson of the industrial section of the KPDES Branch was contacted and a telephone record is included in Appendix A. To Mr. Thompson's knowledge, all but one water treatment plant in Kentucky uses the General KPDES Permit for Wastewater Discharges Associated with Drinking Water Plant Activities. The permits are written and issued on 5-year cycles and will be reissued January 2004.

Only the City of Paducah has a special permit that allows unlimited discharge of total suspended solids (TSS). The basis for this exception was a study conducted for the city that indicated no detrimental effects to the river due to the solids. Similar studies could be conducted by other permittees for KDOW's consideration. A copy of the permit is included in Appendix B.

Special discharges from water treatment plants and other potable water utility activities can be allowed independently of KPDES permitting. Referred to as "special discharge authorizations", typical examples are for construction work, pressure testing, and reservoir draining and are requested on a case-by-case basis. These types of discharges do not have limits or sampling requirements. KDOW would expect these types of requests to be submitted infrequently, probably less than once per year per site.

Mr. Mike Mudd at the Louisville Regional DOW office was contacted and asked about these special discharge authorizations. He indicated that LWC has previously requested permission to discharge chlorinated water associated with water quality flushing and hyperchlorinated water associated with new water main construction. However, he is not familiar with the special discharge authorizations and suggested that Mr. Sam Lester, DOW Field Operations Manager, should handle this type of request.

Mr. Lester was contacted and indicated that he has issued special discharge authorizations. He is willing to cooperatively work with utilities that have infrequent operation and maintenance challenges that involve special discharges. One of the first uses of a special discharge authorization beneficial to LWC would be to allow discharges associated with cleaning the 24-inch drain line to the BE Payne lagoons. Mr. Lester would consider a proposal to temporarily divert CHWTP wastewater to a direct Ohio River discharge if he is given a specific proposal for the overall project which estimates discharge rates, volumes, and water quality. He would be in favor of constructing temporary lagoons to capture wastewater generated during a cleaning operation, such as line pigging or jet cleaning, and would offer a temporary permit for discharging the supernatant from these temporary lagoons. He suggests that the water utility request an initial meeting to describe the proposed project and discuss with DOW the options for managing the wastewater. After this, a more specific proposal requesting the discharge authorization could be submitted to DOW.

The lagoon on the Zorn Avenue PS site previously recommended in the original report may not be required if DOW would allow diversion of CHWTP wastewater to the river during the pipeline cleaning operation. Even if permission for this discharge is obtained, however, one or more other temporary lagoons to capture wastewater from pipeline cleaning may be required. If so, a lagoon at the Zorn Avenue site for temporary use might be the best location, anyway, due to availability of property.

Booster Pump Station on North Side of Crescent Hill WTP

The feasibility of a booster pump station and wet well at Reservoir Avenue to increase the carrying capacity of process wastewater was investigated. This pump station would be used in conjunction with high wastewater flowrate events, such as draining a reservoir or coagulation basin. The pump station would also help clean the pipeline to BE Payne lagoons by imparting high flushing velocities.

Several hydraulic scenarios of the CHWTP drainage system to the BEPWTP lagoons were modeled in the original report. Table 1 (the same as Table 3-3 from the original report) summarized the results of three scenarios all using the 2010 projected drainage rate but with different backwash holding basin pumping rates and friction factors:

1. One backwash holding tank pump operating at 3.47 mgd, total flow rate at 5.87 mgd, and existing friction factors
2. Two backwash holding tank pumps operating at 4.72 mgd, total flow rate at 7.12 mgd, and existing friction factors
3. Two backwash holding tank pumps operating at 5.27 mgd, total flow rate at 7.67 mgd, and improved friction factors

A successful outcome to computer modeling was determined by the following criteria:

- Backwash holding tank pumping rates had to be at least 5 mgd to include an allowance for basin drainage
- The weir boxes at the coagulation basins could not overflow
- If the Node 15 manhole in Reservoir Avenue overflowed, it must be closed off or raised

The first two scenarios had insufficient pumping out of the backwash holding tank. Scenario 2 would have had an overflow at the coagulation basin weir boxes. Scenarios 2 and 3 would have had an overflow at Junction 15 (on Reservoir Avenue) and would have required some type of modification to the manhole in order to function properly. The only difference between Scenario 2 and 3 was that improved friction factors were assumed due to the anticipated beneficial effect of cleaning the pipeline. The improved friction factor would prevent overflow of the coagulation basin weir boxes; however, the ability to increase friction factor to this degree cannot be proven in advance. An inline pump station on the drainage system would increase the carrying capacity of the 24-inch pipeline to BEPWTP lagoons whether or not the friction factor was increased.

Scenario 2 was modified to insert a pump station at the Zorn Avenue Pump Station site to handle total flows draining to BEPWTP. This site is recommended over a site on Reservoir Avenue for the following reasons:

- The site is farther away from the public and less obtrusive
- A much higher suction pressure would be available for an inline pumping configuration a wet well would not be required
- The pump station could also be used to pump out a new lagoon at the Zorn Avenue site and divert to the BEPWTP lagoons via the 24-inch pipeline

Modeling results are shown in Appendix C, which includes a model schematic and the EPANET analysis results with junction and pipe reports. The proposed pump station is labeled "2" in between Junctions N11 and 2. The suction pressure (at Junction N11) is 45 psi when pumping 7.9 mgd at 50 feet of head. The discharge pressure (at Junction 2) is 67 psi. The pressure at the proposed pump station site is 56 psi during the original Scenario 2, which has a flow rate of 7.12 mgd without pumping. The model predicts an increase in pressure from 56 to 67 psi while pumping.

Preliminary sizing criteria for the pump station and wet well were developed. Ideally, a target flow rate of 2.4 mgd from the eight coagulation basins will occur at all times that all basins are in operation. During the peak filter backwashing season, 4 mgd would be pumped out of the backwash holding basin. In addition, softening basin drainage would flow at a rate of 1 mgd. Total wastewater flow rate would be 7.4 mgd. An approximate firm capacity of 8 mgd was selected for a new drainage pump station. This capacity assumes no modifications are made to the stormwater drainage system at CHWTP.

Hydraulic M[illegible]ing Summaries from Original Report

Results	Scenario 1	Scenario 2	Scenario 3
EPANET Filename	2010 basecondition-1 pump.net	2010 basecondition-2 pumps.net	2010 improved friction-2 pumps.net
No. of BW holding tank pumps	1	2	2
Pumped flow rate, mgd	3.47	4.72	5.27
Flow rate to lagoons, mgd [a]	5.87	7.12	7.67
HGL @ Node 11 (Zorn Ave. at River Rd.), ft	533.5	570.9	560.1
HGL @ Node 15 (manhole on Reservoir Ave.), ft [b]	534.6	572.1	561.3
HGL @ Nodes 17 & 18 (N. Coag Basins' weir box), ft [c]	534.8	572.2	561.4
HGL @ Nodes 19 & 20 (S. Coag Basins' weir box), ft [d]	535.0	572.4	561.6
Conclusions			
Pumped flow rate sufficient for design year? [e]	No	No	Yes
Overflow at Node 15 manhole?	No	Yes	Yes
Overflow at coag. basin weir boxes?	No	Yes	No

Notes:
[a] Assumes sludge flows from N. & S. Coag. Basins are 0.25 mgd and 0.35 mgd per basin, respectively
[b] Grade at manhole is 557.4 feet
[c] Weir box elev is 562.5 feet
[d] Weir box elev is 562.7 feet
[e] Assumes that 5 mgd is required based on total sludge flow rate being set to 1.0 mgd for partial softening

Preliminary sizing indicates that three 3,000-gpm pumps at a total dynamic head of 50 feet with 60-hp motors could provide this capacity. A construction cost estimate for a pump station with wet and dry wells, CMU/brick veneer walls, pumps, piping, valves, electrical, and instrumentation would be approximately $1 million. Appendix D contains the construction cost estimate. Including engineering and a contingency would increase the estimated cost to $1.5 million for budgeting purposes.

The original report recommends inspection and cleaning the 24-inch pipeline to increase its carrying capacity. If the pipeline can be successfully cleaned to increase its capacity to the extent shown in Scenario 3 of the original report, the pump station is not required. If the pipeline capacity cannot be increased beyond its present capacity by cleaning, the pump station will be required. The pump station would not need to operate continuously. During off-peak drainage and filter backwashing, the drainage by gravity flow would be adequate and the pump station would not need to operate.

Stormwater Management on South Side Crescent Hill WTP

The Crescent Hill Water Treatment Plant (CHWTP) site is divided in two by Frankfort Avenue. The north and south parts have separate stormwater drainage systems. The south part drains into Louisville/Jefferson MSD's Beargrass Creek combined sewer system. Stormwater on a small portion of the south side site is collected into the filter backwash holding basin. In normal operation all of this water is pumped from the basin to the north side of CHWTP into a gravity drainage system that drains to the BE Payne WTP lagoons.

The purpose of this analysis is to evaluate the impact of collecting more of the stormwater runoff from the south part of the site to divert from the MSD combined sewer system into the backwash holding basin. Various scenarios of stormwater intensity and diversion were analyzed. The increased flow due to stormwater would require a larger backwash holding basin and pump station, so the capacity of these facilities was investigated, too.

Approach

This analysis evaluates stormwater runoff on the south side of Crescent Hill WTP, bounded by Frankfort Avenue to the south and Grinstead Drive to the east. The south side area is approximately 34.4 acres. This site is subdivided into four smaller areas for analysis:

Area A	Site area where stormwater is currently collected and diverted to the backwash holding basin
Area B	Site area that drains to a 15-inch stormwater pipeline that runs by the backwash holding basin
Area C	Site area on west side that drains directly to MSD combined sewer in Grinstead Drive
Area D	Site area that drains over land to catch basins off site

See the attached map in Appendix E for a delineation of drainage areas. Current MSD design criteria for new site developments within the MSD service area require stormwater detention facilities to restrict flow rates. Within MSD's combined sewer area, developers are required to keep the 100-year post-development runoff rate equal to or less than the 10-year pre-development runoff rate. Because these stormwater conditions are often used for stormwater management, these flow rates were developed for the south side of CHWTP.

Approximately 9 acres of the site have open basins that capture rainwater and eliminate stormwater flows from the post development condition.

Peak discharges were calculated and compared for three scenarios:

1. Undeveloped conditions (pre-LWC facilities),
2. Existing conditions (with current LWC facilities)
3. Diverting 15" storm line into backwash holding basin

Two tributary areas were analyzed: (1) at backwash holding basin and (2) at LWC Grinstead Drive entrance. Storm runoff from the remainder of the site that sheet flows off the property along other perimeter areas (Area D) was also calculated and combined with the Grinstead Drive tributary area, to illustrate the overall impact of stormwater runoff from the LWC site. The 10- and 100-year storm events were analyzed to compare the 10-year pre-developed runoff with the 100-year post-developed (existing conditions) runoff.

Design Criteria

Hydrologic and hydraulic methodology utilized is in accordance with Chapter 10 "*Stormwater Facilities Design*" of MSD's Design Manual.

Since the site is less than 50 acres the Rational Method was used for calculating peak runoffs for the 10- and 100-year storm events:

$$Q=CIA,$$

Where:

> Q=peak runoff (cfs),
> C=runoff coefficient,
> I=rainfall intensity (inch/hr),
> A=tributary area (acres),

Since this method calculates peak discharge only, it uses the basic assumption that the design storm has constant rainfall intensity for the time period (storm duration), which is equal to the time of concentration (T_c). For this site, a minimum T_c of 10 minutes was utilized for deriving rainfall intensities. This equates to I_{10}=5.4 in/hr and I_{100}=7.3 in/hr. A "weighted" runoff coefficient, C, was utilized for each drainage area using 0.95 for impervious areas (rooftops and pavement) and 0.35 for grassy areas.

Results

The results are summarized in Table 2. For a 10-year storm event the backwash holding basin would currently receive a peak stormwater flow of 7.6 cfs, or 3,400 gpm. It would be simple to divert Area B to the backwash holding basin by relocating the 15-inch stormwater pipe to discharge into the backwash holding basin rather than the MSD combined sewer in Grinstead Avenue. However, this would increase peak stormwater from 3,400 to 11,700 gpm, or 16.8 mgd, to collect Areas A+B. The current pump station capacity is about 4 mgd, or 2,800 gpm. Previous modeling scenarios show that 4 mgd is the approximate flow rate limit for this pump station when discharging to the gravity system simultaneously with the coagulation basin sludge flows. To increase the pumping rate from the backwash holding basin significantly more than 4 mgd would require a much greater capacity backwash basin pump station, a new force main to the north side of CHWTP in parallel with the existing 12- and 20-inch force mains, and improvements to the 24-inch gravity pipeline to BEPWTP, such as an inline booster pump station. Even with a new booster pump station, handling this flow rate combined with coagulation basin sludge flows would be questionable. Currently, peak flows to the BEPWTP lagoons are approximately 8 mgd. Diverting Areas A+B would capture about 34 percent of the total south side area of CHWTP, which is 25.3 acres (excluding open basins).

TABLE 2
Potential Stormwater Flow Rates

Stormwater Collected at Backwash Holding Basin

Scenario	Area (acres)	Weighted C	Q_{10yr} (cfs)	Q_{10yr} (gpm)
Existing Conditions (A)	1.5	0.94	7.6	3,400
Diversion Improvements (A+B)	8.6	0.56	26.0	11,700

Stormwater Discharge into MSD Combined Sewer System near Grinstead Drive Entrance

Scenario	Area (acres)	Weighted C	Q, 10 year		Q,100 year	
			cfs	gpm	cfs	gpm
Undeveloped Conditions	24.1	0.35	45.6	20,400	61.6	27,600
Existing Conditions (B+C)	16.0	0.56	48.4	21,700	65.4	29,400
Diversion Improvements (B)	8.9	0.54	26.0	11,700	35.1	15,800

Stormwater Runoff from Remainder of Site

Scenario	Area (acres)	Weighted C	Q, 10 year		Q, 100 year	
			cfs	gpm	cfs	gpm
Undeveloped Conditions	10.3	0.35	19.5	8,740	26.3	11,800
Existing Conditions (D)	9.6	0.35	18.1	8,120	24.5	11,000

Stormwater Runoff for Entire Site (Grinstead Drive Entrance Area + Remainder of Site, excluding basins)

Scenario	Area (acres)	Weighted C	Q, 10 year			Q, 100 year		
			cfs	gpm	% Change	cfs	gpm	% Change
Undeveloped Conditions	34.4	0.35	65.1	29,200	-	88.0	39,500	-
Existing Conditions (B+C+D)	25.6	0.48	66.5	29,800	+2.1%	89.9	40,300	+2.1%
Diversion Improvements (C+D)	18.5	0.44	44.1	19,800	-32.3 %	59.6	26,700	-32.3%

Note: In the scenario column, the letter in brackets ( ) represents the drainage area delineated on the attached map.

Collecting Area C stormwater, in addition to Areas A+B, would total 15.7 acres (62 percent of the south side area excluding open basins), and would further divert stormwater from MSD sewers on the south side. This would result in a peak storm flow rate of 25,100 gpm, or 36 mgd, going into the backwash holding basin. For this level of diversion, a new 36-inch force main would be required to divert the 10-year storm event to the north side of CHWTP.

However, the 24-inch pipeline to BEPWTP lagoons would be undersized for this flow rate, even if an inline booster pump station was built. Collecting Area C would also require a new pump station located at the west side of the property, near the stormwater discharge to sewers in Grinstead Drive, to intercept the stormwater.

Collecting the final 9.6 acres of stormwater in Area D would require significant site work to collect the water and divert to a common point.

Conclusion

Stormwater from only 6 percent of the south area of CHWTP (excluding basins) is currently diverted to the backwash holding basin for pumping to the north side gravity drain system. Area B could easily be diverted to the backwash holding basin, too, since the 15-inch stormwater pipeline runs adjacent to the basin. However, peak stormwater flow rates from Areas A+B combined would significantly increase the required pumping rate out of the basin. A new inline booster pump station would also be required. Even if these modifications were made, only 34 percent of the stormwater on the south side would be captured. Capturing stormwater flows for more than Areas A+B would require a parallel pipeline to BEPWTP lagoons, if stormwater were diverted to the lagoons.

A preferred approach to managing stormwater on the south side of CHWTP might be to construct a detention pond to capture stormwater for Areas A+B+C. The pond would need to have adequate volume to receive the 100-year storm flow and release it at the 10-year, pre-developed flow rate. Probably the most suitable location for a pond would be adjacent to the southwest corner of the site at Stiltz Avenue and Hermann Court. Further analysis would be required to determine volume, elevation, and pumping requirements.

Measure BE Payne Lagoon Solids

In the July report, solids in the four BE Payne WTP lagoons were estimated by visual observation. For this report supplement, the surveying firm Mindell Scott & Associates, Inc made a more accurate measurement of the solids by surveying one profile and two cross sections in each lagoon. Sludge samples were obtained from each lagoon and taken to the CHWTP laboratory for solids content analysis.

Lagoons 1 and 4 had dense brush and small trees that had to be cleared for the survey. Flynn Brothers Contracting, Inc. cleared the brush to provide line-of-sight for the profiles and cross sections. Because Lagoons 2 and 3 had standing water, the survey crew used a boat to probe and measure bottom elevations over much of each lagoon.

The results of the survey and solids volume computations are summarized in Table 3, which compares the initial estimates with the surveyed measurements. Overall, the life remaining changed from the estimated 7.7 years to 12.5 years, based on a 90-percent maximum filling that was assumed in both instances. Although the lagoons' effluent structure and berms are designed to have a liquid level perhaps two feet higher than the inlet pipe invert, we do not recommend operating the lagoons at this high of a level. Additionally, for a given maximum liquid level the solids would not be capable of accumulating over the entire depth. As the maximum solids level is reached, the lagoon will become less effective in capturing solids and they will be carried out in the effluent supernatant. As a result, the 90-percent fullness was defined as 90-percent of the height from the lagoon bottom to the invert of the inlet pipe, and that is the assumed maximum solids content.

TABLE 3
Lagoon Volume Available

Lagoon No.	Total Lagoon Volume, cf	Estimated Percent Filled	Surveyed Percent Filled	Estimated Available Capacity, %	Surveyed Available Capacity, %	Solids Content, %	Estimated Life Remaining, Yr
1	10,497,000	62.9%	52.4%	27.1%	37.6%	63.0%	2.6
2	9,476,000	77.0%	45.8%	13.0%	44.2%	35.5%	1.5
3	10,610,000	67.9%	72.1%	22.1%	17.9%	37.7%	0.7
4	23,753,000	54.3%	41.1%	35.7%	48.9%	63.0%	7.6
Total/Avg.	54,336,000	62.6%	50.2%	27.4%	39.8%	51.5%	12.5

Note:
Projected wastewater flow residuals, both WTPs with pH adjustment at CHWTP, dry lbs/yr: 60,035,000

The maximum percent fullness could be tested by monitoring suspended solids in the active lagoon's effluent. The KPDES permit requires one grab sample a month for monitoring total suspended solids, but this infrequent monitoring would not necessarily indicate a trend in declining lagoon performance, due to the temporary effects of weather and other factors on the lagoon's performance.

Lagoon 2 was thought to have the least amount of useful life based on the estimates, but now, Lagoon 3 has the least amount of useful life based on surveyed results. The fullness of Lagoon 2 was initially overestimated due to a surface layer of uncompacted silt making the lagoon appear fuller.

In the original report, an overall solids content was assumed for all lagoons to be 50 percent based on previous studies. During the surveying, three samples from each lagoon were taken for solids content analysis. Readings in each lagoon were fairly consistent except for Lagoon 2, in which two of the samples were obtained with difficulty from under water and were given a lower weighting than the third sample which was taken away from standing water.

The results are reported in Table 3. Lagoons 1 and 4, which are inactive, both had 63 percent average solids content, whereas Lagoons 2 and 3, which are both in use with standing water inside, have 35.5 and 37.7 percent average solids, respectively. The samples taken with standing water nearby were shallow so the results may not be as reliable as those that were taken from the inactive lagoons. Textbook values given for water treatment sludge dryness indicate ranges of up to 45 percent solids for alum coagulants to more than 60 percent solids for lime (calcium carbonate), so the measured values in the BEPWTP lagoons are very good for the inactive lagoons. Regardless what the actual percent solids content might be in Lagoons 2 and 3, once they are removed from service the solids content would be expected to increase to a similar level as the other lagoons after a dormant period of several months to a year for drying.

The solids content analyses, volume computations, sample locations, and hand sketches showing channelization of flows through Lagoons 2 and 3 are included in Appendix F.

Figure 1 shows a schematic of the four lagoons and all piping that enters or exits from each. As stated in the original report, a comprehensive, long-term operation and maintenance plan needs to be developed for the lagoons, scheduling the following recommendations:

1. The schematic shows that the BEPWTP filter backwash wastewater and coagulation and softening sludge can only drain into Lagoon 3 and drainage from general water use at the raw water pump station can only discharge to Lagoon 2. Lagoon 3 has very little service life remaining, so piping modifications to divert backwash water to one or more other lagoons is needed as soon as possible.
2. The original report recommended cleaning Lagoon 2 first. Lagoon 3 was found by surveying to be the fullest and is presently the only lagoon that can receive BEPWTP backwash wastewater. As a result, a new backwash pipe diversion to another lagoon should be installed as soon as possible so that Lagoon 3 can be removed from service and allowed to dry for cleaning.
3. A geotechnical investigation of Lagoons 1 and 4 is needed to determine if leakage is occurring at suspected sites. If leakage is confirmed these lagoon berms need to be repaired as soon as possible while Lagoons 2 and 3 are still usable.
4. Once Lagoon 3 has sufficiently dried, it should be cleaned. Based on the degree of solids content in Lagoons 1 and 4, it appears that all lagoons can be cleaned using conventional mechanical earthmoving equipment if given an adequate time of dormancy for drying.
5. After Lagoon 3 is ready to be placed back into service, Lagoon 2 can be removed from service for drying and cleaning.
6. The active lagoons that are filling with solids should be monitored for TSS more frequently than required by the KPDES to help indicate settling performance and determine more precisely when the lagoons should be shut down for cleaning.

The comprehensive plan should schedule lagoon cleanings in such a way that ample drying time (up to 1 year) occurs, contractors who bid the project have ample time to complete the work so that bids will be more competitive, and the annual costs for cleaning are as uniform as possible to result in an even cash flow.

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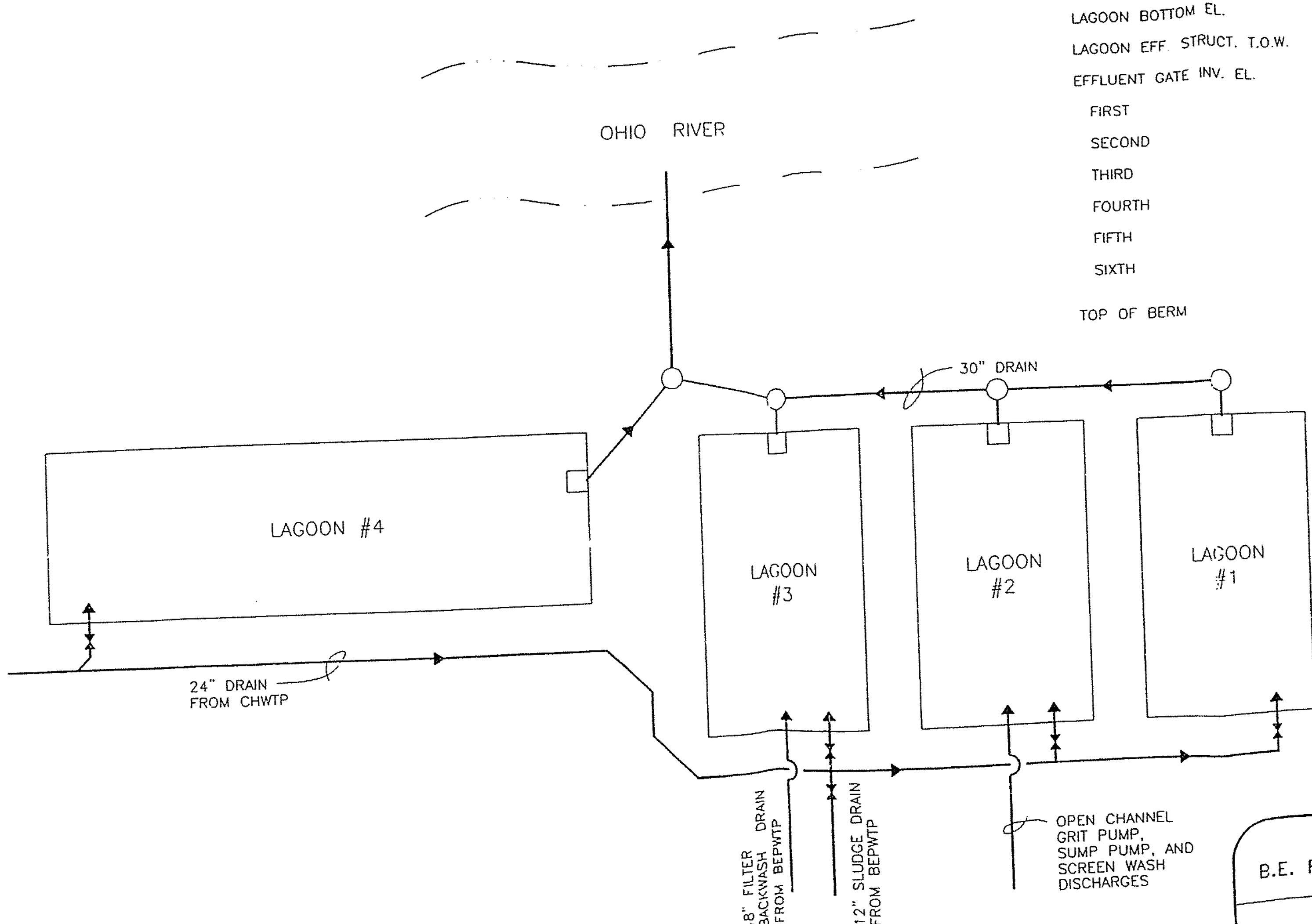

NOTES:

	LAGOONS #1 – 3	LAGOON #4
INLET INV. EL.	444.0	445.0
LAGOON BOTTOM EL.	430.0	423.5
LAGOON EFF. STRUCT. T.O.W.	450.5	450.5
EFFLUENT GATE INV. EL.		
FIRST	431.0	424.5
SECOND	437.0	429.0
THIRD	440.0	433.5
FOURTH	443.0	438.0
FIFTH	446.0	442.5
SIXTH		446.0
TOP OF BERM	450.0	450.0

FIGURE 1
B.E. PAYNE LAGOON SCHEMATIC

CH2M HILL/Quest

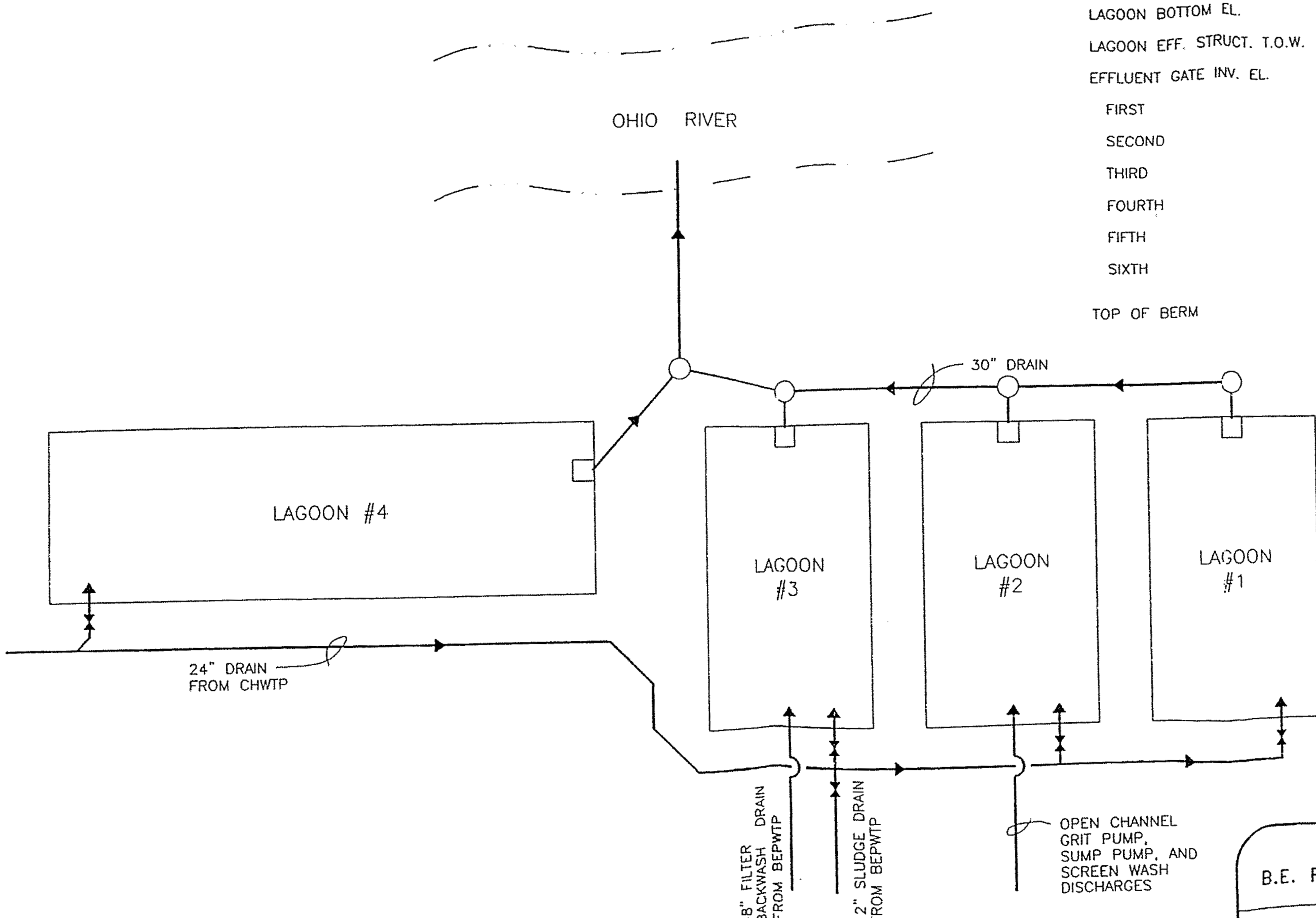

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FIGURE 1
B.E. PAYNE LAGOON SCHEMATIC

CH2M HILL/Quest

Appendix A

Telephone Record

CH2MHILL TELEPHONE CONVERSATION RECORD

Call To:	Ronnie Thompson, Industrial Section,		KPDES Branch, KY Division of Water
Phone No.:	502-564-3410 x423	**Date:**	February 21, 2001
Call From:	Jerry Anderson	**Time:**	04:28 PM
Message Taken By:	JLA, CH2M HILL		
Subject:	Wastewater discharges from WTPs		

Do all water treatment plants in the State of KY have the same "General KPDES Permit for Wastewater Discharges assocatied with Drinking Water Plant Activities"?

Yes, unless special WTP conditions require a unique discharge permit. For example, the city of Paducah, whose WTP discharges to the Ohio River, discharges without a TSS limit, because a study was conducted demonstrating that itsTSS had no detrimental effect on the river. Project title is "*Padacah Water Works Backwash Discharge River Impact Study*". One condition on the permit (No. KY 0073113) is that an existing diffuser in the river must be maintained and used.

All other WTPs, as far as he knows, use the statewide permit for all types of WTP process wastewater, such as sedimentation sludge, filter backwashing, membranes, etc., and have a TSS limit of 30 mg/L, average, and 50 mg/L maximum daily concentration.

Could a similar study be conducted for another WTP utility that discharges to the Ohio River?

Yes. If done, submit the study report to him.

Could I get a copy of the Paducah report?

Obtain a Freedom of Information request form from fileroom contact, Anita Young, at Ext. 522, KPS Branch, DOW, fax no. 502-564-5105, 14 Raleigh Road, Frankfort, 40601.

How long will the statewide WTP permit be effective?

Until January 31, 2004. They are reissued in 5-year cycles.

Are special infrequent discharges from WTPs ever allowed?

Yes, such as water for construction work used for pressure or leak testing. Rather than issuing a permit, the DOW regional office would issue an individual authorization letter. These types of discharges usually do not have limits or sampling requirements. For a special discharge occurring as often as annually, he would have to review on a case-by-case basis to determine whether or not the statewide permit or a special discharge authorization would be appropriate for an infrequent discharge.

Are seasonal variations offered or discharge limitations ever waived on permits based on higher flow rates in the receiving stream providing more favorable dilution rates?

Not that he is aware of.

What about discharges of chlorinated water from water main flushing?

He believes at least one utility in the state, LWC, has a general "areawide" permit for discharging potable water from WTPs, storage tanks, and water mains. That way, the utility does not need to apply for approval for routine operations such as these.

General closing comment

Water treatment plants are not on KDOW's priority list of polluters.

Appendix B

KPDES Permit for City of Paducah WTP

Appendix B

KPDES Permit for City of Paducah WTP

ICKFORD
SECRETARY

PAUL E. PATTON
GOVERNOR

COMMONWEALTH OF KENTUCKY
NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION CABINET
DEPARTMENT FOR ENVIRONMENTAL PROTECTION
FRANKFORT OFFICE PARK
14 REILLY RD
FRANKFORT KY 40601

NOV 29 1999

Glen Anderson
ıcah Water Works
Box 2377
ıcah, Kentucky 42002-2377

Re: Paducah Water Treatment Plant
KPDES No.: KY0073113
McCracken County, Kentucky

Mr. Anderson:

Enclosed is the Kentucky Pollutant Discharge Elimination System (KPDES) permit the above-referenced facility. This action constitutes a final permit issuance r 401 KAR 5:075, pursuant to KRS 224.16-050.

his permit will become effective on the date indicated in the attached permit that no request for adjudication is granted. All provisions of the permit be effective and enforceable in accordance with 401 KAR 5:075, unless stayed by learing Officer under Sections 11 and 13.

Any demand for a hearing on the permit shall be filed in accordance with the :dures specified in KRS 224.10-420, 224.10-440, 224.10-470 and any regulations .lgated thereto. Any person aggrieved by the issuance of a permit final decision lemand a hearing, pursuant to KRS 224.10-420(2), within thirty (30) days from the of the issuance of this letter. Two (2) copies of request for hearing should be tted in writing to the Natural Resources and Environmental Protection Cabinet, e of Administrative Hearings, 35-36 Fountain Place, Frankfort, Kentucky 40601 and ommonwealth of Kentucky, Natural Resources and Environmental Protection Cabinet, ion of Water, 14 Reilly Road, Frankfort, Kentucky 40601. For your record keeping ses, it is recommended that these requests be sent by certified mail. The en request must conform to the appropriate statutes referenced above.

If you have any questions regarding the KPDES decision, please contact Courtney . Inventory and Data Management Section, KPDES Branch, at (502) 564-2225, sion 465.

Further information on procedures and legal matters pertaining to the hearing st may be obtained by contacting the Office of Administrative Hearings at (502) 312.

Sincerely,

Jack A. Wilson, Director
Division of Water

:ng

ORSANCO
Paducah Regional Office
Division of Water Files

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GOVERNOR

COMMONWEALTH OF KENTUCKY
NATURAL RESOURCES AND ENVIRONMENTAL PROTECTION CABINET
DEPARTMENT FOR ENVIRONMENTAL PROTECTION
FRANKFORT OFFICE PARK
14 REILLY RD
FRANKFORT KY 40601

STATEMENT OF BASIS

)ES No.: KY0073113 **Permit Writer:** Ronnie Thompson **Date:** October 8, 1999

:ility Name: Paducah Water Treatment Plant

ility Location: 1800 North Eighth Street
Paducah, McCracken County, Kentucky

mitting Action: This is a reissuance of a permit to an existing source Water Treatment Plant (SIC Code 4941).

mit Duration: This permit shall become effective February 1, 2000. The effective date of this permit is being delayed to place the facility in the correct 5-year cycle, as per the Kentucky Watershed Management Framework. During the interim period, the current permit will remain effective, in accordance with 401 KAR 5:060, Section 1(5)(c). In this instance, the permit is scheduled for reissuance in February 2005 for the Tennessee/Mississippi/Cumberland Basin Management Unit.

ription of Discharge: Filter backwash water and sedimentation basin wastewater at the rate of 109,808 gpd.

tment Provided: None

iving Stream: The Ohio River at mile point 45

am Segment Use Classification: Warmwater Aquatic Habitat and Primary/Secondary Contact Recreation

am Low Flow Condition: 7Q10 = 13,000 cfs

: Quality or Effluent Limited: This permit is effluent limited.

.fication of Permit Conditions:

:ollowing regulations are pursuant to KRS 224.10-100, 224.70-100, and 224.70-110.

. Suspended Solids and Total Residual Chlorine
monitoring requirements for these parameters are consistent with 401 KAR 5:065, on 2(8).

: limitations for this parameter are consistent with 401 KAR 5:031, Section 4.

ıdegradation:

: conditions of 401 KAR 5:029, Section 2(1) and (3) have been satisfied by this mit action. A review under Section 2(2) and (4) is not applicable.

KPDES

KENTUCKY POLLUTANT DISCHARGE ELIMINATION SYSTEM

PERMIT

PERMIT NO.: KY0073113

AUTHORIZATION TO DISCHARGE UNDER THE KENTUCKY POLLUTANT DISCHARGE ELIMINATION SYSTEM

Pursuant to Authority in KRS 224,

Paducah Water Works
P.O. Box 2377
Paducah, Kentucky 42002-2377

authorized to discharge from a facility located at

Paducah Water Treatment Plant
1800 North Eighth Street
Paducah, McCracken County, Kentucky

to receiving waters named

The Ohio River at mile point 45

in accordance with effluent limitations, monitoring requirements, and other conditions set forth in PARTS I, II, and III hereof. The permit consists of this cover sheet, and PART I 2 pages, PART II 1 page, and PART III 1 page.

This permit shall become effective on February 1, 2000.

This permit and the authorization to discharge shall expire at midnight, January 31, 2005.

NOV 29 1999
Date Signed

Jack A. Wilson, Director
Division of Water

Robert W. Logan
Commissioner

DEPARTMENT FOR ENVIRONMENTAL PROTECTION
Division of Water, Frankfort Office Park, 14 Reilly Road, Frankfort, Kentucky 40601

A. EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

During the period beginning on the effective date of this permit and lasting through the term of this permit, the permittee is authorized to discharge from Outfall serial number: 001 - Filter backwash water and sedimentation basin wastewater. 1/

Such discharges shall be limited and monitored by the permittee as specified below:

EFFLUENT CHARACTERISTICS	DISCHARGE LIMITATIONS kg/day(lbs/day) Monthly Avg.	Daily Max.	Other Units(Specify) Monthly Avg.	Daily Max.	MONITORING REQUIREMENTS Measurement Frequency	Sample Type
Flow, m^3/day (MGD)	Report	Report	N/A	N/A	1/Month	Instantaneous
Total Suspended Solids (mg/l)	N/A	N/A	Report	Report	1/Month	Composite 2/
Total Residual Chlorine (mg/l)	N/A	N/A	Report	Report	1/Discharge	Grab

The pH of the effluent shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored 1/Month by Grab sample.

There shall be no discharge of floating solids or visible foam or sheen in other than trace amounts.

Samples taken in compliance with the monitoring requirements specified above shall be taken at the following location: nearest accessible point after final treatment, but prior to actual discharge or mixing with receiving waters.

1/ In order to avoid the creation of sediment deposits and excessive turbidity in the receiving stream, the discharge shall be directed through a diffuser and flow at a regulated rate. See PART III-C.

2/ The composite sample shall contain equal volume samples taken over the period of discharge.

Schedule of Compliance

The permittee shall achieve compliance with all requirements on the effective date of this permit.

PART III

ER REQUIREMENTS

Reporting of Monitoring Results

itoring results must be obtained for each month and reported on a preprinted charge Monitoring Report (DMR) Form which will be mailed to you each quarter for upcoming quarter. The completed DMRs for each month must be sent to the Division Water at the address listed below (with a copy to the appropriate Regional Office) tmarked no later than the 28th day of the month following the completed quarter.

Division of Water
Paducah Regional Office
4500 Clarks River Road
Paducah, Kentucky 42003
ATTN: Supervisor

Kentucky Natural Resources and
Environmental Protection Cabinet
Dept. for Environmental Protection
Division of Water
Inventory & Data Management
14 Reilly Road, Frankfort Office Park
Frankfort, Kentucky 40601

Reopener Clause

s permit shall be modified, or alternatively revoked and reissued, to comply with applicable effluent standard or limitation issued or approved under 401 KAR 5:050 ough 5:080 and KRS 224, if the effluent standard or limitation so issued or ed:

1. Contains different conditions or is otherwise more stringent than any effluent limitation in the permit; or

2. Controls any pollutant not limited in the permit.

permit as modified or reissued under this paragraph shall also contain any other irements of KRS Chapter 224 when applicable.

Special Conditions

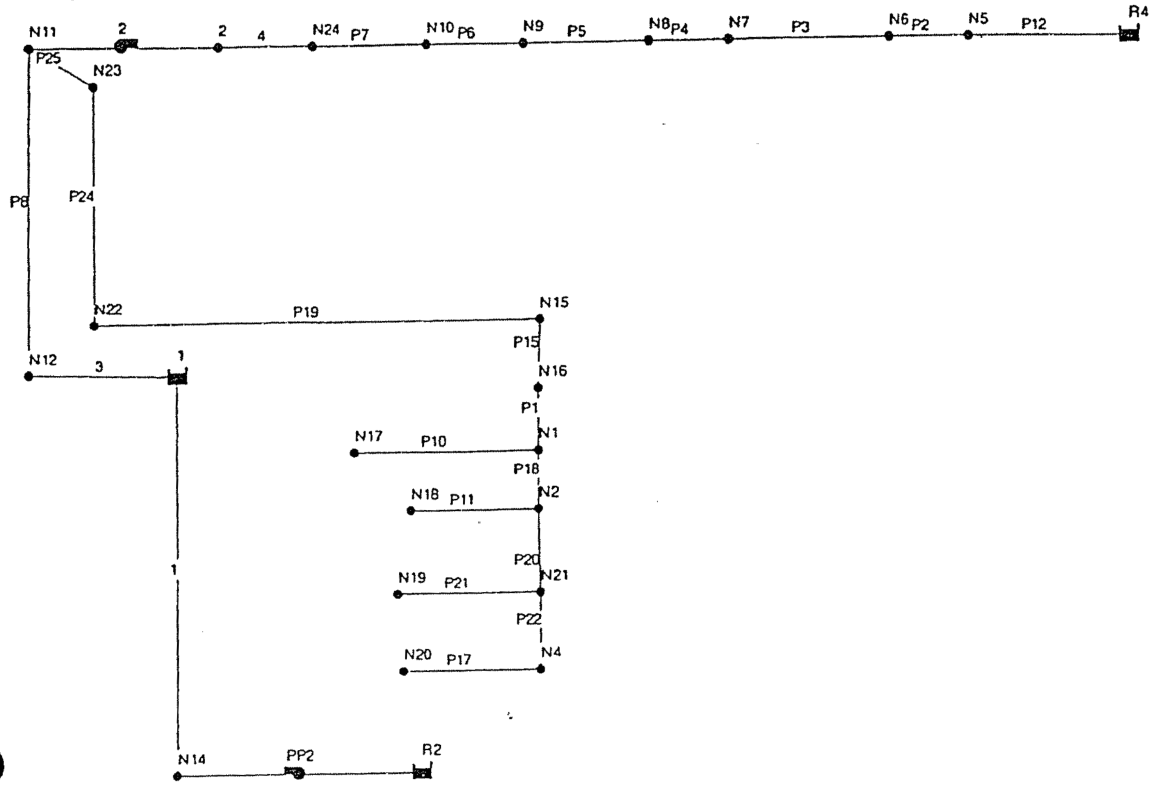
discharge shall take place through a fully functional and properly designed iport diffuser at least equivalent in performance to the diffusion structure inally approved by the Division of Water on April 6, 1993.

Appendix C

EPANET Modeling Results

Crescent Hill Drainage System

Day 1, 12:00 /



Crescent Hill Drainage System--Addendum

Network Table - Nodes

Node ID	Elevation ft	Demand MGD	Head ft	Pressure psi
Junc N5	424	0.00	474.03	21.68
Junc N6	437	0.00	475.13	16.52
Junc N7	438	0.00	484.03	19.95
Junc N8	438	0.00	484.36	20.09
Junc N9	440	0.00	490.96	22.08
Junc N10	440	0.00	493.60	23.22
Junc N11	442	0.00	546.33	45.21
Junc N12	556	0.00	550.94	-2.19
Junc N14	549	0.00	557.43	3.65
Junc N15	557.4	0.00	547.45	-4.31
Junc N16	556	0.00	547.47	-3.70
Junc N1	556	0.00	547.51	-3.68
Junc N2	556	0.00	547.54	-3.67
Junc N4	556	0.00	547.78	-3.56
Junc N17	562.5	-0.50	547.57	-6.47
Junc N18	562.5	-0.50	547.58	-6.46
Junc N19	562.7	-0.70	547.81	-6.45
Junc N20	562.7	-0.70	547.82	-6.45
Junc N21	556	0.00	547.78	-3.56
Junc N22	556	0.00	547.33	-3.76
Junc N23	442	0.00	546.34	45.21
Junc N24	451	0.00	517.67	28.89
Junc 2	442	0.00	596.33	66.87
Resvr R2	530	-5.32	530.00	0.00
Resvr R4	446	7.90	446.00	0.00
Resvr 1	551	-0.18	551.00	0.00

Network Table - Links

Link ID	Length ft	Diameter in	Roughness	Flow MGD	Velocity fps	Headloss ft/Kft
Pipe P2	200	24	76	-7.90	3.89	5.48
Pipe P3	2700	24	100	-7.90	3.89	3.30
Pipe P4	100	24	100	-7.90	3.89	3.30
Pipe P5	2000	24	100	-7.90	3.89	3.30
Pipe P6	800	24	100	-7.90	3.89	3.30
Pipe P7	7300	24	100	-7.90	3.89	3.30
Pipe P8	8100	30	100	-5.50	1.74	0.57
Pipe P12	8500	24	100	7.90	3.89	3.30
Pipe P15	100	30	100	-2.40	0.76	0.12
Pipe P1	50	20	100	-2.40	1.70	0.88
Pipe P10	100	12	100	-0.50	0.98	0.58
Pipe P11	75	12	100	-0.50	0.98	0.58
Pipe P17	30	12	100	-0.70	1.38	1.08
Pipe P18	50	20	100	-1.90	1.35	0.57
Pipe P20	250	16	100	-1.40	1.55	0.96
Pipe P21	30	12	100	-0.70	1.38	1.08
Pipe P22	20	16	100	-0.70	0.78	0.27
Pipe P24	8100	30	100	2.40	0.76	0.12
Pipe P25	20	24	100	2.40	1.18	0.36

Link ID	Length ft	Diameter in	Roughness	Flow MGD	Velocity fps	Headloss ft/Kft
Pipe P19	1000	30	100	2.40	0.76	0.12
Pipe 1	2540	21.8	100	5.32	3.18	2.53
Pipe 3	100	30	100	5.50	1.74	0.57
Pipe 4	14700	24	77	7.90	3.89	5.35
Pump PP1	#N/A	#N/A	#N/A	5.32	0.00	-27.43
Pump PP2	#N/A	#N/A	#N/A	0.00	0.00	0.00
Pump 2	#N/A	#N/A	#N/A	7.90	0.00	-50.00

Appendix D

Construction Cost Estimate

CONTINGENCY =				
	5.00%	5.00%	5.00%	10.00% / 5.00%
	20.00%	20.00%	20.00%	20.00%

NO.	DESCRIPTION	QTY	UNIT	MATERIALS UNIT $	MATERIALS AMOUNT	LABOR MH	LABOR RATE	LABOR AMOUNT	CONST. EQUIP. UNIT $	CONST. EQUIP. AMOUNT	INSTL or S/C UNIT $	INSTL or S/C AMOUNT	TOTAL	RESOURCE
	SITE WORK													
	CLEAR & GRUB AS REQUIRED	0.2	AC								$2,500.00	$500	$500	CH2M HILL COST DATA
	EROSION CONTROL	1	LS								$1,000.00	$1,000	$1,000	CH2M HILL COST DATA
	STRIP TOP SOIL, 6" DEEP 200' HAUL	889	SY				$0.06	$53	$0.16	$142			$196	MEANS 98 022 286 0200
	SPREAD TOP SOIL WITH FRONT END LOADER 6" DEEP	772	SY				$0.27	206	$0.25	$196			$401	MEANS 00 02310 460 0020
	SODDING	210	SY								$2.10	$441	$441	CH2M HILL HISTORICAL DATA
	SEEDING & MULCH	618	SY								$0.31	$192	$192	CH2M HILL HISTORICAL DATA
	STRUCTURAL EXCAVATION	2741	CY								$8.00	$21,928	$21,928	
	STRUCTURAL BACKFILL	2257	CY								$10.00	$22,575	$22,575	
	LOAD AND HAUL EXCAVATED MATERIAL OFF SITE	484	CY								$4.00	$1,934	$1,934	
	PLACE #57 STONE 6" DEEP	97.1	CY								$22.50	$2,185	$2,185	
	DEWATERING WELL POINTS	160	LF								$253.00	$40,480	$40,480	
	CONCRETE													
	49' x 27.33' x 2' THICK CONCRETE SLAB ON GRADE	99.0	CY								$220	$21,780	$21,780	
	1.67' THICK CONCRETE WALLS	186.0	CY								$342.00	$63,612	$63,612	CH2M HILL COST DATA
	45' X 23.33' X 1' SUSPENDED SLABS	39.0	CY								$375.00	$14,625	$14,625	
	CONCRETE BEAMS	5	CY								$600.00	$3,000	$3,000	
	MISCELLANEOUS CONCRETE	10	CY								$400.00	$4,000	$4,000	
	BUILDING													
	23.33' X 45' BUILDING CMU WITH BRICK VENEER	1049.85	SF								$110.00	$115,484	$115,484	
	MONORAIL AND HOIST	1	LS								$15,000.00	$15,000	$15,000	
	EQUIPMENT													
	VERTICAL CENTRIFUGAL 3000 GPM, @ 50 FT TDH	3	EA	$28,000.00	$84,000		$4,200.00	$12,600					$96,600	QUOTE
	VARIABLE FREQUENCY DRIVE 60 HP	3	EA	$14,100.00	$42,300		$2,346.00	$7,038	$365.00	$1,095			$50,433	MEANS 2001 16220 900 1200
	MECHANICAL													
	20" GATE VALVE FLG	3	EA	$4,680.00	$14,040	21.15	$32.00	$2,030	$142.34	$427			$16,497	CH2M HILL COST DATA
	16" GATE VALVE FLG	3	EA	$2,970.00	$8,910	16.14	$32.00	$1,549	$108.62	$326			$10,785	CH2M HILL COST DATA
	16" CHECK VALVE	3	EA	$3,726.00	$11,178	11.94	$32.00	$1,146	$80.36	$241			$12,565	CH2M HILL COST DATA
	16" X 8" CLDI FXF CON REDUCER	3	EA	$420.00	$1,260	11.02	$32.00	$1,058	$52.90	$159			$2,477	CH2M HILL COST DATA
	20" X 10" CLDI FXF CON REDUCER	3	EA	$620.00	$1,860	11.80	$32.00	$1,133	$79.41	$238			$3,231	CH2M HILL COST DATA
	24" X 16" CLDI FXF TEE	3	EA	$2,137.50	$6,413	28.13	$32.00	$2,700	$189.31	$568			$9,681	CH2M HILL COST DATA

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ESTIMATE [illegible]ARY

PROJECT : [illegible]ISVILLE WATER COMPANY

FACILITY : 8.6 MGD PUMP STATION

FILE NAME: J:\CONSTRUC\CMS\LOUISVILL\LouisvillePumpStation0.xls

ESTIMATOR: D JONES
PROJ. MANAGER:
PROJ. NO.: 157683.DR.00
ESTIMATE NO. : 2001016
REV. NO. : 0
DATE: 2/23/01

MARK-UPS:

	MATL.	LABOR	EQUIP.	INSTL or S/C
OVERHEAD =	10.00%	10.00%	10.00%	10.00%
PROFIT =	10.00%	10.00%	10.00%	10.00%
MOB/BOND/INS. =	5.00%	5.00%	5.00%	5.00%
CONTINGENCY =	20.00%	20.00%	20.00%	20.00%

NO.	DESCRIPTION	QTY	UNIT	MATERIALS UNIT $	MATERIALS AMOUNT	LABOR MH	LABOR RATE	LABOR AMOUNT	CONST. EQUIP. UNIT $	CONST. EQUIP. AMOUNT	INSTL or S/C UNIT $	INSTL or S/C AMOUNT	TOTAL	RESOURCE
	16" CLDI FXF 90 DEG ELBOW	3	EA	$1,010.00	$3,030	12.10	$32.00	$1,162	$58.08	$174	°		$4,366	CH2M HILL COST DATA
	20" CLDI FLGD WALL PIPE WITH SUCTION BELL	3	EA	$1,000.00	$3,000	10.00	$32.00	$960	$48.00	$144			$4,104	
	24" BLIND FLANGE	1	EA	$550.00	$550	9.00	$32.00	$288	$43.20	$43			$881	CH2M HILL COST DATA
	16" X 1' CLDI CL 53 FXF SPOOL	3	EA	$496.46	$1,489	6.38	$32.00	$612	$30.61	$92			$2,193	CH2M HILL COST DATA
	16" X 2' CLDI CL 53 FXF SPOOL	3	EA	$565.97	$1,698	7.35	$32.00	$706	$35.29	$106			$2,510	CH2M HILL COST DATA
	16" X 3' CLDI CL 53 FXF SPOOL	3	EA	$608.39	$1,825	8.33	$32.00	$799	$39.97	$120			$2,744	CH2M HILL COST DATA
	16" X 12' CLDI CL 53 FXF SPOOL	3	EA	$1,008.57	$3,026	17.11	$32.00	1,642	$82.12	$246			$4,914	CH2M HILL COST DATA
	20" X 1' CLDI CL 53 FXF SPOOL	3	EA	$631.32	$1,894	8.63	$32.00	$829	$58.11	$174			$2,897	CH2M HILL COST DATA
	20" X 2' CLDI CL 53 FXF SPOOL	3	EA	$726.02	$2,178	9.77	$32.00	$938	$65.75	$197			$3,315	CH2M HILL COST DATA
	24" X 5' CLDI CL 53 FXF SPOOL	2	EA	$1,091.41	$2,183	14.97	$24.55	$735	$100.72	$201			$3,119	CH2M HILL COST DATA
	24" X 8' CLDI CL 53 FXF SPOOL	1	EA	$1,295.02	$1,295	18.87	$24.55	$463	$126.96	$127			$1,885	CH2M HILL COST DATA
	AIR RELEASE VALVE	1	EA								$1,400.00	$1,400	$1,400	
	SMALL BORE PIPING	1	LS								$15,000.00	$15,000	$15,000	
	PIPE SUPPORTS	1	LS								$14,000.00	$14,000	$14,000	
A	SUBTOTAL				$192,129			$38,649		$5,017		$359,135	$594,929	
B	OVERHEAD & PROFIT	(A*ohd)+((A+ohd)*p)			$40,347			$8,116		$1,054		$75,418	$124,935	
C	MOB / BOND / INSUR.	(% of A)			$9,606			$1,932		$251		$17,957	$29,746	
D	CONTINGENCY	(% of A)			$38,426			$7,730		$1,003		$71,827	$118,986	
E	TOTAL ESTIMATED CONSTRUCTION COST				$280,508			$56,427		$7,325		$524,336	$868,600	

ALLOWANCES	5.00%	$53,000
I&C	10.00%	$107,000
ELECTRICAL	2.00%	$21,000
FINISHES	1.50%	$16,000
METALS		
TOTAL ESTIMATED CONSTRUCTION COST		$1,066,000

Appendix E

Drainage Area Map of CHWTP

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ESTIMATE SUMMARY

PROJECT : LOUISVILLE WATER COMPANY
FACILITY : 8.6 MGD PUMP STATION
FILE NAME: J:\CONSTRUC\CMS\LOUISVILL\LouisvillePumpStation0.xls

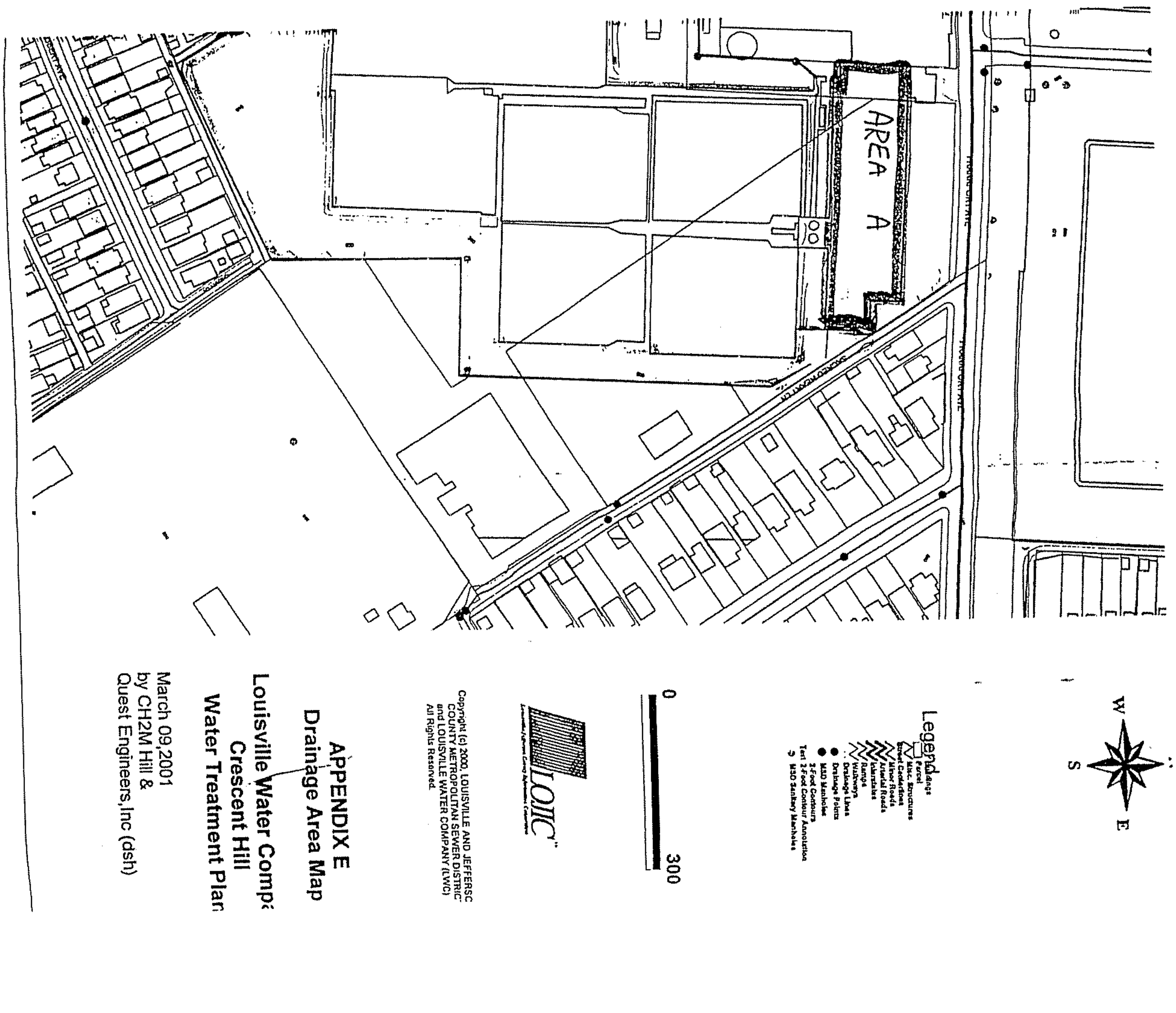
ESTIMATOR: D JONES
PROJ. MANAGER:
PROJ. NO.: 157683 DR 00
ESTIMATE NO.: 2001016
REV. NO.: 0
DATE: 2/23/01

MARK-UPS:

	MATL.	LABOR	EQUIP.	INSTL or S/C
OVERHEAD =	10.00%	10.00%	10.00%	10.00%
PROFIT =	10.00%	10.00%	10.00%	10.00%
MOB/BONDANS. =	5.00%	5.00%	5.00%	5.00%
CONTINGENCY =	20.00%	20.00%	20.00%	20.00%

NO.	DESCRIPTION	QTY	UNIT	MATERIALS UNIT $	MATERIALS AMOUNT	LABOR MH	LABOR RATE	LABOR AMOUNT	CONST. EQUIP. UNIT $	CONST. EQUIP. AMOUNT	INSTL or S/C UNIT $	INSTL or S/C AMOUNT	TOTAL	RESOURCE
	16" CLDI FXF 90 DEG ELBOW	3	EA	$1,010.00	$3,030	12.10	$32.00	$1,162	$58.08	$174			$4,366	CH2M HILL COST DATA
	20" CLDI FLGD WALL PIPE WITH SUCTION BELL	3	EA	$1,000.00	$3,000	10.00	$32.00	$960	$48.00	$144			$4,104	
	24" BLIND FLANGE	1	EA	$550.00	$550	9.00	$32.00	$288	$43.20	$43			$881	
	16" X 1' CLDI CL 53 FXF SPOOL	3	EA	$496.46	$1,489	6.38	$32.00	$612	$30.61	$92			$2,193	CH2M HILL COST DATA
	16" X 2' CLDI CL 53 FXF SPOOL	3	EA	$565.97	$1,698	7.35	$32.00	$706	$35.29	$106			$2,510	CH2M HILL COST DATA
	16" X 3' CLDI CL 53 FXF SPOOL	3	EA	$608.39	$1,825	8.33	$32.00	$799	$39.97	$120			$2,744	CH2M HILL COST DATA
	16" X 12' CLDI CL 53 FXF SPOOL	3	EA	$1,008.57	$3,026	17.11	$32.00	1,642	$82.12	$246			$4,914	CH2M HILL COST DATA
	20" X 1' CLDI CL 53 FXF SPOOL	3	EA	$631.32	$1,894	8.63	$32.00	$829	$58.11	$174			$2,897	CH2M HILL COST DATA
	20" X 2' CLDI CL 53 FXF SPOOL	3	EA	$726.02	$2,178	9.77	$32.00	$938	$65.75	$197			$3,315	CH2M HILL COST DATA
	24" X 5' CLDI CL 53 FXF SPOOL	2	EA	$1,091.41	$2,183	14.97	$24.55	$735	$100.72	$201			$3,119	CH2M HILL COST DATA
	24" X 8' CLDI CL 53 FXF SPOOL	1	EA	$1,295.02	$1,295	18.87	$24.55	$463	$126.96	$127			$1,885	CH2M HILL COST DATA
	AIR RELEASE VALVE	1	EA								$1,400.00	$1,400	$1,400	
	SMALL BORE PIPING	1	LS								$15,000.00	$15,000	$15,000	
	PIPE SUPPORTS	1	LS								$14,000.00	$14,000	$14,000	
A	SUBTOTAL				$192,129			$38,649		$5,017		$359,135	$594,929	
B	OVERHEAD & PROFIT	(A*ohd)+((A+ohd)*p)			$40,347			$8,116		$1,054		$75,418	$124,935	
C	MOB / BOND / INSUR.	(% of A)			$9,608			$1,932		$251		$17,957	$29,746	
D	CONTINGENCY	(% of A)			$38,426			$7,730		$1,003		$71,827	$118,986	
E	TOTAL ESTIMATED CONSTRUCTION COST				$280,508			$56,427		$7,325		$524,336	$868,600	

ALLOWANCES		
I&C	5.00%	$53,000
ELECTRICAL	10.00%	$107,000
FINISHES	2.00%	$21,000
METALS	1.50%	$16,000
TOTAL ESTIMATED CONSTRUCTION COST		$1,066,000



Appendix F

BE Payne Lagoon Computations and Support Data

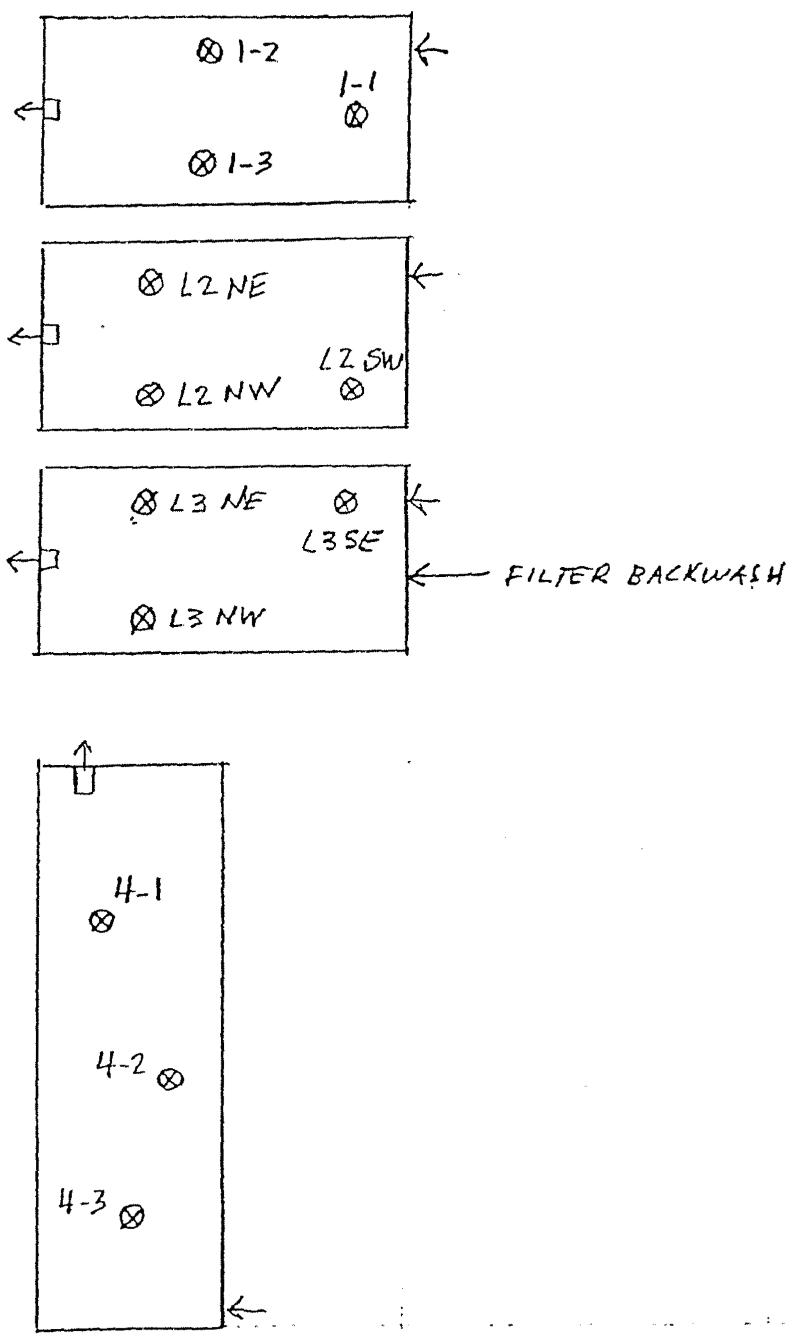
Appe F
BE Payne Sludge Lagoon Solids Samples
Jan-2001

Sample Location: L2 NW Dish # 1 Dried sample + dish weight 155.4628 grams Dish tare weight 153.3339 grams Sample weight 2.1289 grams Wet sample amount 29.67 grams **7.18%** % dried solids	Sample Location: L2 SW Dish # 2 Dried sample + dish weight 168.1116 grams Dish tare weight 153.7088 grams Sample weight 14.4028 grams Wet sample amount 29.34 grams **49.09%** % dried solids	Sample Location: L2 NE Dish # 3 Dried sample + dish weight 158.0495 grams Dish tare weight 155.3263 grams Sample weight 2.7232 grams Wet sample amount 28.19 grams **9.66%** % dried solids
Sample Location: L3 NW Dish # 6 Dried sample + dish weight 165.4276 grams Dish tare weight 154.6526 grams Sample weight 10.7750 grams Wet sample amount 25.37 grams **42.47%** % dried solids	Sample Location: L3 SE Dish # 5 Dried sample + dish weight 166.6867 grams Dish tare weight 156.8212 grams Sample weight 9.8655 grams Wet sample amount 27.87 grams **35.40%** % dried solids	Sample Location: L3 NE Dish # 4 Dried sample + dish weight 168.6537 grams Dish tare weight 157.7274 grams Sample weight 10.9263 grams Wet sample amount 29.14 grams **37.50%** % dried solids
Sample Location: 1-1 Dish # 1 Dried sample + dish weight 158.3531 grams Dish tare weight 153.3342 grams Sample weight 5.0189 grams Wet sample amount 7.35 grams **68.28%** % dried solids	Sample Location: 1-2 Dish # 2 Dried sample + dish weight 160.5213 grams Dish tare weight 153.7050 grams Sample weight 6.8163 grams Wet sample amount 11.12 grams **61.30%** % dried solids	Sample Location: 1-3 Dish # 3 Dried sample + dish weight 161.5481 grams Dish tare weight 155.3239 grams Sample weight 6.2242 grams Wet sample amount 10.48 grams **59.39%** % dried solids
Sample Location: 4-3 Dish # 6 Dried sample + dish weight 160.3518 grams Dish tare weight 154.6509 grams Sample weight 5.7009 grams Wet sample amount 10.38 grams **54.92%** % dried solids	Sample Location: 4-2 Dish # 5 Dried sample + dish weight 162.6721 grams Dish tare weight 156.8202 grams Sample weight 5.8519 grams Wet sample amount 9.92 grams **58.99%** % dried solids	Sample Location: 4-1 Dish # 4 Dried sample + dish weight 165.8020 grams Dish tare weight 157.7254 grams Sample weight 8.0766 grams Wet sample amount 11.34 grams **71.22%** % dried solids

CH2M HILL

LOCATIONS | SHEET NO. ____ of ____ DATE ________

BE PAYNE WTP LAGOONS | PROJECT NO. ________



Appendix F
Lagoon Solids Computations
Lagoon No. 1

		P1	P2
TS1	442.32		
P1	440.94	440.72	436.27
P2	434.09	439.03	436.51
TS2	431.00	440.5	435.08
		440.36	434.92
		440.75	434.53
		441.22	434.02
D1	442	441.61	433.49
D2	468	441.13	433.39
D3	412	441.5	433.56
		441.56	433.42
		441.35	432.81
		441.38	432.45
		441.28	431.84
		440.55	433.5
		441.2	435.5
Area 1	5141.05	440.9427	434.086
Area 2	3516.71		
Area 3	1047.72		

Total Solids Cross-sectional Area	9705.47
Total Cross-sectional Area	18508
Percent Solids	52.44%
Total Volume	10,497,000
Sludge Volume	5,504,558
Available Capacity	4,992,442

Solids content from lab:

No.	Weighting	Percent
1-1	33.33%	68.3%
1-2	33.33%	61.3%
1-3	33.33%	59.4%
Average	**100.00%**	**63.0%**

ɔpendix F
ıg[illegible] Solids Computations
ıgoon No. 2

		P2
TS1	443.14	438.06
P1	437.57	438.28
P2	435.83	438
TS2	428.15	438.38
		438.19
D1	440	434.87
D2	300	434.1
D3	410	434.7
		436.03
		432.78
		432.55
		432.91
		434.8
		437.93
		435.8271

Area 1	4556.20
Area 2	2009.57
Area 3	815.31
ɔtal Solids Cross-sectional Area	7381.09
ɔtal Cross-sectional Area	16100
ercent Solids	45.85%
[illegible] Volume	9,476,000
udge Volume	4,344,296
vailable Capacity	5,131,704

ɔlids content from lab:	No.	Weighting	Percent
	L2 NW	16.67%	7.2%
	L2 SW	66.67%	49.1%
	L2 NE	16.67%	9.7%
	Average	**100.00%**	**35.5%**

Appendix F
Lagoon Solids Computations
Lagoon No. 3

TS1	445.40
P1	441.23
P2	439.48
TS2	432.41
D1	496
D2	226
D3	400

	P2
	441.59
	438.8
	438.28
	438.92
	439.03
	439.68
	439.53
	439.08
	439.27
	439.29
	439.29
	439.43
	439.14
	439.6
	439.8
	441
Average	439.4831

Area 1	6604.24
Area 2	2340.58
Area 3	2378.63
Total Solids Cross-sectional Area	11323.45
Total Cross-sectional Area	15708
Percent Solids	72.09%
Total Volume	10,610,000
Sludge Volume	7,648,446
Available Capacity	2,961,554

Solids content from lab:

No.	Weighting	Percent
L3 NW	25.00%	42.5%
L3 SE	50.00%	35.4%
L3 NE	25.00%	37.5%
Average	100.00%	37.7%

ppendix F
agoon Solids Computations
a[illegible] No. 4

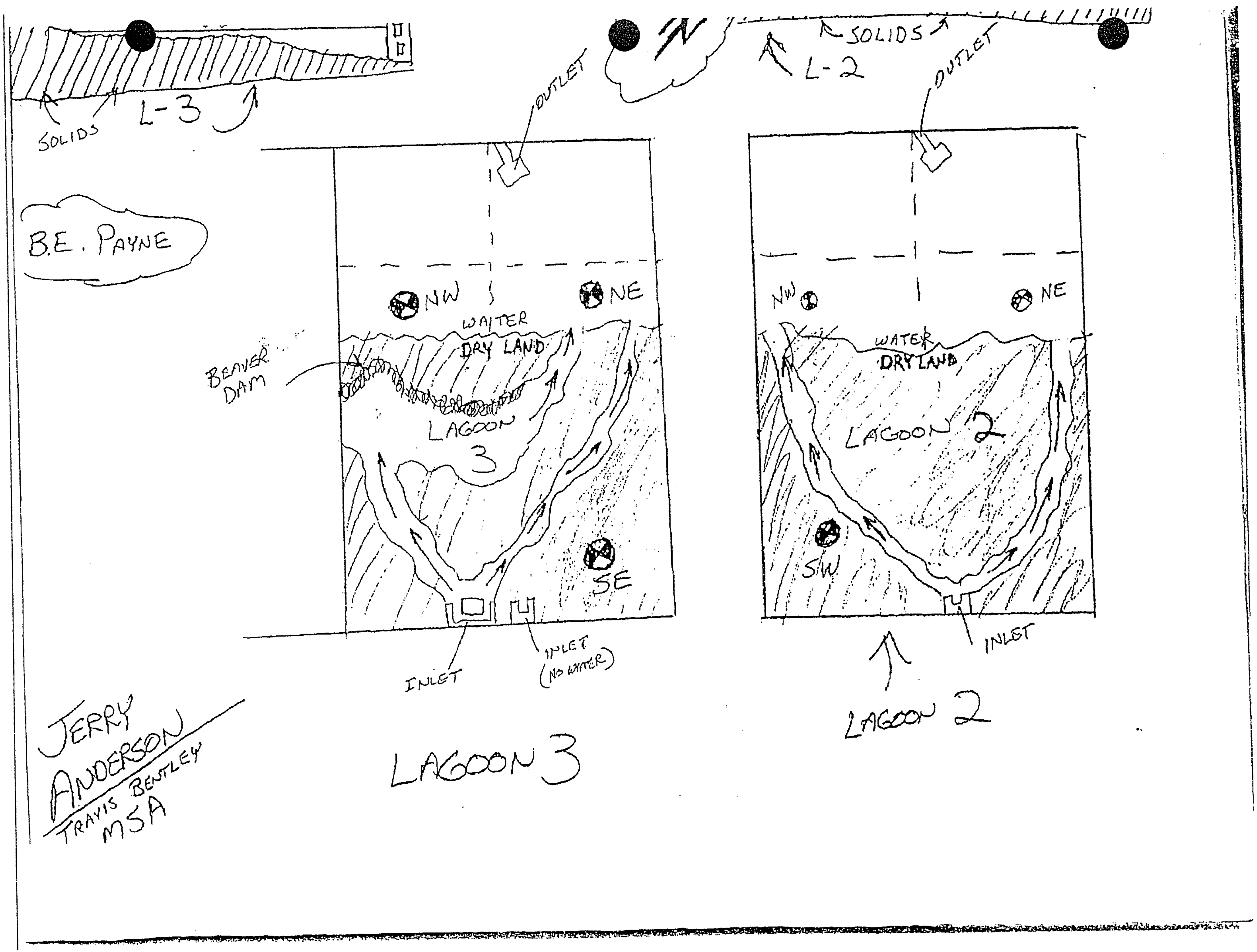
TS1	442.05
P1	437.98
P2	431.20
P3	425.36
TS2	424.01
D1	510
D2	500
D3	504
D4	466

P1	P2	P3
437.32	431.9	426.4
436.53	430.85	424.57
436.51	430.98	424.78
436.17	430.97	424.99
436.44	431.14	424.98
436.59	431	424.98
437.85	431	425.23
438.42	431.34	425.38
439.03	431.12	425.36
439.2	431.23	425.15
439.43	431.14	424.77
439.34	430.94	424.92
438.78	431.02	426.78
440.07	432.1	426.78
437.9771	431.195	425.3621

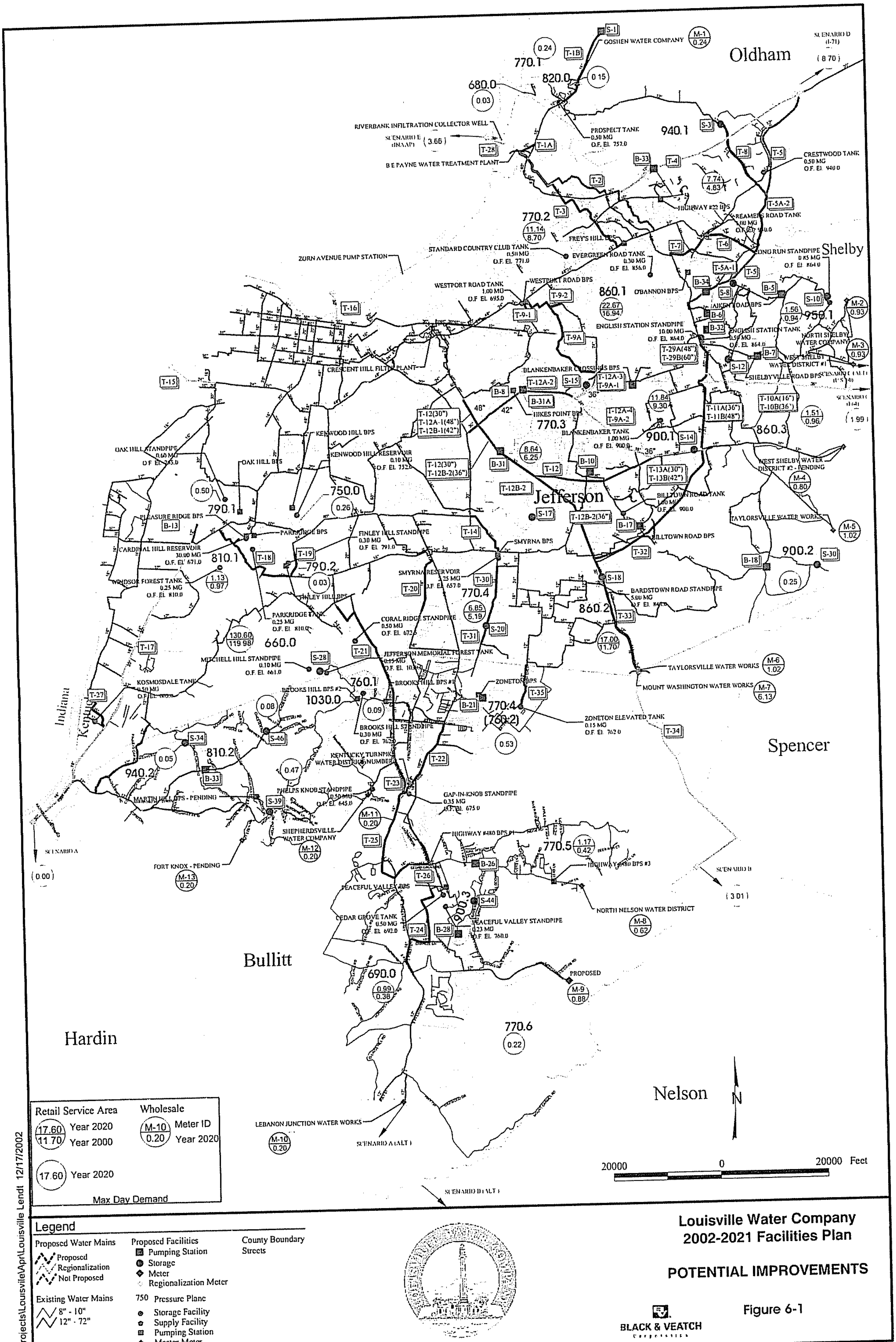
Area 1	8676.92
Area 2	5793.04
Area 3	2660.40
Area 4	785.71
otal Solids Cross-sectional Area	17916.07
otal Cross-sectional Area	43560
[illegible]nt Solids	41.13%
[illegible]Volume	23,753,000
ludge Volume	9,769,521
vailable Capacity	13,983,479

olids content from lab:

No.	Weighting	Percent
4-1	33.33%	68.3%
4-2	33.33%	61.3%
4-3	33.33%	59.4%
Average	100.00%	63.0%



SOLIDS
L-3
OUTLET
N
SOLIDS
L-2
OUTLET
B.E. PAYNE
NW
NE
WATER
DRY LAND
BEAVER DAM
LAGOON 3
SE
INLET
INLET (NO WATER)
NW
NE
WATER
DRY LAND
LAGOON 2
SW
INLET
LAGOON 2
JERRY ANDERSON
TRAVIS BENTLEY MSA
LAGOON 3



Louisville Water Company
2002-2021 Facilities Plan

POTENTIAL IMPROVEMENTS

Figure 6-1

BLACK & VEATCH

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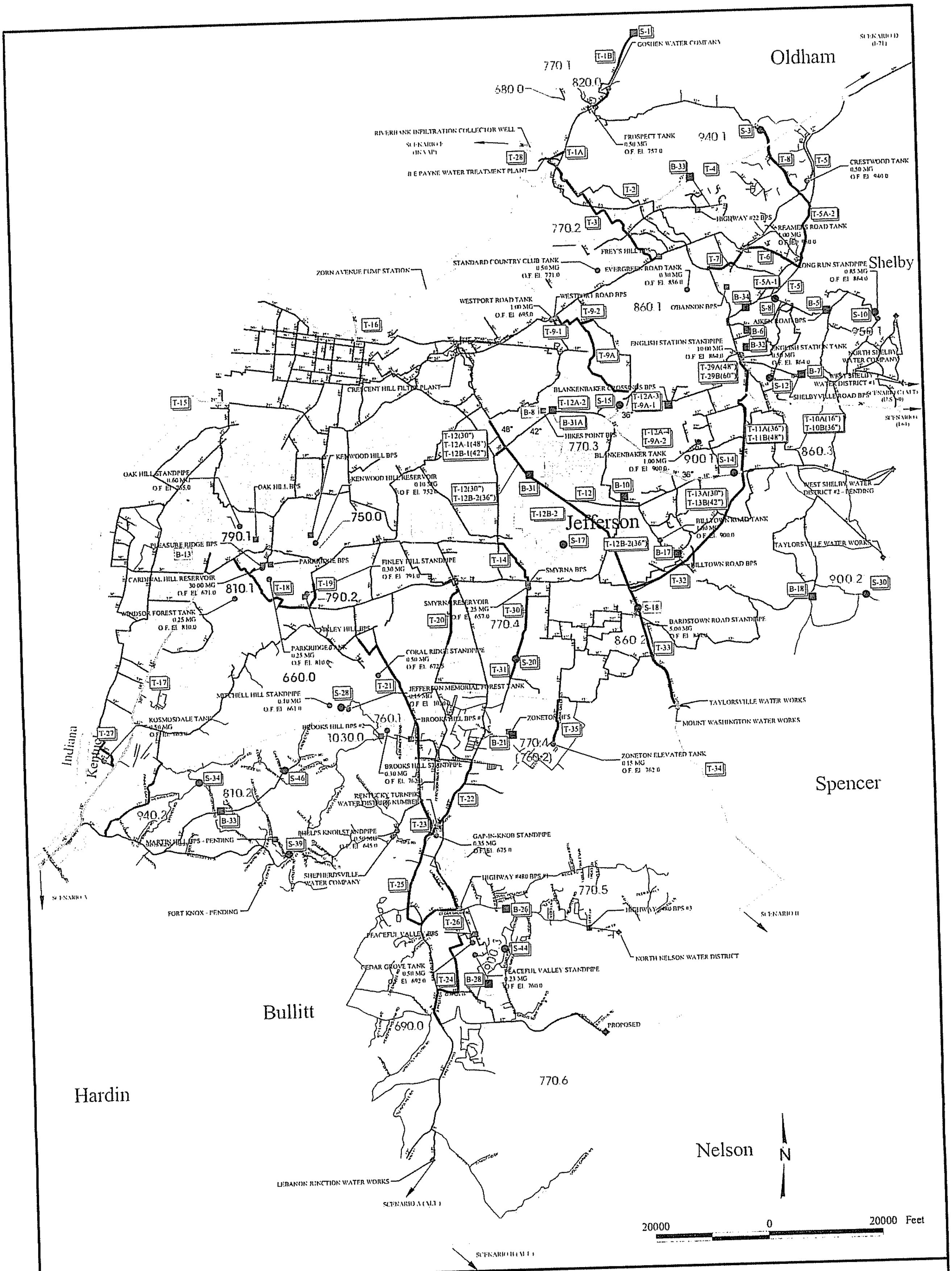

Legend

Proposed Water Mains
Proposed
Regionalization
Not Proposed

Proposed Facilities
Pumping Station
Storage
Meter
Regionalization Meter

County Boundary
Streets

Existing Water Mains
8" - 10"
12" - 72"

750 Pressure Plane
Storage Facility
Supply Facility
Pumping Station
Master Meter

D:\projects\Louisville\Apr\Louisville

Louisville Water Company
2002-2021 Facilities Plan

POTENTIAL IMPROVEMENTS

Figure 6-3

BLACK & VEATCH

BOARD OF WATER WORKS
SPECIAL-CALLED MEETING MINUTES
OCTOBER 1, 2007

Board Members Present:
Mr. Stewart Conner, Chair
Ms. Wendy Welsh, Vice-Chair
Mr. Ed Crooks
Ms. Margaret Harris (via Conference Call)
Mr. Gerald Martin (via Conference Call)
Ms. Marita Willis (via Conference Call)

Not Present:
Mayor Jerry Abramson

Others Present:
Mr. Gregory Heitzman, CEO/President
Mr. Rick Johnstone, Deputy Mayor, Louisville Metro Government
Mr. James Brammell, Vice President and Chief Engineer
Ms. Barbara Dickens, Vice President, General Counsel and Secretary
Ms. Susan Lehmann, Vice President, Human Resources and Organizational Effectiveness
Mr. Robert Miller, Vice President, Business Resources and Treasurer
Ms. Amber Halloran, Business System Owner, Supplying Financial Resources and Controller
Mr. Jim Smith, Business System Owner, Infrastructure Planning and Business Development

Visitors Present:

The special-called meeting of the Board of Water Works was held on Monday, October 1, 2007 at, Louisville Water Company, The John L. Huber Building, 550 South Third Street, Louisville, Kentucky. Chairperson Stewart Conner called the meeting to order at10:00 a.m.

Closed Session Held

Ms. Margaret Harris moved to go into closed session to discuss a specific proposal pursuant to KRS 61.810(1) (g), respectively at 10:00 a.m. Ms. Wendy Welsh seconded, and the motion carried.

Open Session Resumed

On the motion of Ms. Harris, seconded by Ms. Welsh and unanimously carried, the Board resumed open session at 11:01 a.m.

President Authorized to Submit Proposal to Supply Water to Central Kentucky

Mr. Conner moved to authorize President Greg Heitzman to propose a 25 MGD supply of potable water to Central Kentucky by offering to construct and fund a 36-inch transmission main be installed along the Interstate 64 corridor from English Station Road/I-265 in Jefferson County to Kentucky Highway 53 in Shelby County, and further, to collaborate through public/private partnerships in the construction and financing of a 36-inch transmission main to be installed along the Interstate 64 corridor from Kentucky Highway 53 in Shelby County to Kentucky American Water Company's 24-inch water main in Newtown

Pike in Fayette County, including but not limited to, associated pumping stations and storage facilities; said proposal being submitted with the understanding that it embody the specifics outlined in the document "Louisville Water Company Proposal for a Louisville to Lexington Pipeline Along Interstate 64" dated October 1, 2007 and be contingent upon the negotiation of a contract containing those terms and other standard terms and conditions. Ms. Harris seconded, and the motion carried

Ms. Barbara Dickens updated the Directors regarding the schedule of the Public Service Commission matter, in which the Company is an intervening party. Mr. Heitzman briefly discussed the rebuttal testimony being filed today by the Company in that matter.

There being no further business, Ms. Welsh moved to adjourn the meeting, seconded by Ms. Marita Willis, and carried. The meeting was adjourned at 11:10 a.m.

Respectfully submitted,

Barbara K. Dickens

Barbara K. Dickens
Vice President, General Counsel and Secretary