FINAL

TECHNICAL MEMORANDUM NO. 6
TUNNEL AND SHAFT CONSTRUCTION WATER TREATMENT AND DISCHARGE

Water Treatment Plant #4 – Jollyville Transmission Main
Phase B – Final Design
CIP ID: 6935.016

B&V Project 167760
B&V File D-1.2

To: Stacie Long, P.E. – Project Manager, City of Austin

From: Dennis Allen, P.E. – Project Manager, Black & Veatch

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1. Introduction

The purpose of this Technical Memorandum No. 6 is to present applicable methods of treatment and discharge of groundwater inflow intercepted during construction of the shafts and tunnel as part of the Jollyville Transmission Main (JTM). The proposed JTM will convey finished water from Water Treatment Plant No. 4 (WTP4) to the Jollyville Reservoir (JR) for distribution by the City of Austin (COA).

2. Tunnel and Shaft Locations

The tunnel alignment and shaft locations are discussed in detail in TM No. 11 (Evaluation of Tunneling Alternatives Concepts) dated September 22, 2010. Table 1 lists the shaft locations and describes where treatment and discharge of shaft and/or tunnel construction water will be required.

Table 1 JTM Working (W) Shafts or Retrieval (R) Shafts		
Shaft Designation	**General Location**	**General Description**
W-2	Jollyville Reservoir (JR)	At the JR tank site on COA property, in the Rattan Creek Watershed. Manage both shaft and tunnel construction discharges.
R-2	Spicewood Springs Road (PARD property)	Parks and Recreation Department (PARD) property on east side of Spicewood Springs Road where it turns to the south, in the Bull Creek watershed. Manage shaft construction discharge.
W-1	Four Points Area (FPA)	Site northwest of intersection of RM 2222 and River Place Boulevard and southwest of Four Points Drive and River Place Boulevard, in the Bull Creek watershed. Manage both shaft and tunnel construction discharge.
R-1	WTP4	Near medium service pumping station on the west side of the WTP4 site, in the Lake Travis watershed. Manage shaft construction discharge.

3. Background

Groundwater will be encountered during construction of the working and retrieval shafts and the tunnel. Based on site geology and hydrogeology, shafts will straddle overburden soils, Edwards, Walnut Creek, and Glen Rose formations and associated groundwater tables. The tunnel will be constructed entirely within the Glen Rose rock formation. The water will be sediment laden with a milky appearance due to rock cuttings carried with it. In addition to rock cuttings, the water may contain trace amounts of oil and grease, largely due to lubricants used for bearings in the tunnel boring machine (TBM) and other moving machinery. TM No. 5 (Groundwater Inflow Mitigation Plan) contains a thorough discussion of groundwater inflow.

4. Applicable Discharge Regulations

The following regulations would apply for discharge of treated water:

- The Texas Commission on Environmental Quality (TCEQ) regulates stormwater and construction wastewater discharge permits within the State of Texas.
- The City of Austin Environmental Criteria Manual regulates water quality within the city.
- Jollyville and Forest Ridge Transmission Mains, Environmental Commissioning Plan.
- The Austin Water Utility Special Services Division regulates construction water discharges to the sanitary sewer system.

The TCEQ is responsible for issuance of any Texas Pollutant Discharge Elimination System (TPDES) permits within the state of Texas. A TPDES permit will be required for discharge of any water, either to a local body of water or into the local sanitary sewer system. Black & Veatch met with the TCEQ and determined that an individual wastewater discharge permit will not be required for this project. If water is to be discharged to a local water body, the applicable permit is titled TPDES General Permit No TXG830000, which covers waters contaminated by petroleum fuel or petroleum substances. This permit will cover the discharge to surface water of tunnel and construction water for the JTM, due to the expected presence of oil and grease. The permit contains several guidelines for the quality of the discharged water, including a Total Petroleum Hydrocarbon content of 15 mg/L, as well as "no visible oil." These include general requirements prohibiting the discharge of any other substances that could be harmful to human health or the environment, including prohibition of any discharge that could harm endangered species or its habitat. However, there are no specific discharge requirements in the General Permit for total suspended solids, turbidity, or pH.

The Environmental Criteria Manual has basic requirements for construction water encountered during tunneling operations: "*All silted water and slurry generated by the construction can be pumped into one or more temporary earthen pits or metal tanks to allow the sediment to settle before discharging the clean water. These temporary sedimentation facilities must be adequately sized to be most effective and may be constructed in series to improve sediment removal. (Section 1.4.4(4)(G))*" On other local tunneling projects, this rule is enforced in practice by taking grab samples of the discharge water after it has passed through settling tanks, placing it in a clear plastic bottle, and attempting to read a sheet of

paper through the fluid. If an inspector can legibly read the writing through the bottle, it is deemed safe to discharge.

Black & Veatch met with the City of Austin Watershed Protection Department (WPD) to discuss baseline discharge parameters if water were to be discharged to the Bull or Rattan Creek watersheds. WPD proposed discharge parameters based on the ambient water quality in the two creeks. The recommended parameters from the WPD are listed in Tables 2 and 3 below.

Table 2
WPD Recommended Discharge Limitations for Bull Creek

Parameter	Unit	Standard	Reasoning
24-Hour Avg Dissolved Oxygen	mg/L	6.8	ambient average
24-Hour Min Dissolved Oxygen	mg/L	5	TCEQ standard
Benzene	ug/L	0.1	method detection limit
Conductivity	uS/cm	550	ambient average
Ethylbenzene	ug/L	0.1	method detection limit
Max pH	-	8.5	ambient average
Min pH	-	7	ambient average
Oil and Grease	mg/L	0.4	method detection limit
Toluene	ug/L	0.3	method detection limit
Total Disolved Solids	mg/L	350	estimated from conductivity
Total PAH	mg/L	0.013	method detection limit
Total Petroleum Hydrocarbons	mg/L	1	ambient average
Total Suspended Solids	mg/L	2	ambient average
Xylene	ug/L	0.4	method detection limit

Table 3 WPD Recommended Discharge Limitations for Rattan Creek			
Parameter	**Unit**	**Standard**	**Reasoning**
24-Hour Avg Dissolved Oxygen	mg/L	10.75	ambient average
24-Hour Min Dissolved Oxygen	mg/L	5	TCEQ standard
Benzene	ug/L	0.1	method detection limit
Conductivity	uS/cm	599	ambient average
Ethylbenzene	ug/L	0.1	method detection limit
Max pH	-	8.7	ambient average
Min pH	-	6.7	ambient average
Oil and Grease	mg/L	0.4	method detection limit
Toluene	ug/L	0.3	method detection limit
Total Disolved Solids	mg/L	390	estimated from conductivity
Total PAH	mg/L	0.013	method detection limit
Total Petroleum Hydrocarbons	mg/L	0.2	ambient average
Total Suspended Solids	mg/L	2.1	ambient average
Xylene	ug/L	0.4	method detection limit

The Austin Water Utility (AWU) Special Services Division issues temporary discharge permits for construction projects to discharge to the City of Austin sanitary sewer system. Preliminary discussions with AWU indicate discharge limitations of 200 mg/L TSS and 15 mg/L TPH are to be expected.

5. Shaft and Tunnel Construction Water Treatment and Discharge Options

There are three options for discharge of treated tunnel and shaft construction water: (1) surface water; (2) sanitary sewer; and (3) reuse or recycling. On site retention and disposal to remote locations is not feasible due to the quantity of water to be managed. The ability to reuse or recycle any treated water is also limited. The variability of flow quantities and relatively short duration that water is available creates a reliability problem and difficult to arrange for an end user. The contractor could reuse a portion of the treated water for dust control and irrigation at the WTP4 and JR sites, but due to the quantity of water, other discharge locations need to be identified.

Typically for tunneling projects of this type, shaft and tunnel construction water is treated with settling tanks and oil skimming devices, then discharged into local surface waters. However, the EC Team has expressed concern regarding the discharge of construction water to Bull and Rattan Creeks given the potential for high total dissolved solids (TDS) and mobile phosphate levels present in the groundwater and their possible impact on creek habitat, if the water is not treated to background levels in the creek. In addition, the estimated maximum flows from the tunnel construction may overwhelm creek base flows. Therefore, the preferred option for discharge is to the COA sanitary sewer system.

Black & Veatch performed investigations of the available sewer capacities in the vicinity of the JR and FPA shaft locations. This section summarizes the sanitary sewer investigations, treatment and discharge options at each shaft site, and recommendations. The following attachments are included at the end of this technical memorandum:

- Attachment 1 – Table TM6-1 contains a summary of the shaft and tunnel construction water treatment and discharge options, including estimated monthly operating costs
- Attachment 2 – Figure TM6-1 (Four Points Area Shaft Site Plan) and Figure TM6-2 (Jollyville Reservoir Shaft Site Plan) showing the existing sanitary sewers and recommended actions
- Attachment 3 – Operating data for the Four Points Center lift station (#123) and calculations
- Attachment 4 - Calculations for the sanitary sewer evaluation at the JR shaft area
- Attachment 5 – October 2010 corrosivity testing results from five piezometers, including TDS
- Attachment 6 – February 3, 2011 and March 9, 2011 e-mails from AWU accepting Black & Veatch's recommendations and providing guidance regarding the permit process for discharge to the sanitary sewers.

A discussion of the groundwater inflows to be handled is provided in TM No. 5 (Groundwater Inflow Mitigation Plan). Two groundwater inflow conditions have to be handled during tunnel construction: 1) flush inflow, and 2) steady-state inflow. The total maximum flow from the tunnel will be the sum of the steady-state inflow and any flush inflow that is encountered. Inflows were estimated for each shaft construction and along the following reaches of the tunnel:

- Reach 1 – From WTP4 shaft (R-1) to FPA shaft (W-1)
- Reach 2 – From FPA shaft (W-1) to Parks and Recreation Department (PARD) shaft (R-2)
- Reach 3 – From PARD shaft (R-2) to JR shaft (W-2)

The estimated inflows from TM No. 5 are provided below and represent the maximum flows for unmitigated conditions. The contract documents will include mitigation measures to decrease the groundwater inflow into the tunnels and volume of water to be treated and discharged. The unmitigated flows will exist for a period of time until the mitigation measures are implemented. The tunneling contractor will need to be prepared to handle these maximum flows.

5.1 Identification and Evaluation of Discharge Options

5.1.1 Retrieval Shaft (R-1) at Water Treatment Plant #4 (WTP4)

Only shaft construction inflow water will need to be handled at the WTP4 site. Tunnel inflow water from Reach 1 will be treated and discharged at the FPA shaft site. The maximum expected inflow of 80 gpm for construction of the WTP4 shaft would be treated and discharged to a tributary flowing to Lake Travis, similar to the approach and requirements established for the WTP4 raw water tunnel construction. For the raw water tunnel and related construction, all water is required to be treated to achieve a water quality with particles no larger than 30 microns in diameter prior to discharge, if the on-site water quality and detention ponds are not utilized.

The nearest existing sanitary sewer is located along FM 620, approximately 4,000 feet from the shaft. Due to the relatively short duration that construction water needs to be handled at this location, a treatment system adjacent to the shaft would be more cost effective than installation of a temporary discharge line to the existing sanitary sewer along FM 620.

5.1.2 Working Shaft (W-1) at the Four Points Area (FPA) Site

The estimated inflows for the shaft and tunnel construction to be handled at this site are:

- Shaft Construction: 80 gpm
- Tunnel Construction (Reaches 1 and 2):
    - Steady-State Discharge: 407 gpm
    - Flush Flow Discharge: 725 gpm

- Total Maximum Flow (Steady-State and Flush): 1,132 gpm

The following sanitary sewer options were considered in the area of the site (refer to Figure TM6-1):

1. A 30" sewer runs to the southeast along FM 2222, starting at the intersection of River Place Blvd. and FM 2222. All force mains from the existing lift stations in the area discharge into a junction chamber where the 30" line originates. The 30" line continues along RM 2222 where other contributing areas are connected and increases in size to 36" downstream of MH 5802. The controlling pipes appear to be a 30" @ 1.06% slope followed by a 36" line @ 0.4% slope. These sewers have roughly the same capacities, 19,000 gpm when flowing full, and 15,200 gpm when flowing 80% full (COA maximum). The service area to this point in the system is large, and includes WTP4. The system should have sufficient available capacity since the system was built for full development and the area has not yet been fully developed. In addition, sanitary and other waste flows from WTP4 will not be realized until after tunnel construction is complete. AWU has orally informed Black & Veatch that sufficient capacity is available for discharge of the shaft and tunnel construction water at this location. Water would be pumped to the junction chamber in a temporary discharge pipe. Refer to Figure TM6-1 for three alternate routes. The recommended route will be determined after discussions with the City project management team and the owners of the 3M property.

2. A 12" gravity sewer line runs to the northwest along Four Points Drive. The 12" line flows to the Four Points Center lift station (#123) approximately 2,000 feet to the north, which serves a large area including the shopping centers around FM 620 and FM 2222. The lift station contains one duty pump and a standby pump, each with a capacity of 1,080 gpm; thus, the discharge of construction water would be limited to the firm capacity of the lift station of 1,080 gpm, less the existing peak flow in the gravity lines feeding the lift station. To conduct a field investigation and estimate the existing average, peak dry and wet weather flows from this large service area (based on the number of businesses and type of business, and the number of residences) would be extensive and the results may not be very accurate. As an alternative means of estimating the available capacity, lift station operation data was obtained

from AWU and are included in Attachment 3. Evaluating the most recent data (November 1, 2009 through November 30, 2010) reveals the following:

- The average pump run time was approximately 2 hours per day. For the maximum month (July 2010), the average pump run time was approximately 3.25 hours per day.
- Only about 29% of the lift station's firm pumping capacity is needed for the estimated current peak dry weather flow, and about 41% of the LS firm pumping capacity is needed for the estimated current peak wet weather flow.
- The available capacity for construction water is calculated to be 763 gpm during dry weather, and 642 gpm for peak wet weather.

Refer to Attachment 3 for the supporting calculations. Connection to the 12" gravity sewer and relying on the existing lift station would not be adequate for the maximum construction flows expected.

3. Connection to the 12" gravity sewer and upgrading the lift station pumps would increase the capacity. Increasing the capacity of the lift station could be accomplished as follows:

   - Contractor or AWU could provide a spare, ready to install pump for the lift station. Both of the installed pumps could then be operated simultaneously during a peak flow event and the "firm pumping capacity" of the lift station would be increased. It is estimated that with both pumps running together, a total discharge of approximately 1,500 gpm could be achieved. If one of the pumps were to fail, the spare pump would then be installed with minimal downtime. The capacity with both pumps running would need to be verified with as-built data on the pumps (pump curves) and force main system.
   - Contractor or AWU could provide temporary standby pumps on the surface at the lift station site, which would be operated if a lift station pump failed.

   The force main from the lift station is an 8" pipe and is routed southeast along Four Points Drive and then south along River Place Blvd., past the shaft site entrance to the previously mentioned existing junction chamber at the intersection of River Place Blvd. and FM 2222. The capacity of the 8" forcemain is approximately 1,500 gpm

(same capacity of both pumps running together). The velocity in the force main would be slightly under 10 feet per second at this flow rate.

This option would accommodate the total maximum flows expected; however, there would be the potential for overloading the lift station during wet periods, and causing backups in the gravity system.

4. In lieu of connecting to the 12" gravity sewer, the existing force main could be tapped with a temporary pressure pipe from the construction water handling system. This option would eliminate concerns with overloading the lift station; however, the available capacity while the lift station is running would be limited to 763 gpm during peak dry weather flows, and 642 gpm for peak wet weather flows as described in Option 2, but for most of the time, the full capacity of the forcemain could be used (1,500 gpm).

Based on preliminary discussions with AWU and their industrial pre-treatment staff, two sedimentation tanks in series equipped with oil skimming devices would be acceptable for discharge to the sanitary sewer. A third tank for additional capacity or equalization would be prudent. These types of systems typically achieve an effluent quality of approximately 200 mg/L of total suspended solids (TSS) and less than 15 mg/L of oil and grease. There would no discharge fees, unless the water contained excessive amounts of contaminants to classify it as extra strength waste.

5.1.3 Retrieval Shaft (R-2) at the Parks and Recreation Department (PARD) Site

Only shaft construction inflow water will need to be handled at the PARD site. The expected steady-state inflow of 50 gpm will be treated and discharged to the existing sanitary sewer.

There is an existing 15" gravity sanitary sewer that crosses the northwest corner of the site and flows to an existing lift station on the north side of Spicewood Springs Road. A new manhole would be installed to allow for discharge into the system. According to discussions with AWU, sufficient capacity exists in this line for shaft construction discharges. A pre-treatment system would be required as described above.

5.1.4 Working Shaft (W-2) at the Jollyville Reservoir Site

The estimated inflows for the shaft and tunnel construction to be handled at this site are:

- Shaft Construction: 200 gpm (subject to change)
- Tunnel Construction (Reach 3):
  - Steady-State Discharge: 484 gpm
  - Flush Flow Discharge: 970 gpm
  - Total Flow (Steady-State and Flush): 1,454 gpm

Treatment and discharge of all construction water to Rattan Creek via the existing storm sewer system along McNeil Road would be more costly compared to treatment and discharge to the sanitary sewer system due to the discharge requirements. The Rattan Creek standards provided by the EC Team included the following discharge limits for key parameters: 390 mg/L TDS, 2.1 mg/L TSS, and 0.2 mg/L Total Petroleum Hydrocarbons. A packaged water treatment system consisting of oil removal, clarification, and membrane filtration or reverse osmosis equipment may be required to achieve the desired water quality. There would be extensive operation and maintenance requirements, including certified operators and disposal of high strength waste streams.

The following sanitary sewer options were considered in the area of the site (refer to Figure TM6-2):

An 8" gravity sewer line runs along McNeill Road just to the north, a 12" sewer line runs along HWY 183 to the southeast of the site, and a 24" sewer line runs along the opposite side of HYW 183 from the site to the southeast. Black & Veatch performed a field survey of the existing businesses and residences, as well as a review of the COA gridmaps to determine the existing sewer flows within the 8" and 12" lines. Using the COA Utilities Criteria Manual Section 2.9.4, and as-built drawings of existing utilities, the existing capacities and flows were determined to be:

	HWY 183 12" Sewer	McNeill 8" Sewer
Peak Wet Weather Flow, gpm	21	4
Capacity (100% full), gpm	722	565
Available Capacity (100%), gpm	701	561
Capacity (85% full), gpm	614	480
Available Capacity (85%), gpm	592	476

The supporting calculations for these flows are included in Attachment 4. The McNeill 8" sewer flows to the northeast to a lift station approximately 3,000 feet down McNeill Road. The lift station contains one duty pump and a standby pump, each with a capacity of 700 gpm; thus, the discharge of construction water would be limited to the firm capacity of the lift station of 700 gpm, less the existing peak wet weather flow in the gravity lines feeding the lift station. However, the limiting section of the line was determined to be the 8" line segment immediately adjacent to the reservoir site.

The HWY 183 12" sewer flows southeast, from its starting point at a manhole approximately 300 feet from the site boundary, into a 16" main approximately 1,250 feet to the southeast, and thence into the COA collection system. The limiting section of this line was determined to be a section of the 12" sewer constructed at minimum slope of 0.2% just southeast of the site. The Black & Veatch team requests that AWU consider allowing the available full capacity of the sewers used to determine available capacity. It is our understanding that the 85% calculation required by AWU allows for reserve capacity for unanticipated future flows. After construction, the reserve capacity would be available.

An analysis of the capacity within the 24" gravity sewer across HWY 183 from the reservoir site has not been performed. Assuming capacity is available, a temporary HDPE discharge line could be installed beneath the highway (using directional drilling techniques); however, additional surveying and engineering work would be required to design the line. Approval from TxDOT would also be necessary.

Utilizing the McNeill 8" and HWY 183 12" sewers would be sufficient for the steady-state flows and most of the estimated flush flows. There is the possibility that during moderate and severe storm events, the sewers may experience higher flows than the calculated peak wet weather flows resulting in sewer backups. Consideration should be given to discharge of flows to Rattan Creek via the existing storm sewer system during storm events. A higher level of treatment can be provided consisting of filtration (to 50 mg/L of TSS, or no particles greater than 30 microns) and carbon adsorption. No treatment for total dissolved solids (TDS) is proposed. Refer to Attachment 5 for background levels of TDS in the groundwater from the peizometers.

If actual flush flows do exceed the available capacities of 8" and 12" sewers during dry weather periods, discharge to Rattan Creek would also be necessary

5.2 Recommendations

Recommended actions are summarized below:

1. At the WTP4 shaft (R-1), it is recommended that temporary onsite treatment be provided, and shaft construction water be discharged to a tributary of Lake Travis via the stormwater management ponds at the WTP4 site.

2. At the Four Points Area shaft (W-1), it is recommended that discharge up to 2,000 gpm be allowed into the 30" sewer junction box at the intersection of FM 2222 and River Place Blvd. This amount will provide sufficient capacity to receive the steady-state flows, as well as any peaks resulting from possible flush flows. Utilizing the sanitary sewer system for all discharges at this location should alleviate the concerns about contamination to the headwaters of Bull Creek, and possible adverse effects to endangered species and the Jollyville Plateau Salamander. Figure TM6-1 shows three possible routes for a temporary discharge line to be installed from the shaft location to the existing junction box. Selection of a route is contingent on agreements with TxDOT, 3M, or COA for temporary installation of the discharge line. This is currently being pursued.

3. At the PARD shaft (R-2), it is recommended that temporary onsite treatment be provided and a temporary manhole be constructed on the existing 15" line for discharge of water during construction of the working shaft.

4. At the JR shaft (W-1), it is recommended that a discharge of 590 gpm into the 12" sanitary sewer along HWY 183 be allowed as shown on Figure TM6-2. A temporary discharge pipe would be installed using directional drilling techniques within the existing easement along the front of the two lots between the reservoir site and the first manhole on the line. This would provide sufficient capacity to handle all steady-state discharges from the tunnel construction. During wet weather events, and when flush flows are encountered, the contractor should be allowed to discharge into the existing 30" storm sewer along the northern edge of the site. This system flows into Rattan Creek. During these occurrences, the contractor would be required to use filtration, in addition to sedimentation to reduce the loading to the creek. Alternatively, additional capacity is available in the 8" sanitary sewer along McNeil. This line could take a portion of flush flow, it addition to steady-state discharges, however some additional capacity would be required. Utilization of the existing

storm sewer would be preferred, and it should be noted that the level of pollutants within the discharge stream of shaft and tunnel construction water after treatment would probably be substantially less than runoff from streets and parking lots in the area during a storm event. The 24" sanitary sewer across HWY 183 would likely have sufficient capacity for all flows, however it would require additional design time for installation of temporary pipe across the ROW of HWY 183 and would require TxDOT approval. This option could be pursued further upon request.

5. The need to clean and televise the existing sanitary sewer lines prior to and after tunnel construction should be reviewed and conducted by AWU, as necessary.

Following submittal of the first draft of this memorandum dated February 2, 2011 to the City project management team, AWU responded with their concurrence of the findings and recommendations (refer to Joe Hoepken's e-mail dated February 3, 2011 included in Attachment 6). In addition, the Construction Manager-at-Risk or the tunneling contractor will be responsible for acquiring the discharge permit(s) from AWU's Special Services Division (refer to Joe Hoepken's e-mail dated March 9, 2011 included in Attachment 6).

The Contract Documents will be prepared to show installation of temporary discharge lines to the existing sanitary and storm sewers and corresponding capacity limitations and requirements. At the Four Points Area shaft (W-1), a discharge line will be routed along the west side of River Place Blvd within COA ROW into the 30" sewer junction box at the intersection of RM 2222 and River Place Blvd. At the PARD shaft (R-2), a service connection will be installed to the existing 15" line. At the JR shaft (W-1), a discharge line will be routed to the existing 12" line within TxDOT ROW as the former water and wastewater easements were taken when HWY 183 was expanded. In addition, temporary discharge lines will be routed to the existing 8" sanitary sewer and storm sewer along the north side of the site. If treated construction water flows exceed the capacities of the 8" and 12" sanitary sewers, treated water will be diverted to the storm sewer with the following conditions, as outlined in the final Environmental Commissioning Consensus document:

- All discharges will be pretreated for sediment (30 micron filter);
- Tunnel water with possible hydrocarbon contamination due to spills or visible sheen will be treated through a charcoal filter;
- Storm sewer or surface drainage discharge will be for a maximum of 7 calendar days in any 30 day period;

- Contractor must discharge to the sanitary sewer if the capacity is available;
- Discharging to the storm sewer or stream for more than 7 days during any 30 day period requires approval by the EC Lead or Coordinator.

ATTACHMENT 1

Table TM6 - 1 Shaft and Tunnel Construction Water Treatment and Disposal Summary

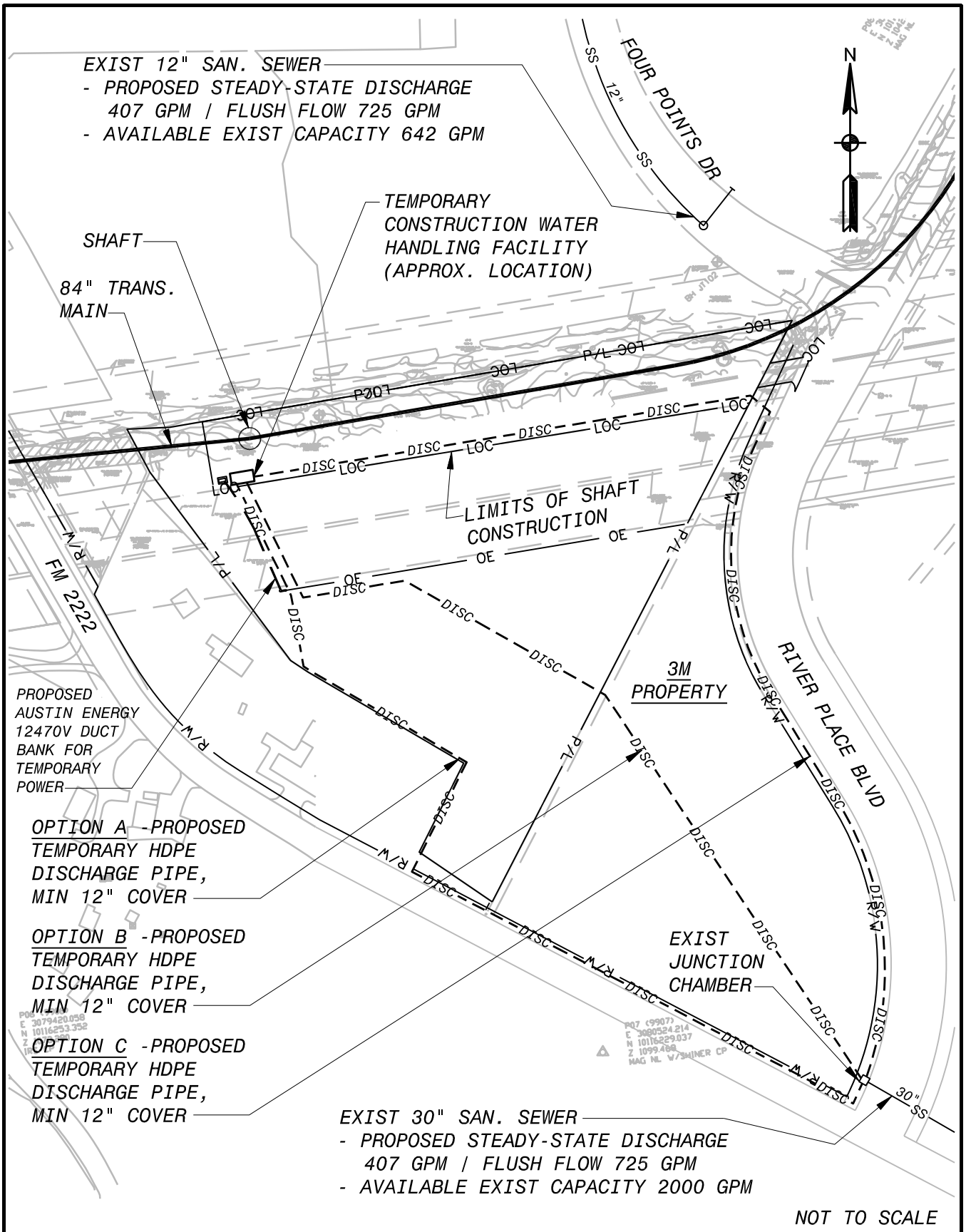
Shaft	Retrieval Shaft (R-1) at Water Treatment Plant 4 (WTP4)	Working Shaft (W-1) at Four Points Area (FPA)		Retrieval Shaft (R-2) at Parks and Recreation Department (PARD)		Working Shaft (W-2) at Jollyville Reservoir (JR)		
Estimated Shaft Construction Inflow (while being mitigated)	80 gpm	80 gpm		50 gpm		200 gpm (subject to change)		
Estimated Tunnel Construction Inflow (Flush/Steady-State/Total Max)[1]	N/A	725 gpm/407 gpm/1132 gpm		N/A		970 gpm/484 gpm/1454 gpm		
Tunnel Reach[2]	N/A	1 and 2		N/A		3		
Discharge Location	Lake Travis	Bull Creek	Sanitary Sewer	Bull Creek	Sanitary Sewer	Rattan Creek (All)	Sanitary Sewer	Rattan Creek (Partial)
Description	Shaft construction water would be routed via the effluent structure of Detention Pond 3 currently under construction on the WTP4 site, to Lake Travis	Construction water would be routed to the existing stormwater conveyance system, which flows into the upper reaches of Bull Creek	Construction water would be pumped to the existing 30" sanitary sewer junction box (COA GIS #236483) which starts at the intersection of River Place Blvd. and FM 2222.	Shaft construction water would flow by overland flow into the tributary of Bull Creek adjacent to the shaft location	Shaft construction water would be introduced to the system via a temporary manhole constructed on the existing 15" sanitary sewer line that crosses the southwest corner of the PARD site	Construction water would be introduced to the existing stormwater conveyance system adjacent to the reservoir site, which flows into Rattan Creek.	Construction water would be piped into a manhole (COA GIS #114232) on the existing 12" sanitary sewer system located just to the southeast of the reservoir site	Flow to Rattan Creek would only occur during storm events, or if steady state flows exceed capacity of sanitary system
Discharge Requirements	Max 30 um particle size TPH = 1 mg/L Benzene = 0.1 ug/l Ethylbenzene = 0.1 ug/l Toluene = 0.3 ug/l Xylene = 0.4 ug/l pH 6.5 - 9.0	TSS = 2 mg/L TPH = 1 mg/L TDS = 350 mg/L Cond. = 550 uS/cm	TSS = 200 mg/L TPH = 15 mg/L	TSS = 2 mg/L TPH = 1 mg/L TDS = 350 mg/L Cond. = 550 uS/cm	TSS = 200 mg/L TPH = 15 mg/L	TSS = 2.1 mg/L TPH = 0.2 mg/L TDS = 390 mg/L Cond. = 600 uS/cm	TSS = 200 mg/L TPH = 15 mg/L	Max 30 um particle size TPH = 1 mg/L Benzene = 0.1 ug/l Ethylbenzene = 0.1 ug/l Toluene = 0.3 ug/l Xylene = 0.4 ug/l pH 6.5 - 9.0
Permit Requirements	WTP4 Stormwater Permit	TXR830000	AWU Discharge Permit	TXR830000	AWU Discharge Permit	TXR830000	AWU Discharge Permit	TXR830000
Potential Treatment Technology Required	Sedimentation tanks in series, equipped with oil skimmers, followed by filtration and carbon adsorption	High-rate clarification water treatment system, equipped with oil skimmers, followed by reverse osmosis	Sedimentation tanks in series, equipped with oil skimmers	High-rate clarification water treatment system, equipped with oil skimmers, followed by reverse osmosis	Sedimentation tanks in series, equipped with oil skimmers	High-rate clarification water treatment system, equipped with oil skimmers, followed by reverse osmosis	Sedimentation tanks in series, equipped with oil skimmers	Sedimentation tanks in series, equipped with oil skimmers, followed by filtration and carbon adsorption
Available Suppliers	Rain-for-Rent, Baker Tank, Deltank Filtration	US FILTER/Siemens, Parkson	Rain-for-Rent, Baker Tank, Deltank Filtration	US FILTER/Siemens, Parkson	Rain-for-Rent, Baker Tank, Deltank Filtration	US FILTER/Siemens, Parkson	Rain-for-Rent, Baker Tank, Deltank Filtration	Rain-for-Rent, Baker Tank, Deltank Filtration
Estimated Operating Cost (per Month)	$60,000	$300,000	$110,000	$150,000	$60,000	$350,000	$140,000	$170,000
Advantages	Low environmental impact to Lake Travis, conservation of water	Conservation of water, proximity to stormwater conveyance system	Low environmental impact, ease of treatment, excess capacity available in sanitary system	Conservation of water, proximity to receiving stream	Low environmental impact, ease of treatment, excess capacity available in sanitary system	Available capacity, conservation of water, proximity to existing storm sewer	Low environmental impact, ease of treatment	Ease of treatment, proximity to storm sewer
Disadvantages	Use of filtration and carbon adsorption systems make operation more complex and costly	Costly, extensive O&M requirements requiring trained operators, high strength waste stream from RO system requiring disposal	Additional loading to sanitary sewer system, contractor would be required to install temporary pipe and pump water to junction box, easements or approvals required for options on TxDOT and 3M	Costly, extensive O&M requirements requiring trained operators, high strength waste stream from RO system requiring disposal	Lift station downstream of shaft site has overflowed in the past, discharge would require construction of new manhole on existing line	Costly, extensive O&M requirements requiring trained operators, high strength waste stream from RO system requiring disposal	Temporary pipe will be required across private property to convey water to sanitary system	Possible environmental impact

NOTE 1 - TUNNEL CONSTRUCTION WATER DISCHARGES ARE ANTICIPATED TO BE AT OR BELOW STEADY-STATE INFLOW ESTIMATES DURING MOST OF DURATION OF CONSTRUCTION. HOWEVER, DUE TO POTENTIAL TO ENCOUNTER FLUSH FLOWS, IT IS PROPOSED THAT CONTRACTOR BE PREPARED TO HANDLE UP TO MAXIMUM BASELINE QUANTITIES THAT ARE PRESENTED ABOVE (STEADY STATE PLUS FLUSH FLOW) AND INCLUDED IN THE GEOTECHNICAL BASELINE REPORT.

NOTE 2 - SEE FIGURE ON NEXT PAGE FOR REACHES.


R-1 AT WTP4
REACH 1
W-1 AT FPA
REACH 2
R-2 AT PARD
REACH 3
W-2 AT JR

ATTACHMENT 2



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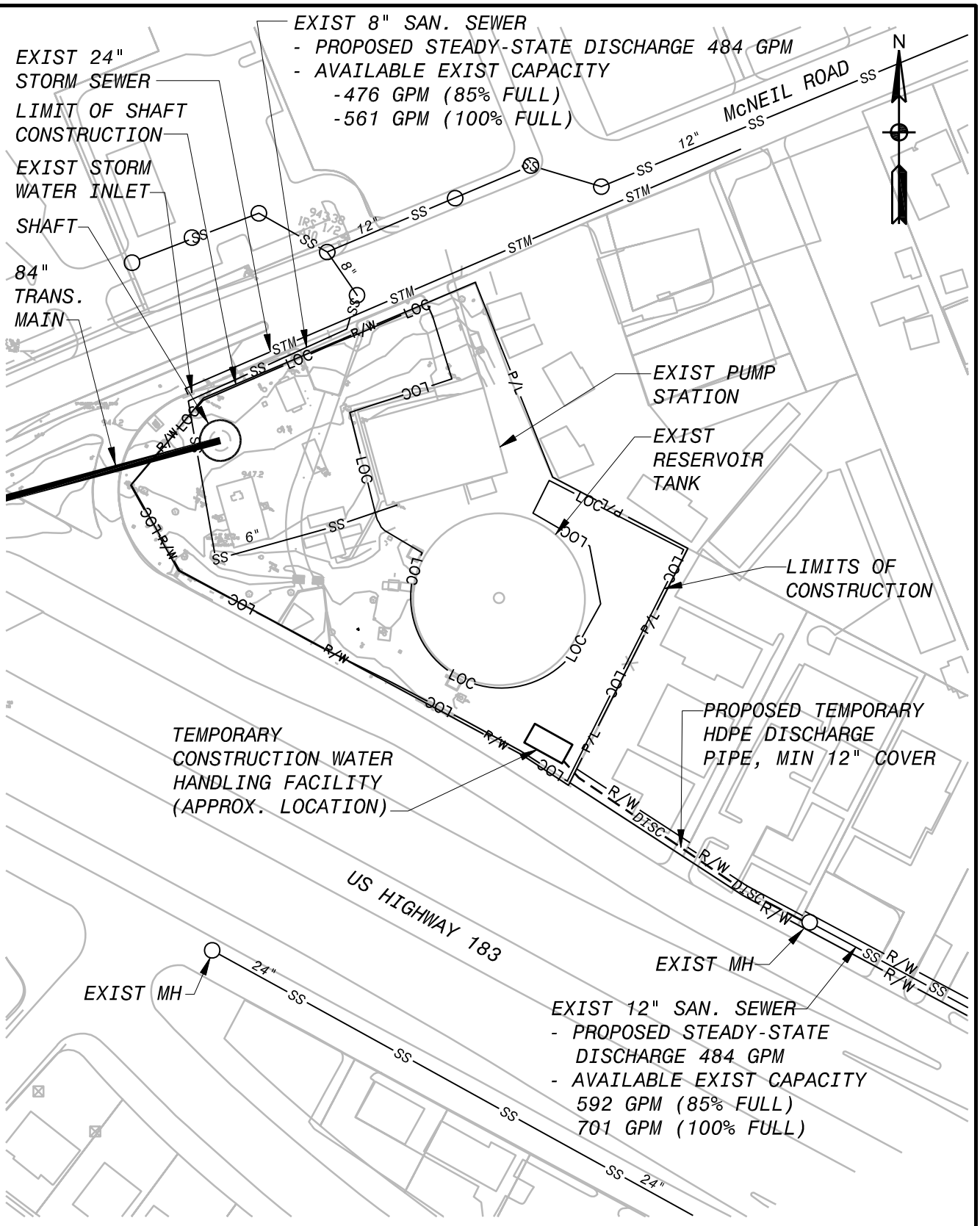
BLACK & VEATCH Corporation PROJECT 167760	CITY OF AUSTIN, TEXAS JOLLYVILLE TRANSMISSION MAIN FROM WATER TREATMENT PLANT NO.4	DRAFT
	FOUR POINTS AREA SHAFT SITE	FIGURE TM6-1

A30005
FA30005



NOT TO SCALE

BLACK & VEATCH Corporation PROJECT 167760	CITY OF AUSTIN, TEXAS JOLLYVILLE TRANSMISSION MAIN FROM WATER TREATMENT PLANT NO.4	DRAFT
	JOLLYVILLE RESERVOIR SHAFT SITE	FIGURE TM6-2

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A30005
FA30005

ATTACHMENT 3

StaNo	StationName	PumpNo	Date	RunHours	GallonsPumped	Pump rate in calc	PumpNo	RunHours	GallonsPumped	Total Run Hours	Total Gal Pumped	
LS-123	***Four Points Ctr***											
		1	12/1/2009	19.20	1244160.00	1080	2	18.00	1166400.00	37.20	2,410,560	1080
		1	1/1/2010	19.50	1263600.00		2	18.50	1198800.00	38.00	2,462,400	
		1	2/1/2010	19.60	1270080.00		2	18.70	1211760.00	38.30	2,481,840	
		1	3/1/2010	25.70	1665360.00		2	24.50	1587600.00	50.20	3,252,960	
		1	4/1/2010	25.40	1645920.00		2	24.60	1594080.00	50.00	3,240,000	
		1	5/1/2010	26.40	1710720.00		2	25.10	1626480.00	51.50	3,337,200	
		1	6/1/2010	34.10	2209680.00		2	32.20	2086560.00	66.30	4,296,240	
		1	7/1/2010	22.50	1458000.00		2	21.30	1380240.00	43.80	2,838,240	
		1	8/1/2010	51.30	3324240.00		2	46.00	2980800.00	97.30	6,305,040	
		1	9/1/2010	44.10	2857680.00		2	34.60	2242080.00	78.70	5,099,760	
		1	10/1/2010	31.80	2060640.00		2	33.60	2177280.00	65.40	4,237,920	
		1	11/1/2010	37.90	2455920.00		2	36.50	2365200.00	74.40	4,821,120	
		1	12/1/2010	39.10	2533680.00		2	38.00	2462400.00	77.10	4,996,080	
									Totals for 2 pumps	768.20	49,779,360	gal
									Averages per day over 13 months	1.94	126,024	gpd
									Totals for 2 pumps in max month (July 2010)	97.30	6,305,040	gal
									Averages per day for max flow month (July 2010)	3.24	210,168	gpd
									Average flow into LS wet well over 13 months		87.52	gpm
									Estimated peaking factor for DWF using formula		3.62	
									Peak DWF		317	gpm
									Estimated peak WWF flow using overall peaking factor of 5.0		437.58	gpm
									Equivalent acreage based on 750 gpd/acre for I/I allowance		232	acres
									Firm pumping capacity of LS		1080	gpm
									% of firm capacity for estimated peak WWF		41%	
									% of firm capacity for estimated peak DWF		29%	
									Capacity available in LS during dry weather for transporting ground water from Four Points Shaft		763	gpm
									Capacity available in LS during wet weathe event for transporting ground water from Four Points Shaft		642	gpm

ATTACHMENT 4

Project No.	167760		
Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	1 of	9	

Based on the City of Austin shapefiles: 2003 building footprints, addresses, lots & row (basemap), a map of the Jollyville Reservoir area was made and attached for this calculation.
Note that not all the lightlighted, brown bulidings on the map were used for the calculation.
Adjustments for accounting current buildings were made for the calculation (if it was deemed necessary).
The calculations were performed according to the City of Austin's:
Living Unit Equivalent (LUE) Guidance Document & Utilities Criteria Manual (UCM 2.9.4 Wastewater)
Note that the I/I calculation below included an entired lot area for each lot where the building on the lot contributed sewer flows. This approach is conservative numerically for computing I/I flows.

1. Research Blvd (Hwy 183)

The limiting pipe for Research Blvd (Hwy 183) was identified to be the 12" pipe at 12621 Research Blvd. The limiting pipe has a slope of -0.20%. The files used are Grid map H36 and as-built plan 93-0009.

A. Building Floor Area and Sewer FLow

Address	Building Type	Floor SF*	LUE Conversion	LUE	SFR (gpd)	Average Flow (gpd)	Cumulative Avg Flow (gpm)	Peaking Factor**	Cum. Peak Dry Flow (gpm)
12741	Office	3272	3000	1.09	245	267.2	0.19		
12741	Office	4019	3000	1.34	245	328.2	0.41		
-	Office	4976	3000	1.66	245	406.4	0.70		
-	Office	7239	3000	2.41	245	591.2	1.11		
12731	Office	1653	3000	0.55	245	135.0	1.20		
12731	Office	3365	3000	1.12	245	274.8	1.39		
12731	Office	2246	3000	0.75	245	183.4	1.52		
12731	Office	8368	3000	2.79	245	683.4	1.99		
12707	Office	7140	3000	2.38	245	583.1	2.40		
12705	Office	1286	3000	0.43	245	105.0	2.47		
12703	Office	4910	3000	1.64	245	401.0	2.75		
12701	Office	4023	3000	1.34	245	328.5	2.98		
12687	Office	3104	3000	1.03	245	253.5	3.15		
12675	Church	150	70	2.14	245	525.0	3.52		
12627	Office	2237	3000	0.75	245	182.7	3.64		
12621	Retail: Jiffy Lube	1892	1660	1.14	245	279.2	3.84	4.27	16.39

*A church is not measured by Floor SF but by Seats.
**Calculation of Peaking Factor:
$PF = (18+(0.0206 \times F)^{0.5}) / (4+(0.0206 \times F)^{0.5})$

Note that the Floor SF values have been adjusted for multiple floors.

Project No.	167760		
Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	2 of	9	

B. Lot Area and Infiltration/Inflow Flow

Address	Lot Area (SF)	Lot Area (Acre)	I/I Rate (g/d/acre)	I/I Flow (gpd)	
12741	34592	0.794	750	595.6	
-	38273	0.879	750	659.0	
12731	35827	0.822	750	616.9	
12707	28942	0.664	750	498.3	
12705	5141	0.118	750	88.5	
12703	28643	0.658	750	493.2	
12701	5482	0.126	750	94.4	
12687	36511	0.838	750	628.6	
12675	71145	1.633	750	1225.0	
12627	51504	1.182	750	886.8	
12621	35560	0.816	750	612.3	
-	19641	0.451	750	338.2	
12611	16141	0.371	750	277.9	
			Total =	7014.6	gpd
				4.87	gpm

C. Peak Wet Weater Flow for the Limiting Pipe

Peak Wet Weater Flow = Peak Dry Weater Flow + I/I Flow = 16.39 + 4.87 = 21.26 gpm

Project No.	167760		
Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	3 of	9	

D. Sewer Capacity

For the limiting pipe:

Peak Wet Weater Flow = 21.26 gpm

Length of Pipe = 324 ft

Slope of Pipe = -0.20 %

Upstream Invert Elevation = 939.54 ft
Downstream Invert Elevation = 938.88 ft

Downstream pipe of the limiting pipe:

Length of Pipe = 493.67 ft

Slope of Pipe = -1.22 %

Upstream Invert Elevation = 938.75 ft
Downstream Invert Elevation = 932.73 ft

StormNet, a commercial software of BOSS International, was used to perform the hydraulic analysis.
StormNet is a fully-hydrodynamic model that can analyze both simple and complex sanitary sewer systems.
A simple alignment model was created that consisted of pipes, manholes, and an outfall.
Roughness for the pipes was set at 0.013.

The modeling results show that the full flow of this limiting pipe is 721.73 gpm.
The input flow of 21.26 gpm for the limiting pipe takes 3% of the full flow at v = 0.92 ft/sec.
For 85% of the capacity of the pipe flowing full, there are 82% (85 - 3 = 82) of the capacity remaining.

The availalble capacity of this limiting pipe for Jollyville Reservoir site construction wastewater:

82% x 721.73 = 591.8 gpm with velocity = 2.5 ft/sec (for Q = 613.5 gpm)

The modeling output report is attached (JR-Research Blvd Modeling Report.pdf).

2. McNeil Drive

The 12" limiting pipe for McNeil Drive appears to be at 6909 McNeil Drive.
This pipe has a slope of -0.25%. The files used are Grid map H36 and as-built plan 88-0015.
The 8" line is located at upstream of the 12" line. The 8" limiting pipe has a slope of -1.08%
The files used are Grid map H36 and as-built plan 95-0008.

A. Building Floor Area and Sewer Flow for the 12" Limiting Pipe

Address	Building Type	Floor SF***	LUE Conversion	LUE	SFR (gpd)	Average Flow (gpd)	Cumulative Avg Flow (gpm)	Peaking Factor	Cum. Peak Dry Flow (gpm)
7410	Retail: Walgreen	16229	1660	9.78	245	2395.2	1.66		
7404	Restraurant	3345	200	16.73	245	4097.6	4.51		
7329	Warehouse	11020	4000	2.76	245	675.0	4.98		
7318a	Office	5000	3000	1.67	245	408.3	5.26		
7318b	Restraurant	3000	200	15.00	245	3675.0	7.81		
7311	Restraurant	6344	200	31.72	245	7771.4	13.21		
7309	Office	1290	3000	0.43	245	105.4	13.28		
7308	Retail: Auto Parts	12000	1660	7.23	245	1771.1	14.51		
7304	Office	11392	3000	3.80	245	930.3	15.16		
7301	Office	3110	3000	1.04	245	254.0	15.34		
7218	Office	12779	3000	4.26	245	1043.6	16.06		
7217	Office	3110	3000	1.04	245	254.0	16.24		
7215	Office	1720	3000	0.57	245	140.5	16.33		
7213	Office	3391	3000	1.13	245	276.9	16.53		
12015*	Office	1912	3000	0.64	245	156.1	16.64		
7212	Office	12894	3000	4.30	245	1053.0	17.37		
7208	Office	10660	3000	3.55	245	870.6	17.97		
7200	Office	1803	3000	0.60	245	147.2	18.07		
7207	Office	2813	3000	0.94	245	229.7	18.23		
12210**	Office	4007	3000	1.34	245	327.2	18.46		
12202**	Single Family	-	-	1	245	245.0	18.63		
12200**	Single Family	-	-	1	245	245.0	18.80		
12140**	Single Family	-	-	1	245	245.0	18.97		
12138**	Single Family	-	-	1	245	245.0	19.14		
12132**	Single Family	-	-	1	245	245.0	19.31		
12130**	Single Family	-	-	1	245	245.0	19.48		
12112*	Office	23670	3000	7.89	245	1933.1	20.82		
7119	Office	1626	3000	0.54	245	132.8	20.92		
7113	Office	1388	3000	0.46	245	113.4	20.99		
12211**	Single Family	-	-	1	245	245.0	21.16		
12207**	Single Family	-	-	1	245	245.0	21.33		
12205**	Single Family	-	-	1	245	245.0	21.50		
12203**	Single Family	-	-	1	245	245.0	21.67		
12137**	Single Family	-	-	1	245	245.0	21.84		

12135**	Single Family	-	-	1	245	245.0	22.01		
12133**	Single Family	-	-	1	245	245.0	22.19		
12131**	Single Family	-	-	1	245	245.0	22.36		
12117**	Single Family	-	-	1	245	245.0	22.53		
12115**	Single Family	-	-	1	245	245.0	22.70		
12214*	Office	1518	3000	0.51	245	124.0	22.78		
12212*	Office	1865	3000	0.62	245	152.3	22.89		
12210*	Office	1500	3000	0.50	245	122.5	22.97		
12208*	Office	779	3000	0.26	245	63.6	23.02		
12206*	Office	1391	3000	0.46	245	113.6	23.10		
12204*	Office	240	3000	0.08	245	19.6	23.11		
12200*	Office	1478	3000	0.49	245	120.7	23.19		
12128*	Office	1020	3000	0.34	245	83.3	23.25		
12201*	Single Family	-	-	1	245	245.0	23.42		
7114	Church	150	70	2.14	245	525.0	23.79		
7101	Office	2699	3000	0.90	245	220.4	23.94		
7001	Reail: Car Dealer	57815	1660	34.83	245	8532.9	29.86		
6917	Office	3418	3000	1.14	245	279.1	30.06		
6915	Office	3479	3000	1.16	245	284.1	30.26		
6914	Office	3047	3000	1.02	245	248.8	30.43		
6909	Animal Hospital	10	5	2.00	245	490.0	30.77	3.92	120.58

*Jekel Circle
**Pecan Street
***A church is not measured by Floor SF but by Seats. A hospital measured by beds.

Project No.	167760		
Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	6 of	9	

B. Lot Area and Infiltration/Inflow Flow for the 12" Limiting Pipe

Address	Lot Area (SF)	Lot Area (Acre)	I/I Rate (g/d/acre)	I/I Flow (gpd)
7410	127552	2.928	750	2196.2
7404	58947	1.353	750	1014.9
7329	139311	3.198	750	2398.6
7318	127075	2.917	750	2187.9
7311	76666	1.760	750	1320.0
7309	12827	0.294	750	220.9
7308	49333	1.133	750	849.4
7304	45774	1.051	750	788.1
7301	63440	1.456	750	1092.3
7218	46277	1.062	750	796.8
7217	15344	0.352	750	264.2
7215	15459	0.355	750	266.2
7213	16058	0.369	750	276.5
12015*	40166	0.922	750	691.6
7212	46335	1.064	750	797.8
7208	53146	1.220	750	915.1
7200	32693	0.751	750	562.9
7207	25795	0.592	750	444.1
7230/12210	25708	0.590	750	442.6
12202**	13165	0.302	750	226.7
12200**	12200	0.280	750	210.1
12140**	13607	0.312	750	234.3
12138**	13649	0.313	750	235.0
12132**	13682	0.314	750	235.6
12130**	14149	0.325	750	243.6
12112*	40500	0.930	750	697.3
7119	11568	0.266	750	199.2
7113	10914	0.251	750	187.9
12211**	13413	0.308	750	230.9
12207**	13469	0.309	750	231.9
12205**	13380	0.307	750	230.4
12203**	13326	0.306	750	229.4
12137**	13372	0.307	750	230.2
12135**	13315	0.306	750	229.3
12133**	13343	0.306	750	229.7
12131**	13287	0.305	750	228.8
12117**	13298	0.305	750	229.0
12115**	18093	0.415	750	311.5
12214*	13291	0.305	750	228.8

12212*	13418	0.308	750	231.0	
12210*	13421	0.308	750	231.1	
12208*	13363	0.307	750	230.1	
12206*	13475	0.309	750	232.0	
12204*	13346	0.306	750	229.8	
12200*	13391	0.307	750	230.6	
12128*	13488	0.310	750	232.2	
12201*	41701	0.957	750	718.0	
7114***	210000	4.821	750	3615.7	
7101	42577	0.977	750	733.1	
7001	327476	7.518	750	5638.4	
6917	37935	0.871	750	653.2	
6915	31642	0.726	750	544.8	
6914	59481	1.366	750	1024.1	
6909	49960	1.147	750	860.2	
			Total =	38010.0	gpd
				26.4	gpm

*Jekel Circle

**Pecan Street

***7714: Gateway Church was a new buidling for the 2003 data. Its area contributing to I/I was an estimate.

Project No.	167760		
Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	8 of	9	

C. Peak Wet Weater Flow for the 12" Limiting Pipe

Peak Wet Weater Flow = Peak Dry Weater Flow + I/I Flow = 120.58 + 26.4 = 146.98 gpm

D. Sewer Capacity for the 12" Limiting Pipe

For the limiting pipe:

Peak Wet Weater Flow = 146.98 gpm
Length of Pipe : 340 ft
Slope of Pipe = -0.25 %

Upstream Invert Elevation = 905.81 ft
Downstream Invert Elevation = 904.96 ft

Downstream Pipe:

Length of Pipe : 349 ft
Slope of Pipe = -0.53 %

Upstream Invert Elevation = 904.81 ft
Downstream Invert Elevation = 903.11 ft

The modeling results show that the full flow of this limiting pipe is 799.55 gpm.
The input flow of 146.98 gpm for the limiting pipe takes 18% of the full flow at v = 1.78 ft/sec.
For 85% of the capacity of the pipe flowing full, there are 67% (85 - 18 = 67) of the capacity remaining.

The availalble capacity of this limiting pipe for Jollyville Reservoir site construction wastewater:

67% x 799.55 = 535.7 gpm with velocity = 2.71 ft/sec (for Q = 679.62 gpm)

The modeling output report is attached (JR-McNeil Drive Modeling Report.pdf).

Subject	Jollyville Reservoir Area Sewer Capacity Calculation		
Computed by	Joseph Dong	Date	Jan. 2011
Checked by		Date	
Approved by		Date	
Sheet No.	9 of	9	

E. Building Floor Area and Sewer Flow for the 8" Limiting Pipe

Address	Building Type	Floor SF	LUE Conversion	LUE	SFR (gpd)	Average Flow (gpd)	Cumulative Avg Flow (gpm)	Peaking Factor	Cum. Peak Dry Flow (gpm)
7329	Warehouse	11020	4000	2.76	245	675.0	0.47	4.42	2.07

F. Lot Area and Infiltration/Inflow Flow for the 8" Limiting Pipe

Address	Lot Area (SF)	Lot Area (Acre)	I/I Rate (g/d/acre)	I/I Flow (gpd)	
7329	139311	3.198	750	2398.6	
				1.67	gpm

G. Peak Wet Weater Flow for the 8" Limiting Pipe

Peak Wet Weater Flow = Peak Dry Weater Flow + I/I Flow = 2.07 + 1.67 = 3.74 gpm

H. Sewer Capacity for the 8" Limiting Pipe

For the limiting pipe:

Peak Wet Weater Flow = 3.74 gpm
Length of Pipe : 71 ft
Slope of Pipe = -1.08 %

Upstream Invert Elevation = 934.80 ft
Downstream Invert Elevation = 934.03 ft

Downstream Pipe:

Length of Pipe : 110 ft
Slope of Pipe = -1.13 %

Upstream Invert Elevation = 933.93 ft
Downstream Invert Elevation = 932.69 ft

The modeling results show that the full flow of this limiting pipe is 564.83 gpm.
The input flow of 3.74 gpm for the limiting pipe takes 0.66% of the full flow at v = 1.02 ft/sec.
For 85% of the capacity of the pipe flowing full, there are 84.34% (85 - 0.66 = 84.34) of the capacity remaining.

The availalble capacity of this limiting pipe for Jollyville Reservoir site construction wastewater:

84.34% x 564.83 = 476.4 gpm with velocity = 3.77 ft/sec (for Q = 480.10 gpm)

The modeling output report is attached (JR-McNeil Drive Modeling Report 8in.pdf).

BOSS International StormNET® - Version 4.15.0 (Build 17034)

****
Analysis Options
****
Flow Units ..... GPM
Link Routing Method ..... Hydrodynamic
Pond Exfiltration..... None
Starting Date APR-06-2010 00:00:00
Ending Date APR-10-2010 20:00:00
Antecedent Dry Days 0.0
Report Time Step 00:10:00
Routing Time Step ..... 30.00 sec

****
Element Count
****
Number of rain gages 0
Number of subbasins 0
Number of nodes 4
Number of links 3
Number of pollutants 0
Number of land uses 0

****
Node Summary
****

Node ID	Element Type	Invert Elevation ft	Maximum Elev. ft	Ponded Area ft²	External Inflow
MH1	JUNCTION	939.54	950.00	0.00	Yes
MH2	JUNCTION	938.75	947.00	0.00	
MH3	JUNCTION	932.40	943.00	0.00	
Out	OUTFALL	932.00	933.33	0.00	

****
Link Summary
****

Link ID	From Node	To Node	Element Type	Length ft	Slope %	Manning's Roughness
exit	MH3	Out	CONDUIT	100.0	0.4000	0.0130
Pipe1	MH1	MH2	CONDUIT	324.0	0.2037	0.0130
Pipe2	MH2	MH3	CONDUIT	493.7	1.2194	0.0130

****
Cross Section Summary
****

Link ID	Shape	Depth/ Diameter ft	Width ft	No. of Barrels	Cross Sectional Area ft²	Full Flow Hydraulic Radius ft	Design Flow Capacity GPM
exit	CIRCULAR	1.33	1.33	1	1.40	0.33	2178.08
Pipe1	CIRCULAR	1.00	1.00	1	0.79	0.25	721.73
Pipe2	CIRCULAR	1.00	1.00	1	0.79	0.25	1765.85

********** Flow Routing Continuity **********	Volume acre-ft	Volume Mgallons
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000

Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.454	0.148
External Outflow	0.453	0.148
Surface Flooding	0.000	0.000
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.001	0.000
Continuity Error (%)	0.056	

****
Node Depth Summary
****

Node ID	Average Depth Attained ft	Maximum Depth Attained ft	Maximum HGL Attained ft	Time of Max Occurrence days	Time of Max Occurrence hh:mm	Total Flooded Volume acre-in	Total Time Flooded minutes	Retention Time hh:mm:ss
MH1	0.15	0.15	939.69	0	06:52	0	0	0:00:00
MH2	0.08	0.08	938.83	0	16:38	0	0	0:00:00
MH3	0.09	0.09	932.49	2	01:46	0	0	0:00:00
Out	0.09	0.09	932.09	0	08:39	0	0	0:00:00

****
Node Flow Summary
****

Node ID	Element Type	Maximum Lateral Inflow GPM	Peak Inflow GPM	Time of Peak Inflow Occurrence days	Time of Peak Inflow Occurrence hh:mm	Maximum Flooding Overflow GPM	Time of Peak Flooding Occurrence days	Time of Peak Flooding Occurrence hh:mm
MH1	JUNCTION	21.26	21.26	0	00:00	0.00		
MH2	JUNCTION	0.00	21.26	0	06:53	0.00		
MH3	JUNCTION	0.00	21.26	0	06:03	0.00		
Out	OUTFALL	0.00	21.26	0	08:39	0.00		

****
Outfall Loading Summary
****

Outfall Node ID	Flow Frequency (%)	Average Flow GPM	Peak Inflow GPM
Out	99.85	21.24	21.26
System	99.85	21.24	21.26

****
Link Flow Summary
****

Link ID	Element Type	Time of Peak Flow Occurrence days	Time of Peak Flow Occurrence hh:mm	Maximum Velocity Attained ft/sec	Length Factor	Peak Flow during Analysis GPM	Design Flow Capacity GPM	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Surcharged Minutes
exit	CONDUIT	0	08:39	1.09	1.00	21.26	2178.08	0.01	0.07	0
Pipe1	CONDUIT	0	06:53	0.92	1.00	21.26	721.73	0.03	0.12	0
Pipe2	CONDUIT	0	06:03	1.70	1.00	21.26	1765.85	0.01	0.08	0

****
Flow Classification Summary
****

Link	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
	--- Fraction of Time in Flow Class ----								
exit	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.63	0.0000
Pipe1	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.48	0.0000
Pipe2	0.00	0.00	0.00	0.00	0.00	0.00	1.00	1.08	0.0000

****
Time-Step Critical Elements
****
None

****
Highest Flow Instability Indexes
****
All links are stable.

****
Routing Time Step Summary
****
Minimum Time Step   :   30.00 sec
Average Time Step   :   30.00 sec
Maximum Time Step   :   30.00 sec
Percent in Steady State   :   0.00
Average Iterations per Step :   2.00

Analysis begun on: Mon Jan 24 11:03:47 2011
Analysis ended on: Mon Jan 24 11:03:48 2011
Total elapsed time: 00:00:01

BOSS International StormNET® - Version 4.15.0 (Build 17034)

****
Analysis Options
****
Flow Units ..... GPM
Link Routing Method ..... Hydrodynamic
Pond Exfiltration..... None
Starting Date APR-06-2010 00:00:00
Ending Date APR-10-2010 20:00:00
Antecedent Dry Days 0.0
Report Time Step 00:10:00
Routing Time Step ..... 30.00 sec

****
Element Count
****
Number of rain gages 0
Number of subbasins 0
Number of nodes 4
Number of links 3
Number of pollutants 0
Number of land uses 0

****
Node Summary
****

Node ID	Element Type	Invert Elevation ft	Maximum Elev. ft	Ponded Area ft²	External Inflow
MH1	JUNCTION	905.81	925.00	0.00	Yes
MH2	JUNCTION	904.81	918.00	0.00	
MH3	JUNCTION	903.10	910.00	0.00	
Out	OUTFALL	903.00	904.33	0.00	

****
Link Summary
****

Link ID	From Node	To Node	Element Type	Length ft	Slope %	Manning's Roughness
exit	MH3	Out	CONDUIT	100.0	0.1000	0.0130
Pipe1	MH1	MH2	CONDUIT	340.0	0.2500	0.0130
Pipe2	MH2	MH3	CONDUIT	349.0	0.4871	0.0130

****
Cross Section Summary
****

Link ID	Shape	Depth/ Diameter ft	Width ft	No. of Barrels	Cross Sectional Area ft²	Full Flow Hydraulic Radius ft	Design Flow Capacity GPM
exit	CIRCULAR	1.33	1.33	1	1.40	0.33	1089.04
Pipe1	CIRCULAR	1.00	1.00	1	0.79	0.25	799.55
Pipe2	CIRCULAR	1.00	1.00	1	0.79	0.25	1116.06

********** Flow Routing Continuity **********	Volume acre-ft	Volume Mgallons
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000

Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	3.139	1.023
External Outflow	3.136	1.022
Surface Flooding	0.000	0.000
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.004	0.001
Continuity Error (%)	0.007	

****
Node Depth Summary
****

Node ID	Average Depth Attained ft	Maximum Depth Attained ft	Maximum HGL Attained ft	Time of Max Occurrence days	Time of Max Occurrence hh:mm	Total Flooded Volume acre-in	Total Time Flooded minutes	Retention Time hh:mm:ss
MH1	0.33	0.33	906.14	0	01:12	0	0	0:00:00
MH2	0.24	0.25	905.06	0	01:12	0	0	0:00:00
MH3	0.35	0.35	903.45	0	02:12	0	0	0:00:00
Out	0.33	0.33	903.33	0	05:09	0	0	0:00:00

****
Node Flow Summary
****

Node ID	Element Type	Maximum Lateral Inflow GPM	Peak Inflow GPM	Time of Peak Inflow Occurrence days	Time of Peak Inflow Occurrence hh:mm	Maximum Flooding Overflow GPM	Time of Peak Flooding Occurrence days	Time of Peak Flooding Occurrence hh:mm
MH1	JUNCTION	146.98	146.98	0	00:00	0.00		
MH2	JUNCTION	0.00	146.98	0	01:17	0.00		
MH3	JUNCTION	0.00	146.98	0	01:12	0.00		
Out	OUTFALL	0.00	146.98	0	05:09	0.00		

****
Outfall Loading Summary
****

Outfall Node ID	Flow Frequency (%)	Average Flow GPM	Peak Inflow GPM
Out	99.97	146.91	146.98
System	99.97	146.91	146.98

****
Link Flow Summary
****

Link ID	Element Type	Time of Peak Flow Occurrence days	Time of Peak Flow Occurrence hh:mm	Maximum Velocity Attained ft/sec	Length Factor	Peak Flow during Analysis GPM	Design Flow Capacity GPM	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Surcharged Minutes
exit	CONDUIT	0	05:09	1.17	1.00	146.98	1089.04	0.13	0.25	0
Pipe1	CONDUIT	0	01:17	1.78	1.00	146.98	799.55	0.18	0.28	0
Pipe2	CONDUIT	0	01:12	1.97	1.00	146.98	1116.06	0.13	0.29	0

****
Flow Classification Summary
****

Link	--- Fraction of Time in Flow Class ---- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
exit	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.36	0.0000
Pipe1	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.59	0.0000
Pipe2	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.56	0.0000

****
Time-Step Critical Elements
****
Link exit (99.94%)

****
Highest Flow Instability Indexes
****
All links are stable.

****
Routing Time Step Summary
****
Minimum Time Step   :   16.76 sec
Average Time Step   :   16.77 sec
Maximum Time Step   :   30.00 sec
Percent in Steady State   :   0.00
Average Iterations per Step :   2.00

Analysis begun on: Mon Jan 24 10:58:37 2011
Analysis ended on: Mon Jan 24 10:58:38 2011
Total elapsed time: 00:00:01

BOSS International StormNET® - Version 4.15.0 (Build 17034)

****
Analysis Options
****
Flow Units ..... GPM
Link Routing Method ..... Hydrodynamic
Pond Exfiltration..... None
Starting Date APR-06-2010 00:00:00
Ending Date APR-10-2010 20:00:00
Antecedent Dry Days 0.0
Report Time Step 00:10:00
Routing Time Step ..... 30.00 sec

****
Element Count
****
Number of rain gages 0
Number of subbasins 0
Number of nodes 4
Number of links 3
Number of pollutants 0
Number of land uses 0

****
Node Summary
****

Node ID	Element Type	Invert Elevation ft	Maximum Elev. ft	Ponded Area ft²	External Inflow
MH1	JUNCTION	934.80	943.00	0.00	Yes
MH2	JUNCTION	933.93	940.00	0.00	
MH3	JUNCTION	932.60	940.00	0.00	
Out	OUTFALL	931.00	932.00	0.00	

****
Link Summary
****

Link ID	From Node	To Node	Element Type	Length ft	Slope %	Manning's Roughness
exit	MH3	Out	CONDUIT	100.0	1.6000	0.0130
Pipe1	MH1	MH2	CONDUIT	71.0	1.0845	0.0130
Pipe2	MH2	MH3	CONDUIT	110.0	1.1273	0.0130

****
Cross Section Summary
****

Link ID	Shape	Depth/ Diameter ft	Width ft	No. of Barrels	Cross Sectional Area ft²	Full Flow Hydraulic Radius ft	Design Flow Capacity GPM
exit	CIRCULAR	1.00	1.00	1	0.79	0.25	2022.72
Pipe1	CIRCULAR	0.67	0.67	1	0.35	0.17	564.83
Pipe2	CIRCULAR	0.67	0.67	1	0.35	0.17	575.85

********** Flow Routing Continuity **********	Volume acre-ft	Volume Mgallons
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000

Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.080	0.026
External Outflow	0.080	0.026
Surface Flooding	0.000	0.000
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.045	

Node Depth Summary

Node ID	Average Depth Attained ft	Maximum Depth Attained ft	Maximum HGL Attained ft	Time of Max Occurrence days	Time of Max Occurrence hh:mm	Total Flooded Volume acre-in	Total Time Flooded minutes	Retention Time hh:mm:ss
MH1	0.04	0.04	934.84	0	00:53	0	0	0:00:00
MH2	0.04	0.04	933.97	0	04:30	0	0	0:00:00
MH3	0.03	0.03	932.63	2	08:06	0	0	0:00:00
Out	0.03	0.03	931.03	0	00:58	0	0	0:00:00

Node Flow Summary

Node ID	Element Type	Maximum Lateral Inflow GPM	Peak Inflow GPM	Time of Peak Inflow Occurrence days	Time of Peak Inflow Occurrence hh:mm	Maximum Flooding Overflow GPM	Time of Peak Flooding Occurrence days	Time of Peak Flooding Occurrence hh:mm
MH1	JUNCTION	3.74	3.74	0	00:00	0.00		
MH2	JUNCTION	0.00	3.74	0	00:51	0.00		
MH3	JUNCTION	0.00	3.74	2	03:14	0.00		
Out	OUTFALL	0.00	3.74	0	00:58	0.00		

Outfall Loading Summary

Outfall Node ID	Flow Frequency (%)	Average Flow GPM	Peak Inflow GPM
Out	99.91	3.74	3.74
System	99.91	3.74	3.74

Link Flow Summary

Link ID	Element Type	Time of Peak Flow Occurrence days	Time of Peak Flow Occurrence hh:mm	Maximum Velocity Attained ft/sec	Length Factor	Peak Flow during Analysis GPM	Design Flow Capacity GPM	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Surcharged Minutes
exit	CONDUIT	0	00:58	1.10	1.00	3.74	2022.72	0.00	0.03	0
Pipe1	CONDUIT	0	00:51	1.02	1.00	3.74	564.83	0.01	0.06	0
Pipe2	CONDUIT	2	03:14	1.03	1.00	3.74	575.85	0.01	0.06	0

****
Flow Classification Summary
****

Link	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
	--- Fraction of Time in Flow Class ----								
exit	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.0000
Pipe1	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.91	0.0000
Pipe2	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.93	0.0000

****
Time-Step Critical Elements
****
None

****
Highest Flow Instability Indexes
****
All links are stable.

****
Routing Time Step Summary
****
Minimum Time Step   :   30.00 sec
Average Time Step   :   30.00 sec
Maximum Time Step   :   30.00 sec
Percent in Steady State   :   0.00
Average Iterations per Step :   2.00

Analysis begun on: Mon Jan 24 10:29:45 2011
Analysis ended on: Mon Jan 24 10:29:46 2011
Total elapsed time: 00:00:01

ATTACHMENT 5

Table 2 below is an excerpt from TM7 (Corrosion Protection). Samples were taken in Oct. 2010

	WTP4 Shaft (JT-109)	FPA Shaft	PARD Shaft (JT-120-A)	JR Shaft
TDS, mg/L	1,100 (Note 1)	Note 2	687	Note 2
1. Sample from Edwards formation. All other samples from the Glen Rose formation				
2. Results from the piezometers located at the FPA and JR shaft sites should be available by February 14				

Table 2.
Results of Piezometer Water Sample Analyses

Parameter	Units	Water Sample Location				
		JT-104-A	JT-109	JT-110-A	JT-120-A	JT-125-A
Depth of Water Sample Collection	feet	91	38	222	12	102
Calcium	mg/L	90.6	725	117	82.3	61.8
Magnesium	mg/L	ND	ND	63	78.2	43
Potassium	mg/L	ND	ND	ND	ND	ND
Sodium	mg/L	237	26.7	ND	35.7	29
Sulfur	mg/L	39.7	ND	52.6	86.8	ND
Alkalinity, Bicarbonates ($CaCO_3$)	mg/L	354	ND	286	270	282
Alkalinity, Total	mg/L	354	879	286	270	282
Hardness, As $CaCO_3$	mg/L	311	1,890	552	528	331
pH	pH units	8.11	11.6	7.63	7.25	7.27
Total Dissolved Solids	mg/L	1,600	1,100	560	687	335
Specific Conductance	µmhos/cm	890	4,070	724	916	566
Resistivity	ohm-cm	1,124	246	1,381	1,092	1,767
Chloride	mg/L	46.7	22.7	15.9	28	13.9
Nitrate	mg/L	ND	ND	ND	ND	ND
Nitrite	mg/L	0.653	ND	ND	ND	ND
Ortho-phosphate as P	mg/L	ND	ND	ND	ND	ND
Sulfate	mg/L	186	37.2	139	243	16
Larson's Ratio		1.6	132	1.3	2.5	0.2
Langelier Saturation Index		0.70	5.53	0.02	-0.24	-0.27

ATTACHMENT 6

From:	Hoepken, Joe
To:	Long, Stacie; Bybel, Jason; Lesniak, Charles; Perkins, Thais; Smith, Robyn K; Schnettgoecke, Gary; Allen, Dennis L.; Brainard, Ray C.; Anderson, David; Ross.r.webb@mwhglobal.com; Larry Laws
Subject:	RE: JTM - Shaft and Tunnel Construction Water Discharge Options Summary DRAFT FOR YOUR REVIEW
Date:	Wednesday, March 09, 2011 2:18:13 PM

FYI, I called Tony Canales, Special Services Division Manager (under Daryl), yesterday to get direction on next steps with construction/tunnel water discharge to the sanitary sewer. MWH and/or the tunnel contractor (party responsible for the discharge) needs to contact John Milligan with the Special Services office (972-1060) to submit an application for the requested discharge permits. Typically, a sedimentation tank is required prior to discharge to reduce solids loading, along with a flow meter.

Along with the table below, BV should now have adequate information on discharge limits/permit process for incorporation into the contract documents. If not, please let me know.

Thanks,
Joe

From: Hoepken, Joe
Sent: Friday, February 11, 2011 2:31 PM
To: Canales, Antonio; Houston, David; Milligan, John; Neberman, Michael; Fuhr, Susan; McCulloch, John
Cc: Long, Stacie; Shropshire, Paul; Ellison, Tom F.; Schrader, Steven; Lesniak, Charles; Perkins, Thais; Smith, Joe [WCC7]; Nguyen, Dong; Smith, Robyn K; Conrad, William; 'Larry Laws'; 'Schnettgoecke, Gary'; 'Allen, Dennis L.'; 'Oksuz, Faruk'
Subject: FW: JTM - Shaft and Tunnel Construction Water Discharge Options Summary DRAFT FOR YOUR REVIEW

Industrial Waste Team:

Our project team has determined projected flowrates for tunnel construction water along with manholes we propose to discharge into as follows:

Shaft Site	Manhole No.	Max Discharge Flow (gpm)
Jollyville Reservoir	114159	476
	114232	592
PARD Tract	New MH on 82-0600	50
Four Points	87305	1132

See attached for more detailed information, calcs, etc.

Please advise on the next step for obtaining necessary discharge permits. Construction will begin in Fall 2011. Thanks for your assistance.

Joe

From: Hoepken, Joe
Sent: Thursday, February 03, 2011 3:08 PM
To: 'Schnettgoecke, Gary'; Long, Stacie; Smith, Robyn K
Cc: Anderson, David; Allen, Dennis L.; Oksuz, Faruk; Brainard, Ray C.; Weeks, Zachary; Breck Plauche; joseph dong
Subject: RE: JTM - Shaft and Tunnel Construction Water Discharge Options Summary DRAFT FOR YOUR REVIEW

Gary,

Here is my feedback on the Recommendations section:

- I agree with Items 1 and 2.
- For Item 3, what is proposed for locating the line? Typically, we would want to do this right now to determine location relative to pavement and the PARD LOC. However, in the interest of keeping things quiet in the neighborhood, I suggest we delay this until start of construction. When the time comes, I can have one of our maintenance crews locate and mark the alignment using a sonde, or defer this task to the contractor (probably for the best to go with the latter to avoid any finger-pointing – and would extend our official LOC to the u/s and d/s manholes).
- I agree with Item 4, and do not want to pursue discharge to the 24-inch sanitary line on the other side of Hwy 183.
- I agree with Item 5 – any cleaning and televising of sanitary lines is outside of BV's scope. I will check to see what sewer inspection videos we have of the proposed receiving lines.

Also, I am investigating the following and should have a response soon: "The Black & Veatch team requests that AWU consider allowing the available full capacity of the sewers used to determine available capacity. It is our understanding that the 85% calculation required by AWU allows for reserve capacity for unanticipated future flows. After construction, the reserve capacity would be available."

Thanks,
Joe

From: Schnettgoecke, Gary [mailto:SchnettgoeckeG@bv.com]
Sent: Wednesday, February 02, 2011 7:27 PM
To: Long, Stacie; Hoepken, Joe; Smith, Robyn K
Cc: Anderson, David; Allen, Dennis L.; Oksuz, Faruk; Brainard, Ray C.; Weeks, Zachary; Breck Plauche; joseph dong
Subject: JTM - Shaft and Tunnel Construction Water Discharge Options Summary DRAFT FOR YOUR REVIEW

Attached is our first draft of the subject document for your review. The document contains our evaluation of the existing sanitary sewer capacities (as requested by Joe), summary of surface water and sewer discharge options, and our preliminary recommendations. At your earliest convenience, we would like to discuss the summary to gain consensus on the recommended actions and incorporate the work into the contract documents.

Please contact me or Dennis with any questions.