US Army Corps of Engineers®
Memphis District

GRAND PRAIRIE REGION AND BAYOU METO BASIN, ARKANSAS PROJECT

BAYOU METO BASIN, ARKANSAS

GENERAL REEVALUATION REPORT

VOLUME 3

APPENDIX B

ENGINEERING INVESTIGATIONS & ANALYSES AGRICULTURAL WATER SUPPLY COMPONENT

SECTION I – HYDRAULICS AND HYDROLOGY

NOVEMBER 2006

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PART I

HYDRAULICS AND HYDROLOGY

PART I - HYDROLOGY AND HYDRAULICS

TOPIC A - INTRODUCTION

1-A-01. GENERAL

This section addresses the hydraulics of two water systems that will occupy the project area landscape: the existing natural drainage system and a surface water delivery system. A design was performed that permits water to be pumped from the Arkansas River and distributed throughout the project area using man-made canals and existing ditches modified to accommodate flood flows and additional irrigation flows.

The irrigation delivery system is designed to take water from the Arkansas River, and distribute it through a main canal system to numerous smaller canals and ditches. The fundamental concept of the delivery system design is that the water is to be lifted so that it can be conveyed within the project area by gravity flow wherever possible. New canals and laterals will typically follow high ground, wherever possible, in order to take full advantage of opportunities to convey withdrawals by gravity flow.

Topic B consists of a brief description of the hydrology of existing streams within the project area and an explanation of the analysis of water needs including a need for a supplemental source of water to meet the demands necessary. Hydrologic analysis of these streams was required in order to ensure that the project components do not cause flooding and to determine that certain existing streams can be incorporated into the delivery system. The hydraulics of the delivery system components is the subject of Topic C, including pump stations, regulation reservoirs, man-made canals, check structures, existing ditches, pipelines, inverted siphons, pump-type turnouts, gate-well structures, weirs, drop inlet structures, and road crossings. Topic D briefly describes the control system setup.

TOPIC B – HYDROLOGY AND WATER NEEDS OF THE BAYOU METO AREA

1-B-01. GENERAL.

There is a need to supplement the water needs in the Bayou Meto area. Looking at runoff in the basin and water needs, additional water will be introduced into the system via a delivery system. It was necessary to determine how irrigation flows and the man-made canals used to convey these flows might affect the natural runoff characteristics in the Bayou Meto watershed. It was also necessary in developing the delivery system that we determine if certain existing streams could be incorporated into the delivery system. A hydrologic study was made of the project area's existing streams. The study provided estimates of the magnitude and frequency of natural flows in the existing streams. This information was used in the hydraulic design of inverted siphons and weirs. The siphons permitted natural streams to flow unimpeded across canal alignments, while the weirs pool water to be pumped to farms. The results of the hydrologic study indicated that the siphons and weirs could be designed to function without causing flooding.

The project area boundaries do not coincide with one watershed, but instead encompass portions of the watersheds of several creek-sized streams and minor drains, as shown in Plate I-B-1.

The topography of the project area is that of the Arkansas River bottom and its tributaries including old river runs and a relatively small remnant of the Grand Prairie north and east of the Bayou Meto channel in Lonoke County. Many existing streams dissect the topography creating a complicated drainage system. The largest stream within the project area is Bayou Meto, which originates in the northwest corner of the project area and flows toward the southeast, passing east of the Bayou Meto Wildlife Management Area and exiting into the Arkansas River through gated drainage structures. Most of the project area drains toward the south, but the portion north of Interstate 40 primarily drains to the northeast.

The climate of the project area is characterized by cool, wet winters and hot, humid summers. Average annual precipitation recorded at the National Weather Service (NWS) gage at Stuttgart, Arkansas was 49.2 inches for the years 1948 through 1986. The lowest and highest recorded NWS temperatures at Stuttgart, Arkansas for the years 1948 through 1986 were -6 degrees and 107 degrees Fahrenheit [1]. A detailed description of project area climate is presented in the water balance section of this volume.

Project area soils and vegetative cover affect infiltration of rainfall. The soils are primarily silt loams underlain by a compact subsoil [2]. Land use in the project area is primarily agricultural, with most farmland devoted to crop production. Cropland vegetative cover varies throughout the year, depending on the state of crop growth at a given season.

1-B-02. DATA AVAILABLE.

The existing streams within the project area are ungaged, so the Vicksburg District's hydrologic analysis focused on estimating discharge-frequency relationships for the streams. Information required to perform the hydrologic analysis included topographic and land use data, distance measurements used to support unit hydrograph computations, and rainfall data.

1-B-03. ADDITIONAL WATER NEEDED.

The design flows used in the development of the delivery system for the Bayou Meto project were derived from demand flows furnished by the Natural Resource Conservation Service (NRCS). Water needs were determined for each tract of land in the project area using the water balance approach, which takes into account climatological data, on-farm storage, and crop needs derived from land-use information, evapotransporation, and infiltration. Since the entire delivery system will consist of unlined man-made canals, existing ditches, and existing streams, additional flow will be needed from the supply source to account for seepage losses and evaporation losses. The maximum demand flow for any 10-day period based on the demand analysis at any reach in the delivery system will govern the size of the delivery system in that reach.

The decision to select an irrigation design flow in the delivery system was based on the reliability of the source. The reliability of the source determines the availability of water on a 10-day basis taking into account minimum water requirements for navigation, water quality, and fish and wildlife. Authority to establish minimum streamflow was granted to the Arkansas Soil and Water Conservation Commission by the Act 1051 of 1985 and the Act 469 of 1989. The minimum streamflow is defined in the legislation as the quantity of water required to meet the largest of the following instream flow needs as determined on a case-by-case basis for 1) interstate compacts, 2) navigation, 3) fish and wildlife, 4) water quality, and 5) aquifer recharge. The only quantifiable flow rates for navigation, fish and wildlife, and water quality in Pool No. 6 of the Arkansas River are 2000 cfs, 4645 cfs, and 684 cfs, respectively. The instream flow that meets the largest requirement stated by the legislation would be the 4645 cfs for fish and wildlife. Period of record data shows that some years have had days when there was not enough flow in the Arkansas River to meet the minimum flow requirements, which would mean no water would be available to meet additional demands. These critical months were part of the statistical analyses used in determining the design.

The crop demand data spreadsheet provided by the NRCS consists of synthetic period-of-record demands in 10-day increments. This was derived from a water balance approach using a historical set of data between 1940 and 1996. Duration analyses were performed on this data to determine the design flow for each 10-day increment. Flow-duration analyses were used to determine flows with an exceedance frequency of 0.01% based on the 57 year period-of-record. The results of these statistical analyses show the peak demands for each 10-day period during the year. By taking the demands and the source reliability, and looking at when water is needed versus when water is available, a determination can be made on the reliability of the delivery system. To increase the reliability of the project during peak crop needs, additional on-farm storage will be developed to help meet crop needs and reduce demands from the delivery system. These on-farm reservoirs will be filled during the winter/spring, and during non-peak usage times when water from the river is available. So the determination of an on-farm storage/delivery system design had to be based on the expected amount of additional on-farm storage. This was on based the NRCS's knowledge of the region, information database necessary to develop crop demands, and the day-to-day interactions with local farmers. From these discussions, the optimal design was determined to be 25% additional on-farm storage in the northeastern half of the improvement project area and 10% additional on-farm storage in the southwestern half of the project improvement area.

Hydraulic and geotechnical personnel made decisions to account seepage and evaporation losses from regional correlation of the Grand Prairie area, as well as extensive field and laboratory works. Using numerous borings to identify soil strata, preliminary information on canal depths, widths and alignments, standard operation losses of 5%, and information dealing with pan evaporation in the area, values were developed to increase 10-day demand flows through the design year. Analyses show that a 8.5% increase for seepage and a 0.5% increase for evaporation, plus the 5% for operational losses, would accommodate the losses in the system and allow the system to delivery necessary demands. With losses accounted for, the peak demand flow at the inlet for the entire delivery system of 1750 cfs. Even though the crop needs for agricultural farms are the greatest during the summer months, design demand flows were influenced by spring month unmet needs because of the potential of future water needs in the area of baitfish farming. The system was designed to accommodate today's needs as well as needs in the future.

Using Little Rock District gage information of Pool No. 6, which is formed by David D. Terry Lock and Dam No. 6, duration analyses were performed on 57 years of historical data to determine the reliability of having certain flow amounts during different times of the year. These flow amounts would be in addition to the minimum flow requirement of 4645 cfs. The analyses were also conducted for flow amounts available 100% of the time, 99.8% of the time, 98% of the time, 95% of the time, 90% of the time, and 80% of the time for the design year.

The design flows are plotted along with these durations in Plate I-B-2 to show the reliability of each 10-day design demand flow in relation to the availability of the source.

TOPIC C - HYDRAULICS

1-C-01. INTRODUCTION.

A complex system of structural components is required to convey water in a controlled manner through the delivery system. It was necessary to determine the types of structures to include in the system design, as well as the dimensions and locations of these structures. Twelve delivery system components were selected for the system: pump stations, regulation reservoirs, man-made canals, check structures, existing ditches, pipelines, inverted siphons, pump-type turnouts, gate-well structures, weirs, drop inlet structures, and road crossings.

A large pump station at the Arkansas River lifts water up into a regulation reservoir, which will in turn supply the main canal with irrigation water. Smaller canals, existing ditches, and pipelines will be used convey water across the project area to individual farms. Water levels are maintained in the main canal by check structures. Water is directed to the secondary system through turnout structures. Pipelines are used to get water to the farms not adjacent to the delivery canals and ditches. Bridge and culvert road crossings allow irrigation flows across public and private roads.

Analyses were performed on all of these components and to determine sizes, alignments, capacity, functionality, etc. The system was looked at in three general ways, all existing channels and ditches with fewer man-made canals and pipelines, all man-made canals and pipelines, and balance between existing ditches and man-made canals and pipelines. From these analyses, the latter ended up being the selected plan.

When the project was first being developed, it was envisioned that the irrigation system would use a gravity inlet from the Arkansas River to feed all of the existing streams in the project area, via some man-made canals to tie everything together. Well the first concern arose in trying to gravity feed water from the Arkansas River. Levels in Pool No. 6 didn't appear to be high enough to effectively feed a canal system, the topography seemed to high. After investigation and cost analyses, it was cheaper to build a pump station with necessary canals and turnouts to streams instead of trying to gravity feed the system, with very deep man-made canals and relifts at every stream. The results of the analyses are shown on Plate I-C-1 and I-C-2.

After resolving the pump vs. gravity issue, the more difficult task of how to get water to the farms needed to be addressed. After looking at unmet needs all the way to the farm level, the streams initially laid out to get water to these farms were too small, and work would have to be done in a lot of them to get the necessary conveyance. Environmental interests were had concerns about this type of work in some the streams. Some of the streams they felt like should not be touched at all, limiting what we could convey. Plus, there would have to be some way to measure waters in these streams, the more streams used, the more complicated and costly. It was decided that another means of getting water to the farms needed to be looked at.

From lessons learned on the Grand Prairie Area Demonstration Project about the reliability and effectiveness of a man-made canal system, a design was looked at that would move water throughout the project area, completely transparent to the natural runoff characteristics of the Bayou Meto basin. Although there are some tremendous benefits to having this type of system, the major fall back was the amount of acreage needed to put a system in place for the project area. Farmers weren't happy on how this alternative would affect them. Too much land would come out of production when they had land they couldn't farm along streams adjacent to there land. The final alternative was to determine which streams we could use, possibly enlarge, and then determine what would be the best way to get water to the farms.

Field trips were taken with environmental team members to see which of the streams we could use as part of the delivery system. The first thing identified was that some of the streams were actually man-made ditches used for flood relief. These ditches were straight ditch segments with very little substantial vegetation. And the channels had filled in with sediments and the ditches were dry in places. Because of the environmentally poor conditions of the ditches, it was acceptable to use these ditches to convey irrigation water throughout the system. Avoiding streams that were pristine in nature and using ditches and pipelines made this alternative the most acceptable to farmers and environmental groups. Of course there still had to be man-made canals to tie the whole system together, but minimizing the number was an important consideration. So as the system developed over time using these considerations, we have an inlet channel from the Arkansas River to Pump Station No. 1, which pumps against 20 foot of head to a regulation reservoir which in turn feeds the main canal through a check structure built in the reservoir levee. The canal proceeds to the east through additional check structures, turning out into other man-made canals, ditches, and pipelines via gatewell structures and pump type turnouts to supply areas to the south. These areas south use weirs to hold pools that are pumped into pipelines that supply irrigation water to the farms. On the main canal, we reach a point in the Bayou Meto basin where we must get irrigation water up to Long Prairie, and then later cross Bayou Two Prairie to feed the remainder of the project. Using the same type of delivery components discussed early, we pump up to regulation reservoirs in the two locations just mentioned, and move water to the east and north through man-made canals, check structures, gatewell structures, pump type turnouts, ditches, and pipelines. Using sophisticated control software and water data gathering software and hardware, this system will not only move irrigation water throughout the system, but it will do it in such a way that water will get to the farmer as quickly as possible, always fair and equitable, providing a usable and much needed resource for the farmer, as well as the environmental interests who understand the importance of this resource.

The following sections discuss the design of these components of the project and how they play an integral part in getting surface water the river to the farm, everyone benefiting from the much needed water.

1-C-02. PUMP STATIONS.

1. General. Four pump stations are included in the project. Pump Station No.1 is located in Pool No. 6 of the Arkansas River, just upstream of David D. Terry Lock and Dam. All water required by the project delivery system passes through this pump station. Pump Station No.2 is located at Bayou Meto just south of Lonoke, AR. This pump station takes irrigation water across Bayou Meto and up onto the Long Prairie. From there, Canal 3000 goes to the north, just west of Lonoke, Arkansas, crosses Interstate 40, and continues north to Bayou Two Prairie. Pump Station No.3 is used to pump water across Bayou Two Prairie and feed the northeastern portion of the project. Pump No. 4 is a 125 cfs capacity pump serving as a relift station from Caney Creek. These pump stations are part of a system that will move water from the Arkansas River eastward in a very timely and efficient manner. Sophisticated control software will play a crucial role in moving irrigation water across the basin.

Pump Station No.1 will have a maximum capacity of 1750 cfs. It was assumed that sources of water used by farmers, primarily groundwater, would be limited in the future. The size of Pump Station No.1 this station was based on the projected maximum unmet needs stemming from this development. Maximum unmet needs were based on average needs for 10-day periods for a long period of record. Looking at 56 years of recorded water usage, analyses were performed on the data to develop a 10-day average demand curve for unmet needs in the project improvement area. The final demand curve is shown on Plate I-C-3.

Plate I-C-4 shows how the inlet channel directs water from the Arkansas River to the pump station, and this inlet channel must be able to take in water from a fluctuating pool, even when pool levels are low. In order to determine design elevations for the inlet channel, an analysis was made of gage data at David D. Terry Lock and Dam. The 0.01 non-exceedance probability discharge at the lock and dam is 720 cfs, which can be passed at a pool of 231.2 ft, NGVD. The pump station inlet elevation is 227.0 ft, NGVD, which will provide sufficient inlet head for the intakes at Pump Station No.1.

Table I-C-1 presents the relationship between exceedance frequency and pool elevation at Pool No. 6. The frequency-elevation relationship was used as a guide in setting minimum elevations of the Pump Station No.1 components.

Table I-C-1. Frequency vs. Pool Elevation
for Pool No. 6 from the Arkansas River

Exceedence Frequency	Computed Pool Elevation ft-NGVD
1.01	231.2
2	231.3
5	233.4
10	237.3
25	244.3
50	248.5
100	253.4

Pump Station No.1 and other pump stations in the system have only been described to the extent necessary to show design delivery system functionality considerations. A more detailed description of pump station design is provided in the Appendix B, Section 4, Structural, Electrical, and Mechanical portion of the project report.

1-C-03. CANALS.

1. General. Pump Station No.1 will fill a regulation reservoir, which will in turn supply the man-made canal system, existing ditch system, and numerous pipelines. The man-made canals have very mild slopes with average flow velocities of 2 feet per second (fps) or less. Check structures in the man-made canal system serve to form and to control the canal pools.

2. Design Data. Information required to design the canals included demand data, geographical data, soils data, losses, and hydraulic data. Regarding demand and geographical data, tract irrigation demands were provided by the NRCS, while aerial photography and USGS 7.5 minute mapping provided locations of improvements and project area topography.

Soil properties affected canal design with respect to slope stability, channel regime, seepage, and earthwork balance. The Geotechnical Section recommended 3 foot horizontal: 1 foot vertical (3H:1V) canal side slopes for stability. The silty texture of the soil was a factor in determination of the proper silt factor, f, used in the Lacey regime computations; a silt factor of 0.357 was found for "Lower Mississippi silt" [3]. Delivery system water loss due to seepage, evaporation, and operational waste was estimated at approximately 15% of the demand. This loss value was based on USBR guidelines for preliminary estimation of losses in unlined earthen canals [4] and on consideration of the silty texture of project area soils, which will not seep excessively. Adopting recommendations of the Geotechnical Section, a shrinkage value of 1.3 was used for computation of earthwork balance, such that 1.3 cubic feet of channel excavation was equated to 1.0 cubic feet of earthfill.

Canal hydraulic computations required values for roughness, slope, canal depth limits, and required freeboard. A Manning channel roughness of 0.035 was selected for design, which is appropriate for a maintained earthen canal [5]. Since the delivery system is designed to operate essentially as a series of pools, small channel slopes were assumed in the range of 0.001 ft/ft to 0.00005 ft/ft. Regarding turnout inlet submergence requirements, minimum desired flow depth for the smaller canals was determined to be about 3.0 feet. Due to maintenance considerations, a maximum canal depth of 13 feet was adopted. USBR [6] canal freeboard guidelines were followed, which express freeboard as a function of canal discharge. As shown in Plate I-C-5, the recommended height of the canal bank above the water surface increases slowly with increasing discharge. For example, the minimum recommended freeboard is 1.2 feet, a freeboard of 3.55 ft is recommended for a canal discharging 1000 cfs, and a canal discharging 2000 cfs requires only an additional 0.55 feet of freeboard.

3. Design Method. Important canal design considerations were to (1) provide service to all irrigated tracts within the project area, (2) maximize gravity flow distribution, (3) determine stable canal proportions required to convey the design discharge, and (4) to obtain balanced earthwork quantities.

Preliminary canal alignments were established, based on location of irrigated tracts, on topography, and on the location of roads, utilities, buildings, and other improvements. As much as possible, the canals were located along ridges in order to permit withdrawals by gravity flow while minimizing land-rights conflicts. In most cases the preliminary canal alignments were found to be feasible and were adopted. In a few cases land rights concerns resulted in changes of alignment or in substitution of pipelines for open channels. Irrigation demands were matched with canal alignments to determine the demand discharge of each canal. Since losses were estimated at 15 percent, the actual design discharge of a canal exceeded demand discharge.

Based on alignment and design discharge, canal dimensions and hydraulics were determined. First, a value of canal bed slope was assumed. With slope assumed and the Manning n-value of 0.035 selected, the remaining factor to determine was canal cross-section. A satisfactory canal cross-section simultaneously satisfied the requirements of side slope stability, required hydraulic radius and area, and an appropriate width/depth ratio for channel regime stability. By adding freeboard to maximum pool depth, the overall depth of the canal section was determined.

The Lacey regime method was used to determine stable channel cross-section proportions. Unlike side slope stability, which addresses the structural ability of sloping earth to support its own weight, regime stability is a sediment transport matter. In general, for a given discharge and soil type there is a combination of bed slope and cross section dimensions for which the quantity of sediment transported into the reach is equaled by the quantity transported out of the reach. If this balance does not exist, the channel will perform work on its boundaries until balanced sediment transport is achieved, at which time it is said to be "in regime"[7]. The regime concept was developed in India as a result of studies on man-made canals. The Indian canals tended to be operated for long periods at design discharge, which is similar to the project operating conditions.

The basic Lacey equation is, [8]:

$$U = 1.17(Rf)^{0.5}$$

where: U = velocity, fps
R = hydraulic radius, ft
f = silt factor.

The equation states that for a stable channel the average velocity is equal to a constant times the square root of the product of hydraulic radius and a silt factor. Recommended silt factors vary from a low of 0.357 for "Lower Mississippi silt," to 1.00 for "Standard Kennedy silt," to a high of 39.60 for "massive (24") boulders". Lacey manipulated the basic stability equation to provide solutions for channel area, perimeter, and bed slope. For project conditions, using a silt factor of 0.4 and a maximum allowable velocity of 2.0 fps, the Lacey equations resulted in the following two relationships:

$$2 \leq W/D \leq 3 \quad \text{for } Q \leq 400 \text{ cfs}$$
$$3 \leq W/D \leq 5 \quad \text{for } Q > 400 \text{ cfs}$$

where W = canal bottom width, ft
D = canal flow depth, ft
Q = design discharge, cfs.

The maximum velocity of 2 fps was selected in order to minimize erosion, to minimize head loss, and to produce a controllable pool-like canal system. For a given channel width, maximum downstream and minimum upstream values of flow depth were calculated for the regime proportions. The calculated depths were large enough to provide the turnout inlet heads required. The allowable range in canal depth divided by the assumed slope length yielded the maximum permissible length for that canal, to be established by installation of check structures.

An invert elevation intended to result in balanced earthwork quantities was assumed for the canal, and hydraulic computations were performed using the calculated cross-section dimensions. A check was then performed to ensure that the Lacey W/D proportions were satisfied all along the length of the canal, particularly at locations where a change in channel dimensions occurred.

Additional considerations in canal design included the elevation of the canal water surface, the maximum drop across check structures, and design details for small canals. It was desired that the water surface in the canal be above the elevation of natural ground in order to facilitate gravity flow for withdrawals and also to minimize excavation quantities. The drop across check structures was limited to about 5 or 6 feet from upstream bed to downstream bed. For small canals conveying 20 cfs or less, a minimum depth of about 3 feet was maintained and stable canal proportions were selected to minimize pool surface area and associated evaporation loss. However, in locations where excavated material was needed for fill, stable canal dimensions were selected to provide the material even if pool surface area was not minimized.

Erosion control was also considered in canal design. Types of erosion evaluated were general scour due to flow, wave-induced erosion, and localized erosion due to sudden changes in canal configuration. Due to the selection of design flow velocities of 2 fps or less and the use of regime canal design principles, canals were found to require no general riprap protection against flow-related scour. Since the canal pools will be maintained at an essentially constant elevation and the long, straight canals have significant fetch, there is potential for long-term wave-induced erosion similar to that occurring along lake shorelines. Wave-induced erosion will be resisted somewhat by vegetation on the embankment, but near the water line the growth of vegetation will be suppressed. In an extreme case, wave erosion could seriously damage canal embankments, but the most likely extent and rate of wave erosion is difficult to estimate. Although lining the canal waterlines with riprap is physically possible, this approach was not seriously considered due to the high cost. In the expectation that wave erosion will not be a significant problem, it was decided to recommend provision for occasional wave erosion repair in project operating costs and for coverage of the topic in the O&M manual. Regarding local scour, velocities are low enough that even sharp canal bends or abrupt changes in dimensions are not expected to require riprap protection against flow. It is possible, however, that certain exposed locations in bends may require protection from direct wave attack. During design for construction, if it should become apparent that isolated canal locations require riprap protection for whatever reason, design of localized riprap protection will be performed at that time.

4. Design Results. The delivery system design process resulted in a system of canals/ditches and of pipelines with canal designations. The canal-designated pipelines are described under the pipeline portion of this report. The canals were trapezoidal in cross section and the design table is shown on Plate I-C-6. Plate I-C-7 shows a typical canal cross-section. Main canal levees were designed with 12 ft and 15 foot working berms. Channel cuts and levee slopes were designed at 3H:1V.

1-C-04. CHECK STRUCTURES.

1. General. Check structures feature a gate and are used to regulate the water surface elevation in the canal pool upstream of the structure and to release flow to the downstream canal pool. The details of check structure operation depend on the canal system operation concept being applied, i.e. downstream or upstream operation [9]. With either control concept, check structures are operated to maintain the canal water surface elevations required for deliveries at turnout structures. The procedure used to determine check structure spacing and location is described in 3-C-03. The structure design varies according to the maximum discharge capacity. Main canal sites with high flow rates are controlled with large gated check structures, while sites with lower discharges are controlled with closed conduit check structures.

2. Design Data. Design data included design discharge, loss coefficients and conduit roughness, and upstream and downstream water surface elevations.

3. Design Method of Main Canal Check Structures. Main canal gated check structures were required on Canals 1000, 2000, 2500, and 3000, due to the magnitude of discharge, which ranged from 228 cfs to 1750 cfs. A typical gated check structure is shown in plan view in Plate I-C-8. The structure features the gates themselves, a stilling basin, and overflow weirs to either side of the gates.

The types of gate typically used for check structures are the sluice gate (vertical lift) and the tainter (radial) gate. Tainter gates were selected for these main check structures because of their ability to accommodate the need for small flow changes that would require small gate movements. These small changes are necessary for downstream system control. Gate sizes are dependant on the required demand flows in the downstream canal under maximum water surface elevation conditions. Gate discharges were computed using the standard orifice equation for free flow conditions, as shown below:

$$Q = cA(2gH)^{0.5}$$

where "c" is the contraction coefficient for tainter gates, "A" is the cross sectional area of the gate opening in square feet, "g" is the acceleration of gravity in feet per second squared, and "H" is the difference between the upstream water surface elevation and the centerline of the gate opening in feet.

Under submergence conditions, the contraction coefficient is a function of the tailwater depth (h_s), and gate opening (g_o). Head (H) is the difference between the upstream and downstream water surface elevations. The submergence adjustment was conducted following U.S. Army Corps of Engineers guidance [10], [11]. The submerged contraction coefficient, c_s, and its relationship with tailwater depth and gate opening are shown below:

$$c_s = c\left(\frac{h_c}{g_o}\right)$$

Stilling basin design was conducted following USBR guidance [12] for Type-I hydraulic jump stilling basins without baffle blocks. EM 1110-2-1605 suggests the stilling basin should be designed for a single gate fully open with normal headwater and minimum tailwater. An array of gate openings and tailwater conditions were analyzed for stilling basin design to ensure all ranges of velocities anticipated were considered during operations. The initial condition analyzed was a 1.0 ft. gate opening assuming no tailwater in the receiving canal. Subsequent gate openings were increased until the gates were eventually unsubmerged. Maximum stilling basin lengths resulted from the maximum gate openings. Riprap protection and limits were designed both upstream and downstream of the structures using procedures outlined in EM-1605. Approach riprap limits were set at 50 feet for all checks and the downstream limits were set at 10 times the conjugate depth, y_2, as outlined in the EM.

4. Design Results.

A complete list of main canal check structures is provided in Plate I-C-6, including the identification of each structure, minimum upstream pool elevations, bottom elevation, and explanatory remarks.

1-C-05. EXISTING DITCHES.

1. General. Selected existing ditches were designed to convey irrigation water from the canals to the landowners, via pipelines. Ditches were selected based on the considerations of condition, vegetative cover, and advantageous access to irrigated tracts using pipelines. To address questions concerning the ditches to be used, existing hydraulic models developed by the Vicksburg District, used for flood control, were run by the Vicksburg District to examine the addition of irrigation flows. The models were run to obtain water surface profiles for Indian Bayou, Indian Bayou Ditch, Caney Creek, Crooked Creek Ditch, Blue Point Ditch, Skinner Branch, Shumaker Branch, Oak Branch, and Rickey Branch. The models were used to determine frequency flowlines using the irrigation project information, and to aid in locating and sizing weirs. Weirs are structures necessary to adapt the existing streams to serve as delivery system components and are discussed in 3-C-09.

2. Design Data. Field survey cross-section data was used to build HEC-RAS models for the natural streams. Discharges for input to these models were derived using HEC-HMS watershed models for each natural stream. Manning's roughness coefficients were determined through field reconnaissance, guided by standard references [13], [14]. Road crossing information for bridges or culverts was obtained from survey information and was coded into the HEC-RAS model.

3. Design Method. The existing stream computer models were used to compare the magnitude of the frequency flows to bankfull discharges, and to predict how small increases in discharge, due to the addition of irrigation flows, would affect frequency flowline elevations. The largest irrigation flows introduced into the channels were only about 10% of the 1.01-year event, which is above bankfull. This resulted in very small changes to frequency flowlines. Since the bankfull elevation is also the zero flood damage elevation, changes in flowlines were considered in the design of the weirs. This was a critical concern, because the weirs must pool water in the existing streams without significantly increasing water surface elevations at bankfull discharge. Design information and typical design computations are shown on Plates I-C-9 and I-C-10.

1-C-06. PIPELINES.

1. General. At some locations within the project area it is more efficient to deliver water through pipelines than through open channels. In some instances pipelines fit into right-of-way restrictions better than open channels. Some reaches feature unfavorable depths of cut for an open channel and some reaches have an adverse grade requiring pumped conduit flow. The pipelines are typically used from existing ditches through the project. There are enough pumping pools in the existing ditches so that discharges to individual farms are low enough to be readily conveyed through a conduit of economical size. However, in some cases pipelines were substituted for segments that had been originally planned for canals due to right-of-way limitations and topography constraints.

2. Design Data. Data required for designing the pipelines included topography, irrigation flow demands, head losses, and pump characteristics. Table I-C-3 presents the values of minor losses associated with pipeline fittings and appurtenances. Table I-C-4 presents values of Hazen-Williams pipe

roughness for PVC and concrete pipelines. Pumped pipeline criteria are presented in Table I-C-5. The motor and pump efficiencies were provided in Appendix B, Section IV, Structural, Electrical, and Mechanical portion of the project report and are to be used as a generalization for the size of pumps used in this project. The maximum pipe velocity, kinematic viscosity, and roughness coefficient were taken from design guidelines.

Table I-C-3 Minor Loss Coefficients [15]

FITTING	COEFFICIENT
45° Elbow	0.2
90° Elbow	0.22
Tee, Line Run	0.35
Tee, Branch Run	1.28
45° Wye, Line Run	0.30
45° Wye, Branch Run	0.50
Coupling	0.33
Squared Entrance	0.50
Exit	1.0

Table I-C-4 Hazen-Williams Discharge Coefficients [16]

PIPE MATERIAL	H.W. COEF.
Poly-Vinyl Chloride Pipe (PVC)	150
Reinforced Concrete (RCP)	140

Table I-C-5 Pumped Pipeline Criteria [17]

Motor Efficiency	90%
Pump Efficiency	75%
Minimum Pipe Diameter	12"
Minimum Motor Horse Power	.75 hp
Maximum Velocity in Pipe	5 fps
Kinematic Viscosity	1×10^{-5}
Roughness Coefficient	0.0018

3. Design Method. Due to a large number of pipelines, minimum depth, outlet head, and velocity limits were standardized to aid in design. The pipelines were designed at a pipe invert depth of 5 feet below natural ground in order to obtain adequate cover. Final design of pipelines will provide the

minimum cover so as not to impede farming operations. The pipelines were sized to deliver water at design discharge to an elevation 5 feet above natural ground elevation at the pipeline outlet as a design safety factor to ensure that adequate head was provided at the ground level. The minimum design velocity desired in the pipe network was 2 fps in order to prevent settling of sediment. The maximum desired design velocity was 5 fps in order to protect the pipe from flow-related damage. In several cases the calculated velocities exceed 5 fps and appropriate measures, according to manufacture's specifications should be taken to adequately protect these pipelines against surge. Pipes were sized to minimize pumping horsepower requirements.

The scale and complexity of the pipeline system warranted analysis aided by specialized computer software. Pipe and pump sizes were determined using the CYBERNET program developed by Haestad Methods. This process consisted of graphically constructing a pipe network schematic for each site. Once the schematic was completed, various components such as static nodes, junction nodes, pumps, flow control valves (FCV), check valves, and minor loss components were added by snapping to their desired location on a schematic. Pull-down menus for each type of component were accessed, and values were added for pipe lengths; diameters; and roughness coefficients; static node water surface elevations; junction node elevations; and pump horsepower (hp). Values for minor loss coefficients were assigned to each type of fitting and were applied to the network calculations when a fitting was inserted into the schematic. Behind the basic editing environment of CYBERNET are numerical models, which will yield a complete and accurate simulation of the system network pressures, pipe flow rates, hydraulic grades, and pumping rates using Hazen Williams friction equations. These numerical models incorporate the algorithms contained in the standard KYPIPE2™ computer program. A sample computation and sample network schematics are shown in Plate I-C-6.

4. Design Results. Pipeline design results for the project area are presented in Plate I-C-11. It summarizes pipelines that are identified by the standard project numbering system for pipelines. The plate presents pipeline length, required capacity, material selected, pipe diameter, pump horsepower, and actual capacity. Some segments of the delivery system were originally designated to be served by canals, but during design were converted to pipeline systems, due to unfavorable site conditions such as limited right-of-way or unfeasible depth of cut. These new pipelines retain the original canal numbering convention.

1-C-07. INVERTED SIPHONS.

1. General. It is necessary to keep delivery system flow separate from natural stream flow (except where the natural streams are intentionally used to receive canal flow as components of the delivery system). The alignment of the delivery system canals would block intersecting natural streams if no provision were made to reroute flow in either the canal or the stream. Locations were identified on USGS 7.5' quadrangle maps where this would occur, and inverted siphons were sized for these locations. A conceptual drawing of an inverted siphon is provided in Plate I-C-12. In most cases the design discharge for the natural stream was less than the design discharge for the canal and a smaller siphon resulted by passing the natural stream under the canal.

Where the canal was passed under the natural stream, the required capacity of the siphon was known. For the typical case where natural streams were routed under canals, it was necessary to determine the required capacity. Therefore, the natural stream drainage areas were delineated in order to determine runoff for the design storm. Based on a 50-year project design life, a 100-year design event was selected to size the siphons. A head of 0.3 foot on the siphons was adopted as the allowable head that would produce insignificant increases in upstream water elevations. Using the design event and maintaining this minimum head on the siphons, a spreadsheet was built in order to analyze each of the structures using the NRCS equation for time of concentration, the Rational Method for predicting peak discharge, and the Hazen-Williams head loss equation for flow in pipes.

2. Design Data. The inverted siphon design procedure required the following values as input: drainage basin area, either the runoff coefficient and rainfall intensity for the Rational Method (C and i) or the Q_{100} from a previous study if available, and energy loss coefficients for entrance, exit, bend, and friction losses. Based on previous studies using the Rational Method to compute runoff for a basin, small sub-basins (1 square mile or less) produced peak discharges that were comparable with gage records. Larger sub-basins tended to yield peak discharges that were higher than gage records. Because most of the sub-basins were small, it was decided that the Rational Method would be used for runoff determination and that larger than recommended sub-basins would be slightly over-designed to account for inaccuracies in quadrangle maps used in the design.

Drainage basin areas for each siphon were planimetered from USGS quadrangle maps. In the cases where one basin drained into another, the value entered in the spreadsheet reflected the total combined area.

The runoff coefficient selected for the Rational Method was C=0.5, which is suitable for flat, cultivated land with clay and silt loam [18]. In order to approximate rainfall intensity, five representative siphon locations were selected for their catchment characteristics. The time of concentration was calculated for each drainage area according to the NRCS equation [19], which was developed for agricultural watersheds and is well suited for application to the project. The equation is:

$$tc = \frac{100(L)^{0.8}\left(\frac{1000}{CN} - 9\right)^{0.7}}{1900(S)^{0.5}}$$

where "L" is the length of the watercourse in feet, "S" is the average slope in percent, and "CN" is the NRCS runoff curve number. The time of concentration, t_c, is equal to 1.67 times the basin lag. A curve number of 90 was adopted [20]. This value corresponds to fallow agricultural lands having slow to very slow infiltration rates, and is representative of project cropland during periods of little vegetative cover. Values for time to concentration were calculated for the five areas and were averaged to obtain a single value of 3.2 hours. Based on a time of concentration of 3.2 hours, a rainfall intensity of 1.66 inches/hour was derived. Using the TP-40 rainfall maps [21], this was found equivalent to a 100-year, 3-hour storm.

The Rational Method is considered applicable to catchments smaller than 1 square mile, but some siphon locations had drainage areas that exceeded this limit. Since the values given by the Rational Method could overestimate the design discharge, the design discharge determined from a previous study was used for design if available.

Tailwater depth for the siphons was assumed to equal the diameter of the selected pipe size unless a flowline elevation was available for use from a previous study.

Values for energy loss coefficients came from a number of different sources. An entrance loss coefficient of K=0.8 was adopted for pipes projecting from fill [22]. The exit loss coefficient used was K=1.0 [23]. The value adopted for a 45 degree bend was equal to K=0.16 [24]. All energy losses mentioned above were calculated by using:

$$h_L = K\left(\frac{V^2}{2g}\right)$$

The conduit friction loss was estimated by the Hazen-Williams head loss equation:

$$h_L = 3.02(L)(D)^{-1.167}\left(\frac{V}{C}\right)^{1.85}$$

where "L" was set equal to 250 feet, "D" was the pipe diameter in feet, and "V" was the velocity in fps. The length, L, was standardized at 250 ft in order to provide for head losses associated with the longest siphon lengths expected in the project area. The coefficient "C" for a corrugated metal pipe (CMP) was C=60 [25].

The data used to design each inverted siphon is presented in Plate I-C-9. This plate presents a summary of design data, as well as resultant siphon dimensions, for the inverted siphons that convey natural stream flows under a canal. Design data includes drainage area, Rational Method "C" values, the reference discharge determined during the previous study, the design discharge, head and tailwater, and loss coefficients.

3. <u>Design Method</u>. The design was based on the criterion that headwater would be limited to 0.3 feet above the top of the pipe when passing the Q_{100} through the siphon. Based on the largest canal size in the water delivery system, the inverted siphons were assumed to have a total length of 250 feet, a conduit slope of 1%, and four 45-degree bends. This configuration translates to vertical drop of about 2.34 feet between the entrance and the exit. For conditions where no tailwater was available or tailwater is equal to the diameter of the pipe, the following equation was used:

$$HW = (h_{entrance} + h_{exit} + h_{bends} + h_{friction}) - 2.34$$

where "HW" is headwater is feet above the top of the pipe, and head losses are expressed in feet. Where a tailwater was available and not equal to the diameter, the equation for headwater was modified to the following:

$$HW = TW - D + (h_{entrance} + h_{exit} + h_{bends} + h_{friction}) - 2.34$$

where "TW" is tailwater above the downstream invert in feet, "D" is the diameter of the pipe in feet, and all other values remain as previously defined. Values of available diameter CMP were 24, 30, 36, 42, 48, 54, 60, 66, 72, 78, 84, 90, and 96 inches. An iteration was performed looking at one pipe at each size and increasing the size, then going to two pipes and increasing the size, and so on until the 0.3 feet above the top of the pipe on the upstream end criterion was obtained.

4. Design Results. Design results are presented in Plate I-C-13. Design results consist of a conduit diameter and the number of parallel conduits required.

1-C-08. TURNOUTS (PUMP TYPE AND GATEWELL STRUCTURES)

1. General. Turnouts are used to divert water from a segment of the delivery system toward one or more water users. Water may be diverted from a canal, from an existing stream, or from a pipeline. Typically the water exiting a turnout enters a smaller receiving channel, but some turnouts divert water into pipelines. The USBR publication "Design of Small Canal Structures" was used as a guide in designing project turnouts [26].

Five types of project turnouts are provided for, as described below:

Type-1. This gravity flow steel pipe structure is the most typical turnout, where water is drawn from the side of a canal and is diverted through the canal bank into the upstream end of a receiving channel. As shown in Plate I-C-14, a horizontal inlet conduit conveys water to a gate housed in a riser. Flow passes through the gate and horizontal outlet conduit to the receiving channel. The plate also shows the riprap outlet protection, which protects the channel from scour. Plate I-C-15 is a plan view of a Type-1 turnout, showing the relationship between the canal embankments and the embankments of the receiving channel.

Type-2. This type differs from Type-1 in that the water is diverted into a buried pipeline instead of an open channel, as shown in Plate I-C-16. No riprap protection is required.

Type-3. This type differs significantly from Types-1 and -2, due to the presence of a pump for forcing diverted flow through a buried pipeline. It is used at locations where a high pipe outlet elevation or intervening high ground prevents gravity flow from the turnout. As shown in Plate I-C-17, no gate is required in this type, since the pump itself provides control.

Type-4. This type is identical to Type-2, except the turnout is located at the very end of a canal, rather than being located along the side of a canal, as shown in Plate I-C-18.

Type-5. Unlike all previously described types, this turnout receives water from a buried pipeline and diverts flow into an open channel. As shown in Plate I-C-19, the construction is very similar to the Type-1 turnout, except no riprap protection is provided.

Type-3 turnouts were designed in the 3-C-06, dealing with pipeline and pump design, and are considered in this document to be a pump-type turnout. All other turnouts in this document will be considered gatewell structures. All design data stated in the following paragraphs deal with the design of gatewell structures.

2. Design Data. Data required for turnout design included design discharge, inlet head, and tailwater depth. Also required data included estimated values of minor loss coefficients and pipe roughness.

3. Design Method. For Type-1 turnouts pipe flow velocity affects outlet stability and also affects design pipe diameter. It was necessary to select a typical velocity in order to proceed with other aspects of turnout design. The USBR recommends that structures discharging into an unlined channel should have a maximum pipe velocity of approximately 3.5 feet per second [27]. Alternatively, turnouts with protected outlets have a maximum permissible pipe velocity of approximately 5 feet per second. Given the erodible project area soils, it was believed that scour would occur even at the lower velocity, so riprap protection and a velocity of 5.0 feet per second was selected. The USBR recommends a minimum turnout pipe diameter of 18 inches [28], which was adopted as the minimum diameter for the project.

Once the pipe size was determined, the turnout elevations were designed to overcome the head loss for the required maximum delivery discharge and minimum upstream pool elevation. Total energy (head) loss through a turnout is comprised of pipe entrance loss, friction loss, and exit loss. The equation for head loss is shown below:

$$h_l = 0.78h_v + LS_f + h_v$$

where h_l is total head loss in feet, h_v is the velocity head in feet, L is the length of the horizontal conduit, S_f is the friction slope in feet per feet. A factor of 1.78 times the velocity head was added to the total energy loss and subtracted from the control water surface to set the final turnout invert. The USBR recommends this factor be used to ensure proper submergence of the turnout to promote smoothness of flow and accuracy of water measurement devices incorporated into the structure [29].

For gravity flow turnouts, the gate and appurtenances used to control flow rate are housed in a steel pipe riser. In order to house the vertical gate and to facilitate attachment to the horizontal conduit, it is desirable for the riser to have a greater diameter than the horizontal conduit. A riser diameter 1.5 times the diameter of the horizontal conduit economically provides the needed enlargement and was selected as a standard proportion in the turnout design. Gate sizes were approximated as equal to the horizontal conduit diameter.

4. Design Results. The results of gatewell structure design are presented in Plate I-C-20, including associated canal number, diameter of conduit, gate size, and number of parallel conduits. For pump-type turnouts, design results are located on Plate I-C-7. Additional information used to design the gatewell structures and weirs is located on Plate I-C-21.

1-C-09. WEIRS.

1. General. As described in 3-C-08, it is essential that a sufficient depth of water be pooled at the inlet of each turnout in order for the turnout to attain its design discharge. This requirement applies along existing ditches, as well as along canals. Weirs were designed to create the necessary pools along the existing ditches.

Under existing conditions, the existing ditches in the project area convey flows resulting from rainfall-runoff events. At a given point along an existing ditch, a maximum bankfull discharge capacity exists, and landowners conduct their farm operations accordingly. Since the with-project existing ditches will continue to convey storm runoff flows, a reduction in bankfull capacity would tend to cause more frequent out of bank flows and consequently increase flood damages. Therefore, the goal of the weir design process was to establish a pool for irrigation operations without significantly reducing the bankfull discharge capacity.

2. Design Data.

Data required for design of weirs included topography, weir height, and irrigation flow demands. From survey information, HEC-RAS and HEC-HMS models were constructed and used to analyze weir hydraulics. Irrigation water flows were added to the flows used in the HEC-RAS models to show with-project conditions. The following table, Table I-C-6, shows bankfull discharges (< 1.01 year) with, and without, irrigation flows for each existing waterway in the project.

Table I-C-6

STREAM NAME	STREAM NUMBER	PEAK IRRIGATION ADDED (CFS)	NO. OF WEIRS
Indian Bayou Ditch/Manmade Canals	1500	222	4
	1530	51	1
Caney Creek Ditch/Manmade Canals	2100	654	2
	2120	188	5
	2140	125	1
	2160	10	1
Crooked Creek Ditch	2200	394	6
Big Ditch	2240	70	4

Blue Point Ditch	2532	76	4
Skinner Branch	2530	144	11
Shumaker Branch	2540	37	6
White Oak Branch	2511	68	9
Rickey Branch	4110	187	2

Site-specific water demand and changes in the project boundaries affected the process of weir design.

Data required to size was pool elevation, minimum pool elevation, and the weir's drop height, H, for the design discharge. These values for each weir location are included in the plates summarizing design results (Plate I-C-22).

3. Design Method.

Weirs were sited in the following locations: (1) locations along the natural waterway that would provide service to tracts not serviced by a canal or by a major pipe system, (2) intermediate locations providing the deepest pool to as many tracts as possible, and (3) locations where the pool from the next downstream weir ends, provided that location is not in the middle of a tract already supplied.

In order to obtain maximum performance from each weir, it was desirable to set weirs at the highest elevation possible. It was decided to set weir elevations by determining the highest elevation that can convey the existing bankfull discharge with no significant increase in flowline elevation.

Structure dimensions were based in part on a standard design for the United States Bureau of Reclamations Water Measurement Sructures. The weir structures to be used in the Bayou Meto Irrigation Project are the Cipolletti weirs. Weirs have advantages and disadvantages, but the ability to accurately measure a wide range of flow while forming the necessary pumping pools made these weirs an important part of the delivery system. The equation that will be used to measure flow over the Cipolletti weirs is:

$$Q = 3.367LH^{3/2}$$

where L is the length of notch and H is difference between upstream pool elevation and the elevation of the notch [30].

The length of a stilling basin can be established determining the most critical discharge/tailwater condition, and using the following equations:

$$X_b = \left[3.54 + 4.26\left(\frac{H}{Y_c}\right)\right] Y_c$$

and

$$L_{b=} 2(X_b)$$

where "L_b" is the length of the stilling basin, measured from the weir to the beginning of the downstream channel. "X_b " is defined as the distance between the weir and the baffle pier or plate. Although baffle piers and plates are not used on the project stilling basins, because of low velocities, it was necessary to calculate the X_b value to be used in the L_b calculations. "H" is the absolute drop height and "Y_c "is the critical depth for the design discharge.

Velocity of flow over the weirs was calculated using a standard step backwater approach in order to determine the riprap size required for stability. The flow was considered subcritical when the computed water surface elevation (CWSEL) was greater than the critical water surface elevation (CRIWS) and/or the channel Froude number (FRCH) was less than unity, while for supercritical flow the converse was true. The occurrence of supercritical flow affects the solution technique for determination of flow lines, but also from an operational standpoint the significance of supercritical flow is that it typically has more potential to cause erosion or to displace riprap than does subcritical flow. Velocities in the upstream and downstream regions of the weir were found to be between 0.50 and 2.50 feet per second respectively and typically about one foot per second.

Rip-rap thickness and gradations were designed using to Standard LMVD Gradation Tables [31], Hydraulic Design Criteria [32], and EM-11-2-1601 [33], using velocities based on a range of low and high flows in an HEC-2 model. According to the Standard Gradation Table, for a specific weight of stone of 155 pounds per cubic foot, and using the Isbach Equation for highly turbulent flow, a rip-rap layer 18" thick and a median stone weight within the range of 20 to 40 pounds sustain velocities as high as 6.7 feet per second. This far exceeds any velocities encountered in the HEC-2 runs, but it is standard Corps practice to use an 18" minimum thickness and this thickness was adopted.

4. Design Results. The results of weir design are presented in on Plate I-C-22 and shows dimension data for the Cipolletti weirs; Plate I-C-23 shows a typical weir section; and Plate I-C-24 shows a typical channel profile for Indian Bayou Ditch, showing how the weirs form pumping pools along the stream.

1-C-10. DROP INLET STRUCTURES.

1. General. The ditches being used in the irrigation delivery system have numerous small field drains the convey field runoff to the ditch. There was a concern that if measures were not taken to reduce the sediment introduced into ditches from these field drains, and then the sediment would begin to fill in ditches used in the irrigation delivery system to rapidly. The drop inlet structures will reduce the amount of sediment entering the ditches because it will drop out behind the structure.

2. Design Results. After field investigations and topographic map evaluations, locations for these structures were determined and a standard drop inlet structure was designed, since the field drains selected were uniform in size and runoff characteristics. Plate I-C-25 shows the typical design.

1-C-11. ROAD CROSSINGS.

1. General. The open channel portion of the delivery system intersects many roadways, including state highways, county roads, and private on-farm roads. It was necessary to provide for the conveyance of canal flows past these roadways. Bridges were designed for road crossings over most of the canals, and bridges were used across existing ditches where enlargement of the ditch or increased flow made it necessary to replace the existing structure with a bridge.

2. Design Data. The information required to design the bridges included the maximum demand discharge of the canal and the amount of canal freeboard available on the upstream side of the bridge. Bridge hydraulic design required low chord elevations and pier dimensions, and loss coefficients.

3. Design Method. Following the determination of canal alignment, design freeboard, and flowlines, the road crossings were designed. Road crossings were designed to have very low head losses, such that the effect on canal hydraulics would be negligible and the previously computed flowlines would remain valid.

TOPIC D – CONTROL SYSTEM

1-D-01. GENERAL.

In order to efficiently manage water in the irrigation delivery system, a control system is being designed for the Bayou Meto area. The system consists of actuated control gates, level and flow measurements and setpoints, control signals, calculations, consequent gate movements, and consequent hydraulic dynamics. Based on downstream control and the concept of feedforward and decoupling as described in Jan Schuurmans' thesis, an integral model of a hydraulic model and a real time control system is under development. The hydraulic model is being used to simulate the response of the system under a variety of flow conditions and various operational schemes. The model gains insight into the operation of the system by analyzing the system characteristics and developing control algorithms for the system. The real time control system is applied to operate the hydraulic model through a real time control interface in the model. The system is composed of two major components: (1) the setting of the scheduled flow to the (lateral) off take structures; and (2) the master slave controller that controls the water levels of the main channel pools. With this system, the water deliver system is able to simulate water flowing in each delivery structure and assure users on the last reach having an equal basis with users on the first reach.

Downstream control is a water control method of determining the water level on a check structure by looking at the level of the downstream structure. In general, downstream control is more complicate than upstream control, but it has less time delay (lag time), more flexible in flow adjustment and more equitable access to water user. The operation of this downstream control system depends on the hydraulic configuration in the whole system. In the Bayou Meto project, the hydraulic components include:

1. 105 miles of canal
2. 116 miles of ditch
3. 472 miles of pipeline
4. 56 weirs
5. 183 pumps
6. 86 inverted siphons
7. 14 turnouts
8. 92 drop structure

1-D-02. DESIGN DATA.

Information required for control system design included cross sections of the channel, design discharges, upstream and downstream water surface elevations, types of control structure, dimensions of control structure, locations of the structure, and the operation capacities of the structure.

1-D-03. DESIGN METHOD

The design of a control system is based on the flow discharges and surface water elevations, which is developed in the Sobek hydraulic model. The flow data will be sent to the control system and to perform the operation simulations in the water delivery system. The control system model is a flexible implementation model written in Matlab. Matlab is interfaced to the Sobek flow module by the Sobek Real Time Control module. As the real time control module reports all levels and flows needed by the controller to Matlab at each timestep, Matlab calculates control actions needed at the gates and passes that action back to the flow module for inclusion of any new flow calculations in the next timestep. A flow schedule will send anticipated flows for each check structure, lateral gate or pump to the main canal controller. The controller will send flow settings to checks, gates and turn pumps according to the schedule.

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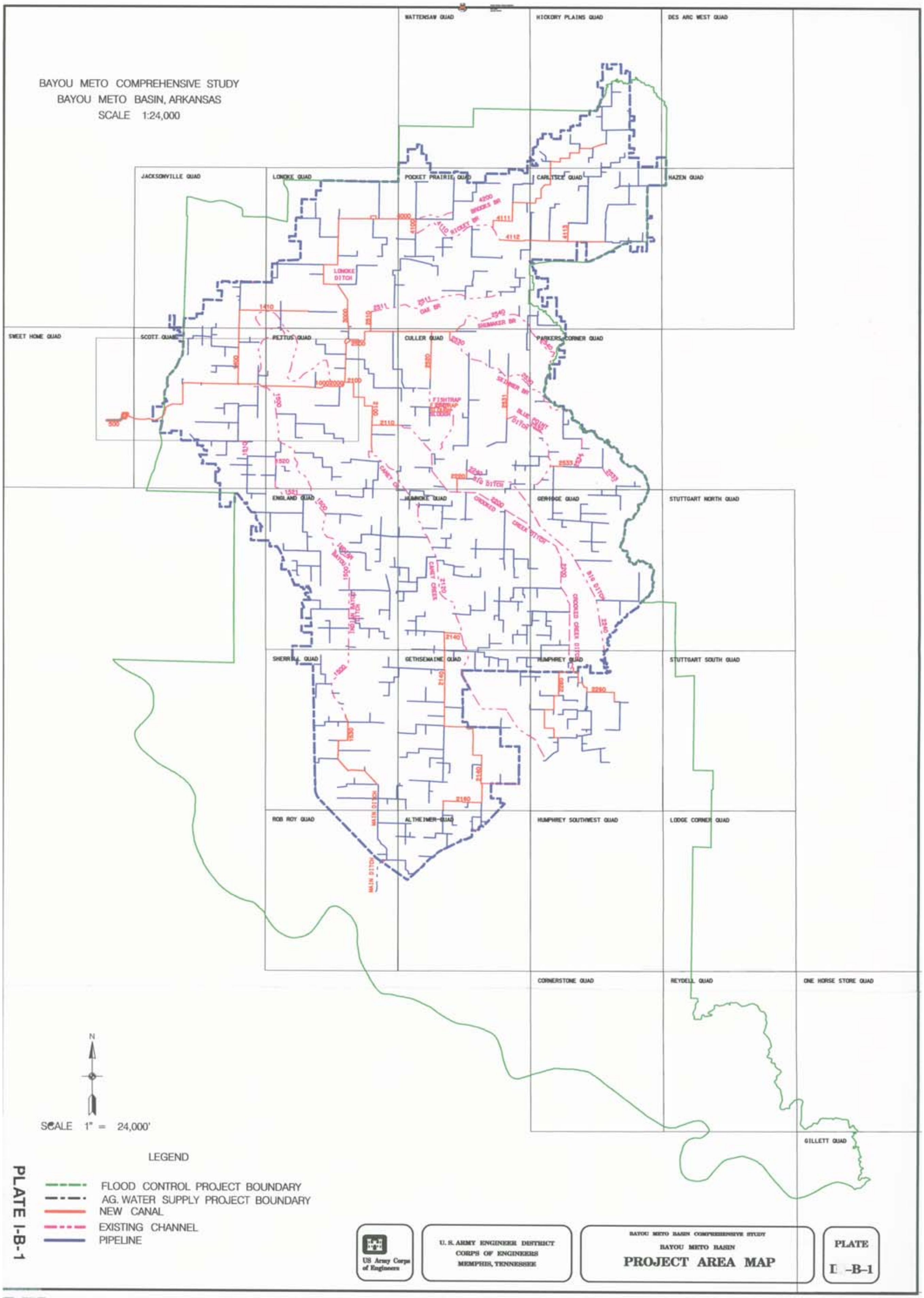

BAYOU METO COMPREHENSIVE STUDY
BAYOU METO BASIN, ARKANSAS
SCALE 1:24,000
WATTENSAW QUAD
HICKORY PLAINS QUAD
DES ARC WEST QUAD
JACKSONVILLE QUAD
LONOKE QUAD
POCKET PRAIRIE QUAD
CARLISLE QUAD
HAZEN QUAD
SWEET HOME QUAD
SCOTT QUAD
PETTUS QUAD
CULLER QUAD
PARKERS CORNER QUAD
ENGLAND QUAD
HUMNOKE QUAD
GERMANIA QUAD
STUTTGART NORTH QUAD
SHERRILL QUAD
GETHSEMAINE QUAD
HUMPHREY QUAD
STUTTGART SOUTH QUAD
ROB ROY QUAD
ALTHEIMER QUAD
HUMPHREY SOUTHWEST QUAD
LODGE CORNER QUAD
CORNERSTONE QUAD
REYDELL QUAD
ONE HORSE STORE QUAD
GILLETT QUAD
LONOKE DITCH
BROOKS BR
RICKEY BR
OAK BR
SHUMAKER BR
SKIMMER BR
BLUE POINT DITCH
FISHTRAP SLOUGH
CANEY CR
CANEY CREEK
BIG DITCH
CROOKED CREEK DITCH
INDIAN BAYOU DITCH
MAIN DITCH
500
1410
1500
1520
2100
2110
2140
2160
2250
2510
2511
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2533
2540
4100
4111
4112
4200
5000
N
SCALE 1" = 24,000'
LEGEND
FLOOD CONTROL PROJECT BOUNDARY
AG. WATER SUPPLY PROJECT BOUNDARY
NEW CANAL
EXISTING CHANNEL
PIPELINE
US Army Corps of Engineers
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MEMPHIS, TENNESSEE
BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN
PROJECT AREA MAP
PLATE
I-B-1

PLATE I-B-1

Bayou Meto Irrigation Study - Source Reliability Plot
Arkansas River - Period of Record (1940-1996)

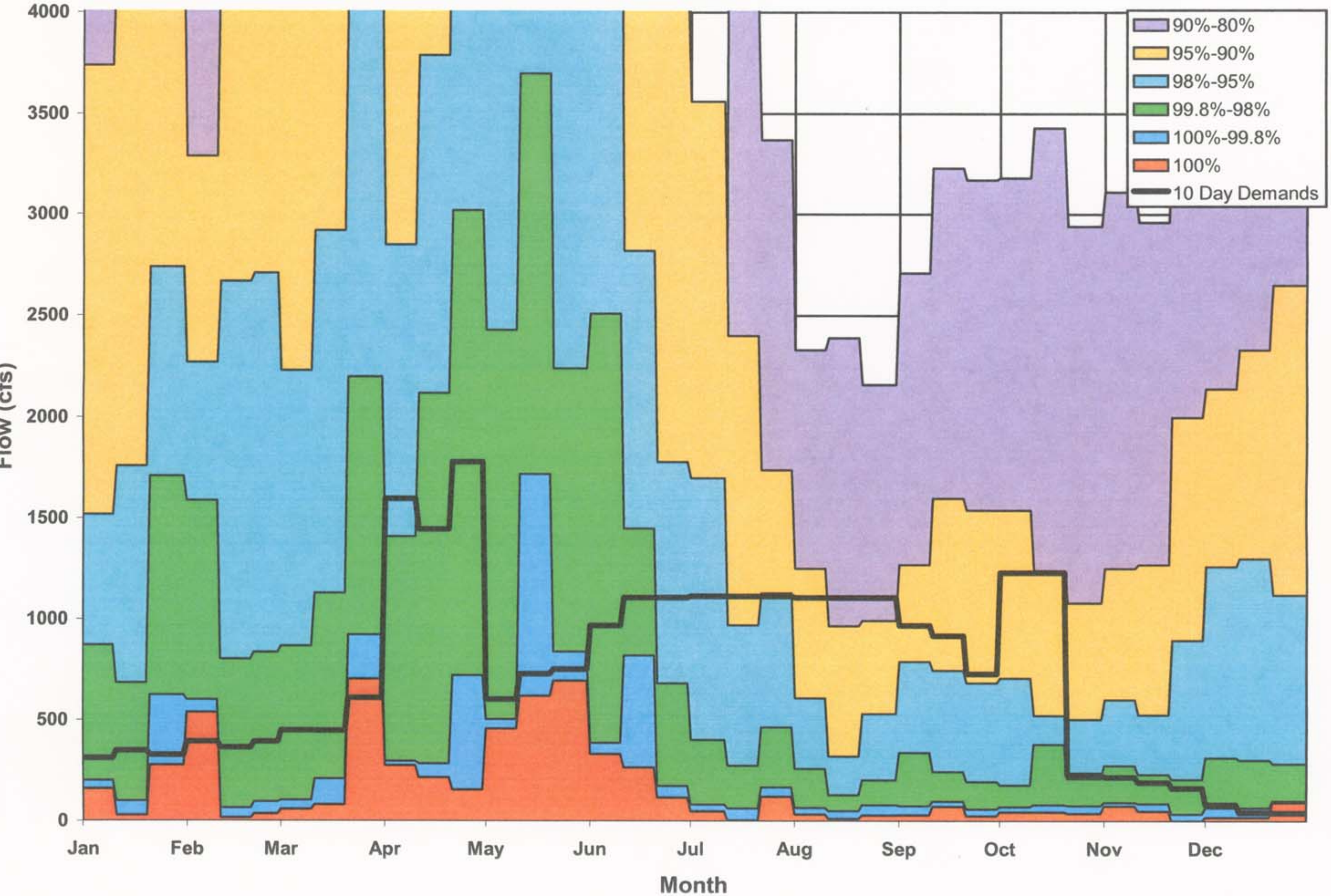


PLATE 1-B-2

Canal 1000/2000 Profile Comparison

08 March 2000

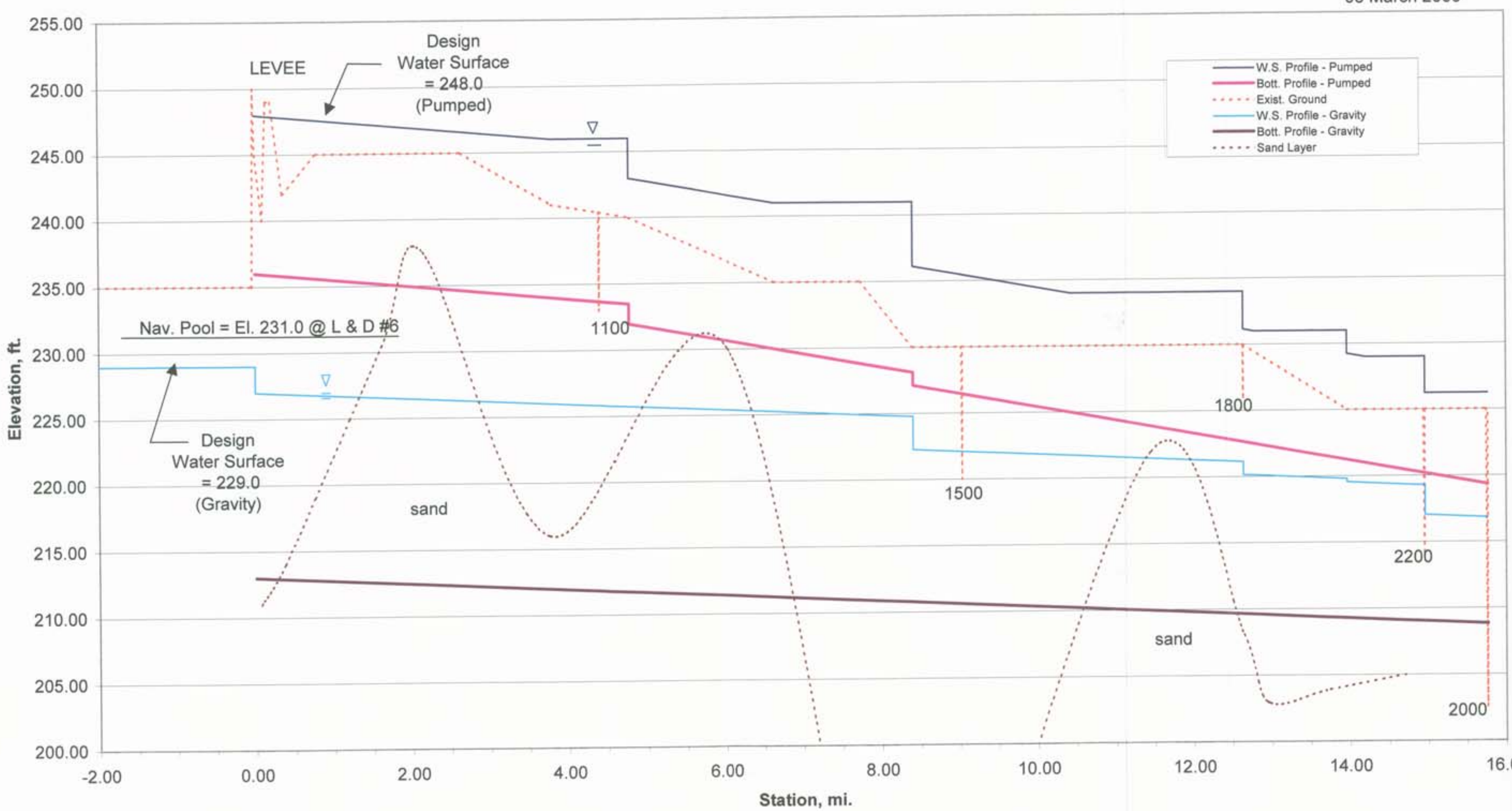


PLATE I-C-1

Bayou Meto
Pumped Flow Alternative
Canal 1000/2000

9-Mar-00

PRELIMINARY

a.) Pumping Station:

	First Cost	Annual O&M Cost	Annual O&M Cost per cfs
1700 cfs	$26,647,500	$935,000	$550
sub-total:	$26,647,500	$935,000	

b.) Re-lift Pumps Required:

	Canal	cfs.	Lift	First Cost	Annual O&M Cost	Annual O&M Cost per cfs
1.	1421	343	18'	$6,534,150	$220,549	$643
2.	2400	135	30'	$3,048,705	$88,425	$655
sub-total:		478		$9,582,855	$308,974	$646

c.) Relocations:

	First Cost
Exist. Roads, Utilities, Bridges, etc. =	$7,386,000
sub-total:	$7,386,000

d.) Real Estate:

	First Cost
Est. 580 ac. R.O.W. =	$1,440,000
sub-total:	$1,440,000

e.) Excavation:

	First Cost
Est. 3,000,000 CY @ $1.50 =	$4,500,000
sub-total:	$4,500,000

	First Cost	Annual O&M Cost
TOTAL:	$49,556,355	$1,243,974

PLATE I-C-2 (1)

9 MARCH 00

First and Annual Costs for Pumping Station

FY	N	Pumping Station	Re-lift Pumps	Relocations	Real Estate	Excavation	Total Const	PV Factor @ 6.625%	PV First Cost
1	5			1,846,500	360,000		2,206,500	1.37815	3,040,888
2	4	5,329,500	1,916,571	1,846,500	360,000	900,000	10,352,571	1.29252	13,380,905
3	3	5,329,500	1,916,571	1,846,500	360,000	900,000	10,352,571	1.21221	12,549,490
4	2	5,329,500	1,916,571	1,846,500	360,000	900,000	10,352,571	1.13689	11,769,734
5	1	5,329,500	1,916,571			900,000	8,146,071	1.06625	8,685,748
6	0	5,329,500	1,916,571			900,000	8,146,071	1.00000	8,146,071
		26,647,500	9,582,855	7,386,000	1,440,000	4,500,000	49,556,355		57,572,836
								0.06625	3,814,000
								0.00279	161,000
								0.06904	3,975,000

Interest	3,814,000
Sinking Fund	161,000
Operation and Maintenance	1,243,974
Total	5,219,000

Bayou Meto
Gravity Flow Alternative
Canal 1000/2000

9-Mar-00

PRELIMINARY

a.) Slide Gate Structure:	First Cost	Annual O&M Cost
8 Gate Structure	$6,000,000	$15,000
sub-total:	$6,000,000	$15,000

b.) Re-lift Pumps Required:

	Canal	cfs.	Lift	First Cost	Annual O&M Cost	Annual O&M Cost per cfs
1.	1100	43	12'	$967,586	$28,810	$670
2.	1200	31	12'	$708,629	$20,615	$665
3.	1400	483	12'	$9,291,954	$304,773	$631
4.	1410	34	12'	$773,160	$22,644	$666
5.	1421	343	18'	$6,534,150	$220,549	$643
6.	1500	168	12'	$3,564,288	$110,040	$655
7.	1700	19	12'	$441,104	$12,578	$662
8.	1800	39	12'	$886,900	$25,935	$665
9.	2100	123	12'	$2,648,682	$80,688	$656
10.	2200	298	12'	$6,086,650	$191,837	$644
11.	2400	135	30'	$3,048,705	$88,425	$655
	sub-total:	1716		$34,951,808	$1,106,894	$645

c.) Relocations:	First Cost
Exist. Roads, Utilities, Bridges, etc. =	$10,602,000
sub-total:	$10,602,000

d.) Real Estate:	First Cost
Est. 1200 ac. R.O.W. =	$2,660,000
sub-total:	$2,660,000

e.) Excavation:	First Cost	
Est. 10,500,000 CY @ $1.50 =	$15,750,000	
Est. 1,500,000 CY @ $3.00 =	$4,500,000	
sub-total:	$20,250,000	

	First Cost	Annual O&M Cost
TOTAL:	$74,463,808	$1,121,894

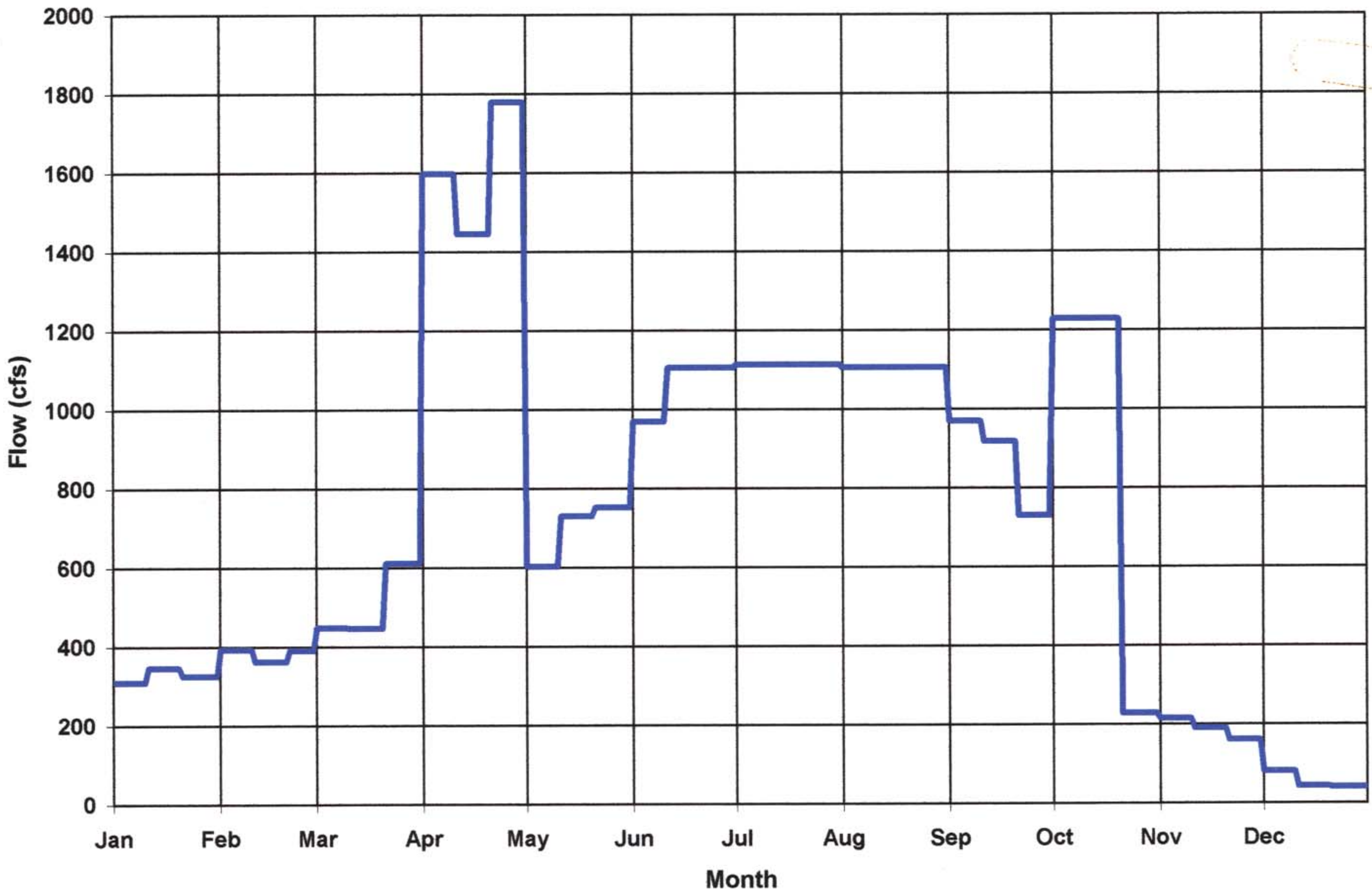
9 MARCH 00

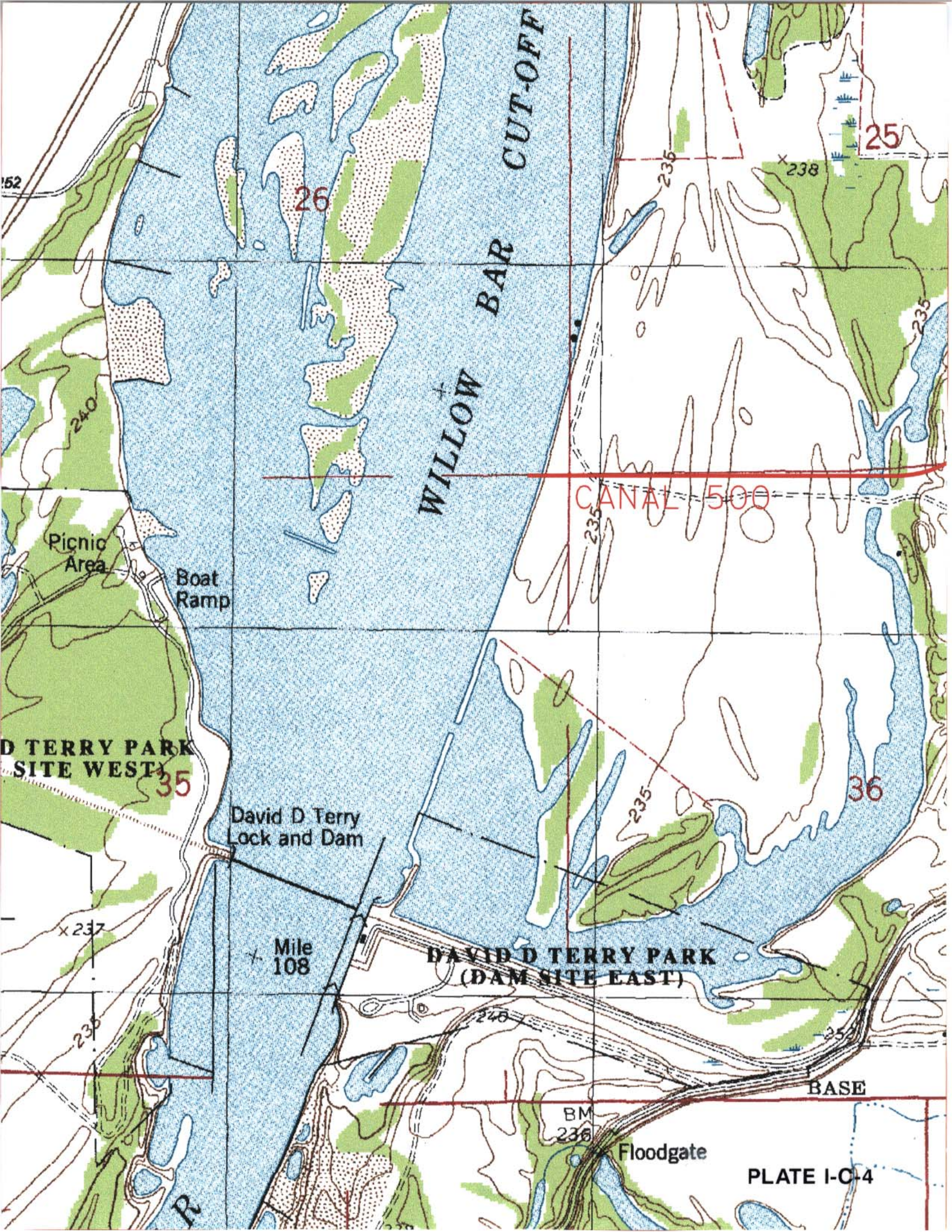
First and Annual Costs for Gated Structure

FY	N	Gated Structure	Re-lift Pumps	Relocations	Real Estate	Excavation	Total Const	PV Factor @ 6.625%	PV First Cost
1	5			2,650,500	665,000		3,315,500	1.37815	4,569,256
2	4	1,200,000	6,990,362	2,650,500	665,000	4,050,000	15,555,862	1.29252	20,106,262
3	3	1,200,000	6,990,362	2,650,500	665,000	4,050,000	15,555,862	1.21221	18,856,971
4	2	1,200,000	6,990,362	2,650,500	665,000	4,050,000	15,555,862	1.13689	17,685,303
5	1	1,200,000	6,990,362			4,050,000	12,240,362	1.06625	13,051,286
6	0	1,200,000	6,990,362			4,050,000	12,240,362	1.00000	12,240,362
		6,000,000	34,951,808	10,602,000	2,660,000	20,250,000	74,463,808		86,509,440
								0.06625	5,731,000
								0.00279	242,000
								0.06904	5,973,000

Interest	5,731,000
Sinking Fund	242,000
Operation and Maintenance	1,121,894
Total	7,095,000

Bayou Meto Irrigation Study - Design Irrigation Demand Flows
Period of Record (1940-1996) - 10 Day Increments

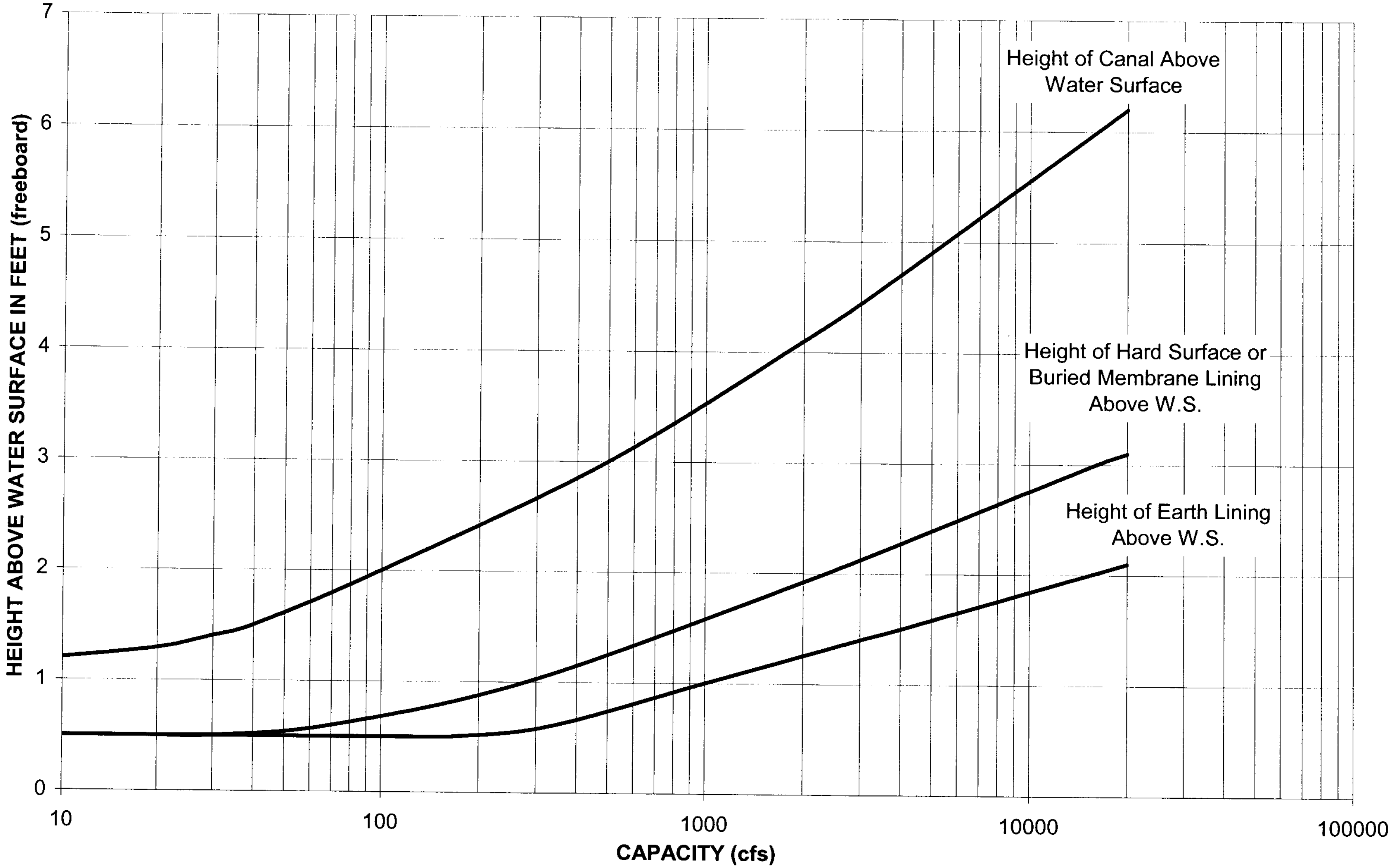




WILLOW BAR CUT-OFF
26
25
238
235
240
252
CANAL 500
Picnic Area
Boat Ramp
D TERRY PARK
SITE WEST)
35
36
David D Terry
Lock and Dam
237
Mile
108
DAVID D TERRY PARK
(DAM SITE EAST)
240
BASE
BM
236
Floodgate
R

PLATE I-C-4

GENERAL REQUIREMENTS AND DESIGN CONSIDERATIONS



Bayou Meto Main Canal Elevations — 1/29/2004

Canal Profiles.xls

CANAL 500 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	217.50	River; Q=1750	11.50	n/a	229.00	n/a	0.0001	4819	217.00	228.50	60	1.72
48+19	217.00	P-1000R	11.50	n/a	228.50	n/a					60	

CANAL 1000/2000 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	234.00	C-1000-R; Q=1750	11.00	4.0	245.00	249.00	0.0002	10000	232.00	243.00	40	2.27
100+00	232.00	Begin Constant Levee Grade	11.00	4.0	243.00	247.00	0.0002	13500	229.30	243.00	40	2.27
235+00	229.30	C-1000.1; Q=1750	11.00	4.0	240.30	244.30	0.0002	11500	227.00	238.00	40	2.27
350+00	227.00	Begin Constant Levee Grade	11.00	4.0	238.00	242.00	0.0002	7000	225.60	238.00	40	2.27
420+00	225.60	C-1000.2; Q=1357	9.00	4.0	234.60	238.60	0.0002	13000	223.00	232.00	35	2
550+00	223.00	Begin Constant Levee Grade	9.00	4.0	232.00	236.00	0.0002	18300	219.34	232.00	35	2
733+00	219.34	C-2000.1; Q=681	8.00	3.5	227.34	230.84	0.0002	1700	219.00	227.00	25	1.8
750+00	219.00	Begin Constant Levee Grade	8.00	3.5	227.00	230.50	0.0002	9179	217.16	227.00	25	1.8

CANAL 1400 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	231.00	T-1400.1; Q=166	5.00	2.5	236.00	238.50	0.0001	10000	230.00	235.00	20	0.96
100+00	230.00	Begin Constant Levee Grade	5.00	2.5	235.00	237.50	0.0001	10970	228.90	235.00	20	0.96
209+70	228.90	T-1410; Q=98	6.10	2.5	235.00	237.50						

CANAL 1410 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	228.00	T-1410; Q=98	4.00	2.0	232.00	234.00	0.0001	10000	227.00	231.00	20	0.85
100+00	227.00	Begin Constant Levee Grade	4.00	2.0	231.00	233.00	0.0001	9610	226.04	231.00	20	0.85
196+10	226.04	P-1410.02	4.96	2.0	231.00	233.00						

CANAL 1530 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	210.20	P-1530	3.50	2.0	213.70	215.70	0.0001	30287	206.10	209.60	15	0.76
302+87	206.10	P-1530.05	3.50	2.0	209.60	211.60						

CANAL 2100 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	218.00	C-2000.1; Q=668	8.00	3.5	226.00	229.50	0.0002	14979	215.00	223.00	25	1.8
149+79	215.00	Begin Constant Levee Grade	8.00	3.5	223.00	226.50	0.0002	5150	213.97	223.00	25	1.8
201+29	213.97	C-2110; Q=221	5.00	2.5	218.97	221.47	0.0002	4854	213.00	218.00	20	1.35
249+83	213.00	Begin Constant Levee Grade	5.00	2.5	218.00	220.50	0.0002	2900	212.42	218.00	20	1.35
278+83	212.42	Exist. Chan. 2100	5.58	2.5	218.00	220.50						

CANAL 2110 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	213.87	C-2110; Q=426	7.00	3.0	220.87	223.87	0.0002	4335	213.00	220.00	25	1.67
43+35	213.00	Begin Constant Levee Grade	7.00	3.0	220.00	223.00	0.0002	4100	212.18	220.00	25	1.67
84+35	212.18	T-2200; Q=410	7.82	3.0	220.00	223.00						

CANAL 2140 and 2160 (Rev Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	196.00	P-2140; Q=126	4.50	2.0	200.50	202.50	0.0001	31783	192.80	196.80	20	0.9
317+83	192.80	W-2140.01; Q=45	4.00	2.0	196.80	198.80	0.0001	30818	189.70	193.70	10	0.78
626+01	189.70	W-2160.01; Q=11	2.50	2.0	192.20	194.20	0.0001	7000	189.00	190.80	5	0.56
696+01	189.00	Begin Constant Levee Grade	2.50	2.0	191.50	193.50	0.0001	7633	188.30	191.50	5	0.56
772+34	188.30	P-2160.03	3.20	2.0	191.50	193.50						

CANAL 2220 (completed RCT)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	204.00	P-2220; Q=70	4.00	2.0	208.00	210.00	0.0001	5000	203.50	207.50	15	0.82
50+00	203.50	Begin Constant Levee Grade	4.00	2.0	207.50	209.50	0.0001	3975	203.10	207.50	15	0.82
89+75	203.10	T-2240	4.40	2.0	207.50	209.50						

CANAL 2260 (Completed RCT)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	194.00	P-2260; Q=64	4.00	2.0	198.00	200.00	0.0001	15000	192.50	196.50	10	0.78
150+00	192.50	Begin Constant Levee Grade	4.00	2.0	196.50	198.50	0.0001	6054	191.89	196.50	10	0.78
210+54	191.89	P-2260.05	4.61	2.0	196.50	198.50						

CANAL 2280 (Completed RCT)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	191.00	P-2280; Q=53	4.00	2.0	195.00	197.00	0.0001	20000	189.00	193.00	10	0.78
200+00	189.00	Begin Constant Levee Grade	4.00	2.0	193.00	195.00	0.0001	8645	188.14	193.00	10	0.78
286+45	188.14	P-2280.08	4.86	2.0	193.00	195.00						

CANAL 2500 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	233.00	C-2500R; Q=306	6.00	3.0	239.00	242.00	0.0002	5000	232.00	238.00	20	1.5
50+00	232.00	Begin Constant Levee Grade	6.00	3.0	238.00	241.00	0.0002	4000	231.20	238.00	20	1.5
90+00	231.20	C-2500-1	5.00	2.5	236.20	238.70	0.0002	6000	230.00	235.00	20	1.35
150+00	230.00	Begin Constant Levee Grade	5.00	2.5	235.00	237.50	0.0002	7000	228.60	235.00	20	1.35
220+00	228.60	C-2500-2	5.00	2.5	233.60	236.10	0.0002	5500	227.50	232.50	20	1.35
275+00	227.50	Begin Constant Levee Grade	5.00	2.5	232.50	235.00	0.0002	3556	226.60	232.50	20	1.35
310+56	226.60	T-2530-1	5.90	2.5	232.50	235.00	0.0002	2381	226.12	232.50	20	1.35
334+37	226.12	T-2530-2	6.38	2.5	232.50	235.00						

CANAL 2510 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	232.50	T-2510	4.00	2.0	236.50	238.50	0.0002	2500	232.00	236.00	8	1.08
25+00	232.00	Begin Constant Levee Grade	4.00	2.0	236.00	238.00	0.0002	3590	231.28	236.00	8	1.08
60+90	231.28	T-2511	4.72	2.0	236.00	238.00						

CANAL 2520 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	226.00	T-2520	3.50	1.5	229.50	231.00	0.0001	10000	225.00	228.50	8	0.71
100+00	225.00	Begin Constant Levee Grade	3.50	1.5	228.50	230.00	0.0001	3700	224.63	228.50	8	0.71
137+00	224.63	P-2520	3.87	1.5	228.50	230.00						

CANAL 2531 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	221.00	P-2531	3.00	1.5	224.00	225.50	0.001	10000	211.00	214.00	10	2.11
100+00	211.00	Begin Constant Levee Grade	3.00	1.5	214.00	215.50	0.001	2009	209.00	214.00	10	2.11
120+09	209.00	T-2532	5.00	1.5	214.00	215.50						

CANAL 2533 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	204.25	P-2533	4.00	2.0	208.25	210.25	0.0001	2500	204.00	208.00	15	0.82
25+00	204.00	Begin Constant Levee Grade	4.00	2.0	208.00	210.00	0.0001	1096	203.89	208.00	15	0.82
35+96	203.89	P-2533.02	4.11	2.0								

CANAL 3000 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	232.00	T-3000R; Q=316	6.00	3.0	238.00	241.00	0.0002	10000	230.00	236.00	20	1.5
100+00	230.00	Begin Constant Levee Grade	6.00	3.0	236.00	239.00	0.0002	17920	226.42	236.00	20	1.5
279+20	226.42	C-3000; Q=297	6.00	3.0	232.42	235.42	0.0002	7100	225.00	231.00	20	1.5
350+20	225.00	Begin Constant Levee Grade	6.00	3.0	231.00	234.00	0.0002	14806	222.04	231.00	20	1.5
498+26	222.04	P-3000R: Q=260										

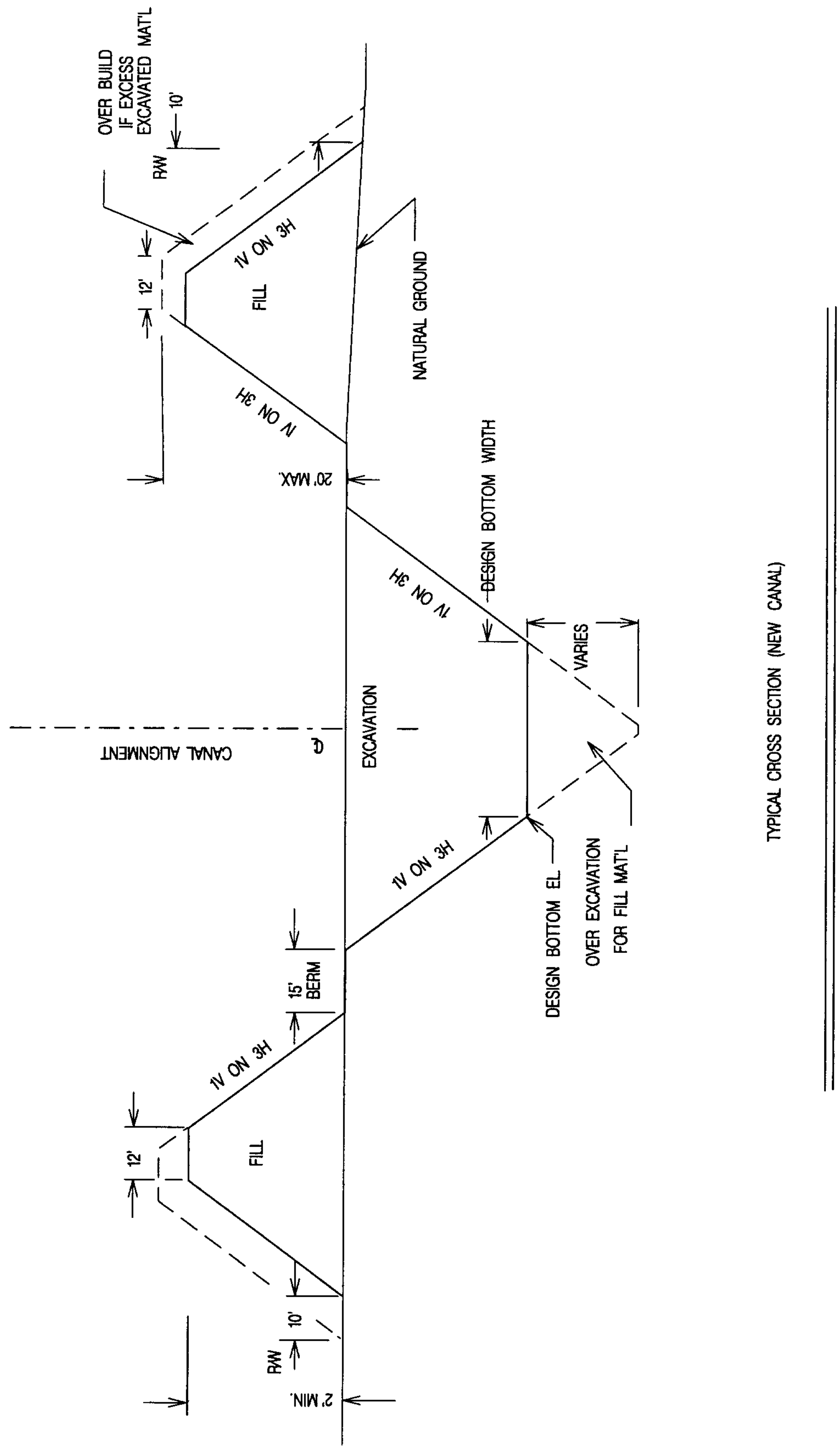
CANAL 4000/4100 (Completed SRO)

BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	241.00	C-4000; Q=241	5.00	2.5	246.00	248.50	0.0002	10000	239.00	244.00	20	1.35
100+00	239.00	Begin Constant Levee Grade	5.00	2.5	244.00	246.50	0.0002	1437	238.71	244.00	20	1.35
114+37	238.71	Begin Canal 4100	5.29	2.5	244.00	246.50	0.0002	3844	237.94	244.00	18	1.35
152+81	237.94	T-4110; Q=189										

CANAL 4111 (Completed SRO)												
BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	230.00	P-4111; Q=60	4.00	2.0	234.00	236.00	0.0001	20000	228.00	232.00	10	0.78
200+00	228.00	Begin Constant Levee Grade	4.00	2.0	232.00	234.00	0.0001	7500	227.25	232.00	10	0.78
275+00	227.25	C-4111.01	4.00	2.0	231.25	233.25	0.0003	20833	221.00	225.00	10	0.78
483+33	221.00	Begin Constant Levee Grade	4.00	2.0	225.00	227.00	0.0003	6168	219.15	225.00	10	0.78
545+01	219.15	P-4111.09										

CANAL 4112 (Completed SRO)												
BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	233.00	P-4112; Q=109	5.00	2.0	238.00	240.00	0.0001	15000	231.50	236.50	20	0.96
150+00	231.50	Begin Constant Levee Grade	5.00	2.0	236.50	238.50	0.001	15048	230.00	236.50	20	0.96
300+48	230.00	P-4112.07										

CANAL 4113 (Completed SRO)												
BEGINNING			DEPTH	FB	ELEVATION		SLOPE	DISTANCE	END		B.W.	VEL.
STA	ELEV	NOTE	(ft.)	(ft.)	W.S.	LEVEE	(ft./ft.)	(ft.)	ELEV	W.S.	(ft.)	(ft/s)
0+00	231.50	T-4113; Q=35	3.50	1.5	235.00	236.50	0.0001	2000	231.30	234.80	8	0.71
20+00	231.30	Begin Constant Levee Grade	3.50	1.5	234.80	236.30	0.0001	3255	230.97	234.80	8	0.71
52+55	230.97	P-4113.01										



US Army Corps of Engineers

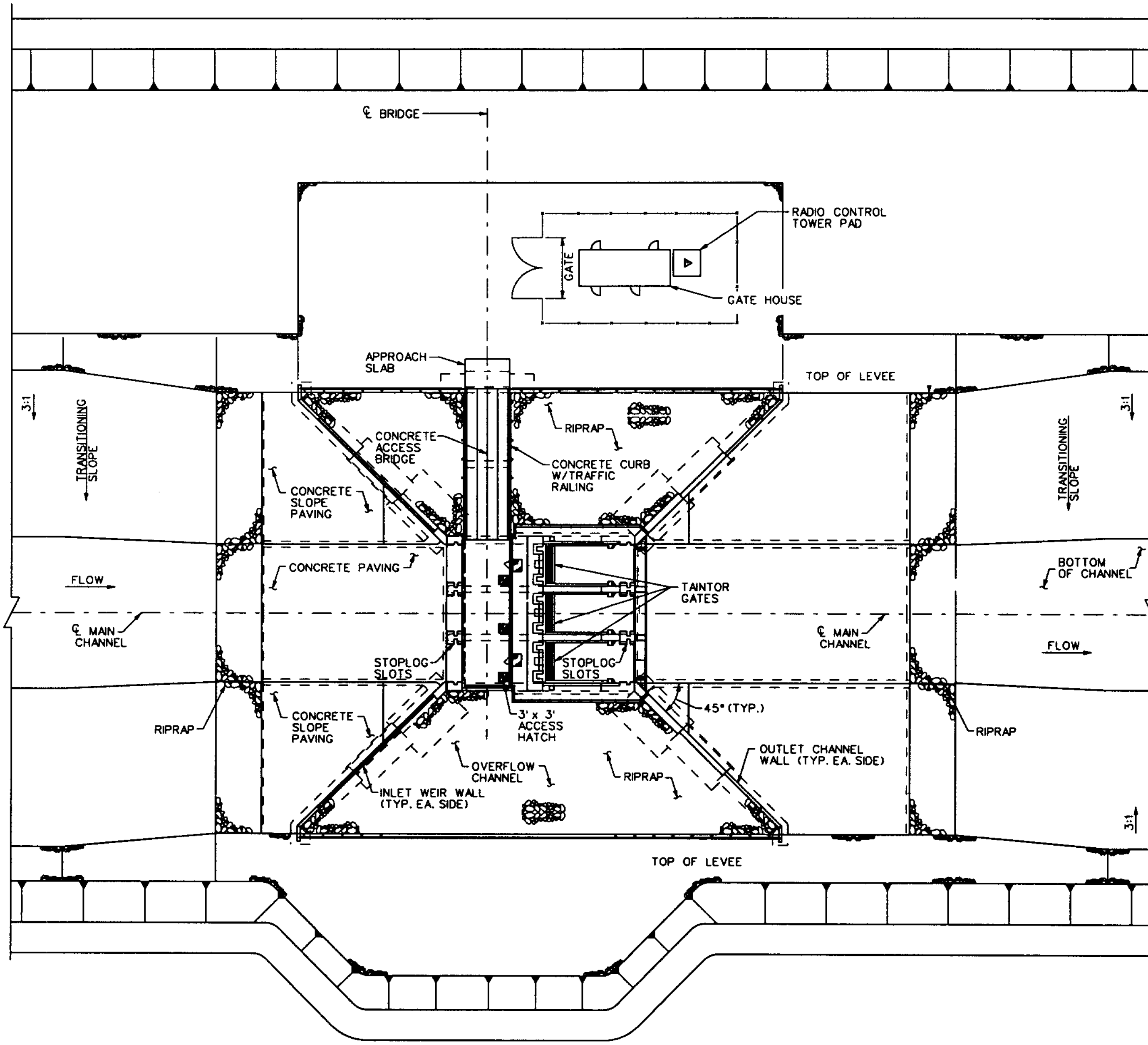
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEER
MEMPHIS, TENNESSEE

BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN

TYPICAL CANAL SECTION

PLATE
I-C-7

PLATE I-C-7



SITE PLAN

US Army Corps of Engineers

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEER
MEMPHIS, TENNESSEE

BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN
TYPICAL CHECK STRUCTURE
PLAN

PLATE
I–C–8

Miles (Reaches)	Channel Work	Side Slope	
Indian Bayou			
28.3 - 20.34	20' Bottom Width	1V:3H	Ag Water
20.34 - 16.4(58.3)	10' Bottom Width	1V:3H	Ag Water
58.3 - 49.8	15' Bottom Width	1V:3H	Flood Control
Caney Creek Ditch			
11.94 - 10.84	30' Bottom Width	1V:3H	Ag Water
10.84 - 8.34	20' Bottom Width	1V:3H	Ag Water
8.34 - 0.0	10' Bottom Width	1V:3H	Ag Water
Crooked Creek Ditch			
18.64 - 12.0	50' Bottom Width	1V:3H	Ag Water
12.0 - 10.1	50' Bottom Width	1V:3H	Ag Water
10.1 - 0.0	50' Bottom Width	1V:3H	Flood Control
Big Ditch			
17.1 - 6.84	Cleanout	Existing	Ag Water
Skinner Branch			
9.1 - 5.54	20' Bottom Width	1V:3H	Ag Water
5.54 - 3.3	20' Bottom Width	1V:3H	Ag Water
3.3 - 0.0	Cleanout	Existing	Ag Water
White Oak Branch			
7.74 - 0.34	20' Bottom Width	1V:3H	Ag Water
Shumaker Branch			
7.5 - 3.24	15' Bottom Width	1V:3H	Ag Water
Rickey Branch			
9.04 - 6.02	60' Bottom Width	1V:3H	Ag Water
6.02 - 5.32	40' Bottom Width	1V:3H	Ag Water
5.32 - 3.3	30' Bottom Width	1V:3H	Ag Water
Blue Point Ditch			
5.8 - 1.74	30' Bottom Width	1V:3H	Ag Water
1.74 - 0.0	Cleanout	Existing	Ag Water

Worksheet
Worksheet for Irregular Channel

Project Description	
Worksheet	Caney Creek EC - Mile 11.5
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Slope	0.000095	ft/ft
Water Surface Elevation	218.40	ft

Options	
Current Roughness Method	mproved Lotter's Method
Open Channel Weighting Method	mproved Lotter's Method
Closed Channel Weighting Metho	Horton's Method

Results		
Mannings Coefficient	0.050	
Elevation Range	212.96 to 219.43	
Discharge	76.68	cfs
Flow Area	118.0	ft²
Wetted Perimeter	35.12	ft
Top Width	32.49	ft
Actual Depth	5.44	ft
Critical Elevation	214.49	ft
Critical Slope	0.040114	ft/ft
Velocity	0.65	ft/s
Velocity Head	0.01	ft
Specific Energy	218.41	ft
Froude Number	0.06	
Flow Type	Subcritical	

Roughness Segments

Start Station	End Station	Mannings Coefficient
100+76	101+11	0.070
101+11	101+46	0.050
101+46	102+27	0.070

Natural Channel Points

Station (ft)	Elevation (ft)
100+76	219.05
100+87	219.24
101+11	219.31
101+16	216.01
101+19	214.73
101+30	213.42
101+36	212.96
101+41	215.66
101+46	219.20
101+52	219.43

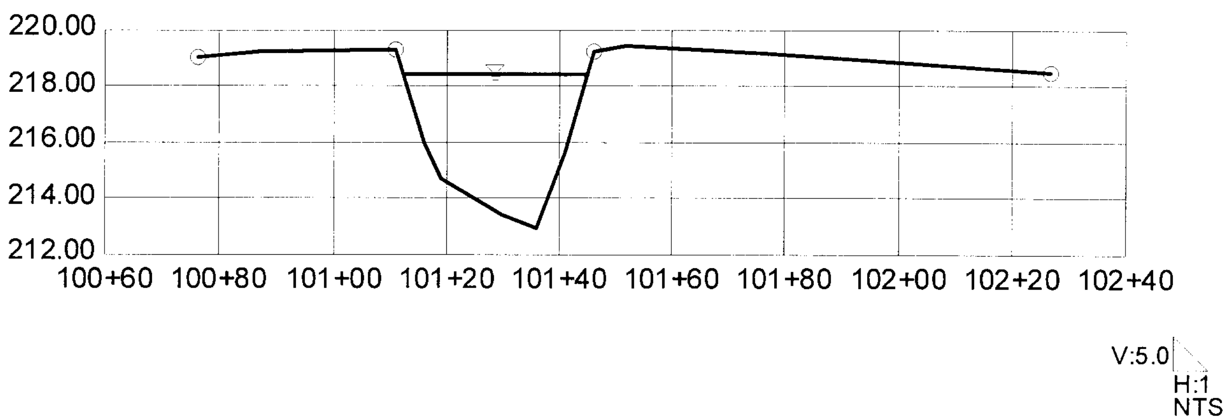
Worksheet
Worksheet for Irregular Channel

Natural Channel Points	
Station (ft)	Elevation (ft)
101+77	219.16
102+02	218.81
102+27	218.48

Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Caney Creek EC - Mile 11.5
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Discharge

Section Data		
Mannings Coefficient	0.050	
Slope	0.000095	ft/ft
Water Surface Elevation	218.40	ft
Elevation Range	212.96 to 219.43	
Discharge	76.68	cfs



Worksheet
Worksheet for Irregular Channel

Project Description	
Worksheet	Caney Creek IC - Mile 11.5 w/ Ag Water
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Slope	0.000095	ft/ft
Water Surface Elevation	218.40	ft

Options	
Current Roughness Method	mproved Lotter's Method
Open Channel Weighting Method	mproved Lotter's Method
Closed Channel Weighting Metho	Horton's Method

Results		
Mannings Coefficient	0.035	
Elevation Range	212.96 to 221.00	
Discharge	342.70	cfs
Flow Area	332.2	ft²
Wetted Perimeter	84.40	ft
Top Width	82.84	ft
Actual Depth	5.44	ft
Critical Elevation	214.36	ft
Critical Slope	0.017261	ft/ft
Velocity	1.03	ft/s
Velocity Head	0.02	ft
Specific Energy	218.42	ft
Froude Number	0.09	
Flow Type	Subcritical	

Roughness Segments

Start Station	End Station	Mannings Coefficient
100+76	106+50	0.035

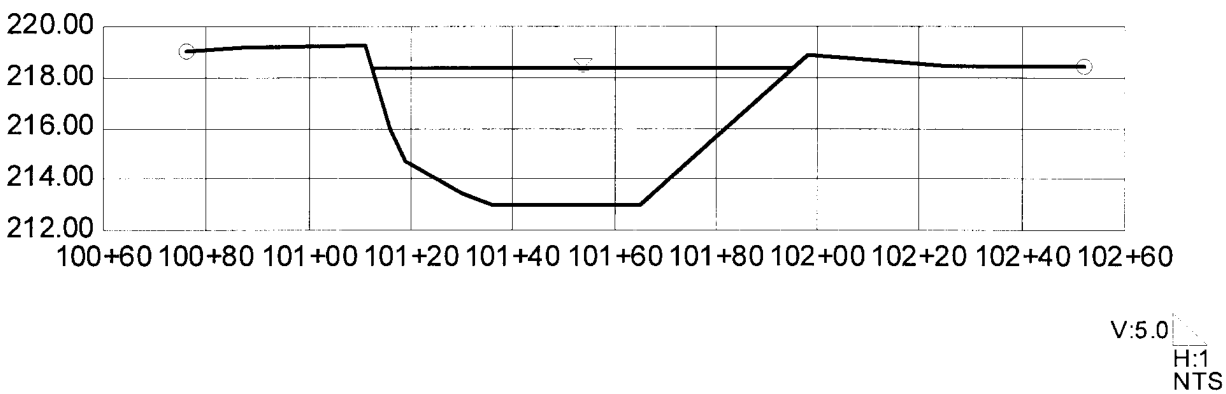
Natural Channel Points

Station (ft)	Elevation (ft)
100+76	219.05
100+87	219.24
101+11	219.31
101+16	216.01
101+19	214.73
101+30	213.42
101+36	212.96
101+65	212.96
101+98	218.90
102+26	218.48
102+52	218.46
106+50	221.00

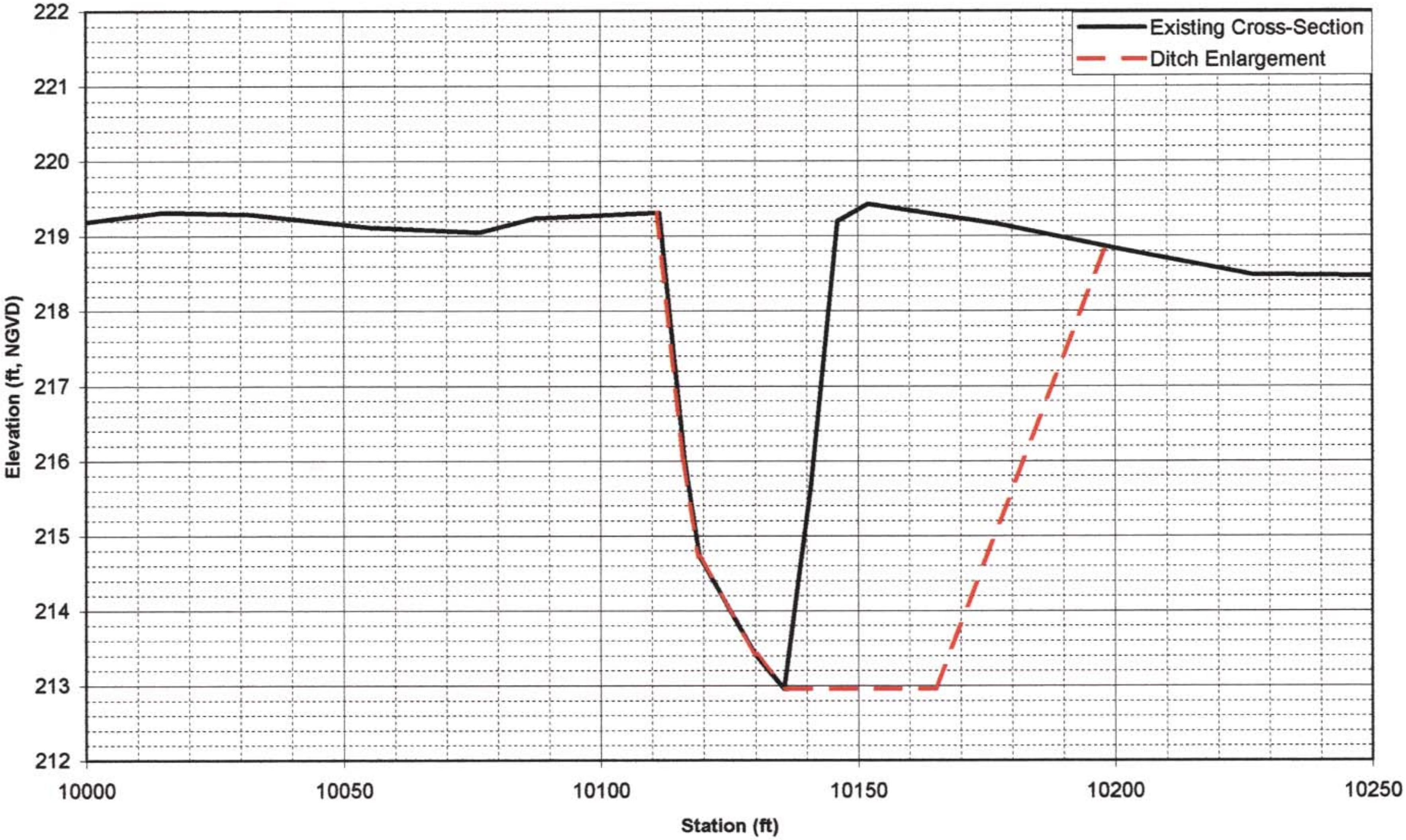
Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Caney Creek IC - Mile 11.5 w/ Ag Water
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Discharge

Section Data		
Mannings Coefficient	0.035	
Slope	0.000095	ft/ft
Water Surface Elevation	218.40	ft
Elevation Range	212.96 to 219.31	
Discharge	342.70	cfs



Caney Creek Ditch
Mile 11.5



Bayou Meto Comprehensive Study, Pump and Pipe Sizing

PIPE NAME	PIPE Length (feet)	Inlet Pool Elev	Outlet Elev NGVD	REQUIRED FLOW (cfs)	DELIV FLOW (cfs)	PUMP NAME	Min. Pump HP	12"	PVC 16"	PVC 18"	PVC 21"	PVC 24"	PVC 27"	Concrete 30"	Concrete 33"	Concrete 36"
1000.01	1817	232	240	0.51	0.77	P1000.01	1	1817								
1000.02	488	232	238	0.221	0.6	P1000.02	0.75	488								
1000.03	3283	232	238	0.867	1.3	P1000.03	2	3283								
1000.031	4047		234	0.085	0.3	P1000.031	BRANCH	4047								
1000.04	4320	227	228	1.275	1.6	P1000.04	2	4320								
1400.01	7632	235	238	8.362	9.1	P1400.01	20				1309	6323				
1400.011	3208		240	0.816	1.2	P1400.011	BRANCH	3208								
1400.02	15158	235	245	13.07	13.4	P1400.02	55	4337		489			10332			
1400.021	2717		240	1.088	1.2	P1400.021	BRANCH	2717								
1400.022	11440		240	7.631	7.7	P1400.022	BRANCH	8738			2702					
1400.0221	2634		240	0.816	1.5	P1400.0221	BRANCH	2634								
1400.0222	2717		240	4.878	5.3	P1400.0222	BRANCH			2717						
1400.023	1454		240	0.308	3.2	P1400.023	BRANCH	1454								
1400.03	5750	235	238	2.345	2.4	P1400.03	5	5750								
1400.031	1427		238	0.612	0.7	P1400.031	BRANCH	1427								
1400.04	7651	235	236	18.866	19.4	P1400.04	30			1248						6403
1400.041	352		230	0.034	4	P1400.041	BRANCH	352								
1400.042	856		238	0.255	2.8	P1400.042	BRANCH	856								
1400.043	2154		236	4.283	8.7	P1400.043	BRANCH				2154					
1400.044	4299		230	1.122	3.9	P1400.044	BRANCH			4299						
1410.01	2691	227	226	3.824	4.1	P1410.01	5		2691							
1410.02	8057	226	226	6.679	7.6	P1410.02	15				8057					
1410.03	5518	227	236	0.323	0.5	P1410.03	0.75	5518								
1500.01	20864	218	230	19.307	19.49	P1500.01	110			5254					15610	
1500.011	3473		228	2.294	2.3	P1500.011	BRANCH	3473								
1500.012	7681		228	2.566	2.6	P1500.012	BRANCH	1697	5984							
1500.0121	2659		236	1.853	1.9	P1500.0121	BRANCH	2659								
1500.013	12231		236	1.568	1.6	P1500.013	BRANCH	12231								
1500.0131	1785		236	0.168	0.17	P1500.0131	BRANCH	1785								
1500.014	14482	218	240	3.304	3.6	P1500.014	15	6896		7586						
1500.0141	6907		230	1.071	1.1	P1500.0141	BRANCH	6907								

(Note: 'Branch' under PUMP HP denotes no pump required.)
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1500.0142	1402			234		0.204		0.21		P1500.0142	BRANCH	1402								
1500.0143	238			238		0.187		0.19		P1500.0143	BRANCH	238								
1500.02	14185	213		222		22.095		22.51		P1500.02	70							5952		8233
1500.021	18874			230		7.257		8.81		P1500.021	BRANCH								18874	
1500.03	29325	213		230		12.26		14.43		P1500.03	50			3946					25379	
1500.031	2912			236		0.051		0.06		P1500.031	BRANCH	2912								
1500.032	6171			236		1.207		1.3		P1500.032	BRANCH	6171								
1500.033	3987			236		0.561		0.57		P1500.033	BRANCH	3987								
1500.034	1364			230		0.612		5.5		P1500.034	BRANCH			1364						
1500.04	18262			226		1.292		1.33		P1500.04	5	18262								
1500.041	2677			228		0.034		0.126		P1500.041	BRANCH	2677								
1500.05	19463	213		224		14.796		15.01		P1500.05	70									19463
1500.051	13873			226		1.896		1.92		P1500.051	BRANCH	13873								
1500.0514	5263			230		0.374		0.92		P1500.0514	BRANCH	5263								
1500.052	1369			230		0.357		0.4		P1500.052	BRANCH	1369								
1500.053	8712			226		0.68		3		P1500.053	BRANCH			8712						
1500.054	13305			220		8.566		9.69		P1500.054	BRANCH					5465		7840		
1500.0541	6645			230		1.836		3.19		P1500.0541	BRANCH	6645								
1500.5411	4318			224		0.204		0.3		P1500.05411	BRANCH	4318								
1500.05412	6039			228		0.901		2.88		P1500.05412	BRANCH	6039								
1500.054121	1351			226		0.085		0.1		P1500.054121	BRANCH	1351								
1500.06	10451	213		224		4.47		4.85		P1500.06	15		6067	4384						
1500.061	2641			224		0.867		1.49		P1500.061	BRANCH	2641								
1500.07	142	213		216		0.51		0.81		P1500.07	0.75	142								
1500.08	13704	213		222		1.955		2.53		P1500.08	5	0	13704							
1500.081	1337			222		0.068		1.94		P1500.081	BRANCH	0	1337							
1500.09	2764	213		218		0.272		0.62		P1500.09	0.75	2764								
1500.1	11640	213		214		7.189		7.2		P1500.10	25				11640					
1500.101	2659			218		0.187		0.2		P1500.101	BRANCH	2659								
1500.102	12728			218		3.773		6.98		P1500.102	BRANCH		12728							
1500.1021	4312			216		0.255		1.29		P1500.1021	BRANCH	4312								
1500.11	13628	210		222		3.501		3.88		P1500.11	15	5477		8151						
1500.111	6910			230		0.017		1.7		P1500.111	BRANCH	6910								
1500.12	3773	210		218		8.243		8.26		P1500.12	20	1917				1856				
1500.121	15113			218		8.056		8.08		P1500.121	BRANCH					15113				
1500.1211	2617			216		1.139		4.76		P1500.1211	BRANCH			2617						
1500.13	8665	210		222		8.804		9.08		P1500.13	25	3210					5455			

(Note: 'Branch' under PUMP HP denotes no pump required.)
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1500.131	7239			224		6.968		7.37		P1500.131	10		3097	4142						
1500.14	753	210		216		0.017		0.59		P1500.14	0.75	753								
1500.15	1925	210		216		0.782		1.36		P1500.15	2	1925								
1500.17	8407	207		220		4.572		4.73		P1500.17	20	5390		3017						
1500.171	9394			218		3.773		3.78		P1500.171	BRANCH		9394							
1500.1711	1941			222		0.816		0.98		P1500.1711	BRANCH	1941								
1500.18	279	207		214		0.017		0.55		P1500.18	0.75	279								
1500.19	1726	207		214		0.068		0.54		P1500.19	0.75	1726								
1500.02	75	203		212		1.819		2.79		P1500.20	5	75								
1500.21	1478	203		212		0.323		1.16		P1500.21	2	1478								
1500.22	10159	203		224		2.464		2.81		P1500.22	15	10159								
1500.221	1226			214		0.017		0.5		P1500.221	BRANCH	1226								
1500.222	1440			216		0.187		0.5		P1500.222	BRANCH	1440								
1500.23	10213	203		208		4.895		5.02		P1500.23	15		6112	4101						
1500.231	2652			214		0.493		1.15		P1500.231	BRANCH	2652								
1510.01	1473	228		228		0.408		1.1		P1510.01	0.75	1473								
1520.01	4065	220		224		4.266		4.64		P1520.01	12		4065							
1520.011	4141			228		0.544				P1520.011	BRANCH	4141								
1520.02	2594	222		224		0.95		1		P1520.02	0.75	2594								
C1530/1531	31805	199		195		18		18.6		P1530	30									31805
1530.01	2807	203		212		0.323		0.46		P1530.01	0.75	2807								
1530.02	9032	203		210		1.054		1.22		P1530.02	2.5	9032								
1530.03	14649	203		216		4.062		4.79		P1530.03	15			14649						
1530.04	1035	199		208		0.561		0.62		P1530.04	1	1035								
1530.05	12111	199		214		4.283		4.38		P1530.05	15			12111						
1530.051	1664			210		0.102		1		P1530.051	BRANCH	1664								
1530.06	11616	199		212		6.594		6.87		P1530.06	20					11616				
1530.07	8041	199		206		2.991		3.83		P1530.07	15	2720	5321							
1530.071	1464			206		0.051		0.06		P1530.071	BRANCH	1464								
1530.072	5378			210		0.561		0.97		P1530.072	BRANCH	5378								
1530.0721	735			214		0.17		0.57		P1530.0721	BRANCH	735								
1530.073	2708			206		1.309		2.01		P1530.073	BRANCH	2708								
1530.08	9446	196		206		3.593		3.87		P1530.08	15	1551		7895						
1530.081	5879			200		2.302		2.67		P1530.081	BRANCH	5879								

(Note: 'Branch' under PUMP HP denotes no pump required.)
USACE Hydraulics Branch, Updated 26 Jun 2002, filename: PUMPS.xls

1530.0811	5414			208		1.248	1.87		P1530.0811	BRANCH	5414								
1530.09	2574	196		208		0.102	0.51		P1530.09	1	2574								
1530.1	1221	196		208		4.47	4.86		P1530.10	10			1221						
1531.01	3835	196		208		0.459	0.5		P1531.01	1	3835								
2000.01	5518	229		224		0.731	0.83		P2000.01	0.75	5518								
2000.011	6773			230		0.153	0.21		P2000.011	BRANCH	6773								
2000.02	2918	227		228		2.023	2.2		P2000.02	3	2918								
2000.03	12415	227		231		8.209	8.4		P2000.03	15					12415				
2100.02	11032	218		222		1.513	1.75		P2100.02	5	11032								
2100.06	12284	213		230		5.966	6.3		P2100.06	35	11539		745						
2100.061	11755			229		4.691	5		P2100.061	BRANCH	2665		9090						
2100.0611	16378			231		2.685	2.8		P2100.0611	BRANCH		16378							
2100.07	2788	210		214		3.664	4.1		P2100.07	7.5		2788							
2100.08	2749	207		214		0.85	1.2		P2100.08	2	2749								
2100.09	9261	207		208		2.855	3.3		P2100.09	7.5		9261							
2100.1	3561	207		214		1.649	2.6		P2100.10	5	3561								
2100.101	2723			212		0.918	1		P2100.101	BRANCH	2723								
2100.11	11461	204		206		7.376	7.4		P2100.11	30		3975	7486						
2100.111	470			210		0.323	0.5		P2100.111	BRANCH	470								
2100.112	1405			210		0.51	0.6		P2100.112	BRANCH	1405								
2100.113	6459			210		2.14	3.3		P2100.113	BRANCH	3636	2823							
2100.1132	3959			210		1.173	1.5		P2100.1132	BRANCH	3959								
2120.01	4028	201		210		1.173	1.28		P2120.01	2.5	4028								
2120.02	24695	201		222		10.782	11.03		P2120.02	50					11433		13262		
2120.021	2806			214		0.176	0.5		P2120.021	BRANCH	2806								
2120.022	13984			224		0.2158	3.12		P2120.022	BRANCH	13984								
2120.0221	1364			218		0.051	0.1		P2120.0221	BRANCH	1364								
2120.023	2392			224		0.408	0.75		P2120.023	BRANCH	2392								
2120.03	5289	201		206		6.841	7.04		P2120.03	15		1369			3920				
2120.031	2744			208		3.816	4.34		P2120.031	BRANCH				2744					
2120.0311	4098			208		3.34	4.34		P2120.0311	BRANCH	1453	2645							
2120.04	21157	196		216		10.402	10.48		P2120.04	60					2736	6363	12058		
2120.041	11695			220		1.666	1.98		P2120.041	BRANCH	11695								

(Note: 'Branch' under PUMP HP denotes no pump required.)

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2120.0411	2726			216		0.034		0.05		P2120.0411	BRANCH	2726								
2120.0412	2377			218		0.034		0.05		P2120.0412	BRANCH	2377								
2120.0413	2695			214		0.935		1		P2120.0413	BRANCH	2695								
2120.042	3952			218		0.051		0.44		P2120.042	BRANCH	3952								
2120.05	18834	196		198		8.821		9.07		P2120.05	55			5445		7368	6021			
2120.051	2793			206		1.173		2.11		P2120.051	BRANCH	2793								
2120.052	680			206		0.629		1.28		P2120.052	BRANCH	680								
2120.06	780	192		200		0.595		0.67		P2120.06	1	780								
2120.07	7429	192		200		7.733		8.31		P2120.07	20			1880		5549				
2120.071	9434			200		2.838		3.12		P2120.071	BRANCH				9434					
2140.01	11111	200		218		2.702		2.705		P2140.01	15	11111								
2140.011	1345			212		0.085		0.09		P2140.011	BRANCH	1345								
2140.012	9254			214		1.037		1.2		P2140.012	BRANCH	9254								
2140.0121	2328			216		0.306		0.4		P2140.0121	BRANCH	2328								
2140.013	1316			216		0.085		0.1		P2140.013	BRANCH	1316								
2140.02	9339	200		214		1.326		1.8		P2140.02	5	9339								
2140.021	954			212		1.02		1.1		P2140.021	BRANCH	954								
2140.022	1244			212		0.153		0.16		P2140.022	BRANCH	1244								
2140.03	18936	200		198		24.95		25.26		P2140.03	60			1523						17413
2140.031	2739			200		0.51		0.52		P2140.031	BRANCH	2744								
2140.032	6969			196		3.331		3.4		P2140.032	BRANCH			6974						
2140.033	8621			202		1.751		2.84		P2140.033	BRANCH			8626						
2140.0331	3479			196		0.034		0.04		P2140.0331	BRANCH	3484								
2140.034	2609			196		0.357		0.4		P2140.034	BRANCH	2614								
2140.035	5908			194		13.427		14.1		P2140.035	BRANCH									5908
2140.0351	8139			190		12.747		14.1		P2140.0351	BRANCH			480	2581					5078
2140.3511	5582			196		5.014		5.6		P2140.03511	BRANCH					5582				
2140.35111	1293			194		0.017		1.97		P2140.035111	BRANCH	1293								
2140.04	17571	200		216		4.572		5		P2140.04	20	4612			12959					
2140.041	1275			204		0.034		0.1		P2140.041	BRANCH	1275								
2140.042	9698			214		1.53		2.15		P2140.042	BRANCH	2705	6993							
2140.0421	5237			206		0.782		0.8		P2140.0421	BRANCH	5237								
2140.043	2781			214		0.221		0.3		P2140.043	BRANCH	2781								
2140.05	10579	200		204		2.345		2.56		P2140.05	5	5241	5338							
2140.051	3984			204		1.258		1.36		P2140.051	BRANCH	3984								
2140.06	6492	200		196		8.345		8.7		P2140.06	5						6492			

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2140.07	4697	200		202		1.904		2.15		P2140.07	2.5		4697							
2140.08	15940	200		212		1.717		1.79		P2140.08	7	15940								
2140.081	2701			206		0.289		0.3		P2140.081	BRANCH	2701								
2140.09	15800	200		206		9.6		10.1		P2140.09	30						15800			
2140.091	15128			210		0.51		0.6		P2140.091	BRANCH	15128								
2140.092	6651			208		1.615		1.7		P2140.092	BRANCH		6651							
2140.093	6684			206		0.102		0.1		P2140.093	BRANCH	6684								
2140.094	10810			200		4.98		5		P2140.094	BRANCH	5773	3891	1146						
2140.0941	6146			202		1.938		2		P2140.0941	BRANCH		6146							
2140.09411	986			206		0.119		1.7		P2140.09411	BRANCH	986								
2140.1	12505	195		190		6.832		7.17		P2140.10	10				9831	2674				
2140.101	1246			196		0.697		2.23		P2140.101	BRANCH	1246								
2140.102	2653			194		0.833		0.94		P2140.102	BRANCH	2658								
2140.11	6583	195		194		1.683		1.73		P2140.11	2.5	6583								
2140E	10260	195		190		4.011		4.57		P2140E	10				8066	2194				
2140.12	2717	195		190		1.258		1.71		P2140.12	0.75	2717								
2140.13	5400	195		186		2.753		2.86		P2140.13	0.75			5400						
2160.01	6766	195		200		1.97		3.4		P2160.01	10	6766								

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2160.011	6500			198		1.8		2.2		P2160.011	BRANCH	6500								
2160.0111	500			202		0.102		1.7		P2160.0111	BRANCH	500								
2160.012	2256			205		0.986		1		P2160.012	BRANCH	2256								
2160.02	2101	195		202		0.238		0.53		P2160.02	0.75	2101								
2160.03	13335	195		204		5.8		6		P2160.03	35			13335						
2160.031	6832			200		0.493		0.6		P2160.031	BRANCH	6832								
2160.032	5258			210		1.139		4		P2160.032	BRANCH	5258								
2160.033	2770			207		3.671		2		P2160.033	BRANCH	2770								
2160.0331	12645			204		1.989				P2160.0331	BRANCH	12645								
										XXXXXXXXXX										
2200.01	2578	206		208		0.544		0.83		P2200.01	0.75	2578								
2200.02	2441	206		196		1.343		2.6		P2200.02	0.75	2441								
2200.03	3752	206		208		14.158		14.7		P2200.03	25			1535			2217			
2200.031	2640			202		5.4		8.4		P2200.031	BRANCH			2640						
2200.05	4827	206		210		1.938		2.5		P2200.05	5	4827								
2200.051	1399			208		0.49		0.6		P2200.051	BRANCH	1399								
2200.06	2943	203		206		0.969		1.5		P2200.06	2	2943								
2200.07	7934	200		208		5.15		5.3		P2200.07	15	345		7589						
2200.071	3294			206		1.598		3.2		P2200.071	BRANCH	3294								
2200.08	16962	197		204		25.001		25		P2200.08	85				5845					11117
2200.081	1271			201		0.51		0.6		P2200.081	BRANCH	1271								
2200.082	21580			202		10.3		16.2		P2200.082	BRANCH			8061		13519				
2200.0821	5682			204		0.663		2.4		P2200.0821	BRANCH	5682								
2200.0822	9451			199		4.028		5.4		P2200.0822	BRANCH				9451					
2200.083	1210			206		0.374		1.6		P2200.083	BRANCH	1210								
2200.09	18466	188		200		6.866		7.2		P2200.09	35				18466					
2200.091	2690			196		0.493		0.5		P2200.091	BRANCH	2690								
2200.092	3283			200		0.017		0.1		P2200.092	BRANCH	3283								
2200.093	7948			198		3.416		6.6		P2200.093	BRANCH	4757		3191						
2200.0931	2640			200		0.51		0.5		P2200.0931	BRANCH	2640								
2200.0932	13889			200		2.277		2.3		P2200.0932	BRANCH	13889								
2200.09321	2540			200		0.17		0.8		P2200.09321	BRANCH	2540								
2200.1	17308	188		200		15.942		16		P2200.10	75	1338		5407		3950				6613
2200.101	9460			198		7.206		9		P2200.101	BRANCH	9460								
2200.1011	1350			198		1.411		3.2		P2200.1011	BRANCH	1350								
2200.102	1377			200		0.034		2		P2200.102	BRANCH	1377								
2200.104	5306			196		1.683		3.2		P2200.104	BRANCH	5306								

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2200.1041	6686			196		1.292		2.2		P2200.1041	BRANCH	6686								
2200.11	20028	188		196		8.634		8.8		P2200.11	30				4133	15895				
2200.111	1322			200		0.595		1		P2200.111	BRANCH	1322								
2200.112	4007			198		0.17		1.2		P2200.112	BRANCH	4007								
2200.12	2441	188		196		0.748		0.9		P2200.12	1.5	2441								
RELIFT		203		208		70		70		P2220	50									
2220.01	1705	193		204		2.006		2.2		P2220.01	1	1705								
2240.01	2946	193		204		4.249		4.48		P2240.01	10			2946						
2240.02	4964	193		204		2.057		2.24		P2240.02	5		4964							
2240.03	4634	193		202		2.277		2.43		P2240.03	5		4634							
2240.04	3498	193		200		1.275		1.45		P2240.04	2.5	3498								
2240.041	1325			200		0.629		0.75		P2240.041	BRANCH	1325								
2240.05	10372	188		192		10.062		10.22		P2240.05	20						10372			
2240.06	20118	185		200		15.313		15.91		P2240.06	50									20118
2240.061	3986			200		0.731		1.63		P2240.061	BRANCH	3986								
RELIFT		188		198		64		64		P2260	80									
2260.01	4175	196.5		196		1.309		1.7		P2260.01	2	4175								
2260.02	1173	196.5		194		0.034		1.6		P2260.02	0.75	1173								
2260.03	1752	196.5		194		1.139		1.5		P2260.03	0.75	1752								
2260.04	7492	196.5		194		5.915		6.5		P2260.04	20				7492					
2260.041	7072			194		1.105		2.4		P2260.041	BRANCH	7072								
2260.042	7752			194		2.04		4.1		P2260.042	BRANCH		7752							
2260.05	13580	196.5		194		18.917		20		P2260.05	40			11844					1736	
2260.051	2384			192		7.92		15.5		P2260.051	BRANCH				2384					
RELIFT		188		195		53		53		P2280	45									
2280.01	1270	193		198		1.168		1.5		P2280.01	2	1270								
2280.02	7240	193		196		1.394		1.6		P2280.02	3	7240								
2280.03	5356	193		194		2.396		2.4		P2280.03	5	5356								
2280.04	1356	193		194		0.459		1		P2280.04	0.75	1356								
2280.05	5408	193		190		5.456		5.7		P2280.05	7.5			5408						
2280.06	1533	193		190		0.476		1.6		P2280.06	0.75	1533								
2280.07	8705	193		188		9.059		9.1		P2280.07	15	7425				1280				
2280.071	1253			188		4.912		7.1		P2280.071	BRANCH		1253							

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2280.08	5801	193		192		18.135		18.13		P2280.08	60	395								5406
2280.09	13937	193		194		15.024		1513		P2280.09	BRANCH	8263						4210	1464	
2280.091	16172	193		190		4.691		5.1		P2280.091	BRANCH			13433	2739					
2280.0911	1330			190		0.153		0.5		P2280.0911	BRANCH	1330								
2280.913	1312			192		0.578		3		P2280.0913	BRANCH	1312								
2280.092	2686			192		4.045		0.3		P2280.092	BRANCH		1346		1340					
2280.0921	623			188		0.595		7.2		P2280.0921	BRANCH		623							
2300	15000	227		222.5		13.784		15.4		P2300	15						15000			
2300.01	27540	227		216		13.784		15.4		P2300.01	15						27540			
2500.01	22257	235		213		4.946		5.1		P2500.01	7.5	681		21576						
2500.011	1119			210		0.748		3		P2500.011	BRANCH	1119								
2500.03	5460	236.2		235.2		1.2		5		P2500.03	0.75	5460								
2511.01	30457	227		222		12.934		13.02		P2511.01	20								30457	
2511.011	2559			221		0.0561		0.06		P2511.011	BRANCH	2559								
2511.02	837	215		216		0.68		0.96		P2511.02	0.75	837								
2511.03	13288	215		220		26.497		27.2		P2511.03	80									13288
2511.031	6604			224		0.527		0.6		P2511.031	BRANCH	6604								
2511.04	3090	212		220		0.153		0.5		P2511.04	0.75	3090								
2511.05	5991	210		226		8.328		8.47		P2511.05	35						5991			
2511.051	1082			214		0.442		0.5		P2511.051	BRANCH	1082								
2511.052	1324			214		3.739		3.8		P2511.052	BRANCH			1324						
2511.053	3923			234		3.705		4.17		P2511.053	BRANCH					3923				
2511.531	3911			234		2.243		3.5		P2511.0531	BRANCH			3911						
2511.0532	1378			232		0.663		0.67		P2511.0532	BRANCH	1378								
C2520	12905	228		225		23		24		P2520	15									12905
2521.01	5444	207		209		4		4.26		P2521.01	15	5444								
2521.011	2597			214		0.153		0.16		P2521.011	BRANCH	2597								
2521.02	5299	207		208		3.484		3.5		P2521.02	BRANCH	5299								
2530.01	30967	218		210		21.993		22.41		P2530.01	65									30967
2530.011	2987			224		0.425		0.5		P2530.011	BRANCH	2987								
2530.012	2726			224		0.255		0.35		P2530.012	BRANCH	2726								

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2530.02	968	218		225		0.68		0.86		P2530.02	1.5	968								
2530.03	4931	210		226		2.328		2.57		P2530.03	7.5		4931							
2530.04	773	207		212		0.544		0.65		P2530.04	0.75	773								
2530.05	6614	207		220		1.241		1.68		P2530.05	5	6614								
2530.06	2836	199		200		0.425		0.63		P2530.06	0.75	2836								
2530.07	17890	196		200		10.928		11.01		P2530.07	20							17890		
2530.071	2797			204		0.561		0.6		P2530.071	BRANCH	2797								
RELIFT		206.5		224		88		88		P2531	180									
2531.01	12235	204		206		7.24		7.28		P2531.01	10						12235			
2532.01	876	198		206		1.36		1.54		P2532.01	2.5	876								
2532.02	2730	198		206		0.408		0.5		P2532.02	0.75	2730								
2532.03	6535	198		204		3.161		3.23		P2532.03	10	3865	2670							
2532.031	10104			202		2.6		2.64		P2532.031	BRANCH	771	9333							
2532.0311	1398			204		0.119		0.4		P2532.0311	BRANCH	1398								
2532.0313	1346			208		0.068		0.24		P2532.0313	BRANCH	1346								
2532.04	1920	195		210		0.119		0.65		P2532.04	1.5	1920								
2532.05	1261	195		196		0.96		1		P2532.05	0.75	1261								
C2533	16582	208		208		36		36.5		P2533	100									16582
2533.01	5397	190		203		8.933		9.2		P2533.01	25					5397				
2533.02	9278	190		206		12.186		12.8		P2533.02	45							9278		
2533.021	13247			206		8.719		12.8		P2533.021	BRANCH			7974	5273					
2533.0211	1236			206		0.374		1.98		P2533.0211	BRANCH	1236								
2533.03	6699	190		203		5.507		5.8		P2533.03	20	1546		5153						
2533.031	1288			200		0.561		3.4		P2533.031	BRANCH	1288								
C2535	15076	208		190		25		27		P2535	15									15076
2535.01	2740	190		200		3.74		4.1		P2535.01	BRANCH		2740							
2535.02	2712	190		198		0.561		0.65		P2535.02	BRANCH	2712								
2535.03	17385	190		202		18.203		18.2		P2535.03	BRANCH	6159						3214		8012
2535.031	3238			200		1.207		1.3		P2535.031	BRANCH	3238								
2535.032	6846			196		5.388		5.4		P2535.032	BRANCH		4577		2269					
2535.0321	7239			198		2.821		2.82		P2535.0321	BRANCH	7244								
2535.033	10216			198		8.43		8.5		P2535.033	BRANCH			5527	4689					
2535.0331	3232			198		0.408		0.9		P2535.0331	BRANCH	3232								

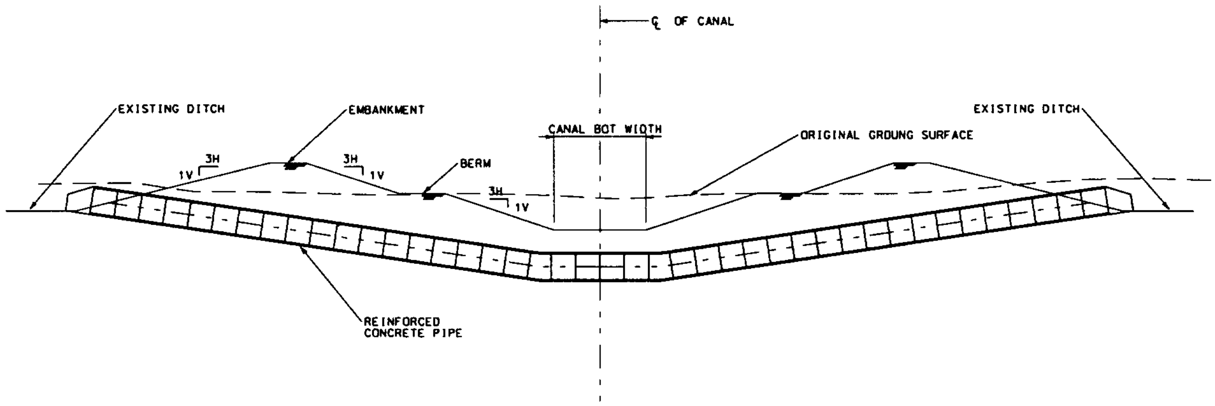
(Note: 'Branch' under PUMP HP denotes no pump required.)

2535.0332	3971			204		0.816	0.9		P2535.0332	BRANCH	3971								
2540.01	2884	214		210		0.714	1.53		P2540.01	0.75	2884								
2540.02	8410	214		220		3.263	3.76		P2540.02	7.5		8410							
2540.03	6331	214		188		3.569			P2540.03	0.75	1564		4767						
2540.031	8085			194		3.467			P2540.031	BRANCH		8085							
3000.01	2810	236		242		0.782	1.3		P3000.01	2	2810								
3000.02	798	236		240		0.714	0.71		P3000.02	0.75	798								
3000.03	16380	236		225		0.92	1.2		P3000.03	0.75	16380								
3000.04	3752	232		225		1.67	2.2		P3000.04	0.75	3752								
3000.05	20433	232		230		3.3	3.5		P3000.05	15		20433							
3000.06	6730	232		232		4.8	5		P3000.06	15		6730							
3000.07	15571	231		257		8.9	8.9		P3000.07	45		3798			11773				
3000.072	4472			250		no specific	8.9							4472					
3000.08	2718	231		244		2.6	3		P3000.08	7.5		2718							
4000.01	17102	246		234		13.7	14.1		P4000.01	15							17102		
4000.02	20548	243		246		7.4	7.8		P4000.02	20						20548			
4000.03	10496	243		238		11.642	12.2		P4000.03	15							10496		
4000.031	621			236		0.714	5.6		P4000.031	BRANCH				621					
4000.032	3273			230		2.26	6.7		P4000.032	BRANCH				3273					
4100.01	6710	243		234		5.235	5.6		P4100.01	7.5			6710						
4100.011	5419			244		0.051	0.9		P4100.011	BRANCH	5419								
4100.012	3090			234		0.051	2.3		P4100.012	BRANCH	3090								
4100.013	6575			226		1.938	2.4		P4100.013	BRANCH	6575								
4110.01	345	238		240		5.201	5.5		P4110.01	10	335			10					
4110.011	12890			234		2.957	3.3		P4110.011	BRANCH	7770	5120							
4110.0111	1951			240		0.136	1		P4110.0111	BRANCH	1951								
4110.0112	2664			240		0.306	0.9		P4110.0112	BRANCH	2664								
4110.02	17798	235		234		7.733	7.9		P4110.02	20			16990	808					
4110.021	2082			240		1.547	3.5		P4110.021	BRANCH	2082								
RELIFT		235.5		234		60	60		P4111	10									
4111.01	13431	232		222		2.515	2.8		P4111.01	3		13431							
4111.02	2497	232		224		0.119	2.2		P4111.02	0.75	2497								

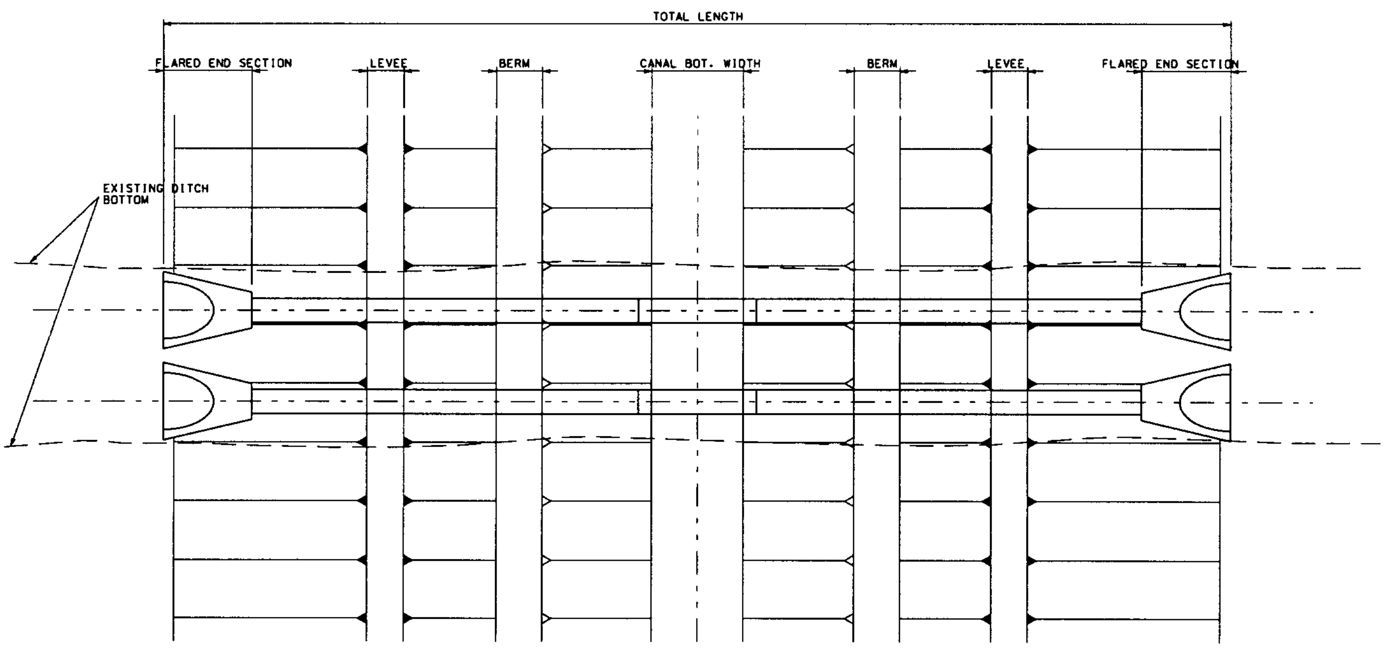
(Note: 'Branch' under PUMP HP denotes no pump required.)

4111.05	6531	229		228		0.255		1		P4111.05	0.75	6531								
4111.06	3165	229		224		1.819		1.9		P4111.06	1	3165								
4111.061	3649			226		0.204		0.2		P4111.061	BRANCH	3649								
4111.07	6329	229		218		3.382		3.8		P4111.07	3	6329								
4111.08	5028	229		214		0.153		2.4		P4111.08	0.75	5028								
4111.09	3892	229		220		23.386		23.6		P4111.09	60									3892
4111.091	17454			190		5.898		14.5		P4111.091	BRANCH	2620			9665		5169			
4111.0911	1491			214		0.068		8		P4111.0911	BRANCH				1491					
4111.0912	2654			220		2.209		3.5		P4111.0912	BRANCH		2654							
4111.092	12732			216		3.96		9.1		P4111.092	BRANCH				12732					
RELIFT		235.5		238		108		108		P4112	40									
4112.01	8013	236.5		238		1.768		2.1		P4112.01	5	8013								
4112.02	8820	236.5		230		0.68		1.3		P4112.02	0.75	8820								
4112.03	13233	236.5		244		4.113		4.7		P4112.03	15			13233						
4112.04	2839	236.5		246		1.224		1.5		P4112.04	3	2839								
4112.05	18433	236.5		214		16.083		16.12		P4112.05	20				1665					16768
4112.051	5260			220		6.757		8.1		P4112.051	BRANCH					5260				
4112.06	13560	236.5		216		5.737		7.8		P4112.06	BRANCH	4386		9174						
4112.061	3902					0.901		1		P4112.061	BRANCH	3902								
4112.062	665			204		0.289		0.3		P4112.062	BRANCH	665								
4112.063	4597					0.272		2.4		P4112.063	BRANCH	4597								
4112.07	11315	236.5		238		17.506		17.7		P4112.07	25							4006		7309
4112.071	924			236		4.028		8.1		P4112.071	BRANCH			924						
4112.072	2446			228		4.113		9.1		P4112.072	BRANCH			2446						
4113.01	15895	235		236		26.769		26.8		P4113.01	85									15895
4113.011	3967			230		0.408		7		P4113.011	BRANCH			3967						
4113.012	2614			228		0.204		12		P4113.012	BRANCH					2614				
4113.013	7627			224		5.014		7.8		P4113.013	BRANCH			1271		6356				
4113.013	1299			232		0.034		1.8		P4113.0131	BRANCH	1299								
4113.0132	10599			198		1.547		2.5		P4113.0132	BRANCH	10599								
C4200	10794	238		235		14		15.6		P4200	15							10794		
4200.01	10036	238		230		9.892		9.8		P4200.01	15		6713					3323		
4200.011	18397			230		5.609		5.6		P4200.011	BRANCH				18397					

(Note: 'Branch' under PUMP HP denotes no pump required.)



PROFILE
NO SCALE



PLAN
NO SCALE

US Army Corps of Engineers

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEER
MEMPHIS, TENNESSEE

BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN

INVERTED PIPE SIPHON

NATURAL DRAINAGE CROSSING @ CANAL

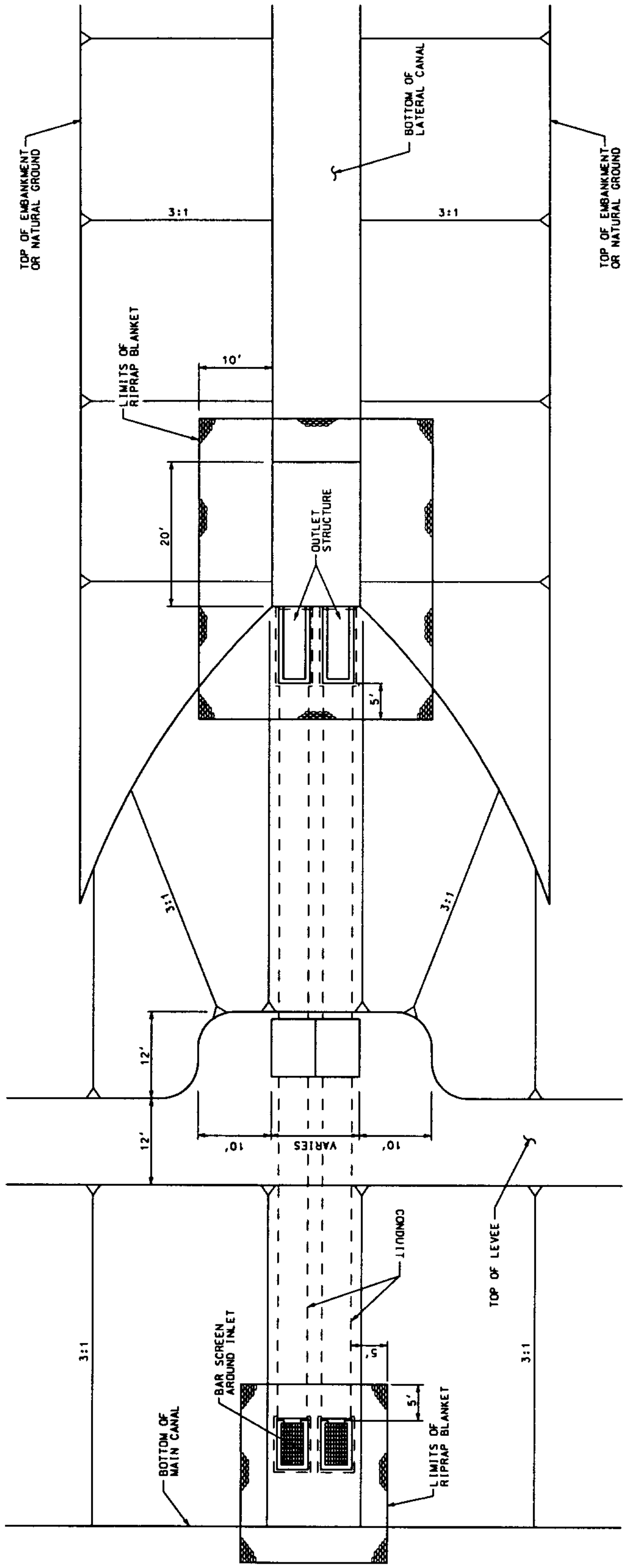
PLATE
I–C–12

Bayou Meto Water Supply Study
Inverted Siphon Design for Natural Drainage

Canal No.	Siphon No.	Drainage basin Area (sq mi)	Calculated Q100 (cfs)		Selected Q100 (cfs)	Head (ft.)	Tailwater Depth (ft.)	Energy Losses (ft)				Selected Siphon Diameter (in.)	Number of Siphons
			Rational Method	Previous Study				Entrance K=0.8	Exit K=1.0	Bend K=0.16	Friction C=60		
1000	1000-02	1.151268	616		616	0.1	0.0	0.467	0.584	0.374	0.981	96	2
1000	1000-04	0.356364	191		191	0.0	0.0	0.411	0.513	0.329	1.110	78	1
1000	1000-05	0.217367	116		116	0.0	0.0	0.266	0.333	0.213	1.310	48	2
1000	1000-06	0.783033	419		419	0.0	0.0	0.369	0.461	0.295	0.921	84	2
1000	1000-07	1.980821	1061		1061	0.0	0.0	0.447	0.559	0.358	1.016	90	4
1000	1000-08	3.40179	1821		1821	0.0	0.0	0.453	0.566	0.362	0.953	96	6
1000	1000-09	0.777749	416		416	0.3	0.0	0.424	0.530	0.339	1.389	66	3
1000	1000-10	0.210876	113		113	0.0	0.0	0.251	0.313	0.201	1.239	48	2
1000	1000-11	4.459647	2388		2388	0.0	0.0	0.438	0.548	0.350	0.924	96	8
1400	1400-01	0.591769	317		317	0.0	0.0	0.390	0.488	0.312	1.161	72	2
1400	1400-02	0.613791	329		329	0.2	0.0	0.420	0.525	0.336	1.242	72	2
1400	1400-03	0.202163	108		108	0.0	0.0	0.230	0.288	0.184	1.146	48	2
1400	1400-04	0.460312	246		246	0.1	0.0	0.331	0.414	0.265	1.398	54	3
1400	1400-05	0.8075	432		432	0.0	0.0	0.323	0.403	0.258	0.975	72	3
1410	1410-01	0.367759	197		197	0.2	0.0	0.437	0.547	0.350	1.176	78	1
1410	1410-02	0.729623	391		391	0.1	0.0	0.430	0.538	0.344	1.159	78	2
1530	1530-01	0.18323	98		98	0.3	0.0	0.266	0.332	0.213	1.829	36	3
2000	2000-01	0.37703	202		202	0.3	0.0	0.460	0.575	0.368	1.231	78	1
2000	2000-02	0.282429	151		151	0.0	0.0	0.281	0.351	0.225	1.199	54	2
2000	2000-03	0.17572	94		94	0.3	0.0	0.297	0.371	0.238	1.693	42	2
2000	2000-04	0.127709	68		68	0.0	0.0	0.157	0.196	0.126	0.938	42	2
2100	2100-01	0.593187	318		102	0.0	0.0	0.335	0.419	0.268	1.249	60	1
2100	2100-02	0.14447	77		77	0.0	0.0	0.165	0.207	0.132	1.179	36	3
2100	2100-03	0.182382	98		98	0.0	0.0	0.307	0.384	0.246	1.152	60	1
2100	2100-04	0.057736	31		31	0.0	0.0	0.238	0.297	0.190	1.649	36	1
2100	2100-05	0.333246	178		178	0.0	0.0	0.359	0.449	0.287	0.980	78	1
2110	2110-01	0.078569	42		42	0.3	0.0	0.228	0.285	0.182	1.964	30	2
2140	2140-01	0.982318	526		526	0.0	0.0	0.440	0.550	0.352	1.001	90	2
2140	2140-02	0.687665	368		368	0.0	0.0	0.382	0.478	0.306	1.038	78	2
2140	2140-03	0.402924	216		216	0.2	0.0	0.375	0.469	0.300	1.385	60	2
2140	2140-04	0.290452	156		156	0.0	0.0	0.297	0.371	0.238	1.262	54	2
2140	2140-05	0.282421	151		151	0.0	0.0	0.281	0.351	0.225	1.199	54	2
2140	2140-06	0.485136	260		260	0.0	0.0	0.371	0.464	0.297	1.228	66	2
2140	2140-07	0.730536	391		391	0.1	0.0	0.432	0.539	0.345	1.161	78	2
2140	2140-08	0.463542	248		248	0.1	0.0	0.336	0.420	0.269	1.416	54	3
2160	2160-01	0	0	4	4	0.0	0.0	0.020	0.025	0.016	0.270	24	1
2220	2220-01	0.205413	110		110	0.0	0.0	0.238	0.297	0.190	1.180	48	2
2220	2220-02	0.285265	153		153	0.0	0.0	0.286	0.358	0.229	1.221	54	2
2220	2220-03	0.464913	249		249	0.1	0.0	0.338	0.423	0.271	1.424	54	3
2300	2300-01	0.354073	190		190	0.0	0.0	0.406	0.507	0.324	1.096	78	1
2500	2500-01	0.204269	109		109	0.0	0.0	0.235	0.294	0.188	1.168	48	2

Bayou Meto Water Supply Study
Inverted Siphon Design for Natural Drainage

Canal No.	Siphon No.	Drainage basin Area (sq mi)	Calculated Q100 (cfs)		Selected Q100 (cfs)	Head (ft.)	Tailwater Depth (ft.)	Energy Losses (ft)				Selected Siphon Diameter (in.)	Number of Siphons
			Rational Method	Previous Study				Entrance K=0.8	Exit K=1.0	Bend K=0.16	Friction C=60		
2500	2500-02	0.621043	333		333	0.2	0.0	0.430	0.537	0.344	1.270	72	2
2500	2500-03	0.652853	350		350	0.0	0.0	0.299	0.374	0.239	1.005	66	3
2520	2520-01	1.07721	577		577	0.0	0.0	0.409	0.511	0.327	0.867	96	2
2520	2520-02	0.511331	274		274	0.3	0.0	0.412	0.516	0.330	1.353	66	2
2531	2531-01	1.484296947	795		795	0.0	0.0	0.447	0.558	0.357	1.015	90	3
2280	2280-01	0.331194	177		177	0.0	0.0	0.275	0.344	0.220	1.348	48	3
2280	2280-02	0.291156	156		156	0.0	0.0	0.298	0.373	0.239	1.268	54	2
2280	2280-03	0.309023	165		165	0.1	0.0	0.336	0.420	0.269	1.416	54	2
2280	2280-04	0.240798	129		129	0.2	0.0	0.327	0.409	0.262	1.583	48	2
3000	3000-01	0.19134	102		102	0.0	0.0	0.206	0.258	0.165	1.035	48	2
3000	3000-02	0.177595	95		95	0.3	0.0	0.303	0.379	0.243	1.726	42	2
3000	3000-03	0.364809	195		195	0.1	0.0	0.430	0.538	0.344	1.159	78	1
3000	3000-04	0.101485	54		54	0.0	0.0	0.232	0.290	0.186	1.154	48	1
3000	3000-05	0.567705	304		304	0.0	0.0	0.331	0.414	0.265	1.234	60	3
3000	3000-06	0.567705	304		304	0.0	0.0	0.331	0.414	0.265	1.234	60	3
3000	3000-07	0.405825	217		217	0.2	0.0	0.380	0.475	0.304	1.404	60	2
3000	3000-10	0.411257	220		220	0.3	0.0	0.391	0.488	0.312	1.438	60	2
4000	4000-01	0.324107	174		174	0.0	0.0	0.263	0.329	0.211	1.296	48	3
4000	4000-02	0.269322	144		144	0.0	0.0	0.255	0.319	0.204	1.098	54	2
4000	4000-03	0.309534	166		166	0.1	0.0	0.337	0.422	0.270	1.420	54	2
4111	4111-01	0.333555	179		179	0.0	0.0	0.279	0.349	0.223	1.366	48	3
4111	4111-02	0.164183	88		88	0.0	0.0	0.259	0.324	0.207	1.493	42	2
4111	4111-03	0.321321	172		172	0.3	0.0	0.363	0.454	0.291	1.522	54	2
4112	4112-01	0.124684	67		67	0.0	0.0	0.255	0.318	0.204	1.144	52	1
4112	4112-02	0.431044	231		231	0.1	0.0	0.447	0.558	0.357	1.100	84	1
4112	4112-03	0.431044	231		231	0.0	0.0	0.291	0.363	0.233	1.238	54	3
4112	4112-04	0.113641	61		61	0.0	0.0	0.230	0.288	0.184	1.601	36	2
4112	4112-05	0.042308	23		23	0.0	0.0	0.161	0.202	0.129	1.851	24	2
4112	4112-06	1.016647	544		544	0.2	0.0	0.472	0.589	0.377	1.067	90	2
4112	4112-07	0.704862	377		743	0.0	0.0	0.390	0.488	0.312	0.896	90	3
4112	4112-08	1.398732	749		749	0.0	0.0	0.397	0.496	0.317	0.909	90	3
4112	4112-09	2.347766	1257		1257	0.2	0.0	0.486	0.607	0.388	1.017	96	4
4113	4113-01	1.162117	622		622	0.1	0.0	0.476	0.595	0.381	0.998	96	2



GATE WELL TURNOUT PLAN
(MULTIPLE CELL SHOWN, SINGLE CELL TYPICAL)
NOT TO SCALE

NOTE:
GATE WELL TURNOUT SITE PLANS VARY FROM ONE TO THREE CELL STRUCTURES WITH 30" TO 60" DIA. PIPES.

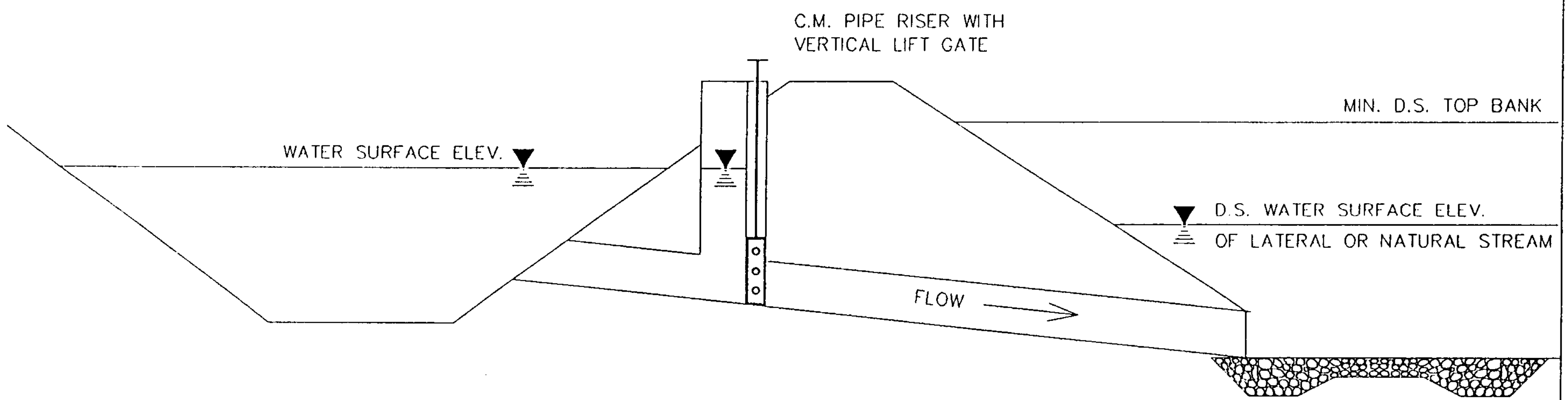
US Army Corps of Engineers

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEER
MEMPHIS, TENNESSEE

BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN
GATEWELL TURNOUT PLAN

PLATE
I-C-14

TYPE 1 TURNOUT STRUCTURE
Side (Gravity) Flow to Lateral Or Natural Stream



U.S. ARMY CORPS OF ENGINEERS
Memphis District

EASTERN ARKANSAS REGION
COMPREHENSIVE STUDY
TYPICAL TURNOUT
STRUCTURE (TYPE 1)

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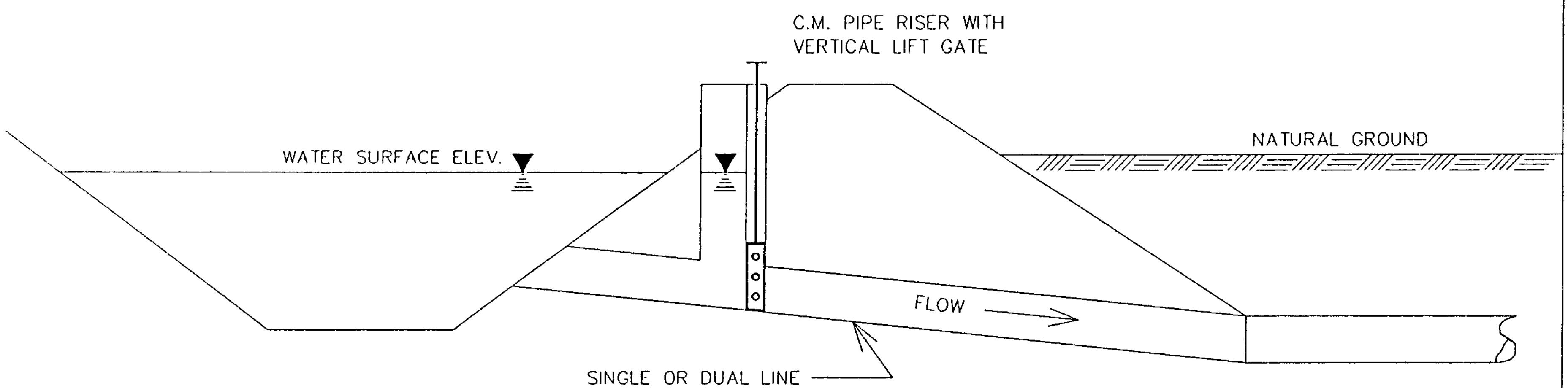
TYPE 2 TURNOUT STRUCTURE
Side (Gravity) Flow to Pipeline

2a – Single Line
2b – Dual Line



U.S. ARMY CORPS OF ENGINEERS
Memphis District

EASTERN ARKANSAS REGION
COMPREHENSIVE STUDY
TYPICAL TURNOUT
STRUCTURE (TYPE 2)

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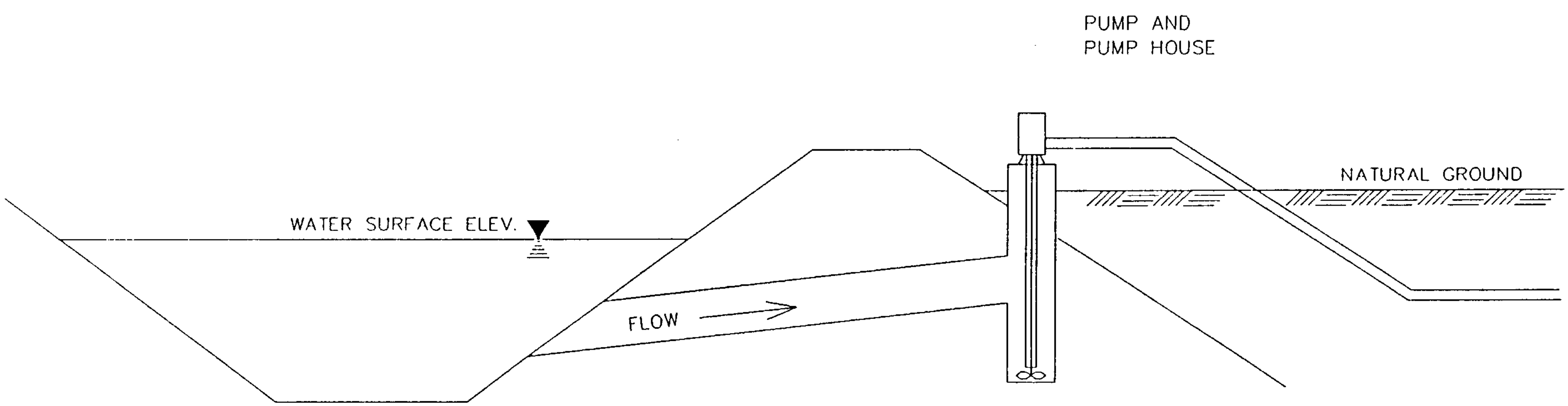
TYPE 3 TURNOUT STRUCTURE
Side (Pump) to Pipeline



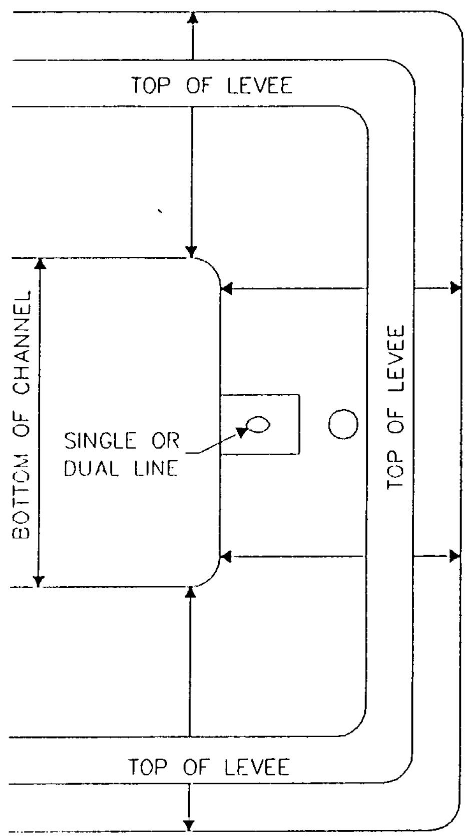
U.S. ARMY CORPS OF ENGINEERS Memphis District
EASTERN ARKANSAS REGION COMPREHENSIVE STUDY TYPICAL TURNOUT STRUCTURE (TYPE 3)

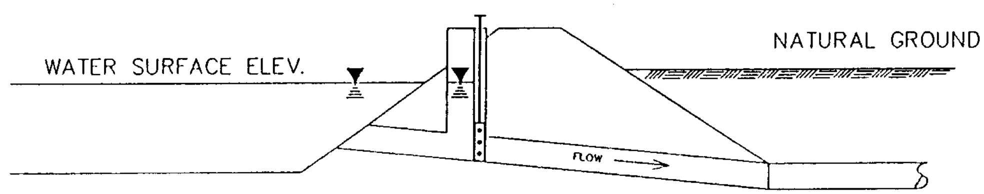
\COMMON\EARK\CANALS\CYBERNET\TURNOUT.DWG

TYPE 4 TURNOUT STRUCTURE

End (Gravity Flow to Lateral, Natural Stream or Pipeline

4a – Single Line
4b – Dual Line





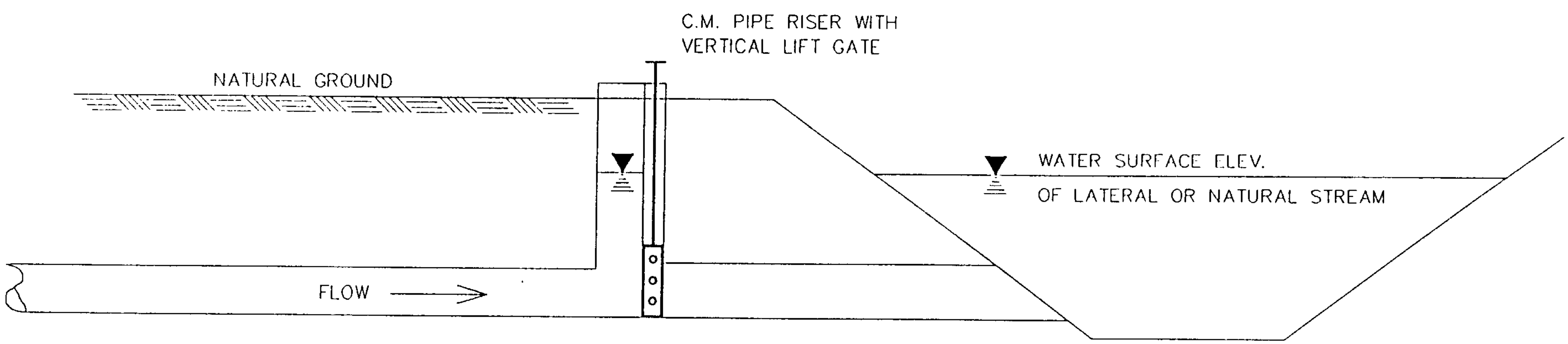
U.S. ARMY CORPS OF ENGINEERS Memphis District
EASTERN ARKANSAS REGION COMPREHENSIVE STUDY TYPICAL TURNOUT STRUCTURE (TYPE 4)

\COMMON\EARK\CANALS\CYBERNET\TURNOUT.DWG

TYPE 5 TURNOUT STRUCTURE

Gravity Flow From Pipe to Lateral or Natural Stream

5a – Single Line
5b – Dual Line



U.S. ARMY CORPS OF ENGINEERS
Memphis District

EASTERN ARKANSAS REGION
COMPREHENSIVE STUDY
TYPICAL TURNOUT
STRUCTURE (TYPE 5)

\COMMON\EARK\CANALS\CYBERNET\TURNOUT.DWG

GATEWELL STRUCTURES

Gatewell Structure	Headwater	Tailwater	Flow (cfs)	Diameter (in)	Number of Pipes	Velocity	Riprap Thickness (in)	Max Stone (lb)
T-1400	238	236	166	60	2	4.2	18	90
T-1410	237.5	235	82	48	2	4.0	18	90
T-1500	238	228	222	54	2	7.0	18	90
T-2200	220	215	370	60	3	6.3	18	90
T-2240	207.5	197.5	70	30	2	7.1	24	200
T-2510	238.5	229.5	68	30	2	6.9	18	90
T-2511	238.5	229.5	68	30	2	6.9	18	90
T2520	234	229.5	35	30	1	7.1	24	200
T-2530-1	233	229	144	48	2	5.7	18	90
T-2530-2	233	229	144	48	2	5.7	18	90
T-2532	215.5	207	76	48	1	6.0	18	90
T-4110	244	240	187	54	2	5.9	18	90
T-4113	237	235	33	36	1	4.7	18	90
T-3000R	242	238	316	60	3	5.4	18	90

CANEL NUMBER	STRUCTURE		U.S. POOL ELEV.	BOTTOM ELEV.	REMARKS
	NAME	TYPE			
1000	P-1000R	PUMP STATION			PUMP TO RESERVOIR R-1000
1000	C-1000R	CHECK STR.	245.00	234.00	
1000	C-1000.1	CHECK STR.	240.30	229.30	
1000	P-1000.01	TURNOUT STR. (TYPE 3)			PUMP TO 1000.01
1000	P-1000.02	TURNOUT STR. (TYPE 3)			PUMP TO 1000.02
1000	P-1000.03	TURNOUT STR. (TYPE 3)			PUMP TO 1000.03
1000	C-1000.2	CHECK STR.	234.60	225.60	
1000	P-1000.04	TURNOUT STR. (TYPE 3)			PUMP TO 1000.04
1400	T-1400	TURNOUT STR. (TYPE 1)			TO NEW STREAM 1400
1400	P-1400.01	TURNOUT STR. (TYPE 3)			PUMP TO 1400.01
1400	P-1400.02	TURNOUT STR. (TYPE 3)			PUMP TO 1400.02
1400	P-1400.03	TURNOUT STR. (TYPE 3)			PUMP TO 1400.03
1400	P-1400.04	TURNOUT STR. (TYPE 3)			PUMP TO 1400.04
1410	T-1410	TURNOUT STR. (TYPE 1)			TO NEW STREAM 1410
1410	P-1410.01	TURNOUT STR. (TYPE 3)			PUMP TO 1410.01
1410	P-1410.02	TURNOUT STR. (TYPE 3)			PUMP TO 1410.02
1410	P-1410.03	TURNOUT STR. (TYPE 3)			PUMP TO 1410.03
1500	T-1500	TURNOUT STR. (TYPE 1)			
1500	P-1500.01	TURNOUT STR. (TYPE 3)			PUMP TO 1500.01
1500	P-1500.014	TURNOUT STR. (TYPE 3)			PUMP TO 1500.014
1500	P-1500.02	TURNOUT STR. (TYPE 3)			PUMP TO 1500.02
1500	P-1500.03	TURNOUT STR. (TYPE 3)			PUMP TO 1500.03
1500	W-1500.01	WEIR	218.00	215.00	
1500	P-1500.04	TURNOUT STR. (TYPE 3)			PUMP TO 1500.04
1500	P-1500.05	TURNOUT STR. (TYPE 3)			PUMP TO 1500.05
1500	P-1500.06	TURNOUT STR. (TYPE 3)			PUMP TO 1500.06
1500	P-1500.07	TURNOUT STR. (TYPE 3)			PUMP TO 1500.07
1500	P-1500.08	TURNOUT STR. (TYPE 3)			PUMP TO 1500.08
1500	P-1500.09	TURNOUT STR. (TYPE 3)			PUMP TO 1500.09
1500	P-1500.10	TURNOUT STR. (TYPE 3)			PUMP TO 1500.10
1500	W-1500.02	WEIR	213.00	210.00	
1500	P-1500.11	TURNOUT STR. (TYPE 3)			PUMP TO 1500.11
1500	P-1500.12	TURNOUT STR. (TYPE 3)			PUMP TO 1500.12
1500	P-1500.13	TURNOUT STR. (TYPE 3)			PUMP TO 1500.13
1500	P-1500.131	TURNOUT STR. (TYPE 3)			PUMP TO 1500.131
1500	P-1500.14	TURNOUT STR. (TYPE 3)			PUMP TO 1500.14
1500	P-1500.15	TURNOUT STR. (TYPE 3)			PUMP TO 1500.15
1500	W-1500.03	WEIR	210.00	207.00	
1500	P-1500.17	TURNOUT STR. (TYPE 3)			PUMP TO 1500.17
1500	P-1500.18	TURNOUT STR. (TYPE 3)			PUMP TO 1500.18
1500	P-1500.19	TURNOUT STR. (TYPE 3)			PUMP TO 1500.19
1500	W-1500.04	WEIR	207.00	204.00	
1500	P-1500.20	TURNOUT STR. (TYPE 3)			PUMP TO 1500.20
1500	P-1500.21	TURNOUT STR. (TYPE 3)			PUMP TO 1500.21
1500	P-1500.22	TURNOUT STR. (TYPE 3)			PUMP TO 1500.22
1500	P-1500.23	TURNOUT STR. (TYPE 3)			PUMP TO 1500.23
1510	P-1510.01	TURNOUT STR. (TYPE 3)			PUMP TO 1510.01
1520	P-1520.01	TURNOUT STR. (TYPE 3)			PUMP TO 1520.01
1520	P-1520.02	TURNOUT STR. (TYPE 3)			PUMP TO 1520.02
1530	P-1530	TURNOUT STR. (TYPE 3)			PUMP TO NEW CANAL 1530
1530	P-1530.01	TURNOUT STR. (TYPE 3)			PUMP TO 1530.01
1530	P-1530.02	TURNOUT STR. (TYPE 3)			PUMP TO 1530.02
1530	P-1530.03	TURNOUT STR. (TYPE 3)			PUMP TO 1530.03
1530	W-1530.01	WEIR	202.50	199.50	
1530	P-1530.04	TURNOUT STR. (TYPE 3)			PUMP TO 1530.04
1530	P-1530.05	TURNOUT STR. (TYPE 3)			PUMP TO 1530.05
1530	P-1530.06	TURNOUT STR. (TYPE 3)			PUMP TO 1530.06
1530	P-1530.07	TURNOUT STR. (TYPE 3)			PUMP TO 1530.07
1530	P-1530.08	TURNOUT STR. (TYPE 3)			PUMP TO 1530.08

1530	P-1530.09	TURNOUT STR. (TYPE 3)			PUMP TO 1530.09
1530	P-1530.10	TURNOUT STR. (TYPE 3)			PUMP TO 1530.10
1531	P-1531.01	TURNOUT STR. (TYPE 3)			PUMP TO 1531.01
2000	P-2000.01	TURNOUT STR. (TYPE 3)			PUMP TO 2000.01
2000	C-2000.1	CHECK STR.	227.34	219.34	
2000	P-2000.02	TURNOUT STR. (TYPE 3)			PUMP TO 2000.02
2000	P-2000.03	TURNOUT STR. (TYPE 3)			PUMP TO 2000.03
2000	P-2000R	PUMP STATION			PUMP TO RESERVOIR R2500/3000
2100	P-2100.02	TURNOUT STR. (TYPE 3)			PUMP TO 2100.02
2100	P-2100.06	TURNOUT STR. (TYPE 3)			PUMP TO 2100.06
2100	P-2100.07	TURNOUT STR. (TYPE 3)			PUMP TO 2100.07
2100	W-2100.02	WEIR	210.00	207.00	
2100	P-2100.08	TURNOUT STR. (TYPE 3)			PUMP TO 2100.08
2100	P-2100.09	TURNOUT STR. (TYPE 3)			PUMP TO 2100.09
2100	P-2100.10	TURNOUT STR. (TYPE 3)			PUMP TO 2100.10
2100	W-2100.03	WEIR	207.00	204.00	
2100	P-2100.11	TURNOUT STR. (TYPE 3)			PUMP TO 2100.11
2110	C-2110	CHECK STR.			
2120	W-2120.01	WEIR	204.00	201.00	
2120	P-2120.01	TURNOUT STR. (TYPE 3)			PUMP TO 2120.01
2120	P-2120.02	TURNOUT STR. (TYPE 3)			PUMP TO 2120.02
2120	P-2120.03	TURNOUT STR. (TYPE 3)			PUMP TO 2120.03
2120	W-2120.02	WEIR	201.00	198.00	
2120	P-2120.04	TURNOUT STR. (TYPE 3)			PUMP TO 2120.04
2120	P-2120.05	TURNOUT STR. (TYPE 3)			PUMP TO 2120.05
2120	W-2120.03	WEIR			
2120	W-2120.04	WEIR	196.00	193.00	
2120	P-2120.06	TURNOUT STR. (TYPE 3)			PUMP TO 2120.06
2120	P-2120.07	TURNOUT STR. (TYPE 3)			PUMP TO 2120.07
2120	W-2120.05	WEIR	192.00	189.00	
2140	P-2140	PUMP STATION			PUMP TO NEW CANAL 2140
2140	P-2140.01	TURNOUT STR. (TYPE 3)			PUMP TO 2140.01
2140	P-2140.02	TURNOUT STR. (TYPE 3)			PUMP TO 2140.02
2140	P-2140.03	TURNOUT STR. (TYPE 3)			PUMP TO 2140.03
2140	P-2140.04	TURNOUT STR. (TYPE 3)			PUMP TO 2140.04
2140	P-2140.05	TURNOUT STR. (TYPE 3)			PUMP TO 2140.05
2140	P-2140.06	TURNOUT STR. (TYPE 3)			PUMP TO 2140.06
2140	P-2140.07	TURNOUT STR. (TYPE 3)			PUMP TO 2140.07
2140	P-2140.08	TURNOUT STR. (TYPE 3)			PUMP TO 2140.08
2140	P-2140.09	TURNOUT STR. (TYPE 3)			PUMP TO 2140.09
2140	W-2140.01	WEIR	200.00	197.00	
2140	P-2140.10	TURNOUT STR. (TYPE 3)			PUMP TO 2140.10
2140	P-2140.11	TURNOUT STR. (TYPE 3)			PUMP TO 2140.11
2140	P-2140E	TURNOUT STR. (TYPE 3)			PUMP TO EXISTING CANAL 2140
2140	P-2140.12	TURNOUT STR. (TYPE 3)			PUMP TO 2140.12
2140	P-2140.13	TURNOUT STR. (TYPE 3)			PUMP TO 2140.13
2160	W-2160.01	WEIR	195.00	192.00	
2160	P-2160.01	TURNOUT STR. (TYPE 3)			PUMP TO 2160.01
2160	P-2160.02	TURNOUT STR. (TYPE 3)			PUMP TO 2160.02
2160	P-2160.03	TURNOUT STR. (TYPE 3)			END OF CANAL--PUMP TO 2160.03
2200	T-2200	TURNOUT STR. (TYPE 1)			To LATERAL 2200
2200	P-2200.01	TURNOUT STR. (TYPE 3)			PUMP TO 2200.01
2200	P-2200.02	TURNOUT STR. (TYPE 3)			PUMP TO 2200.02
2200	P-2200.03	TURNOUT STR. (TYPE 3)			PUMP TO 2200.03
2200	P-2200.05	TURNOUT STR. (TYPE 3)			PUMP TO 2200.05
2200	P-2200.06	TURNOUT STR. (TYPE 3)			PUMP TO 2200.06
2200	W-2200.02	WEIR	202.67	199.67	
2200	P-2200.07	TURNOUT STR. (TYPE 3)			PUMP TO 2200.07
2200	W-2200.03	WEIR	199.67	196.67	
2200	P-2200.08	TURNOUT STR. (TYPE 3)			PUMP TO 2200.08

2200	W-2200.04	WEIR	196.67	193.67	
2200	W-2200.05	WEIR	193.88	190.88	
2200	W-2200.06	WEIR	190.88	187.88	
2200	P-2200.09	TURNOUT STR. (TYPE 3)			PUMP TO 2200.09
2200	P-2200.10	TURNOUT STR. (TYPE 3)			PUMP TO 2200.10
2200	P-2200.11	TURNOUT STR. (TYPE 3)			PUMP TO 2200.11
2200	P-2200.12	TURNOUT STR. (TYPE 3)			PUMP TO 2200.12
2200	W-2200.07	WEIR	187.88	184.88	
2220	P-2220	TURNOUT STR. (TYPE 3)			PUMP TO 2220
2220	P-2220.01	TURNOUT STR. (TYPE 3)			PUMP TO 2220.01
2240	T-2240	TURNOUT STR. (TYPE 1)			TO LATERAL 2240
2240	P-2240.01	TURNOUT STR. (TYPE 3)			PUMP TO 2240.01
2240	P-2240.02	TURNOUT STR. (TYPE 3)			PUMP TO 2240.02
2240	P-2240.03	TURNOUT STR. (TYPE 3)			PUMP TO 2240.03
2240	P-2240.04	TURNOUT STR. (TYPE 3)			PUMP TO 2240.04
2240	W-2240.01	WEIR	193.03	190.03	
2240	W-2240.02	WEIR	190.77	187.77	
2240	P-2240.05	TURNOUT STR. (TYPE 3)			PUMP TO 2240.05
2240	W-2240.03	WEIR	187.77	184.77	
2240	P-2240.06	TURNOUT STR. (TYPE 3)			PUMP TO 2240.06
2240	W-2240.04	WEIR	184.77	181.77	
2260	P-2260	TURNOUT STR. (TYPE 3)			PUMP TO 2260
2260	P-2260.01	TURNOUT STR. (TYPE 3)			PUMP TO 2260.01
2260	P-2260.02	TURNOUT STR. (TYPE 3)			PUMP TO 2260.02
2260	P-2260.03	TURNOUT STR. (TYPE 3)			PUMP TO 2260.03
2260	P-2260.04	TURNOUT STR. (TYPE 3)			PUMP TO 2260.04
2260	P-2260.05	TURNOUT STR. (TYPE 3)			PUMP TO 2260.05
2280	P-2280	TURNOUT STR. (TYPE 3)			PUMP TO 2280
2280	P-2280.01	TURNOUT STR. (TYPE 3)			PUMP TO 2280.01
2280	P-2280.02	TURNOUT STR. (TYPE 3)			PUMP TO 2280.02
2280	P-2280.03	TURNOUT STR. (TYPE 3)			PUMP TO 2280.03
2280	P-2280.04	TURNOUT STR. (TYPE 3)			PUMP TO 2280.04
2280	P-2280.05	TURNOUT STR. (TYPE 3)			PUMP TO 2280.05
2280	P-2280.06	TURNOUT STR. (TYPE 3)			PUMP TO 2280.06
2280	P-2280.07	TURNOUT STR. (TYPE 3)			PUMP TO 2280.07
2280	P-2280.08	TURNOUT STR. (TYPE 3)			PUMP TO 2280.08
2300	P-2300	TURNOUT STR. (TYPE 3)			PUMP TO 2300.01
2300	P-2300.01	BOOSTER PUMP			PUMP THE REST OF 2300.01
2500	C-2500R	CHECK STR.			TO CANAL 2500
2500	C-2500-1	CHECK STR.			CANAL 2500
2500	P-2500.01	TURNOUT STR. (TYPE 3)			PUMP TO 2500.01
2500	C-2500-2	CHECK STR.			CANAL 2501
2500	P-2500.03	TURNOUT STR. (TYPE 3)			PUMP TO 2500.03
2510	T-2510	TURNOUT STR. (TYPE 1)			TO LATERAL 2510
2511	T-2511	TURNOUT STR. (TYPE 1)			TO LATERAL 2511
2511	W-2511.01	WEIR	232.56	229.56	
2511	W-2511.02	WEIR	229.56	226.56	
2511	P-2511.01	TURNOUT STR. (TYPE 3)			PUMP TO 2511.01
2511	W-2511.03	WEIR	226.56	223.56	
2511	W-2511.04	WEIR	223.56	220.56	
2511	W-2511.05	WEIR	220.56	217.56	
2511	W-2511.06	WEIR	217.56	214.56	
2511	P-2511.02	TURNOUT STR. (TYPE 3)			PUMP TO 2511.02
2511	P-2511.03	TURNOUT STR. (TYPE 3)			PUMP TO 2511.03
2511	W-2511.07	WEIR	214.56	211.56	
2511	P-2511.04	TURNOUT STR. (TYPE 3)			PUMP TO 2511.04
2511	W-2511.08	WEIR	212.27	209.27	
2511	P-2511.05	TURNOUT STR. (TYPE 3)			PUMP TO 2511.05
2511	W-2511.09	WEIR	209.78	206.78	

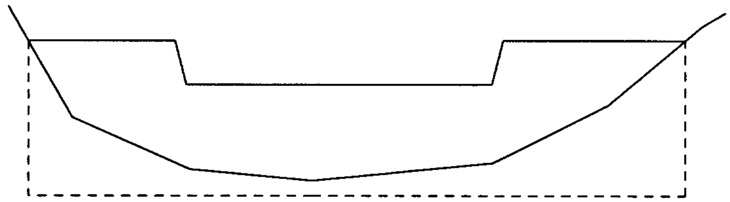
2520	T-2520	TURNOUT STR. (TYPE 1)			TO LATERAL 2520
2520	P-2520	TURNOUT STR. (TYPE 3)			PUMP TO 2520.01
2520	P-2521.01	TURNOUT STR. (TYPE 3)			PUMP TO 2521.01 AND 2521.02
2530	T-2530-1	TURNOUT STR. (TYPE 1)			TO LATERAL NEW CANAL 2530
2530	T-2530-2	TURNOUT STR. (TYPE 1)			TO LATERAL EXIST. CHANNEL 2530
2530	W-2530.01	WEIR	223.93	220.93	
2530	W-2530.02	WEIR	220.93	217.93	
2530	P-2530.01	TURNOUT STR. (TYPE 3)			PUMP TO 2530.01
2530	P-2530.02	TURNOUT STR. (TYPE 3)			PUMP TO 2530.02
2530	W-2530.03	WEIR	217.93	214.93	
2530	W-2530.04	WEIR	215.79	212.79	
2530	W-2530.05	WEIR	212.79	209.79	
2530	P-2530.03	TURNOUT STR. (TYPE 3)			PUMP TO 2530.03
2530	W-2530.06	WEIR	209.46	206.46	
2530	P-2530.04	TURNOUT STR. (TYPE 3)			PUMP TO 2530.04
2530	P-2530.05	TURNOUT STR. (TYPE 3)			PUMP TO 2530.05
2530	W-2530.07	WEIR	206.46	203.46	
2530	W-2530.08	WEIR	205.04	202.04	
2530	W-2530.09	WEIR	202.04	199.04	
2530	P-2530.06	TURNOUT STR. (TYPE 3)			PUMP TO 2530.06
2530	W-2530.10	WEIR	199.04	196.04	
2530	P-2530.07	TURNOUT STR. (TYPE 3)			PUMP TO 2530.07
2530	W-2530.11	WEIR	196.05	193.05	
2531	P-2531	TURNOUT STR. (TYPE 3)			PUMP TO 2531
2531	P-2531.01	TURNOUT STR. (TYPE 3)			PUMP TO 2531.01
2532	T-2532	TURNOUT STR. (TYPE 1)			TO LATERAL 2532
2532	W-2532.01	WEIR	203.70	200.70	
2532	W-2532.02	WEIR	200.70	197.70	
2532	P-2532.01	TURNOUT STR. (TYPE 3)			PUMP TO 2531.01
2532	P-2532.02	TURNOUT STR. (TYPE 3)			PUMP TO 2531.02
2532	P-2532.03	TURNOUT STR. (TYPE 3)			PUMP TO 2531.03
2532	W-2532.03	WEIR	197.70	194.70	
2532	P-2532.04	TURNOUT STR. (TYPE 3)			PUMP TO 2531.04
2532	P-2532.05	TURNOUT STR. (TYPE 3)			PUMP TO 2531.05
2532	W-2532.04	WEIR	194.74	191.74	
2533	P-2533	TURNOUT STR. (TYPE 3)			PUMP TO NEW CANAL 2533
2533	P-2533.01	TURNOUT STR. (TYPE 3)			PUMP TO 2533.01
2533	P-2533.02	TURNOUT STR. (TYPE 3)			PUMP TO 2533.02
2533	P-2533.03	TURNOUT STR. (TYPE 3)			PUMP TO 2533.03
2535	P-2535	TURNOUT STR. (TYPE 3)			PUMP TO 2535
2540	W-2540.01	WEIR	229.38	226.38	
2540	W-2540.02	WEIR	226.45	223.45	
2540	W-2540.03	WEIR	223.45	220.45	
2540	W-2540.04	WEIR	220.45	217.45	
2540	W-2540.05	WEIR	217.45	214.45	
2540	P-2540.01	TURNOUT STR. (TYPE 3)			PUMP TO 2540.01
2540	P-2540.02	TURNOUT STR. (TYPE 3)			PUMP TO 2540.02
2540	P-2540.03	TURNOUT STR. (TYPE 3)			PUMP TO 2540.03
2540	W-2540.06	WEIR	214.45	211.45	
3000	T-3000R	TURNOUT STR. (TYPE 2)			TO LATERAL 3000 PIPE
3000	P-3000.01	TURNOUT STR. (TYPE 3)			PUMP TO 3000.01
3000	P-3000.02	TURNOUT STR. (TYPE 3)			PUMP TO 3000.02
3000	P-3000.03	TURNOUT STR. (TYPE 3)			PUMP TO 3000.03
3000	P-3000.04	TURNOUT STR. (TYPE 3)			PUMP TO 3000.04
3000	P-3000.05	TURNOUT STR. (TYPE 3)			PUMP TO 3000.05
3000	C-3000	CHECK STR.			
3000	P-3000.06	TURNOUT STR. (TYPE 3)			PUMP TO 3000.06
3000	P-3000.07	TURNOUT STR. (TYPE 3)			PUMP TO 3000.07
3000	P-3000.08	TURNOUT STR. (TYPE 3)			PUMP TO 3000.08
3000	P-3000R	PUMP STATION			PUMP TO RESERVOIR R-4000

4000	P-4000.01	TURNOUT STR. (TYPE 3)			PUMP TO 4000.01
	C-4000	CHECK STR.			
	P-4000.02	TURNOUT STR. (TYPE 3)			PUMP TO 4000.02
	P-4000.03	TURNOUT STR. (TYPE 3)			PUMP TO 4000.03
4100	P-4100.01	TURNOUT STR. (TYPE 3)			PUMP TO 4100.01
4110	T-4110	TURNOUT STR. (TYPE 1)			TO LATERAL 4110
4110	P-4110.01	TURNOUT STR. (TYPE 3)			PUMP TO 4110.01
4110	W-4110.01	WEIR	238.21	235.21	
4110	P-4110.02	TURNOUT STR. (TYPE 3)			PUMP TO 4110.02
4110	W-4110.02	WEIR	235.46	232.46	
4111	P-4111	TURNOUT STR. (TYPE 3)			PUMP TO NEW CANAL 4111
4111	P-4111.01	TURNOUT STR. (TYPE 3)			PUMP TO 4111.01
4111	P-4111.02	TURNOUT STR. (TYPE 3)			PUMP TO 4111.02
4111	C-4111.01	CHECK STR.			
4111	P-4111.05	TURNOUT STR. (TYPE 3)			PUMP TO 4111.05
4111	P-4111.06	TURNOUT STR. (TYPE 3)			PUMP TO 4111.06
4111	P-4111.07	TURNOUT STR. (TYPE 3)			PUMP TO 4111.07
4111	P-4111.08	TURNOUT STR. (TYPE 3)			PUMP TO 4111.08
4111	P-4111.09	TURNOUT STR. (TYPE 3)			PUMP TO 4111.09
4112	P-4112	TURNOUT STR. (TYPE 3)			PUMP TO NEW CANAL 4112
4112	P-4112.01	TURNOUT STR. (TYPE 3)			PUMP TO 4112.01
4112	P-4112.02	TURNOUT STR. (TYPE 3)			PUMP TO 4112.02
4112	P-4112.03	TURNOUT STR. (TYPE 3)			PUMP TO 4112.03
4112	P-4112.04	TURNOUT STR. (TYPE 3)			PUMP TO 4112.04
4112	P-4112.05	TURNOUT STR. (TYPE 3)			PUMP TO 4112.05
4112	P-4112.07	TURNOUT STR. (TYPE 3)			PUMP TO 4112.07
4113	T-4113	TURNOUT STR. (TYPE 1)			TO LATERAL 4113
4113	P-4113.01	TURNOUT STR. (TYPE 3)			PUMP TO 4113.01
4200	P-4200	TURNOUT STR. (TYPE 3)			PUMP TO 4200
4200	P-4200.01	TURNOUT STR. (TYPE 3)			PUMP TO 4200.01

WEIR DESIGN AND DIMENSIONS

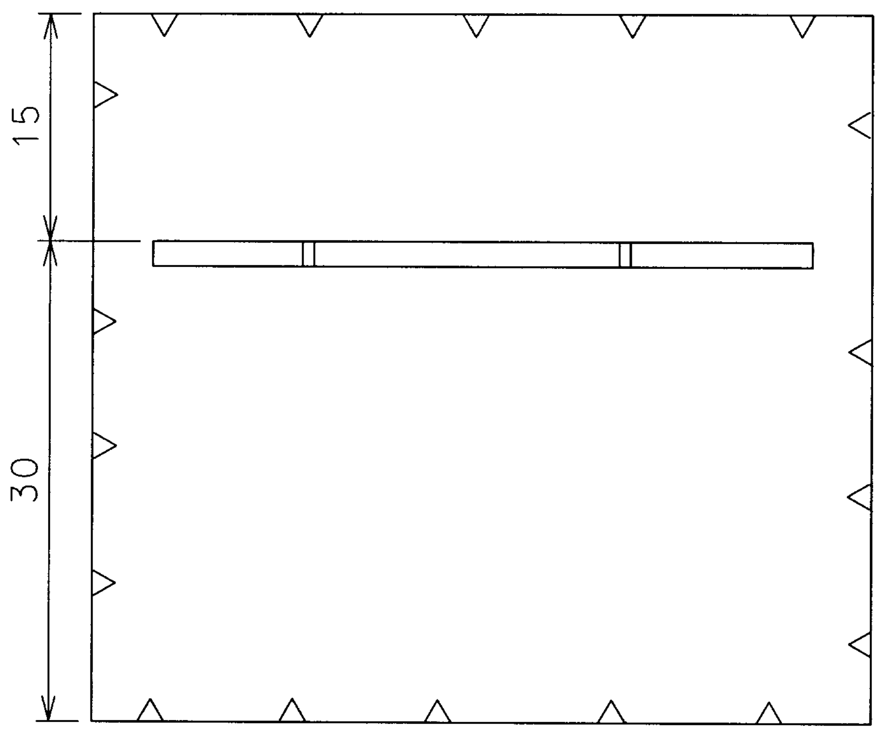
DITCH	WEIR NUMBER	CHANNEL DIMENSIONS			WEIR DIMENSIONS		NOTCH DIMENSIONS		
		Bottom Width (ft)	SS	Avg. Depth (ft)	Length (ft)	Height (ft)	Width (ft)	Height (ft)	Slope X/Y
INDIAN BAYOU	W-1500.01	20	1V:3H	7	62	6.6	20	3	0.25
	W-1500.02	20	1V:3H	10	65	8.5	15	3	0.25
	W-1500.03	20	1V:3H	10	60	7.5	10	3	0.25
	W-1500.04	20	1V:3H	13	75	10	10	3	0.25
	W-1530.01	15	1V:3H	6	39	5	10	3	0.25
CANEY CREEK	W-2100.02	20	1V:3H	10	68	9	25	3	0.25
DITCH	W-2100.03	20	1V:3H	13	56	7	25	3	0.25
	W-2120.01	20	1V:3H	11	62	8	20	3	0.25
	W-2120.02	20	1V:3H	8	50	6	20	3	0.25
	W-2120.03	20	1V:3H	14	56	7	20	3	0.25
	W-2120.04	20	1V:3H	11.5	59	7.5	20	3	0.25
	W-2120.05	20	1V:3H	12.5	62	8	15	3	0.25
	W-2140.01	20	1V:3H	7	62	6	15	3	0.25
	W-2160.01	5	1V:3H	5	35	4	10	3	0.25
CROOKED	W-2200.02	50	1V:3H	7	80	6	25	3	0.25
CREEK DITCH	W-2200.03	50	1V:3H	7	80	6	25	3	0.25
	W-2200.04	50	1V:3H	7	80	6	25	3	0.25
	W-2200.05	50	1V:3H	7	80	6	25	3	0.25
	W-2200.06	50	1V:3H	7	80	6	20	3	0.25
	W-2200.07	50	1V:3H	18	92	8	10	3	0.25
BIG DITCH	W-2240.01	30	1V:3H	12.5	72	8	10	3	0.25
	W-2240.02	40	1V:3H	14	82	8	10	3	0.25
	W-2240.03	50	1V:3H	13	92	8	10	3	0.25
	W-2240.04	40	1V:3H	14	82	8	10	3	0.25
WHITE OAK	W-2511.01	20	1V:3H	8	56	7	10	3	0.25
BRANCH	W-2511.02	20	1V:3H	8	56	7	10	3	0.25
	W-2511.03	20	1V:3H	8	56	7	10	3	0.25
	W-2511.04	20	1V:3H	8	56	7	10	3	0.25
	W-2511.05	20	1V:3H	8	56	7	10	3	0.25
	W-2511.06	20	1V:3H	8	56	7	10	3	0.25

	W-2511.07	20	1V:3H	6	44	5	10	3	0.25
	W-2511.08	20	1V:3H	6	44	5	5	3	0.25
	W-2511.09	20	1V:3H	6	44	5	5	3	0.25
SKINNER	W-2530.01	20	1V:3H	10	62	8	20	3	0.25
BRANCH	W-2530.02	20	1V:3H	10	62	8	20	3	0.25
	W-2530.03	20	1V:3H	10	62	8	20	3	0.25
	W-2530.04	20	1V:3H	10	62	8	20	3	0.25
	W-2530.05	20	1V:3H	10	62	8	20	3	0.25
	W-2530.06	20	1V:3H	10	62	8	20	3	0.25
	W-2530.07	20	1V:3H	10	62	8	10	3	0.25
	W-2530.08	20	1V:3H	6	44	5	10	3	0.25
	W-2530.09	20	1V:3H	6	44	5	10	3	0.25
	W-2530.10	20	1V:3H	6	44	5	10	3	0.25
	W-2530.11	20	1V:3H	6	44	5	10	3	0.25
BLUE POINT	W-2532.01	30	1V:3H	14	78	9	10	3	0.25
DITCH	W-2532.02	30	1V:3H	14	78	9	10	3	0.25
	W-2532.03	30	1V:3H	14	78	9	10	3	0.25
	W-2532.04	30	1V:3H	14	78	9	10	3	0.25
SHUMAKER	W-2540.01	15	1V:3H	6	44	5	10	3	0.25
BRANCH	W-2540.02	15	1V:3H	6	44	5	10	3	0.25
	W-2540.03	15	1V:3H	6	44	5	10	3	0.25
	W-2540.04	15	1V:3H	6	44	5	10	3	0.25
	W-2540.05	15	1V:3H	6	44	5	5	3	0.25
	W-2540.06	15	1V:3H	6	44	5	5	3	0.25
RICKEY BRANCH	W-4110.01	40	1V:3H	7	70	6	25	3	0.25
	W-4110.02	30	1V:3H	7	60	6	25	3	0.25



TYPICAL WEIR PROFILE

NO SCALE



TYPICAL WEIR PLAN

NO SCALE

US Army Corps of Engineers

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEER
MEMPHIS, TENNESSEE

BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN

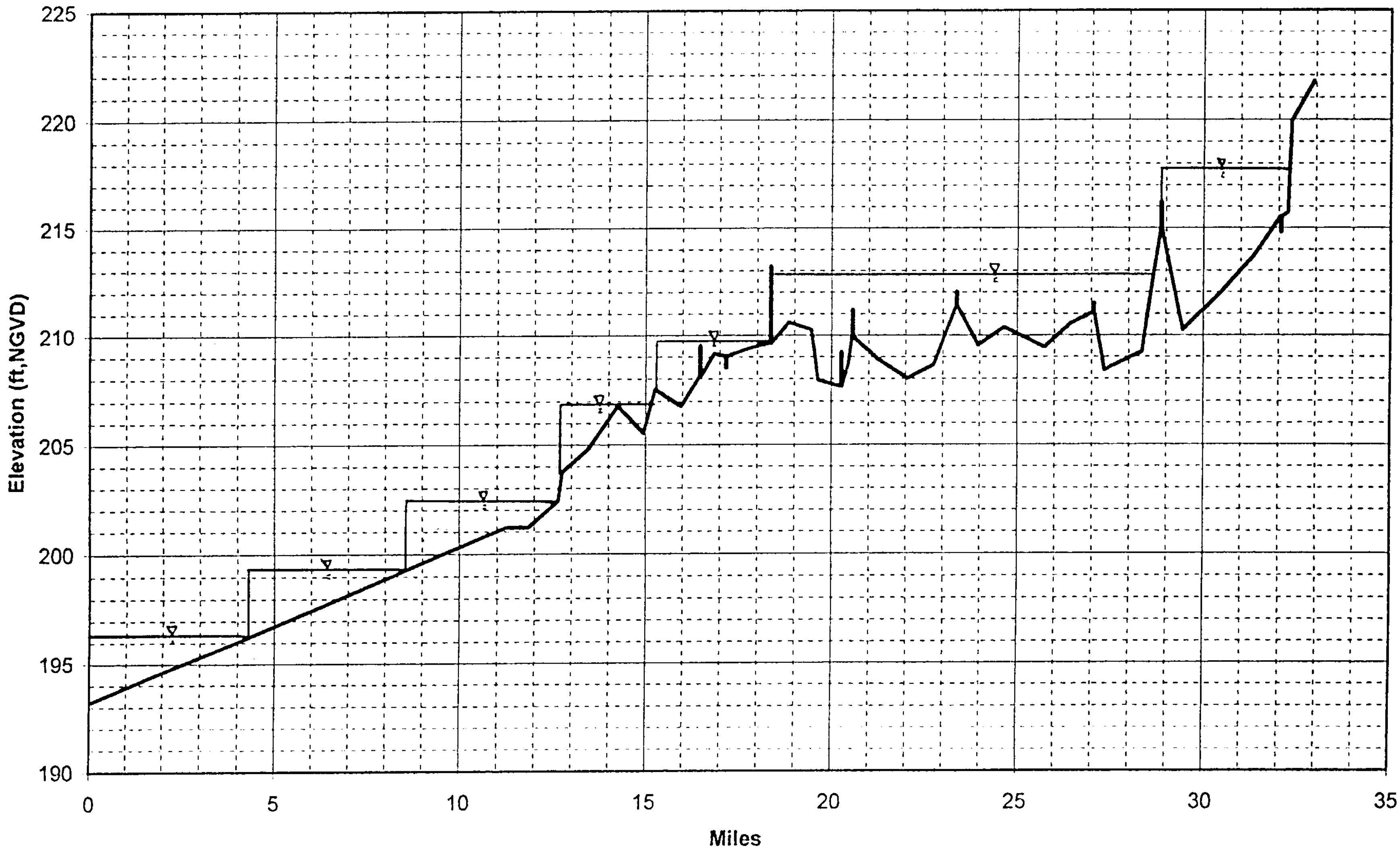
TYPICAL WEIR

PLATE

I-C-23

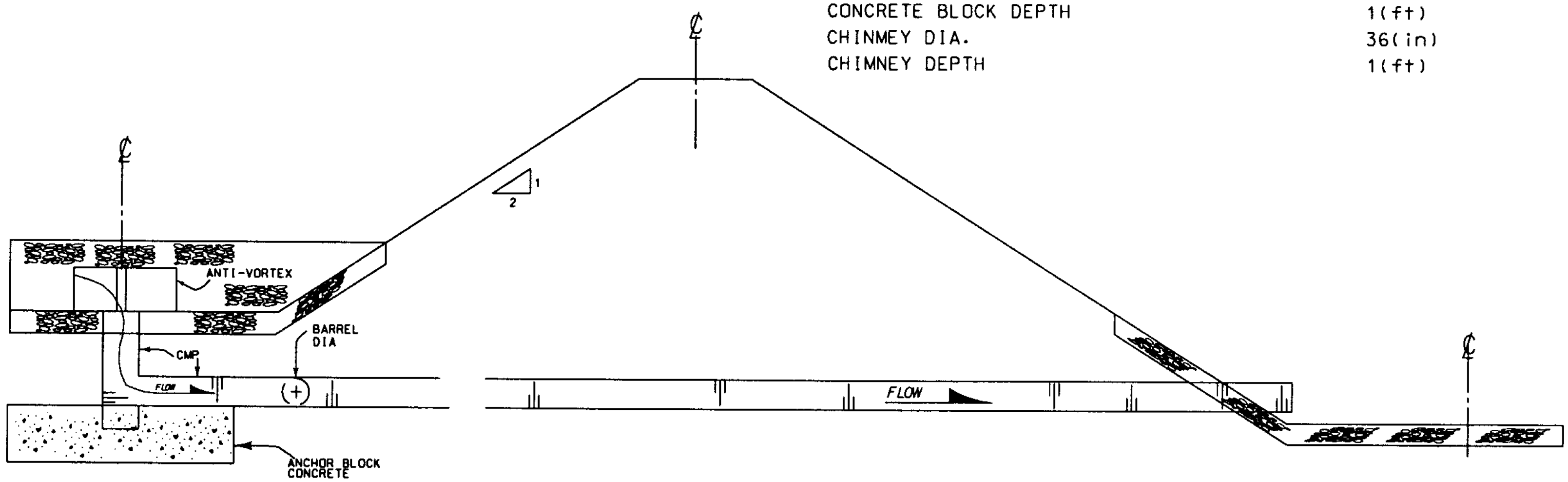
Indian Bayou Ditch

Weir Locations



DRAINAGE CHIMNEY DROP DIMENSIONS

LEVEE DEPTH	5(ft)
LEVEE WIDTH	5(ft)
SIDE SLOPE(X/Y)	2
BARREL DIA.	24(in)
RISER DIA.	30(in)
CONCRETE BLOCK	4(ft) x 4(ft)
CONCRETE BLOCK DEPTH	1(ft)
CHINMEY DIA.	36(in)
CHIMNEY DEPTH	1(ft)



PIPE DROP STRUCTURE

PROFILE
NOT TO SCALE

US Army Corps of Engineers

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BAYOU METO BASIN

DROP STRUCTURE

PLATE
I-C-25

PART II

SEDIMENT TRANSPORT

PART II - SEDIMENT TRANSPORT

TOPIC A – INLET CHANNEL

2-A-01. OVERVIEW.

1. General. Excess sediment accumulation at river diversionary canal headworks can interfere with canal operation, resulting in reduced conveyance capacity or requiring dredging maintenance. The purpose of this overview is to qualitatively characterize the nature and location of these sediment deposits and the mechanics for their formation. In addition, a limited literature search verified existence of models to predict stream sedimentation, although no model was found which professed to accurately predict sedimentation at intake structures. And, finally, research on sediment diversion vanes at channel inlets indicates that these structures work effectively to prevent sediment accumulation. A computer program was found to design the vane arrangement required for a specific geometry and flow profile.

2. Qualitative Assessment. Two types of sediment transport can be defined: bed load and suspended load. Bed load is sediment in contact with the bed during transport. Suspended load is sediment moving together with the water flow without contact with the bottom. Wash load is a subset of suspended load, and consists of cohesive and very fine sediments.

At a diversion channel intake, there is a flow separation boundary between downstream and diversion channel streamlines. This dividing stream-surface extends further out into the main channel near the bed than near the surface. Since most of the suspended sediment is carried in the lower 1/3 of the vertical flow [ASCE, 1977], as shown in Figure 1, this results in a disproportionate volume of sediment being carried with the flow into the diversionary channel. Flow structure of the water-sediment fluid is three-dimensional, with centrifugal forces acting on the non-uniform flow profile in the main channel and on the flow diverted into the canal [Neary et al., 1994]. A stagnation point tends to occur at the downstream face of the canal intake such that the pressure at that point is similar to upstream pressure. A separation zone in the diversion channel is caused by a secondary circulation pattern--a recirculating eddy causing velocity reduction and associated settling out of sediments (see Figure 2).

Jacob Odgaard, professor of Hydraulics at the University of Iowa, has conducted extensive research on sediment transport and deposits in diversionary channels. His research indicates that there is a direct relationship between the angle of the diversionary channel and the ratio of sediment transported into the intake. Research results show that a diversion operating within a Q2/Q1 range of 0.05 to 0.13 can reduce sand bar growth rates by roughly 70% by diverting water at 45-degrees instead of 90-degrees (where Q2 is diversionary channel discharge and Q1 is stream discharge) [Neary et al., 1994].

Model experimentation conducted by Odgaard et al. showed that streamwise convection dominated for a 90-degree diversion channel when the separation zone in the diversion channel was largest, suggesting a "squeezing" effect in which the transverse migration of deposited sediment bars toward the separation zone increased with increasing discharge ratio, roughness and diversion angle. The elliptical sediment bars normally reached their equilibrium position just within the near-bed boundary of the separation zone.

According to Melone, et al. [1975], velocity fluctuations in open channel flow, especially in the vicinity of the diversion headworks, are of the same magnitude as the settling velocities of silt and clay particles, thus the fine material load will remain in suspension and be transported with the diversion water. Heavier particles will tend to settle as described above.

3. Quantitative Measurement. Research for this analysis focused on identifying quantitative methods to determine the following four items:

1) Sediment movement. Identification of conditions for sediment movement in streams and quantification of the amount of sediment to expect in transport

2) Sediment distribution. Analysis of vertical distribution pattern of sediment in streams

3) Width of flow separation zone. Characterization of the flow separation zone at diversionary inlets

4) Sediment deposit at diversion channel inlet. Analysis of how much of the sediment carried into the diversionary channel could be expected to be deposited as sediment bars at the inlet.

Sediment movement: The presence of sediment in stream flow is a function of the tractive force of the water on the channel as related to the conditions for bed material transport. Tractive force is shown by Chow [1959] to be:

$$\tau = \gamma RS \tag{1}$$

g = unit weight of water
R = hydraulic radius = P/A
P = wetted perimeter
A = cross-sectional area of water
S = friction slope

Actual field measurements are the most reliable source of data for sediment movement. A suspended load sampler is most common. Measurements are taken to within 0.3 ft of the bed, and an estimate is made of the lowest 0.3 ft, generally assuming that 5 - 10% of bed material is carried in this regime. Wide, shallow channels typically carry more bed load than deep narrow channels. Streams with high turbulence tend to have a smaller bed load.[Melone, et al. 1975]

Many authoritative references are available concerning sediment transport in channels. The ASCE Manual on Sedimentation Engineering [1977] and the US Army Corps of Engineers Engineer Manual, Sedimentation Investigations of Rivers and Reservoirs [1989] provide a general treatment of the subject.

David T. Williams [1995] identified and conducted a comprehensive, well-documented comparison of four sediment transport relations, which predict total bed material load in sand-bed channels. The results of this investigation show that no one sediment transport relationship is best for all applications. It is possible, however, to judiciously select the best sediment transport model for any known set of conditions.

Williams reconstructed the original data sets used in development and evaluation of each of the four selected sediment transport relations, then tested all methods using each of the input data sets. It should be noted that data were excluded which were outside the applicable ranges cited by the developers of the relations, so that only appropriate results were used in comparison of the methods. The four methods, and results of Williams' comparison are as follows:

1) Ackers and White (1973). This transport function is based upon three dimensionless parameters: particle size, mobility and transport. The method is based upon total stream power for fine sediments, and on the product of net grain shear and stream velocity for coarse sediments and assumes that transport efficiency increases as sediment mobility increases.

2) Brownlie (1981). This method is based upon a relation between depth, discharge and bed slope developed over a wide range of uniform flow situations, then applied to unsteady, non-uniform flow. The method was calibrated based upon laboratory and field data and statistical analysis. Brownlie based his analytic approach on the recognition that flow depth is a critical factor in sediment transport predictions. His method was developed by comparing the methods of Ackers and White (1973); Bagnold (1966); Bishop, Simons, and Richardson (1965); Einstein (1950); Engelund and Fredsoe (1976); Engelund and Hansen (1967); Graf (1971); Laursen (1958); Ranga Raju, Garde, and Bhardwaj (1981); Rottner (1959); Shen and Hung (1971); Toffaleti (1968); and Yang (1973). These methods were compared for laboratory and field results in development of the Brownlie relation.

3) Engelund and Hansen (1967). This method is based upon the postulate that bed load results from effective bed shear acting directly on the bed surface, and that suspended load results from agitation by fluid turbulence. Wash load (very fine suspended particles) is considered negligible and is not included in the model. Engelund and Hansen utilize equilibrium of exposed grains to determine whether sediment motion will occur. Mobility of grains depends on relative size of the drag force, lifting force, and immersed grain weight. Horizontal drag force on each particle, τ_*, induced by the flow is proportional to bed shear. Vertical forces include lifting force due to hydrostatic pressure distribution around the grain and the immersed weight of the grain. In summary, the method relies upon classic soil mechanics theory.

4) Yang (1973). The Yang unit stream power sediment transport relation is based upon the hypothesis that the rate at which energy is dissipated in an open channel is related to the concentration of sediment being transported. This is expressed as a velocity-slope product, which physically can be considered the rate of potential energy dissipation per unit weight of water. The Yang relation is derived from basic fluid mechanics and turbulence theories. The

Yang dimensionless unit stream power equations can be expressed in a manner that includes criteria for incipient motion.

Williams' evaluation produced the following four general conclusions:

1) The Ackers and White relation is fairly good for laboratory data but tends to slightly over predict for most cases. However, it under predicts for field data, especially for rivers with medium sand (0.25 - 0.5 mm) and small flow depths (less than 1 foot). The method is not applicable for fine sediment sizes (<0.125 mm) or for large flow depths.

2) The Brownlie relation is at its best for low flow velocities and small depths with medium sands (0.25 - 0.5 mm). This method over predicts for laboratory data with concentrations less than 50 ppm.

3) The Engelund and Hansen relation over predicts at low shear values and generally over predicts for field data except for medium sands (0.25-0.5 mm) where the results under predict actual sediment transport. The relation does well for light weight sediments (specific gravity, G_s, less than 2.65).

4) The Yang relation is not applicable for fine sediments (<0.125 mm). It is good for flume and small rivers, although it under predicts for large flow depths. It is highly applicable for medium sands (0.25 - 0.5 mm), but does not do well for lightweight sediment (G_s less than 2.65), and over predicts for coarse sands (>1.0 mm).

Williams developed a method that numerically rates the predictability of these four commonly used sediment transport relations for given hydraulic and sediment conditions. This method is used to select the appropriate sediment transport relation for any laboratory or field data based upon an assessment of dimensionless parameters for grain diameter, D_{gr}; grain Froude number, F_{gr}; stream power, P; and flow depth, Z.

Sediment distribution: Vertical distribution of sediment was shown by Rouse [1950] as :

$$C/C_a = [\{(D - y)/y\}\{a/(D-a)]^z \qquad (2)$$

C = concentration at a distance y from the bed
C_a = concentration at a point a above the bed
D = depth of flow
$z = \omega/\beta K v^*$ = Rouse number
ω = particle fall velocity
β = coefficient relating diffusion coefficients
K = von Karman velocity coefficient (often 0.4)
v^* = shear velocity = $(gRS)^{1/2}$

Small values of Rouse number correspond with small particles and turbulent flow, tending toward uniform sediment distribution. Large values of Rouse number produce sediment concentrations in the lower 1/3
of the flow depth, with little sediment at the water surface. This is characteristic for larger particles and less turbulent conditions.

Width of flow separation zone: Neary [1992] uses mass, momentum and energy conservation laws to predict the maximum width of the separation zone at a diversionary channel and the distance that the dividing streamline extends into the main channel.

Governing equations:

$$q = Q_2/Q_1 \tag{3}$$

Q_1 = main channel discharge
Q_2 = diversion channel discharge

$$S = W_1 \times q \tag{4}$$

W_1 = Width of streamline (main channel width)
S = Distance the stream line extends into the main channel

$$p = \text{pressure at a section} = p_{(hydrostatic)}/(\rho g) \tag{5}$$

$$p_s = \text{stagnation pressure} = p_1 + Q_1^2/(2gW_1^2) \tag{6}$$

(stagnation pressure is approximately the same as pressure immediately upstream of the flow division)

$$\omega = \tan -1[(u_2 \sin \delta)/u_1] \tag{7}$$

ω = diversion channel stream inflow deflection
δ = diversion angle (angle of departure from main channel centerline)
u_1 = main stream velocity in the direction of the main channel
u_2 = diversion stream velocity in the direction of the diversion channel

Applying the momentum equation in the direction of the lateral branch and substituting for stagnation pressure yields

$$p1\text{-}p2 = Q_2^2/(g\mu W_2^2) - (Q_1Q_2 \cos \varepsilon)/gW_1W_2 - (1/4)(Q_1^2/W_1^2) \tag{8}$$

μ = contraction ratio (fraction of the branch channel not occupied by the separation zone)
ε = the angle $\delta - \omega$

Applying the energy equation and substituting for stagnation pressure yields

$$p1\text{-}p2 = Q_2^2/2g\{(1/\mu^2 W_2^2) - [(3/2)(1/q^2\ W_1^2)]\} \tag{9}$$

Setting (8) and (9) equal and simplifying yields the quadratic equation

$$(1/\mu)^2 - 2\,(1/\mu) + 2(w/q)\cos\varepsilon - (w/q)^2 = 0 \tag{10}$$

This is solved for μ. The maximum width of the separation zone, H, is

$$H = W_2\,(1 - \mu) = S_{max} \tag{11}$$

This set of equations must be solved for each of the vertical flow strata, considering the effect of differential sediment load from surface to stream bed, to arrive at an estimate of the three-dimensional flow separation boundary and width of the separation zone at the midpoint of each stratum.

Sediment deposit at diversion channel inlet: Having calculated the sediment load in the main channel (selecting the transport model per Williams), the vertical sediment distribution (per Rouse), and the width of the separation zone at each of the flow strata (per Neary), another analytic step is required to assess how much of the sediment could be expected to be deposited at the inlet. No relation was found which modeled the recirculating eddy current, which is responsible for streamflow velocity reduction and consequent sediment deposition at the inlet. There is evidence that a 90-degree diversion angle exacerbates the conditions for sediment deposit, and an adverse slope for the diversion canal would also contribute to these conditions.

4. Available Sediment Transport Methods. Mathematical relations for the simulation of non-equilibrium, suspended sediment transport in open channels are based on the solution of 2-D or 3-D convection-diffusion equation and depth-integrated models. Several of these relations have been incorporated into computer programs and are available on the market. The previously mentioned Williams [1995] method could be used to select the best analytic model for the conditions at the inlet. Mendez [1996] cites several available mathematical models for analyzing suspended sediment transport. He compared 5 methods (Ackers-White, Brownlie, Engelund-Hansen, Van Rijn, and Yang) and found that the Ackers-White and Brownlie methods seem best suited to predict sediment transport rate in irrigation canals. It was noted that all methods performed better for high sediment concentrations than for concentrations lower than 500 ppm.

5. Cross-section Analysis. The WinXSPRO channel cross-section analyzer software program has capability for estimating sediment transport at a stream cross-section using the Ackers & White transport relation [WEST, 1996].

The USCOE Sediment Analysis Method, SAM, [Copeland et al., 1996] provides a methodology for assessing sediment transport relations for stable channel design. The SAM package provides the computation capability to perform a channel assessment incorporating four sedimentation processes at a cross-section: erosion, entrainment., transportation and deposition. The package is intended to be used as a planning tool. A brief summary of SAM capabilities follows:

SAM.hyd - Calculates normal depth and composite hydraulic parameters for a cross section with variable roughness. It will calculate stable channel dimensions --channel width, depth and slope-- for a prescribed discharge and sediment load.

SAM.sed - Calculates the bed material sediment discharge-rating curve by size class using hydraulic parameters calculated in SAM.hyd or user specified input. SAM.sed applies the sediment transport functions at a point, with no temporal or special variability in the size class distribution.

SAM.yld - Calculates sediment yield passing a cross-section during a period of time. The time period can be a single event or an entire year. Flow input can be a flow duration curve or a hydrograph. Sediment discharge rating curve can be specified as either a ratio of sediment vs water discharge or as a concentration.

SAM.aid - Guides the user in selection of the proper transport functions based upon mean grain diameter, bed slope, velocity, channel width and flow depth.

SAM.avg - Allows for the creation of input files for SAM.sed using an HEC-2 TAPE95 input file.

6. Hydrodynamic models. HEC-6 [USACOE, 1991] is a one-dimensional numerical model that computes scour and deposition by simulating the interaction between the hydraulics of the flow and the rate of sediment transport. HEC-6 simulates the ability of a stream to transport sediment, using various sediment transport functions. The model was designed to simulate long-term river and reservoir behavior rather than the response of stream systems to short-term, single event, floods.

MIKE 11 is a 1-D hydrodynamic model developed by the Danish Hydraulic Institute, which permits the computation of non-cohesive sediment transport capacity together with corresponding accumulated erosion/sedimentation rate using several transport and calculation models. The model does not take into account topographical changes during the simulation period, with no feedback from the sediment transport computation to the hydrodynamic computations. MIKE 11 allows prediction using 5 sediment transport methods: Engelund-Hansen, Ackers-White, Smart-Jaeggi, Engelund-Fredsoe and van Rijn [Mendez, 1996].

SOBEK is a 1-D open channel dynamical numerical modeling system developed by Delft Hydraulics and the Institute for Inland Water Management and Waste Water Treatment of the Netherlands. The SOBEK system can be used to study the effect of proposed river training works, dredging optimization, indication of initial aggradation and degradation reaches, long term morphological changes [Mendez, 1996].

7. Diversionary Structures. Sediment can be diverted away from intake structures by means such as training walls, skimming weirs, guide vanes, or tunnel type sediment diverters.

Odgaard et al. have conducted extensive research on use of guide vanes in combination with training walls. The theory consists of solving the equations of the motion of water and sediment using appropriate boundary conditions. Vane-induced bed-shear stresses are calculated using airfoil theory [Wang, et al. 1996]. For fully developed flow at the equilibrium state the equations are reduced to

$\rho g S d = \tau_{bs} + \tau_{Vs}$ (12)

$\rho g S_n d = \tau_{bn} - \tau_{vn} - \rho u^2 d/r$ (13)

$d(d)/dn = -[m/\{\rho \kappa u B(\theta \Delta\ gD)^{1/2}]\ \tau_{bn}$ (14)

s and n = streamwise and transverse coordinates
r = radius of channel curvature
d = flow depth
u = depth-averaged velocity
S, S_n = water surface slopes in s and n directions,
τ_{bs}, τ_{bn} = bed shear stresses in s and n directions
τ_{Vs}, τ_{vn} = vane induced shear stresses in s and n directions
ρ = density of water
m = flow resistance factor = $\kappa\ (8/f)^{1/2}$
κ = Karman constant
f = Darcy Weisbach friction factor
D = median grain diameter
θ = Shields parameter
$\Delta = (\rho_s - \rho)/\rho$
ρ_s = density of sediment
g = acceleration due to gravity
B = function of Coulomb friction and of the ratio of lift-force to drag-

A computer program was developed at the University of Iowa to design vane arrangement for an intake sill based upon the following basic flow and sediment parameters: (1) Pre-vane average flow depth, d_o; (2) velocity u_o and resistance parameter m; (3) the channel's width-depth ratio b/d_o and radius-width ratio r/b; and (4) the sediment Froude number F_D, which is defined as

$$F_D = u_o/(gd)^{1/2} \quad (15)$$

Input to the computer program includes mean flow velocity, mean depth, sediment properties, and vane dimensions and spacing. The output is velocity and depth distributions.

Odgaard has collected data for several applications including the water intake at the Duane Arnold Energy Center on the Cedar River in Palo, Iowa. These results demonstrate that submerged vanes serve as an effective sediment management tool at water intakes. By introducing relatively small changes in the bed-shear stresses, arrays of vanes can generate significant changes in the distributions of velocity and depth in river channels. These changes can be calculated. The calculations apply only when the intake flow is small enough that the withdrawal causes little change in the river flow velocity in front of the intake (e.g., ratio of withdrawal velocity to river flow velocity less than 0.2).

8. Summary. Sediment is likely to accumulate at the intake of a 90-degree diversionary channel, especially if the diversion canal is at an adverse slope. 1-D and 2-D models exist to analyze sediment transport, and models exist to predict the flow separation at a branch channel, but no model was found which specifically addresses sediment in separated flow at diversionary channels. Submerged guide vanes have been shown as effective in diverting sediment, thereby avoiding accumulation at intake structures. Design charts and a computer design aid are available to assist in designing guide vanes.

2-A-02. INLET CHANNEL SEDIMENT ASSESSMENT.

1. General. A cursory channel scour/deposition analysis was conducted for the inlet channel leading up to the pump station based on information developed for the Arkansas River. This analysis was used to qualitatively estimate the amount of scour or deposition in the inlet channel with the project in place.

2. Design Methodology. Due to the adverse slope of -0.004 in the inlet channel the WES program SAM was not applicable. The channel was studied using a settling tank analysis method. The inflowing sediment concentration and gradations were assumed to be evenly mixed and to be the same as the Arkansas River as reported in the canal sedimentation section of this report.

The settling tank method determines the distance a particle can travel laterally before falling out due to gravitational effects. The controlling properties are the overflow rate V_o (sometimes called surface loading or critical velocity) determined by dividing the water discharge by the surface area of the "tank", and by the velocity in the vertical direction, (settling velocity), which is determined using Figure g1. If it is assumed that the horizontal water velocity is a uniform V_f, then all particles with $> V_o$ will settle out. The procedure to calculate the quantity of material deposited is described below:

1) Inlet channel discharges of 2,570 cfs, 1,750 cfs and 450 cfs were based on the discharges for canal C1000 as presented in the Canal Sedimentation Analysis section of this report. The discharges represent bank full, high flow and low flow conditions of canal reach C1000, respectively. The water surface in the inlet channel will maintain the same elevation as the Arkansas River with the exception of the localized effects caused by pump drawdown. Since the discharge and water surface elevation will remain constant through the channel velocity will increase as the flow area decreases. This will cause the channel velocity to be lowest at the Arkansas River and highest at the pump inlet. However, because even the highest velocities are relatively small (less than one ft/s), the uniform horizontal velocity assumption described above is reasonable.

2) The Arkansas River HEC-RAS model was run and a flowline elevation was computed using an average summer flow. Given the water surface elevation at the Arkansas River and the inlet canal geometry provided, the surface area was calculated for the inlet canal (the area did not vary by flow rate since the river water surface elevation was assumed constant). The discharge divided by the surface area yielded the overflow rate V_o.

3) The fall velocities were determined using Figure g1 based on a D_{84} of 0.062 mm and a D_{92} of 0.15 mm. These grain size diameters were based on the canal sedimentation analysis presented in this report.

4) Using the overflow rate, V_o, and the vertical velocity the percent removed was determined by the equation $(XXX//V_o)*100\%$. This equation makes the simplifying assumption of a horizontal flow tank which will tend to slightly underestimate the sediment deposition. A minimum and maximum percent removed was calculated for each discharge.

$$C(mg/l) = \frac{Q_s(tons/day)}{Q(cfs)} * \frac{2000lbs}{ton} * \frac{1day}{86400\sec} * \frac{10^6}{62.4lb/ft^3}$$

5) Once the percent removed was determined, the sediment discharge (Q_s) was found by converting the concentration to tons/day. The conversion was based on the following equation where C (mgl) is 74 mgl and Q (cfs) is equal to one of the three discharges above. The quantity Q_s was then multiplied by the percent of total material, i.e., 84% or 16%. The total amount deposited presented in the Summary Table on Plate II-A-1 was determined by adding the amount deposited in each size class for a given discharge.

4. Conclusions. The inlet channel scour/deposition analysis results the amount deposited are reported for each of the three discharges for two size classes. The Summary Table presents the total amount expected to deposit for each of the three discharges in tons/day and tons/year. This analysis is based on the pump on period only and does not account for the deposition, which will occur during the pump off period. The results indicate that all sediment drawn into the inlet canal from the river will deposit; therefore, the pumps would deliver clear water to the irrigation delivery canals. However, if the intake canal is not maintained, it is likely that some equilibrium condition would be reached where the pumped water would contain some fine sediments. A more detailed sedimentation analysis would need to be performed to fully analyze the quantity of deposition.

Further discussion pertaining to sedimentation within the inlet channel was presented in a review of literature written about diversionary channels and a qualitative assessment was made to characterize the nature and location of deposition. The adverse slope of the inlet channel served a two-fold purpose: 1) to enhance deposition trends near the Arkansas River while decreasing deposition near the pump intake, and 2) to substantially reduce the probability or larval fish being trapped in the inlet channel by reducing velocities. As current literature indicated, deposition would be primarily at the upstream (Arkansas River) bank of the inlet channel near its convergence with the Arkansas River. No quantitative estimate of this sediment could be made with available data. Periodic removal of shoaling in this area of the inlet channel will be necessary; however, the adverse slope should significantly extend the period of such maintenance. Dredging operations on the Arkansas River could be utilized to complete required maintenance.

Because of the wide variability in Arkansas River discharges and pumping station diversions, a quantitative estimate for annual maintenance was not possible with available data. The settling basin approach presented provided an estimate of potential daily deposition rates for several discharges. Since the Arkansas River stage greatly impacts the inlet channel velocities (inlet channel velocities could potentially be reversed for out-of-bank Arkansas River discharges [the inlet channel sweeps toward the downstream direction of Arkansas River flow] even with peak pumpage at the pumping station). An average daily rate of deposition was estimated as 9.5 t/day from available data.

Riprap bank and toe protection, 24 inches of R200 riprap, for the inlet channel will be required at the Arkansas River to ensure bank stability, particularly for high Arkansas River discharges.

TOPIC B - CANAL SEDIMENTATION

2-B-01. CANAL SCOUR / DEPOSITION ANALYSIS.

Because of the similarities between the Grand Prairie Demonstration Project and the Bayou Meto Irrigation Project in main canal design, velocities, and operation, it was determined that results derived in the Grand Prairie study were applicable to Bayou Meto. Sediment loading is actually lower on the Arkansas River than on the White River, so it was expected that impacts would be lower, but response in the system would be very similar. So the following analyses describe the Grand Prairie effort with conclusions being derived for Bayou.

A channel scour/deposition analysis was performed on canal reaches C1000, C2000, C3000 and C4000. Canal limits and geometry were obtained from the UNET geometry file for the main canal system. The Stable Channel routine in the HYDRAULIC DESIGN PACKAGE FOR CHANNELS (SAM) computer program was used to evaluate the potential for scour or deposition within each canal reach. SAM Model input requirements were determined as follows:

1) Canal geometry from the UNET model was extracted and analyzed for average geometric conditions. Channel slope, bank slope, area and roughness were considered in the selection of break points. However, canal reaches were uniform and did not require

further subdivision (break points were set at the limits between canal reaches). Average channel parameters were then determined for each canal reach.

2) Channel discharges representing high flow, low flow and bank full conditions were run in each canal reach. High and low flow discharges were estimated from the delivery system demands. Bank full discharges were calculated using Manning's equation.

3) The stable channel routine in SAM calculated the equilibrium inflowing sediment concentration for each canal reach. For the high and intermediate flows for each canal reach, the energy slope input for SAM was equal to the canal slope (0.00005). However, for the low flows, backwater due to intermediate controls (gates) becomes important, and the normal depth assumption is no longer valid. To model this situation with SAM (which always assumes normal depth), energy slopes were approximated using water surface slopes from the UNET model output for the main canal system.

4) SAM models were set up to run fixed inflowing sediment concentrations of 20 milligrams/liter (mgl), 40 mgl and 60 mgl and the results were tabulated and plotted. Design channel conditions were compared to these curves to determine the amount of canal scour or deposition.

5) SAM offers nine different methods for n-value calculation on the channel bed (Keulegan, Strickler, Limerinos, Brownlie, SCS A-E). The Brownlie bed roughness equations were chosen due to applicability to project conditions and minimal sediment information requirements compared to the other methods (which require a gradation curve). When using the Brownlie method SAM requires D_{50} and the geometric standard deviation of the bed sediment (computed using D_{84} and D_{16}). These grain size diameters were estimated by extracting the values from the White River sediment data that were within ±2,000 cfs from the average discharge for the pump on period of 12,500 cfs. The variability of ±2,000 cfs was based on variations in discharge during pump on period. The resulting 16 data points were plotted to determine the average percentage of material less than 0.062 mm. This average of 82% was used as the D_{84} input value in SAM. D_{50} and D_{16} were estimated as 0.02 mm and 0.006 mm respectively.

6) A canal sediment sensitivity analysis was performed to determine the effects of different gradations on the SAM output. Canal C1000 with the bank full discharge of 2,570 cfs was chosen for the analysis. The D_{84}, D_{50} and D_{16} were varied ±30% in 10% increments. The resulting plot exhibits the effects that variations in grain size have on the stable channel dimensions reported by SAM. The sensitivity of the output to changes in grain size is representative of the other channel reaches as well.

2-B-02. CANAL SCOUR / DEPOSITION ANALYSIS RESULTS.

SAM stable channel routine output represents equilibrium channel conditions, i.e., no channel scour or deposition. The existing channel conditions were plotted on these figures for determination of potential scour or deposition. Any inflowing concentration curve that plots above the existing channel condition will result in channel deposition and conversely, any curve below the existing channel condition will result in channel scour. Actual inflowing sediment concentration for the canal cannot be determined from the data available.

An estimate of the inflowing sediment concentration can be made if the flow in the canal is assumed to carry the same concentration as the White River. The sediment data from the USGS gage #07077800 at Clarendon was used to determine the sediment concentration in the White River during the months of canal inlet pump operation. The canal inlet pump operation period was determined from the delivery system demands. This operation period was plotted on the White River flow hydrograph in order to determine an average discharge for the months May through August. The average discharge was found to be 12,500 cfs at the Clarendon gage. To determine the concentration for a discharge of 12,500 cfs, a plot of sediment discharge vs. water discharge was prepared using White River sediment data from the Clarendon gage (Oct. 1974 - Sept. 1975). An average concentration of 2,500 tons/day was found to correlate to the average discharge of 12,500 cfs. It is assumed that the canals will have the same concentration as the White River. This assumption does not account for any filtering of sediment at the canal inlet. A conversion from tons/day to milligrams/liter (mg/l) was required due to the input requirements of SAM. The following conversion equation was used:

Using the conversion equation, the concentration was found to be approximately 74 mg/l. All canals will experience deposition at this concentration level. By observing the results of the sensitivity to grain size analysis it becomes evident that the canals will still have deposition at this concentration even if the grain sizes are off by ±30%.

A sedimentation basin analysis was performed on Canal 1000 using a discharge of 450 cfs to see how the results compared to the output from the stable channel analysis for low flows performed with SAM. SAM predicted 33,293 tons/year while the sedimentation basin analysis predicted about half as much, 17,868 tons/year.

2-B-03. SUMMARY OF GRAND PRAIRIE USED FOR BAYOU METO.

The deposition/scour results predict deposition in canals for a Arkansas River concentration level of 74 mgl. As the concentration level is decreased, the probability of deposition also decreases. If clear water is input to the system, the potential for scour exists. In addition, as the particle size decreases the probability of deposition also decreases. The analysis did not consider inflowing sediment from outside sources nor outflows of sediment along the canal at diversions.

The potential for deposition, as presented, represents worst-case conditions. Sediment data in the White River and Arkansas River indicates suspended sediment concentrations of approximately 74 mg/l. Inlet channel deposition was predicted to remove the vast majority of this material; therefore, water actually entering the canal system should be relatively sediment free. As such, deposition rates shown for concentrations of 40 mg/l and 60 mg/l would significantly overstate depositional tendencies. Actual deposition within the canal system should be minimal. Any scour potential should be limited by the cohesive nature of the canal banks (constructed to minimize seepage) and vegetative cover above the waterline. Additionally, the canal system will function as a series of pools, between check structures, except when at peak capacity. Average canal velocities at peak capacity will be 1.6 feet per second or less.

TOPIC C - DELIVERY DITCH SEDIMENTATION.

2-C-01. DITCH SEDIMENTATION.

General. A channel sedimentation analysis was conducted in the Bayou Meto Basin, Eastern Arkansas. The purpose of the study is to determine sediment yields, bed scour/deposition, and water profiles in the channel. A 10-year simulation period was selected for this sediment analysis. The result of this study will be used to demonstrate the current and future sediment erosion/deposition pattern in the Bayou Meto irrigation canals and ditches.

Several hydrology and hydraulics softwares were applied in this study. These are:

1. HEC-6T: Sedimentation in stream network program is a one-dimensional numerical sedimentation model, which can be used to compute non-cohesive sediment transport and deposition in a network channel. Based on the energy and momentum conservations, the model can also be used to calculate water profiles in the channel. Both water profile (or fixed bed) and sediment analysis (or movable bed) simulations were performed using HEC-6T model.

2. HEC-RAS: River Analysis System is a one-dimensional flow model for water profile calculation. This model is able to compute water surface profiles in subcritical, supercritical, and mixed flow regimes for steady and unsteady state conditions. The intention of using this model was to assist in verifying water profiles developed by the HEC-6T fixed bed model.

3. HUXRAIN: HUXRAIN is an in-house hydrological model developed by the U.S. Army Corps of Engineers, Memphis District. The program was to determine the daily discharge data for HEC-6T continuous simulations.

2-C-02. METHODOLOGY.

The pilot channel selected for this study is located in the Bayou Meto Basin, Eastern Arkansas. The channel, called Indian Bayou Ditch, begins River Mile 50.1 and ends at River Mile 58.3 within the Little Bayou Meto (Plate II-C-01). It was determined that this ditch was representative of the ditches in the basin and arguments supporting this assumption are stated within this section. The selection criterion of this channel were based on the following facts:

(1) The channel is a typical agriculture channel in Eastern Arkansas;

(2) The channel consists of man-made channel and natural channel;

(3) Simple runoff flow attributes to the channel and no significant backwater effect occurs in the channel; and

(4) Several typical bridges are located in the channel.

Although the size of the selected channel is relative small, sediment transport and sediment load pattern in this channel should be identical to other channels in the study basins. Colby and Laursen suggested that sediment transport load is a function of the mean flow velocity, channel condition, viscosity, water temperature, sediment shear stress, and concentrations of the sediment. In HEC-6T model, these data are essentially required for sediment analysis and these data among the channels should most likely be the same. The detailed description of these data is listed as follows:

(1) Geometric data in the HEC-6T model includes stations and elevations for each cross section, reach lengths, and the Manning coefficients. As mentioned, the selected channel is a typical agriculture irrigation channel. This channel geometric condition should have the similar condition as other irrigation channels in the study area. In addition, the channel consists of similar the Manning coefficients, reach length, and the contraction and expansion coefficients.

(2) Sediment data need to be specified in the HEC-6T model. Because no sediment gage station is located in the study area, the sediment data of the St. Francis River and the Cache River were adopted. From the sediment property analysis, the sediment property data indicated that there is no significant variation between these two rivers. Therefore, the sediment characteristics among the study channels, including the Indian Bayou Ditch and others, should have the similar pattern as the St. Francis River and the Cache River.

(3) At a certain temperature, water parameters, such as viscosity, density, etc., in the HEC-6T model are the same. This is a valid assumption for the sedimentation analysis. During the course of this study, the defaulted temperature of 60°F was used. Other water parameters were determined based on this temperature.

(4) Having specified the geometric data and the sediment data for the channel, the hydrologic data is the last required component for sediment analysis. The hydrologic parameters include the flow rate or discharge, water temperature, and a simulation period.

In order to illustrate the pilot channel that can be represented other channels in the study area, it needs to acquaint with if the selected channel has received the same amount of runoff from the attributed watershed. An analysis was conducted to verify the ratio of the discharge flow and drainage area within the Bayou Meto Basin. From the HEC-HMS model, the average discharge per drainage area was approximately 8.48 cfs/sq mi, ranging from 2.15 cfs/sq mi to 21.25 cfs/sq mi. Over 95% samples were between 3.91 cfs/sq mi and 13.05 cfs/sq mi. The Indian Bayou Ditch had the value of 6.19 cfs/sq mi. It clearly indicated that the Indian Bayou Ditch has received the average discharge ratio from its tributary as compared to other channels in the basins.

According to the above assumptions, the sedimentation analysis of the Indian Bayou Ditch was performed. The analysis procedure is listed below:

1. Channel geometry of the Indian Bayou Ditch was retrieved from the Bayou Meto HEC-RAS model developed by the U.S. Army Corps of Engineers, Vicksburg District for the Bayou Meto Irrigation project. The Indian Bayou Ditch consists of twenty-eight cross sections and four bridges. For the sedimentation analysis, the channel geometric parameters including channel cross-section coordinates, the Manning coefficient, reach length, and the contraction/expansion coefficients were identified and imported into the HEC-6T model.

2. Sediment data were obtained from the St. Francis Sedimentation Study. The majority of particle were identified and classified as sand.

3. Channel discharges were created either from the HEC-HMS or from the HUXRAIN model. Using a recurrence interval of 1.01, 2, 5, 10, 25, 50, 100-year rainfall data, the peak flow in the channel was established by the HEC-HMS model. Downstream water rating curve generated from the HEC-HMS model and the HEC-RAS model was considered as the boundary conditions in the HEC-6T. To determine the daily flow discharge, the HUXRAIN simulation model was used. Based on the rain gage data and a specific unit hydrograph in the region, the HUXRAIN model produced a 10-year daily flow rate.

4. HEC-6T was developed for both fixed bed and movable bed simulations. In fixed bed simulations, geometric data of each cross-section and channel discharge data were compiled into the HEC-6T model. Due to HEC-6T does not use the bridge modeling approach as the HEC-RAS model does, the geometric data of the deck and roadway cannot directly be coded into the HEC-6T model. Therefore, a "S-type" of

coding technique was used to model bridge cross sections. An example of the bridge coding is showed on Plate II-C-02. Using the standard step method, the HEC-6T fix bed model established water surface elevation for each cross section. A comparison was finally conducted for determining the discrepancy of water profiles between the HEC-RAS model and the HEC-6T fixed bed model. Since no significant difference between these models was found, the geometric data were then applied to the HEC-6T movable model.

5. Two movable bed models were set-up for this sedimentation analysis. The first model was based on the assumption of the St. Francis sedimentation study conducted by Mr. William A. Thomas. The study suggested that a one-year sediment yield in the St. Francis basin is equal to the sediments produced by 24.5 days of the 2-year peak discharge. In his study, it also suggested that multiplying the 24.5-day-interval by 50 times can represent 50 years of normal sediment yield.

   Because the Indian Bayou Ditch or Bayou Meto Basins is adjacent to the St. Francis basins, the first movable bed model adopted a 245-day simulation period of the 2-year peak discharge for 10-year sedimentation simulation. The sediment concentration data were also taken from the St. Francis sedimentation study. To verify the sediment data, the sediment concentrations of the Cache River at the U.S.G.S. gage # 070775000 were plotted and applied to the HEC-6T movable model (Plate II-C-03).

   The second movable bed model was created for traditional continuous-daily-discharge simulation. The geometric data and the sediment concentrations were the same as the first movable model. The hydrologic data were replaced by the daily flow discharges obtained from the HUXRAIN model. Based on rational assumptions that the discharge generated in the basin is directly run into the ditch, no branch flows or local flows were found in this HEC-6T model.

2-C-03. RESULTS.

Surface water elevations of the HEC-6T fixed model and the HEC-RAS model are plotted on Plate II-C-04. In general, the HEC-6T produced lower water surface profile than the HEC-RAS did because of the different modeling approach in ineffective area. From the selected 2-year-peak-flow discharges, the results indicated that two models had only 0.5 ft discrepancy, suggested that two models do agree with each other. Since two models had such high consistency, no further calibration or justification of the models were necessary. The geometric information was directly applied to the movable bed models.

The first HEC-6T movable model was performed by repeating 24.5 days of the 2-year-peak-flow discharges for 10 times. During the course of this study, no significant sediment scour or deposition was found in the study channel except RM 58.3, 57.2, and 52.3 (See Plate II-C-05). A total of 55836 cubic yards (or 6727 cubic yards/mile) of sediment was produced in 10 years. At RM 58.3, the model indicated that 15198 cubic yards of sediments or 2.41 feet of sediment deposition were accumulated. Due to the inflowing boundary condition, the RM 58.3 cross section has been changed from the fixed bed boundary to movable bed boundary. As a result, high sediment loads took placed in this section. The second highest sediment deposition localities were found in RM 57.2 and 52.3 that may cause by high local flows discharged to the sections. According to the HEC-HMS model, flow discharges in RM 57.2 and in RM 52.3 were increased from 342 cfs to 1032 cfs and from 1032 cfs to 1732, respectively. These inflows counted for 45160 and 46133 cubic yards of sediment yields in these two sections. Because of sediment associated with high local inflows, high depositions have effected on both sections. Based on the assumptions of the St. Francis study, it appears that the iterations of the 2-year-peak-flow discharge for a number of days, such as 245 days for 10-year simulation, was not proper approaches for high local inflows. It can only be used in single reach channel with low local flows. If the study channel contains a number of high local inflows, several peak sediment loads will expect to occur at the junction or following sections.

From the HUXRAIN continuous simulation, 3562 days of the daily flows in the Indian Bayou were imported into the HEC-6T model. The model was run when the initial flow elevation or base flow elevation in the channel was defaulted to be 214.5 ft. The results indicated that no significant sediment was built or scoured in the study channel. A total of 21902 cubic yards (or 2639 cubic yards/mile) of sediment was produced in 10 years. In Plate II-C-06, it appeared that very little aggradations and degradations were found in the study channel. Similar to the first movable bed model, the RM 58.3 cross section had the highest sediment deposition in the channel. The result showed that a 1.24 ft of sediment or a 15953-cubic-yards of sediment was deposited at RM 58.3.

2-C-04. CONCLUSIONS.

The sedimentation analysis was conducted on the Indian Bayou Ditch in the Bayou Meto Basin, Eastern Arkansas. Two HEC-6T movable models were developed for this sedimentation analysis. Both models contained the same geometric data and sediment data, but different hydrologic data. The first model was based on the assumptions of St. Francis study. A 245-day of the 2-year peak discharge was performed for a 10-year simulation. The second model applied traditional continuous sediment simulation approach and assumed that no local flow is taking place in the area. Based on the different assumptions, it appeared low sediment accumulation on the bottom of the channel, suggested that no progressive aggradations or degradations will be occurred in the next 10 years. Because the effects of variations are very insignificant, the channel design parameters, such as slope, depth, velocity, top width, will be remained as its origin state. Although some depositions are expected to occur in the channel, these are mainly due to the boundary and local flow conditions. As these rational assumptions have been modified, the discrepancy should be very limit.

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BAYOU METO IRRIGATION PROJECT (ARKANSAS RIVER)

PUMP INLET CHANNEL DEPOSITION QUANTITIES

(1)	(2)	(3)	(4)	(5)	(6)	(7)=(6)/(3)*100	(8)=74*(1)/371	(9)=(8)*(7)*(4)/1E4
Discharge (cfs)	Surface Area (sq. ft.)	V_O (ft/s)	% of Total Material	Particle Size (mm)	Settling Velocity (ft/s)	% Removed	Qs (t/day) for C=74mgl	Amount Deposited (t/day)
2570	282268	0.009	16	0.15	0.0410	100	512.61	82.0
1780	282268	0.006	16	0.15	0.0410	100	355.04	56.8
450	282268	0.002	16	0.15	0.0410	100	89.76	14.4
2570	282268	0.009	84	0.064	0.0098	100	512.61	430.6
1780	282268	0.006	84	0.064	0.0098	100	355.04	298.2
450	282268	0.002	84	0.064	0.0098	100	89.76	75.4

Column Description

(1) Discharges from Canal C1000 analysis
(2) Assumes constant WSEL of 159.54 in canal
(3) Column (1) divided by column (2)
(4) % of material represented by particle size (5)
(5) Material particle size
(6) Settling velocity for particle size (5) ref: Fig. 2.2 ASCE Man. & Reports in Eng. Practice #54

Summary Table		
	Amt. Deposited	
Q	min (t/day)	(t/yr)
2570	512.6	187,228
1780	355.0	129.676
450	89.8	32.783

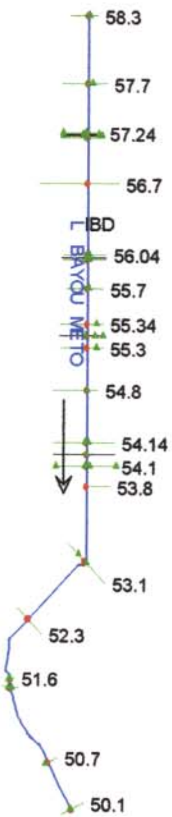

Figure 1. Schematic Diagram of Indian Bayou Ditch

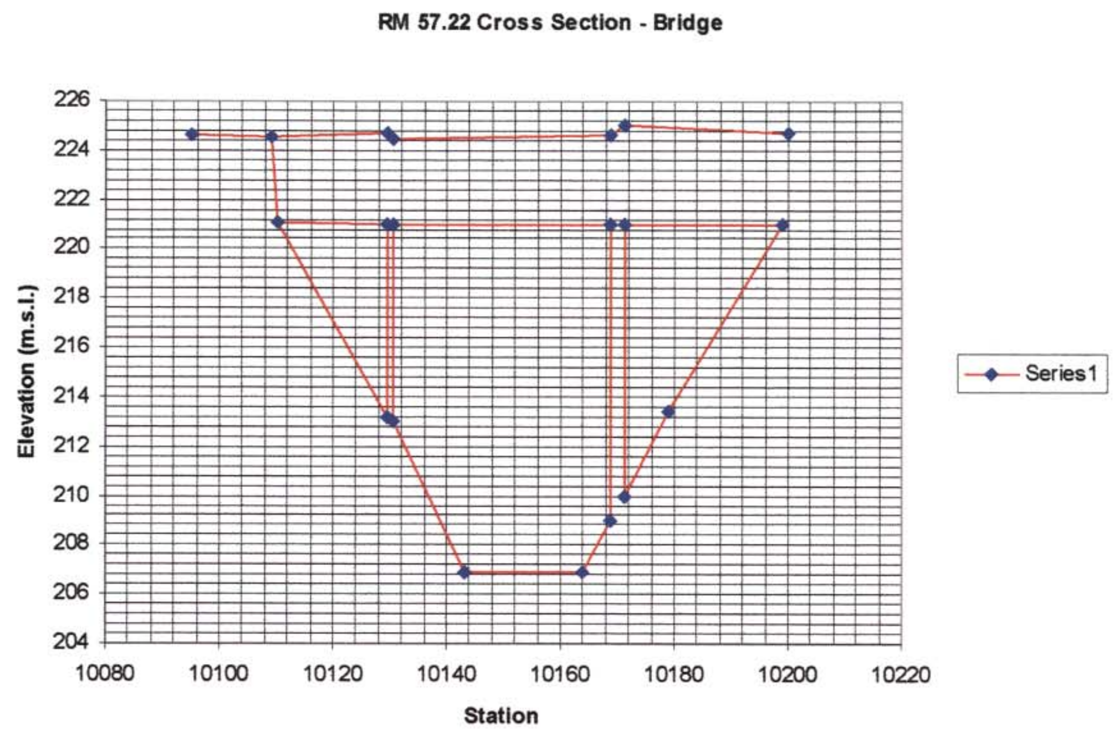

Figure 2. S-Type of Cross Section for Bridge Modeling

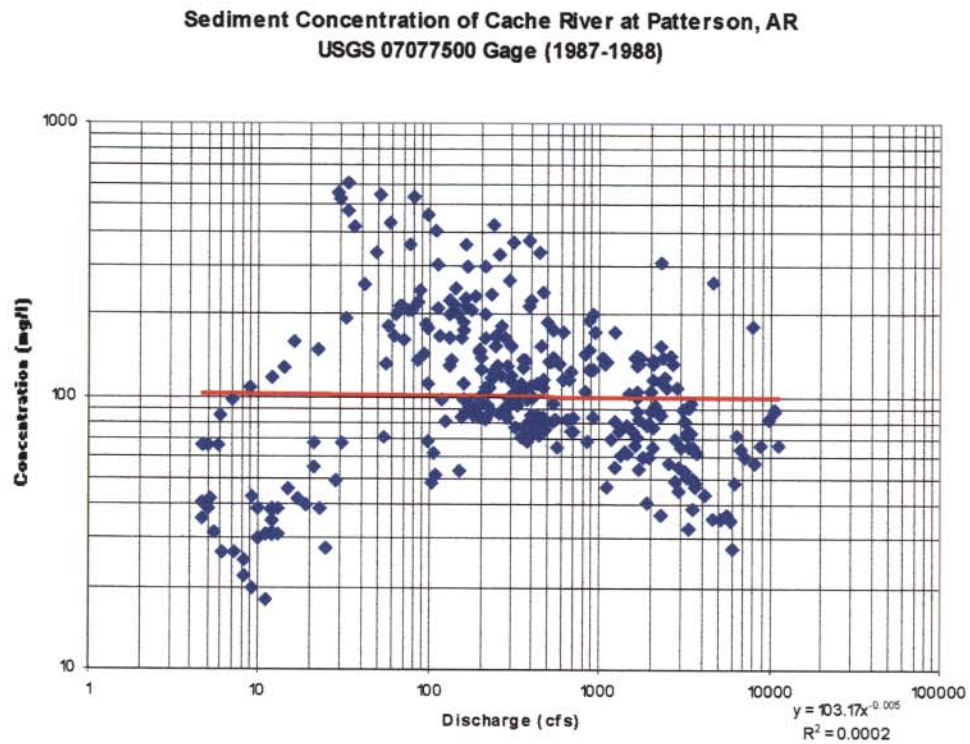

Figure 3. Measured Sediment Concentrations at USGS Gage 07077500

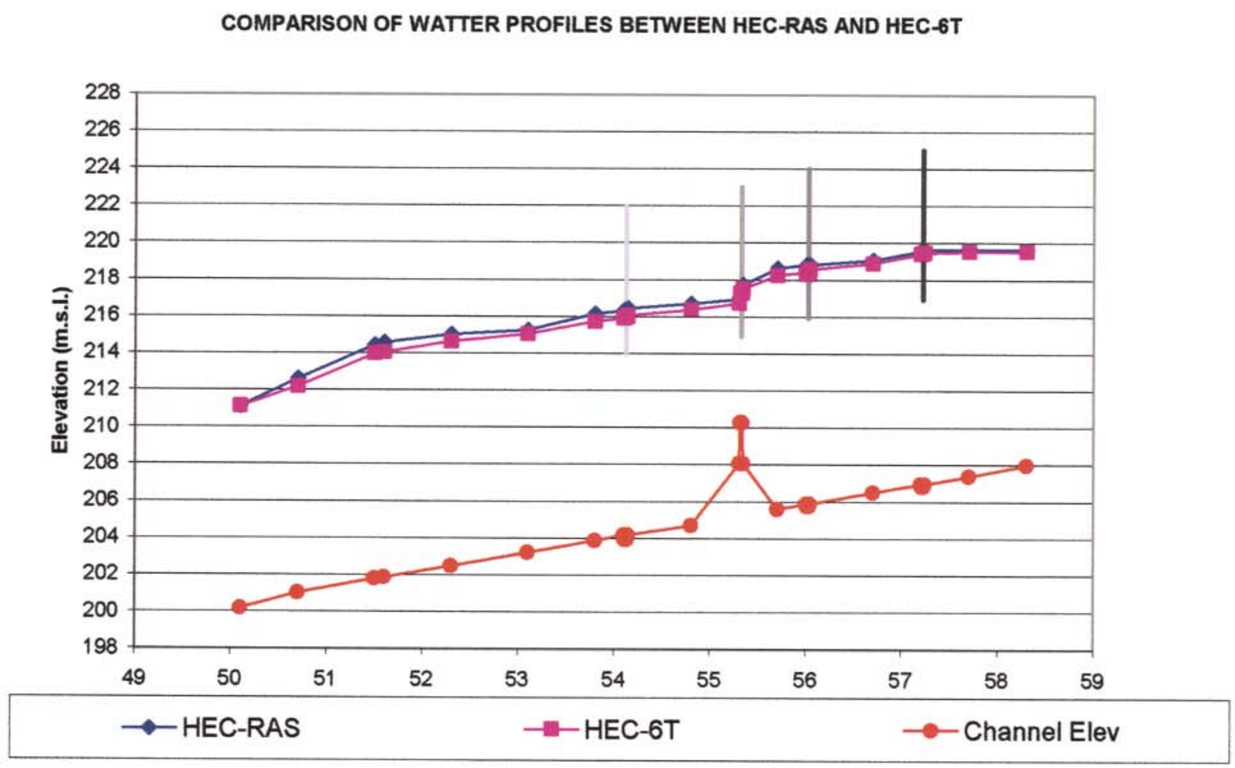

Figure 4 Comparisons of Water Profiles between HEC-RAS and HEC-6T

PLATE II-C-04

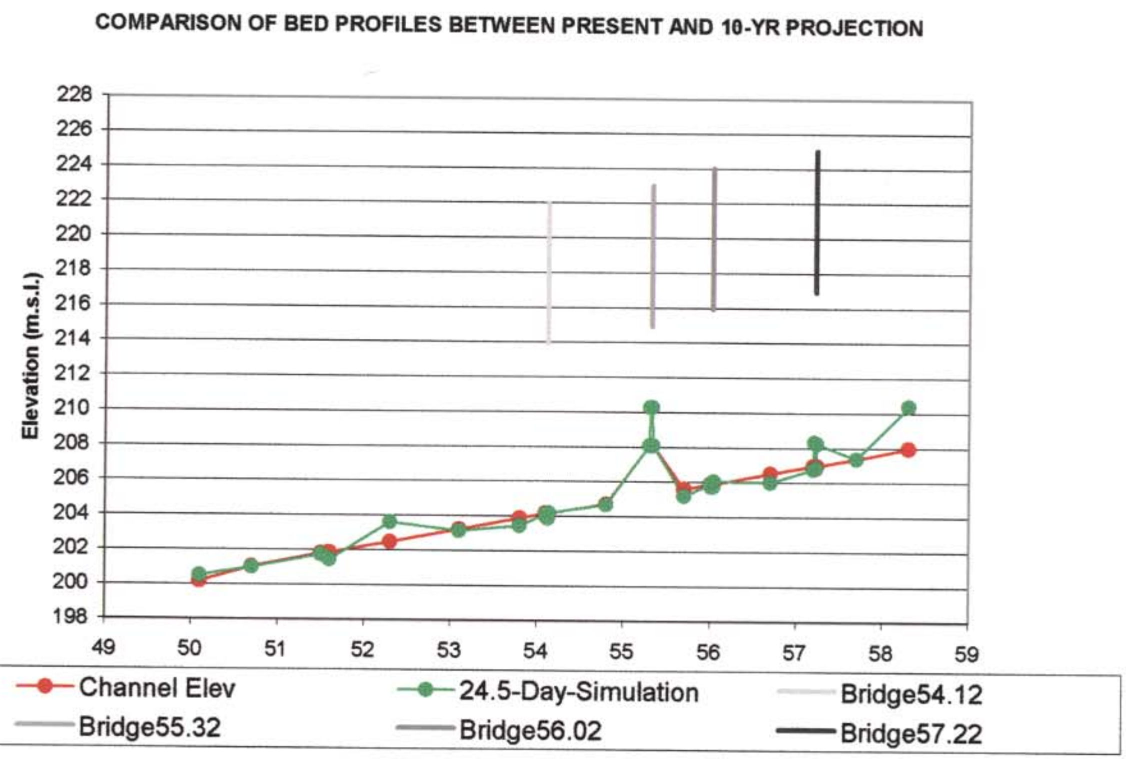

Figure 5 245-Day Simulation of HEC-6T Model in Indian Bayou Ditch

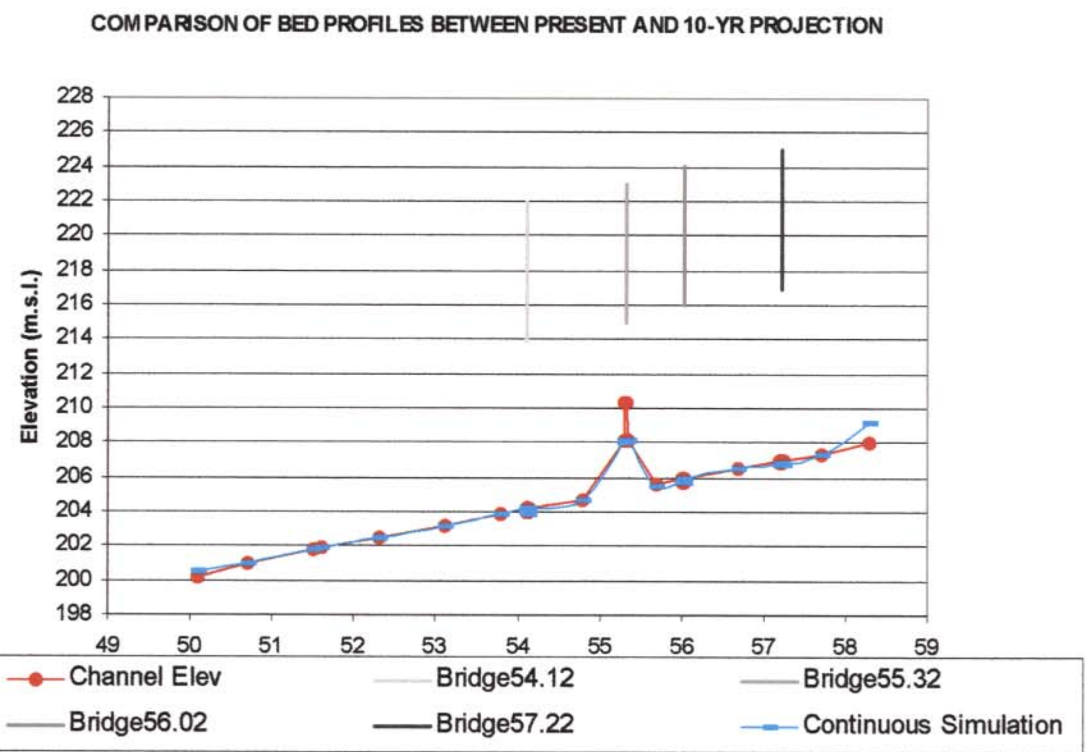

Figure 6 Daily Continuous Simulation of HEC-6T Model in Indian Bayou Ditch

PART III

OPERATION MANUAL

OPERATION MANUAL

PART IV BAYOU METO IRRIGATION PROJECT OPERATION PROCEDURES

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CELMM-ED-HD 20 September 2002

Bayou Meto Irrigation Project

A Rough Draft of an Operation Manual for the Bayou Meto Irrigation Project (BMIP) is provided for input from the Bayou Meto Irrigation District, (BMID), the NRCS, and others as necessary.

Several sections are incomplete and/or under development. These sections will be inserted/completed as the information is developed. Information from the BMID, the NRCS, and others will be necessary to complete this manual. The goal is to develop a guide to what structural and organizational requirements will be necessary for the BMID to implement and operate the project. Further development of this manual will continue as detailed plans and specifications are begun and completed.

PRELIMINARY DRAFT OF THE BAYOU METO IRRIGATION PROJECT OPERATION MANUAL

PART I INTRODUCTION

1. Summary

Institutional Framework

The Bayou Meto Irrigation District is a public non-profit corporation organized under Arkansas law. The co-operative plans to provide local sponsorship for the construction, operation, and maintenance of the Bayou Meto Irrigation Project (BMIP). The BMIP consists of canals, pumping stations, check structures, and turnout structures designed to divert water from the Arkansas River at David D. Terry Lock and Dam, Pool No. 6, for the purpose of agricultural water supply. By the Act under which the District is incorporated, sale of water constitutes its only source of revenue. Water will be sold to landowners through water-use contracts. These executed contracts to buy water will be the basis for the District's assurances for payment of construction, operation, and maintenance of the project.

For the BMIP diversion channels to be built, receive diversion waters, and legally carry those waters for irrigation purposes, certain permits must be granted and certified by select state and federal agencies. The Corps of Engineers has federal oversight through the issuance of Section 404 and Section 10 permits. The Arkansas Soil and Water Conservation Commission (ASWCC) have legislative empowerment to issue non-riparian water-use permits. Other possible applicable state review or permit requirements could include Water Plan Compliance Review, determination of minimum instream flows, and dam safety permit issuances for on-farm reservoirs.

Bayou Meto Irrigation Water Supplies

Sources of water in the BMIP area consist of an alluvial aquifer, several confined aquifers, abundant rainfall, and a major river system. Additional water is available to downstream users when irrigation water is released from fields. Historically, the majority of irrigation water was taken from the alluvial aquifer. Heavy use of this ground water resource has resulted in a decline of up to **100** feet within the BMIP. Declining aquifer levels has spurred farmers to construct numerous storage reservoirs and lakes to augment ground water supplies. An average annual rainfall of 48 inches, although unevenly distributed through the year, provides ample opportunities to capture large quantities of runoff. Currently, 14 percent of the average annual crop demands are provided from stored water. Increased storage is projected for the future. Where the alluvial aquifer no longer provides sufficient quantities for pumping, deep wells, penetrating the underlying confined aquifer, have been constructed. Municipal and Industrial water supplies typically come from the confined aquifers.

The Need and Demand for Water

The need for agricultural water stems from the fact that rainfall, although abundant, is unevenly distributed throughout the year--greater rainfall depths generally occur in the winter to early-spring and lesser rainfall depths occur during the cropping season. Demands for water increased historically as additional acreage was placed under irrigation. Future demands for irrigation water are limited by the type of crops raised. Essentially all of irrigatable acres are currently irrigated; therefore, additional increases in demand are unlikely. Municipal and industrial (M&I) water demands continue to grow as population increases. M&I needs are significantly less than agricultural needs. Confined aquifers in the region provide most M&I users needs. As more of the confined aquifer is developed for agricultural use, competition for this source will become acute.

Will There Be Enough Water?

2. **Institutional Framework for Water Resource Management**

Allocation and Management of Arkansas' Water Supplies

Allocation and management of Arkansas' water resources falls under the auspices of the Arkansas Soil and Water Conservation Commission. Current regulatory and legislative statutes are described as follows.

<u>Arkansas Water Law (Common Law Roots).</u> The Arkansas General Assembly, by statute, adopted the common law of England, including the English Doctrine of Riparian Rights for surface water, as law governing water usage in the State. The Arkansas Supreme Court indicated a preference for the reasonable use theory of Riparian Rights. Under the reasonable use framework, the court stated: (i) the right to use water for strictly domestic purposes is superior to many other uses of water; (ii) other than the use mentioned above, all other uses of water are equal. Some of the lawful uses of water recognized by the State are fishing, swimming, recreation, and irrigation; (iii) when one lawful use of water is destroyed by another lawful use, the latter must yield, or it may be enjoined; (iv) when one lawful use interferes with or detracts from another lawful use, then a question arises as to whether, under all the facts and circumstances of that particular case, the interfering use shall be declared unreasonable and as such enjoined, or whether a reasonable and equitable adjustments should be made, having due regard to the reasonable rights of each.

Concerning ground water usage, the reasonable use rule provides that if two or more persons are pumping from a common aquifer, "each has a common and correlative right to use of this water on his land, to the full extent of his needs if the common supply is sufficient, and to the extent of a reasonable share thereof, if the supply is so slant that the use by one will affect the supply of the others."

<u>Registration of Withdrawals ACA 15-22-302</u>. Reporting of withdrawals to the Arkansas Soil and Water Conservation Commission is required for all persons who withdraw underground water except for individual household wells and except from wells having a maximum flow rate of less than fifty thousand gallons per day. ACA 15-22-215 requires the registration of diversion of surface waters.

Right To Take Impounded Water ACA 15-22-216. Any person constructing a dam shall have the exclusive right to take water from the reservoir so long as the dam is maintained and operated under permit from the Arkansas Soil and Water Conservation Commission.

Allocation During Shortages ACA 15-22-217.

a. Whenever a shortage of water exists to the extent that there is not sufficient water to meet the requirements of all water needs, the Arkansas Soil and Water Conservation Commission may allocate the available water among the uses of water affected by the shortage of water in a manner that each of them may obtain an equitable portion of the available water.

b. In allocating water, the Commission may consider the use that each person involved is to make of the water allocated.

c. In making such allocations of water, reasonable preference should be given to different uses in the following order of preference:

1) Sustaining life;
2) maintaining health; and
3) increasing wealth

d. Water needs shall include domestic and municipal water supply needs, agriculture and industrial water needs, navigation, recreational, fish and wildlife, and other ecological needs.

e. The following priorities shall be reserved prior to allocation:

1) Domestic and municipal domestic
2) Minimum streamflow
3) Federal water rights

Permit Required for Dam Construction ACA 15-22-210. No person shall have the right to construct or own a dam to impound water until he obtains a permit from the Arkansas Soil and Water Conservation Commission. ACA 15-22-214 exempts dams meeting the following conditions:

a. Impounds less than fifty (50) acre-feet of water or is of a height less than twenty-five (25') feet.

b. The height of which is at or below the ordinary high water mark on the stream.

Transfer of Excess Surface Water To Non-Riparian ACA 15-22-304. The Arkansas Soil and Water Conservation Commission may authorize the transportation of excess surface water to non-riparians for their use. "Excess surface water" means twenty-five percent (25%) of that amount of water available on an average annual basis from any watershed amount, as determined by the Commission, required to satisfy all of the following:

a. Existing riparian rights as of June 28, 1985;

b. The water needs of federal projects existing on June 28, 1985;

c. The firm yield of all reservoirs in existence on June 28, 1985;

d. Maintenance of instream flows for fish and wildlife, water quality, and aquifer recharge requirements; and

e. Future water needs of the basin of origin as projected in the state water plan developed pursuant to ACA 15-20-207 and 15-22-501 et seq.

Delegation Of Allocation Authority ACA 15-22-221. The Arkansas Soil and Water Conservation Commission may delegate the power to allocate water during times of shortage to conservation districts and regional water districts.

Establishment of Minimum Stream Flows ACA 15-22-222. The Arkansas Soil and Water Conservation Commission shall establish and enforce minimum stream flows for the protection of instream water needs.

Surface Water Management--Regulatory and Permit Requirements

Various State and Federal permits must be obtained prior to construction and operation of the BMIP. Specific requirements are outlined as follows.

Section 404. The authority to regulate discharges of dredged (excavated) or fill material in waters of the United States (including wetlands) was given to the Corps of Engineers with the passage of the Federal Water Pollution Control Act Amendments. This act later was changed to the Clean Water Act in 1977. The regulation of materials into or from rivers, streams, lakes, and wetlands is intended to "restore and maintain the integrity of the Nation's waters".

Section 10. Since 1899, the Corps has had authority to regulate any work activity performed over our Nation's navigable waters. Structures, intakes, and any other impact are included under this jurisdictional mandate.

Non-Riparian Permit. Upon completion of an application by a non-riparian, the ASWCC can authorize the transportation of "excess surface waters" to non-riparians under the provisions of Statute 15-22-304. Procedures for application as well as delineation of "excess surface waters" are contained in this statute.

Water Plan Compliance Review. Under Title VI in the ASWCC's "Rules For Water Development Project Compliance With The Arkansas Water Plan", all water development "projects", excluding sewage disposal, industrial waste, or other waste treatment systems, shall be subject to review and approval by ASWCC. A written application in accordance with Subtitle II must accompany the filing correspondence.

Dam Safety Permit. If a dam or levee is 25 feet or more in height and impounds 50 acre-feet or more, issuance of a dam permit is required by the ASWCC. The permit should be obtained before actual construction begins on the dam.

PART II WATER SOURCES

3. Surface Water Supply

Rainfall

Rainfall occurring over the BMIP is abundant with an annual average rainfall of 48 inches. Although abundant, rainfall is unevenly distributed throughout the year. This results in excess water during the winter and early spring months and a deficit during the summer and early fall months. The deficit through the summer and early fall months significantly affects crop yields. Surface reservoirs of have been constructed to capture a portion of the excess runoff, especially during late winter and early spring months. Tailwater recovery systems have been installed to further enhance capture of runoff and to re-capture water released from irrigated fields. Surface diversions from natural streams have increased dramatically in recent years.

Evaporation

Evaporation from the Grand Prairie Area is extensive, especially in the summer months. Although mean annual rainfall is 48 inches, total evaporation often produces a net loss in available water.

Import

Importation of water from external sources (water diverted from watersheds outside the BMIP) currently constitutes only a very small fraction of water used for irrigation in the White River Regional Irrigation Water Distribution District. The BMIP will divert up to 1800 cfs from the White River from a point upstream of DeValls Bluff, AR.

4. Ground Water Supplies

Ground Water Defined

Ground Water is subsurface water occurring in a zone of saturation. In that zone, water fills the pore spaces in sediment. The sediments are randomly interspersed mixtures of fine to coarse-grained material and constitute the aquifers within a zone of saturation. Several distinct aquifers exist in the BMIP area, the alluvial aquifer and various confined aquifers. The alluvial aquifer is a relatively shallow unconfined aquifer most commonly used for irrigation. The confined aquifers are deeper and are used primarily for municipal and industrial purposes.

Historical decline of the alluvial aquifer has resulted from pumpage in excess of recharge capacity. This has led to use of the confined aquifers for irrigation. Ground water currently provides approximately 86 percent of irrigation water. Water quality within the alluvial aquifer is generally good. However, in some areas within the BMIP, ground water quality has declined due to increased salinity concentrations caused by overdraft. Water quality within the confined aquifers is generally much superior to that of the alluvial aquifer. The ASWCC requires wells to be registered and permitted.

Safe Sustained Ground Water Yield

Short-term and long-term ground water yields were projected from finite difference groundwater models developed by the USGS and the University of Arkansas. Simulations were conducted utilizing present and projected water needs. An estimate of the minimum saturated thickness of the alluvial aquifer was made during development of the ground water model. A minimum saturated thickness of 20 feet was estimated as the minimum to sustain the aquifer characteristics. Based on a 20 foot saturated thickness as a minimum, long-term sustained yield from the alluvial aquifer was estimated to be approximately 40,000 acre-feet per year.

5. Conservation Measures

Tailwater Recovery Systems

Installation of tailwater recovery systems greatly reduces the amount of water released from fields directly as stream runoff. Tailwater recovery systems can typically increase irrigation efficiencies by 10 percent or more. Tailwater recovery systems also increase the volume of rainfall captured and stored in reservoirs thereby reducing the quantity of imported water.

Irrigation Methods

Different irrigation techniques can greatly affect irrigation efficiencies. Various irrigation methods commonly used in other parts of the US are center pivot, furrow irrigation, surge irrigation (a variation of furrow irrigation), drip irrigation, and flood irrigation. Flood irrigation generally is the least efficient method of irrigating row crops such as corn, milo, soybeans, etc. Where topography permits, furrow irrigation is a more efficient method of irrigating row crops. Surge irrigation simply increases the efficiency of furrow irrigation by reducing the travel time down the furrow and by reducing water lost at the end of the row. Sprinkler, center pivot, and drip irrigation typically reduce water demands further, however, at a much-increased cost due to higher head requirements. With each increase in efficiency, system installation costs increase. Selection of appropriate irrigation methods must balance cost effectiveness.

PART III WATER USES

6. Water Demands

Average Year

The average year, for determining demands, was determined by averaging total rainfall depths for the months of July and August for the period 1965 to 1981. Ten-day crop demands were computed utilizing ten-day average temperature, evapo-transpiration, and rainfall data at the NWS gaging station at Stuttgart, AR. Crop patterns were assumed to be consistent with current rotation practices.

Drought Year

The average drought demand was estimated from climate data for 1981. Ten-day crop demands were computed for current crop rotation patterns.

7. Operations for Enhancing Water Supply Reliability

Farmer Owned Storage Reservoirs

Water available in the area is abundant from a variety of sources. Capture of surface runoff is accomplished by diversions into elevated reservoirs and lakes. Historical trends show that surface storage has increased as aquifer levels drop (and pumping costs escalate). Increased storage also increases supply reliability. As an integral component of the BMIP, surface storage will utilized to reduce the peak diversion rate thereby increasing the reliability of the system to deliver full demands. The reliability of the system, from the perspective of an individual water contractor, depends on the amount of time that water is available for use. Limits imposed by available flows from the White River require a minimum of twenty-five percent of the total average year demand to be stored. At the twenty-five percent storage level White River flows are projected to provide full diversions percent of the time; Lesser diversions will be available for a larger percent of the time. Increasing storage beyond the twenty-five percent level will yield even greater reliability. Increased storage will serve to augment less than full diversions in the late-summer time period.

8. Balancing Water Supply and Demands

Average Year Supply: The average annual supply of a water development system over a long period. For the BMIP average year supply is the average annual delivery capability of the Arkansas River over a 57-year study period (1940-1996).

Drought Supply: The average annual supply of a water development system during a defined drought period. For the BMIP the drought period is 1980 and 1981. For dedicated natural flow, it is environmental flows as required under specific agreements or state mandated minimum in-stream flows.

PART IV BAYOU METO IRRIGATION PROJECT OPERATION PROCEDURES

9. General Organization

District Headquarters

The BMIP will establish a central headquarters facility where operations and maintenance activities will be coordinated and administered. Primary operations activities will consist of the following: all operations of the irrigation project River pumping stations, full operation of all check structures located on the main canal (major check structures) and lateral canals (minor check structures), complete monitoring of water levels and gate openings at all check structures, turnout structures, and weir structures throughout the system. All Water Demands from Sub-districts will be consolidated and Diversion rates from the Arkansas River and check gate openings modified accordingly to provide the required flow, if available. Primary maintenance activities will consist of the following: budgeting of necessary resources, finances and manpower; conducting ordinary maintenance of canal embankments and access roadways; maintaining electronic devices (computers, gate controllers, and water level monitors) for the entire delivery system; scheduling major maintenance activities at pump stations, check structures, turnout structures, weir structures, and drop inlet structures; and conducting annual inspections of canals and ditches and periodic inspections of canals, ditches, and pumping station structures.

Sub-District Structure

The BMIP will be further subdivided into sub-districts to facilitate requests for water. Each sub-district will consist lateral canals and/or pump type turnouts including farmer turnouts and will provide water for approximately **400** water contracts or customers. Each sub-district(s) will have ditch riders who will be responsible for coordinating water demands with the headquarters office and for monitoring operation of farmer-operated field turnouts. The ditch rider will also make flow-meter readings and report the findings to the headquarters. The ditch rider will also provide continuous visual observation of canal, ditch, and structure status relaying maintenance requirements to the headquarters office.

10. System Controls and Water Level Monitoring

Computerized System Controls

Computers will be utilized for monitoring pump station activity and all canal, ditch, and regulation reservoir water levels. Upstream and/or downstream water levels will be monitored at check, turnout, weir structures. Communications will utilize either buried cable or radio transmitter/receiver(s). Data will be collected at regular intervals as part of the control system operation. All water level readings will be stored in a permanent database to be used in establishing delivery efficiencies, for billing verification, and for other future archival purposes. Current status at each structure (gate, pump, or weir) will be actively displayed and updated on the operators screen or system board to facilitate operations and system performance. Displays will consist of tabular (tables) and graphics (bar charts, Icons, etc) data to reflect the functional status of gates and pumps (on, off, open, closed, error) and the water levels upstream and/or downstream of canal and ditch structures. Actual pumping station delivery rates and check structure and major turnout gate settings will be controlled remotely through computer controls. Complete automatic control by computers, both flow and water levels controlled by software logic, is anticipated. Gate opening and closing sequences will consist of the desired gate opening for each gate at a structure determined by the control software SOBEK. This program will insure that required flow off takes occur while maintaining delivery system stability. This is done by reducing flow miss matches and letting gated structures operate using high level decoupling techniques to prevent oscillations in pool set points. Audible alarms should activate within the control room to indicate gate failure, pump failure, extreme water levels, monitoring system error, or critical situations of other system components. The alarms should indicate the individual structure and the cause of the alarm(s). The alarms are essential to real-time monitoring of the delivery system.

Manual System Controls

Each structure will have the capability for manual operations. Farmer-operated field turnouts will be operated manually. All manual operations will be coordinated with the sub-district or headquarters office prior to implementation.

11. Pump Stations Operations

Pump stations in the project area will be operated continuously to provide the demands through the system. Four pump stations will be operating to move water to meet system demand. Pump stations consist of various pumps. Because of fluctuating Arkansas River stages, constant monitoring of water levels at the pump intakes and reservoirs is essential to maintain required deliveries. A flow-measuring device will be required at or near the pumping station to adequately assess the quantity of water pumped in a 24-hour period. Check structures at outlets to canals will be necessary to adequately adjust pump operations for daily operations in the delivery system.

Pumped Diversions

Operation of the small pumps will be based on the actual demand. Since pumps are electric, the pumps will be operated based on flow passing weir structures to provide equitable distribution of the water throughout the system.

12. Check Structure Operations/Major Turnout Structures

Maximum Allowable Water Level Changes

Maximum allowable water level changes in canals while filling pools will be 0.25 foot per hour until sufficient tailwater is established to completely submerge the hydraulic jump below the gates; after the jump becomes submerged, the rate can be increased to 0.5 foot per hour. Gates will not be operated in a manner that creates excessive velocities in the downstream canal.

Maximum water level changes in the Main Canal will less than 1 foot per day.

Maximum water level changes in the Lateral Canals will be 1 foot per day if greater than 5 feet in normal water depth and 0.5 foot per day if less than or equal to 5 feet in normal water depth.

Gate Settings

Gate Settings will be made to evenly distribute flow through the structure. Individual gates will be operated at a tolerance needed for control of the system. Special exceptions are permitted for emergency conditions when control is not an issure. Gates will not be operated in a manner that creates excessive velocities in the downstream canal at any time.

13. Turnout Structure Operations

The responsibility for turnout structure operations varies. For major turnouts, those diverting water to lateral canals, the BMIP headquarters office has jurisdiction. Setting changes at the major turnout structures will be a part of the automated control of the system and coordinated with the Watermaster on requested deliveries. The headquarters office via computer controls will initiate gate changes with input from the appropriate ditch rider. Pump-type turnouts from man-made canals and existing ditches used in the delivery system will be under automated control and adjusted to insure that water is distributed fair and equitably throughout the system. Individual farmer-operated off-takes must be coordinated prior to gate changes. The individual will make contact with the sub-district (ditch rider) who then contacts the headquarters office to adjust the system for the necessary water delivery. The farmer will then make gate adjustments at his off-take. Maximum water level changes and gate operations will be in accordance with requirements for system control.

Timing of Deliveries

Timing of deliveries will vary slightly from one end of the system to the other. Deliveries made along the canals will generally take less lead-time than those made along natural streams.

14. Water Delivery Scheduling

Water Call Procedures

Delivery scheduling will require an advance notice or "Water Call" from the customer to the Ditch Rider in each sub-district.

For canal deliveries, a **1 to 2** day advance Water Call will be required.

For Stream deliveries, varying advance notice will be required. See Table **14-1** for Water Call notice.

Delivery Projections

Actual delivery projections may be made from previous day(s) deliveries, Water Calls, and amount of previous day(s) rainfall totals ratioed to the average year rainfall totals (average year is defined as the average year rainfall data used for developing the average year demand curve). In general, water will be delivered at the rate projected by the ten-day average year demand curves for each canal segment. Water delivery may be reduced below that projected by the average year demand depending on flow availability from the White River. Water delivery may be increased during the early spring up to the maximum projected by the average year demand if flow is available from the White River.

Deliveries will generally follow the schedule shown in Table **14-2**. Specific deliveries will generally be programmed using the average year ten-day demand values adjusted for actual rainfall amounts. Deliveries will also be based on Water Calls. However, for optimum performance of the BMIP, the system cannot function strictly as an on-demand system. Water must be delivered when most readily available; and then the water may in turn be applied directly to crops, if needed, or placed into storage at the discretion of the customer. If a customer (or customers) does not need water for crops and does not have storage available, the water being delivered through the BMIP will be reduced accordingly. The customer must realize that water foregone early during the year may result in their experiencing a water shortage later during the irrigation season. If storage is utilized early in the irrigation season for deliveries (while canals are operating at less than full capacity and additional Arkansas River flow is available for diversion), shortages in surface water imports will likely result, particularly in late July through September.

Table 14-1 Advance Notice Requirements

Canal or Stream Name	Days Notice Required
Main Canal System (includes Canal 1400 and 2500)	1
Upper Indian Bayou Ditch and Canal 1530	1
Lower Indian Bayou Ditch	2
Canal 2100 and Upper Caney Creek Ditch	1
Lower Caney Creek Ditch and Canal 2140	2
Crooked Creek Ditch and Canals 2260 and 2280	2
Big Ditch	2
Canal 2520 and Fishtrap Slough	1
Skinner Branch	1
Canal 2531 and Blue Point Ditch	2
Shumaker Branch	1
White Oak Branch	1
Rickey Branch	1
Canals 4111 and 4112	2

Table 14-2 General Operation Scheduling

Step	Management Concept	Month(s) of Operation	General Procedure
1	Fill Storage Prior to irrigation season if it has not already been filled.	Mar - Apr	Diversion pumps operate as needed. Water is released to customers as needed.
2	Irrigation season begins. Use canal water first. Use stored water only if canal capacity does not meet crop demands.	May - Jun	Water released to customer upon demand for use directly on crops or to fill storage.
3	Middle of irrigation season. Continue using canal water first then stored water. Water use is at maximum. If canal capacity and stored water do not meed full demand, ground water may be utilized.	Jul - Aug	Diversion pumps are operated continuously to meet demand. Volume pumped may be determined by the demand or by other operation tools such as a "Water Budget".
4	Late irrigation season. Rice irrigation has ended. Soybeans and other row crops continue to be irrigated.	Sep-mid Oct	Continue as in Step 2. The diversion system will be at full capacity during this period. Actual diversions may be dictated by Arkansas River Conditions. Continue as in Step 3.
5	End of irrigation Season.	Mid Oct- Nov	Operate diversion pumps according to needs for special purposes such as wildlife or other environmental needs.
6	General maintenance and inspection and major maintenance conducted.	Dec	No system operations.

15. Water Shortage Procedures

Arkansas River Availability

Full deliveries will be provided to the customers, up to the maximum projected by the average year demand, provided flows are available from the Arkansas River. When the Arkansas River discharge at David D. Terry Lock and Dam falls to 4645 cfs (based on information from Little Rock District Water Control), no flow is available for diversion. For Arkansas River discharges greater than 4645 cfs, flow may be diverted. The peak demand projection of 1,750 cfs would, therefore, require 6395 cfs for full diversions to occur. During critical periods, the actual delivery will be adjusted. Deliveries will be reduced based on the actual computed percent reduction uniformly for all customers.

16. Reporting Requirements

Internal Reports

Daily reports will be maintained of gate operations and flow rates through check, turnout, and weir structures.

Monthly Reports to ASWCC

Monthly reports of the daily diversion rates, Arkansas River stages and discharges (based on information from the Little Rock District), and the total volume of water diverted will be provided to the ASWCC.

Monthly Billing Reports to Customers

Monthly reports of water usage will be provided to the customer. Billing procedures and payment will depend on the contract established between the customer and the Bayou Meto Irrigation District.

17. Expectations for Farmer Owned Reservoirs

Use of Existing and Planned Storage

Storage available from existing reservoirs will continue to be filled from natural runoff and tailwater recovery systems as much as practical. New storage constructed as a project feature will also be filled from natural runoff and tailwater recovery as much as practical. The combination of existing and new storage should approximate nineteen percent of the average annual demand volume. Should rainfall be insufficient to fill all on-farm storage, imported water may be used to complete filling reservoirs. To maximize water availability and system reliability during the summer months, storage should not be used until crop demands exceed available diversions. Should customers use stored water while diversions can supply the full crop needs, those customers will experience less system dependability (from their point of reference) than those customers that utilize storage as late as possible. To yield the highest system reliability crop needs must be taken first from the canals then reservoirs. However, if storage is depleted at any time during the year, the customer may, at their discretion, use purchased water to replenish the storage if available.

18. Emergency Procedures

Drowning Incidents and Reports

Incidents involving the public will be documented by a permanent electronic report filed immediately after the occurrence. The electronic report will include the time of the report, time logged by the computer system, and will be structured such that no editing may be done to the report. A written report will also be completed and filed.

Canal Failure

Canal failure will be prevented by regular maintenance and inspection. Should canal failure occur, several procedures must be followed.

a. If catastrophic failure of an embankment occurs, emergency shutdown of the pumping station is implemented immediately and gates just upstream and downstream of the failure are closed completely. Water deliveries may continue through the system upstream of the failure. Repairs are made before placing the canal back in service.

b. If severe failure of an embankment occurs such that immediate failure is not expected, emergency shutdown of the pumping station should be implemented immediately and gates just upstream and downstream of the failure are closed completely. Water deliveries may continue through the system upstream of the failure. Repairs are made before placing the canal back in service.

c. If minor failure of an embankment occurs such that no additional failures will result, the system continues to operate at full delivery. Repairs are delayed until the non-irrigation season.

Gate Failure

Gate failure will be prevented by regular maintenance and inspection. Should gate failure occur, several procedures must be followed.

a. If complete failure of the gate or operating mechanism occurs, limited operation of the canal can continue by use of the remaining gates in a manual mode. Gate repair should be accomplished by use of stoplogs to evacuate water from the gate bay. Failure of single gated structures will require removing the downstream canal from service until gate repairs can be completed.

b. Failure of automated control mechanisms require the gates to be operated manually until repairs can be implemented.

Weather Forecasts

Problems induced by rainfall will generally consist of rising stages in natural streams, particularly severe rainfall events. Accurate advance weather information--extending over days, weeks, or months--would be invaluable in adjusting water operations in all types of years whether wet, dry, or normal. Potential benefits of dependable weather forecasts could be a reduction in water diversions and, to some degree, flooding impacts. Long-term weather forecasts could result in considerable savings in operation costs and in greater system reliability. The National Weather Service routinely issues both short-term and long-term weather forecasts. However, current predictions are not sufficiently reliable for project operation. Operations must, therefore, react to actual weather conditions rather than anticipate conditions based on predictions. Best-management practice will involve close monitoring of water levels at all times.

19. Policing

Water Usage

Unauthorized water use must be minimized to allow equitable distribution and billing of imported water. Primary policing will be the responsibility of Ditch Riders. Users found making unauthorized withdrawals will be subject to penalties that increase with number of occurrences and the volume and rate of water illegally diverted. Individual customers will also be responsible for reporting suspected illegal diversions.

Trespass/Vandalism

Arrangements with local police and sheriff departments will be made for patrolling headquarters facilities, Sub-district offices and maintenance facilities, pumping stations, and other structures. Fencing will be constructed in appropriate locations around building compounds.

PART IV

GLOSSARY

PART IV - GLOSSARY

Parts I, II, and III of this report makes use of special irrigation and water-supply industry terminology, and also uses terms and abbreviations specific to the Bayou Meto Irrigation Project. Selected terms and abbreviations are defined below.

aquifer
: A water-bearing geologic formation able to yield water in usable quantities.

ASCS
: See FSA.

ASWP
: Arkansas State Water Plan.

automatic control
: A procedure or method used to regulate mechanical or electrical equipment without human observation, effort, or decision.

automation
: A procedure or method used to regulate a water system by mechanical or electronic equipment that takes the place of human observation, effort and decision; the condition of being automatically controlled.

balanced operation
: Operation of a canal system where the water supply exactly matches the total flow demanded.

canal
: An man-made open channel delivering water to subsidiary components of the delivery system.

canal check gate structure
: A structure designed to control the water surface level and flow in a canal, maintaining a specified water depth or head on outlets or turnout structures. Most canal check structures have moveable gates.

canal freeboard
: The amount of canal lining available above maximum design water depth.

canal lining
: A layer of material intended to enhance the performance of the canal by reducing seepage, resisting erosion, etc. A lining is often made of concrete, but in some instances may be a layer of specially compacted earth.

canal pool
: That portion of a canal between check structures.

canal prism
: The cross sectional shape of a canal.

canal reach
: Segment of main canal system consisting of a series of canal pools between major flow control structures.

canal system control concepts
: The fundamental strategy of canal flow control--either upstream or downstream control.

canal system control methods
: The way in which the selected control concept is implemented--local manual, local automatic, or supervisory control.

canal system operation
: Water transfer from its source to points of diversion for irrigation, municipal and industrial, fish and wildlife, and drainage purposes.

canal system operation concepts
: Downstream operation or upstream operation.

canal system operation methods
: Constant downstream depth, constant upstream depth, constant volume, controlled volume.

CE
: U. S. Army Corps of Engineers.

centralized control
: Control of a canal project from a central location generally by a master station, communications network, and one or more remote terminal units (RTU's).

centralized headquarters
: Control of a canal project from a central location by the watermaster.

check gate
: A gate located at a check structure used to control flow.

classification of a water conveyance system
: A general classification based on the objective of the system. Water conveyance systems may be classified as delivery systems, collector systems, or connector systems.

constant volume operation method
A canal operation that maintains a relatively constant water volume in each canal pool.

control
To exercise restraining or directing influence over: a mechanism used to regulate or govern operation of a system.

control element
A part of a control system through which the system's process is regulated.

control system
An arrangement of electronic, electrical, and mechanical components that commands or directs the regulation of a canal system.

controlled volume operation method
An operation in which the volume of water within a canal reach between two check structures is controlled in a prescribed manner for time variable inflows and outflows such as off-peak pumping or canal side deliveries.

conventional method
The control of a canal system onsite by operations personnel (ditchrider and watermaster). Labor saving devices and machinery may be used to assist in the control of the canal facilities.

crop distribution
For the project area, the percentage of cropland area devoted to the production of each crop species.

CRP
Conservation Reserve Program; A USDA program of contracts with landowners to keep designated tracts of land out of crop production and in good vegetative cover for a period of a few years.

delivery system
Conveys water from a single source such as a storage reservoir to a number of individual points of use. The delivery system is a common water conveyance system classification. It is associated with irrigation, municipal and industrial, and fish and wildlife canal systems.

demand delivery
Unrestricted use of the available water supply with limitations imposed only by maximum design flow rate and total water allotment, i.e. the availability of water at the source is not the limiting factor.

distribution system
Delivers water from the main canalside turnout to individual water users or to other smaller distribution systems.

ditchrider

A member of the canal system operation personnel. The person responsible for controlling the canal system onsite, based upon the flow schedule established by the watermaster.

downstream control

Canal control structure adjustments are based upon information from downstream. The required information is measured by a sensor located downstream or based upon the downstream water schedule established by the watermaster.

evapotranspiration

The loss of water from topsoil to the atmosphere, due to the combined effects of evaporation from the ground surface and from the transpiration of water by vegetation.

farm base acreage

Under the USDA farm program, and based on the cropping history of a given farm, the reference or "base" acreage of cropland for selected crop species used to determine crop subsidy payments by the FSA.

farm number

A number assigned by FSA that identifies a given farm in a given county for the administration of the USDA farm program. For the project the standard farm number has been adapted to accommodate farms from the four counties involved.

FSA

Farm Service Agency (formerly ASCS--the Agricultural Stabilization and Conservation Service). The USDA agency responsible for the administration of the federal farm program.

fish reservoir

In the project area, ponds or reservoirs designed and operated for commercial production of fish.

gate position sensor

A device, such as an analog or digital sensor, that can measure the mechanical position of a gate and provide a signal representing the position.

hydraulic gradient pivot point

A location along the water surface in a canal reach where the water level remains essentially constant during changes in flow.

imported water

Surface water pumped from the Arkansas River to satisfy water demand and to reduce withdrawals from groundwater.

inlet channel

A short open channel connecting the Arkansas River to the intakes of the main pump station.

inline reservoir

A large pool comprising an enlarged segment of the canal, used to regulate flow for balanced operation.

inverted siphon

A pipe used to convey a canal under drainage channels, depressions, roadways, or other structures. Alternately, the siphon may conduct natural drainageways under a canal. The term "siphon" is slightly misleading, since there is no actual siphon action in this type of structure; the term siphon is adopted because it is commonplace. Inverted siphons are also referred to as sag pipes.

irrigation efficiency

An expression of the amount of delivered water that actually benefits the irrigated crop. It is the ratio of the amount of irrigation water that actually benefits the crop to the gross amount of water at a point of reference in the delivery or distribution system. The efficiency will always be less than 100%, due to losses.

lateral

A branch in a canal or pipeline system that diverges from the main canal or other branches.

local automatic control

Onsite control by control equipment without human intervention.

local manual control

Onsite control by a human operator (ditchrider).

main canal system

Delivers water from a primary source of supply to several points of diversion or canalside turnouts to smaller distribution systems.

manual control

Control of equipment requiring direct intervention of a human operator.

master station

The centralized facility with communications to remote terminal units for the purpose of information retrieval, control of apparatus, system control, and operation optimization.

mismatch

A condition in which water supplied to a given point in a conveyance or distribution system does not equal the demand for water at that point.

natural stream

Existing drainageways, natural or modified, in the project area that will be incorporated into the delivery system.

normal year (climatic)

NRCS

Natural Resources Conservation Service (formerly the SCS--Soil Conservation Service). The USDA agency responsible for providing soil and water conservation technical assistance to landowners.

offline reservoir

Constructed to the side of the main canal usually in a natural drainage channel. In canal system, used to store surplus water runoff during the winter season for use during the irrigation season.

operational spill

A loss or waste of water in an irrigation system occurring during operation and caused by operator error or by insufficient control capability.

Peralta cell

Groundwater levels for the entire Eastern Arkansas Region were modeled by the researcher Peralta. Peralta divided the region into a grid of squares three miles on a side (nine square miles in area) and used a groundwater computer program to produce output for each square. Afterwards the squares have been referred to as "Peralta cells".

pipeline

Closed-conduit components of the delivery system, operating under either gravity flow or pressure flow provided by pump turnouts.

pump station

An installation housing pumps, motors, controls, and appurtenances required to accomplish the lifting of water to an elevation high enough to permit gravity flow to points downstream.

regime

For a stream or canal conveying a given discharge, the attainment of an equilibrium or stable condition with respect to bed slope and cross-section. Canals may be designed with slopes and cross-sectional dimensions anticipated to satisfy regime requirements so the canal does not experience unacceptable changes in dimensions due to operation at design flow.

remote monitoring

Periodic or continuous measuring of quantities at remote sites for transmission and dissemination at another location.

remote terminal unit (RTU)

Supervisory control equipment at the remote site that performs data collection, executes control commands, performs automatic control functions, and communicates with a master station.

response time

The time required for a desired result (such as attaining a specified canal water depth) to occur after a control correction has been initiated to obtain that result.

riparian
: The condition of being located along the bank of a river or stream. An owner of riparian land may have rights to water in the adjacent stream, depending on state law.

riprap
: Stone placed in a layer to provide protection to hydraulic structures from erosion or scour. Typical locations include bridges, culverts, siphons, check structures, and turnouts.

safe yield
: The maximum rate at which groundwater may be removed from an aquifer without causing unacceptable results. For example, if it were desired to maintain the level of groundwater in an aquifer, then the safe yield might be set equal to the expected rate of groundwater replenishment.

sag pipe
: See inverted siphon.

scheduled delivery
: Operation of a water delivery system to meet predetermined needs, generally based upon user water orders.

SCS
: See NRCS.

seepage
: The loss of water from the delivery system due to the downward movement of water through the earth.

segment
: That length of a canal, stream, or pipeline extending from one delivery system discharge point to the next delivery system discharge point. For example, a segment of a main canal would exist between one two laterals off the main canal.

self regulation
: A controlled system requiring virtually no operator intervention (see automatic control).

sensor
: A device for measuring water level, flow, gate position, etc., for input to a local automatic controller or RTU.

setpoint
: A value of water level, flow, etc., that the control system maintains, also called the target.

soil association

A group of two or more soil series present in typical proportions that, together with minor inclusions of other soil series, constitute a characteristic landscape. Typically, a detailed county soil survey may be condensed into a county soil association map, which locates the dominant landscapes present in the county.

steady flow
: If at a specified location along a water conveyance, there is no change in flow rate (e.g. in cubic feet per second) over time, then the flow is steady at that location.

stilling basin
: A downstream-end component of an hydraulic structure, such as a check structure or weir, that allows the destructive energy of the flow to be dissipated harmlessly before release to the channel downstream.

storage reservoir
: The project term for off-line reservoirs.

supervisory control
: The control of a canal system from a centralized location (master station) over a communication system and using remote terminal units (RTU's) at the canal structure sites.

tailwater recovery
: The collection and directing to reservoir storage of water that has reported to the downstream end or borders of irrigated land. The water may be runoff from rainfall or may be excess water applied for irrigation. Typically, the landowner must pump the recovered water uphill to the reservoir.

target
: A value of water level, flow, etc., that a control system maintains, also called the setpoint.

telemeter
: To sense, encode, and transmit data to a distant point.

tract
: An area of farmland identified by the FSA as a being part of a farm. The project associates demand for water with tracts.

turnout
: A structure provided to divert water from a main or primary irrigation canal to a distribution canal or farm delivery point. For example, turnouts are used at the head of canal laterals.

unsteady flow
: If at a specified location along a water conveyance, there is a change in flow rate (e.g. in cubic feet per second) over time, then the flow is unsteady at that location.

upstream control

Control structure adjustments based upon information from upstream. The required information is measured by a sensor located upstream, or based upon the upstream water schedule established by the watermaster.

USGS
: United States Geological Survey.

water budget
: An accounting of water inflows, losses, and productive uses based on a selected time span, such as one year.

waterfowl flooding
: The intentional shallow flooding in Autumn of project land area for the benefit of waterfowl.

water level pivot point
: A location along the water surface in a canal reach where the water level remains essentially constant during changes in flow.

watermaster
: The person responsible for operation of the entire canal project.

wave erosion
: The removal of earth from canal inside slopes due to the lapping of waves in the zone just above and below the canal pool elevation. The waves may be caused by wind or by operation of the canal system. In extreme cases the wave erosion may compromise the integrity of the canal embankment.

wedge storage
: The volume of water contained between two different water surface profiles (flow changes) within a canal pool.

weir
: A structure designed to maintain water levels in the project natural stream segment immediately upstream of the weir. The weir is essentially a small hump in the bed of the chute that pools water upstream. The weir maintains the water depth required by the turnout(s) to divert water at design peak discharge.