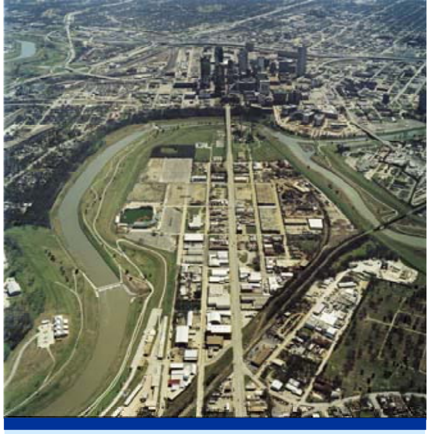

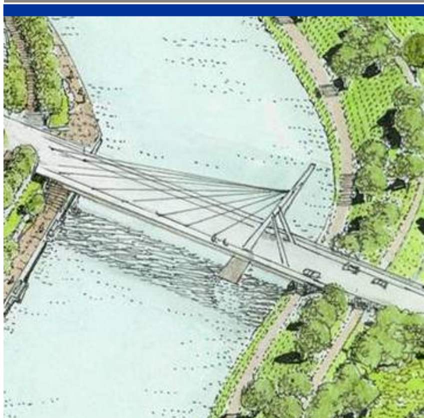

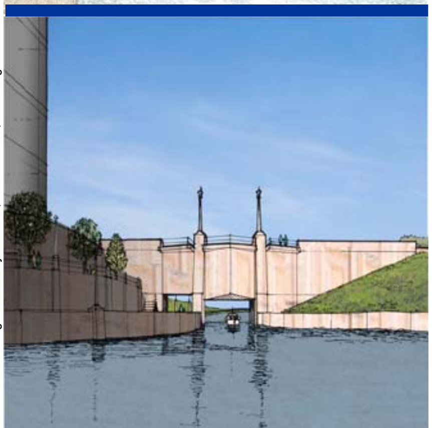

Images courtesy of CDM, Gideon Toal, and Bing Thom Architects

Fort Worth Central City Preliminary Design

Civil/Structural Preliminary Design

Draft Environmental Impact Statement

Appendix C

May 2005

Volume I - Report

Fort Worth Central City Preliminary Design

Civil/Structural Preliminary Design

Draft Environmental Impact Statement

Appendix C

May 2005

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Executive Summary

General

This report presents the preliminary Civil and Structural Design of the major components of the Fort Worth Central City Project. These components have been identified, evaluated, and preliminary designs developed for the bypass channel configuration, excavation and earthwork, relocation of utilities, modification to existing roadways, new vehicular and pedestrian bridges, control dam, isolation gates, flood control components, and storm water pumping.

The proposed bypass channel is approximately 8,400 feet long and approximately 300-feet wide. Water levels in the bypass channel will be controlled by a dam with crest gates to create a controlled, quiescent watercourse. Flood isolation gates will be incorporated into the levee system to protect the interior area. Three vehicular bridges are proposed over the bypass channel at North Main St., Henderson St. and White Settlement Rd., as well as other roadway modifications. Two pedestrian bridges are also proposed

Earthwork

The bypass channel sections consist of four general elements: the base flow channel, soft edge (levee), hard edge (walls), and maintenance/staging area. The soft edge will contain recreational trails, sloped vegetation, and access for maintenance and emergency vehicles. Bermuda grasses will be planted and maintained on the soft edge levee side for aesthetic and slope erosion protection purposes.

The hard edge is located on the eastern side of the bypass channel. This edge section is planned to contain a series of tiered retaining walls, multiple walkways, and landscape areas. Construction of the bypass channel will require adequate space for the various contractors to stage, move equipment, and construct the work.

In order to mitigate a loss of floodplain or valley storage due to the diversion of flood flows through the bypass channel, valley storage mitigation sites are included in the preliminary design. Valley storage mitigation sites will be provided in three areas, along the West Fork of the Trinity River upstream of the project area, in the vicinity of the Samuels Ave. Dam, and in proximity to Riverside Park.

Forty (40) individual sites were identified and subsequently investigated for feasibility and to estimate the potential amount of valley storage that could be created on each site. The amount of valley storage was then compared to the cost of property and the cost of necessary site improvements.

Utility Relocation

Construction of the bypass channel, levee system and valley storage mitigation will require relocation or abandonment of utilities located in the project area. The majority of utilities currently located in the project area are predominantly a result of past development. Major sewer, water and electrical transmission mains will be relocated.

Transportation Systems

The bypass channel will affect the existing network of local streets, roads, and bridges the project area. New bridge crossings, as well as the modification of existing roads, are proposed to maintain existing traffic patterns. The replacement of existing roadways will be "in kind".

Henderson St. and Main St. will maintain their existing horizontal alignments but require modifications in vertical alignment. White Settlement Rd. is proposed to maintain its existing horizontal alignment over the bypass channel, but then be realigned across the new water feature to merge with 5th St. Grade separations are proposed over the Fort Worth and Western Railroad (FW&WRR) in conjunction with the Henderson and White Settlement bridges.

Bridge Structures

Preliminary profile grades have been prepared to meet various design constraints at each new channel crossing location. Bridges along Henderson and White Settlement are planned to receive aesthetic enhancements to a "medium-high" level, while the North Main St. Bridge is planned as a "signature" bridge with a cable-stayed superstructure.

At the Henderson and White Settlement crossings the bridges will span over the FW&WRR in addition to the bypass channel. Several factors control not only the length of the bridge but also the spans between columns, the height of the bridge, and the approach grades.

Pedestrian Bridge Structures

Two pedestrian bridges are proposed to serve a dual role of providing access and recreational connectivity. One pedestrian bridge is proposed over the bypass channel approximately 1,050 feet downstream of the Henderson Rd. The second pedestrian bridge will be located on the West Fork Trinity River is approximately 560 feet upstream of the Fort Worth and Western Railroad crossing.

Isolation Gates

Three isolation gate structures are planned for the Central City Project. The primary objective of these isolation gate is to protect the interior area from becoming inundated during periods of high flows.

Several different alternative gate types were evaluated. Fixed-wheel (roller) gates are preferred over other types, due to their ability to close during a power outage; relative ease to conceal gate leaves in a concrete structure; and maintenance accessibility while in the open position. The top of each structure will be set at four feet above the Standard Project Flood (SPF) elevation. Each structure will include one 24′x17′ channel gate for boating, and at least one 12′x10′ pedestrian walkway gate.

The gates are located upstream at the confluence of the bypass channel and the Clear Fork (Clear Fork Gate), at the midpoint of the bypass channel and the West Fork confluence (Trinity Point Gate), and downstream at the confluence of the bypass channel and the West Fork (TRWD Gate).

Samuels Avenue Dam

A gated dam is proposed on the main stem of the West Fork of the Trinity River just east of the Samuels Ave. Bridge. The dam was sited downstream from Samuels Ave. and the adjacent three railroad bridges, approximately 1,300 feet downstream from the confluence of Marine Creek.

A review of several alternative configurations and types of gates for the dam was conducted. These included leaf, crest, or bascule, gates that operate by lying down with released water flowing over the top of the gate, and radial gates that operate by rotating upwards, allowing floodwaters to flow underneath.

During normal dry weather operation the dam will maintain the normal water pool level elevation of 524.3. Based on hydraulic modeling the dam was sized to operate with seven 48-feet wide and 18-feet high gates. The gate width was chosen as the maximum reasonable width, enhancing the hydraulic capacity, while providing reasonable operable gates. The structure will also incorporate 4-feet wide by 6-feet high low flow conduits located in each of the three interior piers to minimize the use of the large flood gates and simplify operations.

Storm Water Pumping Station

A storm water pump station is planned to pump storm water from the interior area over the levee to the channel and for dewatering of the interior water feature area for maintenance.

The proposed location of the pump station is adjacent to and upstream of the proposed TRWD isolation gate, located on the east bank of the existing West Fork channel. A preliminary analysis performed determined a required pumping capacity of approximately 300 cubic feet per second (cfs).The preliminary pump selection for the storm water pump station is a vertical axial or mixed flow type.

Retaining Walls

New concrete retaining walls will be provided within the new flood bypass channel on the east side of the bypass channel (i.e. the "hard edge"). In general, there are three tiers of walls. Lower level walls will retain earth and contain the normal pool. Mid-level walls will retain earth above the normal pool level and below the Standard Project Flood (SPF) level. Upper level walls retain earth and extend to an elevation 4 feet above the SPF.

Structural design parameters are selected based on the requirements and recommendations of applicable USACE Engineering Manuals. Retaining walls have been evaluated for sliding, overturning, and foundation bearing capacity, by using the USACE computer program CTWALL.

Operation and Maintenance

TRWD Fort Worth Operations performs a variety of maintenance activities, similar to that expected for the FWCC Project. These practices include turf maintenance which includes mowing, weed removal, fertilizing, and tree removal. These and other operational considerations are discussed.

Section 1
Introduction

1.1 Project Description

The Fort Worth Central City Project consists of a bypass channel, levee system and associated improvements to divert flood flows around a segment of the existing Trinity River adjacent to downtown Fort Worth. The proposed bypass channel is approximately 8,400 feet long and approximately 300 feet wide between the top of levees. The bypass channel will be approximately 30 feet below existing grade. Figure 1-1 shows the bypass channel and other significant project components.

Water levels in the bypass channel will be controlled by a dam with crest gates. The dam is proposed on the West Fork of the Trinity River just east of the Samuels Avenue Bridge and will be designed to maintain normal water level of approximately 525 feet above sea level in the bypass channel and interior area. Flood isolation gates will be incorporated into the levee system to protect the interior area, otherwise known as Trinity Uptown. The gates are located upstream at the confluence of the bypass channel and the Clear Fork (Clear Fork Gate), at the midpoint of the bypass channel and the West Fork confluence (Trinity Point Gate), and downstream at the confluence of the bypass channel and the West Fork (TRWD Gate).

Construction of the bypass channel, dam and isolation gates will create an approximately two-mile segment of the existing West Fork Trinity River as a controlled, quiescent watercourse. A water feature or urban lake, approximately 900 feet long, is proposed for the interior area (Trinity Uptown). The water feature will extend from the bypass channel southeast to the existing West Fork and Clear Fork confluence of the Trinity River.

Six bridges are proposed for the project, including four vehicular bridges and two pedestrian bridges. Vehicular bridges are proposed over the bypass channel at North Main St., over the bypass channel and Fort Worth and Western Railroad (FW&WRR) at Henderson St. and White Settlement Rd., and on the White Settlement Rd. extension over the urban lake. Two pedestrian bridges are also proposed, across the bypass channel downstream of Henderson St., and across the West Fork, approximately 500 feet upstream of the existing FW&WRR Bridge.

The project also includes proposed modifications to University Dr., which will effectively raise the roadway approximately 10 feet from existing grade and out of the 100 year floodplain. The proposed modifications begin north of the existing bridge over the West Fork extending to Jacksboro Highway (State Highway 199).

The project could result in a loss of floodplain or valley storage due to the diversion of flood flows through the bypass channel as the bypass channel is shorter than the existing river channel. To prevent this potential loss of floodwater retention, valley storage mitigation sites are included in the preliminary design. Valley storage mitigation sites will be provided in three areas, along the West Fork of the Trinity River upstream of the project area, in the vicinity of the Samuels Avenue Dam, and slightly downstream of the dam in proximity to Riverside Park. Construction of the bypass channel and associated valley storage sites will not increase downstream water surface elevations or downstream flows.

1.2 Project Objectives

The objective of this study was to develop a preliminary understanding of the civil and structural components of the Central City Project. The civil components of the project include:

- Earthwork activities for construction of the channel and valley storage mitigation
- Utility relocation
- Modifications to transportation systems

The structural components of the project include:

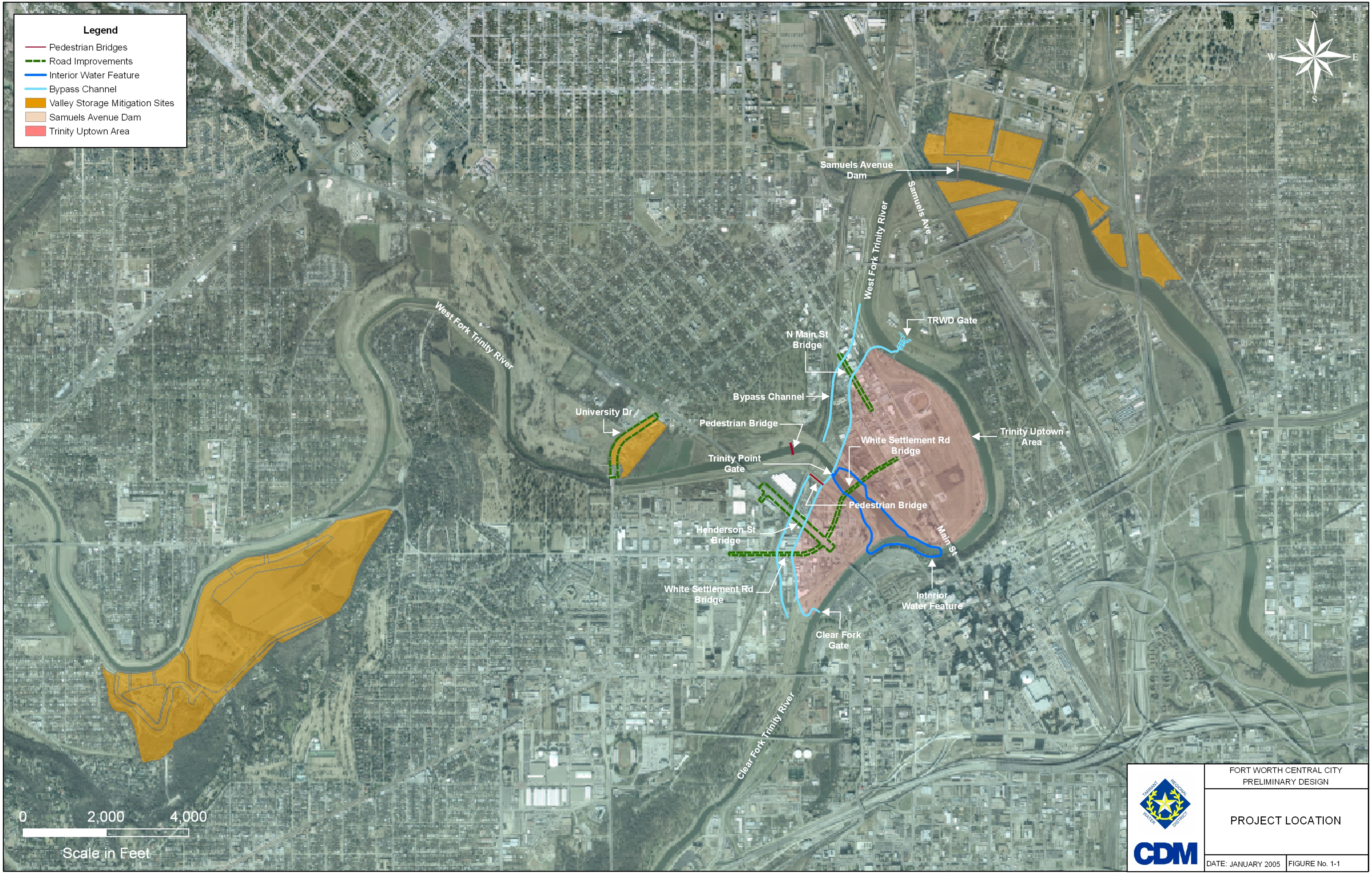
- Bridge Structures
- Pedestrian Bridge Structures
- Isolation gates
- Gated dam
- Storm water pumping station
- Bypass channel retaining walls

This report provides a description and conceptual level design and layout of the required components.

1.3 Scope of Work

The construction of a bypass channel and accompanying flood control components requires channel configuration and earthwork calculations, relocation of utilities, evaluation of the impact on existing roadways, creation of new roadway and pedestrian bridges, design of flood control components, and design of a storm water pumping station. The specific scope of work associated with each component is described in more detail in their respective sections.

Section 2 of this report provides a description of the earthwork activities. Section 3 provides the required utility relocation activities. The impacts of the project on transportation systems are outlined in Section 4. The bridge structures are described in Section 5. Pedestrian bridges are described in Section 6. Section 7 provides a description of the isolation gates. Section 8 provides detail on the Samuels Avenue Dam to be constructed. Section 9 provides information on the storm water pumping station and Section 10 describes the proposed channel's retaining walls. Section 11 provides the preliminary plan to operate and maintain the proposed bypass channel levee system, Samuels Avenue Dam, three Isolation Gate Structures, a Storm Pump Station, and Valley Storage Mitigation areas.



Legend
Pedestrian Bridges
Road Improvements
Interior Water Feature
Bypass Channel
Valley Storage Mitigation Sites
Samuels Avenue Dam
Trinity Uptown Area
N
E
S
W
Samuels Avenue Dam
Samuels Ave
West Fork Trinity River
TRWD Gate
N Main St Bridge
Bypass Channel
West Fork Trinity River
University Dr
Pedestrian Bridge
White Settlement Rd Bridge
Trinity Uptown Area
Trinity Point Gate
Pedestrian Bridge
Henderson St Bridge
Main St
White Settlement Rd Bridge
Interior Water Feature
Clear Fork Gate
Clear Fork Trinity River
0
2,000
4,000
Scale in Feet
TARRANT REGIONAL WATER DISTRICT
CDM
FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN
PROJECT LOCATION
DATE: JANUARY 2005
FIGURE No. 1-1

Section 2
Earthwork

2.1 General Description

2 .1.1 Purpose

The purpose of this section is to identify and develop the major earthwork components of the project including: 1) bypass channel configuration; 2) earthwork requirements for the bypass channel; 3) potential disposal sites for excess material; 4) general grading at structures, and 5) potential sites for mitigation of valley storage

2.1.2 Scope of Work

The scope of work includes the finalization of the channel alignment and typical bypass channel sections as the result of coordination between Hydrology and Hydraulics, Geotechnical, Utilities, and Urban Design Standards. Included in this effort is the determination of final alignment, typical bypass channel sections, grading limits, earthwork cut and fill volumes and general design parameters for the design of the bypass channel and valley storage mitigation sites.

2.1.3 Criteria

The following items were considered during development of the bypass channel sections and alignment:

- Urban Design Consultant conceptual drawings;
- Requirements from hydraulic modeling;
- Constructability of the bypass channel section;
- Optimize grading limits to maximize future redevelopment opportunities;
- Maximum levee slopes (3 horizontal: 1 vertical);
- Minimize off-site transport of excavation material ;
- Easement and right-of-way constraints;
- Adhere to Local, State and Federal Regulations, Codes and Laws; and
- Adhere to U.S. Army Corp of Engineer (USACE) Standards.

The following criteria were considered during the selection of potential valley storage mitigation sites:

- Relationship of existing ground surface elevation to existing water surface elevations;

- Proximity to the project area;
- Site availability and accessibility;
- Limited space opportunities and utility conflicts; and
- Synergy with ecosystem restoration.

2.2 Design Requirements

2.2.1 Bypass Channel

2.2.1.1 General Parameters

The bypass channel typical sections and alignment were developed to reflect the current urban design considerations discussed below. These cross sections will be further refined during the design development stage to meet the final project hydraulic and hydrological requirements.

Alignment

The bypass channel location and alignment is shown on Drawing C-1, Volume II. This drawing shows the current alignment used for the development of preliminary plan and profile, grading plans, and development of earthwork volumes. The bypass channel consists of two segments defined as the Lower and Upper segments, for a total length of approximately 8,400 feet. These segments are:

Lower Segment

The Lower bypass segment starts on the West Fork Trinity River at approximately Sta 2458+70, which is approximately 1,100 feet south of the East Northside Dr. Bridge (Sta 2447+66). The bypass channel alignment proceeds southwest until it intersects with the West Fork Trinity River for the second time. The intersection of the Lower segment and the West Fork Trinity River occurs east of the existing Fort Worth & Western (FW&WRR Bridge (Sta 2575+46) at approximately Sta 2570+25. The horizontal alignment (on the Lower segment) is generally controlled by the proposed bridge crossing of Main St. and the FW&WRR right-of-way (ROW) to the northwest.

The bypass channel alignment maintains the existing at-grade street crossing of Main St. and the FW&WRR with a maximum bridge approach slope of 5.0%. The alignment accounts for the need to preserve the Ellis Pecan Building located at 1012 North Main St., which is listed on the National Register of Historic Places. As configured, the Ellis Pecan Building and parking lot are located outside of the grading limits.

Upper Segment

The Upper bypass segment starts at the confluence with the West Fork Trinity River east of the existing FW&WRR Bridge (Sta 2575+46) and proceeds southwest until it intersects with the Clear Fork Trinity River (Sta 34+65) approximately 940 feet north of the West 7th St. Bridge (Sta 44+02). The horizontal alignment from the confluence with the West Fork Trinity River to approximately White Settlement Rd. is controlled primarily by ROW constraints to the northwest by the FW&W ROW. Along this portion of the alignment, drainage between the bypass channel and the railroad ROW is proposed to be provided by a drainage swale (approximately 10 feet in width) between the toe of the levee slope and existing railroad ROW.

South of White Settlement Rd. to its intersection with the Clear Fork Trinity River, the bypass channel alignment is controlled primarily by the Greenleaf St. ROW on the west side. A small portion of Greenleaf St. will be impacted near White Settlement Rd. However, access is intended to be maintained to the existing business located at the intersection of Greenleaf and Kansas Sts. A drainage swale was investigated in this area, but is not required as the existing ground surface drains towards Greenleaf St.

Profile

The bypass channel profile of the Upper segment is controlled by the existing elevations of the Clear Fork and West Fork Trinity Rivers at the confluence locations with the bypass channel as shown in Table 2-1. In an effort to control bypass channel velocities and limit rock excavation, the Lower segment was held at a constant slope of 0.2% until approximately 600 feet north of Main St., Sta 9+80. An approximate five foot drop structure is proposed to be constructed into the adjacent bedrock at Sta 9+80 in order to limit the bedrock excavation required and allow the bypass channel to intersect the West Fork Trinity River at the existing elevation at the downstream end of the project.

Geotechnical information at this location is very limited so a preliminary design of this drop structure cannot be completed at this time. The intent is to construct a reinforced concrete structure, which is locked into the anticipated bedrock at this location. The structure will be entirely underwater and be used to control undercutting of the bypass channel bottom and banks and has been included in the hydraulic model and cost estimate for this project. As additional geotechnical information is obtained, a structural design will be developed that takes into consideration the hydraulic requirements and geotechnical findings.

Table 2-1
Existing Elevations of Clear Fork and West Fork Trinity Rivers

River Section at Confluence Points	River Station / Bypass channel Section	Approx. Existing Channel Invert
West Fork Trinity River (Downstream)	WF 258+70/ 0+00	495.5
West Fork Trinity River (Upstream)	WF 2570+25/ 40+00	507.75
Clear Fork Trinity River	CF 34+65/ 83+95	515.1

Drawings CG-3 to CG-10, Volume II, show the bypass channel plan and profile along the alignment.

Bypass Channel Geometry
Six typical bypass channel sections (Drawings CG-11, CG-12, and CG-13, Volume II) were developed based on preliminary discussion with the urban design consultant and hydraulic modeling requirements. The typical sections were applied along the proposed alignment to define the approximate grading limits and real estate requirements. Additional iterations and refinement of these preliminary sections will occur during final design development.

The bypass channel typical sections consist of four general elements: the base flow channel, soft edge (levee), hard edge (walls), and staging area. Included as Figure 2-1, is a schematic of the bypass channel typical section.

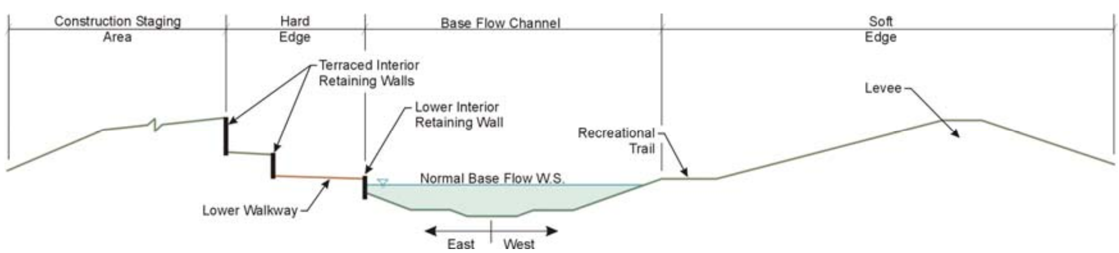


Figure 2-1
Bypass Channel Schematic, Looking South (Upstream)

The following is a general description of these four elements:

Base Flow Channel
The base flow channel is intended to contain the base flow (i.e. normal flow) and flood events of approximately a 25-year frequency event. Moderate fluctuations, not to exceed 1-2 feet, of the normal water surface (EL 524.3) are anticipated during less severe storm events. This normal water surface and ability to maintain this elevation with moderate fluctuations will be controlled

by the proposed Samuels Avenue Dam, shown on Drawings C-1 and SS-2, Volume II. Operation of the dam and maintenance of the water surface elevation during storm events will be further developed during the design phase.

The intent of the base flow channel is to maintain flow velocities, so that armament or other protection of the bypass channel bottom and side slopes will not be required.

Soft Edge

The soft edge of the bypass channel will be designed to be "park-like" or natural while providing adequate side slope erosion protection. The soft edge is located on the western side of the bypass channel, and incorporates the earthen levee. The soft edge will contain a recreational trail, sloped vegetation, and access for maintenance and emergency vehicles on both the top and bottom of the levee.

The 20 feet wide recreational trail will be located approximately 5-feet above the normal base flow water surface (EL 524.3), and comply with ADA requirements. A maximum cross slope of 2% and longitudinal slope of 5% or less are recommended. The recreational trail is envisioned to allow bikers, walkers, and roller-blade access to the “park-like” area. Portions of the recreational trail may also be used for equestrian use depending on the final outcome of the recreational use plan for the project. The recreational trail may also be used for emergencies and maintenance as necessary.

In addition to the recreational trail, access will be provided by construction of a gravel roadway on top of the levee to provide emergency and maintenance access during major storm events when the lower recreational trail is unavailable. Ramps or other means of street access will be provided to the top of the levee at a maximum horizontal spacing of 2,500 feet. If access cannot be provided at this frequency then turnouts will be provided. Ramps to the access roads will be designed to have a maximum slope of 10%. All access roads and ramps will be designed and constructed in accordance to USACE requirements.

Primarily, Bermuda grasses will be planted and maintained on the foreslope of soft edge levee side, above the recreational trail, for aesthetic and slope erosion protection purposes. Selection of the landscaping in this area will be appropriate native species and could include a combination of medium to tall shade tress and low lying bushes. Consideration will be given in selecting the landscaping that will be able to survive occasional storm flows in the bypass channel without impairing the integrity of the levee embankment. The foreslope of the levee and vegetation will be actively maintained for aesthetic and hydraulic purposes.

Native or Bermuda grasses will be planted on the backside of the levee and maintained in accordance with current operating procedures. On the backside of levee, at toe of slope, over land drainage will be provided through existing drainage swales or the construction of new swales where necessary. Additional ground survey will be required to fully develop the local drainage plan.

Hard Edge

The hard edge is located on the eastern side of the bypass channel. This edge section is planned to contain a series of tiered retaining walls, multiple walkways, and landscape areas. The design of these retaining walls is discussed in Section 10 of this report. Included as Figure 2-2, is the preliminary hard edge concept as developed by the Urban Design consultant.

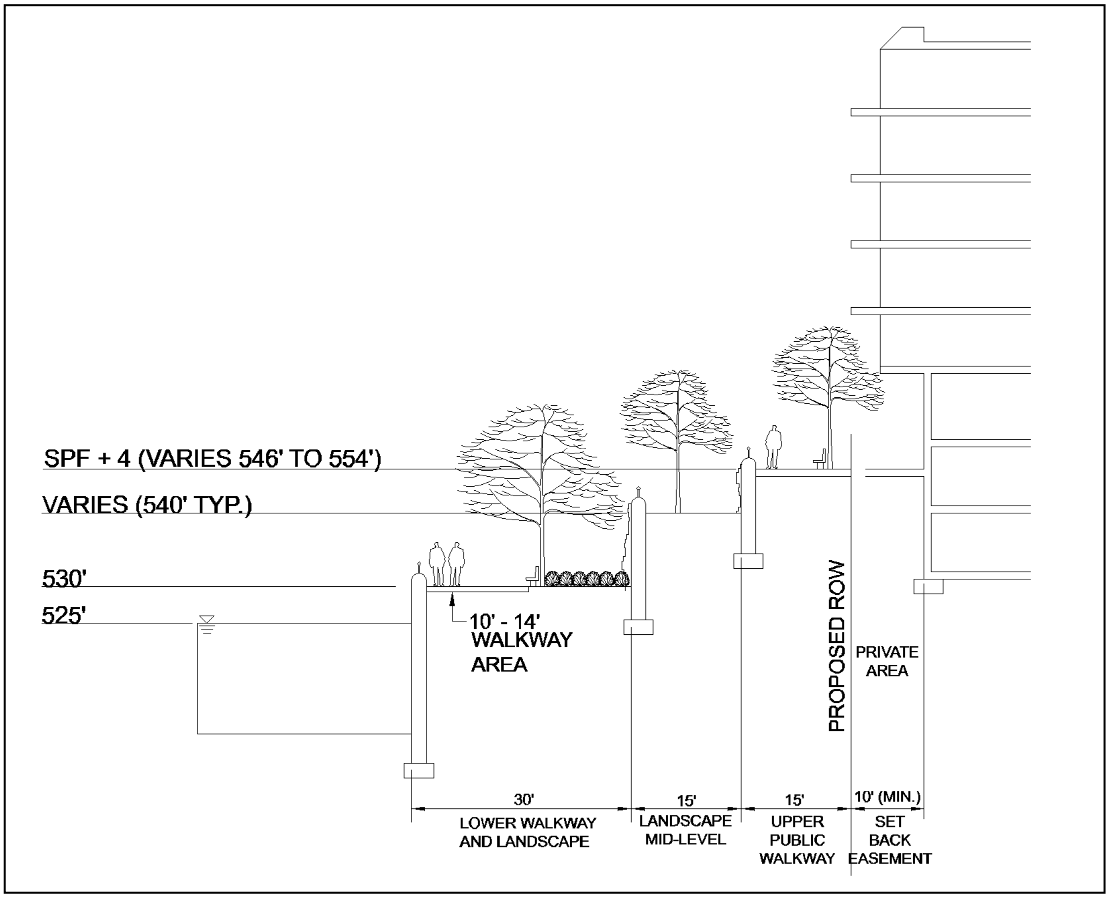


Figure 2-2
Typical Hard-Edge

The lower interior retaining wall is located adjacent to the base flow channel, with a top of wall elevation of approximately 530.0. This wall will vary in height but will generally have an exposed surface height of 10 feet. Immediately adjacent and parallel to the lower interior wall will be a walkway. Similar to the recreational trail on the soft edge, the lower walkway will allow pedestrian access to the "park-like" environment of the bypass channel. Along the lower walkway and interior wall, various amenities including park benches, sitting, or picnic areas will be present and surrounded by various landscaping features. The configuration and extent of these walkway and landscape areas will be further developed during the design development stage consistent with the hydraulic modeling assumptions.

Access to this lower walkway will be provided at various entry points by means of a combination of stairs and ramps which will be ADA compliant. These access points and physical layout and geometry will be determined during the design development stage.

The remainder of the grade separation from the lower interior wall and walkway to an upper interior wall will be accomplished with a series of terraced retaining walls. The height of retaining walls on the hard edge will vary along the length of the bypass channel but will typically consist of 3 walls, each varying in height from 5-10 feet. To break up and enhance the tiered walls walkways, landscape planters and other landscaping options will be considered.

Staging Area

During the construction of the bypass channel it will be necessary to have adequate space for the contractor to stage and construct the work. Construction of the bypass channel requires multiple stages and construction operations. Significant activities include the excavation of the base channel, construction of the soft edge levee, excavation of the hard edge and retaining walls. These construction activities are likely to occur concurrently or with significant overlap to compress the total construction schedule. Therefore adequate working space must be provided.

Consideration must also be given to the erosion control, dewatering, excavated material stockpile and handling, and materials staging operations required for each stage of the project. These operations were used to determine the temporary construction limits on the east side of the bypass channel (interior area). Included as Figure 2-3 is a preliminary layout for the staging area.

Formal staging areas will be largely dependent on the project sequencing, number of construction contractors, and staging of the project. Project staging and sequencing is discussed in Section 2.2.4 of this report and Section 3.2.2.8 of Appendix A, Hydrology and Hydraulic.

Designated staging areas will be provided for each construction stages with the layout and final configuration determined by each respective contractor.

2.2.1.2 Samuels Avenue Dam

The construction of the Samuels Avenue Dam will require a combination of temporary and permanent grading. Drawings SS-1 and SS-2, Volume II show the preliminary temporary and final grading plans for the structure. During construction of the dam a temporary diversion channel is to be constructed to the south of the future dam. The temporary diversion channel is proposed to the south of the dam due to the elevated rock that was encountered in the soil boring near the northern abutment.

Coffer dams will be constructed upstream and downstream during construction of the dam to divert low flows around the structure. The unnamed channel west of the railroad ROW, as shown on the grading plan, will also be routed around the dam structure as shown in Drawing SS-2, Volume II. Upon the completion of Samuels Avenue Dam the coffer dams will be removed and the excess material will be used to partially fill the temporary diversion channel.

Upstream of the dam, earthen levee embankments will also be constructed on both the north and south side of the existing UPRR Bridge crossing to maintain the normal pool elevation upstream of the dam. Some off-site materials may be required if suitable materials are not encountered during excavation. Additional geotechnical investigations prior to the construction of the dam will more adequately quantify the volume of suitable existing soil material for the embankment. Excavation of the adjacent overbanks for valley storage mitigation will provide additional embankment materials for both the coffer dams and levee embankments should insufficient or unsuitable materials be encountered. The Samuels Avenue Dam is discussed in more detail in Section 8 of this report.

2.2.1.3 Isolation Gate Structures

The construction of Clear Fork, Trinity Point and TRWD Isolation gates as shown on Drawing GI-2, Volume II, will require a combination of temporary and permanent grading. Constructed immediately adjacent and tied into the bypass channel are the Clear Fork and TRWD gates as shown on Drawings CG-14 and CG-15, Volume II. The Trinity Point gate is shown on Drawing CG-2, Volume II. The isolation gate structure design is discussed in more detail in Section 7 of this report.

During construction coffer dams will be constructed to protect the structures work zones. Final project staging will determine if diversion channels are required to divert flow around these structures during construction. Preliminary sequencing of the project intends to have the bypass channel functional prior to the start of construction of the gate structures, which would minimize the need for temporary diversions channels at all of the structures. The TRWD gate with its downstream location of the existing West Fork will require a pump station or diversion channel for diverting any interior drainage.

The Trinity Point gate will be tied directly into the bypass channel hard edge on both sides. Construction of this gate will include the construction of the adjacent hard edge walls until they intersect the existing levees. The Clear Fork and TRWD gate structures will be tied into the bypass channel by a series of retaining walls. As shown on the Drawings CG-14 and CG-15, Volume II, the opposite sides of the gates will be composed of earthen levees tied into the adjacent ground surface at an elevation 4 feet above the Standard Project Flood (SPF). The existing levee adjacent to the TRWD gate does not currently meet the SPF plus 4 feet requirement and is proposed to be raised as part of the gate construction to meet this requirement. Material will be stockpiled from the excavation of the bypass channel to construct these levees. Appropriate stage areas for material stockpiles will be reserved outside of the existing floodway as to not constrict flow within the existing floodway.

A storm water pump station will be constructed concurrently with the TRWD gate to address the future interior drainage needs once the gates are constructed. The Storm Water Pump Station is discussed in Section 9 of this report.

2.2.1.4 Interior Water Feature

A water feature is proposed along the existing West Fork between its confluence with the Clear Fork and the Trinity Point gate. The interior water feature is shown on Drawing CI-1, Volume II. Construction of the water feature will consist of the excavation of the existing banks of the West Fork to widen the channel except the southeast portion closest to the TXU power plant property, which will be left in its existing condition. Future investigation of the soil and groundwater and coordination with TCEQ is proposed before finalizing the future design in this area, due to the contamination concerns.

Retaining walls similar to the lower interior walls constructed as part of the bypass channel will be constructed along a majority of the banks as shown on the drawings. To reduce the height of the retaining walls it is proposed to fill the bottom of the existing West Fork channel (approx EL 509) to EL 515. Excavation of the interior water feature will be conducted primarily under dry conditions as the area will be protected by the Trinity Point gate and Clear Fork gates.

2.2.1.5 Earthwork Calculations

Preliminary earth work volume calculations were performed based on the current bypass channel alignment and typical sections. Approximately 1.4 million cubic yards (CY) of material are anticipated to be excavated for the construction of the bypass channel. Of the 1.4 million CY, approximately 130,000 CY is expected to be rock excavation. In addition, approximately 490,000 CY are anticipated to be excavated from the areas around the isolation gates and interior water feature.

The geotechnical investigation found that suitable soils were encountered for use on the project. Construction of the levee and embankment immediately adjacent to the retaining walls will require specific materials that will need to be segregated from other excavated soils. The remaining materials not used for levee and embankment construction will be used to fill additional interior areas and available spoil sites. Further discussion on materials disposal is included in Section 2.2.1.6. A potential exists that some near and deep surface soils may be contaminated and require special handling. Soils handling and management are discussed in Section 2.3.1.

Preliminary excavation and fill volumes are summarized in Table 2-2.

Table 2-2
Preliminary Excavation Summary

Description	Excavation (CY)	
	CUT	FILL
Bypass Channel		
Unclassified Excavation	1,280,000	900,000
Rock Excavation	130,000	
SUBTOTAL	1,410,000	900,000
Isolation Gates	55,000	500,000
Interior Water Feature	435,000	160,000
TOTAL (CY)	1,900,000	1,560,000
NET EXPORT (CY)	340,000	

2.2.1.6 Materials Disposal

Excess quantities of excavated material that cannot be disposed of along the bypass channel will be disposed of outside the bypass channel project limits. As shown in Table 2-2, the majority of the excavated materials from the bypass channel will be used to construct the levees and embankment behind the hard edge retaining walls. Excavated rock may be suitable for use as on-site structural backfill for the retaining walls. Preliminary analysis of the excavation quantities found that a complete balancing of the bypass channel excavation within the proposed grading limits would not be feasible unless the interior embankment fill area is expanded. The intent is to retain all excavated material within the project limits to the extent possible because future redevelopment of the interior area is anticipated to require additional fill.

Fill requirements were identified for the University Dr. mitigation site as part of the preliminary design and hydraulic modeling effort. The University Dr. mitigation site is discussed in more detail in Section 2.2.2.5. University Dr. is anticipated to be able to accommodate approximately 300,000 CY.

An additional fill site is located on the northside of Henderson St. and west of the FW&WRR. This site will require fill to allow the construction of the Henderson St. Bridge and is close to the bypass channel.

Excess materials can also be hauled to the fill site B near Northside Dr. and stockpile it in this location until additional fill operations occur on the interior area. This site is also to be used for stockpiling fill from valley storage mitigation work. Valley storage mitigation is discussed in Section 2.2.2.

Considering the possibility that storing excess material on-site may not be practical, additional disposal sites were located for potential use. Included as Figure 2-4, are the potential sites identified during this initial investigation process. Adequate sites exist within 10-15 miles or less of the project that are capable of handling material disposal. However, options exist that would minimize or eliminate the need to transport additional material off-site (i.e., temporary raising of fill areas and stockpiling on TRWD property). These options will be pursued during detailed design and scheduling

2.2.1.7 Coordination with Outside Agencies

The design of the various bypass channel components will require coordination with outside agencies including but not limited to the City of Fort Worth, USACE, Texas Department of Transportation, impacted private utility providers, and railroad operators.

2.2.2 Valley Storage Mitigation

Construction of the bypass channel requires the mitigation of floodplain storage, referred to as "valley storage" to compensate for the loss caused by the bypass channel. The amount of valley storage mitigation required for the project was determined by the hydraulic modeling analysis and compliance to the criteria established by the Corridor Development Certificate (CDC) guidelines. The hydraulic analyses quantified the approximate volume of valley storage lost as 5,250 acre-ft (8.47 million cubic yards) with no action. The Relevant Design Criteria and Hydraulic Analysis detailing the mitigation storage requirements are included in Sections 1.5 and 3.2, respectively, of the Hydrology and Hydraulics Report.

The valley storage loss caused by the construction of the bypass channel is comprised of two components. Routing of the existing Clear Fork and West Fork through the bypass channel instead of the interior area reduces the total length of channel which subsequently results in less in-line floodplain storage. The shorter channel length also creates a drawdown affect on both the Clear Fork and West Fork 100-year and SPF water surface elevations upstream of the bypass channel which reduces the upstream valley storage.

2.2.2.1 Valley Storage Approach

Two primary approaches were considered in developing alternatives to mitigate the valley storage loss. One approach is overbank storage mitigation by providing addition locations which could be modified for additional volume. Storage mitigation is characterized as either off-line or on-line.

The other approach is drawdown mitigation where the intent is to minimize the amount of water surface drawdown on the existing West Fork and Clear Fork channels. Drawdown mitigation typically consists of investigating potential locations where the 100-year and SPF flows can be constricted to restore the upstream water surface elevations to pre-bypass channel conditions.

Both approaches require a detailed investigation along the West Fork and Clear Fork. The purpose of the investigation is to locate suitable mitigation options and areas which are in the best interest of the community and project.

The West Fork was investigated from downstream of Riverside Dr. to upstream of Westworth Boulevard. The Clear Fork was investigated from its confluence with the West Fork to U.S. Interstate 30. The investigation included a review of aerial photography, existing USACE topography, parcel ownership information, and utilities. Site reconnaissance trips were made along the existing levee system to identify, confirm and visually evaluate potential mitigation sites.

Several drawdown alternatives and numerous storage mitigation sites were investigated in an effort to determine the most suitable alternatives and storage sites based on cost effectiveness, proximity to the core project, and other factors.

The following is a summary of the storage mitigation sites and drawdown mitigation alternatives.

2.2.2.2 Storage Mitigation

The evaluation of storage mitigation sites included three phases: 1) identification/investigation, 2) ranking, and 3) findings. The following is a summary of the site identification/investigation, ranking rationale, and findings which determined the most suitable storage mitigation sites.

Site Identification/Investigation

The primary emphasis during the site identification and investigation was to select undeveloped sites in the immediate vicinity of the Trinity River. Aerial photographs and existing site topography were used to develop a set of preliminary valley storage mitigation sites which could be investigated by the project team. Property ownership and existing site utilities were researched and identified for each of the potential sites.

A total of forty (40) individual sites were identified and subsequently investigated to estimate the potential amount of valley storage that could be created on each site. The amount of valley storage was then compared to the cost to acquire the property and the cost of necessary site improvements to create the additional storage. The valley storage mitigation sites were divided into two groups and referred to as the Valley Storage Mitigation Sites - Lower West Fork and Upper West Fork as shown on Figures 2-5 and 2-6, respectively.

Site visits were conducted by both CDM and TRWD personnel to further quantify the viability and desirability of each of the sites. The following is a summary of the steps taken in determining the total site improvement costs for each of the preliminary sites.

Site Improvements

Each valley storage mitigation site was investigated for its potential storage capacity based on 100-year and SPF water surface elevations from the hydraulic modeling and USACE topographic data. Based upon this information, each site was evaluated for cuts and fills to determine the potential valley storage volumes on a balanced site basis. Balancing the site meant that all excavation cut materials would be retained on-site, considered to be the most economically favorable alternative, if feasible.

The existing topography initially determined which sites could be balanced or required haul-off. Excavation is required below the SPF water surface elevation and fill limited to above SPF on each site to create a net gain in valley storage. A majority of the valley storage mitigation sites did not provide substantial benefit from balancing the excavated material and were subsequently investigated as haul-off sites.

After evaluating sites as either balanced, haul-off or a combination, the preliminary excavation and valley storage volumes were tabulated for each of the sites. These volume quantities and respective areas impacted by the various cut and fill operations were tabulated and units prices assigned for each element of work.

Parcel Ownership

Initial parcel ownership identification was performed using parcel ownership information provided by the TRWD geographical information system. Parcel queries were performed on an individual site basis to determine ownership and the current assessed parcel value including site improvements if applicable. Valley storage sites that share off-site fill sites had the parcel costs prorated based on the approximate volume of material to be disposed of at each site.

Additional parcel ownership checks were made through the Tarrant Appraisal District (TAD) website if the initial query did not cover the full extents of the valley storage mitigation sites. Parcel acquisition costs were then tabulated and grouped as either public or private.

Utilities

Public and private utilities conflicts and impacts were initially screened using available City-wide data from the geographical information system. Once the initial screening was completed and the most suitable sites were identified, public and private utility carriers were contacted to confirm the extent and nature of utilities on the preferred sites. Utilities were considered either regional or local depending on the service area they covered. Regional utilities serve larger service areas than that of each individual valley storage mitigation site or serve as major transmission facilities. If the utilities only serve the immediate area of the valley storage mitigation site, then they are considered local.

For estimation purposes local, utility facilities were considered to not require replacement since they could be abandoned or removed at minimal cost without significantly impacting the overall utility service grid. Regional utility facilities given their system-wide importance were assumed to be fully replaced or protected if within the impact limits of each of the valley storage mitigation sites.

Ranking Rationale

The site improvements, costs, parcel ownership and values, and utility relocation costs were tabulated for each site and are shown on Table 2-3. The sites were then ranked using the ratio of total cost versus storage ($/ AC-FT) with the intent of identifying the most economical sites. These rankings are shown on Table 2-4.

Site Findings

Cost was one component in the overall evaluation of the storage mitigation sites. Other factors, such as proximity to other improvements, project staging, impacts to existing vegetation, implementation, and ecosystem enhancement opportunities were also considered in the final findings. Based on the rankings shown in Table 2.4, storage mitigation sites XXXIX and XXXVIII were found to be the most suitable, economically.

Although site XXXIX is the highest ranked site, several distinct disadvantages exist with this site. The proximity to the Greenwood Cemetery and heavy forestation makes the site less favorable due to the impact of constructing a new levee through the site, which would require extensive clearing. Site XXXVIII (also referred to as Riverbend) is considered a very suitable site for valley storage mitigation, given the large contiguous footprint. The site also provides a unique opportunity to provide some ecosystem enhancement. Site XXXVIII is located on the upper end of the project and provides a major portion of the necessary storage mitigation required.

Sites II and III, located adjacent to the Samuels Avenue Dam site, are also considered very favorable due to the construction work that will occur at this location as a result of the dam construction and location downstream of the bypass channel. Mitigation sites IV, V, VI and XVI are also in the same general proximity of the dam site and are fairly contiguous. These sites were not ranked as high as others but were deemed suitable, given there continuity to one another and the fact that there will be minimal impacts during construction and fewer impacted property owners. This continuity provides a much greater value from the hydraulic modeling and construction staging and site economy viewpoint. Collectively, these sites are now referred to as the Downstream Mitigation Sites.

2.2.2.3 Drawdown Mitigation

As discussed in the Hydrology and Hydraulics report the amount of valley storage upstream on the Clear Fork and West Fork is directly related to the drawdown effect of the bypass channel. In an effort to minimize this drawdown effect five potential drawdown mitigation options were considered as part of the hydraulic analysis. These four drawdown mitigation options are:

- Option 1 - Construction of a large impoundment, (i.e. bridge) on the West Fork upstream of the FW&WRR crossing (with partial storage mitigation).

- Option 2 - Construction of a split channel with an island configuration on the West Fork upstream of the FW&WRR crossing (with partial storage mitigation).

- Option 3 - Raising University Dr. out of the 100-year flood plain (with partial storage mitigation).

- Option 4 - Constricting flow in the bypass channel (with partial storage mitigation).

Each alternative is hydraulically feasible and provide varying levels of drawdown mitigation. Partial storage mitigation is required for each of the alternatives to fully compensate the total valley storage loss. A cost evaluation of Options No. 1 through 4 indicates that the most cost effective alternative is to raise University Dr. and provide additional storage mitigation. The raising of University Dr. provides the additional benefit of removing the roadway out of the 100-year flood plain.

2.2.2.4 Mitigation Site Design

Preliminary design of the mitigation sites includes the development of preliminary grading plans and utility site plans. The mitigation areas are divided into three groupings of mitigation sites: Riverbend, Downstream, and University Dr. The following is a summary of these mitigation sites.

Riverbend Mitigation Site

The Riverbend mitigation site is located on the West Fork, upstream of White Settlement Rd. Grading of the Riverbend site consists of the removal of portions of the existing levee, as shown on Drawings CG-16, CG-17, CG-18 and CG-19, Volume II. A new levee will be constructed on the south and east sides of the site to provide appropriate SPF protection. Construction of the new levee requires the removal and replacement of the existing flood control gate and the rerouting of the existing drainage to the existing sump at the southern end of the site. Cross sections of the site are included on Drawings CG-20 and CG 21, Volume II. Construction of the levee and other spoil areas balance the total cut excavation on the site.

In addition to the grading operation required to construct the new levee, additional grading is proposed to provide added ecosystem benefits. This grading includes the cutting of an oxbow and swale through the middle portions of the site. Secondary swales are also proposed from the other notches in the existing levee to allow the interior areas to be inundated at increased frequencies.

The extensive grading operation on this site will impact the utilities located on the site as shown on Drawings CU-VS-38 and CU-VS-39, Volume II. Utilities impacted by the site improvements include a 24 inch sanitary sewer line that crosses the middle of the site in a northeast to southwest direction. The 24 inch sewer line also has several tie-ins with smaller 6 inch lateral lines generally running northwest to southeast along the length of the line and will need special consideration as a new levee is to be constructed in this area as shown on Drawing CG-17 and CG-18, Volume II. Based on the depth information known a larger 39 inch sanitary sewer line runs northeast to southwest on the eastern side of the site but does not appear to require relocation based on planned excavation depths. Additional verification of the line depths will be required to confirm and protection will needed given the reduction in cover and large amounts of fill place on top of it in some locations. A larger 60 inch sanitary sewer line located on the southern portion of the site, shown on Drawing CU-VS-39 appears to be abandoned and will be removed as required to accommodate the new grading.

Other utilities include an 8 inch water line, 36 inch storm drain, and a number of electrical lines and poles throughout the site. As final design of the mitigation site proceeds these facilities will be relocated or protected as needed or grading will be adjusted to minimize the total impacts.

University Drive Mitigation Site

Preliminary grading and plan and profile for University Avenue are included as Drawings CP-8 to CP-10, Volume II. The intent of the University Dr. mitigation site is to raise the profile of the existing roadway approximately 10 feet to a low point elevation of 549.5 to constrict flow which currently overtops University during 100-year and SPF flood events. Imported fill will be required to raise the roadway as well relocation and modification of existing utilities. Imported fill is proposed to come from the excavation of the bypass channel.

Utility impacts as a result of the University Dr. modifications are shown on Drawing CU-VS-40, Volume II. Two water lines (20 inch and 6 inch) travel northeast to south throughout the site, as well as a 36 inch storm drain and 6 inch sanitary sewer line which run parallel these water lines. These lines and any electrical service in the ROW are assumed at this time to be replaced for the extent of the roadway modification due to the profile adjustment in the roadway. Several other sanitary sewer lateral cross perpendicular to the roadway including a 66 inch sewer main on the southern portion of the site. These facilities are assumed to be left in place and protection measures utilized.

Downstream Mitigation Sites

The Downstream mitigation sites include a total of six separate mitigation sites (II, III, IV, V, VI and XVI). Grading and cross-section plans for the downstream mitigation sites are included as Drawings CG-22 to CG-28, Volume II. Excess materials from the excavation of these sites will be used to fill the spoil areas shown on CG-22, CG-23, and CG-24. Additional excess materials from mitigation sites V, VI, and XVI will be disposed of to the west of mitigation site XV, as shown on Figure 2-5.

Grading work on the downstream mitigation sites will affect a small number of the existing utilities as shown on Drawings CU-VS-41 and CU-VS-42, Volume II. Electrical, gas, and sewer facilities shown on Sites II, III, and IV and V are intended to be preserved and grading operation are not anticipated to affect these facilities. Electrical lines and poles running northwest to southeast on the eastern portion of the Site XVI are anticipated to be impacted and will be relocated. Spoil materials from Site V and VI placed on an adjacent fill site may also impact additional cable and electrical facilities and have been assumed to be relocated as part of the work.

2.2.3 Operation and Maintenance

Primary responsibility for the operation and maintenance of the bypass channel will rest with the TRWD. However, it is anticipated that the TRWD, through a series of Memorandums of Understanding (MOU) or Intergovernmental Cooperation Agreements (ICA), will share the responsibility for the maintenance and operation of non-flood control critical facilities with such entities as the City of Fort Worth and/or a development authority.

To provide for maintenance and operation, two areas will be established behind the hard edge on the typical sections, as shown on Drawings CG-11, CG-12, and CG-13, Volume II. This includes the area set aside, between the retaining wall face and proposed TRWD property for any physical components of the retaining wall such as tie-backs, footings, etc. The width of this area is currently assumed to be 14 feet on the upper portion of the bypass (approx Sta 40+00 to Sta 84+21) and 6.5 feet on the lower portion of the bypass (Sta 0+00 to Sta 40+00), based on the initial retaining wall design parameters. This area may also be used as a public walkway. Final determination of this width will be based on the final retaining wall design and may be reduced in some areas.

A second area shown on the typical sections is a building setback easement. The final determination of this width will depend on the final retaining wall design and urban design standard developed for buildings that will front the bypass channel. This area will be located beyond the proposed TRWD property line and be used to restrict building development. This area would be available for low impact facilities such as patios by adjacent land owners, but would restrict large-scale development, such as buildings or parking structures. The intent of this area is to allow for significant maintenance work, if required, on the retaining walls and also provide suitable space for necessary excavation support that would be required to construct foundations for new buildings below and elevation of the adjacent retaining wall footings.

Operation and Maintenance is discussed in Section 11 of this report.

2.2.4 Control of Water and Project Sequence

Sequencing of the bypass channel construction and other elements will be critical to protect the environment and maintain comparable flood protection levels. Care will be taken in planning the construction activities to minimize any potential negative impacts on the river. Separate erosion and water control plans will be prepared for various construction contracts and elements of the project. The plans will include requirements and guidelines for contractor staging and equipment maintenance areas.

Mass excavation and grading will be planned and sequenced to minimize in-channel and bank excavation. Where in-channel or swale excavation is required, the excavation will be scheduled from downstream to upstream and major equipment and supplies removed from the floodway each day. Dewatering discharges from excavations will not be allowed to discharge directly to the river or storm sewer. Discharges will be directed to sedimentation basins outside of the existing floodway, prior to discharge to the river. Discharge of the contaminated water is addressed in Section 2.3.2.

Buffer zones and barriers will be provided in excavation and fill areas to minimize erosion and siltation to water courses and/or the storm sewer system. Seeding of new levees will be completed as soon as possible to produce rapid establishment and maturity of cover. Temporary biodegradable erosion control blankets will be used in selected areas to help minimize erosion and facilitate the growth of vegetative cover.

Consideration was given, during the development of the sequence of work, to minimizing construction impacts to waterways. A preliminary sequence of construction has been established based on assumptions that environmental assessments, land acquisition, permitting, and funding activities will not adversely impact the schedule. Key issues and objectives considered and factored into the development of this preliminary sequence include:

- Minimizing the duration of construction activities within or directly connected to the River channel.
- Maintaining a comparable level of flood protection during construction.
- Phasing of improvements to have valley storage mitigation areas on-line at the appropriate time.
- Maximizing construction opportunities under dry conditions.

For discussion purposes, the construction sequence can be described in eight basic segments. Actual contract packages, construction contract size, and specific timing will be developed in more detail as the project detailed design progresses. The overall sequencing requirements and constraints are shown by the following construction segment overview:

- ***Construction Segment No. 1: Roadway Bridges:*** Construct temporary roadway bypasses at Henderson, Main St., and White Settlement. Construct bridge piers, bridges, and roadway approaches at all three locations. Complete roadway improvements and tie-in to new bridges. This will allow for the construction of the bridges and roadways "in the dry" without the need for temporary bridgeworks.

- ***Construction Segment No. 2: Interior Bypass Channel:*** Construct the interior portions of the upper and lower bypass channels without breaching the existing levees to the river. Complete excavation, utility relocations, new levee construction, and interior retaining walls. This will allow for a major portion of the channel to be constructed "in the dry" condition, except for potential groundwater.

- ***Construction Segment No. 3: Riverbend Mitigation:*** Complete the Riverbend mitigation site grading, ecosystem restoration and levee modifications. This will provide additional valley storage to compensate for the drawdown when the bypass channel is initially opened.

- ***Construction Segment No. 4: Bypass Channel Tie-ins:*** Construct the remaining reaches of the upper and lower bypass channel excavation, levee, and retaining walls. Breach levees and tie-in new bypass channel beginning from lower to upper channel connections. This will minimize the amount of construction within the existing channel and reduce the amount of coffer dam construction.

- ***Construction Segment No. 5: Construct University Drive Mitigation:*** Reconstruction of University Dr. to raise it out of the 100 year flood elevation and to provide for valley storage mitigation will closely follow the completion of the bypass channel. This component is required to partially restore the 100-year and SPF flood elevations from the drawdown effect of the bypass channel on the West Fork. Construction will be deferred until the bypass channel is complete so there will not be an increase in flood elevations during construction.

- ***Construction Segment No. 6: Construct Isolation Gates:*** After the completion of the bypass channel and "upstream" valley storage mitigation the existing West Fork interior channel can be taken out of service for major flow events. This will allow for the construction of the isolation gates for the interior area. Coffer dam construction is envisioned to segregate the construction area and provide protected working conditions from river flows. This segment will include the construction of all three isolation gates, tie-ins to the bypass channel retaining walls, levees, and the stormwater pump station at the TRWD gate.

- ***Construction Segment No. 7 Samuels Avenue Dam:*** Construction of the Samuels Avenue Dam will also include the remaining downstream valley storage mitigation sites. Construction of these improvements will be concurrent with the construction of the isolation gates thus providing the remaining valley storage when the interior area is completely isolated.

- ***Construction Segment No. 8 Interior Water Feature and Connector:*** Completion of the isolation gates and valley storage sites will enable the re-routing of flows from the interior area to the new bypass channel. This will allow for the construction of the interior water feature and the completion of the White Settlement Connector.

2.3 Environmental

Excavation activities, as part of the construction of the bypass channel and valley storage mitigation site, through existing industrial and commercial area raises the concern of encountering environmental issues. To evaluate the potential risk posed by these activities a limited Phase I Environmental Site Assessment (ESA) was conducted to determine if widespread contamination would be encountered in the construction areas, and to identify potential Recognized Environmental Conditions (RECs) on parcels within and immediately adjacent to the construction areas. Included in Appendix D is a summary of the investigation of these sites.

2.3.1 Soils Handling and Management

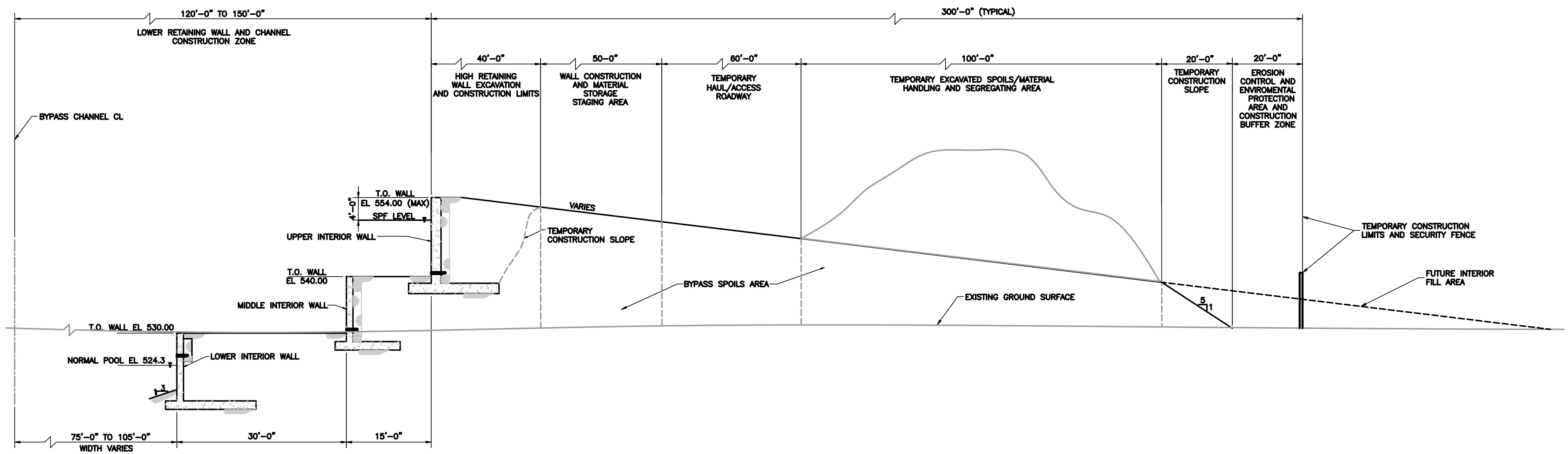
If contaminated soils are found during construction that exceed regulatory standards then they will be properly handled and disposed of as follows: (1) placed within the earthen levees; (2) placed within the earthen levees after on-site treatment; (3) placed in a Subtitle D landfill; (4) placed in a Subtitle D landfill after on-site treatment; or (5) placed in a Subtitle C hazardous waste landfill/disposal facility. Disposal method will be determined by chemical characteristics of the soil and cost effectiveness. Soil handling and disposal will be conducted in accordance with the TCEQ 30 Texas Administrative Code (TAC) Chapter 350 – Texas Risk Reduction Program (TRRP) methods. Project coordination with the TCEQ – Region 4 in Fort Worth and the TCEQ Remediation Division in Austin, TX will help guide the soils excavation, remediation, and disposal efforts in accordance with 30 TAC § 350 during the establishment of the Trinity River channel realignment corridor. A detailed soil management plan, based upon the latest available data, will be developed during subsequent design phases of this project.

2.3.2 Groundwater Handling and Management

During the initial phase of the geotechnical investigation, groundwater was encountered at various depths. Based upon preliminary observations, groundwater levels were at an approximate elevation of 520 feet. Soils encountered consisted of alluvial soils varying from plastic clay, clayey silty sand, and gravels. These soils are conducive to a number of dewatering methods. The determination of a method of dewatering during construction will be based upon subsequent investigation and evaluation.

Initial groundwater sampling at eight locations along the proposed bypass channel indicated the presence of arsenic and bis (2-ethylhexyl) phthalate above regulatory standards. However, these compounds were not present during a subsequent groundwater sampling event and the groundwater in the areas defined by the eight wells is not expected to require treatment during dewatering.

Prior to discharging any water during dewatering of excavations, the water will be sampled and characterized for contamination. Any groundwater that exceeds applicable regulatory standards (30 TAC § 350) will be treated in accordance with Texas Pollutant Discharge Elimination System (TPDES) or with local publicly-owned treatment works (POTW) requirements prior to discharge to State waters or to the sanitary collection system. Treatment methods will be determined based on chemical characteristics of the discharge water.



ELEVATION

NTS

FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

BYPASS CHANNEL
CONSTRUCTION WORK LIMITS

CDM

DATE: JANUARY 2005

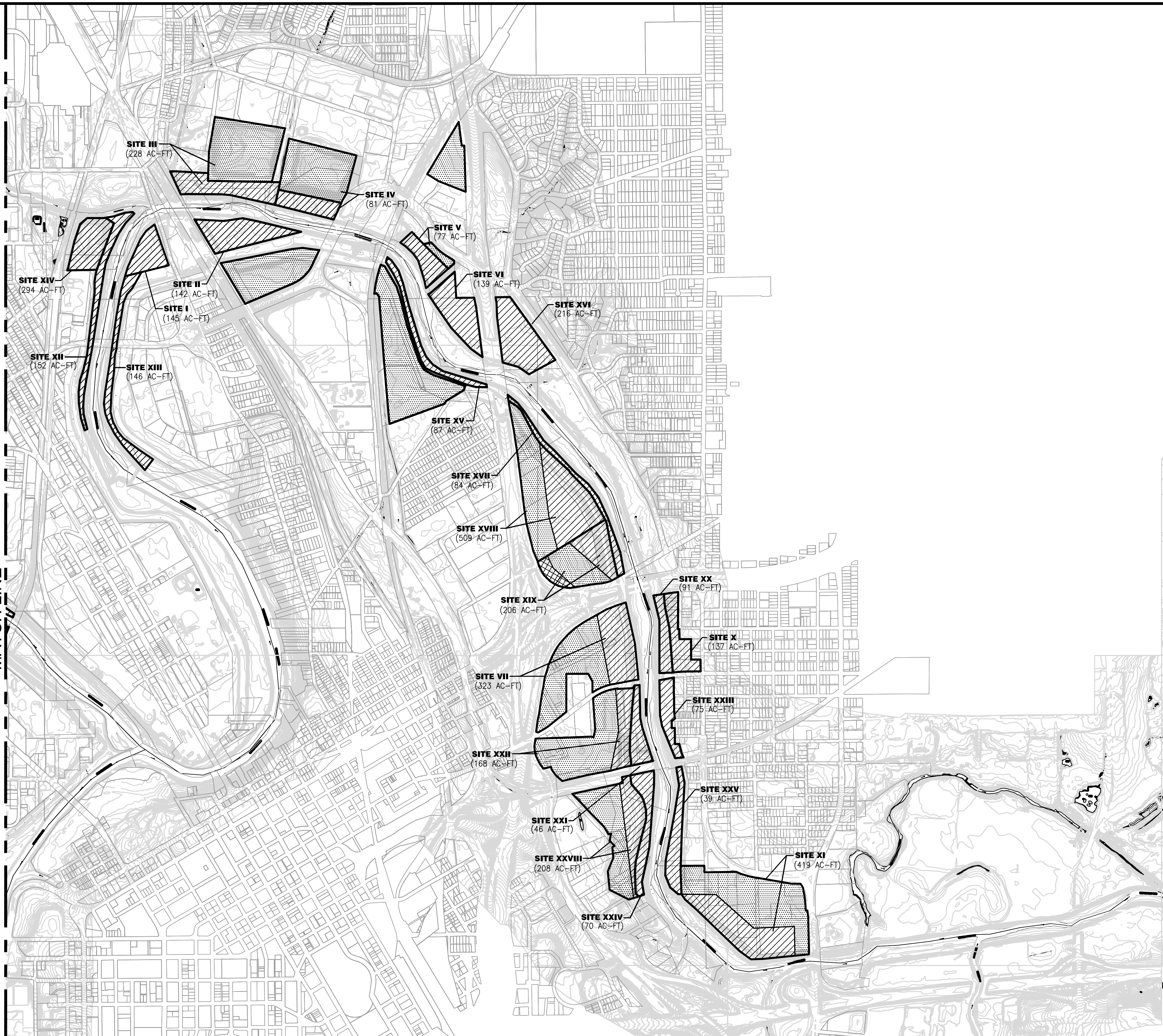
Figure No. 2-3

FORT WORTH CENTRAL CITY PRELIMINARY DESIGN

OFFSITE DISPOSAL SITES

DATE: JANUARY 2005

FIGURE No. 2-4



N

1600 800 0 800

SCALE: 1" = 1600'-0"

LEGEND

VALLEY STORAGE MITIGATION CUT SITES

PROPOSED VALLEY STORAGE MITIGATION FILL SITES

PROPOSED INTERIOR SUMP LOCATION

SITE BOUNDARY

NOTES:

1. VALLEY STORAGE MITIGATION STORAGE VOLUMES SUBJECT TO REVISION BASED ON FINAL GRADING REQUIREMENTS AND H & H HEC-RAS MODEL ANALYSIS.

FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

VALLEY STORAGE SITE
ALTERNATIVES
LOWER WEST FORK

TARRANT REGIONAL WATER DISTRICT

CDM

DATE: JANUARY 2005

Figure No. 2-5

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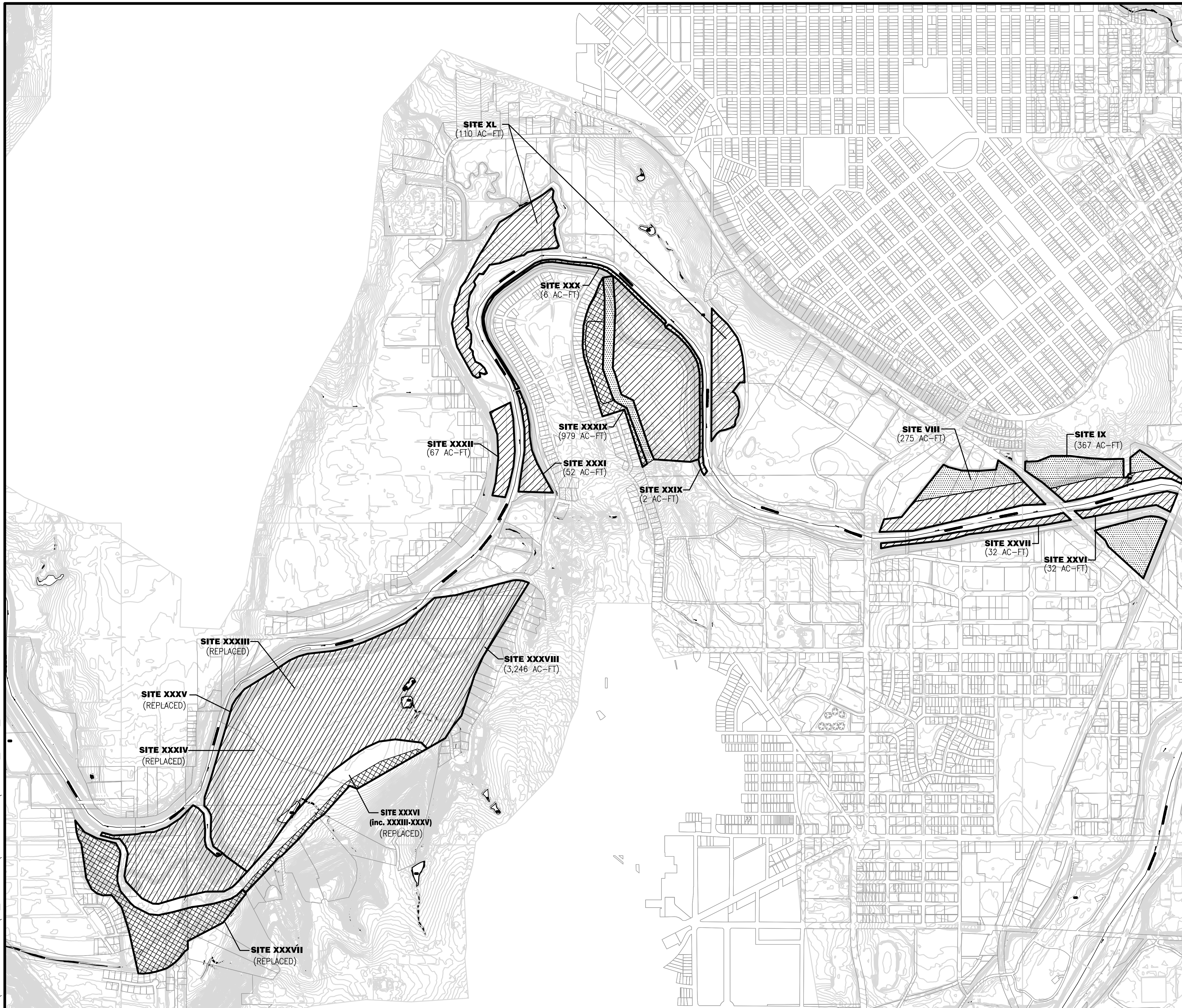

N

1600 800 0 1600

SCALE: 1" = 1600'-0"

LEGEND

- VALLEY STORAGE MITIGATION CUT SITES
- PROPOSED VALLEY STORAGE MITIGATION FILL SITES
- PROPOSED INTERIOR SUMP LOCATION
- SITE BOUNDARY

NOTES:

1. VALLEY STORAGE MITIGATION STORAGE VOLUMES SUBJECT TO REVISION BASED ON FINAL GRADING REQUIREMENTS AND H & H HEC-RAS MODEL ANALYSIS.
2. VALLEY STORAGE MITIGATION SITES XXXIII, XXXIV, XXXV, XXXVI, AND XXXVII ARE REPLACED BY SITE XXXVIII.

TARRANT REGIONAL WATER DISTRICT

CDM

FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

VALLEY STORAGE SITE
ALTERNATIVES
UPPER WEST FORK

DATE: JANUARY 2005

Figure No. 2-6

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Table 2-3
Preliminary Valley Storage Site Improvement Costs

Site Name	Property Acquisition	Demolition	Excavation	Fill	Export	Site Restoration	Utility Relocation	Grand Total
I	$135,000	$0	$655,000	$397,000	$2,338,000	$66,000	$0	$3,591,000
II	$633,000	$29,000	$355,000	$215,000	$0	$289,000	$42,000	$1,563,000
III	$1,712,000	$0	$1,184,000	$559,000	$948,000	$339,000	$0	$4,742,000
IV	$1,224,000	$0	$367,000	$223,000	$0	$255,000	$40,000	$2,109,000
V	$116,000	$0	$144,000	$23,000	$379,000	$69,000	$60,000	$791,000
VI	$85,000	$0	$643,000	$0	$2,297,000	$172,000	$0	$3,197,000
VII	$193,000	$0	$1,005,000	$624,000	$0	$348,000	$832,000	$3,002,000
VIII	$1,928,000	$159,000	$1,244,000	$0	$4,444,000	$300,000	$1,015,000	$9,090,000
IX	$2,806,000	$0	$307,000	$55,000	$771,000	$222,000	$398,000	$4,559,000
X	$2,168,000	$463,000	$772,000	$469,000	$2,759,000	$72,000	$54,000	$6,757,000
XI	$2,226,000	$927,000	$943,000	$640,000	$0	$471,000	$0	$5,207,000
XII	$82,000	$0	$686,000	$417,000	$2,450,000	$79,000	$13,000	$3,727,000
XIII	$254,000	$0	$658,000	$399,000	$2,349,000	$76,000	$17,000	$3,753,000
XIV	$225,000	$0	$1,266,000	$769,000	$4,523,000	$92,000	$133,000	$7,008,000
XV	$161,000	$0	$393,000	$238,000	$0	$239,000	$259,000	$1,290,000
XVI	$56,000	$10,000	$976,000	$0	$3,486,000	$143,000	$19,000	$4,690,000
XVII	$24,000	$0	$380,000	$231,000	$0	$55,000	$511,000	$1,201,000
XVIII	$1,709,000	$194,000	$1,395,000	$847,000	$0	$375,000	$675,000	$5,195,000
XIX	$233,000	$0	$593,000	$360,000	$0	$186,000	$826,000	$2,198,000
XX	$8,000	$0	$412,000	$250,000	$1,470,000	$47,000	$163,000	$2,350,000
XXI	$33,000	$0	$207,000	$126,000	$0	$55,000	$0	$421,000
XXII	$408,000	$19,000	$607,000	$369,000	$0	$289,000	$785,000	$2,477,000
XXIII	$6,000	$0	$339,000	$206,000	$1,211,000	$53,000	$147,000	$1,962,000
XXIV	$141,000	$0	$317,000	$192,000	$0	$73,000	$189,000	$912,000
XXV	$298,000	$0	$175,000	$106,000	$0	$72,000	$157,000	$808,000
XXVI	$220,000	$0	$147,000	$89,000	$524,000	$56,000	$473,000	$1,509,000
XXVII	$82,000	$0	$147,000	$89,000	$524,000	$56,000	$1,985,000	$2,883,000
XXVIII	$922,000	$0	$642,000	$390,000	$0	$216,000	$1,290,000	$3,460,000
XXIX	$101,000	$0	$10,000	$6,000	$36,000	$27,000	$88,000	$268,000
XXX	$149,000	$0	$26,000	$16,000	$93,000	$39,000	$0	$323,000
XXXI	$221,000	$0	$234,000	$143,000	$790,000	$61,000	$78,000	$1,527,000
XXXII	$303,000	$0	$304,000	$186,000	$1,002,000	$79,000	$345,000	$2,219,000
XXXIII	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
XXXIV	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
XXXV	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
XXXVI	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
XXXVII	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
XXXVIII	$12,193,000	$0	$1,064,000	$722,000	$0	$1,074,000	$1,237,000	$16,290,000
XXXIX	$2,943,000	$0	$276,000	$337,000	$0	$325,000	$891,000	$4,772,000
XL	$1,573,000	$0	$540,000	$26,000	$1,775,000	$411,000	$42,000	$4,367,000

N/A- Site replaced by Site XXXVIII.

Table 2-4
Preliminary Valley Storage Site Rankings

Site Name	Flood Storage AC-FT	Total Cost/ ac-ft $/ AC-FT	Total Cost Ranking
I	144.9	24,800	21
II	142.0	11,000	8
III	228.0	20,800	15
IV	81.0	26,000	24
V	77.0	10,300	6
VI	139.0	23,000	18
VII	323.2	9,300	4
VIII	275.0	33,100	28
IX	67.0	68,000	33
X	136.5	49,500	31
XI	418.8	12,400	9
XII	151.9	24,500	20
XIII	145.6	25,800	22
XIV	294.4	23,800	19
XV	86.9	14,800	13
XVI	216.0	21,700	17
XVII	84.1	14,300	11
XVIII	509.1	10,200	5
XIX	205.8	10,700	7
XX	91.2	25,800	23
XXI	45.8	9,200	3
XXII	168.1	14,700	12
XXIII	75.1	26,100	25
XXIV	70.1	13,000	10
XXV	38.8	20,800	16
XXVI	32.5	46,500	30
XXVII	32.5	88,800	34
XXVIII	208.0	16,600	14
XXIX	2.2	119,600	35
XXX	5.7	56,300	32
XXXI	51.9	29,400	26
XXXII	67.2	33,000	27
XXXIII	NA	NA	NA
XXXIV	NA	NA	NA
XXXV	NA	NA	NA
XXXVI	NA	NA	NA
XXXVII	NA	NA	NA
XXXVIII	3,246.0	5,000	2
XXXIX	979.0	4,900	1
XL	110.0	39,700	29

N/A- Site replaced by Site XXXVIII.

Section 3
Utility Relocation

3.1 General Description

3.1.1 Purpose

The purpose of this section is to provide information on the type and location of utilities which will be abandoned or relocated, establish design criteria for the relocation or abandonment, and provide initial estimated quantities.

3.1.2 Scope of Work

As a result of construction of the bypass channel and levee system and valley storage mitigation utilities located in the direct route and adjacent to the project area will require relocation or abandonment. The utilities currently located in the project area are predominantly a result of past development in the vicinity, with the exception of major sewer mains and electrical transmission mains. The following types of utilities are located within the project area:

- Water (Potable)
- Sanitary Sewer
- Storm Drainage
- Natural Gas (including service and high pressure distribution)
- Low Pressure Petroleum (Abandoned)
- Electric (including distribution and transmission)
- Cable Television
- Telephone service
- Fiberoptics

3.1.3 Criteria

The criteria used to determine if existing utilities require relocation or abandonment is based on the following:

- Underground utilities should be located a minimum of 3 feet below the bottom of the proposed bypass channel bottom.
- Underground utilities should not interfere with the structural integrity of the bypass channel levee system.
- Above ground utility poles should not be located closer than 15 feet from the top of the channel back slope.

- Aerial utility lines should be a minimum of 20 feet above the top of the levee for crossings.
- Utility crossings should meet requirement of USACE.
- Utility crossings should meet criteria established by Owner of utility.
- Pipeline crossings of the FW&WRR, which will be abandoned as a result of the project, will be required to be filled with concrete or a flowable fill acceptable to FW&WRR.   Crossing permits for abandoned crossings should be negated.
- New pipeline crossings of the FW&WRR will require new permits in accordance with the owner's requirements.

3.2 Design Requirements

The design requirements for utility relocations and abandonment consist of two major components. First, any modifications to utilities must meet the design and construction requirements established by the Owner of the utility. Secondly, the modifications must meet the design and construction requirements established by the USACE.

3.2.1 General Parameters

Listed below are the general parameters common to all utilities:

1. Design and construction shall meet the requirements established by the Owner of the utility.

2. The USACE criteria for levee and channel crossings shall be followed. Local criteria for additional construction within the limits of existing floodways (pamphlet No. 11650-2-1), prepared by the Fort Worth District, and Chapter 8 of the Design and Construction of Levees Manual (EM 1110-2-1913, 30 April 2000) will be used.

   Presented below is general criteria provided by the USACE. If the Owner of the utility has more stringent criteria, that should be followed.

   a. Overhead Wire Crossings:

      i. No structure (poles or otherwise) should be located closer than 10 feet from the toe of any levee.

      ii. No structure (poles or otherwise) should be located closer than 15 feet from the top of any channel back slope.

      iii. A minimum vertical clearance of 20 feet between the crown of the levee and the low wire at the low point of the wire at the levee crossing computed under the most adverse conditions (temperature, wind, load, etc.) should be provided.

iv. A minimum vertical clearance of 20 feet should be provided between the natural ground and the low wire at the low point of the sag in the area of the floodway channel, or three feet above the floodway design water surface level, which ever is higher. Electrical code criteria should also be met.

v. Guy wires and anchors should be located in such a manner that they do not interfere with the operation and/or maintenance of the channel, levees or related structures.

b. Gravity Storm and Sanitary Sewer Lines Under Levees:

i. A positive cut-off structure shall be located on the riverside of the levee crown. The cut-off structure shall extend to the levee crown elevation.

ii. Seep rings will be provided when required by ETL 1110-2-126, 28, June 1971.

iii. No pervious bedding shall be provided except as specified in ETL 1110-2-126.

iv. Gravity storm drains discharging into the floodway shall be provided with flap gates at the discharge end of the line and energy dissipaters, as required.

c. Pump Discharge Lines over the Levees

i. The invert of the discharge lines should be at the top of the protective works (levee) and shall be free-vented at the highest point.

ii. No flap gates will be provided on the end of the discharge lines.

iii. Energy dissipaters shall be provided, as required.

d. Crossings Under Rivers and Channels

i. A minimum of 3 feet between the river or channel bottom and the top of the pipe or conduit shall be provided.

ii. Where pipe encasement is used as an alternative to lowering the pipe, the top of encasement should be 1 foot below the lowest grade of the channel bottom.

e. Crossings over Rivers and Channels (utilities or bridges)

   i. A minimum free board of three feet between the low point of the crossing and the design water surface level.

   ii. Minimum obstruction of piers with no longitudinal cross-bracing.

f. General

   i. All manholes within the floodway having tops below design water surface level shall be provided with water-tight sealed manhole covers.

   ii. Any lines to be abandoned under existing levees are to be filled with concrete.

3. Major utility relocations for the bypass channel will occur before excavation of the channel and construction of the levees. Therefore, open cut (trench) methods of construction can be used. In the event that horizontal direction drilling is used in lieu of the open-cut method, the following USACE criteria should be used:

   Guidelines for Installation of Utilities beneath Corps of Engineers Levees Using Horizontal Direction Drilling, ERDC/GS: TR-02-9, June 2002, Geotechnical and Structures Laboratory.

4. The utility relocations and abandonment should not result in extended disruption of service to customers.

5. Customers should be notified a minimum of 24 hours in advance of any temporary shut down of utilities.

6. The replacement utility lines shall provide the capacity, pressure and level of service supplied by the original line, as a minimum.

7. Replacement lines shall be located in existing or future street right-of-way to minimized acquisition of easements.

8. Local, state, and federal regulations, codes and laws shall be followed.

In addition to these general parameters, each type of utility has specific functional and technical requirements, as discussed in the following section.

3.2.2 Functional and Technical Requirements

3.2.2.1 Potable Water

The City of Fort Worth Water Department owns and operates the water distribution system which will be affected by the construction of the bypass channel. Contained on Drawing GU-WA-1, Volume II, is an overall plan of the existing waterlines in the project area, the proposed bypass channel and levee system, and the relocated and abandoned waterlines. Larger scale drawings of the overall plan are shown on Drawings CU-WA-2 through CU-WA-5, Volume II. The following City of Fort Worth documents will be the basis for design of utility relocations and abandonment:

- General Contract Documents and Specifications for Water Department Projects (January 1, 1978); and

- Policy and Procedure for Processing Water and Wastewater Projects for Design and Construction (April 1999).

Existing water lines, 12 inches in diameter and smaller, in the project area will be abandoned by cutting and plugging the existing line. Abandoned water lines immediately adjacent to the bypass channel that have any pervious bedding material will have clay dams to prevent seepage from the bypass channel. In some cases the dead-end lines created by the abandonment, will be connected to adjacent water lines to create loops in the system. Existing water lines, larger than 12-inch in diameter, will be relocated appropriately, under the bypass channel.

Relocated waterlines crossing the levees and channel will be installed in steel casing pipe, or encased in concrete. Pipes will be restrained in the casing with casing spacers or hold down jacks. The annual space between the outside of the water pipe and inside of the steel casing may require grouting depending on the type of water pipe used. Air release valves may be needed on one or both ends of the cased crossing.

It is anticipated that the utility relocations will be constructed prior to the excavation of the bypass channel and construction of the levees. Therefore, the casing pipe and carrier pipe will be installed by open-cut (trench method) in lieu of a bored or tunneled crossing.

Presented below, in Table 3-1, is a listing of the new relocated pipes required for construction of the bypass channel.

Table 3-1
Water Pipeline Replacements

Diameter (inch)	Location	Length (feet)
20” Channel Crossing	North Commerce St.	1,000
16” Channel Crossing	White Settlement Rd.	870
12” Channel Crossing	North Main St. Rd.	1,290

3.2.2.2 Sanitary Sewer

The City of Fort Worth Water Department owns and operates the sanitary sewer system which will be affected by the construction of the bypass channel. Contained on Drawing CU-SS-6, Volume II, is an overall plan of the existing sanitary sewer line in the project area, the proposed bypass channel and levee system, and the relocated and abandoned sewer line. Larger scale drawings of the overall plan are shown on Drawings CU-SS-7 through CU-SS-10, Volume II. As with the water lines, the following City of Fort Worth documents will be the basis for design of utility relocations and abandonment:

- General Contract Documents and Specifications for Water Department Projects (January 1, 1978); and
- Policy and Procedure for Processing Water and Wastewater Projects for Design and Construction (April 1999).

The existing sewer lines in the project area convey sewage by gravity. Due to the depth of the bypass channel, the sewer lines will require a siphon system to relocate them adequately below the bypass channel bottom. Relocated sewer lines crossing the levees and channel will be encased in steel casing pipe, or encased in concrete.

Inside the casings, the carrier pipes will be restrained with casing spacers or hold down jacks. The annular space between the outside of the sewer pipe and inside of the steel casing will require grouting.

Presented below, in Table 3-2, is a listing of the new relocated pipes required for construction of the bypass channel.

Table 3-2

Wastewater Pipeline Replacements

Diameter (inch)	Location	Quantity
Segment:M-54 Two-Barrel Siphon		
15" siphon	North Calhoun St.	1,130 l.f.
21" siphon	North Calhoun St.	1,130 l.f.
2 Junction Boxes w/ Gates	North Calhoun St.	2 ea.
Segment:M-248 Two-Barrel Siphon		
39" siphon	NW 7th St.	1,150 l.f.
54" siphon	NW 7th St.	1,150 l.f.
2 Junction Boxes w/ Gates	NW 7th St.	2 ea.
Segment:M-545ext* Two-Barrel Siphon		
39" siphon	North of Henderson St.	800 l.f.
66" siphon	North of Henderson St.	800 l.f.
2 Junction Boxes w/ Gates	North of Henderson St.	2 ea.
Segment:M-439* Two-Barrel Siphon		
39" siphon	Arthur St.	1,050 l.f.
54" siphon	Arthur St.	1,050 l.f.
2 Junction Boxes w/ Gates	Arthur St.	2 ea.
Segment: Miscellaneous		
8" Sewer	NE 10th St & North Commerce St.	650 l.f.
8" Sewer	NW 7th St.	610 l.f.
12" Sewer	From Greenleaf St. to Viola	1,480 l.f.
24" Sewer	M-54 Siphon Connection	60 l.f.
54" Sewer	M-248 Siphon Connection	100 l.f.
66" Sewer	M-545ext* Siphon Connection	70 l.f.
54" Sewer	M-439* Siphon Connection	690 l.f.
Manhole	-	12 ea.

3.2.2.3 Storm Drainage System

The City of Fort Worth, Transportation and Public Works Department owns and operates the storm drainage systems in the project area. Contained on Drawing CU-ST-11, Volume II, is an overall plan of the existing storm drainage system for the project area, the proposed bypass channel and levee system, the relocated and abandoned storm drainage lines, and new outfall facilities. Larger scale drawings of the overall plan are shown on Drawings CU-ST-12 through CU-ST-15, Volume II. Existing storm drainage pipe sizes range from 24-inch to 72-inch in diameter. The construction of the bypass channel and levees will change the drainage patterns in the bypass channel project area. The following City of Fort Worth Public Works

Department documents will be the basis for design of utility relocations and abandonment:

- Storm Drainage Criteria and Design Manual (1967), including amendments; and
- Standard Specifications for Streets and Storm Drain Construction, including amendments.

The collection of storm water in the area includes sheet flow on the ground and in streets and some underground storm drainage conduits. All of the excess storm water is ultimately directed to the Trinity River. The area to the north and west of the project area drain by overland flow and storm drainage pipe systems towards the bypass channel. New storm drainage outfall structures are shown on Drawings CU-ST-12 though CU-ST-15 at the points where the existing drainage system will be severed. The outfall structures will transfer storm water from the existing system into the bypass channel, either through gravity systems, pumped system or a combination. The upstream conditions of the outfall structure and the discharge conditions in the bypass channel will need to be further evaluated during final design to determine the appropriate method of discharge.

Drainage in the peninsula area will drain into the proposed urban channels or interior section of the Trinity River located between the isolation gate structures. The isolation gate structures include Clear Fork Gate, Trinity Point Gate and TRWD Gate. The gates will separate the interior water feature and channels from the bypass channel and any potential flood events that would occur in the river. With the gates closed, the interior water feature and channel would be subject to localized flood events. The contributing area will drain approximately 612 acres, including some of the northern downtown Central Business District.

Final hydrologic analyses and operating controls will dictate the design capacity of the storm water pump station needed to protect the property adjacent to the isolated interior pool. Preliminary analysis indicates a pumping capacity of approximately 300 cubic feet per second (cfs) will utilize some of the interior pool's storage capacity, while retaining a relatively constant pool elevation, and accommodate a 24-hour, 100-year rainfall event.

The proposed location of the pump station is adjacent to the TRWD gate, located on the east side of the existing West Fork channel. This location will allow a short discharge length across the TRWD Gate levee. The intake structure from the river (sump) channel will provide two intake elevations (normal operations at elevation 513 and a sump draining operation level at elevation 505.5). Preliminary capacity of the pump station is 4 - 100 cfs pumps (one as a standby), which would operate at approximately 20 feet of total dynamic head, typical of a stormwater pump station. A single large discharge header directs the flow from the pump station across the levee to the river side, which will be at flood stage. Additional discussion on the storm water pump station is included in Section 9.

3.2.2.4 Natural Gas Distribution Systems

Natural gas service is extensive in the project area, and is currently provided by Atmos Energy. Atmos Energy announced the acquisition of TXU Gas, a wholly owned subsidiary of TXU Corp in June 2004. The gas service lines in the project area range in size from 2-inch to 24-inch diameter. Atmos Energy also owns and operates a pressure regulating station located on NE 9th St. between Main and Commerce Sts. The pressure regulating station is in the path of the bypass channel and levee system; therefore, will need to be relocated. Larger scale drawings of the overall plan are shown on Drawings CU-NG-17 and CU-NG-18, Volume II. Contained on Drawing CU-NG-16 is an overall plan of the existing gas lines in the project area and the proposed bypass channel and levee system. Presented below, in Table 3-3, is a listing of the relocated natural gas pipes required for construction of the bypass channel.

Table 3-3
Gas Pipeline Replacements

Diameter (inch)	Location	Length (feet)
24”	Main St. at Railroad	1,300
6”	North Commerce St.	1,340
4”	North Henderson St.	880
2”	North Henderson St.	530
6”	White Settlement Rd.	1,590
3”	White Settlement Rd.	980

3.2.2.5 Petroleum Gas Pipeline

A 20-inch diameter low pressure petroleum gas distribution line is located in North Calhoun St. The line is owned by Gulf South Pipeline, and has been abandoned in place. Abandonment of the line includes purging the line with nitrogen gas - cleaning the line with a pipe pig. Gulf South Pipeline is currently in the process of releasing the easement to the landowner. The line will need to be cut and plugged on each side of the proposed bypass channel.

3.2.2.6 Electrical Transmission and Distribution

Oncor Energy owns and operates the electrical transmission and distribution lines in the project area. According to Oncor staff, all final engineering, design and construction will be done by Oncor personnel. Contained on Drawing CU-EL-19, Volume II, is an overall plan of the existing electrical distribution and transmission lines in the project area and, the proposed bypass channel and levee system. Presented below are the tasks required to relocated and abandon 12,470 Volt and 138,000 V electrical lines:

- **For distribution lines (12,470V):** Poles, conductors and dead end existing lines will be removed along with existing underground lines and pad mounted transformers. New poles and conductors to backfeed specified line segments and to maintain service continuity where backfeed provisions are impossible will be installed. The bypass channel alignment will be bored wherever necessary for distribution lines to cross. Pole lines will be relocated as necessary to maintain electric service integrity during and after construction of the bypass channel.

- **For transmission lines (138,000V):** A new river crossing will be constructed, utilizing insulated cable at White Settlement Rd. Approximately 3,000 feet of three - conductor underground cable will be required along with termination areas, as needed for the high voltage cable installation.

Listed below are the distribution circuits, which will be affected by the relocation and abandonment of the affected electrical lines. Demolition of electrical lines are shown on Drawings CU-EU-20 through CU-EU-23, Volume II, and new lines are shown on Drawings CU-EU-24 and CU-EU-25, Volume II.

Circuit NMAIN/FDR0211

All overhead and underground facilities will be removed on Greenleaf St. to White Settlement Rd (just north of Kansas St.), this includes one underground service and pad mounted transformer. A new line reconnecting this feeder will be constructed on the west side of the levee.

All overhead facilities will be removed on Dakota St., east of Greenleaf St. and in the alley between Greenleaf St. and Arthur St. north of Dakota St. Since the bypass channel alignment will be in this area, and since there are no isolated circuit sections, these facilities will not be replaced.

All overhead facilities will be removed on Kansas St., between Greenleaf St. and Commercial St. The feeder to the eastern portion of this circuit will be maintained with a new bore under the bypass channel along Kansas St.

All overhead facilities will be removed along the south side of White Settlement Rd., between the FW&WRR crossing and Commercial St. The feeder will be replaced with a line and will be attached to the White Settlement Rd. Bridge, crossing the bypass channel.

All overhead facilities will be removed on Henderson St. between the FW&WRR crossing and North Commercial St. The main feeder will be replaced with an underground line under the bypass channel.

The single phase overhead feeder will be removed from the FW&WRR to the Calvert Ct. area. This feeder presently serves the fire and police department training area. Since most of this area will be taken by the bypass channel alignment, any new service will come from Woodward St.

Overhead distribution facilities will be modified near the area of the FW&WRR river crossing. Poles and lines will be removed, and will be replaced to accommodate the bypass channel alignment. Any river crossings in this area will be underground.

Circuit NMAIN/FDR0231:

All overhead and underground facilities will be removed along Throckmorton St. north of NW 6th St. and on NW 7th St. west of Houston St. Also removed will be facilities on Houston St. north of NW 7th St. and on NW 8th St. west of Houston St. Since the bypass channel alignment will be in this area, these facilities will not be replaced.

All overhead facilities will be removed along Calhoun St. just north of NE 8th St. and NE 10th St. The main feeder will be replaced with an underground feeder under the bypass channel.

Circuit NMAIN/FDR0252:

All overhead facilities will be removed along the north side of White Settlement Rd., between the FW&WRR crossing and Commercial St. The feeder will be replaced with an underground line and routed under new White Settlement Rd. Bridge. This circuit will parallel circuit NMAIN / FDR0211 which is on the south side of the street.

Circuit NMAIN/FDR0272:

All overhead facilities will be removed along Commerce St. between NE 8th St. and NE 10th St., and along NE 9th St. west of Commerce St. The main feeder will be replaced with an underground feeder under the bypass channel. The bore will be shared with circuit NMAIN / FDR0231, which runs along Calhoun St.

North Main-Calmont 138 kV Transmission Line:

This line is along the south side of White Settlement Rd., near the present river crossing into the switchyard at the abandoned TXU generating facility. This line will need to be rebuilt, due to the bypass channel location and land use. The rebuild will include the installation of insulated conductor. At the east end of that bridge, an underground termination area will be constructed, and the line will be routed in public right-of-way underground to the west end of the development area. An underground termination area will be constructed, and the insulated line will continue along the bridge structure at the new water feature into the switchyard. Alternatively, the underground portion will continue in a bore under the existing river channel, and the underground termination will be at the existing switchyard.

All electrical distribution facilities affected by the bypass channel realignment will be removed. This includes overhead primary lines, poles, underground electric lines (primary and secondary), transformers (pole mounted and pad mounted), guy wires, switches, capacitor banks and all associated pole hardware. This removal of facilities will be accompanied by the installation of new facilities such that electric service will be continued to all customers without disruption. Presented below in, Table 3-4, is a listing of the electrical utility removal and replacements for construction of the bypass channel.

Table 3-4
Electrical Removal and Replacements

Item	Quantity
Remove pole-mounted transformer	98
Remove pad-mounted transformer	4
Remove overhead conductors	38,500 ft.
Remove underground conductors	1,650 ft.
Remove pole	143
Install line pole	8
Install Riser pole	12
Install insulated conductors	5,200 ft.
Install overhead conductors	5,130 ft.
Bore	1,850 ft.
Cut & fill	100 ft.
Remove 138kV steel poles	10
Remove 138kV overhead conductors	3,000
Install termination areas	2
Install insulated conductors	3,000

3.2.2.7 Fiberoptics Cable

Charter Communications and Xespedius Communications own and operate the fiberoptics cable in the project area, which will be affected by the construction of the bypass channel. Contained on Drawing CU-FO-28, Volume II, is an overall plan of the existing fiberoptics line in the project area, the proposed bypass channel and levee system, and the relocated and abandoned fiberoptics line. A larger scale drawing of the overall plan is shown on CU-FO-29, Volume II. The fiberoptics line is aerial and will be relocated on the proposed bridge. Presented below, in Table 3-5, is a listing of the demolished and replacement fiberoptics lines.

Table 3-5
Fiberoptics Line Replacements

Item	Length (FTS)
Demolish Aerial Line at White Settlement Rd.	1,780
Replacement of Line at White Settlement Rd. Bridge	1,800

3.2.2.8 Cable TV

Charter Communications owns and operates the cable TV lines in the project area which will be affected by the construction of the bypass channel. The location of existing cable lines are shown on Drawing CU-CP-30, Volume II. Larger scale drawings of the overall plan are shown on Drawings CU-CP-31 and CU-CP-32, Volume II. Much of the cable lines are located on existing power poles. The relocation of the cable TV lines will follow the relocated electrical distribution lines. Presented below, in Table 3-6, is a listing of the demolished and replacement cable lines.

Table 3-6
Cable Line Replacements

Item	Length (feet)
Demolition	4,170
Replacement	2,590

3.2.2.9 Telephone

SBC owns and operates the phone lines in the project area, which will be affected by the construction of the bypass channel. Much of the telephone lines are located on existing power poles. The relocation of the telephone lines will follow the relocated electrical distribution lines.

3.2.3 Design Objectives

The design of the utility relocations and abandonment should include the following objectives:

- Meet the criteria established by the USACE for channel and levee crossings;
- Meet the criteria established by the Owner of the Utility;
- Meet or exceed current capacity, pressure and level of service; and
- Not result in extended disruption of service.

3.2.4. Calculations

All calculations will follow the requirements of USACE, Owner of Utility and standard industry practice.

3.2.5 Coordination with Outside Agencies

Coordination with outside agencies for the relocation and abandonment of utility lines will be minimum. However, all transmission line outages must be coordinated through ERCOT and the Texas PUC.

Section 4
Transportation Systems

4.1 General Description

4.1.1 Purpose

Construction of the bypass channel will affect the existing network of local streets, roads, bridges and other transportation elements in the project area. New bridge crossings, as well as the modification of existing roads, will be necessary to maintain existing traffic patterns throughout the project area. This section addresses the impacts to existing roadways due to the construction of the bypass channel. The future redevelopment within the project area and subsequent traffic needs are not addressed as part of this project.

4.1.2 Scope of Work

This preliminary design submittal for transportation identifies impacted roadways, relevant design criteria, and proposed mitigation of these roadway impacts. Modifications to the roadway network will be limited to maintaining and reconstructing portions of the major thoroughfares. Only minor modifications will be required on secondary side streets being terminated at the grading extents with temporary improvements.

It is anticipated that future redevelopment will reconfigure a significant number of the remaining minor roadways. Future transportation improvements will be coordinated with the Traffic Impact Study (prepared by others) and as supplemented by the City's own Impact Study which is being developed.

4.1.3 Criteria

The following criteria were used during the preliminary transportation system design:

- Conceptual drawings provided by the Urban Design Consultant.
- Maintain traffic flow on the major thoroughfares (Main St., Henderson St., White Settlement Rd.) during the project;
- Minimize impact to project area commercial/industrial businesses;
- Provide bridge crossings (lanes/sidewalks) suitable for redevelopment traffic needs;
- Minimize easement and right-of-way acquisition;
- Minimize vertical and horizontal changes in alignment;
- Adhere to Local, State and Federal Regulations, Codes and Laws;

- Adhere to City of Fort Worth Roadway Standards;
- Adhere to TxDOT Roadway Design Manual; and
- Adhere to TxDOT Bridge Development Manual.

4.2 Design Requirements

4.2.1 General Parameters

The current transportation system is primarily oriented toward the movement of truck and automobile traffic, with minimal pedestrian facilities. A single set of active railroad tracks are located within the project area west of the proposed bypass channel. This set of tracks serves an excursion tourist passenger train and some local freight traffic. Additional spur railroad track facilities from this line are present which do not appear, through current research, to be in service. A public transit within the area is limited to the Main St. corridor. The current transportation system does not provide well-kept facilities for pedestrian mobility, with the exception of the arterial streets in the project area. Transportation improvements will need to be coordinated with the Traffic Impact Study (prepared by others) and as supplemented by the City's own Impact Study, which is in the process of being developed.

4.2.2 Design Objectives

The design objective for the existing and proposed transportation systems located within the project area is to maintain the existing transportation network as much as possible, while providing the bridge crossing facilities over the new bypass channel to handle projected future traffic needs. The replacement of existing roadways will be "in kind" to the existing facilities as they are intended to be temporary until additional upgrades are made to the interior area as part of the redevelopment.

4.2.3 Functional and Technical Requirements

4.2.3.1 Roadway Classification

Construction of the bypass channel will impact two designated State highways and one major City roadway. Existing roadways in the project area are shown on Figure 4-1. Classification of the streets to be impacted in the project area was made using the City of Fort Worth's "Proposed Street Development Standards, Roadway Standards and Master Thoroughfare Plan", February 2002 as defined below:

- Principle Arterial: The main function of principal arterial streets is to carry traffic within the community and between major activity centers of the region. The principal arterial street system carries most of the traffic entering and leaving the urban area, as well as most of the through movement bypassing the central city. Principal arterials carry 30,000 to 45,000 vehicles per day (vpd) and serve high-density residential, retail, service, and industrial uses.
- Major Arterial: The major arterial street system connects with the principal arterial system to accommodate trips of moderate length with a lower level of travel

mobility and a higher level of land access. The major arterial street system distributes trips to geographic areas and serves major commercial and industrial districts. Such facilities may carry local bus routes and provide inter-community continuity, but should not penetrate identifiable neighborhoods. Major arterials are generally designed to carry 15,000 to 35,000 vpd.

- Minor Arterial: Minor arterials are commonly located along neighborhood borders and collect traffic from residential areas and direct vehicles to the major arterial system. These streets are designed to carry 4,000 to 20,000 vpd.

- Industrial streets: Industrial streets are located in Industrial-zoned areas to recognize different types of vehicles with larger turning radii and heavier industrial type traffic. These roadways are basically minor arterials that route industrial vehicles from the arterial system to and within industrial districts.

- Residential streets: Residential streets serve traffic within neighborhoods and should carry low traffic volumes, 200 to 4,000 vpd at slower speeds. There are three types of residential streets: collector, local, and limited local. These streets are used in subdivisions based on varying sizes and numbers of residential lots.

Presented on Table 4-1 is the preliminary classification and listing of the streets to be impacted by the bypass channel alignment.

Table 4-1

Streets to be Impacted by Bypass Channel Alignment Route

St. Name	Preliminary Classification
Arthur St.	Industrial
Calhoun St.	Industrial
Calvert St.	Industrial
Calvert Ct.	Industrial
Commerce St.	Industrial
Commercial St.	Industrial
Cullen St.	Industrial
Dakota St.	Industrial
Greenleaf St.	Industrial
Henderson St. (Hwy. 199)	Principle Arterial
Houston St.	Industrial
Kansas St.	Industrial
Lexington St.	Residential
North Main St. (Hwy. 496)	Major Arterial
N 7^{th} St.	Industrial
N 8^{th} St.	Industrial
N 9^{th} St.	Industrial
N 10^{th} St.	Industrial
Refinery St.	Industrial
Rockwood Lane	Residential
Rockwood Parkway	Residential
Rupert St.	Residential
Throckmorton St.	Industrial
White Settlement Rd.	Minor Arterial
Viola St.	Industrial

In addition to the streets impacted by construction of the bypass channel, University Dr. will be improved as part of the valley storage mitigation work. Lexington St. will be improved by construction of the earthen embankment for the Clear Fork isolation gate. The modifications to University Dr. subsequently impact the connecting roadways of Rockwood Lane and Rockwood Parkway.

4.2.3.2 Roadway Impact

Main St., Henderson St., White Settlement Rd., and University Dr. are the roadways of main concern for maintaining traffic flow, during and after the construction of the bypass channel and valley storage mitigation, as they are arterial streets and carry the largest volume of traffic within the project area. Therefore, these roadways will be maintained with new bridge crossings over the bypass channel, with the exception of University Dr. The bridge crossings are further discussed in Section 4.2.3.4 and Section 5 of this report. Modifications to University Dr. are discussed further in Section 2.2.2.5.

The construction of the new bridges on Henderson St., Main St. and White Settlement Rd. will require the construction of temporary detours during construction. Detours will be sized and located to carry equivalent traffic volumes.

The remaining impacted streets are considered local streets and have very limited traffic volumes. Given the potential future redevelopment within the bypass channel project area, significant investments in new roadways in this area are not warranted as part of this project. Hence, the remaining local streets impacted by the bypass channel will be terminated in one of the following manners:

1) Terminated with a cul-de-sac; or

2) Terminated at the closest intersection.

Roadways which are severed by the bypass channel and have active parcels (parcels which will not be acquired as part of the project) beyond the closest continuous intersection will be terminated with a cul-de-sac to allow continued access and suitable turning radii for emergency vehicles. A minimum outside radii of 50 feet is applied in commercial and industrial areas.

Roadways that do not have any active parcels or driveways beyond the nearest continuous intersection will be terminated at the intersection with the remainder of the street vacated/abandoned and pavement removed prior to construction of the bypass channel.

Roadways impacted as a result of the University Dr. valley storage mitigation work will be fully reconstructed and restored to service.

Illustrated on Figure 4-2, shown on Drawing CP-1, Volume II, and summarized below, is a preliminary reconfigured roadway system based on the current bypass channel alignment.

The following streets will be reconnected across the bypass channel with new bridge crossings:

- Henderson St. (State Highway 199);
- Main St. (State Highway 496); and
- White Settlement Rd.

Henderson St. and Main St. will maintain their existing horizontal alignments but require significant modifications in vertical alignment. White Settlement Rd. is proposed to maintain its existing horizontal alignment over the bypass channel, but the approach on the east side of the channel will include a curve to the northeast until it intersects with Henderson St., at a perpendicular angle roughly 500 feet southeast of the existing Commercial St. intersection with Henderson St. Traffic volumes on White

Settlement Rd. and Henderson St. will necessitate the need for traffic signals at this new intersection location. The bridge crossings at these locations are further discussed in Section 4.2.3.4 and Section 5 of this report.

In addition railroad separations are proposed at White Settlement Rd. and Henderson St. The bridges over the bypass channel will be extended over the FW&WRR to eliminate the at-grade crossings and improve safety.

The following streets will be terminated at the bypass channel with a cul-de-sac:

- Kansas St. near Viola St.;
- Cullen southwest of Henderson St.
- Commerce St. southeast of 10th St. (west of bypass channel);
- Rupert St. south of White Settlement Rd.; and
- Throckmorton St. southeast of 7th St. (east of bypass channel).

The following streets will be terminated at the nearest intersection:

- Calvert Ct. at private roadway;
- Calvert St. at private roadway;
- Calhoun St. at 8th St. (east of bypass channel);
- Calhoun St. at 10th St. (west of bypass channel);
- Commerce St. at 8th St. (east of bypass channel);
- Dakota St. at Greenleaf St.;
- Kansas St. at Greenleaf St. (west of bypass channel);
- Lexington St. at W Peach St.;
- Houston St. at 7th St.;
- White Settlement Rd. (existing) at Viola St.;
- 8th St. at Commerce St.; and
- 7th St. at Houston St.

The following streets will be vacated due to their direct location within the grading limits of the bypass channel/ Clear Fork gate:

- Dakota St. (between Greenleaf St. and Arthur St.);
- Kansas St. (between Greenleaf St. and Commercial St.);
- Arthur St. (between Dakota St. and White Settlement Rd.);
- Commercial St. (between Kansas St. and Henderson St.);
- Portions of Calvert Court;
- Portions of Calvert St.;
- Greenleaf St. (north of Kansas St. to White Settlement Rd.);
- Lexington St. (north of Peach St.);
- Throckmorton St. (north of 6th St. to 8th St.);
- Houston St. (north of 7th St.);
- 7th St. (between Throckmorton St. and Houston St.);
- 8th St. (between Throckmorton St. and Houston St.);
- 8th St. (between Main St. and Commerce St.);
- 9th St. (between Main St. and Calhoun St.);
- Refinery St. (between Main St. and Houston St.);
- Commerce St. (north of 8th St. to south of 10th St.); and
- Calhoun St. (between 8th St. and 10th St.).

4.2.3.3 Roadway Standards

Roadways which are replaced with a cul-de-sac will incorporate a pavement design “in-kind” with the adjacent existing pavement cross section. All new roadways associated with the University Dr. mitigation and bridge crossings will meet City of Fort Worth minimum pavement standards as provided in Table 4-2. Texas DOT Roadway standards apply to Henderson St. (Hwy. 199) and Main St. (Hwy. 496).

Table 4-2
City of Fort Worth Minimum Pavement and Cross-Sections

Street Classification	Construction Standard
Principle Arterial and Industrial Streets	8" Reinforced Concrete 6" Stabilized Sub grade
Major and Minor Arterial Streets	7" Reinforced Concrete 6" Stabilized Sub grade
Local, Collector and Private Streets	6" Reinforced Concrete 6" Stabilized Sub grade
	6" H.M.A.C. 8" Stabilized Sub grade

The City of Fort Worth requires sidewalks on both sides of new streets, 4-feet minimum, or 5-feet minimum if adjacent to curb. The new roadways at Main St., Henderson St. and White Settlement Rd. will be reconstructed to City of Fort Worth standards with pedestrian sidewalks which are ADA compliant.

4.2.3.4 Bridges

Bridge structures crossing the bypass channel are proposed at Main St., Henderson St., and White Settlement Rd. Approach grades to the structure will not exceed 5% and will maintain minimum clearance of 4 feet over the SPF water surface elevation. Proposed bridge structures will maintain a minimum clearance of 24 feet from the railroad to the superstructure of the bridge deck.

The proposed bridge structures will have some bridge piers located within the bypass channel, which have been incorporated into the hydraulic model. During preliminary design these structure will be further defined and final bridge types, pier locations, span lengths, and impacts to adjacent structures will be defined.

4.2.3.5 Railways

The FW&WRR has a single set of tracks located along the western limits of the bypass channel construction corridor.

The following at-grade railroad crossings will be removed and replaced with a vehicular overpass as part of the roadway bridge crossings:

- Henderson St. (State Highway 199); and
- White Settlement Rd.

The following at-grade railroad crossings will not be impacted by construction of the bypass channel:

- Main St. (State Highway 496);
- Commerce St.; and

- Calhoun St..

Additional spur railroad track facilities are present within Commerce St. and Houston St. right-of way, which do not appear to be in service. Further investigation of these facilities will be included as part of preliminary design.

4.2.4 Coordination with Outside Agencies

The design of the various transportation components will require coordination with outside agencies including, but not limited to, the City of Fort Worth, impacted private utility carriers, and railroad operators. Due to Henderson St. and Main St. having state highway designations, design coordination with the Texas Department of Transportation (TXDOT), as well as the City, will be required.

4.3 Operation and Maintenance

The operation and maintenance of the proposed transportation system reconfiguration will not change from the existing operations and maintenance responsibilities.

4.3.1 Roadway System

In general, the City of Fort Worth is responsible for the maintenance of the transportation system pavement, signage, signals, and repair of the roadways within the project area. After implementation of the proposed project, the maintenance and operation responsibilities of the roadway system will remain with the City of Fort Worth.

TARRANT REGIONAL WATER DISTRICT

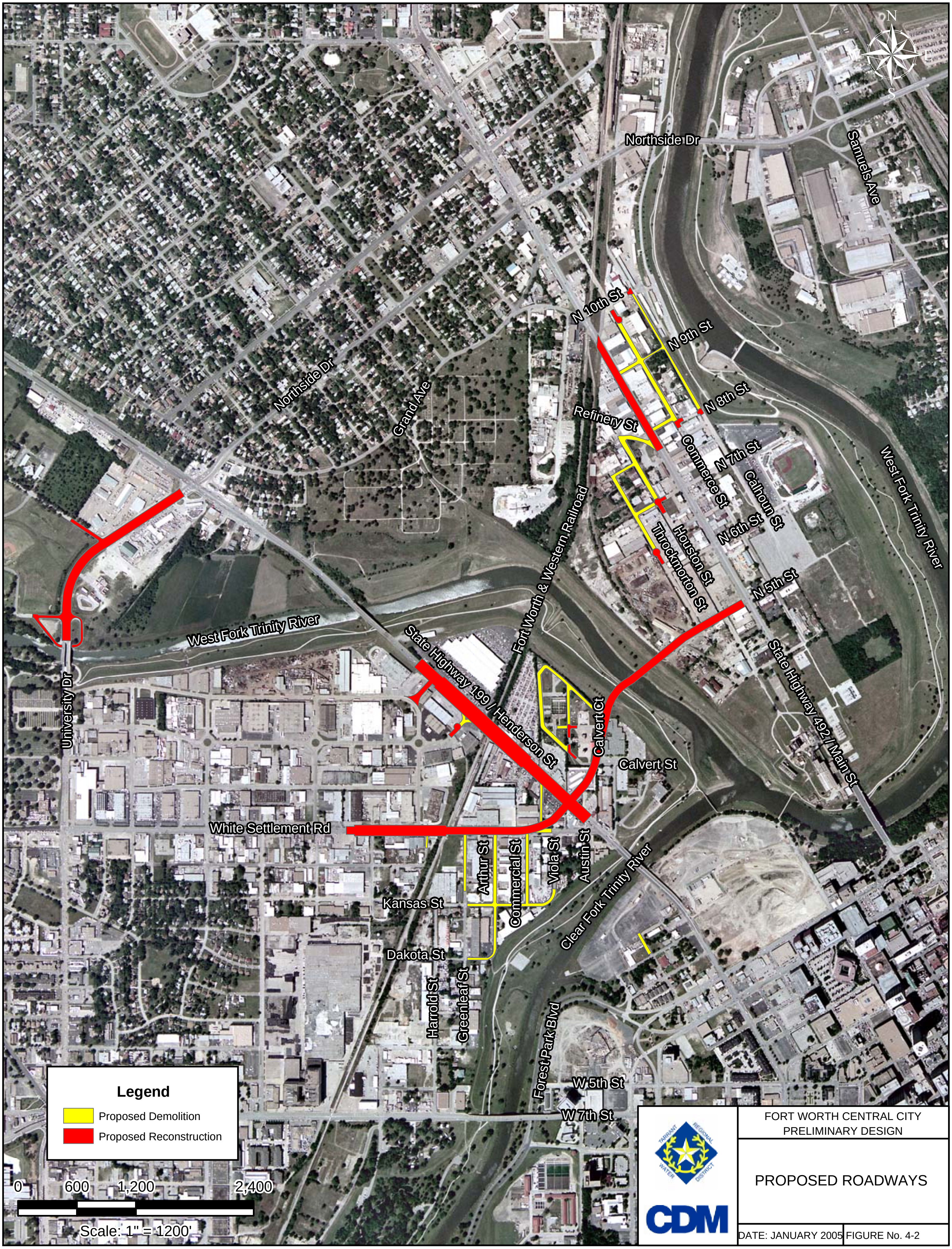
CDM

FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

EXISTING ROADWAYS

DATE: JANUARY 2005

FIGURE No. 4-1



FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

PROPOSED ROADWAYS

DATE: JANUARY 2005 | FIGURE No. 4-2

Section 5
Bridge Structures

5.1 General Description

This section provides a preliminary level analysis of the bridge structure crossings at the proposed bypass channel along North Main St., Henderson St., and White Settlement Rd. The analysis includes preliminary profile grades required to meet various design constraints at each crossing locations. Bridges along Henderson and White Settlement are planned to receive aesthetic enhancements to a "medium-high" level, while the North Main St. Bridge is planned as a "signature" bridge with a cable-stayed superstructure. The various bridge structure traffic configurations are planned in accordance with the latest version of the City of Fort Worth Master Thoroughfare Plan. Embankment and retaining wall options of placing fill for the approach roads on the west side of the bypass channel are reviewed and discussed. A combination of detours, street closures and temporary roads will be required to construct these three bridges. Also, temporary transition roads may be required to continue access after completion of the bridge and prior to final grading of the interior area east of the bypass channel. In addition to impacts during construction, this section discusses the impacts the new bridges and retaining walls/embankment will have on future access and traffic circulation.

5.1.1 Purpose

The purpose of this section is to describe, at a preliminary level, the bridge structures that are required to maintain the existing street network over the proposed bypass channel. At this level, the preliminary type, size, location and associated features of the bridge structures are identified. In addition, impacts to adjacent property, traffic and access are also discussed. Required approach roads and associated geometrics are also analyzed.

5.1.2 Scope of Work

The scope of work for the bridge structure and approach road portion of the project has four main objectives: 1) identify controlling criteria, constraints, and parameters, 2) determine preliminary type, size, and location, and 3) analyze approaches, grades, embankment, and/or retaining wall options. The three new bridge structures over the proposed bypass channel are:

1. North Main St. (Business US 287/ State Highway 496)

2. Henderson St. (State Highway 199)

3. White Settlement Rd.

5.1.3 Criteria

The following criteria were identified as controlling the preliminary structure planning:

- Conceptual drawings and aesthetic directives, provided by the urban design consultant;
- AASHTO Bridge Specifications (LRFD);
- TxDOT Guidelines, Requirements and Practices (for Henderson and North Main);
- Constructability of the bridge structures in coordination with the bypass channel;
- Limit impacts, to the maximum extent possible, in order to preserve future development opportunities;
- ADA requirements, including limiting approach roadway grades to 5% maximum;
- Availability of Easements and Right-of-Way;
- Local, State and Federal Regulations, Codes and Laws;
- Preservation of Historic Structures;
- Trinity Point Redevelopment Conceptual Plans; and
- USACE requirements.

5.2 Design Requirements

5.2.1 General Requirements

The proposed bypass channel levee system will be constructed at a higher elevation than the existing ground, interrupting the existing street network. The three bridges will maintain critical connections across the bypass channel to the north and west. The proximity of the bypass channel to the existing railroad tracks at Henderson and White Settlement does not allow for at-grade crossings due to the top of levee elevation. At these two crossings, the bridges will span over the FW&WRR in addition to the bypass channel. Several factors control not only the length of the bridge but also the spans between columns, the height of the bridge, and the approach grades. These factors are discussed in further detail later in this section.

5.2.2 Aesthetic Requirements

The project urban consultant determined that the North Main St. Bridge should be a "symbolic" or "gateway" type structure; preferably featuring a cable-stayed superstructure. The Henderson and White Settlement bridges were recommended to be of "medium-high" aesthetic interest. The urban consultant also determined that vertical bridge supports should be avoided in the main channel, but could be included between the main bypass channel edge and the levee or hard edge retaining walls. Subsequently, the bridge supports were located to blend into the bypass channel soft edge/hard edge configuration.

Aesthetics for the bridge substructure, superstructure, and roadway surface will be coordinated with the treatment of the bypass channel edges. In general, supports will be placed in the overbank of the levees on the soft edge side and near the lower wall on the hard edge side. The cable-stayed North Main St. structure will have two large supports located off-center in the soft edge levee.

5.2.3 Hydraulic Requirements

The bypass channel will likely be constructed after the completion of the bridges to minimize the impacts to motorists and transit. Therefore, the usual hydraulic requirements for bridge structures during construction and waterway flow interaction for a new crossing will not be necessary. However, Coordination between the structural and hydraulic design efforts will be critical during the final design process to ensure that all hydraulic considerations are met. Although conceptual pier sizes and locations have been determined, final pier sizes and locations will have to be incorporated into the hydraulic model. Note that the decision to place bridge piers outside the main channel, as mentioned above, is based on aesthetics developed specifically for this project and not specifically hydraulic considerations. General practice for bridges crossing waterways in North Texas is to place piers in the channel, incorporate the bridge characteristics into the hydraulic modeling and still maintain economical bridge span lengths. A design constraint controlling the vertical alignment and height of the bridge requires that the low chord of the bridge be constructed at an elevation above the SPF.

5.2.4 Geometric Requirements

All three streets will maintain their existing horizontal alignment over the bypass channel. Bridge construction will incorporate the City's Master Thoroughfare Plan's street classifications and be constructed to the ultimate configuration including number and widths of lanes and acceptable sidewalk widths. Turning lanes, however, would not be carried across the bridge structures.

Preliminary vertical profiles are set to accommodate a design speed of 35 mph. ADA requirements for new construction with sidewalks limit the approach grades to 5.0% maximum. As part of the urban design characteristics, each bridge section incorporates 10' wide sidewalks on each side. The vertical clearance (24 feet) required over the railroad tracks for Henderson and White Settlement increases the length of their approaches on the west side of the bypass channel. On the east side of the channel, it is assumed that the areas behind the hard edge of the levee will be filled in with embankment above the existing ground level so that the future street level will be closer to the top of levee. The proposed profiles reflect the described scenario as shown on Drawings CP-2, CP-3 and CP-4, Volume II.

5.2.5 Structural Requirements

All new bridge structures will be designed to meet current AASHTO specifications and TxDOT requirements, as applicable. Beginning in 2007, all bridge design will be governed by AASHTO's Load and Resistance Factor Design (LRFD). Preliminary calculations and structural component sizing in this section incorporated the LRFD method. Specific bridge types and preliminary support sizes are discussed further in Section 5.3.

5.2.6 Coordination with Outside Agencies

As the design progresses and project funding is identified, extensive coordination will be required with the City of Fort Worth and TxDOT regarding the bridge structures. In addition, coordination will be required with the railroad and permits must be obtained prior to construction.

5.3 Preliminary Type, Size & Location

Preliminary designs defining structure type, size and location for the three structures have been developed based on the requirements outlined above. Profiles are shown on Drawings CP-1, CP-2, and CP-3, Volume II. Typical deck sections are shown on Drawing CP-4, Volume II.

5.3.1 Main Street

The Main St. Bridge will have an asymmetrical cable-stayed superstructure. Cable-stayed bridges feature a fan arrangement of straight cables supporting the deck suspended from tall piers. The bridge is currently anticipated to have two columns that "frame" the downtown skyline. Each column will have an approximate height of 200 feet. The bridge will have four traffic lanes and two 10′ wide sidewalks / bike paths. The piers will be placed off-center just east of the soft edge levee. The main span over the channel will be 297 feet long with a 109 foot back span for a total length of 406 feet. The south abutment will be at the top of the hard edge levee. Drawing CP-2, Volume II shows a preliminary profile of the Main St. Bridge.

As a symbolic or gateway structure, this bridge will be a prominent feature of the project when viewed from any angle, but especially approaching along North Main from Paddock Viaduct or from the Stockyards. Other aesthetic treatments, pedestrian features and lighting will be defined as the design progresses.

5.3.2 Henderson Street

The Henderson St. Bridge over the bypass channel will be situated between existing bridges that will remain over the Clear Fork and the West Fork of the Trinity River. Although a definitive structure type has not been determined, this report models a superstructure consisting of trapezoidal steel girders with a concrete deck for its clean sight lines and spanning capability. Preliminary calculations indicate approximate column sizes of 4′ x 8′ with a structure depth of 7′ based on the required spans. Since Henderson is classified as a Principal Arterial based on the 2004 Master Thoroughfare

Plan, six traffic lanes are provided. It should be noted that the existing street has four lanes at various locations, and in some areas fewer than four lanes. A separate traffic study may justify fewer lanes on this bridge.

The northwest approach roadway will begin approximately 1,000 feet north of the FW&WRR tracks. This distance is required to provide adequate clearance over the railroad tracks and to conform to current ADA standards for the maximum longitudinal grade of 5%. The bridge spans both the railroad and proposed channel. Preliminary pier locations and span lengths are shown on Drawing CP-4, Volume II. The total length of the bridge is estimated to be 700 feet. Aesthetic treatments would be added to enhance the visual appeal of the structure.

5.3.3 White Settlement Road

The new structure carrying White Settlement Rd. over the bypass channel is similar to the Henderson Bridge described above. Bridge columns, like the Henderson Bridge, are approximately 4′ x 8′ in size. The anticipated structure depth is approximately 7′. Grade differences and clearances similar to the Henderson Bridge will require the approach to start approximately 1,000 feet west of the railroad. The span arrangement and structure type for the White Settlement Bridge are similar to Henderson. Preliminary pier locations and span lengths are shown on Drawing CP-4, Volume II. The total length of the bridge is estimated to be 735 feet. This bridge will have four traffic lanes and 10′ wide sidewalks.

5.3.4 Bridge Approach Roads

This section addresses the roads which begin at the onset of the positive grade change leading up to the connection with each bridge. These "approach" roads end at the bridge and are typically located within the limits of the embankment or retaining walls. It is assumed that redevelopment behind the hard edge or east side of the bypass will substantially raise the ground elevation; therefore, the final approach road embankment for each of the three bridges east of the levee at completed ultimate anticipated build-out is minimal. However, should the bridges be constructed prior to completion of the mass fill operations a significant amount of embankment along with temporary pavement will be required to allow for continuous unimpeded traffic on an interim basis. Upon completion of the fill operations and as the roads east and south of the levee are brought to final grade, the temporary pavement may be replaced with the permanent pavement structure. Drawings CP-2, CP-3, and CP-4, Volume II depict the ultimate conditions. Due to the required approach length on the west side and associated vertical clearances over the railroad tracks, fill heights, both temporary and permanent, approach 25′ near the bridge structure. As a result, a significant volume of fill will be required for the approach roads. Two alternate scenarios were reviewed for the permanent approach roads: (1) the use of embankment with 4:1 side slopes and (2) the use of retaining walls. Only the use of embankment with 4:1 side slopes was reviewed for temporary roads.

Embankment, utilizing typical 4:1 sideslopes, requires approximately twice the amount of fill as required for the retaining wall option, as shown in Table 5-1 below. Note that this table is for the ultimate condition only.

Table 5-1
Fill Volume

Road	Embankment (4:1) Option (cy)	Retaining Wall Option (cy)
North Main	12,100	7,600
Henderson	47,500	26,000
White Settlement	38,600	20,800
Total	***98,200***	***54,400***

In addition to requiring a significant amount of additional fill, the embankment option requires a much larger footprint and acquisition of additional ROW. Although additional ROW may be desired to accommodate an upgraded roadway section, implementation of the embankment alternative would require additional ROW acquisition from several property owners. In addition to the ROW required, the embankment alternative would encompass several existing structures, requiring the purchase and demolition of these structures. Table 5-2 below shows the additional ROW area required and the number of structures potentially affected. Note that, due to the anticipated amount of redevelopment south and east of the levee, Table 5-2 is only for the west and northwest approaches.

Table 5-2
Embankment Option

Road	Additional ROW Area Required (ac)*	Existing Structures
North Main	0.8	3
Henderson	1.9	2
White Settlement	1.7	5
Total	***4.4***	***10***

* Note: Additional ROW area assumes a 10′ parkway area from the toe of the embankment slope to the edge of the ROW.

5.3.4.1 North Main Street Approach Road

The northwest approach to the new bridge begins near the intersection with the UPRR and FW&WRR track ROWs. The vertical alignment transitions from an approximate grade of -1.7% up to the maximum allowable grade of 5.0% through a vertical curve. The total length of approach, requiring fill, is approximately 430 feet. The approach road reaches approximately 15 feet in height. The embankment option results in a footprint of 220 feet in width at the beginning of the bridge. The retaining wall option results in a footprint 76 feet in width (2-12′ lanes, 2-15′ lanes, 2-10′ sidewalks & 2-1′ guardrails).

From the maximum height of the bridge, the grade changes from 5.0% to -5.0% through a vertical curve and continues at this grade until it reaches the proposed future roadway elevation. The southeast approach road to the new bridge is approximately 500 feet in length with a maximum height of 21 feet based on the interim and ultimate scenarios.

5.3.4.2 Henderson Street Approach Road

The northwest approach to the new bridge will begin approximately 1,000 feet north of the FW&WRR tracks. The vertical alignment transitions from an existing approximate grade of -3.9% up to the maximum allowable grade of 5.0% through a vertical curve. The new bridge begins approximately 230 feet northwest of the railroad tracks. The total length of approach, requiring fill, is approximately 600 feet. The approach road reaches approximately 25 feet in height. The embankment option results in a footprint 300 feet in width at the beginning of the bridge. The retaining wall option results in a footprint 100 feet in width (4-12′ lanes, 2-15′ lanes, 2-10′ sidewalks, & 2-1′ guardrails).

Traveling southeast across the bridge, from the maximum height of the bridge, the grade changes from 5.0% to -3.4% through a vertical curve and continues at this grade until it reaches the proposed future roadway elevation. The southeast approach road to the new bridge is approximately 275 feet in length with a maximum height of 7 feet based on the ultimate scenario. The interim approach road is much like the northwest approach, as described above, in length and height. It is anticipated that this temporary road will be constructed at the maximum allowable slope or with the use of a retaining wall in order to minimize the impacts to the surrounding areas and also to minimize the amount of required pavement and embankment to be constructed. At this time the interim approach has been included as the preferred approach option, as it minimizes the impact to private property.

5.3.4.3 White Settlement Approach Road

The west approach to the new bridge begins approximately 1,000 feet west of the railroad tracks. The vertical alignment transitions from an approximate grade of 1.2% up to the maximum allowable grade of 5.0% through a vertical curve. The new bridge begins approximately 250 feet west of the railroad tracks. The total length of approach, requiring fill, is approximately 750 feet. The approach road reaches approximately 25 feet in height. The embankment option results in a footprint 300 feet in width at the beginning of the bridge. The retaining wall option results in a footprint 76 feet in width (2-12′ lanes, 2-15′ lanes, 2-10′ sidewalks & 2-1′ guardrails).

Traveling east across the bridge, from the maximum height of the bridge, the grade changes from 5.0% to -2.8% through a vertical curve and continues at this grade until it reaches the proposed future roadway elevation. The southeast approach road to the new bridge is approximately 275 feet in length with a maximum height of 7 feet based on the ultimate scenario. The interim approach road is much like the west approach, as described above, in length and height. It is anticipated that this temporary road will be constructed at the maximum allowable slope on retaining walls in order to minimize the impacts to the surrounding areas and also to minimize the amount of required pavement and embankment to be constructed. At this time, the interim approach has been included as the preferred approach option, as it minimizes the impact to private property.

5.4 Construction Sequencing and Traffic Control

Because each of these streets is heavily used and available ROW is limited, bridge construction should be staged in a manner that will minimize traffic impacts.
The three bridge structures planned for the bypass channel will be constructed prior to the construction of the bypass channel. By installing the bridges before the construction of channels, traffic can be more easily detoured until the bridges are complete. Traffic can then be moved to the completed bridges during the construction of the bypass channels. In order to maintain an aggressive construction schedule, it is assumed that all three bridges will be under construction at essentially the same time period.

This section presents traffic control alternatives to be implemented during the construction of each bridge. The options presented below assume that the bridges are constructed prior to the ultimate fill condition of the interior area. It should be noted that the tie-in of the roadway and the footprint resulting from the embankment shown on Drawings Cp-1, CP-2, and CP-3, Volume II, assumes the ultimate fill condition on the east side of the bypass channel.

It should also be noted that when the island side of the bypass channel is filled with embankment, detours will be required for all the affected roadways within the fill area. The location and extent of the detours will depend on the phasing of the construction of the interior area. The temporary approaches for the three bridges will have to be reconstructed to their final configuration, and during the reconstruction, the traffic for the three roadways will have to be detoured again.

5.4.1 Main Street

If the Main St. Bridge is to be of the cable-stayed type, it will not be possible to maintain traffic along the existing North Main St. ROW during construction. Existing parallel streets however provide a reasonable detour alternative. While Main St. is closed for construction between Northeast 7th St. and Northeast 11th St., northbound traffic could be detoured to Commerce St. via Northeast 7th St. The detour could extend along Commerce St. up to Northeast 11th St. where traffic could then be diverted back onto North Main St. Southbound traffic along Main St. could be diverted to Commerce St. via Northeast 11th St. thereby utilizing the same detour route as northbound traffic.

The current configuration of Northeast 7th St., Northeast 11th St., and Commerce St. accommodates one lane of traffic in each direction with parking lanes on each side. Utilization of this detour route will most likely require upgrades in the form of surface overlays and/or pavement and subgrade improvements to accommodate the increased traffic volume, but the width of the existing streets appears sufficient to allow the four lane traffic currently on Main St., with the elimination of the parking areas along the route. A significant benefit of this route is that it uses an existing FW&WRR crossing.

In the event that a <u>non</u>-cable stayed bridge is desired at this location, similar to the ones proposed at Henderson and White Settlement, it may be possible to facilitate construction-phase traffic handling by building the bridge in halves (e.g., first the NB lanes, then the SB lanes). This would provide room to maintain at least 2 lanes of traffic on North Main St. at all times. If the level of traffic along North Main dictates the need to maintain 4 lanes, it would be possible to use North Main and North Commerce as two one-way pairs.

5.4.2 Henderson Street

Because Henderson St. is heavily used and provides an important link to downtown from the NW portion of Fort Worth, it is assumed that it will be essential to maintain 4 lanes of traffic during construction. Although the trapezoidal bridge and approach roads planned at this location could be constructed one-half at a time, the construction time and cost would be greatly reduced if a 4-lane detour around the construction area is utilized.

The detour could be constructed to the east of Henderson St. on the site currently used for the Henderson St. Bazaar. This site is planned to be purchased and used as a spoil disposal site for excess materials excavated during the construction of the bypass channel and renovated as part of the Trinity Uptown Plan. This portion of the detour would allow for construction of the northwestern approach of the Henderson St. Bridge. From the Henderson St. Bazaar property, the detour could then be extended to the south, crossing the FW&WRR using the existing railroad crossing to the extent possible. Securing an additional railroad crossing would most likely prove extremely costly and may significantly impact the project schedule. From the existing railroad crossing, the detour could extend around the proposed fill area of the southeastern bridge approach, and then connect to the existing Henderson St. approximately 400 feet northwest of the current intersection with White Settlement Rd. This portion of the detour will allow for the construction of the southeastern Henderson St. Bridge approach.

This detour option requires that acquisition of the Henderson St. Bazaar property and property required for channel construction will precede bridge construction.

5.4.3 White Settlement Road

At this location it may be desirable to construct the approach roads, retaining walls, and bridge one half at a time. In this scenario, one half (eastbound or westbound lanes) of the bridge and approaches could be constructed first, leaving sufficient room on the existing pavement to accommodate 2 lanes of traffic (one lane in each direction). This option would require the contractor to provide temporary shoring to stabilize the approach road embankment. Once the new lanes are complete, traffic can be shifted to the new bridge and construction of the remainder of the bridge can be completed. Disadvantages of this option are the increased costs associated with phased bridge construction and the resultant increase in construction time.

As an alternative, a detour could be constructed on the property immediately south of White Settlement Rd. Due to planned approach road retaining wall construction in this area, access will be restricted to several lots on the south side of White Settlement Rd. Therefore it is anticipated that the affected lots will be acquired as part of the project development. A detour in this location would allow for the construction of the western approach of the White Settlement Rd. Bridge. From here, the detour could then be extended to the east, crossing the FW&WRR using the existing railroad crossing to the extent possible. Securing an additional railroad crossing would most likely prove extremely costly and may significantly impact the project schedule. From the existing railroad crossing, the detour could extend north around the proposed fill area of the eastern bridge approach, and then connect to the Henderson St. detour, as described in Section 5.4.2 above. It is anticipated that this intersection would be signalized. This portion of the detour will allow for the construction of the eastern White Settlement Rd. Bridge approach.

This detour option requires that property acquisition for channel and detour road construction will precede bridge construction.

5.5 Access

The construction of the bridges and approach roadways will have important consequences to existing businesses and will impede access to adjacent property and intersecting streets. The assumption of redevelopment of the area on the interior side of the bypass channel means that the desired access can be designed to serve the future development. On the west side of the channel, the limits of redevelopment are not yet defined and impacts to access for adjacent properties and streets will have to be considered as the design progresses. Specific access issues are discussed below as they relate to each bridge.

5.5.1 Main Street

At the north end of the proposed Main St. Bridge, access to the extension of NE 10th St. may be impacted. The existing intersection is shown to be located at the end of the proposed bridge. Depending on the available sight distance to the south, the intersection could remain open, closed at Main St. to provide local circulation, or abandoned and removed.

If the extension of NE 10th St. is abandoned and removed then an alternative route (NE 11th St.) exists to the north. Vehicles along NE 10th St. could easily proceed north on North Commerce St. and make a left turn on NE 11th St. and access Main St.

Another option to explore is the creation of cul-de-sacs. However, with the restrictive right-of-way along some of the streets, this may require the purchase of additional properties.

5.5.2 Henderson Street

At the north end of the Henderson St. Bridge, access to the adjacent properties will be impacted by the proposed overpass. To address local access, a u-turn type roadway may be provided under the bridge. This u-turn roadway would be one-way and provide circulation to the adjacent properties. The u-turn roadway would enter and exit Henderson St. via slip ramps. These ramps could start and end at the southern end of the existing bridge over the West Fork of the Trinity River to the west.

To maintain access to Cullen St., Cullen St. could intersect with the u-turn roadway. This would only provide access to and from the north on Henderson St. Another option to provide access to the Cullen St. area would be to extend Shamrock Avenue so that it intersects with Henderson St. This would result in an intersection between the two bridges and sight distance may be restrictive, especially to the south, since the proposed bridge is required to provide clearance to an existing railroad line.

At the south end of the Henderson St. Bridge, access to North Commercial St. will also be impacted by the proposed overpass and bypass channel work. North Commercial St. could be terminated and left in place to temporarily provide local access or the roadway can be realigned to form a “t” intersection with Henderson St. This may or may not be a feasible solution depending on the resultant sight distance to the north.

Another option to explore is the creation of cul-de-sacs. However, with the restrictive right-of-way along some of the streets, this may require the purchase of additional properties. However, it should be noted that the construction of the bypass channel and ultimate fill conditions of the interior area will require a complete reconstruction of Commercial St., so any work to maintain traffic at this time would be temporary.

5.5.3 White Settlement Road

On the west end of the White Settlement Bridge, access to Rupert St. and Adolph will be impacted by the proposed overpass. The sections of Rupert St. just north and south of White Settlement can be modified and terminate into an existing driveway to provide local access or each can be abandoned all together. Circulation to the existing properties can be obtained via Whitmore to the south and Tillar to the north. Another option on the west side of the White Settlement Bridge is to provide a u-turn type roadway under the bridge. With the u-turn roadway in place, it may be possible to maintain the Rupert St. circulation via the one-way u-turn roadway.

On the east side of the White Settlement Bridge, access to South Commercial St. will be impacted by the bridge given the grade differential. South Commercial St. will be terminated at the embankment for the new White Settlement Rd. Access may be maintained temporarily through Kansas St. and Viola St. until the bypass channel work begins.

Section 6
Pedestrian Bridge Structures

6.1 General Description

6.1.1 Purpose

The purpose of this section is to provide a conceptual level analysis of the pedestrian bridge crossings proposed along the West Fork Trinity River and bypass channel as part of the Fort Worth Central City project. Preliminary recommendations for the size and general configuration of the pedestrian bridge structures are included herein.

6.1.2 Scope of Work

The scope of work for evaluation of the pedestrian bridges has three main objectives: 1) identify the controlling criteria, constraints, and parameters for design; 2) determine the preliminary locations, sizes, and types of the pedestrian bridges; and 3) develop preliminary quantities to be used in estimates of construction costs.

6.1.3 Criteria

The following criteria were identified for preliminary design of the pedestrian bridge structures:

- AASHTO Guide Specifications for Design of Pedestrian Bridges
- ADA requirements
- USACE requirements
- Local, State and Federal Regulations, Codes and Laws
- Conceptual drawings developed by the urban design consultant

6.2 Design Requirements

6.2.1 General Requirements

The pedestrian bridges are intended to serve a dual role of providing access and recreational connectivity. Enhancement of access across the Trinity River is a primary consideration in the placement of the pedestrian bridges, while providing additional connectivity between the two sides of the bypass channel for pedestrian and recreational use.

These primary considerations and additional requirements control the location, overall length and width of the bridges, spans between support piers, and the height of the bridge above the normal water surface on the bypass channel and West Fork. These additional requirements are further discussed herein.

6.2.2 Aesthetic Requirements

The pedestrian bridges will be located in the heart of the City of Fort Worth and adjacent to the Trinity River Uptown area. The aesthetics for the bridges are a primary consideration to the selected bridge alternative, given the visibility of the bridge structures in this high profile area. Final design of the pedestrian bridges will be a collaborative effort between the design engineers, urban planner, and architects in developing structures that blend in appropriately with their surroundings. During final design, consideration will be given to the other proposed structures and existing nearby structures, such as the FW&WRR trestle.

6.2.3 Hydraulic Requirements

Evaluation of the interaction of the bridge structures and the waterway, under flooding conditions, will be incorporated into the design of the bridges. Conceptual pier sizes and locations, as well as deck and railing heights, have been determined and incorporated into the hydraulic model. Final bridge geometry, pier sizes and locations will need to be updated within the hydraulic model once final selection of the bridge type is determined. Current general practice for bridge crossings over waterways in North Texas is to place piers within the channel, with the bridge characteristics incorporated into the hydraulic modeling, to maintain shorter and more economical bridge span lengths.

6.2.4 Geometric Requirements

The proposed clear walkway width of the pedestrian bridge at the bypass channel is 10 feet The clear walkway width of the West Fork Bridge is proposed to be 22 feet, to accommodate equestrian uses and to provide for light maintenance vehicle access across the river. The bridge heights will be set to provide 10 to 15 feet of clearance above the normal water surface (El 524.3), for boats to pass underneath the bridges.

Total bridge lengths vary by the site alternatives under consideration, but generally range from 180 feet to 220 feet. Site alternatives are discussed in Section 6.3.1.1.

Railing heights for the pedestrian bridges will be set based on the prescribed use. Equestrian use of the West Fork Pedestrian Bridge is anticipated, but special railing requirements are not anticipated given the limited concern for visual distractions.

6.2.5 Structural Requirements

The bridge structures are to be designed in accordance with the current AASHTO Guide Specifications for Design of Pedestrian Bridges, as applicable. As recommended by AASHTO, the bridges will be designed for either an H-5 or an H-10 truck load depending upon bridge width, as well as for required pedestrian loads, wind loads, and hydraulic loads during flooding events. The required 85 psf uniform live load and H-10 truck load for the 22 feet wide bridge are anticipated to be adequate for equestrian use (ref. Caltrans Section 3.14.1.3). Access to the bridges will be controlled by spacing bollards so that inappropriate vehicles do not use the bridge.

Specific structural consideration will be given to the unique loading characteristics caused by the hydraulic loading on the pier and superstructure due to a major flooding event. In the event of an unanticipated failure of the bridge during a flood, it is anticipated that the structure will sink prior to affecting any existing downstream structures.

6.3 Preliminary Type, Size and Location

6.3.1 Bridge Alternatives

6.3.1.1 Site Alternatives

Several locations were considered for the pedestrian bridges along the upper bypass channel and West Fork Trinity River upstream of the FW&WRR crossing. Based upon the continuity of the trails and desired connectivity of the bypass channel, two pedestrian bridge locations were selected. The proposed pedestrian bridge locations are shown on Figure 6-1 and Drawing CP-1, Volume II and further described below.

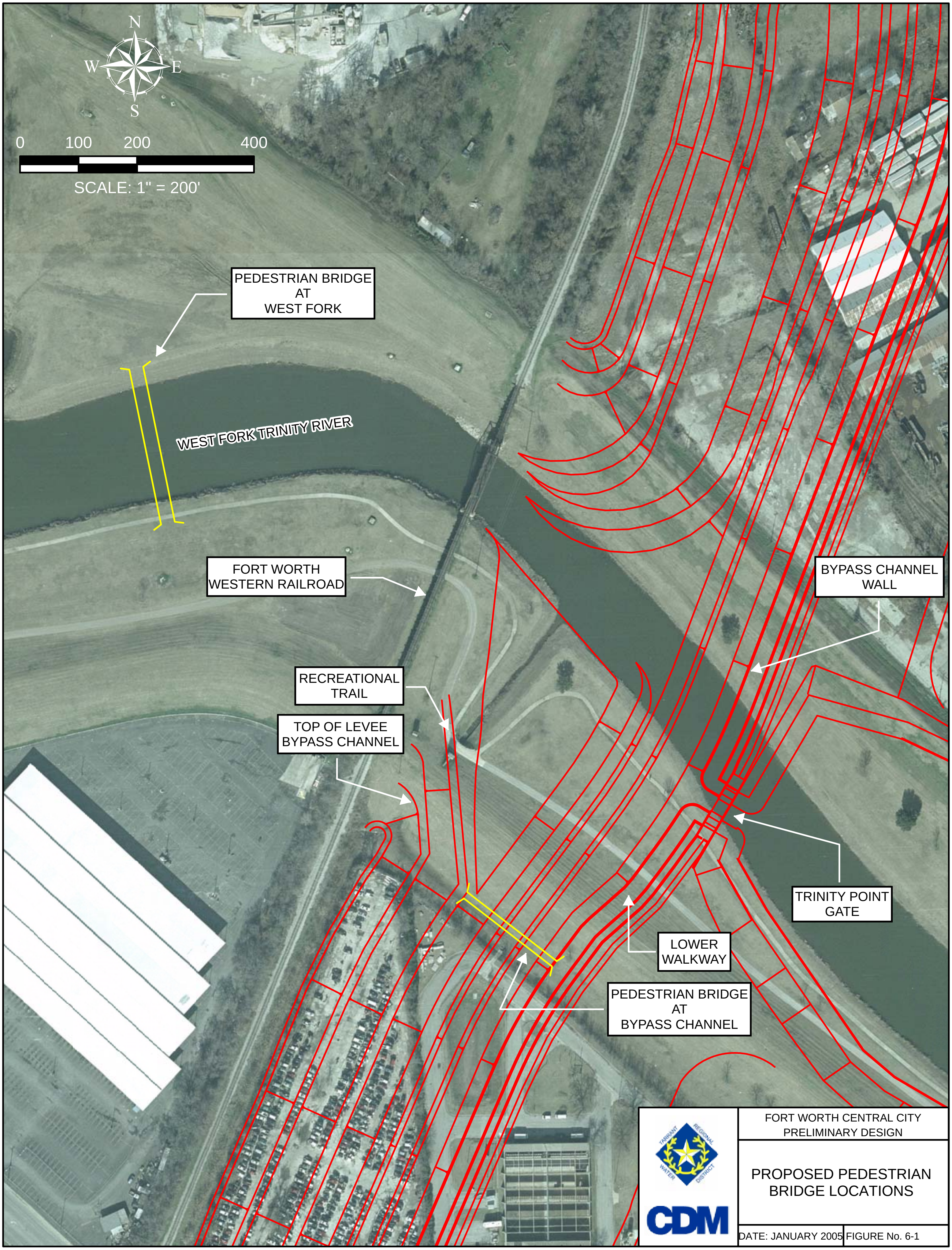
The final location of the pedestrian bridge over the bypass channel is approximately 1,050 feet downstream of the Henderson Rd. crossing at bypass channel Sta 44+26. The location for the bypass channel pedestrian bridge was determined after comparing the spacing between the North Main St. and Henderson Rd. crossings (approx. 3,900 LF) and the other crossings along the bypass channel. The distance between the North Main St. and Henderson Rd. crossings was the greatest of all the other crossings. Location of the pedestrian bridge upstream of the West Fork confluence was also deemed suitable for the hydraulic model.

Final location of the pedestrian bridge on the West Fork Trinity River is approximately 560 feet upstream of the FW&WRR crossing. The location of the pedestrian bridge upstream of the railroad crossing on the West Fork is considered beneficial in improving the connectivity of the west side of the bypass channel, while also providing benefits to the hydraulic model. The west side of the bypass channel, without the placement of the pedestrian bridge on the West Fork, would otherwise be cutoff by the confluence of the West Fork and bypass channel. The alternative to the pedestrian bridge is that pedestrians and other users could travel an additional 1,200 feet upstream to the existing Henderson Rd. Bridge to cross over the West Fork. This alternative was deemed unsuitable given the current traffic volume on Henderson Rd. and increased travel distance.

6.3.1.2 Bridge Alternatives

Several bridge materials and bridge configurations have been considered, including: steel truss bridges, prestressed concrete I-beam bridges, and concrete arch bridges. Steel bridges can economically make use of "weathering steel", but this option may not meet the aesthetic requirements for the project and may cause rust stains on concrete supports and retaining walls. Steel bridges may also be painted, but this option has the potential disadvantage of requiring repainting in the future for maintenance and aesthetics requirements. The concrete arch bridge and concrete I-beam bridges offer greater latitude for aesthetic enhancement and durability vs the steel truss options.

The preliminary design of the pedestrian bridges, defining the structure locations, sizes, and types, was developed based on the requirements outlined above. The total lengths of the two pedestrian bridges are estimated to be approximately 190 feet for the bypass channel bridge and 220 feet for the West Fork Trinity River Bridge. Sidewalk or trail approaches will be required at each end of the bridge structures. The proposed bridge profiles and cross-sections are shown on Drawings CP-6 and CP-7, Volume II.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SCALE: 1" = 200'
PEDESTRIAN BRIDGE
AT
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WEST FORK TRINITY RIVER
FORT WORTH
WESTERN RAILROAD
BYPASS CHANNEL
WALL
RECREATIONAL
TRAIL
TOP OF LEVEE
BYPASS CHANNEL
TRINITY POINT
GATE
LOWER
WALKWAY
PEDESTRIAN BRIDGE
AT
BYPASS CHANNEL
TARRANT REGIONAL WATER DISTRICT
CDM
FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN
PROPOSED PEDESTRIAN
BRIDGE LOCATIONS
DATE: JANUARY 2005
FIGURE No. 6-1

Section 7
Isolation Gates

7.1 General Description

7.1.1 Purpose

This section summarizes the structural and mechanical design requirements for the isolation gates planned for the Fort Worth Central City Project. The scope of this section includes developing gate selection criteria, structural and mechanical design criteria for the proposed gate facilities, gate construction details, functional descriptions and a preliminary layout of each closure structure. Three isolation gate structures are planned for the Central City Project. The location plan in Figure 7-1 illustrates the proposed Trinity River bypass channel alignment and the location of each proposed gate structure.

7.2 Hydraulic Gate Selection

CDM collaborated with GE Hydro to evaluate flood isolation gate alternatives that meet the functional and aesthetic requirements of the project. A copy of the report prepared by GE Hydro, including illustrations of each gate type, is included in Attachment A. The isolation gates will be used to protect the interior areas, vulnerable to damage from flood flows during storm events. The gates will be installed within concrete structures to be constructed within the proposed levee and floodwall system. The gates will remain in the fully-open (un-submerged) position under normal conditions, and lowered to the fully-closed position under balanced head prior to flooding during significant rainfall events.

The gate types identified as alternatives for the project include:

- Fixed-Wheel (Roller) Gates
- Radial Gates (Lower-to-Close)
- Radial Gates (Raise-to-Close)
- Vertically-Hinged Sector Gates
- Mitre Gates
- Bottom-Hinged Flap Gates

At a minimum, each gate type was evaluated based upon the following operating and aesthetic requirements:

- Gate design must have self-cleaning characteristics.
- Gate must provide a clear opening, capable of passing small watercraft in the boat channels, and pedestrian traffic along walkways proposed for the project.
- Gate must be easily hidden by the isolation gate structure.
- Gate should require a minimal amount of civil concrete to support the gate structure and operating equipment.
- Gate must be capable of closing under its own weight without power.

The following paragraphs provide descriptions of the advantages and disadvantages of each gate alternative. Each alternative was compared based upon the criteria listed above, and a preferred alternative was then selected. Illustrations of each gate are included in Attachment A.

7.2.1 Fixed-Wheel (Roller) Gates

Fixed-wheel gates are vertical-lift gates that have been used as flood control gates at many locations around the world. These gates are typically operated by wire rope hoists; hydraulic hoist can also be used. The advantages of these gates in flood control service include:

- The gates are easily concealed within an overhead structure.
- Required civil concrete support structures required are relatively small.
- The gates can be lowered to the closed position under their own weight without power.
- Gate wheels provide a greater distribution of the hydrostatic load.
- Gates can be easily accessed in the raised position for inspection and maintenance.
- Sill design is typically self-cleaning, although the gate slots require cleaning (See below).

Disadvantages include:

- Gates require an overhead structure to support the hoisting mechanism and house the gate.
- Gate slots are required, which can collect debris that may jam the load wheels.

7.2.2 Radial Gates (Lower-to-Close)

Radial gates (or tainter gates) are typically used for discharge regulation in free-surface flow over spillways, for water level control in canals and as flood isolation gates. Radial gates can be operated by hydraulic cylinders or wire rope hoists. Advantages of these gates in flood control service include:

- Gate slots are not required.
- Gates are mechanically simpler and usually require less hoisting capacity.
- Gates are structurally stiffer than other gates.
- Gates can be lowered to the fully-closed position without power.

Disadvantages include:

- Requires large civil works to support the gate trunnions.
- High concentrated load is transferred to the structure through the trunnion bearings.
- Difficult to conceal the gate in a structure.
- Gates have complex structures that are difficult to fabricate.
- Seals can be subject to damage due to freezing.

7.2.3 Radial Gates (Raise-to-Close)

Advantages of this radial gate include:

- Gate is submerged and concealed from view when in the fully-open position.

Disadvantages of this gate include:

- Gate requires line or standby electric power to close.
- Gate is normally submerged and will be subject to corrosion.
- Gate can be unstable when closed and may require dogging in the closed position.

7.2.4 Vertically-Hinged Sector Gates

Vertically-hinged sector gates have traditionally been used on locks and yacht marinas, but have also been used for storm surge barriers and flood control. Sector gates are typically hydraulically operated. The advantages of these gates include:

- Operating machinery is easily accessible for maintenance when the gate is in the fully-open position.

- Gates leaves are easily concealed within recesses in the side walls when they are in the fully-open position.
- Gates are stable in the fully-closed position.
- Gate sill contact is narrow preventing the collection of debris and jamming of the gate.

Disadvantages of these gates include:

- Civil structures are large and complex due to the sidewall, which contain the gate leaves when they are in the open position.
- Gates require line or standby electric power to close.
- Both gate leaves are required to successfully close.
- Gate leaf maintenance requires the installation of stoplogs.

7.2.5 Mitre Gates

Mitre gates are vertically-hinged, and are constructed of two leaves that form a three-hinged arch when the gates are in the closed position. These gates are typically hydraulically operated and used for large openings in locks, levees and flood walls. The advantages of these gates include:

- Weigh less than other gates types used for large openings.
- Gates are easily hidden in the sidewalls when open.
- Center support is not required for two-leaf gates.

Disadvantages of these gates include:

- Gates require a debris-free sill to prevent jamming when closing.
- Both gate leaves need to close successfully.
- Gates require line or standby electric/hydraulic power to close.
- Gate leaf maintenance will require stop logs.

7.2.6 Bottom-Hinged Flap Gates

Bottom-hinged flap gates are typically crest-mounted and used for water level control in rivers, canals and spillways for dams. These gates are completely submerged in a recess when in the fully open position, and are typically operated with hydraulic cylinders. Advantages of these gates include:

- Easily concealed below the water level when in the fully-open position.

- Hoists are accessible for maintenance in the fully-open position.

Disadvantages of these gates include:

- Gate requires line or standby electric power to close.

- Stop logs are required to access the gate leaf for routine maintenance.

7.2.7 Preferred Alternative

Fixed-wheel (roller) gates were preferred over other types, due to their ability to close during a power outage; relative ease to conceal gate leaves in a narrow concrete structure; and maintenance accessibility while in the open position. The detailed design aspects of these gates are included in the ensuing sections.

7.3 Flood Closure Structures

Preliminary layouts and details of each gate structure are included in Volume II Drawings. The site layouts are based upon the current urban planning scheme for each location. Figure 7-1 and the isolation gate civil structure drawings, included in Volume II, should be reviewed concurrently with these sections. The following isolation gate structures are included:

- TRWD Isolation Gate Structure – located within the existing channel of West Fork Trinity River, upstream of the confluence with the northernmost reach of the new bypass channel. There will be one pedestrian walkway gate and one channel gate at this location.

- Trinity Point Isolation Gate Structure – located near the middle of the new bypass channel, where the bypass channel intersects the existing channel of West Fork Trinity River. There will be two pedestrian walkway gates and one channel gate at this location

- Clear Fork Isolation Gate Structure – located within the existing channel of Clear Fork Trinity River, downstream of the southernmost reach of the new bypass channel. There will be one pedestrian walkway gate and one channel gate at this location.

The primary objective of the isolation gate structures is to retain flood waters during periods of high flows. The top of each structure will be set at four feet above the SPF elevation. The following design elevations are assumed for preliminary design of isolation gate structures:

- SPF El 552.5 at Clear Fork Structure, El. 545.5 STET Trinity Point Structure, and El. 540.0 at TRWD Structure.

- Normal pool elevation at El 524.3.
- Minimum operational pool elevation in the protected areas at El 520.00 under normal gravitational operation.
- Top elevation of flood closure structure at each site provides a minimum of 4-foot of freeboard.

Each structure will include fixed-wheel gates, for pedestrian walkways at sill elevation 530.0, and the small boat channels at sill elevation 520.0. The walkway gates will seal a 12 feet W x 10 feet H clear opening, and the small boat channel gates will seal a 24 feet W x 17 feet H clear opening in the isolation gate structure. The designs for each gate type will be identical. The gates will remain in the open position during normal flow conditions, and be closed by operators prior to and during high flow events in the bypass channel.

Stop log guides will be provided in the openings, on the flood side of each structure, to permit isolation of the gates from the bypass channel during extended gate maintenance periods. One set of stop logs will be provided for each gate facility. Stop logs, and their lifting devices, will be stored on the equipment deck or in the protected area adjacent to each closure structure. Concrete parapet walls are proposed for the equipment deck, to shield the mechanical equipment and stored materials from view by pedestrians. The proposed storm water pumping station will enable the operators to lower the pool elevation in the protected area, which will allow inspection and maintenance of the channel gate slots and sills.

While in the open position, the wheel-gate leaf will be accessible on both sides by maintenance personnel from the gate chamber within the flood closure structure. Entry to the gate chamber is gained from the equipment deck through a roof scuttle and access ladder. A passageway will be provided between the walkway gate and channel gate chambers. The layouts, shown in Volume II, provide adequate reach and clearance for installation and removal of all gate equipment by a mobile crane. The crane can be mobilized on the levee, or in the walkway on the terraced side of the channel.

A control room will be constructed within the isolation gate structure to securely house the gate control panels and other electrical equipment. Control room access will vary depending upon the levee and terrace finished grade elevation at each site. The TRWD control room will be accessed through a security door on the south side of the structure, and the equipment deck will be accessed by a ladder to the roof scuttle above. The Trinity Point control room will be accessed by entrance through a security door and climbing a ladder to the equipment deck; from there access to the control room will be gained through a roof scuttle. The Clear Fork control room will be accessed by a stairway to the equipment deck, then entry through a roof scuttle.

7.4 Design Requirements

Gate structural design will be in conformance with the following USACE Engineering Manuals:

- EM 1110-2-2105 Design of Hydraulic Steel Structures.
- EM 1110-2-2701 Vertical Lift Gates.
- EM 1110-2-2705 Structural Design of Flood Closure Structures for Local Flood Control Projects.

Additional engineering and material selection guidelines in the German Standard, *DIN 19704 (1998): Hydraulic Steel Structures; Criteria for Design and Calculation,* may also be used.

The gate leaf will be of all-welded ASTM A-36 steel plate construction, consisting of horizontal plate girders arranged on the protected side of the gate. The girders will act compositely with the skin plate on the flood side of the gate. Horizontal and vertical framing members will transfer the hydraulic load to the concrete closure structure. Vertical intercostals will be furnished between girders, where required. All assembly hardware will be austenitic (300 series) stainless steel.

7.4.1 Load Wheel Requirements

Load wheels will be fabricated from steel forgings, have turned cylindrical treads, and operate on flat track fabricated from stainless steel plate. The load wheels will operate on self-aligning, antifriction bearings. Bearings will be mounted on cantilevered stainless steel axles, which will be gun-bored for pressure application of grease to the wheel bearing case. Grease fittings will be connected to the axle at the support end, allowing application of grease by maintenance personnel from the gate chamber, when the gate is in the raised position.

7.4.2 Seal Requirements

The gates will be designed to operate under a differential head of 30 feet. Gate seals will be arranged on the skin plate for flood side (upstream) sealing. The allowable leakage rate at this head pressure will be no greater that 1-fl.oz/ft-of-seal/sec. Seals will have a 1/8” to 1/4” preset, which will engage the side seal bulb to the seal plate during periods when the flood stage is below the lintel elevation. Although the leakage rate will be slightly higher than that required for sealing under full head, the gates will still provide flood protection. Sealing will occur in one direction. Seals will be fabricated from neoprene extrusions having a Shore-A Durometer hardness of 65. Seals exposed to sliding friction will have Teflon facings to minimize the hoist friction load.

Seals will be clamped to the skin plate by stainless steel retaining strips, machine screws and nuts. Stainless steel ferrules will be inserted into the screw holes, passing through the retaining strips and seal material, to ensure even clamping of the seals. Machine screws and nuts will be attached with elastomer faced washers to prevent leakage through the screw holes.

Seal splices will be straight butt joints of neoprene extrusions, having identical section dimensions, and vulcanized at the shop or in the field. Seal corners will be fabricated of neoprene moldings, specially designed for the seal shapes to be joined.

Sill seals will be formed by a rectangular extrusion, as shown on Drawing S-17, Volume II. The bottom downstream framing member will include a sloping flange plate, slanted in the upstream direction, terminating at the bottom of the skin plate. This provision will enable arrangement of the bottom seal in the most downstream position, and in the same plane as the side seal and lintel seal.

Side seals will be formed by Teflon faced single-stem bulb seals (J-type), as shown on Drawing S-17, Volume II. Retaining strip machine screws for the side seal will be counter-sunk to allow clearance between the retaining strip and the side seal contact plate.

Lintel seals will be formed by Teflon faced double-stem center-bulb seals, as shown on Drawing S-17, Volume II. The lintel seal is pressed against the contact plate by the flood side hydraulic pressure, due to water entering the passageway between the bulb and the skin plate. Retaining strip machine screws for the lintel seal will be counter-sunk to allow clearance between the retaining strip and the lintel seal contact plate.

7.4.3 Embedded Parts Requirements

The flood closure structures will be constructed with dove-tailed or keyed cut-out sections in the primary concrete, for the installation of prefabricated embedded parts. The embedded parts used in conjunction with the fixed-wheel gates are shown on Drawing S-17, Volume II. These parts include the sill seal contact plate and beam assembly, side seal contact plate, load wheel and guide wheel plate assembly and the lintel seal contact plate and reaction beam. The bottom corners of the gate slots will be angled to prevent accumulation of debris. Seal contact plates, guide plates, and wheel tracks will be so arranged in the gate slots, to prevent tampering or damage by vandals. Galvanized alignment anchors and leveling nuts will be embedded in the primary concrete cut-out sections, allowing alignment of the embedded parts prior to the placement of fill concrete.

All exposed embedded parts with seal contact and sliding faces will be fabricated of 300 series stainless steel. In some instances, plate stock used as contact faces will be part of a plate fabricated beam. Seal contact and sliding faces will be machined to a 125 micro-in rms surface roughness.

7.4.4 Coatings Requirements

Coating selection, surface preparation, and application will be in accordance with USACE Engineering Manual: EM 1110-2-3400 *Painting: New Construction and Maintenance.* In general, the gate leaf will receive a surface preparation after fabrication meeting SSPC-SP-10 (near white blast); a zinc-rich primer will be applied to a 3.5 mil dry film thickness (DFT); and two finish coats of a flouropolymer epoxy finish will be applied, each to 3.0 mil DFT. During the detailed design phase, hot-dip galvanizing or thermal spraying will be evaluated as a coating option. Based upon the surface area ratios between the anodic and cathodic components, the need for cathodic protection of the gate leaf and embedded parts is not anticipated.

7.5 Gate Operating and Control Equipment

7.5.1 Functional Description

The fixed-wheel gates will be lowered into position in anticipation of high water events in the new bypass channel. The gates will remain in the full-open or full-closed position during normal operation. The passageways through the isolation gate structures require additional safety provisions in the operating procedures, to protect boaters and pedestrians while the operating equipment is in use. These conditions will require local operation of the gates and special safety features as described below.

7.5.2 Gate Operating Equipment

Gate operating equipment will be designed in conformance with the following USACE Engineering Manuals:

- EM 1110-2-2610 Lock and Dam Operating and Control Systems
- EM 1110-2-3200 Wire Rope Selection Criteria for Gate Operating Devices

The fixed-wheel gates will be operated by electric motorized, dual-drum, wire rope hoists mounted on the equipment deck, as shown in Volume II. The gates are lowered by gravity at a controlled rate that is regulated by the hoist. The normal speed of operation will be 1.5 feet per minute, adjustable to 3 feet per minute for emergency operation.

The gate hoists will be comprised of custom fabricated and commercially available machinery components, integrated into a skid-mounted self-contained unit. The gate hoist and control panels will be designed, fabricated and assembled by the gate manufacturer. Wire ropes from each drum will be connected by forged shackles and sheave blocks, to two lifting eyes located at the top of the gate leaf. The gate hoists will have a single center-mounted, squirrel-cage induction motor drive. The motors will operate on 480V, 3-phase, 60Hz power.

The drive design will include self-locking speed reducers; a spring-set, shoe-type stop brake, with a DC magnet release; and a passive eddy-current braking system, which will allow manual closure of the gate without AC power. Gate movement will be controlled automatically by field adjustable mechanical limit switches, which will automatically stop the gate hoist when the gate arrives at the full-open or full-closed position.

7.5.3 Safety Provisions

Dogging devices will be furnished with the gates, and manually installed in the gate slots, to secure the gates while in the raised position. This device will prevent an unplanned closure of the gate due to failure of the hoist system. The dogging device will be removed by operators prior to lowering the gate during normal operations.

The dogging device will be capable of releasing the gate under full load for emergency lowering without the hoist system. This will be accomplished by providing a self-locking, centralized worm-gear drive with parallel shafts linked to the wire rope drums. The worm will be mechanically linked to a secondary shaft. During normal operations, the secondary shaft will be disengaged and the system will operate by the electric motor. During emergency operation, with no standby power, the secondary shaft will be engaged and a hand crank will be attached. The hand crank will be used to manually raise the gate by several inches, and the dogging devices will be removed for lowering.

Several visual and audio warning devices will be activated by a timer-controlled relay interlocked with the gate hoist controls. The timer will operate a visual beacon and audible horn, and simultaneously delay opening and closing of the gate, thereby providing a warning to pedestrians in the vicinity of the structure during operation. The warning beacon and horn will operate until the gate is in the full-closed or full-open position. Closed circuit surveillance cameras will be positioned around the flood closure structure, allowing operators to monitor pedestrian and boat activity around the structure from the control room.

7.5.4 Emergency Operation

The fixed-wheel gate operating system will be triple-redundant, i.e. having three methods of gate closure during periods of high flow. Emergency engine-driven generator sets will be installed in small prefabricated buildings, located in the protected areas adjacent to the flood closure structures, to provide emergency power to the gates in the event of a power outage. If the generator equipment fails to work during a power outage, gates will be lowered manually using the gravity lowering feature in the drive train.

In addition to the alternative methods of emergency operation, the gates will be designed to close with a single wire rope if one of the ropes is disabled or damaged. The provision of side guide rollers will accomplish this. The system will be designed to permit this operation, with AC power or manually, without exceeding the working stress of any component in the drive train.

7.6 Civil Structures

7.6.1 Design Requirements

Design of structures for isolation gates will be in conformance with the following USACE Engineering Manuals:

- EM 1110-2-1913 *Design and Construction of Levees*
- EM 1110-2-2104 *Strength Design for Reinforced- Concrete Hydraulic Structures*
- EM 1110-2-2502 *Retaining and Flood Walls*
- EM 1110-2-2705 *Structural Design of Closure Structures for Local Flood Control Projects*
- EM 1110-2-2906 *Design of Pile Foundations*

For preliminary design, each isolation gate structure has been analyzed for the following load cases:

- Usual Load Case: water level on the driving side at SPF elevation, with gates closed, and water level on the resisting side at the lowest gate sill elevation. Sliding Factor of Safety = 1.50.
- Unusual Load Case: water level on the driving side at lowest adjacent top of levee elevation, with gates closed, and water level on the resisting side at the lowest gate sill elevation. Sliding Factor of Safety = 1.33.
- Extreme Load Case: water level on the driving side at normal pool elevation, with gates closed, and water level on the resisting side at the lowest gate sill elevation, with lateral earthquake forces applied. Sliding Factor of Safety = 1.10.

During final design, an additional load case will be considered as follows: water level at the normal pool level at the downstream face of gate structure and water level at the lowest drawdown level at Samuels Dam at the upstream face of gate structure. This is considered to be a maintenance condition (unusual load case).

Gate structures have been evaluated for sliding, overturning, and foundation bearing capacity, as well as for functional requirements for flood gate operation. In general, retaining walls in the vicinity of the isolation gate structures have been designed in accordance with the criteria given in the Preliminary Structural Submittal for Retaining Walls. However, the deep training walls that form the flow channel just upstream and downstream of the gate openings will be supported on steel piles, similar to the gate structures. At the TRWD gate structure, these training walls will be continuous with the walls forming the inlet for the storm water pump station.

General environmental loads (wind, snow, seismic, etc.) will be based on the 2000 International Building Code (IBC). Wind loads will be based on a Basic Wind Speed of 90 mph (3-second gust). Ground snow load is 5 psf and minimum foundation depths will be 12 inches.

Seismic considerations: According to the seismic maps included with the 2000 IBC, Fort Worth has a short period ground motion value (S_s) of 0.112g and a one-second (S_2) value of 0.055g based on a 2-percent probability of exceedance within a 50-year period. For Site Class D (stiff soil), the corresponding design spectral response accelerations are S_{ds} = 0.12g and S_{d1} = 0.09g. The corresponding effective ground acceleration is 0.0473g. Accordingly, a 0.05g ground acceleration will be used in design.

For preliminary analyses, the following foundation design parameters have been used:

- Maximum foundation bearing capacity: 10,000 psf mass concrete on rock or on roller compacted concrete.

- Friction angle equal to 35-degrees for rock and 45-degrees for roller compacted concrete.

For steel H-piles, the allowable stress is assumed to be 10 ksi, as recommended in EM 1110-2-2906.

7.6.2 Functional and Technical Requirements

The primary objective of the isolation gate structures is to retain flood waters during periods of high flows and to protect adjacent land from becoming inundated. As such, the gate structures will act as "flood walls". Based on geotechnical evaluation, cutoff walls may be required below the gate structures in order to control seepage.

The top of each gate structure will be established, as required, for complete housing of the raised vertical gates within the concrete structure. The adjacent levee will be set at four feet above the SPF elevation. The following design elevations have been assumed for preliminary design of isolation gate structures (based on estimated SPF elevations rounded up at least to the nearest 0.5 feet):

- SPF at El 552.5 at Clear Fork Gate Structure.

- SPF at El 545.5 at Trinity Point Gate Structure.

- SPF at El 540.0 at TRWD Gate Structure.

- Minimum operational pool elevation in the protected areas at El 520.00 for all gate structures (i.e., sill elevation for channel gates, 5 feet below Normal Pool Elevation)

A sheetpile cutoff wall is shown below the Clear Fork Gate Structure (including the abutments) due to the granular soil conditions between the top of bedrock and the bottom of the concrete structure. A bentonite-soil slurry wall (approximately 3 feet in thickness) may be considered as an alternate to the sheetpile wall. The cutoff wall is shown centrally located under the structure to avoid interference with the battered H-piles.

Although there is a sand and gravel layer deep below the TRWD Gate Structure, and there may be such a layer deep below the Trinity Point Gate Structure, it is not anticipated that a cutoff wall will be required below these structures. There appears to be an adequate layer of clay (i.e, in excess of 10 feet) below the bottom of concrete and the top of sand or gravel to act as a suitable cutoff against seepage at these structures.

The abutment structures at the sides of the primary gate structures are shown as retaining walls, with buttress walls at the downstream face and with bases supported on steel H-piles. This is the proposed configuration for each of the three gate structure sites. The abutments are separated from the gate structures by expansion joints due to the difference in structural configuration. The end walls of the abutment structures are battered at 1H:10V at the interface with the levee, as recommended in Section 8-14 of EM 1110-2-1913.

Approximate thicknesses of concrete elements have been estimated based on preliminary calculations and are shown to scale on the attached drawings. For final design, gate structures and abutment structures will be evaluated for internal concrete stresses and required reinforcement, as well as sliding, overturning, and foundation bearing capacity.

7.6.3 Calculations and Results

Preliminary analyses of gate structures have been made for sliding, overturning, and foundation bearing capacity. Initial analyses considering gate structures founded on soils indicated excessively large concrete structures in order to meet the sliding factor of safety. Consequently, subsequent analyses considered either the concrete structure to extend down to bedrock or deep foundations using steel H-piles or drilled concrete shafts.

For each structure, the initial stability analysis considers the structural concrete foundation to extend down to bedrock, assuming sound bedrock at 4-feet below measured top of rock. For the Trinity Point Gate Structure, the soil boring did not extend to bedrock depth; bedrock has been assumed at 20 feet below the bottom of boring for preliminary analysis of this structure. Due to the volume of structural concrete required, a second alternative was developed assuming roller compacted concrete for the lower concrete foundation.

Since bedrock is generally a considerable distance below the channel bottom, a third alternative was developed using battered steel H-piles for support of the gate structure. Piles battered in the downstream direction were sized to resist the maximum lateral load, based on the given load cases. Then, an equal number of piles battered in the upstream direction were added to resist the maximum vertical dead load without fluid pressures. The rows of piles were set to correspond to the resultant location for each of these load conditions. During final design, more detailed computer analyses of pile groups and pile interactions will be required.

The potential use of a foundation drainage system was evaluated during preliminary design. Although some reduction in concrete volume (less than 10 percent) could be achieved, the added costs, maintenance, and risks associated with a foundation drainage system do not appear to be warranted.

Stability analyses of gate and abutment structures were performed using spreadsheets developed by CDM for this project. Summaries of gate stability analysis results are included in Tables 7-1 and 7-2, and related stability analysis loading diagrams are shown on Drawings S-14, S-15, and S-16 in Volume II for the primary load cases for each gate structure on piles. Summaries of abutment stability analysis results are included in Table 7-4. The stability analyses consider differential fluid pressures on the structures. Lateral soil loads are not considered for these preliminary analyses since soil levels are expected to be balanced on the upstream and downstream sides of the structures, but soil loads will be considered during final design.

Stability analyses of the deep training walls adjacent to the gate structures were performed using CTWALL. Summaries of the training wall stability analysis results are included in Table 7-5.

Detailed stability analyses performed for gate structures, abutment structures, and training walls are included in Volume III. Volume III also includes printouts of the spreadsheets with cell formulas displayed and includes manual calculations for design of steel H-piles. Table 7-3 shows the required volume of concrete, additional volume of roller compacted concrete (RCC), and number of piles required for each design condition for the gate structures and for the abutment structures.

Layout drawings for isolation gate structures are included in Volume II. Drawings S-1, S-2, S-3, S-4A, S-4B, and S-4C show the layout of the gate structure and abutments at the TRWD Isolation Gate Facility. Drawings S-5, S-6, S-7, S-8A, S-8B, and S-8C show the layout of the gate structure and abutments at the Trinity Point Isolation Gate Facility. Drawings S-9, S-10, S-11, S-12, S-13A, S-13B, S-13C, and S-13D show the layout of the gate structure and abutments at the Clear Fork Isolation Gate Facility.

Three alternatives are shown for the each isolation gate structure: 1) concrete foundation to rock; 2) concrete structure over roller compacted concrete foundation to rock; and, 3) concrete structure supported by battered steel piles. For the Clear Fork Isolation Gate, a fourth alternative is shown: concrete foundation to rock, with a foundation drainage system. Where a heel or toe is shown, steel reinforcement will be required to resist tensile stresses in the concrete foundation.

For the TRWD Isolation Gate Structure, rock is estimated to be approximately 32 feet below groundwater level and 66 feet below SPF level. For the Trinity Point Isolation

Gate Structure, rock elevation is unknown but has been estimated to be approximately 55 feet below groundwater level and 80 feet below SPF level. For the Clear Fork Isolation Gate Structure, rock elevation is estimated to be about 32 feet below groundwater level and 65.5 feet below SPF level.

Four rows of battered steel piles are shown on drawings S-4C, S-8C, and S-13D for resistance to lateral and vertical loads. The two downstream rows are designed to resist the maximum lateral load (maximum water level case), with piles loaded axially in compression and neglecting potential tension in the two upstream rows of piles. The two upstream rows are provided to support the dead weight of the structure when fluid loads are not present. By using equal numbers of battered piles upstream and downstream, the lateral components in the rows of piles will balance each other for the vertical dead load case. The number and rows of piles have been set such that the centroid of the pile reaction is approximately at the location of the foundation reaction resultant from the stability analysis of the upper structure, for both the maximum water level case and for the vertical dead load case.

The TRWD Gate Structure concrete mass is shown to extend down to elevation 496.0 for support on piles. This was based on extending to 4 feet below the existing channel bottom. However, it is anticipated that the channel bottom will be filled up to elevation 512.0 or higher during construction, thus a shallower base elevation for the concrete structure may be considered during final design.

The abutment structures at each side of the gate structures are proposed to be supported on battered steel piles. The base of these structures is much shallower, but much more massive structures would be required to resist sliding if founded on clays.

The deep training walls that form the flow channel just upstream and downstream of the isolation gates are also proposed to be supported on steel piles. The proposed training wall configuration is shown in Drawing S-17, Volume II.

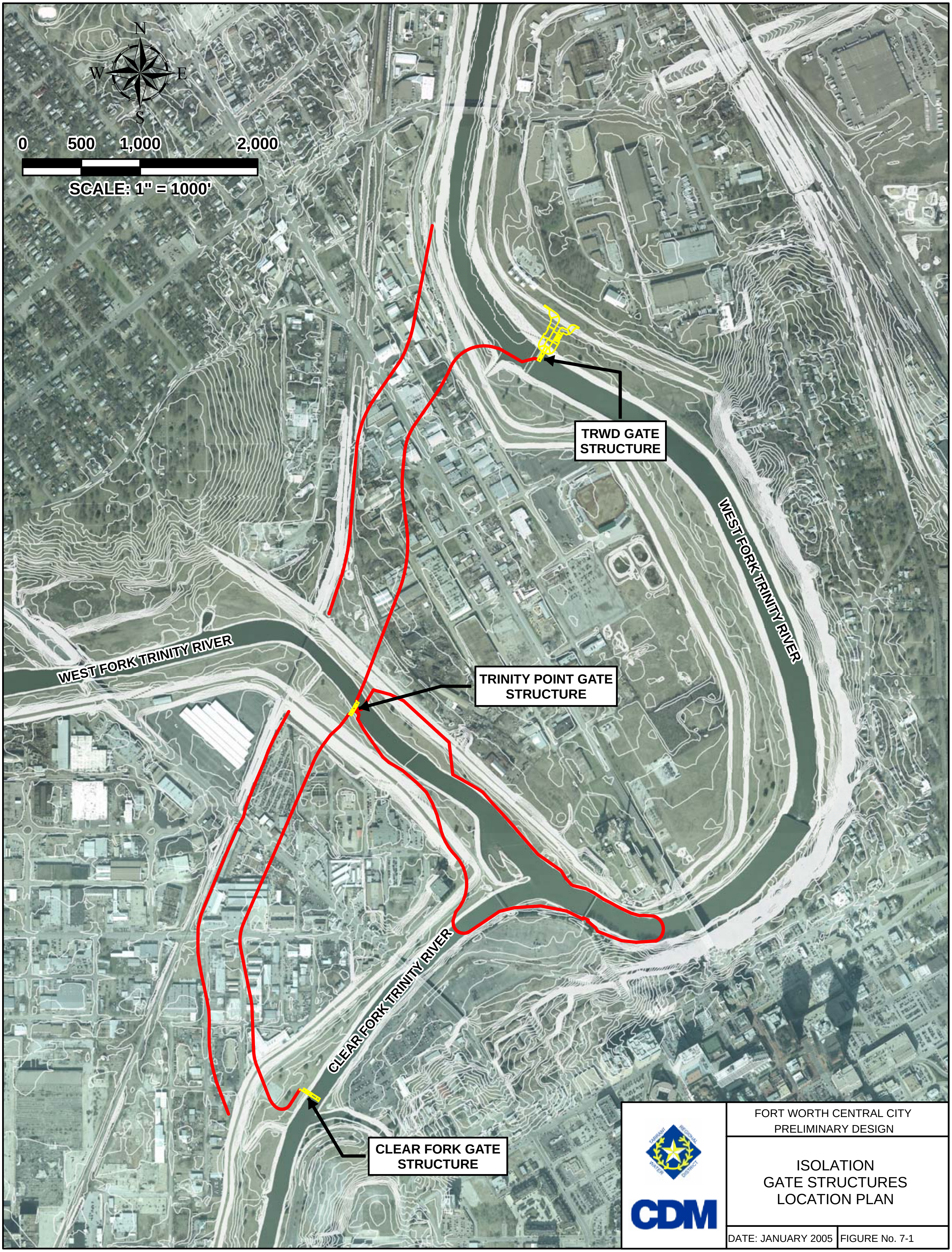
The Unusual Load Condition, with water level at the top of levee, controlled the design for each gate structure and abutment structure. Seismic analyses were performed for the proposed configurations using H-piles, using the normal pool elevation on the driving side and water level at the gate sill elevation on the resisting side. Lateral forces due to seismic are much lower than lateral forces due to maximum water levels, so the Extreme Load Case is not considered to be critical for these structures.

7.7 Recommendations

It is recommended that battered steel piles be used for support of each of the gate structures, abutment structures, and deep training walls. This is based on the assumption that the alternatives with mass concrete or roller compacted concrete down to bedrock are relatively more expensive, due to the added costs of excavation and dewatering, as well as significant added concrete mass for a concrete foundation to rock. However, if the roller compacted concrete alternative is found to be relatively close in cost to the steel H-piles, it is recommended to use roller compacted concrete for the gate structures, due to improved reliability. Nevertheless, steel H-piles would still be proposed to be used at abutments and deep training walls.

The proposed cross-sections for gate structures and abutments on plies are shown on drawings S-3, S-4C, S-7, S-8C, S-12, and S-13D. The proposed cross-section for the training walls is shown on Drawing S-17, Volume II.

Drilled concrete shafts may also be further considered during final design. Steel H-piles were chosen for preliminary design due to ease of installation for the battered pile configuration. Concrete shafts may not be able to be drilled at as steep a batter and may not be practical with any batter at all. Thus, concrete shafts likely would need to rely on lateral soil resistance and on fixity into rock with flexural resistance. However, if concrete shafts are shown to be practical and more economical, they could be reconsidered during final design.



FORT WORTH CENTRAL CITY
PRELIMINARY DESIGN

ISOLATION
GATE STRUCTURES
LOCATION PLAN

DATE: JANUARY 2005 | FIGURE No. 7-1

Table 7-1
Isolation Gate Structures Stability Analyses - Pile Foundations

Load Case	Base Width (ft)	Overturning Ratio	Eccentricity from Centerline (ft)	Comments
TRWD Gate Structure:				
Standard Project Flood (SPF)	35	1.83	2.40	Maximum Water Level Case Controls
Maximum Water Level	35	1.58	4.84	100% Base Compression Controls
Trinity Point Gate Structure:				
Standard Project Flood (SPF)	37	1.83	3.45	Maximum Water Level Case Controls
Maximum Water Level	37	1.57	5.77	100% Base Compression Controls
Clear Fork Gate Structure:				
Standard Project Flood (SPF)	48	1.46	2.60	Maximum Water Level Case Controls
Maximum Water Level	48	1.29	7.56	100% Base Compression Controls

Note: Refer to Stability Analyses Loading Diagrams on Drawings S-14, S-15, and S-16.

Table 7-2
Isolation Gate Structures Stability Analyses Results - Alternate Foundations

Load Case	Base Width (ft)	Sliding Ratio	Max. Bearing Pressure (psf)	Comments
TRWD Gate Structure:				
Mass Foundation on Rock				
Standard Project Flood (SPF)	47	1.44	6805	Maximum Water Level Case Controls
Maximum Water Level	47	1.27	8376	100% Base Compression Controls
RCC Foundation to Rock				
At Top of RCC				
Standard Project Flood (SPF)	35	2.00	5145	Maximum Water Level Case Controls
Maximum Water Level	35	1.72	6442	100% Base Compression Controls
At Base of RCC				
Maximum Water Level	50	1.31	7868	100% Base Compression Controls
Trinity Point Gate Structure:				
Mass Foundation on Rock				
Standard Project Flood (SPF)	60	1.15	6856	Maximum Water Level Case Controls
Maximum Water Level	60	1.06	8274	100% Base Compression Controls
RCC Foundation to Rock				
At Top of RCC				
Standard Project Flood (SPF)	34	1.56	3222	Maximum Water Level Case Controls
Maximum Water Level	34	1.33	4184	100% Base Compression Controls
At Base of RCC				
Maximum Water Level	60	1.09	8752	100% Base Compression Controls
Clear Fork Gate Structure:				
Mass Foundation on Rock				
Standard Project Flood (SPF)	71	1.05	4961	Maximum Water Level Case Controls
Maximum Water Level	71	1.00	5413	Sliding Controls
With Foundation Drains				
Maximum Water Level	67	1.01	6425	Sliding Controls
RCC Foundation to Rock				
At Top of RCC				
Standard Project Flood (SPF)	69	1.35	3417	Maximum Water Level Case Controls
Maximum Water Level	69	1.26	3737	100% Base Compression Controls
At Base of RCC				
Maximum Water Level	81.5	1.01	4370	Sliding Controls

Table 7-3
Isolation Gate Structure Alternatives - Volume of Concrete / RCC and Number of Piles Required

Load Case	Concrete Volume (CY)	Addl RCC Volume (CY)	No. of Piles Required
TRWD Gate Structure:			
Mass Foundation on Rock	5,112		
With Foundation Drains	5,013		
RCC Foundation to Rock	2,736	2,516	
Pile Foundation	2,736		60 HP 14x102 x 29-FT LG *
Trinity Point Gate Structure:			
Mass Foundation on Rock	9,378		
With Foundation Drains	9,265		
RCC Foundation to Rock	2,557	6,969	
Pile Foundation	2,597		58 HP 14x102 x 53-FT LG *
Clear Fork Gate Structure:			
Mass Foundation on Rock	5,363		
With Foundation Drains	5,148		
RCC Foundation to Rock	2,736	2,688	
Pile Foundation	2,169		64 HP 14x102 x 27-FT LG *

* Includes 5-ft rock embedment

Table 7-4
Abutment Structures Stability Analyses - Pile Foundations

Load Case	Base Width (ft)	Overturning Ratio	Eccentricity from Centerline (ft)	Comments
TRWD Abutment Structure:				
Maximum Water Level, East Abut.	25	1.76	1.81	
Maximum Water Level, West Abut.	25	1.91	1.69	
Trinity Point Abutment Structure:				
Maximum Water Level, East & West Abut.	31	1.42	4.72	100% Base Compression Controls
Clear Fork Abutment Structure:				
Maximum Water Level, East Abut.	47	1.41	7.45	100% Base Compression Controls
Maximum Water Level, West Abut.	45	1.52	7.14	100% Base Compression Controls

Table 7-5
Deep Training Wall Stability Analyses Results
(26-ft Base Length)

Load Case	Total Lateral Force (kips/ft)	Overturning Ratio	Eccentricity from Centerline (ft)	Comments
I1 Normal Loading Condition:				
I1NSPILE	15.5	2.10	1.57	Full depth water-filled crack
I1NLPILE	31.9	1.54	4.33	100% Base Compression Controls
I1FSPILE	18.2	2.27	1.88	Full depth water-filled crack
I1FLPILE	34.2	1.67	4.18	100% Base Compression Controls
I4 Construction / Maintenance Loading Condition:				
I4CSPILE	25.3	5.26	-0.05	Full depth water-filled crack
I4CLPILE	37.6	2.15	3.47	

Section 8
Samuels Avenue Dam

8.1 General Description

8.1.1 Purpose

The bypass channel is intended to pass significant floods without affecting the constant water levels in the original channel quiescent segment. A gated dam on the main stem of the West Fork will be located approximately 1,300 feet downstream from the confluence of Marine Creek and 1150 feet downstream from Samuels Ave. Bridge. The dam, shown on Drawing SS-2, Volume II, will maintain the normal water level elevation of 524.3 feet, during non-flooding conditions, throughout the upstream area and will have sufficient gate discharge capacity to pass the appropriate design flood flows, while maintaining flood levels within current conditions.

8.2 Dam Location

The dam was sited downstream from Samuels Ave. Bridge and the three adjacent railroad bridges for multiple reasons, including:

- Safety – It was assumed that a safety plan could be more easily implemented to keep the public away from the dam, particularly during flooding situations, by installing a buoy system immediately upstream from the Samuels Ave. Bridge and placing the dam downstream from the bridges. This will also allow for more open, usable water surface upstream from Samuels Ave.
- Aesthetics – During workshop discussion, the Urban Designer Consultants expressed a preference to keep the dam downstream from the bridges in order to provide the appearance of a larger lake surface, a more attractive perspective of the bridges, and to reduce the visual impact of the dam.

- Cost – Estimates showed the cost of the downstream site was slightly higher, but not substantially different from placing the dam upstream from the Samuel Ave. Bridge.

After the decision was made to place the dam downstream from the bridges, the actual location was specifically set with the front, upstream edge of the structure as approximately 600 feet downstream from the centerline of the south abutment of the easternmost railroad bridge. As shown on Drawings SS-1 and SS-2, Volume II, this location was selected in order to provide sufficient room for the structure to be constructed with appropriately sized training walls that transition the approximately 250-foot wide channel to the 390-foot wide dam structure (inside dimension) and back on the downstream side, as well as for a suitable diversion channel and coffer dams. The dam was oriented so that its discharges would line up with the downstream channel. This also allowed the training walls to have relatively consistent lengths and shortened the total length needed between the coffer dams.

The diversion channel was sized to be 75 feet wide with 2: 1 side slopes. This would pass the 10-year flood of 24,400 cfs with less than a 1-foot rise through the diversion channel. The diversion channel is to be located in the south abutment in order to avoid a large amount of rock excavation that would be needed in the north abutment. The coffer dams were assumed to have 10 feet top widths at an approximate elevation of 520 feet and 2:1 side slopes with riprap on the outside faces. Actual dimensions and locations of the diversion channel and the coffer dams will be set by the contractor.

8.3 Gate Configuration

Several alternatives for the dam and its means of handling flood discharges were reviewed, both in previous studies and in preparation for this submittal, including the following:

- Leaf, crest, or bascule, gates that operate by lying down with released water flowing over the top of the gate;
- Radial gates that operate by rotating upwards, allowing floodwaters to flow underneath;
- A rubber dam, that would hold the water when inflated and release it as it deflates; and
- An Obermeyer gate, which is a steel leaf gate supported and controlled by an inflatable rubber bladder.

Based upon input from various workshops, it was determined that leaf gates would be preferable since they:

- Were estimated to cost the same or less than the other alternatives;
- Would require much less of a visible structure over the lake than the radial gates;
- Would provide more flexible release control during smaller flood events; and
- Would be somewhat more dependable than the rubber dam or the Obermeyer structure.

Based on hydraulic modeling of the various flood flows on the river, including the SPF, the dam was sized to operate with seven 48-feet wide and 18-feet high gates. The gate width was chosen as the maximum reasonable width, enhancing the hydraulic capacity, while having operable gates that aren't too heavy. This assumes a concrete weir crest elevation, with the gate in its lowered, open position, of 507.0 feet. Operating equipment for each gate will be located on each pier, accessed by a bridge across the structure set just above the SPF elevation. The gates can be operated either by a hydraulic system or by a lifting cable and drum system. The advantages and disadvantages of each are briefly described below.

- Both would be considered dependable, assuming that hydraulic lines are readily accessible for maintenance and repair. This will require a bridge across the structure with all hydraulic lines run along the bridge.

- The hydraulic system would be better suited for the large force needed to lift the weight of the gate when water is flowing over it, though either system would be appropriate.

- External controls that might be impacted by flowing water would consist of two hydraulic cylinders for each gate or a series of wire ropes from the drum to the gate. Each would require an offset in the shape of the intermediate concrete piers, to minimize the potential for debris being caught in the system.

Based on manufacturer's recommendations, a hydraulic lift system is recommended for further detailed study. The drawings show the hydraulic lift, though only minor modifications would be needed for a cable drum system.

Stop logs and appropriate slots will be constructed to allow for future maintenance of the gates. The stop logs can be stored on the front side of the bridge. These could be put in place either from a barge mounted crane or a separate crane located on the flat access road located immediately downstream from the gate piers. A ramp allowing access to the downstream side of the gates would be located in the south abutment, as shown in Drawing SS-1, Volume II. Two alternatives for consideration would be the construction of a monorail system above the stop logs that would allow for the placement of the stop logs without the mobilization of a large crane or to size the bridge to hold a suitable crane.

A separate small control building to house the operational controls for the gates will be needed in the area. Possible locations would be attached to the dam itself, on or near the south levee, or adjacent to the nearby railroad embankments, out of the effective flow area of the channel. Its floor will likely need to be set above the SPF. Its final location has not been determined.

8.4 Dam Features

In addition to the gates described above, the dam will have several features that are recommended at this time. Their layout and conceptual design criteria are described below.

The stilling basin for the dam was sized to fully contain a hydraulic jump for energy dissipation of the gate releases. In order to contain the hydraulic jump, the basin was set to an elevation of 491.0 feet, with the downstream exit channel graded to 495.0 feet. The lower portion of the stilling basin was set to a length of 105 feet, measured from the top of the slope at elevation 503.5 feet to the downstream exit channel. The critical configuration was based on two gates fully opened, which would pass slightly less than the 10-year flood, or 24,400 cfs. At higher flows, the tailwater rises sufficiently so that less stilling basin length would be required. The basin would not be of sufficient length to fully dissipate the energy from one gate fully open, which would release approximately 11,300 cfs, slightly less than the 2-year flood. For this reason, the gates are recommended to be operated using partial gate openings for multiple gates before any one gate is opened fully.

Vertical training walls will be required for both the approach and the exit to transition to and from the 390 feet wide structure to the approximately 250-foot wide channel. The walls on the upstream side will reach from the approach floor elevation of 500-feet to the top of the structure at 530 feet at the dam, and slope downward and into the channel with a 3:1 sloped grade behind the wall, as shown on Drawing SS-6, Volume II. Typical sections of the training walls are shown on Drawings SS-1 and SS-2, Volume II.

The dam structure will be constructed primarily of reinforced concrete, but will also incorporate RCC. The RCC will be used in the mass concrete portion of the structure underneath the gates. In addition to being more cost effective, the RCC can be placed much more quickly than reinforced concrete, facilitating construction while the river is being passed through the diversion channel. The preliminary geotechnical explorations showed that competent rock is located at a relatively high level in the left abutment, minimizing the amount of RCC needed in that area. However, the top of competent rock is about 20 to 25 feet below the bottom of the channel, which will require a large amount of overexcavation of poor material and replacement with RCC. The surface of the rock will need to be cleaned and treated prior to placement of the RCC. This will also require an extensive dewatering system as the top of the rock is well below the water table.

Both abutments require a positive cut off in the floodplain adjacent to the structure to minimize seepage losses once the normal water level is reached. The right, or south, abutment has as much as 50 feet of sands and gravels above the competent rock. This entire zone would need to be cut off. The most cost effective method would be the use of a soil-bentonite slurry trench cut off wall. It will be constructed from the back side of the structure wall to the toe of the levee. The left, or north, abutment has a relatively high rock level in the foundation and so only a relatively shallow cut off trench would be needed. This could be done either with an open cut off backfilled with compacted clay or a soil bentonite slurry trench, similar to the right abutment.

The spillway will require a bridge across the top of the piers for maintenance. The low girder elevation would be set at elevation 538 feet, just above the SPF level. The bridge would allow access to all operating systems as well as provide a means for routing the hydraulic lines to each gate operator. The 12-foot wide concrete bridge would be sized for foot traffic and possibly small vehicle loads. If vehicular traffic is allowed, a ramp to the bridge for access will be added. This is not shown in the figures or indicated in the cost estimates.

Another site grading issue is the inclusion of a short, wide berm on both the north and south bank of the river that will tie the dam to the base of the railroad bridge. The ground in this area on the south side is currently lower than the projected normal water level of 524.3 feet. This area will be graded up to about elevation 530 feet once the construction of the spillway is complete and the diversion channel can be backfilled. The area on the north side will be graded similarly, but is not dependent on completing the diversion channel. This is shown on Drawing SS-2, Volume II.

8.5 Spillway Operations

As part of the conceptual design of the dam and gates, several factors in the planned operation of the spillway gates were developed, including the following:

- Since a single gate will release water at a rate high enough to almost pass the 2-year flood when fully open and because downstream interests will need to be protected from sudden large releases of water, the stilling basin was sized on the assumption that multiple gates would be partially opened prior to any gate being fully opened. Though this will complicate the operational sequencing, it will provide for much smoother operations, both for the structure and downstream interests. For example, a 2-year flow of 12,100 cfs could be released with no rise in the reservoir if all seven gates were lowered 4.9 feet. The 5-year flood, or 18,800 cfs, would require all gates to be lowered 6.6 feet. The main gates are sized so that the 100-year flood, or 50,500 cfs, can be passed without a significant rise in the pool. The gates would have to be lowered approximately 13 feet to pass the 100-year flows.

- The gates are sized to have their tops at elevation 524.3, with no freeboard above the normal pool elevation. The gates will be designed to allow for water to flow over the top either when fully raised or partially lowered. Allowing flow over the top of the gates for smaller, more frequent rain events will simplify operations and reduce the frequency with which the gates are operated.

- A 4 -foot wide by 6-feet high low flow conduit will be located in each of the three interior piers, as shown on Drawings SS-3 and SS-4, Volume II. Each gate will pass approximately 530 cfs at the normal pool level. This configuration would allow for small rises in the pool to be absorbed and then released through the low flow gates in addition to small flows over the top of the gates. Once the water surface has risen an appropriate amount, which has yet to be determined, at least one of the flood control gates will need to be partially lowered to maintain the lake level, and flood operational sequences will apply beyond that point. This will minimize the use of the large flood gates and simplify the frequent operations. The gates would also allow for some limited flushing of silt from the bottom of the impoundment. The level of this gate would have to be determined based on water quality considerations. The final sluice gate configuration is still to be finalized.

A physical model study of the dam and its gate operations is recommended as part of the final design process. This will help in the design of the final configuration of the structure, particularly the stilling basin and adjacent erosion protection measures;

fine-tune its hydraulic control parameters, and validate appropriate gate operations and sequencing procedures.

8.6 Structural Design

8.6.1 Loads

8.6.1.1 Hydrostatic Loads

For the preliminary design, only the normal pool elevations were used. A normal pool headwater elevation of 524.3 feet was assumed. A design level of 525.0 feet was used for calculations, assuming some flow over the top of the gates. The tailwater elevation was taken as the downstream channel elevation of 495.0 feet, relative to the stilling basin elevation of 491.0 feet.

8.6.1.2 Silt Loads

To account for sediment build-up during flooding conditions, the silt elevation is assumed to be equal to the crest elevation, at 507.0 feet, below the gates on the crest section and on the upstream training walls. The submerged unit weight of silt used was 40 pcf with a coefficient of lateral earth pressure at rest of 0.6. Thus, the silt is assumed to apply a pressure based on 24 psf per feet of depth.

8.6.1.3 Training Wall Loads

Refer to section *8.7 Geotechnical Considerations* for geotechnical wall design criteria.

The design of the walls was controlled by consideration of the wall loads resulting from a rapid drawdown following a general inundation flood event. It was assumed that the water table in the retained soil was 10 feet higher than the water on the channel side of the dam. The walls were evaluated for a range of water elevations, where the highest water elevation on the retained soil side was at the grade elevation at the wall and the lowest elevation was five feet above the bottom of channel elevation. The water on the channel elevation was simultaneously considered to be 10 feet lower, following a rapid drawdown event, but never lower than the bottom of the channel. Wall heights ranged up to a maximum retained height of 36 feet. The 10 feet maximum differential for use in the preliminary study was intended as a conservative estimate. The actual magnitude of the differential head is a function of backfill permeability, the rate of drawdown of the river, and the proximity of drainage provided by adjacent slopes. Thus, a simple application of drain efficiency was not considered realistic since the driving head varies with time as the river level lowers. It is anticipated that final design will be based on the estimated river drawdown rate and permeability of the wall backfill. Preliminary soil data is limited; however, additional data should be an achievable design parameter. During final design, this assumption will be verified or a suitable backfill capable of meeting this parameter will be specified. It is not anticipated that the efficiency of the drain itself, exclusive of backfill, will be the controlling factor.

8.6.1.4 Uplift Loads

Uplift potential on the dam section was computed based on a linear transition (measured horizontally) from full headwater pressure at the upstream edge of the dam to full (unreduced) tailwater elevation at the downstream sill of the stilling basin. A line of relief wells, combined with a continuous drain, was provided at a point within the dam and a continuous drain was provided at the toe of the dam. It is anticipated that the relief wells will only be utilized at dam sections without a significant amount of RCC below. Each line of drains was assumed to have an efficiency of 50%. The uplift at the sections with the drains was assumed to vary linearly from the upstream edge to the drains and then to the downstream sill with the head at the drains being midway between the straight-line transition from headwater to tailwater and the tailwater elevation.

8.6.2 Analysis

Overturning and sliding stability of the dam and abutments were analyzed using the methodology and stability criteria of USACE document Gravity Dam Design, EM 1110-2-2200, dated 30 June 1995. The training walls, which are not integral to the dam structure, were analyzed using the methodology and stability criteria of USACE document Retaining and Flood Walls, EM 1110-2-2502, dated 29 September 1989. Copies of Design calculations are included in Volume IV.

8.6.3 Materials

Concrete: 28-day compressive strength of 4,000 psi
Reinforcing steel: ASTM A615, Grade 60

8.6.4 Concrete Design

Concrete member design was based on ultimate strength design. Load factors were based on the USACE document Strength Design for Reinforced-Concrete Hydraulic Structures, EM 1110-2-2104, dated 30 June 1992. All dead and live loads were multiplied by a basic load factor of 1.7 and a hydraulic load factor of 1.3.

8.7 Geotechnical Considerations

8.7.1 Geologic Setting

The proposed dam site lies near the middle of a large U-shaped bend of the Trinity River. The river valley is entrenched in the sedimentary bedrock formations of the Lower Cretaceous Age. At the dam site these may include the Kiamichi , Goodland and Walnut Formations. The Kiamichi is primarily weak shale with thin limestone beds. The Goodland is mostly massive beds of relatively strong limestone, but with thin layers of shale. The Walnut includes both limestone and shale. The river has filled this trench in the bedrock with alluvium that includes clay, sand, and coarse materials ranging from gravel to large boulders. The alluvium-filled floodplain is nearly a mile wide at the dam site.

The geotechnical data provided to date consists of two borings drilled on the banks by the USACE. These borings show drastically different subsurface conditions. The boring on the north bank encountered limestone bedrock at a depth of 17 feet, well above the elevation of the river bed. The boring on the south bank encountered limestone bedrock at a depth of 53 feet, about 27 feet below the river bed and about 40 feet lower in elevation than the north boring. The change in the bedrock surface must occur mostly below the river and within the length of the proposed dam. The surface is probably stepped, and the intervening rock layers may include shale much weaker than the limestone that was sampled.

8.7.2 Design Issues

The dam construction will require rock excavation at the north end. The preliminary design assumes that the entire structure will be supported on bedrock, requiring excavation more than 25 feet below the riverbed to expose and prepare the bedrock and placement of roller compacted concrete (RCC) up to the bottom of the reinforced concrete dam structure. It will be necessary to define the bedrock configuration with some modest level of detail to enable reasonably accurate excavation cost estimates. The geotechnical investigation should address the question whether the dam might be supported adequately on a combination of rock and soil to reduce overexcavation and RCC costs.

Sliding stability is an important issue for concrete dams. The geotechnical investigation needs to determine the nature of the bedrock and soil materials throughout the possible range of elevations of the dam foundation and evaluate their shear strength adequately to allow a safe design that is not excessively conservative.

The soils encountered above the bedrock in both borings included sand and gravel with cobbles and boulders. Some of these layers could be highly permeable, posing seepage control issues both for the permanent condition and for temporary dewatering of the excavation during construction. Underseepage control can affect hydrostatic uplift, which influences the sliding stability. Seepage around the dam involves issues of water loss, unstable banks where the water re-enters the river, and the possible effects of the raised water table on existing facilities that may be several thousand feet from the dam. The investigation should attempt to evaluate the permeability of the soils in-situ. This will probably require controlled pumping tests of carefully installed wells. The coarse nature of the soils may necessitate the use of rather large-diameter wells and large drilling equipment. The presence of cobbles and boulders will also affect the selection and construction cost of a hydraulic cut off such as a slurry trench or sheet pile diaphragm. It is difficult to accurately assess the presence of scattered boulders or nested cobbles with a limited number of comparatively small borings.

The strength and cost of RCC depends on the mix design and the cost and source of the aggregates, cement and fly ash used. Preparation and testing of trial mixes is useful in developing a consistent, workable and economical mix design.

The preliminary design of the dam section was controlled by overturning and sliding criteria. The sliding design of the south end of the dam below the RCC was based on a friction angle between the RCC and bedrock of 25.0° with no cohesion. This assumption was based on the expectation that the bedrock contains horizontal layers of shale. The sliding design of the north end of the dam, where the rock is relatively shallow, was based on friction angles of 20° and 50° for horizontal and inclined failure planes, respectively, with no cohesion. The low friction angle for the horizontal failure plane was based on the presence of thin shale seams within otherwise competent limestone. It is assumed that the shear strength along an inclined failure plane would be controlled by the strength of the majority of material, which is limestone rock.

The preliminary design of the abutments was controlled by overturning stability since sliding resistance was provided by the dam and basin. The preliminary design of the upstream and downstream training walls was controlled by the overturning and sliding stability analyses. Due to the lack of quantifiable information about the soils behind and below these walls, it was assumed that the material retained by the walls and below the walls was uniform. The limited information in the soil borings indicated a medium dense sand/gravel alluvium. The stability of the existing channel banks with an angle of 33.7° from horizontal indicates fairly competent materials within the sloped bank zones. The borings provided were not considered adequate for determining material stratification. In order to facilitate analysis without assuming both material strengths and stratification, an engineering judgment was made to assume a uniform material with an internal friction angle of 32° and no cohesion.

8.7.3 Recommended Geotechnical Exploration and Geologic Studies

A search should first be made of applicable geologic reports and papers, and a search for available information developed for other infrastructure projects in the area. This may allow for a reduction in the amount of new information that will be needed. The proposed exploration of the area should be as follows:

Borings should be located at a nominal 100 feet maximum spacing across the upstream and downstream limits of the dam/RCC foundation, and additional borings should be located along the centerline or within the interior of the dam footprint. These borings should all be extended at least 10 feet into bedrock. Several borings where the bedrock is shallow should extend at least 10 feet below the lowest bedrock surface found on the site to be sure all intervening formations are sampled. At least two borings should continue at least 30 feet below the lowest bedrock surface. The bedrock should be cored, preferably with HQ or larger core barrels. If weak or highly weathered rocks that recover poorly are encountered, some borings should be augered and the rock evaluated with the Texas cone penetrometer.

Borings extending at least 10 feet into unweathered bedrock should be drilled at nominal 100 feet spacings along the two proposed cofferdams.

At least six borings should be drilled on several cross-sections along the diversion channel to well below the excavation depth.

Borings extending at least 10 feet into bedrock should be drilled along the extended centerline into the abutments to evaluate conditions for the cut off. A 100-foot wide spacing can be planned close to the structure, and a few borings at 200- to 400-foot wide spacing should extend well beyond the dam, especially on the right side, where seepage around the dam may be a critical issue.

The presence of cobbles and boulders poses challenging problems for advancing the borings, stabilizing the boreholes, and obtaining useful samples. Cable-tool drilling equipment may prove very useful for this site. The USACE may be the only local agency so equipped. Large rotary drills can also be useful. Ordinary geotechnical drilling equipment may be of limited effectiveness.

A tentative list of boreholes is presented below:

FEATURE	BORINGS	AVERAGE TOTAL DEPTH	AVERAGE CORING	REMARKS
Main Dam	10	65	25	6 require barge equipment
Cofferdams	8	45	10	4 require barge equipment
Diversion	8	30	NA	
Abutments	6	65	10	

At least one test well and at least four piezometers or observation wells are needed to evaluate the permeability of the coarse materials in the right abutment. Most of the piezometers can be installed in the exploratory boreholes listed above.

8.7.4 Laboratory Testing

Testing of soil samples should include classification tests (PI and -#200 sieve), grain size distribution for coarse-grained soils, and unconfined compression for cohesive soils. Drained triaxial compression testing of coarse-grained soils may be desirable if a foundation supported on soil is to be analyzed. Consolidation tests will be needed if compressible clays are encountered below foundation level. Rock testing should include moisture content, density and unconfined compression testing of representative specimens of each major rock type encountered. Direct shear testing of shale specimens and of model RCC/rock interfaces will be needed for the sliding stability analysis.

A preliminary list of estimated laboratory tests is shown below:

TYPE OF TEST	NUMBER OF TESTS	REMARKS
% passing #200 sieve	90	
Liquid and plastic limits	60	
Sieve analysis	90	
Unconfined – soil	60	Include water content and density
Consolidation	6	
Unconfined - rock	60	Include water content and density
Triaxial compression	4	Multi-stage
Direct shear - rock	12	
RCC trial mix	12	Assumes 2 aggregates, 6 cement-ash %
RCC compression	72	3 ages with duplicates

8.7.5 Geophysical Investigations

Downhole geophysical logging can be used to help define the stratigraphy if coarse materials or weak rock prevents adequate sampling and logging. Resistivity or seismic profiling can be considered to add detail to the rock surface profiles.

Section 9
Storm Water Pumping Station

9.1 General Description

9.1.1 Purpose

The construction of the bypass channel, isolation gate structures, and levee system will change the storm drainage patterns within the project area. Since the bypass channel and new levee system will convey the flood flows, the existing levees on the river segment contained within the isolation gates will no longer be needed. It is proposed that the existing interior levees will be removed to enhance access to the waterfront once the bypass channel is in place. The purpose of this section is to provide information on the requirements for a storm water pumping facility which will transport storm water from within the interior area, (within the isolation gates), maintain a fixed pool elevation and drain the interior for maintenance purposes.

9.1.2 Scope of Work

Included in this preliminary submittal are the following:

- Capacity analysis of the pump station.
- Revised drainage area map for the project area.
- Preliminary concepts for the storm water pumping facility.
- Preliminary design for a pumping facility, including; design assumptions, sizing parameters and criteria, layouts, and equipment recommendations.

9.1.3 Criteria

Design criteria for the storm water pumping facility shall include:

- General Principles of Pumping Station Design and Layout, USACE, EM-1110-2-3102.
- Mechanical and Electrical Design of Pumping Stations, USACE, EM-1110-2-3105.
- Safety and Health Requirements Manual, USACE, EM385-1-1.
- Design and Construction of Levees, USACE, EM-1110-2-1913.
- Hydrologic Analysis of Interior Areas, USACE, EM-1110-2-1413.
- City of Fort Worth Floodplain Development Permit (FDP).
- Tarrant Regional Water District design criteria.
- Hydraulic Institute Standards

- The USACE criteria for construction within the limits of existing Federal Flood Protection Projects (pamphlet No. 1150-2-1), prepared by the Fort Worth District.
- Planning, design, construction and maintenance requirements of Federal Emergency Management Agency (FEMA) including 44 CFR Chapter 1, Section 65.10.
- The structural design of the storm water pump station will be performed in accordance with following agency requirements and documentation:
    - City of Fort Worth;
    - International Building Code (IBC) 2003; and
    - American Concrete Institute (ACI) ACI-350R Code Requirements for Environmental Engineering Concrete Structures (as noted).
- USACE requirements, including;
    - EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures;
    - EM 1110-2-3104, Structural and Architectural Design of Pumping Stations; and
    - EM 1110-2-2906 Design of Pile Foundations.
- Local, state, and federal regulations, codes and laws shall be followed.

9.2 Storm Drainage Area and System

9.2.1 Existing Storm Drainage Area and Collection System

The collection of storm water in the project area includes sheet flow on the ground and in streets and some underground storm drainage conduits. All of the excess storm water is ultimately directed to the Clear Fork and the West Fork of the Trinity River. The area to the north and west of the project area drains by overland flow and storm drainage pipe systems towards the bypass channel. The existing storm drainage basins are shown on Drawing CU-SD-36, Volume II.

Immediately north of downtown Fort Worth, in the area of the Trinity River, there are three distinct drainage areas, shown as the Existing River, Existing Main, and Existing Southwest (SW) drainage areas on drawing CU-SD-36, Volume II. The Existing River basin (351 acres) consists of numerous small drainage areas, each of which contribute directly, as separate outfalls, into the Trinity River either from small systems within the northern downtown area, or from the banks of the existing Clear Fork and North Main Levees. The Existing Main basin (473 acres) is shown to exist as three sub-basins: the South Main sub-basin (193 acres), the West Main sub-basin (225 acres) and the North Main sub-basin (55 acres), which combines as a single discharge through the North Main Levee (Sta 108+50), through the existing Flood Gate Structure No. 26. The existing Southwest basin (129 acres) is shown to exist as three sub-basins: the SW-1 sub-basin (72 acres), which combines with the SW-2 sub-basin (40 acres) to discharge through the Clear Fork Levee (Sta 69+00), through the existing Flood Gate Structure No. 25, and the separate SW-3 sub-basin (17 acres), which discharges separately through the Clear Fork Levee (Sta 56+50) via an unnamed outlet.

The existing portion of the Clear Fork and West Fork Trinity River has been mapped by FEMA under its Flood Insurance program, and a FEMA regulated 100-year floodplain has been established. The project will change the current 100-year floodplain boundaries with the addition of the bypass channel and new levee system. Also, the removal of the existing levees contained within the gated area will change the 100-year floodplain delineation. A Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR) will be required for the project.

9.2.2 Proposed Storm Drainage Area and Collection System

New storm drainage outfall structures will be constructed at points where the existing drainage system will be intercepted by the bypass channel. The outfall structures will provide transfer of storm water from the existing system into the bypass channel, either through gravity systems, a pumped system or a combination. See discussion contained in the Earthwork, Utility Relocation sections of this volume and Drawings, Volume II for details on the bypass channel. The following paragraph describes the delineation of the existing and proposed drainage basins.

The location and orientation of the proposed bypass channel is such that it bisects the three primary drainage areas, creating a new single drainage area (612 acres), shown on Drawing CU-SD-37, Volume II. The new drainage area will be isolated by the three isolation gates when the river is in flood stage. This interior area will drain into the proposed urban channels or into the section of the Trinity River that is located between the isolation gate structures. The three isolation gate structures, the Clear Fork Gate, the Trinity Point Gate and the TRWD Gate are also shown on Drawing CU-SD-37, Volume II. With the gates closed, the interior pool would then be subject to localized rainfall events within the 612-acre pool drainage area, which includes some of the northern downtown Central Business District.

9.3 Storm Water Pumping Station

9.3.1 Location

The site location of the storm water pump station should be near the bypass channel to provide the most direct and efficient transfer of storm water to the floodway. This will reduce the head requirements for the pumping units and minimize the amount of discharge piping. The pump station should also be located to minimize environmental and water quality impacts.

The proposed location of the pump station is adjacent to and upstream of the proposed TRWD isolation gate, located on the east bank of the existing West Fork channel. The pump station location is shown on Drawings CI-2 and CI-3, Volume II. The proximity of the pump station to the proposed TRWD Isolation Gate Levee is such that pump discharge length will be less than 50 feet. The location is also in close proximity to the TRWD Main Office and Maintenance and Operations Center.

For ease of access, a portion of the site will be filled to approximately the same elevation as the levee, 546.6 feet. The working pump platform area will be at a lower elevation of 539.5 feet, to minimize the pump column height. Access to this platform will be via stairs. Plans and sections of the storm water pump station are contained on drawings SS-20, SS-21 and SS-22, Volume II.

9.3.2 Capacity and Operation

Detailed hydrologic analysis was used to determine the preliminary sizing of the storm water pump station. Final hydrologic analyses and operating controls will dictate the ultimate design capacity of the storm water pump station needed to protect the property adjacent to the isolated interior pool. A preliminary analysis has been performed for determination of the pumping capacity of approximately 300 cubic feet per second (cfs). This capacity will utilize some of the interior pool's storage capacity, while retaining a relatively constant pool elevation (rise no greater than 3 feet). The hydrologic analysis for the storm water pump station is contained in, Appendix A - Hydrology and Hydraulics.

After the bypass channel is constructed the existing river channel contained within the isolation gates will be filled to an approximate elevation of 515 feet.
The layout of the storm water pump station is such that it can be utilized, either during a localized rainfall event occurring simultaneously while the river is in some flood stage, or as a maintenance drain service. Total preliminary capacity of the pump station is 300 cubic feet per second (cfs), which will be supplied by 4 (four) 100 cfs pumps. The fourth pump, which will serve as a back-up to the three main pumps, will function primarily to drain the interior pool, as required for maintenance purposes, to an elevation of approximately 515 feet. As such, its intake channel elevation is at a lower elevation of 505.5 feet.

The intake channels for these pumps are located at a higher elevation (flow line = 513.0 feet) which is appropriate for the proposed pool elevation (524.3 feet). The intake structure is 20 feet by 50 feet wide; with the area at each pump roughly 20 feet by 12.5 feet wide. Stop log guides will be included to allow dewatering of the pump slots for maintenance and repair.

The storm water pump station will be operated remotely through the use of a SCADA system controlled by specific rainfall and/or local lake levels. Appropriate back-up systems will be in place. Manual operation of the pump station will also be allowed.

9.3.3 Pumps and Equipment

The preliminary pump selection for the storm water pump station is a vertical type, with axial or mixed flow. The pump station will operate at approximately 24 feet of total dynamic head (TDH), under normal conditions, typical of a storm water pump station. During draining operations, the TDH is approximately 32 feet. Since the pump station is a part of a flood-protection project, the pumping units shall be highly reliable and require minimum maintenance. The pump motor capacity is estimate to be less than 400 Hp.

Individual conduits direct the flow from the pump station across the top of the levee to the river side. The conduits consist of 48-inch steel pipe, minimum 0.25 inch thickness, with a mortar lining and coal tar coating. The pipes will be embedded in concrete on the top of the levee to handle traffic loads on the levee. A siphon breaker shall be installed at the high point of the pipeline at 545.6 foot elevation. The flowline of the steel pipe is 541.3 feet, which is 2.3 feet above SPF elevation of 539.6 feet. The storm water discharge piping is terminated at the discharge structure.

The pumps, motors and other equipment will be housed in the building. The building will have a roof hatch to remove the pumps and motors for maintenance and repair. Mobile cranes will be used to remove and replace pumps and motor units as necessary. A monorail hoist will also be included in the building.

The intake channels will be isolated from the river channel utilizing buoys or other safety screening devices , connected across the intake structure with cabling, thereby, preventing direct access via the channel into the pump station. Additionally, trash racks will further isolate the pump intake from the intake channels. The trash racks will prevent large floating debris from entering the pump suction, causing operational and maintenance problems. Trash racks should be equipped with a cleaning device that is power operated due to the size of the trash racks. Stops will be included at the intake structure to allow dewatering for repair and maintenance of the trash racks. Disposal containers will be brought into the site periodically for proper disposal of trash. The trash racks should be inclined for ease of maintenance. The size and spacing of the trash racks should be determined after performing field survey of the type and potential quantity of trash expected.

Electrical supply at the capacity needed for the pump station is available near the proposed pump station site. The pump station will be equipped with emergency power supply, consisting of a diesel generator or other reliable and feasible supply facilities. It is anticipated that the local utility supplier will require power factor correction capacitors and soft start for the pumping unit motors.

9.4 Structural Design Requirements

Structural design parameters are selected based on the requirements and recommendations applicable to USACE Engineering Manuals (as referenced). Geotechnical design parameters are based on preliminary estimates of soil properties;

wall and foundation designs will be updated based upon site specific subsurface investigations and evaluation of soil properties when available.

The foundation design for the pump station is based upon the data from soil boring F-1. The location of the boring is shown on Drawing SS-20, Volume II and the boring log is included within the Geotechnical Appendix. This boring indicates the possibility of a limestone material at elevation 478, approximately 25 feet beneath the lowest level of the structure It also indicates soft, weak sandy clay for some distance above elevation 482. The unconfined compressive strength of this clay layer is estimated to be in the range of 0.5 tons per square foot.

Due to the weak soils below the base of the structure, steel H-piles are proposed to be used for vertical and lateral support of the pump station. The proposed foundation would consist of 120 battered steel H-piles configured to resist the combination of lateral and vertical forces. Some of the adjacent retaining walls, with heights greater than 25 feet, will also require H-piles. The H-piles are to be founded into the bedrock. Deeper borings will need to be drilled to confirm foundation design assumptions. Concrete drilled shafts may be considered as an alternative to the steel H-piles in the final design.

For the exterior walls at the pump station, the levee fill material is anticipated to be plastic clay. Wall design must consider saturated soil conditions, due to drawdown conditions as well as maximum flood levels. Drawdown conditions may occur prior to a flood as part of the floodway operations or for routine maintenance of facilities. Thus, preliminary wall design includes a lateral pressure from the undrained soil combined with the lowest possible pool level within the pump station. As a result of the large lateral loads, the structure will require counterfort walls to stiffen the wall panels. The counterforts will be provided on the soil side of the intake walls to avoid interference with the flow within the intake and to avoid internal beams/struts with column supports, which potentially would interfere with the trash racks. The interior divider walls between the pumps will act as buttresses for the end wall.

Exterior walls have been designed for a minimum surface surcharge pressure equal to 2 feet of soil. This will be confirmed in final design if a paved area is to remain within a horizontal distance from the top of wall equal to one-half the wall height (Ref. AASHTO Bridge Specifications, Section 5.5.2).

The three-sided configuration of the pump station limits the load cases to be investigated. The walls, that contain the normal pool within the inlet area, will likely have water on both sides of the wall during normal operations and will generally extend below existing groundwater levels. The following loading conditions have been considered:

> Loading Condition No. 1, Construction: Dead loads and equipment loads. No live loads, soil loads, or water loads (for maximum vertical loads without lateral loads on piles.)

Loading Condition No. 2, Construction or Maintenance: Dead loads with undrained (saturated) soil loads and uplift. No equipment loads, live loads, or internal water loads (for minimum vertical loads with lateral soil loads).

Loading Condition No. 3, Normal: Dead loads, equipment loads, live loads, internal water loads, and drained (dry) soil loads, excluding uplift (for maximum vertical loads on piles).

Loading Condition No. 4, Normal: Dead loads and equipment loads with undrained (saturated) soil loads and uplift. No live loads or internal water loads (for potential uplift on piles).

A seismic loading condition was also evaluated using an effective ground acceleration of 0.05g along with the seismic variables calculated for the floodwalls. It was determined that the seismic load condition is not a critical case.

The stability analysis for the storm water pump station is contained in Volume V.

9.5 Calculations

9.5.1 Storm Water Pump Station Calculations

The City of Fort Worth Floodplain Development Permit dictates adherence to the Federal Emergency Management Agency's requirements of providing protection of properties within the limits of the Federal Insurance Rate Maps to a 100-year flood level of protection. As such, using the U. S. Weather Bureau's Technical Paper No. 40 (Rainfall Frequency) for a 100-year 24-hour rainfall for Tarrant County, Texas, the hydraulic capacity of the pump station is determined thru hydrologic analyses. The hydrologic analyses are contained in Attachment B, Appendix A - Hydrology and Hydraulics. Considered in the analyses were the drainage area, available pool storage, and the desire to minimize the rise in water surface from the constant pool elevation. Additional pump station design criteria includes utilizing a TxDOT Type II, synthetic 100-yr 24-hr rainfall distribution, a flood hydrograph, routed through a reservoir using a storage-indication routing method, and the assumptions that the isolation gates are operated in a manner such that they are closed at a water surface elevation of 526.0 feet and the minimum pool water surface elevation is 523.3.

9.5.2 Structural Calculations

Preliminary concrete design for the pump station was in conformance with ACI-350 Code Requirements for Environmental Engineering Concrete Structures using Attachment A, Alternative Design Method (working stress design method). Final design of concrete elements should be in accordance with EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures.

The following unit weights have been assumed in the analyses for pump station:

- Concrete unit weight = 150 pcf
- Water unit weight = 62.5 pcf
- Soil unit weight = 120 pcf

Although final site-specific soil parameters have not yet been determined for this project, estimated soil design parameters have been established based on preliminary soil's investigations. For preliminary analyses, the following soil design parameters have been used:

- Drilled shaft bearing capacity (end bearing plus skin friction) = 5 tsf + 1.25 tsf/LF.
- Equivalent fluid pressure based on an "at-rest" condition = 120 psf per foot of depth.

Detailed structural calculations are contained in Appendix C, Volume III of this report.

Section 10
Retaining Walls

10.1 General Description

10.1.1 Purpose

This section summarizes the preliminary structural design of the concrete retaining walls to be constructed within the proposed bypass channel for the Fort Worth Central City Project.

10.1.2 Scope of Work

This section includes descriptions of proposed concrete retaining walls, a summary of design requirements, and a summary of results of the preliminary analyses performed to date.

The concrete retaining walls within the new flood bypass channel generally form the east side of the bypass channel (i.e. the "hard edge"). In general, there are three tiers of walls, including:

- Lower level walls (also referred to as Lower Interior Wall), which retain earth and contain the normal pool.

- Mid-level walls (also referred to as Middle Interior Wall), which retain earth above the normal pool level and below the SPF level.

- Upper level walls (also referred to as Upper Interior Wall), which retain earth and extend to an elevation 4 feet above the SPF.

Refer to Drawing SB-1, Volume II for the general configuration of the three tiers of retaining walls. (The two elevations drawn show the configuration with a taller upper level wall, which is typical of the upper bypass channel, and a shorter upper level wall, which is typical of the lower bypass channel.).

10.1.3 Criteria

The retaining walls are designed to meet the following general criteria:

- Conform to the wall configurations proposed by the urban design consultant.

- Conform to the channel configurations proposed in the Preliminary Civil Submittal; and

- Adhere to USACE Standards.

10.2 Design Requirements

10.2.1 General Design Parameters

The primary objectives of the bypass channel retaining walls are to retain earth and to form the channel profile, while meeting the intent of the urban design configuration and appearance. The walls must also be durable and resistant to flood flows. Since these walls assist in containing the floodway, they have been designed in accordance with USACE criteria for "Inland Flood Walls". The fill materials placed behind the concrete flood walls (retaining walls) will also assist in containing the floodway.

The walls must consider drawdown conditions as well as maximum flood levels. Drawdown conditions will occur at the upper level and mid-level walls when a flood is receding. At the lower level walls, drawdown may occur prior to a flood as part of the floodway operations or for routine maintenance of facilities. For this condition, consideration must be given to the lowest possible pool level at Samuel Avenue Dam.

Structural design parameters are selected based on the requirements and recommendations of applicable USACE Engineering Manuals, as referenced. Geotechnical design parameters are based on preliminary estimates of soil properties. Wall designs will be updated based upon site specific subsurface investigations and evaluation of soil properties, when available. Long-term and short-term stress conditions are considered for clays, based upon expected drainage conditions as noted for the various loading conditions.

Where applicable for final designs, general environmental loads (wind, snow, seismic, etc.) should be based upon the 2000 International Building Code (IBC). Wind loads should be based on a Basic Wind Speed of 90 mph (3-second gust). Ground snow load is 5 psf and minimum foundation depths should be 12 inches.

It is proposed to specify a minimum 28-day concrete compressive strength of 4000 psi, with a maximum water-cementitious materials ratio of 0.50, and Grade 60 reinforcing steel.

10.2.2 Functional and Technical Requirements

Retaining walls have been designed in conformance with EM 1110-2-2502, Retaining and Flood Walls. Final design of concrete elements should be in accordance with EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, (in lieu of concrete design requirements specified in Chapter 9 in EM 1110-2-2502).

Retaining walls have been evaluated for sliding, overturning, and foundation bearing capacity. For concrete strength and foundation bearing capacity at retaining walls, resisting pressures have been limited to 50% of unfactored passive pressure. For sliding stability, driving and resisting earth pressures are based on the multiple wedge method with applicable factors of safety applied to soil properties.

Consideration is given to uplift, due to seepage along the base of structure, where applicable. The line of creep method for seepage analysis has been used in accordance with the USACE computer program CTWALL, Volume VI.

All channel retaining walls have been designed for a minimum surface surcharge pressure of 100 psf under normal operating conditions. A surcharge equal to two feet of soil may be considered in final design if paved streets are to be provided within a horizontal distance from the top of wall equal to one-half the wall height (Ref. AASHTO Bridge Specifications, Section 5.5.2).

The lower level walls, which contains the normal pool within the bypass channel, will likely have water on both sides of the wall during normal operations and will generally extend below existing groundwater levels. Due to potential rapid drawdown of the pool, these walls are proposed to be designed for the undrained condition with hydrostatic pressure included with the lateral earth pressures on the driving side, and with a low water condition on the pool side.

The mid-level and upper level retaining walls will only be exposed to channel flows and submergence during flooding and will generally be above normal groundwater elevations. These walls have been designed assuming partially drained conditions on the soil side, with consideration of a differential hydrostatic pressure for rapid drawdown conditions. For the drawdown condition, the water level on the driving side is assumed to be at 50% of the distance between the top of wall and the water level on the resisting side. Varied water levels are considered on the resisting side to determine the most critical design condition. A drainage system consisting of free-draining fill and slotted drainage pipes are proposed to be provided behind such walls to facilitate rapid drainage.

It is anticipated that the velocities due to flooding along the mid-level and upper-level retaining walls will generally not exceed 12 feet per sec under SPF conditions. Lower velocities are anticipated for the less severe but more frequent flooding events. The surfaces between the walls will be protected from erosion by a combination of walkways, native vegetation, such as turf grass, and other suitable vegetation for the anticipated velocity ranges each area may be exposed too. The vegetation will also be selected based upon the anticipated frequency of inundation from flooding. Thus, it has been assumed that passive pressures may be included in the evaluation of retaining wall stability during flooding.

In addition to local stability analyses of individual walls, overall slope stability has been evaluated by the Geotechnical Engineer.

Conceptual building foundations are shown on Drawing SB-1, Volume II near the upper level walls. Provided that excavations for building foundations do not extend past the limits of the upper wall footing and provided that properly designed braced excavation techniques are used for deep excavations, it is anticipated that building foundations can be installed safely. It is proposed to set a limit of excavation at 2 feet from the edge of retaining wall footing for a margin of safety.

10.2.3 Calculations and Results

10.2.3.1 General Design Assumptions

The following unit weights have been assumed in the stability analyses for retaining walls:

- Concrete unit weight = 150 pcf
- Water unit weight = 62.5 pcf
- Soil unit weight = 100 pcf dry weight and 130 pcf moist weight

Based on recommendations in the Initial Geotechnical Investigation, Appendix B, the following preliminary soil design parameters have been established:

- Maximum foundation bearing capacity on soil:
  - 2,000 psf for Usual Loading Conditions
    (Bearing Capacity Safety Factor = 3.0)
  - 3,000 psf for Unusual Loading Conditions
    (Bearing Capacity Safety Factor = 2.0)
  - 6,000 psf for Extreme Loading Conditions
    (Bearing Capacity Safety Factor = 1.0)
- Short-term design conditions (undrained soil): 1,000 psf cohesion with φ = 0-degrees (unfactored values).
- Long-term design conditions (drained soil): internal friction angle, φ = 27 degrees with 100 psf cohesion at lower level and mid-level walls, and φ = 27 degrees with 250 psf cohesion at upper level walls (unfactored). The cohesion values noted were included for resistance to sliding but were not included in the determination of lateral earth pressures; it is conservative to consider only friction for applied earth pressures.

For the seismic loading condition, an effective ground acceleration of 0.05g was used. According to the seismic maps included with the 2000 IBC, Fort Worth has a short period ground motion value (S_S) of 0.112g and a one-second (S_1) value of 0.055g based on a 2-percent probability of exceedance in within a 50-year period. For Site Class D (stiff soil), the corresponding design spectral response accelerations are S_{ds} = 0.12g and S_{d1} = 0.09g. The corresponding effective ground acceleration (0.40*S_{ds}) is 0.0473g.

Preliminary seismic analyses for stability are included herein, based on approximate methods using CTWALL. Estimated lateral forces due to earthquake were manually calculated and compared to the forces already included in CTWALL. Where the manually calculated seismic forces exceeded the static forces included in CTWALL, an added lateral force was input using the horizontal line load input under "Surcharge Loads".

Stability analyses of retaining walls were performed using the USACE computer program CTWALL. The factors of safety and stability criteria applied are in accordance with Table 4-2 of EM 1110-2-2502, for Inland Flood Walls.

10.2.3.2 Stability Analyses of Lower Level Retaining Walls

The lower level retaining walls have a 10-foot clear wall height and a 3.5-foot extension below grade. The lower level (LL) walls have been evaluated for the following loading conditions:

I1-N. Normal Loading Condition (See Drawing SB-2, Volume II):

Hydrostatic pressure for full height on driving side (El 530); water at normal pool level (El 524.3) on resisting side; passive soil pressure neglected on resisting side (due to sloped channel fill); 100 psf vertical surcharge pressure applied.

I1NSLL: Short-term soil properties assumed with partial-depth water-filled crack.

I1NLLL: Long-term soil properties assumed.

I1-F. Flood / Drawdown Loading Condition:

Water at any level on the resisting side, with water level on the driving side at 50% of the distance between the top of wall and the resisting side water level; passive soil pressure neglected (conservatively) on the resisting side; no surcharge pressure included.

I1FLLL: Long-term soil properties assumed.

I1FSLL: "Study" using short-term soil properties and a full-depth water-filled crack.

I3-E. Extreme (Earthquake) Loading Condition:

Similar to the Normal Loading Condition (normal pool level), except that no vertical surcharge pressure is included. Lateral seismic forces applied (0.05 g).

I3ESLL: Short-term soil properties assumed with partial-depth water-filled crack.

I3ELLL: Long-term soil properties assumed.

I4-C. Construction / Maintenance Loading Condition (See Drawing SB-2, Volume II):

Hydrostatic pressure for full height on driving side; water at any level below the normal pool elevation on the resisting side (including levels at or below the bottom of base); passive soil pressure neglected on the resisting side; 100psf surcharge pressure.

I4CSLL: Short-term soil properties assumed with partial-depth water-filled crack.

I4CLLL: Long-term soil properties assumed.

10.2.3.3 Stability Analyses of Mid-Level Retaining Walls

The mid-level retaining walls have a 10-foot clear wall height and a 3-foot extension below grade. The mid-level (ML) walls have been evaluated for the following loading conditions:

I1-N. Normal Loading Condition (See Drawing SB-2, Volume II):

Drained soil conditions above the drainage pipe on the driving side; passive soil pressure included on the resisting side (per noted limitations); 100 psf vertical surcharge pressure applied.

I1NML: Long-term soil properties assumed.

I1-F. Flood / Drawdown Loading Condition (See Drawing SB-2, Volume II):

Water at any level on the resisting side, with water level on the driving side at 50% of the distance between the top of wall and the resisting side water level; passive soil pressure included on the resisting side; no surcharge pressure included.

I1FLML: Long-term soil properties assumed.

I1FSML: “Study” using short-term soil properties and a full depth water-filled crack.

I3-E. Extreme (Earthquake) Loading Condition:

Similar to the Normal Loading Condition (normal pool level), except that no vertical surcharge pressure is included. Lateral seismic forces applied (0.05 g).

I3ELML: Long-term soil properties assumed.

I4-C. Construction Loading Condition (See Drawing SB-2, Volume II):

Soil conditions on the driving side as noted below; passive soil pressure neglected on the resisting side; 100 psf vertical surcharge pressure applied.

I4CSML: Short-term soil properties assumed with a full-depth water-filled crack on the driving side.

I4CLML: Long-term soil properties assumed, with drained soil conditions on the driving side (i.e. no hydrostatic pressure).

10.2.3.4 Stability Analyses of Upper Level Retaining Walls

The upper level retaining walls have 6-foot to 14-foot clear wall heights and a 3-foot extension below grade. The upper level walls are to extend 4 foot above the SPF. The upper level (UL) walls have been evaluated for the following loading conditions:

I1-N. Normal Loading Condition (See Drawing SB-3, Volume II):

Drained soil conditions above the drainage pipe on the driving side; passive soil pressure included on the resisting side (per noted limitations); 100 psf vertical surcharge pressure applied.

I1NL14UL: 14-foot clear height wall with long-term soil properties assumed.

I1NL6UL: 6-foot clear height wall with long-term soil properties assumed.

I1-F. Flood / Drawdown Loading Condition (See Drawing SB-3, Volume II):

Water at any level on the resisting side up to SPF level, with water level on the driving side at 50% of the distance between the top of wall and the resisting side water level; passive soil pressure included on the resisting side; no surcharge pressure included.

I1FL14UL: 14-foot clear height wall with long-term soil properties assumed.

I1FS14UL: "Study" using short-term soil properties and a full-depth water-filled crack.

I1FL6UL: 6-foot clear height wall with long-term soil properties assumed.

I1FS6UL: "Study" using short-term soil properties and a full-depth water-filled crack.

I3-E. Extreme (Earthquake) Loading Condition:

Similar to the Normal Loading Condition (normal pool level), except that no vertical surcharge pressure is included. Lateral seismic forces applied (0.05 g).

I3EL14UL: 14-foot clear height wall with long-term soil properties assumed.

I3EL6UL: 6-foot clear height wall with long-term soil properties assumed.

I4-C. Construction Loading Condition (See Drawing SB-3, Volume II):

Full earth retention on the driving side; passive soil pressure neglected on the resisting side; 100 psf vertical surcharge pressure applied.

I4CS14UL: 14-foot clear height wall with short-term soil properties assumed and with saturated soil and a partial depth water-filled crack on the driving side.

I4CL14UL: 14-foot clear height wall with long-term soil properties assumed and with drained soil conditions on the driving side.

I4CS6UL: 6-foot clear height wall with short-term soil properties assumed and with a full depth water-filled crack on the driving side.

I4CL6UL: 6-foot clear height wall with long-term soil properties assumed and with drained soil conditions on the driving side.

10.2.3.5 Results of Stability Analyses

Refer to Table No. 10-1 for a summary of the results of the stability analyses for the noted loading conditions. The summary shows the total base width of retaining wall used in the analysis for each load case, the calculated factor of safety against sliding, the maximum calculated soil bearing pressure, and comments regarding the results. Refer to Volume VI of this report for detailed output from the program CTWALL, including figures of applied loads at walls.

For the lower level walls, the Normal Loading Condition governed the design, due to the required sliding factor of safety (when using the long-term soil parameters). For the mid-level, the Flood / Drawdown Loading Condition governed the design due to the required sliding factor of safety. For the 14-foot clear height upper level walls, the Normal Loading Condition governed the design due to the maximum allowable soil bearing pressure. For the 6-foot high upper level walls, the requirement for 100-percent of base in compression governed the base width design for several loading conditions.

For the earthquake condition, the base width was set at the maximum width required by other loading conditions and the effective factor of safety calculated. Development of seismic loads used in CTWALL is included in Volume VI. Since the driving force applied in CTWALL for sliding analysis is based on the “calculated” factor of safety rather than the “required” factor of safety, the applied driving forces are higher than the actual applied loads. Where the manually calculated combined lateral static plus seismic force was less than the driving wedge force already included for sliding stability (with an effective factor of safety in excess of 1.1),the analysis was considered to be conservative for earthquake sliding analysis. Such cases where a negative seismic force was calculated, are noted in Table 10-1. The negative forces were neglected in the analysis for earthquake, yielding conservative results. However, where the manually calculated combined lateral static plus seismic force was greater than the driving wedge force included in CTWALL for sliding stability, additional analysis was necessary. CTWALL was then run with the option to calculate forces for the required factor of safety of 1.10. A horizontal surcharge load was then included to account for the difference between the manually calculated lateral forces and the driving force included in CTWALL. The analysis methods used are judged adequate for evaluation of sliding stability, but they are not accurate for overturning analysis for earthquake. Further analysis of overturning is proposed to be completed during the final design phase. For a lateral seismic coefficient of 0.05g, all earthquake load conditions had a factor of safety against sliding in excess of the required minimum.

Some “study” cases, using short-term soil properties for the flood/drawdown case have been included. (The case determined as the most critical drawdown case for long-term soil properties was analyzed using the short-term soil properties.) It is felt that the short-term soil properties do not need to be applied for this design condition, but the analyses are included for information only. The results indicate that these conditions would not control the wall designs.

It should also be noted that analyses in addition to those included in Volume VI were performed as “studies” using CTWALL. The walls were checked using various water levels on the resisting side, to determine the most critical drawdown design condition, using long-term soil properties. A “reverse” load for the SPF was also analyzed at the upper wall level, assuming flood as a driving force and zero passive resistance beyond the back edge of the footing. This is to account for potential future earth excavation beyond the heel of the retaining wall. The stability analyses results were satisfactory for this case.

Limited concrete analysis was performed using approximate design data from the results of these stability analyses. Resulting wall thicknesses and slab thicknesses are shown on the Drawings SB-2 and SB-3, Volume II.

In conclusion, the wall configurations depicted on the enclosed Drawings are considered to be representative of the required retaining walls (flood walls) along the bypass channels. The configuration of these walls may be refined during the final design phase.

Table 10-1
Inland Flood Wall Stability Analyses Results

Load Case	Base Width (ft)	Factor of Safety for Sliding	Max. Bearing Pressure (psf)	Comments	
LOWER LEVEL WALL (LL):					
I1 Normal Loading Condition (Required Factor of Safety = 1.50, Bearing Pressure = 2,000 psi):					
I1NSLL	10	2.64	1817	100% Base Compression Controls	
I1NLLL	16	1.53	1726	Sliding Controls	**Critical Load Case**
I1FSLL	12	2.11	2005	Bearing Pressure Controls	
I1FLLL	13	1.53	1655	Sliding Controls	
I3 Extreme / Earthquake Loading Condition (Required Factor of Safety = 1.10, Bearing Pressure = 6,000 psi):					
I3ESLL	16	3.10	1779	1.36 Kip Horiz. Load Included	
I3ELLL	16	1.10	2129	Forces As Req'd For Factor of Safety	
I4 Construction / Maintenance Loading Condition (Required Factor of Safety = 1.33, Bearing Pressure = 3,000 psi):					
I4CSLL	10.5	1.84	2234	100% Base Compression Controls	
I4CLLL	12.5	1.34	2447	Sliding Controls	
MID-LEVEL WALL (ML):					
I1 Normal & Flood Loading Conditions (Required Factor of Safety = 1.50, Bearing Pressure = 2,000 psi):					
I1NLML	10	1.72	1969	Bearing Pressure Controls	
I1FLML	11.5	1.54	1720	Sliding Controls	**Critical Load Case**
I1FSML	7.5	3.02	2005	100% Base Compression Controls	**Study Case**
I3 Extreme / Earthquake Loading Condition (Required Factor of Safety = 1.10, Bearing Pressure = 6,000 psi):					
I3ELML	11.5	2.36	1912	Negative EQ Force	
I4 Construction Loading Condition (Required Factor of Safety = 1.33, Bearing Pressure = 3,000 psi):					
I4CSML	10	1.89	2145	100% Base Compression Controls	
I4CLML	9	1.40	2402	100% Base Compression Controls	

Table 10-1
Inland Flood Wall Stability Analyses Results

Load Case	Base Width (ft)	Factor of Safety for Sliding	Max. Bearing Pressure (psf)	Comments	
UPPER LEVEL WALL - 14-FT SOIL DIFFERENTIAL (14-UL):					
I1 Normal & Flood Loading Conditions (Required Factor of Safety = 1.50, Bearing Pressure = 2,000 psi):					
I1NL14UL	16	1.86	1978	Bearing Pressure Controls	**Critical Load Case**
I1FL14UL	14.5	1.51	1914	Sliding Controls	
I1FS14UL	11	2.36	1983	Bearing Pressure Controls	**Study Case**
I3 Extreme / Earthquake Loading Condition (Required Factor of Safety = 1.10, Bearing Pressure = 6,000 psi):					
I3EL14UL	16	2.13	2729	Negative EQ Force	
I4 Construction Loading Condition (Required Factor of Safety = 1.33, Bearing Pressure = 3,000 psi):					
I4CS14UL	12.5	1.38	2540	Sliding Controls	
I4CL14UL	11	1.37	2773	Sliding Controls	
UPPER LEVEL WALL - 6-FT SOIL DIFFERENTIAL (6-UL):					
I1 Normal & Flood Loading Conditions (Required Factor of Safety = 1.50, Bearing Pressure = 2,000 psi):					
I1NL6UL	6.5	1.90	1608	100% Base Compression Controls	**Critical Load Case**
I1FL6UL	6.5	1.78	1306	100% Base Compression Controls	**Critical Load Case**
I1FS6UL	4.5	6.08	1155	100% Base Compression Controls	**Study Case**
I3 Extreme / Earthquake Loading Condition (Required Factor of Safety = 1.10, Bearing Pressure = 6,000 psi):					
I3EL6UL	6.5	2.24	1654	Negative EQ Force	
I4 Construction Loading Condition (Required Factor of Safety = 1.33, Bearing Pressure = 3,000 psi):					
I4CS6UL	6.5	2.57	1375	100% Base Compression Controls	**Critical Load Case**
I4CL6UL	6	1.56	1686	100% Base Compression Controls	

Section 11
Operation & Maintenance

11.1 General Description
This Section discusses the preliminary plan to operate and maintain the proposed bypass channel levee system, Samuels Ave. Dam, three Isolation Gate Structures, a Storm Pump Station, and Valley Storage Mitigation areas.

Channel and levee side slopes are tentatively planned for 3 horizontal to 1 vertical slope, similar to what has been successfully used and maintained for the existing Trinity River channel. Retaining wall structures are proposed along the east side of the bypass channel in three tiers; lower level interior walls at about normal pool level, mid-level interior walls above normal pool level and below Standard Project Flood (SPF) level, and upper level interior walls above the SPF level.

A dam with leaf gates and three separate isolation gates are planned to control water levels in the bypass channel and interior area. The dam with leaf gates is proposed in the vicinity of Samuels Ave. The isolation gates are planned to control the quiescent river segment of the old West Fork River channel at the upper, lower, and middle confluences with the bypass channel

11.2 Existing Organization Operations & Maintenance
Currently the TRWD Fort Worth Operations group performs a variety of maintenance activities, similar to that expected for the FWCC Project. These practices include turf maintenance which includes mowing, weed removal, fertilizing, tree removal, fencing, litter control, walkway and trail maintenance and the trimming of areas not accessible to mowers.

The equipment and facilities currently maintained by the Fort Worth Operations include building facilities, and equipment used by personnel at the operations. The overall operations and maintenance effort is extensive, but currently does not include pump stations, isolation gate structures, or dam structures within the Fort Worth Operations group. However, elsewhere in the district, TRWD personnel are engaged in maintaining dam structures, gates and pump stations. Therefore, knowledge and expertise for maintaining these types of structures is high.

11.3 Samuels Avenue Dam, Isolation Gates and Pump Station Operations
The following is a discussion on the operation of the major hydraulic elements of the FWCC Project bypass channel and interior waterway. The major elements include the Samuels Ave. Dam, three isolation gate structures and storm water pump station. This section describes operation practices that may be used to coordinate the opening and closing of Samuels Ave. Dam and the isolation gates during periods of operations.

11.3.1 Standard Operations Procedures

Once a final decision has been made on the hydraulic equipment, specific Standard Operating Procedures (SOPs) associated with each piece of equipment should be developed. This information will be part of a comprehensive operations manual which will include equipment manuals, parts specifications and operations procedures.

TRWD has a Computerized Maintenance Management System (CMMS) that has all of the functional capabilities typically provided in a state-of-the-art CMMS software package. For example, the current MAXIMO software program offers TRWD the following asset maintenance and management tools:

- Asset Inventory with an asset register tracking relationships between equipment and physical location.
- Document and track equipment specifications, associated costs, histories and failures, to enable effective repair or replace decisions.
- Equipment hierarchies to "roll up" maintenance costs.
- Enter and document work requests from multiple users.
- Enter, record and view detailed planning information, work plans, schedule, costs, labor, materials, equipment, failure analysis, and related documents via the Work Order Tracking screen.
- Automatically issue pre-schedule preventive maintenance work orders.
- Define and sequence work for multiple procedures and assets.
- Attach safety plans, hazards, precautions and lock-out/tag-out to work plans.
- Create purchase requisitions or orders for materials and services.
- Track stocked and non-stocked items through multiple stores.

It is anticipated that TRWD will develop SOPs to provide district personnel with the safety, health, environmental and operational information necessary to perform the work on the new assets properly. This will ensure that operations are performed consistently to maintain quality control of processes and maintenance procedures. The SOPs will also serve as a historical record of the how, why and when of steps in an existing process so there is a factual reason for revising those steps when a process or equipment is changed.

11.3.2 Samuels Avenue Dam Operations

The bypass channel is intended to pass significant floods without affecting the constant water levels in the original quiescent channel segment. The Samuels Ave. Dam located on the main stem of the West Fork will be located approximately 1,300 feet downstream from the confluence of Marine Creek and 1150 feet downstream from Samuels Ave. Bridge. The dam will maintain the normal water level elevation of 524.3 during non-flooding conditions throughout the upstream area and will have sufficient gate discharge capacity to pass the appropriate design flows, while maintaining flood levels within current conditions.

The operational assumption of the dam is that multiple gates would be opened partially prior to any single gate being opened fully. This will provide for much smoother and controllable operations, both for the structure and downstream interests. For example, a 2-yr flow of 12,100 cfs could be released with no rise in the upstream reservoir if all seven gates were lowered 4.9 feet. The 5-yr flood, or 18,800 cfs, would require all gates to be lowered 6.6 feet.

Initially three operating conditions have been established for the hydraulic structures, as a guide on how to manage the system. The first is the normal day to day operations of the structures; the second is the operations during moderate amounts of rainfall and the third is the operations during significant amounts of rainfall. It is also anticipated that a chart will be developed that describes specific actions to be taken during these three operating scenarios.

Normal dry weather operation of the dam will maintain the normal water pool level elevation of 524.3 during non-flooding conditions. The dam will have sufficient gate discharge capacity with the lower regulating gates to pass the appropriate dry weather flows. During the normal operations of the dam, certain preventive maintenance efforts should be planned and scheduled. Any problems that are identified during these inspections should be corrected as soon as possible.

Moderate rainfall will range between 1 to 3 inches of rainfall within a given period of time. Prior to this rainfall, the leaf gates of the dam will be opened to reduce the level by approximately two to five feet in anticipation of the rain event. The operation of the dam will be automatic but may also include provision for manual operation.

During periods of heavy rainfall, it is anticipated that data from upstream rain gauges and water level sensors will feed information to the centralized SCADA system to provide information to lower the dam water level to an appropriate level in anticipation of a significant event. This data will assist the staff at TRWD to operate and maintain the dam. This is to minimize the impact lowering the dam that would overdraft and maintaining at a level.

It is critical to operate and regulate the flow of water through the dam during periods of significant rainfall, but it is equally important not to release unnecessary amounts of water during drought conditions. Optimal operation of the Dam gates requires managing the storage space in anticipation of future inflows and multiple needs for water.

The Samuels Ave. Dam will be designed and operated using leaf gates to provide flexibility in the operations and control of the Dam. The Dam gates are recommended to be operated using partial gate openings of multiple gates before any one gate is opened fully. Operating equipment for each gate will be located on each pier, accessed by a bridge across the structure set above the SPF elevation. The gates can be operated either by a hydraulic system or by lifting cable and drum system.

In addition 4' wide by 6' high low flow conduits will be located in each of the three interior piers. Each gate will pass approximately 530 cfs at the normal pool level. This configuration will allow for small rises in the pool to be absorbed and then released through the low flow gates in addition to small flows over the top of the gates. Once the water surface has risen an appropriate amount, at least one of the flood control gates will need to be partially lowered to maintain the normal pool level and flood operational sequences will apply beyond that point. This will minimize the use of the large flood gates and simplify the frequent operations. The smaller gates will also allow for some limited flushing of silt from the bottom of the impoundment.

11.3.3 Samuels Avenue Dam Instrumentation and Monitoring

Instrumentation, proper monitoring and evaluation are extremely valuable in determining the performance of the Samuels Ave. Dam, the isolation gate structures and storm pump station. Instrumentation will be used to operate the hydraulic systems of the bypass channel in the following manner:

- The instrumentation data will provide information to the extent of a problem in the dam structure, i.e. hydraulic problem. Such information from monitoring will provide staff with corrective actions.
- Monitoring of Data – Instrumentation can remotely monitor and control the dam and isolation gate sites. For instance, a dam operator will remotely monitor the dam operation not requiring personnel to be sitting in front of a monitor watching controls. Even with instrumentation, the operator can manually operate the system if necessary.
- Warning and Analyzing Problems – Instruments can detect unusual changes, such as water level fluctuations. Also the need to repair a hydraulic hoses controlling the opening and closing of the dam structures may be detected by level sensors and send a readings via SCADA to the operator.

For the purpose of the dam, water level and flow monitoring is critical for managing the water level in the bypass channel. It is important to monitor the water level in the channel to determine the quantity of water in the channel. The water level in the channel will be measured by elevation gauges – staff gauges or level sensing devices.

Weather and precipitation monitoring at the bypass channel will provide valuable information about both day-to-day performance and developing problems.

11.3.4 Isolation Gate and Storm Water Pump Station Operation

The three interior isolation gates, Clear Fork, Trinity Point, and TRWD are intended to operate infrequently, only under major flood conditions. The gates will be designed to allow normal boat and pedestrian traffic to pass when in the raised position. The sill elevation will be set at el 520 for small boat passage with adjacent walkways set at el 530. All gates will be similar in design and operation.

It is anticipated that the combination of the lowering of Samuels Ave. Dam and the Bypass Channel hydraulics will convey most storm events with little water surface fluctuation within the interior area. Additional hydraulic modeling will be performed prior to final design to determine the resultant water surface conditions from various frequency storm events. This information will be used to determine a more detailed operating plan setting criteria for anticipating gate closures. Prior to significant rainfall or flooding events the isolation gates will be lowered. It is anticipated that the operation of the gates will be manual.

The storm water pump station is envisioned to operate under two conditions. The first is during major flood or river stage events when the isolation gates are closed. In this condition the pump station will pump storm water from the interior area over the levee to the channel. The second operating condition is from maintenance of the interior water feature area. In this condition the channel is isolated from the interior either by lowering or shutting the gates, the pump station is then used to lower the water in the interior area.

11.4 Soft Edge

The proposed soft edge is located on the western side of the bypass channel, and incorporates the earthen levees. This section is envisioned to be “park-like” or natural, while providing adequate side slope erosion protection. The soft edge will contain a recreational trail, sloped vegetation, and access for maintenance and emergency vehicles.

The recreational trail will be located approximately 5-feet above the normal base flow water surface, and comply to ADA Requirements with a maximum cross slope of 2% and maximum longitudinal slope of 5%. The recreational trail is envisioned to allow bikers, walkers, and roller-blade access to the park like area. The recreational trail will be approximately 20 feet in width and may also be used for emergencies, as necessary.

In addition to the recreational trail, an access road will be constructed on top of the levee to provide maintenance access for routine maintenance and during major storm events when the lower recreational trail is unavailable. Ramps or other means of street access will be provided to the top of the levee.

Bermuda grass will be planted and maintained on the soft edge levee side slopes above the recreational trail to improve aesthetics and provide slope erosion protection. Selection of the landscaping in this area will be appropriate and could include a combination of medium to tall shade trees and low lying bushes. Consideration will be given to selecting the landscaping that will be able to survive occasional storm flows in the channel without impairing the integrity of the levee embankment.

Native or Bermuda grasses will be planted on the backside of the levee and maintained in accordance with current operating procedures. The levee toe will be sloped to provide for over land drainage through existing swales where they do not currently exist.

11.4.1 Turf Maintenance

Turf maintenance practices at the Fort Worth Operations are currently seasonal, increasing significantly during the period April through November, which is a high growth time with significant pedestrian use of the trails. Although heavy rainfall during the year affects the facility, the District's aggressive preventive maintenance practice has prevented major failures.

The turf management performed at the Fort Worth Operations includes mowing, fertilization, repair and renovation. Grass height is maintained according to species and variety of grass. Aeration, reseeding or sodding and weed control are practiced as needed. The cost for mowing and weed abatement is estimated on a cost per acre. Currently, data is being tracked using a Computerized Maintenance Management System (CMMS). This system tracks costs on a per acre basis including the equipment being used, fuel, labor and benefits, and any supplies.

It is anticipated that the frequency of mowing of the soft edge will be 12 times per year, which equals the current mowing frequency performed on the existing levees. However, it is anticipated that this area will attract additional visitors and may require an increase in mowing, if necessary.

11.4.2 Levee Debris Removal

The current debris removal program requires TRWD personnel to provide weekend supervision with a lead position supervising the weekend, both Saturday and Sunday work release program from the Sheriff's Department. The areas along the trails and paths usually are the primary place for debris collection and removal.

11.5 Hard Edge

The hard edge will be located on the eastern side of the bypass channel. This section is envisioned to contain a series of tiered retaining walls, multiple walkways, and landscaping areas. There are three areas within the hard edge requiring maintenance. A local contractor may be used to clean and provide landscape services in this area. However, the lower walkway area may be subject to flooding and as a result, experience a significant amount of mud and silt build up.

11.5.1 Lower Walkway and Landscape Area

The Lower Walkway and Landscape Area is 30 feet wide and approximately 8,400 feet long equaling 252,000 sf. In this area, the walkway area is approximately 14 feet wide and 8,400 feet long equaling 117,600 sf and the landscape area is approximately 16 feet wide and 8,400 feet long equaling 134,100 sf. Similar to the recreational trail on the soft edge, the lower walkway will allow pedestrian access to the "park-like" environment of the channel. Along the lower walkway and interior wall, various amenities including park benches; sitting or picnic areas will be present and surrounded by various landscaping.

The work in the lower walkway and landscape areas will involve maintaining the sidewalks, and shrubs, to be weed free and clean in appearance. Any debris from mowing, trimming, or pruning will be removed after work. The landscape area will consist of shrubs and flowers that will need to be thinned and pruned. There will be trees located in the area requiring very little maintenance in the beginning. Application of fertilizer will be required during the growing season to maintain a healthy green color throughout the year. Additionally, lawn herbicides will be applied in areas to control weeds.

11.6 Riverbend Site

The purpose of the Riverbend site is to establish valley storage mitigation. This section covers the operation and maintenance of this location, specifically the grasslands, the levees and woodlands in this location. The following are the maintenance requirements:

- Planting of seedlings and irrigation of these trees during the first five years using a temporary irrigation system.
- Debris removal that may occur from visitors at the location.
- Trail maintenance, these will be natural trails that will require maintenance as a result of erosion and wear.
- Levee maintenance to prevent slope failure.

11.6.1 Riverbend Grassland Maintenance

The preliminary design of the ecosystem areas grassland should provide brush cover for small animals. The area will consist of native grasslands, where possible, replacing Bermuda and Johnson grass communities. A mowing schedule in these areas shall not interfere with the tall-grass nesting birds. Mowing of the grasslands will be performed after July 15th of each year, preferably in August or September and be cut back to a one foot height.

Besides mowing, the grassland maintenance performed at the Fort Worth Operations for this area will include minimal fertilization, repair and renovation. The cost for mowing and weed abatement is estimated on a cost per acre. Mowing and maintaining of this turf requires the use of Bat Wing Mowers and small finish mowers to keep areas attractive and meet the districts quality guidelines.

The total grassland area represents approximately 66 acres and will require a combination of maintenance activities from mowing once a year and debris removal approximately three times per year.

11.7 Samuels Avenue Dam, Isolation Gates and Pump Station Maintenance

Routine maintenance will be performed on the Samuels Ave. Dam, the isolation gates and pump station's equipment to ensure operational reliability and to maximize the useful life. The maintenance program will focus on preventive maintenance. The organization and staffing to support the maintenance program will require an understanding of the following types of systems:

- Electrical and electronic systems.
- Mechanical systems.
- Hydraulic and pneumatic systems.

The maintenance for each of these systems requires a different set of skills and varying levels of knowledge. Because it would be extremely unlikely for any single employee to possess the detailed knowledge required to operate and maintain all such systems, it is typical for an agency to separate or create specialized maintenance groups. Alternatively, agencies establish maintenance contracts with companies with personnel having the skills to perform these specialized tasks.

11.7.1 Inspection Program

An effective inspection program for the Samuels Ave. Dam, isolation gates and pump station is essential to identify problems early and to provide for safe maintenance of the structures. The inspection program will involve the following three types of inspections:

- Periodic technical inspections which involve inspections with specialists familiar with the design and construction of dams, isolation gates and pump station including assessments of structure safety

- Periodic maintenance inspections which are performed more frequently than technical inspections in order to detect, at an early stage, any detrimental developments in the dam, isolation gates and pump station; they involve assessment of operational capability as well as structural stability.

- Informal observations, which are continuing efforts by onsite personnel and performed in the course of normal duties.

Attachment A
Isolation Gate G.E. Hydro Report

GE Hydro

David M. Lake
Account Manager

May 21, 2004

Camp Dresser & McKee, Inc.
777 Taylor Street
Suite 1050
Fort Worth, TX 76102

Attention: *Mr. D. Funderlic – Project Engineer*

Subject: Trinity Point – Forth Worth, Texas

Dear Don,

The General Electric Company, Hydro business unit, is pleased to provide this document as application alternatives for the Trinity Point Redevelopment Project. Our Center of Excellence for the design and components utilized for such projects as this has compiled the attached information for your preliminary use to explore various solutions to the control of waters that may be expected to flow through this river system.

We understand the project is currently being considered as part of an overall redevelopment project that is expected to go forwarding the Fort Worth, Texas, area within the next few years. As such, the budgetary information submitted is to be considered preliminary in nature and for feasibility purposes only. The amounts are present day amounts and factors such as escalation, delivery and other commercial items would need to be further developed based on the specific requirements of the ultimate design and parameters. They are presented to provide a more graphic comparison between the various designs and selections that we believe would be suitable for this application.

We trust this information will be helpful in your current activities for this project. After you have had a chance to review the report, associated conceptual drawings, and reference list, I would like to arrange for a discussion with you and my colleagues to address any questions that you may have.

Please feel free to call me at your convenience.

Sincerely,

David M. Lake

David M. Lake

General Electric Company
2025 W. Beltline Rd. Suite 100
Carrollton, TX 75006
U.S.A.

T 972 389 7754
F 972 323 9918

1. Introduction.

The following document represents a study of the possible gate types suitable for 3 identical but independent flood barriers on the Trinity River, Fort Worth.

The gated flood barriers will be designed to protect low lying industrial real estate from floodwater.

Two basic overall layout concepts need to be considered

- Limited headroom option.
- Open channel option.

In each of the above concepts consideration needs to be given to the following

- Operational reliability
- Aesthetics
- Overall Cost (including Civil works)
- Maintenance

Operational reliability should focus on any reason why the gate will be prevented from closing then to mitigate these risks. Gate operation in anger is anticipated as very infrequent i.e. long periods of non-use with the gate in the fully open position.

A further issue that will also affect the gate type is the gates ability or non- ability to close under flowing water conditions. Early flood warning means the gates close in near balanced flow conditions. Late warning means closing under increasing flood flow.

For flood barrier applications 100% gate sealing is not considered necessary, however gate-sealing systems are recommended to minimise leakage.

Navigation is required for small craft only.

2. Basic design parameters

- Number of flood barrier locations = 3
- Gate clear span at each location = 60ft
- Channel widths = 2 at 140ft 1 at 250ft
- Minimum water depth for navigation by small craft = 5ft
- Minimum headroom thro' gates = 12-15ft
- Gate sill level = 520ft
- Normal pool water level = 525ft
- River bed level = 515ft varies
- Flood level = 540 ft
- Flood protection level = 554ft
- Protection depth at gates = 34ft
- Number of flood operations per year = 2 (to be confirmed)
- Gates are open or closed type and not used for regulation.

3.Limited Headroom (12-15ft) options

3.1 Vertical Roller gate

Gate size needed
- 60ft span x 20ft deep.

Normal Operation
- Gate closes by lowering due to own mass in say 5 minutes,
- AC power required for opening in say 10 minutes.
- Gate can open or close in unbalanced flow condition

Operating mechanism
- Normally wire rope winches (or hydraulic cylinder hoists).

Backup requirements
- Gate gravity closure needs UPS/battery DC signal to release gate brake to allow controlled closing. Closing speed control by fixed orifice hydraulic retarder.

Operational reliability
- High reliability as gate does not need AC power to close.

Aesthetics
- Needs concrete headwall or bridge to hide majority of gate structure when open and to reduce gate depth.

Civil comments
- Headwall can include pedestrian walkway or vehicle bridge.
- Civil layout is compact.

Other comments
- These are normal flood barrier gates with many references.
- Gate sill contact is narrow so debris is unlikely to prevent closure.
- Sill is self-cleaning under flow.
- Gate is stable when closed.

Maintenance
- Majority of the gate structure and hoist are accessible for inspection and maintenance in the fully open position.

3.2. Radial gate (Lowering)

Gate size needed
- 60ft span x 20ft deep.

Normal Operation
- Gate closes by lowering due to mass in say 5 minutes,
- AC power required for opening in say 10 minutes.
- Gate can open or close in unbalanced flow condition.

Operating mechanism
- Normally hydraulic cylinder hoists.

Backup requirements
- Gate gravity closure needs UPS/battery DC signal to release hydraulics to allow controlled closing. Closing speed controlled by hydraulic orifice.

Operational reliability
- High reliability as gate does not need AC power to close.

Aesthetics
- Needs concrete headwall or bridge to hide majority of gate structure when open.

Civil comments
- Headwall can include pedestrian walkway or vehicle bridge.

Other comments
- These are normal flood barrier gate with many references.
- Gate sill contact is narrow so debris is unlikely to prevent gate closure.
- Sill is self-cleaning under flow.
- Gate is stable when closed.

Maintenance
- Majority of the gate structure and hoist are accessible for inspection and maintenance in the fully open position.

4.Open channel options

4.1. Radial gate (Rising)

Gate size needed

- 60ft span x 34ft deep.

Normal Operation

- Gate rises to close. AC power opened and closed in say 15 minutes.
- Gate can open or close in unbalanced flow conditions.

Operating mechanism

- Hydraulic cylinders and oil power units

Backup requirements

- Requires diesel generator for back up AC power (Hydraulic accumulators are not practical in this case.)

Reliability

- Reduced, as gate needs power to close therefore backup systems are vital.

Aesthetics

- Gate submerged when open so good open channel aesthetics.

Civil comments

- Rebate in bed is required for the gate in the open position.

Other comments

- This type of gate forms the basis of the Thames barrier, UK. We show a simplified version sometimes used in locks and more in keeping with this size/application.
- Gate leaf protects the rebate in the riverbed when open so debris should not prevent closure.
- Gate is unstable when closed so needs to be latched in the closed position, as the gate mass tends to open the gate if oil pressure fails. (Sufficient size counterweights cannot be included to avoid this.)

Maintenance

- Hoist will be accessible for maintenance with gate open.
- Gate leaf/seals maintenance will require stoplogs.

4.2. Vertical Sector gates

Gate size needed
- 60ft span x 34ft deep. Two leaves 30ft span each

Normal Operation
- AC power opened and closed in 5 minutes.
- Gates can open or close in unbalanced flow conditions

Operating mechanism
- Hydraulic cylinders and oil power units

Backup requirements
- Requires diesel generator for back up AC power (or optional hydraulic accumulators).

Operational reliability
- Reduced, as gate needs power to close therefore back up systems are vital.

Aesthetics
- Gates are hidden in sidewalls when open so good open channel aesthetics.

Civil comments
- Gate sidewall rebates means the civil layout is complicated and may be high cost.

Other comments
- Traditionally these gates are used on locks and yacht marinas where fast locking and balancing of the water levels is needed. (This feature is not required at this site.)
- Both gate leaves need to successfully close.
- Gate sill contact is narrow so debris is unlikely to prevent closure.
- Gates are stable in the closed position.

Maintenance
- Hoist will be accessible for maintenance with gate open.
- Gate leaf/seals maintenance will require stoplogs.

4.3. Mitre lock gates

Gate size needed

- 60ft span x 34ft deep. Two leaves approx 35ft span each.

Normal Operation

- AC power needed to close. Open and close in 5 minutes.
- Gates can only open or close in balanced no flow conditions

Operating mechanism

- Hydraulic cylinders and oil power units.

Backup requirements

- Requires diesel generator for back up AC power (or optional hydraulic accumulators.)

Reliability

- Reduced, as gate needs power to close therefore back up systems are vital.

Aesthetics

- Gate hidden in sidewall when open so good open channel aesthetics.

Civil comments

- Rebate needed in sidewalls and sill.

Other comments

- Both gate leaves need to successfully close.
- Traditional lock gates, but occasionally used as floodgates.
- The gate when open does not protect the rebate in the riverbed so debris could prevent gates from closing.
- The gates sweep a large area of bed, which needs to be free of debris to ensure gate closure.
- Gates open with reverse head but are stable with on seat head.

Maintenance

- Hoist will be accessible for maintenance with gate open.
- Gate leaf maintenance will require stoplogs.

4.4 Flap gate (Bottom hinged)

Gate size needed
- 60ft span x 34ft deep.

Normal Operation
- AC power needed to close. Open and close in 15 minutes.
- Gate can only open and close in partially balanced flow conditions.

Operating mechanism
- Hydraulic cylinders and oil power units.

Backup requirements
- Requires diesel generator for back up AC power. (Accumulators are not practical in this case)

Reliability
- Reduced, as gate needs power to close.

Aesthetics
- Gate hidden in bed when open so good open channel aesthetics.

Civil comments
- Rebate needed in bed.

Other comments
- Traditionally dock and lock gates, but also used as flood barriers.
- Gate leaf protects the rebate in the riverbed when open so debris should not prevent closure.

Maintenance
- Hoist will be accessible for maintenance with gate open.
- Gate leaf/seals maintenance will require stoplogs.

5.Gate Maintenance/Inspection.

1. Provision needs to be made in the plant layout and preventative maintenance schedules to adequately inspect and maintain the selected gate type.
These gates are infrequently operated for flood protection and will require exercising say once every 2 months to verify operational status and check the backup system reliability.

2. Stoplog slots upstream and downstream of the gates are recommended for gate maintenance.
One set of stop logs for all 3 sites should be made available. The stoplogs can be stored off site and installed using a temporary mobile crane as and when required. Stoplogs depth up to 530ft level should be sufficient to allow maintenance to the gate sill level during non-flood periods, whilst not affecting the normal water level either side of the gate.

6. Other gate types

Other gate types that have been considered **but dismissed** as either much less reliable, or outside the normal size range or not suiting the operational requirements for this flood prevention project are: -

- Rubber dams (outside the depth range and very slow closing by inflating with compressed air)
- Double leaf roller gates (Complicated and no better than other limited headroom gate options shown and high cost)
- Drum gates (These are normally for water level regulation and require 11m deep excavations in the river bed)

7. Budget Costs

The following short listed gates are proposed for budget purposes. Budget costs include design, supply, transportation and installation of the three sets of identical gates, embedded parts, operating equipment and local controls, but exclude civil costs. Budget costs do not include import duties, if applicable, and are based upon having no sourcing restrictions. We also identify added costs for the normal diesel generator back up system to be added where relevant and stoplogs for maintenance.

• Vertical Roller gate	$ 3.7 Million US Dollars
• Radial gate (Lowering)	$ 3.2 Million US Dollars
• Radial gate (Rising)	$ 5.0 Million US Dollars
• Vertical Sector gates	$ 4.8 Million US Dollars
• Mitre gates	$ 4.3 Million US Dollars
• Flap gate	$ 4.6 Million US Dollars

Additional Items

• Sets of Stoplogs & Frames	$ .82 Million US Dollars
• Diesel Generators	$ .48 Million US Dollars

See appendix A - Outline Drawings
See appendix B - GE gate brochure
See appendix C - GE reference lists

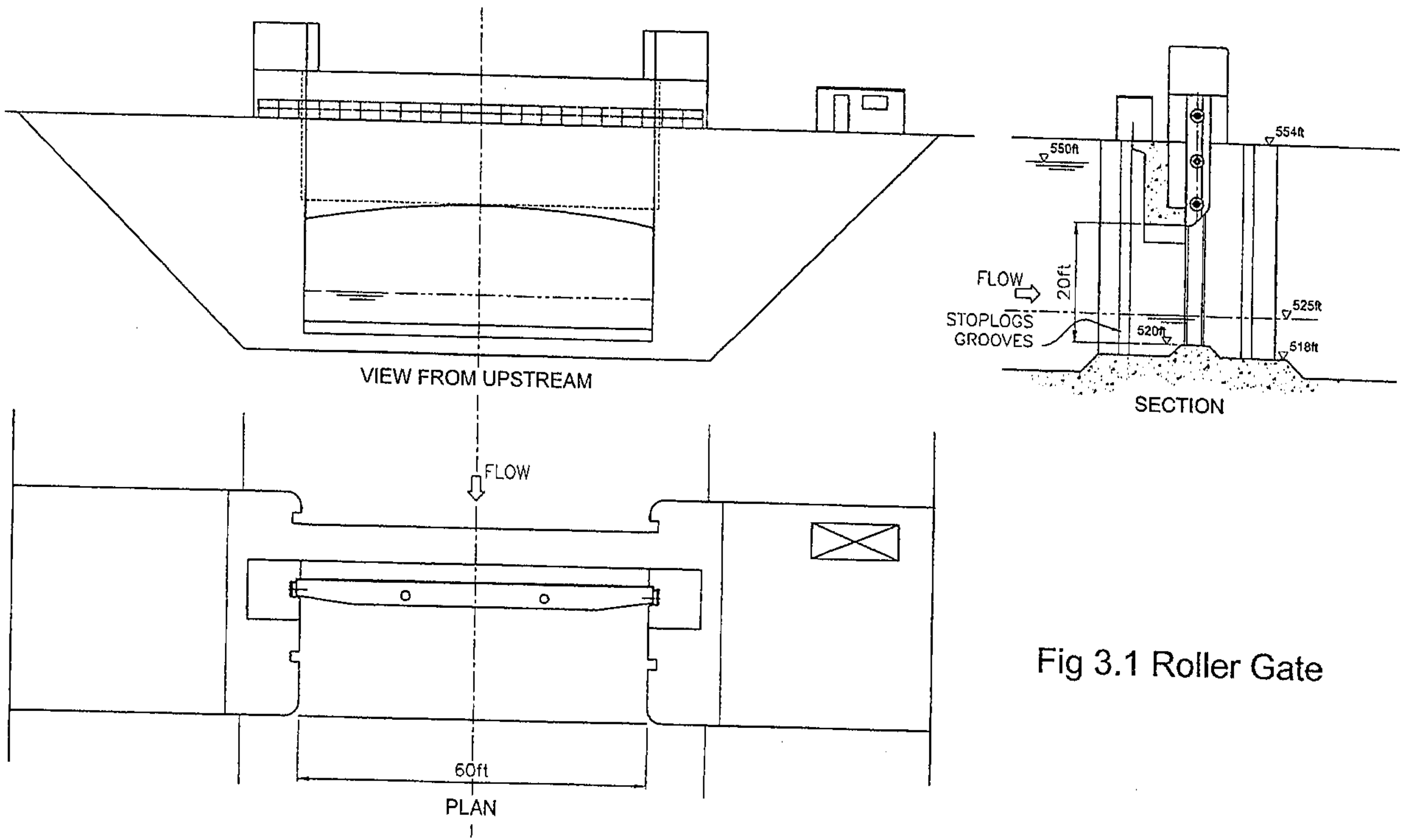


Fig 3.1 Roller Gate

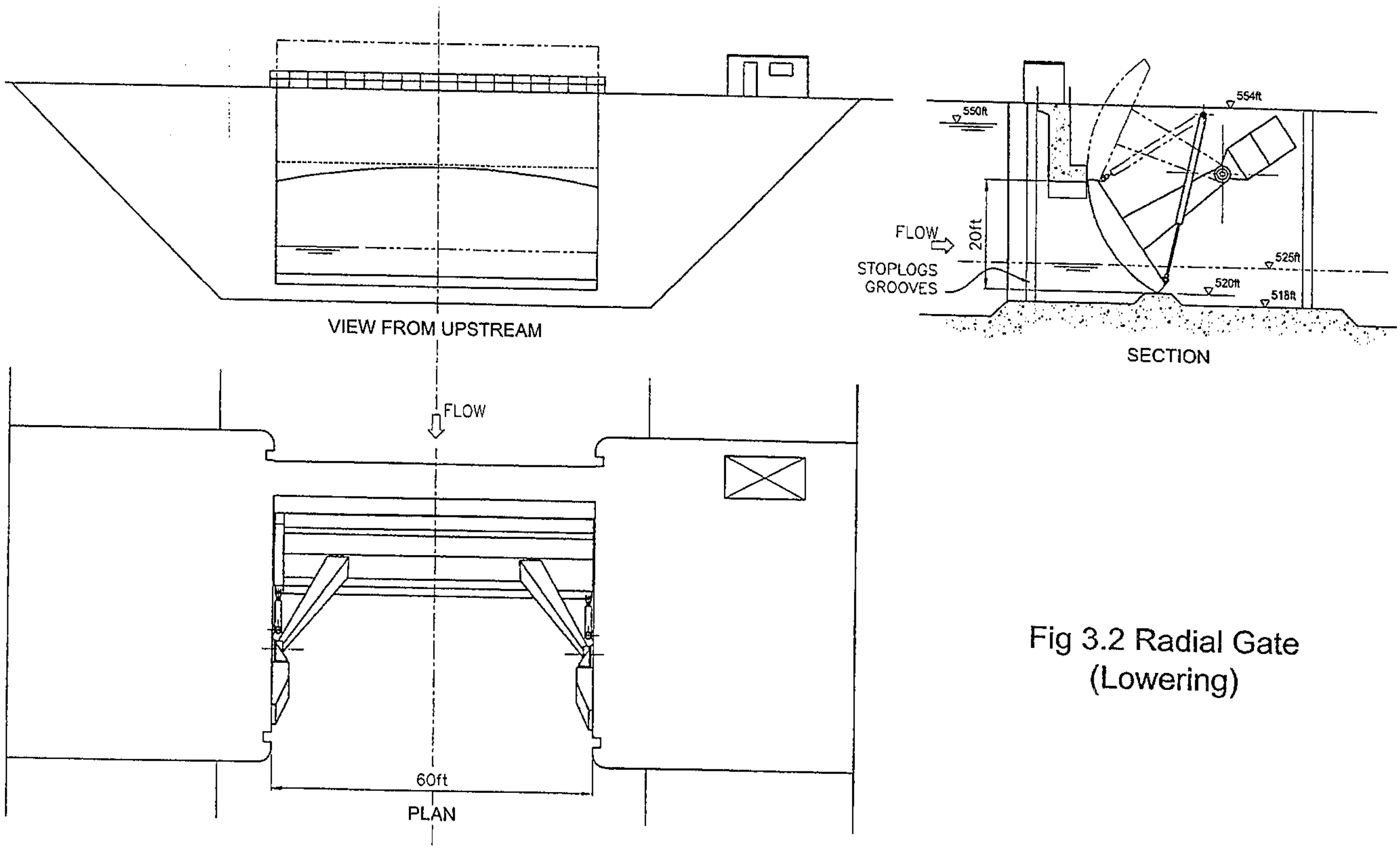


Fig 3.2 Radial Gate
(Lowering)

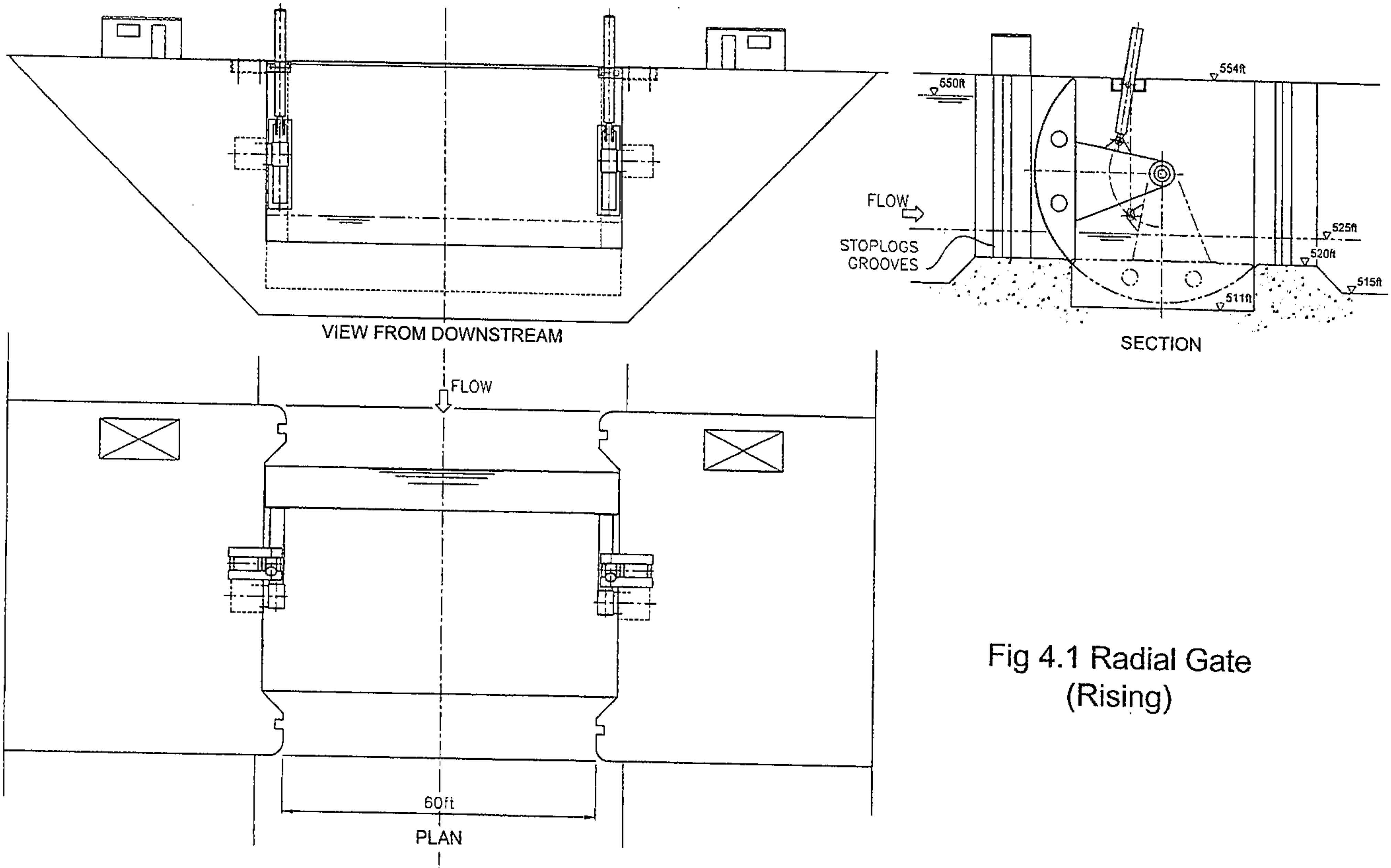


Fig 4.1 Radial Gate
(Rising)

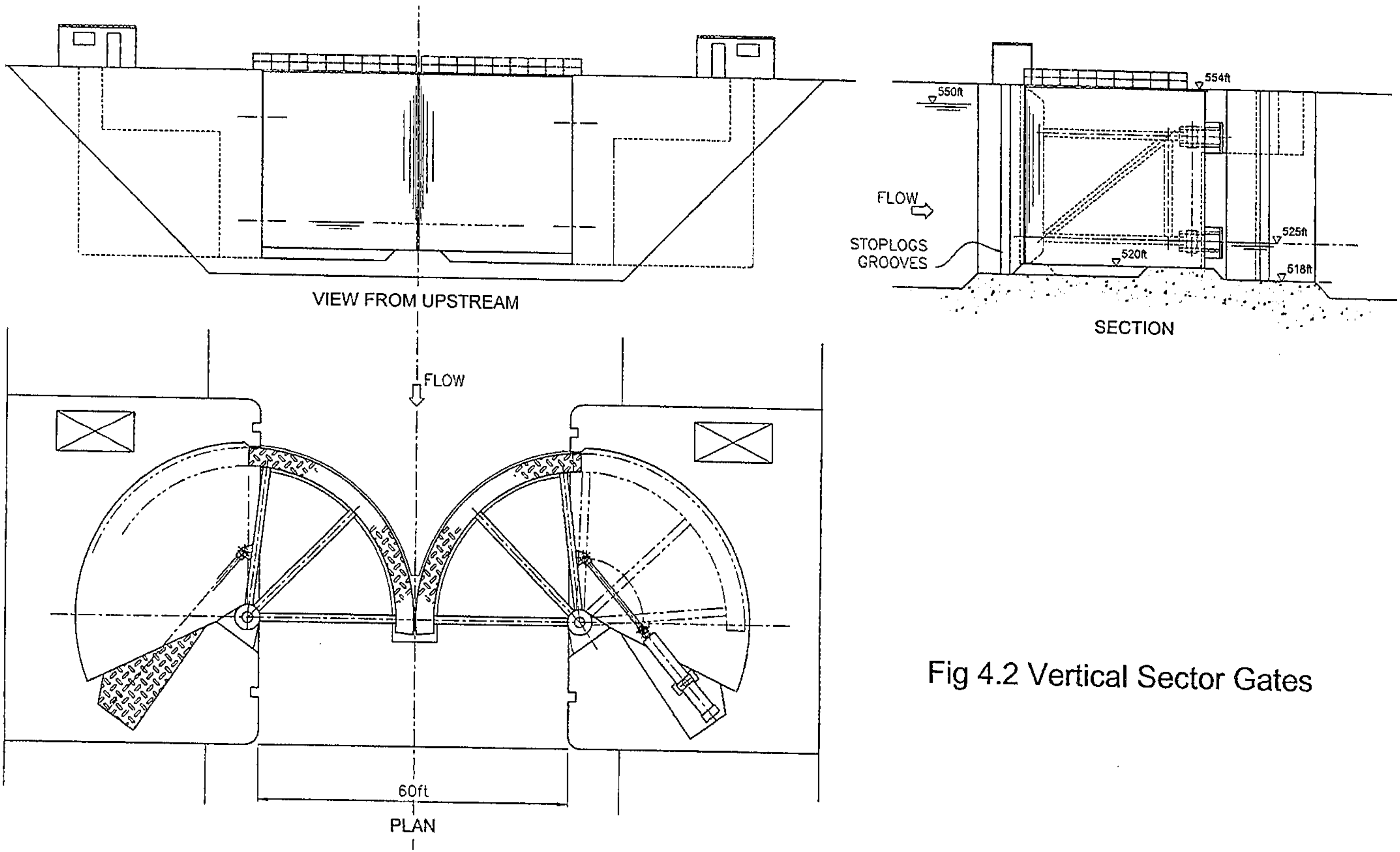


Fig 4.2 Vertical Sector Gates

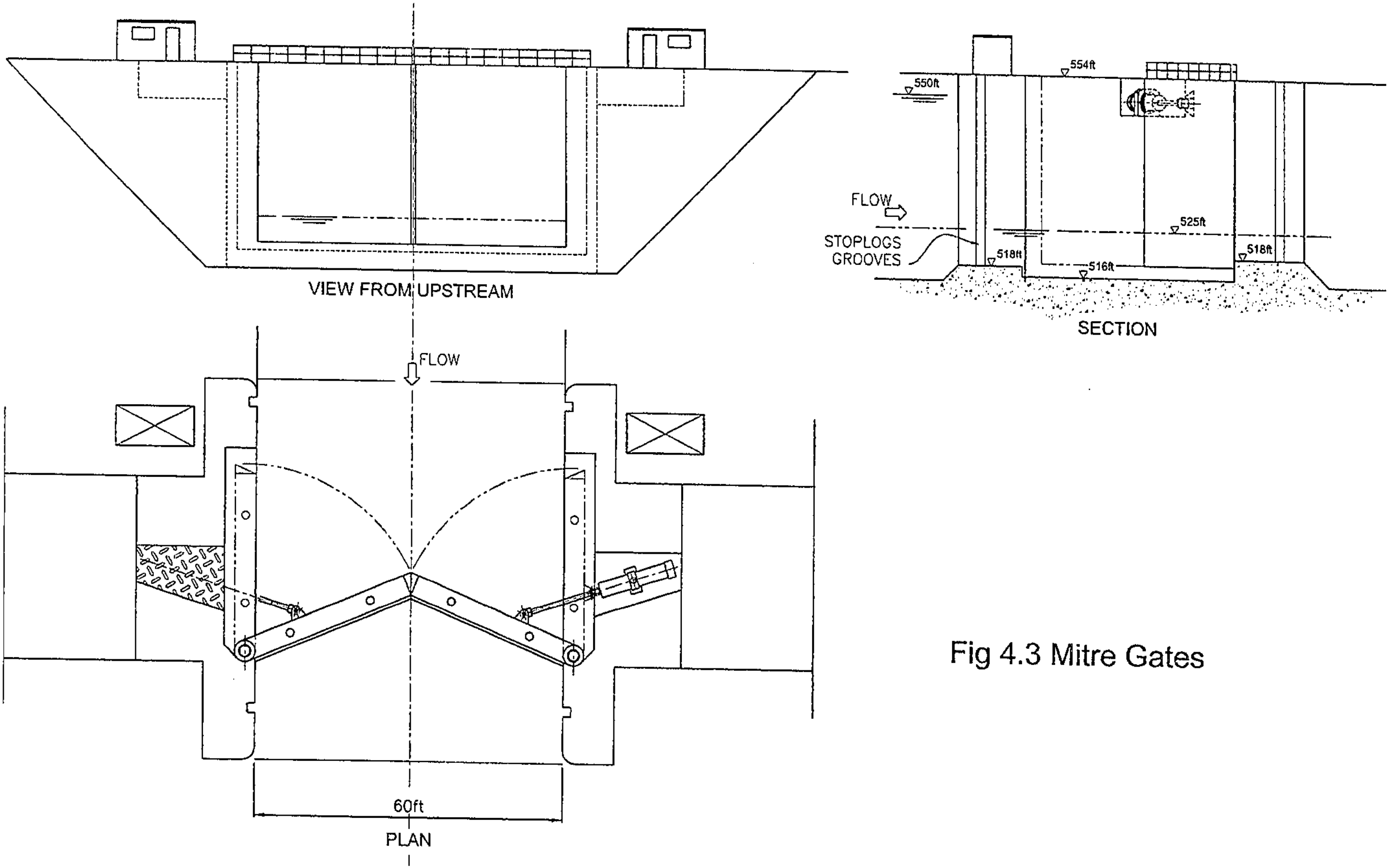


Fig 4.3 Mitre Gates

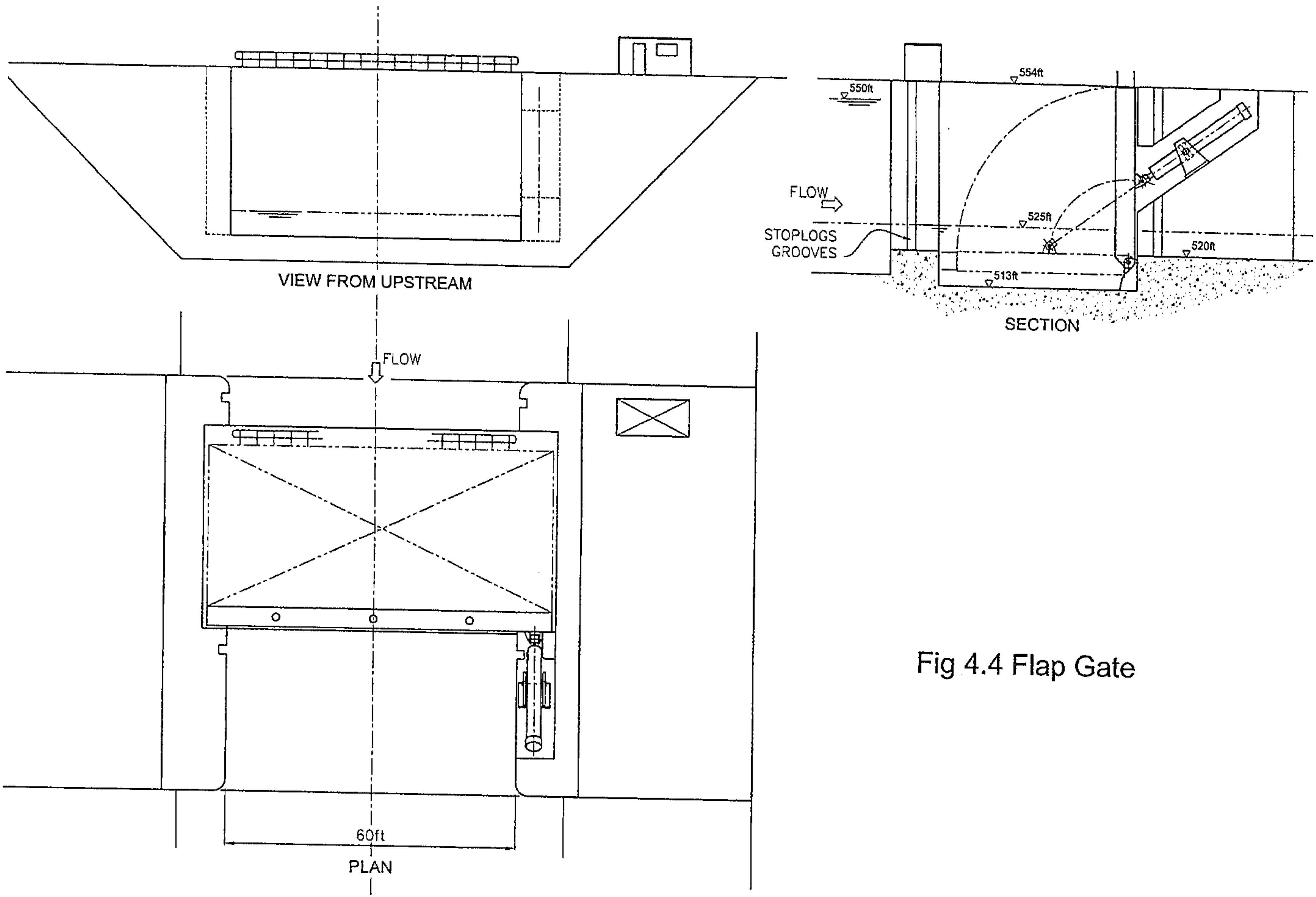


Fig 4.4 Flap Gate