

# Fort Worth Central City Preliminary Design

## Civil/Structural Preliminary Design

Final Supplement No. 1  
to the Final Environmental  
Impact Statement

Appendix C

Volume I- Narrative

March 2008

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#### Appendix C

March 2008

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# FORT WORTH CENTRAL CITY Preliminary Design Supplement

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# Appendix C

## Civil/Structural

### 1.0 Introduction

This Appendix supports Supplement No. 1 to the Final Environmental Impact Statement (FEIS) for the Central City Project, Upper Trinity River. The supplement to the FEIS is limited to those components of the project which are modified as a result of the evaluation of alternatives to merge the authorized Central City Project with the Riverside Oxbow Ecosystem Restoration Project. Specifics of the previously approved plans are summarized in the text of the FEIS for the Authorized Central City project and Interim Feasibility Report and Integrated Environmental Assessment for the Riverside Oxbow Ecosystem Restoration Project (April 2003) and Addendum dated April 2005.

### 1.01 Background

The following paragraphs provide a brief summary of the two projects, Central City and Riverside Oxbow Ecosystem Restoration.

#### 1.01.01 Central City Project

The Central City Project is a multi-agency endeavor involving several Federal agencies (primarily the US Army Corps of Engineers) and at least three non-Federal partners (Tarrant Regional Water District, City of Fort Worth and Streams and Valleys). The primary focus of the Central City Project is to enhance existing levels of flood protection while restoring components of the natural riverine system that were sacrificed in construction of the existing flood control system and facilitating urban revitalization. Authorization for Federal participation for construction was provided by Public Law 108-447 dated 8 December 2004. The FEIS was completed for the Authorized Central City Project in January 2006 and the Project Report was completed in March 2006. The Record of Decision (ROD) was signed, and the Project Report recommending the Community-Based Alternative was endorsed as being technically sound and environmental acceptable, by the ASA(CW) on 7 April 2006.

The Community-Based Alternative consisted of the following elements:

- construction of an approximately 8,400-foot long bypass channel extending from just downstream of 5<sup>th</sup> Street in the Clear Fork to just upstream of Northside Drive on the West Fork;
- construction of a dam on the West Fork, approximately 1,100 feet downstream of Samuels Avenue;
- construction of three isolation gates to direct flood flows through the bypass channel and to create a controlled, quiescent watercourse in the

approximately two-mile stretch of the existing West Fork channel adjacent to downtown Fort Worth;

- construction of numerous street and highway improvements, including raising University Drive out of the 100-year floodplain; and
- valley storage mitigation for 5,250 acre-feet of storage at various sites.

### **1.01.02 Riverside Oxbow Ecosystem Restoration Project**

The Riverside Oxbow Ecosystem Restoration project encompasses approximately 1,060 acres with a focus on restoring both the biological integrity of wetland and bottomland hardwood communities and the integrity and function of floodplain habitats and adjacent uplands. An Interim Feasibility Report and Integrated Environmental Assessment for the Riverside Oxbow Ecosystem Restoration Project were completed in April 2003 and a Finding of No Significant Impact (FONSI) was signed by the USACE, Fort Worth District Commander, on 22 May 2003. The recommended Locally Preferred Plan was approved by the USACE Chief of Engineers on 29 May 2003. An Addendum dated April 2005 was prepared to respond to comments from the Assistant Secretary of the Army. Neither construction funding nor authorization for implementation of this project has been provided to date.

The Locally Preferred Plan included:

- construction of a notched control structure in the existing floodway channel to allow flows through the old cutoff oxbow;
- demolition and replacement of the existing Beach Street Bridge;
- construction of a new parkway entrance and bridge; and
- restoration of almost 600 acres of diverse aquatic and riparian woodland habitats previously impacted by the channel improvements.

### **1.01.03 Modified Central City Project**

In June 2006 the City of Fort Worth submitted a letter requesting that the Fort Worth District Corps of Engineers conduct an evaluation to consider the potential benefits of combining the authorized Central City Project with the Riverside Oxbow Ecosystem Restoration Project. Following an initial evaluation of the merits of combining the two projects, the Corps of Engineers determined that merging the two projects had the potential to increase hydraulic efficiency and to provide additional environmental restoration opportunities. Based on this determination, the Corps of Engineers proceeded with the detailed evaluation of alternatives to prepare a supplement to the FEIS for the Central City Project.

The civil components of the Modified Central City Project that have changed due to the result of merging the two existing projects include:

- Alternative valley storage sites at:

- Rockwood Park- West
- Riverside Park
- Ham Branch
- Riverside Oxbow
- Gateway Park
- Modification to previously approved valley storage sites at:
  - University Drive
  - Samuels Avenue

The design criteria, assumptions and findings are discussed in Sections 1.02, 1.03 and 1.04, respectively.

The structural components of the Modified Central City Project that have changed or been added as a result of merging the two projects include:

- Relocation of the Samuels Avenue Dam,
- Addition of a low water fixed dam on Marine Creek, and
- Construction of a controlled lock structure to accommodate movement of small boat traffic from the Interior Water Feature to Marine Creek.

The structural design criteria, assumptions, loading conditions, method of analysis and results of computations for these components are discussed in Sections 1.05 through 1.08. In addition to the above components, these sections also present reference drawings and data from the Riverside Oxbow Project related to the Beach Street Bridge and Park Road Entrance Bridge.

## **1.02 Civil References**

The design for the valley storage mitigation sites, Samuels Avenue Dam and Marine Creek low water dam is in accordance with standard engineering practices and guidance as set forth in various manuals as published by the USACE as follows and applicable:

EM 1110-2-301, Guidelines for Landscape Planting at Floodwalls, Levees & Embankment Dams, 01 Jan 00

EM 1110-2-410, Design of Recreation Areas and Facilities - Access and Circulation, 31 Dec 82

- EM 1110-2-1205, Environmental Engineering and Local Flood Control Channels, 15 Nov 89
- EM 1110-2-1206, Environmental Engineering for Small Boat Basins, 31 Oct 93
- EM 1110-2-1601, Hydraulic Design of Flood Control Channels Change 1 ENG 4794-R, 30 Jun 94
- EM 1110-2-1913, Design & Construction of Levees, 30 Apr 00
- EM 1110-2-2607, Planning and Design of Navigation Dams, 31 July 95
- EM 1110-2-2610 Change 1, Lock and Dam Gate Operating and Control Systems, 02 Apr 04
- EM 1110-2-2902, Conduits, Culverts and Pipes, 31 Mar 98
- EP 1130-2-550, Recreation Operations and Maintenance Guidance and Procedures, CH 3, 15 Aug 02
- Pamphlet No. 11650-2-1, X Criteria for Construction within the Limits of Existing Federal Flood Protection Projects, Fort Worth District, 31 Oct 03

### 1.03 Civil Specifications

Design criteria and base assumption for the civil design are as follows:

#### Excavation

Maximum Cut Slope	3H:1V
Maximum Fill Slope	3H:1V
Aerial Utilities Vertical Clearance	Min 20 ft
Pole Setback from Toe	Min 10 ft
Seeding	Bermuda Grass

#### Roads and Bridge

Cross Slope	Max 2%
Park Road Maximum Design Speed	25 mph
Beach Street Maximum Design Speed	45 mph

#### Recreational Trails and Ramps

Cross & Longitudinal Slope	Max 2% and 5%
Ramp Slope	Max 10%

### 1.04 Valley Storage Mitigation Civil Design

#### 1.04.01 Background

As noted in Appendix C of the Central City FEIS dated January 2005, construction of the bypass channel required mitigation of lost floodplain storage, referred to as “valley storage”, due to increased efficiency of flow through the bypass channel rather than through the existing channel. The valley storage loss caused by the construction of the bypass channel is comprised of two components. First, routing the existing Clear Fork and West Fork through the bypass channel instead of the

interior area reduces the total length of channel resulting in less in-line floodplain storage. In addition, the hydraulic efficiency of the shortened channel length also creates a drawdown effect on both the Clear Fork and West Fork 100-year and Standard Project Flood (SPF) water surface elevations upstream of the bypass channel which reduces the upstream valley storage.

The amount of valley storage mitigation required to compensate for this loss was determined by hydraulic modeling analysis in compliance with the criteria established by the Corridor Development Certificate (CDC) guidelines. The original hydraulic analyses (contained in Appendix A of the Central City Project FEIS) quantified the approximate volume of valley storage lost as 5,250 acre-ft without mitigation. This volume consisted of an estimated loss of 2,850 acre-feet due to the construction of the bypass channel and approximately 2,400 acre-feet due to upstream drawdown.

#### **1.04.02 Alternative Analysis**

As part of the analysis for the Modified Central City Project, valley storage mitigation was re-evaluated on the basis of eliminating the Riverbend Site and incorporating the Riverside Oxbow/Gateway Park Site. Primary goals of the re-evaluation of the valley storage mitigation sites were to maximize use of existing public lands rather than privately owned land for valley storage and, at the same time, enhance the opportunities for ecosystem restoration and recreation.

The amount of valley storage mitigation required to compensate for valley storage loss by the Modified Project was determined by hydraulic modeling analysis in compliance with the criteria established by the Corridor Development Certificate (CDC) guidelines. The design criteria and hydraulic analyses (contained in Supplement No. 1 Appendix A, Hydrology and Hydraulics) quantify the approximate volume of valley storage lost as 5,250 acre-ft without mitigation.

Potentially available areas included all of the areas previously considered for the Authorized Central City Project, lands identified within the footprint of the former Riverside Oxbow Ecosystem Restoration Project, sites identified by the USACE during evaluation of the merits of combining the two projects and a few additional areas not previously considered. Potential valley storage sites within the combined project area were ranked based on the ability of the site to accommodate additional valley storage while minimizing both adverse effects on significant habitat areas and the need to acquire privately owned land. Further evaluation also included analysis of potential constraints such as utility relocations, constructability and hazards associated with prior land use.

The results of the re-evaluation of valley mitigation storage sites indicated that the majority of the required valley storage mitigation provided by the original Riverbend Mitigation Site could be accommodated within the confines and project area of the Riverside Oxbow Ecosystem Restoration project. In addition to the

proposed Riverside Oxbow Sites, the University Drive Mitigation Site and the Samuels Avenue Sites, the remaining valley storage would come from three newly identified sites. The following storage mitigation sites, discussed in further detail in the next section, comprise the Locally Preferred Valley Storage Plan proposed for the Modified Project:

- Rockwood Park -West Site
- Samuels Avenue Sites
- Riverside Park Site
- Ham Branch Site
- Riverside Oxbow/Gateway Park Sites and
- University Drive Modification.

### **1.04.03 Rockwood Park - West Site**

#### **1.04.03.01 *Site Description and Constraints***

Rockwood Park- West is a 23 acre site, located between River Stations 2702+00 and 2724+00, within the existing Trinity River floodplain on the southwestern portion of the existing City of Fort Worth Rockwood Park Golf Course. The 27-hole golf course is located south of Henderson Street (Jacksboro Hwy) on the West Fork Trinity River between the White Settlement Road and University Drive bridges. The site is bounded by the Trinity River on the east and existing federal levee to the west. The proposed mitigation area is within an inactive portion of the golf course. Vegetative cover on the site is primarily grassland with minimal tree coverage. This mitigation site was analyzed as part of the original Central City FEIS, but was excluded from the final valley storage mitigation plan due to the selection of the Riverbend Site.

Within the reaches of the site, the top of bank elevation ranges from 524 to 526. Immediately upstream of the site, the northern bank, from the river to the toe of the levee, is comprised of an 'upper shelf' with an elevation of 540. Most of the Rockwood Park-West Site is on a 'lower shelf' with an elevation ranging from 528 to 538. The elevation of the top of levee is approximately 554.

The proposed work includes grading the site to gently slope towards the river, to a new top of bank elevation of 526, which is approximately 1 foot above the proposed normal pool elevation, to obtain optimum valley storage mitigation. A minimum 30 foot buffer is to be provided from the base of the levee to the proposed excavation to maintain the integrity of the levee and provide a maintenance road and trail access in front of the levee. Excavated materials will be transported and disposed of off-site. The toe of the levee from Station 2723+00 to Station 2710+00 is marked by a 2-wire cable fence, with 4-foot high metal posts, to

prevent vehicular access. There are no apparent surface obstructions or structures within the site limits.

#### **1.04.03.02 Utility Relocations**

There are no proposed utility relocations associated with the Rockwood Park – West Site. An existing 36-inch sanitary sewer line that runs along and parallel to the toe of the levee traverses the site from Station 2723+00 to 2710+00. Field observations revealed no visible manholes, although, based on existing utility maps, several are known to exist throughout the reaches of this site. The top of slope of the proposed excavation will establish a minimum 10 foot offset from the sanitary sewer alignment to protect any potential erosion into the limits of the sewer.

#### **1.04.03.03 Material Handling**

##### **Earthwork Quantities**

The top of the river bank excavation is set at Elevation 526.0. Excavation quantities for the Rockwood Park – West Site, based on the contours presented on grading plan sheet CG-02, using 3:1 side slopes with a 1% cross fall slope across the site, from toe of levee to river bank are 148,000 CY. These are presented as raw quantities with no shrinkage or swell factors considered.

##### **Haul Routes & Disposal Sites**

The majority of the spoil material generated by the proposed excavation at Rockwood Park – West will be transported to the University Drive Modification site to be used in filling the site to raise the roadway. Of the estimated 148,000 CY generated, 130,000 CY will go to the University Drive Site. The remaining 18,000 CY will be transported to the Bypass Channel construction zone for use in backfilling the hard edge or Bazaar Fill Site. The proposed haul route from Rockwood Park – West to University Drive will be through the use of a temporary access road along the edge of the existing Rockwood Golf Course to Jacksboro Highway (SH 199) and south approximately 1.25 miles to University Drive. The haul route to the Bypass Channel/Bazaar area will be the same, but continuing an additional one mile south on Jacksboro Highway. Transportation of the spoil materials to the University Drive will need to be coordinated with the City of Fort Worth and their roadway improvements as well as within the overall staging of the project.

#### **1.04.04 Samuels Avenue Sites**

##### **1.04.04.01 Site Description and Constraints**

The Samuels Avenue Sites cover approximately 40 acres within the Trinity River floodplain and are located downstream of the Samuels Avenue Bridge. The sites

lie along the north and south banks of the West Fork Trinity River and were previously analyzed and included as part of the Central City FEIS. The sites are bounded by Brennen Ave to the north, Northside Drive to the east and south, and the Union Pacific Railroad right-of-way to the west. The southern site is flanked by a federal levee while the northern site is flanked by two old construction waste landfills. Property ownership is a combination of City of Fort Worth and Tarrant Regional Water District.

The Samuels Avenue North Site is an approximately 15 acre site along the north bank of the West Fork of the Trinity River (River Station 2392+00 to 2417+00). It abuts an existing inactive City of Fort Worth Landfill located on Brennan Drive from Station 2403+00 to 2412+00 and a completed (i.e., capped) portion of the landfill from Station 2393+00 to 2403+00. Being immediately adjacent to the river and within the floodway, the area is well maintained and regularly mowed.

Within the reaches of this site, the elevation of the top of riverbank ranges from approximately 524 to 529, with the top side relatively flat (less than 1% slope). The proposed lower excavation elevation (new top of bank) is 502.0.

The 'back' property line (adjacent to the existing and completed landfill properties) from Station 2493+00 to Station 2417+00 is marked by a 6-ft cyclone fence. Outside the fence, from Station 2404+00 to 2413+00 is a 20 ft asphalt paved landfill road, including street lighting (20 ft light poles on 200 ft centers). The only apparent obstructions or structures within the site limits are described below in the Utility Impacts section.

The Samuels Avenue South Site is an approximately 25 acre triangular-shaped site along the south bank of the West Fork of the Trinity River from River Station 2398+00 to 2414+00. The site contains approximately 1,600 linear feet of concrete-paved trails within its limits that are part of the Trinity Trails system. Being immediately adjacent to the river and within the floodway, the area is well maintained and regularly mowed.

Within the reaches of this site, the elevation of the top of bank ranges from approximately 516 to 522 and extends at a gentle slope to the south to the edge of a small upper shelf. A high-voltage electric transmission line is located at the southern edge of the lower shelf at a ground surface elevation ranging from 523 to 526. The top of levee along the south side of the site is at approximate elevation 538. The proposed lower excavation elevation (new top of bank) is 502.0.

Scattered along the 1,600 ft reach of this site are approximately 20-30 transplantable (4" to 6") oak trees, that have been planted for landscape purposes in recent years. In addition, there is a Monument Boulder (approximately 6-8 ft in length), adjacent to the trail, near Station 2410+00.

Proposed work includes grading the sites to gently slope towards the river to a bank elevation approximately 1 foot above the static water elevation (EL 501) which is controlled by the 4<sup>th</sup> Street low water dam. The existing high voltage

transmission lines will have a minimum 50-foot wide set-back between the power lines and the beginning of the grading. The existing maintenance road and access point to recreation trails will be reconstructed within the set-back area to provide access and continuity of the existing trail system. The existing trail along the river bank will also be reconstructed to provide scenic access closer to the river.

#### **1.04.04.02 Utility Relocations**

The Samuels Avenue North Site has an existing concrete-lined stormwater discharge channel at Station 2403+00, which drains an area north of the river, including part of the landfill and adjacent properties along Brennan Avenue. According to historical topographic map sources, this channel originally had a dirt road crossing along the top of the slope, but that crossing no longer exists.

There is also a concrete structure, located approximately 100 feet back of the top of the bank, near Station 2415+00, that is suspected to be a vent structure for an abandoned sanitary sewer siphon as no structure exists on the other bank. To date, no historical plans have been located to document this approximately 8-foot by 8-foot by 8-foot concrete structure which has a "sewer" manhole lid. City of Fort Worth personnel have been made aware of this site and have made site visits to assess the situation. The city staff believes that the manhole is for an abandoned storm sewer line, rather than a sanitary sewer. Regardless of the nature of the structure, it and the associated piping will require removal and replacement if determined through additional investigation to be active. Along the northwest corner of the northern property an existing 42-inch sanitary sewer (M-106 R\*), runs across Lebow Creek and will not be impacted.

The Samuels Avenue South Site is traversed from Station 2412+00 to 2398+00 by an existing high voltage electric transmission line that runs parallel to the top of the proposed excavated slope. This line is supported on three large four-legged steel towers on approximate 650 foot centers. A minimum 50-foot set-back from the nearest steel tower leg to the top of slope of the proposed excavation was established to protect against any potential erosion into the limits of the tower base.

In addition, the Samuels Avenue South Site has an existing area inlet, located near the top of existing bank at Station 2404+00 to prevent erosion due to localized run-off. The Samuels Avenue North Site has a similar existing area inlet, located near the top of existing bank at Station 2396+00. Both inlets will be removed due to the significant lowering of the bank elevation.

### **1.04.04.03 Material Handling**

#### **Earthwork Quantities**

The top of the river bank excavation is set at Elevation 502.0. Excavation quantities for the Samuels Avenue Sites, based on the contours presented on grading plan sheets CG-06 and CG-07, using 3:1 side slopes with a 1% cross fall slope across the site, from toe of levee to river bank are 552,000 CY for the north site and 315,000 CY for the south site for a total of 867,000 CY. These are presented as raw quantities with no shrinkage or swell factors considered.

#### **Haul Routes & Disposal Sites**

The Samuels Avenue Sites (both north and south) have immediately adjacent City-owned properties that lend themselves to serve as disposal sites. The north site is adjacent to an existing City Landfill, which has two separate cells (west and east). The east cell has been completed and capped at an approximate elevation of 555 while the west cell is still partially active. A significant portion of the east side of the west cell has been completed and capped to an approximate elevation of 550. It is intended to use both of these landfill sites for disposal of the excavated soil. Filling to an elevation of 572 for both cells yields a fill capacity of 560,000 CY for the west cell and 330,000 CY for the east cell, for a total volume of 890,000 CY.

In addition to the landfill area, there is a tract of land, located north and west of Northside Drive and south of the Samuels Avenue South site that is currently being used as a City of Fort Worth auto impoundment lot and is graded to an elevation of approximately 525. Filling the impound site to an elevation of 550 yields a fill capacity of 490,000 CY. Some adjustment in final elevations is anticipated but current grading plans indicate a surplus capacity so reductions in fill elevations should not significantly impact the site.

The proposed grading plan will use the west landfill site to accommodate excavation from the Samuels Avenue North site (approximately 552,000 CY), which is within 8,000 CY capacity of balancing, and to utilize the south auto impound lot to accommodate excavation from the Samuels Avenue south site. Given an estimated excavation quantity of 315,000 CY and a fill capacity of and a fill capacity 490,000 CY, there will be a surplus fill capacity of approximately 175,000 CY. This surplus fill capacity and the northeast landfill area (capacity of 330,000 CY) will provide a total excess fill capacity of 505,000 CY. This remaining capacity of the Samuels Avenue north site, east landfill and south auto impound lot area would be available as a disposal area for Riverside Park excavation and/or other spoils materials, if needed.

### **1.04.05 Riverside Park Site**

#### **1.04.05.01 Site Description and Constraints**

The Riverside Park Site is a 20 acre site, publicly owned (City of Fort Worth) property located on the east bank of the West Fork Trinity within the Trinity River

floodplain (River Station 2316+00 to 2334+00). The site is located immediately north of East Belknap Street and is bounded by the Oakhurst Scenic Drive on the east. The north side of the site is defined by an area of large old growth trees which are to be preserved. Existing park features include an asphalt and gravel parking lot, a lighted softball field, two soccer fields and trail facilities. Being currently used as a park and immediately adjacent to the river and within the floodway, the area is well maintained and regularly mowed. Current site elevations vary from 518 to 524 and slope gradually towards the river.

Within the reaches of this site, the elevation of the top of bank ranges from approximately 518 to 520. The proposed lower excavation elevation (new top of bank) varies throughout the site from 504.0 to 510.0, depending on the location. By varying the excavation limits, the existing park facilities (soccer fields, softball field and parking lot) can be set at elevations that will be appropriate for each use, thereby maximizing the amount of excavation and valley storage generated.

Excavation elevations were based on providing 1 foot above the 2-year frequency event flood elevation of 508 for the main athletic fields while maintaining a minimum of 3 feet above the static water elevation (EL 501) which is controlled by the 4<sup>th</sup> Street low water dam. An existing 18-inch sanitary sewer (M-1728) located on the east side of the site near Oakhurst Scenic Drive will remain in place. An existing 30-inch storm water outfall and box culvert under Oakhurst Scenic Drive, located on the south of the site, will also remain in place. Overhead power lines crossing the site will need to be relocated to accommodate the proposed work. Excavated materials will be transported off-site for disposal.

The entire site is marked by a 2-wire cabled fence along the riverbank, with 4 ft metal posts and a paved trail located outside (toward the river). The existing trail will be replaced along the bank with a combination maintenance and recreational trail. The City of Fort Worth has future plans to tie in the trail with planned sidewalk improvements along Race Street. The parking and athletic facilities have been shifted to the north end of the site for this reason. Access to the parking lot will be provided by a new access road aligned with Race Street. The maximum slope will be 12% or less.

#### **1.04.05.02 Utility Relocations**

The Riverside Park Site is crossed by an existing high voltage electric transmission line, located at Station 2330+00. The line is conveyed on three single wooden poles, one located near the Oakhurst Scenic Drive ROW and two within the property boundary, on approximate 125 ft centers. Near the river bank is a dual pole, with dual guy wires, that serves as an anchor for the 450 ft span across the river to the west. The transmission line and pole will be need to relocated to accommodate the excavation work.

In addition, there is a storm sewer that crosses the site, near Station 2318+00, from an outfall of a 3' x 3' box culvert, along Oakhurst Scenic Drive, immediately into a 36" RCP storm sewer. There is a 2' x 2' Y-inlet, in an open area in the middle of the parking area, which connects to the 36" storm line, which outfalls at the river's edge. The 2' x 2' inlet will be removed by the grading operations which will also remove the existing parking lot. The existing 36-in storm drain will be replaced.

The Riverside Park Site is traversed from Station 2334+00 to 2316+00 by an existing 18" sanitary sewer line (M-1728), that runs along and parallel to the west ROW line of Oakhurst Scenic Drive, to near Station 2326+00, where it jogs to the toe of the slope (downhill of the tree line) along the east side of the site. Near Station 2321+00, the line jogs slightly into the parking area to an observed sanitary sewer manhole, located near the aforementioned Y-inlet, then exits the site just west (and downhill) of the building located south of the south end of the site. Approximately 1600 ft of the 18" sanitary line will need to be re-laid due to grading encroachments as shown on plan sheet CG-09. The sanitary sewer alignment will be shifted to the south travel lane of Oakhurst Scenic Drive to avoid impacting the existing trees along the roadway.

#### **1.04.05.03 Material Handling**

##### **Earthwork Quantities**

Excavation quantities for the Riverside Park site, based on the contours presented on grading plan sheet CG-10, using 3:1 side slopes with a 1% cross fall slope across the site, from toe of levee to river bank are 302,000 CY. These are presented as raw quantities with no shrinkage or swell factors considered.

##### **Haul Routes & Disposal Sites**

The excavation from Riverside Park will be placed at the Samuels Avenue east landfill site which is less than one mile away via a combination of Oakhurst Scenic Drive and Northside Dr. With a cut quantity of 302,000 CY and an available fill quantity of 505,000 CY at the Samuels Avenue Sites which are less than one mile haul. Spoils material that is suitable for levee construction will also be transported to the Ham Branch Site via a combination of Belknap, Sylvania Blvd, and 4<sup>th</sup> Street for use in reconstruction of the back levee.

#### **1.04.06 Ham Branch Site**

##### **1.04.06.01 Site Description and Constraints**

The Ham Branch site is a 100 acre property located east of US Hwy 287/Spur 280 (Martin Luther King Freeway), midway between Interstate 30 to the south and Highway 121 (Airport Freeway) to the north. The site is further bounded by the West Fork Trinity River to the east and the northern extents of the site are approximately 150 ft to the north of the railroad centerline. The property is currently protected by a system of levees on the east and south sides. The site is

primarily owned by the City of Fort Worth and used as a City park identified as Harmon Park. Transecting the site is a small creek that runs diagonally from the northwest to southeast prior to discharging into the West Fork Trinity River through a gate controlled structure. The creek is lined by dense vegetation and is included as a component of the Central City for aquatic habitat mitigation. The site also functions as a storm water sump (Sump No. 31). Other significant site features include a recreation center, three competition soccer fields and a baseball field. A network of sanitary sewer lines along with gas and fiber optic lines exist on the property with a majority being located on the eastern side of the site. Elevations within the enclosed sump area vary from 512 to 520.

The proposed work includes lowering a portion of the existing levee to allow inundation of the site during greater than 100-yr storm events on the Trinity River. Restoration of a portion of a former levee is proposed to the north of the railroad embankment to maintain existing levels of protection to areas outside of the Ham Branch site. Outside of the levee footprint, minimal grading work is proposed because of the site's relatively low grade and ecosystem benefits. The recreational features will be maintained by rerouting a portion of the existing trails to accommodate the levee lowering. Several manholes and inspection chambers will require modification to raise and/or seal their elevation above the SPF water surface elevation.

#### **1.04.06.02 Utility Relocations**

The Ham Branch site has five large (10 ft + x 10 ft+) concrete structures, that function as utility manholes and/or siphon structures, that will be required to be adjusted, in height, to an elevation above the SPF flood elevation of 530. This will require an extension of height of up to 15 ft, for some of the structures.

The site is also traversed by an existing high voltage electric transmission line that runs northwest to southeast across the site. This line is supported on large four-legged steel towers and will not be impacted by the proposed work. Existing gas lines and fiber optic lines that cross the site should not be impacted by the proposed exposure to floodwaters.

#### **1.04.06.03 Material Handling**

##### **Earthwork Quantities**

The lower limits of the river levee 'notch' excavation is set at Elevation 524.0. The proposed northern levee elevation is set at Elevation 534.0. Fill quantities for the Ham Branch site, based on the contours presented within the grading plan, using the 10:1 slopes, along the levee 'notch' and 3:1 side slopes, along the levee, are 15,500 CY (3,300 CY excavation, along the notch and 18,800 CY fill, along the levee). These are presented as raw quantities with no shrinkage or swell factors

considered. The additional fill will come from the Riverside Park Site should suitable levee material be available on site.

### **Haul Routes & Disposal Sites**

The excavation from the Ham Branch levee 'notch' will be used if suitable for construction of the new levee. If this material is deemed to be unsuitable it will be spoiled to the backside of the proposed levee. Additional spoil materials that are suitable for levee constructions will be transported to the Ham Branch Site from the Riverside Park site for construction of the proposed levee.

## **1.04.07 Riverside Oxbow/ Gateway Park Sites**

### **1.04.07.01 Site Description and Constraints**

Located immediately north of Interstate 30 and bounded by Beach Street on the east and Riverside Drive on the west, the Riverside Oxbow site consists of approximately 200 acres that are entirely within the existing 100-yr floodplain. The site is primarily within the confines of the current river channel and the old river oxbow, however, portions of the site extend to the north beyond the oxbow boundaries which are proposed for ecosystem restoration purposes. The Oxbow site also includes some property on the south bank near the former Sycamore Creek channel. The Oxbow property is primarily publicly owned with the exception of a gas drilling site located in the northeast corner of the property that is currently in use. No excavation is planned for this area as shown on the grading sheets. Existing site elevations vary from 510 to 514.

The Gateway Park site is located east of the Riverside Oxbow site. The approximately 225 acre site is bounded by Beach Street on the west, East 1<sup>st</sup> St on the north and the Trinity River Channel on the east and south. Northeast and eastern portions of the site are characterized by woodland vegetation while the central and southern portions of the site are predominantly park and athletic facilities. The northwest portion of the site is largely vacant land with some commercial development along Beach Street. The site includes an abandoned wastewater treatment plant (WWTP) in the southeast quadrant. Property ownership is a combination of public (City of Fort Worth and Tarrant Regional Water District) and private entities. This site is also entirely within the existing 100-yr and SPF floodplain with the exception of a small area along Beach St and the existing ball field and old WWTP which have some protection due to existing fill embankment. Existing site elevations generally vary from 506 to 510.

Proposed work for the Oxbow/Gateway Park area includes grading the two sites to elevations ranging from 5-year to less than 2-year frequency event flood elevations to maximize valley storage benefits. Existing woodland vegetation near the Gateway Park drive, along the Trinity River, and northeastern portions of the site will be preserved and enhanced as part of the ecosystem restoration activities.

Proposed recreational features which will be constructed include recreational fields, restrooms and snack bar facilities, covered basketball courts, boat launches, and splash park. As a related future project the City intends to construct additional recreational fields, parking lots and picnic areas. Recreational trails will be constructed as part of the grading work and include a combination of hard and soft paved trails and equestrian trails. Critical infrastructure facilities will be constructed at or above the 2yr flood frequency elevation. Associated access roads, maintenance road, and parking will also need to be constructed.

Ecosystem enhancements are proposed throughout the site including riparian woodlands, buffer, and native grassland areas. A series of four rock weir structures, large riprap placed in a vane arrangement, are proposed along the old oxbow channel and reconstituted Sycamore Creek channel to create a series of pools and riffles. Dredging of portions of the old channel will be required to create the proposed pools and riffles. All work will be maintained within the existing channel.

Proposed work on the Oxbow portion of the site will encounter an 84-inch sanitary sewer (M-245P\*) and an 18-inch main (M-126) will need to be protected during excavation activities in some areas. A portion of the 18-inch (M-126) line will be replaced to accommodate excavation and grading work. Existing natural gas line and liquid fuel lines which transect the property will remain in place and will also need to be protected during construction. A 54-in water main (E-21) adjacent and to the west of the Beach St bridge may need to be relocated pending survey of the sewer and old oxbow channel during detailed design. A proposed north-south gas line has been proposed from the existing gas well on the northeast corner of the site to the existing Sycamore Creek area. Coordination activities are continuing with the gas company so that the line will not interfere with future grading work on the site. Portions of the excavated material from this site will need to be transported off-site for disposal.

On the Gateway portion of the site the pond area on the southeast will be constructed by the City of Fort Worth as part of their on-going park development activities. The abandoned wastewater treatment plant site is proposed for demolition and subsequent disposal of excavated materials to minimize transport expenses. Demolition and clean-up activities of the abandoned wastewater treatment plant are to be conducted by the City of Fort Worth under a separate project. The site contains an 84-inch sanitary sewer (M-245P\*), 72-inch (M-245-AR), 60-inch (M-389), and 21-inch (M-181A) s which now carry wastewater flow to the Village Creek Wastewater Treatment plant will need to be protected during excavation activities in some area. An existing 10-inch water main will also need to be protected. Additional utilities include an overhead high voltage transmission line.

### 1.04.07.02 *Utility Relocations*

The Riverside Oxbow Site is crossed by an existing high voltage electric transmission line on the southeastern portion of the site. The line is supported on a series of large four legged steel towers. The line crosses Beach Street to the north of the existing Beach Street Bridge over the West Fork. Relocation of the line is not proposed with existing grades being maintained at each respective tower. The electrical utility corridor will be maintained and free of trees and large woody vegetation.

An 84-inch sanitary sewer (M-245-P\*) runs parallel to I-30 along the south edge of the Oxbow site until it crosses the West Fork to the west of Sycamore Creek. The line proceeds east, parallel to and north of the West Fork until crossing the old oxbow channel with an above grade crossing where the line continues east to the old WWTP. A 60-in (M-389) sewer main lies parallel and south of the West Fork until it crosses the West Fork, downstream of the existing Beach Street low water dam, and continues east parallel to the M-245-P\* line to the old WWTP. Both of these lines will require special precautions during excavation work to protect these lines, see utility sheets CU-02 and CU-03. The southeast corner of the site has an 18-inch main (M-126) vitrified clay sanitary line that is proposed to be replaced and relayed at its existing grade, see utility sheet CU-03. Replacement is necessary due to adjacent excavation activities and concerns that this line may be disturbed and damaged during construction activities. Two water lines, 36-in (E-11) and 46-in (E-21) are located within the Beach Street ROW near the Beach Street Bridge crossing over the West Fork. Proposed improvements to Beach Street will require existing water vales to be adjusted. Modification of the old oxbow channel and replacement of the existing box culvert with a clear span bridge over the channel, see Section 2.03, may necessitate the lowering of a 54-in (E-21) the water main to accommodate the channel and bridge improvements, see utility sheet CU-04. The City has also planning for a new parallel 54-inch water main in Beach Street which will need to be coordinated with this project. Construction of this line is outside of the context of this project.

The Gateway Site is crossed by the same high voltage electric transmission line as previously stated. The line diagonally crosses the middle of the site. Relocation of the line or towers will not be required based on the proposed grading.

Several sanitary sewer lines cross the site including the 84-inch (M-245-P\*) and 60-inch (M-389) line previously discussed. A 72-inch (M-245-AR) line crosses the site from Beach Street to the old WWTP on the north side of the existing park road entrance. These sanitary sewer lines converge at junction boxes located on the southwest corner of the abandoned WWTP site. Relocation or adjustment of the junction structure will not be required based on the proposed grading. A 21-inch (M-181-A) sanitary sewer line and 10-inch water line run north-south through the middle of the site as shown on sheet CU-01. These lines will require special precautions during excavation work to protect these lines, see utility sheets CU-04 and CU-05. In addition, two sanitary lines, 90-inch (M-388-B) and 54-inch (M-280-B) lie on the eastern edge of the site. Both of these lines are outside the current

grading limits but will need to be reviewed during future design efforts to protect them from impacts which could be caused by hauling of excavated materials. One 18-inch abandoned sewer line (M-131) is planned to be removed to allow grading work, see sheet CU-04.

### **1.04.07.03 Roadways**

The Riverside/ Gateway site includes three new roadways and one roadway improvement as shown on sheets CP-01 to CP-06. The roadway improvement consists of replacing the existing box culvert on Beach Street with a new clear span bridge over the old oxbow. As previously noted, a potential utility conflict with a 54-inch (E-21) water main exists to the west side of the bridge. Detailed survey during final design will be required to confirm the location and depth of the line and the required action. In addition the project includes the relocation of the existing park entrance, north of the bridge, to a location between the West Fork and old oxbow. Replacement of the bridge and entrance will necessitate reconstruction of the roadway. New roadways will include the aforementioned new park entrance, referred to as the East Park Road which will connect Beach St to the Gateway site. On the Riverside Oxbow sites a new entrance roadway and spur, West Park Road and West Park Road Spur, will be constructed to provide access to the proposed recreational facilities.

### **1.04.07.04 Material Handling**

#### **Earthwork Quantities**

Excavation quantities for the Riverside Oxbow, are based on the contours presented on grading plan sheets CG-21 to CG-29 are 2,215,000 CY. Total excavation quantities from the Gateway Park Site are 860,000 CY which does not include the proposed grading by others on the southeast corner of the site. See Sheets CG-28 and CG-29, Appendix C, Vol. II Supplemental Plans. In addition, approximately 13,500 CY is anticipated to be excavated from the old oxbow channel to develop the riffle pools for the ecosystem restoration work. These are presented as raw quantities with no shrinkage or swell factors considered.

#### **Haul Routes & Disposal Sites**

The excavation spoils from Riverside Oxbow will be placed at a combination of an off-site disposal site and the old WWTP site. Approximately 1,163,500 CY is estimate to be transported to an offsite disposal site with the remaining 1,925,000 CY being used to fill on-site at the old WWTP, area adjacent to Beach Street and hill area north of the existing Dog Park. Offsite material will be transported primarily on Beach and 1<sup>st</sup> Street.

### **1.04.08 University Drive Modification**

The University Drive Modification was previously included in the original approved Central City FEIS and Technical Appendix C. Since no changes to the proposed roadway raise are proposed the site it is not discussed in this Supplement.

## **2.0 Structural Component Design**

The following sections provide design information on the structural components of the Modified Central City project.

### **2.01 Structural References**

The Structural design for the relocated Samuels Avenue Dam, Marine Creek Low Water Dam and lock structure is in accordance with standard engineering practices and guidance as set forth in various published manuals as follows and applicable:

#### **US Army Corps of Engineers**

EM 1110-1-1905, Bearing Capacity of Soils, 30 Oct 92

EM 1110-2-1418, Channel Stability Assessment for Flood Control Projects, 31 Oct 94

EM 1110-2-1901, Seepage Analysis and Control for Dams, CH1, 30 Apr 93

EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures, CH2, 31 Mar 01

EM 1110-2-2006, Roller-Compacted Concrete, 15 Jan 00

EM 1110-2-2100, Stability Analysis of Concrete Structures, 01 Dec 05

EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures, CH1, 20 Aug 03

EM 1110-2-2105, Design of Hydraulic Steel Structures, CH1, 31 May 94

EM 1110-2-2200, Gravity Dam Design, 30 Jun 95

EM 1110-2-2400, Structural Design and Evaluation of Outlet Works, 02 Jun 03

EM 1110-2-2502, Retaining and Flood Walls, 29 Sep 89

EM 1110-2-2504, Design of Sheet Pile Walls, 31 Mar 94

EM 1110-2-2602, Planning and Design of Navigation Locks, 30 Sep 95

EM 1110-2-2607, Planning and Design of Navigation Dams, 31 July 95

EM 1110-2-2610, Lock and Dam Gate Operating and Control Systems, CH1, 02 Apr 04

EM 1110-2-2703, Lock Gates and Operating Equipment, 30 Jun 94

EM 1110-2-2704, Cathodic Protection Systems for Civil Works Structures, 01 Jan 99

EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Protection Projects, 31 Mar 94

EM 1110-2-2906, Design of Pile Foundations, 15 Jan 91

ER 1110-2-110, Instrumentation for Safety – Evaluations of Civil Works Projects, 08 July 85

ER 1110-2-1806, Earthquake Design & Evaluation of Civil Works Projects, 31 July 95

TL 1110-2-256, Sliding Criteria for Concrete Structures, 24 Jun 81

### **Unified Facilities Guide Specifications**

UFGS 09 97 02 (09964), Painting: Hydraulic Structures

### **American Concrete Institute Specifications**

ACI 318-05, Building Code Requirements for Structural Concrete

### **American Institute of Steel Construction**

AISC 360-05, Specification for Structural Steel Buildings

### **American Association of State Highway Officials**

LRFD Bridge Design Specifications, 3<sup>rd</sup> Edition, 2004

## **2.02 Structural Specifications**

Loading parameters, unit weights and properties of materials used in the design of the relocated dam and lock structures, as well as the allowable stresses calculated, are as follows:

Unit weight of water	62.5 pcf
Unit weight of concrete	150 pcf
Unit weight of steel	490 pcf
Concrete compressive strength ( $f'c$ ):	
Typical:	4,000 psi
Access bridge prestressed girders	5,000 psi

Steel:

Wide Flanges:	ASTM A992
Channels:	ASTM A36
Pipes:	ASTM A53, TYPE E OR S, GRADE B
HSS:	ASTM A500, GRADE B
Plates:	ASTM A36
Misc.:	ASTM A36
Anchor Rods:	ASTM F1554, Grade 36 or 55, WELDABLE
Reinforcing steel:	ASTM A615, Grade 60

Backfill materials:

Moist unit weight	120 pcf
Saturated unit weight	120 pcf
Submerged unit weight	57.5 pcf
Cohesion (Q-case)	0 psf
Internal friction angle (Q-case)	32 degrees
Cohesion (S-case)	0 psf
Internal friction angle (S-case)	32 degrees

Foundation Materials:

Moist unit weight	120 pcf
Saturated unit weight	120 pcf
Submerged unit weight	57.5 pcf
Cohesion (Q-case)	0 psf
Internal friction angle (Q-case)	32 degrees
Cohesion (S-case)	0 psf
Internal friction angle (S-case)	32 degrees

Roller Compacted Concrete (RCC):

Moist unit weight	140 pcf
Saturated unit weight	140 pcf
Submerged unit weight	77.5 pcf
Cohesion	300 psf
Internal friction angle	45 degrees

Many of the preceding material properties are based on properties which are reasonably anticipated or based on very limited geotechnical investigations. As

the design progresses and geotechnical design parameters are developed, these properties will require reevaluation.

## **2.03 Beach Street Bridge**

The proposed Beach Street Bridge was previously included in the Riverside Oxbow Ecosystem Restoration Study and no changes are proposed at this time. Plans for the bridge from the previous study are included by reference in Appendix C, Volume II - Supplemental Plans. The preliminary design for the Beach Street Bridge is a 115-foot long clear span bridge consisting of a concrete deck on prestressed concrete girders and supported on 16 3'-diameter drilled shafts, based on current, limited geotechnical data. The design includes four lanes of traffic (two each way) with pedestrian walkways on each side for a total width of 80 feet. The bridge replaces an existing 10' X 12' box culvert. One of the pedestrian walkways will be 10 feet wide to accommodate both pedestrian and bicycle traffic.

## **2.04 Park Road Bridge**

The proposed Park Road Bridge was previously included in the Riverside Oxbow Ecosystem Restoration Study and no changes are proposed at this time. Plans for the bridge from the previous study are included by reference in Appendix C, Volume II - Supplemental Plans. The preliminary design for the Park Road Bridge is a 103-foot wide clear span bridge consisting of a concrete deck on prestressed concrete girders and supported on 6 3'-diameter drilled shafts, based on current, limited geotechnical data. The design includes two lanes of traffic (one each way) with pedestrian walkways on each side for a total width of 44 feet. One of the pedestrian walkways will be 10 feet wide to accommodate both pedestrian and bicycle traffic.

## **2.05 Samuel Avenue Dam and Lock**

### **2.05.01 Location**

The Fort Worth Central City Project included an in-channel dam to achieve the urban design objective of maintaining water levels in the project interior at a relatively constant normal water surface elevation of approximately 525 NGVD. The original dam site location resulted in potential impacts on Marine Creek due to both the high backwater elevation as well as a resulting increase in operations to pass flood flows on the Marine Creek watershed. The original site also impacted the lower segment of Lebow Creek by loss of habitat resulting from rerouting of the creek downstream of the dam.

As part of the evaluation of the Modified Central City Project, relocation of the Samuels Avenue Dam was considered for potential impacts on valley storage requirements, to address potential upstream impacts on Marine Creek and the hydraulic constraints posed by the original location. Alternative sites for the dam

on the West Fork upstream of the Marine Creek confluence, ranging from immediately at the confluence to just downstream of Northside Drive, were evaluated. Placing the dam to close to the Marine Creek confluence could introduce scour potential at the Samuels Avenue Bridge, while placing it further upstream towards Northside Drive reduced or eliminated options to maintain hydraulic connectivity with Marine Creek. Sites south of Northside Drive were eliminated from consideration due to impacts on Northside Drive, limited area, and conflicts with the bypass channel.

The selected revised location of the gated dam is proposed on the main stem of the West Fork of the Trinity River just upstream of the confluence with Marine Creek. The dam is still referred to as the Samuels Avenue dam due to its proximity to the Samuels Avenue Bridge. The dam is sited approximately 1,750 feet downstream of Northside Drive, immediately upstream from the confluence of Marine Creek. The dam was sited upstream from Samuels Avenue Bridge and the adjacent three railroad bridges in order to allow for a lower, separately maintained water level on Marine Creek.

The downstream end of the northern stilling basin wall will connect to a low water dam located on Marine Creek which will maintain a normal water pool level elevation of 516.5. The location was set with the front, upstream edge of the structure 670 feet upstream from the centerline of Samuels Avenue Bridge. As shown on sheet SS-01, this location provides sufficient room for the structure to be constructed with appropriate grading that transitions both the 390-foot wide main dam and the 200-foot wide Marine Creek dam to the 250-foot wide channel. The two dams were oriented so that their discharges could be directed and aligned with the downstream channel.

The two pools will maintain hydraulic connectivity through the use of a lock and channel located on the west side of the dam, allowing small boat traffic to travel upstream and downstream of Samuels Avenue Dam. The lock structure will be approximately 40-feet long by 16-feet wide and have a maximum hydraulic lift of 8.5 feet.

The benefits of this dam site include reduced backwater impacts to Marine Creek as well as simplifying the operational demands of Samuels Avenue Dam by allowing Marine Creek flood flows to pass without affecting the urban lake pool elevation. In addition, hydraulic connectivity between the Stockyards and Downtown is maintained so that all project objectives are met. A secondary benefit to the revised dam site is the elimination of adverse impacts to Lebow Creek and associated habitat.

### 2.05.02 Design Assumptions

The Corps of Engineers is currently in the process of acquiring additional geotechnical information. Due to unavailability of this additional information at the present time, geotechnical data from the previous investigation (contained in Appendix B of the Central City Project FEIS) as well as historical boring information from the Northside Drive bridge and previous USACE floodway work was used in the following feasibility-level design. For the basis of this design, the parameters were considered conservative. As additional data becomes available in future design phases, these parameters will be reviewed and refined as appropriate.

Based on the available data, the top of rock elevation was assumed to be uniformly at elevation 474.0 NGVD. Should the actual top of rock be determined to be substantially higher than this, other structural systems such as roller compacted concrete (RCC), as was proposed at the original location, may become more viable. If the depth to rock is deeper, the only impact to the proposed design should be a marginal increase in cost due to increase drilled shaft lengths. The same soil parameters used in the previous FEIS were used in this design analysis.

### 2.05.03 Design Criteria

A series of hydraulic loads were reviewed, ranging from static normal pool with minimal base flows to the 100-year and up to the SPF. Selected combinations of headwater and tailwater that would prove to be the critical design loading were utilized in the structural design

#### Hydrostatic Loads:

Case	Classification	Head Water Elev. (ft-MSL)	Tail Water Elev. (ft-MSL)
No flow	Usual	524.3	499.0
10-yr	Usual	524.3	509.8
25-yr	Unusual	524.4	520.8
100-yr	Unusual	526.5	525.0
500-yr	Extreme	533.6	531.4
SPF	Extreme	540.7	537.8

Silt Loads: Silt loads are based on silt up to the gate sill elevation. Silt load is computed using a buoyant weight of 40 pcf and a lateral coefficient of pressure of 0.6.

Dynamic Water Loads: Hydraulic loads such as wave action will be considered as appropriate. Unless the actual hydraulic jump profile is modeled, the tailwater contribution to lateral stability will be based on an aerated unit weight of 60% of

normal. If the actual hydraulic jump profile is modeled, the full unit weight of the tailwater will be used.

Uplift Loads: Uplift loads will be computed based on cutoff location, drain location, drain efficiency and linear reduction to tailwater pressure.

Seismic Loads: Seismic Loads are expected to be negligible in the project area and are not anticipated to provide any controlling factor in the final design. Once suitable geotechnical information is available, this assumption will be verified.

#### **2.05.04 Design Methodology**

The dam structure will be constructed primarily of reinforced concrete, with much of the structure supported on drilled shafts that extend down to bedrock, which was assumed to be at elevation 474 NGVD.

Stability design will be based on USACE criteria (EM 1110-2-2100, Stability Analysis of Concrete Structures) except where reinforced concrete strength is provided by lateral bearing in soil and rock. Where lateral bearing in soil and rock is used, the software program Lpile Plus, version 4.0 by Ensoft, Inc. will be used in general conformance with the methodology of USACE criteria (EM 1110-2-2906, Design of Pile Foundations).

It should be noted that seepage data for the soils on site were not available for the preliminary design. Underdrains and wall drains are indicated where anticipated by the preliminary design. The type and extent of these drains will be determined in the final design based on the seepage data available at that time.

#### **2.05.05 Design of Foundations**

The preliminary design of the Dam on the main channel is based on a drilled shaft supported platform. The assumed depth of rock was such that the previous foundation approach to expose the rock and place roller compacted concrete was no longer feasible. The revised design philosophy is to found the platform on rock using drilled shafts embedded into the bedrock below a sufficient depth to develop the moment capacity of the shafts at the top of rock elevation. Due to the indeterminate nature of the platform bearing on the underlying soils and the drilled shafts, it is conservatively assumed that the platform receives all of its support from the drilled shafts. It is consistently assumed that the soils resist lateral movement without benefit of any overburden pressure due to the presence of the structural platform.

The foundation analyses were generally performed using MathCad software version 13.1 by MathSoft. MathCad provides a versatile automated calculation sheet. Calculation methodology was performed in conformance with the principles and procedures in the referenced design manuals.

The drilled shafts are designed to resist the net lateral load in the structural platform. The drilled shafts are designed to bear laterally in the soil and rock below the dam. The behavior of the shafts is identified using Lpile Plus, version 4.0 by Ensoft, Inc. The lateral bearing pressure of the shafts in the soil will be integrated along each shaft and summed for the group of shafts and compared to the lateral shear capacity of the surrounding soil to check shaft group behavior. Distribution of lateral load within the group is believed to be very uniform. The shafts have relatively large spacings and all shafts are of the same size, length and have similar surrounding soil. Additionally, the dam slab has a relatively large size and stiffness when compared to the drilled shaft and soil system. The slab should be capable of transmitting lateral load to the entire drilled shaft group in a reasonably uniform manner.

The stilling basin is cast-in-place concrete and has been thickened to resist uplift. The joint between the stilling basin and the dam superstructure will be an expansion joint and will not be dowelled to allow for differential settlement. The stilling basin side of the joint will be stepped down slightly to avoid water impacting the joint.

The preliminary design is based on the material below the dam being cohesionless and having an angle of internal friction of 32-degrees.

## **2.05.06 Design of Superstructure**

The design of the concrete superstructure is based on EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures, EM 1110-2-2906, Design of Pile Foundations and the principles of EM 1110-2-2200, Gravity Dam Design. Steel elements will be designed per AISC 360-05, Specification for Structural Steel Buildings.

The forces in the superstructure are statically determinate except for those resulting from the direct connection to the drilled shafts. No other changes to the superstructure are required due to the revised dam location.

### **2.05.06.01 Gate Configuration**

No changes to the proposed gates or their operating system are needed due to the change in dam location. For additional information on the previous gate analysis, refer to Section 8.3, Gate Configuration, in Appendix C of the Central City Project FEIS.

A separate small control building to house the operational controls for the gates and the lock system will be needed in the area. The proposed location is on the west abutment as shown on the plans. This location would be above SPF flood levels but have a direct line-of-sight with both the lock and gates.

The spillway bridge will provide access to the gates for routine maintenance and operations. Primary gate operations, both low flow and flood gates, are planned from the control building, but manual override capabilities will necessary at the gate. This bridge is assumed to be designed for light vehicular traffic only. Cranes for heavy repair or maintenance will access the gates from the downstream side.

### 2.05.06.02 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, with the downstream exit channel graded to 495.0. The lower portion of the stilling basin was set to a length of 60 feet. The critical configuration was based on two gates fully opened, which would pass slightly less than the 10-yr flood, or 21,300 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 10,600 cfs, slightly less than the 2-yr 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. This is described more fully in the following sections.

A sheet piling system will be utilized as a positive seepage cutoff for the dam and in each abutment. It will also be used for diversion and construction sequencing, as described in section 2.07. The front face of the dam will be connected to the final top of the sheetpiling when cut to final grade, in a water tight manner, as shown on Sheet SS-4. Since subsurface information is not available, the viability of the sheetpiling providing a positive cutoff with the rock foundation is unknown. Additional options that will be considered once geotechnical information is available include:

- In the abutments and where the sheetpiling has a moderate contact with the rock, grouting of the top of the rock and the material behind the sheetpiling can be performed.
- If boulders are encountered or other evidence that the sheetpiling has a poor contact with the rock, jet grouting would be a feasible additional item to provide a cutoff in these zones.
- If sheetpiling is determined to be an ineffective cutoff, a slurry trench, either concrete or soil bentonite, could be used for the subsurface cutoff. However, sheetpiling will still likely be needed for diversion and phasing of construction, though likely at a reduced cost.

The Samuels Avenue Dam will be constructed on soil well above the bedrock and underseepage control will be a design issue. The seepage flow rate is not critical to the hydraulic design. The design is anticipated to be primarily a structural concern; however the length of the cutoff could trigger environmental issues that must be incorporated in the final choice of seepage control. According to USACE's EM 1110-2-1901, methods for control of underseepage include horizontal drains, cutoffs (compacted backfill trenches, slurry walls, concrete walls, and steel sheetpiling), upstream impervious blankets, downstream seepage berms, toe drains, and relief wells.

The method selected for the conceptual design is a combination of a steel sheetpile cutoff and a horizontal drain. Steel sheet piles are well suited to serve both as the permanent cutoff and as the construction cofferdam. Although the USACE does not encourage the use of steel sheet pile cutoffs to prevent underseepage because relatively minor flaws may reduce the efficiency, precedents do exist. In our opinion, the steel sheet pile cutoff can be effective for this project if the subsurface deposits do not include boulders that can damage the sheets, the bedrock surface is not too irregular, and the bedrock near its surface is reasonably intact and impervious. Design of sheet piling will be in accordance with EM 1110-2-2504, Design of Sheet Pile Walls. If exploration shows the subsurface conditions are unfavorable, a slurry trench cutoff or a concrete wall constructed in a slurry-filled trench may be necessary. The soils are expected to be too permeable and deep for economical construction of a compacted backfill trench, particularly with staged construction, and the staging does not favor construction of an upstream pervious blanket. An initial efficiency of 25% has been assumed for the cutoff.

A horizontal drainage blanket is planned to control hydrostatic uplift and prevent piping by providing a filtered outlet for water that passes through or around the cutoff. Uplift and stability analyses were based on an assumed straight-line distribution of head between the cutoff and the toe of the structure. This assumption is conservative because the drain will extend under the entire structure downstream of the cutoff, providing much shorter seepage paths.

The quantity of underseepage will be highly dependent on the efficiency of the cutoff. Seepage loss of stored water is not a critical design issue for these dams.

Both abutments require a positive cutoff in the floodplain adjacent to the structure to minimize seepage losses once the normal water level is reached. The right, or east, abutment was assumed to have about 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 the same sheet piling that is to be used for diversion. An alternative would be a soil-bentonite slurry trench cutoff. It will be constructed from the back side of the structure wall to the toe of the levee. The cutoff is not likely to be able to be extended laterally far enough to reach an impervious zone and will, therefore, need to extend sufficiently into the abutment to reduce the final seepage gradient down to an acceptable level. The left, or west,

abutment was assumed to have a similar foundation and will also have a sheet pile cutoff, though it will also be integral with the lock system. It is known that this abutment also contains similar pervious material but that it does not extend as far until competent rock is at a suitable elevation to provide a full positive cutoff. It is assumed that the sheet pile cutoff will be extended to a point in the west abutment at which an open cutoff trench filled with compacted clay will be more economical. This cutoff trench will extend to a point where the rock is high enough to complete the positive cutoff. This location can only be determined once the geotechnical exploration is complete and the depth to rock, the depth to the water table, and the overburden soil properties are determined. Assumed extents of the sheet piles are shown in the drawings.

A physical model study of the Samuels Avenue Dam, the Marine Creek Low Water Dam and the gate operations is recommended as part of the final design process. This will aid in designing the final configuration of the structure, particularly the stilling basin and adjacent erosion protection measures; fine tuning the hydraulic control parameters, and validating appropriate gate operations and sequencing procedures. The physical model study will also confirm flow patterns as the water transitions to the current channel under the Samuel Avenue Bridge and the railroad bridge.

### **2.05.06.03 Spillway Operation**

Preliminary sequencing was assumed to consist of partial gate openings for up to four gates, assumed to be gates 2 through 5, assuming the gates are numbered sequentially starting at the east end. This will align most flows better with the original channel and the opening of Samuels Ave. bridge. Once tailwater is sufficiently high, the remaining gates can be opened in the order of 1, 6, and 7. Once the last gate is fully opened, at approximately the 100 year flood level, the tailwater will be sufficiently high to prevent scour and erosion at the left downstream training wall, which also serves as the right training wall for the Marine Creek Dam. At the 100 year flood level, the difference between headwater and tailwater at the gates is only about 1.5 feet and the average velocity in the downstream channel is only about 4 fps. Though local turbulence will exist, significant scouring is not anticipated. In addition, at these higher flow levels, the angled training wall will also serve to redirect the higher flows more towards the Samuels Ave. bridge, reducing erosion potential there. These assumptions will need to be verified in a physical hydraulic model study of the dams. A full gate operating plan has not been developed at this time and will be developed as part of the recommended physical model study.

### **2.05.07 Design of Lock and Appurtenances**

This section summarizes the conceptual design criteria and major mechanical components of the water connectivity lock. The following subsections include the overall lock dimensions and operational narrative; basis of lock gate selection; illustrative details of the lock gate components; lock gate structural and

mechanical design; lock gate operating machinery design; lock pumping system hydraulic and mechanical design; and miscellaneous lock structures and appurtenances.

The proposed lock will be constructed within a channelized section of the Trinity River adjacent to the Samuels Avenue Dam and will provide connectivity for light vessels between the Central City reach of the Trinity River and Marine Creek, providing access to the Stockyards. Design of the lock will be based on EM 1110-2-2602, Planning and Design of Navigation Locks, as applicable to this structure.

The structural design of the lock is based on EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures. The lock will be founded on on-site soils and has a relatively low bearing stress. The sheet pile cutoff will pass under the upstream end of the lock structure. In order to prevent differential movement, the lock will be rigidly connected to the sheet piles. Because this connection creates a possibility of the sheet piles becoming a support point, the lock design should be conservatively designed as if the upstream end is supported entirely by the sheet piles.

#### 2.05.07.01 Lock Dimensions and Operation Narrative

The maximum lift of the proposed lock is 8.5 feet, which classifies the structure as a *very-low-lift* type based upon USACE nomenclature. The dimensions and design features of the lock are as follows:

Upper pool elevation (Trinity River):	525.0
Lower pool elevation (Marine Creek):	516.5
Maximum navigation lift:	8.5 ft
Design vessel maximum length:	39 ft
Design Vessel width:	12 ft
Design Vessel Side clearance:	2ft
Design vessel draft:	4 ft
Sill clearance:	3 ft
Lock chamber width:	16 ft
Usable length of lock:	40 ft
Operation Time (estimated)	
Fill (raise):	10 min

Drain (lower):	10 min
Lock chamber volume at maximum lift:	55,000 gal
Lock sill monolith spacing:	46 ft 8 in
Downstream sill elevation:	509.5
Upstream sill elevation:	518.0

The lock system will be operated manually by a lock system operator. The lock will be filled and drained through a side-port-flume. The flume will be constructed as part of the shore-side lock wall. The flume will be filled from the upstream pool and drained to the downstream pool.

Flow into and out of the flume will be controlled by two hydraulically-operated stainless steel slide gates installed at the upstream and downstream ends of the flume. The hydraulic cylinder operators on the slide gates will be controlled by the operator from the lock control panel. The hydraulic cylinders will be mounted on support beams installed above the side-port-flume. A single dedicated hydraulic power unit (HPU) will be installed in the lock house. The HPU system will be custom designed with each component individually specified for heavy duty industrial service. A detailed description of the hydraulic power unit design is provided below.

#### **2.05.07.02 Lock Gate Selection and Design Criteria**

For locks in this size classification, the standard Corps design utilizes sector gates. However, for this application, miter gates were preferred over sector gates due to their simplicity in design and successful implementation on similar very-low-lift dam and lock schemes. In addition miter gates require less civil concrete and overall width of the lock structure when compared to sector gates. A detailed structural and mechanical design will be prepared for the lock gates, suitable for fabrication by a firm experienced in the construction of steel hydraulic structures. Firms meeting the experience qualifications will be listed in the specifications. All components and materials will be selected to minimize custom fabrication of the major wear component replacement parts. Consideration shall be given during bid/ contract document development for the procurement of selective spare parts which are custom or may become obsolete in the future.

In addition, a detailed corrosion assessment will be included during the next phase of design. Where necessary, corrosion resistant materials will be selected and a detailed cathodic protection system will be added to the design. Cathodic protection system will be designed in accordance with the guidelines included in EM 1110-2-2704 "Cathodic Protection Systems for Civil Works Structures".

The following is a summary of design data for the proposed lock miter gates:

Miter Angle:	24 degrees
Pintle-to-Pintle spacing:	54 ft 11¾ in
Clear opening at each gate:	16 ft
Upstream Gate Leaf Dimensions:	
Width:	113 in
Height:	115 in
Downstream Gate Leaf Dimensions:	
Width:	113 in
Height:	217 in
Gate Leaf Bottom Clearance:	4 in

Gate structural design will be based upon the Load and Resistance Factored design (LRFD) load criteria included in EM 1110-2-2105, "Design of Hydraulic Steel Structures", with the exception that the ice load criteria will not be used, given the local weather conditions. Additional loading and other design criteria will be adapted from EM 1110-2-2703, "Lock Gates and Operating Equipment".

The miter gate design for this project will differ from the typical USACE design since the size of the gate is much smaller than the standard miter gates illustrated in the Corps manuals. Illustrative details of the miter gates, including plan and elevation views, and important details of construction are included on Sheet SS-10, and Figure C-1 to C-7. These drawings and the following descriptive paragraphs, describing the lock gates and operating machinery, should be reviewed concurrently.

The miter gates will be horizontally framed; constructed of structural steel as specified above; and fit into lock wall recesses when in the open position. The major components of each gate leaf include the following:

1. Skin Plate
2. Horizontal girders
3. Diagonals
4. Miter contact post assembly
5. Leaf quoin
6. Quoin pivot shaft

7. Upper quoin pivot shaft gudgeon bracket and torque arm assembly
8. Bottom quoin pivot shaft gudgeon bracket
9. Quoin, bottom and miter contact seal assemblies
10. Access walkway
11. Gate leaf fenders

The miter gate skin plate will be reinforced by horizontal girders and diagonal members (see Figure C-1 and C-2). Wherever possible open members that are not susceptible to trapping mud and other debris will be used. Where the use of open framed members is not practical, horizontal members will include drain holes, outside the locations of the largest moment reactions, to allow easy removal of mud and other trapped debris by high pressure hose. Other locations susceptible to silt and debris build-up will be reviewed and provisions made for maintenance during the next phase of design.

The materials of construction for the gate leaf, embedded parts and hardware will be reviewed in detail and selected during final design. The evaluation of these materials will be economics and performance-based. Miter gates are typically constructed of A36 structural steel and coated with a high performance epoxy spray coating system. Alternatives to this approach include the use of stainless steels, carbon steel coated with thermal sprayed corrosion resistant metal overlays. Hardware will be constructed of corrosion resistant materials selected to meet the mechanical requirements of the installation, and protect against excessive material loss during service.

The leaf quoin will be constructed of a tubular steel section fitted with welded steel guide plates for the pivot shaft. The tubular members will be seal welded to prevent moisture infiltration and corrosion to the member. The use of open framing members for the leaf quoin, having less potential to trap moisture requiring frequent maintenance, will also be evaluated in subsequent phases of design.

The quoin shaft will be constructed of 400 series or 17-4PH stainless steel, machined to a reduced diameter at the top to form drive shafts with square key-seats for linkage to the torque-arm (see Figure C-3). The torque arms will operate on ultra-high molecular weight polyethylene (UHMW-PE) thrust bearings mounted above the upper pivot shaft bearing barrel. The bearing barrel will be welded to the upper bearing bracket, which will be anchored into the lock wall. The sleeve bearings will have machined grooves for grease lubrication. The bottom of the pivot shafts will be machined to form a smaller diameter hinge pin, which will operate in a bottom hinge bracket fitted with bronze sleeve and thrust bearings (See Figure C-6).

The gate leaf mitered ends will be constructed of vertical structural sections, welded to reinforced contact blocks fabricated from break-formed welded steel plate. The contact blocks will be fitted with elastomer pads as shown in Figure C-4.

Fenders will be incorporated into the gate leaf to provide protection to boats in the lock during operation. Fenders will be EPDM rubber arch-type utilized on larger USACE lock gates. The fenders will be specified with adapted language from UFGS specification Section 39.59.13.19, "Arch-Type Rubber Marine Fenders".

### **2.05.07.03 Miter Gate Seals**

The gate seals will be constructed of neoprene rubber, containing reinforcing carbon black, zinc oxide, accelerators, antioxidants, vulcanizing agents, and plasticizers. The gates will be designed to operate under a differential head of 19 ft. The allowable leakage rate at this head pressure will be no greater than 1-fl.oz./ft-of-seal/sec. Gate seals will be arranged on the skin plate for upstream sealing. Sealing will occur in one direction. Seals will be fabricated from molded neoprene having a Shore-A Durometer hardness of 65 (See Figure C-4 and C-5).

Seals will be clamped to the skin plate by stainless steel retaining strips and UHMW-PE spacers, 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.

### **2.05.07.04 Miter Gate Operating Machinery**

A lock house will be constructed on the parapet adjacent to the lock chamber. The lock house will contain all of the hydraulic operating machinery for the lock gates. The miter gates will be hydraulically operated via the torque arm linkage to the pivot shaft (see Figure C-7). An electrically-operated hydraulic power unit (HPU) will be installed in the lock house. A gimbal-mounted double-acting hydraulic cylinder will operate the gate from a recess at the top of the lock wall. Hydraulic cylinders will be constructed of stainless steel.

The HPU will control both the hydraulically operated miter gates and slide gates at the side-port-flume. The HPU controls will be PLC-based, and operator interface terminals will be installed inside the lock house and locally at the lock wall. A pedestal-mounted local control panel will be installed outside the lock house adjacent to the shore-side lock wall. During final design, consideration will be given to programming the PLC for automated operation with a human machine interface (HMI) installed on the lock parapet for use by boaters when the lock operator is not available or potentially phased out in the future. The PLC will also be programmed to prevent accidental opening of the lock gates prior to equalization of the pools on both sides. A process instrumentation diagram and detailed control descriptions will be developed during the next phase of design.

Two hydraulic power units will be custom designed for the project, one for the stainless steel slide gates on the side-port-flume, and the other for the lock gate mechanism. A system of accumulators will be provided for emergency gate operation during a power outage. This will be accomplished by manual override controls provided on the valve stacks. Accumulators will be the piston type with auxiliary nitrogen bottles provided as necessary. The system will be provided with variable volume pumps or constant volume pumps having adjustable frequency drives to meet the load demand during gate operation.

## **2.05.08 Design of Ancillary Walls**

The basic configuration of ancillary retaining and flood walls is a cast-in-place concrete inverted T. The design of retaining walls will be based on stability criteria consistent with EM 1110-2-2100, Stability Analysis of Concrete Structures. The strength of these structures will be based on EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures.

The walls will be designed to bear on and be backfilled with onsite soils. It may be necessary to use select on site soils as backfill to meet required soil parameters used in the analysis. The final design will confirm that the parameters used are consistent with the data from the geotechnical investigation.

In general, the walls will be designed to have independent foundation systems from the adjacent structures and will be provided with expansion joints at the transition points. The exception to this is abutment walls at the dam and basin sections. These walls receive adequate sliding stability from the adjoining structure. Rotational stability and strength design for abutment walls will be similar to those of the typical retaining walls. Detailing of articulation and transition points will be provided during final design.

## **2.06 Marine Creek Low Water Dam**

### **2.06.01 General**

A fixed low water dam is proposed on Marine Creek at the confluence with the main stem of the West Fork of the Trinity River to meet project objectives of water connectivity between the Central City Project area and the Stockyards. Several alternatives were evaluated for the Marine Creek dam including both the use of a gated or fixed structure as well as varying the crest width and height. A fixed structure is recommended on Marine Creek as it is able to meet the design requirements of not increasing the existing 100-year water surface elevations on Marine Creek while also reducing both construction and operation and maintenance costs.

The fixed dam on Marine Creek also addresses the hydraulic constraints associated with the Samuels Avenue Dam location downstream of the Marine Creek confluence. The dam structure will have a crest elevation of 516.5 and a crest length of 200 feet. The Marine Creek channel will need to be widened by

approximately 50 feet near the dam location in order to accommodate the 200 feet of crest length needed to pass the 100-year flow without causing increases in water surface elevations upstream.

Although upstream impacts on Marine Creek are reduced by lowering the pool elevation, several existing railroad bridge piers will be inundated by the proposed 516.5 pool elevation, but the impacts are much less than the previously proposed FEIS pool elevation of 525. Individual owners of these structures have been previously notified of the potential inundation created by the project and not indicated a concern at this time. An analysis of existing storm drain systems was conducted to ensure these systems are not impacted by the proposed pool elevation. Hydraulic modeling results indicate that backwater impacts from the low water dam are minimal and the existing and proposed water surface profiles converge in the vicinity of the sidewalk bridge in the Saunders Park area.

An existing low water dam in Saunders Park maintains a pool elevation of approximately 518.5. Since the revised Marine Creek pool elevation of 516.5 is below the existing Saunders Park elevation, no impacts are anticipated further upstream. Widening of Marine Creek and construction of a turnaround basin is proposed just upstream of 23<sup>rd</sup> Street at the limits of the 516.5 pool elevation.

The proposed combination of structures meets the goals and objectives of the Trinity River Vision Master Plan to enhance neighborhood linkages by impounding water upstream on Marine Creek, thus providing water connectivity between the Cultural District, Downtown, and the Rockwood Park area to the Stockyards area.

## **2.06.02 Location Analysis**

The dam was sited upstream from the Samuels Avenue Bridge and the adjacent three railroad bridges primarily in order to allow for the lower, separately maintained water level on Marine Creek. The actual location was specifically set with the front, upstream edge of the structure 450 feet upstream from the centerline of Samuels Avenue Bridge. As shown on Sheet SS-1, this location was selected in order to provide sufficient room for the structure to be constructed with appropriate grading that transitions back to the approximately 250-foot wide channel. The two dams were oriented so that their discharges could be directed and aligned with the downstream channel.

The initial dam location, downstream from the Samuels Ave. bridge did not allow for separate water levels on the main stem and on Marine Creek. The higher level needed on the river created unacceptable flood levels on Marine Creek. By siting the structure upstream of both the bridge and the confluence, a separate lower water level could be maintained on Marine Creek. However, this would require a separate structure on Marine Creek as well as a lock system to allow boat traffic to travel between the two lakes. After the decision was made to place the dam

upstream from the bridges, the actual location was specifically set with the front, upstream edge of the structure 450 feet upstream from the centerline of Samuels Avenue Bridge. As shown in Figure SS-01, this location was selected in order to provide sufficient room for the structure to be constructed with appropriate grading that transitions back to the approximately 250-foot wide channel at the bridge. The two dams were oriented so that their discharges could be directed and aligned with the downstream channel.

Conflicting information has been found on a 45-inch (M-279) sanitary sewer line that runs east west near the north abutment of the dam. City GIS information suggests that it is located immediately adjacent to the abutment however site reconnaissance indicates a manhole further to the north which would place the line outside of the immediate vicinity of the dam work. The mitigation of the large sanitary sewer line will depend on its actual depth and location. Verification of the precise location of the line, both horizontally and vertically is needed in order to develop an appropriate remedy but indications are that it can be accommodated without relocation.

**2.06.03 Design Criteria**

Hydrostatic Loads: Similar to the main dam, a range of hydraulic loads were reviewed in order to determine the critical combination of headwater and tailwater loading conditions. The tailwater levels listed are different from those for the main dam because of the widely varying drainage areas and discharges on the Trinity River and Marine Creek. The cases used frequency flood levels for Marine Creek with tailwater on the main stem based on an assumed limited contribution to flows from the main stem. This provides a conservatively low potential tailwater level. The preliminary design is based on the following water levels.

Case	Classification	Head Water Elev. (ft-MSL)	Tail Water Elev. (ft-MSL)
Normal Pool	Normal	516.5'	499.0'
10-yr	Normal	522.6	510.0
50-yr	Unusual	524.8	513.0
100-yr	Unusual	525.7	515.0
500-yr	Extreme	526.9	517.0
Top of Walls	Extreme	530.0	521.0

Silt Loads: Silt loads are based on silt up to elevation 511.0 ft-MSL. Silt load is computed using a buoyant weight of 40 pcf and a lateral coefficient of pressure of 0.6.

Dynamic Water Loads: Hydraulic loads such as wave action will be considered as appropriate. Unless the actual hydraulic jump profile is modeled, the tailwater contribution to lateral stability will be based on an aerated unit weight of 60% of

normal. If the actual hydraulic jump profile is modeled, the full unit weight of the tailwater will be used.

Uplift Loads: Uplift loads will be computed based on cutoff location, drain location, drain efficiency and linear reduction to tailwater pressure.

Seismic Loads: Similar to the main dam, seismic Loads are expected to be negligible in the project area and are not anticipated to provide any controlling factor in the final design. Once suitable geotechnical information is available, this assumption will be verified.

#### **2.06.04 Design Methodology**

Stability design will be based on USACE criteria in EM 1110-2-2100, Stability Analysis of Concrete Structures and EM 1110-2-2006, Roller-Compacted Concrete.

It should be noted that seepage data for the soils on site were not available for the preliminary design. Underdrains and wall drains are indicated where anticipated by the preliminary design. The type and extent of these drains will be determined in the final design based on the seepage data available at that time.

#### **2.06.05 Design of Foundations**

The preliminary design of the channel dam on Marine Creek is based on a reinforced concrete capped section. It is assumed that the excess soil material to be removed from the main channel dam will provide an excess of material to select good quality granular fill material for the channel dam. It is also assumed that the materials below the dam location are granular without any significant amount of clay. Based on these assumptions, the anticipated settlement is anticipated to be low and to occur during construction.

The bulk of the dam is composed of RCC. A cast-in-place facing is provided at the dam and stilling basin. A thickened RCC mass at the toe of the dam and within the stilling basin was provided to resist uplift forces.

Similar to the Samuels Avenue, the Marine Creek Dam will be constructed on soil well above the bedrock, and underseepage control will be a design issue. For reasons discussed in the previous sections, the same combination of a steel sheetpile cutoff and a horizontal drain will be utilized, as shown in the drawings.

Both abutments require a positive cutoff in the floodplain adjacent to the structure to minimize seepage losses once the normal water level is reached. The right, or south, abutment will have a sheet piling system continuous with that of the main dam that will be part of the connecting training wall. The left, or north, abutment will likely have a similar foundation though competent rock is likely to be

relatively close, as was found at the original downstream dam location. It is assumed that the sheet pile cutoff will be extended to a point in the west abutment at which an open cutoff trench filled with compacted clay will be more economical. This cutoff trench will extend to a point where the rock is high enough to complete the positive cutoff. This location can only be determined once the geotechnical exploration is completed and the depth to rock, the depth to the water table, and the overburden soil properties are determined. Assumed extents of the sheet piles are shown in the drawings.

#### **2.06.06 Design of Superstructure**

The bulk of the dam will be RCC. A cast in place facing will be provided at all faces and in the stilling basin with the uppermost portion of the structure rounded/ curved. The stilling basin will be structurally continuous with the dam superstructure. The sill wall and baffle blocks will be structurally connected to the concrete facing layer.

#### **2.06.07 Design of Ancillary Walls**

The basic configuration of ancillary retaining and flood walls is a cast-in-place concrete inverted T. The design of retaining walls will be based on stability criteria consistent with EM 1110-2-2100, Stability Analysis of Concrete Structures. The strength of these structures will be based on EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures.

The walls will be designed to bear on and be backfilled with on site soils. It may be necessary to use select on site soils as backfill to meet required soil parameters used in the analysis. The final design will confirm that the parameters used are consistent with the data from the geotechnical investigation.

In general, the walls will be designed to have independent foundation systems from the adjacent structures and will be provided with expansion joints at the transition points. The exception to this is abutment walls at the dam and basin sections. These walls receive adequate sliding stability from the adjoining structure. Rotational stability and strength design for abutment walls will be similar to those of the typical retaining walls. Detailing of articulation and transition points will be provided during final design.

### **2.07 Temporary Diversion Construction Sequences**

The construction of the Samuels Ave Dam (SAD) and the Marine Creek Dam (MCD) will be performed in three primary stages to allow for temporary diversion of stormwater flows on the West Fork of the Trinity River and Marine Creek at all times during construction. In general, Phase I will consist of the western half of the SAD and the lower portions of the southern half of the MCD. Phase II would consist of the eastern and northern portions of the respective dams. Phase III would consist of the upper portion of the southern half of the MCD. The sheetpiling used for diversion will be incorporated into the final structure as a

permanent foundation cutoff. The phased construction sequencing of the two dams is described below.

Phase I:

1. Install sheetpiling as shown on Figure SS-02, outlining the southern half of the MCD and the western half of the SAD. Sheetpiling would be extended from the rock foundation, assumed to be at elevation 474, up to at least elevation 520, which would prevent overtopping from approximately a 10-year flood.
2. Flow would continue through the primary channels limited by the sheetpiling.
3. Construct the western 3½ bays of the SAD, appropriate portions of the stilling basin and the western retaining wall, which also forms the southern retaining wall of the MCD.
4. Construct the southern 100 ft of the MCD up to elevation 503.5 and the entire stilling basin.

Phase II:

1. Install sheetpiling as shown on Figure SS-02, outlining the northern two-thirds of the MCD and the eastern half of the SAD. Sheetpiling would be extended from the rock foundation, assumed to be elevation 474, up to at least elevation 520 and will use the common wall that divides the structures.
2. Cut/remove appropriate portions of the sheetpile walls from Phase I.
3. Flow would travel through the low flow piers and the three westernmost gates on the SAD and over the 100-foot section at elevation 503.5 on the MCD.
4. Construct the eastern 3½ bays of the SAD, appropriate portions of the stilling basin, and the eastern retaining wall.
5. Construct the northern 100 ft of the MCD including its portion of the stilling basin.

Phase III:

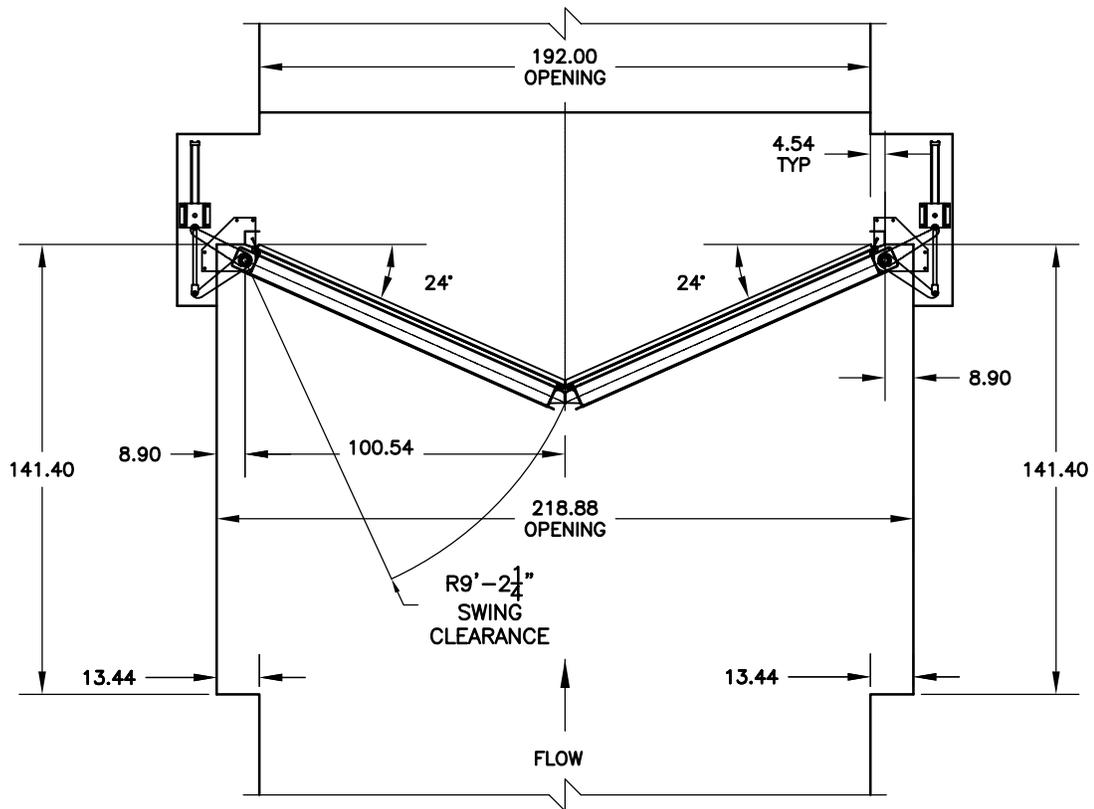
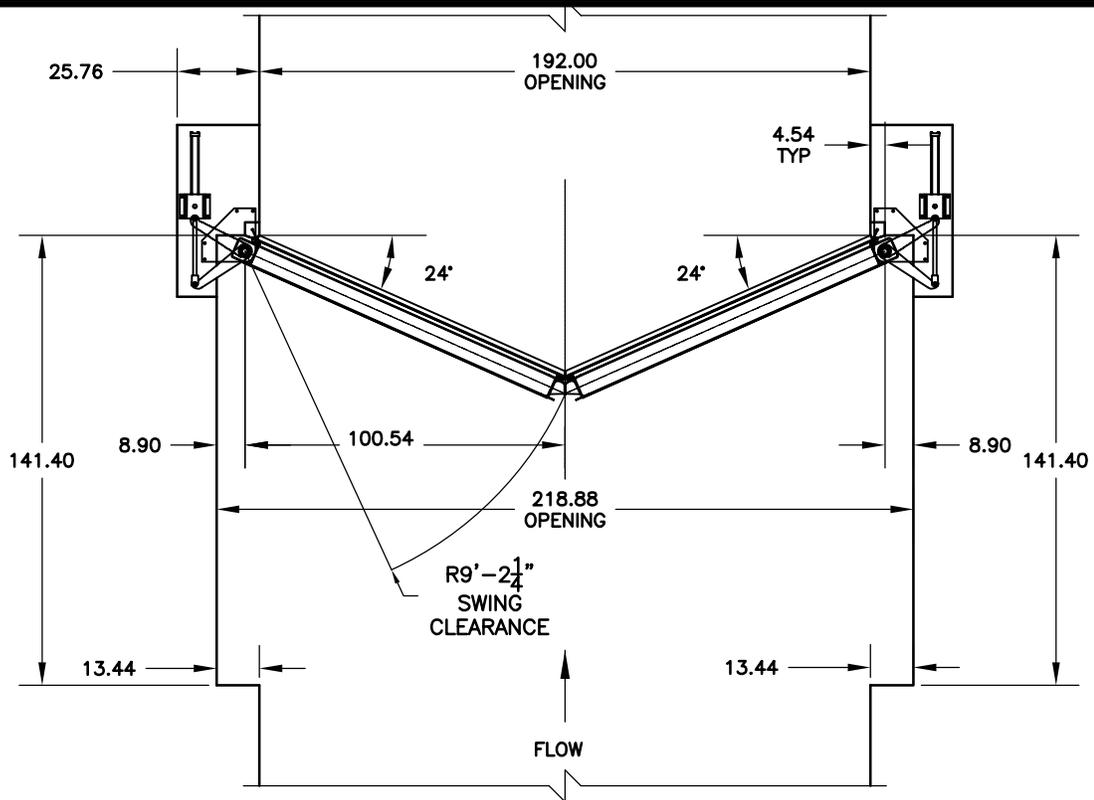
1. Cut/remove appropriate portions of the sheetpile walls from Phase I.
2. Flow would travel through all of the gates on the SAD and through the low flow release structure at the MCD.
3. Construct the remaining portions of the southern 100ft of the MCD above elevation 503.5.

Other:

- The lock system and related retaining walls can be constructed as part of Phase I, II, or both, as its work area can be separated from the river flow in both phases.
- All required excavation upstream from the structures and their related sheetpiling can be performed during any phase, as the excavated surfaces are above the normal water level in the river created by the 4<sup>th</sup> Street dam downstream, which is about elevation 500.

- The majority of the required excavation downstream from the designated sheetpiling can be excavated in the dry during any phase. A portion will require a temporary lowering of the water level at the 4<sup>th</sup> Street dam for the final exaction and placement of riprap.
- Gates on the main dam can be constructed either during the appropriate phase or behind individual stop logs at appropriate times in the sequence.

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PLAN VIEW



US Army Corps  
of Engineers  
Fort Worth District



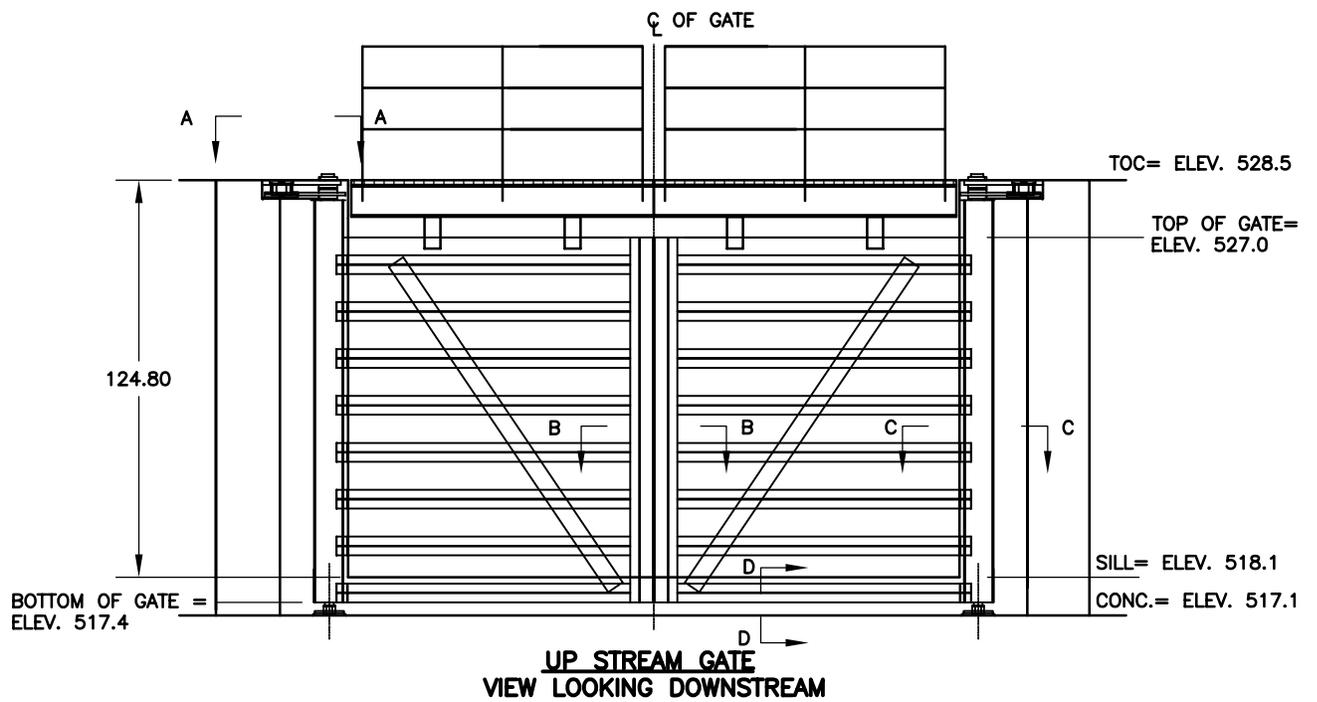
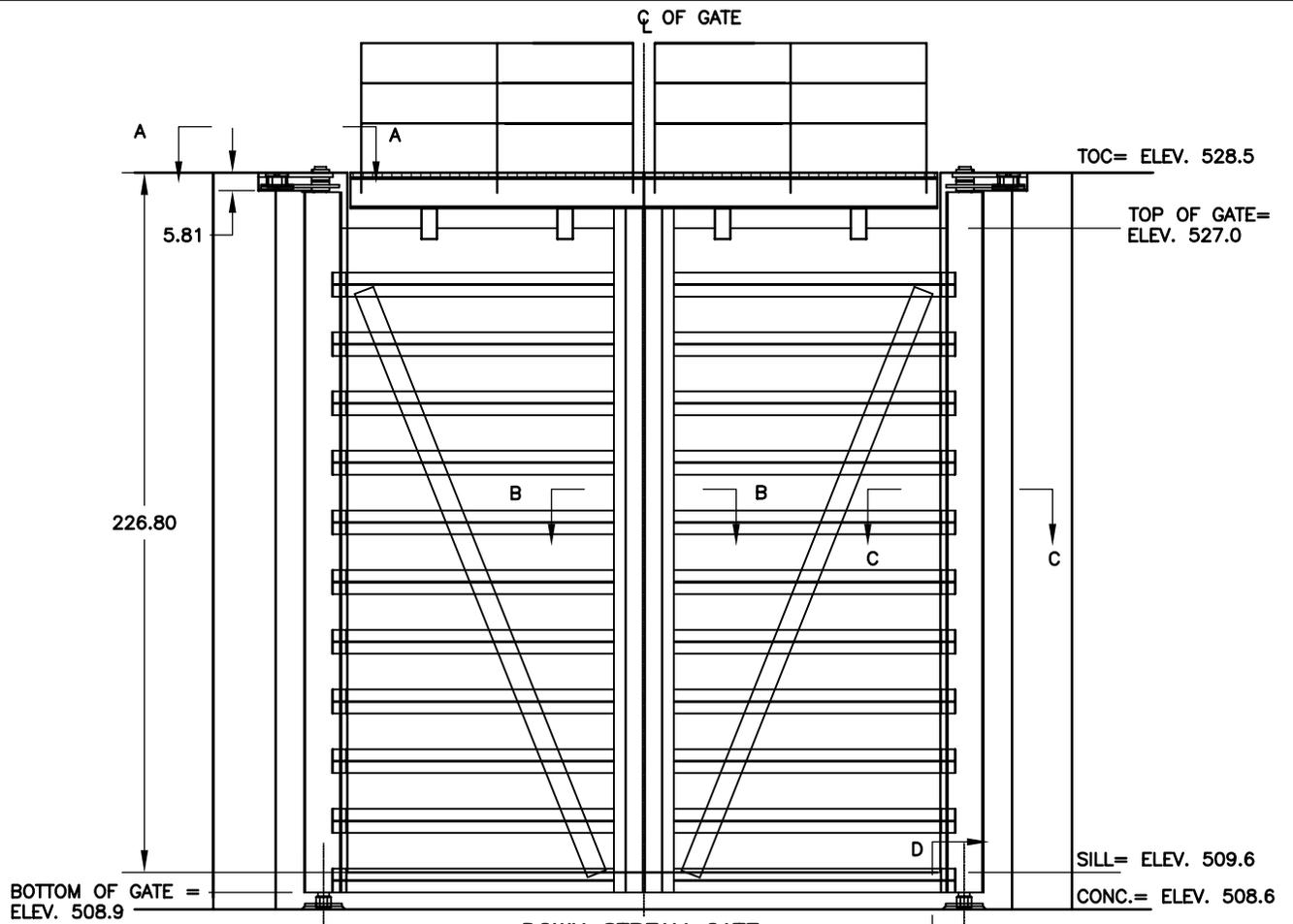
FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

SAMUELS AVENUE LOCK  
MITER GATE PLAN VIEW

DATE NOV 2007

FIGURE No. C-1

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US Army Corps  
of Engineers  
Fort Worth District

**CDM**

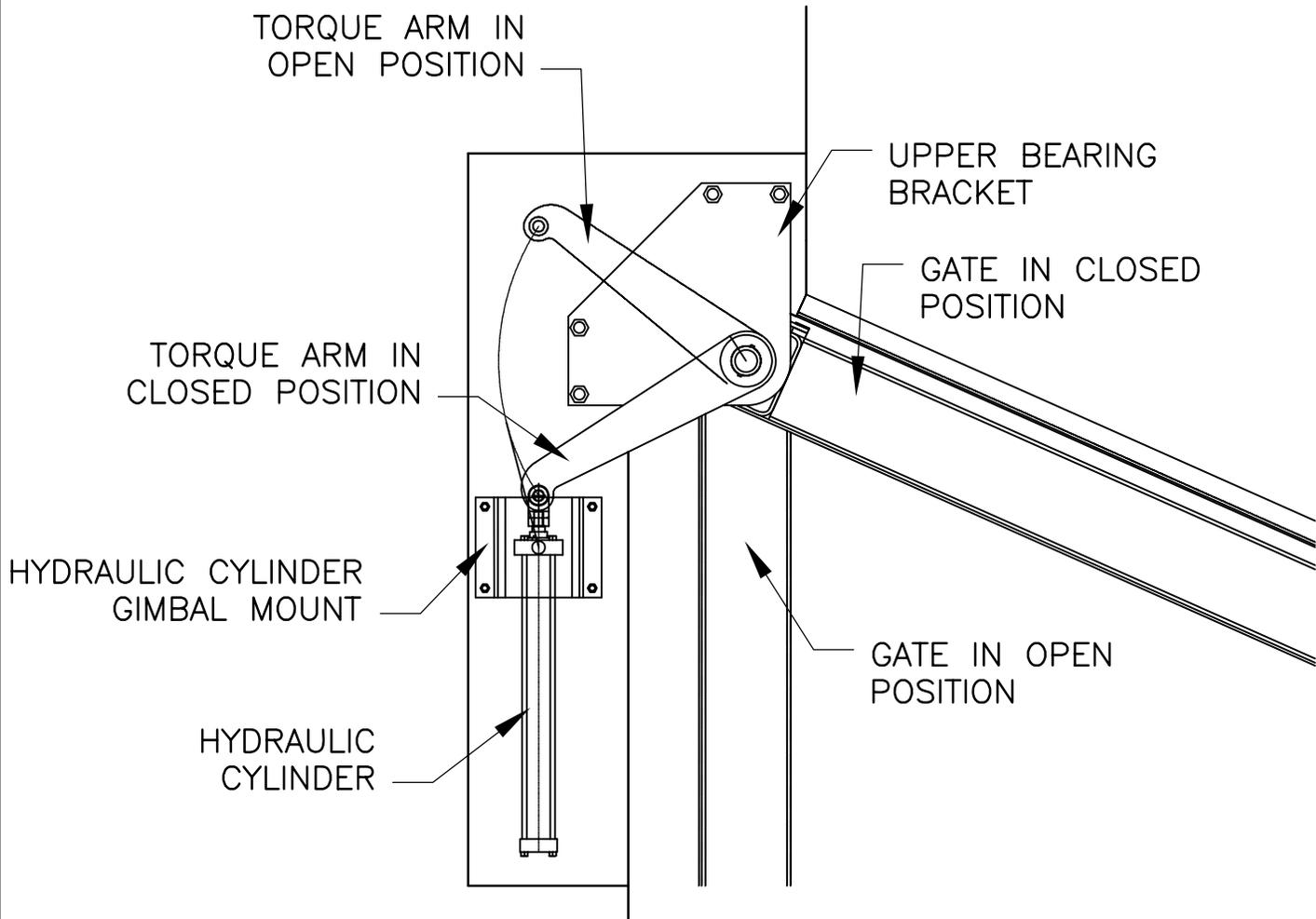
**FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN**

**SAMUELS AVENUE LOCK  
MITER GATE ELEVATIONS**

DATE NOV 2007

FIGURE No. C-2

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SECTION A-A



US Army Corps  
of Engineers  
Fort Worth District

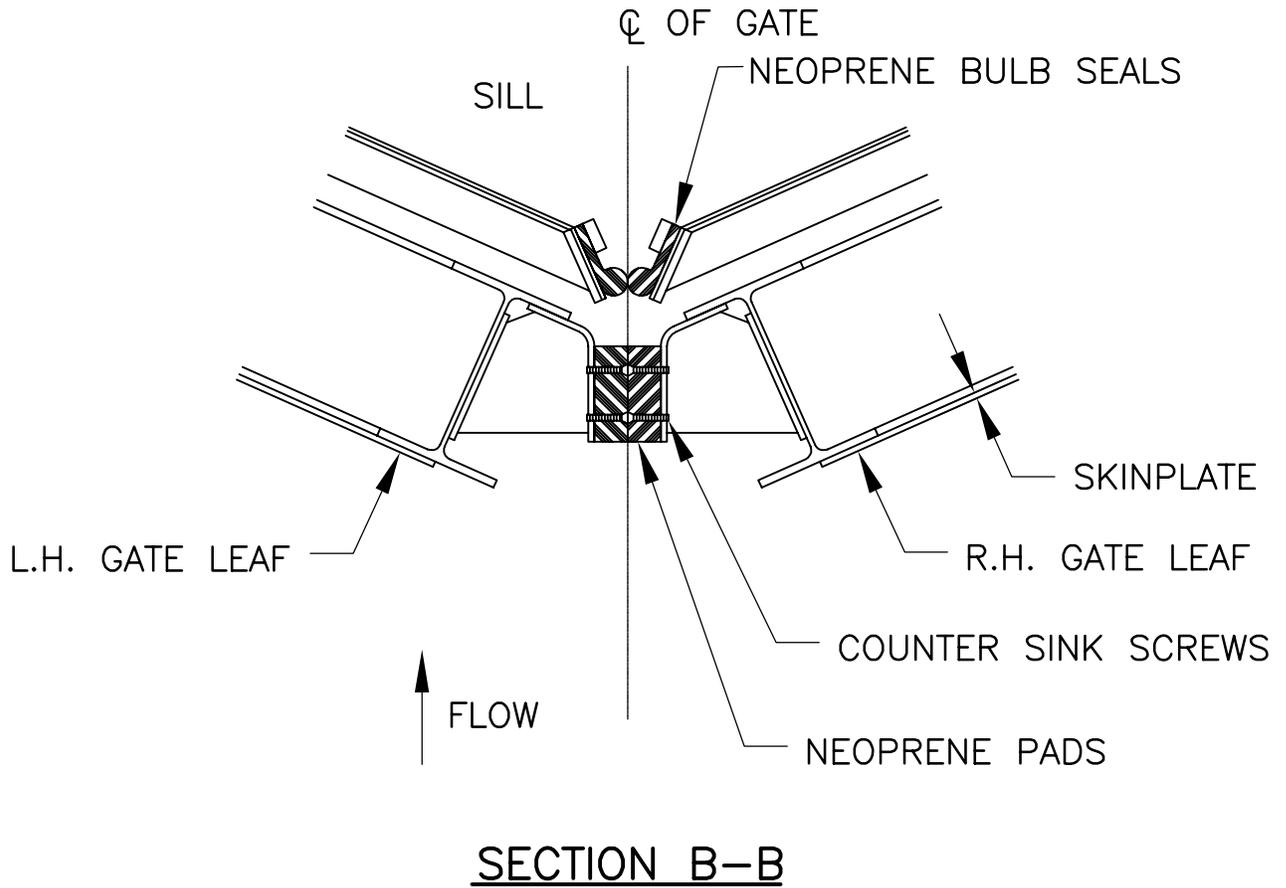


FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

SAMUELS AVENUE LOCK  
MITER GATE  
TORQUE ARM ASSEMBLY

DATE NOV 2007

FIGURE No. C-3



US Army Corps  
of Engineers  
Fort Worth District

**CDM**

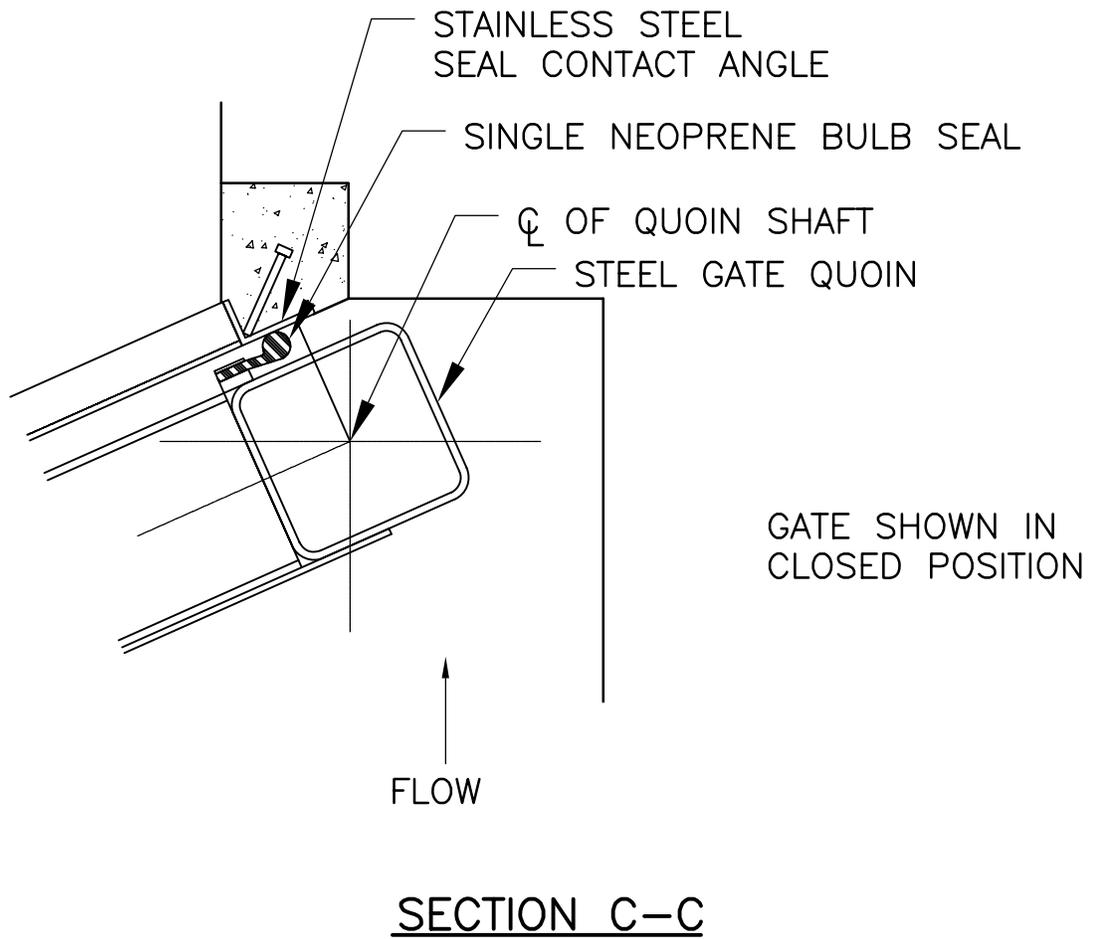
FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

MITER CONTACT  
DETAIL

DATE NOV 2007

FIGURE No. C-4

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Adobe PDF.pc3, Letter, 1:32



US Army Corps  
of Engineers  
Fort Worth District

**CDM**

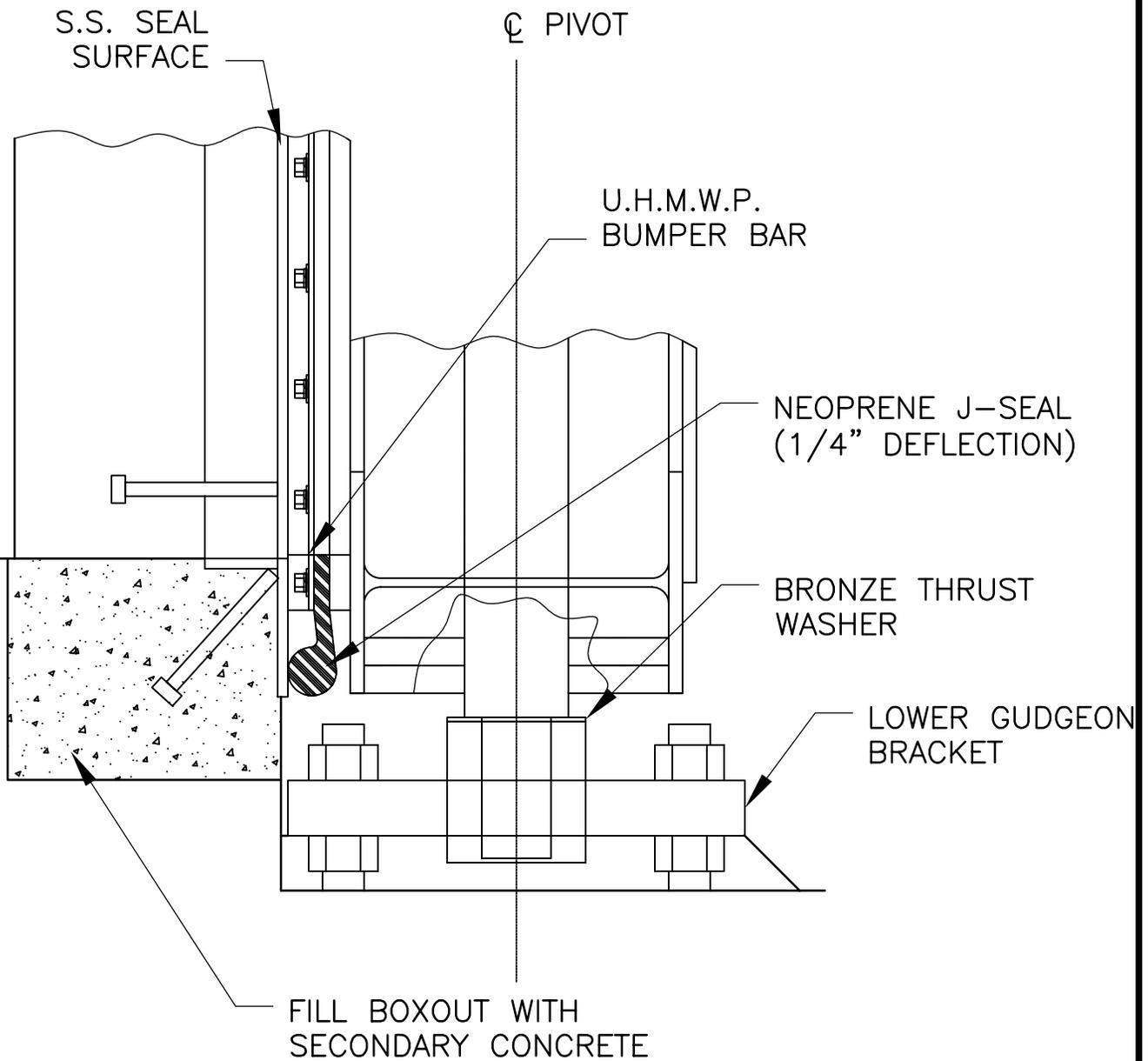
FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

SAMUELS AVENUE LOCK  
GATE QUOIN SEAL ASSEMBLY

DATE NOV 2007

FIGURE No. C-5

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SECTION D-D



US Army Corps  
of Engineers  
Fort Worth District

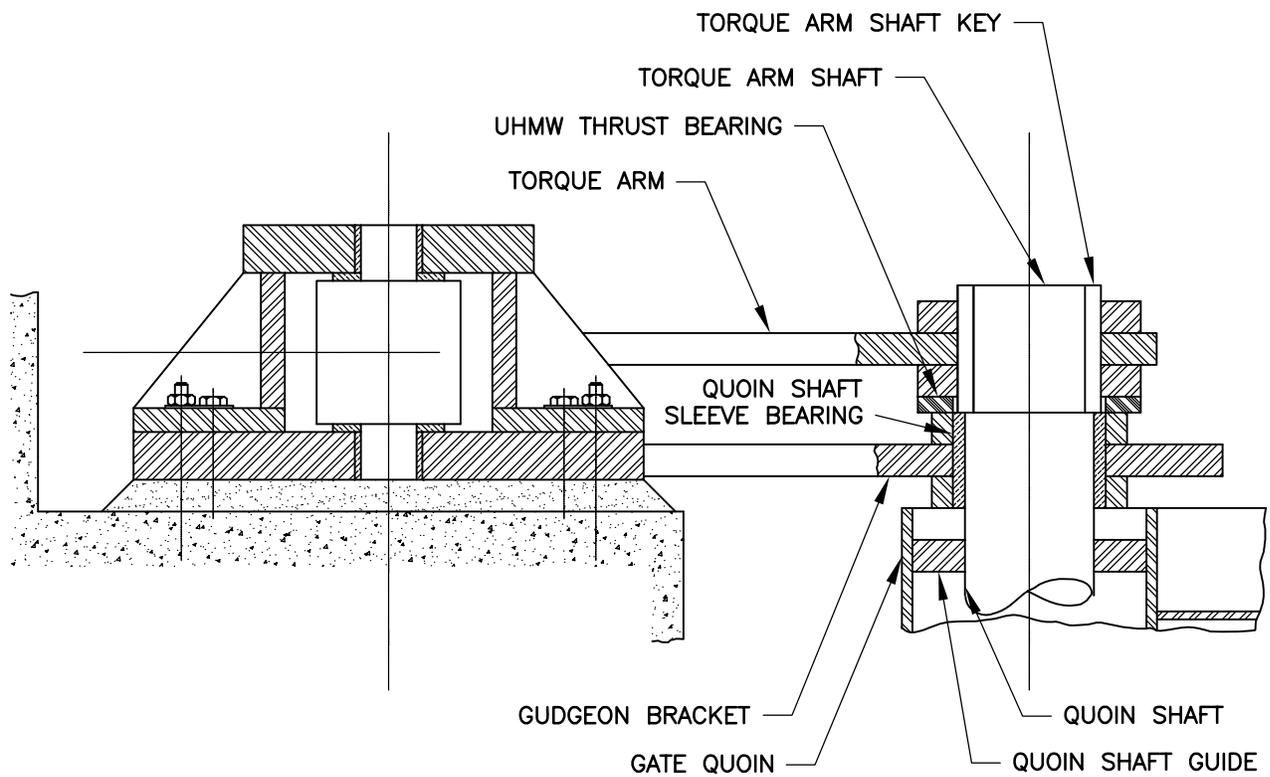


FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

SAMUELS AVENUE LOCK  
BOTTOM PIVOT  
SHAFT BRACKET

DATE NOV 2007

FIGURE No. C-6



US Army Corps  
of Engineers  
Fort Worth District



FORT WORTH CENTRAL CITY  
PRELIMINARY DESIGN

TORQUE ARM ASSEMBLY  
AND QUOIN SHAFT BRACKET

DATE NOV 2007

FIGURE No. C-7