DEVELOPMENT OF WITHOUT PROJECT HYDROLOGY AND HYDRAULICS MODELS

The without project (restoration) conditions hydrologic and hydraulic models used for the Park Reach and the Mission Reach were developed for the San Antonio River Federal Emergency Management Agency (FEMA) Limited Map Maintenance Program (LMMP) Study. The LMMP study consisted of the development of new San Antonio River and San Pedro Creek basin hydrology models using the HEC-1 Flood Hydrograph Package and new San Antonio River and San Pedro Creek hydraulic models using the HEC-RAS River Analysis System. The limits of the new San Antonio River HEC-RAS model extend from downstream of IH 410 to Olmos Dam (approximately 16.81 river miles). The development of the models are the end result of a compilation of several years of modeling efforts by Freese and Nichols, Inc, - Fort Worth, HDR Engineering, Inc. - San Antonio, the San Antonio River Authority (SARA), the City of San Antonio (CSA), and the US Army Corps of Engineers Fort Worth District (USACE).

The modeling process incorporated the best available topographic data, bridge data, and channel data, and incorporated the San Antonio River Tunnel (SART) and San Pedro Creek Tunnel (SPCT) capacity rating curves. SART physical model data, developed by the St., Anthony Falls Laboratory at the University of Minnesota in November 2001, was incorporated into the HEC-1 and HEC-RAS models. The HEC-RAS model consists of eight plans representing the following flood events: 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 250-year, and 500-year.

A brief discussion of the major portions in the development of the San Antonio River LMMP model follows to serve as background information.

Hydraulic Model - Topographic Mapping. A Digital Base Map (DBM) of the San Antonio area, utilizing the best available aerial topographic information from several sources, was developed for use as the base mapping for the entire San Antonio River LMMP Study. All of the topographic files were based on NAD 27 horizontal datum and the 1927 vertical datum. SARA provided detailed information along the San Antonio River channel from Houston Street upstream to SH 281 (McAllister Freeway) in the form of 18 digital sheets, which were spliced into the digitized background. SARA provided recently flown aerial data from Myrtle Street to the San Pedro Creek/San Antonio River confluence. In the 8-5-2 Reach, recent aerial topography from January 1998 was provided for the Brackenridge Park and San Antonio Zoo area from U.S. 281 upstream to Hildebrand Avenue – this information was also spliced into the background. The final DBM consisted of two-foot contour interval mapping.

Hydraulic Model - Development of Cross-Sections. Cross-section geometric information was generated from the Digital Base Map using the program BOSS-RMS for AutoCAD. The BOSS-RMS program allowed for increased accuracy of the cross-section horizontal stations and the development of a layer indicating the actual cross-section lines in digital format. Cross-sections for San Antonio River were generated at approximately 500-foot intervals. Adjustments to the cross-section interval were made at bridges, culverts, low water crossings, and channel dams/weirs that required a more detailed analysis. A minimum of four cross-sections were generated in the vicinity of these structures - two providing the physical configuration of the stream channel directly upstream and downstream of the structure, and one upstream and downstream of the flow expansion/contraction caused by the structure. The cross-sections were extended, where necessary, to include the entire 100-year floodplain top width. The cross-sections were incorporated into the HEC-RAS model oriented looking downstream left-to-right. Cross section locations are referenced by the stream centerline stationing established for the study. The centerline river stationing established for the hydraulic modeling is common to all of the discussions and references to locations of features in this report.

Hydraulic Model - River Channel Flow Line (Bathymetry) Determination. The aerial photographic mapping used in the development of the Digital Base Map did not include the area of the river channel obscured by standing water (bathymetry). Representation of the geometry below the standing water surface was estimated based on available surveyed and spot elevations along the river channel, and previous hydraulic models. Additional bathymetry survey data, from Hugman Dam (upstream of Lexington Street to the San Antonio River Tunnel Inlet, was provided by SARA and used in the analysis. The flow line elevations were interpolated between these available known points.

Hydraulic Model - Manning's Coefficients of Roughness (n Values) Determination. Manning's coefficients of roughness were distributed using horizontal variations across the cross-section stations and were used to define the relative roughness of overbanks, main channel, and transition areas. Aerial photographs and field reconnaissance were used to establish roughness characteristics of the channel and floodplain. The San Antonio River LMMP used the City of San Antonio Unified Development Code (UDC) as a guide to determine the Manning's coefficients of roughness for the model. Table C.1-1 contains the acceptable Manning's n values recognized by the City of San Antonio.

TABLE C.1-1
City of San Antonio Unified Development Code

Channel Description	Manning's n Value
Concrete-lined channel	0.015
Grass-lined channel with regular maintenance	0.035
Grass-lined channel without regular maintenance	0.050
Vegetated channel with trees, little or no underbrush	0.055
Natural channel with trees, moderate underbrush	0.075
Natural channel with trees, dense underbrush	0.090

Overbank Description	Manning's n Value
Pasture	0.050 – 0.055
Trees, little or no underbrush, scattered structures	0.060 – 0.075
Dense vegetation, multiple fences and structures	0.075 – 0.090

The Manning's n values ranged from 0.015 in the downtown channel section of the San Antonio River to 0.100 in overbanks in the southern part of Bexar County. SARA and the City of San Antonio provided estimates of Manning's "n" values in both the San Antonio River and San Pedro Creek. In many river sections, it was determined that modeling the floodplain using three values was insufficient to represent the cross section roughness variations, therefore, in these cases a horizontal variation method was used. This methodology was chosen primarily to provide for the roughness change from the channel section to an overgrown overbank in the direct vicinity of the channel to a golf course beyond the overgrowth. An example of this can be found in the Brackenridge Park/San Pedro Golf Course portion of the model.

Hydraulic Model - San Antonio River Tunnel and San Pedro Creek Tunnel. Capacity rating curve information for the San Antonio Tunnel and the San Pedro Creek Tunnel was incorporated into the HEC-RAS and HEC-HAS models. SART physical model data, developed by the St. Anthony Falls Laboratory at the University of Minnesota in November 2001, was used as part of the development of a new rating curve for the SART. The development of the rating curve used the San Antonio River tailwater rating curve at the SART outlet near Lone Star Boulevard and the San Antonio River inlet rating curve near Josephine Street, with the computed friction losses in the tunnel, and the inlet structure rating curve determined by the physical model. The SART and SPCT final rating curves computed the diversion of flow for a given flood discharge into the SART and SPCT and the resulting discharge downstream to the San Antonio River and San Pedro Creek. The discharge-diversion rating curve was incorporated in the HEC-1 model and the resultant flood discharges incorporated into the HEC-RAS model.

Hydraulic Model - Calibration. Calibration of the HEC-RAS model was accomplished through comparison of model water surface results with surveyed high water marks along the San Antonio River and San Pedro Creek. Collection of high water marks came from four sources: SARA, City of San Antonio, the U.S. Army Corps of Engineers, and the USGS. The October 1998 storm event was used as the calibration flood event to verify model parameters. Elevations at cross-sections upstream and downstream of bridges were given significant weight because the bridges act as hydraulic control points.

Hydraulic Model - Uncertainty in Gage Information and Model Parameters. Four stream gages have been constructed in the San Antonio River Basin, three of them on the San Antonio River and one on Olmos Creek. During the October 1998 storm event, the Olmos Creek, Alamo Street, and Mitchell Street gage were in operation. The South Loop 410 gage was not operating during the 1998 event. The USGS provided the Summary for Indirect Measurement reports for the Mitchell Street Gage and the Loop 410 Gage. The data for the Olmos Gage and Alamo Street Gage were obtained from USGS personnel. Discussions with the USGS personnel indicated that their evaluation of the accuracy of all four gages ranged from fair to poor (10% accuracy to greater than 20%). From the Summary for Indirect Measurement reports, the Mitchell Street evaluation rating is good (10%) while the Loop 410 gage is poor (greater than 20%). The USGS personnel indicated that they were not comfortable with the data at the control cross-section at the Olmos Gage and the Alamo Street Gage to provide an evaluation at these points. Additional sources of error may occur from limiting the Manning's 'n' values to those specified in the City of San Antonio UDC. Due to the large variation in undergrowth and fauna in the overbank areas and channel sections, the actual roughness may vary upwards or downwards from the specified value. Many cross-sections in the lower reaches of the San Antonio River exhibit roughness factors that may lie between the specified UDC acceptable values. Uncertainties involved in flow estimation will also present themselves as systematic uncertainties in the hydraulic model.

Hydraulic Model - Calibration Results. The City of San Antonio desired to use the model parameters based on the UDC criteria in order to maintain parameter consistency with other modeling efforts in the San Antonio area. The UDC criteria are also adopted as the local development criteria for analysis by code. This provided more conservative results for floodplain mapping and the design of hydraulic structures.

Hydrologic Basin Model Development. The base hydrologic model for the San Antonio River watershed incorporates the watershed for the San Antonio River and tributaries to the San Antonio River including: San Pedro Creek, Zarzamora Creek, Alazan Creek, Olmos Creek, Apache Creek, Martinez Creek, Six Mile Creek, and the Catalpa-Pershing Channel. The San Antonio River hydrologic model was originally developed using the HEC-HMS Hydrologic Modeling system and HEC-GeoHMS modeling package. HEC-GeoHMS was used to process terrain information and extract watershed parameters. The source terrain information is a 30-meter Digital Elevation Model (DEM) obtained from the USGS National Elevation Dataset (NED). The DEM represents a seamless combination of six USGS 7.5-minute quadrangle sheets (Castle Hills, Longhorn, San Antonio East, San Antonio West, Terrell Wells, and Southton). The model watershed delineation was checked against available topographic and infrastructure mapping. The City of San Antonio reviewed the existing watershed delineation and provided a modified watershed delineation map that accounted for the City's storm water system, roadway embankments, flow impediments, and other infrastructure.

A second watershed model was developed using HEC-1 in order to use the storm centering and index hydrograph interpolation routines that are available in HEC-1 but not provided in HEC-HMS. The HEC-1 model was then adopted as the hydrologic model of choice for the study. Estimation of loss rates were generated from the Soil Survey geographic spoils coverage for Bexar County, which was used to analyze the types and distribution of the soils in the San Antonio River watershed. The land use and land cover for Bexar County were obtained from 1:250,000 scale land use/land cover GIRAS Spatial Data. The calculated storage volume for the Modified-Puls routing was adjusted for each basin. The Modified-Puls methodology applied to the San Antonio River, San Pedro Creek, and the Catalpa-Pershing channel. The HEC-RAS cross-section layout was overlaid onto the watershed coverage to determine the bounding cross-sections. The HEC-1 model includes three reservoirs: Olmos Dam; Woodlawn Lake, and Elmendorf Lake. Elevation-storage-outflow data for each of the lakes were incorporated into the HEC-1 model. Computed flow diversions for the SART and the SPCT were developed and also incorporated into the HEC-1 model.

The HEC-1 model generated peak flood discharges for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year floods. The 250-year flood peak discharges were interpolated between the 100-year and 500-year flood discharges.

PARK REACH

San Antonio River. The following descriptions include references to locations either by reference to a prominent feature along the San Antonio River or the river centerline stationing that has been established for the hydraulic modeling. The Park Reach extends from U.S. 281 McAllister Freeway (Sta. 2343+50) to Hildebrand Avenue (Sta. 2458+20), approximately 2.17 miles. The river channel flow line elevation varies from approximately 669.4 at Hildebrand Avenue to approximately 646.1 at U.S. 281 – a channel slope of 0.0020 ft/ft or 0.20 %. The river is an earthen channel with banks approximately 10-15 feet high. Manning's n values used in the HEC-RAS model were 0.050 for the river channel and ranged from 0.060 – 0.075 in the overbanks. The major features of the reach floodplain are the San Antonio Zoological Gardens (located on the right bank downstream of Hildebrand Avenue), the Witte Museum (located on the left bank downstream of Hildebrand Avenue), the Brackenridge Park and Golf Course, and the River Road Residential Community. The bridges and channel dams within the Park Reach are shown in Table C.1-2.

Table C.1-2

Structure Type	River Station Location
Low water crossing upstream of inlet	2346+42
Culvert at golf course	2353+54
Grade control structure at golf course	2358+90
Mulberry Avenue Bridge	2386+62
Zoo low water crossing	2409+00
Drop structure at iron bridge	2429+17
Hildebrand Avenue Bridge	2458+20
Low water crossing	2479+25

Catalpa-Pershing Channel. The Catalpa-Pershing channel extends from U.S. 281 McAllister Freeway to upstream of Mulberry Avenue (approximately 5300 feet). The river channel flow line elevation varies from approximately 656.9 at the upstream end approximately 645.0 at U.S. 281 – a channel slope of 0.00226 ft/ft. Manning's n values used in the HEC-RAS model ranged from 0.015 – 0.035 for the river channel and from 0.050 – 0.075 in the overbanks. Approximately 2300 feet of the upstream end of the channel is concrete-lined, the downstream portion is an earthen natural channel. The major feature of the reach that the channel flows through is the Brackenridge Park and Golf Course. The channel depth is approximately 15 feet - the bottom width varies to approximately 20 feet. The channel parallels the highly developed area along Broadway Boulevard to the east, and joins the San Antonio River upstream of U.S. 281. The bridges and channel dams within the Park Reach are as follows:

Structure Type	Cross-section Location
Parking area bridge	1600
Mulberry Avenue Bridge	4180

Water Surface Profiles. Water surface profiles were developed for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 250-year, and 500-year flood events. Table C.1-3 is a listing of the San Antonio River water surface elevations computed for flood events at each cross-section in the Park Reach. Table C.1-4 is a listing of the Catalpa Channel water surface elevations computed for flood events at each cross-section in the Park Reach.

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
2-year	234563	2402	646.00	652.99
5-year	234563	3361	646.00	654.07
10-year	234563	4044	646.00	654.91
25-year	234563	4919	646.00	657.32
50-year	234563	5561	646.00	658.87
100-year	234563	6303	646.00	659.99
250-year	234563	7241	646.00	661.12
500-year	234563	7990	646.00	662.06
	234642	low water crossing u/s inlet		
2-year	234669	2402	646.29	653.69
5-year	234669	3361	646.29	654.47
10-year	234669	4044	646.29	655.04
25-year	234669	4919	646.29	657.33
50-year	234669	5561	646.29	658.87
100-year	234669	6303	646.29	660.00
250-year	234669	7241	646.29	661.13
500-year	234669	7990	646.29	662.07
2-year	235121	2402	647.53	655.91
5-year	235121	3361	647.53	657.03
10-year	235121	4044	647.53	657.66
25-year	235121	4919	647.53	658.70
50-year	235121	5561	647.53	659.75
100-year	235121	6303	647.53	660.63
250-year	235121	7241	647.53	661.68
500-year	235121	7990	647.53	662.49
2-year	235287	2402	648.00	657.26
5-year	235287	3361	648.00	658.80
10-year	235287	4044	648.00	659.76
25-year	235287	4919	648.00	660.98
50-year	235287	5561	648.00	661.73
100-year	235287	6303	648.00	662.48
250-year	235287	7241	648.00	661.45

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
500-year	235287	7990	648.00	662.57
	235354	culvert at golf course		
2-year	235496	2402	648.99	657.60
5-year	235496	3361	648.99	659.17
10-year	235496	4044	648.99	660.19
25-year	235496	4919	648.99	661.41
50-year	235496	5561	648.99	662.20
100-year	235496	6303	648.99	662.46
250-year	235496	7241	648.99	662.27
500-year	235496	7990	648.99	662.49
2-year	235709	2402	650.00	658.07
5-year	235709	3361	650.00	659.44
10-year	235709	4044	650.00	660.38
25-year	235709	4919	650.00	661.54
50-year	235709	5561	650.00	662.30
100-year	235709	6303	650.00	662.55
250-year	235709	7241	650.00	662.41
500-year	235709	7990	650.00	662.63
	235890	grade control structure		
2-year	235910	2402	650.49	659.30
5-year	235910	3361	650.49	660.02
10-year	235910	4044	650.49	660.75
25-year	235910	4919	650.49	661.84
50-year	235910	5561	650.49	662.56
100-year	235910	6303	650.49	662.83
250-year	235910	7241	650.49	662.81
500-year	235910	7990	650.49	663.05
2-year	236058	2402	650.83	659.45
5-year	236058	3361	650.83	660.21
10-year	236058	4044	650.83	660.92
25-year	236058	4919	650.83	661.95

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
50-year	236058	5561	650.83	662.63
100-year	236058	6303	650.83	662.90
250-year	236058	7241	650.83	662.90
500-year	236058	7990	650.83	663.14
2-year	236391	2402	651.62	660.34
5-year	236391	3361	651.62	661.36
10-year	236391	4044	651.62	662.11
25-year	236391	4919	651.62	663.03
50-year	236391	5561	651.62	663.64
100-year	236391	6303	651.62	664.03
250-year	236391	7241	651.62	664.35
500-year	236391	7990	651.62	664.68
2-year	236705	2402	652.35	661.07
5-year	236705	3361	652.35	662.13
10-year	236705	4044	652.35	662.85
25-year	236705	4919	652.35	663.70
50-year	236705	5561	652.35	664.26
100-year	236705	6303	652.35	664.68
250-year	236705	7241	652.35	665.11
500-year	236705	7990	652.35	665.46
2-year	236995	2402	653.04	661.57
5-year	236995	3361	653.04	662.69
10-year	236995	4044	653.04	663.42
25-year	236995	4919	653.04	664.25
50-year	236995	5561	653.04	664.80
100-year	236995	6303	653.04	665.25
250-year	236995	7241	653.04	665.67
500-year	236995	7990	653.04	666.01
2-year	237396	2402	653.98	662.02
5-year	237396	3361	653.98	663.07
10-year	237396	4044	653.98	663.74
25-year	237396	4919	653.98	664.51
50-year	237396	5561	653.98	665.05

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	237396	6303	653.98	665.50
250-year	237396	7241	653.98	665.93
500-year	237396	7990	653.98	666.28
2-year	237696	2402	654.19	662.38
5-year	237696	3361	654.19	663.46
10-year	237696	4044	654.19	664.13
25-year	237696	4919	654.19	664.89
50-year	237696	5561	654.19	665.34
100-year	237696	6303	654.19	665.74
250-year	237696	7241	654.19	666.10
500-year	237696	7990	654.19	666.34
2-year	238008	2402	654.41	662.93
5-year	238008	3361	654.41	664.07
10-year	238008	4044	654.41	664.79
25-year	238008	4919	654.41	665.62
50-year	238008	5561	654.41	666.02
100-year	238008	6303	654.41	666.45
250-year	238008	7241	654.41	666.88
500-year	238008	7990	654.41	667.17
2-year	238298	2402	654.61	663.31
5-year	238298	3361	654.61	664.40
10-year	238298	4044	654.61	665.10
25-year	238298	4919	654.61	665.91
50-year	238298	5561	654.61	666.31
100-year	238298	6303	654.61	666.73
250-year	238298	7241	654.61	667.15
500-year	238298	7990	654.61	667.45
2-year	238530	2402	654.77	663.88
5-year	238530	3361	654.77	665.09
10-year	238530	4044	654.77	665.85
25-year	238530	4919	654.77	666.71
50-year	238530	5561	654.77	667.17
100-year	238530	6303	654.77	667.62

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
250-year	238530	7241	654.77	668.09
500-year	238530	7990	654.77	668.40
2-year	238611	2402	654.83	664.05
5-year	238611	3361	654.83	665.29
10-year	238611	4044	654.83	666.07
25-year	238611	4919	654.83	666.97
50-year	238611	5561	654.83	667.49
100-year	238611	6303	654.83	668.02
250-year	238611	7241	654.83	668.61
500-year	238611	7990	654.83	669.02
	238662		Mulberry Avenue	
2-year	238713	2402	654.90	664.34
5-year	238713	3361	654.90	666.04
10-year	238713	4044	654.90	666.63
25-year	238713	4919	654.90	667.18
50-year	238713	5561	654.90	667.57
100-year	238713	6303	654.90	668.74
250-year	238713	7241	654.90	669.20
500-year	238713	7990	654.90	669.54
2-year	238807	2402	654.97	664.64
5-year	238807	3361	654.97	666.33
10-year	238807	4044	654.97	666.92
25-year	238807	4919	654.97	667.47
50-year	238807	5561	654.97	667.85
100-year	238807	6303	654.97	668.85
250-year	238807	7241	654.97	669.32
500-year	238807	7990	654.97	669.66
2-year	239106	2402	655.18	664.89
5-year	239106	3361	655.18	666.49
10-year	239106	4044	655.18	667.10
25-year	239106	4919	655.18	667.66
50-year	239106	5561	655.18	668.05

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	239106	6303	655.18	669.01
250-year	239106	7241	655.18	669.46
500-year	239106	7990	655.18	669.79
2-year	239502	2402	655.45	665.23
5-year	239502	3361	655.45	666.76
10-year	239502	4044	655.45	667.38
25-year	239502	4919	655.45	667.99
50-year	239502	5561	655.45	668.40
100-year	239502	6303	655.45	669.27
250-year	239502	7241	655.45	669.72
500-year	239502	7990	655.45	670.05
2-year	239890	2402	655.72	665.65
5-year	239890	3361	655.72	667.08
10-year	239890	4044	655.72	667.70
25-year	239890	4919	655.72	668.32
50-year	239890	5561	655.72	668.77
100-year	239890	6303	655.72	669.57
250-year	239890	7241	655.72	670.02
500-year	239890	7990	655.72	670.35
2-year	240300	2402	656.01	666.18
5-year	240300	3361	656.01	667.58
10-year	240300	4044	656.01	668.22
25-year	240300	4919	656.01	668.74
50-year	240300	5561	656.01	669.15
100-year	240300	6303	656.01	669.87
250-year	240300	7241	656.01	670.31
500-year	240300	7990	656.01	670.62
2-year	240696	2402	658.99	666.94
5-year	240696	3361	658.99	668.25
10-year	240696	4044	658.99	668.91
25-year	240696	4919	658.99	669.53
50-year	240696	5561	658.99	669.99
100-year	240696	6303	658.99	670.68

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
250-year	240696	7241	658.99	671.21
500-year	240696	7990	658.99	671.59
2-year	240840	2402	661.49	667.15
5-year	240840	3361	661.49	668.41
10-year	240840	4044	661.49	669.06
25-year	240840	4919	661.49	669.68
50-year	240840	5561	661.49	670.13
100-year	240840	6303	661.49	670.82
250-year	240840	7241	661.49	671.36
500-year	240840	7990	661.49	671.75
	240900	zoo low water crossing		
2-year	241096	2402	662.99	669.33
5-year	241096	3361	662.99	669.83
10-year	241096	4044	662.99	670.14
25-year	241096	4919	662.99	670.48
50-year	241096	5561	662.99	670.72
100-year	241096	6303	662.99	671.05
250-year	241096	7241	662.99	671.47
500-year	241096	7990	662.99	671.82
2-year	241498	2372	663.79	669.85
5-year	241498	3215	663.79	670.53
10-year	241498	3829	663.79	670.95
25-year	241498	4669	663.79	671.41
50-year	241498	5269	663.79	671.72
100-year	241498	5894	663.79	672.07
250-year	241498	6716	663.79	672.48
500-year	241498	7368	663.79	672.80
2-year	241899	2372	664.59	671.04
5-year	241899	3215	664.59	671.88
10-year	241899	3829	664.59	672.37
25-year	241899	4669	664.59	672.92
50-year	241899	5269	664.59	673.27

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	241899	5894	664.59	673.65
250-year	241899	6716	664.59	673.99
500-year	241899	7368	664.59	674.22
2-year	242194	2372	665.18	671.73
5-year	242194	3215	665.18	672.64
10-year	242194	3829	665.18	673.19
25-year	242194	4669	665.18	673.84
50-year	242194	5269	665.18	674.26
100-year	242194	5894	665.18	674.69
250-year	242194	6716	665.18	675.09
500-year	242194	7368	665.18	675.41
2-year	242592	2372	665.98	672.99
5-year	242592	3215	665.98	673.85
10-year	242592	3829	665.98	674.33
25-year	242592	4669	665.98	674.92
50-year	242592	5269	665.98	675.29
100-year	242592	5894	665.98	675.67
250-year	242592	6716	665.98	675.77
500-year	242592	7368	665.98	676.04
	242917	drop structure at iron bridge		
2-year	243002	2372	666.30	673.26
5-year	243002	3215	666.30	674.04
10-year	243002	3829	666.30	674.48
25-year	243002	4669	666.30	675.01
50-year	243002	5269	666.30	675.36
100-year	243002	5894	666.30	675.72
250-year	243002	6716	666.30	675.84
500-year	243002	7368	666.30	676.09
2-year	243398	2372	666.61	673.91
5-year	243398	3215	666.61	674.79
10-year	243398	3829	666.61	675.27
25-year	243398	4669	666.61	675.81

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
50-year	243398	5269	666.61	676.16
100-year	243398	5894	666.61	676.47
250-year	243398	6716	666.61	676.71
500-year	243398	7368	666.61	676.95
2-year	243796	2372	666.92	674.70
5-year	243796	3215	666.92	675.53
10-year	243796	3829	666.92	675.99
25-year	243796	4669	666.92	676.52
50-year	243796	5269	666.92	676.85
100-year	243796	5894	666.92	677.12
250-year	243796	6716	666.92	677.40
500-year	243796	7368	666.92	677.64
2-year	244192	2372	667.23	675.21
5-year	244192	3215	667.23	676.05
10-year	244192	3829	667.23	676.52
25-year	244192	4669	667.23	677.01
50-year	244192	5269	667.23	677.30
100-year	244192	5894	667.23	677.44
250-year	244192	6716	667.23	677.72
500-year	244192	7368	667.23	677.94
2-year	244591	2372	667.54	675.59
5-year	244591	3215	667.54	676.46
10-year	244591	3829	667.54	676.92
25-year	244591	4669	667.54	677.41
50-year	244591	5269	667.54	677.65
100-year	244591	5894	667.54	677.82
250-year	244591	6716	667.54	678.13
500-year	244591	7368	667.54	678.34
2-year	245191	2372	668.46	675.90
5-year	245191	3215	668.46	676.73
10-year	245191	3829	668.46	677.18
25-year	245191	4669	668.46	677.68
50-year	245191	5269	668.46	677.93

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	245191	5894	668.46	678.14
250-year	245191	6716	668.46	678.51
500-year	245191	7368	668.46	678.72
2-year	245494	2372	668.92	676.01
5-year	245494	3215	668.92	676.84
10-year	245494	3829	668.92	677.28
25-year	245494	4669	668.92	677.78
50-year	245494	5269	668.92	678.04
100-year	245494	5894	668.92	678.27
250-year	245494	6716	668.92	678.65
500-year	245494	7368	668.92	678.86
2-year	245774	2372	669.35	676.16
5-year	245774	3215	669.35	676.96
10-year	245774	3829	669.35	677.40
25-year	245774	4669	669.35	677.89
50-year	245774	5269	669.35	678.16
100-year	245774	5894	669.35	678.39
250-year	245774	6716	669.35	678.75
500-year	245774	7368	669.35	678.96
	245820	Hildebrand Avenue		
2-year	245872	2372	669.50	676.56
5-year	245872	3215	669.50	677.43
10-year	245872	3829	669.50	677.95
25-year	245872	4669	669.50	678.57
50-year	245872	5269	669.50	678.97
100-year	245872	5894	669.50	679.35
250-year	245872	6716	669.50	679.85
500-year	245872	7368	669.50	680.36
2-year	246198	2372	670.00	677.63
5-year	246198	3215	670.00	678.66
10-year	246198	3829	670.00	679.31
25-year	246198	4669	670.00	680.06

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
50-year	246198	5269	670.00	680.47
100-year	246198	5894	670.00	680.88
250-year	246198	6716	670.00	681.40
500-year	246198	7368	670.00	681.98
2-year	246698	2372	672.00	678.67
5-year	246698	3215	672.00	679.88
10-year	246698	3829	672.00	680.58
25-year	246698	4669	672.00	681.32
50-year	246698	5269	672.00	681.72
100-year	246698	5894	672.00	682.11
250-year	246698	6716	672.00	682.60
500-year	246698	7368	672.00	683.04
2-year	246998	2372	672.37	679.30
5-year	246998	3215	672.37	680.51
10-year	246998	3829	672.37	681.16
25-year	246998	4669	672.37	681.84
50-year	246998	5269	672.37	682.22
100-year	246998	5894	672.37	682.60
250-year	246998	6716	672.37	683.03
500-year	246998	7368	672.37	683.43
2-year	247398	2372	672.87	680.06
5-year	247398	3215	672.87	681.15
10-year	247398	3829	672.87	681.77
25-year	247398	4669	672.87	682.44
50-year	247398	5269	672.87	682.82
100-year	247398	5894	672.87	683.12
250-year	247398	6716	672.87	683.44
500-year	247398	7368	672.87	683.80
2-year	247898	716	673.50	680.75
5-year	247898	767	673.50	681.69
10-year	247898	800	673.50	682.27
25-year	247898	861	673.50	682.91
50-year	247898	861	673.50	683.27

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	247898	893	673.50	683.46
250-year	247898	1737	673.50	683.83
500-year	247898	2784	673.50	684.22
	247925	crossing		
2-year	248298	716	674.00	680.76
5-year	248298	767	674.00	681.70
10-year	248298	800	674.00	682.27
25-year	248298	861	674.00	682.92
50-year	248298	861	674.00	683.27
100-year	248298	893	674.00	683.46
250-year	248298	1737	674.00	683.84
500-year	248298	2784	674.00	684.23
2-year	248698	716	674.29	680.97
5-year	248698	767	674.29	681.84
10-year	248698	800	674.29	682.40
25-year	248698	861	674.29	683.03
50-year	248698	861	674.29	683.37
100-year	248698	893	674.29	683.56
250-year	248698	1737	674.29	684.14
500-year	248698	2784	674.29	684.79
2-year	248998	716	674.51	681.20
5-year	248998	767	674.51	682.00
10-year	248998	800	674.51	682.55
25-year	248998	861	674.51	683.16
50-year	248998	861	674.51	683.47
100-year	248998	893	674.51	683.66
250-year	248998	1737	674.51	684.43
500-year	248998	2784	674.51	685.27
2-year	249398	716	674.80	681.54
5-year	249398	767	674.80	682.25
10-year	249398	800	674.80	682.76
25-year	249398	861	674.80	683.34

TABLE C.1-3

SAN ANTONIO RIVER
PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
50-year	249398	861	674.80	683.63
100-year	249398	893	674.80	683.81
250-year	249398	1737	674.80	684.80
500-year	249398	2784	674.80	685.90
2-year	249679	716	675.00	681.76
5-year	249679	767	675.00	682.42
10-year	249679	800	675.00	682.90
25-year	249679	861	675.00	683.46
50-year	249679	861	675.00	683.73
100-year	249679	893	675.00	683.92
250-year	249679	1737	675.00	685.05
500-year	249679	2784	675.00	686.30

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
2-year	239	1248	645	653.04
5-year	239	1820	645	654.14
10-year	239	2242	645	654.99
25-year	239	2793	645	657.37
50-year	239	3106	645	658.9
100-year	239	3359	645	660.01
250-year	239	3828	645	661.14
500-year	239	4199	645	662.07
2-year	391	1248	645.3	653.19
5-year	391	1820	645.3	654.29
10-year	391	2242	645.3	655.13
25-year	391	2793	645.3	657.44
50-year	391	3106	645.3	658.95
100-year	391	3359	645.3	660.05
250-year	391	3828	645.3	661.18
500-year	391	4199	645.3	662.11
2-year	573	1248	645.67	653.26
5-year	573	1820	645.67	654.35
10-year	573	2242	645.67	655.18
25-year	573	2793	645.67	657.47
50-year	573	3106	645.67	658.97
100-year	573	3359	645.67	660.08
250-year	573	3828	645.67	661.2
500-year	573	4199	645.67	662.13
2-year	735	1248	645.99	653.36
5-year	735	1820	645.99	654.42
10-year	735	2242	645.99	655.23
25-year	735	2793	645.99	657.49
50-year	735	3106	645.99	658.98

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
100-year	735	3359	645.99	660.09
250-year	735	3828	645.99	661.21
500-year	735	4199	645.99	662.13
2-year	952	1248	646.55	653.45
5-year	952	1820	646.55	654.5
10-year	952	2242	646.55	655.28
25-year	952	2793	646.55	657.5
50-year	952	3106	646.55	658.99
100-year	952	3359	646.55	660.11
250-year	952	3828	646.55	661.21
500-year	952	4199	646.55	662.14
2-year	1230	1248	647.26	653.67
5-year	1230	1820	647.26	654.63
10-year	1230	2242	647.26	655.35
25-year	1230	2793	647.26	657.49
50-year	1230	3106	647.26	658.96
100-year	1230	3359	647.26	660.12
250-year	1230	3828	647.26	661.22
500-year	1230	4199	647.26	662.14
2-year	1516	902	648	653.82
5-year	1516	1292	648	654.81
10-year	1516	1573	648	655.53
25-year	1516	1949	648	657.58
50-year	1516	2247	648	659.02
100-year	1516	2556	648	660.11
250-year	1516	2953	648	661.19
500-year	1516	3272	648	662.11
2-year	1567	902	648.18	653.77
5-year	1567	1292	648.18	654.73
10-year	1567	1573	648.18	655.44

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
25-year	1567	1949	648.18	657.51
50-year	1567	2247	648.18	658.96
100-year	1567	2556	648.18	660.05
250-year	1567	2953	648.18	661.13
500-year	1567	3272	648.18	662.05
	1600		parking area bridge	
2-year	1619	902	648.37	654.63
5-year	1619	1292	648.37	656.46
10-year	1619	1573	648.37	658.2
25-year	1619	1949	648.37	661.96
50-year	1619	2247	648.37	663.24
100-year	1619	2556	648.37	664.05
250-year	1619	2953	648.37	664.8
500-year	1619	3272	648.37	665.32
2-year	1658	902	648.51	654.59
5-year	1658	1292	648.51	656.42
10-year	1658	1573	648.51	658.17
25-year	1658	1949	648.51	661.94
50-year	1658	2247	648.51	663.23
100-year	1658	2556	648.51	664.04
250-year	1658	2953	648.51	664.79
500-year	1658	3272	648.51	665.31
2-year	1875	902	649.3	654.63
5-year	1875	1292	649.3	656.46
10-year	1875	1573	649.3	658.19
25-year	1875	1949	649.3	662.01
50-year	1875	2247	649.3	663.31
100-year	1875	2556	649.3	664.14
250-year	1875	2953	649.3	664.92
500-year	1875	3272	649.3	665.45

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
2-year	2069	902	650	654.71
5-year	2069	1292	650	656.52
10-year	2069	1573	650	658.26
25-year	2069	1949	650	662.01
50-year	2069	2247	650	663.31
100-year	2069	2556	650	664.13
250-year	2069	2953	650	664.92
500-year	2069	3272	650	665.46
2-year	2226	902	650.17	654.62
5-year	2226	1292	650.17	656.49
10-year	2226	1573	650.17	658.24
25-year	2226	1949	650.17	662.01
50-year	2226	2247	650.17	663.31
100-year	2226	2556	650.17	664.13
250-year	2226	2953	650.17	664.91
500-year	2226	3272	650.17	665.45
2-year	2378	902	650.34	654.96
5-year	2378	1292	650.34	656.65
10-year	2378	1573	650.34	658.32
25-year	2378	1949	650.34	662.04
50-year	2378	2247	650.34	663.33
100-year	2378	2556	650.34	664.16
250-year	2378	2953	650.34	664.94
500-year	2378	3272	650.34	665.48
2-year	2684	902	650.68	655
5-year	2684	1292	650.68	656.61
10-year	2684	1573	650.68	658.25
25-year	2684	1949	650.68	661.97
50-year	2684	2247	650.68	663.26
100-year	2684	2556	650.68	664.1

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
250-year	2684	2953	650.68	664.89
500-year	2684	3272	650.68	665.44
2-year	2987	902	651.01	655.66
5-year	2987	1292	651.01	657.03
10-year	2987	1573	651.01	658.5
25-year	2987	1949	651.01	662.03
50-year	2987	2247	651.01	663.31
100-year	2987	2556	651.01	664.14
250-year	2987	2953	651.01	664.92
500-year	2987	3272	651.01	665.46
2-year	3308	902	651.37	655.86
5-year	3308	1292	651.37	657.17
10-year	3308	1573	651.37	658.59
25-year	3308	1949	651.37	662.06
50-year	3308	2247	651.37	663.34
100-year	3308	2556	651.37	664.16
250-year	3308	2953	651.37	664.92
500-year	3308	3272	651.37	665.46
2-year	3647	902	651.75	656.1
5-year	3647	1292	651.75	657.32
10-year	3647	1573	651.75	658.65
25-year	3647	1949	651.75	662.06
50-year	3647	2247	651.75	663.32
100-year	3647	2556	651.75	664.14
250-year	3647	2953	651.75	664.87
500-year	3647	3272	651.75	665.39
2-year	3871	902	652	656.11
5-year	3871	1292	652	657.28
10-year	3871	1573	652	658.6
25-year	3871	1949	652	662.02

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
50-year	3871	2247	652	663.29
100-year	3871	2556	652	664.1
250-year	3871	2953	652	664.82
500-year	3871	3272	652	665.33
2-year	4025	902	652.52	656.34
5-year	4025	1292	652.52	657.17
10-year	4025	1573	652.52	658.49
25-year	4025	1949	652.52	661.99
50-year	4025	2247	652.52	663.26
100-year	4025	2556	652.52	664.06
250-year	4025	2953	652.52	664.79
500-year	4025	3272	652.52	665.3
2-year	4123	902	652.86	657.19
5-year	4123	1292	652.86	658.03
10-year	4123	1573	652.86	658.57
25-year	4123	1949	652.86	661.97
50-year	4123	2247	652.86	663.22
100-year	4123	2556	652.86	664.01
250-year	4123	2953	652.86	664.7
500-year	4123	3272	652.86	665.18
	4180		Mulberry Avenue	
2-year	4207	902	653.14	659.62
5-year	4207	1292	653.14	662.22
10-year	4207	1573	653.14	663.21
25-year	4207	1949	653.14	669.24
50-year	4207	2247	653.14	670.15
100-year	4207	2556	653.14	670.69
250-year	4207	2953	653.14	671.17
500-year	4207	3272	653.14	671.47

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
2-year	4286	902	653.41	659.12
5-year	4286	1292	653.41	662.06
10-year	4286	1573	653.41	663.1
25-year	4286	1949	653.41	669.32
50-year	4286	2247	653.41	670.25
100-year	4286	2556	653.41	670.81
250-year	4286	2953	653.41	671.33
500-year	4286	3272	653.41	671.66
2-year	4517	606	654.2	660.32
5-year	4517	870	654.2	662.57
10-year	4517	1059	654.2	663.59
25-year	4517	1311	654.2	669.44
50-year	4517	1510	654.2	670.23
100-year	4517	1714	654.2	670.85
250-year	4517	1976	654.2	671.38
500-year	4517	2186	654.2	671.71
2-year	4762	606	655.03	660.38
5-year	4762	870	655.03	662.62
10-year	4762	1059	655.03	663.64
25-year	4762	1311	655.03	669.45
50-year	4762	1510	655.03	670.23
100-year	4762	1714	655.03	670.79
250-year	4762	1976	655.03	671.31
500-year	4762	2186	655.03	671.63
2-year	5044	606	656	660.44
5-year	5044	870	656	662.64
10-year	5044	1059	656	663.66
25-year	5044	1311	656	669.45
50-year	5044	1510	656	670.24
100-year	5044	1714	656	670.8
250-year	5044	1976	656	671.31

TABLE C.1-4

CATALPA-PERSHING CHANNEL

PARK REACH
EXISTING CONDITIONS WATER SURFACE PROFILES

Flood	River Station	Discharge (cfs)	Min Ch El (ft)	WSEL (ft)
500-year	5044	2186	656	671.63
2-year	5300	606	656.87	660.57
5-year	5300	870	656.87	662.7
10-year	5300	1059	656.87	663.71
25-year	5300	1311	656.87	669.46
50-year	5300	1510	656.87	670.24
100-year	5300	1714	656.87	670.8
250-year	5300	1976	656.87	671.32
500-year	5300	2186	656.87	671.64

FLOOD DAMAGE REDUCTION PROCESS

Several measures were evaluated to reduce the frequency and depth of flooding. Each is briefly described below.

Bridge Replacement at Mulberry Avenue and Millrace Road. Based on the "without-project" assessment, the Mulberry Avenue and Millrace Road Bridge are constrictions to flood flow. The Mulberry Avenue Bridge has 4 feet of head loss and the Millrace Road Bridge 6.7 feet. It is from these channel constrictions that most of the flood damages along Catalpa occur. In order to incrementally justify the NED plan, this bridge replacement only alternative was modeled first, in an attempt to see what benefits accrue from bridge replacement only. The Mulberry Avenue Bridge was opened up from the existing 2 – 10 x 5.5 box culverts, which conveyed 1856 cfs of the total 2556 cfs of the 100-year flood event, to a 25-foot bottom width, 2:1 side slopes channel which conveys the total 2556 cfs. The Millrace Road Bridge was opened from the existing 3 – 8 x 6 box culverts, which conveyed 1819 cfs of the total 2556 cfs, to a 25-foot bottom width grass lined channel with 2:1 side slopes, which conveys the total flood flow for the 100-year event. This bridge only alternative resulted in a 99.5% reduction in flood damages along Catalpa channel and when considered with cost, economically feasible. From this, we are able to include this increment in all others in formulating the NED plan. The bridge modifications will have no effect or impact to the San Antonio River or the flooding that is occurring in the Museum, zoo, park or River Road community. In an attempt to reduce flooding in the main damage centers, a diversion channel and hydraulic channel modifications were considered in conjunction with this plan.

Smaller Diversion Channel – at Tuleta. This plan includes hydraulic improvements including a new diversion structure and Catalpa channel modifications in an attempt to reduce flood damages. San Antonio River modifications include bank stabilization only. This weir diversion structure was modeled just upstream of Tuleta Drive, which conveys floodwater from the San Antonio River down a new diversion channel through Brackenridge Park and empting into the existing Catalpa channel headwall. A 50-foot long, sharp crested weir, elevation 666.9 feet, was designed using the standard weir equation, while assuming a free outfall condition with no submergence. The flow distribution from the San Antonio River to the new diversion structure is detailed in the Table C.1-5.

Table C.1-5

	Total Flow cfs	San Antonio River cfs	Catalpa Diversion cfs
2 Year	2372	2052	320
5 Year	3215	2615	600
10 Year	3829	3049	780
25 Year	4669	3649	1020
50 Year	5269	4069	1200
100 Year	5894	4524	1370
250 year	6716	5116	1600
500 Year	7368	5593	1775

Small Diversion Channel - Flow Distribution at Weir. A 75-foot wide opening through Tuleta Drive requires a new bridge structure which transitions into a 30-foot wide grass-lined channel with 3:1 side slopes. In addition to the new bridge at Tuleta Drive, three others will be required at Parfun Way and two railroad crossings for the Eagle railroad. The existing Catalpa channel will be modified to include the new bridges at Mulberry Avenue (station 41+80) and Milrace Road (station 16+00), with low chords set at 665 and 660 feet, respectively. The concrete channel at the Mulberry Avenue Bridge will be modified to a 20-foot bottom width with 1.5:1 side slopes. The concrete channel through the Millrace Road Bridge will be 25-foot bottom width with 2:1 side slopes. The existing Catalpa channel was modified to remove the concrete on the rightbank (looking downstream) while gently sloping back the natural ground to 3.5:1 side slopes. This provides added conveyance to the existing channel along city-owned lands, while also providing for a more natural appearance. Modifications to Catalpa channel in the left bank were limited due to the close proximity of Avenue B and other real estate constraints. This plan reduced damages by 31-percent along the San Antonio River and 99-percent along Catalpa. In an attempt to divert more water from the San Antonio River, a larger diversion channel was modeled.

Larger Diversion Channel at Tuleta. This plan calls for hydraulic modifications including a diversion from the San Antonio River to Catalpa channel, Catalpa channel modifications, and bank stabilization to the San Antonio River. A weir diversion structure just upstream of Tuleta Drive will divert this water down a new diversion channel through Brackenridge Park and empty into the existing Catalpa channel. This 80-foot, sharp crested weir is placed in-line of the proposed diversion channel at elevation 667 feet and designed using the standard weir flow equation assuming a free outfall condition. A new bridge is required at Tuleta Drive to construct this diversion. The channel continues through the Park area for 1645 feet with varying bottom widths between 85 and 150 feet. The channel has alternating side slopes that transition from 3.5:1 grass-lined slopes to vertical walls. The diversion empties the diverted water into Catalpa channel at the existing headwall. The existing Catalpa channel was modified to remove major portions of the existing concrete, while laying back the natural channel side slope on the right bank (looking downstream). Portions of the concrete do remain at the recreation facilities at Lions Field, the Senior Citizens Center, Mulberry Avenue Bridge. Two existing Catalpa channel bridges are replaced in this alternative which were previously analyzed and deemed economically feasible: Mulberry Avenue and Millrace Road, at station 41+80 and 16+00 respectively. This is required in order to pass existing flood flows, including the diverted flood flow from the San Antonio River. The new bridges for Mulberry Avenue and Milrace Road will be set a low chord elevation of 666.0 and 662.0 feet, respectively. The grass-lined channel through the Mulberry Avenue will have a 65-foot bottom width and 3.5:1 side slopes. The grass-lined channel through the Milrace Road Bridge will have a 45-foot bottom width with 3:1 side slopes. Three new bridges are required across the new diversion channel, two for the Eagle railroad and one at Parfun Way. Changes to the San Antonio River were limited to bank stabilization along the main stem, since previous discussions discounted main stem modifications, due to possible destruction of the riparian corridor. Existing condition damages along the main stem occur at the Witte Museum, the park, the zoo and the River Road community. The analysis was to determine how much the water surface elevation is lowered in these damage centers by scalping flood flows at Tuleta at the first event damage event. Table C.1-6 details the diverted flow from the San Antonio River to the Catalpa diversion.

Table C.1-6

	Total Flow cfs	San Antonio River cfs	Catalpa Diversion cfs
2 Year	2372	1807	563
5 Year	3215	2411	802
10 Year	3829	2864	963
25 Year	4669	3499	1170
50 Year	5269	3967	1300
100 Year	5894	4455	1437
250 year	6716	5107	1607
500 Year	7368	5639	1729

Larger Diversion Channel – Flow Distribution at Weir Diversion. This plan reduced damages by 45% along the San Antonio Rive and 99% along Catalpa Channel, but was not effective in reducing damages at the Witte Museum, which is the largest damage center in the reach. In an attempt to reduce damages even more, another diversion channel was analyzed but for this plan, upstream of the Witte Museum at station 2445+45.

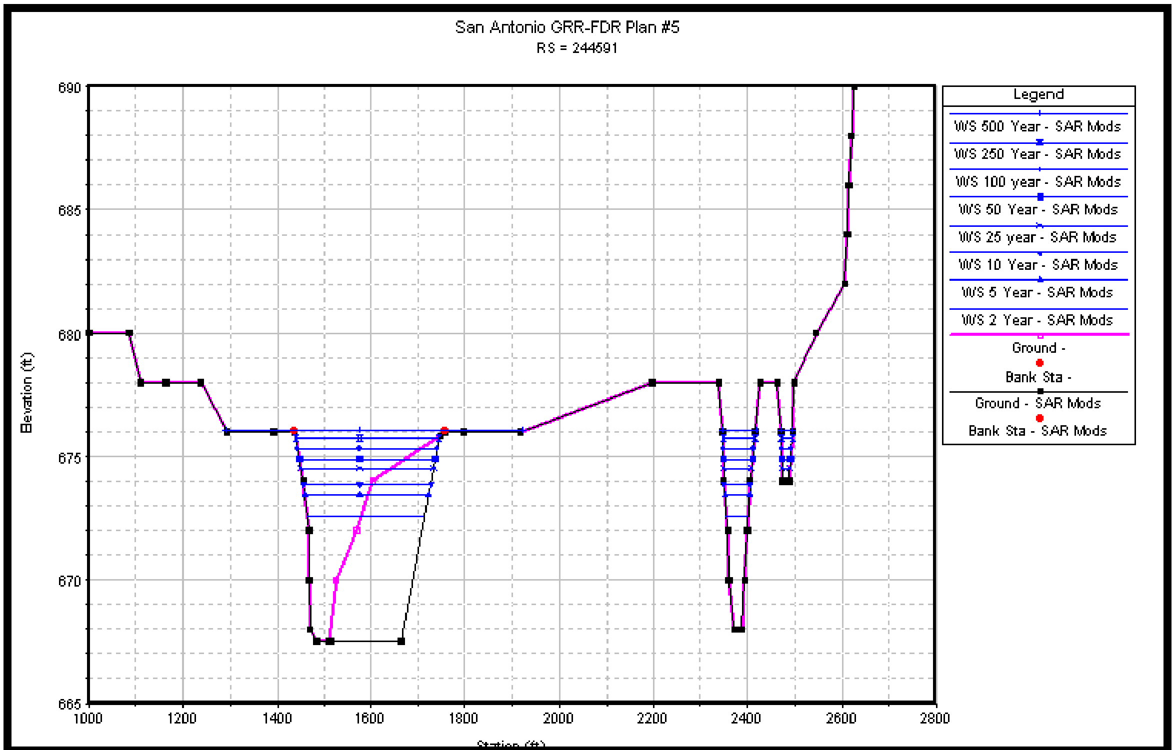
Diversion Channel Upstream of Witte Museum. This plan calls for hydraulic modifications including a diversion from the San Antonio River to Catalpa channel, Catalpa channel modifications, and bank stabilization to the San Antonio River. The weir diversion structure is located upstream of the previously modeled diversion at station 2445+45 of the San Antonio River. This sharp crested weir, set at elevation 667.54 feet, is 50-feet in length and designed for a free outfall condition. The distribution of flows can be seen in Table C.1-7.

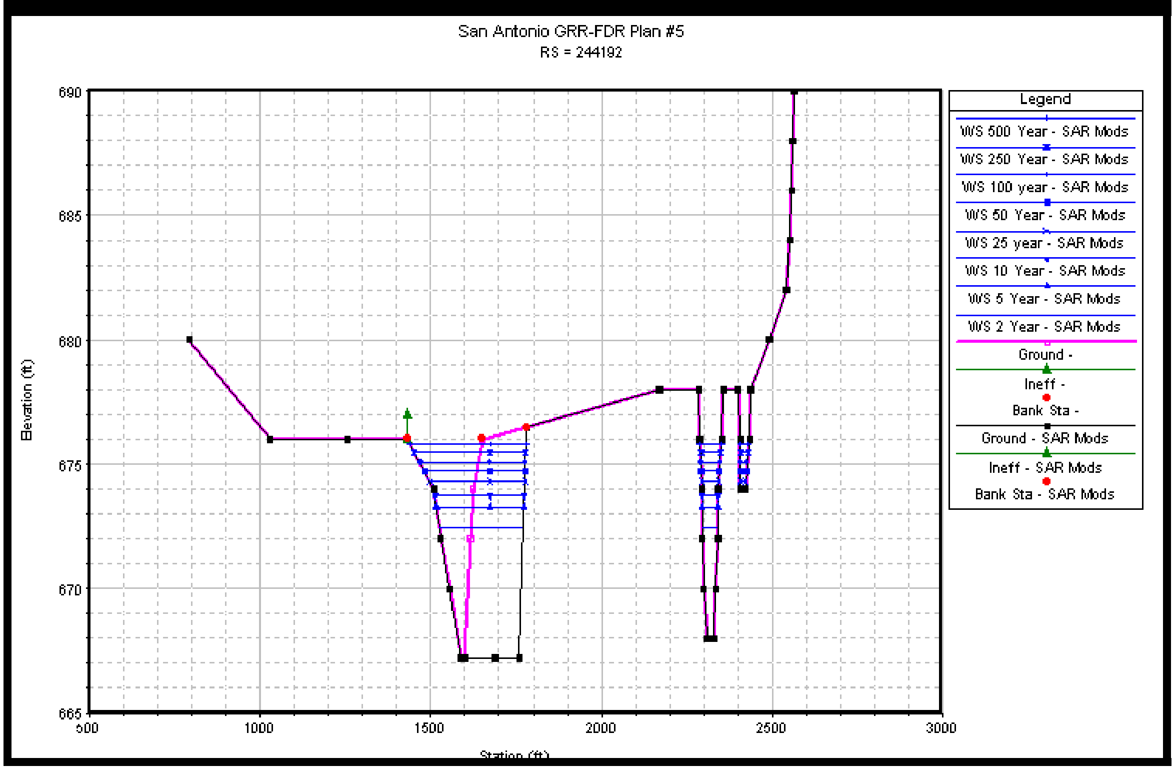
Table C.1-7

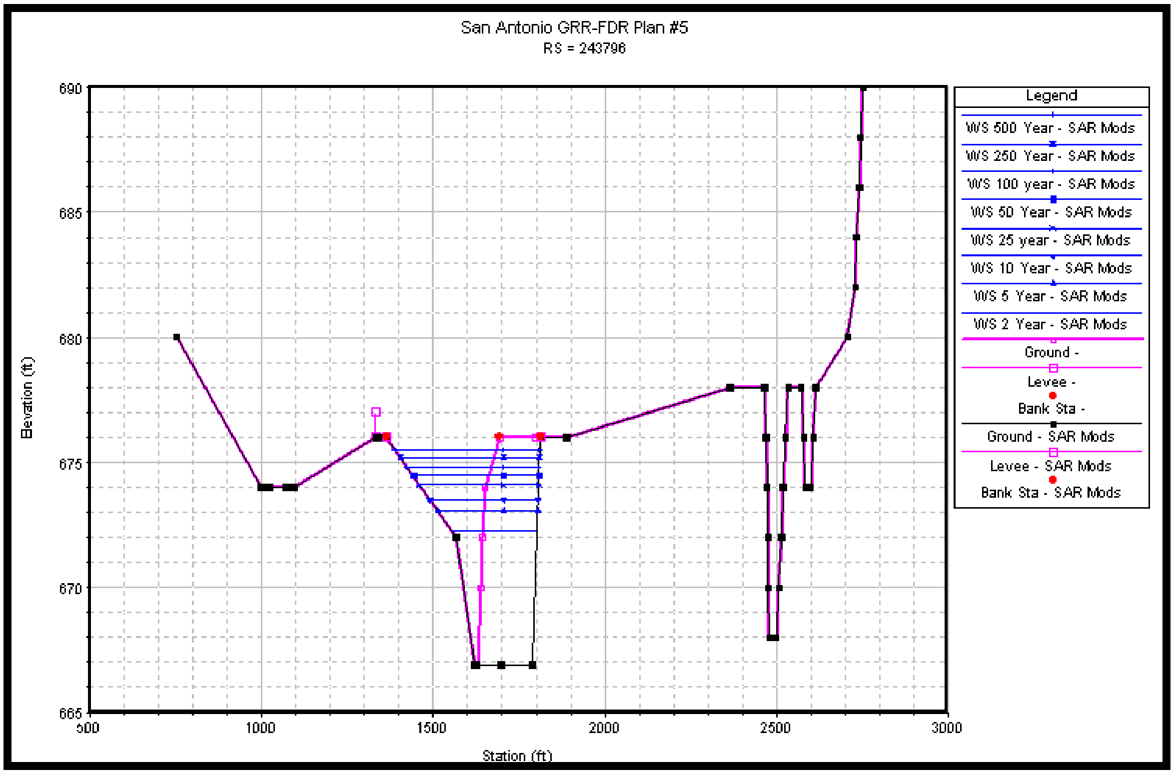
	Total Flow cfs	San Antonio River cfs	Catalpa Diversion cfs
2 Year	2372	1200	1172
5 Year	3215	1675	1540
10 Year	3829	2014	1815
25 Year	4669	2499	2170
50 Year	5269	2869	2400
100 Year	5894	3264	2630
250 year	6716	3816	2900
500 Year	7368	4298	3070

Upstream Diversion – Flow Distribution. The diverted water from the San Antonio River flows in an open channel around the Witte Museum through the existing parking area, in a concrete trapezoidal channel with 2:1 side slopes and a 4-foot vertical retaining wall for both top banks. This vertical segment will make the appearance of this channel, closer resemble the historic vertical walls of the San Antonio River through this area. This diversion channel is 1380 feet in length at a relatively flat slope. This diversion channel continues through Tuleta Road, requiring a new bridge structure, and through the current Nursery area of the Park. At this point the diversion channel follows the alignment of the previously discussed diversions, in a grass-lined channel with a 50-foot bottom width and 3:1 side slopes. The length of this diversion is 1285 feet with a relatively flat slope. This channel empties into the existing Catalpa headwall just downstream of Parfun Way, which will require a new bridge structure. The existing channel will be modified to include concrete removal on the right bank (looking downstream) while gently sloping the side to 3:1. The channel bottom width varies from 30 feet upstream of Mulberry Avenue to 40 feet downstream. The existing channel runs parallel to Avenue B, restricting slope modifications on the left bank. The Mulberry Avenue and Millrace Road bridges will require modification to allow for the additional conveyance needed to carry the current as well as the now diverted flood flows. The concrete channel at Mulberry Avenue will have a 40-foot bottom width, 2:1 side slopes and have a low chord elevation set to 665.0. The channel at Millrace Road will be modified to concrete lined with a 25-foot bottom width and 2:1 side slopes. The low chord will be set at 660.00. This plan reduces flood damages on the San Antonio River by 73% and 99% along Catalpa channel. Damages still do occur and at frequent flood events at the Witte Museum. In order to lower damages more, a final alternative was developed with modifications along the west bank of the San Antonio River in attempt to contain more frequent flooding.

San Antonio River Modification with Diversion at Tuleta. This plan includes channel modifications to the San Antonio River, a diversion weir and channel and modifications to Catalpa channel. This plan is the only plan that includes modifications to the San Antonio River and they occur over a length of 890 feet. The modifications are needed in an attempt to alleviate the damages caused by frequent flood events from the San Antonio River and will done by increasing the channel bottom width, from the existing left bank toe of slope, to 170 feet and laying back the right bank side slope to 2.5:1. These modified sections are 2445+91, 2441+92 and 243796 and can be seen in detail below:







Modified Sections from the San Antonio River

In addition to the modifications to the main stem, a diversion structure and channel is modeled to now diverted flood flows out of the main stem to Catalpa channel. The flow distribution is detailed in Table C.1-8.

Table C.1-8

	Total Flow cfs	San Antonio River cfs	Catalpa Diversion cfs
2 Year	2372	1462	910
5 Year	3215	1955	1260
10 Year	3829	2319	1510
25 Year	4669	2840	1829
50 Year	5269	3215	2054
100 Year	5894	3630	2264
250 year	6716	4170	2546
500 Year	7368	4610	2758

This sharp crested weir is 50 feet long and designed for a free outfall condition. The crest is set at 666.90 feet. The new channel will require a new bridge structure at Tuleta Drive, which will allow a concrete channel with a 75-foot bottom width and vertical side slopes. The diversion channel has a 50-foot bottom width, 3:1 side slopes and is 1615 feet in length. The flood flow enters Catalpa channel at the existing headwall. This will require a new bridge at Parfun Way and for 2 railroad crossing for the Eagle railroad. The existing Catalpa channel was modified to remove the concrete on the right bank (looking downstream) while gently sloping back the natural ground to 3.5:1 side slopes. This provides added conveyance to the existing channel along city-owned lands, while also providing for a more natural appearance. Modifications to Catalpa channel in the left bank were limited due to the close proximity of Avenue B and other real estate constraints.

Modeling Characteristics. Manning roughness values were taken at .035 for grass-lined channels and .015 for concrete lined channels. Hydraulic model ran with a downstream boundary condition of a rating curve at the tunnel inlet. This rating curve was developed for the LMMP model. Flood damage reduction benefits obtained using HEC-FDA. Hydraulic water surface profiles were verified for no crossing profiles and no dips in profile. Info imported into HEC-FDA. Hydraulic models run using HEC-RAS. Study reaches defined in the Economics portion of the Appendices. Final NED plan not identified in this document. Bridges modeled using the Energy method (Standard Step). The flow distribution resulting from the diversion weirs was balanced by trial and error method rather than using split-flow optimization tool in HEC-RAS.

MISSION REACH

San Antonio River. The Mission Reach of the San Antonio River extends from the downstream side of the Lone Star Boulevard Bridge at Sta. 2121+24 to approximately 3800 feet downstream of Interstate Highway 410 at Sta. 1698+70 in the southern part of the city of San Antonio. The Lone Star Boulevard bridge is located just downstream of the San Antonio River Tunnel (SART) Outlet. The upstream and downstream limits of the Mission Reach result in a total river flow line distance of approximately 42,300 feet or 8.0 miles and comprises the downstream portion of the San Antonio River Channel Improvement Project (SARCIP) constructed by the U.S. Army Corps of Engineers. This reach of the historic San Antonio River has been extensively modified by the construction of the trapezoidal grass-lined floodway channel for the purpose of flood damage reduction. The downstream limit of the Mission Reach corresponds to the downstream end of the transition of the SARCIP floodway channel to the original San Antonio River channel. Downstream of this location, the river channel has not been modified for the purpose of flood damage reduction. The floodway project has resulted in the cutoff of a number of the historic river meander bends that existed in this reach prior to the project. Some of these historic meander bends still remain off-channel from the floodway but some segments have been filled for other land uses. The river channel flow line elevation ranges from elevation 598.4 at the Lone Star Boulevard bridge to about elevation 486.0 at the downstream end of the channel modification resulting in an overall channel slope of 0.0027 ft. / ft. or 0.27 % for the entire reach. However the channel slope within the Mission Reach varies from about 0.24 % in the southern part to 0.62% in the northern part.

As-Built Floodway Channel Description. The San Antonio River floodway project was designed as a grass-lined trapezoidal flood conveyance channel with a centrally located base flow channel for most of its length, which has straightened the historic river flow path and increased its gradient. The increased gradient of the river has resulted in the downgrading of the pilot channel from its original design at many locations and has required substantial armoring to maintain channel stability. The floodway channel was constructed to bottom widths within the Mission Reach varying from 50 feet to 300 feet but generally has side slopes constructed to a ratio of 2.5 horizontal to 1 vertical (2.5H:1V). The base flow channel was constructed in varying widths, generally to a depth of 2.5 feet below the floodway channel centerline and with 2H:1V side slopes. The base flow channel has been highly modified over the years since construction due to erosion and implementation of erosion control measures. The following is a description of the floodway channel project as originally designed and a description of the floodway channel and base flow channel in its present condition to follow after. The base flow channel as described in this as-built channel description or “pilot channel” as referred to in as-built construction drawings is not to be confused with the pilot channel described later as a smaller channel within the larger floodway channel designed for the purpose of conveying the “Effective Flow” for sediment transport continuity. For clarity the as-built “pilot channel “ will be referred to as the base flow channel in this description since the as-built base flow channel was designed and constructed originally for the purpose of confining the base flow to the central area of the floodway channel for ease of maintenance.

The floodway channel project within the Mission Reach of the San Antonio River is most notably divided by the confluence with San Pedro Creek. The upper reach between the Lone Star Blvd Bridge and the San Pedro Creek confluence is characterized by a much steeper flow line gradient and a much narrower channel than downstream due to the substantial flood flow combining with the San Antonio River at the San Pedro Creek confluence. Beginning downstream of Lone Star Blvd Bridge this upper reach consists of a 60 ft. wide floodway channel bottom width and has 2.5H: 1V side slopes, but does not have a constructed base flow channel. The channel has a transition to a 50 ft. bottom width at Sta. 2110+00 and a transition back to a 60 ft. bottom width over a distance of 480 feet upstream from the confluence with San Pedro Creek. There are ten (10) sheet pile grade control structures located within this upper reach referred to as Check Dams Nos. 1 through 10 beginning downstream near the San Pedro Creek confluence. These check dams were originally placed with the crest approximately at the elevation of the flow line of the constructed floodway channel. Each sheet pile structure was placed adjacent to a concrete local drainage chute and the tops of the sidewalls of the chute stilling basins were constructed to the same grade as the original channel bottom. The steepest channel bottom segment at 0.6235% within this reach is located between Mitchell Street and the San Pedro Confluence and three of the ten sheet pile check dams constructed for grade control are located within this segment. The flattest channel bottom segment at 0.3560% is between the City Public Service (CPS) vehicle access bridge at Sta. 2102+02 and the Southern Pacific Railroad Bridge at Sta. 2098+97. The average slope for this upper reach between the Lone Star Blvd Bridge (Elev. 601.2) and the San Pedro Creek confluence (Elev. 573.5) as constructed is 0.44%.

Downstream of the San Pedro Creek confluence, the floodway channel was constructed with a bottom width of 200 feet, 2.5H:1V side slopes and a 30 ft. bottom width base flow channel. These dimensions extend downstream to the Davis Lake area downstream of S.E. Military Parkway. A channel bottom width transition from 200 feet to 280 feet occurs through the bend of the floodway channel just downstream of S.E. Military Pkwy from Sta. 1866+00 to Sta. 1872+00. This transition is 600 feet in length. The 280 feet bottom width floodway channel extends downstream from Sta. 1872+00 through the Espada Dam area to about 370 feet downstream of the Ashley Road Bridge at Sta. 1793+00 where the bottom width further expands to 300 feet. The 300 ft. bottom width of the floodway channel extends downstream to the limit of the floodway channel project and terminates in the transition of the floodway channel to the existing channel at Sta. 1698+70. This transition is from a floodway bottom width of 300 feet down to 50 feet and occurs over a channel length of 500 feet.

As previously mentioned, the upper reach of the Mission Reach from the Lone Star Blvd Bridge to the San Pedro Creek confluence was not constructed with a base flow channel. However, beginning at the confluence with San Pedro Creek, a 30 ft. wide base flow channel was constructed extending downstream from the San Pedro Creek confluence to a concrete lined base flow channel at the San Juan River Remnant low flow diversion structure upstream of Ashley Road at Sta. 1809+40. This concrete lined base flow channel segment has a 30 ft. bottom width and is about 800 feet in length. The concrete pilot channel terminates at the headwall of an underground culvert that diverts low flow from the floodway channel through the culvert to the San Juan River Remnant channel just downstream of Ashley Road and the historic Bergs Mill Bridge on the east side (left bank) of the floodway. The culvert consists of three 42-inch reinforced concrete pipes (RCP), has an invert flow line elevation of 508.0, and is approximately 750 feet in length. The culvert is located near the alignment of the original San Antonio River channel and this portion of the river channel was filled to cover the culvert when the floodway channel was constructed. Downstream of this diversion structure to Sta. 1783+00, no base flow channel was constructed due to the base flow being entirely diverted to the San Juan River Remnant. However, a 30 ft. bottom width base flow channel was constructed at the confluence of Las Piedras Creek (Six Mile Creek) at Sta. 1783+00 to divert flow from Six Mile Creek to the center of the floodway channel. This base flow channel segment extends downstream to a concrete lined base flow channel segment at Sta. 1775+80. The concrete lined base flow channel segment at Sta. 1775+80 is a return flow structure for the San Juan River Remnant that is diverted upstream of Ashley road. The length of this concrete lined base flow channel segment is approximately 310 feet and transitions from a bottom width of 30 feet to a bottom width of 40 feet at the downstream end at Sta. 1772+70. From Sta. 1772+70 the 40 ft. bottom width base flow channel extends downstream to a 40 ft. bottom width concrete lined base flow channel segment downstream of I.H. 410 at Sta. 1733+90 and forms a structure for diverting low flow to a remnant of the old San Antonio river channel to the west of the floodway channel. This concrete lined channel segment is located about 160 feet downstream of I.H. 410 and extends downstream about 190 feet to Sta. 1732+00. Low flow is diverted at this structure to an old river channel remnant to the west (right bank) of the floodway channel through a culvert that extends along a portion of the old channel that was filled to cover the culvert. The culvert consists of four 24-inch RCP's for a length of about 180 feet to a junction box and three 24-inch RCP's from the junction box to the outlet. The total length of the culvert is about 570 feet. Flow from this old river remnant is returned to the floodway channel at Sta. 1726+00. From the downstream end of the concrete lined base flow channel segment at Sta. 1732+00, the 40-foot wide floodway base flow channel was originally constructed downstream to Sta. 1718+70 and terminated at a flow line elevation of 490.0. The floodway channel invert was constructed to a flow line elevation of 490.0 from Sta. 1718+70 downstream to the end of the channel project at Sta. 1698+70. Later in the 1970's, the base flow channel was extended downstream to the end of the floodway channel at Sta. 1698+70 with a bottom width of 40 feet and also 1600 feet downstream of the floodway channel to a bottom width of 20 feet.

EXISTING CHANNEL DESCRIPTION

The following is a detailed description of the present condition of the floodway channel and base flow channel with emphasis on the erosion and sediment deposition processes that have altered the channel. Much of this descriptive segment was excerpted from the Geomorphic and Sediment Transport Technical Memorandum (GSTTM) that was prepared for this study. The complete GSTTM is included in this report as Appendix . For clarity, the division of the Mission Reach into sub-reaches used in the GSTTM has been retained for this description.

Below Tunnel Outlet Sub-Reach. The San Antonio River downstream of the San Antonio River Tunnel Outlet Structure and Lone Star Blvd located at Sta. 2121+24 is the beginning of a steeper bed profile and includes a series of vertical drop structures needed for grade control. Below Lone Star Blvd, both the base flow channel banks and bed are armored with concrete riprap, but the right, concave bank has far more volume of riprap protection. Banks and floodplain surfaces are generally comprised of 1 to 2 feet of fine silt loam overlying a 1- foot layer of gravels with a concrete riprap toe. The floodway channel is an earthen channel and the slopes are comprised of grass that is routinely mowed. No woody vegetation is established through this upper segment of floodway. A well-defined herbaceous vegetation line is visible at the top of bank, about 5 feet above the bed. The left bank adjacent to Roosevelt Park offers potential to create an inset floodplain.

Vertical grade control structures known as Check Dam #10 (Sta. 2116+28) and Check Dam #9 (2113+10) are comprised of metal sheet piles embedded in the channel and supported by riprap along the bank margins. The crest elevation of Check Dam #10 and #9 is approximately 599.9 and 598.7 respectively. Vertical drop at each structure is about 2 feet to 4 feet. Check dams #9 and #10 are partially failed, with the sheet piles bowing downstream due to the weight and hydraulic forces exerted from upstream. Thus, at flood stage flow is being directed towards the banks, increasing shear forces. Despite the improper alignment of the sheet pile weirs, lateral bank erosion is minimal due to high volumes of riprap below the structure and adjacent to storm outfalls.

Land acquisition on the left floodway, downstream of Roosevelt Park, offers potential to open up the floodplain and improve flood retention values (Sta. 2140+00 to 2110+00). Local bank instability and floodplain surface scour is found at a storm outfall on the right bank (Sta. 2104+00).

Check Dam structures #8 at Sta. 2104+28 and #7 at Sta. 2098+67, help to maintain grade and stabilize adjacent storm sewer outfalls on both banks. At Sta. 2103+50, bed slope increases, creating uncommon riffle features as the channel passes under two bridge structures (a pipeline/utility crossing and the CPS vehicle bridge). About 4 feet of bridge pier scour has occurred at the CPS bridge piers based on the original design grade and the current survey of the channel. Photo of this site is available in addendum.

The base flow channel width decreases as slope increases through this heavily encroached segment. The channel passes under the Southern Pacific Railroad line (Sta. 2098+97), Steves Avenue (Sta. 2095+10), and Interstate Highway 10 (Sta. 2090+00). On the right floodplain, a secondary overflow channel has formed upstream of Steves Avenue. Between I.H.10 and Mitchell Street, the majority of both bank segments are heavily armored with riprap, which extends 10 to 12 feet above the channel bed. The exposed, near vertical right bank underneath the IH-10 overpass is eroding and provides a moderate sediment source.

Check dam sheet pile grade control structures #6, #5, and #4 are used to prevent further channel incision and undermining at storm sewer outlets. However, at Sta. 2082+25, a tributary storm outlet on the right bank forms a concrete rectangular pad, perpendicular to the channel flow. This structure forms an abrupt ledge or sill, and a hydraulic jump at flood flows likely causes bed scour immediately downstream on the main channel. Large concrete riprap maintains relative stability in this segment. Again, each sheet pile structure was placed adjacent to a concrete local drainage chute and the tops of the side-walls of the chute stilling basins were constructed to the same grade as the original channel bottom. This provides evidence of the erosion that has occurred in the floodway channel bottom.

The number of grade control structures is numerous through this reach. The opportunity exists to replace and add additional drop structures, increasing the number of structures, but decreasing their vertical drop (hydraulic head). The entrenched and laterally confined, highly urbanized reach offers limited changes to channel alignment and thus, sinuosity will remain the same for the new design channel. Additional grade control structures built as riffle features would help distribute the slope more evenly over a greater channel distance.

Concepcion Sub-Reach. The confluence with San Pedro Creek at Sta. 2060+50 marks a significant decrease in bed slope and an increase in channel width and cross sectional area downstream. The tremendous increase in flow volumes and sediment input would indicate San Pedro Creek watershed generally dictates the dominant hydrologic/hydraulic condition and geomorphic characteristics of the downstream reaches. At the confluence, both floodways are heavily armored with riprap comprised of 2 to 4 foot slabs of concrete. A large cobble and gravel bar has formed along the right channel margin. To prevent vertical incision and head cutting, concrete grade control structures are placed perpendicular to both the San Pedro and San Antonio River channels immediately upstream of the confluence. The enormous volume and size of riprap and extent of poured concrete on both banks and floodway terrace is indicative of extremely high stream power, and subsequent erosion, and degradation that can occur during major flood events.

Below E. Theo Road, the historic San Antonio River channel pathway is visible on the left bank at Sta. 2048+00. On the right bank, a large concrete-lined tributary outfall enters the main channel (Sta. 2047+35). Local erosion is very high. A well armored, but active nick point at the confluence gives evidence of channel instability, bed scour and ongoing degradation. The invert elevation of the tributary outfall is about 9 feet above the invert of the San Antonio River channel at the confluence.

Downstream to the Mission Road Bridge, the pilot channel is a straight trapezoid, and lined with riprap. The quantity of concrete riprap in and adjacent to the channel is substantial. Although the flood channel lacks variability, the channel has formed subtle riffle features that are evenly spaced at about 100 to 200 feet, with marginal pool habitat (depth < 2 feet) along the outer, concave channel margins. A veneer of finer-grained alluvium is deposited on top of the armored bed. Further downstream to Mission Road, visible aquatic channel features disappear due to a backwater caused by a 5- foot vertical, concrete grade structure upstream of Mission Road at Sta. 2007+68.

The Concepcion Park channel reach between the San Pedro River confluence and Mission Road is a relatively wide floodway with far less infrastructure or lateral constraints compared to upstream reaches. The Conception Park property on the left floodway (east) offers additional width to create an inset floodplain environment adjacent to the active channel, in concert with flood detention opportunity.

Mission Sub-Reach. The Mission sub-reach extends from upstream of Mission Road Bridge (Sta. 2006+30) to a more depositional segment above the San Juan Diversion (Sta. 1910+45). Channel margins are almost entirely armored with concrete riprap through this sub-reach, which correlates with high shear stress values and erosive potential. A far greater volume of concrete is placed on the outer, concave left bank. The inside, convex right channel margin has formed a series of small gravel point bars between Sta. 2003+00 to Sta. 1993+00.

On the left bank, the tributary outfall (Sta. 1993+00) from the Riverside Municipal golf course is comprised of a concrete, rectangular 3-section box culvert with an apron of grouted boulder riprap on the upstream and downstream side. Opportunities exist to re-vegetate and enhance this outfall.

Between Mission Road and White Avenue (1944+07), the Mission Parkway parallels the floodway. As the parkway approaches the Roosevelt Avenue underpass, a three arch bridge, the road embankment encroaches on the floodway. An unstable concrete sill grade structure spans the channel at Sta. 1978+70 above the bridge and storm drain outfalls draining the Riverside Municipal golf course enter from the right bank at Sta. 1978+30.

The longitudinal profile or bed slope begins to decrease as the channel nears the E. Southcross Blvd Bridge. Floodway surfaces show evidence of recent maintenance with topsoil-capped floodplains. Banks are generally 4-feet vertical where riprap is not present.

Some woody plant species are established about 3 to 4 feet above the toe of the bank within the heavily riprap banks. This subtle woody vegetation line provides potential reference when establishing planting zones relative to water surface stage, inundation rates, and bank shear profiles.

The volume of concrete riprap slabs and poured concrete on the floodplain and banks poses design challenges and high costs to remove and/or replace with properly sized native rock materials. Past operation and maintenance efforts include capping the floodplain with topsoil and reseeding with grass. Scoured floodplain segments are evident where capped soil and sod have been ripped and plucked from the floodway surface, exposing the layers of poured concrete that inhibit further scour and erosion. These segments serve as explicit indicators of high stream power and shear stress outside the base flow channel on the floodplain/floodway bench.

Downstream of White Avenue, a storm drain tributary outfall enters the main channel at Sta. 1937+00. Outfall discharge may be utilized as a water source above the San Antonio River base flow that may support riparian and wetland plant communities and habitat.

San Juan Sub-Reach. As a result of the San Juan Dam diversion structure at Sta. 1910+45, channel dimensions and sediment transport characteristics gradually change to a more depositional reach. Concrete riprap bank margins end temporarily near Sta. 1939+50 on the right bank and 1929+00 on the left bank. The active channel width increases and depth decreases. The San Juan diversion structure reduces flow velocities, creating a backwater environment and the development of a long point/side gravel bar deposit that is exposed at base flow (Sta. 1929+00 to 1922+00) on the left side of the channel. Sediment accretion and backwatering through this segment has also reduced relative bank heights from about 6 to 3 feet on the left bank, providing greater flood accessibility to a lower floodplain surface area. Banks are comprised of cohesive silt loam and clay with intermittent layers and lenses of coarse gravelly alluvium. A well-developed flow line is present on the right side of the channel, an uncommon channel feature in a project reach generally characterized by a uniform, trapezoidal base flow channel and floodway.

The San Juan Dam creates approximately a 6 –foot vertical change in bed elevation. The old San Juan Dam and acequia (irrigation system) is located immediately downstream on the left floodway.

The old San Juan Dam structure on the main channel is located at Sta. 1899+52, with the upper end of Symphony Lane and remnant historic channel on the right floodplain. The right channel margin is actively eroding for about 400 feet, an 8- foot vertical cut bank composed of silt and sand overlaying a distinct clay geologic formation. The old San Juan Dam acts as a critical grade control structure, maintaining grade and vertical stability upstream. The existing structure is a concrete-lined channel bed and side slopes with a concrete sill that has partially failed. Active lateral instability on both banks and channel scour immediately downstream of the old structure indicates a nick point and head cut potential. The old San Juan Diversion Structure is just upstream of the upper pool level of Davis Lake created by Espada Dam.

The distinct clay formation is exposed on the channel bed and/or banks more frequently downstream. The cohesive clay unit is grayish-blue and uniform in texture. The channel bed has down cut well into the clay 1-4 feet, but remains relatively stable under the current flow regime. The clay is readily soluble with abrasion or rubbing, but does not exhibit massive slab or rotational failure along eroding bank segments. To some unknown threshold, the clay layer likely maintains channel grade and provides a distinct horizontal plane or conveyor belt for sediment movement. Bed load is deposited on the clay boundary and readily entrained and transported during peak flows as a plug of sediment that moves episodically downstream.

Davis Lake Sub-Reach. Backwater from Espada Dam contributes to extensive depositional bar features through the Davis reach. Large gravel and cobble deposits are formed, an anomaly from upstream conditions. At higher flows, the bars are submerged, but at base flow conditions, side and mid-bar deposits are readily visible, which likely form on the receding limb of peak flow events. At some critical discharge, the backwater reduces flow velocities and causes sediment deposition. However, sediment transport analysis indicates the Davis reach is still quite competent transporting sediment through the reach during high flows. A comparison of 1960's ACOE channel cross sections from as-built plans and current 2002 topographic surveys indicate several feet of sediment deposition has occurred on various segments of the Davis reach.

The remnant, historic San Antonio River channel re-enters the main pilot channel at Sta. 1880+00 on the right bank, and the Asylum Creek confluence is at Sta. 1877+66 on the left bank. Asylum Creek is a large tributary with a concrete side slope/apron and a concrete dam grade structure at the mouth. Asylum Creek is a trapezoidal concrete-lined inset channel with mowed grass floodway for an undetermined distance upstream.

Beginning at approximately Sta. 1883+00, a mid-channel bar deposit forms diagonally across the channel. Through this segment, a well developed side gravel bar deposit on the inside, convex left bank margin extends from well above Asylum Creek, under the S. E. Military Drive Bridge to the upper end of the Espada Dam structure, about 3,200 feet in length. The bar extends from the distal side of the left floodway to the base flow channel. The base flow channel's left margin is well defined by an abrupt and steep submerged ledge. Channel maintenance, sediment dredging and removal above Espada Dam likely are responsible for this bank feature anomaly. An extensive sediment dredging operation in Davis Lake was completed in 2003.

Below Espada Dam Sub-Reach. Immediately below Espada Dam, a low water ford provides vehicular crossing and additional vertical bed stability. On the right channel margin, the historic San Antonio River channel re-enters the floodway pilot channel at Sta. 1846+00. The historic Espada Dam is a historic landmark located on the secondary (historic) channel on the right floodway. On the main pilot channel, banks below the dam are heavily armored with large slabs of concrete riprap. The riprap on the left bank beginning at Sta. 1828+00 constricts and narrows the channel width for several hundred feet. At base flow, the constriction creates riffle features and a change from laminar to turbulent flow conditions. Local hydraulics and in-channel riprap structure increases aquatic habitat complexity compared to upstream and downstream segments. Riprap confinement on the left bank margin ends and bank materials change to non-armored alluvial fine-grained materials composed of sand, silt and clay.

At Sta. 1809+25, the channel bed changes to a concrete-lined segment, which extends about 800 feet downstream. A remnant channel swale outlet converges with the main channel on the left bank at Sta. 1806+00. The concrete channel is used to control and divert base flows through culverts (Sta. 1802+00) under the Mission Parkway Road embankment, which outlet at the historic bridge and remnant San Antonio River channel on the left bank (east) of the floodway. This structure maintains base flows to the historic channel segment that re-enters the pilot channel at Sta. 1774+00. This structure effectively diverts all of the normal base flow from the floodway pilot channel and the floodway channel is usually dry downstream to Sta. 1791+00 below Ashley Rd. under low flow conditions.

A distinct change in bed elevation is evident beginning at approximately Sta. 1805+00. As the longitudinal profile indicates, the downstream reach (Six Mile) is more uniform and constant in slope for about 3,000 feet.

Six Mile (Piedras) Creek Sub-Reach. Loss of a well-defined channel due to low flow diversions indicates sediment accumulation through the upper segment of the reach. The floodway continues south under Ashley Road (Sta. 1797+53). The pilot channel is not well defined through this dewatered segment of the floodway. The active floodplain is wider (about 300 feet) with a poorly defined secondary channel that cuts through haphazard riprap materials. The floodplain is scattered with large (4- foot plus) slabs of concrete riprap. Marginal woody plant growth along the floodway bottom indicates the altered hydrology (dewatered at base low) allows some woody plants to colonize the floodway. Imbrication of massive pieces of concrete bed material gives evidence that high stream power associated with large flood events still impacts this reach.

At Sta. 1791+00, Six Mile Creek or Piedras Creek by-pass channel enters the San Antonio River floodway. The addition of flows from this major tributary begins to re-form a pilot channel. The historic remnant channel confluence with the pilot channel is located downstream at Sta. 1774+28. Between Sta. 1776+50 to 1773+00, the main pilot channel is concrete-lined before returning to a well-defined trapezoidal pilot channel with armored riprap and alluvial banks. A well-armored riffle segment, immediately below the outlet, extends for several hundred feet, an indicator of distinct grade break and less steep sub-reach downstream.

410 Sub-Reach. Near Sta. 1770+00, the channel bed slope decreases dramatically, as shown on the longitudinal profile. Historic channel incision and over bank deposition is evident throughout this sub-reach. I.H. 410 is located at Sta. 1736+67. Flood debris caught in the bridge piers indicates floodwater surface elevations nearly 18 feet high above the channel bed. Below the interstate overpass, a remnant channel is maintained with flow diversions on the right floodplain. Where the side channel converges with the main pilot channel, a large gravel bar has formed at Sta. 1726+00, below the Camino Coahuilteca low water crossing downstream, the right bank remains unprotected, whereas the left bank is heavily armored with concrete riprap to the end of the project reach. As a result, the right channel margin is characterized as an 8 to 15- foot near vertical eroding cut bank composed of fine silt and sand material.

Below Project Area. Downstream from the project reach, the floodway trapezoidal channel ends. A natural channel configuration with steep fine-grained alluvial banks and a gravel bed and a dense woody riparian plant community characterizes the San Antonio River. Active floodplain and lateral bar development, aquatic habitat, and vegetative complexity are high when compared to the upstream project reach. Steep terrace walls laterally confine the channel and provide a natural sediment source to the system.

Extremely flashy runoff driven by rain storm events moves large volumes of sediment and re-deposits material as a massive wedge or plug in short durations, episodically migrating downstream over time. A 1.5- inch rainstorm event on March 19, 2002 caused stream flows at the I.H.410 USGS gage station to rise from 75 cfs to nearly 3,000 cfs in less than one hour. The following day, fresh deposits of sand and small gravels up to 20 inches in depth were observed on lateral point and side bars.

MISSION REACH MAJOR FEATURES

The significant features within the Mission Reach floodway channel, such as, bridges, grade control structures, channels dams, low water crossings, diversion structures, and pilot channel characteristics have been discussed above. However there exist a number of significant features that lie adjacent to the floodway channel that have some impact on the design of the Mission Reach environmental restoration.

Lone Star Boulevard Bridge and Tunnel Outlet. The San Antonio River Tunnel (SART) outlet structure is located just upstream from the Lone Star Blvd Bridge. The SART diverts flood flow from the river channel at the tunnel inlet located just upstream of Josephine Street. The SART provides flood protection for the downtown San Antonio area. The channel at the outlet location and downstream through the Lone Star Blvd. Bridge opening is concrete lined. This portion of the channel is a high energy flow area and is effectively a stilling basin combining flow from the tunnel outlet to the intervening flow from the river channel between the SART inlet and outlet structure. The invert of this concrete lined channel segment is at elevation 598.37.

Lone Star Brewery. The Lone Star Brewery site is located adjacent to the floodway channel on the right bank (west) side of the floodway channel between Sta. 2107+00 and Sta. 2121+00.

Roosevelt Park. The Roosevelt Park is located adjacent to the floodway channel on the left bank (east bank side of the floodway channel between Sta. 2109+00 and Sta. 2121+00.

City Public Service Facility. The CPS Facility is located on the left bank side of the floodway channel between Sta. 2101+00 and Sta. 2109+00 and also between Sta. 2078+00 and Sta. 20183+00. The facility is also located on the right bank side between Sta. 2099+00 and Sta. 2107+00 and between Sta. 2063+00 and Sta. 20168+00. The pipeline bridge over the floodway channel at Sta. 2103+50 and the vehicle maintenance bridge at Sta. 2102+10 are part of the CPS Facility and are to remain in place.

San Antonio Water Supply Facility. The San Antonio Water Supply (SAWS) facility is located on the left bank of the floodway channel between Sta. 2061+00 and Sta. 2075+00 and also between Sta. 2083+00 and Sta. 2088+00.

Concepcion Park. The Concepcion Park is located on the left bank of the floodway channel between Sta. 2019+00 and Sta. 2061+00.

E. Theo Street. E. Theo Street is located adjacent to the floodway channel on the left bank side between Sta. 2052+00 and Sta. 2061+00. The street lies between the top bank of the floodway channel and the Concepcion Park.

H Street and B Street Mobile Home Residential Area. A mobile home residential area lies adjacent to the floodway channel on the left bank side between Sta. 2007+00 and Sta. 2018+00 just upstream from the Mission Road bridge. B Street is located near the top bank of the floodway channel between Sta. 2007+00 and Sta. 2013+00 and residential structures are located along H Street between Sta. 2013+00 and Sta. 2018+00 with the structures and property lines located at the top bank of the floodway channel.

Mission Parkway. The Mission Parkway and adjacent trail is located along the right bank side of the floodway channel between the E. White Ave. bridge at Sta.1944+00 and the Mission Road bridge at Sta. 2006+50. The Mission Parkway and trail is located at the top of bank of the floodway channel for most of this length but the portion between Sta. 1957+00 and Sta. 1981+00 is located within the floodway channel and passes under the Roosevelt Ave. Bridge and the E. Southcross Blvd Bridge.

Riverside Golf Course. The Riverside Golf Course is located downstream of the Mission Road Bridge on the left bank side of the floodway channel between Sta. 1992+00 and Sta. 2006+00. The golf course is also located on the right bank side of the floodway channel between Sta. 1978+00 and Sta. 2006+00.

Riverside Drive Residential Area. A residential area along Riverside Drive is located adjacent to the left bank side of the floodway channel upstream of the Roosevelt Drive Bridge between Sta. 1981+00 and 1992+00.

Mission County Park. Mission County Park is located on the right bank of the floodway channel downstream of E. White Road between Sta. 1935+00 and 1944+00.

Padre Park. Padre Park is located on the right bank of the floodway channel between Sta. 1902+00 and 1935+00. The downstream limit of Padre Park is near the upstream portion of the historic river remnant surrounding the Symphony Lane community.

Hot Wells Bath House. The historic Hot Wells Bath House is located on the left bank of the floodway channel near Sta. 1925+00. This historic structure is less than 200 feet from the top of the floodway channel.

Symphony Lane Area River Remnant. A historic river remnant known as the Symphony Lane River Remnant is located in the right bank floodplain between Sta. 1880+00 and Sta. 1902+00. The river remnant is cut off from the floodway channel at the upstream end near Sta. 1902+00 but low flows are maintained through the river remnant by gravity flow through a culvert outlet at Sta. 1902+00. The downstream end of the river remnant discharges to the floodway channel by means of an open channel to the Davis Lake near Sta. 1880+00.

E. Pyron Road Residential Area. A residential area is located adjacent to the floodway channel and E. Pyron Rd. on the left bank side between Sta. 1885+00 and Sta. 1891+00.

Espada Park. The Espada Park is a national park located on the right bank of the floodway channel between Sta. 1846+00 and Sta. 1862+00 and surrounds a historic river remnant. The historic Espada Dam remains on this river remnant and is located in the right overbank area of the floodway channel at Sta. 1851+00 lateral to the newer Espada Dam on the floodway channel. The floodway channel dam is also known as Davis Lake Dam and maintains a pool level of approximately 633.7 on the floodway channel. This river remnant is cutoff from the floodway channel at the upstream end near Sta. 1862+00 but low flow is maintained through the river remnant channel by culverts from Davis Lake. Low flow through the river remnant is returned to the floodway channel downstream of Davis Lake Dam at Sta. 1846+00.

Acequia Park. The Acequia Park is located in the left bank area of the floodway channel from Ashley Road at Sta. 1798+00 to S.E. Military Drive at Sta. 1871+00. The park is named for the historic San Juan Acequia that is located nearby. Mission Parkway is located within the park and a portion of Mission Parkway is located adjacent to the top bank of the floodway channel from Sta. 1806+00 to Sta. 1824+00.

Mission San Juan Capistrano. The historic Mission San Juan Capistrano is located on the left overbank area of the floodway channel downstream of Ashley Road at approximately river Sta. 1790+00. This historic structure is located east of a historic San Antonio River remnant known as the San Juan River Remnant.

San Juan Acequia. The historic San Juan Acequia is located near the left top bank of the floodway channel between Sta. 1740+00 and Sta. 1747+00.

Camino Coahuilteca. The Camino Coahuilteca is a park road leading to the Mission Espada and is a low water crossing of the floodway channel at Sta. 1727+60. Box culverts beneath the low water crossing convey the normal low flows without overtopping the roadway.

Mission San Francisco de la Espada. The historic Mission San Francisco de la Espada is located on right overbank area of the floodway channel downstream of I.H. 410 near the historic river remnant at Sta. 1726+00.

MISSION REACH WITHOUT-PROJECT HYDRAULIC ANALYSIS

The San Antonio River LMMP HEC-RAS hydraulic model has been used for analysis of existing conditions for the Mission Reach. For the purposes of this analysis of potential habitat improvement measures, the hydraulic analysis results from the existing conditions model will be referred to as the "Without Project" condition. The model has been developed primarily for analysis of rare flood event flows such as the 10-year, 50-year, 100-year, and 500-year events as required by FEMA. However, the model also has been used for analysis of low flows such as the predominant base flow of 20 cfs and the "effective flow" determined in the Geomorphic and Sediment Transport Study. The "effective flow" as described in the GSTTM is the hypothetical value of flow that is presumed to transport most of the sediment over time. The flow values for the effective discharge as determined in the GSTTM are used in the design of a stable pilot channel to maximized long-term channel stability.

For the purposes of this study, three basic flow conditions are primarily used for design and analysis. The water surface profile resulting from the 100-year flood event for the Without Project condition has been selected as the benchmark from which all proposed channel modifications are compared. These comparisons ensure that proposed plans do not increase flood risk compared to the existing floodway channel. Secondly, the effective flows as determined in the GSTTM are used to maximize the pilot channel stability over time, minimize costs to maintain, and provide improved and consistent long term channel bed substrate for aquatic habitat. Thirdly, the base flow is used to analyze the function of riffle structures, low water crossings, and long-term conditions for aquatic and riparian habitats including river chutes, riffles and pools. A base flow value of 20 cfs has been determined to be the median base flow using analysis of the available stream gage records for the San Antonio River in the Mission Reach. This analysis of base flow was performed as part of the San Antonio River sediment transport study and is documented in the GSTTM. However, the flow values for both 100-year flood event and the effective flows vary considerably within the Mission Reach. The values for the effective flow are found in Table T.4 on Page 98 of the GSTTM and shown in Table C.1-9 for reference.

The Manning's roughness coefficients used in the Without Project hydraulic model for the Mission Reach range from 0.035 to 0.100 in the overbank areas, but are consistently 0.035 within the floodway channel. A consistent roughness coefficient for the floodway channel has been used based on periodic maintenance of the floodway being continuously performed. The required maintenance of the floodway channel is generally comprised of mowing the channel bottom and slopes and removal of woody plants to ensure the flood carrying capacity of the floodway is maintained as designed. The use of a roughness coefficient of 0.035 for the base flow channel where mostly open water is generally found would normally be considered slightly higher than recommended. However, substantial amounts of concrete and rock riprap have been placed both on the side slopes of the pilot channel and the bed of the base flow channel. Therefore, the use of a roughness coefficient of 0.035 for the entire width of the floodway channel including the base flow channel has been considered a reasonable estimate for the overall floodway channel and compares well with the model calibration results.

The hydraulic analysis for the 100-year flood event indicates that there is a wide variation in flow depth throughout the Mission Reach. The following is a discussion of the flow depth variations through the Mission reach as indicated on the 100-year water surface profile. The depths are expressed as the maximum flow depth measured from the water surface to the invert of the base flow channel. It should be noted that due to the significant variation in the base flow channel depth of

Table C.1-9

	Sub-Reach Effective Discharge Estimates	
	Subreach	**Effective Discharge (cfs)**
North Reach	BELOW OLMOS	510
	PARK 1	740
	PARK 2	740
	URBAN	1,034
	DOWNTOWN	1,076
South Reach	EAGLELAND	1,106
	BELOW OUTLET	1,180
	CONCEPTION	2,391
	MISSION	2,559
	SAN JUAN	2,630
	DAVIS	2,639
	BELOW ESPADA	2,758
	SIX MILE	2,882
	410	3,200

approximately 2 to 10 feet, there exists a wide variation between the base flow channel invert and the elevation of the lower terrace of the floodway channel throughout the Mission Reach.

In the upper reach between Lone Star Blvd and the San Pedro Creek confluence, the flow depth ranges between 21 to 24 feet. Downstream of the San Pedro Creek confluence the flow depth is between 30 and 33 feet to the San Juan Diversion Dam. Downstream of the San Juan Dam through the Davis Lake area to the Espada Dam, the flow depth is generally 21 to 23 feet. Downstream of the Espada Dam to I.H. 410 the flow depth is generally 30 to 33 feet with the exception of the reach between the San Juan River Remnant diversion structure upstream of Ashley Road and the river remnant return structure at Sta. 1774+00. Within this reach the flow depth is substantially less at 21 to 23 feet. Downstream of I.H. 410 the flow depth increases to 36 feet for the 100-year flood event. Flow depths can be seen graphically using the water surface profiles of the Mission Reach. The water surface profiles of the Mission Reach for the Without Project Condition are shown on Plates C.1-1 through C.1-3.

The water surface profiles for the 100-year flood event also provides an indication of a number of bridges that cause a significant hydraulic head loss at the higher flows. This is indicated on the water surface profile by an abrupt and significant rise in the water surface profile at the bridge location. At these locations, bridge or channel modifications may be considered for potential opportunities to lower the flood levels upstream of the bridge and provide opportunity for increased vegetation within the channel or other measures that increase habitat values. Some of these potential bridges as evidenced by the Without Project 100-Year water surface profile are: Ashley Rd., E. Southcross Blvd., E. White Ave., and E. Mitchell St.

The hydraulic analysis for the 100-year event within the Mission Reach has shown that because of the relatively steep slope of the floodway channel, average flow velocities are high. High flow velocities are typical throughout the range of higher flows and translate to high shear stress values at many locations that may result in erosion. In the upper part of the Mission Reach below Lone Star

Blvd down stream to the San Pedro Confluence average flow velocities within the floodway channel are typically 9 to 12 feet per second (fps) except for a short reach of about 1000 feet just below Lone Star Blvd that is in the range of 6 to 8 fps. From the San Pedro Creek confluence downstream to the Roosevelt Ave. Bridge, flow velocities are typically about 13 fps but are slightly lower at 9 to 12 fps downstream to the I.H. 410 bridge. Below the I.H. 410 Bridge, flow velocities are typically 5 to 8 fps because of the backwater effects of the downstream natural channel.

MISSION REACH DESIGN APPROACH AND DESIGN CONDITIONS

Three basic “Design Conditions” have been framed to address the wildlife habitat deficiencies and opportunities outlined in the Problem Statement. These design conditions focus primarily on significantly varying scales of earthwork involved to create habitat improvements and provide for channel stability within the applicable design constraints imposed upon the project reach. A general consensus has been reached between the U.S. Army Corps of Engineers (USACE) and the local sponsor, San Antonio River Authority (SARA), on some basic design constraints that applies to all plans developed for the Mission Reach.

These design constraints are:

(1) Project designs will not increase the Without Project 100-year water surface profile. The Without Project 100-Year water surface profile will be the primary guideline for determining the effects of habitat measures.

(2) Project elements shall be designed to withstand the erosive energies associated with the 100-year flood event

The framework for three basic “Design Conditions” has been used to provide a step-wise progression of habitat improvement measures for the Mission Reach. The Design Conditions are based upon three significantly varying scales of earthwork required to develop wildlife habitat improvement plans. The unique requirements of each Design Condition are discussed below and the common design procedures for all are provided following.

Design Condition 1. The development strategy for Design Condition 1 (DC-1) was to implement the appropriate number and types of habitat improvement measures that result in habitat unit (HU) gains and other ecosystem benefits without excavation beyond the limits of the existing floodway channel federal right-of-way (ROW) and without a rigorous sediment transport design constraint. DC-1 seeks to improve wildlife habitat and provide as much total ecosystem benefits as reasonably attainable without requiring additional lands or easements beyond the current floodway ROW. This design condition also does not have the requirement for a within floodway pilot channel designed with rigorous adherence to the sediment transport guidelines as specified in the GSTTM. Excavation designed for measures developed under DC-1 would be that necessary to construct in-stream riffle structures, excavation to increase the depth of pools and/or increase conveyance within the floodway channel, excavation required to enhance or create wetlands, excavation needed to improve the channel longitudinal slopes for improvement in the long term dominant substrates, and excavation required to remove undesirable materials, such as concrete rubble, from the channel.

Design Condition 2. The development strategy for Design Condition 2 (DC-2) is to implement the appropriate number and types of habitat improvement measures that result in habitat unit (HU) gains and other ecosystem benefits in conjunction with the creation of a new pilot channel designed to convey the “effective discharge” or “effective flow” as defined in the San Antonio River Geomorphic & Sediment Transport Technical Memorandum (GSTTM). The “effective flow” is the flow for which the frequency and sediment transport capacity are maximized. The goal of the pilot channel design for DC-2 is to provide equilibrium of sediment transport and minimize the damaging effects of sediment accumulation and erosion within the system while providing for improved habitat and ecosystem values.

The sediment transport pilot channel designed for DC-2 is to be excavated within the current floodway channel and excavation is to be primarily limited to the bottom width of the floodway channel and the existing project right-of-way. The existing floodway channel will not be modified in overall width in order to gain hydraulic conveyance and no additional lands, easements or rights-of-way will be required.

A base flow channel will be constructed within the pilot channel to convey the average low flow of 20 cubic feet per second (cfs) and located primarily within the river chutes (runs). Base flow channels are not applicable within pools or areas backwatered by riffle structures. Base flow channels are not used within riffle structures so that the habitat potential of the riffle structure can be maximized. Riffle structures will be constructed at various points along the river and at various heights to control grade and attain the reach average sediment transport equilibrium slope as recommended in the GSTTM. The findings and conclusions of the GSTTM have been used as a guide for the design of the pilot channel and base flow channels.

Design Condition 3 (3A and 3B). The development strategy for Design Condition 3 has been divided into two subdivisions to better analyze the habitat improvement opportunities within this Design Condition (DC-3A and DC-3B). DC-3 basically follows the same rationale as DC-2, but will allow for excavation and modification to the existing floodway channel and existing federal right-of-way. The floodway channel is to be excavated beyond the existing right-of-way limits where opportunities exist in order to gain flood conveyance that will allow more extensive habitat improvement measures to be implemented within the floodway without compromising the flood carrying capacity. Both DC-3A and DC-3B have the same basic pilot channel design incorporating the design guidelines established in the GSTTM and is very similar to the pilot channel design incorporated in DC-2. The difference between the pilot channel design in DC-3 and DC-2 is primarily the pilot channel in DC-3 is wider in some reaches due to the enlargement of the floodway channel in DC-3 that was not afforded in DC-2.

DC-3A was developed initially and DC-3B was developed as a second step to analyze the effects of modifying certain measures to determine if significant habitat gains within this design condition could be attained. The habitat measures modified in DC-3B from DC-3A. are: 1) Riffle structures have an inset base flow channel within them in DC-3A but are removed in DC-3B, 2) Some larger pool areas in DC-3A have been reduced in size to allow more riparian vegetation, 3) Vegetation types and locations have been modified in order to result in higher overall HU gains.

COMMON DESIGN PROCEDURES FOR EACH DESIGN CONDITION

A number of common design procedures have been applied to each of the Design Conditions for the Mission Reach. The following is a discussion of these common procedures and the methods of applying the various identified habitat improvement measures to the hydraulic analysis.

Baseline Water Surface Profile. Habitat improvement measures that include the addition of trees and other woody types of vegetation to the floodway channel generally reduce the flood carrying capacity or conveyance of the floodway channel by increasing the channel surface roughness and thereby increase water surface elevations and the flood risk. The primary design limitation for all of the design conditions is that habitat measures must not increase the flood risk over the present condition. The baseline condition that all Design Conditions have used for design and comparison is the Without Project 100-year water surface profile developed for the San Antonio River LMMP study as discussed in the “Development of Existing Conditions Hydrology and Hydraulics Model” section above. The design procedures in each Design Condition utilize the hydraulic analysis in a trial and error methodology to determine the effects of the vegetation on the hydraulic performance of the floodway channel and simultaneously analyze the effects of any measures to reduce these effects. The Without Project water surface profiles are shown on Plates C.1-1 through C.1-3. The water surface profile comparisons for each of the Design Conditions is shown on Plates C.1-4 through C.1-15.

Vegetation Types Criteria. In order to facilitate consistent hydraulic design and analysis, common hydraulic analysis criteria for the establishment of vegetation types with specific characterization of tree planting spacing and associated under-story growth was established prior to the initiation of this study. A document was prepared entitled " Assigning Manning's "n" Values for Vegetation Associations" that characterizes various vegetation types or habitat improvement measures to be used for the hydraulic analysis and the environmental design. This document was prepared in coordination with the Corps of Engineers, the City of San Antonio, the San Antonio River Authority and local engineering firms. The document contains recommended Manning's roughness coefficients to be used in the hydraulic analysis for the various planting zones for mature trees and plants and has been carefully reviewed by all parties involved to insure that sufficiently conservative values would be used for design. The document is included in this report in Appendix .

Hydraulic Influence for Riparian Zones. It was determined that the riparian influence to aquatic habitats would have some variability within the floodway channel due to the hydrologic regimes within this system. This variability is due in part to the relative frequency of inundation of the various riparian zones. A methodology to include this variability into the habitat analysis was based on the flood frequency analysis that was completed for the study of Without Project conditions. This methodology is based on establishing a ratio of variability of the frequency of inundation between the lower zone of riparian vegetation and the upper zone. Some basic assumptions were required in order to adjust the habitat values for the riparian zones. First, the inundation level of the lower zone of vegetation was assumed to be just above the flood level of the pilot channel flowing full. This level was assumed to be a flow slightly greater than the "effective flow" which was used to size the pilot channel. Secondly, the inundation level of the upper zone was assumed to be approximately half way up the side slopes of the floodway channel. A low flow frequency analysis was done to facilitate the sediment transport analysis documented in the GSTTM. A flood flow frequency referred to as the 0.25-Year event was extrapolated from the flood frequency analysis completed for the San Antonio River LMMP Study and closely approximates the criteria of being slightly greater than the pilot channel capacity. Therefore, this frequency was selected as being representative of the frequency of inundation for Riparian Zone 1. The water surface profile for the 2-year flood event was analyzed and found to be representative for the frequency of inundation for Riparian Zone 2. The relative frequency difference between these two flood events is a factor of 8 and this value was used in the habitat analysis to account for the variability of inundation between Riparian Zone 1 and Riparian Zone 2 vegetation.

Conveyance Balance Analysis. The hydraulic analysis uses the Manning's roughness coefficient, which is a measure of surface roughness used in the Manning's equation, in the computation of water surface elevations. A component of the Manning's equation, which defines the physical characteristics of the stream, is "conveyance ". Conveyance is a measure of the flood carrying capacity of the stream and is a function of primarily three variables. These three variables are; surface roughness, available flow area, and the wetted perimeter of the cross section. Because the addition of trees or other woody vegetation in the floodway increases the surface roughness and to some degree reduces the available flow area, conveyance is reduced and flood levels will generally rise. Each of the design conditions includes measures that will provide increased or compensating conveyance in combination with the additional vegetation such that the flood carrying capacity is not reduced. Primarily the compensating factor for the increased roughness used in each of the Design Conditions is by excavation to increase the channel flow area. In the design process, generally measures that increase conveyance within the river channel, such as the sediment transport pilot channel, are modeled first and vegetation measures are then located to produce habitat gains and are modeled iteratively until the habitat gains are approximately maximized such that the 100-Year flood water surface profile for Without Project conditions is not exceeded.

Comparison with Existing River Channel. Although in some cases, compensating for increased channel roughness within various parts of the channel can be accomplished by reducing roughness in other areas within the channel, this is not possible within the San Antonio River floodway. This is because the hydraulic analysis of the existing floodway channel includes the use of a Manning's "n" value of 0.035 to model the hydraulic effects of the predominant grass-lined surfaces. This value has been used in the analysis for the entire floodway channel width and also for the entire Mission Reach.

The use of a Manning's "n" value of 0.035 for the floodway channel represents a typical floodway design value for grass-lined channels with minimal surface irregularities and low-level vegetation throughout. Because of the size of the floodway channel, this value is a predominant factor in the determination of the baseline 100-year water surface profile that was used for comparisons of alternatives. Since this value has been used to analyze the hydraulic performance of the existing floodway channel for both grass surfaces and open water areas, this value was also selected for analyzing the effects of habitat measures for the same types of ground cover whether they are water areas or grass areas. Therefore, the opportunity to lower roughness values to increase conveyance simply by changing to a hydraulically smoother ground cover compared to the existing floodway channel does not exist in the analysis.

One measure to improve habitat values within the river channel that was considered was to only convert the predominantly Bermuda grass-lined existing channel to native grass cover. Because roughness values would not be changed by conversion to native grasses, this measure would result in no change to the hydraulic performance of the floodway. Therefore, the evaluation of this measure alone did not require hydraulic analysis for comparison to the Without Project condition.

Riffle Structures. All of the Design Conditions utilize some form of a "riffle structure" both as a direct habitat improvement measure and an indirect measure. Riffle structures are used to provide many habitat restoration functions and also provide some essential hydraulic functions. Riffle structures are hardened surface structures placed in the stream with the ability to resist the erosive velocity associated with large floods without erosion or significant degradation. Riffle structures generally have surfaces with higher than normal stream slopes such that under low flow conditions a very shallow white water or "riffle" effect is created in the flow. Most of the riffle structures for each of the design conditions are constructed from graded stone or riprap. The stone surfaces provide an economical means of resisting erosive energies in the stream and as a direct habitat improvement measure, provide for additional substrate diversity. The water filled spaces between the stones provides habitat diversity by providing harbor for invertebrates and provides for the interaction between the soil beneath the stones, the ground water and the active stream flow. As an indirect habitat improvement measure, riffle structures are used as dams to create an upstream pool that increases the flow depth to desirable levels and reduces low flow velocity within the pool. As a byproduct of reducing upstream flow velocity within the pool areas generally stream sheer stress is reduced for the flows that influence the sediment transport functions and the dominant substrate is generally improved. Hydraulically, riffle structures perform an essential function as drop structures or grade controls where a relatively abrupt drop in channel grade can occur over a short distance and resist the erosive energy associated with that grade change without channel degradation. Riffle structures provide for the ability to effectively reduce the river slope between structures. Reducing the channel slope provides the benefit of enhanced sediment transport continuity by reducing the damaging effects of degradation and sediment deposition. Reducing the channel slope also provides habitat improvement by reducing low flow velocities in river chute reaches.

Pilot Channel for Sediment Transport. Design Conditions DC-2, DC-3A, and DC-3B all have as a prerequisite an excavated pilot channel within the floodway channel designed to convey the "effective flow", for the purposes of maintaining sediment transport continuity and reducing the negative effects of erosion and sediment deposition. This pilot channel was designed in accordance with the parameters set forth in the GSTTM and is functionally identical for each of these Design Conditions. The sediment transport pilot channel includes excavation to establish the proper stream width and gradient and includes structures to maintain this gradient.

Hydraulic Parameters for Aquatic Habitats. Several hydraulic parameters for the river have been used in the analysis of the aquatic habitats. These parameters are indicators of long-term stream conditions that influence the quality of the aquatic habitat within the river. The parameters that have used in the analysis are: cumulative water surface area, average flow velocity, maximum flow depth, water surface elevation, and flow top width. These parameters have been computed using the same basic HEC-RAS hydraulic model that was used for the Without Project condition with minor revisions to better reflect the base flow channel for low flow. The base flow value of 20 cfs has been used to determine the hydraulic parameters for each of the Design Conditions.

Dominant Substrate Analysis. The aquatic habitat evaluation for each of the Design Conditions has included a methodology for comparing the expected dominant substrate of each of the Design Conditions. This methodology has been derived from the basic analysis techniques documented in the GSTTM due to the dominant substrate being strongly influenced by the sediment transport characteristics of the channel. The methodology is based on the use of the Shield's equation discussed ion pages 85-86 of the GSTTM. Shield's equation describes the hydraulic condition at which motion of individual sediment particles is initiated. The hydraulic conditions at which incipient motion occurs can be described as the critical shear stress on the bed material. Shield's equation expresses the relationship between the critical shear stress of the flow and the sediment size of the bed material. Therefore, the methodology uses the Shields equation and the computation of the shear stress from the HEC-RAS model for each of the Design Conditions to solve for a dominant sediment size by reach. The flow rate used to compute the shear stress with the HEC-RAS model is 10% of the "effective flow". This flow rate was found to reasonably approximate the actual dominant sediment size within the channel for Without Project conditions. The dominant sediment size for the without project conditions were established by field investigations and sampling. This dominant substrate methodology is not intended to accurately predict the expected dominant substrate but provide a reasonable means of comparing the changes expected for each design condition and to the without project condition.

SPECIFIC CHANNEL FEATURES INCLUDED IN THE HYDRAULIC ANALYSES

Riffle Structures. The riffle structures used in DC-1, DC-2, and DC-3B have typically the same basic configuration with a level crest (DC-3A has a "notched" crest) that is lateral to the river channel and has downward sloping surfaces both upstream and downstream of the crest. In most structures, an anchored concrete inverted T-wall is used in the core of the structure. The concrete T-wall core provides the impervious membrane needed to ensure that a continuous pool level upstream of the structure is maintained under a wide range of flows. The T-wall limits the flow of ground water and through-flow that would occur if only placed stones were used and reduces the risk of piping of fine soil particles beneath the structures that could result in failure or movement of the structure. The level crest of the T-wall also provides for a maximum width of water surface area in the pool immediately upstream from the structure. The T-wall crest and the adjacent rock surfaces serve to function as a weir by spreading and equalizing the flow energy across the channel bottom evenly such that the risk of potential movement of the stones is minimized under high flow conditions. The T-wall also serves to spread the base flow evenly across the channel to maximize the water contact with the rock surfaces of the riffle downstream of the crest. The concrete top of the T-wall will be virtually unnoticeable under low flow conditions. The stone riprap layers upstream and downstream of the T-wall combined with the anchored T-wall is an economical means of providing for a stable structure under a variety of flow conditions.

Pools. Pools are created upstream of the riffle structures to varying widths and depths ranging from 2- to 4-feet maximum. Pools are maintained continuously by the riffle structures and the base flow. The maximum pool depth is generally at the upstream side of the riffle structure and gradually reduces in depth in the upstream direction due to the channel slope. Pools are defined in the habitat evaluation along the river extending upstream from a riffle structure to a point where the water surface is no longer level and effectively terminate where the flow transitions into a more narrow river chute base flow channel or in some cases terminates at the downstream slope of the next upstream riffle structure.

Weirs. In DC-2, DC-3A, and DC-3B, two weirs where included in the sediment transport hydraulic model. These are located at River Stations 1773+78 and 1798+50.

Invert Slope Protection. In DC-2, DC-3A, and DC-3B, two areas of invert slope protection are included in the sediment transport hydraulic model. These hardened bottom areas are needed due to the localized higher shear stress values in the flow. There are located between River Sta 1786+13 and 1793+71, and 1796+13 and 1798+03.

Bridge Modifications. In DC-3A and DC-3B, the channel plan calls for pilot channel modifications under two bridges, East White Ave. and East Southcross Blvd. The bridge modifications provide additional flow conveyance allowing for the placement of additional vegetation.

Vegetation Types. The vegetation types with trees and under-story have been generally located near the waters edge where possible for pools and river chutes. The hydraulic modeling has used the horizontal variation of Manning's "n" values methodology in HEC-RAS to model the placement of these vegetation types within the channel and uses the "Assigning Manning's "n" Values for Vegetation Associations" memo as criteria for the selection of roughness values.

Four different vegetation types were established in this study, ranging from native grasses to forests with varying density. The roughness of these types, related to hydraulic resistance is measured by the Manning's roughness coefficients ("n" values) applied in the HEC-RAS model. The layout of this vegetation was established through an iterative process of varying the n-values in the HEC-RAS model based on optimized zoning, while comparing the resulting water surface elevations to the existing 100-year water surface elevations. A rise of no more than 0.10 feet was allowed. In order to ensure and maximize the restoration around key features such as embayments, tributary mouths and riffle pool complexes, these areas were first established. Team members developed vegetation placement design criteria, which provided a process of "adding" and "reducing" the vegetation area or density in order to ensure that the resulting 100-year profile is not above the baseline profile. Water surface profile dip computational anomalies occasionally occur at very rapidly changing cross sectional areas of the stream, such as at bridges. These dips in the profiles are where the water surface appears to be lower in the upstream direction for a short distance and dips in the water surface profiles generally were excluded from comparison.

Embayments. Embayments are used to provide intermittent shallow pools and are excavated adjacent to an in-stream pool or a river chute base flow channel. Embayments are excavated within the lower level of the floodway channel but generally above the level of the base flow channel or the pools. They have a generally trapezoidal cross section shape and extend linearly along the river's edge for varying distances. The shape of the embayments slightly enhance the flood conveyance of the channel due to the additional excavation, but are modeled with slightly higher Manning's roughness coefficients due to the expectation of vegetation that is denser than the native grasses within the wetted areas of the embayment. A Manning's roughness coefficient of 0.045 has been used to model the hydraulic effects of the wetland areas of the embayments. Embayments are generally from 1 to 3 feet in depth and are recharged with water by periodic higher river flow or periodically by local runoff through adjacent outfall structures. Table C.1-5 displays the approximate location by river station for each embayment in each design condition analyzed in the hydraulic models.

Tributary Mouth Restoration. Tributary mouth restoration measures are small pools created at the mouths of existing tributary streams flowing into the main stem of the San Antonio River, which are excavated to allow a still water pool to remain following a storm event, enhancing the aquatic habitat. An existing tributary mouth is located at River Sta. 1756+00 and will be excavated and re-graded in conjunction with an adjacent embayment. This tributary mouth area will be graded to retain local runoff to an approximate maximum depth of 3 feet. The pond area is separated from the base flow channel by a ridge to maintain wetland conditions during dry periods. The water surface of the tributary mouth is common with the adjacent embayment. This tributary mouth area extends from River Sta. 1655+00 to Sta. 1656+50.

An existing concrete culvert local runoff outfall is located at River Sta. 1937+00. This outfall consists of a grouted riprap trapezoidal outfall chute at a large culvert pipe headwall. The culvert pipe invert at the headwall is approximately 10 feet above the streambed. The grouted riprap chute is to be removed from the existing base flow channel bank to the existing ROW, a distance of approximately

150 feet. A new chute is to be constructed of rock riprap with 3H:1V side slopes and a 3H:1V invert slope from elevation 550.0 down to the streambed elevation of 540.0.

The Concepcion Creek tributary mouth outfall structure extends from River Sta. 2043+00 to Sta. 2045+60. This existing tributary outfall is a concrete lined trapezoidal channel extending from the tributary channel into the floodway channel. This modification extends to the existing ROW and involves removal of the existing concrete channel lining to the existing project right-of-way. The removal of the concrete lined channel is to be terminated at the existing toe (elevation 571.0) of a drop chute in the concrete channel and this location is approximately coincident with the ROW limit. A new concrete chute channel extension is to be constructed at this point effectively extending the existing drop chute down on the same slope to elevation 565.0. The sloping part of the new concrete chute will be approximately 30 feet in length and a level portion of the new concrete chute at elevation 565.0 will extend from the toe of the sloping portion another 30 feet. Therefore, the flow line length of the new concrete structure is about 60 feet. The side slopes of the new concrete chute will match the old concrete structure at the juncture of the old and the new and are approximately 2H:1V. The side slopes of the new concrete chute will vary from 2H:1V to 2.5H:1V. Rock riprap protection will be placed to extend from the end of the new concrete chute out into the adjacent embayment excavation to form a tributary mouth pool. The top of the riprap layer will be at elevation 565.0 forming the base of the pool and will also form an outlet sill at elevation 567.0 along the right bank of the proposed base flow channel. The riprap layer will extend up the existing side slopes of the tributary and the floodway channel to elevation 580.0. The rock riprap layer will also extend out across the base of the proposed base flow channel from River Sta. 204300 to River Sta. 204560.

River Remnant Restoration. The existing San Juan River Remnant Diversion structure is located upstream from Ashley Road at River Sta. 1801+83. Currently, low flows are diverted at this site from the floodway channel by means of a concrete lined channel segment which directs flow to a concrete headwall on the left bank side of the floodway channel into a 3 x 42” diameter concrete pipe culvert. Low flow is diverted through the culvert pipe under ground approximately 750 feet to the culvert termination downstream of Ashley Road. Approximately 580 feet of the upstream portion of the culvert pipe and the upstream headwall is to be removed and an open channel will be excavated along the path of the historic river. This channel will consist of re-vegetated natural soil. The excavated bottom width of the restored channel will be 30 feet and side slopes will vary from 2H:1V to 5H:1V. An approximate 170 feet length of the existing pipe culvert and the downstream headwall is to remain in place to convey flow beneath Ashley Road. A new concrete headwall is to be constructed at the upstream end of the remaining culvert pipe and will be located about 60 feet upstream of Ashley Road. Low flow will continue to be diverted from the floodway channel to the river remnant by the proposed Riffle Structure No. 6A and 6B. This river remnant was included in the hydraulic model for all design conditions.

A second river remnant restoration was included in DC-3A and DC-3B and is located at River Sta 1733+00 near the Espada Mission and I.H. 410. This river remnant was partially filled and low flow was maintained to the original downstream natural portion of the remnant by a concrete culvert pipe when the floodway channel was constructed. This culvert pipe is approximately 550 feet in length and conveys low flow from the floodway channel to the remaining natural portion of the river remnant. This restoration consists of complete removal of the concrete culvert and headwalls and excavation of an open channel to the same approximate slope and shape as the original river channel. Riparian zones will be re-established on the excavated slopes.

Concrete Channel Removal. An existing concrete channel segment at River Sta. 1732+00 was constructed originally for the purpose of diverting low flow to the existing river remnant on the right bank floodplain. The concrete channel segment diverts low flow from the floodway channel to the headwall of an existing culvert pipe and flow is carried through the pipe underground approximately 550 feet to the existing old river remnant channel. This existing concrete channel lining for the base flow channel extends approximately 195 feet along the San Antonio Floodway channel for River Sta. 1732+00 to Sta. 1733+95 and is about 55 feet in width with an arm that extends to the remnant diversion culvert on the right bank. The diversion arm of the structure is about 33 feet in width. This structure is to be completely removed as part of the floodway channel excavation for all Design Conditions

Another existing concrete channel lining segment extends from River Sta. 177270 to Sta. 177580. This segment conveys the low flow from the San Juan River Remnant to the center of the floodway channel and extends approximately 310 feet along the San Antonio Floodway channel and is about 55 feet in width at the downstream end and 45 feet in width at the upstream end. The structure has a diversion arm that extends to the remnant channel on the left bank. The diversion arm of the structure is about 50 feet in width. This structure is to be completely removed as part of the floodway channel excavation for all Design Conditions

The existing concrete channel segment located from River Sta. 1801+70 to Sta. 1809+40 extends approximately 800 feet along the San Antonio River Floodway channel and is about 60 feet in width. This river remnant diversion structure conveys the base flow from the center of the floodway channel to the culvert headwall on the left bank side of the floodway channel and diverts flow from the floodway to the San Juan River Remnant near Ashley Road. This concrete channel segment is to be completely removed as part of the floodway channel excavation for all Design Conditions.

The Old San Juan Diversion Dam and concrete channel lining extends from River Sta. 1898+85 to Sta. 1900+40. This concrete structure is to be removed entirely and the floodway channel side slopes restored to original grades as part of the floodway channel excavation for all Design Conditions.

The hydraulic performance results from the development of the each Design Condition is shown in the water surface profiles for the base flow, the effective flow, and the 100-year flood event on Plates C.1-4 through C.1-15.

NATIONAL ECOSYSTEM RESTORATION PLAN

The National Ecosystem Restoration (NER) Plan that has been selected is described above as DC-3B. This plan has been described in detail, however, some additional hydraulic performance information is provided herein. Table C.1-11 provides a comparison of water surface elevations and average floodway channel flow velocities for the 100-year flood event and the water surface profiles for DC-3B are shown on Plates C.1-13 through C.1-15.

Table C.1-11

100-Year Flood Event

River Station	Flow	Minimum Channel Elevation	NER Plan W.S. Elev.	Without Project W.S. Elev.	W.S. Comparison		NER Plan Chan Velocity	Without Project Chan Velocity	Velocity Comparison
	(cfs)	(ft)	(ft)	(ft)	(ft)		(ft/s)	(ft/s)	(ft/s)
213817	9965	603.97	620.24	621.43	1.19		6.34	5.68	-0.66
213495	9965	603.81	619.61	621.1	1.49		7.12	5.83	-1.29
213195	9965	603.66	618.94	620.66	1.72		7.54	6.33	-1.21
212921	9965	600.8	619.03	620.71	1.68		5.44	4.74	-0.7
212786	9965	600.53	619.01	620.7	1.69		5.19	4.48	-0.71
212680	9965	600.32	619	620.71	1.71		4.82	4.13	-0.69
212655	Bridge								
212630	9965	600.22	618.81	620.58	1.77		5.06	4.29	-0.77

212576	9965	600.11	618.69	620.49	1.8		5.55	4.73	-0.82
212523	9965	600	618.62	620.44	1.82		5.76	4.88	-0.88
212489	Inl Struct								
212377	9965	601.2	618.57	620.41	1.84		4.59	3.95	-0.64
212259	16435	598.51	617.97	619.96	1.99		7.42	6.35	-1.07
212198	16435	598.32	617.18	619.51	2.33		10.01	8.12	-1.89
212161	Lone Star Blvd								
212124	16435	600.29	616.33	618.66	2.33		10.95	8.75	-2.2
212015	16435	599.82	616.25	618.74	2.49		8.26	6.37	-1.89
211912	16435	599.36	616.03	618.58	2.55		7.76	6.37	-1.39
211633	16435			618.37				5.37	
211628	16435			618.35				5.48	
211623	16435			618.35				5.38	
211428	20876	596.43	615.83	618.22	2.39		7.49	6.01	-1.48
211330				617.97				6.69	
211310				617.91				6.88	
211247				617.8				6.98	
211212				617.87				6.1	
211113				617.79				6.11	
211028	20876	594.77	615.82	617.58	1.76		7.04	6.99	-0.05
210663	20876	593.86	615.04	616.43	1.39		6.1	8.75	2.65
210428	20876	592.12	614.36	615.3	0.94		8.39	10.61	2.22
210359	20876	591.82	613.83	615.35	1.52		9.6	9.74	0.14
210346.5	Aerial Pipeline Bridge								
210334	20876	591.7	613.62	615.06	1.44		9.73	9.98	0.25
210300	20876	591.8	613.05	614.83	1.78		10.84	10.37	-0.47
210247	20876	590.98	612.29	614.64	2.35		11.6	10.57	-1.03
210220	20876	591.1	612.21	614.53	2.32		11.49	10.69	-0.8
210202	CPS Bridge								
210184	20876	590.94	611.53	614.12	2.59		11.83	10.86	-0.97
210113	20876	590.63	611.47	614.05	2.58		10.66	10.24	-0.42
209997	20876	590.79	611.82	613.98	2.16		7.17	9.42	2.25
209908	20876	588.83	610.64	613.95	3.31		10.68	8.8	-1.88
209897	Southern Pacific Railroad								
209886	20876	588.73	609.67	613.57	3.9		11.59	8.86	-2.73
209867	20876			613.24				9.64	
209848	20876	588.56	609.83	613.45	3.62		9.73	8.47	-1.26
209732	21284	587.95	609.19	612.91	3.72		10.18	9.54	-0.64
209651	21284	587.59	608.97	612.68	3.71		9.78	9.76	-0.02

209563	21284	587.2	608.09	612.63	4.54		11.48	9.27	-2.21
209510	Steves Ave.								
209457	21284	586.74	607.02	612.2	5.18		11.8	8.89	-2.91
209336	21284	587.87	607.23	611.69	4.46		8.38	9.73	1.35
209176	21284	585.5	606.82	611.41	4.59		8.64	8.36	-0.28
208986	21284	584.66	605.28	610.42	5.14		10.37	9.29	-1.08
208804	21284	585.03	604.24	606.65	2.41		10.05	15.57	5.52
208735				608.21				8.43	
208730				607.52				10.5	
208725				607.88				8.71	
208414	21284	582.14	604.23	608.03	3.8		4.63	5.1	0.47
208250				607.93				5	
208245				607.9				5.24	
208240				607.91				5.02	
208128	21284	582.91	603.67	607.35	3.68		5.81	7.6	1.79
207861	21284	580.2	602.5	605.91	3.41		10.01	10.39	0.38
207817	21284	579.5	601.69	605.23	3.54		11.73	11.85	0.12
207778	E. Mitchell St.								
207738	21284	578.9	599.07	603.63	4.56		15.15	11.03	-4.12
207671	21284	578.29	599.71	603.42	3.71		10.28	11.09	0.81
207585				603.59				9.32	
207580				600.8				15.79	
207575				602.27				10.58	
207558	21284	579.26	599.02	601.46	2.44		10.8	12.49	1.69
207204	21284	574.48	599.01	600.75	1.74		5.95	11.1	5.15
207182				600.34				12	
207173				600.49				11.35	
206824	21284	572.83	598.86	600.11	1.25		4.12	9.68	5.56
206815				600.16				9.36	
206810				600.13				9.44	
206805				599.45				11.33	
206595	21284	569.99	598.76	599.85	1.09		4.21	7.83	3.62
206361	21284	568.1	598.49	599.68	1.19		4.79	7.53	2.74
206199	21284	571	598.6	600.19	1.59		2.76	2.62	-0.14
206062	70396	566.4	596.37	598.4	2.03		11.41	10.55	-0.86
205888	70396	565.11	595.83	597.34	1.51		11.21	12.48	1.27
205698	70396	565.5	595.11	597.81	2.7		11.54	9.85	-1.69
205454	70396	563.02	594.3	596.3	2		12.94	12.43	-0.51
205261	70396	562.48	593.62	595.71	2.09		12.44	12.94	0.5
205210	70396	562.33	593.11	595.57	2.46		13.28	13.03	-0.25
205173	E. Theo Ave.								
205136	70396	562.14	592.44	594.92	2.48		13.68	13.45	-0.23

205079	70396	561.97	592.11	594.81	2.7		13.9	13.37	-0.53
204930	70396	561.55	591.48	594.56	3.08		12.96	13.06	0.1
204735	75660	561	591.2	593.91	2.71		10.64	13.61	2.97
204149	75660	558.61	591.19	592.62	1.43		8.92	13.5	4.58
203916	75660	560.85	590.28	592.07	1.79		9.16	13.57	4.41
203644	75660	557.63	590.08	591.4	1.32		5.86	13.67	7.81
203277	75660	555.6	589.81	589.96	0.15		4.26	14.62	10.36
202893	75753	557.52	589.12	589.29	0.17		5.95	13.56	7.61
202531	75753	553.71	588.14	588.77	0.63		7.44	12.84	5.4
202156	75753	552.66	587.75	588.27	0.52		7.3	12.29	4.99
201849	75753	555.6	587.57	587.57	0		6.95	12.68	5.73
201465	75753	551.72	585.69	586.68	0.99		11.39	13.08	1.69
201090	75753	549.97	584.95	586.14	1.19		11.17	12.75	1.58
200773				585.83				12.05	
200768				585.71				12.34	
200763				585.76				12.1	
200723	75753	552.11	584.02	585.77	1.75		11.28	11.85	0.57
200666	75753	551.75	584.04	585.77	1.73		10.7	11.38	0.68
200630	Mission Road								
200593	75753	551.4	583.67	585.15	1.48		11.02	11.79	0.77
200356	75753	548.45	583.81	583.92	0.11		8.12	13.37	5.25
200065	75753	547.93	582.91	583.34	0.43		9.66	13.27	3.61
199779	75753	550.42	582.49	582.88	0.39		9.29	12.95	3.66
199486	78088	547.09	581.91	582.09	0.18		9.71	13.38	3.67
199181	78288	546.34	580.9	581.54	0.64		11.09	13.11	2.02
198848	78288	548.74	580.47	580.98	0.51		10.33	12.81	2.48
198530	78288	545.37	580.67	581.16	0.49		7.36	10.72	3.36
198260	78288	544.88	580.6	581.32	0.72		6.19	9.1	2.91
197991	78288	544.2	580.24	580.36	0.12		6.47	10.88	4.41
197744	78288	546.57	578.23	579.14	0.91		11.8	12.75	0.95
197678	78288	546.55	578.49	579.77	1.28		10.68	10.57	-0.11
197643	Roosevelt Ave.								
197608	78288	545.51	577.7	578.97	1.27		10.65	10.56	-0.09
197526	78288	545.36	577.61	578.56	0.95		10.5	11.33	0.83
197319	78288	542.19	577.18	578.08	0.9		10.7	11.82	1.12
197119	78288	545.63	577.15	577.73	0.58		9.2	12.02	2.82
196926	78439	542.48	577.3	577.69	0.39		6.53	11.57	5.04
196612	78439	541.71	576.92	577.54	0.62		6.17	10.59	4.42
196378	78439	541.29	576.59	577.18	0.59		6.36	10.81	4.45
196322	78439	542.44	575.62	577.28	1.66		9.82	10.05	0.23
196287	E. Southcross Blvd								

196251	78439	542.31	574.21	575.97	1.76		10.65	10.9	0.25
196178	78439	541.13	574.48	575.2	0.72		8.29	12.31	4.02
196039	78416	540.68	573.78	574.88	1.1		9.25	12.48	3.23
195663	78416	543	572.76	574.32	1.56		8.74	12.29	3.55
195240	78416	540.24	570.61	574.06	3.45		10.86	11.07	0.21
194847	78416	539.53	570.18	573.57	3.39		8.82	11.01	2.19
194500	78416	538.61	570.18	573.36	3.18		7	10.39	3.39
194442	78416	538.51	569.1	573.55	4.45		10.41	9.24	-1.17
194407	E. White Ave.								
194371	78416	538.28	568.74	570.63	1.89		9.43	12.23	2.8
194285	78416	538.23	568.64	569.47	0.83		9.45	14.02	4.57
194143	78416	537.97	568.44	569.11	0.67		9.22	14.05	4.83
193617	78767	537.02	565	567.96	2.96		14.32	13.56	-0.76
193207	78531	539.08	565.62	565.92	0.3		8.3	15.46	7.16
192847	78531	536.43	565.47	565.84	0.37		6.67	13.22	6.55
192613	78531	535.36	565.27	566.17	0.9		6.43	9.77	3.34
192256	78531	534.72	564.45	565.4	0.95		8.22	10.47	2.25
191800	78531	533.9	563.67	563.61	-0.06		8.57	12.67	4.1
191487	78531	533.34	561.81	562.69	0.88		12.17	13.6	1.43
191201	78531	538	561.16	562.02	0.86		12.05	13.62	1.57
191045	78531	538	560.52	561.4	0.88		12.71	14.35	1.64
190605	78531	529.76	560.06	559.89	-0.17		10.64	13.74	3.1
190297	78531	529.21	560.12	560.26	0.14		8.27	9.67	1.4
190011	78531	528.7	559.72	560.65	0.93		7.72	5.96	-1.76
189952				558.21				13.51	
189951				558.36				12.93	
189860	78531	528.6	558.81	558.71	-0.1		9.73	10.8	1.07
189747	78531	528.5	558.01	558.7	0.69		11.13	9.62	-1.51
189430	78531	528.2	556.54	556.53	-0.01		11.68	13.58	1.9
189121	78531	527.74	554.83	555.48	0.65		13.17	13.95	0.78
188680				554.6				12.75	
188605	78531	526.7	553.44				12.69		
188243	78531	526.16	552.82	554.71	1.89		10.46	9.74	-0.72
187766	79485	525.44	552.93	554.26	1.33		7.08	9.88	2.8
187252	79485	524.77	550.51	552.53	2.02		12.24	11.75	-0.49
187159	79485	526.63	550.42	552.5	2.08		11.74	11.24	-0.5
187105	S.E. Military Dr.								
187050	79485	526.47	549.31	550.95	1.64		12.5	12.05	-0.45
186917	79485	526.27	549.15	548.75	-0.4		10.96	15.36	4.4
186657	79485	525.88	547.54	548.05	0.51		10.47	14.77	4.3
186165	79485	525.14	546.95	548.54	1.59		8.71	10.07	1.36

185849	79485	524.94	546.59	548.36	1.77		9.08	9.33	0.25
185533	79485	524.47	546.32	547.54	1.22		6.68	10.34	3.66
185300	79485	524.12	545.53	547.24	1.71		8.48	10.13	1.65
185150	79572	523.9	545.24	547.35	2.11		9.03	8.62	-0.41
185095	Espada Dam								
185075	79572	516.26	543.83	544.84	1.01		8.79	8.55	-0.24
184915	79572	515.89	543.88	544.76	0.88		7.83	8.53	0.7
184723	79572	515.54	543.48	544.69	1.21		8.47	7.94	-0.53
184696	Mission Pkwy L/W Crossing								
184669	79572	514.6	543.18	544.42	1.24		8.44	7.86	-0.58
184414	79572	513.19	542.77	542.79	0.02		8.69	11.35	2.66
184150	79572	514.77	541.51	542.4	0.89		9.98	11.22	1.24
183850	79572	514.29	541.22	541.75	0.53		8.62	11.64	3.02
183527	79572	510.97	541.1	541.26	0.16		6.98	11.56	4.58
183238	79572	511.31	540.76	540.72	-0.04		7.02	11.65	4.63
182939	79572	510.83	540.32	540.4	0.08		7.69	11.13	3.44
182620	79572	512.32	539.73	539.74	0.01		8.48	11.49	3.01
182274	79572	508.97	538.65	539.31	0.66		9.76	11.08	1.32
181984	79572	508.51	537.28	538.43	1.15		10.78	12.01	1.23
181692	79572	508.84	537.42	538.26	0.84		7.08	10.88	3.8
181354	79572	508.3	536.91	537.91	1		6.68	10.43	3.75
181085	79572	509.87	535.59	537.58	1.99		9.3	10.64	1.34
180806	79572	508.17	534.59	536.61	2.02		10.72	11.4	0.68
180478	79572	506.4	534.55	536.18	1.63		8.68	11.24	2.56
180405				536.08				11.16	
180400				535.9				11.61	
180395				535.98				11.24	
180158	79572	504	534.88	536.12	1.24		5.11	9.52	4.41
179974	79572	504	534.71	535.95	1.24		5.45	9.6	4.15
179859	79572	504.2	534.33	535.95	1.62		6.19	8.81	2.62
179786	79572	509.87	533.12	535.86	2.74		10.5	8.69	-1.81
179753	Ashley Road								
179720	79572	509.72	531.08	531.37	0.29		12.57	14.33	1.76
179613	79572	509.5	531.46	531.79	0.33		8.81	11.57	2.76
179588	79572	506.65	531.45	532.39	0.94		8.28	8.11	-0.17
179423	79572	507.25	531.21	531.65	0.44		7.3	10.3	3
179232	79572	506.85	530.62	531.39	0.77		7.8	10.03	2.23
178942	90920	505.25	529.67	530.5	0.83		8.72	11.48	2.76
178613	90920	505.55	528.93	530.48	1.55		8.28	9.77	1.49
178376	90920	502.5	528.68	530.42	1.74		6.53	8.62	2.09
178105	90920	501.38	528.05	530.33	2.28		7.9	7.74	-0.16

177795	90920	498.45	527.65	529.82	2.17		8.03	8.68	0.65
177428	90920	498.44	527.44	528.36	0.92		7.1	11.05	3.95
177300				528.89				6.41	
177091	90920	495.17	526.98	527.67	0.69		7.9	9.53	1.63
176742	90920	495.48	526.54	526.91	0.37		7.7	9.88	2.18
176372	90920	495.16	525.95	526.08	0.13		8.59	10.93	2.34
175923	90920	496.33	526.04	526.5	0.46		6.04	7.24	1.2
175614	90920	493.9	525.69	526.03	0.34		6.67	8.06	1.39
175320	90920	492.49	525.34	525.35	0.01		7.3	9.69	2.39
175022	90920	492.07	525.18	525.13	-0.05		6.71	9.48	2.77
174756	90920	493.7	524.73	524.8	0.07		7.45	9.73	2.28
174478	90920	493.31	523.98	524.57	0.59		8.87	9.55	0.68
174178	90920	490.09	523.64	524.14	0.5		8.82	9.82	1
173842	90920	492.42	523.7	524.24	0.54		7.22	8.28	1.06
173796	90920	492.36	523.04	523.66	0.62		9.05	9.6	0.55
173667	I.H. 410								
173537	90920	492	522.74	523.08	0.34		9.06	9.94	0.88
173465	90920	491.15	522.98	523.33	0.35		7.24	8.34	1.1
173290	90920	489.51	523.06	523.32	0.26		5.49	7.72	2.23
173023	90920	489.89	523.05	523.47	0.42		3.73	5.43	1.7
172827	90920	490	522.9	523.47	0.57		3.8	4.8	1
172780	90920	490	522.85	523.46	0.61		4.04	4.79	0.75
172760	Camino Coahuilteca L/W Crossing								
172740	90920	491.55	522.77	523.46	0.69		4.13	4.48	0.35
172684	90920	491.5	522.74	523.41	0.67		4.25	4.77	0.52
172446	90920	488.5	522.45	523.09	0.64		5.89	6.77	0.88
172150	90920	491.9	522.06	522.8	0.74		6.8	7.18	0.38
171889	90920	491.53	521.68	522.48	0.8		7.48	7.72	0.24
171513	90920	488.2	521.2	521.91	0.71		8.05	8.93	0.88
171124	90920	488.46	520.83	521.69	0.86		7.96	7.91	-0.05
170757	90920	489.95	520.69	521.46	0.77		7.12	6.79	-0.33
170487	90920	489.57	520.45	521.21	0.76		7.35	6.64	-0.71
170079	90920	485.2	520.15	520.71	0.56		7.53	7.29	-0.24
169740	90920	484.97	519.04	519.11	0.07		10.82	11.51	0.69
169380	90920	485.1	518.62	518.62	0		8.4	8.44	0.04
169157	90920	484.68	518.29	518.28	-0.01		7.77	7.82	0.05
169009	90920	484.4	518.24	518.21	-0.03		7.09	6.96	-0.13
168853	90920	484.11	518.07	518.07	0		7.49	7.52	0.03
168684	90920	484.29	517.87	517.85	-0.02		6.96	6.99	0.03
168543	90920	484.03	517.72	517.7	-0.02		6.77	6.83	0.06

168201	90920	483.89	517.68	517.66	-0.02		5.19	5.21	0.02
167914	90920	484.35	517.6	517.58	-0.02		4.89	4.9	0.01
167798	90920	484.13	517.53	517.51	-0.02		4.91	4.92	0.01
167702	90920	483.95	517.51	517.49	-0.02		4.7	4.71	0.01
167399	90920	483.38	517.41	517.41	0		3.74	3.74	0
167099	90920	482.82	517.32	517.32	0		4.24	4.24	0

The NER Plan has been analyzed for changes to the Without Project condition and has shown to generally lower the average flow velocities and computed shear stress values. The effect of these change results in a reduced need of erosion protection and a general improvement in the dominant substrate characteristics in the aquatic environment toward finer sediment sizes. A comparison of the computed shear stress values for the Effective Flow is shown in Table C.1-12. Areas of increased shear stress values are generally located at riffle structures that are designed to resist erosion.

Table C.1-12

River Station	W.S. Elev	Flow	NER Plan Ave. Velocity	Without Project Ave. Velocity	Ave. Velocity Comparison		NER Plan Shear Stress	Without Project Shear Stress	Shear Stress Comparison
	(ft)	(cfs)	(ft/s)	(ft/s)	(ft/s)		(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
213817	610.36	1120	2.88	2.88	0		0.18	0.18	0
213495	610.03	1120	3.42	3.42	0		0.26	0.26	0
213195	609.09	1120	5.75	5.75	0		0.81	0.81	0
212921	609.27	1120	2.4	2.4	0		0.04	0.04	0
212786	609.26	1120	2.31	2.31	0		0.04	0.04	0
212680	609.24	1120	2.27	2.27	0		0.04	0.04	0
212655	Bridge								
212630	609.21	1120	2.25	2.25	0		0.04	0.04	0
212576	609.2	1120	2.27	2.27	0		0.04	0.04	0
212523	609.2	1120	2.27	2.27	0		0.04	0.04	0
212489	Inl Struct								
212377	605.75	1120	3.29	3.11	-0.18		0.05	0.04	-0.01
212259	605.81	1120	2.18	2.1	-0.08		0.02	0.02	0
212198	605.76	1120	2.7	2.61	-0.09		0.03	0.02	-0.01
212161	Lone Star Blvd								
212124	605.38	1120	4.59	4.2	-0.39		0.48	0.4	-0.08
212015	605.21	1120	4.14	3.28	-0.86		0.37	0.22	-0.15
211912	605.19	1120	3.13	3.64	0.51		0.21	0.28	0.07
211633				4.57				0.49	
211628				8.18				1.83	
211623				5.79				0.83	

211428	603.21	1180	3.47	5.4	1.93		0.26	0.68	0.42
211330				4.45				0.45	
211310				7.8				1.57	
211247				4.8				0.53	
211212				4.42				0.44	
211113				4.53				0.47	
211028	599.61	1180	7.52	4.89	-2.63		1.38	0.55	-0.83
210663	599.59	1180	2.89	5.36	2.47		0.18	0.66	0.48
210428	598.09	1180	5.66	5.63	-0.03		0.72	0.75	0.03
210359	598.04	1180	4.61	4.77	0.16		0.47	0.53	0.06
210346.5	Aerial Pipeline Bridge								
210334	597.98	1180	4.29	4.53	0.24		0.4	0.47	0.07
210300	597.74	1180	5.18	5.51	0.33		0.6	0.74	0.14
210247	597.65	1180	4.83	4.85	0.02		0.51	0.55	0.04
210220	597.54	1180	5.11	4.59	-0.52		0.58	0.48	-0.1
210202	CPS Vehicle Bridge								
210184	596.4	1180	6.67	4.28	-2.39		1.03	0.4	-0.63
210113	595.93	1180	7	5.34	-1.66		1.15	0.67	-0.48
209997	596.25	1180	2.58	3.71	1.13		0.14	0.29	0.15
209908	594.37	1180	5.79	3.41	-2.38		0.77	0.24	-0.53
209897	Southern Pacific Railroad								
209886	594.03	1180	6.21	3.36	-2.85		0.89	0.24	-0.65
209867				8.39				1.89	
209848	593.86	1180	6.2	4	-2.2		0.89	0.34	-0.55
209732	593.7	1180	4.66	4.14	-0.52		0.49	0.37	-0.12
209651	593.57	1180	4.39	4.94	0.55		0.43	0.57	0.14
209563	593.45	1180	4.13	3.99	-0.14		0.37	0.36	-0.01
209510	Steves Ave.								
209457	593.22	1180	3.88	3.27	-0.61		0.33	0.23	-0.1
209336	593.2	1180	2.3		-2.3		0.12		-0.12
209266	Inl Struct			3.83				0.34	
209176	591.11	1180	4.21	3.62	-0.59		0.43	0.32	-0.11
208986	590.63	1180	4.19	4.35	0.16		0.44	0.45	0.01
208804	590.54	1180	2.54	3.53	0.99		0.15	0.29	0.14
208735				2.11				0.08	
208730				7.94				1.76	
208725				3.69				0.28	
208414	588.32	1180	4.61	3.79	-0.82		0.46	0.31	-0.15
208250				2.86				0.17	

208245				6.41				1.06	
208240				2.99				0.18	
208128	588.26	1180	2.46	3.84	1.38		0.13	0.31	0.18
207861	585.72	1180	5.64	3.42	-2.22		0.72	0.23	-0.49
207817	585.66	1180	5.19	6.1	0.91		0.58	0.86	0.28
207778	E. Mitchell St.								
207738	585.41	1180	4.79	2.77	-2.02		0.49	0.15	-0.34
207671	585.44	1180	3.18	2.63	-0.55		0.21	0.13	-0.08
207585				1.84				0.06	
207580				8.06				1.8	
207575				3.1				0.19	
207558	585.32	1180	3.4	5.5	2.1		0.24	0.69	0.45
207204	578.72	1180	5.7	4.24	-1.46		0.81	0.39	-0.42
207182				8.81				2.02	
207173				5.67				0.72	
206824	578.86	1180	1.6	5.9	4.3		0.05	0.84	0.79
206815				5.12				0.61	
206810				5.69				0.78	
206805				10.46				2.58	
206595	574.07	1180	6.06	10.33	4.27		0.94	2.51	1.57
206361	573.61	1180	4.14	9.24	5.1		0.39	2.06	1.67
206199	573.05	1180	4.81	3.17	-1.64		0.66	0.24	-0.42
206062	571.83	2320	7.11	3.78	-3.33		1.15	0.28	-0.87
205888	571.9	2320	3.71	5.78	2.07		0.29	0.68	0.39
205698	571.75	2320	3.28	6.87	3.59		0.26	0.98	0.72
205454	569.92	2320	5.11	5.59	0.48		0.54	0.62	0.08
205261	569.61	2320	4.87	6.01	1.14		0.49	0.72	0.23
205210	569.5	2320	5.02	6.59	1.57		0.52	0.89	0.37
205173	E. Theo Ave.								
205136	569.29	2320	5.04	6.36	1.32		0.52	0.83	0.31
205079	569.19	2320	4.96	6	1.04		0.51	0.71	0.2
204930	569.13	2320	3.75	6.24	2.49		0.28	0.78	0.5
204735	569.05	2400	3.04	5.9	2.86		0.18	0.71	0.53
204149	565.08	2400	4.61	5.51	0.9		0.47	0.62	0.15
203916	564.22	2400	5.81	5.44	-0.37		0.81	0.59	-0.22
203644	563.06	2400	6.04	4.26	-1.78		0.84	0.34	-0.5
203277	562.27	2400	4.77	8.52	3.75		0.48	1.64	1.16
202893	561.14	2480	5.65	5.54	-0.11		0.76	0.64	-0.12
202531	560.54	2480	3.96	4.63	0.67		0.34	0.43	0.09
202156	559.95	2480	4.47	4.92	0.45		0.41	0.52	0.11
201849	558.95	2480	6.04	5.08	-0.96		0.87	0.54	-0.33
201465	557.58	2480	5.63	5.01	-0.62		0.71	0.51	-0.2
201090	556.97	2480	4.44	4.88	0.44		0.41	0.5	0.09

200773				4.92				0.55	
200768				8.98				2.06	
200763				7.93				1.47	
200723	556.03	2480	5.58	6.93	1.35		0.71	1.07	0.36
200666	556.03	2480	4.34	5.12	0.78		0.42	0.56	0.14
200630	Mission Road								
200593	555.76	2480	4.82	4.64	-0.18		0.51	0.45	-0.06
200356	555.37	2480	4.52	5.72	1.2		0.42	0.67	0.25
200065	555.03	2480	4.14	5.42	1.28		0.35	0.59	0.24
199779	554.44	2480	4.99	5.22	0.23		0.56	0.53	-0.03
199486	554.02	2520	4.2	5.63	1.43		0.37	0.63	0.26
199181	553.6	2560	4.37	6.49	2.12		0.39	0.87	0.48
198848	552.97	2560	4.88	4.89	0.01		0.53	0.49	-0.04
198530	552.44	2560	4.53	4.87	0.34		0.42	0.51	0.09
198260	552.06	2560	4.48	4.13	-0.35		0.41	0.36	-0.05
197991	551.82	2560	3.94	4.69	0.75		0.31	0.43	0.12
197744	551.52	2560	4.16	4.08	-0.08		0.37	0.34	-0.03
197678	550.73	2560	7.36	3.53	-3.83		1.22	0.25	-0.97
197643	Roosevelt Ave.								
197608	550.38	2560	6.83	3.65	-3.18		1	0.29	-0.71
197526	550.3	2560	5.25	3.49	-1.76		0.59	0.23	-0.36
197319	550.12	2560	4.11	4.46	0.35		0.34	0.45	0.11
197119	549.42	2560	6.07	4.05	-2.02		0.85	0.35	-0.5
196926	549.12	2580	4.62	5.97	1.35		0.45	0.87	0.42
196612	548.69	2580	4.34	5.95	1.61		0.39	0.73	0.34
196378	548.48	2580	3.9	6.02	2.12		0.3	0.77	0.47
196322	548.3	2580	4.77	5.34	0.57		0.24	0.61	0.37
196287	E. Southcross Blvd								
196251	548.22	2580	4.77	4.4	-0.37		0.24	0.37	0.13
196178	548.15	2580	4.67	5.59	0.92		0.45	0.61	0.16
196039	548.02	2600	4.21	5.32	1.11		0.35	0.55	0.2
195663	547.51	2600	4.32	5.28	0.96		0.4	0.57	0.17
195240	546.97	2600	4.18	4.5	0.32		0.35	0.42	0.07
194847	546.83	2600	2.82	5.12	2.3		0.15	0.57	0.42
194500	546.02	2600	4.35	4.06	-0.29		0.38	0.36	-0.02
194442	545.96	2600	4.28	3.34	-0.94		0.37	0.24	-0.13
194407	E. White Ave.								
194371	545.8	2600	4.32	4.78	0.46		0.38	0.5	0.12
194285	545.7	2600	4.29	4.75	0.46		0.37	0.5	0.13
194143	545.3	2600	5.44	5.86	0.42		0.61	0.79	0.18

193617	544.22	2630	5.49	3.67	-1.82		0.61	0.27	-0.34
193207	543.54	2630	4.54	4.4	-0.14		0.45	0.42	-0.03
192847	542.6	2630	5.49	4.74	-0.75		0.65	0.54	-0.11
192613	542.43	2630	3.99	4.55	0.56		0.31	0.45	0.14
192256	542.16	2630	3.75	3.44	-0.31		0.27	0.25	-0.02
191800	541.9	2630	3.43	2.35	-1.08		0.22	0.11	-0.11
191487	541.78	2630	3.02	3.7	0.68		0.17	0.3	0.13
191201	541.39	2630	4.28	2.98	-1.3		0.44	0.18	-0.26
191045	539.98	2630	7.65	8.02	0.37		1.66	1.63	-0.03
190605	536.36	2630	3.91	4.52	0.61		0.3	0.48	0.18
190297	536.13	2630	3.7	4.39	0.69		0.27	0.46	0.19
190011	536.03	2630	2.89	2.01	-0.88		0.16	0.08	-0.08
189952				4.48				0.49	
189951				3.24				0.23	
189860	535.97	2630	2.9	3.32	0.42		0.16	0.24	0.08
189747	535.92	2630	2.87	3.96	1.09		0.15	0.35	0.2
189430	535.8	2630	2.85	4.48	1.63		0.15	0.49	0.34
189121	535.72	2630	2.53	6.2	3.67		0.12	0.92	0.8
188605	535.64	2630	2	5.63	3.63		0.07	0.72	0.65
188243	535.61	2630	1.63	3.71	2.08		0.05	0.3	0.25
187766	535.59	2640	1.19	3.17	1.98		0.02	0.22	0.2
187252	535.57	2640	1.29	1.81	0.52		0.03	0.06	0.03
187159	535.56	2640	1.33	1.67	0.34		0.03	0.05	0.02
187105	S.E. Military Dr.								
187050	535.54	2640	1.35	1.65	0.3		0.03	0.05	0.02
186917	535.53	2640	1.46	1.67	0.21		0.04	0.05	0.01
186657	535.52	2640	1.24	1.39	0.15		0.03	0.03	0
186165	535.51	2640	0.93	1.43	0.5		0.01	0.04	0.03
185849	535.5	2640	0.9	1.58	0.68		0.01	0.04	0.03
185533	535.5	2640	0.83	1.3	0.47		0.01	0.03	0.02
185300	535.49	2640	0.78	1.14	0.36		0.01	0.02	0.01
185150	535.49	2750	0.86	0.85	-0.01		0.01	0.01	0
185095	Espada Dam								
185075	524.37	2750	1.52	1.44	-0.08		0.01	0.04	0.03
184915	524.34	2750	1.9	2.33	0.43		0.06	0.12	0.06
184723	524.24	2750	2.5	2.46	-0.04		0.13	0.12	-0.01
184696	Mission Pkwy L/W Crossing								
184669	519.94	2750	4.97	2.76	-2.21		0.52	0.15	-0.37
184414	519.59	2750	4.04	5.33	1.29		0.39	0.59	0.2
184150	519.15	2750	4.2	4.35	0.15		0.42	0.35	-0.07

183850	518.64	2750	4.22	3.73	-0.49		0.36	0.25	-0.11
183527	518.05	2750	4.25	4.25	0		0.56	0.35	-0.21
183238	517.42	2750	4.4	3.78	-0.62		0.47	0.25	-0.22
182939	517.08	2750	3.83	4.72	0.89		0.25	0.49	0.24
182620	516.57	2750	4.33	4.2	-0.13		0.48	0.38	-0.1
182274	516	2750	3.98	3.47	-0.51		0.39	0.25	-0.14
181984	515.5	2750	4.35	3.88	-0.47		0.46	0.28	-0.18
181692	515.04	2750	4.31	2.75	-1.56		0.37	0.14	-0.23
181354	514.57	2750	3.81	3.05	-0.76		0.39	0.21	-0.18
181085	513.85	2750	5.12	5.43	0.31		0.59	0.69	0.1
180806	513.29	2750	4.62	3.71	-0.91		0.46	0.32	-0.14
180478	513.01	2750	3.62	3.94	0.32		0.26	0.38	0.12
180405				3.75				0.35	
180400				7.32				1.56	
180395				4.26				0.46	
180158	513.11	2750	1.06	3.27	2.21		0.02	0.25	0.23
179974	513.08	2750	1.59	4.83	3.24		0.06	0.59	0.53
179859	513.05	2750	1.68	4.68	3		0.06	0.58	0.52
179786	512.82	2750	3.58	3.84	0.26		0.31	0.36	0.05
179753	Ashley Road								
179720	512.58	2750	3.97	5.98	2.01		0.39	0.98	0.59
179613	511.24	2750	7.17	5.88	-1.29		3.58	0.91	-2.67
179588	511.39	2750	2.48	1.22	-1.26		0.33	0.05	-0.28
179423	511.09	2750	2.86	3.87	1.01		0.49	0.39	-0.1
179232	510.6	2750	2.98	3.83	0.85		0.53	0.39	-0.14
178942	509.93	3200	3	4.37	1.37		0.51	0.49	-0.02
178613	507.44	3200	7.45	5.08	-2.37		3.36	0.68	-2.68
178376	506.59	3200	3.33	4.08	0.75		0.5	0.41	-0.09
178105	505.6	3200	5.57	3.77	-1.8		0.68	0.33	-0.35
177795	505.43	3200	3.56	5.06	1.5		0.24	0.65	0.41
177428	505.42	3200	1.92	11.16	9.24		0.07	2.67	2.6
177300				3.87				0.35	
177091	503.01	3200	4.75	3.52	-1.23		0.45	0.24	-0.21
176742	502.57	3200	4.63	4.56	-0.07		0.42	0.41	-0.01
176372	502.14	3200	4.33	4.13	-0.2		0.38	0.36	-0.02
175923	501.55	3200	4.37	4.41	0.04		0.4	0.41	0.01
175614	500.88	3200	5.37	4.56	-0.81		0.59	0.47	-0.12
175320	500.41	3200	4.99	4.4	-0.59		0.5	0.43	-0.07
175022	500.16	3200	4.14	4.01	-0.13		0.33	0.35	0.02
174756	499.92	3200	4.06	4.87	0.81		0.33	0.51	0.18
174478	499.58	3200	4.37	4.74	0.37		0.38	0.44	0.06
174178	499.36	3200	3.88	4.34	0.46		0.28	0.4	0.12
173842	499	3200	4.32	4.49	0.17		0.37	0.45	0.08
173796	498.83	3200	5.08	2.81	-2.27		0.38	0.18	-0.2

173667	I.H. 410								0
173537	498.44	3200	5.1	4.31	-0.79		0.38	0.42	0.04
173465	498.39	3200	4.72	4.82	0.1		0.44	0.53	0.09
173290	498.44	3200	2.71	4.83	2.12		0.12	0.52	0.4
173023	498.38	3200	2.49	4.14	1.65		0.11	0.34	0.23
172827	498.34	3200	2.38	2.99	0.61		0.1	0.2	0.1
172780	498.32	3200	2.37	2.53	0.16		0.14	0.14	0
172760	Camino Coahuilteca L/W Crossing								
172740	498.21	3200	2.77	2.02	-0.75		0.07	0.08	0.01
172684	497.78	3200	5.27	1.93	-3.34		0.56	0.08	-0.48
172446	497.67	3200	3.83	2.74	-1.09		0.28	0.15	-0.13
172150	497.27	3200	4.56	3.6	-0.96		0.43	0.24	-0.19
171889	496.89	3200	4.59	3.51	-1.08		0.44	0.23	-0.21
171513	496.44	3200	4.39	3.83	-0.56		0.37	0.28	-0.09
171124	496.06	3200	4.23	4.02	-0.21		0.35	0.7	0.35
170757	495.67	3200	4.26	4.1	-0.16		0.37	0.76	0.39
170487	495.32	3200	4.4	3.87	-0.53		0.4	0.68	0.28
170079	494.95	3200	4.05	4.18	0.13		0.3	0.79	0.49
169740	494.5	3200	4.86	5.63	0.77		0.48	1.47	0.99
169380	494.21	3200	3.28	3.52	0.24		0.49	0.57	0.08
169157	493.85	3200	3.69	3.92	0.23		0.65	0.76	0.11
169009	493.75	3200	2.67	2.74	0.07		0.32	0.36	0.04
168853	493.62	3200	2.81	3.04	0.23		0.35	0.43	0.08
168684	493.46	3200	2.66	2.78	0.12		0.36	0.4	0.04
168543	493.41	3200	1.73	1.83	0.1		0.14	0.16	0.02
168201	493.27	3200	1.86	1.89	0.03		0.17	0.18	0.01
167914	492.98	3200	3.06	3.06	0		0.48	0.48	0
167798	492.81	3200	3.15	3.15	0		0.49	0.49	0
167702	492.65	3200	2.86	2.86	0		0.44	0.44	0
167399	491.99	3200	3.59	3.59	0		0.63	0.63	0
167099	491.51	3200	3.17	3.17	0		0.5	0.5	0
River Station	W.S. Elev	Flow	NER Plan	Without Project	Ave. Velocity		NER Plan	Without Project	Shear Stress

Erosion Control. There are two design conditions that need to be addressed, as far as erosion control is concerned. The first is the "newly constructed" condition, when the plant materials, are not fully established. The second is the mature condition after a few years. The condition that is the most susceptible to soil erosion on a widespread basis is the "newly constructed" condition. Therefore, this condition forms the design parameters for where erosion-limited measures are used.

A shear stress analysis was performed. Computation of shear stress is way to a measure of the erosive forces in the flow subjected to the bed material in a channel or floodplain and is a factor of flow volume and velocity, bed material, and the vegetative cover. Generally it was found that, in the fully developed condition, the channel velocities are significantly reduced from the existing conditions for the 100-year design storm. For the Effective Flow, the pilot channel velocity and shear stresses were found to be within a generally acceptable range. The most important concern is for conditions within the project limits outside the pilot channel if a flood occurs immediately after the project is completed and before the vegetation can become established. Erosion protection has been designed to address this condition.

Making some slight alterations to the design model, a shear stress analysis of the proposed channel in newly finished condition was performed. The hydraulic model for each design condition marks the top-of-bank locations for each cross-section at the top of the floodway channel. This allows a direct comparison to the without project condition model for the 100-year design storm event. For the shear stress analysis, the top-of-bank locations were moved to the top of the pilot channel. This allows the model to calculate the pilot channel and areas outside the pilot channel (but within the floodway channel) shear stresses separately.

As a conservative approach, another modification was made to the proposed channel “n” or roughness coefficients. All values outside the pilot channel but inside the proposed limits of construction were changed to 0.03. This value represents a “bare earth” type condition. The analysis used these revised conditions for a variety of design storm events between the effective flow and the 100-year event. The events used included the 1-year, 5-year, 10-year and 25-year design storms. The designs were based on trends related to this range of storm events.

The results of this analysis were utilized to determine erosion control measures that were incorporated into the design. Three categories were used to gauge the need. Area with shear stresses less than or equal to 0.5 pounds per square feet (lb/sf) are considered safe from significant erosion without additional protection. Reaches labeled “low” do not need anything more than temporary erosion protection from localized impacts. Reaches labeled “medium” evidenced shear stresses ranging between 0.5 lb/sf and 1.0 lb/sf. Temporary protection until vegetation is established is was considered required for within-floodway overbank areas in this range. The third range was labeled “high” because they exhibited shear stress values exceeding 1.0 lb/sf. Virtually all of the pilot channel values in storms exceeding the effective flow fell into this category. However, that was expected and the pilot channel design will reflect various methodologies for permanent erosion protection.

There are a number of locations where the within-floodway overbank areas exhibit high shear stresses. These are usually associated with riffle structure slopes, weirs, or drop sections already identified for extra protection. Permanent protection is included in the design for these areas. It is known that mature, established vegetation can withstand much higher shear. In fact, the existing channel consists of only Bermuda grass above the pilot channel and yet exhibits very few signs of erosion problems. However, the proposed river improvements will introduce many elements that will contribute to added turbulence. These include the added riffles and dams, the ever-changing vegetation, the sinuosity of the pilot channel and the varying side slopes of the main channel. Past experiences of the design team have shown that it is important to protect the constructed improvements until the new vegetation can establish itself. Any flood event that exceeds the capacity of the pilot channel can potentially damage the plantings and land shaping. Native grasses will not cover to the extent that Bermuda does and will take longer to fully establish. A more permanent protection is recommended for the higher shear areas because it may take 5 or more years to fully establish the desired natural protection.

For the purposes of preliminary design, the shear stresses were divided into 3 categories:

Low (< 0.5 psf shear) - temporary erosion control blanket
Med (0.5 psf < X < 1.0 psf shear) - erosion control mat and planting
High (> 1.0 psf shear) - permanent erosion control mat

The medium and high categories may be addressed by a variety of different measures. For the purposes of the Preliminary Design cost estimate, one measure was used for each category. Those measures are listed above. Other measures may be considered during later design phases. The later design phases will include a detailed 2d SMS model which will yield more data on shear as well as turbulence at a variety of storm flows.

A key consideration is to be conservative in the approach, and to assume that the "newly constructed" condition will be subject to storm flows. The shear stress analysis was performed on the entire floodway channel and addresses the 100-year event.

"Temporary erosion control blanket" is similar to Curlex, is typically biodegradable, and is designed to hold soil on a temporary basis until the groundcover vegetation takes root. It is typically found on slopes greater than 3:1 where there is not a substantial amount of sheet flow or concentrated flow across the soil.

"Erosion control mat and planting" is similar to Contech TRM C-35, is a permanent mat, and will be resident below the soil surface.

"Permanent erosion control mat" is a permanent mat, will be resident just below the soil surface, and is similar to Contech Pyramat.

Quantities were derived by examining the shear in selected cross-sections and by translating that to plan view, and calculating the area of each shear stress range. A summary spreadsheet of those totals is shown in Table C.1-7, and it is broken out by construction phases and by the sub-reaches used in the IWR-PLAN cost analysis.

	Channel (acres)		Overbanks (acres)		Outside limits
	Medium	High	Medium	High	
Sta. 170079 to Sta. 173796	13.59				
Sta. 173796 to Sta. 177378		12.13	34.02		
Sta. 177378 to Sta. 179786		8.79	1.52	3.39	
Sta. 179786 to Sta. 180158	5.87				
Sta. 180158 to Sta. 185075		19.11		43.47	
Sta. 185075 to Sta. 186657	8.83				
Sta. 186657 to Sta. 189121		12.45	18.58		
Sta. 189121 to Sta. 194371		16.12		45.56	
Sta. 194442 to Sta. 196251		6.17	12.02	6.84	
Sta. 196251 to Sta. 196926		3.33			
Sta. 196926 to Sta. 197744		4.14		3.40	
Sta. 197744 to Sta. 200953		8.57	18.93	1.81	
Sta. 200953 to Sta. 205888		22.43		29.19	
Sta. 205888 to Sta. 207204		4.31			
Sta. 207204 to Sta. 209563		4.20		11.84	
Sta. 209563 to Sta. 210593		1.86		2.70	
Sta. 210593 to Sta. 212124		3.28	9.12		
Total (Acres)	28.29	126.89	94.19	148.2	13.46
Phase I	8.83	21.07	18.58	21.94	4.21
Phase II		13.35	9.12	14.54	0.26
Phase III		53.85	30.95	69.94	2.12
Phase IV	19.46	38.63	35.54	41.79	6.87
	28.29	126.9	94.19	148.21	13.46