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Executive Summary

This draft technical memorandum provides a summary of findings to support preliminary design of the San Antonio River Improvements Project. This document mainly focuses on the geomorphic assessment and sediment transport analyses that are supported by historic channel assessment and hydrology & hydraulics sections. The final section of this memo provides preliminary design criteria for the South Reach of the San Antonio River.

The geomorphic assessment summarizes existing watershed and channel conditions from the basin's headwaters to below the Interstate (I) 410 Bridge. The heavily urbanized basin creates a rapid runoff response in a region that is known for intense storms. Basin hydrology is characterized as a bimodal, flashy ephemeral system superimposed on a spring-fed base flow condition. The watershed's transformation to an urban character appears to have had a typical effect on sediment supply conditions where sediment is released by construction activity as the watershed becomes developed and then supply is reduced as a built out condition is approached. Channel conditions reflect the basin's urbanized hydrologic response and sediment supply characteristics. Channel incision (vertical erosion), and reduced sediment supply, is observed throughout the river despite concerted channel armoring efforts.

The historic channel assessment compares historic channel conditions to existing channel conditions to assess the tendency for future adjustments in channel planform, slope, and cross section geometry. Historically the San Antonio River was a dynamic system with a natural sediment supply and frequent flooding over a much wider floodplain. The historic river channel was wider, shallower, gentler sloped, and more sinuous than it is today. Flood control efforts have straightened the river, increased its gradient, and confined flood flows to a relatively narrow floodway allowing urbanization to encroach upon the river's historic floodplain. For the North reach, this included construction of meander bend cutoffs in the late 1920's. For the South reach the construction of the floodway severely manipulated the channel planform by straightening virtually nine continuous miles of river. The geomorphic impact of these works has resulted in increased channel instability and degraded habitat, while conveying flood flows more rapidly downstream. The historic channel assessment also compares floodway as-built cross sections available in the South reach to recently surveyed cross sections. The cross sectional comparison documents that pilot channel erosion (incision) has occurred while deposition is evident in overbank areas within the floodway. This section estimates volumes associated with channel erosion and floodway deposition since the floodway was constructed. Profile comparisons illustrate that channel slope has been reduced over time and is interpreted as the river's natural response to alterations of the channel and watershed in an attempt to reach an equilibrium condition.

Much of the hydrologic and hydraulic data used in this study were derived from models developed for federal flood insurance purposes to estimate flood flows and corresponding water levels throughout the San Antonio River basin. In addition to these data, supplementary information was obtained from USGS gaging stations in the area. For the North Reach the most significant features

that impact flood hydrology are Olmos Dam constructed in 1926, and the San Antonio Tunnel constructed in 1995. For the South Reach the most significant influences are the return flow from the San Antonio Tunnel, contribution of flow from San Pedro Creek, and the constructed floodway. This section develops reach average hydraulics for use in the sediment transport analysis. In addition to flood flow hydrology, flow duration and percent exceedance probabilities were developed for instantaneous flows to support the sediment transport analysis in determination of effective discharge. Base flow conditions were also evaluated to aid in developing design criteria for the base flow channel.

The sediment transport analysis was performed to assess current channel stability and to provide information for design of the proposed channel and stabilization structures. This section provides an effective discharge that will be used to size the pilot channel. The effective discharge is a channel forming flow resulting from a moderate flood. The effective discharge recognizes that minor floods do not carry enough sediment to significantly alter channel form, while large floods, that carry substantial sediment loads, occur too infrequently to be a dominant channel forming event. The analysis provides approximate average channel dimensions and energy gradients (width, depth and slope associated with the effective discharge) that would achieve a condition of sediment continuity based on the assumptions of alluvial channel theory and sediment supply from the contributing watershed. The sediment budget used to estimate stable channel dimensions was based on observed historical channel erosion and sediment transport rates for existing conditions. There are uncertainties associated with the stable channel slopes and dimensions due to the limited data available. Measuring suspended load and bed load concentrations through an extensive range of flows would reduce uncertainties. As urbanization affects future hydrology and sediment load to the system, modification to the stable channel geometry estimates will be required.

The design criteria section refines the SWA Group's Design Guidelines based on the geomorphic and sediment transport analyses. Design criteria refinements cover flood conveyance, base (minimum) flows, water quality, sediment transport, channel geometry, channel alignment, stormwater outfalls, spill structures, bank stabilization and reconstruction materials, fish and wildlife habitat, geomorphic function, floodway vegetation, construction, maintenance and monitoring. While efforts have been made to follow the Design Guidelines regarding alignment and channel geometry, changes have occurred based on a more thorough analysis. The design criteria have incorporated elements from a variety of investigations and analyses to promote the development of a multi-objective project to meet diverse design goals while considering site constraints

Geomorphic Assessment

This geomorphic assessment provides a reach-by-reach inventory of channel conditions for the San Antonio River Basin from its headwaters to below the Interstate 410 crossing. Geomorphic maps provided in Appendix A supplement information in this section.

San Antonio River Basin

The headwaters of the San Antonio River are located in Bexar County in south-central Texas, a heavily urbanized watershed. The City of San Antonio grew at a rate of 22% between 1990 and 2000, and by population is ranked the ninth largest city in the United States. In addition to the influences of urbanization, the local geology of the San Antonio River Basin has a tremendous impact on surface water hydrology. Typically, the surface geology of the upper basin consists of Cretaceous limestone overlain by a very thin soil cap. As a result, the runoff is naturally an extremely flashy system. Peak flood flows are driven by large rainstorm events that occur most frequently in the spring and fall months. Drainages north and west of San Antonio are largely ephemeral washes that have little to no infiltration capacity. These streams flow southward through the "recharge zone", the fractured limestone fault zones on the southern edge of the Edwards Plateau. The headwater springs of the San Antonio River are located on the down gradient (southern) edge of the recharge zone where conditions are artesian. In general, the hydrology may be expressed as bimodal, a flashy ephemeral system superimposed on a spring-fed base flow condition.

As the basin drainages that originate in the hill country head southward onto the relatively flat limestone benches, they have historically dumped their sediment loads due to a reduced slope, forming broad fluvial terrace deposits during the early Holocene epoch (10,000 years bp). These terraces, first visible through Brackenridge Park, naturally confine the present San Antonio River channel in the upper project reaches. The river channel accessibility to these terrace deposits provides a natural sediment source for the Museum North project reach. The San Antonio River flows southward towards the city's downtown area, and the cumulative effects of urbanization, flood control projects, and encroachment on the historic floodplain confine the active channel to a well entrenched narrow floodway corridor with both relatively natural alluvial banks, riprap segments, and concrete-lined, channelized segments in the Downtown Riverwalk section of the city.

The Mission South project reach of the San Antonio River begins on the southern end of the metropolis and extends about nine miles within an Army Corps of Engineers straightened, trapezoidal floodway and pilot channel whose construction began in the late 1950's through the 1970's. The floodway width ranges between 300 to 500 feet and was excavated into the fluvial Holocene terrace deposits that extend from the toe of Edwards Plateau hill country to the south along the major drainage corridors of the Basin. Below the Mission South project reach, the San Antonio River returns to meandering channel configuration with access to a local sediment supply derived from the historic terrace deposits.

Change in Hydrology and Sediment Supply

Urbanization and the development of storm drainage systems results in increased quantity of runoff, more efficient conveyance of storm run-off to receiving channels, and thus, higher magnitude and shorter duration floods. The more frequent and greater magnitude floods associated with urbanization generally cause channel incision and cross sectional enlargement. Urban development can be represented as a two-phase process; a large initial increase in sediment input when bare soil is exposed to runoff on construction sites and channel sections enlarge, followed by a decline in sediment supply and increase in flood magnitude as an increase in impervious surfaces decrease soil infiltration rates (Knighton, 1984).

Urbanization of a watershed causes fundamental changes in watershed hydrology. An increase in impervious surfaces in the watershed, such as roads, roofs, and parking lots, reduces infiltration rates. More rapid movement and delivery of storm runoff from uplands to the active channel through storm sewer drainages, gutters, and pipes, creates a "flashier" system, increasing flood hazard and producing channel and bank instability. An increase in magnitude and frequency of floods adds more energy to the system, which is exemplified by higher shear stress values and changes in channel geometry and bed slope. With a rapid increase in stream power or energy, a channel bed that is not held in place by bedrock, immobile bed material, or grade control structures is likely to initially reduce its slope through bed downcutting. Banks become more susceptible to mass erosion and failure as bed downcutting increases local bank heights, and channels attempt to laterally migrate to achieve a wider and larger channel area capacity.

The hydrologic regime of the San Antonio River is controlled by rain storm-driven events. These systems are considered "flashy" where a steep rising and falling limb of the hydrograph is typical. When the water surface in the channel or stage drops at a much faster rate than saturated stream banks can drain, the water in the channel no longer supports the saturated, upper banks, and the banks become prone to failure. In this environment, mass wasting and bank failure is often a result of rapid water surface level drawdowns and is exacerbated by the increase in bank height due to incision.

The concentration of runoff from urbanization also causes large changes in erosion and sediment delivery to the stream channel. Typically, high sediment yields during the construction phase are followed by reduced yields once infrastructure and storm sewer systems are fully built and channel sections have been enlarged by incision and bank failure (Kondolf and Keller, 1991). As outlined by Leopold et al. (1964), the channel dimensions and geometry of stream channels are dictated by the sediment load and runoff regimes of the particular drainage. Changes to these variables associated with urbanization usually induce channel downcutting and widening.

As sediment yields from surficial sources decline and discharges increase, high rates of bank erosion and instability can continue. Channel instability is often influenced by human activities that directly alter the channel bed, banks, or floodplain. The historic morphological response and human intervention of the San Antonio River follows this trend. Decades of channel modification (channelization, straightening, widening), maintenance, sediment removal, and dumping riprap on eroding channel bank margins have been used to control the river and its flooding potential.

As the hydrology of the San Antonio River Basin changes with urbanization, the Holocene derived fluvial deposits became much more underfit relative to the escalating dominate discharge regime. As a result, the balance between the resistive forces of the naturally derived geology and the erosional forces of flowing water are out of sync in favor of the hydrology. Human derived changes in hydrologic regime need concurrent changes in fluvial geology to restore equilibrium.

San Pedro Creek Watershed

Major sub-watersheds in the upper basin of the San Antonio River were identified and evaluated by project team members in spring 2002. These include the following drainages: Olmos Basin, San Pedro Creek and its tributaries: Apache Creek, Zarzamora Creek, Alazan Creek, and Martinez Creek. Brief descriptions of these sub-watersheds of the San Antonio River Basin do not represent a comprehensive inventory or understanding of the dynamics and impacts from the respective drainages. Instead, a 'windshield' survey and brief field visit of several tributary channel and floodplain segments were conducted to determine their relative influence on the San Antonio River project reach with regards to hydrology, sediment supply, transport, and deposition.

San Pedro Creek is an Army Corps of Engineers (ACOE) concrete-lined and heavily riprap trapezoidal flood control channel for most its length throughout the greater San Antonio metropolis area. The San Pedro Creek contribution of flow to the San Antonio River is significant. The discharge of the San Pedro is approximately 2 times that of the San Antonio River at their confluence. This twofold increase in flows, and thus, substantial increase in river power has a tremendous influence on the sediment transport and geomorphic characteristics of the San Antonio River downstream through the lower end of the Mission South project reach.

Apache Creek converges with Alazan Creek approximately 2,000 feet upstream of its confluence with San Pedro Creek. Apache Creek is a mostly concrete-lined ACOE channel that provides no significant local sediment supply from the upper San Pedro Creek watershed. Zarzemora Creek is located within the uppermost portion of the Apache Creek watershed that drains the southern end of the hill country west of the city center and flows into Elmendorf Lake, southeast of Rosedale Park. Active channel incision and widening was observed along segments of Zarzemora Creek upstream of the lake. A thin layer of topsoil overlying coarse gravel alluvial deposits overlying limestone bedrock characterize a typical bank cross section on this tributary (Photo G.1). Elmendorf Lake acts as a flood control and flood detention facility and recreational water body. The lake also acts as a sediment sink and likely inhibits significant sediment loading into San Pedro Creek.

Alazan Creek drains north of Apache Creek in a southeast direction. Approximately one mile north of Elmendorf Lake is Woodlawn Lake, which impounds the uppermost reaches of Alazan Creek. A tributary, Maritinez Creek, flows north to south and parallels I-10 just west of downtown for much of its length before its converges with Alazan Creek. At the confluence, substantial concrete riprap along bank margins and a high sediment supply were observed on both drainage corridors (Photo G.2).

Photo G.1 Zarzamora Creek downstream from the Ingram Road Bridge shows active bank and bed instability, bed incision, and channel widening.

Photo G.2 Alazan Creek (left) and Martinez Creek confluence approximately 1.5 miles north of the San Pedro Creek.

Upper Olmos Creek Basin

The Olmos Creek Basin extends to the northwest from its confluence with the headwaters of the San Antonio River at the University of Incarnate Word. The upper Olmos Creek basin watershed is heavily urbanized with residential and commercial development throughout the watershed. The uppermost channel reaches of Olmos Creek are ephemeral.

Rock quarries are prevalent in the upper Olmos Creek Basin near Huebner Road, which may provide a large sediment supply from adjacent uplands and disturbed, eroding terraces. A thin (6-inch) dark topsoil overlies an 18-inch gravelly layer, followed by a more cohesive and homogeneous clayey (15 feet thick) layer. The channel bed material is composed of both large limestone 1 to 5- foot boulders, riprap, and sub-angular to angular gravels, and sand. Exposed, vertical, 10 to 15- foot, non-vegetated banks suggest that highly erosive flow characteristics withmass failure during rainstorm events and undercutting of the toe of the banks are active processes.

Little consideration has been given towards adjacent watershed impacts within the surrounding drainage system. Sediment supplies from overland flow, channel headcutting, and storm drainage systems should be evaluated further. Historic and active channel incision is present upstream from Huebner Rd. Short bank segments adjacent to a quarry have been laid back and haphazardly armored with concrete grout. Sufficient flow volumes to transport and deposit substantial sediment farther downstream are either infrequent or not a concern. However, episodic rainstorm events likely cause considerable erosion and sedimentation problems. Establishment of a minimum buffer zone for development or rock quarry and gravel mining adjacent to the channel should be considered. Changes in floodplain width from reach to reach is high, and therefore delineation of a non-developed corridor by sub-reach is recommended.

Farther downstream at Dreamland Dr., Olmos Creek remains ephemeral. About 500 feet above Dreamland Dr., the channel splits. The west branch is an engineered, trapezoidal, 200 feet wide floodway with grass-lined floodplain and a narrow fringe of trees and shrubs on the outer limits. The north branch also resembles a constructed floodway but appears narrower. The Texas & New Orleans railroad runs parallel to the west branch on the left bank, its embankment effectively cutting off the historic floodplain between the two channels. The railroad bridge is constructed on an exposed layer of limestone bedrock. Below Dreamland Dr., the channel is less well-defined with a high width to depth ratio. The channel bed is composed of bedrock. The banks are poorly defined moderately sloped 1 to 2- feet with soil composed of dark silty clay and angular gravel lenses. A dense canopy of oak trees with an under story of grass and herbaceous ground cover characterize the plant types here.

Photo G.3 An ephemeral segment of Olmos Creek adjacent to active gravel mines and quarries upstream of Huebner Road north of San Antonio.

Approximately two miles downstream at the Jackson –Keller Road Bridge, Olmos Creek is a large concrete-lined channel with a top width of about 80 feet. Less than 1 cfs was observed flowing in the channel. A veneer of gravel and sand deposits is found along the bed.

Farther downstream along the Olmos Basin golf course, perennial flows were observed in the channel (<5 cfs). The channel is trapezoidal, fairly uniform in shape with cohesive clayey banks that support predominantly annual grasses and weeds. The golf course creek corridor is planted with mature deciduous trees. The base of the trees show substantial flood scour while flood debris and trash was observed in branches over 15 feet above the channel bed. A series of concrete bridges with six 24- inch culverts per bridge gives golfers cart access to either side of the floodplain. Plugs of large gravels to sand-sized material are found on the upper and lower end of the bridges. Several of the culverts are plugged with sediment. Towards the bottom of the golf course, channel maintenance, and clearing/cutting woody plants is evident, however, crews have failed to remove woody materials from the channel. The creek flows through the Olmos Basin Park after flowing under the railroad bridge and McAllister Freeway (I-281).

Olmos Park to Olmos Dam

Olmos Park is a large outdoor recreational field facility surrounded by deciduous forest that likely stores most incoming sediment. The active channel is a trapezoid ditch with deposits of sand and gravel overlying a uniform flat channel bed. Banks are composed of fine silt and clay, well vegetated by herbaceous plant cover and deciduous trees. Evidence of spoil piles and recent levee construction along banks, and temporary maintenance roads, indicate routine dredging and removal of excessive sediment occurs near the dam. As a result of Olmos Dam and the surrounding forested wetland complex, it is likely that base flows transport insignificant volumes of suspended or bed load sediment downstream. However, after the October 1998 record flood, large volumes of sediment were transported downstream from Olmos Dam. Indeed, Olmos Creek below Olmos Dam is characterized by low flow velocities at base flow, a relatively narrow, deep channel (2 to 4 ft) with substantial sediment deposited on the channel bed. Banks are well vegetated with evidence of recent aggradation, including mid-channel island formation and vertical bank accretion.

Photo G.4 Looking upstream at Olmos Creek approximately 200 feet above the Olmos Dam structure and outlet.

Photo G.5 Looking downstream at the Olmos Dam and outlet structure.

San Antonio River Project Reach Delineation

Based on similar geomorphic and hydraulic characteristics, the Museum North and Mission South project reaches were delineated into smaller sub-reaches as shown in Figures G.1 and G.2. A summary of project sub-reaches, their Hec-Ras model station numbers, and relative lengths in feet are presented in Table G.1. A set of geomorphic maps from spring 2002 field mapping and inventory efforts are included in Appendix A.

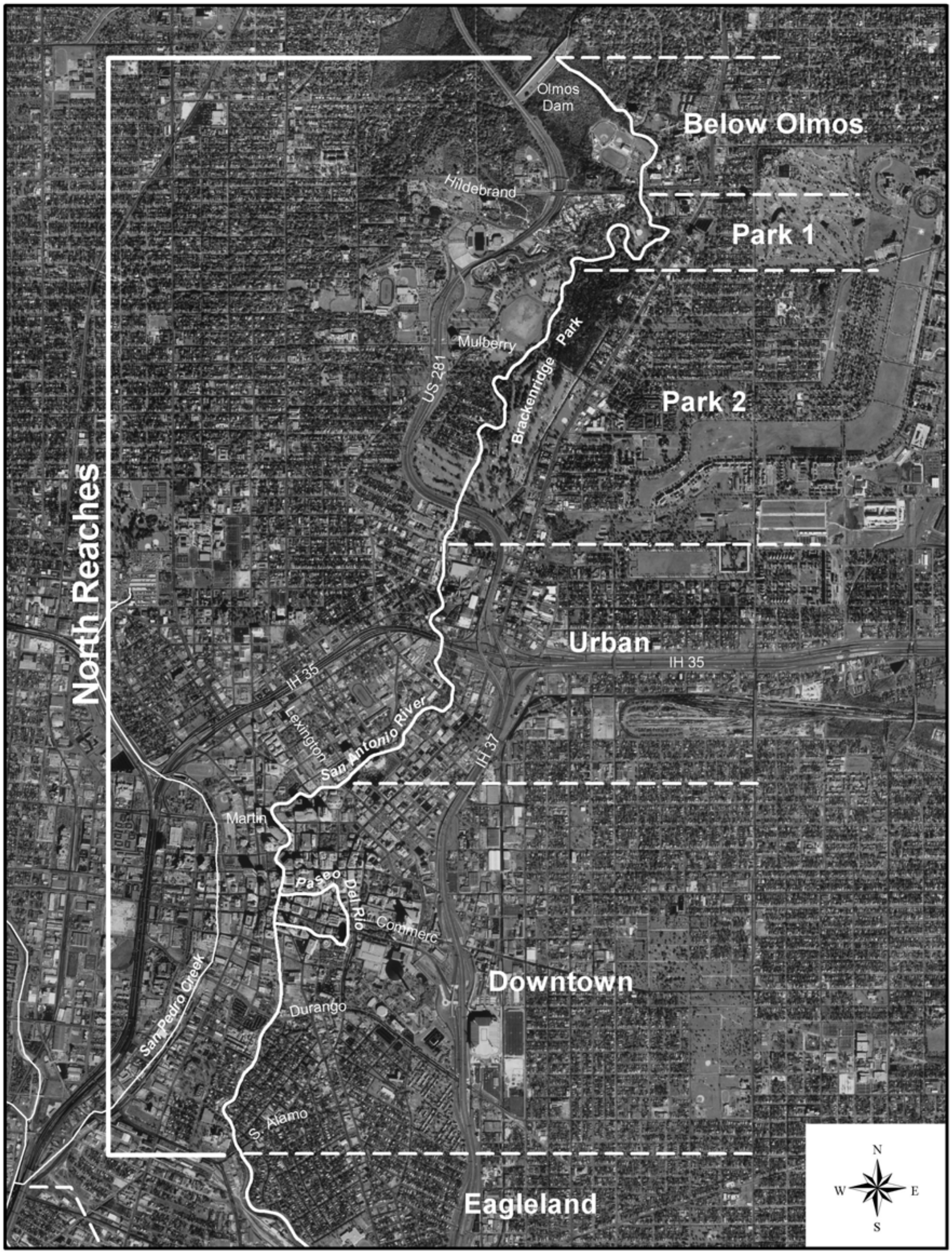

Figure G.1 Delineation of sub-reaches for the North Reach

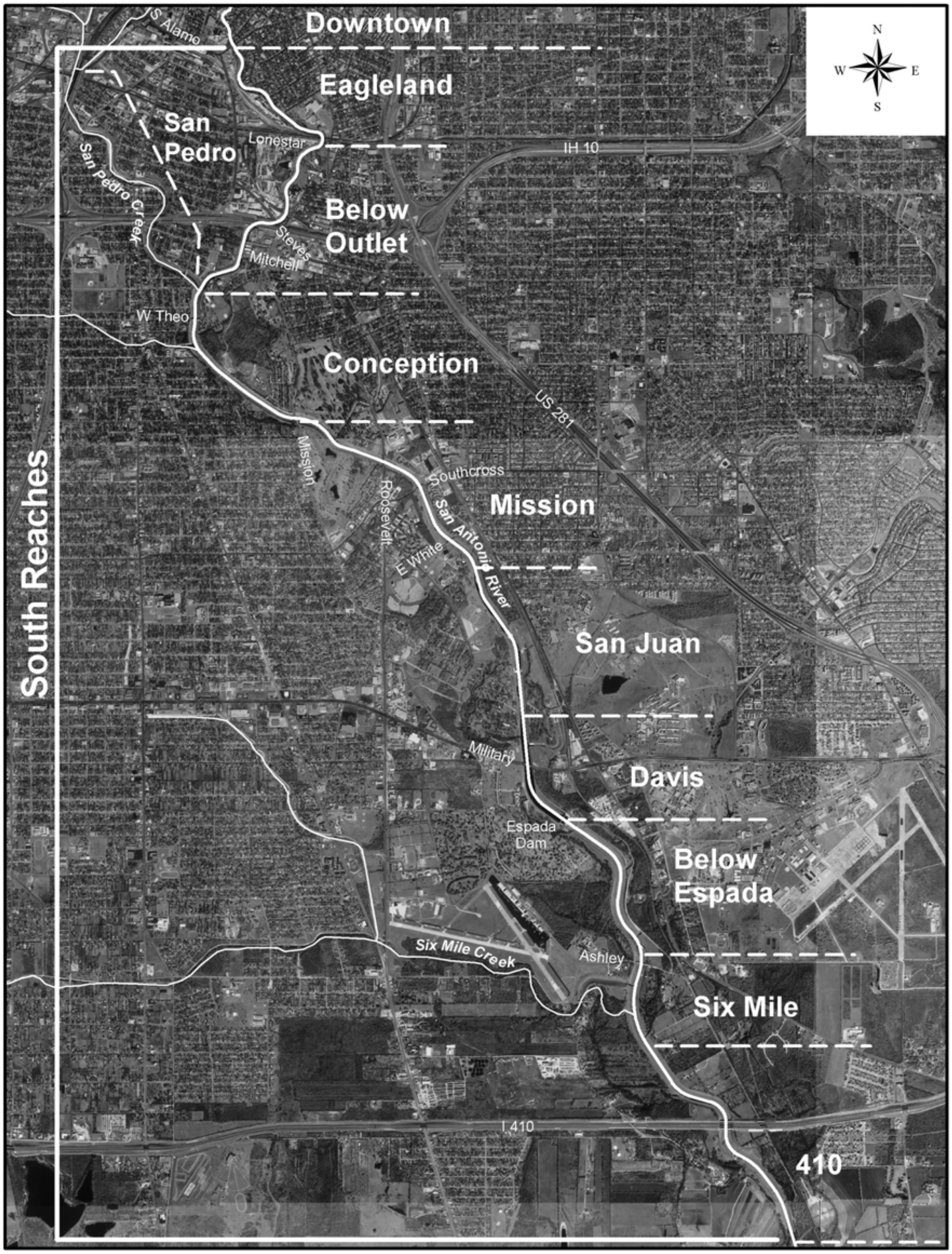

Figure G.2 Delineation of sub-reaches for the South Reach.

Table G.1. San Antonio River Project Sub-reach Delineation

Subreach	Location	Hec-Ras Sta. No.	Reach Length (ft)
OLMOS	Top of Olmos Basin	3188+00	69121
BELOW OLMOS	Olmos Dam	2496+79	3859
PARK 1	Hildebrand Ave	2458+20	4724
PARK 2	Low water crossing at Zoo	2410+96	7368
URBAN	Tunnel Inlet	2337+28	6907
DOWNTOWN	Low Head Dam U/S of Lexington Street	2268+21	9815
EAGLELAND	S. Alamo Street (Gate 6)	2170+06	4482
BELOW OUTLET	Tunnel outlet	2125+24	6474
CONCEPTION	San Pedro Creek confluence	2060+50	5327
MISSION	Grade Control Structure u/s Mission Road	2007+23	7516
SAN JUAN	Begin Sedimentation from San Juan Diversion	1932+07	4527
DAVIS	Begin Davis Lake	1886+80	3530
BELOW ESPADA	Espada Dam	1851+50	4672
SIX MILE	Old Meander Diversion inlet/Ashley Road	1804+78	3050
410	Old Meander Diversion outlet	1774+28	7688
BELOW PROJECT	End project reach	1697+40	8823
	End of Model	1609+17	

Below Olmos Dam

At the confluence with the San Antonio River and Olmos Creek at 200 Patterson Condominiums (Sta.2476+00), over 5 feet of channel aggradation was observed by local residents and employees after the 1998 flood. Where Olmos Creek meets the San Antonio River, a large mid-channel gravel bar deposit has formed, a likely remnant 1998 flood deposit derived from Olmos Creek (Photo G.6). Adjacent banks are well vegetated with woody species along non-developed segments. A condominium apartment complex maintains a turf grass and concrete-bag revetment wall along the channel margin on the left bank upstream from the converging drainages. Farther upstream, contributing flows from spring-fed PVC pipe outlets are frequent. Well-vegetated mid-channel bars and vegetated islands are also abundant, forming split flows having an anastomosed channel pattern. Infrastructure including pedestrian asphalt pathways, storm outlets, floodwalls, and bridges laterally constrains the channel to a narrow floodplain corridor (< 80 feet).

Park 1

The San Antonio River flows southward from E. Hildebrand Avenue, the beginning of the Museum North project reach. Immediately downstream is a lagoon pond complex adjacent to the active channel. An acequia, and irrigation channel constructed during the period of Spanish Colonial Mission settlement, is located on the right historic floodplain and flows in the opposite direction (northward) as the river. Through this reach, mason WPA limestone rock walls confine the low flow channel. These vertical mortared walls form the active channel margin. Local instability and wall failure is common (Photo G.7). Deposits of fine sediment and organics form inset point and side bars with a maximum height of about 4 feet from the channel bed. The low flow channel meanders laterally from one wall boundary to the other (Photo G.8).

Construction and earthwork associated with pedestrian pathways and bridges, masonry, piped irrigation systems, and associated bank disturbance have impacted this reach. Construction materials are susceptible to entrainment by flood flows given a large rainstorm event. Eroding banks and concrete walls indicate local instability through this reach. Past bank stabilization efforts include about 200 feet of rock gabion and 300 feet of geocells (Sta. 2452+00-2447+00)) on the left bank.

Bank instability is evident on the right margin (Sta. 2448+50). Mason limestone rock WPA wall failures are common. Left channel wall margin (Sta. 2443+00-2441+00) needs repair and/or replacement. Additional failures include 2426+80 on the right wall, and 2412+00 on the left wall. Substantial impaired channel segments and/or sites through the Museum North project reach are summarized in Table G.2.

Near Sta. 2446+00, alluvial banks end, and concrete mason walls begin on both channel margins downstream to Sta. 2408+00. The low flow channel is laterally confined by the walls and held vertically stable by a concrete sill grade control structures at Sta. 2429+00.

Photo G.6 As a result of poor sediment conveyance and aggradation, a large mid-channel bar has formed at the confluence of Olmos Creek and the headwaters springs of the San Antonio River.

Photo G.7 WPA wall failure along the left channel margin through the Park 1 sub-reach.

Photo G.8 Base flow of San Antonio River channel with side and point bar deposits of organics and fine material confined by vertical wall margins.

Table G.2. List of impaired channel segments on the San Antonio River, Museum North Reach.

Sub-Reach	HEC-RAS Station	Location/ Length (ft)	Impairment Type/Description
Below Olmos	247400- 700	CH/RB/300	aggradation, sediment sink, mid-channel bar/ poorly defined channel
Park 1	244850	RB/100	severe bank erosion 10 ft VB
Park 1	244300-244200	LB/ 100	WPA wall failure
Park 1	242600	RB/20	WPA wall failure
Park 1	241200	LB/20	WPA wall failure
Park 1	241000	CH/LB/RB	Gate 2 and low water ford, walled island, high hydraulic forces
Park 2	240600-235900	LB/700	denuded, bare banks and floodplain, high human traffic/use
Park 2	240400	RB/100	bank erosion 12 ft-VB terrace
Park 2	238000	LB/50	outer meander bend, severe bank erosion 10 ft VB
Park 2	235700	LB/100	moderate bank erosion, sediment source, 6ft VB
Park 2	235400	LB/ 50	moderate bank erosion, sediment source, 6ft VB
Urban	232100	LB/100	moderate bank erosion, 4 ft VB d/s Pearl Brewery
Urban	231900	RB	failed storm sewer outlet pipe 2 ft diam.
Urban	231100	RB/50	bank erosion 8 ft-VB fine sediment source
Urban	230850	RB/50	Thirteenth Street storm pipe outfall and concrete apron (local erosion)
Urban	229900	LB	asphalt riprap segment

LB= Left Bank
RB=Right Bank
CH= Channel
VB= near-vertical bank slope

The low flow channel through Park 1 is 15-20 feet wide with inset side and lateral point bar deposits (2 to 4 feet in depth) of fine sand silt, clay, and high in organic matter. Gravels and cobbles were observed under the fine material on the channel bed. Some minor segments of the floodwalls have failed and need maintenance and repair. In their current condition, the failed walls are susceptible to additional degradation and bank erosion; bank scalloping and lateral erosion from peak flow events is evident.

Adjacent floodplain surfaces have been landscaped and maintained in a park-like setting with benches, trees, and walkways. However, adjacent banks are completely denuded of all vegetation in many segments. Lack of ground cover and bank strength may be attributed to high impact from human trampling, high nutrient loading and herbivory from waterfowl, and shading from a mature tree canopy.

At Sta. 2411+00 bank and bed instability are high. The grade and hydraulic transition from concrete wall margins to alluvial banks and floodplain, coupled with a walled island and in-channel infrastructure, has created an unstable channel segment at the reach break between Park 1 and Park 2 (Photo G.9).

Design considerations should consider high stream power and complex hydraulics associated with infrastructure and a tight bend configuration near Sta. 2411+00. Flood flows outside the boundaries of the active channel (floodwalls) provide sufficient hydraulic force to pluck and entrain large quantities of walkway/patio bricks. In addition, evidence of recent hydraulic scour around the tree trunks located 10-15 feet above the channel bed was observed. Bank and floodplain treatments to improve stability and aesthetics must consider high shear values well beyond the walled confines of the active base flow channel.

Photo G.9 A tight meander bend and local hydraulic forces near Sta. 2411+00 cause local instability and degraded channel and floodplain conditions.

Park 2

Downstream of the grade control structures and the low water ford, the channel margins are no longer concrete/rock walls, but return to natural alluvial banks at Sta. 2407+00. Banks are composed of fine silt and sand with stratified layers of gravels along exposed, eroding terrace banks. Banks are well vegetated by annual grasses, forbs, and large woody shrub and tree species, which provide a relatively high floodplain roughness value. Channel form shows a developed pool/riffle sequence through this lower, more “natural” segment with cobble and gravel substrate and a veneer of fine silts in low velocity channel zones. However, near Mulberry Street a backwater environment extends approximately 1000 feet upstream. Storage of fines on a relatively deep channel bed is evident. Low flow velocities upstream of Mulberry Street limit fluvial process and subsequent habitat complexity at base flow.

Dense riparian woody and herbaceous vegetation are well established through this reach, including mature trees and woody vines. Although riparian plant community composition and type has been altered, the relation to plant community types and hydrologic stages are evident here. This sub-reach may provide valuable insight towards revegetation efforts for the future. Access to and recreational use of the river corridor through Park 2 is high, and as a result, denuded banks are common from pedestrian traffic (Sta. 2406+00 to 2359+00).

Historic channel incision is more apparent downstream of Mulberry Street The right bank is composed of a historic terrace composed of fine silt and clay with a distinct layer of well-stratified gravels. Coarse to fine-grained layers of well-sorted alluvium are exposed along concave bank segments.

Active floodplain development is occurring along channel margins through this reach. The historic terrace on the right bank constrains the channel while an inset floodplain surface is developing, often creating additional backwater lagoon habitat of emergent wetland plants and cattails. Aggrading bars appear to be the result of local, short-term sediment transport and storage conditions. Relatively frequent rainstorm events are associated with an effective discharge responsible for transporting the greatest volume of sediment on an annual basis. Sediment deposition on the receding limb of these frequent floods form transient side bars, which promote floodplain development and a growing medium for emergent wetland and riparian plant communities. Channel shading from riparian cover is high (Photo G.10).

At Station 2359+00, a grade control structure/low water ford contributes to backwater conditions upstream for approximately 1,500 feet. Immediately downstream, a mid-channel gravel bar is well vegetated with dense perennial herbaceous and annual weed species. Depositional islands are likely a function of change in channel slope and floodplain confinement. Downstream to McAllister Freeway, there are two low water concrete crossings and two bridges associated with the golf course pathways. Approximately 100 ft. of the left bank margin, beginning at Sta. 2357+00, is eroding. Immediately below the crossing at Sta. 2354+00, the left bank margin is also eroding. Both banks are about 6- ft high and nearly vertical, composed of fine silt and sand overlying coarse gravelly alluvium. Typical of floodplain margins devoid of woody vegetation, the golf course grass turf margins are especially prone to erosion (Photo G.11). The channel becomes more entrenched with steep 10-12 ft. vertical banks between the golf cart bridges, limiting revegetation work without laying back the banks.

Photo G.10 San Antonio River looking upstream towards Mulberry Street bridge in the background.

Photo G.11 Incised channel and eroding bank conditions along the golf course segment upstream of McAllister Freeway.

Catalpa-Pershing Ditch

Upstream Broadway Street culvert

The Catalpa-Pershing Ditch is a tributary of the San Antonio River and drains from the northeast of the Brackenridge Park golf course through a high-density residential and commercial area. The drainage begins on the east side of Broadway Street. The channel is trapezoidal with high volume of gravel aggradation within the bed. The channel has a bankfull top width of about 15 feet and depth of 2 feet. The channel bed is coarse gravel alluvium, and banks are fine silt, sand, and clay with intermittent lenses of gravel. The floodway is a relatively open grass swale and parkway with intermittent stands of trees. Active headcutting is rapidly changing existing channel geometry immediately upstream from the culvert inlet at Broadway Street Upstream from the 4- foot grade break or nick point, the floodway changes to a grassy swale with no well-defined channel (Photo G.12).

Head cutting upstream of the Broadway Street culvert inlet indicates geomorphic instability, a condition likely triggered by changes in urban hydrology and subsequent channel response and adjustment. Higher magnitude and more frequent flood events is contributing to channel instability and rapid change in channel geometry. The master plan presents the potential to remove concrete and renaturalize the Catalpa-Pershing Ditch with some minor meander bends and more physical variability. Restoration opportunities of the concrete-lined ditch section must consider high stream power values and almost no sediment supply from upstream.

Downstream Broadway Street culvert

The drainage ditch passes under Broadway Street and daylights on the east side of Brackenridge Park, and flows straight south. The downstream section of channel is a concrete-lined channel with a veneer of sand and gravel deposits and some established woody vegetation (Photo G.13).

Natural channel to confluence San Antonio River

The last 1,500 feet of the ditch returns to natural bank material as the channel bends to the southwest before flowing into the San Antonio River immediately upstream of McAllister Freeway at Sta. 2345+00.

Photo G.12 Active head cutting indicates instability on the upper portion of Catalpa-Pershing Ditch.

Photo G.13 Catalpa-Pershing Ditch concrete-lined channel on east side of Brackenridge golf course.

Urban

Immediately below the tunnel inlet, both the left and right banks of the river are constrained by vertical concrete walls for approximately 100 feet, followed by stacked vertical rock gabions for about 100 feet. The channel is uniform with no remarkable bed features and well entrenched and channelized through this reach. The channel bed appears armored with 6- inch to 1- ft cobbles and boulders. A large fraction of bed material is riprap and/or failed concrete pieces of infrastructure, most visible near bridge crossings. A layer of fine alluvial material, (sand, silt and clay) with a high organic content overlay coarser alluvium where minimal low flow velocities create stagnant base flow conditions.

Beginning 200 ft above Josephine Street Bridge (Sta. 2337+00), woody and herbaceous riparian cover is dense with a high roughness value from channel's edge to top of bank. The vegetation provides extensive bank protection and stability. At Myrtle Street (Sta. 2326+00), a large drain structure enters the channel on the right bank. At base flow conditions, velocity is minimal, which allows emergent plants to form on the main channel here, also indicating some potential nutrient loading at the storm drain outfall and/or marginal sediment conveyance.

Below Grayson Street Bridge, the percentage of impervious surface area in the surrounding urban drainage is high. Surface overland flow from adjacent asphalt and roof cover has contributed to the formation of several steep ephemeral side gullies that head cut perpendicular to the main channel. High vegetation and root density provides bank strength to the fine, cohesive bank materials. The erosion rate and sediment input is likely minor. The riparian and aquatic habitat quality is relatively high at their interface, considering the entrenched and channelized conditions. This may be partially attributed to the high floodplain roughness and complexity of plant communities that overhang and impede flow.

Moderate left bank erosion on the outside of a meander bend at Sta. 2321+00 extends approximately 100 feet, and immediately downstream a gravel bar has developed at Sta. 2320+00. These localized attributes suggest a large volume of the eroded bank has formed this coarse-bedded bar deposit, which likely remain immobile during frequent annual flow events.

Through this reach, one potential challenge for designing the expanding the riverwalk theme will be the frequent storm sewer outlets. Outfall structures could be integrated into a biological, structural component of the design work, as envisioned in the Concept Master Plan. At Sta. 231850, a 24-inch diameter storm pipe outlets on the right bank, one segment of the pipe is failed.

Between Newell Street Bridge (Street 2318+00) and I-35 Bridge (Sta. 2312+00), the channel remains uniform with 20- ft high, 2H:1V slope banks, heavily vegetated by perennial woody trees and herbaceous weed and grass cover (Photo G.14). Camden Street Bridge intersects the channel diagonal at (Sta. 2315+50) and a railroad line (Sta. 2314+00) bridge also crosses the channel, contributing to a high frequency of infrastructure that impacts channel hydraulics. Under the piers and abutments of the I-35 underpass, a large plug of gravel and sand material forms a relatively mobile and unstable bar deposit, sparsely vegetated with weeds. I –35 abutments appear stable. The change in channel dimensions downstream to a more constricted and incised channel geometry may influence sediment continuity. Likewise, local hydraulics generated from adjacent infrastructure-bridge piers- affects sediment continuity. No floodplain development exists, but a constructed bench or historic right-of-way of fill forms the right bank. A 50- foot right bank segment (Sta. 2311+00) is actively eroding, exposing fine-grained materials. Bank height is approximately 8 feet, and will likely necessitate future stabilization measures.

The existing Thirteenth Street storm outfall at Sta.2308+50 on the left bank poses some hydraulic constraints that may impair local stability. Historic channel incision has exposed a large volume of the outfall concrete collar or apron above the existing bed and bank elevation. Immediately downstream of the outfall, the channel bed is lined with boulder-sized riprap as local grade control for about 20 to 30 feet. At Sta. 2305+00, another storm outfall drains the Museum lots on the right bank. Bank stability and vegetation cover is good. Between Sta. 2301+00 and 2295+00 the channel makes a U-shaped bend to the southwest, with the left bank forming the outer, more erosive concave margin. Asphalt-poured grout or riprap is used to stabilize approximately 200 feet of the left bank upstream of W. Jones on the outside meander bend. Aesthetic and ecological impairment is high due to urban encroachment and riprap. On the contrary, the left bank has formed a heavily vegetated inset floodplain about 50 feet wide. Alignment of W. Jones Bridge crossing and the entrenched channel is marginal, which may attribute to deposition upstream and poor sediment continuity, as exemplified by an aggrading floodplain surface on the right bank.

Downstream of W. Jones (Sta. 2298+00), the channel forms another constrained bend way, with the outer left bank armored by concrete wall and large boulder riprap for approximately 300 feet. Towards the Ninth Street Bridge, (2287+00) the channel remains well entrenched and confined and uniform in geometry with heavily vegetated, steep floodway banks. Below Brooklyn Avenue (Sta. 2279+00) turf grass and intermittent, mature deciduous trees replace native woody vegetation on the floodway slopes (photo 65). Minor bank erosion is occurring associated with storm outlet drainage on the left bank (Sta. 2271+00). The channel at base flow shows no significant fluvial features or aquatic habitat and minimal flow velocity. The backwater environment (low flow velocity) is caused by a sheet pile grade control dam/diversion structure used to dewater the river at Sta. 2272+00, immediately downstream of McCullough Avenue. The structure has a vertical height of about 4-6 feet. Farther downstream at Sta. 2268+30, another slope break and grade structure is constructed perpendicular across the channel. This 2– foot vertical grade structure is concrete and/or limestone mason blocks and marks the farthest most upstream extent of current Riverwalk construction project near Lexington Avenue (Photo G.15).

Photo G.14 Looking downstream at Camden Street and I-35 Bridges in the Urban sub-reach.

Photo G.15 Looking downstream at Lexington Avenue Bridge, the end of the Museum North project reach and beginning of the Downtown Riverwalk.

Downtown

The Downtown San Antonio River sub-reach extends from Lexington Avenue to S. Alamo Street The River loop bypass channel diverges from the main channel at Sta. 2228+00 and re-enters at Sta. 2222+00. The main channel is a concrete-lined flood control channel from 2228+00 downstream to Nueva Street (Sta. 2216+00).

Eagleland

At South Alamo Street Bridge, Gate 6 marks the beginning of the Mission South project reach. The gate structure controls flow volumes downstream. Below S. Alamo Street and Gate 6, the right bank is comprised of a 15 foot high concrete wall for approximately 100 feet and the left bank is armored with 1 to 2- foot boulder riprap. About 200 feet below Gate 6, a concrete weir or sill grade control structure with about 1- foot of vertical drop crosses the channel. Bank margins downstream are armored with large 2 to 4- foot diameter concrete riprap slabs and shorter (10 to 100 feet) bank segments of unprotected fine-grained bank material with occasional lenses of gravelly alluvium (Photo G.16). Non-armored alluvial banks are generally steep cut banks, 3-5 feet high, cohesive and sparsely to well vegetated with perennial grass cover. Many large storm-drain outfalls and/or remnant tributaries enter the main channel on both banks in this sub-reach. Downstream of Sta.2152+50, the pilot channel dimensions change. Banks are lower with greater floodplain area and the channel area appears to decrease proportionately. Near Sta. 2144+00, the backwater effects of the tunnel outlet dam structure are evident and wetted channel area increases once again. A lack of geomorphic features and poor aquatic habitat may be contributed in part to stagnant flow conditions through this sub-reach. Floodplain surfaces (2139+00) are capped with poured concrete along isolated riprap channel segments. Approximately 250 feet above the S. P. Co. Railroad Bridge (Sta, 2126+55), the channel banks and bed are concrete-lined, which continues downstream past the tunnel outlet (Sta.2125+24) and Lone Star Blvd. (Sta. 2121+61). Both ends of the concrete-lined channel segment are armored with 1- ft + diameter boulder riprap. The high frequency of railroad bridge piers suggests potential flow and sediment impedance (Photo G.17). There are twelve vertical piers with eight beams per pier supporting the railroad line across the floodway. Given the by-pass tunnel conveys flood flows, the railroad structure upstream from the outlet may be hydraulically insignificant.

Eagleland sub-reach provides the opportunity to design a new pilot channel with low amplitude meanders and an inset compound channel and floodplain surface. Floodway boundaries and adjacent residences on the left (east) bank limit lateral changes to channel planform. Several storm sewer outfalls on both the left and right bank offer additional planning and design challenges related to storm water delivery to the pilot channel and floodplain design. Variables such as distance to water’s edge, infrastructure constraints, and depth of scour will warrant further evaluation. Revegetation supported by storm effluent above the low water table offers good opportunity to diversify plant communities, increase roughness and habitat complexity to the system. Efforts to reduce the backwater effects caused by the tunnel outlet weir would ultimately increase geomorphic and aquatic habitat complexity upstream.

Photo G.16 Looking upstream at the ACOE floodway and riprap pilot channel below S. Alamo Street and Gate 6.

Photo G.17 The S. P. Co. Railroad Bridge crosses the concrete-lined channel segment on a tight floodway bend upstream from the tunnel outlet.

Below Outlet

The Tunnel Outlet at Sta. 2125+24 is the beginning of a more steep bed profile a series of vertical drops in channel bed elevation. The channel remains well entrenched. Below Lone Star Blvd, both channel banks and bed are armored with concrete riprap, but the right, concave bank has far more volume of riprap protection. Banks and floodplain surface are generally comprised of 1 to 2 feet of fine silt loam overlying a 1- foot layer of gravels with a concrete riprap toe. Floodway 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 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- check dams 10 (Sta. 2116+50) and 9 (2113+00) are comprised of metal sheet pile embedded in the channel and supported by riprap along the bank margins. Bed elevation change at each structure is about 2 to 4 feet. Check dam 9 is partially failed, with the sheet pile 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, lateral bank erosion is minimal due to high volumes of riprap below the structure and adjacent to storm outfalls. As outlined in the Master Plan, 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 maintain grade at Sta. 2104+00 and 2098+70, respectively, and help stabilize adjacent storm sewer outfalls on both banks (Photo G.18). At Sta. 2103+50, bed slope increases, creating uncommon riffle features as the channel passes under two bridge structures (a pipeline/utility crossing and CPS Bridge). Nearly six feet of bridge pier scour is visible on the CPS bridge piers (Photo G.19). The channel width decreases as slope increases through this heavily encroached segment. The channel passes under the S. P. Co. Railroad line (Sta. 2098+97), Steve's Avenue (2095+10), and IH-10 (2090+00). On the right floodplain, a secondary overflow channel has formed upstream of Steve's Avenue. Between IH- 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.

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.


Photo G.18 Check dam grade control structures built from metal sheet pile maintain channel stability and grade through the Below Outlet sub-reach.


Photo G.19 Looking upstream at the CPS bridge with substantial pier scour.

Conception

The confluence with San Pedro Creek at Sta. 2060+50 marks a significant decrease in bed slope and an increase in channel width and area downstream. The tremendous increase in flow volumes and sediment input would indicate San Pedro Creek watershed generally dictates the dominant hydrologic hydraulic, and geomorphic characteristics of the downstream reaches. At the confluence, both floodways are heavily armored with riprap, 2 to 4- foot slabs of concrete (Photo G.20). A large cobble and gravel bar has formed along the right channel margin. To prevent vertical incision and headcutting, 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 occurs 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 (Photo G.21). 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 enormous. 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. 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+50 (Photo G.22).

The Conception 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.

Photo G.20 Confluence of San Pedro Creek (left) and the San Antonio River.

Photo G.21 At a tributary outfall on the right bank, active head cutting and scour are evident near Sta. 2044+00.

Photo G.22 Looking upstream from the Mission Road Bridge at the lower end of Conception.

Mission

The Mission reach extends from upstream Mission Road Bridge to a more depositional segment above the San Juan Diversion. 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 1994+00 (Photo G.23). On the left bank, the tributary outfall (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. Opportunity to re-vegetate and enhance this outfall is high.

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. A concrete sill grade structure spans the channel 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 Southcross Avenue 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 (Photo G.24). 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 pilot 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.

Photo G.23 A typical pilot channel and floodway segment of the Mission reach.

Photo G.24 Large volumes of concrete riprap, periodically capped with topsoil, pose high cost and effort to remove and replace with more aesthetic bank protection.

San Juan

As a result of the San Juan Dam diversion structure (Photo G.25) at Sta. 1910+50, 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 thalweg is present on the right side of the channel, an uncommon channel feature in a project reach generally characterized by a uniform, trapezoidal pilot 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+00, 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. Active lateral instability on both banks and channel scour immediately downstream of the old structure indicates a nick point and head cut potential.

The distinct clay formation (Photo G.26) 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.

Photo G.25 Looking upstream at the San Juan Diversion structure.

Photo G.26 An eroding bank segment below the old San Juan Dam exposes a distinct clay formation at the toe and channel edge.

Davis

A 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 section as-builts and current 2002 topographic surveys indicate several feet of aggradation 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+00 on the left bank. Asylum Creek is a large tributary with a concrete side slopes/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 Sta1883+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 (Photo G.27). 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 likely responsible for this bank feature anomaly.

Photo G.27 Looking downstream at Asylum Creek confluence and S.E. Military Drive with extensive side bar deposit that extends to Espada Dam.

Below Espada

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 ACOE 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 rare heavily armored with large slabs of concrete riprap. The riprap on the left bank beginning at Sta. 1828+00 constricts and narrows 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 700 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 (Photo G.28), which outlet at the historic bridge and remnant San Antonio River channel on the left (east) floodway. This bypass maintains base flows to the historic channel segment that re-enters the pilot channel at Sta. 1774+00. 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.

Photo G.28 A concrete-lined segment is used to divert base flows to a remnant channel and historic bridge on the left floodway corridor at the lower end of Below Espada and the start of Six Mile reach.

Six Mile (Piedras) Creek

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 again. The historic remnant channel confluence with the pilot channel is located downstream at Sta. 1774+28. Between Sta. 1776+50 to 167773+00, the main 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.

Photo G.29 Looking upstream at Six Mile, a dewatered segment at base flow, characterized by large scattered riprap slabs and scoured floodplain surfaces.

410

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-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 (Photo G.30). . 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 Coahuliteca low water ford 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 (Photo G.31).

Photo G.30 I-410 Bridge with pier scour and high water debris nearly 18 feet above the channel bed.

Photo G.31 Looking upstream towards I-410 Bridge in the background. Bank erosion is high along non-riprap banks to the end of the project reach.

Below Project

Downstream from the project reach, the ACOE trapezoidal channel and floodway 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-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.

Photo G.32 Downstream from the project reach and ACOE floodway, the San Antonio River remains a more natural system with a riparian corridor, local sediment supply, meandering pattern, and more accessible floodplain.

Historic Channel Assessment

Comparison of historic channel changes can provide insight to dominant river processes and therefore offer a basis for channel design for the future. In this study, a recent chronology of channel conditions was developed to identify the geomorphic change that has occurred in the San Antonio River project reaches in the last century. Significant flood hazard reduction and channel modification projects were implemented on the San Antonio River during the 1900's. This included construction of Olmos Dam, channelization in Brackenridge Park, and meander cut-offs through downtown in the 1920's; construction of the Hugman concept in the form of riverwalks and walls through downtown from 1935 to 1941; and construction of the south reach channelization and floodway that commenced in 1950's and continued through the 1970's. The latest flood reduction project is the San Antonio Tunnel, which allows the by-pass of flood flows around the downtown area.

The river conditions prior to these efforts were of an equilibric, less altered system with a natural sediment supply, flooding over a much wider floodplain, higher width/depth ratio, lower gradient and greater sinuosity. The construction efforts have straightened the river course, increased its gradient, and confined flood flows to a relatively narrow floodway allowing urbanization to encroach upon the river's historic floodplain. The geomorphic impact of these works has resulted in increased channel instability and degraded habitat, while conveying flood flows more rapidly downstream. Therefore, to deduce the physical processes that affect the river system and to evaluate how the river will respond to existing and future construction efforts, a historical assessment of channel conditions was performed. A comparison of previous to existing conditions was performed to assess the tendency for future adjustments in channel planform, slope, and cross section geometry. Moreover the analysis provides information to support the sediment transport analysis and design of channel stabilization measures. Data used in the investigation included as-built plans, recent surveys, and information contained in the archaeological background reports.

Channel Planform Comparison

Comparison of channel alignments was performed to determine the amount of induced planform change and to assess how it has affected channel stability in the north and south project reaches. Most of the planform change resulted from human induced flood reduction efforts conducted after the flood of 1913. For the north reaches, this included construction of meander bend cutoffs in the late 1920's. For the southern reaches the construction of the floodway severely manipulated the channel planform by straightening virtually nine continuous miles of river. Much of the historical planform data was generated from the channel alignment represented in the as-built plans for the south reaches. Additional historic planform data for the Urban Reach was obtained from the Archaeological Background Report (Cox and Fox, 2002) developed for the North Reach study. The pre-construction and current channel alignments are shown in Figures C.1 and C.2. The historical planform shown represents the alignment obtained from readily available information and does not represent a comprehensive description of river conditions prior to any improvements.

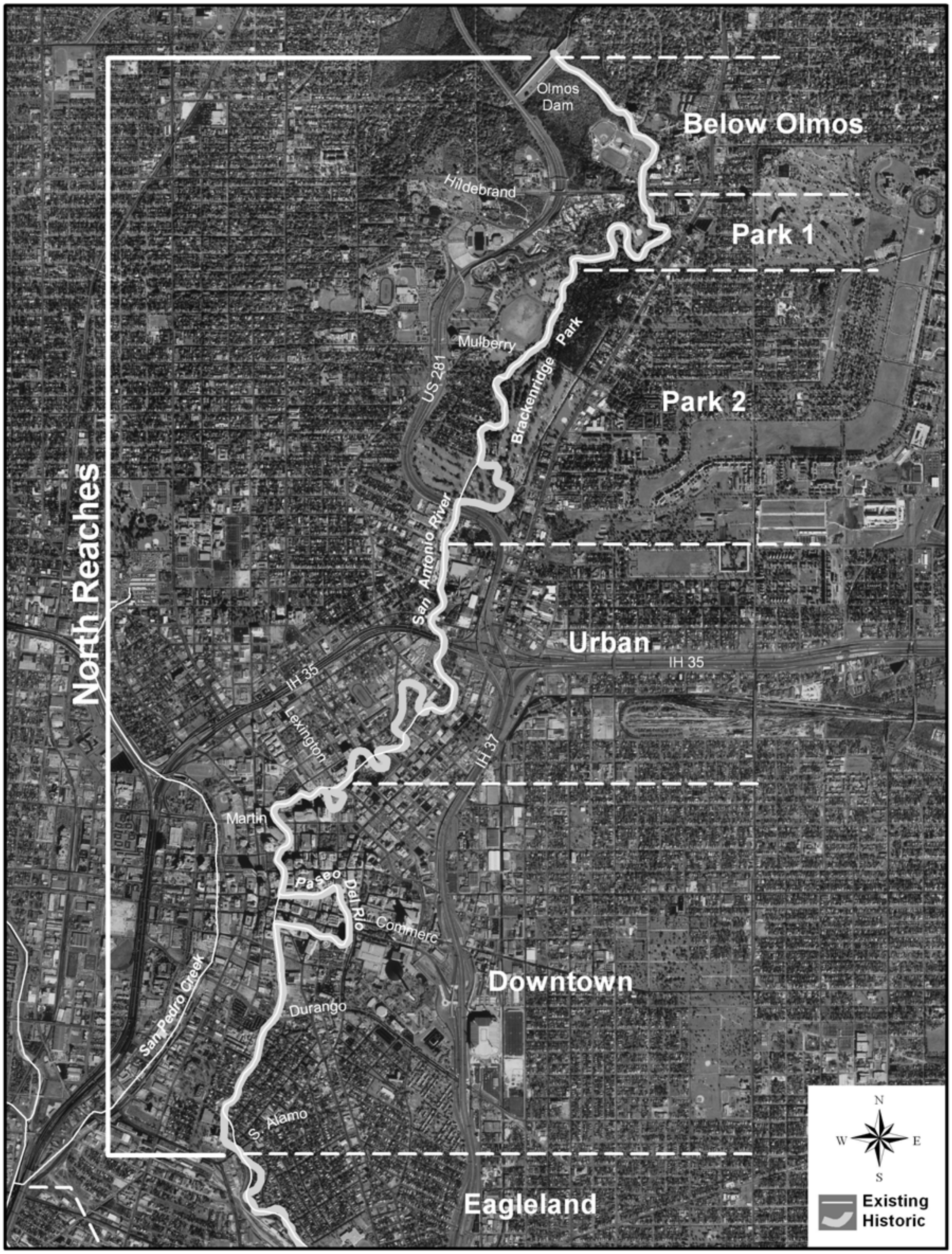

Figure C.1 Existing and Historic Planform of the San Antonio River – North Reach

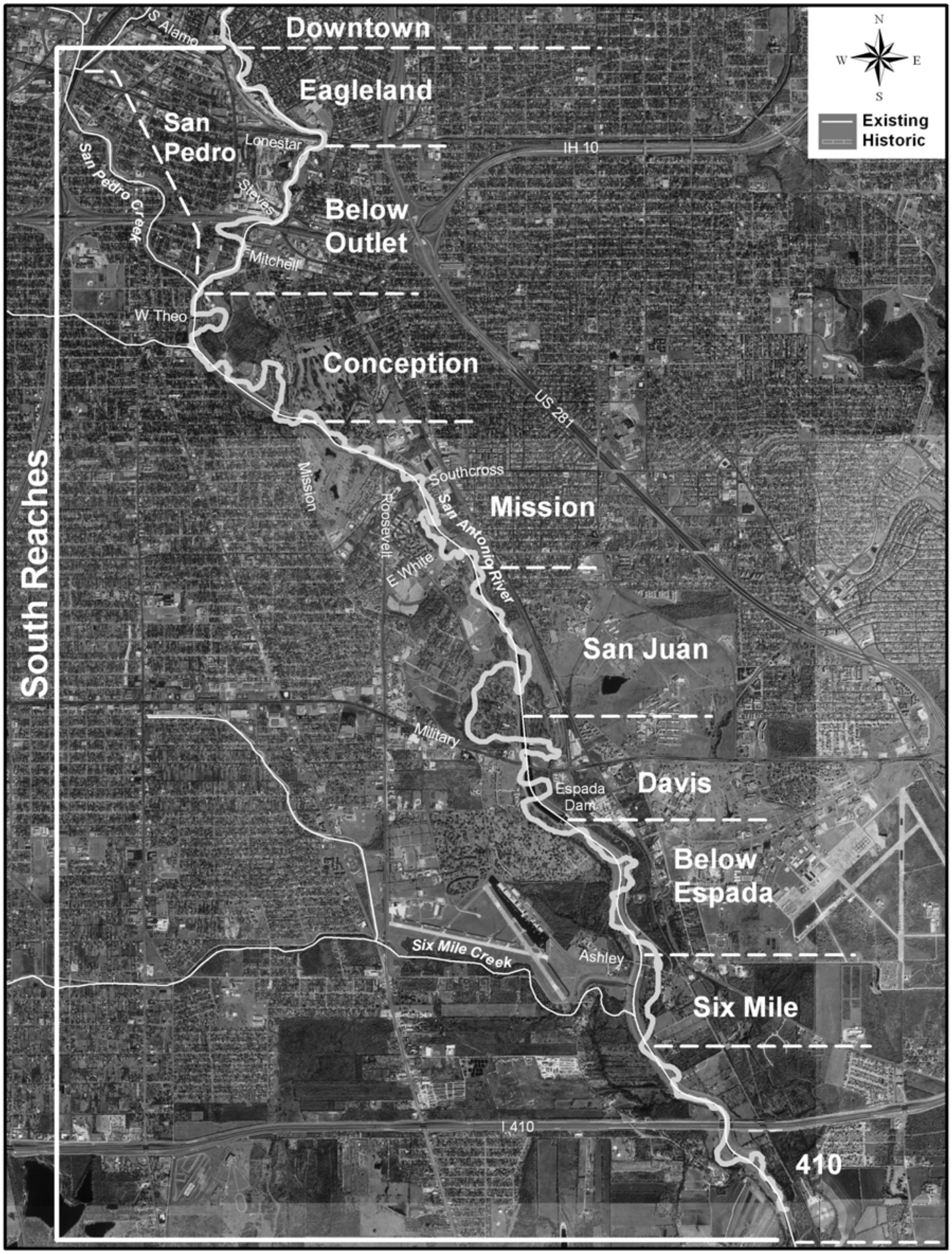

Figure C.2 Existing and Historic Planform of the San Antonio River – South Reach

The planform data were used to calculate historic and existing channel sinuosity values for each of the project subreaches. Sinuosity is computed as the channel length divided by the valley length or channel slope divided by valley slope and can be used as a measure of the amount of meandering in a river system. The historical planform resulted in a sinuosity of approximately 1.5 for the North Reach and 1.7 for the South Reach. The current channel has a sinuosity of 1.2 for the North Reach and 1.0 for the South Reach. This verifies a significant amount of channelization resulting from the flood control projects. The reach based channel sinuosity values are listed in Table C.1 and illustrated in Figure C.3.

Table C.1 Historic and Existing Channel Sinuosity

		Sinuosity		
	Subreach	**Historic**	**Existing**	**Difference**
North Reach	BELOW OLMOS	1.1	1.1	0.0
	PARK 1	1.9	1.9	0.0
	PARK 2	1.3	1.1	-0.2
	URBAN	1.7	1.2	-0.5
	DOWNTOWN	1.6	1.1	-0.5
South Reach	EAGLELAND	1.2	1.1	-0.1
	BELOW OUTLET	1.4	1.1	-0.3
	CONCEPTION	2.0	1.0	-1.0
	MISSION	1.6	1.0	-0.6
	SAN JUAN	1.7	1.0	-0.7
	DAVIS	2.6	1.0	-1.6
	BELOW ESPADA	1.6	1.1	-0.5
	SIX MILE	1.1	1.1	0.0
	410	1.4	1.1	-0.3
	BELOW PROJECT	1.7	1.3	-0.4
	North Reach Average	**1.5**	**1.2**	**-0.3**
	South Reach Average (below San Pedro)	**1.7**	**1.0**	**-0.7**

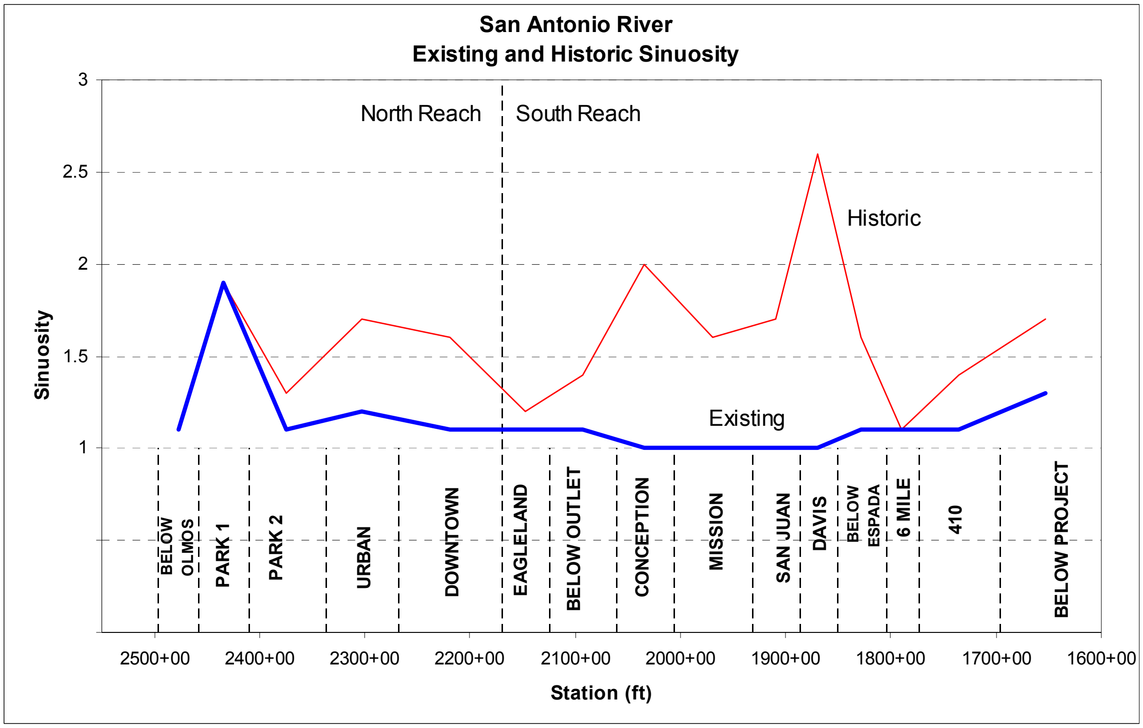

Figure C.3 Comparison of Channel Sinuosity

Valley length has remained relatively unchanged over time, and therefore, the change in sinuosity can be used as a measure of the change in channel slope and energy dissipation as a result of the flood control improvements. Increases in channel slope generally result in increased channel shear stress without the implementation of grade control. Since the channel energy is not being absorbed through meanders the result would be incision until a base level control is obtained. Comparison of existing and historic channel elevations shows this to be the case in the southern reaches and incised channel conditions evident in the northern reaches. A discussion of channel slope and profile comparisons is provided in the flowing section. The change in sinuosity values provides an estimate of the amount of channel incision that may be apparent in the reaches today and what could be expected in the future. The greatest reductions in channel sinuosity have occurred in the Urban, Downtown, Conception, Mission, San Juan and Davis subreaches. Since construction of the meander cutoffs in the North Reaches significant channel incision has been observed in the Urban subreach. Channelization and grade control in the Downtown subreach has inhibited incision in this subreach. In the South reaches, significant incision of the pilot channel has been observed in the Conception and Mission subreaches. Grade controls in the form of Espada Dam and the San Juan Diversion structure have limited incision in these subreaches. Therefore, the geomorphic response has been typical following channel straightening in the affected project reaches.

Another method to quantify the extent of historic change in channel planform is the measurement of the mean radius of curvature from the historic (pre-1957) channel planform as shown in Figures C.1 and C.2. The radius of curvature is defined as the linear distance between the center of the bend and the center of the channel. The radius of curvature is closely related to meander wavelength and sinuosity. As wavelength shortens and sinuosity increases, meander bends tend to tighten, and the radial arm consequently decreases.

The mean radius of curvature was measured at representative meander bends from the historic channel planform of the San Antonio River through the North and South project reaches. The radius of curvature of the same floodway segments was measured from the existing channel alignment where applicable. Many of these segments, most notably in the South reaches below the San Pedro Creek confluence, are straightened, and therefore, do not maintain a meandering form. These segments were labeled (NA) for not applicable. The average values are shown in Table C.2, which verifies the relative effects of channel and floodplain manipulation and flood control projects over time.

Table C.2 Mean radius of curvature values in feet.

	Subreach	Historic	Existing
North Reach	BELOW OLMOS	300	300
	PARK 1	215	215
	PARK 2	198	210
	URBAN	177	260
	DOWNTOWN	167	NA *
South Reach	EAGLELAND	197	520
	BELOW OUTLET	125	520
	CONCEPTION	150	990
	MISSION	208	1080
	SAN JUAN	194	NA *
	DAVIS	227	1110
	BELOW ESPADA	230	1200
	SIX MILE	435	1900
	410	210	880
	North Reach Average	**211**	**246**
	South Reach Average (below San Pedro Creek)	**222**	**1097**

* Note: Not applicable (NA) refers to channel segments that are straight.

The existing channel planform mean radius of curvature has significantly increased from its historic condition as a result of urbanization and flood control projects. Specific impacts include meander cutoffs and channel straightening, channelization, channel maintenance and sediment removal, channel and floodplain encroachment, vertical grade control, and lateral confinement with concrete and riprap. Many existing project reach segments have no measurable bend radii (NA) based on a straightened channel planform while other channel segments are passively meandering, meaning the existing meander bends are locked in place and limited by the existing floodway alignment and immobile lateral channel boundaries of the pilot channel. The exception to this trend is found in the North reaches where the planform geometry has remained relatively stable.

Comparative Cross Sections Analysis

Comparison of channel cross sections provides evidence of the potential for adjustment in channel geometry. Of most relevance to this study is the change in channel geometry and profile since the South Reach floodway was constructed. This information indicates whether the channel and overbank areas have been erosive or depositional and what type of stabilization measures may be required for the current design. Cross sections representative of the as-built (1957 – 1970) condition were obtained from plans provided by SARA. The U.S. Army Corps of Engineers (COE) completed the designs for the South Reach floodway and the channelization of San Pedro Creek. As-built drawings for the north reach channelization efforts were not available. Existing conditions were obtained from recent survey information and current hydraulic models. The San Antonio corridor was recently surveyed in 1998 and 2001 using orthophotogrametry to define the above water topography in the floodway. In addition, SARA surveyed cross sections below the waterline using conventional methods. The exception is the area behind Espada Dam up to the San Juan Diversion that was not surveyed in the recent efforts. However, survey data obtained in 1993 for the dredging of Davis Lake was utilized to define the exiting condition behind Espada Dam. A total of 33 cross sections were selected at intervals that are representative of the overall reach conditions. Additionally, comparative cross sections where obtained for the San Pedro Creek for evaluation of sediment load from this tributary. Locations of the comparative cross sections are represented in Figure C.4

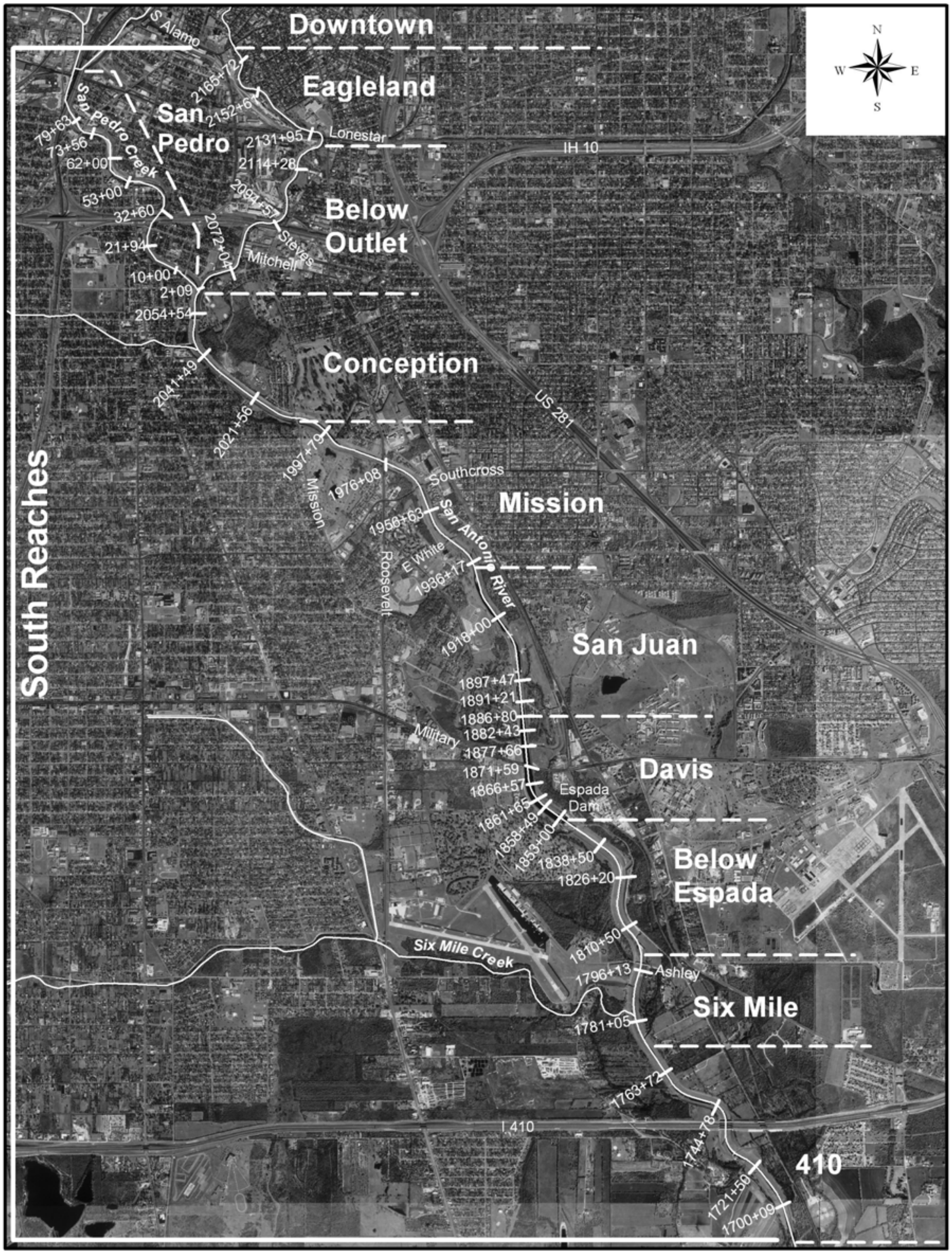

Figure C.4 Comparative Cross Section Locations – South Reach

Comparative Cross Sections - Eagleland Subreach

Comparative cross sections in the Eagleland subreach downstream of S. Alamo Street indicate deposition of material that has defined an inset channel and interior over bank area now evident in this subreach. Channel bathymetry was not surveyed in this subreach, therefore the existing condition is defined by the above water topography. The as-built information did not show a pilot channel or interior overbank, therefore it is assumed that this configuration has been developed from the long-term cycle of deposition and channel erosion in this subreach. Bank sediment sampling in this reach indicates that the interior floodplain consists primarily of silt and clay material. Based on this information, this reach would be described as depositional. Further, construction of the San Antonio River by-pass tunnel outlet weir completed in 1995 located at the downstream end of this subreach also contributes to increased backwater and deposition in Eagleland subreach. A sample comparative cross section for the Eagleland subreach is shown in Figure C.5.

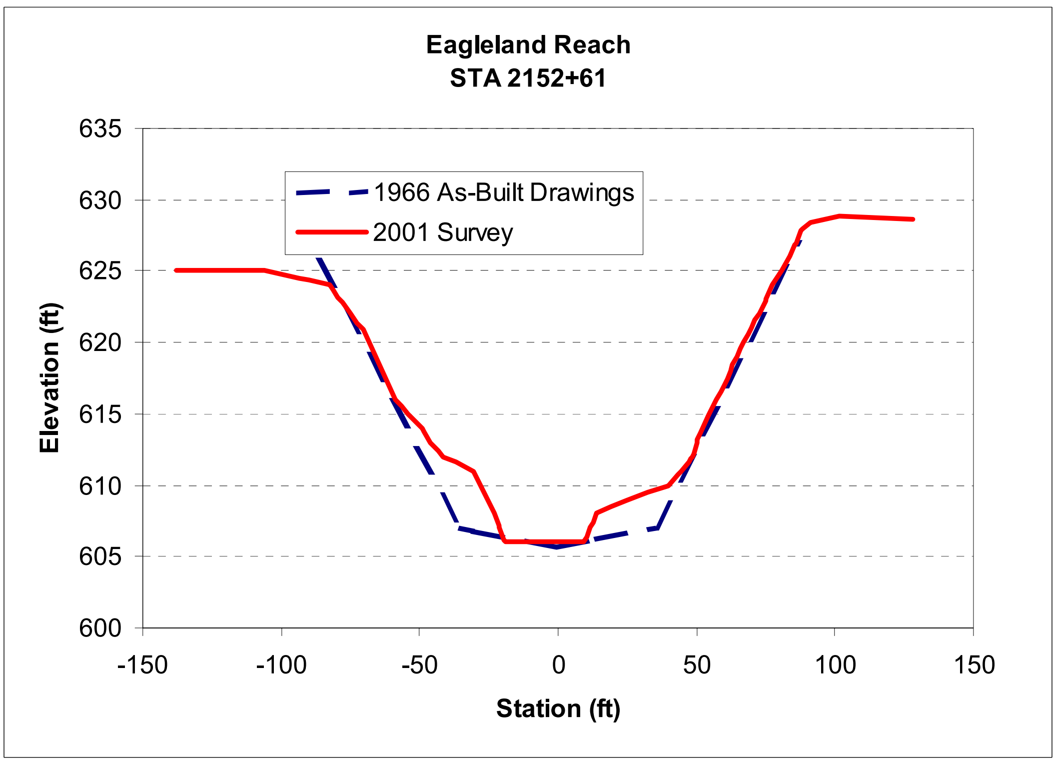

Figure C.5 Comparative Cross Sections in the Eagleland Subreach

Comparative Cross Sections – Below Outlet Subreach

Comparative cross sections in the Below Outlet subreach downstream of the tunnel outlet indicate a confined floodway (~160 feet) with bed scouring that has resulted in a more defined inset channel. The as-built information did not show a pilot channel or interior overbank in this area. The floodway through this reach has a steeper gradient (~0.4%) than the other project subreaches, but was designed with sheet piling grade control structures, which have been exposed over time. The channel has eroded approximately 3 feet since construction. Based on this information this reach would be described as slightly erosive, but the potential has been reduced by the numerous grade control structures in this subreach. A sample comparative cross section for the Below Outlet subreach is shown in Figure C.6.

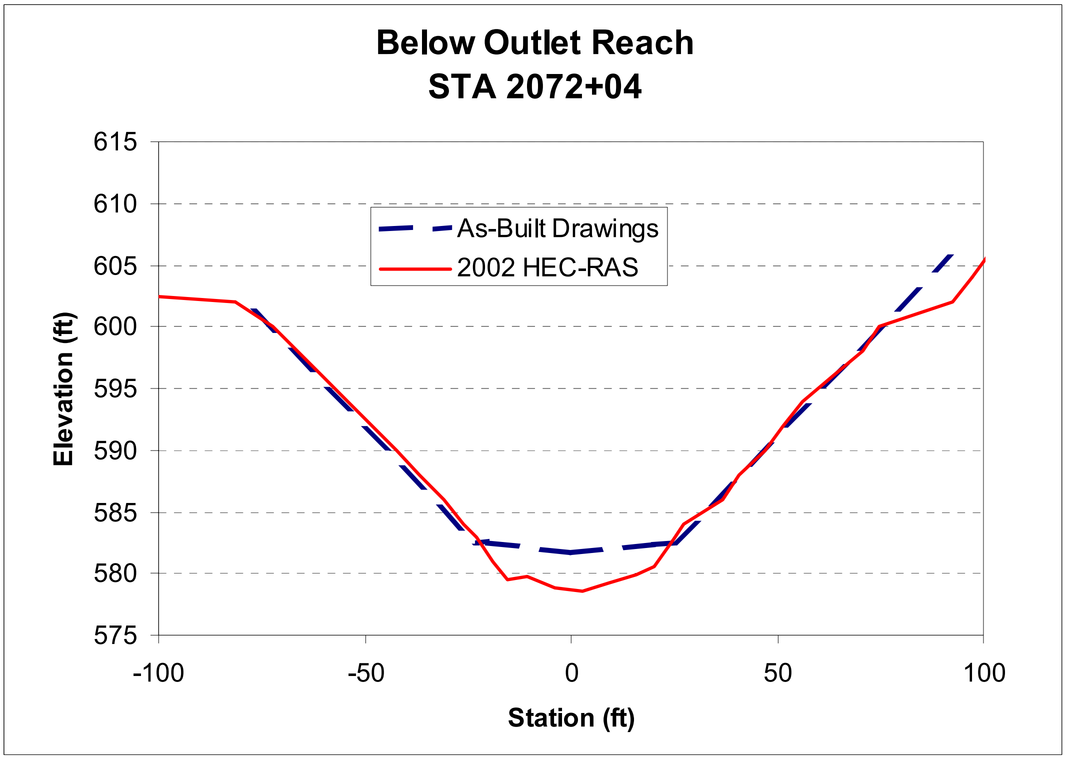

Figure C.6 Comparative Cross Sections in the Below Outlet Subreach

Comparative Cross Sections – San Pedro Creek Subreach

Although the San Pedro Creek is not in the current design, it is significant with respect to sediment loading on the San Antonio River. Comparative cross sections in the San Pedro Creek subreach indicate scouring of the pilot channel and some deposition in the interior overbank area. Therefore, the pilot channel is considered erosive where as the interior overbank area is depositional. The pilot channel has widened from the as-built top width of 40 feet to nearly 70 feet and the channel bed elevation has lowered by more than 5 feet in some areas within this subreach. A pilot channel was constructed from the confluence with the San Antonio River upstream approximately 1100 feet and then transitioned to a single channel geometry. However, the cumulative volume of sediment scoured from the main channel and deposited in the overbanks are nearly equivalent through the lower 5000 feet of the San Pedro Creek. A sample comparative cross section for the San Pedro Reach is shown in Figure C.7.

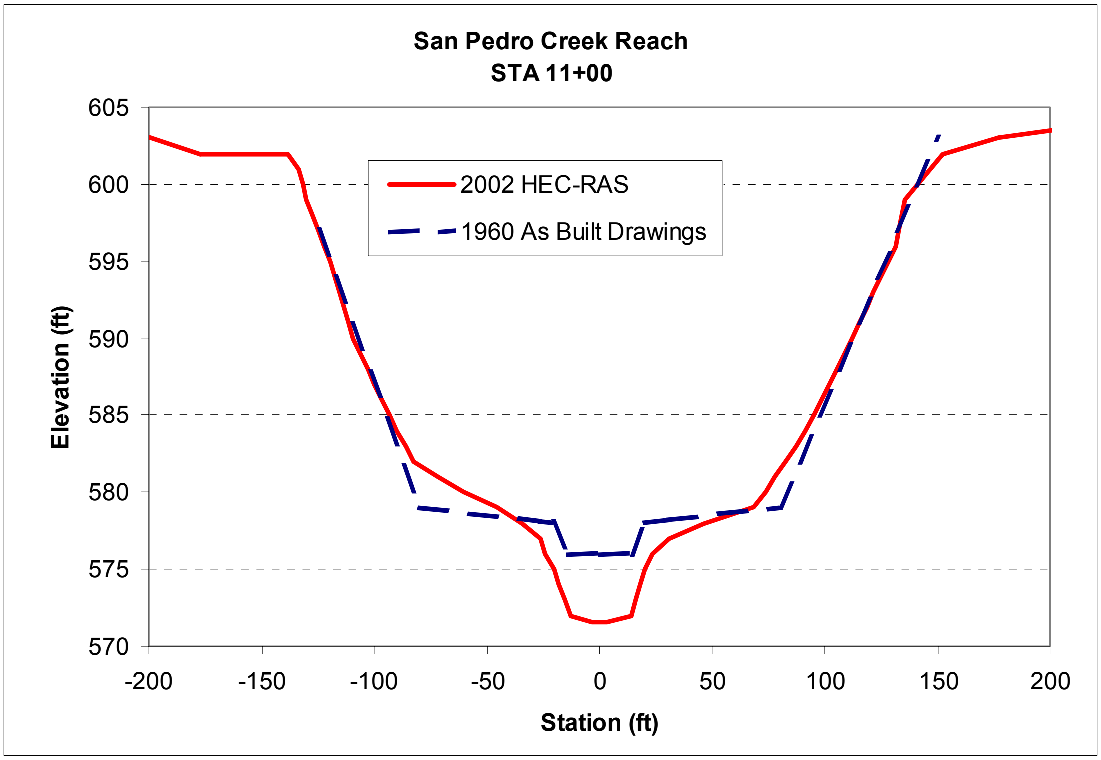

Figure C.7 Comparative Cross Sections in the San Pedro Creek Subreach

Comparative Cross Sections – Conception Subreach

Comparative cross sections in the Conception subreach downstream of confluence with San Pedro Creek indicate significant scouring of the pilot channel and minimal deposition in the interior overbank area. The pilot channel has widened from the as-built top width of 40 feet to as much as 110 feet and the channel bed elevation has lowered nearly 9 feet at the upstream end of this subreach. The existing floodway is approximately 300 feet in this subreach. The contribution of flow from the San Pedro Creek significantly impacts this channel geometry as the drainage area for the San Antonio River nearly doubles at the confluence location. The amount of scour of the main channel and lack of deposition in the overbanks indicate a highly erosive condition throughout this subreach. A sample comparative cross section for the Conception subreach is shown in Figure C.8.

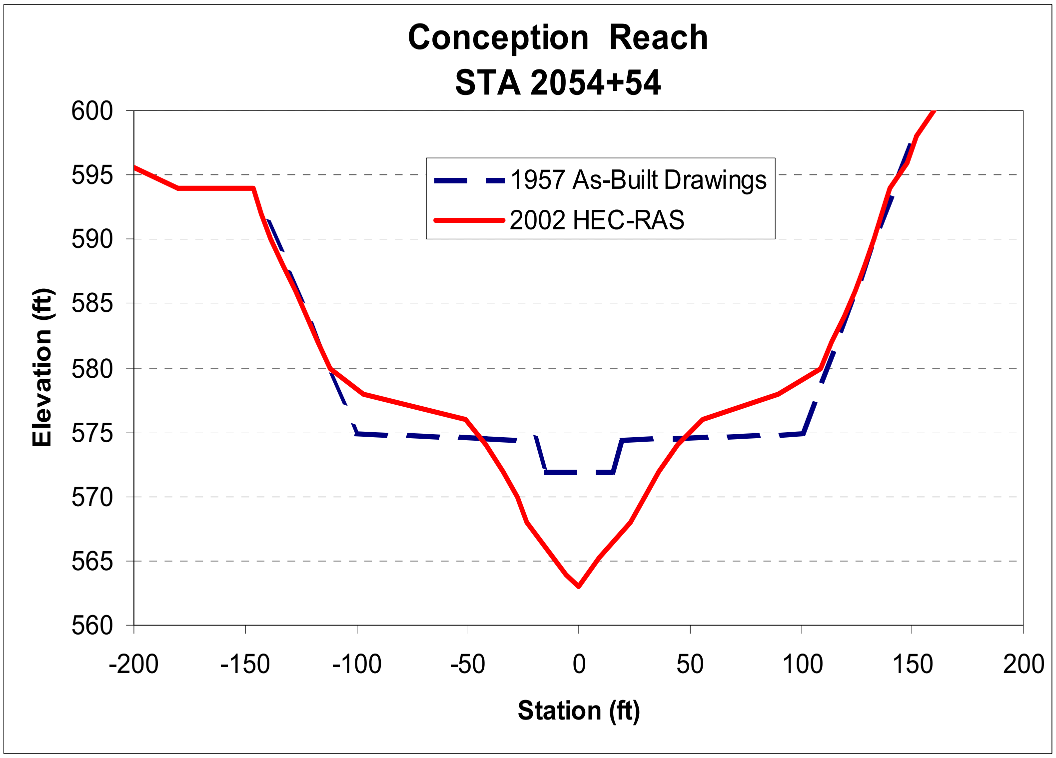

Figure C.8 Comparative Cross Sections in the Conception Subreach

Comparative Cross Sections – Mission Subreach

Comparative cross sections in the Mission subreach downstream of Mission Road and upstream of the influence of the San Juan Diversion structure also indicate significant scouring of the pilot channel and minor deposition in the interior overbank area. As with the Conception subreach, the pilot channel has widened to more than 100 feet in some areas and lowered in elevation more than 8 feet at the upstream end of this subreach. The amount of scour of the main channel and small amount of deposition in the overbanks indicate a highly erosive condition throughout the subreach. A sample comparative cross section for the Mission subreach is shown in Figure C.9.

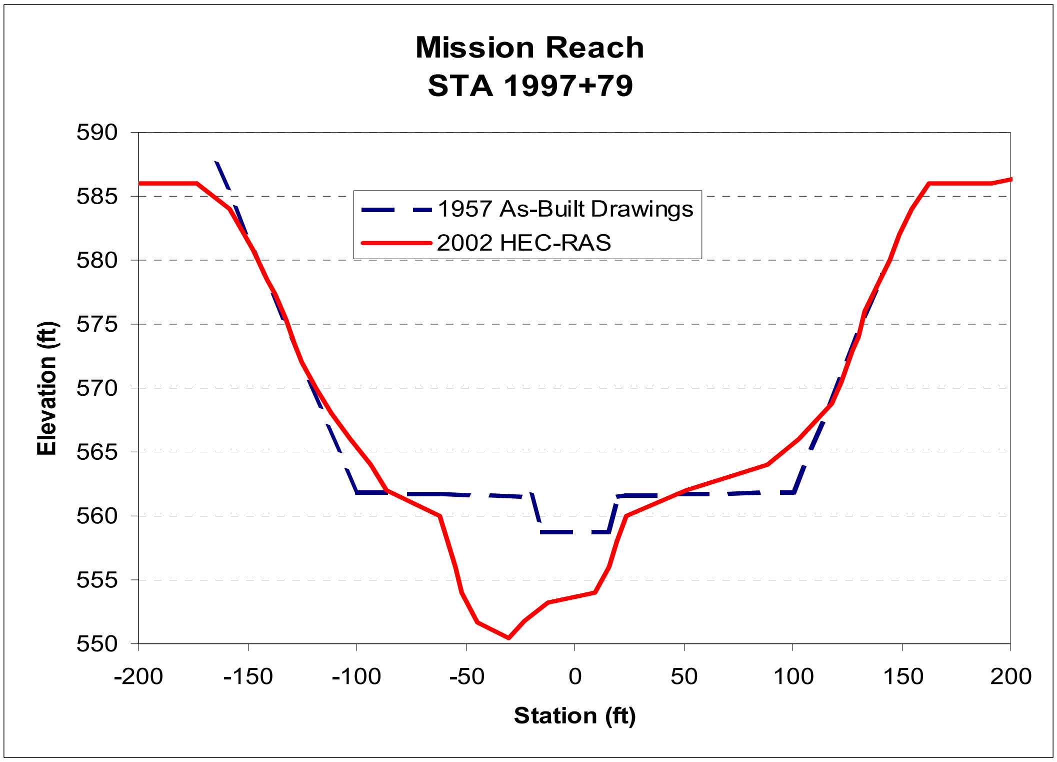

Figure C.9 Comparative Cross Sections in the Mission Subreach

Comparative Cross Sections – San Juan Subreach

Comparative cross sections in the San Juan subreach in the vicinity of the historic and new San Juan Diversion structures indicate some scouring of the pilot channel, but overall deposition in this subreach. The deposition is caused by the backwater effect from the diversion structure that was constructed to provide flow to the San Juan Acequia to the east. The channel in this subreach also becomes wider due to the backwater effect of the hydraulic structure. A sample comparative cross section for the San Juan subreach is shown in Figure C.10.

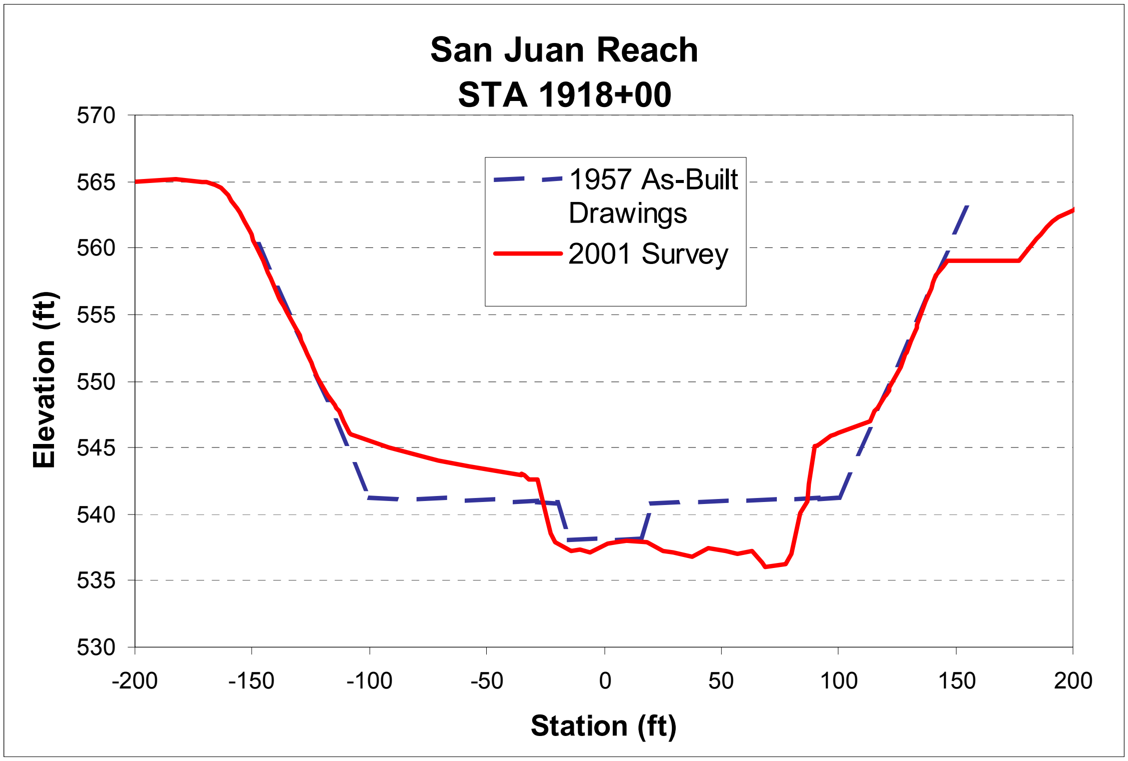

Figure C.10 Comparative Cross Sections in the San Juan Subreach

Comparative Cross Sections – Davis Subreach

Comparative cross sections in the Davis subreach upstream of Espada Dam indicate significant deposition throughout the reach with more than 10 feet of aggradation in the downstream end near the dam. Espada Dam acts as a hydraulic control for moderate to low flows, but becomes submerged at flows above the 10-year event. The pilot channel as constructed no longer exists and the main channel flow has concentrated on the right side of the floodway. A large point bar on the left side of the channel continues from Espada Dam to upstream beyond the confluence with Asylum Creek was evident as early as 1966 from aerial photography. Soil borings conducted in association with dredging efforts indicate mostly silts and clays compose the deposition features with layers of sand and gravel. A grain size analysis of a boring acquired in the large point bar near Espada Dam indicated 68.3% of the material passing the No. 200 sieve. Near surface samples collected in association with the current geomorphic study, had a D_{100} size of 75 mm, which indicates that although generally a depositional subreach, high flows can transport larger material through this subreach. A sample comparative cross section for the Davis subreach is shown in Figure C.11.

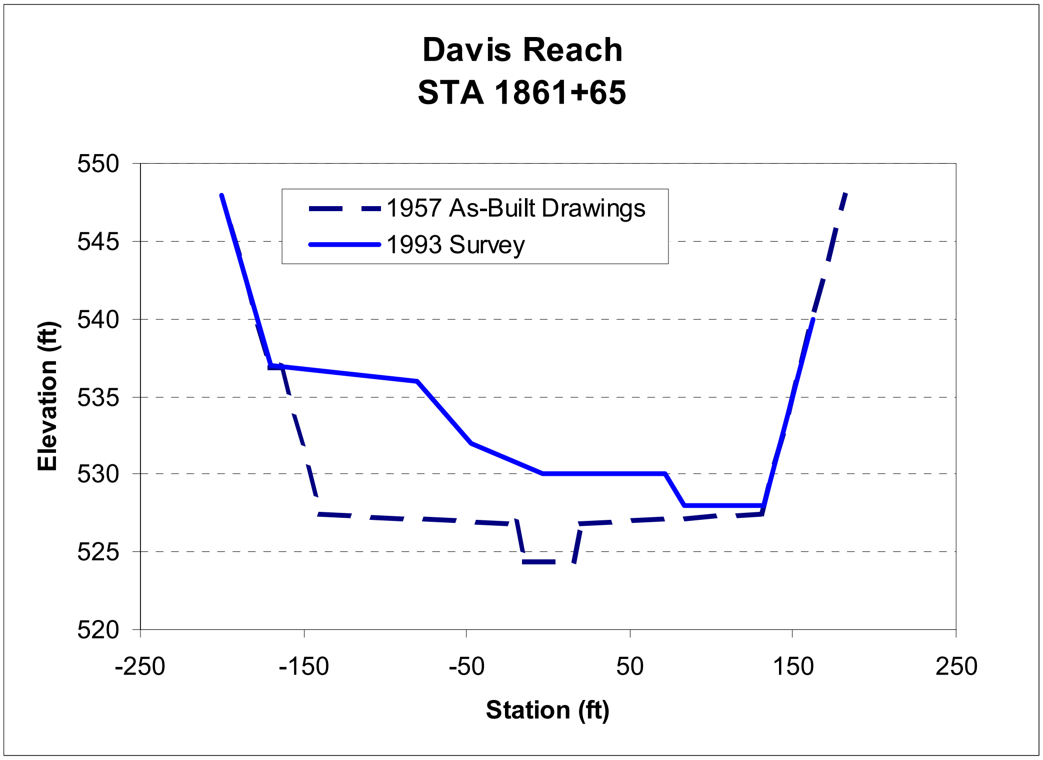

Figure C.11 Comparative Cross Sections in Davis Subreach

Comparative Cross Sections – Below Espada Subreach

Comparative cross sections in the Below Espada Subreach indicate significant scour of the pilot channel and deposition in the overbanks throughout the subreach. Similar to the Conception and Mission subreaches the pilot channel has widened to approximately 110 feet and lowered in elevation up to 8 feet below the as-built channel bed. Some of the long-term channel degradation is attributed to clearwater low-flows leaving Espada Dam. Also, the floodway expands to approximately 400 feet as compared to the 300 foot floodway in the upstream subreaches, which could contribute to some of the overbank deposition. This subreach displays both depositional and erosive characteristics since the overbanks have significantly aggraded and the pilot channel has concurrently degraded. The presence of these erosive and depositional features has been interpreted as follows; high flows deposit sediment in this reach and low flows subsequently head cut back up through the deposits to remove material from the main channel. Overall, the reach is considered depositional, while acknowledging extensive degradation of the pilot channel. A sample comparative cross section for the Below Espada subreach is shown in Figure C.12.

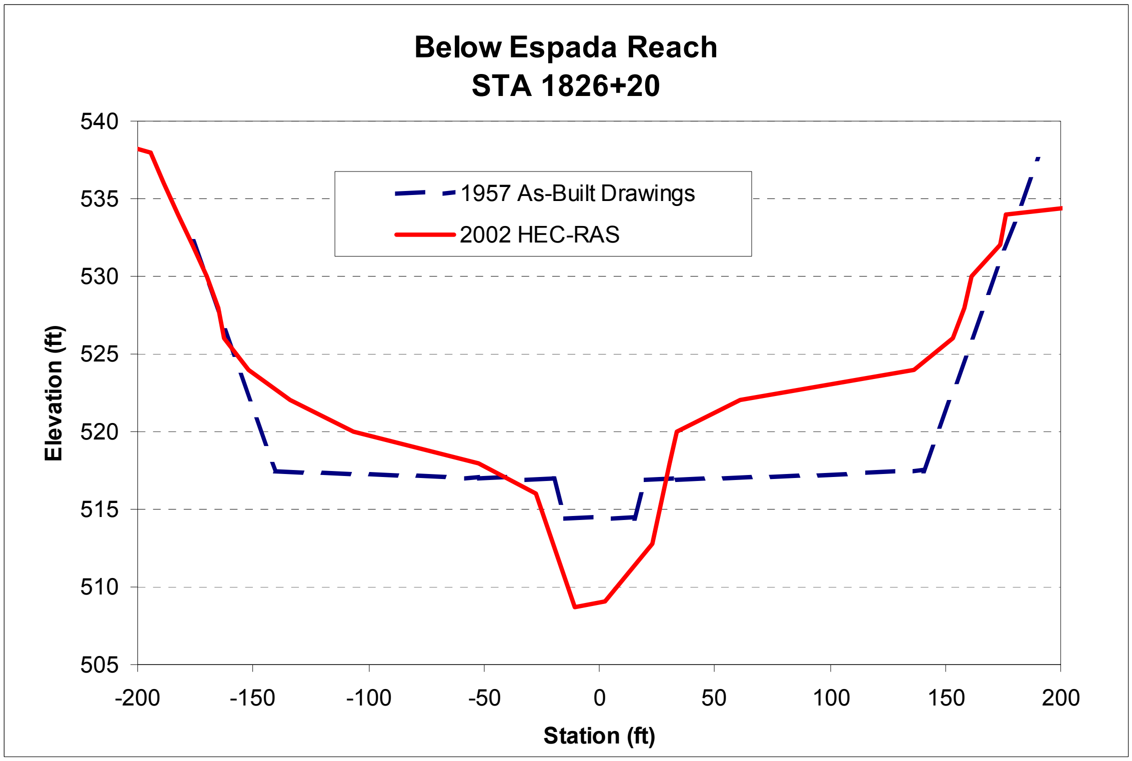

Figure C.12 Comparative Cross Sections in Below Espada Subreach

Comparative Cross Sections – Six Mile Subreach

Comparative cross sections in the Six Mile subreach near the confluence with Six Mile Creek have remained surprisingly stable. This may be attributed to the diversion of low flows into the remnant channel meander near the Ashley road crossing. However, hydraulic modeling indicates that the amount of flow diverted into the remnant channel is relatively small compared to that in the San Antonio main channel during storm events. The as-built plans do not indicate a pilot channel being constructed through much of this reach and channel incision is not apparent. Therefore, the reach is considered overall stable. A sample comparative cross section for the Six Mile subreach is shown in Figure C.13.

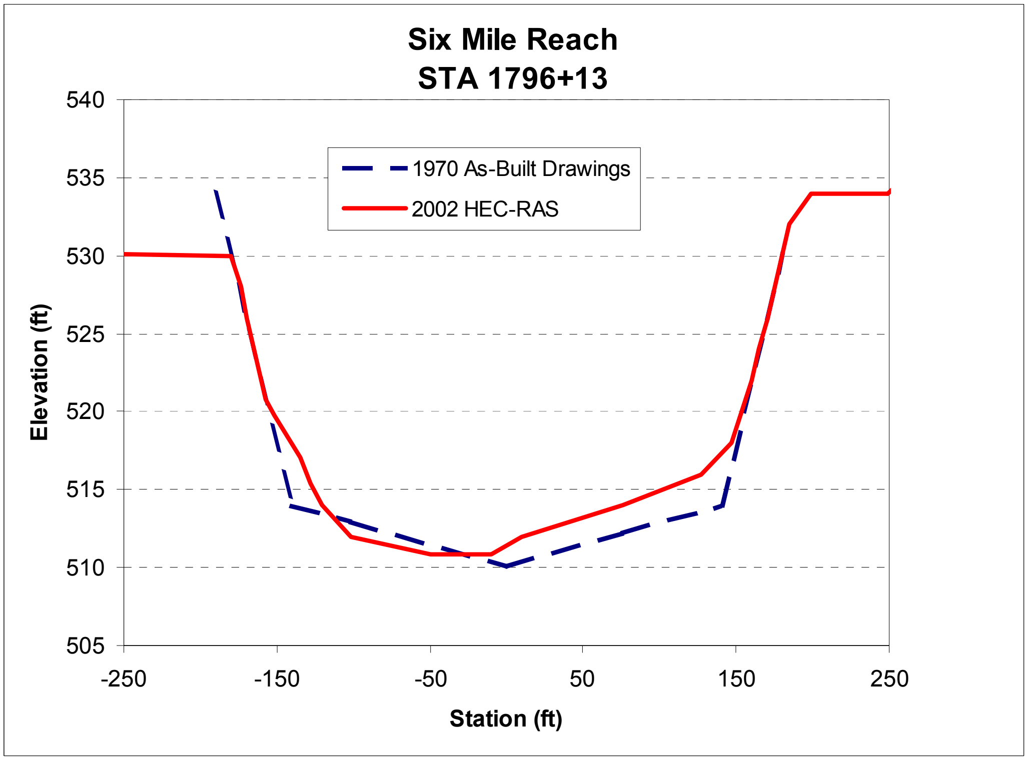

Figure C.13 Comparative Cross Sections in Six Mile Subreach

Comparative Cross Sections – 410 Subreach

Similar to the Below Espada subreach, comparative cross sections in the 410 subreach indicate significant scour of the pilot channel and deposition in the overbanks. The pilot channel has widened to as much as 110 feet and lowered up to nearly 8 feet in some locations as compared to the as-built condition. Because the overbanks have significantly aggraded and the pilot channel has concurrently degraded, it is assumed that high flows deposit sediment in this reach and subsequent low flows remove the excess material from the main channel over time. Therefore the reach is considered overall depositional, while acknowledging excessive degradation and widening of the pilot channel. A sample comparative cross section for the 410 subreach is shown in Figure C.14.

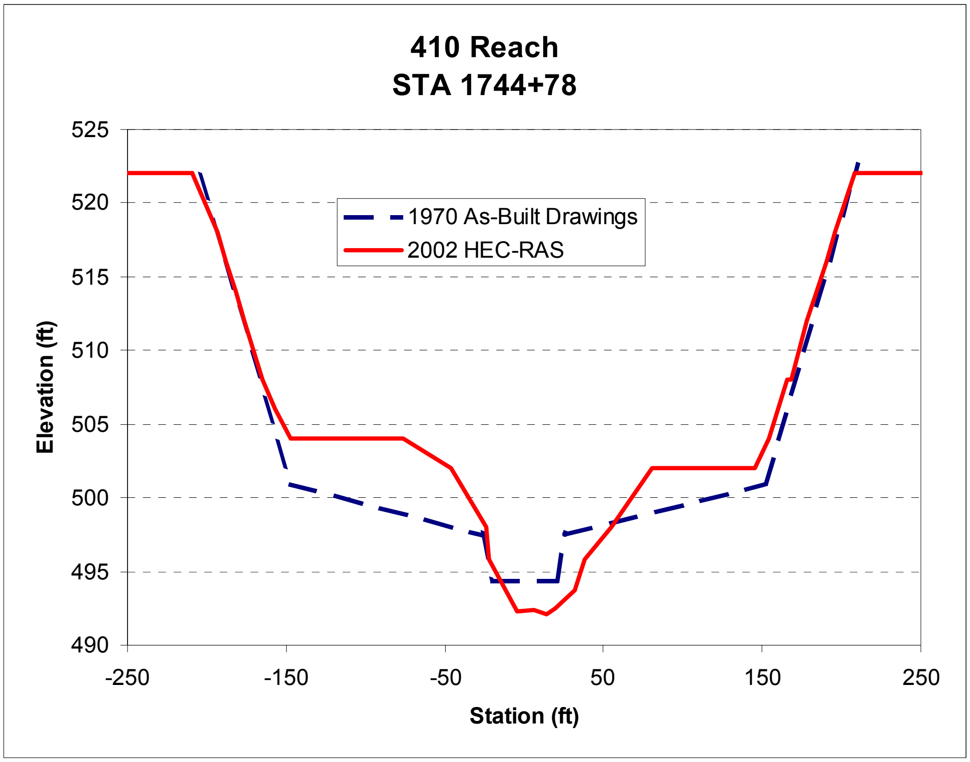

Figure C.14. Comparative Cross Sections in 410 Subreach

Sediment Distribution in the San Antonio River

Analysis of the historical cross sections were used to investigate the amount and distribution of sediment stored or eroded from the in the San Antonio River since construction of the floodway. This information was used to assess erosion and deposition patterns throughout the southern project subreaches. Sediment accumulation values were computed from the change in cross sectional area multiplied by distance along the south reach corridor. The comparative cross sections were subdivided into channel and overbank areas and the difference in section areas were computed. The result is the cumulative sediment accumulation since construction of the San Antonio River floodway as represented in Figure C.15.

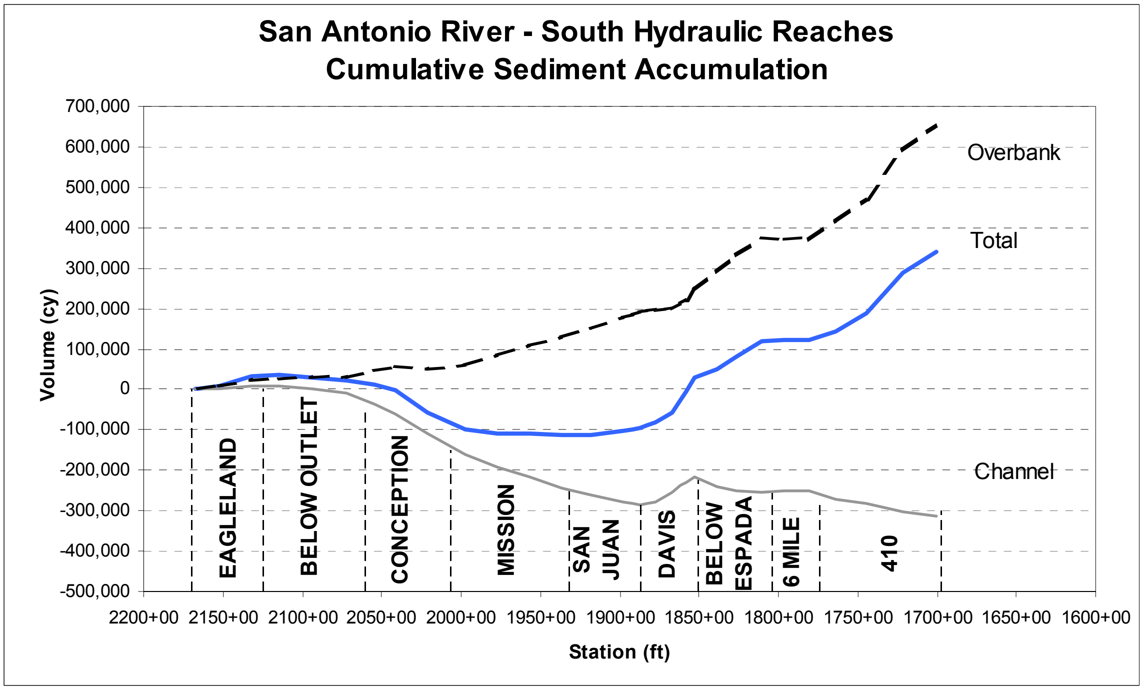

Figure C.15 Historical Sediment Accumulation in the San Antonio River (As-Built to Existing Condition)

A positive slope indicates sediment accumulation and a negative slope indicates sediment erosion. The analysis shows a general loss of material (erosion) in the main channel and deposition in the overbank areas. The cumulative net increase in volume of approximately 350,000 cy^3 was computed for the entire South Reach. Although approximately 300,000 cy^3 of material has been eroded from the channel bed and banks, approximately 650,000 yd^3 of sediment has been stored in the overbank areas. Almost one third of the material stored in the reach since construction (~120,000 cy^3) is behind Espada Dam. However, the analysis indicates there has also been a significant amount of sediment deposited in the overbank areas below the dam. Nearly 100,000 cy^3 has been deposited in the Below Espada subreach and more than 200,000 cy^3 has been stored in the 410 subreach. The sediment accumulation analysis indicates that nearly 300,000 cy^3 of sediment has been eroded from the channel below the San Pedro confluence to Davis Lake. The sediment accumulation analysis was used to compute the average incremental gain or loss of material in each subreach to provide a description of the historic erosion or deposition nature of each subreach since construction of the floodway. This was computed as the average incremental sediment accumulation value divided by the length of reach. The average incremental sediment accumulation was described as the unit volume per length of channel and is summarized in Table C.3

Table C.3 Average Unit Incremental Sediment Accumulation Values			
Subreach	Channel	Overbank	Total Section
	(cy/ft)	(cy/ft)	(cy/ft)
EAGLELAND	1.7	4.9	6.7
BELOW OUTLET	-2.5	1.0	-1.5
CONCEPTION	-22.3	4.9	-17.4
MISSION	-18.0	10.5	-7.5
SAN JUAN	-8.7	12.9	4.2
DAVIS	19.6	16.4	36.0
BELOW ESPADA	-8.3	27.4	19.2
SIX MILE	1.6	-1.2	0.4
410	-8.0	36.7	28.7
SAN PEDRO CREEK	-5.3	6.1	0.8
South Reach Average	**-6.7**	**14.1**	**7.4**

In the subreaches upstream of the San Pedro confluence (Eagleland and Below Outlet) the cross sections have been relatively stable with some minor deposition in the Eagleland subreach and minor scouring of the channel below the tunnel outlet. The average channel and overbank incremental sediment accumulation values are less than 5 cy/ft in these subreaches. It is assumed that the sheet pile grade control structures in the Below Outlet subreach have acted to reduce some of the degradation potential in this subreach. Downstream of the San Pedro confluence in the Conception and Mission subreaches, the historic geomorphic channel response has been significant channel incision of the main pilot channel and low to moderate deposition in the overbank areas. The incremental sediment accumulation values have been near –20 cy/ft for the channel in the Conception and Mission subreaches. In the San Juan subreach the pilot channel has been slightly erosive, but overall depositional as indicated by the total incremental sediment accumulation value of 4.2 cy/ft. In the Davis subreach the trend has been primarily depositional due to long-term aggradation of fine sediments upstream of Espada Dam and deposition on the receding limb of periodic floods and as reflected by the total sediment accumulation value of 36 cy/ft. Below Espada Dam, the trend has been channel incision and widening of the main pilot channel and significant deposition in the overbanks areas. In the Below Espada subreach channel widening and incision of the pilot channel have been observed, but significant deposition on the order of 27.4 cy/ft has occurred in the overbank areas since construction of the floodway channel. In the Six Mile subreach near the confluence with Six Mile Creek the channel bed elevation has remained stable. Channel bed stability through this subreach may be attributed to the diversion of low flows into the old meander near Ashley Road. In the 410 subreach, below the remnant meander confluence to the end of the project; the pilot channel has incised similar to that observed in the Below Espada subreach, but the overbank areas have aggraded resulting in an overbank accumulation rate of 36.7 cy/ft. In San Pedro Creek the channel erosion values where relatively low compared to those computed for the subreaches below the confluence. This could indicate that San Pedro Creek may not have contributed a large amount of sediment as compared to the transport capacity downstream. However comparing to the average values the transport capacities are nearly equivalent.

Channel Profile Analysis

Comparison of channel and overbank profiles was performed to evaluate the degradation or aggradation trend in each subreach and to compute their corresponding slopes. Data from the comparative cross section analysis were used to develop the existing and as-built profiles and channel slopes. Additionally information contained in the as-built plans was used to develop the historic channel profile (pre-1957). The channel profiles were then used to calculate the change in slope that has occurred over the last several decades. This information will be useful in design as the project subreaches should be designed to convey the incoming sediment load without excessive erosion or deposition and likely would not be much flatter than the historic slope. However increases in runoff and reductions in incoming sediment could require a flatter slope than the historic condition for channel stability. Plots of the channel bed profile and average overbank elevations are shown in Figures C.16 and C.17. Historic profiles of overbank elevations were not developed.

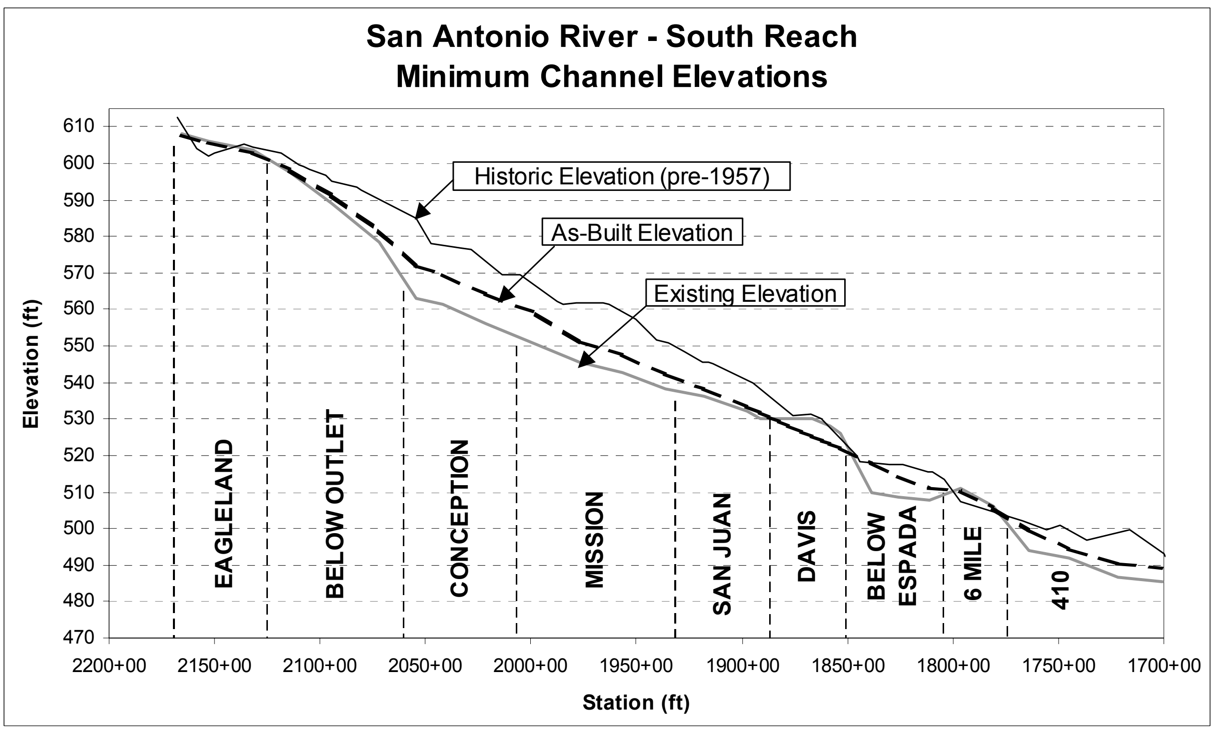

Figure C.16 Historic and existing longitudinal profiles of the South Reach channel bed elevations.

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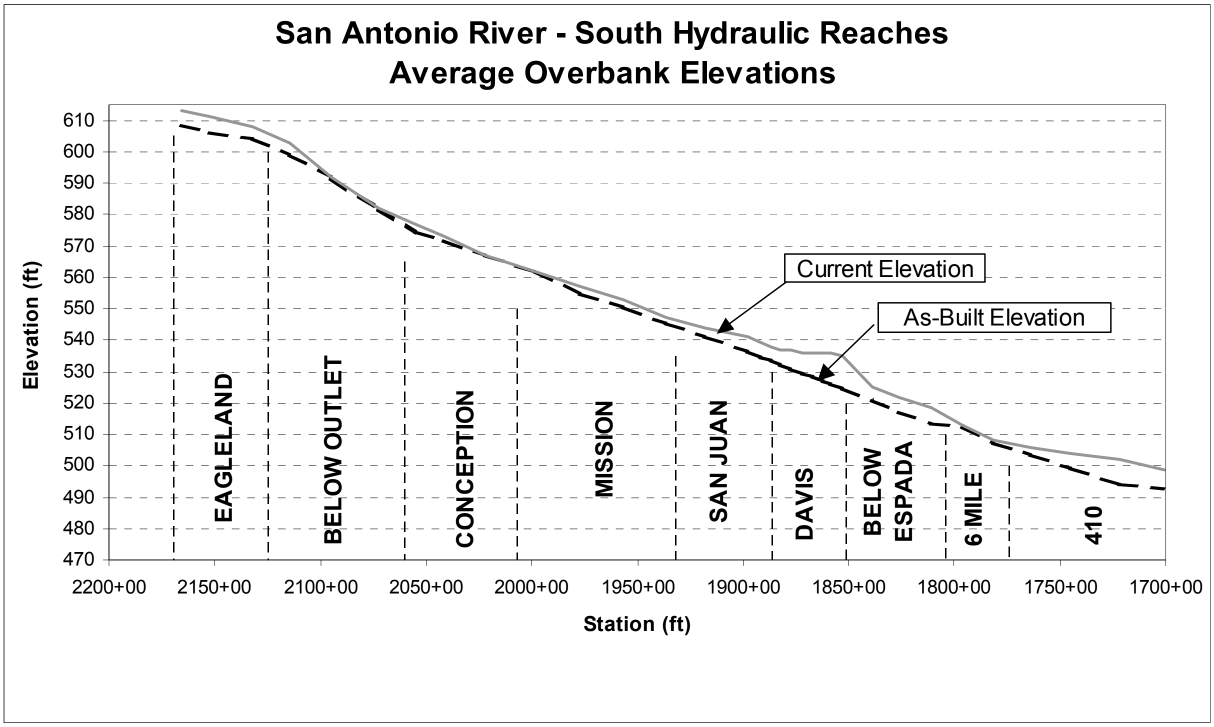


Figure C.17 Historic and existing longitudinal profiles of the South Reach overbank elevations.

Similar to the previous analyses, the profiles show stability in the Eagleland subreach and the Six Mile subreach. There is some erosion potential in the Below Outlet subreach and significant erosion potential in the Conception and Mission subreaches. It can be seen that the channel bed was lowered by approximately 10 feet in these subreaches when the floodway was constructed. The transition area to a more depositional environment occurs in the San Juan and Davis subreaches due to the influence of the San Juan Diversion structure and Espada Dam. Channel incision and significant aggradation in the overbanks is evident in the Below Espada and 410 subreaches as mentioned previously. This is important with respect to channel design, as the Conception and Mission Reaches will likely require substantial channel armoring and grade control and the reaches below Espada Dam will require accommodation for sediment deposition.

Using the channel profiles, channel bed slopes for each of the subreaches were computed. This reach-based analysis is important because river systems can respond to discontinuities in sediment transport by either decreasing or increasing slope. In erosive environments, the channel slope will typically decrease until an equilibrium condition is achieved. In aggrading systems, the typical channel response is to increase the slope until sufficient sediment transport capacity is attained to move the incoming sediment load through the system. The computed channel bed slopes are presented in Table C.4.

Table C.4 Comparison of Channel Bed Slopes			
Subreach	Average Channel Bed Slopes		
	Historic (pre 1957)	As- Built	Existing
EAGLELAND	0.10%	0.14%	0.13%
BELOW OUTLET	0.16%	0.39%	0.45%
CONCEPTION	0.17%	0.23%	0.21%
MISSION	0.16%	0.26%	0.20%
SAN JUAN	0.21%	0.24%	0.23%
DAVIS	0.09%	0.27%	0.29%
BELOW ESPADA	0.14%	0.25%	0.08%
SIX MILE	0.27%	0.30%	0.32%
410	0.11%	0.17%	0.14%
South Reach Average	0.16%	0.25%	0.22%

The Eagleland subreach is influenced by a backwater caused by the tunnel outlet weir, and therefore, Eagleland has remained relatively stable with a bed slope that has not changed significantly. In the Below Outlet subreach, the slope has actually increased because the downstream end has lowered but the upstream end has remained fairly stable. The increase in slope does not indicate an aggrading system, but rather, a response that has been controlled by the grade control structures in this reach. In the Conception and Mission subreaches, the response has been typical of a degrading system with a decrease in slope over time. The channel slope has decreased in response to a deficient sediment load as compared to the channel sediment transport capacity. However, the goal of the subsequent analyses will be to determine at what slope would the channel be relatively stable. Theoretically this should not be less than the historic slope of 0.16% -m 0.17% in this area. The sediment transport analysis will be used to help better define the magnitude of the equilibrium slope required for stability. In the San Juan Reach, the slope has decreased slightly in response to the minor channel incision observed in this subreach. Since the Davis reach has aggraded, an increase in slope has been observed. Below Espada Dam, the channel has degraded to a slope of 0.08%, which could be used as a potential indicator of the equilibrium slope required in this system. The Six Mile subreach has slightly aggraded and increased slope, accordingly. The channel in the 410 reach has degraded to 0.14%. The slope analysis shows the basic trends and where the existing pilot channel has adjusted since its construction, but does not indicate whether this adjustment occurred instantaneously (or episodically) or has occurred steadily over the past several decades. To assess whether or not the channel has continued to degrade in the recent decade, a specific gage analysis was performed as discussed in the next section.

Specific Gage Analysis

To further assess the stability of the South Reach a specific gage analysis was performed using information at the San Antonio River at Loop 410 gage (# USGS 08178565). The specific gage analysis was performed using low-flow measurements obtained from 1987 to 2002 and published by the USGS at this site. Since the gage is located in the 410 subreach and the measurements are from a recent time period, the low flow specific gage analysis will provide a more current assessment of the stability of the pilot channel in this lower reach. A running 5-year average of low flow measurements were used to develop a stage versus discharge relationship for the period of record at this gage. Groups of measurements representing each base year included the two years prior and two years following the base year. The group of measurements was used to develop a regression line relating stage as a function of discharge for each base year. The regression equations were used to compute the stage for a specific discharge selected for the analysis. A discharge of 100 cfs was selected and the corresponding stage for each base year was computed. The results of the analysis and shown in Figure C.18.

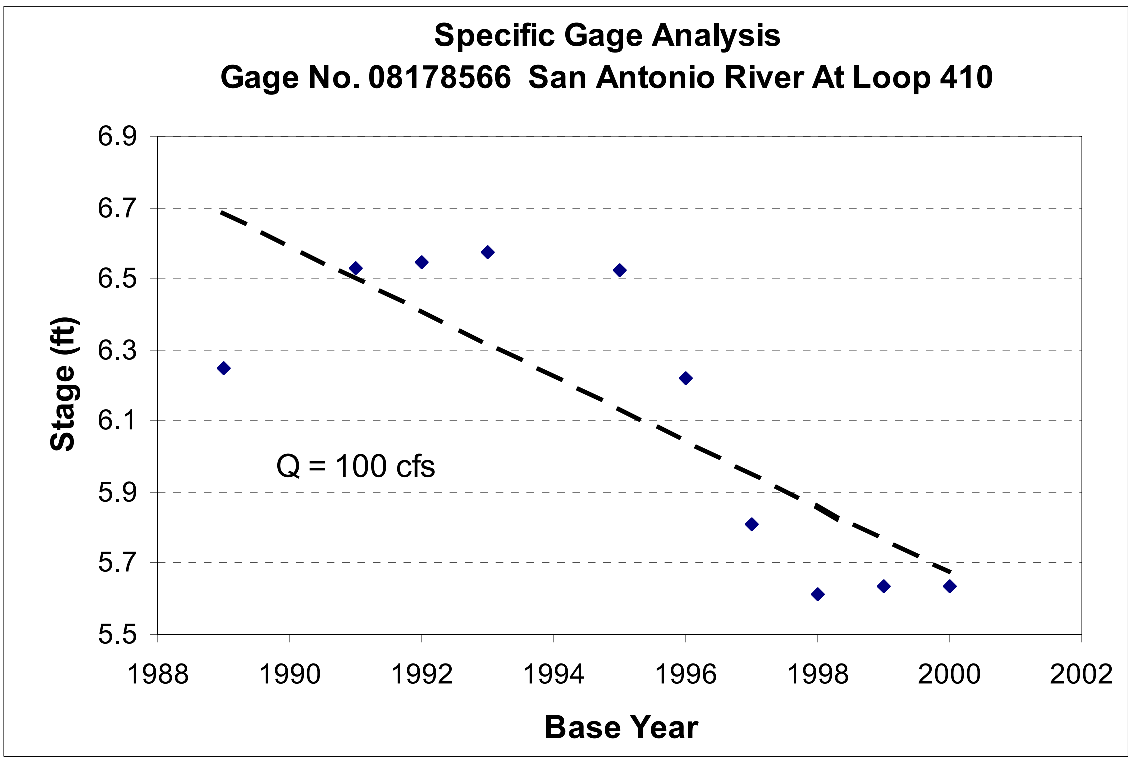


Figure C.18 Specific Gage Analysis Results for San Antonio River at Loop 410

The results indicate a downward trend in stage over the period of record. The decrease in stage is approximately 1 foot over the 14 year period. This corresponds with the amount of degradation observed in the main channel throughout the 410 subreach. The main channel of the 410 reach has experienced on average about 4 feet of degradation in the last 30 years, which could equate to more than 1 foot every 10 years. The specific gage analysis suggests that that the trend has continued to occur over the recent ten year period and could likely be expected into the future.

Historic Channel Assessment Conclusions

The results of the historic channel assessment shows how the San Antonio River has been manipulated from its natural condition and how it has responded to these imposed conditions. Since construction there has been downward trend in channel elevation, channel widening and aggradation in the overbanks of the South Reach especially downstream of Espada Dam. The influence of hydraulic structures such as the San Antonio Tunnel, grade controls in the Below Outlet subreach, the San Juan Diversion and Espada Dam have controlled or reduced the channel incision process in some areas. However on a larger scale, the combined influences of the channel straightening and changes in hydrology have changed the equilibrium between the new flood hydraulics and resistance of the base-level fluvial geology.

The current channel improvement designs will require development of more frequent hydraulic structures or modifications to the base-level fluvial geology to minimize future erosion and/or aggradation. This may be accomplished by designing channel features that provide a condition of sediment continuity over time; meaning the channel will transport the incoming sediment load while concurrently improving the resistive nature of the underlying fluvial geology. Alternatively the channel could be allowed to freely adjust its existing boundaries until the planform, profile, and geometry become such that the incoming sediment load is transported through the system. This alternative is not desirable considering the goal of this project and the objectives defined by SARA and the San Antonio River Oversight Committee. Therefore, improvements to the channel should include consideration for sediment transport and the corresponding geomorphic processes within the geologic confines of the human imposed infrastructure. The following analyses will attempt to quantify the dominant sediment loads, the sediment transport ability of the system and provide hydraulic conditions under which a condition of sediment continuity may be attained. The historical assessment provides the foundation for this analysis and sets some bounds on what can be reasonably achieved.

Hydrology and Hydraulics

Much of the hydrologic and hydraulic data used in this study were derived from models developed for evaluation of flood conditions on the San Antonio River. A limited map maintenance program (LMMP) study was conducted with cooperation between the City of San Antonio, SARA, Bexar County, the ACOE Ft. Worth District and HDR Engineering, Inc. to estimate flood flows and corresponding water levels throughout the San Antonio River basin. In addition to these data supplementary information was obtained from USGS gaging stations in the area. The following sections describe the hydrology and hydraulics used for assessment of channel stability and sediment transport to support the current design of channel improvements on the San Antonio River.

Hydrology

Hydrology of the project area can be described as flashy, with baseflow sustained through interactions with the underlying geology. The San Antonio River basin exists in the subtropical subhumid region of Texas. The San Antonio River receives main channel flow from the Olmos Creek Basin, but the actual headwaters of the San Antonio River are distinguished by springs upstream of Hildebrand Avenue now located in the College of the Incarnate Word. The upper Olmos Creek basin exists over the Edwards Aquifer outcrop and recharge zone, which is comprised primarily of porous limestone and calcareous material on channel gradients of approximately 0.5 - 0.6%. The San Antonio River transitions to milder slopes of approximately 0.1 – 0.2% impacted by springflow from the Edwards aquifer and urban runoff. The lower San Antonio River within the limits of the project is located on the Carrizo-Wilcox outcrop, which is comprised of transmissive sandstones and similar material. With an average annual rainfall of approximately 30 inches, it is common for the San Antonio River basin to experience high intensity storms with several inches of rainfall at any given time. Wet months are typically in the spring and fall seasons. The basin may also experience extended drought periods within the common cycle. Flood flow hydrology for this study was partially derived from hydrologic models developed for the LMMP and flow duration data was developed from information obtained from the USGS gaging stations as described in the following sections.

Flood Flow Hydrology

Peak flow discharge values for the study were derived using rainfall-runoff models and precipitation values defined in the City of San Antonio Unified Development Code (UDC). A runoff model was originally developed for the LMMP using the Hydrologic Modeling System HEC-HMS (USACE, 2000), but was converted to the HEC-1 Flood Hydrograph Package (USACE, 1998) due to limitations in the HMS model. The approved HEC-1 model included hydrology for the 2- through the 500-year storm events. A complete description of the hydrologic model and calibration are provided in other technical memorandum developed for the LMMP (HDR Engineering, 2002). For evaluation of channel stability low flows less than 2-year flood were also computed with the calibrated HEC-1 model. These low flows were generated using rainfall depths extrapolated from the City of San Antonio UDC. Rainfall depths and associated frequencies were extrapolated to compute hypothetical 0.25-, 0.5-, 0.75-, and 1-year storm flows. The frequencies of these events are not imperative, rather it was desired to include a lower end to the flow distribution for use in development of sediment transport rating curves. The UDC values and extrapolated rainfall depths are shown in Figure H.1

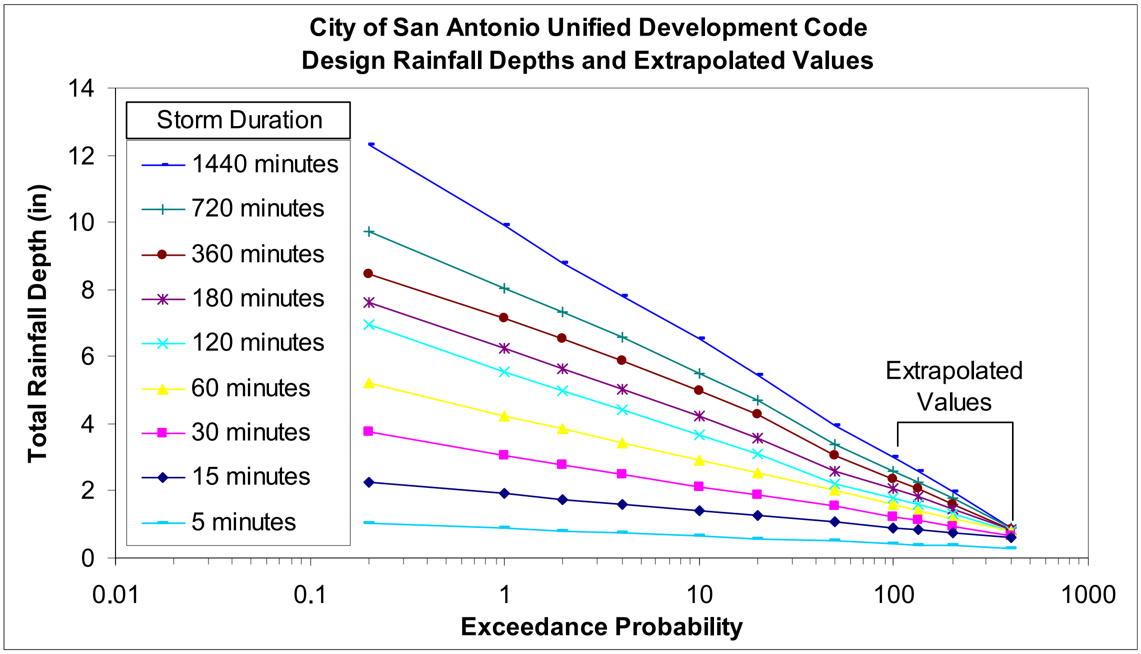


Figure H.1 City of San Antonio Design Rainfall Depths and Extrapolated Values

Including the extrapolated rainfall depths, a total of 11 storm events (0.25- through 500-year) were simulated to provide a range of discharges to be used in the HEC-RAS (HEC, 2001) hydraulic model described in the following section. The distribution of discharges in the San Antonio River basin as represented in the HEC-RAS model are shown in Figures H.2 and H.3.

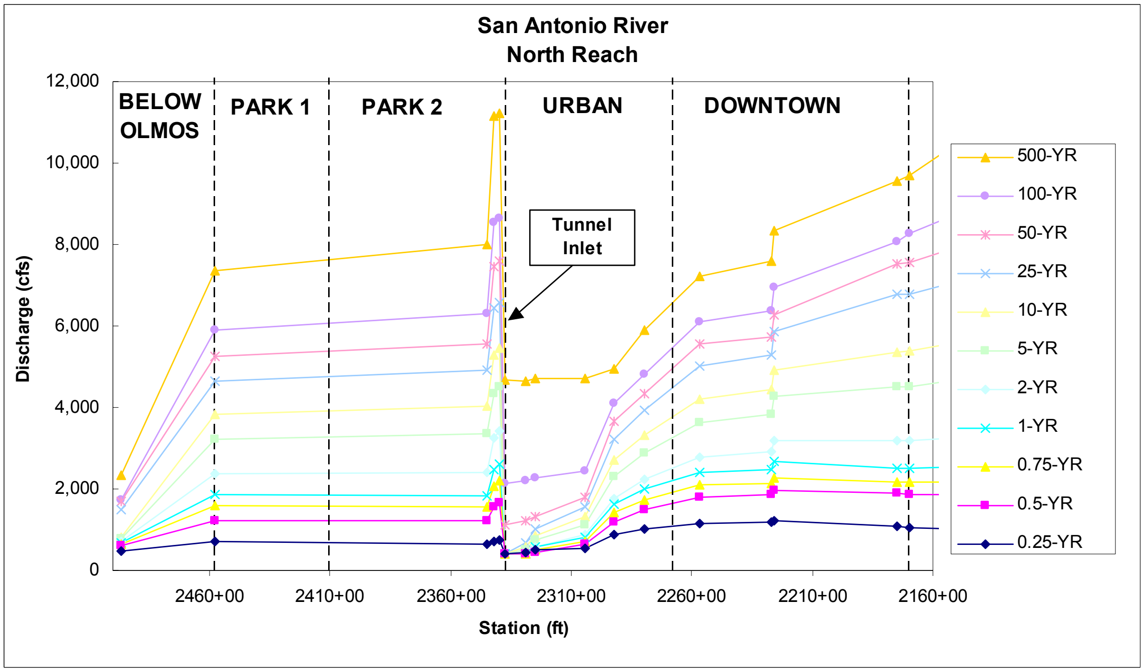


Figure H.2 Hydrology for the San Antonio River – North Reach

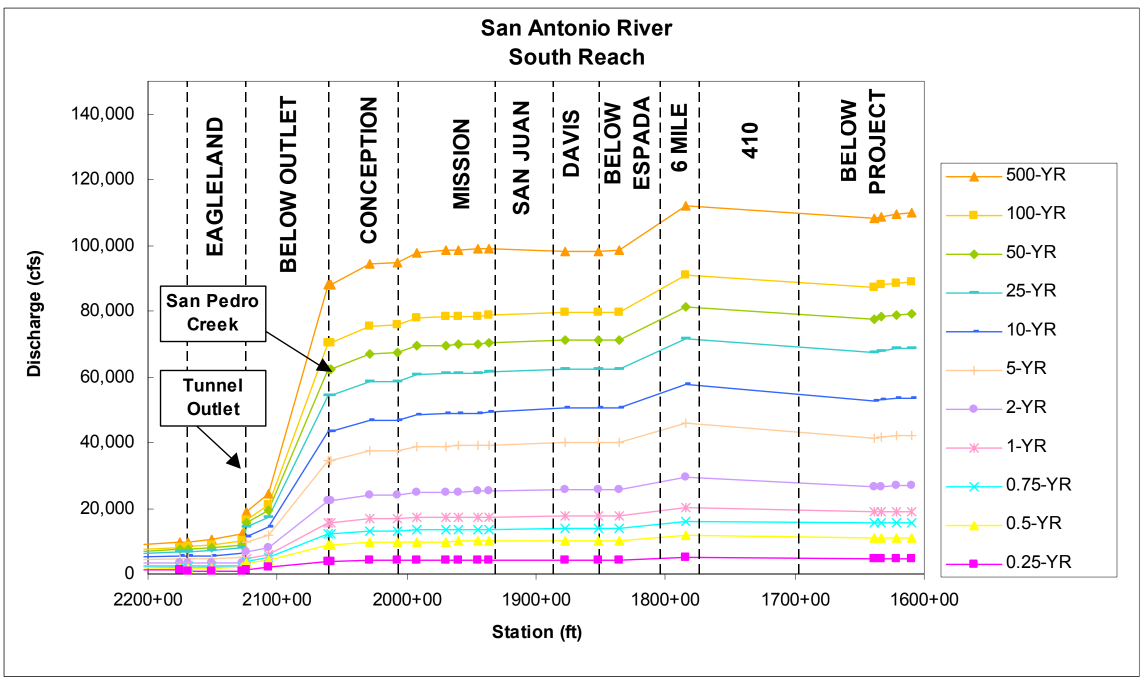


Figure H.3 Hydrology for the San Antonio River – South Reach

For the North Reach the most significant features that impact flood hydrology are Olmos Dam constructed in 1926, and the San Antonio Tunnel constructed in 1995. Olmos Dam exists immediately upstream of the North Reach. With a design capacity of 12,600 ac-ft Olmos Reservoir can capture up to 7.4 inches of rainfall in the upstream 32 square mile basin before overtopping the spillway. Olmos Reservoir is normally empty during dry periods and is used to provide flood storage and during storm events. The San Antonio Tunnel has a design capacity of approximately 6,500 cfs during the 100-year flood and is operated to divert flows around the Urban, Downtown and Eagleland subreaches. Under low to moderate flow conditions (< 50-year) the tunnel is operated such that a maximum of 400 cfs is released downstream to the Urban subreach. For higher flows, the tunnel weir becomes overtopped and the tunnel capacity is maximized. Intervening drainage through the downtown area is significant and can exceed 6,000 cfs during the 100-year flood. For the South Reach the most significant influences are the return flow from the San Antonio Tunnel and the contribution of flow from San Pedro Creek. The tunnel outlet is located at the downstream end of the Eagleland subreach. San Pedro Creek and its tributaries drain most of the urbanized west side of San Antonio. The drainage area for the San Antonio River essentially doubles (~45 to 90 mi^2) at the confluence and the channel width of the San Antonio River increases by more than a factor of 2 at this location.

Flood hydrology can also be evaluated using stream flow data recorded at gaging stations on the river. A flood frequency analysis can be performed using a history of annual peaks recorded at these stations. The gages that would be applicable to the area of interest are the USGS gages at South Alamo Street (USGS 08178000), Mitchell Street (USGS 08178050) and the gage at Loop 410 (USGS 08178565). There is a gage further upstream on Olmos Creek at Dresden Drive (USGS 08177700), but this exists upstream of Olmos Dam. The next gage downstream is the San Antonio River near Elmendorf (USGS 08181800) below the confluence with the Medina River, which has significantly different drainage characteristics than the urbanized upstream basins. The records at loop 410 and Mitchell Street are only 15 years and 9 years, respectively and therefore cannot be used in a flood frequency analysis. The flow record at the S. Alamo gage is more than 85 years, but the influence of urbanization may preclude the use of some of this data. A flood frequency analysis was performed for the S. Alamo gage using the most recent 50 years of data to capture some of the influence of recent urbanization. The HEC-FFA program (HEC, 1995) was used for the analysis. The results of the flood frequency analysis and from the HEC-1 model compared favorably at the S. Alamo gage location. This confirmed the viability of using results from the HEC-1 model for channel stability analysis. Results from the comparison are shown in Figure H.4.

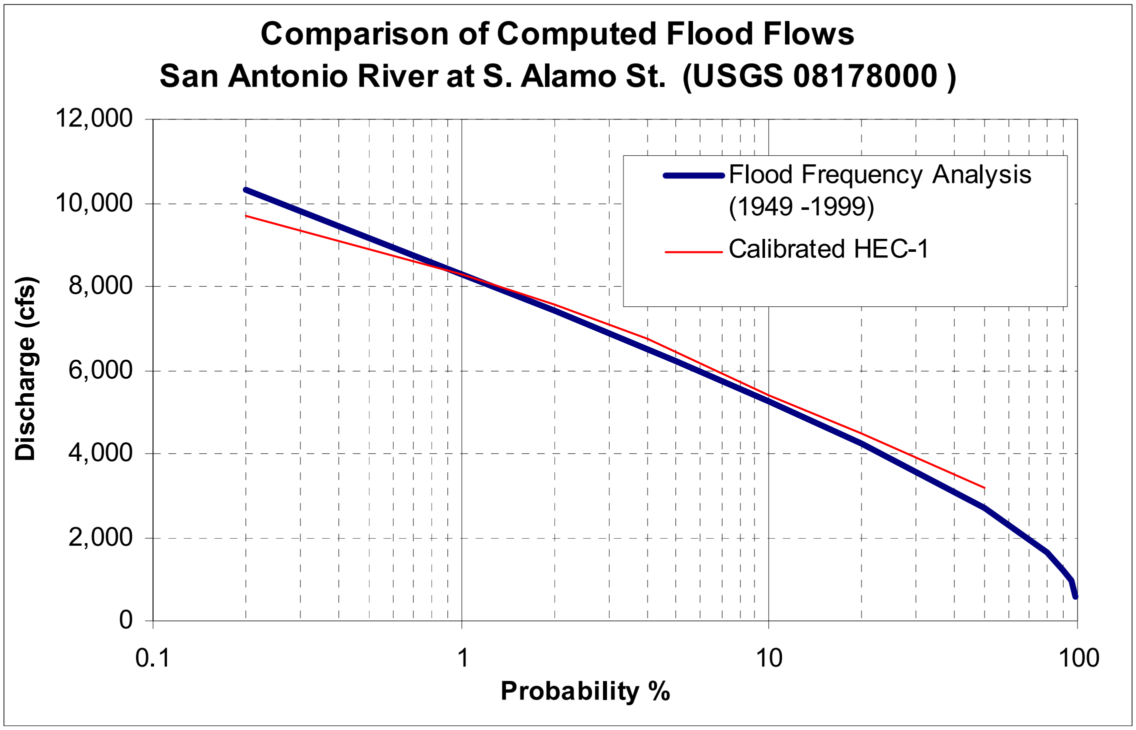


Figure H.4 Comparison of HEC-1 and FFA Flood Frequency Values

The analysis indicates the HEC-1 and FFA compare favorably with less then a 6.5% variance for flows greater than the 2-year flood. The frequencies from the HEC-1 model begin to diverge significantly from the flood frequency values for discharges less than the 2-year return period and therefore, should not be associated with a probability of occurrence.

Long Term Hydrology

In addition to flood flow hydrology long term, flow duration information was developed for the gages at Loop 410 and at S. Alamo. Unit value and mean daily flow data was used to determine exceedance probabilities and percent time occurrences for specific flow ranges. Much of this information was developed as a precursor for the computation of sediment yields as described in the Sediment Transport Analysis section. Daily average flows were available for the period of record at the S. Alamo and 410 gages. In addition approximately 10 years of 15-minute unit values were obtained for the Loop 410 gage. Flow duration curves for the total and base flow were developed for the S. Alamo gage using the period of record. For the 410 gage flow duration curves for total and baseflow were developed using the mean daily average for the period of record and also total flow duration was computed using the 15-minute unit values for water years 1991 - 2001.

The flow duration curves were subdivided using breakpoints to define specific areas of duration. Characteristic of flashy systems, the curves indicate that the most significant portion of high duration flows are those less than approximately 400 - 200 cfs. This could be considered the cutoff for base flow and all flows above this level are most directly storm driven. At the S. Alamo gage flows above 155 cfs comprise less than 8% of the flow record. It is expected that urbanization would increase this number closer to 200 cfs. At the Loop 410 gage flows above 400 cfs comprise less than 5% of the flow record. For the flow ranges defined the percent of time these flows persist were computed as shown in Figures H.5 and H.6.

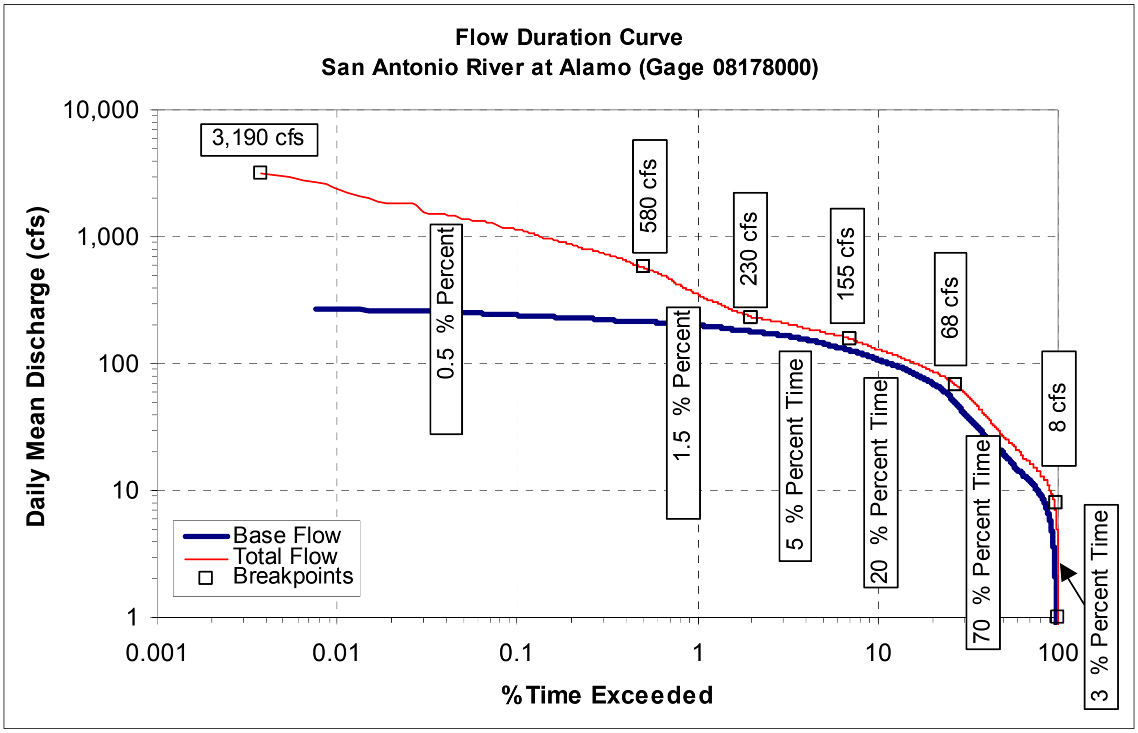


Figure H.5 Flow Duration Curves for San Antonio River at Alamo Gage

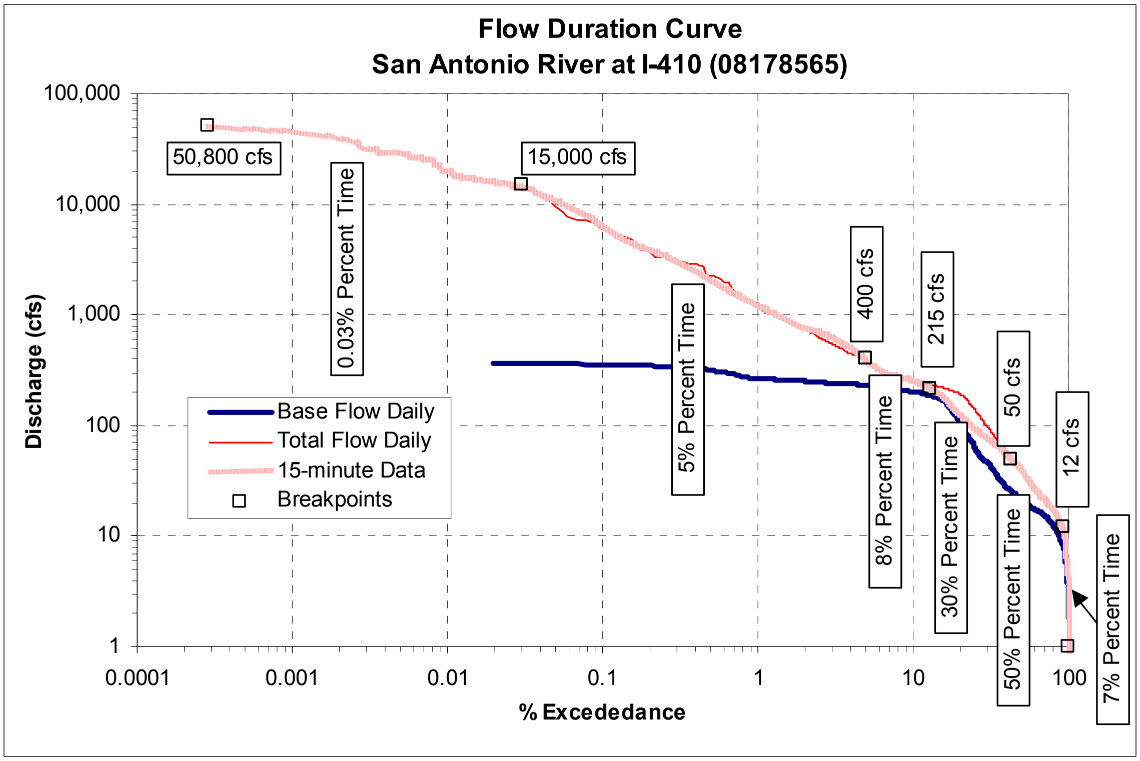


Figure H.6 Flow Duration Curves for San Antonio River at 410 Gage

Analysis of the base flow of the San Antonio River was also performed to provide the parameters required for hydraulic design of low flow features in pools and gradient structures. The base flow of a stream at a given point is an estimate of the cumulative discharge of groundwater from the adjacent aquifers to the stream for all drainage area upstream of the gage, and is representative of flow magnitudes expected during dry weather conditions when no precipitation runoff is occurring. Base flow separation for this study was performed using the Base Flow Index (BFI) computer program, a FORTRAN coded utility program jointly maintained by the USGS and the U.S. Bureau of Reclamation. BFI uses the Standard Hydrologic Institute Method for base flow separation; this method identifies sudden rises in the hydrograph typical of storm-induced runoff, and separates the total stream flow into daily time series of base flow and surface runoff for each gage.

Base flow separation was performed on daily average flow data for the available period of record at the S. Alamo and Loop 410 gages using BFI. Base flow duration curves were generated for the data at each gage and are included in Figures H.5 and H.6. The median baseflow at the S. Alamo gage is 19.4 cfs. The median baseflow for the Loop 410 is 21.2 cfs, indicating slightly gaining conditions over the intervening reach. The implication is that if it is desired to maintain a full channel for low flow conditions, the minimum channel dimensions should consider 20 cfs for the base flow channel.

Hydraulics

Hydraulic data for the current analysis was derived from the existing conditions HEC-RAS model developed for the San Antonio River LMMP. The existing conditions HEC-RAS model was developed primarily for estimation of water surface elevations during flood events, but included many provisions for low flow hydraulics for channel stability analysis and design of channel features. An inventory of hydraulic structures was performed to assess the amount of grade control in the system and to determine what data was needed for inclusion of these controls in model. Within the limits of the North and South project reaches, a total of 36 structures were identified in and are listed in Table H.1.

Table H.1 San Antonio River Hydraulic Structures

Structure #	Station	Description	Subreach
1	2499+00	Gate #1 at Olmos Dam	Below Olmos
2	2479+25	Low Water Xing near 200 Patterson Condos	Park 1
3	2429+17	Stepped Drop below Iron Bridge (Lambert Beach)	Park 1
4	2410+90	Gate # 2 - Mid Channel Island Structure at Zoo	Park 1
5	2409+00	Zoo Low Water Xing (Tuleta Street)	Park 1
6	2387+00	Submerged Structure near Mulberry	Park 2
7	2358+90	Grade Control Structure near Golf Course	Park 2
8	2353+54	3-36" Culvert Crossing at Golf Course	Park 2
9	2346+42	Low Water Xing U/S of Inlet	Park 2
10	2336+67	Tunnel Inlet Weir	Park 2
11	2268+21	Low Head Dam U/S of Lexington St.	Urban
12	2215+50	Gate #5 at Nueva St.	Downtown
13	2169+50	Gate #6 at Alamo St.	Downtown
14	2124+89	Tunnel Outlet Weir	Below Outlet
15	2116+28	Check Dam #10	Below Outlet
16	2113+10	Check Dam #9	Below Outlet
17	2104+50	Check Dam #8 (submerged)	Below Outlet
18	2098+67	Check Dam #7	Below Outlet
19	2095+25	Steves Ave Concrete Bottom	Below Outlet
20	2092+10	Check Dam #6 (filled in)	Below Outlet
21	2087+30	Check Dam #5	Below Outlet
22	2082+45	Check Dam #4	Below Outlet
23	2075+80	Check Dam #3	Below Outlet
24	2071+82	Check Dam #2	Below Outlet
25	2068+10	Check Dam #1	Below Outlet
26	2061+53	Stepped Drop U/S of San Pedro Confluence	Below Outlet

27	2007+68	Grade Control at Mission Road	Mission
28	1978+72	Grade Control at Roosevelt Blvd.	Mission
29	1910+41	San Juan Diversion	San Juan
30	1899+52	Old San Juan Diversion Ruins	San Juan
31	1850+00	Espada Dam	Davis
32	1847+05	Low Water Xing (Mission Pkwy)	Below Espada
33	1804+00	Remnant Meander Diversion Structure	Six Mile
34	1773+00	Remnant Meander Return Structure	Six Mile
35	1730+50	Drop @ end of I-410	410
36	1727+00	Low Water Xing (Camino Coahuliteca)	410

Each of these hydraulic structures were incorporated into the model and represented appropriately based on their geometry. Many of these structures become submerged at high flows and have an insignificant impact on the flood model results. A complete description of the HEC-RAS model developed for the San Antonio River LMMP is provided in the HDR Engineering, 2002 Technical Memorandum.

Additional modifications were made to the HEC-RAS flood model following its submission for use in the sediment transport analysis and channel design. The HEC-RAS model was modified to include bank stations at the edges of the pilot channel rather than at the top of the floodway as in the flood model. Additionally roughness values for the main channel were modified to represent the roughness associated with the bed material rather than the total roughness of the floodway. Results from the sediment sampling effort and the Keulegan equation were used to determine Manning's n values associated with the main channel bed. A roughness height of $3.5D_{84}$ was used in the resistance formula.

Reach Average Hydraulics

With the bank station and bed roughness modifications, the model was run and results evaluated. Computation of reach-average hydraulic parameters for each subreach was performed for use in sediment transport calculations. Results from the existing conditions HEC-RAS model were used to estimate the variation in velocity, depth and channel bed shear stress throughout the North and South project reaches. Reach-average values were computed using length weighted results from the HEC-RAS model with bridges, weirs, and other discrete hydraulic structures omitted from the output.

Minimum and maximum reach average hydraulic variables were computed for the 0.25- through 100-year storm events. Reach-average values were calculated for the main channel and overbank areas. Reach average velocity with channel velocity superimposed for the North Reach are shown in Figure H.7. Individual values from the model cross sections are also superimposed for reference. This was done to illustrate the variation in hydraulics within each subreach and how those affect the reach average values.

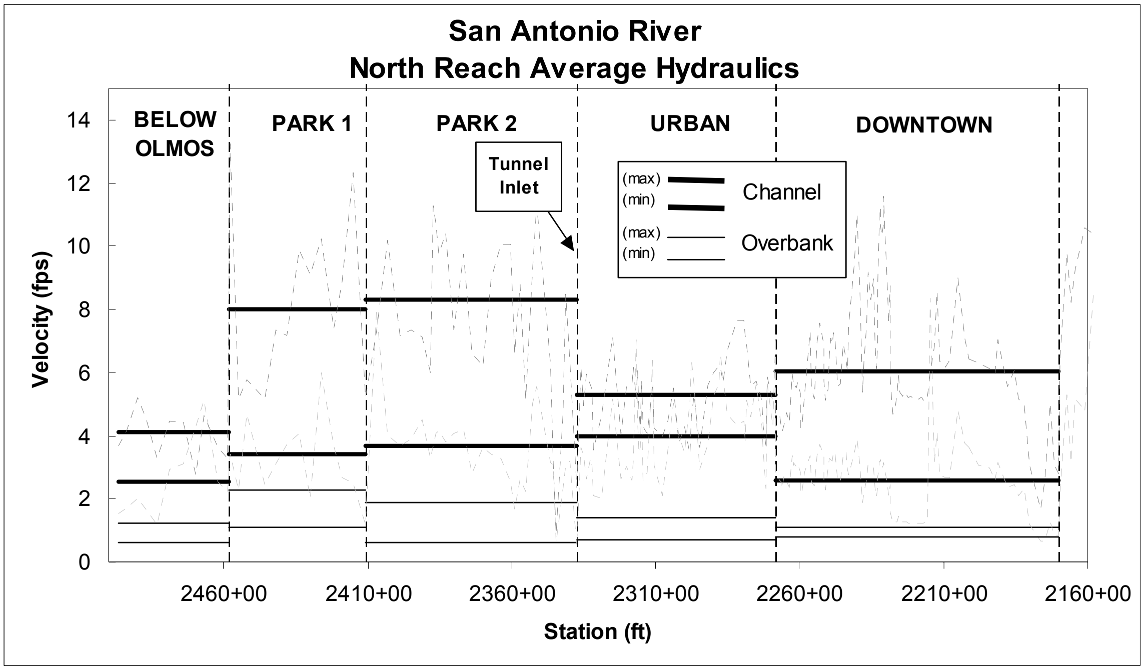


Figure H.7 Reach-Average Velocity for the North Reach (Channel Velocity Superimposed)

During high flows (~ 100-year flood) reach average channel velocities are greatest in the Park 1 and Park 2 subreaches near 8 fps. Individual channel velocities exceed 10 fps in these and the Downtown subreaches during the maximum event. Both the Park 1 and Downtown subreaches are similar in that they are lined with concrete and confine the flow between retaining walls on both sides. Under normal to low flow conditions reach average channel velocities are near 4 fps in the incised Urban subreach. In the natural Park 2 subreach the range of channel velocities are commensurate with the bed material in this area as will be discussed in the Sediment Transport Analysis. With the exception of the Park 1 subreach reach-average overbank velocities are less 2 fps during the maximum event. Reach-average velocities with channel velocity superimposed for the South Reach are shown in Figure H.8.

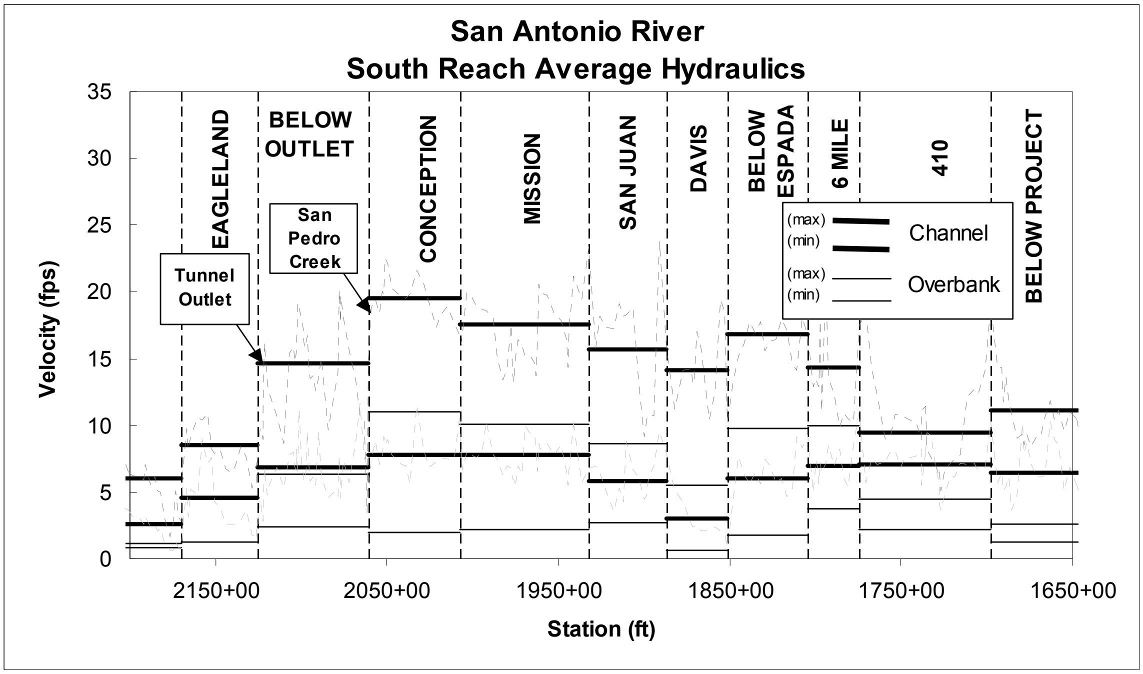


Figure H.8 Reach-Average Velocity for the South Reach (Channel Velocity Superimposed)

Reach-average channel velocities approach 20 fps in the Conception subreach and near 17 fps in the Mission subreach downstream of the confluence with San Pedro Creek during the 100-year event. Individual channel velocities exceed 20 fps in several subreaches of the South Reach during the maximum event. During normal to low flow conditions the reach average channel velocities are approximately 3 fps behind Espada Dam in the Davis subreach. Of concern are reach-average overbank velocities near 10 fps in the Conception, Mission, Below Espada and Six Mile subreaches during the maximum event. This velocity estimate is significant with respect to maintenance of vegetation in these areas. Reach average depth with channel depth superimposed for the North Reach are shown in Figure H.9.

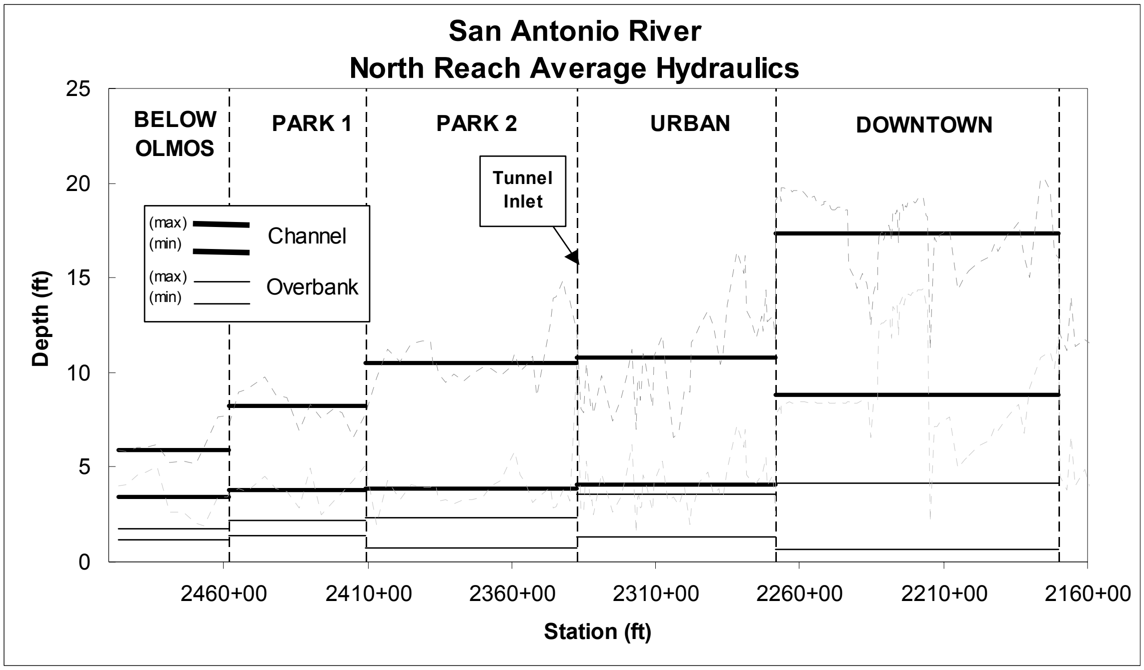


Figure H.9 Reach-Average Depth for the North Reach (Channel Depth Superimposed)

In the North reach depth increases with distance downstream. The significant increase in depth in the Downtown subreach is influenced by hydraulic structures at Nueva (Gate #5) and S. Alamo Streets (Gate #6). The range of reach-average channel depth varies from approximately 6 to 17 feet in the North Reach. Reach-average depths with channel depth superimposed for the South Reach are shown in Figure H.10.

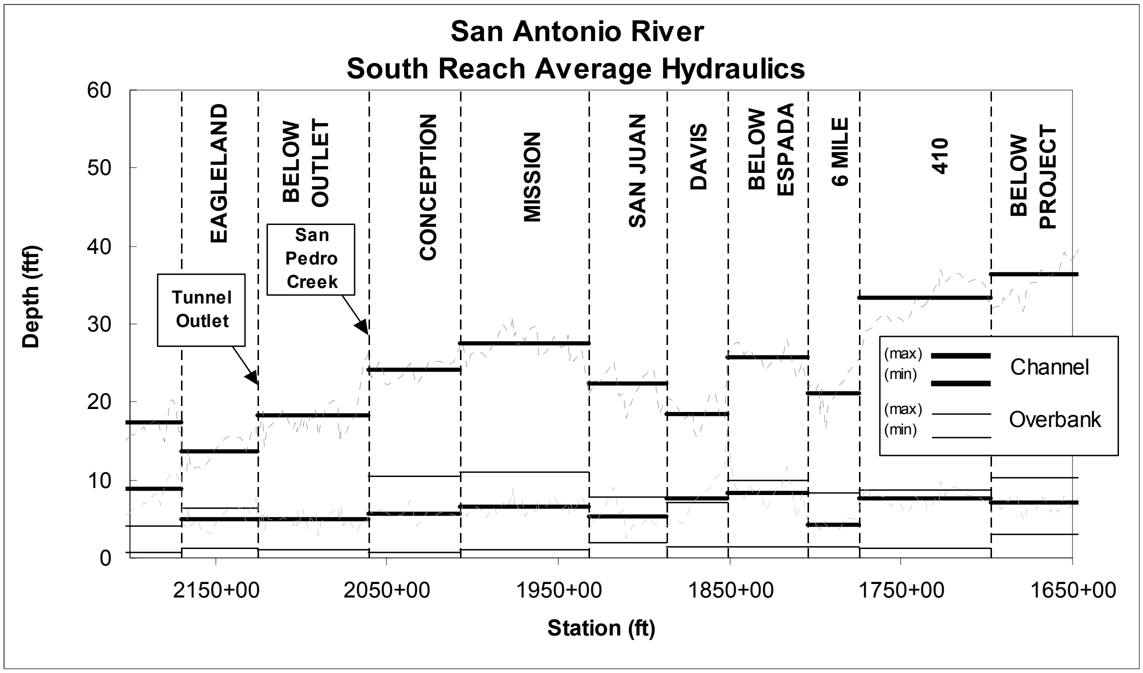


Figure H.10 Reach-Average Depth for the South Reach (Channel Depth Superimposed)

The range of reach average channel depths in the South reach varies from more than 27 feet in the Mission reach to less than 14 feet in the Eagleland Reach during the 100-year event. In general reach average depth also increases in the downstream direction with drainage area. The narrowest range is in the Davis subreach due to the influence of Espada Dam. Of great significance to channel design, sediment transport and maintenance is shear stress. Shear stress is directly related to velocity, depth and boundary roughness. Reach-average shear stress with channel shear stress superimposed for the North Reach are shown in Figure H.11.

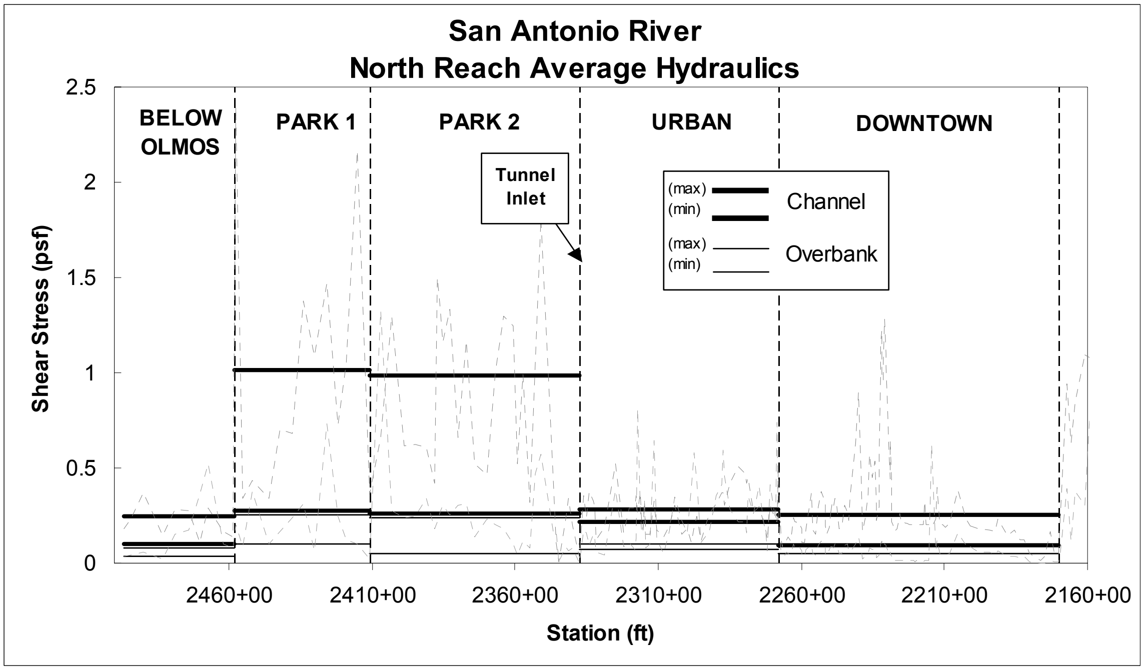


Figure H.11 Reach-Average Shear Stress for the North Reach (Channel Shear Stress Superimposed)

Proportional to velocity and roughness squared, the reach-average shear stress in the North Reach is greatest in the Park 1 and Park 2 subreaches. Channel shear stresses are not much greater than 1 psf for all of the North subreach during the 100-year flood. This indicates a relative state of stability against erosive forces. Although the Downtown reach had considerable velocities and depth, the model results reflect a low shear stress in the Downtown subreach because of a low Manning's roughness value representing concrete that lines the channel in this area. Under normal conditions a layer of silt and sand develops in the Downtown subreach, which could cause dunes to form under certain flow conditions. Development of dunes would increase the roughness and resultant shear stress in this subreach. As mentioned with velocity the shear stress values in the natural Park 2 reach are comparable with the bed material size in this area. Reach average shear stress with channel shear stress superimposed for the South Reach are shown in Figure H.12.

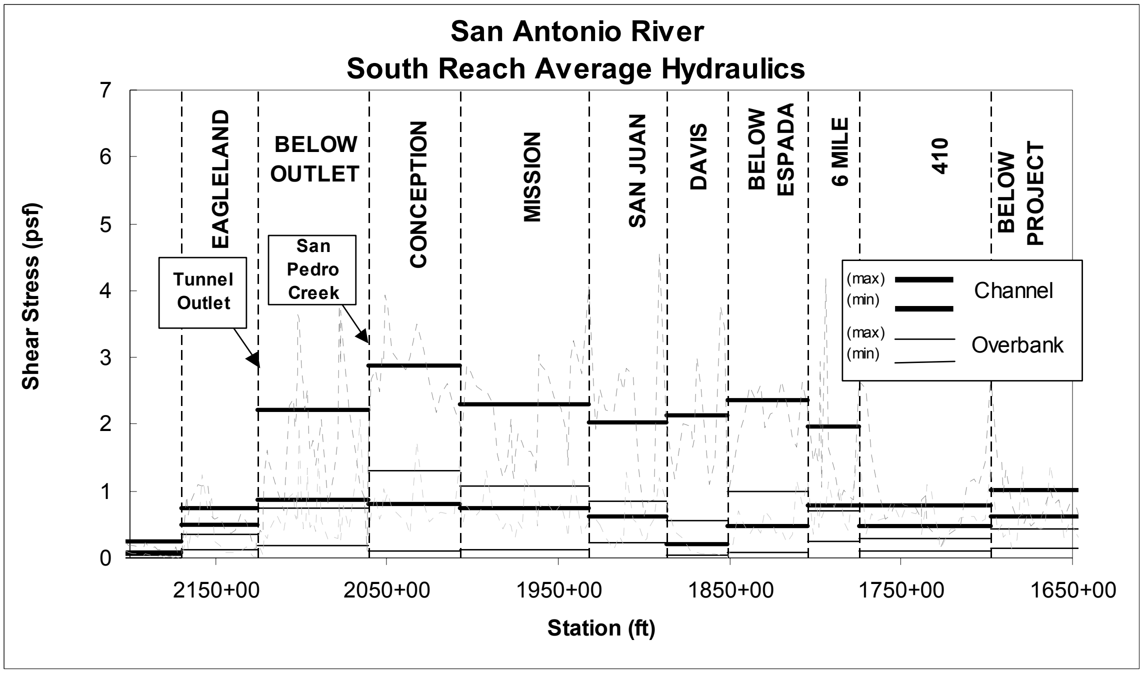


Figure H.12 Reach- Average Shear Stress for the South Reach (Channel Shear Stress Superimposed)

As with velocity, the maximum reach-average shear stress was computed for the Conception subreach. There are exceptions to the direct correlation between shear stress and channel adjustment, but in general, areas where erosion has been observed have higher shear values and areas where deposition has occurred have lower shear stress values, especially during low flows. Below the confluence with San Pedro Creek maximum average channel shear stresses are near or greater than 2 psf with the exception of the 410 subreach. An exception is the Davis reach where the model shows relatively high shear stress values at high flows, but significant deposition has persisted behind Espada Dam. The model results indicate that storm high flow events do have the ability to move larger size sediment through and over the dam, but during low flows fine material accumulates behind the structure. This indicates that big water features in this area will accumulate fine material, but will likely be flushed downstream during high discharge storm events. The sedimentation rate of these areas would likely be equivalent or less to that observed from the historical analysis. Another exception is the Below Espada reach were shear stresses in the overbanks are similar to those in the Mission and Conception reaches, but significant deposition has been observed. This may be attributed the flushing of fine material from Davis Lake and subsequent deposition in this reach during the receding limb of the flood event.

Hydrology & Hydraulics Conclusions

The hydrologic and hydraulic analysis provides fundamental data for design of channel stabilization and enhancement features. Conditions change dramatically from the North Reach to South Reach especially downstream of San Pedro Creek. The influence of flood reduction and hydraulic structures also play a significant role in the way hydrology and hydraulics effect the system. Data derived from hydraulic and hydrologic models, stream gages and the historical channel assessment were used in the Sediment Transport Analysis to determine the current channel stability and what measures are required for future stabilization.

Sediment Transport Analysis

Analysis of sediment transport in the San Antonio River basin was performed to assess current channel stability and to provide information for design of the proposed channel and stabilization structures. The level of the analysis provides approximate average channel dimensions and energy gradients (width, depth and slope) that would achieve a condition of sediment continuity based on the assumptions of alluvial channel theory and sediment supply from the contributing watershed. Estimates of channel parameters required for stability are provided, where stability is defined as a condition where the channel retains its cross sectional geometry and energy grade without excessive erosion or deposition on an engineering time scale (25-50 year time horizon). Because the project goals are to minimize maintenance requirements, whether erosion mitigation or dredging, this criterion is consistent with the proposed definition of stability.

The analysis uses constrained equilibrium methods in prediction of channel parameters and does not consider the temporal variability of adjustment and the full multidimensionality of fluvial systems. The analysis is based on an assumption of one-dimensional flow in the main (pilot) channel and does not consider more complex processes such as secondary currents and flow separation during overbank conditions. The study is limited to evaluation of alluvial bed-load transport in the main channel. Local controls such as the exposed Navarro formation in the lower subreaches will provide temporal stability, but are considered erodible on a longer time scale and are not considered in this analysis. As data becomes more available, further studies should consider the impact of suspended load, complex flow, and temporal adjustment

Data used in the analysis include results from the historic channel assessment and the hydrologic & hydraulic analyses of this memorandum. The sediment transport analysis focuses on developing a sediment budget for the South Reach, but also provides information related to stability and maintenance in the North Reach. The concept of sediment continuity and bed mobility in alluvial systems will be used as the basis for stable channel design. The stable channel analysis provides reasonable estimates of average parameters for design, based on available information. If greater certainty is desired, then sediment concentrations must be measured over an extensive range of flows. Such measurements take a considerable time to obtain, and the schedule and budget of this project does not allow for their collection.

The following discussions provide a description of the concepts, methods, and results from the analysis. Two appendices support the sediment transport analysis, Appendix B provides sediment sample gradations, and Appendix C provides photos of sediment sample locations

Sediment Sampling

A comprehensive sediment sampling effort was conducted to characterize the size, gradation, and distribution of bed material sediment currently in the San Antonio River. The sampling effort encompassed reaches of the San Antonio River Basin beginning in upper Olmos Creek watershed to below the end of the San Antonio River Floodway downstream of Interstate 410. Sediment samples were obtained at locations that would provide a reasonable representation of the existing bed material size and composition in both the north and south project reaches. With the exception of samples taken in the Olmos Basin and those collected below the San Antonio River Floodway, locations of the sediment samples are shown in Figures S.1 and S.2.

Generally, bed material samples were collected below a near base flow water surface when conditions permitted, and alternatively, at the upstream end of exposed point and side bars. It was assumed that samples obtained from the upstream end of bars would be representative of the material comprising control sections or riffles. Riffles are generally comprised of the coarser fraction of bed material that form topographic high points in the longitudinal profile and act as hydraulic controls in rivers. The size and gradation of this material is critical to the stability of the system. A total of 34 sediment samples were collected from the active channel bed with the exception of a sample in the Eagleland reach, which was gathered from the bank. Bed Material samples were obtained from the upper 6 to 8 inches of the channel substrate. Since there was no direct evidence of armoring, subsurface samples were not collected. The only location where a surface and subsurface sample was collected was at the point bar located at the confluence with San Pedro Creek. Some stratification was observed at this location, but the material comprising the subsurface layer was found to be larger than the surface layer.

Sediment samples were collected with a spade, stored in 1-gallon bags, and transported for sieve analyses to be completed by the geotechnical consultant. The results of the sieve analyses are provided in Appendix B. Gradation curves were averaged to provide representative gradations for each of the hydraulic reaches. Further, the individual sample gradations were averaged to provide representative gradations for the north and south hydraulic reaches as a whole. Using the average gradation curves, percent passing values for each of the project reaches were computed. Values for the D_{16}, D_{50} and D_{84} for each of the hydraulic reaches area shown in Figure S.3 and averaged gradation curves for the north and south reaches are shown in Figure S.4.

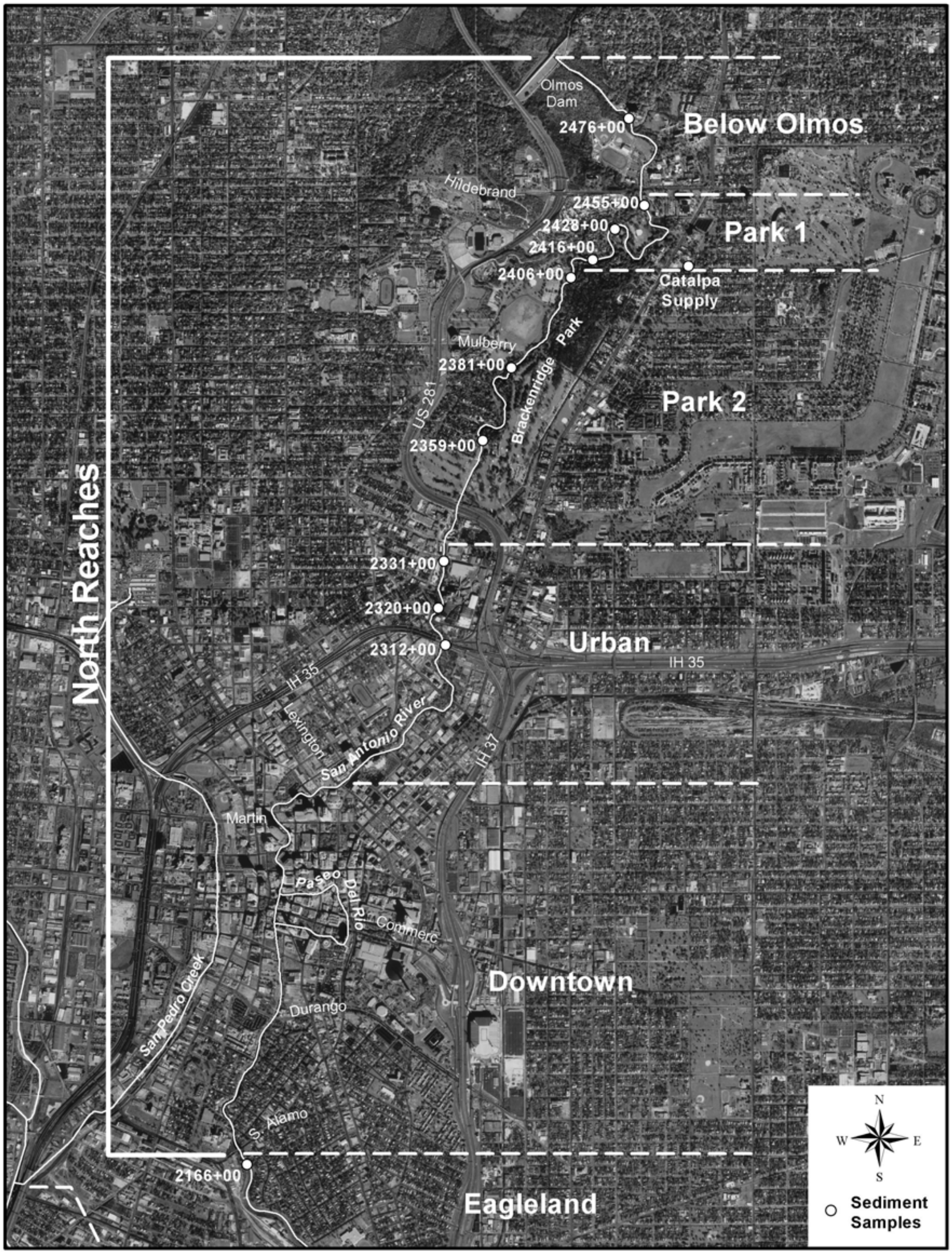


Figure S.1 Sediment Sample Locations – North Reaches

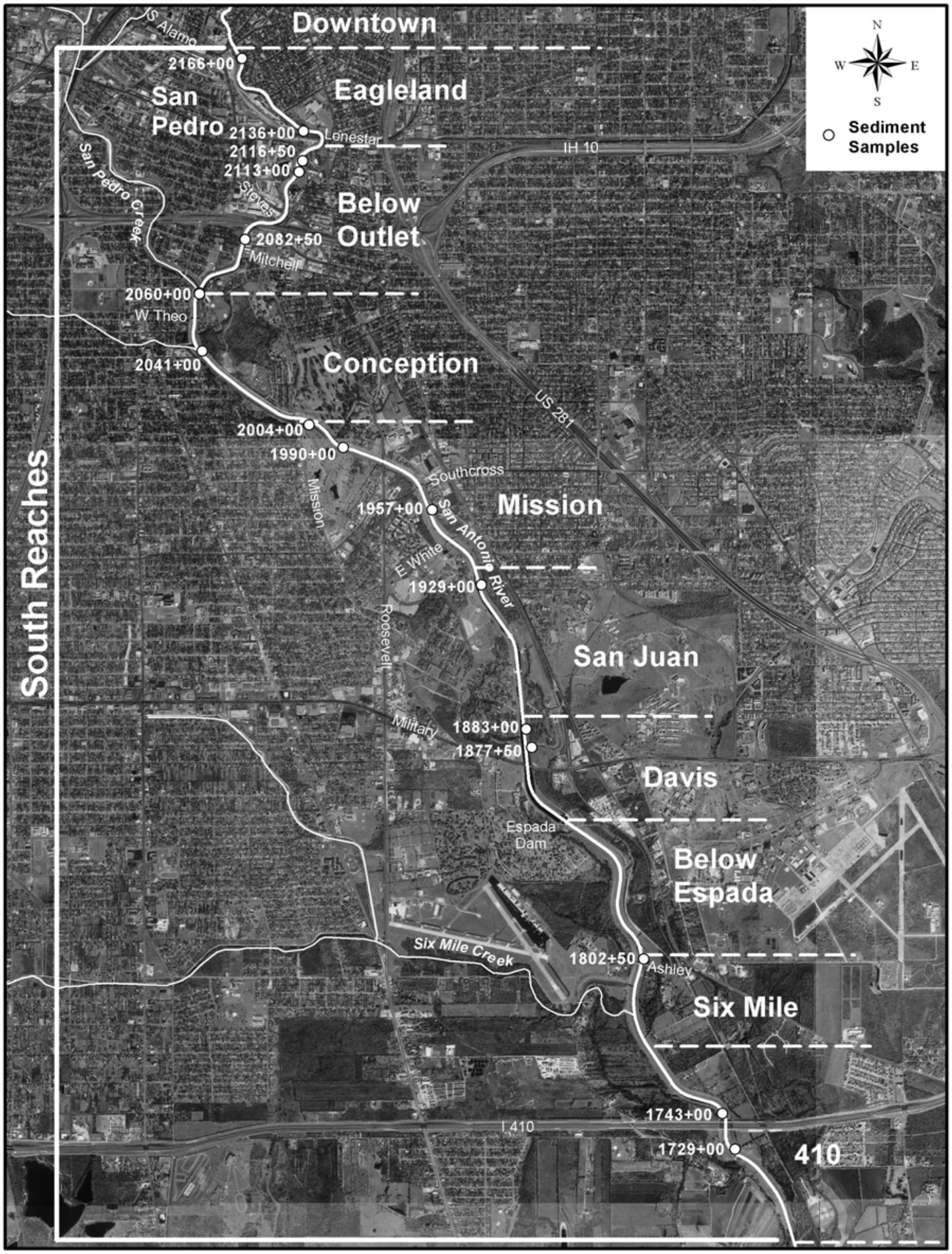


Figure S.2 Sediment Sample Locations – South Reaches

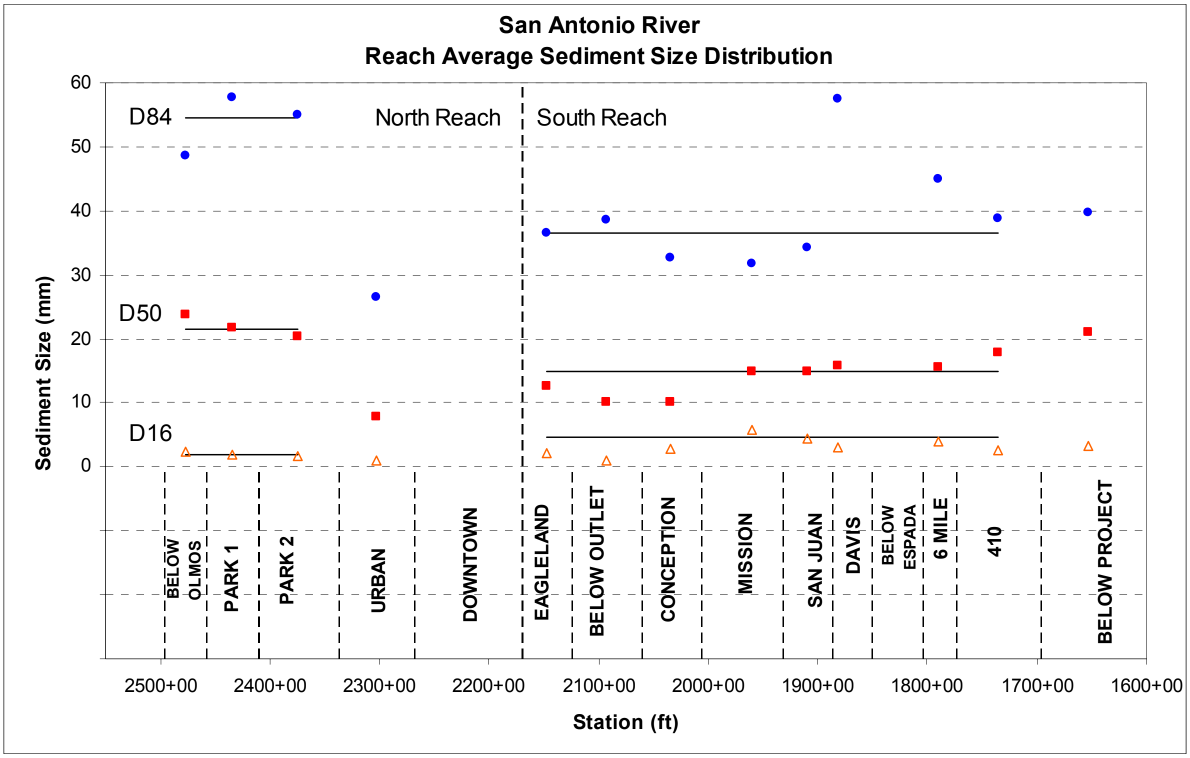


Figure S.3 Reach Average Percent Passing Values

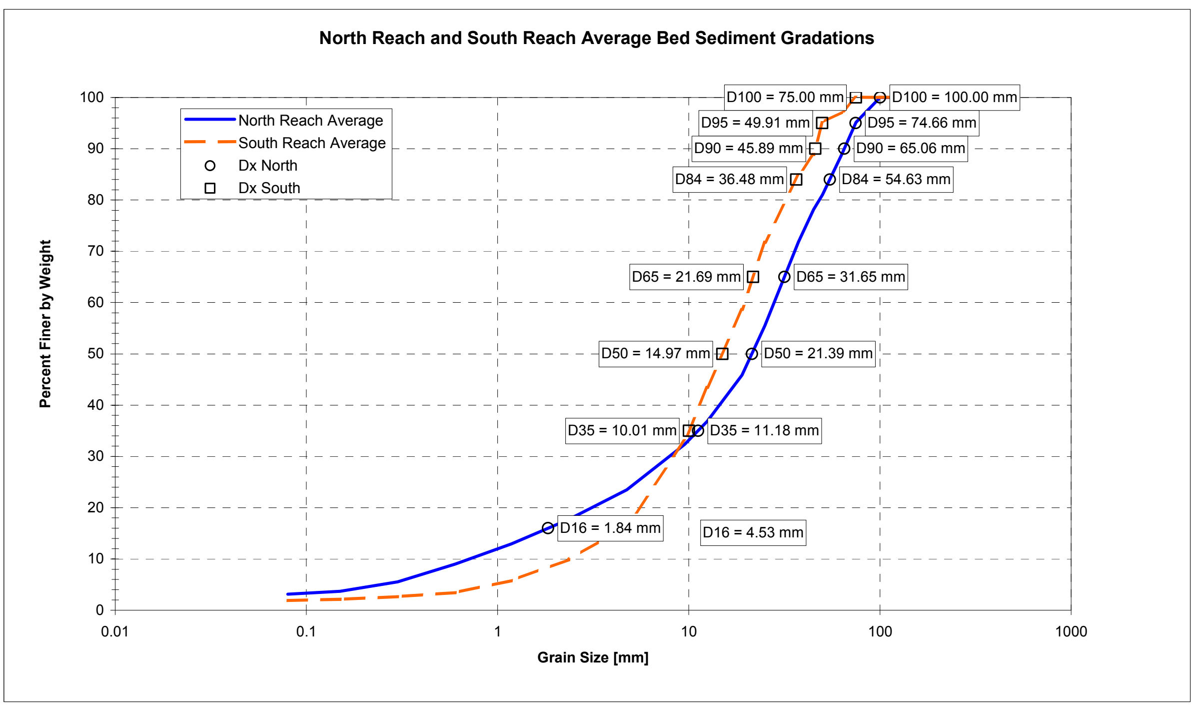


Figure S.4 North and South Reach Average Bed Material Gradation Curves

The North Reach average gradations were developed using three of the reaches excluding the Urban Reach and the samples collected in Olmos Creek upstream of Olmos Dam. These samples were excluded because the average bed material size observed in the Urban Reach was significantly smaller than that found in the upstream reaches. This is attributed to the influence of the San Antonio River Tunnel and weir structure upstream. Additionally, sediment samples obtained in the Olmos Creek Basin were not included because Olmos Dam prohibits the passage of coarse-grained material through its outlet structure. For the South Reach, the average gradation was developed using the reaches below the confluence with San Pedro Creek with the exception of the Conception Reach. The particle size distribution of the Conception Reach samples were smaller than sediment observed downstream, and therefore, were not included in the data set to calculate average South Reach gradations.

The sediment size gradations indicate some variability in bed material size from upstream to downstream. Bed materials in the North Reach are generally larger than material found in the South Reach. In most river systems, bed material size is inversely related to contributing drainage basin area. More coarse bed material is usually found in the upper, higher gradient reaches of the watershed while average particle sizes decrease in the downstream direction. The average D_{84} and D_{50} of the material found in the North Reach is approximately 55 mm and 21 mm, respectively whereas the average D_{84} and D_{50} of the material found in the South Reach is approximately 36 mm and 15 mm, respectively. The average gradation values will be used in the design of the new San Antonio River channel.

Channel Adjustment

Channel adjustment results from the removal or accumulation of sediment in the channel boundary. Results from the historic channel assessment show that portions of the south reach have accumulated sediment where others have been affected significantly by erosion. In erosive reaches the channel has adjusted to a flatter slope and wider cross section as compared to the constructed condition. A relationship proposed by Lane (1955) can be used to qualify some of this response. The Lane relationship can be presented as:

$$Q_w S \propto Q_s D_S \qquad \text{(Equation T.1)}$$

where:

Q_w = water discharge (L^3/L)
S = channel slope (L/L)
Q_s = sediment discharge (L^3/L)
D_s = sediment size (L)

Construction of the floodway and channelization has had a direct impact on the San Antonio River. The result has been increased velocity, shear stress and sediment transport capacity through the system. Further urbanization of the watershed has resulted in increased runoff in both magnitude and frequency. A generalization of the river response following these conditions could satisfy the Lane relationship in the form of:

$$Q_w^{+} S^{--} \propto Q_s^{+} D_s^{+} \qquad \text{(Equation T.2)}$$

The (+) signs indicate an increase in water discharge (Q_w) due to urbanization, an increase in sediment transport capacity (Q_S) due to channelization and coarsening of the channel bed material (D_S) due to winnowing of fine material. The S^{--} indicates a significant decrease in channel slope as has been observed in the South Reach and understood in the North Reach. The historical channel assessment did not include assessment of channel slope in the North Reach, however the reduction in sinuosity and incised conditions in the Urban Reach suggest this occurrence in the unarmored sections. Concurrently channel widening has transpired in the incision process.

The magnitude of channel adjustment depends on the mobility of the channel boundary (bed and banks) material and the amount of sediment supplied from upstream sources. For this study the principle of sediment continuity will be used to quantify this adjustment. The concept of sediment continuity is illustrated as in Figure T.1.

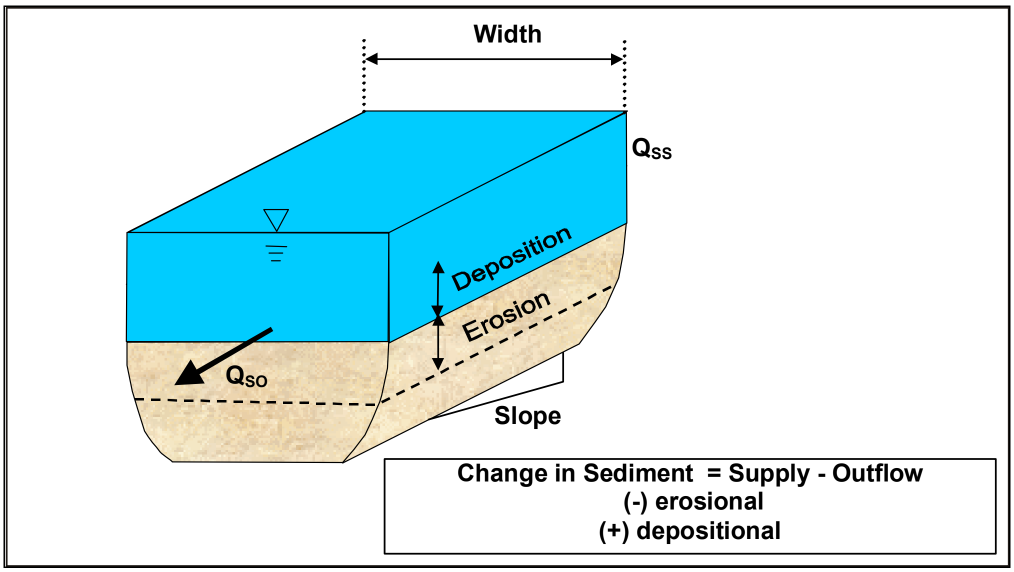


Figure T.1 Concept of Sediment Continuity

When the sediment supply exceeds the outflow (or sediment transport capacity) of a given reach, deposition will likely occur. Alternatively if the reach transport capacity (outflow) is greater than the sediment supply from upstream sources then erosion from the channel boundary will be expected. Sediment transport rates can be estimated through sediment discharge measurement and/or using empirical models. Since sediment discharge measurements were not available for the San Antonio River this study will rely on sediment transport relationships and results from the historical channel assessment.

Channel Stability

For simplicity channel stability will be defined as the condition where the channel retains its cross sectional and energy grade without showing significant reach wide trends toward aggradation or degradation. This stability is affected by the sediment supply to the system and the hydraulic forces that determine its sediment transport capacity. This hydraulic force termed shear stress is the force per unit area acting on the channel boundary. The channel shear stress affects the ability of the channel to mobilize sediment and for this study was computed using the following relationship:

$$\tau_o = \gamma_w R S_f \qquad \text{(Equation T.3)}$$

where:

τ_o	=	channel bed shear stress (lb/ft^2)
γ_w	=	unit weight of water, ~62.4 lb/ft^3
R	=	hydraulic radius (ft)
S_f	=	friction slope or energy slope

Results from the HEC-RAS model were used to calculate reach-average channel shear stress values for each subreach and discharge frequency represented in the model. Hydraulic depth was used for hydraulic radius as an approximation. Reach-average shear stress values are presented in Figures H.11 and H.12 of the Hydrology & Hydraulics section of this memorandum.

For this investigation the mobility of the channel bed material was estimated using the Shield's relationship (1936). The Shields equation describes the hydraulic condition at which motion of individual sediment particles may be initiated. This beginning of motion referred to as incipient motion is extensively used in many sediment transport equations and stability relationships. The hydraulic conditions at which incipient motion occurs can be described as the critical shear stress of the bed material. The critical shear stress can be expressed as:

$$\tau_c = SP\gamma_w (S_g - 1) D_s \qquad \text{(Equation T.4)}$$

where:

τ_c	=	Critical shear stress to initiate motion of the bed material (lb/ft^2)
SP	=	Shields Parameter

S_g = specific gravity of sediment
D_s = sediment size (ft)

For this analysis a Shield's parameter of 0.05 and specific gravity 2.65 were used. Average sediment sizes and critical shear stress values for the bed material in the North and South Reaches are provided in Table T.1

Table T.1 Sediment Size and Critical Shear Stress Values				
	North Reach Distribution		South Reach Distribution	
	Sediment Size (mm)	τ_c (lb/ft^2)	Sediment Size (mm)	τ_c (lb/ft^2)
D_{35}	11.2	0.19	10.0	0.17
D_{50}	21.4	0.36	15.0	0.25
D_{84}	54.6	0.92	36.5	0.62
D_{90}	65.1	1.10	45.9	0.78
D_{100}	100.0	1.69	75.0	1.27

A commonly (mis)used indicator of channel stability is the excess shear stress ratio, which is defined as the ratio of the channel shear stress (Equation T.3) to the critical shear stress (Equation T.4) of the bed material. Ignoring sediment supply, this ratio is used as an indicator of the mobility of the channel bed material. The excess shear stress ratio is described as τ_o/τ_c. Therefore for an excess shear stress ratio greater than 1 the bed material becomes mobilized and for values less than 1 the material is presumed to remain static. At an excess shear stress ratio of 1 the bed material is at a condition of incipient motion.

In gravel bed systems such as the San Antonio River, the concept of equal mobility may be applicable to the selection of the appropriate sediment size for calculation of incipient motion with the Shields equation and to determine the excess shear stress ratio. Equal mobility refers to a hypothesis discussed by Parker et al. (1982) that the preponderance of the bed material does not effectively become transportable until the larger exposed pavement material is mobilized. The research suggests that the D_{84} particle diameter should be used in incipient motion and sediment transport calculations. However, experience has shown that these larger sediment sizes in the exposed surface become mobilized at a lower shear stress value than that expressed with the Shield's equation. In these cases a lower Shield's parameter on the order of 0.03 may be used with the D_{84} particle size. For this analysis utilization of the D_{50} particle diameter and Shield's parameter of 0.05 was used for incipient motion. Excess shear stress ratios using reach-average hydraulic parameters and particle size distributions for the North and South Reaches as a whole are included in Table T.2.

Table T.2 Excess Shear Stress Ratios for the North and South Project Reaches		
	North Reach Average	South Reach Average
Recurrence Interval (yr)	To/Tc D_{50}	To/Tc D_{50}
2	1.18	5.87
5	1.33	6.93
10	1.43	7.55
25	1.54	8.29
50	1.60	8.51
100	1.70	8.45

The results indicate that in general the bed material would be slightly mobile in the North Reach and highly mobile in the South Reach for the range of flows presented. Reach-average excess shear stress ratios for each subreach in the North Reach for the 0.25- through 100-year events are shown in Figure T.2.

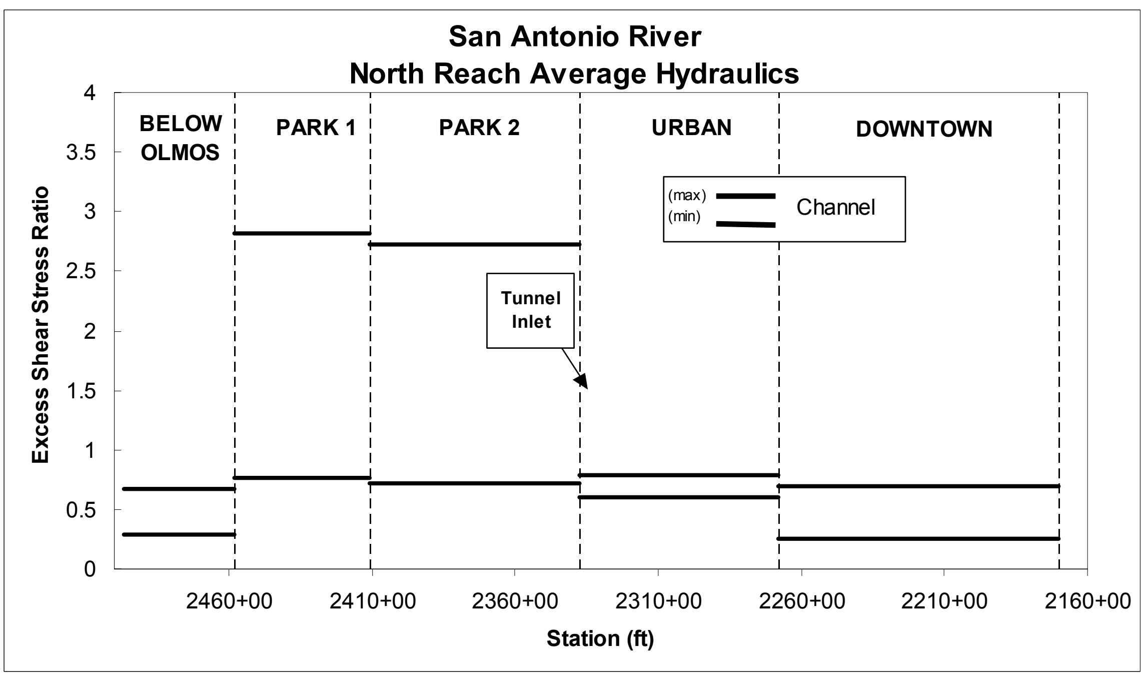


Figure T.2 Reach-Average Excess Shear Stress Ratio for the North Reach

In the North Reach excess shear stress ratios are greatest in the Park 1 and Park 2 subreaches on the order 3 during high flow events. Under low flow conditions excess shear stress ratios are less than 1 indicating a nonerosive environment during these milder storm events. This corresponds to the deposition observed in the Park 1 and Downtown subreaches. Overall the excess ratios do not suggest highly erosive conditions except potentially during extreme floods. Reach-averaged excess shear stress ratios for the South Reach are shown in Figure T.3.

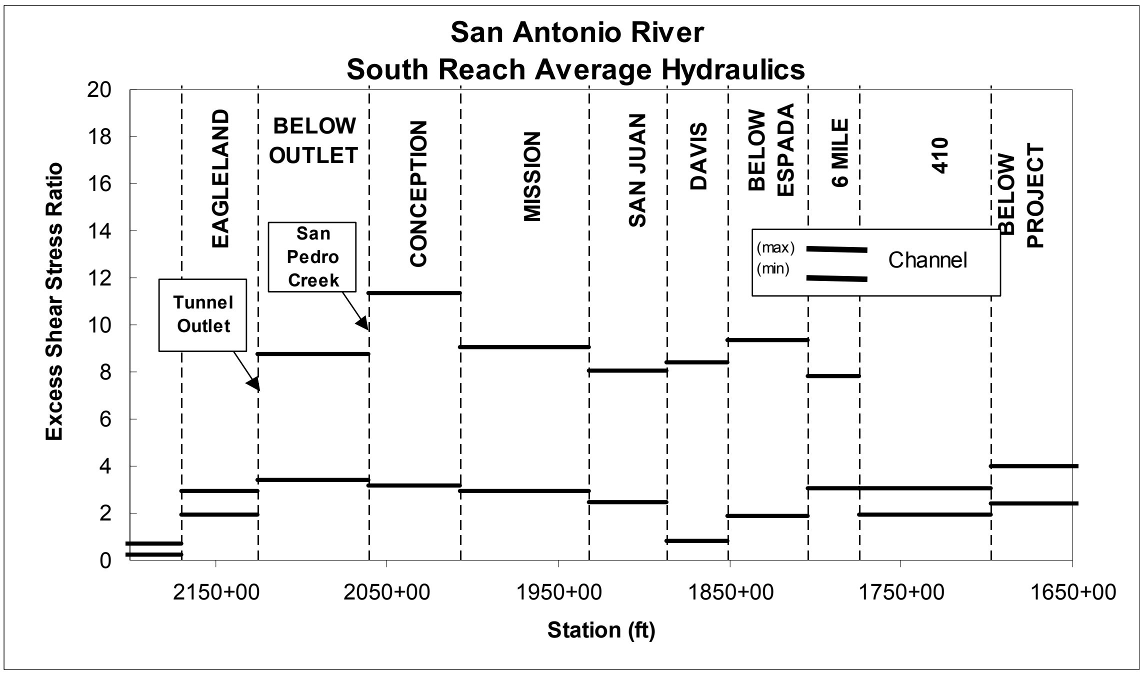


Figure T.3 Reach-Average Excess Shear Stress Ratio for the South Reach

In the South Reach below the confluence with San Pedro Creek, excess shear stress ratios are on the order of 10 during extreme flood events. Even during the milder storm events the ratios are around 3, which is near that computed for the North Reach during low frequency events. This suggests a highly erosive environment, which has been confirmed as discussed in the historical channel assessment. Behind Espada Dam in the Davis subreach the excess shear stress is near 1 or less for high frequency floods, but the material can be flushed downstream as indicated by the excess shear stress ratio near 8 during the maximum event.

Excess shear stress ratios can be used to determine minimum channel dimensions and equilibrium slopes required for channel stability depending on the assumption of sediment supply. Under clear water conditions an excess shear stress ratio less than 1 would need to be provided for all flows including high flow events for to guarantee stability of the channel bed. However this may require continuous armoring and/or excessive grade control in the South Reach of the San Antonio River. An alternative approach is to provide an excess shear stress ratio of 1 at a target discharge. This approach could be considered nonconservative in urban environments because it implicitly assumes some level of some sediment supply for flows above the target discharge. The goal for study is to estimate a realistic quantity of sediment supply from which stable channel parameters can be estimated at the target discharge. For this analysis the target discharge is described as the effective discharge as explained in the following section.

Effective Discharge

Long-term channel response results from the integral effect of all of the hydraulic, hydrologic, meteorological and geologic variables that influence the stream. However, the discharge or discharge range that has been observed to correspond most significantly to the geomorphic channel characteristics has been described as the bankfull, channel-forming, dominant or effective discharge. Leopold et al. (1964) observed a correspondence between the frequency of the bankfull discharge and the discharge that cumulatively transports the most sediment in many natural perennial streams. Because bankfull is less applicable in incised systems, the term effective discharge will be used for this analysis. The effective discharge is a hypothetical value that is presumed to transport most of the sediment over time. It is a convenient tool used for estimating channel parameters for which sediment continuity is provided. In natural systems, the observed recurrence interval has been on the order of 1 to 2 years. However, for incising streams in urban environments the geomorphic characteristics of effective discharge channels show stronger correlation to more frequent flows. In urban streams the inset active channel morphology is more influenced by minor system flows with recurrence intervals less than 1-year (Raymond Chan and Associates 1997). For estimating channel parameters required for stability, effective discharge values were computed for the San Antonio River project subreaches. Methods described by Biedenharn et al. (1999) were used to support the analysis.

The effective discharge calculation attempts to determine the discharge for which the frequency and sediment transport capacity are maximized. To accomplish this flow duration and sediment transport data are required. As discussed in hydrology & hydraulics section flow duration data was available from the gages at S. Alamo St. (USGS 08178000) and Loop 410 (USGS 08178565). Mean daily flow values for the period of record (83 years) were available for the S. Alamo gage and more than 10 years of both daily and 15-minute incremental values were obtained for the Loop 410 gage. Because the flow duration curve for the San Antonio River is strongly skewed from the high incidence of low flows, the USGS flow duration procedure using logarithmic class intervals was used in the effective discharge estimation. A total of 35 logarithmic classes were used to represent log-linear nature of flow duration curve. To capture the influences of urbanization and to correspond with the construction of the floodway the current 40-year period (1957-1997) of data was used for the S. Alamo gage. A 10-year record of 15-minute data was used for the Loop 410 gage. The flow duration curves and average discharge values for each class interval are shown in Figures T.4 and T.5.

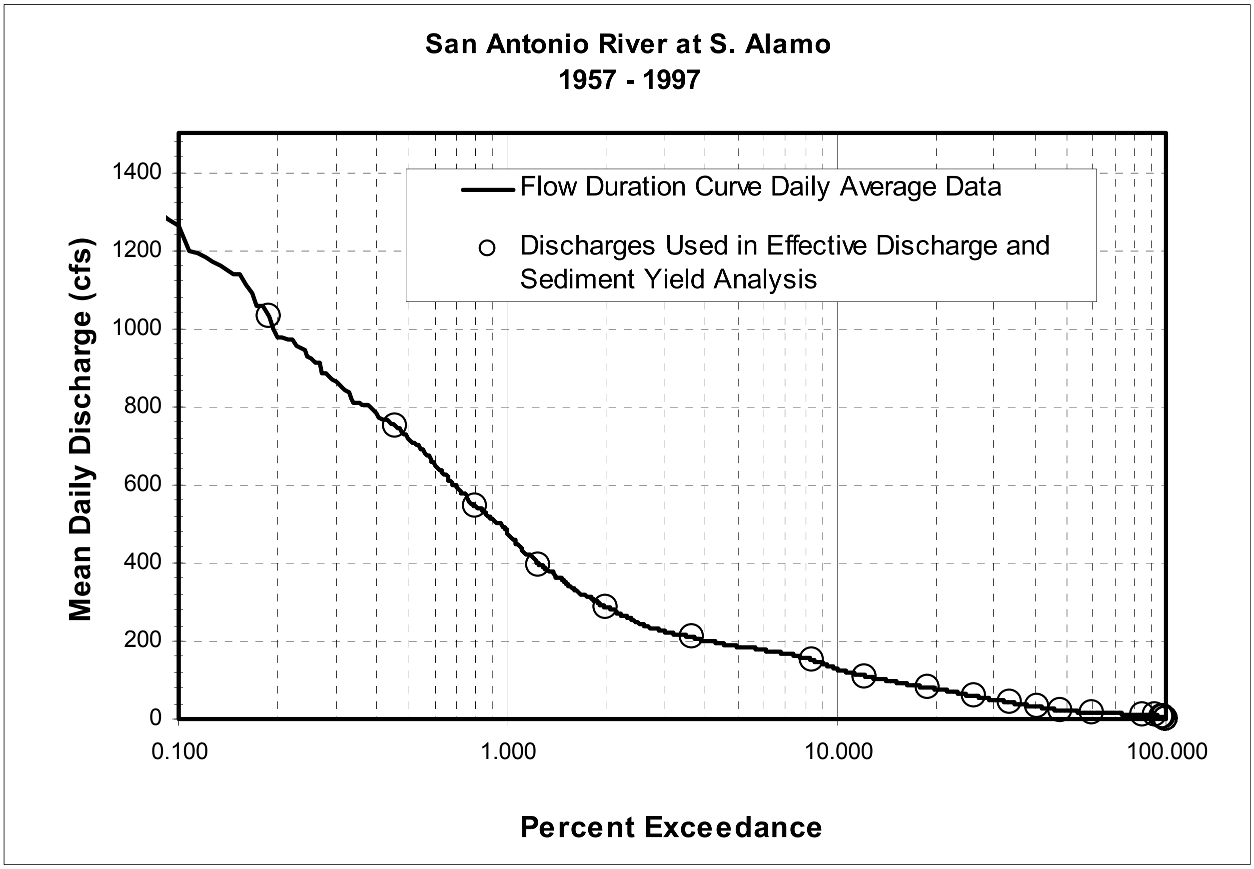


Figure T.4 Flow Duration Curve and Discharges used in the Effective Discharge Analysis - S. Alamo Gage

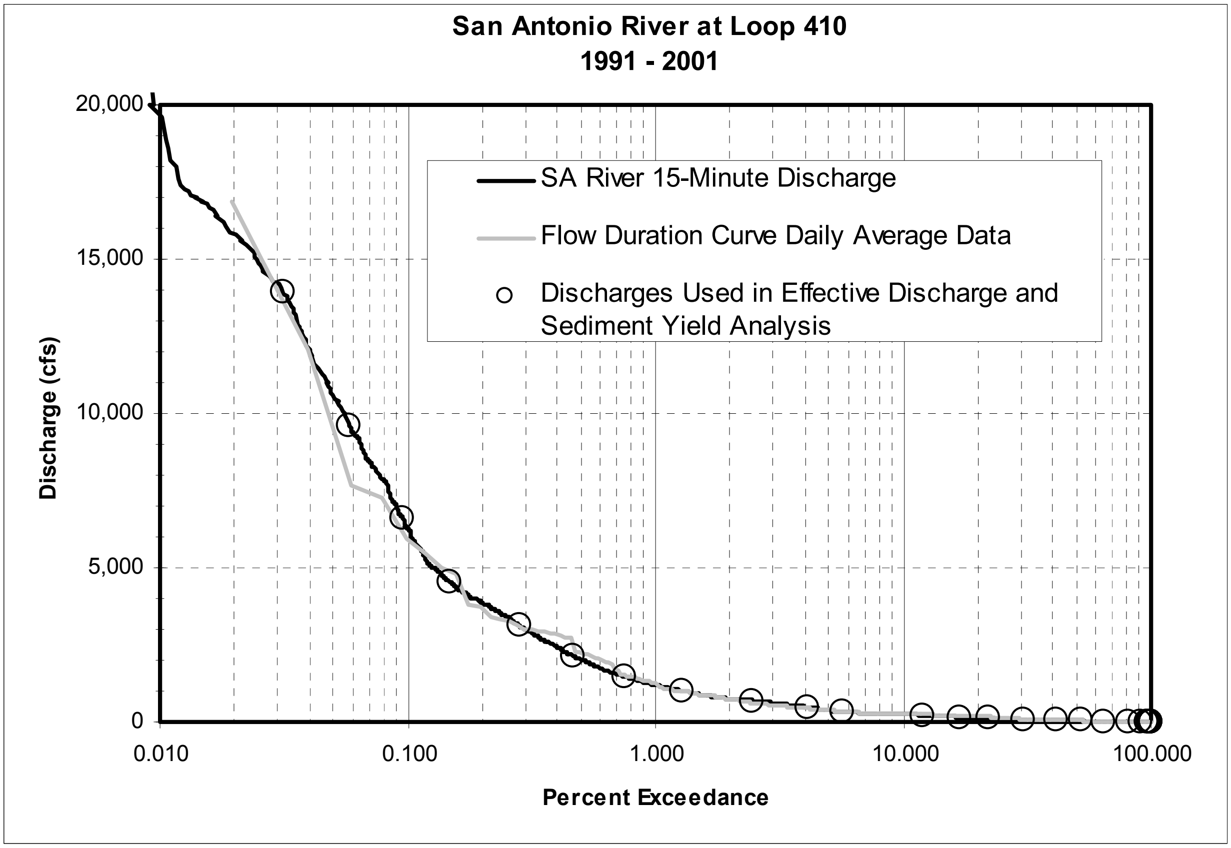


Figure T.5 Flow Duration Curve and Discharges used in Effective Discharge Analysis - Loop 410 Gage

Both the mean daily and 15-minute instantaneous values are plotted for the Loop 410 gage. As expected the lower end of the flow duration curves are nearly identical, but begin to diverge with increasing discharge.

To estimate a rate of sediment transport capacity in each of the flow classes, sediment transport relationships were developed for each of the project subreaches and the North and South Reaches as a whole. Direct measurement of neither bed nor suspended load had been performed on the San Antonio River, therefore this study utilized existing sediment transport equations. For this analysis the DuBoys' equation for bed-load transport as represented in Yang (1996) was used. It was assumed that bed-load was a dominant factor contributing to the main channel form and gradient. The analysis did not attempt to quantify the influence of suspended-load due to the fact that it could not be predicted with any level of confidence. Further, since the analysis was limited to transport in the main channel the additional load not included in the analysis was assumed to be more directly related to accumulation in the overbank areas. The Duboys' equation can be expressed as:

$$Q_s = \frac{0.173}{D_S^{\frac{3}{4}}} \tau_o (\tau_o - \tau_c) W \qquad \text{(Equation T.5)}$$

where:

Q_s	=	Volumetric bed material load, bulked (ft^3/s)
τ_o	=	channel bed shear stress (lb/ft^2)
τ_c	=	critical shear stress to initiate motion of sediment (lb/ft^2)
W	=	channel width (ft)
D_s	=	sediment size (mm)

For this analysis the D_{50} values were used in the Duboy's equation. This equation was used to compute reach-average bed-load sediment transport rates for each subreach and subsequently to define a sediment transport equation of the form of:

$$Q_s = \alpha (\tau_o - \tau_c)^{\beta} W \qquad \text{(Equation T.6)}$$

where:

Q_s	=	Volumetric bed material load, bulked (ft^3/s)
τ_o	=	channel bed shear stress (lb/ft^2)
τ_c	=	critical shear stress to initiate motion of sediment (lb/ft^2)
W	=	channel width (L)
α, β	=	bed material load coefficient and exponent

The (τ_o - τ_c) differential termed "excess shear stress" has been used as the basis for many sediment transport functions. Research from the U.S. Waterways Experiment Station (WES) found that for sand mixtures values of the exponent β are typically confined to a narrow range between 1.5 and 1.8 (Graf 1984). The second equation is considered a DuBoy's type equation and was fit to the results of the first equation. Results of the regression resulted in the following parameters for the North and South Reaches:

Table T.3 Bed Material Load Coefficient and Exponent for the San Antonio River

	North Reach	South Reach
α	0.022	0.028
β	1.54	1.85

Using the Duboys equation, bed-load transport rates were computed for the 11 flow ranges in the HEC-RAS model. Additionally the SAM Hydraulic Design Package for Channels (WES 1994) was used to compute sediment transport rates with the Myer-Peter Muller and Parker bed-load transport equations. The Duboy's equation using D_{50} was compared with averaged results from the SAM model with gradations. In general the results compared favorably in high energy subreaches, but the Duboy's equation produced larger sediment transport rates in the milder energy reaches. Results from the SAM model are provided in Appendix D.

Sediment transport rating curves relating the bed-load to total discharge were computed for each of the subreaches and the North and South reaches as a whole. The sediment transport rating curve for the South Reach below the confluence with San Pedro Creek and upstream of Davis Lake is shown in Figure T.6.

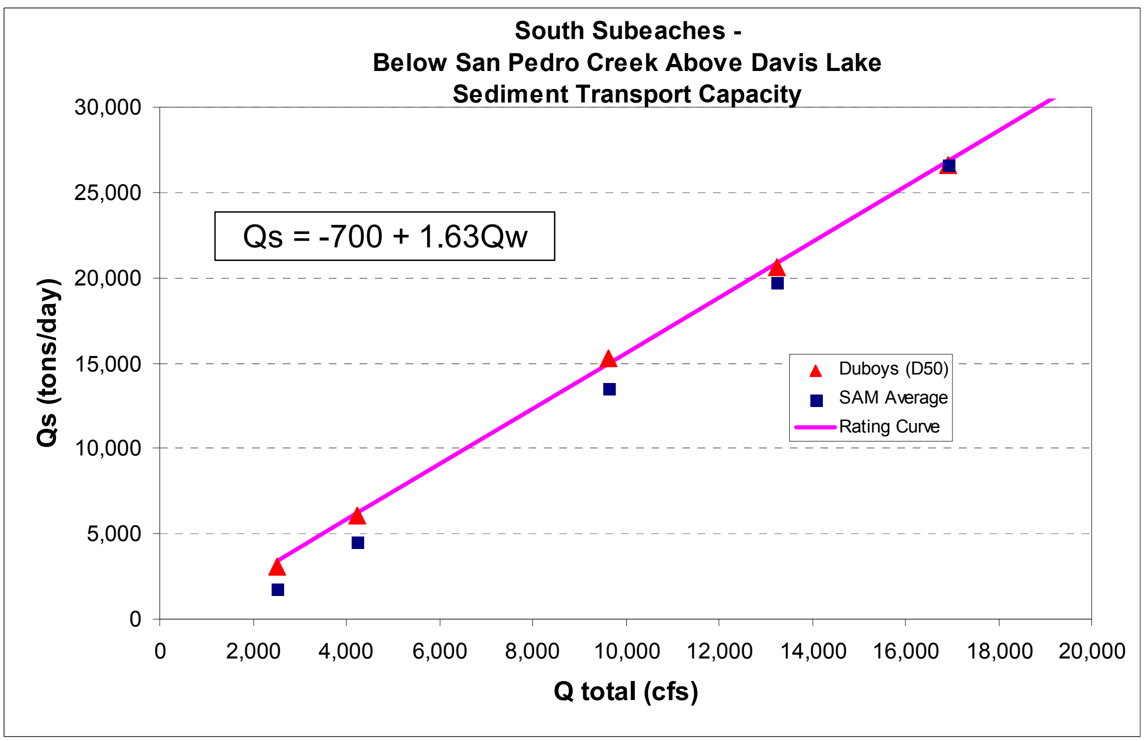


Figure T.6 Bed-Load Rating Curve for the South Reach below San Pedro Creek and Upstream of Davis Lake

The sediment transport capacity of the San Antonio River mainstem upstream of the confluence with San Pedro Creek was computed as the average of the Eagleland and Blow Outlet subreach as shown in Figure T.7.

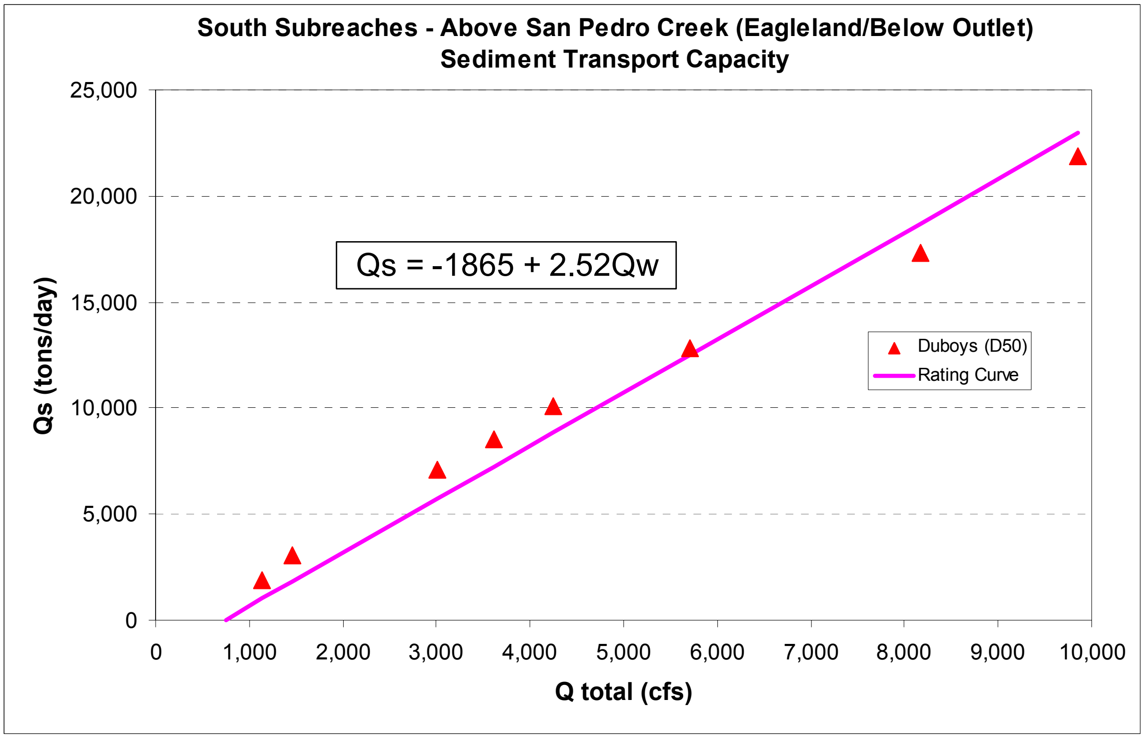


Figure T.7 Bed-Load Rating Curve for South Reach above San Pedro Creek (Eagleland and Below Outlet Subreaches)

To determine the sediment supply loading from San Pedro Creek a sediment transport rating curve was developed for this subreach as shown in Figure T.8.

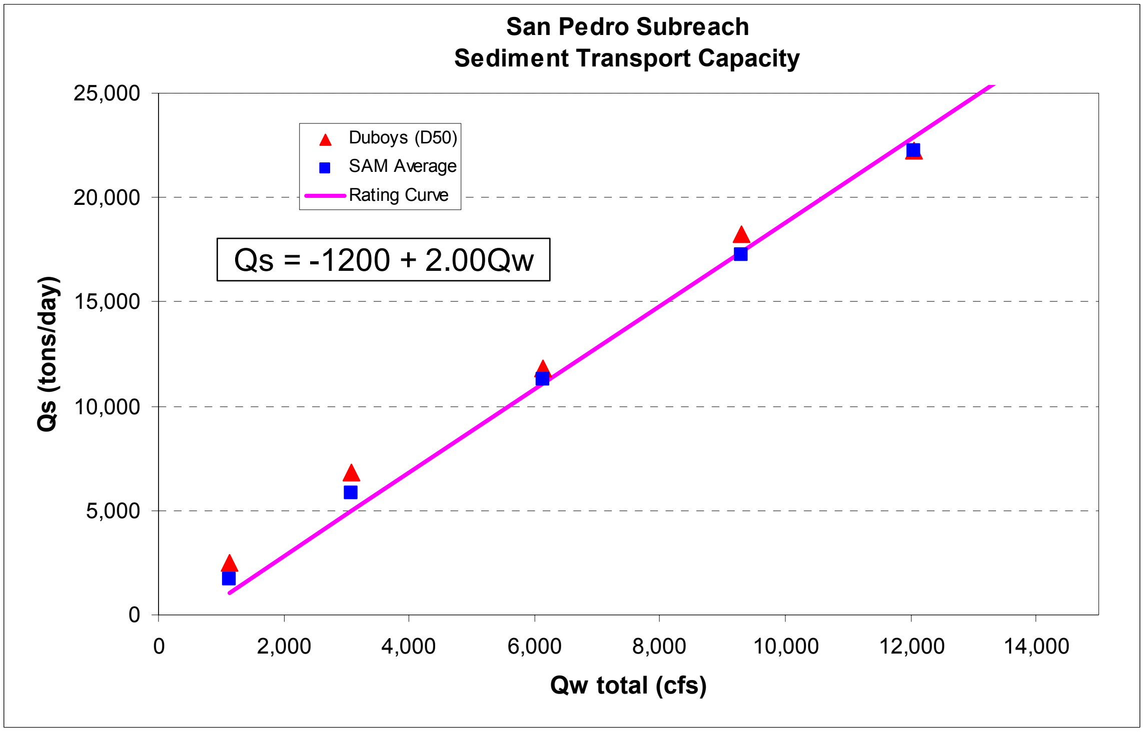


Figure T.8 Bed-Load Rating Curve for San Pedro Creek

The sediment transport rating curves were used to compute sediment transport rates and sediment yield for each of the discharge classes defined on the flow duration curve. The percent occurrence and sediment contribution values using the South Reach average below San Pedro Creek sediment transport rates are presented in Figure T.9.

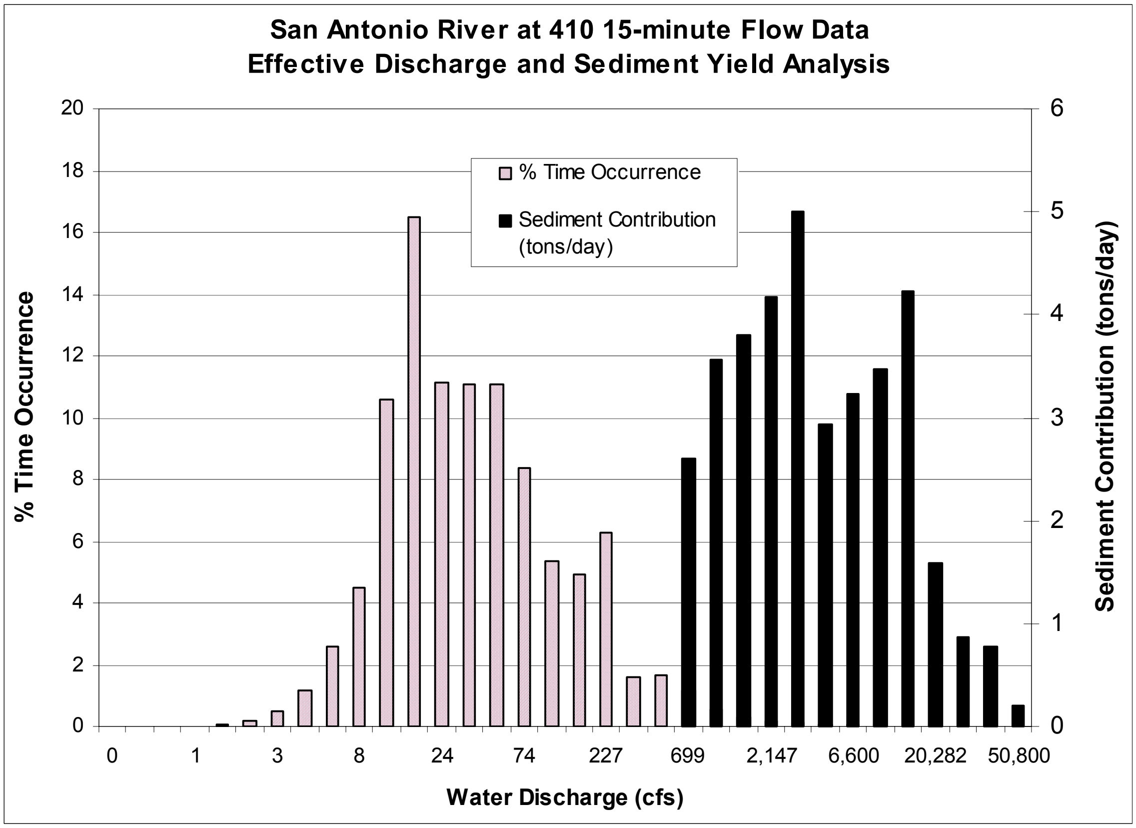


Figure T.9 Effective Discharge and Sediment Yield Estimation at the Loop 410 gage.

The percent time occurrence of flow and sediment contribution is shown. The flow histogram re-illustrates that the baseflow would be in the vicinity of 20 cfs. The sediment contribution was computed as the product of the sediment load rate for each discharge class and the percent time of occurrence. Results from the computation indicate that the effective discharge would be near 3,200 cfs with an average annual sediment yield of approximately 13,000 tons/year. The sediment delivery distribution is somewhat bimodal with a significant percentage of sediment delivery also occurring near 14,000 cfs. However data show that the existing channel has expanded to a capacity near 3,000 cfs, which may be near a new regime type condition. Following comparison with field indicators, hydraulic modeling, and other methods, an effective discharge of the 3,200 cfs was selected for the San Antonio River near the 410 gage.

For the S. Alamo gage the sediment transport rating curve developed for the reaches immediately downstream of the gage were used because it was desired to estimate an effective discharge in these reaches. The percent occurrence and sediment contribution values are presented in Figure T.10.

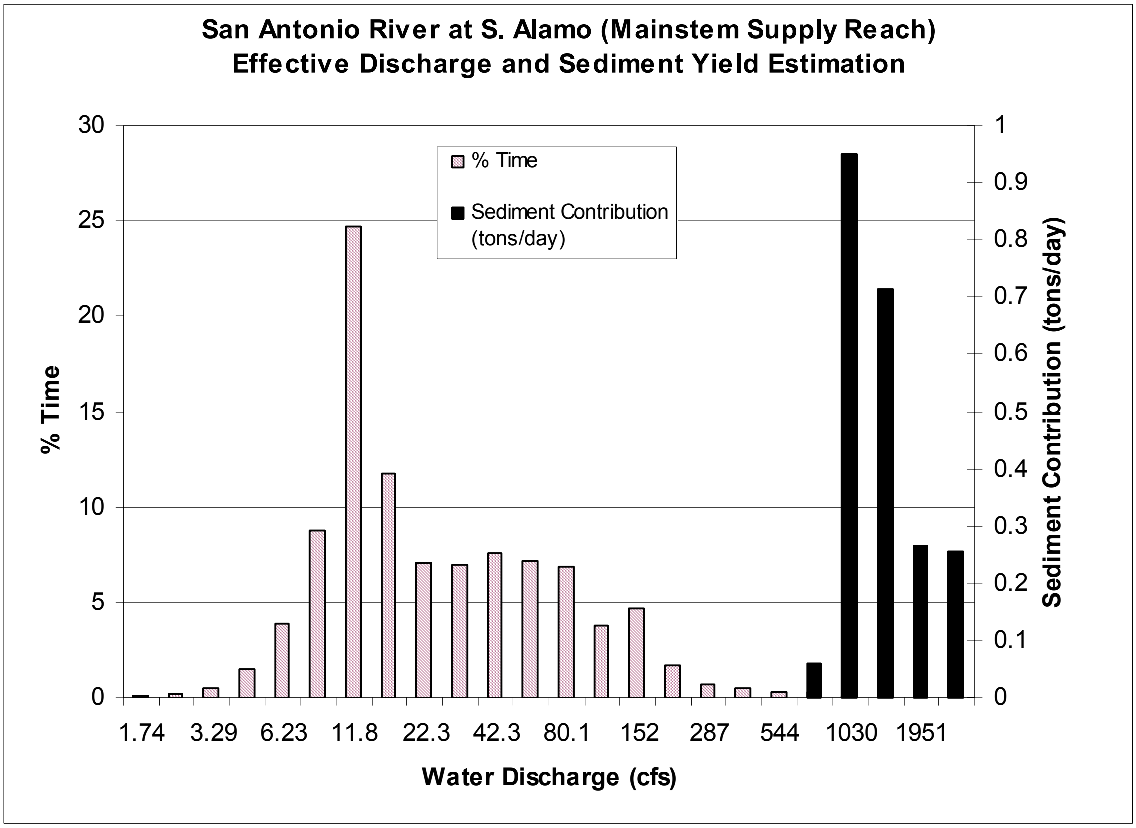


Figure T.10 Effective Discharge and Sediment Yield Estimation at the S. Alamo gage.

Results indicate that the effective discharge would be near 1,100 cfs with an annual sediment yield of approximately 820 tons/year. This location is the mainstem supply for the South Reach below the confluence with San Pedro Creek. Comparing with other methods and hydraulic modeling this effective discharge value was confirmed as reasonable. In both effective discharge calculations the for the S. Alamo and Loop 410 gages the class interval above which sediment transport becomes significant was on the order of 500 cfs. Results from the effective discharge estimation were used to develop flows in the hydraulic model representing an effective discharge at various locations throughout the model. Effective flows at various inflow locations were scaled by drainage area using either the S. Alamo or Loop 410 discharge depending on the proximity to that particular gage. Effective discharge values for each subreach are listed in Table T.4.

	Table T.4 Subreach 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

Sediment Budget

A sediment budget was developed for the South Reach of the San Antonio River using results from the previous analyses and the historical channel assessment. It was desired to determine the sediment supply to the South Reach for prediction of equilibrium conditions to satisfy sediment continuity. The significant sediment supply reaches were identified as the San Antonio River mainstem and San Pedro Creek. Field assessment established that sediment loads from Six Mile Creek and other minor tributaries were relatively negligible. For the receiving reach of the sediment budget the San Antonio River downstream of San Pedro Creek and upstream of Davis Lake (Conception, Mission and San Juan subreaches) were used since Davis Lake acts as a downstream control. The historical channel assessment provided quantities of material that had been eroded from channel segments since construction of the floodway. The floodway construction in this area ended around 1960 and therefore a base period of 40 years was used to estimate annual erosion rates. The quantities of material eroded from the channels and annual rates of erosion for the sediment budget area are listed in Table T.5.

Table T.5 Channel Erosion Rates and Computed Sediment Yields for the San Antonio River and Supply Reaches

	Length	Channel Erosion		Channel Erosion Rate	Computed Sediment Yield
Reach	(ft)	(yd^3)	(tons)	(tons/year)	(tons/year)
Supply 1 - San Antonio River Mainstem Upstream of San Pedro Confluence (Below Outlet Subreach)	6,075	16,847	22,516	563	821
Supply 2 – San Pedro Creek Floodway	8,000	42,747	57,131	1,428	5,540
South – San Antonio River Downstream of San Pedro Creek and Upstream of Davis Lake	16,650	256,495	342,805	8,570	13,096

Channel erosion rates were computed as the observed cumulative channel erosion divided by a period of 40 years. Computed sediment yields were determined from integration of percent time of occurrence and sediment transport capacity of each reach using flow duration curves as also listed in Table T.5. Sediment yields for the receiving reach (South) and for San Pedro Creek were estimated by scaling the flow duration curve from the Loop 410 gage based on uncontrolled drainage area. The ratio of the San Pedro Creek drainage area at the mouth and the uncontrolled drainage area of the San Antonio River (below Olmos Dam) is approximately 0.5. The ratio used for the receiving reach was approximately 0.8. It was assumed that the hydrologic effects of urbanization were similar for both of the San Pedro and San Antonio River basins. The sediment budget developed from this information is shown in Figure T.11.

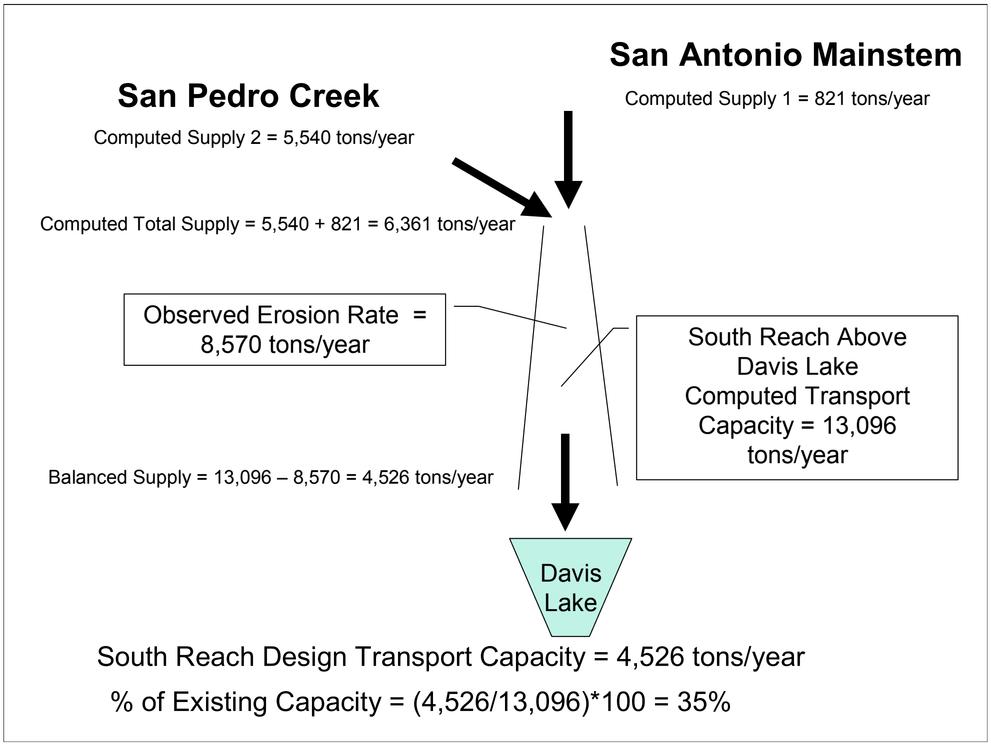


Figure T.11 Conceptual Sediment Budget for the San Antonio River

As illustrated the estimated sediment load to the South Reach below the confluence with San Pedro Creek and upstream of Davis Lake would be approximately 6,400 tons/year based on computed yields from the mainstem and from San Pedro Creek. The computed sediment supply from San Pedro Creek is nearly 7 times that from the mainstem. This appears reasonable since the drainage area for San Pedro Creek at its mouth is approximately 5 times the uncontrolled drainage area for the mainstem at the S. Alamo gage. The total drainage areas for the San Antonio River and San Pedro Creek are nearly equal at the confluence, but Olmos Dam controls 32 square miles of the upstream San Antonio River basin. The two largest impoundments on San Pedro Creek are Elmendorf Lake and Woodlawn Lake are for water supply and do not significantly inhibit sediment transport through the system especially during flood flows.

Using the computed supply based on sediment transport equations and the computed transport of approximately 13,100 tons/year, the South Reach would need to be modified to reduce the existing transport capacity by approximately 50% for sediment continuity. However, that computed sediment budget would result in a erosion rate of approximately 6,700 tons/year (13,096 – 6,361), which is less than 80% of the observed value of approximately 8,600 tons/year. This indicates that the computed supply is conservative and would result in nonconservative estimates of equilibrium channel parameters. In addition, as urbanization increases, it is expected that the available sediment load from upstream sources will decrease over time. Based on the historical channel assessment the observed erosion rate for the design reach was 8,570 tons/year. Balancing this with the computed yield of 13,096 tons/year results in a more realistic supply value of 4,526 tons/year, which is approximately 35% of the existing transport capacity. Based on the assumptions used for determining the pass-through load in the Below Outlet subreach its transport capacity would need to be reduced from 821 tons/year to 258 tons/year for a reduction of 70%. The following sections utilize this information to quantify the channel geometry and gradients required to meet these criteria.

Stable Channel Analysis

Using results from the effective discharge estimation and the sediment budget, equilibrium channel dimensions required for channel stability were computed. The parameters were computed using equilibrium concepts of sediment continuity for alluvial systems. Using Duboy's equation for sediment transport and equating the sediment transport capacity of each subreach to the specified sediment supply provided a relation that satisfies sediment continuity ($Q_{SS}=Q_{SO}$). The equilibrium condition to satisfy the Duboy's equation can be represented with the following equation (Byars et al. 2001).

$$S_{eq} = \left(\frac{CW}{Q_w n}\right)^{\frac{6}{7}} \left[\frac{1}{\gamma}\left(\frac{Q_{SS}}{\alpha W}\right)^{\frac{1}{\beta}} + SP(S_g - 1)D_s\right]^{\frac{10}{7}} \quad \text{(Equation T.7)}$$

where:

S_{eq}	=	equilibrium slope to transport the incoming sediment load
C	=	coefficient for Manning's equation (1.486 for English 1.0 for SI)
Q_{SS}	=	Volumetric bed material load into the reach from upstream sources (L^3/T)
n	=	Manning's roughness coefficient
W	=	Channel width (L)

*all other terms previously defined

Equation T.7 represents the energy slope and channel width required such that the channel bed-load transport capacity matches the sediment supply defined by Q_{ss}. In this analysis the water discharge and corresponding hydraulic variables are those associated with the effective discharge as previously discussed. In the case where the sediment supply is considered negligible a threshold approach may be used. This results in channel dimensions such that transport of bed is impeded ($Q_{SS} = Q_{SO} = 0$) and results in:

$$S_{eq} = \left(\frac{CW}{Q_w n}\right)^{\frac{6}{7}} \left[SP(S_g - 1)D_s\right]^{\frac{10}{7}} \quad \text{(Equation T.8)}$$

Assuming a sediment supply of 4,526 tons/year for the South Reach below San Pedro Creek and a supply of 258 tons/year for the Below Outlet subreach equilibrium channel parameters were computed for each of the subreaches.

Equations T.7 and T.8 can be solved for various widths allowing a range of probable solutions that would satisfy sediment continuity. In design of the main channel consideration for the effective discharge and regime should be included in mobile boundary segments of the river. Because there are no suitable relationships for appropriate channel width in urban systems, especially in Central Texas, the best indicator may be the existing channel width in presumably stable sections. However changing hydrology from urbanization should also be considered in development of design channel widths. Equilibrium channel dimensions computed from Equation T.8 and the assumed sediment supply are listed in Table T.6.

Table T.6 Equilibrium Stable Channel Dimensions for the South Reach of the San Antonio River using Computed Sediment Supply

Subreach	Below Outlet		Conception		Mission		San Juan		Davis		Below Espada		Six Mile		410	
Width	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth
(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)
30	0.16%	5.8	0.10%	10.0	0.09%	10.6	0.09%	10.5	0.08%	11.9	0.08%	11.5	0.09%	10.5	0.07%	13.1
40	0.18%	**4.7**	0.11%	8.1	0.10%	8.6	0.10%	8.5	0.09%	9.7	0.10%	9.4	0.10%	8.5	0.08%	10.7
50	0.20%	4.0	0.12%	6.9	0.11%	7.3	0.12%	7.2	0.10%	8.2	0.11%	7.9	0.12%	7.2	0.09%	9.1
60	**0.22%**	3.4	0.13%	6.0	0.13%	6.4	0.13%	6.3	0.11%	7.2	0.11%	6.9	0.13%	6.3	0.10%	7.9
70	0.24%	3.1	0.14%	5.4	0.13%	5.7	0.14%	5.6	0.12%	6.4	0.12%	6.2	0.14%	5.6	0.11%	7.1
80	0.25%	2.8	**0.15%**	**4.9**	**0.14%**	**5.1**	0.15%	5.1	0.13%	5.8	**0.13%**	**5.6**	0.14%	5.1	0.12%	6.4
90	0.27%	2.5	0.16%	4.4	0.15%	4.7	0.15%	4.7	0.14%	5.3	0.14%	5.1	0.15%	4.7	**0.12%**	**5.8**
100	0.29%	2.3	0.17%	4.1	0.16%	4.3	0.16%	4.3	0.14%	4.9	0.15%	4.7	0.16%	4.3	0.13%	5.4
110	0.30%	2.2	0.18%	3.8	0.17%	4.0	0.17%	4.0	0.15%	4.5	0.16%	4.4	0.17%	4.0	0.14%	5.0
120	0.32%	2.0	0.19%	3.6	0.18%	3.8	0.18%	3.7	0.16%	4.2	0.16%	4.1	**0.18%**	**3.8**	0.14%	4.7
130	0.33%	1.9	0.20%	3.4	0.19%	3.5	0.19%	3.5	0.17%	4.0	0.17%	3.9	0.19%	3.5	0.15%	4.4
140	0.35%	1.8	0.21%	3.2	0.19%	3.4	**0.20%**	**3.3**	0.17%	3.8	0.18%	3.7	0.20%	3.3	0.16%	4.2
150	0.36%	1.7	0.21%	3.0	0.20%	3.2	0.20%	3.2	0.18%	3.6	0.19%	3.5	0.20%	3.2	0.16%	4.0
175	0.40%	1.5	0.23%	2.7	0.22%	2.8	0.22%	2.8	0.20%	3.2	0.20%	3.1	0.22%	2.8	0.18%	3.5
200	0.43%	1.4	0.25%	2.4	0.24%	2.5	0.24%	2.5	**0.21%**	**2.9**	0.22%	2.8	0.24%	2.5	0.19%	3.2
225	0.46%	1.2	0.27%	2.2	0.26%	2.3	0.26%	2.3	0.23%	2.6	0.24%	2.5	0.26%	2.3	0.21%	2.9
250	0.49%	1.1	0.29%	2.0	0.27%	2.1	0.28%	2.1	0.24%	2.4	0.25%	2.3	0.28%	2.1	0.22%	2.7

The highlighted sections are for channel widths closest to the average existing conditions throughout each area. In general the computed equilibrium slopes for the selected widths are near the historic condition with the exception of Davis Lake and the Six Mile subreach. Existing conditions channel dimension at the effective discharge are listed in TableT.7.

Table T.7 Existing Channel Dimensions at the Effective Discharge			
	Width	**Slope**	**Depth**
Subreach	(ft)	(ft/ft)	(ft)
Below Outlet	57	0.28%	4.0
Conception	82	0.22%	4.6
Mission	81	0.19%	5.0
San Juan	136	0.20%	4.2
Davis	213	0.03%	6.9
Below Espada	81	0.08%	7.1
Six Mile	117	0.31%	3.4
410	89	0.14%	6.1

Compared to existing conditions the Below Outlet, Conception, Mission, Six Mile, and 410 subreaches would need to increase width and/or decrease slope to achieve sediment continuity. The Davis subreach slope is well below the required slope to prevent erosion, hence the observed accumulation over the years. The analysis indicates that the San Juan subreach is currently stable which is consistent with the historical channel assessment, which described it as slightly depositional. The Below Espada subreach would be depositional considering the assumed sediment supply. However clear water flows from Espada Dam include much less sediment than that assumed and a negligible supply condition may need to be considered in this subreach. The surprisingly stable Six Mile reach is affected by diversion of low flows to the remnant channel and may not need hydraulic structures as suggested by the analysis.

If a negligible sediment supply were assumed for the design condition and Equation T.8 were used, more extreme hydraulic modification would be required. At incipient motion for the effective discharge the results provided in Table T.8 would be used.

Table T.8 Equilibrium Stable Channel Dimensions for the South Reach of the San Antonio River using Negligible Sediment Supply

Subreach	Below Outlet		Conception		Mission		San Juan		Davis		Below Espada		Six Mile		410	
Width	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth	Slope	Depth
(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)	(ft/ft)	(ft)
30	0.05%	8.2	0.03%	14.6	0.03%	15.4	0.03%	15.3	0.02%	17.3	0.02%	16.8	0.03%	15.3	0.02%	19.1
40	0.06%	6.4	0.04%	11.4	0.03%	12.0	0.03%	11.9	0.03%	13.5	0.03%	13.1	0.03%	12.0	0.03%	15.0
50	0.08%	5.3	0.04%	9.4	0.04%	9.9	0.04%	9.9	0.04%	11.2	0.04%	10.8	0.04%	9.9	0.03%	12.4
60	0.09%	4.5	0.05%	8.0	0.05%	8.5	0.05%	8.4	0.04%	9.6	0.04%	9.3	0.05%	8.5	0.04%	10.6
70	0.10%	4.0	0.06%	7.0	0.05%	7.4	0.05%	7.4	0.05%	8.4	0.05%	8.1	0.05%	7.4	0.04%	9.3
80	0.11%	3.5	0.06%	6.3	0.06%	6.6	0.06%	6.6	0.05%	7.5	**0.06%**	**7.2**	0.06%	6.6	0.05%	8.3
90	0.13%	3.2	0.07%	5.7	0.07%	6.0	0.07%	6.0	0.06%	6.8	0.06%	6.5	0.07%	6.0	0.05%	7.5
100	0.14%	2.9	0.08%	5.2	0.07%	5.5	0.07%	5.4	0.07%	6.2	0.07%	6.0	0.07%	5.5	0.06%	6.8
110	0.15%	2.7	0.08%	4.8	0.08%	5.1	0.08%	5.0	0.07%	5.7	0.07%	5.5	0.08%	5.0	0.06%	6.3
120	0.16%	2.5	0.09%	4.4	0.09%	4.7	0.09%	4.7	0.08%	5.3	0.08%	5.1	0.09%	4.7	0.07%	5.8
130	0.17%	2.3	0.10%	4.1	0.09%	4.4	0.09%	4.3	0.08%	4.9	0.08%	4.8	0.09%	4.4	0.07%	5.4
140	0.19%	2.2	0.10%	3.9	0.10%	4.1	0.10%	4.1	0.09%	4.6	0.09%	4.5	0.10%	4.1	0.08%	5.1
150	0.20%	2.1	0.11%	3.7	0.10%	3.9	0.11%	3.8	0.09%	4.4	0.10%	4.2	0.11%	3.9	0.08%	4.8
175	0.22%	1.8	0.13%	3.2	0.12%	3.4	0.12%	3.4	0.11%	3.8	0.11%	3.7	0.12%	3.4	0.10%	4.2
200	0.25%	1.6	0.14%	2.9	0.13%	3.0	0.13%	3.0	0.12%	3.4	0.12%	3.3	0.13%	3.0	0.11%	3.8
225	0.28%	1.5	0.16%	2.6	0.15%	2.7	0.15%	2.7	0.13%	3.1	0.14%	3.0	0.15%	2.7	0.12%	3.4
250	0.31%	1.3	0.17%	2.4	0.16%	2.5	0.16%	2.5	0.14%	2.8	0.15%	2.7	0.16%	2.5	0.13%	3.1

These conditions may be applicable only to the Below Espada Reach. This more conservative approach may limit the amount of construction due to the increased associated cost.

Conclusions of the Sediment Transport Analysis

Analysis of sediment transport on the San Antonio River was used to determine average channel dimensions that would satisfy a condition of sediment continuity and for negligible sediment supply conditions. The limited analysis considers channel width, depth, and slope as the only degrees of freedom in estimating channel parameters for an alluvial system. The computed stable channel values can be used to layout channel features such as pools and riffles from which they may vary about, but the resultant reach wide average should be near the values presented to prevent aggradation or degradation of the channel. The sediment budget was based on observed historical channel erosion and sediment transport rates for existing conditions. As urbanization affects future hydrology and sediment load to the system, modification to the stable channel geometry estimates will be required. The design implications are that additional structures or modification to designed hydraulic structures may be required. Further, as future channel adjustment and sediment data becomes available for the San Antonio River a refined analysis considering sediment routing and more complex hydraulic representation could be performed. This analysis provides reasonable estimates of average parameters for design, based on available information. Stable channel dimensions could be predicted with much greater certainty if bedload and suspended load concentrations were measured over an extensive range of flows. Such measurements take a considerable time to obtain, however, and the schedule and budget of this project does not allow for their collection.

South Reach Design Criteria

The design criteria provided in this section is a summary of recommendations resulting from the geomorphic and sediment transport analyses. These recommendations recognize that the channel design will be based on SWA Group's Design Guidelines previously prepared in the planning phase of this project. While the Design Guidelines addressed channel alignment and geometries, the basis for analysis lacked substantial geomorphic and sediment transport assessment. Thus, while efforts have been made to follow the Design Guidelines regarding alignment and channel geometry, changes have occurred based on a more thorough analysis. These design criteria have incorporated elements from a variety of investigations and analyses to promote the development of a multi-objective project to meet diverse design goals while considering site constraints. Each subsection of the design criteria contains a bulleted list of major design considerations that is supported by subsequent text.

Channel Design

Channel design must consider numerous interrelated design criteria that consider geomorphology, sediment transport, low flow and flood hydraulics, vegetative condition, habitat value, and recreation. The approach to channel design assumes that the natural character of the constructed channel should be maximized, while recognizing that an urbanized watershed and a narrow meander corridor within the constructed floodway limit the functionality of natural processes in the San Antonio River. Thus, these design criteria strive to maximize natural function while recognizing watershed condition and site constraints.

Flood Conveyance

- Project elements shall be designed to withstand the erosive energies associated with the 100-year interval flood event.
- The project shall be designed so that there will be no increase in height of the 100-year flood elevation unless the affected land is owned by, or flood easements have been granted to, SARA.

Maintaining flood conveyance is necessary for the protection of property and life. Designs shall not have a detrimental effect on flood conveyance. Designs should result in hydraulic conditions that contain the 100-year flood within the floodway where the existing conditions are contained within the floodway. The design should not increase 100-year flood elevation where property and structures are currently at risk.

Erosion potential is another aspect of flood conveyance. In rivers that spread out over a wide floodplain during floods, there are relatively small incremental increases in depth as floods rise. However, when the San Antonio River floods, flows get deeper and faster within the confined floodway. This condition raises erosion potential significantly as flows increase. These design criteria recognize that the 100-year flood can exceed 25 feet of depth and 20 feet per second flow velocities. Subsequent sections reflect considerations for erosion potential during floods.

Channel Alignment

Based on the existing hydrologic regime, geomorphic and hydraulic conditions, and dimensions of the available floodway:

- Floodway, pilot and base flow channels shall be in phase (i.e. pilot and base flow channel bends shall be located at bends of the existing floodway).
- Pilot channel sinuosity shall be within a range of 1.0 to 1.2.
- Pilot channel bend radii shall fall within a range of 400 to 700 feet upstream of San Pedro Creek and 1000 to 2000 feet downstream of San Pedro Creek.
- Pilot channel meander amplitudes shall be slightly less than the floodway bottom width, 280 to 460 feet.
- Pilot channel meander wavelengths shall fall within a range of 1000 to 1500 feet upstream of San Pedro Creek and 2500 to 3000 feet downstream of San Pedro Creek.
- The pilot channel should accommodate existing infrastructure and generally be aligned perpendicular to bridge crossings.
- Through straight reaches of the floodway the pilot channel sinuosity and, wavelength and bend radii shall follow criteria described above considering infrastructure and other constraints.

The channel design will include a 3-phase system that includes the floodway channel, a pilot (or dominant) channel and a base flow channel. The pilot channel will be superimposed on the floodway channel and the base flow channel superimposed within the pilot channel. The floodway channel alignment is somewhat fixed in its current alignment, a relatively straight three to four hundred feet wide corridor. Although increasing sinuosity of the pilot channel within the floodway is desirable from a slope reduction and energy dissipation perspective, it is problematic if it induces turbulent scour at flood flows. Due to the character of the floodway (i.e. flood flows get deeper and swifter instead of spread out), a relatively sinuous pilot channel would generate angular momentum and create vortices on the floodway during large floods and likely result in pilot channel realignment. Designing the pilot channel to be in-phase with the floodway reduces this risk, as the direction of flow in the pilot channel would be somewhat parallel with floodway flows. This is also true for the base flow channel. This in-phase system anticipates that meander wavelengths will be approximately equal for the three channels, that bend radii will slightly decrease for smaller, in-phase, channels, and that channel bends will occur at approximately the same location. Any modifications to the floodway alignment are subject to property acquisition. Where property is acquired adjacent to the floodway, the floodway may be widened which may allow for increased sinuosity of the pilot and base flow channel within the criteria provided herein. Refer to Figure D.1 for conceptual planform alignment.

The Design Guidelines' channel alignment (planform) is based on empirical relationships of river systems throughout the United States. The specific dimensions provided in the Design Guidelines (wavelength, and radius of curvature) were based on the theory of minimum variance (Langbein and Leopold, 1966), the natural tendency for rivers to form geometries that expend the least amount of energy or work. Their study showed river meanders tend to follow a planform pattern with geometry similar to a sine-generated curve.

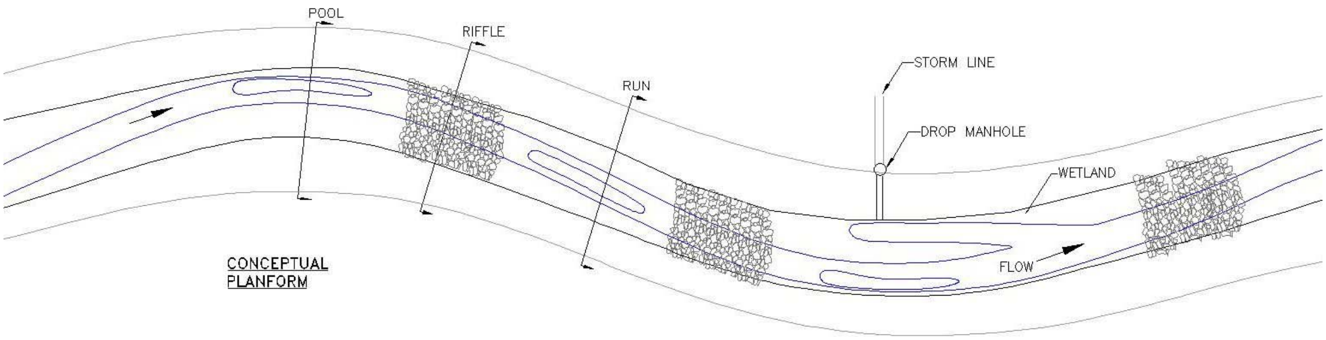


Figure D.1 Conceptual Planform Alignment

These design criteria included a literature search to determine the availability of additional studies or research that established channel geometry regime equations that were more applicable to the south-central plains of the United States, a semi-arid climate that is subjected to frequent and intensive precipitation events, similar to the San Antonio River Basin's flashy hydrology. Hedman and Osterkamp, (1982) provided a method to determine streamflow using channel geometry relations (mainly, active channel width) by applying empirical equations to data of similar stream types and climatic characteristics. Williams (1986) used large sets of empirical data from past landmark channel geometry research (Leopold and Wolman, 1960) to determine the extent in which theory may predict observed relations and to examine the distribution of values of the ratio bend radius of curvature /channel width, and to derive new equations involving meander geometry and channel size. More recent work by the Army Corps of Engineers provided innovative methods to determine planform geometry using a combination of analytical and empirical techniques derived from nine available data sets consisting of 438 sites (Soar and Thorne, 2001, Copeland et. al, 2001). Assuming an average bankfull width (effective discharge) of 80 to 100 feet, empirical equations reviewed in the preceding literature are not directly applicable to determine average channel geometry dimensions for the San Antonio River Mission South Reach. The majority of empirical channel geometry data available, regardless of physiographic region and climate, are collected from mostly non-urbanized rivers and streams whose sinuosity is greater than 1.2. The existing San Antonio River floodway downstream from the confluence with San Pedro Creek to the end of the Mission South project reach has a sinuosity of slightly less than 1.1 to 1.2 depending on the method of calculation. The average belt width of the Mission South Reach ranges from approximately 300 to 500 feet. Given the hydrologic regime and constrained nature of the San Antonio River floodway, empirical equations found in this literature review do not apply when an average bankfull width of 80 to 100 feet is used for the proposed design pilot channel. Geomorphic observation and analysis and hydraulic analysis of the existing channel suggests this range of pilot channel top widths (effective discharge) will provide both flood and sediment conveyance.

Pilot Channel Geometry

- The pilot channel shall be designed to convey the effective discharge as a bank full event. The effective discharge varies according to Table T.4.
- The pilot channel shall be composed of pool, riffle and run sequences.
- Reach average slopes should approach the approximate sediment transport equilibrium slope as highlighted in Table T.6, while recognizing the uncertainty associated with those values.
- Constructed riffle gradient structures shall be the primary means of attaining reach average equilibrium slopes.
- Adjusting channel dimensions and roughness shall be the secondary means of attaining reach average equilibrium slopes.
- The pilot channel shall be sized according the approximate channel dimensions highlighted in Table T.6, while recognizing the uncertainty associated with those values.
- The capacity of the pilot channel shall equal the effective discharge along its throughout each reach.

Designs shall provide continuity of sediment transport through the project reach. The design shall provide a sediment transport equilibrium condition to the extent practicable. Sediment transport equilibrium conditions will be based upon an effective discharge calculated for the reach. The channel design will include appropriate grade control (height and spacing) and channel geometry to achieve the equilibrium slope necessary for sediment continuity. Providing the above considerations for sediment transport, the design will avoid excessive erosion or deposition within the channel and reduce associated maintenance requirements.

Pilot channel geometry shall be based upon reach average equilibrium slopes and a pilot channel comprised of pool, riffle and run sequences within the floodway. Pools will be located at the outside of bends in the pilot channel. Runs will be located within the pilot channel between channel bends. Riffles shall provide hydraulic control of pools and runs and gradient drops to achieve reach average equilibrium slopes. Refer to Figure D.1 for conceptual pool, riffle, and run relative locations. Conceptual cross sections for riffle, pool, and runs are shown on Figures D.2, D.3, and D.4, respectively. Approximate pilot channel dimensions for subreaches are presented in Table T.6 or the Sediment Transport section of this memorandum.

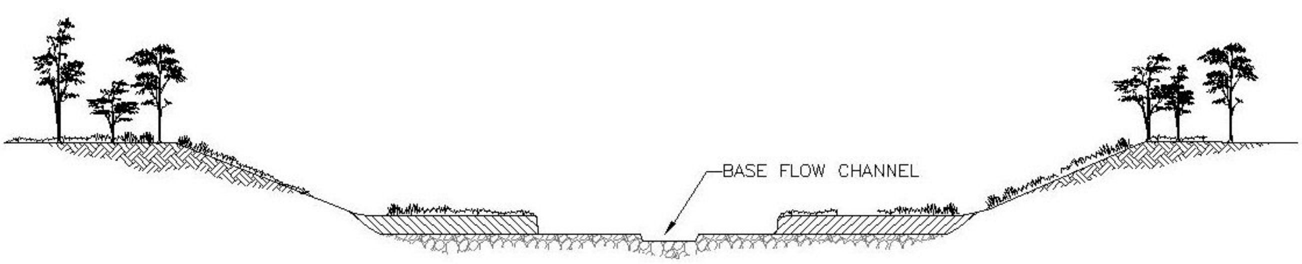


Figure D.2 Riffle Section

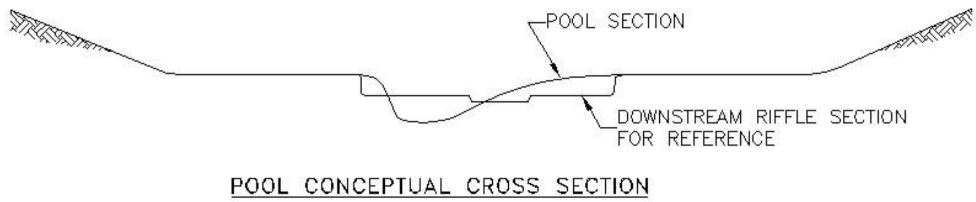


Figure D.3 Pool Section

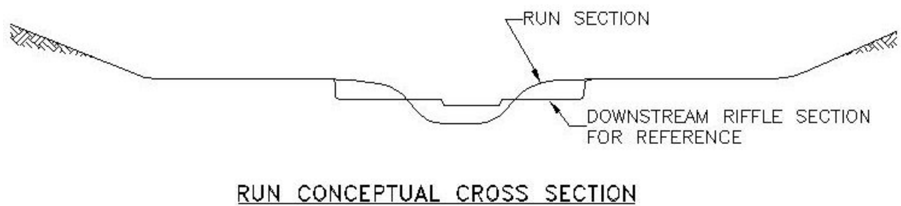


Figure D.4 Run Section

Constructed riffle gradient structures will be the primary means of controlling grade and attaining reach average equilibrium slope. That is, drops in grade will primarily occur along the lengths of riffles, and the length weighted, average slope of the energy grade line through the reach should equal the equilibrium slope recommended in Table T.6, while recognizing the uncertainty associated with those values. Riffle height and length will be adjusted and hydraulically modeled until the length weighted, reach average, energy grade line of the effective discharge flow matches the sediment transport equilibrium slope. This is an iterative design/modeling approach.

Constructed riffle gradient structures will be constructed of stone or concrete of suitable size and density to withstand hydraulics up to the 100-year flood event. These structures will span the floodway and extend up the floodway banks to avoid flanking. The thickness of constructed riffle gradient structures shall exceed anticipated scour and long-term degradation estimates. The constructed riffle gradient structures will accommodate floodway, pilot channel, and base flow channel geometries.

Another aspect of pilot channel geometry with respect to floodway/pilot channel interaction that must be considered is how flows overtop the pilot channel during floods. Pilot channel overtopping must occur simultaneously throughout the entire project to avoid creating conditions where flows are exiting and reentering the pilot channel in uncontrolled locations. Exiting and reentering of flows creates conditions prone to excessive erosion and potential channel avulsion. The pilot channel will be designed to convey the effective discharge of approximately 3200 cfs, at the I-410 gage, as a bank full event. The effective discharge varies according to Table T.4 in the Sediment Transport section of this memorandum.

Pool cross sections shall be asymmetric with pools located at the outside of bends in the pilot channel and point bars located at the inside of bends adjacent to the pools. Base flow channel geometry may be ignored through pools since downstream riffles will create a backwatered, residual pool depth 2 to 4 feet.

Runs shall be located in straight pilot channel reaches, between riffles. Run cross sections shall be approximately symmetric. Base flow channel geometry may be ignored through runs since downstream riffles will create a backwatered, residual depth of 1.5 to 3 feet.

Floodway Channel Geometry

- The floodway channel geometry shall enhance sediment transport continuity at flood flows.
- Transition from floodway to natural channel at the downstream end of project shall be improved.
- Floodway grading to increase the energy grade line through the 410 reach shall be performed.

It is important to distinguish between floodway channel hydraulics, and pilot channel hydraulics since different design criteria are placed upon the floodway and the pilot channel. The pilot channel is sized according to the effective discharge, while the floodway has been designed to convey the 100-year flood. When the pilot channel overtops, floodway flows will carry sediment, and thus, considerations for sediment transport on the floodway will be incorporated into the design. These considerations are different from those for the pilot channel, however. Reach average slopes of high magnitude floodway flows are not easily manipulated through discrete grade drops (constructed riffles) since the effect of these drops tend to wash out as flows increase. A goal of channel design is to avoid excessive erosion or deposition. Floodway erosion will be avoided by providing designs that will withstand erosion potential, while deposition may be avoided through grading the floodplain to promote sediment transport continuity. Creating a more uniform energy grade line for high magnitude flows will aid in promoting sediment transport continuity.

Deposition within the floodway is evident within the 410 subreach. The floodway channel will be reconfigured so that sediment currently deposited in the 410 floodway is transported downstream. The energy grade line of high magnitude flows is relatively flat through the 410 subreach. Creating conditions that increase the energy grade line of flood flows through the 410 subreach will improve sediment transport conditions. This may be accomplished through improving the transition from the floodway to the downstream channel and increasing the grade of the floodway through excavation. Currently, the floodway narrows down to meet the downstream natural channel and approximately 1000 feet downstream the natural channel widens out again. Removing this bottleneck and improving the transition would improve sediment transport through the 410 subreach. Floodway grading will be required to attain the pilot channel sized to carry the effective discharge. An additional aspect of floodway grading will include the goal of creating a 100-year flood, energy grade line of approximately 0.002 (reach average) through all subreaches, including the 410 subreach. This will potentially include narrowing the floodway in the Six Mile and 410 subreaches, as they are about 30 percent wider than upstream subreaches, if flood conveyance is not adversely affected. This is an iterative design/modeling approach.

Base Flow Channel Geometry

- The base flow channel shall be superimposed in constructed riffle gradient structures of the pilot channel. Considerations for the base flow channel geometry may be ignored in pools and runs of the pilot channel as pools and runs shall be backwatered by riffles to provide residual pool depth.
- The base flow channel shall be designed to meander through pilot channel riffles with a sinuosity of 1.1 with respect to riffle length.
- Base flow channel shall be sized to convey 20 cfs.
- Base flow channel dimensions are dependent upon pilot channel riffle slopes, and shall vary with pilot channel riffle slopes.
- Pilot channel riffles shall be designed to minimize losses to subsurface flow at base flow conditions.

Analysis of stream gage records indicates that the median base flow is approximately 20 cfs in the Mission Reach. The analysis indicates that base flow may exceed 200 cfs following rainfall events. The base flow channel will be sized for a 20 cfs flow volume, assuming flows that exceed 20 cfs will spread out over the pilot channel. Designing to the 20 cfs flow will provide a low flow notch in pilot channel riffles for aquatic habitat and recreation such as canoeing.

Base flow channel dimensions will be dependent on pilot channel riffle slopes. Since pilot channel riffle slopes represent an iterative design process that has not yet been performed, it is premature to provide pilot channel dimensions. For conceptual purposes base flow channel width and depth will be approximately 8 feet and 0.7 feet, respectively, based upon a 1% riffle slope. Determination of base flow channel dimensions will consider minimum depth requirements for fish and canoe passage.

Bank Stabilization and Reconstruction Materials

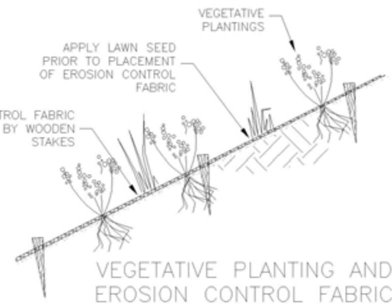
- Bank stabilization methods shall be based upon hydraulic shear, which relates to erosion potential.
- Erosion control treatments shall vary with bank height, since hydraulic shear varies with flow depth.
- Increased shear at channel bends shall be considered.
- Bank stabilization shall be designed to withstand flows up to the 100-year flood.

The intent of erosion protection is to create a channel that will withstand hydraulic forces, yet provide a pleasing natural appearance. The design approach will anticipate maximizing the use of natural materials and/or providing screening (with natural materials) of any man-made materials that might be used. For inundated areas that will not support vegetation, such as the riverbed and lower banks of the pilot channel, it is assumed that native limestone will be the preferred material.

Bank stabilization methods will be based upon hydraulic shear, which relates to erosion potential. Pilot channel and floodway bank treatments at a particular section may change with height up the bank, as shear varies with depth of flow. Toe of slope protection for bank stabilization will extend to a calculated depth of potential scour and long-term degradation. Also, bank protection will be keyed into the channel boundary at termination points to prevent flanking. Examples of bank stabilization and floodway reconstruction materials and methods as related to hydraulic shear are provided below. Note that designs will account for local hydraulic conditions such as, bend shear, local scour, and variations of hydraulic shear with depth.

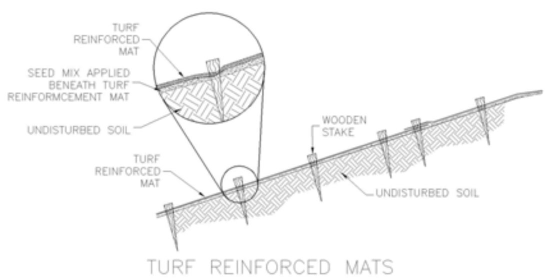
Vegetative Planting and Erosion Control Fabric

Application: Floodway Terrace
Floodway Banks
Recommended
Shear: <0.5 lbs/ft^2



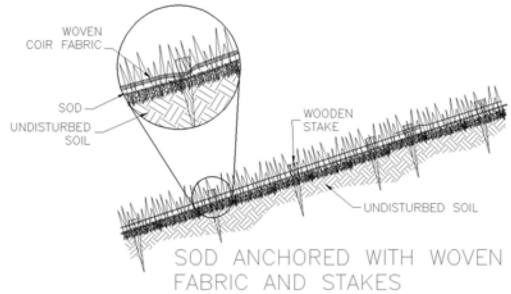
Turf Reinforcement Mats

Application: Pilot Channel Banks
Floodway Terrace
Floodway Banks
Recommended
Shear: Varies with manufacturer
Up to 2 – 2.5 lbs/ft^2 (unvegetated)



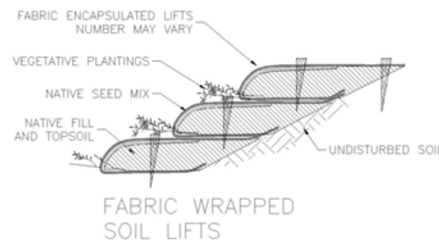
Sod and Woven Coir Fabric

Application: Pilot Channel Banks
Floodway Terrace
Floodway Banks
Recommended
Shear: 0.5 – 1.5 lbs/ft^2



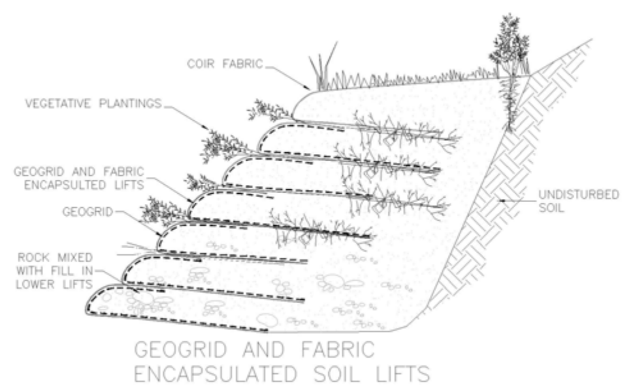
Coir Fabric Wrapped Soil Lifts

Application: Pilot Channel Banks
Floodway Banks
Recommended
Shear: 0.5 – 1.5 lbs/ft^2



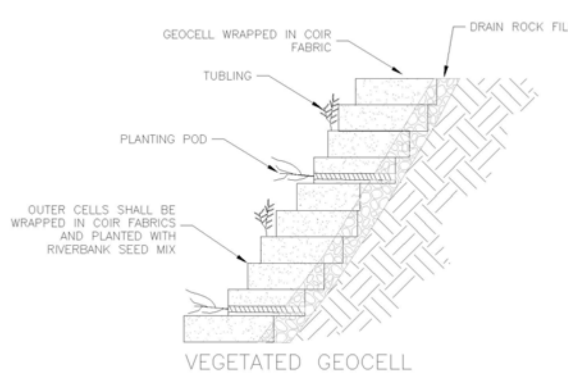
Geogrid and Coir Fabric Wrapped Soil Lifts

Application: Pilot Channel Banks
Floodway Banks
Recommended
Shear: 1.5 – 2.5 lbs/ft^2



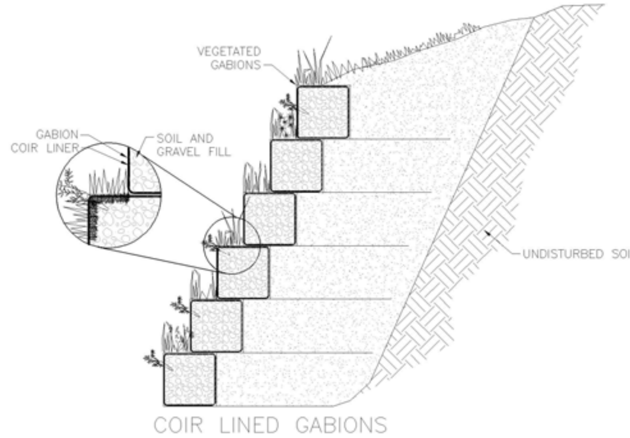
Vegetated Geocell

Application: Pilot Channel Banks
Floodway Banks
Recommended
Shear: 2.0 – 3.0 lbs/ft^2



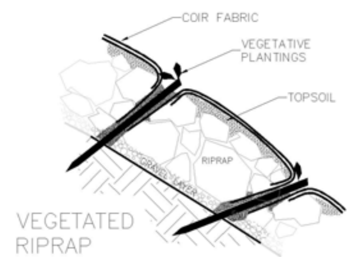
Coir Fabric Lined Gabions

Application: Pilot Channel Banks
Floodway Banks
Recommended
Shear: 2.0 – 3.0 lbs/ft^2



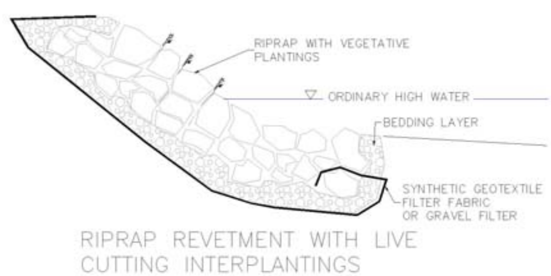
Vegetated Riprap

Application: Pilot Channel Banks
Floodway Banks
Recommended
Shear: 2.0 – 3.0 lbs/ft^2



Riprap Revetment

Application: Lower Pilot Channel Banks
Exclude Vegetation Below Ordinary
High Water
Recommended
Shear: 0.5 – 8.0 lbs/ft^2, depending on rock size



Water Quality

- Vegetation to maximize shade of river flows shall be used to the extent practicable.

- Created wetlands shall provide pretreatment of smaller stormwater outfall flows.

Designs must consider natural processes that enhance water quality. One of the best ways to enhance water quality is to establish riparian vegetation along the river corridor. Stands of riparian vegetation provide shade and nutrient uptake. As such, the channel and landscape design will provide shading of low flows while considering hydraulic roughness and its impact to flood conveyance.

Water quality may also be improved by creating wetlands within the floodway to receive flows from stormwater outfalls. These wetlands will provide some filtering of stormwater flows while creating habitat diversity.

Stormwater Outfalls

- Stormwater outfalls shall be retrofitted to fall into drop manholes with discharge pipes to deliver flow to constructed wetlands in the floodway or to the pilot channel depending upon outfall size.
- Only exiting outfalls that are 16-inch diameter or smaller shall be considered for wetland pretreatment.

Stormwater outfalls are a topic where these design criteria differ from the Design Guidelines. Placing rock structures on a slope exposed to flood hydraulics as proposed in the Design Guidelines is problematic. Design of the structures proposed in the Design Guidelines must account for slope stability, stormwater hydraulics, and stability during flooding conditions including turbulent scour induced by the structures themselves.

Existing storm water outfalls will be retrofitted to fall into drop manholes and discharge pipes to deliver flow to the floodway. Stormwater outfalls under 16-inch diameter will discharge from drop manholes to wetlands created in the floodway. Refer to Figures D.1 and D.5 for a conceptual drawing of a retrofitted stormwater outfall. Stormwater outfalls over 16-inch diameter will discharge directly to the pilot channel as available wetland area is not likely to be sufficient to provide treatment and larger outfalls are more likely to erode wetland areas.

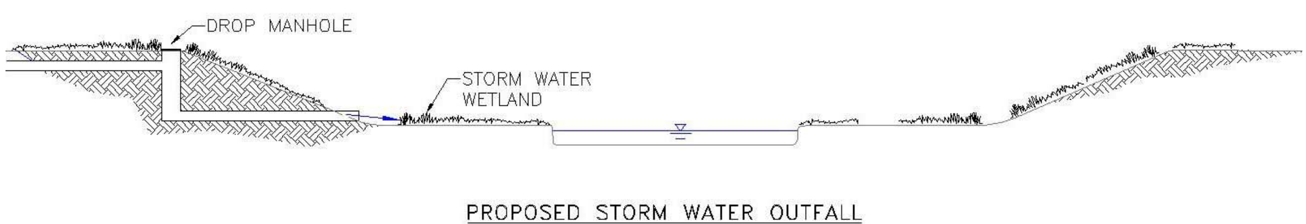


Figure D.5 Conceptual Storm Water Outfall Retrofit Section

Spill Structures

- Spill structures shall be stair stepped and avoid individual vertical drops greater than 1 foot in height.
- Spill structures shall be designed to prevent flanking and undermining considering both lateral and vertical channel adjustments.
- Spill structures shall allow for safe canoe passage at base flow conditions where practicable.

Spill structures provide a means to create "big water" features as proposed in the Design Guidelines. Spill structures may also be applied where tributary flows drop into the main channel. Spill structures will avoid vertical drops and consider application of constructed riffles where possible. If vertical drops are included, they will be stair stepped so that individual drops do not exceed 1 foot in height. The spill structures will be designed to resist hydraulic forces and remain stable through the 100-year flood. Spill structures will be designed to prevent flanking and undermining considering both lateral and vertical channel adjustments. Finally, spill structures will allow for safe canoe passage at base flow conditions where practicable.

Fish and Wildlife Habitat

- Aquatic habitat shall be enhanced by providing pool, riffle, and run sequences throughout the pilot channel.
- Created wetlands shall provide for more diverse habitat conditions.
- Songbird and bat boxes shall be provided.
- Plant selection shall consider those species that are native to the San Antonio River and provide significant food and cover sources for resident wildlife and wildlife using the San Antonio area as a migratory corridor.

Providing pool, riffle, and run sequences within the channel will enhance aquatic habitat. Riffles offer macroinvertebrate habitat that provides a food source for fish, while pools provide escape and cover habitat for fish. Creating backwater wetlands within the floodway will also enhance aquatic and wildlife habitat. These wetlands offer habitat diversity, rearing habitat, and are anticipated to be used by amphibians, waterfowl, and wading birds. Furthermore, designs will incorporate provisions for wildlife habitat such as songbird boxes and bat boxes. Designs will also provide consideration for birds, butterflies, and waterfowl that use the San Antonio area as a migratory corridor. Such considerations will include selecting vegetation that provides a food source for birds, butterflies, and other wildlife. Vegetative selection will also consider providing food sources for resident wildlife.

Floodplain and Riparian Vegetation

- Revegetation efforts shall consist of seeding herbaceous species and planting woody species. The pilot channel banks and floodway side slopes shall be seeded with a mixture of native grasses and forbs. Seeding will be designed to produce varied stands of herbaceous plants.
- Prioritize the use of plant species native to the south-central Texas region to the extent practicable.

- Species selection and planting locations shall be coordinated with anticipated soil moisture conditions, soil textures, inundation regime, and operation and maintenance requirements.
- Plant species shall be selected to produce rapid establishment and maturation, achieve vegetative cover, provide stability to the pilot channel banks and floodway side slopes, and to enhance aesthetic and habitat value in accordance with preceding design criteria.
- Seeding and planting of woody vegetation shall take place during the seasons that will best promote survival.
- Where possible, plantings shall be situated to provide overhanging cover to the base flow channel, particularly in pool sections.
- Vegetation selection shall consider providing food sources for birds, butterflies, and other wildlife.

Designs should include a variety of native and naturalized ornamental plants that provide a pleasing visual character and wildlife habitat, while providing consideration for hydraulic roughness and its influence on flood conveyance. Plant species will be selected with consideration of wildlife habitat, light, water, and hydro period requirements. Plant species selection should also consider competition with noxious weeds and invasive species.

Construction, Maintenance and Monitoring

The contract documents for the project will provide requirements that minimize risk of life, property, and environmental damage. For example, dewatering constructed river reaches should rely on gravity systems where practicable since reliance on pumps may be subject to mechanical failure and greater property loss if flash floods occur. Any trapped fish from dewatering operations should be safely removed, transported, and released within the channel outside the project limits. Construction equipment (excluding dewatering equipment), materials, and supplies should be removed from the floodway at the end of each workday. Also, construction equipment shall include spill kits to contain any hydraulic fluid leaks that occur during construction. Utility locates must be required as well as contract language that requires replacement of damaged utilities and property.

Designs shall incorporate considerations to reduce maintenance including trash removal, vegetative maintenance requirements, and reduced deposition and erosion potential. Reductions may include trash racks for stormwater flows, utilizing native vegetation and providing considerations for sediment transport continuity. Also, long-term vegetative maintenance requirements may be reduced if noxious weeds and invasive vegetation is removed before these plants are allowed the opportunity to develop seeds.

A monitoring period of 5 years should occur after project work. Monitoring should include established photo points with photos taken before, after, and during construction. Annual post-construction monitoring should include photos taken at photo points. Project as-built drawings should include surveyed cross sections at the Federal Emergency Management Agency (FEMA) HEC-RAS model section locations. Annual post-construction monitoring should include surveyed cross sections at selected FEMA, HEC-RAS model sections. Furthermore, vegetative monitoring should not just include percent survival. It should also incorporate measures to account for natural colonization of native vegetation (volunteer plants) and monitor the aerial extent of noxious weeds or invasive species.

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Appendix A, Geomorphic Maps

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Sheet 23 Placeholder

Sheet 24 Placeholder

Appendix B, Sediment Sample Location Photographs

Sample #1 - Right Bank Bar on Olmos Creek Downstream of McCollough

Sample # 1 - Olmos Creek Deposited Material Downstream of McCollough

Sample #2 – Olmos Creek Left Bank Bar Downstream of Tributary Confluence

Sample # 3 Olmos Creek Upstream of Dreamland

Sample # 4 – Olmos Creek Upstream of Huebner

Sample #5 – San Antonio River Bar Sample at STA 2476+00 near 200 Patterson Condominiums

Sample #6 – Submerged Bar at STA 2455+00 Downstream of Hildebrand at Brackenridge Park

Sample #7 – Left Bar at STA 2428+00 downstream of Iron Bridge in Brackenridge Park.

Sample #8 – Riffle at STA 2416+00 in Brackenridge Park Downstream of Pedestrian Bridge

Sample #9 – Riffle at 2406+00 near Vertical Cut Bank

Sample #10 – Riffle at STA 2381+00 near Right Bank Sediment Source

Sample #11 – Mid-channel bar at STA 2359+00 Downstream of Grade Control Structure

Sample #12 – Bar Deposit at STA 2331+00 Downstream of Grayson St.

Sample #13 – Right Bank Bar at STA 2320+00 Behind Pearl Brewery

Sample #14 – Gravel Deposit at STA 2166+00 Downstream of Gate #6 and S. Alamo (Blue Star).

Sample #15 – Bank Material at STA 2136+00 in Eagleland Reach

Sample #16 – Overbank Sample at STA 2116+50

Sample #18 – Below Checkdam at STA 2113+00

Sample #19 – Tributary Sample at STA 2082+50

Sample # 19 & 20 – Point Bar at San Pedro Creek Confluence

Sample #21 - Downstrteam of Tributary Confluence at STA 2041+00

Sample #22 – Right Bank Point Bar Downstream of Mission Road Bridge at STA 2004+00

Sample #23 – Left Bank Point Bar Upstream of Asylum Creek at STA 1877+00

Sample #24 – Bar Deposit in Chute upstream of Ashley Road at STA 1802+50

Sample #25 – Gravel Bar on Right Bank at STA 1743+00

Sample #26 – Bar Deposit Downstream of I-410 at STA 1729+00

Sample #27 – Left Bank Point Bar at STA 1990+00

Sample #28 - Submerged Channel Bed Sample at STA 1956+50

Sample #29 - Mid Channel Bar at STA 1929+ 00

Sample #30 – Mid Channel Bar Upstream of Military Drive at STA 1883+00

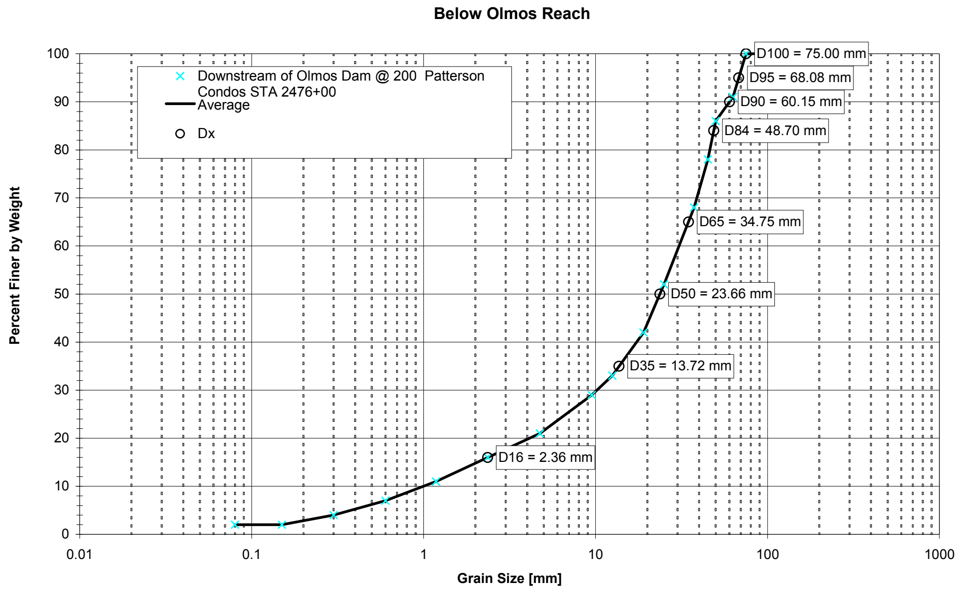
Sample #31 - Under I-35 Bridge at STA 2372+00

Sample #32 - Catalpa Pershing Supply Ditch in Channel Downstream of Headcut

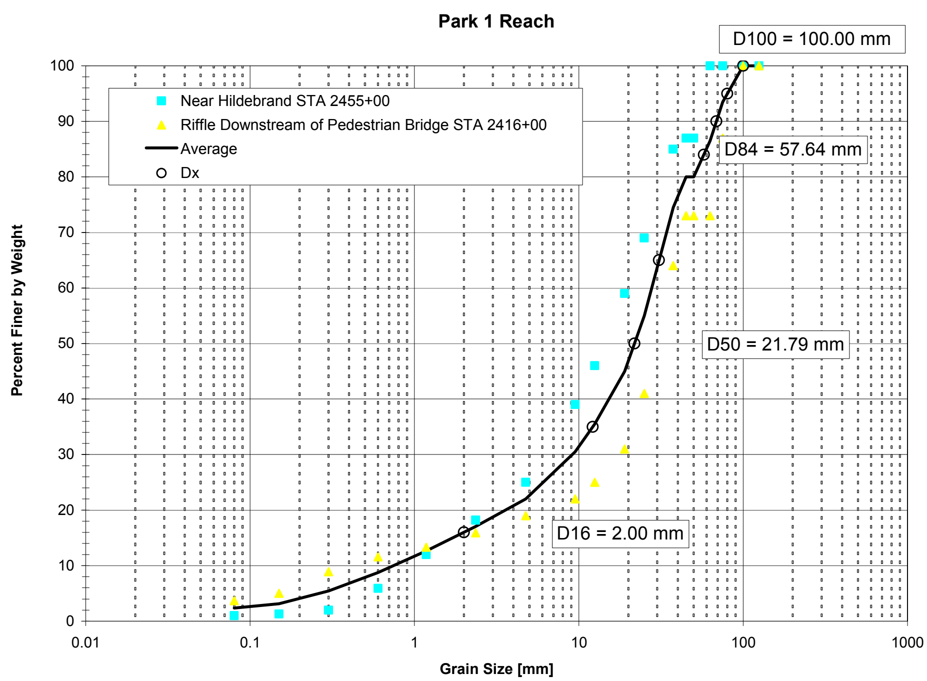
Sample #33 Left Bar downstream of Project Reach at STA 1682+00

Sample #34 – Near Minita Creek at STA 1653+ 00

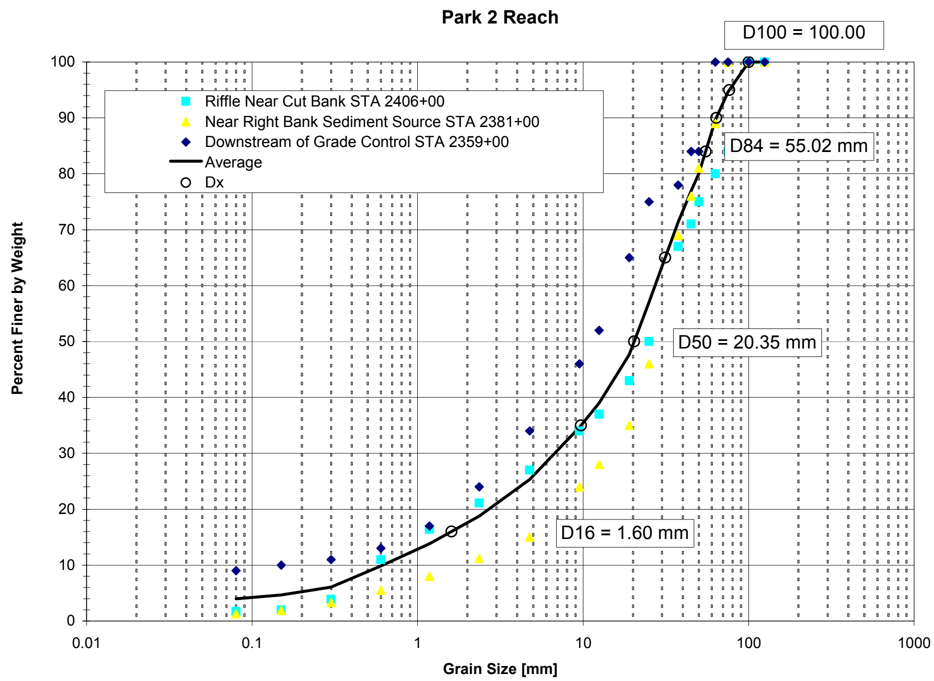
Appendix C, Sediment Sample Gradations



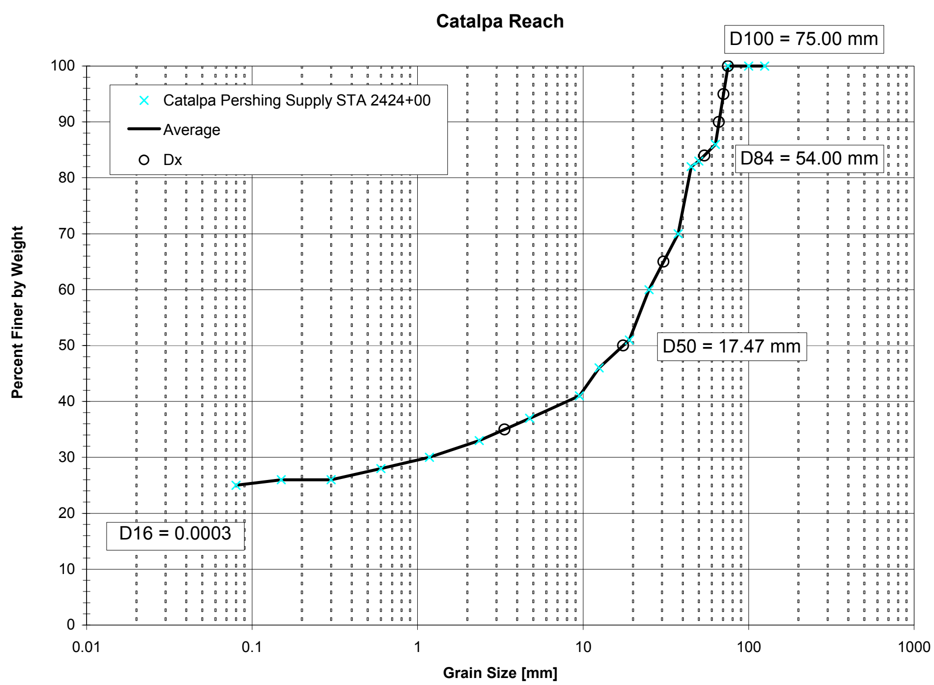
Sediment Sample Gradation Curve, Below Olmos Subreach



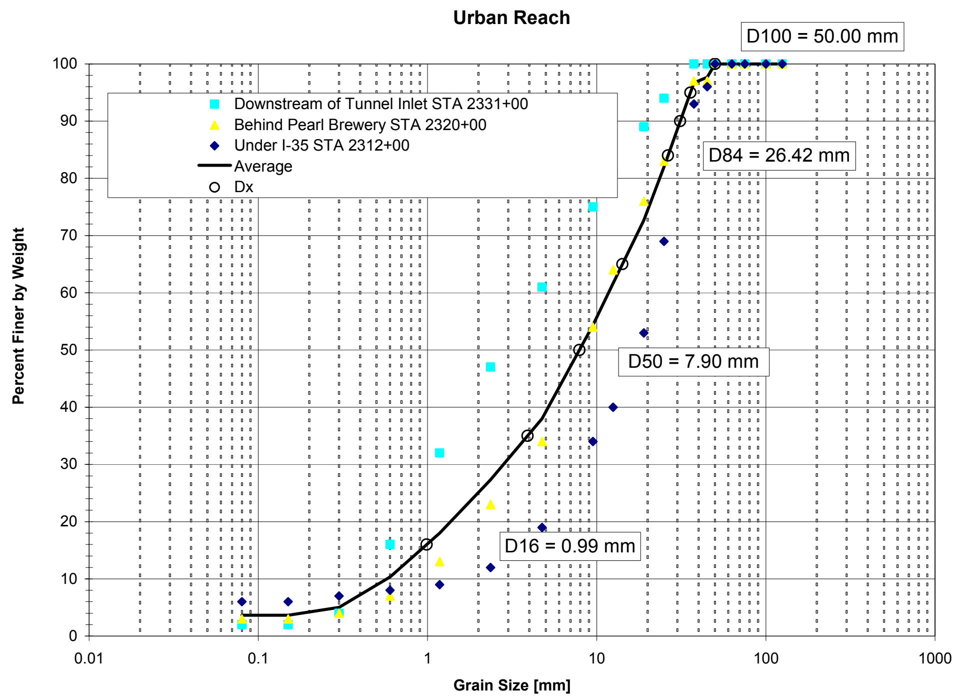
Sediment Sample Gradation Curve, Park 1 Subreach



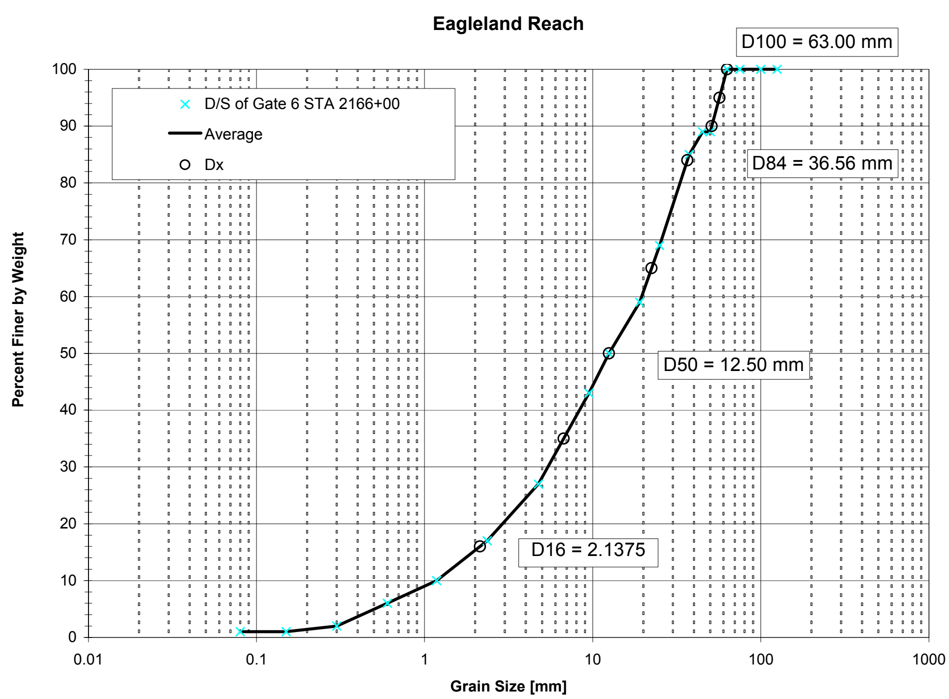
Sediment Sample Gradation Curve, Park 2 Subreach



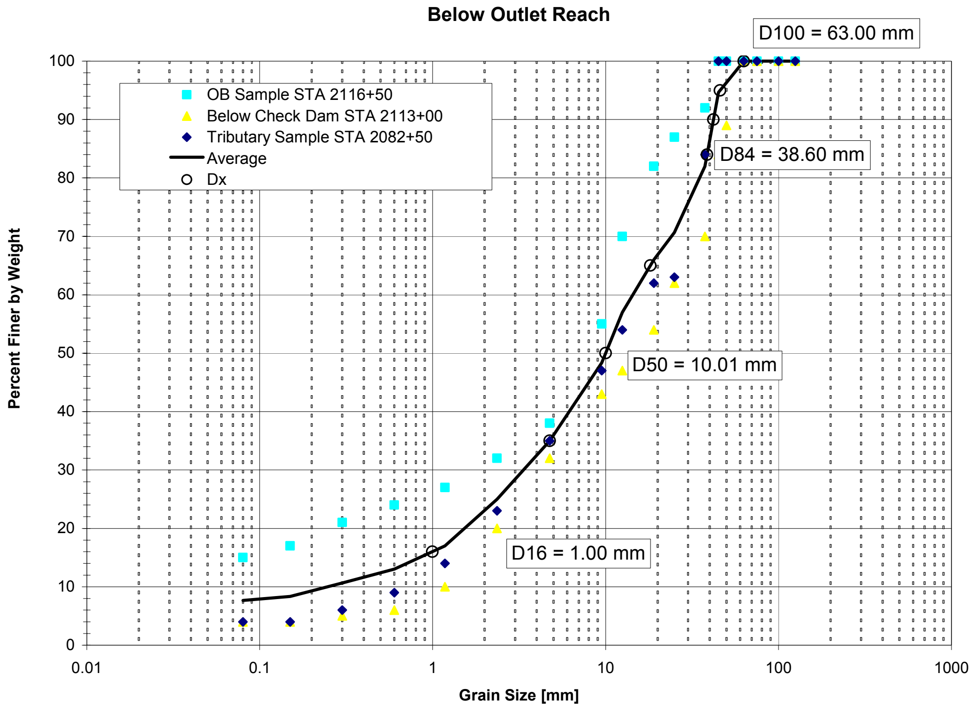
Sediment Sample Gradation Curve, Catalpa Ditch



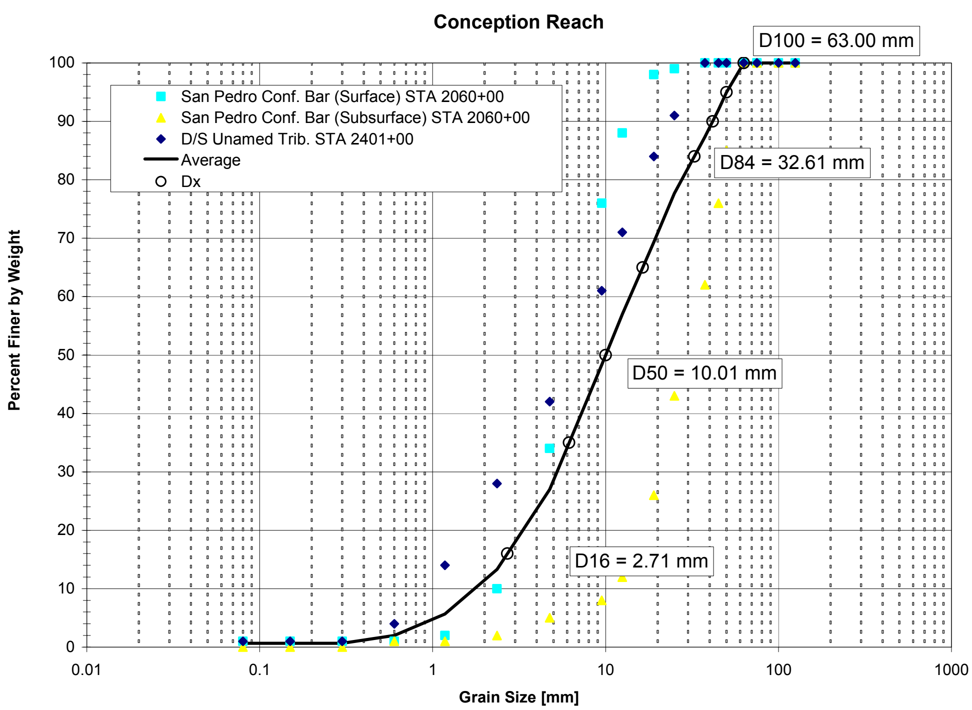
Sediment Sample Gradation Curve, Urban Subreach



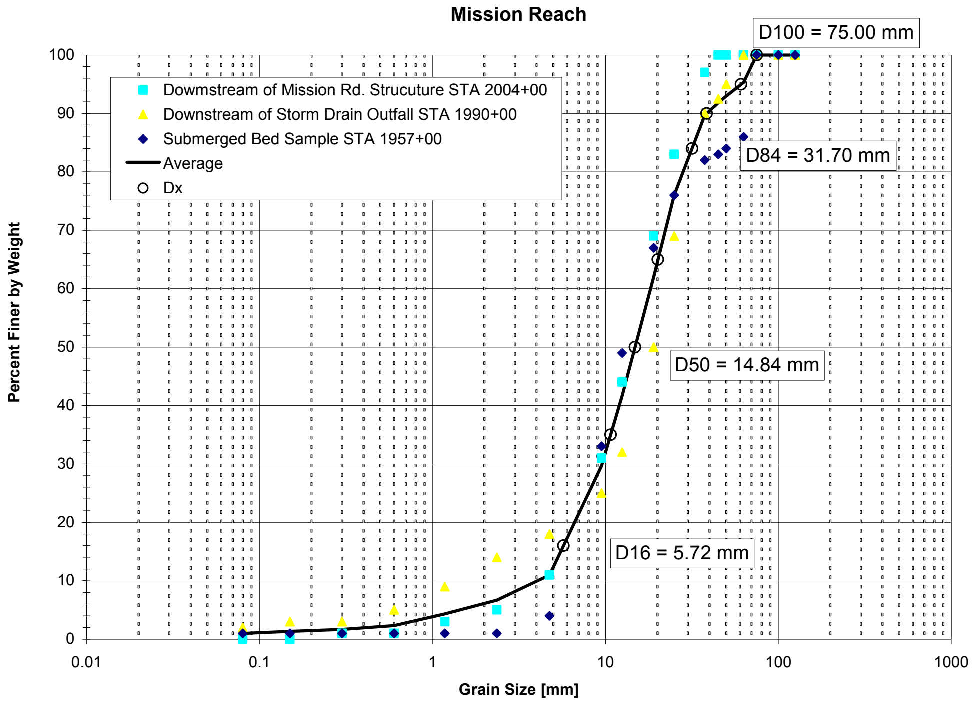
Sediment Sample Gradation Curve, Eagleland Subreach



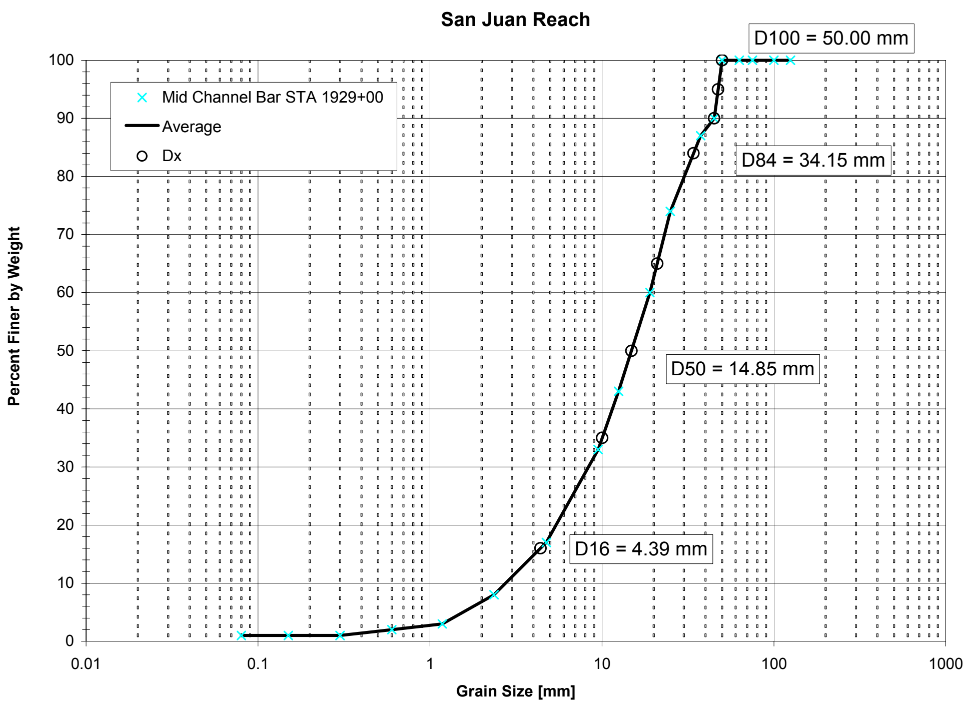
Sediment Sample Gradation Curve, Below Outlet Subreach



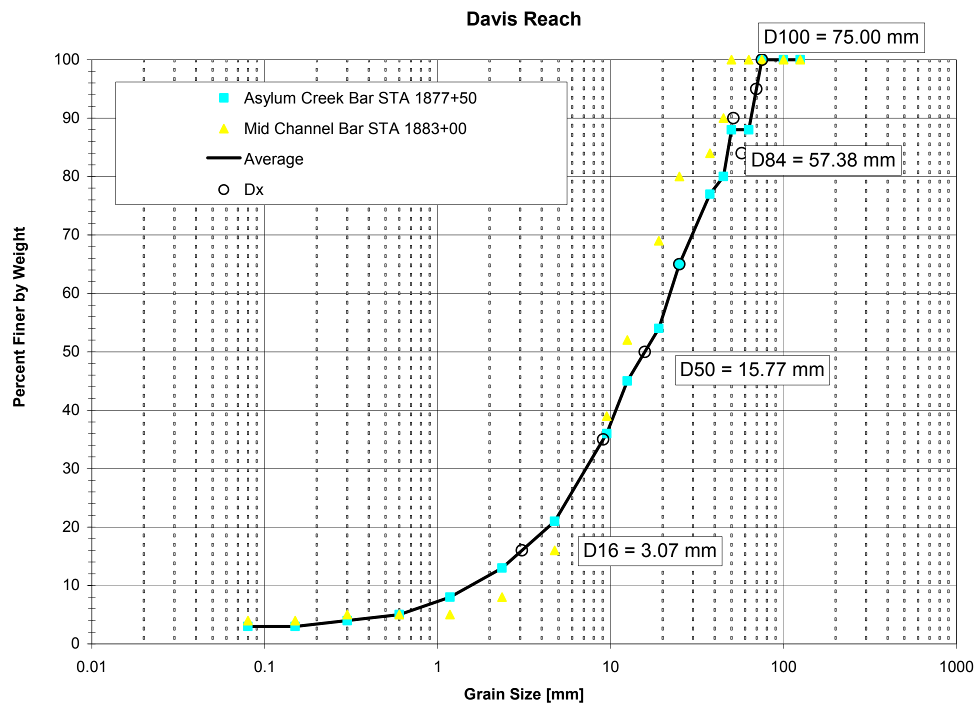
Sediment Sample Gradation Curve, Conception Subreach



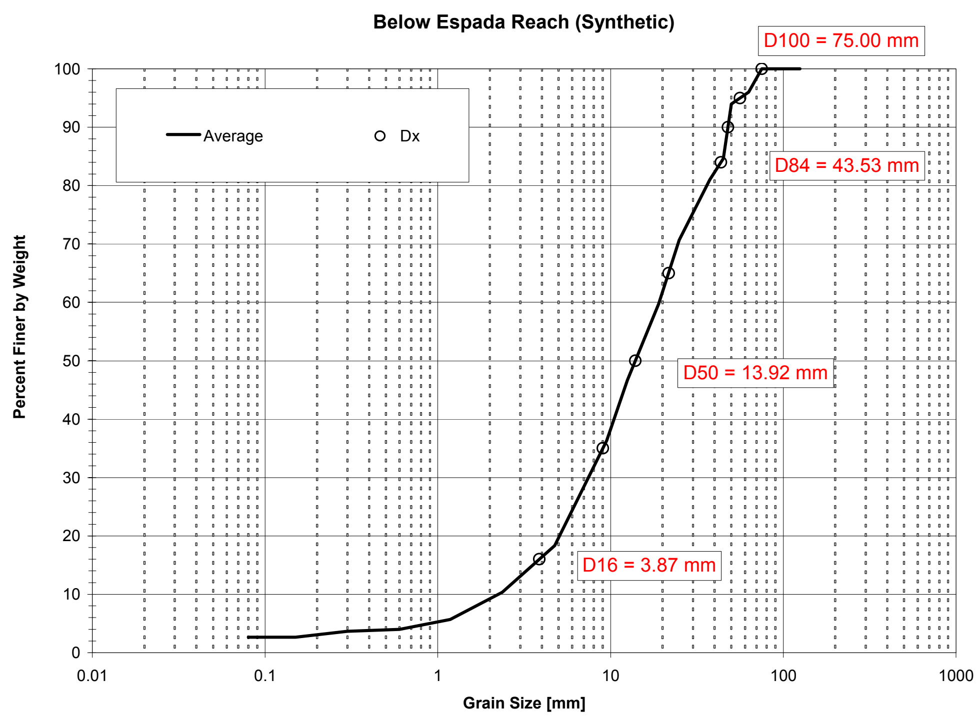
Sediment Sample Gradation Curve, Mission Subreach



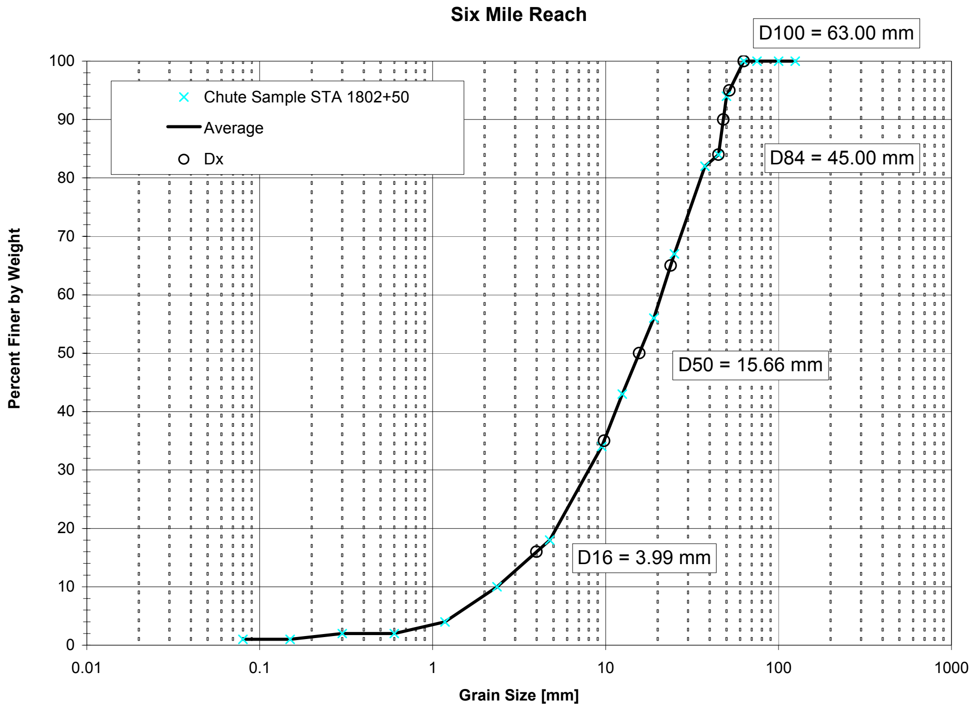
Sediment Sample Gradation Curve, San Juan Subreach



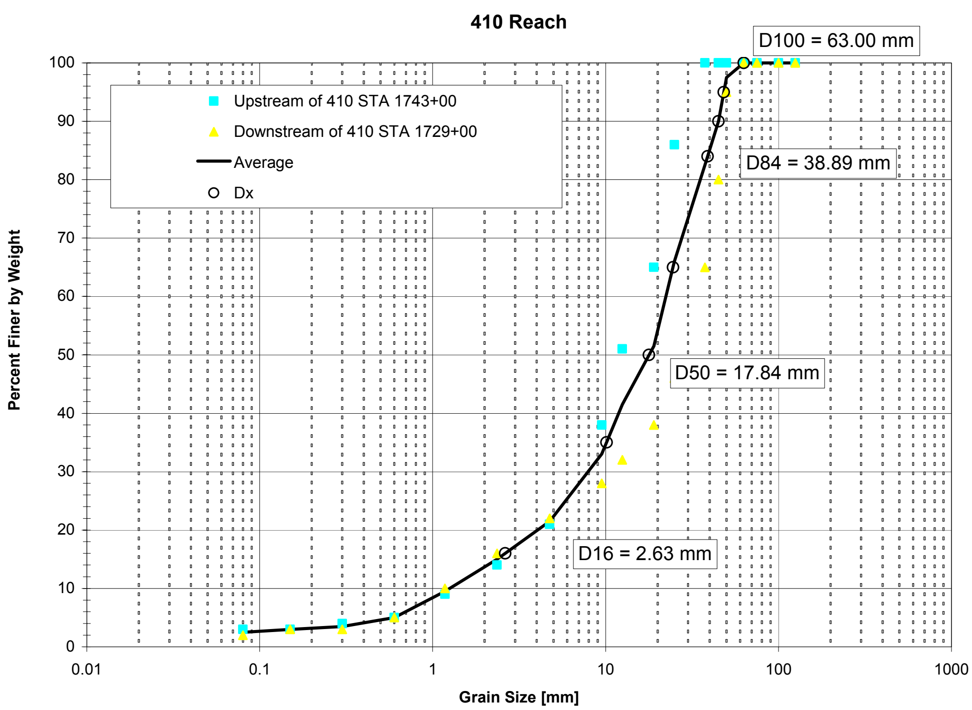
Sediment Sample Gradation Curve, Davis Subreach



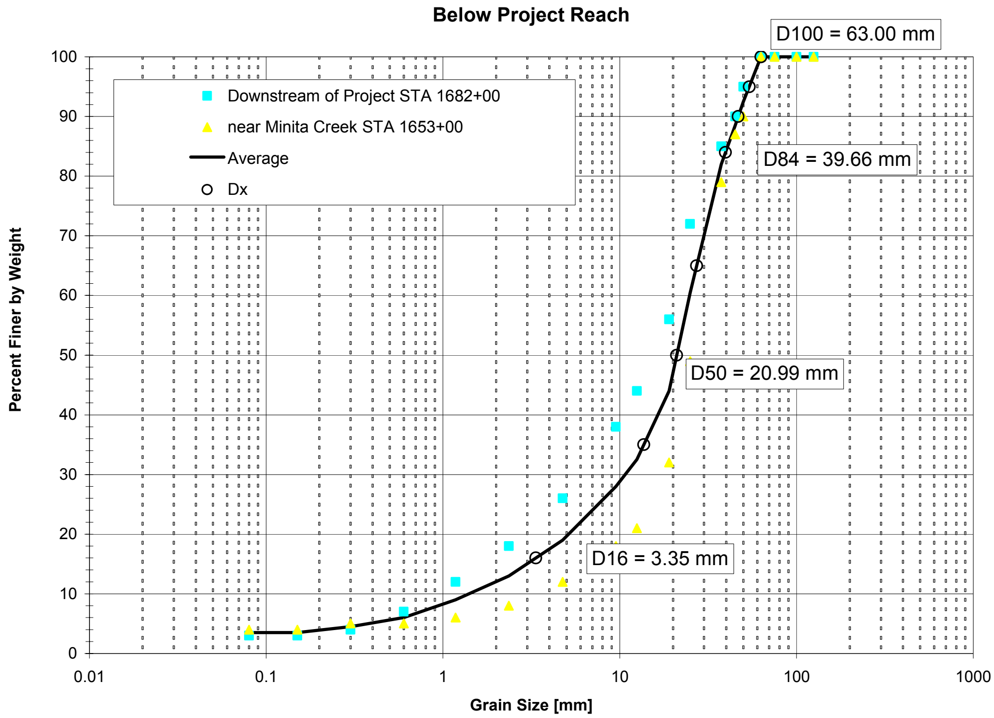
Sediment Sample Gradation Curve, Below Espada Subreach



Sediment Sample Gradation Curve, Six Mile Subreach



Sediment Sample Gradation Curve, 410 Subreach



Sediment Sample Gradation Curve, Below Project Subreach