Appendix C

Fluvial Geomorphology Study Report

ATTACHMENT 8
FLUVIAL GEOMORPHOLOGY STUDY REPORT

PREPARED BY
MUSSETTER ENGINEERING, INC.

UPPER TRINITY
REGIONAL WATER DISTRICT

Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Lake Ralph Hall Project

Submitted to: **Chiang Patel & Yerby, Inc.**
1820 Regal Row, Suite 200
Dallas, Texas 75235

Submitted by: **Mussetter Engineering, Inc.**
1730 S. College Avenue, Suite 100
Fort Collins, Colorado 80525

October 23, 2006

EXECUTIVE SUMMARY

ES.1. INTRODUCTION

The Upper Trinity Regional Water District (UTRWD) is proposing to build a 160,235-acre-foot (ac-ft) water supply reservoir, Lake Ralph Hall, on the North Sulphur River (NSR) about 3.5 miles north of Ladonia in Fannin County, Texas (Figure 1.1). Fannin County is located within the Texas Blackland Prairie physiographic area (NRCS, 2001). The NSR and its tributaries, within the boundaries of the proposed reservoir, as well as upstream and downstream, are deeply incised and eroding. Current conditions are the result of channelization and straightening of the sinuous, meandering river and the lower reaches of its tributaries to prevent frequent overbank flooding on the NSR floodplain in the late 1920s (Williams, 1928; Avery, 1974). Prior to channelization, the NSR was a sinuous (1.7) meandering stream with a slope of about 4.3 ft/mi. In the vicinity of the proposed dam site, the natural channel was about 48 feet wide and 6 feet deep and had a hydraulic capacity of between 700 and 1,000 cfs. The channelized and straightened channel had a top width of 16 to 30 feet, and a depth of 9 to 12 feet with a slope of 6.5 ft/mile (Avery, 1974; Chiang, Patel & Yerby, Inc., 2004; AR Consultants, Inc., 2005) and a hydraulic capacity of about 700 cfs. Currently, at the proposed dam site the NSR is 300 feet wide and about 40 feet deep, the bed and lower portions of the banks of the channel are composed of erodible shale (Ozan Formation), and the channel contains flows well in excess of the 100-year flood peak (38,000 cfs). Between the late 1920s and the present, about 28M tons of sediment have been eroded from the mainstem NSR and its tributaries upstream of the proposed dam site. At the time of the channelization in the late 1920's about 75 percent of the watershed was under cultivation (Williams, 1928), and consequently soil erosion rates were probably very high (up to 16 t/ac/yr) (Baird, 1948, 1964), which may have contributed to loss of channel capacity and increased frequency of overbank flooding that occasioned the channelization. Currently about 21 percent of the watershed that contributes water and sediment to the proposed reservoir is cultivated (Texas State Soil and Water Conservation Board, 1997).

ES.2. OBJECTIVES

The primary objectives of this geomorphic and sedimentation study of the Lake Ralph Hall project, that was conducted by Mussetter Engineering, Inc. (MEI) for the UTRWD under subcontract to Chiang Patel & Yerby, Inc. (CP&Y), were:

1. Quantification of the sediment delivery to the reservoir site for the 50-year project life under pre- and post-project conditions,
2. Evaluation of the downstream effects of the dam on channel conditions and flow capacity, and
3. Assessment of the potential for reducing or managing the upstream sediment supply to the reservoir.
4. Assessment of future conditions in the North Sulphur River and tributaries upstream of the dam site in the absence of the project.

ES.3. METHODOLOGY

Future loss of reservoir capacity due to sedimentation is the primary issue of concern for this investigation of the Lake Ralph Hall project and, therefore, estimates of sediment yield from the 100-square-mile watershed upstream of the proposed dam were required. Potential sources of sediment identified included channel erosion in the mainstem NSR and the incised tributaries (bed and banks) and watershed erosion (sheet, rill, ephemeral gully). Hydrologic analyses of the gage record at the USGS North Sulphur River near Cooper gage (USGS Gage No. 07343000) and HEC-1 models were used to estimate peak flow frequencies (Figures 3.7 and 3.9), mean daily durations and flow volumes (Figure 3.10) for the dam site and the tributaries. One–dimensional HEC-RAS models were developed for the mainstem and for the major tributaries based on the 2-foot contour interval Digital Terrain Model (DTM) provided by CP&Y, and the models were calibrated to field-measured high-water marks for the 2002 (10-year event) and 2003 (25-year event) peak flows. Reach-averaged hydraulic output (effective width, hydraulic depth and average velocity) from the HEC-RAS models was used to compute sediment transport.

ES.4. CHANNEL MORPHOLOGY AND EVOLUTION

Field observations of the NSR and its tributaries indicated that in common with other incised streams, the morphological adjustments of the river and the larger tributaries can be described by a geomorphic model of incised channel evolution (Schumm et al., 1984; Simon and Hupp, 1986; Simon, 1989). A channel evolution model (NSRCEM) was developed for the NSR and its tributaries (Figure 2.19). The model varies substantially from those developed for alluvial streams (Figure 2.4) in that it does not predict an equilibrium end point because both vertical and lateral erosion of the exposed shale outcrop is controlled by wetting and drying cycles (Tinkler and Parish, 1989; Allen et al., 2002) and not hydraulic processes. There is little doubt that following channelization in the late 1920s the NSR incised and widened (Avery, 1974) and followed the typical channel evolution sequence while the channel boundary materials were composed of alluvium (Types I through V). However, exposure of the shale added a significant complicating factor to the evolution of the channel. Based on the flow record at the USGS gage on the NSR near Cooper, there are an average of six wetting and drying cycles per year (Figure 2.3). Flow events in the channel remove the weathering products and re-initiate vertical and lateral erosion into the shale. As a rule, lateral erosion rates exceed vertical erosion rates in bedrock and result in the formation of gravel-covered strath surfaces that become terraces when vertical erosion of the bed occurs (Leopold et al., 1964; Schumm, 1977) (Type VI). Deep-seated slump failures of the overlying alluvium bury the strath surfaces (Type VII) and prevent lateral erosion of the shale. Resulting channel narrowing may actually accelerate erosion of the shale exposed in the bed, which in turn leads to undercutting of the erosion-resistant, root-reinforced alluvium, thereby leading to re-exposure of the shale in the toe of the banks and ongoing lateral retreat of the shale (Type VIII). It is likely that over time the incision into the shale will induce further mass failure of the alluvial valley fill and a Type VII condition will be reestablished at a lower bed elevation and there will be additional channel widening. The NSRCEM applies equally to the larger tributaries that have eroded into the shale.

Between the FM 904 bridge and the upstream end of the watershed, the NSR was subdivided into 10 subreaches (Table 2.2). Based on the NSRCEM, Subreaches 1 through 3 were classified as Type VI, Subreach 4 was classified as Type VII, Subreaches 5 through 8 were classified as Type VIII, and Subreaches 9 and 10 were classified as Type VII. Similar sequences are present in the larger tributaries. Incision in the headwaters of the NSR and the major north-side tributaries has been limited by outcrop of reasonably erosion resistant Roxton/Gober Chalk (Figure 2.2). Currently, the incised channel has the ability to convey in excess of the 100-year flood in-bank (Figures 2.5 through 2.18), the bed of the river is composed of shale, and therefore, the current supply of sediment to the channel is far less than the transport capacity.

ES.5. SEDIMENT TRANSPORT AND YIELD

The primary sources of bed-material size sediment are the exposed shale outcrops in the bed and banks of the river and the tributaries. Based on studies of the erosion of the shale (Allen et al., 2002; Crawford, in prep) and the results of analysis of stage-discharge rating curves for the Cooper gage (Figure 2.36) and comparative bridge profiles (Figure 2.34), erosion rates for shale exposed in the bed and banks of the channel are on the order of 2 to 4 in./yr, respectively. Transport and slaking of the shale clasts results in a temporal and spatial transformation of initially gravel-sized material, which is transported as bed material, to silt-clay-sized wash load (Figure 2.40) that has little or no morphological significance. At the upstream end of the NSR about 80 percent of the bed material that forms a thin veneer over in-situ shale slakes to silt-clay-sized material, whereas in the downstream reaches only about 10 percent of the bed-material slakes (Figure 2.42). Based on a supply-limited model of sediment-transport capacity, calibrated to the area of the bed covered by depositional bars, and incorporating the transformation of the bed material to wash load, the best estimate of sediment yield from channel sources to the dam site under pre-project conditions is 93,100 t/yr. Based on a somewhat unrealistic transport capacity-limited model, the worst-case estimate of sediment yield from channel sources to the dam site is 292,000 t/yr. With the dam in place, the best-case estimate of annual sediment yield from all channel sources to the reservoir is 35,600 tons, and the worst–case estimate is 59,600 tons. The reduced amount of sediment is because the reservoir inundates a high proportion of the contributing channel area and eliminates it as a contributing source.

Estimates of the sheet-and-rill erosion on the watershed were developed with the Modified Universal Soil Equation (MUSLE) with appropriate parameters based on the subbasin topography and soil types (clays and loams) determined from the Soil Survey of Fannin County (NRCS, 2001). Application of the MUSLE with the appropriate parameters underestimated reported gross sheet-and-rill erosion rates on the Blackland Prairie soils (2 t/ac/yr), and therefore the alpha coefficient for the MUSLE was increased by a factor of 2.7. Ephemeral gully erosion for the cropland portions of the watershed was estimated to be equivalent to the sheet-and-rill gross erosion rates on the basis of the soil erosion literature (Laflen et al., 1986). Sediment delivery ratios (SDR) for the sheet-and-rill erosion were estimated with Equation 5.4 (Renfro, 1975) that yields the highest SDR values. For the ephemeral gully erosion the SDR was estimated to be 0.67 (Alan Plummer Associates, 2005). Worst-case watershed sediment yields were estimated with an assumption of 100-percent cropping in the watershed with a gross erosion rate of 3.74 t/ac/yr (Richardson, 1993). The best conservative estimate of the current annual watershed sediment yield at the dam site is about 81,000 t/yr which reduces to about 69,000 t/yr with the reservoir in place. Under worst-case conditions the existing annual watershed sediment yield to the dam site is about 147,000 t/yr, and this reduces to about 90,000 t/yr with the reservoir in place. When placed in the context of reported sediment yields in the Blackland Prairie (Table 5.4), these estimates are very conservative especially because a 100 percent trap efficiency has been assumed for the reservoir.

Although estimated sediment yields to the Lake Ralph Hall reservoir are relatively low, the sediment yields could be further reduced by implementation of soil conservation measures on the watershed and by reducing the exposure of shale in the mainstem of the NSR and the tributaries between the upstream end of the conservation pool and the Roxton/Gober Chalk outcrop (Figure 2.2).

ES.6. DOWNSTREAM IMPACTS

The potential downstream effects of the Lake Ralph Hall project on channel conditions and channel capacity are a concern. Potential problems could include sediment accumulation in the bed of the channel since operation of the reservoir will affect the magnitude and frequency of flows in the downstream channel, but will not affect sediment supply from the watershed, tributary and channel sources below the dam. Field and helicopter reconnaissance of the NSR from its confluence with the South Sulphur River to the headwaters indicates that the channel of the NSR is deeply incised for its entire length, and that the bed of the channel is composed of shale bedrock. Since the rates of bedrock erosion are controlled by the number of wetting and drying cycles (Allen et al., 2002), and not by hydraulic processes, the upstream dam is unlikely to have any effects on bedrock erosion rates. On an average annual basis, the shale will continue to erode vertically at a rate of about 2 inches per year and laterally at a rate of about 4 inches per year. Locally, near the mouths of some of the large tributaries downstream of the dam site (e.g., Hickory and Big Sandy Creeks) there are alternate bars in the bed of the channel, but these reflect local sediment supply and do not extend downstream for any distance. Under existing conditions, the best estimate of the annual total sediment yield to the dam site is about 174,000 tons (Figure 5.8), but only about 25 percent is composed of bed material, the remainder being wash load. Therefore, construction of the dam will reduce the morphologically-significant sediment yield to the channel downstream of the dam by about 25 percent, which will have an insignificant effect on the channel morphology in this sediment supply-limited system.

Based on the geologic map (Figure 2.2), and field observations, the characteristics of the shale exposed in the mainstem NSR and tributaries downstream of the dam site are similar to those upstream of the site, and therefore, it can be assumed that the sediment characteristics are also similar. This being the case, the bulk of the sediments being delivered to the NSR by the tributaries downstream of the dam will be composed of shale clasts that break down into wash-load-sized materials as they are exposed to transport and weathering processes (slaking). Furthermore, the NSR is a supply-limited system that has the capacity to transport considerably more bed material than is currently being supplied to the channel. Consequently, it is unlikely that significant amounts of sediment will accumulate in the bed of the river downstream of the dam. If sediment accumulation does occur it is highly unlikely that there will be significant loss of channel capacity. Even with the loss of channel capacity, flows far greater than the 100-year flood peak can be conveyed in-bank.

ES.7. CONCLUSIONS

The geomorphic, hydrologic, hydraulic and sediment-transport studies conducted for this investigation of the Lake Ralph Hall project allow the following to be concluded:

1. Channelization-induced degradation and widening of the NSR and its principal tributaries upstream of the dam site has resulted in the erosion of about 28M tons of sediment since the late 1920s. Current channel erosion rates are controlled by slaking rates of the exposed shale and not by hydraulic processes and are, therefore, less than historic rates.
2. The conservative estimate of total annual sediment yield to the dam site under pre-project conditions is 86 ac-ft (174,000 tons). With the reservoir in place, the contributing watershed area is reduced, as is the length of channel that is supplying sediment, and therefore, the total annual sediment yield to the reservoir reduces to 51.4 ac-ft (104,000 tons). Therefore, estimated sediment delivery to the 160,235-ac-ft reservoir over a 50-

year period, assuming 100-percent trap efficiency, is about 2,570 ac-ft, which represents a loss of reservoir storage capacity of approximately 1.6 percent.

3. Under the assumptions of the worst-case watershed (100 percent of the watershed under cultivation with no soil conservation measures) and channel sediment yields (transport capacity limited assumption) the estimated total annual sediment yield to the dam site is 217 ac-ft (439,000 tons). With the reservoir in place, the worst-case reduces to an annual sediment yield to the reservoir of 74 ac-ft (150,000 tons). Under these circumstances, estimated sediment delivery to the 160,235 ac-ft reservoir over a 50-year period, assuming 100-percent trap efficiency, is about 3,700 ac-ft, which represents a loss of reservoir storage capacity of approximately 2.3 percent.

4. In the absence of the Lake Ralph Hall project there will be continued erosion of the NSR and its tributaries. On average, where shale is exposed in the bed and banks of the channels, the channel depth will increase by about 8 feet and the channel bottom widths will increase by about 16 feet over a 50-year period. Increased channel depths are also likely to cause further mass failure of the alluvial portions of the banks, thereby increasing channel top widths, as well.

5. No adverse downstream impacts on channel morphology or capacity are expected as a result of sediment trapping in the reservoir, or operation of the reservoir.

6. Watershed sediment yields could be reduced by implementation of best soil conservation management practices, reduction in the area under cultivation and re-establishment of riparian buffer areas along the channel margins where they have been cleared.

7. Channel sediment yields between the elevation of the top of the conservation pool and the downstream extent of the Roxton/Gober Chalk could be reduced by construction of in-channel structures that pond water and prevent weathering of the shale outcrop. Given the existing hydraulic capacity of the channels there is little likelihood that the in-channel structures would cause out-of-bank flooding.

Table of Contents

List of Figures

List of Tables

1. INTRODUCTION

1.1. Background

The Upper Trinity Regional Water District (UTRWD) is proposing to build a 160,235-acre-foot (ac-ft) water supply reservoir, Lake Ralph Hall, on the North Sulphur River (NSR) about 3.5 miles north of Ladonia in Fannin County, Texas (**Figure 1.1**). The NSR and its tributaries, up- and downstream of the proposed reservoir, are deeply incised and eroding. Current conditions are the result of channelization and straightening of the very sinuous (1.7) meandering river and the lower reaches of its tributaries to prevent frequent overbank flooding on the NSR floodplain in the late 1920s (Avery, 1974). Estimates of the initial configuration of the channelized mainstem of the NSR vary from a top width of 16 to 30 feet, and a depth of 9 to 12 feet with a slope of 6.5 ft/mile (Avery, 1974; Chiang, Patel & Yerby, Inc., 2004; AR Consultants, Inc., 2005). It is of interest to note that Mr. Z.F Williams, the State Reclamation Engineer, predicted that the channelization would *cause high velocities and subsequent erosion, and will result in a substantial enlargement to the section as cut* (Williams, 1928). As predicted, the NSR has incised through the alluvial valley fill into the underlying shale bedrock, and currently has a depth and width at the dam site of 40 and 300 feet, respectively. The channel incision and widening caused the loss of agricultural lands, damages to bridges and other utilities, lowering of the water table, loss of riparian habitat and channel biodiversity. Additionally, this has resulted in baselevel lowering for tributaries that were not channelized, that have in turn incised and widened.

Based on measurements of remnants of the natural channel of NSR on the now abandoned floodplain, the width was about 48 feet, the depth was about 6 feet and the slope was about 3.8 ft/mile. Normal-depth calculations based on the geometry of the remnant channel segments indicate that the natural channel of the NSR had a flow capacity of between 700 and 1,000 cfs in the vicinity of the dam site, the channelized river had a flow capacity of about 700 cfs, and the current channel has a capacity in excess of the 100-year flood peak (~38,000 cfs; RJ Brandes Co., 2004). At the time of channelization of NSR, about 75 percent of the watershed was under cultivation (Williams, 1928). Based on Baird's (1948, 1964) estimates of annual gross soil erosion without any conservation measures for the Blackland Prairie Land Resource Area (14.3 to 16.6 t/ac), the annual sediment load at the dam site (100-square-mile drainage area) could have been as high as 1 million tons, which is about 10 times higher than the amount that would be predicted by more recent reservoir sedimentation surveys in the Blackland Prairie area (Alan Plummer and Associates, 2005). As occurred in many parts of the U.S., the high sediment loading from the watershed may have contributed to loss of channel capacity and the frequent (multiple times per year) overbank flooding that occasioned the channelization of NSR (Happ et al., 1940; Trimble, 1974; Schumm et al., 1984; Harvey and Watson, 1986).

Future loss of reservoir capacity due to sedimentation is an issue of concern for the Lake Ralph Hall project and, therefore, estimates of sediment yield from the 100-square-mile watershed upstream of the proposed dam are required. Potential sources of sediment include channel erosion (bed and banks) and watershed erosion (sheet, rill, ephemeral gully, gully). Incised channels generally follow a temporally and spatially based evolutionary sequence from instability back to some form of equilibrium between the supplied water and sediment load and the channel morphology that has been described by a geomorphic model, the Incised Channel Evolution Model (ICEM) (Schumm et al., 1984; Harvey and Watson, 1986; Simon and Hupp, 1986). During the course of the evolutionary sequence, sediment loads derived from erosion of the incised and widening channel can be extremely high (10^3 to 10^6 t/yr), but tend to decease

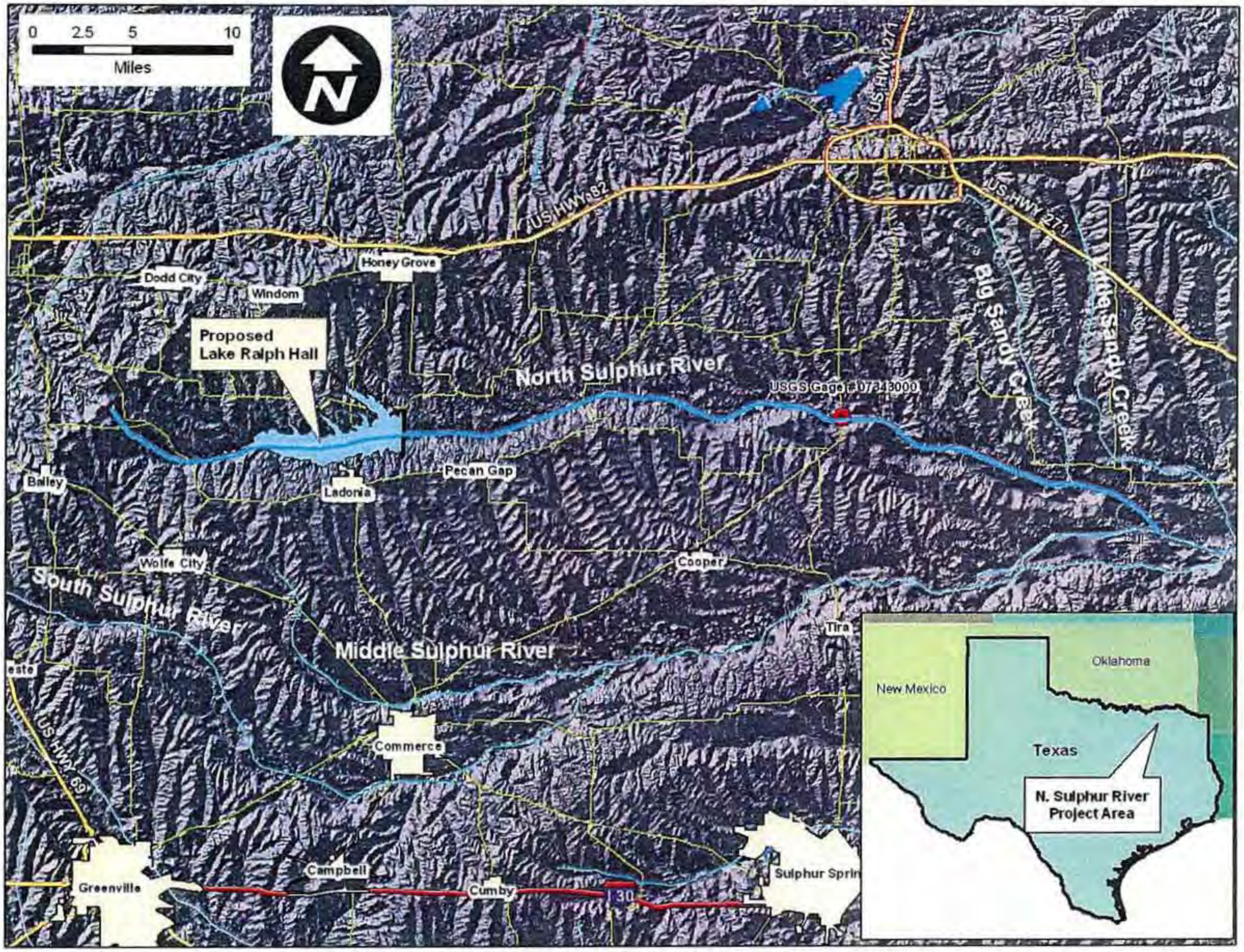


Figure 1.1. Map showing the location of the proposed Lake Ralph Hall on the NSR in Fannin County, Texas.

through time as a new state of equilibrium is approached (Harvey and Watson, 1986; Watson et al., 1986; Watson et al., 1988; Simon and Darby, 1999; Prosser et al., 2000). In the context of the NSR, the current sediment yield from the incised mainstem channel and the tributaries will depend on where these channels are in the evolutionary sequence. Sediment yield from the watershed is dependant on the land use within the watershed. Although approximately 75 percent of the watershed area was under cultivation for primarily row crops in the 1920s and 1930s, the current area in cropland is about 26 percent (Texas State Soil and Water Conservation Board, 1997).

1.2. Project Objectives

The primary objectives of this geomorphic and sedimentation study of the Lake Ralph Hall project conducted by Mussetter Engineering, Inc. (MEI) for Chiang, Patel & Yerby, Inc. (CP&Y) were:

1. Quantification of the sediment delivery to the reservoir site for the 50-year project life under pre- and post-project conditions,
2. Evaluation of the downstream effects of the dam on channel conditions and flow capacity,
3. Assessment of the potential for reducing or managing the upstream sediment supply to the reservoir, and
4. Assessment of future conditions in the North Sulphur River and tributaries upstream of the dam site in the absence of the project.

1.3. Data and Information Sources

Data and information used in this investigation were obtained from a number of sources. Previous project-related investigations that provided relevant information included:

1. Hydrologic and Hydraulic Studies of Lake Ralph Hall (RJ Brandes Co., 2004),
2. Geological Characteristics of Proposed Lake Ralph Hall (CP&Y, 2004),
3. Preliminary Subsurface Exploration, Ralph Hall Dam (Kleinfelder, 2005), and
4. Archaeology and Quaternary Geology at Lake Ralph Hall (AR Consultants, Inc., 2005).

Other data were obtained from a variety of sources, and included:

1. A 2-foot contour interval map and DTM of the proposed reservoir was provided by CP&Y.
2. Mean daily and annual peak flow data were obtained for the USGS North Sulphur River gage near Cooper, Texas (USGS Gage No. 07343000) for the period of record at the gage (1950-2005). Additionally, the 9207 summary discharge gaging data were obtained for the gage, and these were used to develop stage-discharge rating curves for different periods.
3. Bridge profiles were obtained by CP&Y for State Highway 34, FM 2990 and FM904 on the NSR; SH 34 and FM 1550 on Merrill Creek; FM 1550 on Bralley Pool Creek; FM 1550 on Baker Creek.
4. Aerial photography of the watershed for 1956 (1:20,000), 1969 (1:20,000), 1979 (1:40,000), 1989 (1:40,000), USDA.
5. Geologic Maps of Texas, Sherman (1967) and Texarkana (1966) sheets, Bureau of Economic Geology.

6. Soil Survey of Fannin County, Texas, NRCS (2001).

A 2-day helicopter and field reconnaissance of the channel and watershed of the NSR was conducted by Mr. John Levitt, P.E. (CP&Y) and Dr. Mike Harvey (MEI) in October 2005. During the field reconnaissance, four samples of bed material were collected from NSR (3) and Bralley Pool Creek (1) and provided to the Kleinfelder soils laboratory in McKinney, Texas. Because of the very high shale content of the samples, both dry and slaked gradations were determined for the samples. A more detailed field survey of the NSR and the principal tributaries upstream of the proposed dam site was conducted by Dr. Mike Harvey and Mr. Stuart Trabant (MEI) between December 12 and 16, 2005. Geomorphic and geologic features observed during the field survey were recorded, located with hand-held GPS units and photographed. Selected photographs are provided in **Appendix A**. During the course of this field work, a further 11 bed-material samples were collected, 8 in the NSR, 2 in Bralley Pool Creek, and 1 in Baker Creek. Wet and dry gradations and specific gravities were provided by Kleinfelder. All of the gradation data and specific gravities for the samples are provided in **Appendix B**.

1.4. Authorization

This study of the Lake Ralph Hall project was conducted for Chiang, Patel and Yerby, Inc. (CP&Y) and the Upper Trinity Regional Water District (UTRWD) by Mussetter Engineering, Inc. (MEI). CP&Y's project manager for this study was Mr. John Levitt, P.E. and MEI's project manager was Dr. Mike Harvey, P.G. Mr. Stuart Trabant, P.E. (Colorado) was the project engineer and Dr. Stanley A. Schumm, P.G. reviewed the report.

2. GEOLOGY AND GEOMORPHOLOGY

The dynamics of the NSR and its tributaries are intimately linked to the current geomorphic setting of the entrenched valley floor, and the characteristics of both the alluvial valley fill sediments and underlying bedrock that comprise the bed and banks of the incised channels. The details of the bedrock geology and the overlying alluvial valley fill have been described in detail elsewhere (CP&Y, 2004; AR Consultants, Inc., 2005). In the following section, the discussion of the bedrock geology and alluvial valley fill is tailored to their geomorphic significance.

2.1. Geology

The bedrock units that crop out in the North Sulphur River basin are from the Cretaceous-age Gulf Series. Both the land surface and the rock units dip slightly to the southeast (~0.5 degrees), which results in successively younger formations being exposed as the NSR flows east and southeast. From west to east, exposed in ascending order are the Austin and Taylor Groups (**Figure 2.1**). The Roxton Limestone and the Gober Chalk are the two uppermost units of the Austin Group that crop out along the north side of the NSR Basin. Although the geologic map shows a narrow band of Roxton Limestone on the north side of the NSR, field observation and mapping, and the respective lithologic descriptions of the Roxton Limestone and Gober Chalk (Texas Bureau of Economic Geology, 1966, 1967), suggest that it is the Gober Chalk that is actually observed in the beds of the headwaters of the NSR (**Figure A.1**) and the south flowing tributaries (Allen, Bear, Pot, Brushy, Pickle, Davis, Bralley Pool, Merrill, and Baker Creeks). For the purposes of this investigation, the outcrops are referred to as Roxton/Gober Chalk.

The downstream limit of the Roxton/Gober Chalk outcrop provides grade control for the upstream channel and thus limits the upstream extent of the baselevel lowering-induced incision in the tributaries (**Figure 2.2; Figure A.2**). The distance from the upstream extent of the top of the conservation pool elevation (551.0 ft msl) to the downstream limit of the Roxton/Gober Chalk outcrop provides an indication of the upstream extent of the channel incision and also the length of the incised channel that can contribute sediment to the reservoir once the dam is in place (**Table 2.1**). Erosion of the Roxton/Gober Chalk is primarily due to surficial weathering (**Figure A.3**), but the rate of erosion is low. Weathering and erosion tend to produce a low specific gravity (~2.4), sand- and gravel-sized sediment supply to the downstream incised channel (**Figure A.4**).

The uppermost unit of the Taylor Group is the Ozan Formation, a 425-foot thick dark gray calcareous, poorly bedded clay (shale) with varying amounts of silt and glauconite and some thin siltstone and limestone beds. The rock is compact, highly jointed, and highly erodible and ravels (**Figures A.5 and A.6**) when exposed to weathering (Kleinfelder, 2005). The Ozan Formation weathers in situ to a light gray shale and light yellow-brown shaly clay. The results of four borings across the valley at the proposed dam location (Kleinfelder, 2005) indicate that there is relief on the shale surface at the shale-valley fill contact.

Incision of the NSR and its tributaries has exposed the Ozan Formation in the bed (**Figure A.7**) and in the banks (**Figure A.8**) where the streams have eroded into the shale. Erosion into the shale takes place as a result of both hydraulic processes (abrasion, plucking, solution) (**Figure A.9**) and streambed weathering (slaking) (**Figure A.10**) (Howard, 1998; Tinkler and Parish, 1998; Allen et al., 2002). Slaking tests by Crawford (in preparation) indicate that the Taylor Marl has about a 50-percent weight slaking loss following a 2-cycle test. Rates of erosion into the

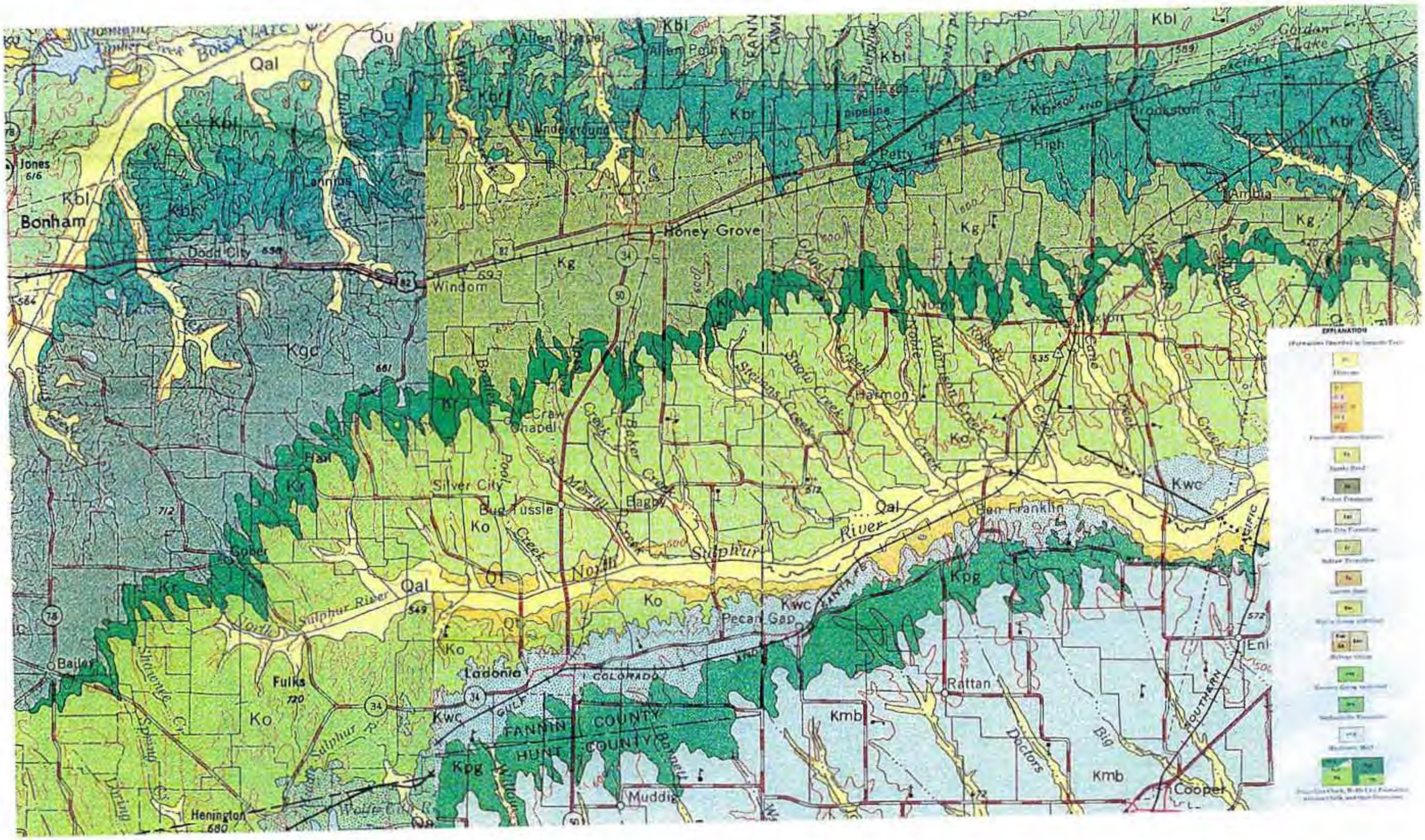


Figure 2.1. Geologic map of the North Sulphur River basin (after Texas Bureau of Economic Geology, 1966, 1967).

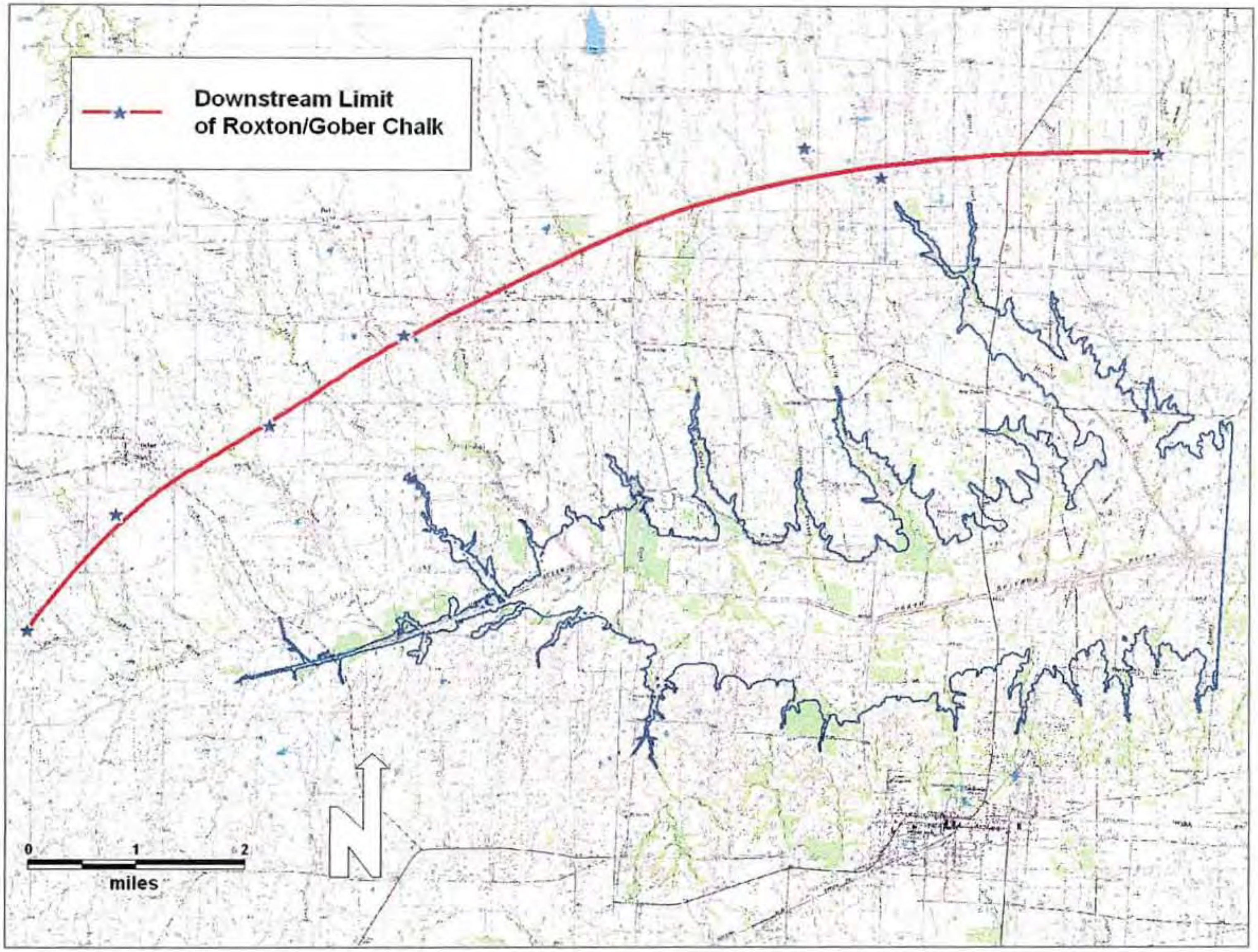


Figure 2.2. Map of the North Sulphur River basin showing the locations of the downstream limits of exposed Roxton/Gober Chalk in the headwaters of the NSR and the north-side tributaries.

weak shale (tensile strength <1 MPa; Crawford, in preparation) may ultimately be controlled by the threshold of motion of a thin mantle of sediment over the bedrock rather than the bedrock hardness (Sklar and Dietrich, 1998; Stock et al., 2005). However, Allen et al. (2002) have measured wetting-drying cycle-driven slaking rates of up to 4 inches per year in the lower bank regions of channels incised into the Taylor Marl, and rates of up to 2 inches per year in the bed. Tinkler and Parish (1998) have documented channel bed erosion rates into shales on the order of 1 inch per year, and have observed that wetting and drying cycles were primarily responsible for fragmenting the exposed shale to a size that could be transported and removed by frequent and moderate high flows. Similar processes have been observed in the bed of the NSR and its tributaries (**Figures A.11 and A.12**), where on average, there are about six wetting and drying cycles per year at the Cooper gage (**Figure 2.3**).

Table 2.1. Lengths of eroding channel between top of conservation pool extent and Roxton/Gober Chalk outcrop.

Channel	Distance to Roxton/Gober Chalk Outcrop (miles)
North Sulphur River	1.8
Allen Creek	1.9
Bear Creek	1.5
Pot Creek	1.3
Brushy Creek	1.1
Pickle Creek	1.0*
Davis Creek	1.0
Leggett Creek	1.0*
Bralley Pool Creek	1.8
Merrill Creek West Branch	0
Merrill Creek East Branch	0.5

*Concrete Box culverts provide grade control downstream of Roxton/Gober Chalk outcrop

Studies of the Quaternary-age alluvial valley fill stratigraphy of the NSR above the Ozan Formation have been conducted by Frye and Leonard (1963), Slaughter and Hoover (1963, 1965) and Rainey (1974), and have been summarized in AR Consultants, Inc. (2005). On average, the alluvial valley fill is about 30 feet thick, but the thickness is variable depending on the underlying relief on the top of the Ozan Formation, and can range from as little as 10 to 32 feet based on field observations (**Figures A.13 and A14**) and the Kleinfelder borings. Tinn clay is the soil unit mapped on the former floodplain of the NSR (NRCS, 2001). Gradation analyses of samples recovered from the floodplain soils indicate that about 90 percent of the soil is smaller than sand (No. 200 sieve) and Atterberg Limits indicate that the soils are classified as high plasticity (CH) and low plasticity (CL) clays (Kleinfelder, 2005). Shallow groundwater is perched on the shale-alluvium contact, and appears to be associated with mass failures of the overlying alluvial materials when it is daylighted in the banks (**Figure A.15**).

2.2. Geomorphology

The NSR originates near the axis of the Preston Anticline and flows east paralleling the general east-northeast strike of the south-southeast dipping Cretaceous-age bedrock (Barnes, 1967).

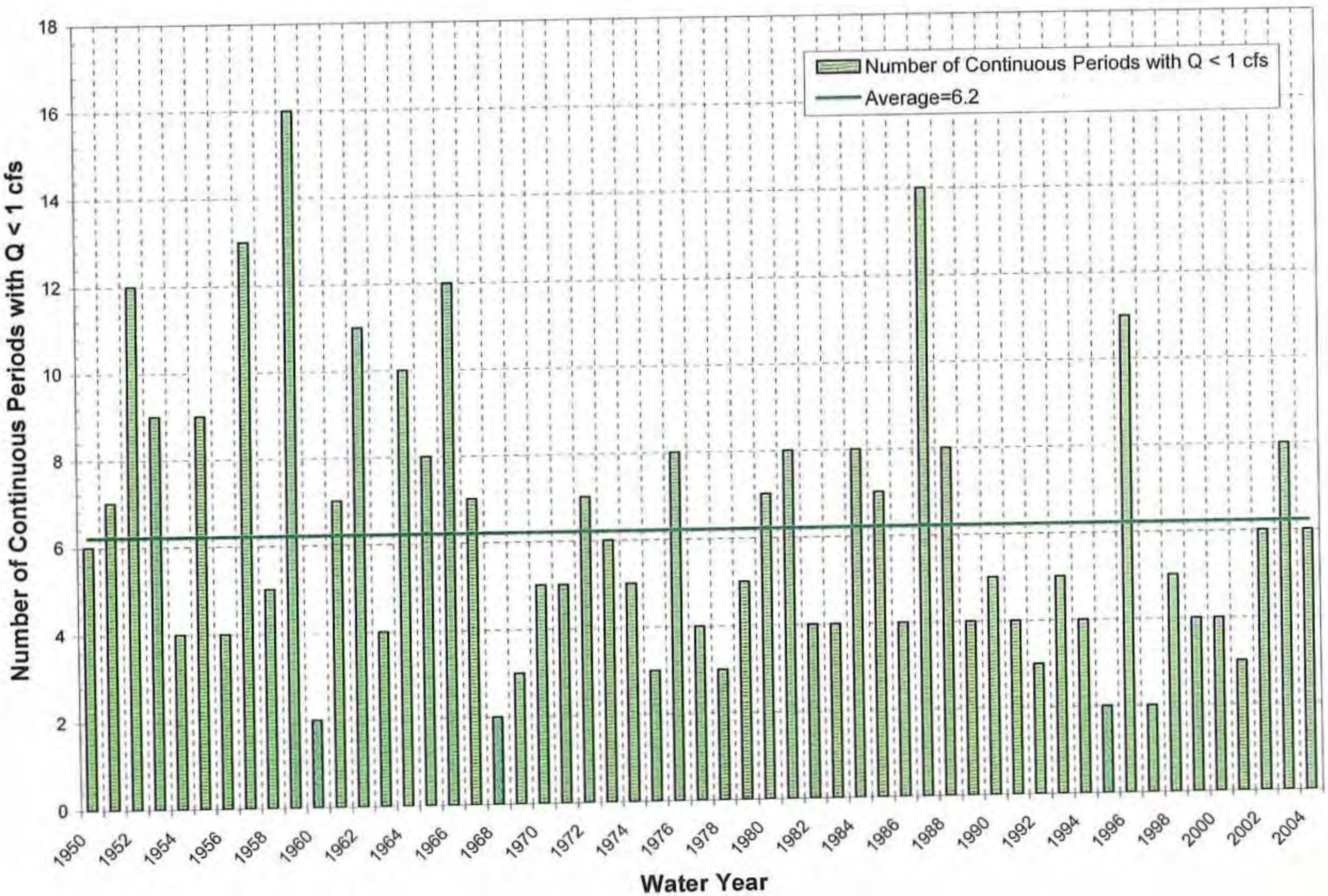


Figure 2.3. Number of continuous periods with flows less than 1 cfs at the USGS gage near Cooper, Texas. On average there are about six wetting and drying cycles per year (Q <1 cfs).

The south-southeast dip of the underlying bedrock is the cause of the asymmetrical valley profile of the NSR. Down-dip preferential erosion has resulted in the south-draining north side tributaries being long and having relatively gentle slopes, while the north-draining south side tributaries are short and steeper (Figure 1.1). Because of the channelization-induced incision, both the north- and south-draining tributaries are currently incised. The pre-channelization floodplains of both the NSR and the incised tributaries are now terraces that are hydrologically disconnected from their channels.

2.2.1. Incised Channel Evolution Models

The dominant characteristic of the present day NSR system is the extent of the incision and the incision-induced widening. In the context of the sediment supply to the system from channel erosion processes, it is necessary to determine whether the system has re-attained equilibrium between the water and sediment supply and the channel morphology 75 years after channelization. Numerous studies of incised channels in alluvial materials in humid regions of the U.S. have shown that following channelization, the channel passes through a consistent, predictable sequence of channel forms with time (Ireland et al., 1939; Schumm et al., 1984; Harvey and Watson, 1986; Simon and Hupp, 1986; Simon, 1989). These systematic temporal adjustments have been collectively referred to as channel evolution, and a number of geomorphic models (Incised Channel Evolution Models—ICEM) have been developed that permit interpretation of past and present channel processes, as well as prediction of future channel processes (Schumm et al., 1984; Simon and Hupp, 1986).

A five-stage ICEM was developed by Schumm et al. (1984), and modified to include the channelized stage by Harvey and Watson (1986). The model describes the systematic evolution of a channelized stream from a state of man-induced disequilibrium (Type II) to a new state of dynamic equilibrium (Type VI) (**Figure 2.4**). The model identifies, quantifies, and integrates four important components of channel evolution: bank stability, the dominant or effective discharge, the hydraulic energy of those discharges and the morphological adjustments of the channel through time and space (Harvey and Watson, 1986; Watson et al., 1988). Through time, the channel incises (Types III and IV), widens as a result of bank failure (Types IV and V), and ultimately aggrades (Type VI), at which point an equilibrium channel that reflects the balance between sediment supply and transport capacity has formed within the over-widened incision into the valley floor. Bank failure occurs when the bank height (h) exceeds the critical bank height (h_c) (Little et al., 1981; Watson et al., 1988). When the banks are steeper slab, or wedge, failures predominate (Type IV), and as the bank angle is reduced deeper seated slump failures predominate (Type V) (Lohnes and Handy, 1968; Harvey and Watson, 1986; Thorne, 1988 and 1999; Simon and Darby, 1999).

Repeat cross-section surveys of an incised channel in northern Mississippi (Schumm et al., 1984), and a computer simulation of the geomorphic evolution of that incised channel (Watson et al., 1986), indicated that total soil loss due to channel erosion (bed and banks) from the 42-square-mile watershed, was on the order of 6.5×10^6 tons over a 15-year period. Initial rates of soil loss were on the order of 0.1×10^6 t/yr (3.7 t/ac/yr), but the maximum rate occurred when the channel was most actively widening and approached 0.5×10^6 t/yr (19 t/ac/yr). Ultimately, channel loss rates diminished to about 0.05×10^6 t/yr (1.9 t/ac/yr) as the channel approached a new state of equilibrium. Simon (1989) showed similar trends with erosion rates eventually returning to less than 2 t/ac/yr. Other studies of incised channels (Simon et al., 1996; Simon and Darby, 1999) have shown that sediment emanating from incised channels can represent up to 80 percent of the total sediment yield from a landscape.

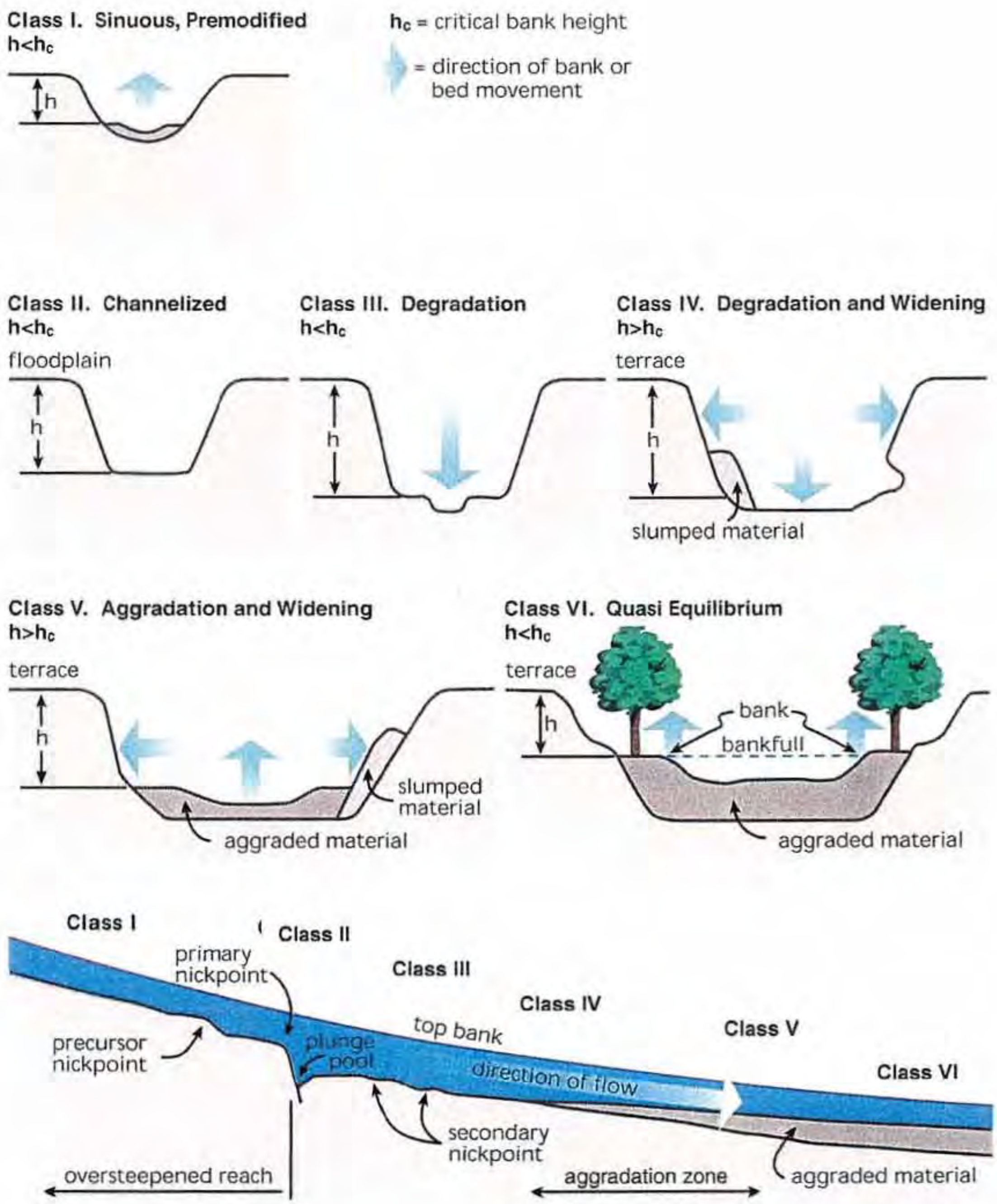


Figure 2.4. Incised channel evolution model (after Schumm et al., 1984).

2.2.2. Channel Evolution in the North Sulphur River

In the context of the current status of the NSR, and sediment yield to the dam site, it is important to know the evolutionary stage of the mainstem and tributaries. In the channelized streams of the humid southeastern U.S., the channel evolution sequence can take about 40 to 50 years (Schumm et al., 1984; Schumm, 1999; Simon, 1989) and over 100 years in the arroyos in the semi-arid southwest (Gellis et al., 1995). Therefore, it could be expected that the NSR, that was channelized about 75 years ago, has completed the evolutionary sequence and might be approaching a new state of equilibrium with the imposed flows and sediment loads. Depending on location, there are indications that this has in fact occurred (**Figure A.16**). However, it is equally apparent that there are sections of the NSR and its tributaries that are still actively widening (**Figure A.17**), and have very little or no sediment accumulation on the bed, which is composed of erodible shale (**Figure A.18**), both conditions which are indicative of ongoing disequilibrium. Similar conditions of apparent disequilibrium (**Figure A.19**), active channel widening (**Figure A.20**) and the presence of shale in the bed and absence of sediment accumulation on the bed can be observed in the tributaries to the NSR. Ongoing degradation below recently replaced bridges across the tributaries also argues for continuing disequilibrium (**Figure A.22**).

The mainstem of the NSR between FM 904 (Sta 00+6) and about 1 mile upstream of SH 68 (the upstream end of the DTM) (Sta 619+66) was subdivided into 10 subreaches, primarily on the basis of the location of the major tributaries (refer to Table 4.1 for subreach boundaries and Figure 2.37 for stationing). Cross sections representing the physical characteristics of the subreaches were developed from the DTM (**Figures 2.5 through 2.18**), and photographs of the NSR at these locations are provided in Appendix A (**Figures A.23 to A.36**). **Table 2.2** summarizes this subreach information.

Table 2.2. Summary of subreach information for mainstem of North Sulphur River.

Subreach Number	Subreach Description	Cross Section Station (ft)	Figure Number	Photograph Number
1	Upstream of SH 68	604+27	2.5	A.23
2	Allen Creek to Bear Creek	562+44	2.6	A.24
3	Bear Creek to Brushy Creek	530+93	2.7	A.25
3	Bear Creek to Brushy Creek	496+42	2.8	A.26
3	Bear Creek to Brushy Creek	468+60	2.9	A.27
3	Bear Creek to Brushy Creek	453+04	2.10	A.28
4	Brushy Creek to Pickle Creek	390+34	2.11	A.29
5	Pickle Creek to Davis Creek	344+08	2.12	A.30
6	Davis Creek to Leggetts Branch	303+01	2.13	A.31
7	Leggetts Branch to Bralley Pool Creek	273+25	2.14	A.32
7	Leggetts Branch to Bralley Pool Creek	246+13	2.15	A.33
8	Bralley Pool Creek to Merrill Creek	187+60	2.16	A.34
9	Merrill Creek to dam site	88+77	2.17	A.35
10	Dam site to FM 904	32+36	2.18	A.36

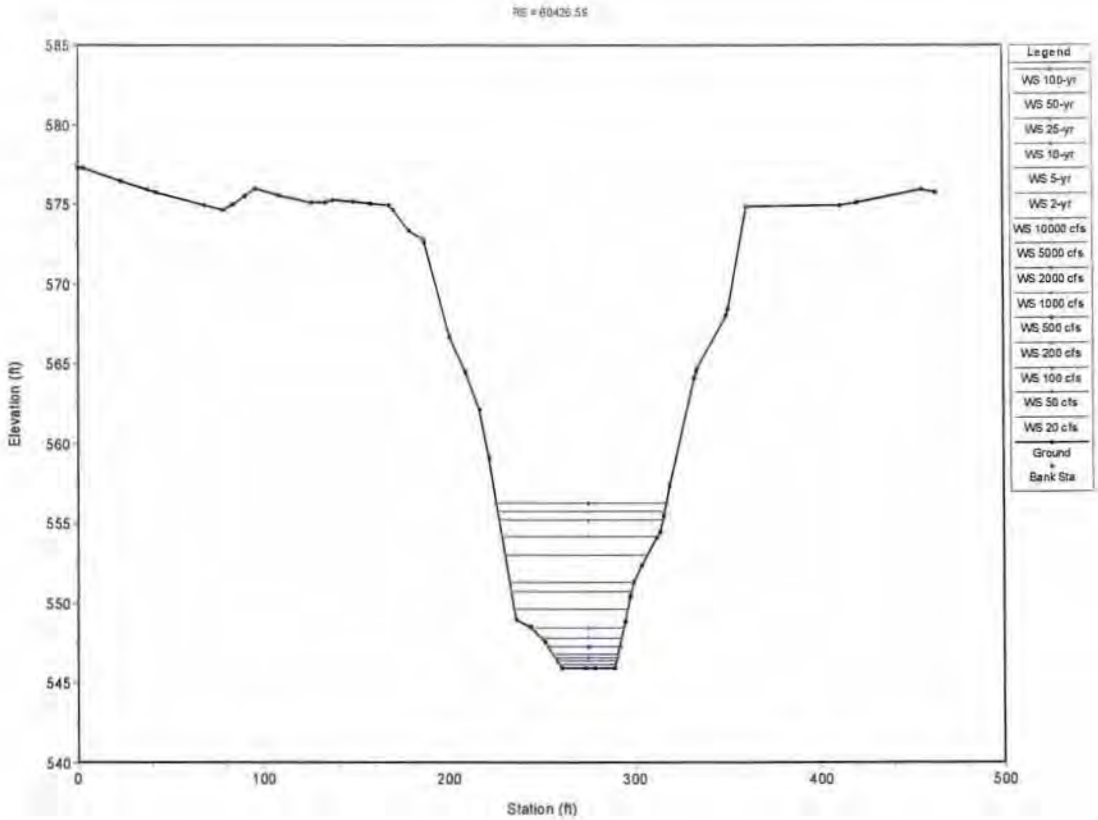


Figure 2.5. Cross section of North Sulphur River in Subreach 1, Sta 604+27.

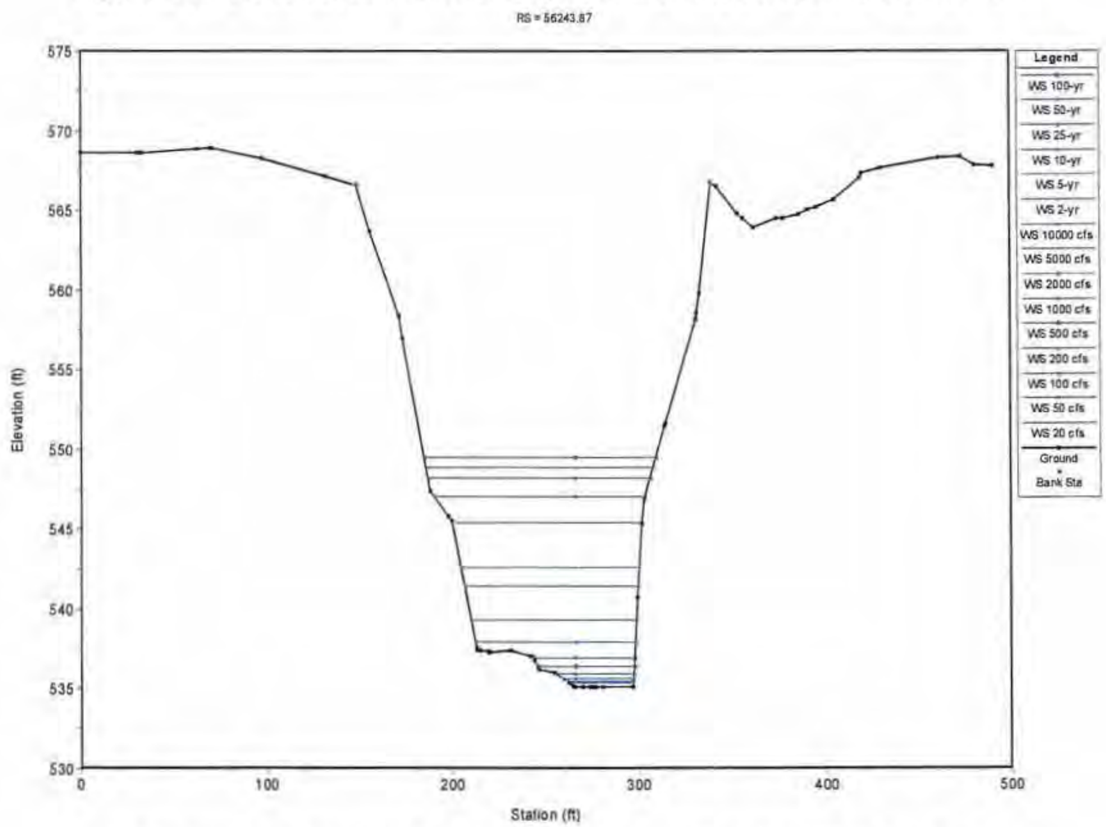


Figure 2.6. Cross section of North Sulphur River in Subreach 2, Sta 562+44.

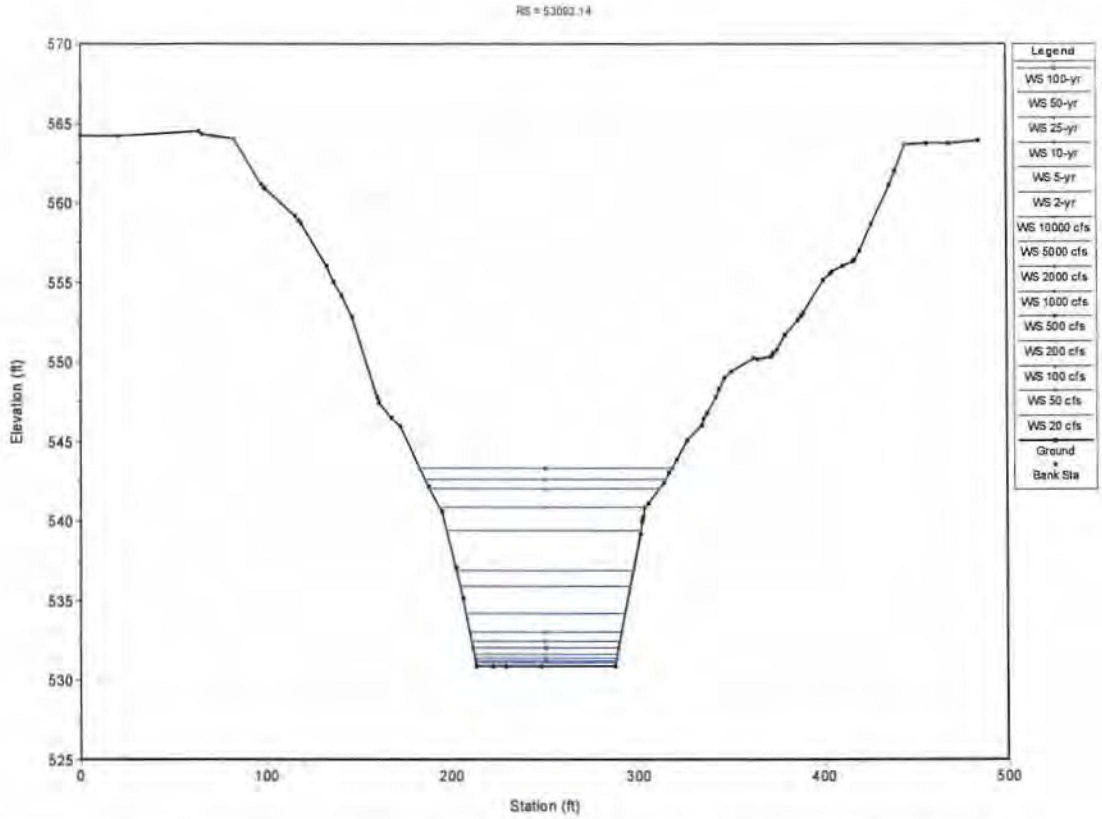


Figure 2.7. Cross section of North Sulphur River in Subreach 3, Sta 530+93.

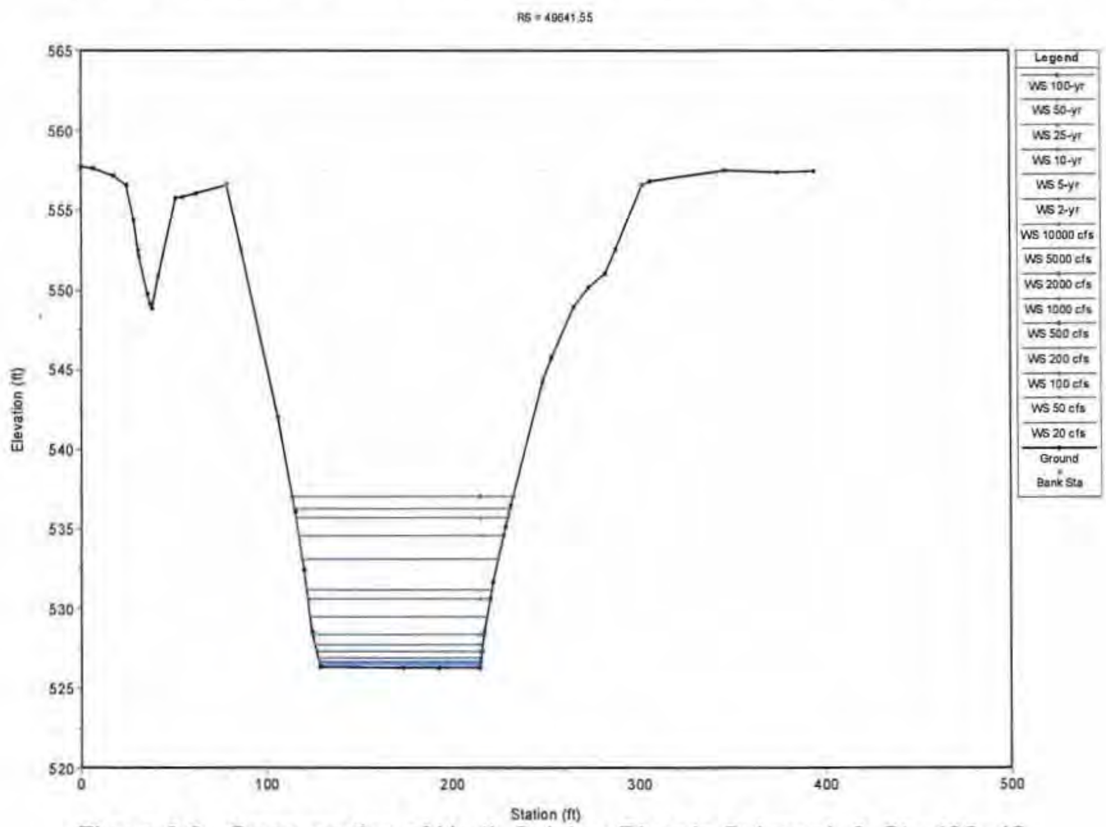


Figure 2.8. Cross section of North Sulphur River in Subreach 3, Sta 496+42.

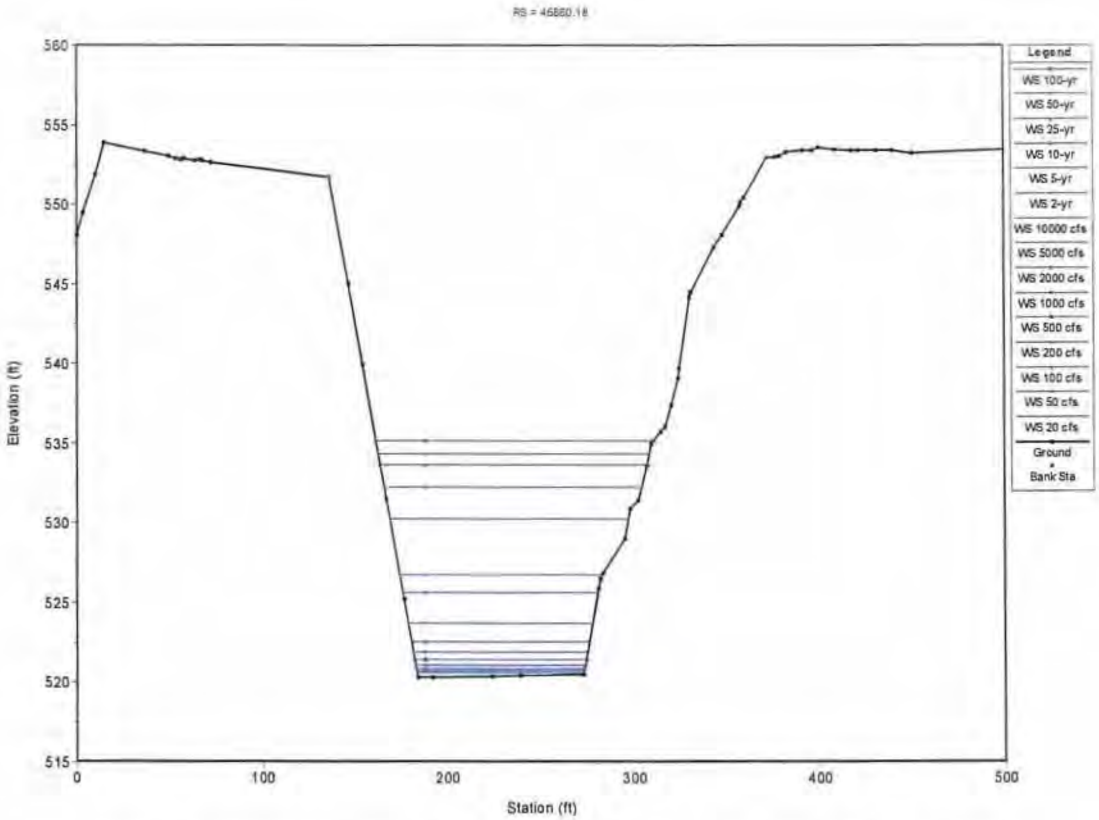


Figure 2.9. Cross section of North Sulphur River in Subreach 3, Sta 468+60.

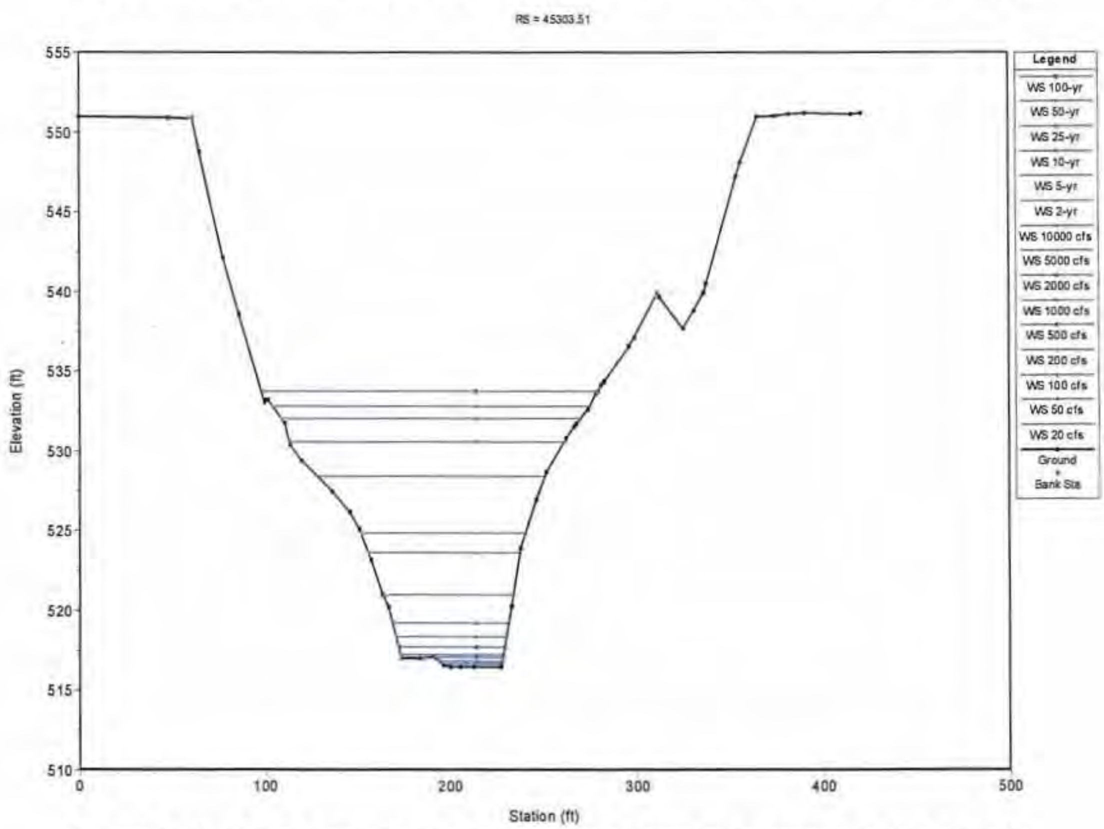


Figure 2.10. Cross section of North Sulphur River in Subreach 3, Sta 453+04.

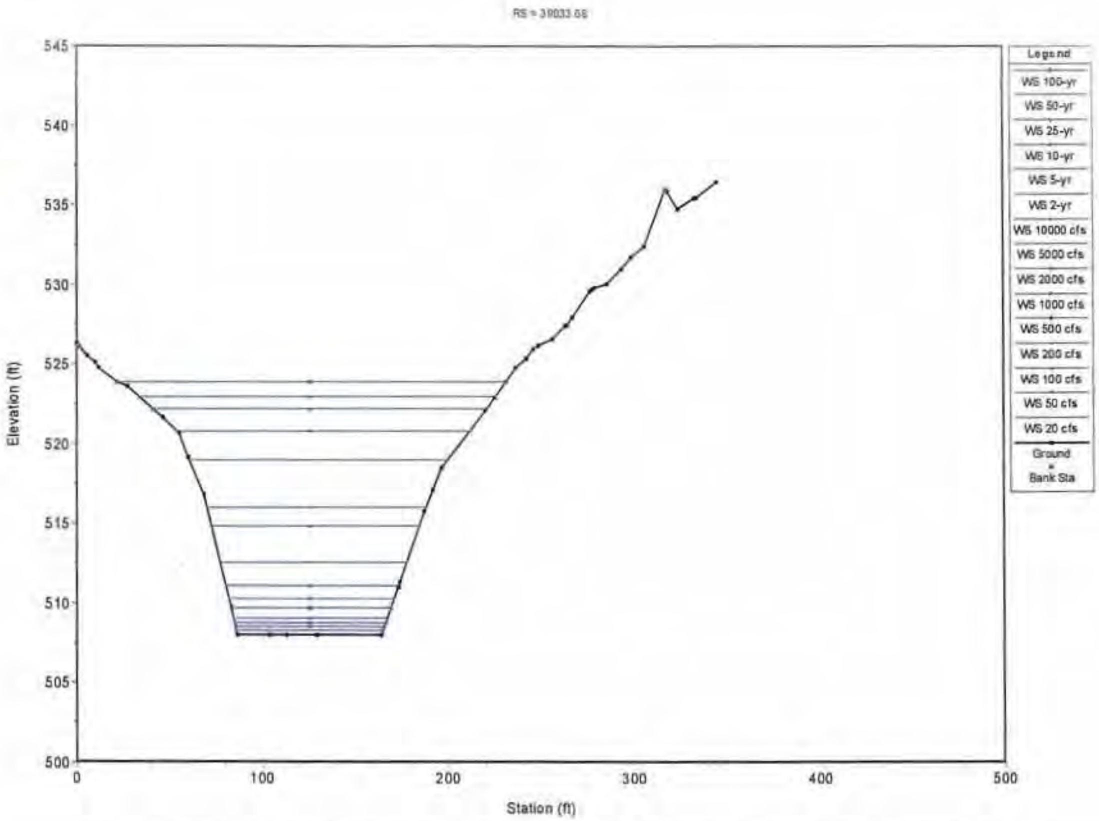


Figure 2.11. Cross section of North Sulphur River in Subreach 4, Sta 390+34.

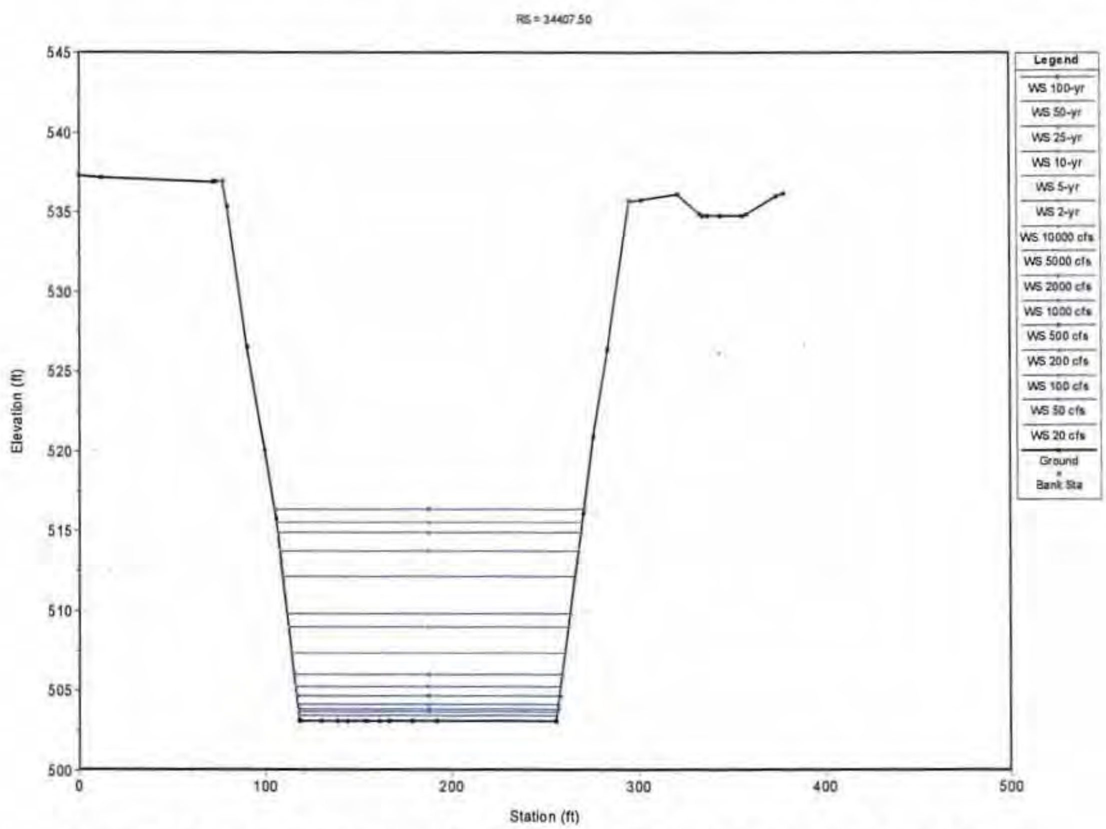


Figure 2.12. Cross section of North Sulphur River in Subreach 5, Sta 344+08.

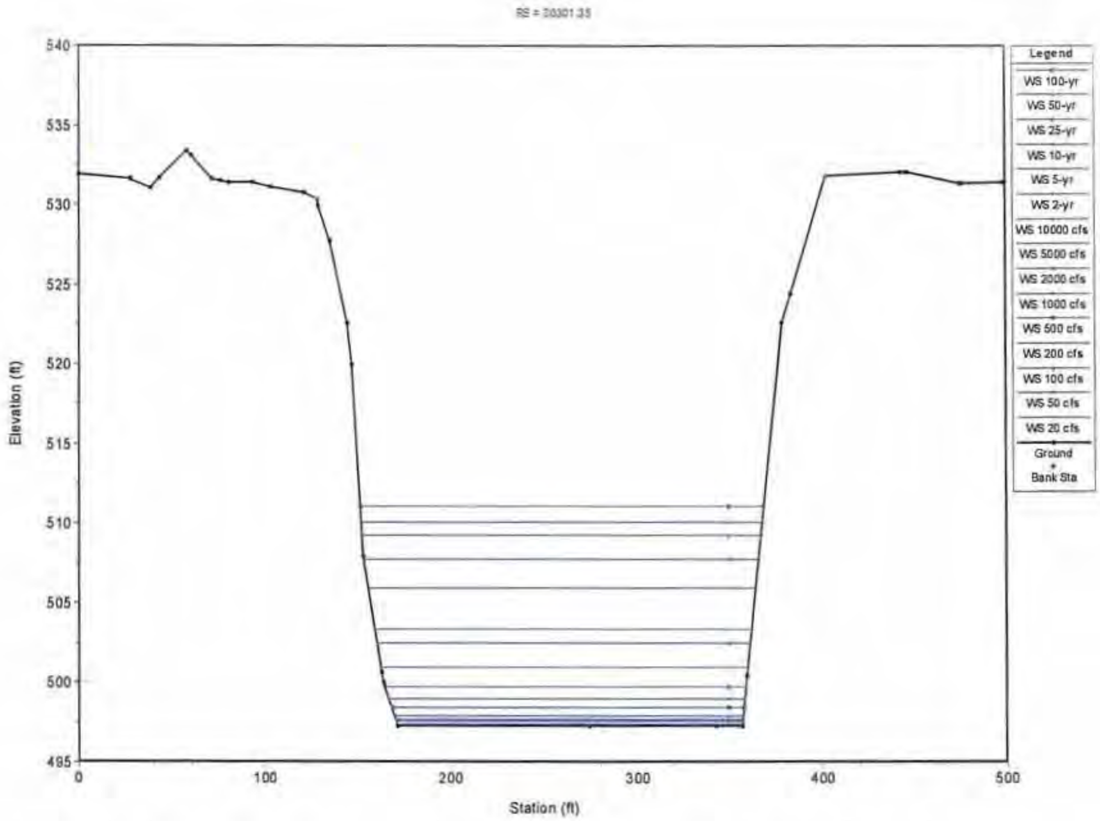


Figure 2.13. Cross section of North Sulphur River in Subreach 6, Sta 303+01.

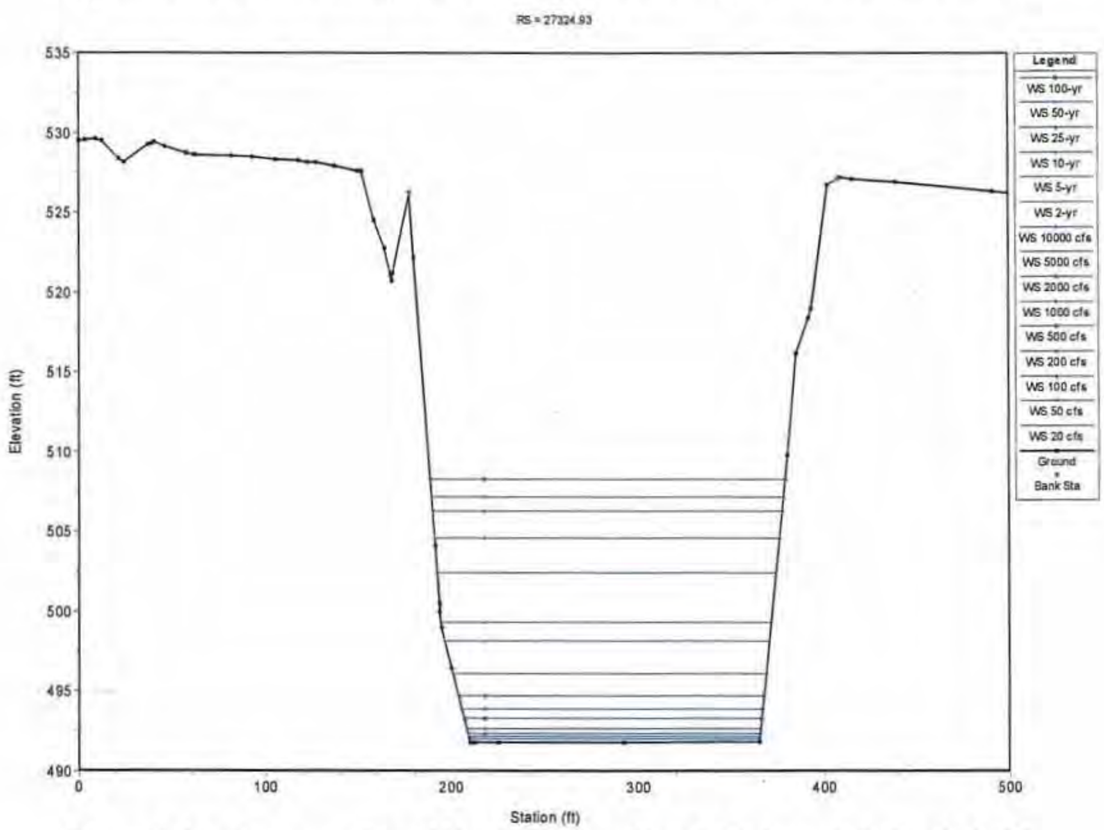


Figure 2.14. Cross section of North Sulphur River in Subreach 7, Sta 273+25.

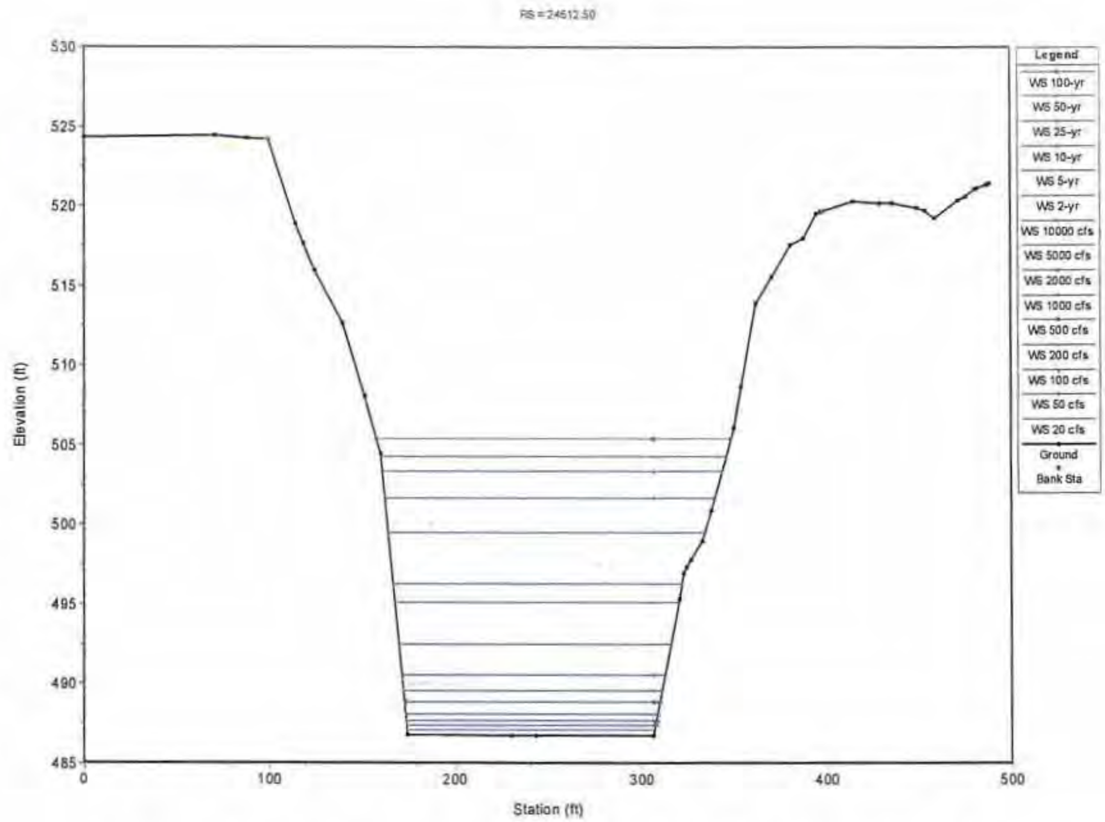


Figure 2.15. Cross section of North Sulphur River in Subreach 7, Sta 246+13.

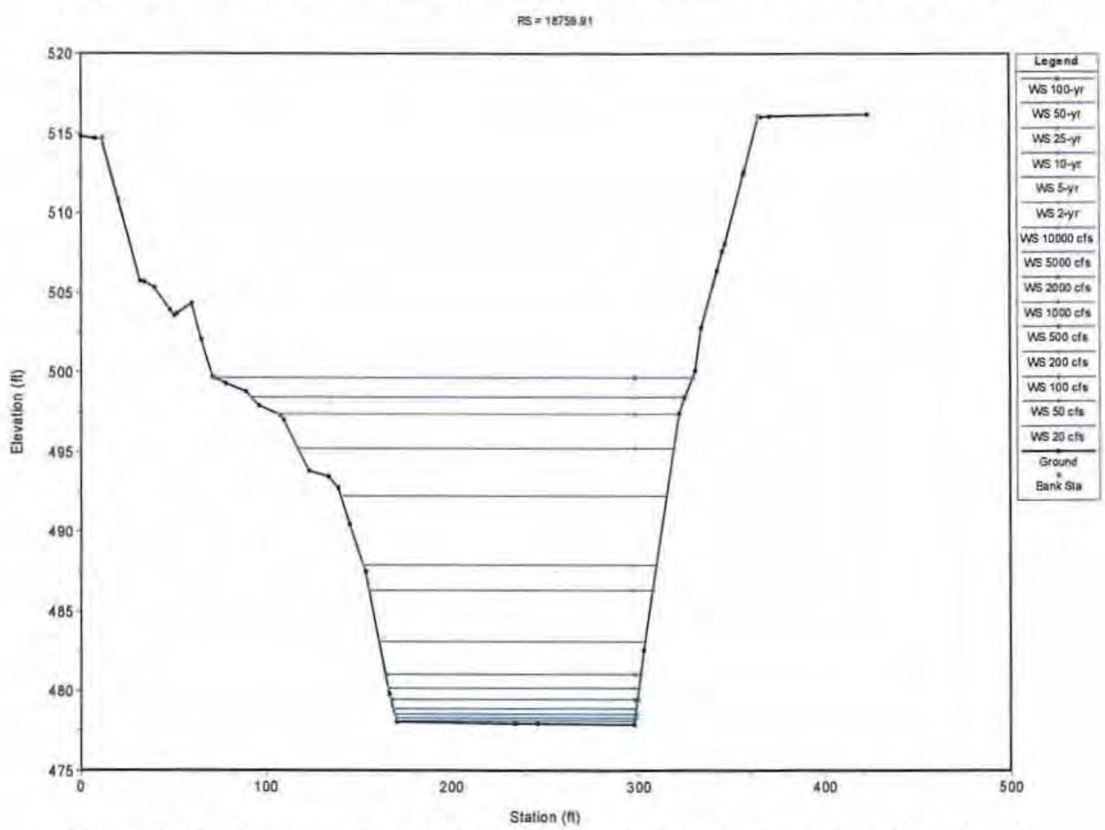


Figure 2.16. Cross section of North Sulphur River in Subreach 8, Sta 187+60.

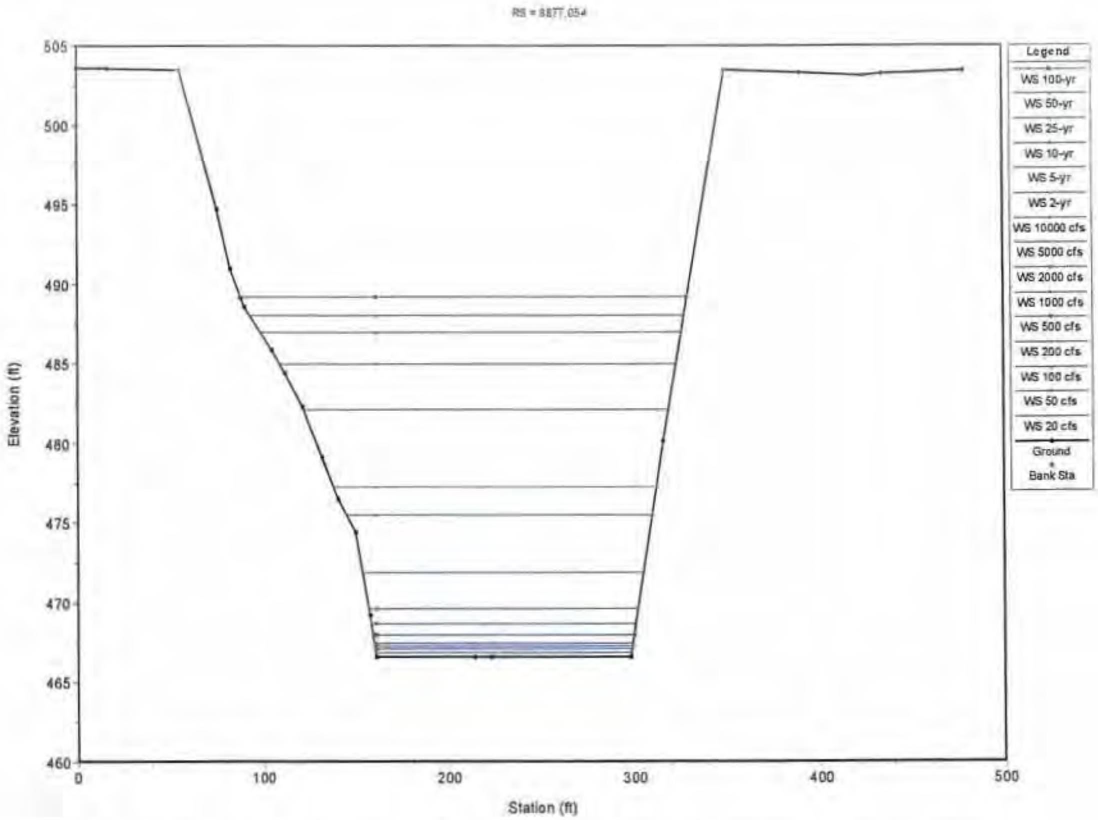


Figure 2.17. Cross section of North Sulphur River in Subreach 9, Sta 88+77.

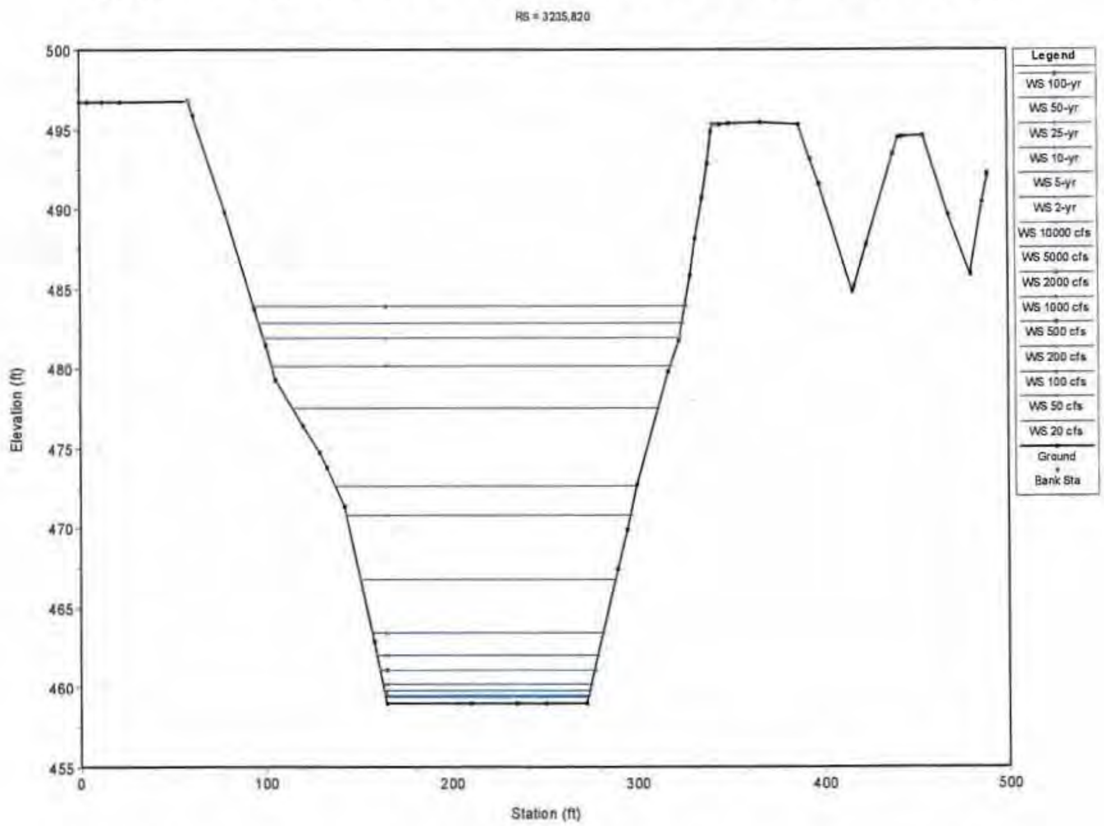


Figure 2.18. Cross section of North Sulphur River in Subreach 10, Sta 32+36.

In Subreach 1 (Figures 2.5 and A.23), the channel has incised at least 10 feet into the shale, and shale forms the lower portion of the banks and the bed. The banks are steep, and weathering of the bed and banks produces a significant amount of gravel-size clasts that are initially transported as bedload, but eventually slake into primarily wash-load-sized material (Tinkler and Parish, 1998; Allen et al., 2002). In Subreach 2 (Figures 2.6 and A.24), the banks are steep, and most of the erosion and channel widening is due to slab failure of the alluvium that overlies the exposed shale (Harvey and Watson, 1986; Thorne, 1988 and, 1999). Subreach 3 (Figures 2.7, 2.8 and A.25, A.26) is characterized by active slab failures of the alluvial fill that maintain a steep bank slope, as well as deeper seated slump failures of the alluvium (Figures 2.9 and A.27). Alternating steeper and flatter bank slopes that create an asymmetrical cross section are characteristic of this subreach (Figure 2.10 and A.28). Subreach 4 (Figures 2.11 and A.29) is characterized by symmetrical cross sections with convex lower slopes and concave upper slopes formed by deep-seated slump failures in the alluvium. The bed of the channel is composed of shale with a veneer of sediment and the banks are composed of displaced, clay-rich alluvium, that is vegetated and root reinforced, and therefore, relatively erosion resistant. Mass failure of both banks, effectively reduces the bottom width of the channel.

Steep banks with slab failures of the alluvium and exposed shale in the lower parts of the banks are characteristic of Subreach 5 (Figures 2.12 and A.30), suggesting that the channel in this subreach has not adjusted as much as in Subreach 4, or that the channel instability has been reactivated by lateral erosion of the mass failed alluvium that had protected the shale from erosion. Lateral erosion of the failed alluvium may be due, in part, to ongoing weathering-driven erosion of the shale in the bed of the channel. Subreach 6 (Figures 2.13, and A.31) has very similar characteristics to Subreach 5, and active channel erosion and widening is ongoing. In Subreach 7, the bank slopes are generally flatter and are indicative of deep seated mass failure of the alluvial fill, but there has been erosion of the toes of the failed banks, and a vertical shale bank that had been buried by the mass failures is now exposed (Figures 2.14, 2.15 and A.32, A.33). Similar conditions are observed in Subreach 8 (Figures 2.16 and A.34), but the degree of erosion of the alluvial toe materials is higher, which might suggests that retreat of the toe is systematic and should progress upstream over time. However, in Subreaches 9 and 10, the toes of the banks are composed of failed alluvium, and there is less sign of toe erosion and retreat (Figures 2.17, 2.18 and A.36, A.36). Therefore, it appears that erosion and retreat of the alluvial material is locally controlled and may be due to the relative erodibility of the shale, which would control the rate of vertical erosion of the bed. Support for the local control of the retreat of the failed alluvial material is provided by variable degrees of failure farther downstream (**Figure A.37**). It is possible that retreat of the failed alluvium (**Figure A.38**), exposure of the shale in the toes of the bank and ongoing degradation into the bed combine to initiate a new cycle of deep seated mass failure of the overlying alluvium (Figure A.15) that results in further widening of the channel top width (**Figure A.39**).

2.2.2.1. North Sulphur River Channel Evolution Model

Field observations permit a channel evolution model (NSRCEM) to be developed for the NSR and its tributaries (**Figure 2.19**), but the model varies significantly from those developed for alluvial streams (Figure 2.4). There is little doubt that following channelization in the late 1920s the NSR incised and widened (Avery, 1974) and followed the typical channel evolution sequence while the channel boundary materials were composed of alluvium (Types I through V). A similar sequence of channel evolution has been observed on Mill Creek, tributary to Chambers Creek in the Blackland Prairie region, but the degradation has yet to expose the underlying shale bedrock (P. Allen, Baylor University, pers. comm., 2006). However, exposure of the shale has added a significant complicating factor to the evolution of

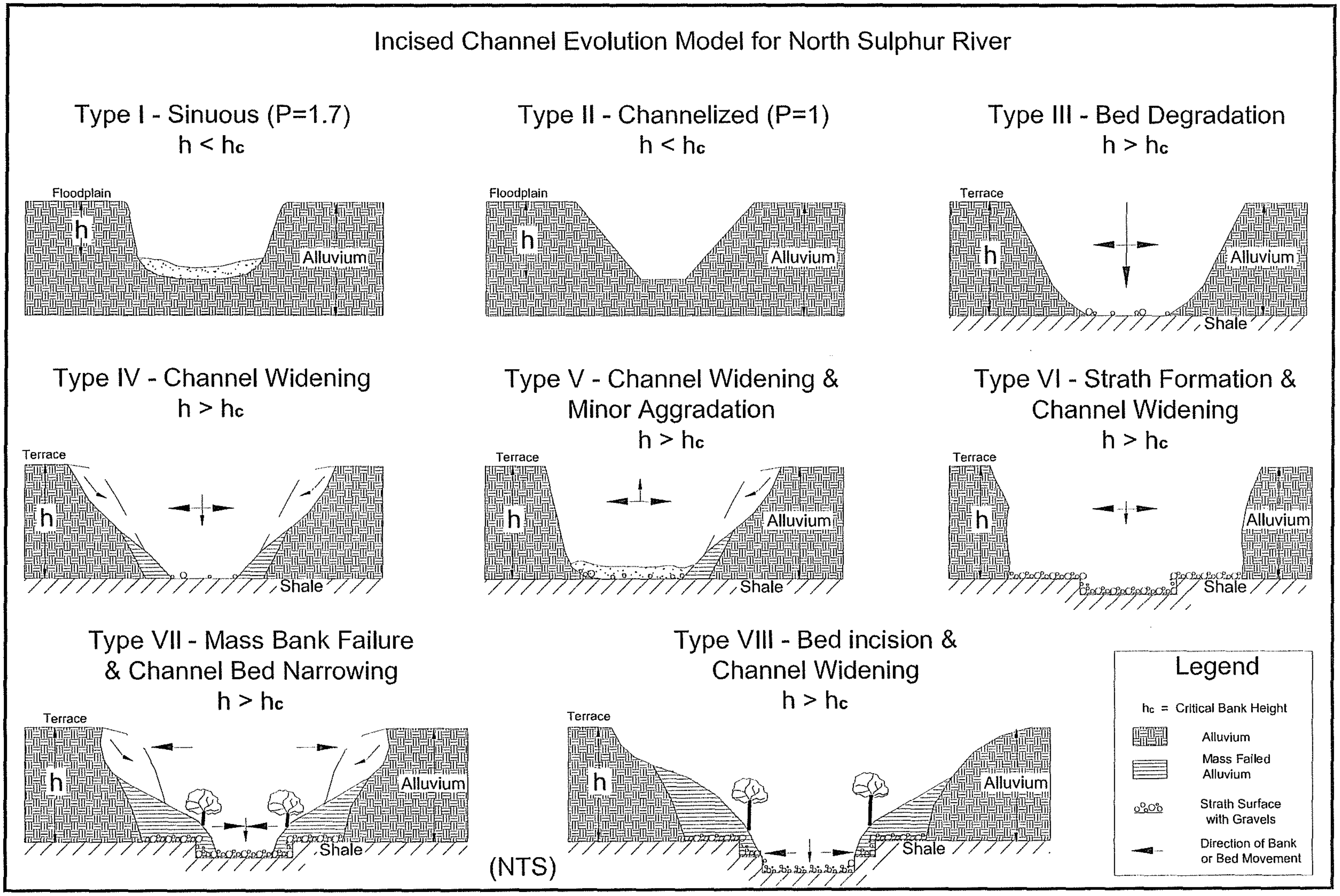


Figure 2.19. Channel evolution model (NSRCEM) for the North Sulphur River.

the channel, to the point where the existing CEMs no longer apply. Ongoing vertical and lateral erosion of the exposed shale in the bed and the banks is dependant primarily on weathering processes that are controlled by wetting and drying cycles (Tinkler and Parish, 1989; Allen et al., 2002) and not hydraulically controlled processes of sediment entrainment and transport. Flow events in the channel remove the weathering products and re-initiate vertical and lateral erosion into the shale. As a rule, lateral erosion rates exceed vertical erosion rates in bedrock and result in the formation of gravel-covered strath surfaces that become terraces when vertical erosion of the bed occurs (Leopold et al., 1964; Schumm, 1977) (Type VI). Deep-seated slump failures of the overlying alluvium bury the strath surfaces (Type VII) and prevent lateral erosion of the shale. Resulting channel narrowing may actually accelerate erosion of the shale exposed in the bed, which in turn leads to undercutting of the erosion-resistant, root-reinforced alluvium, thereby leading to re-exposure of the shale in the toe of the banks and ongoing lateral retreat of the shale (Type VIII). It is likely that over time the incision into the shale will induce further mass failure of the alluvial valley fill and a Type VII condition will be reestablished at a lower bed elevation. The NSRCEM applies equally to the larger tributaries that have degraded into the shale bedrock. Based on the NSRCEM, Subreaches 1 through 3 were classified as Type VI, Subreach 4 was classified as Type VII, Subreaches 5 through 8 were classified as Type VIII, and Subreaches 9 and 10 were classified as Type VII. Similar sequences are present in the larger tributaries.

Based on the current topography of the NSR and the major tributaries as determined from the DTM, and assuming a bulk unit weight of 100 lb/ft^3, approximately 18x10^6 tons of sediment has been eroded upstream of the proposed dam site from the mainstem of the NSR, and a further 10x10^6 tons has been eroded from the major tributaries. Based on the observations of Watson et al. (1986) and Simon (1989), the erosion rates and sediment yields would have varied over time, but on an average annual basis for the period from 1927 to 2005, the channel erosion would have yielded about 3,500 t/sq mi (3.8 t/ac/yr) at the dam site. Suspended-sediment measurements (8 years) at the USGS gaging station on the NSR near Talco, Texas (USGS Gage No. 7343200) showed a maximum annual rate of 2,642 t/sq mi (4.1 t/ac/yr) in 1968 (Texas Dept. of Water Resources, 1979), but this was about 40 years after channelization, and therefore, mostly probably does not reflect the higher sediment loads when the channel was most actively eroding. Currently, the incised channel has the ability to convey in excess of the 100-year flood in-bank (Figures 2.5 through 2.18), the bed of the river is composed of shale, and therefore, the current supply of sediment to the channel is far less than the transport capacity. As a consequence, it is highly unlikely that the NSR will attain a state of equilibrium in the near future. Prevention of further incision and widening of the channels will require significant deposition of sediment on the bed of the river. This can only occur if either the bed-material sediment supply is increased significantly, or the hydraulic capacity is reduced significantly. For example, assuming that sand–sized material would be deposited on the bed of the river, and that velocities less than 2 ft/sec would be required at the 2-year flow to induce deposition of sand on the bed, the effective width of the channel of the NSR would have to increase by an order of magnitude, and shear stresses would have to decrease from between 0.5 and 0.6 lb/ft^2 to less than 0.2 lb/ft^2 (**Figure 2.20**).

2.2.3. Existing Channel Morphology

Existing conditions morphometric characteristics of the mainstem of the NSR and the major tributaries were developed from the DTM. **Figure 2.21** shows the bed and valley floor profiles between FM 904 bridge and the upstream end of the mapping above Allen Creek. The profiles show that on average the channel depth is on the order of 35 feet in the downstream reaches and decreases to about 25 feet in the upstream reaches. The bed slope is about

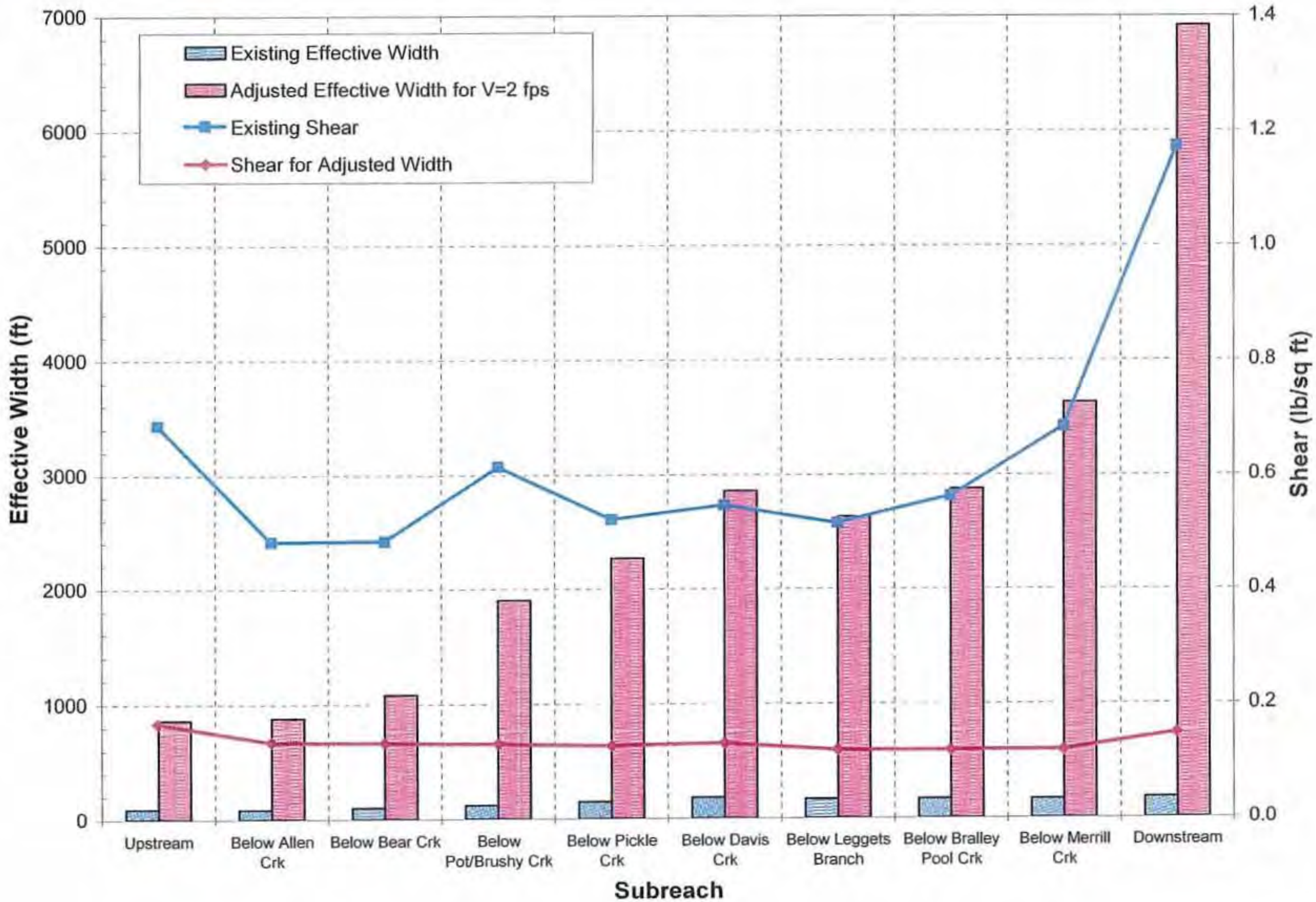


Figure 2.20. Existing and adjusted effective widths and shear stresses for the 10 subreaches of the North Sulphur River at the 2-year peak flow assuming that velocities would have to be < 2 ft/s to induce sediment deposition on the bed.

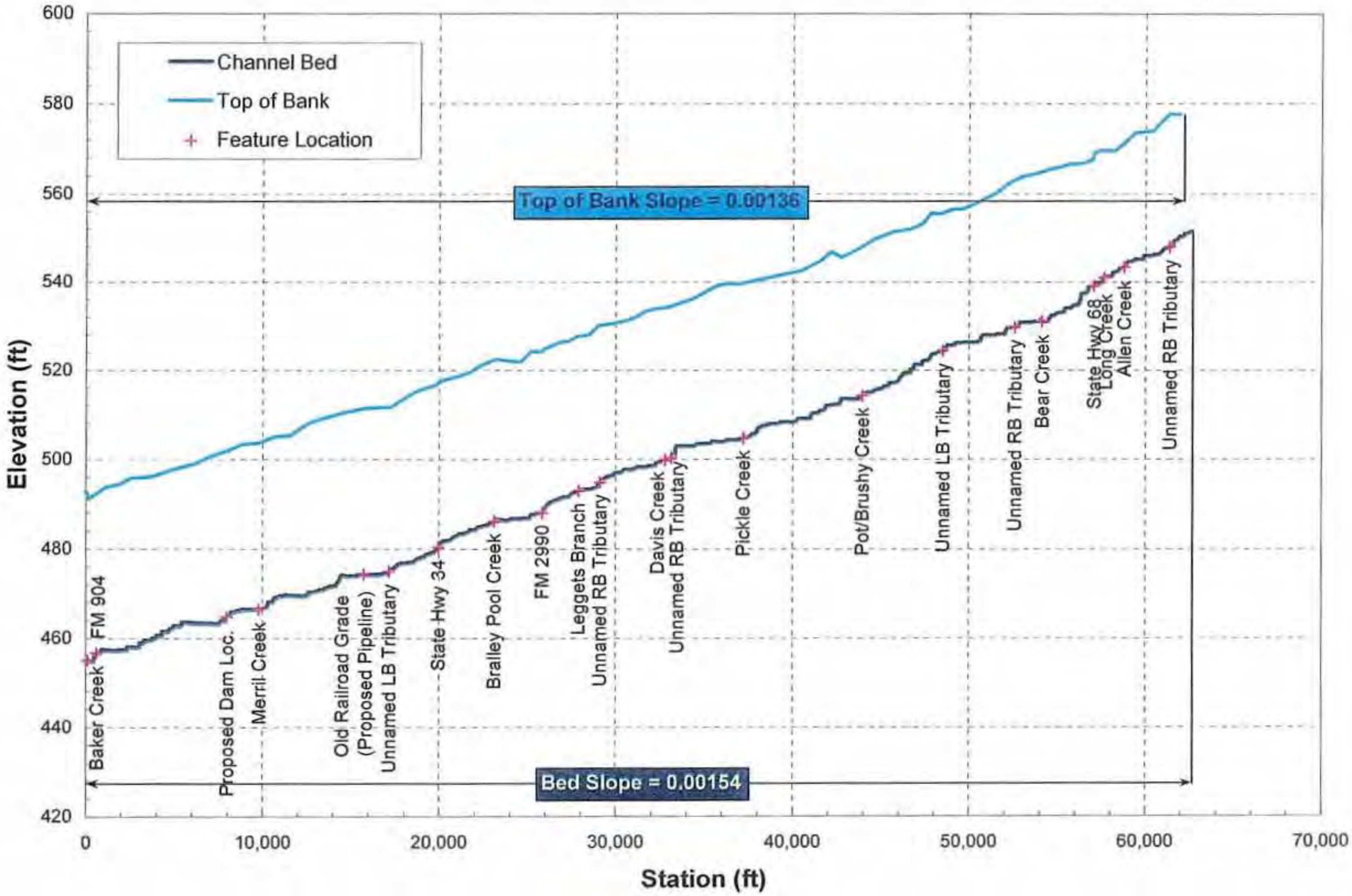


Figure 2.21. Longitudinal profiles of the bed and valley floor of the North Sulphur River between FM 904 Bridge and upstream of Allen Creek.

0.0012 (6.3 ft/mi) between FM 904 and the Brushy Creek confluence (Subreaches 4 through 10) and increases by about 60 percent (0.00195: 10.3 ft/mi) between Brushy Creek and upstream of Allen Creek (Subreaches 3 through 1). The average slope for the valley floor is 0.0014 (7.4 ft/mi) and the average bed slope is 0.0015 (7.9 ft/mi). Channel top widths were identified at cross sections that were used to develop the HEC-RAS model of the NSR (Chapter 4), and were plotted against the distance upstream of the FM 904 bridge (**Figure 2.22**). The data show that in general terms the channel topwidth increases in the downstream direction as would be expected. However, the 5-point moving average shows some interesting patterns. Where the banks tend to be steepest (Subreaches 1, 2, 3, 5, 6) the channel top widths are narrower than where the bank angles are less steep as a result of the deep-seated mass failures (Subreaches 4, 7, 8, 9, 10). This suggests that through time further channel widening should be expected in Subreaches 1, 2, 3, 5 and 6.

Longitudinal profiles of Merrill Creek (**Figure 2.23**), Bralley Pool Creek (**Figure 2.24**), Leggetts Branch (**Figure 2.25**), Davis Creek (**Figure 2.26**), Pickle Creek (**Figure 2.27**), Brushy Creek (**Figure 2.28**), Bear Creek (**Figure 2.29**), Allen Creek (**Figure 2.30**) and Long Creek (**Figure 2.31**) show that all of the tributaries have incised in response to baselevel lowering in the NSR, and channelization of the lower reaches of some of them (Merrill, Bralley Pool, and Davis Creeks). At the mouth of Merrill Creek, the channel depth is about 36 feet, but 4.5 miles upstream (Sta 240+00) the depth has reduced to 22 feet (Figure 2.23). Upstream of the Roxton/Gober Chalk knickpoint, the channel depth is only about 8 feet. Since Merrill Creek is relatively straight, the valley floor slope and the channel slope should be similar, but as can be seen on Figure 2.23 the channel slope is about 2.3 times steeper than the valley slope, and hence further erosion of the bed should be expected. The rate of degradation will depend primarily on the weathering characteristics of the exposed shale. Similar conditions are present in Bralley Pool Creek (Figure 2.24). At the mouth the channel depth is about 36 feet, and it reduces to about 13 feet upstream. Bralley Pool Creek is reasonably sinuous, except in the lower channelized reach, and the channel slope is only about 1.4 times steeper than the valley floor slope, and therefore, some further degradation of the shale bed is to be expected.

A concrete box culvert provides grade control in Leggetts Branch about half a mile upstream of the confluence with NSR (Figure 2.25). Downstream of the box culvert the channel depth is about 35 feet, but upstream it is about 12 feet, which further reduces to about 4 feet upstream of a local bridge crossing. A concrete box culvert is present at the FM 1550 crossing. Provided that the downstream culvert continues to provide grade control, there is little likelihood that there will be significant further degradation of the tributary. Concrete box culverts provide grade control in Davis Creek at the FM 2990 crossing and at the FM 1550 crossing (Figure 2.26). However, before the culverts were emplaced considerable degradation had occurred. At the mouth, the channel depth is about 35 feet, and upstream of FM 2990, it is 22 feet. Further degradation into the shale bed is likely to occur upstream of the FM 2990 crossing. At the mouth, Pickle Creek is about 35 feet deep and this reduces to about 20 feet upstream (Figure 2.27). There is a concrete box culvert at the FM 1550 crossing that provides a measure of grade control for the upstream channel. The presence of a convexity in the bed profile in the downstream portion of the tributary suggests that there will be further degradation into the shale bed in the future.

In common with the other tributaries, the channel depth at the mouth of Brushy Creek is about 35 feet, and the depth reduces in the upstream direction to about 23 feet (Figure 2.28). Further degradation of the shale bed is likely in the future, but the upstream progression of the degradation is likely to be halted by the concrete slab and H-pile grade-control structure at the FM 1550 crossing. The depth of Bear Creek at the confluence with NSR is about 26 feet (Figure 2.29) and this reflects the lesser degree of incision in the mainstem (Figure

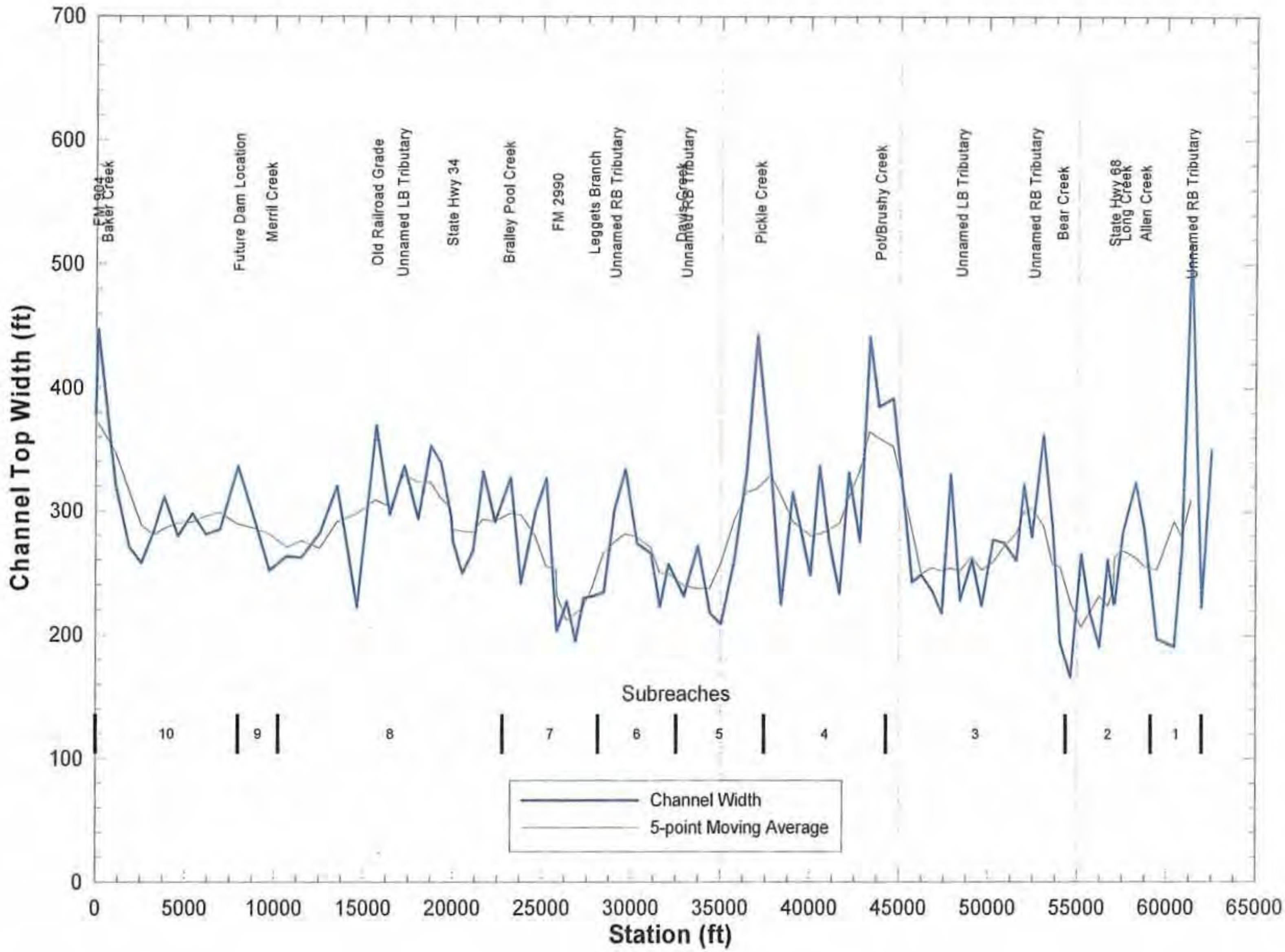


Figure 2.22. Plot of channel top widths along the North Sulphur River between FM 904 Bridge and upstream of Allen Creek.

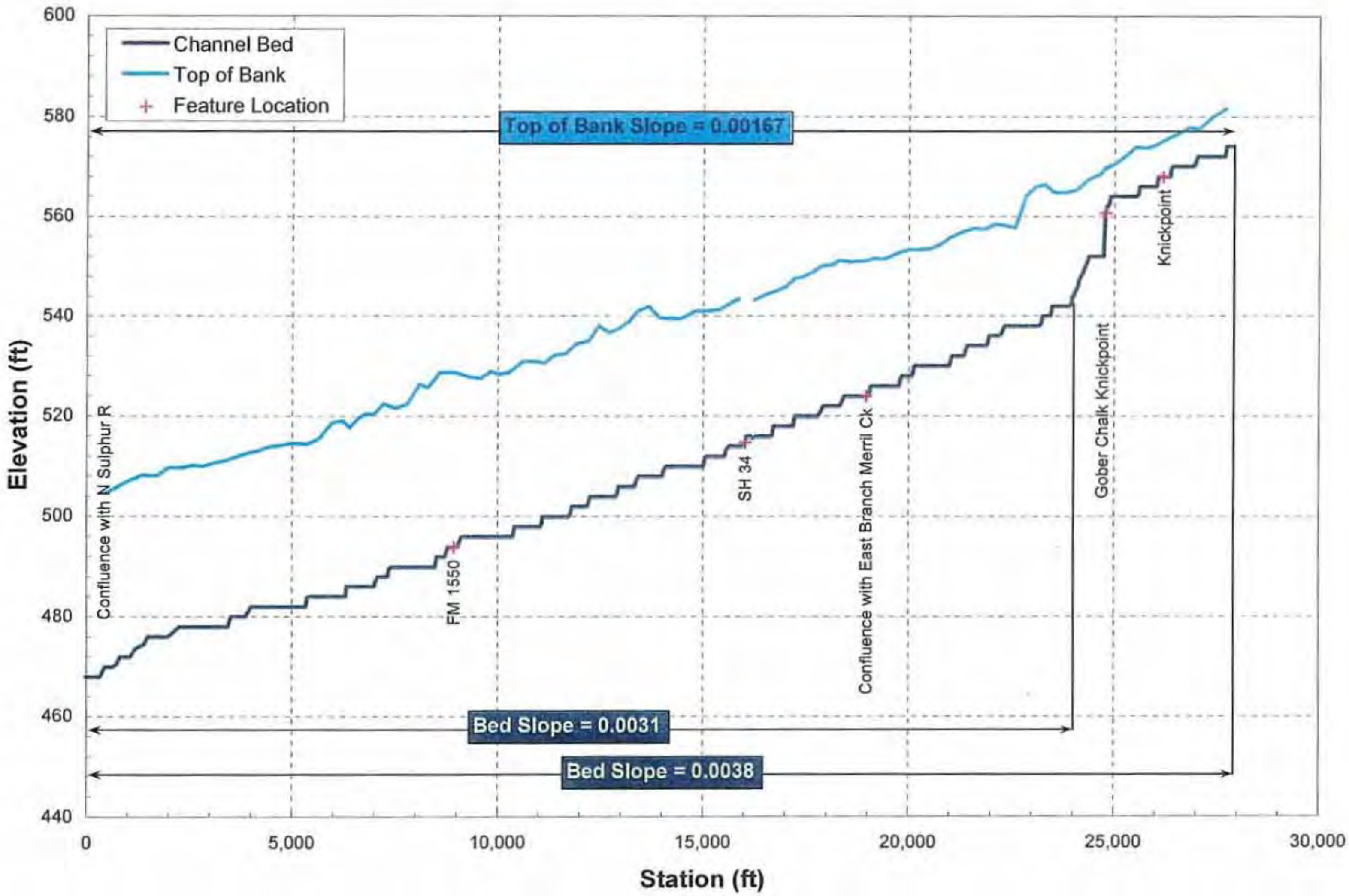


Figure 2.23. Longitudinal profiles of the bed and valley floor of Merrill Creek.

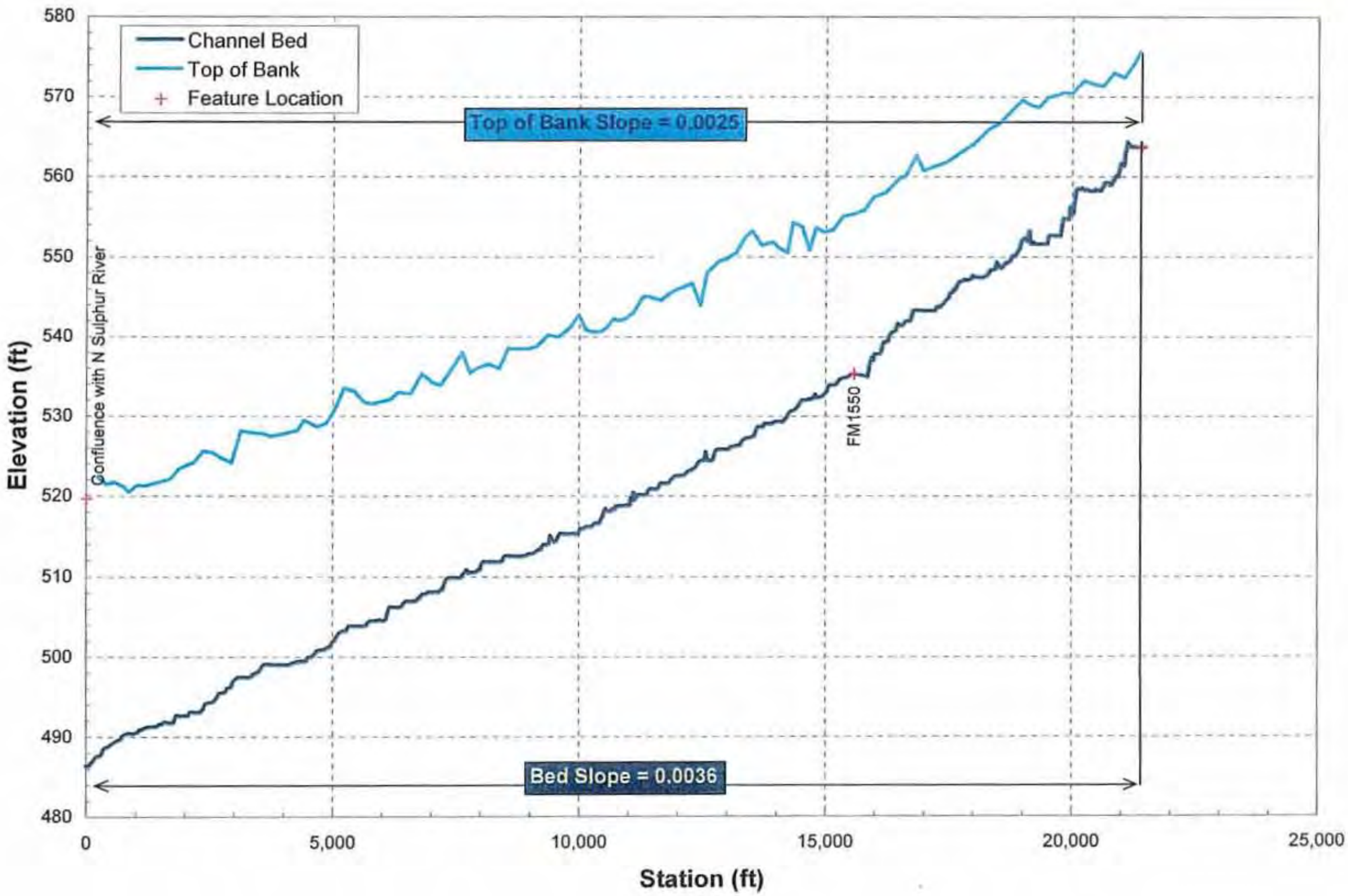


Figure 2.24. Longitudinal profiles of the bed and valley floor of Bralley Pool Creek.

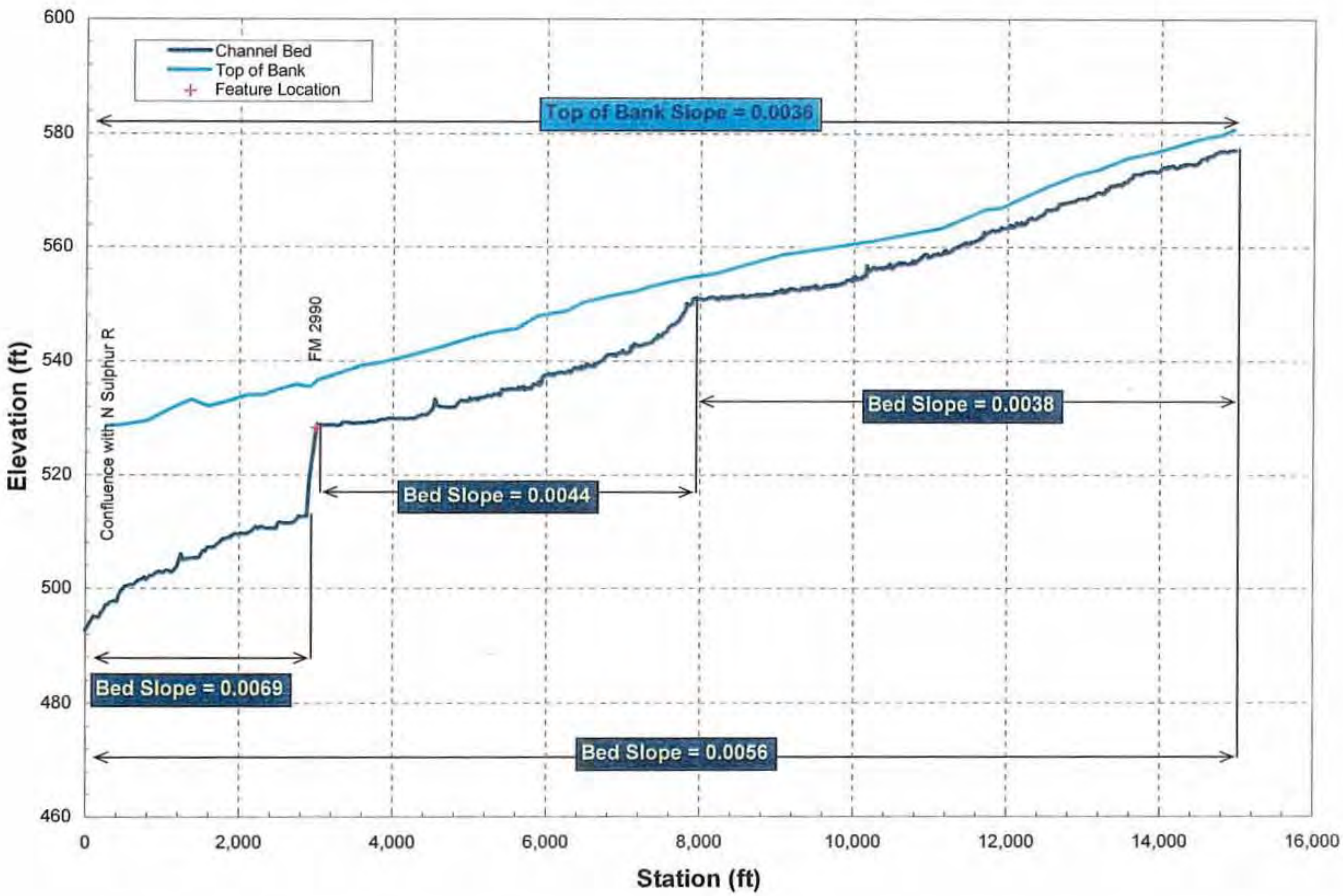


Figure 2.25. Longitudinal profiles of the bed and valley floor of Leggetts Branch.

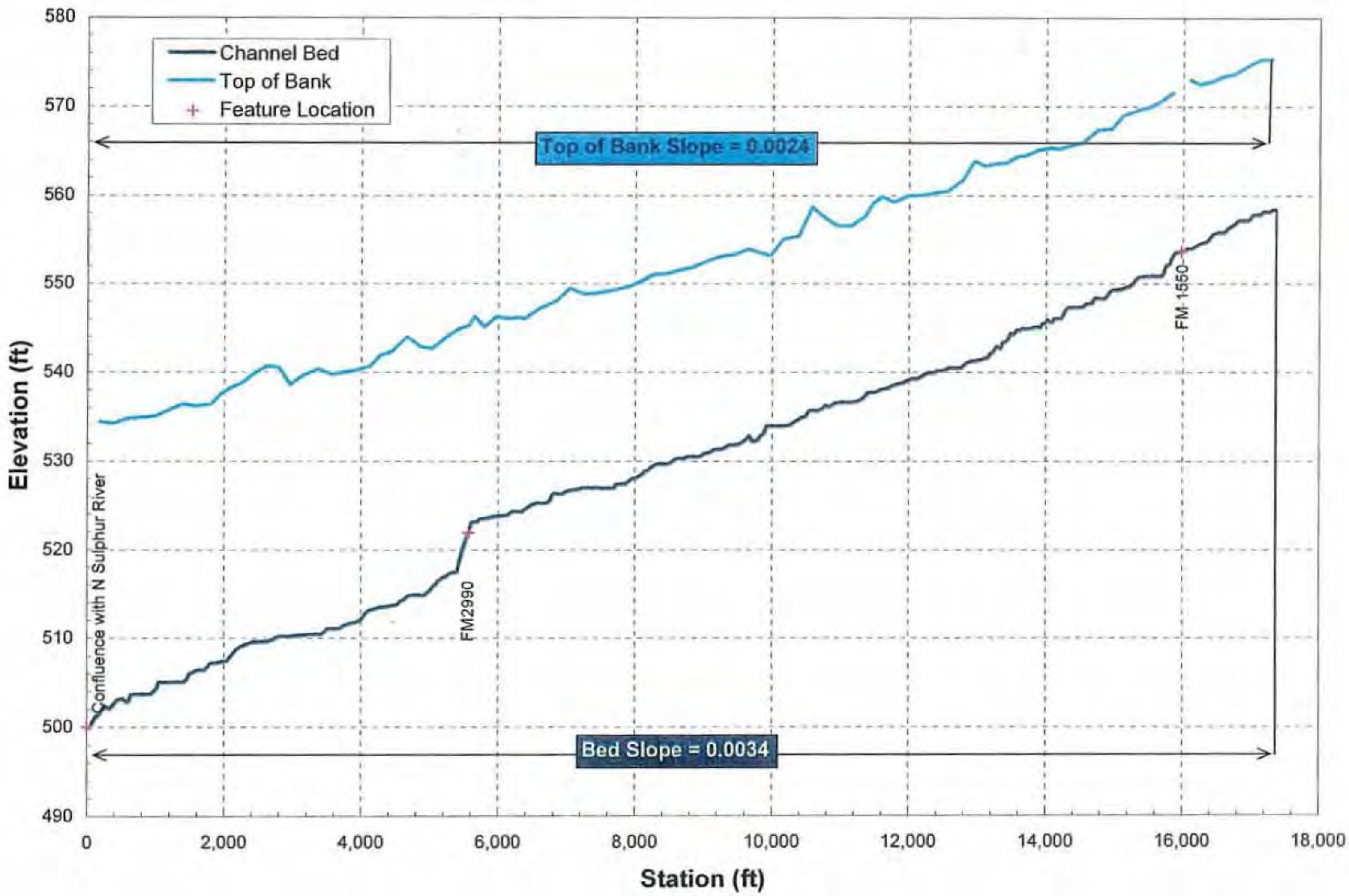


Figure 2.26. Longitudinal profiles of the bed and valley floor of Davis Creek.

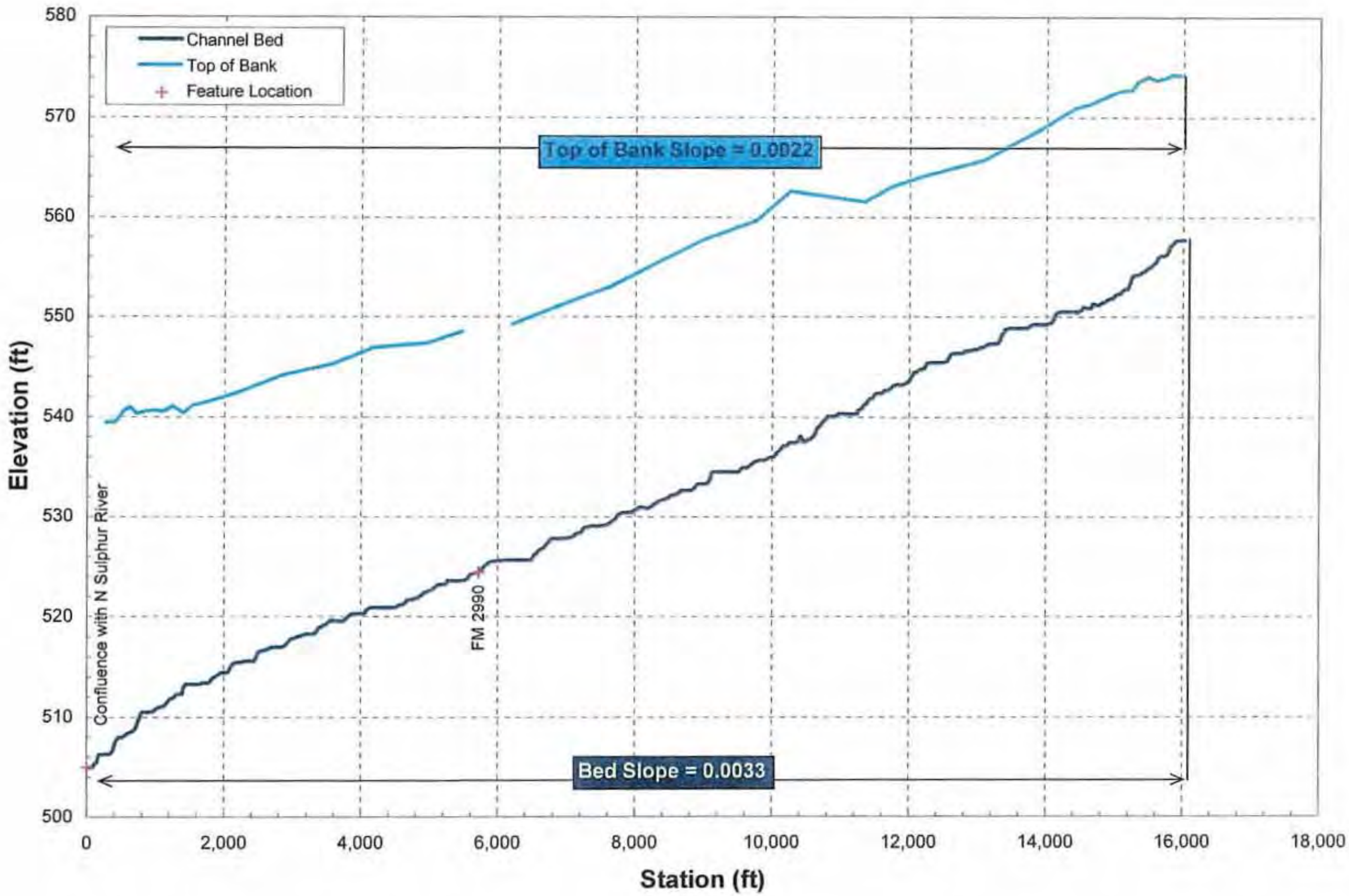


Figure 2.27. Longitudinal profiles of the bed and valley floor Pickle Creek.

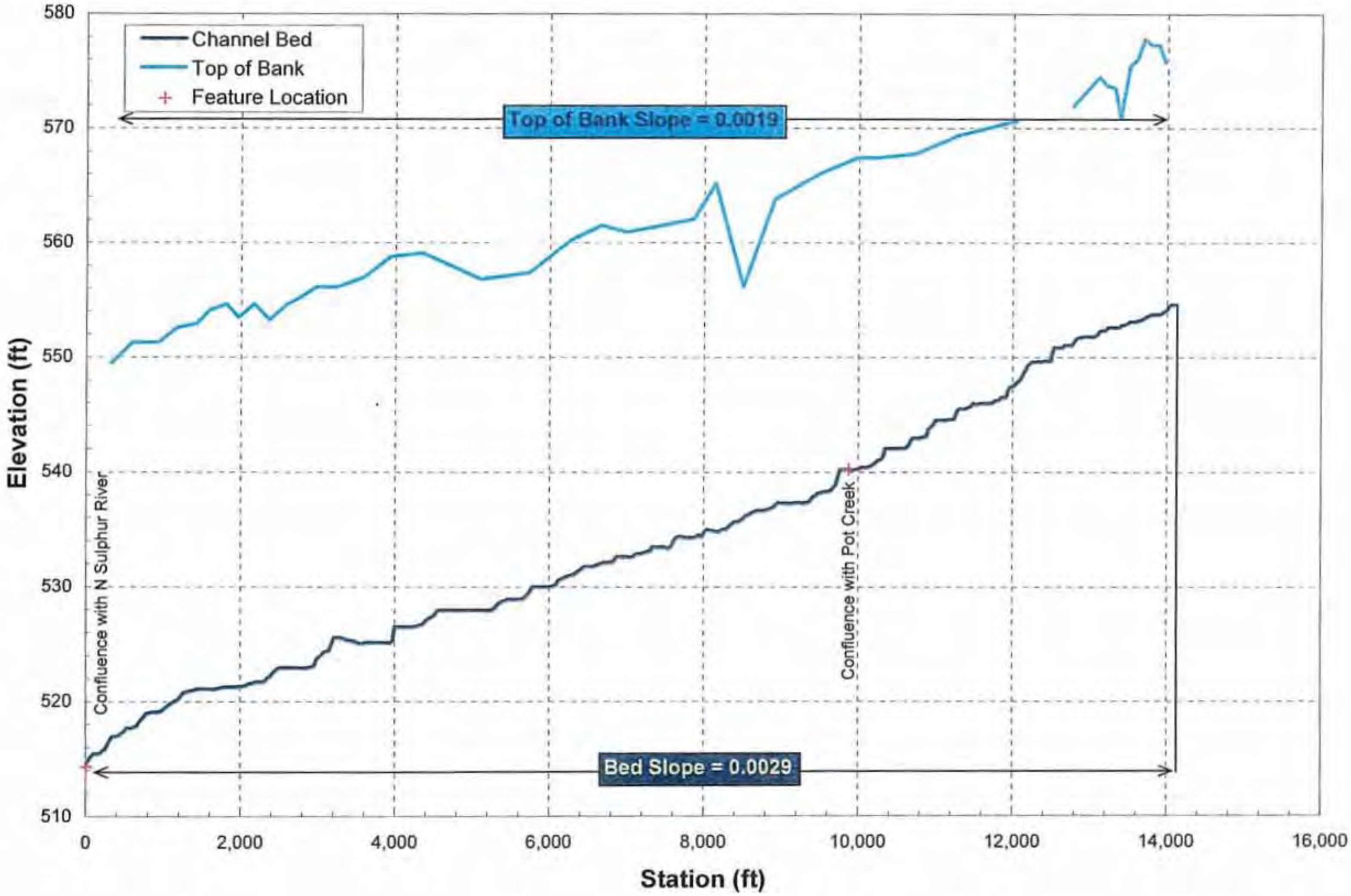


Figure 2.28. Longitudinal profiles of the bed and valley floor of Brushy Creek.

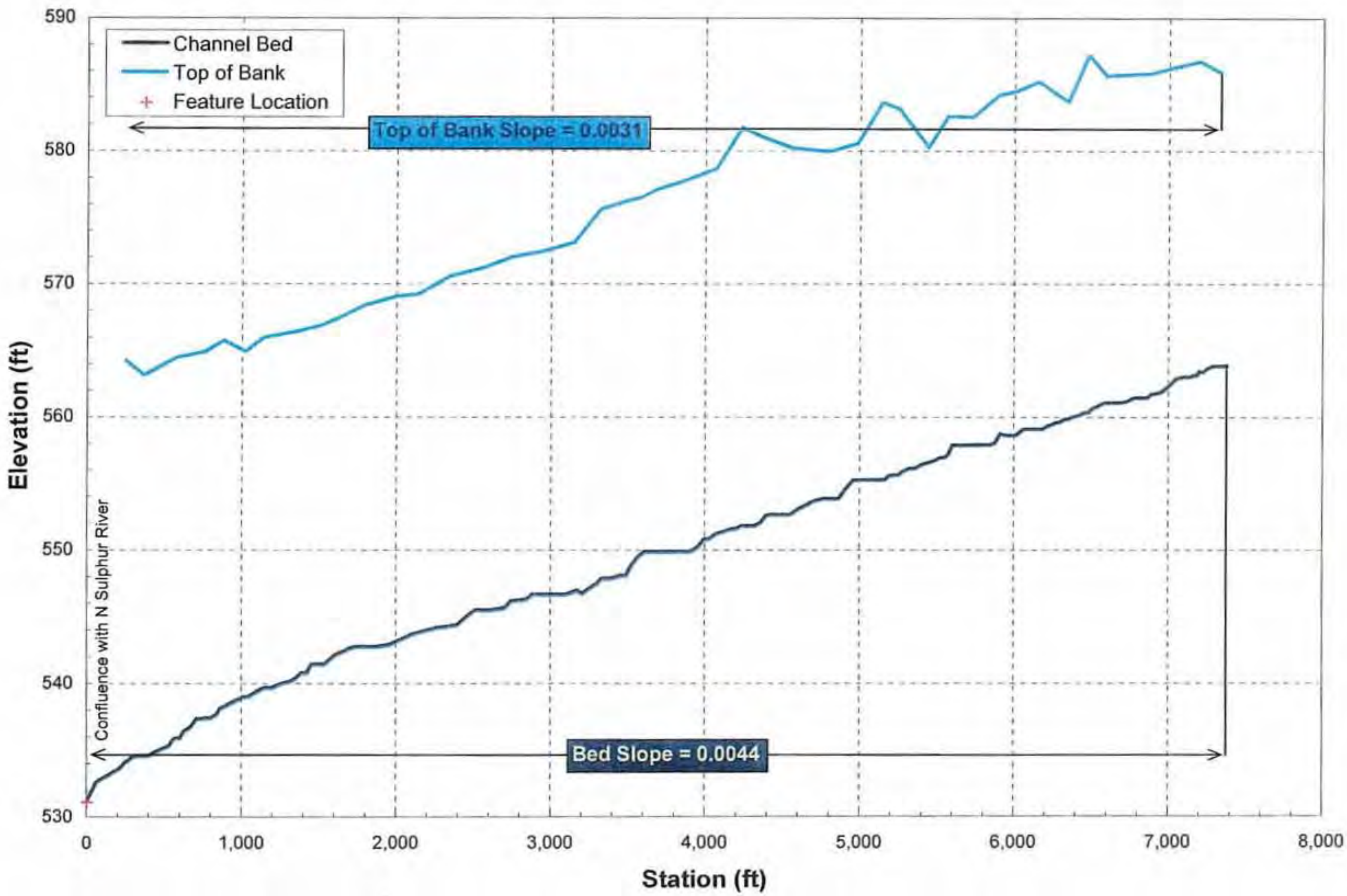


Figure 2.29. Longitudinal profiles of the bed and valley floor Bear Creek.

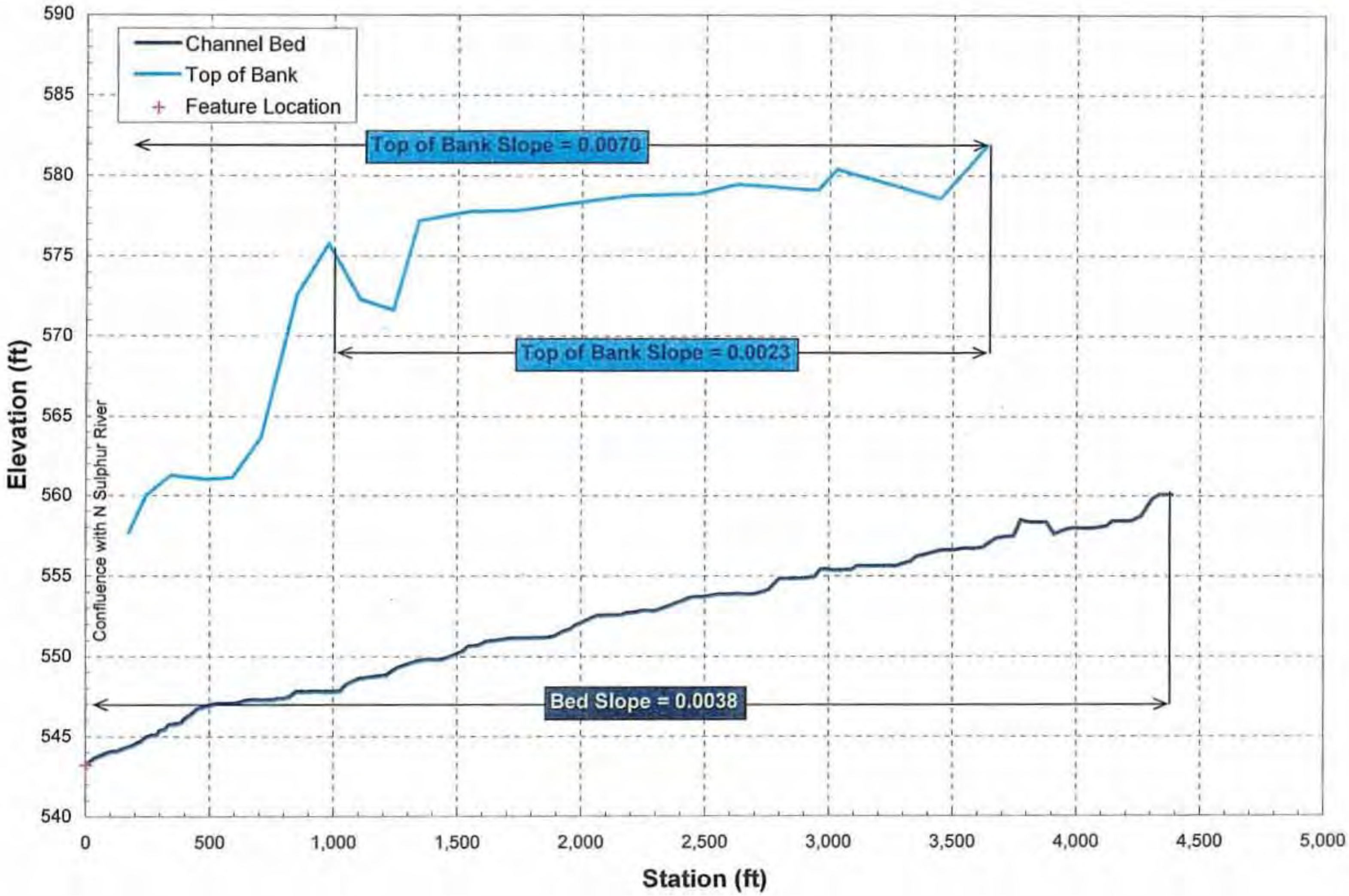


Figure 2.30: Longitudinal profiles of the bed and valley floor of Allen Creek.

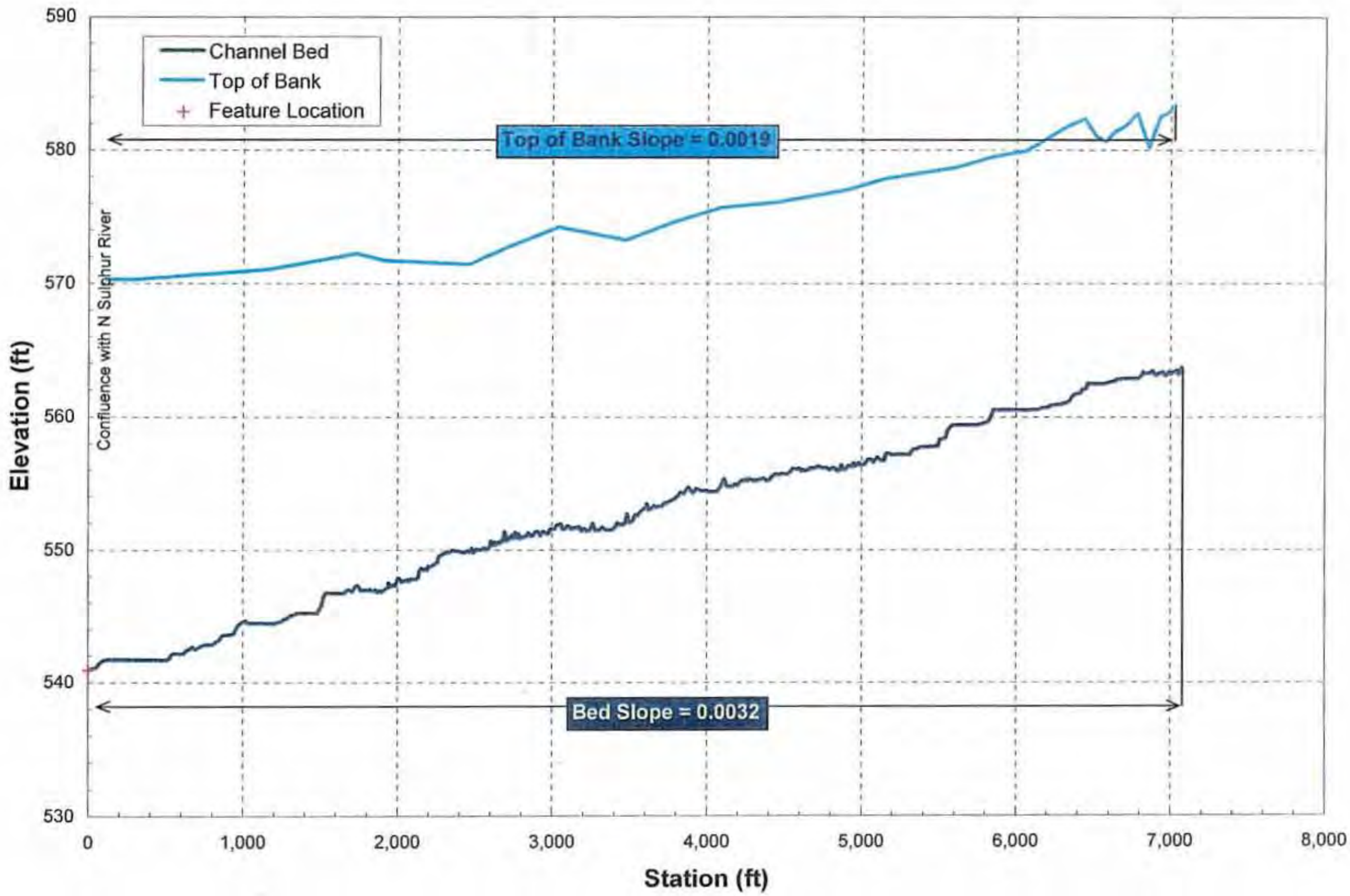


Figure 2.31. Longitudinal profiles of the bed and valley floor of Long Creek.

2.21). Based on the presence of a convexity in the bed profile in the lower reaches, it is highly likely that there will be further incision into the shale bed. At its mouth, Allen Creek is only 15 feet deep which reflects the local depth of the NSR, but the depth increases to about 25 feet farther upstream (Figure 2.30). As the NSR continues to degrade, Allen Creek will also degrade in the future. No grade controls were observed downstream of the Roxton/Gober Chalk outcrop (Figure 2.2) in this tributary. Long Creek is the largest tributary draining the south side of the NSR valley, and it has responded to the lowered baselevel in a similar fashion (Figure 2.31). At the mouth, the channel is about 30 feet deep and this reduces to about 20 feet farther upstream. SCS floodwater retarding structures have been built in the upper reaches of this channel.

In summary, all of the tributaries to the NSR have incised through the valley fill alluvium, and the bed and lower portions of the banks are composed of shale. The inevitable ongoing erosion of the shale in both the bed and banks is primarily the result of weathering processes, and the rate of erosion is governed by the number of wetting and drying cycles. Slab failure of the alluvial materials above the exposed shale does deliver alluvial sediment to the channels, but as failures progress, the upper bank angle becomes flatter, and therefore, more stable since the erosion of the shale toe occurs at a much lower rate. Consequently, through time, the sediment delivery from the alluvial fill declines and the major source of sediment is the weathering of the exposed shale in the bed and banks of the channel. In the lower reaches of the larger tributaries, deep-seated mass failures of the alluvial sediments increase the channel top with, but also bury the exposed shale in the toes of the banks and provide an appearance of stability in a similar manner to the mainstem. As shown in the NSRCEM (Figure 2.19), a mass failure of the alluvial field temporarily protects the exposed shale in the toe of the bank with cohesive and vegetated material, thereby accelerating the bed erosion. In time, the deepened channel causes lateral erosion of the mass-failed toe material and re-exposure of the shale.

2.2.4. Channel Incision Rates

Two sources of information were obtained to evaluate incision rates on the NSR and the tributaries, repeat surveys at bridges and stage-discharge data at the USGS gage at Cooper (USGS Gage No. 07343000). Bridge profiles were obtained by CP&Y for State Highway 34, FM 2990 and FM 904 on the NSR, State Highway 34 and FM 1550 on Merrill Creek, FM 1550 on Bralley Pool Creek, and FM 1550 on Baker Creek. The USGS 9207 summary gaging forms that provide measured stage-discharge data for a range of flows were also obtained for the period from 1950 to the present.

Bridge profiles for the FM 2990 crossing of the NSR (**Figure 2.32**) and the State Highway crossing of Merrill Creek (**Figure 2.33**) provide good examples of historic incision at these structures. Between 1967 and 1985, the bed of the NSR at FM 2990 degraded by about 5 feet at a rate of about 3.3 in./yr (**Figure A.40**). In the following 17 years (1985-2002), there was little if any degradation based on a comparison of the 1985 profile and the cross section derived from the 2002 DTM. However, since the nominal accuracy of the DTM is 1 foot (one-half contour interval), it is possible that there has been up to 1 foot of erosion at this bridge (0.7 in./yr). Review of the 1969 aerial photography (1:20,000), suggests that shale was present in the bed in 1969, and therefore, the erosion rates of up to 3.3 in./yr are consistent with measured rates in shale in other channels in northeast Texas (Allen et al., 2002).

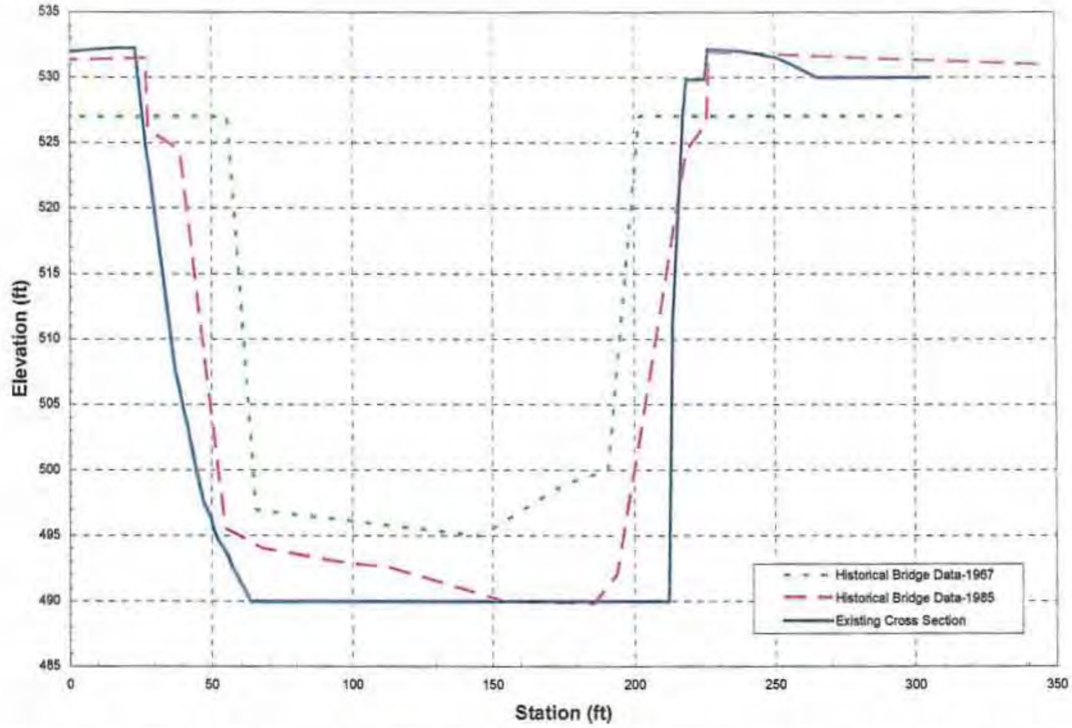


Figure 2.32. Bridge cross-section profiles for the FM 2990 crossing of the North Sulphur River.

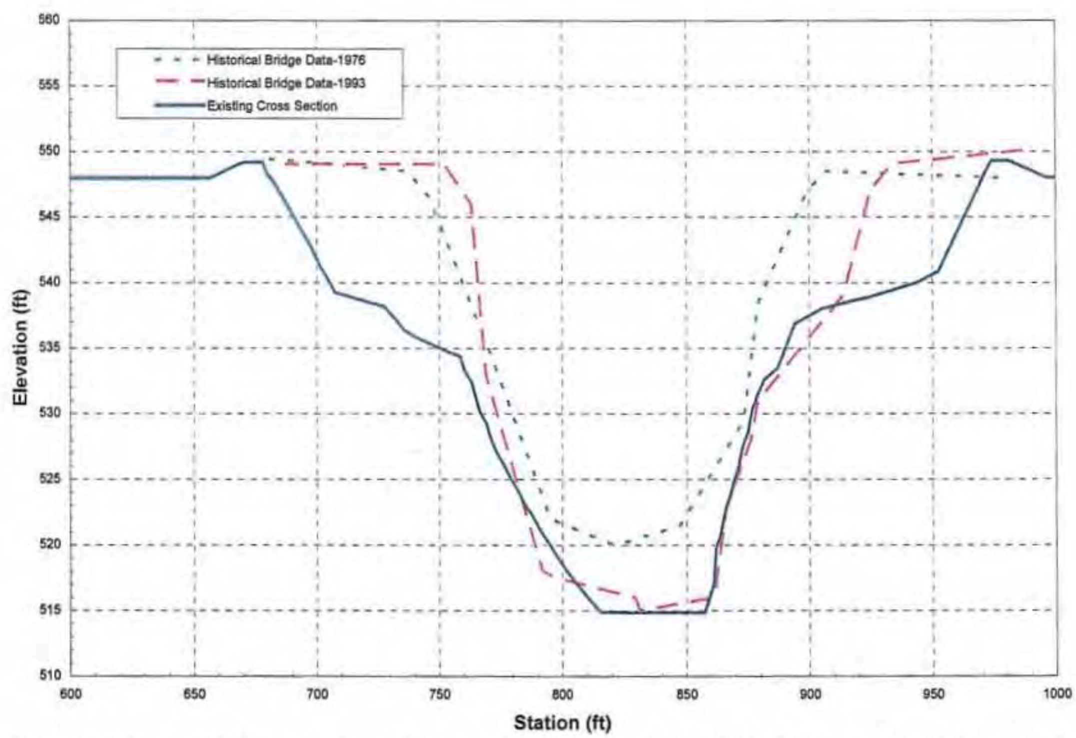


Figure 2.33. Bridge cross-section profiles for the FM 1550 crossing of Merrill Creek.

The bridge profiles at the State Highway 34 crossing of Merrill Creek (Figure 2.33) indicate that Merrill Creek degraded by about 5 feet between 1976 and 1993 at a rate of about 3.5 in./yr. Between 1993 and 2002 there appears to have been little erosion, but it could have been as high as 1.3 in./yr if it is assumed that 1 foot of degradation took place. Both values are consistent with reported values of erosion into the shale (Allen et al., 2002). **Figure 2.34** summarizes the bridge survey data for the seven bridges investigated. With the exception of the FM 1550 bridge at Merrill Creek, rates of incision into the shale average 2 to 3 in./yr, which is very consistent with measured rates in other channels in northeast Texas (Allen et al., 2002). These rates of incision can be expected to occur in the future for as long as the shale is exposed to weathering and slaking processes.

Stage-discharge rating curves were developed from the USGS 9207 summary gaging forms for the Cooper gage for seven periods between 1950 and the present (**Figure 2.35**). The data show that the channel at the gage aggraded between 1950 and 1979, which is consistent with a much higher sediment load from the upstream eroding channels of the NSR and the tributaries (Schumm et al., 1984; Harvey and Watson, 1986; Watson et al., 1986; Simon, 1989). Review of the 1979 aerial photography (1:40,000) suggested that there were a large number of depositional bars on the bed of the NSR at that time. Analysis of the stage-discharge rating curves for flows below 1,000 cfs from 1971 to the present (**Figure 2.36**) indicate that the channel began to degrade after 1985. Degradation rates were about 1.5 in/yr between 1986 and 1993, 1.4 in./yr between 1993 and 1999, and 1 in./yr between 2000 and the present (2005). The bed of the river at the gage is composed of shale (**Figure A.41**), and it is reasonable to conclude that at least the 2000–2005 degradation represents erosion of the shale. Degradation at the gage since 1985 is consistent with a reduced sediment supply due to channel evolution upstream, and the general observation from both helicopter and ground reconnaissance, that there is little sediment stored in the bed of the NSR from its confluence with the South Sulphur River to the headwaters. The bed of the NSR from the confluence with the South Sulphur River to the headwaters downstream of the Roxton/Gober Chalk outcrop is primarily composed of shale with a veneer of alluvial sediment at some locations, generally the mouths of larger tributaries.

2.2.5. Sediment Sources and Bed-material Gradations

Sediment delivery to the NSR is from both watershed and channel erosion sources. Because most of the soils in the watershed are clays and clay loams (NRCS, 2001), the bulk of the sediment supplied to the channels is in the form of wash load that contributes little to channel processes, but is an important component of the annual sediment load. Channel sources include slab (Figure A.13) and slump (**Figure A.42**) failures of the valley fill alluvium, and a variety of shale-related sources. Plucking of the shale in both the bed (Figure A.7) and the banks (**Figure A.43**) produces gravel-cobble sized shale clasts (Figures A.11 and A.12) that are initially transported as bed material. In situ weathering of the shale in the bed tends to produce gravel- and finer-sized clasts (Figure A.10) that are readily transported at the onset of flow in the channel, and probably contribute to the very high initial sediment concentrations (100,000 ppm) reported for similar channels in northeast Texas (Allen et al., 2002). Weathering (**Figure A.44**) and mass failure (Figure A.8) of shale exposed in the banks also produces gravel- and cobble-sized shale clasts that are initially transported as bed material, but eventually slake (Figure A.6) and are transported as part of the wash load. Most of the larger, non-shale, clasts observed in the channel are derived from sandstone and limestone stringers exposed by erosion of the shale (**Figure A.45**), or from poorly cemented, weathered gravels interbedded in the exposed shale (**Figure A.46**). Low-density chalk sands and gravels are derived from the Roxton/

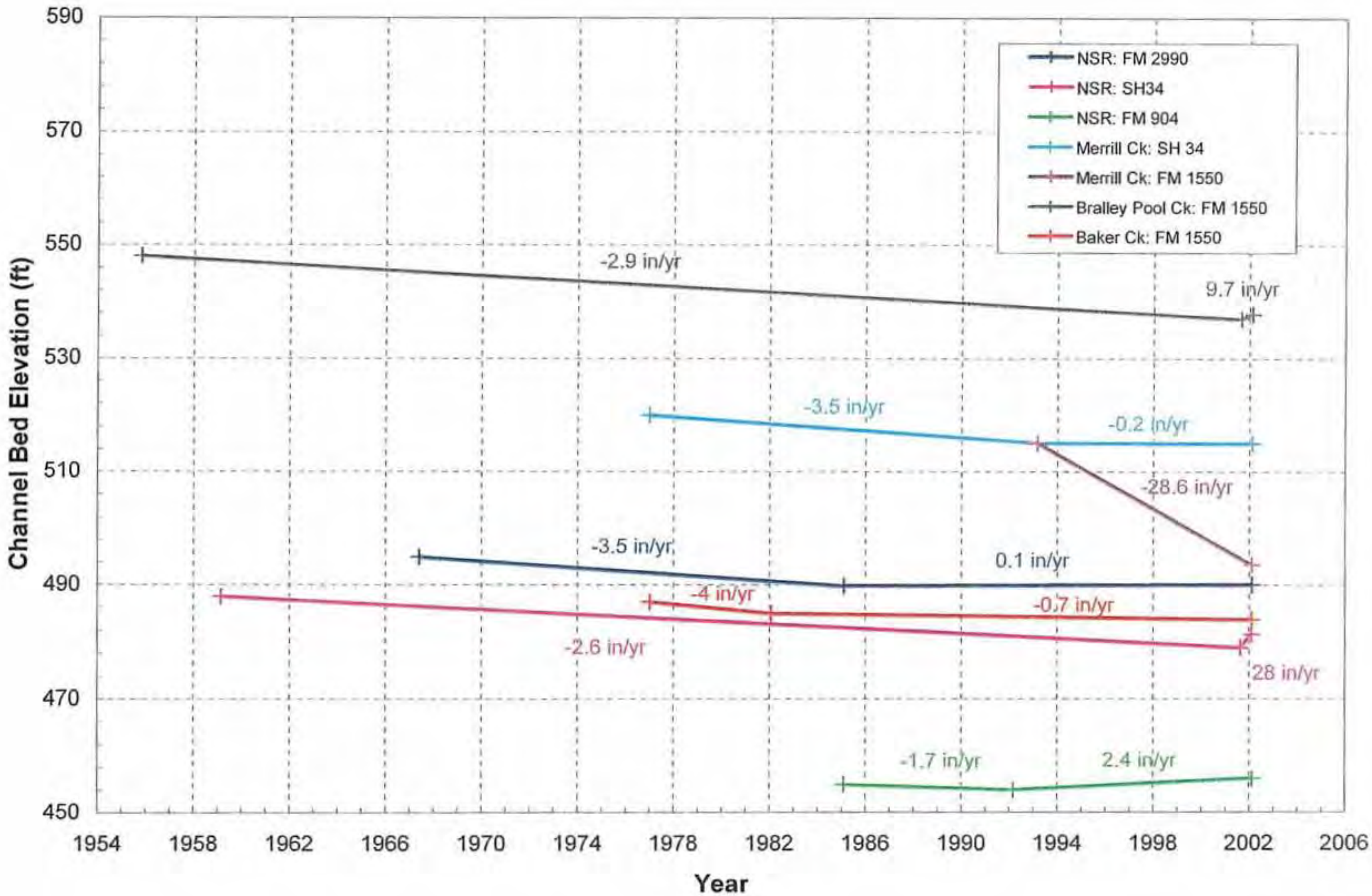


Figure 2.34. Bed degradation rates at the seven evaluated bridges in the upper North Sulphur River basin.

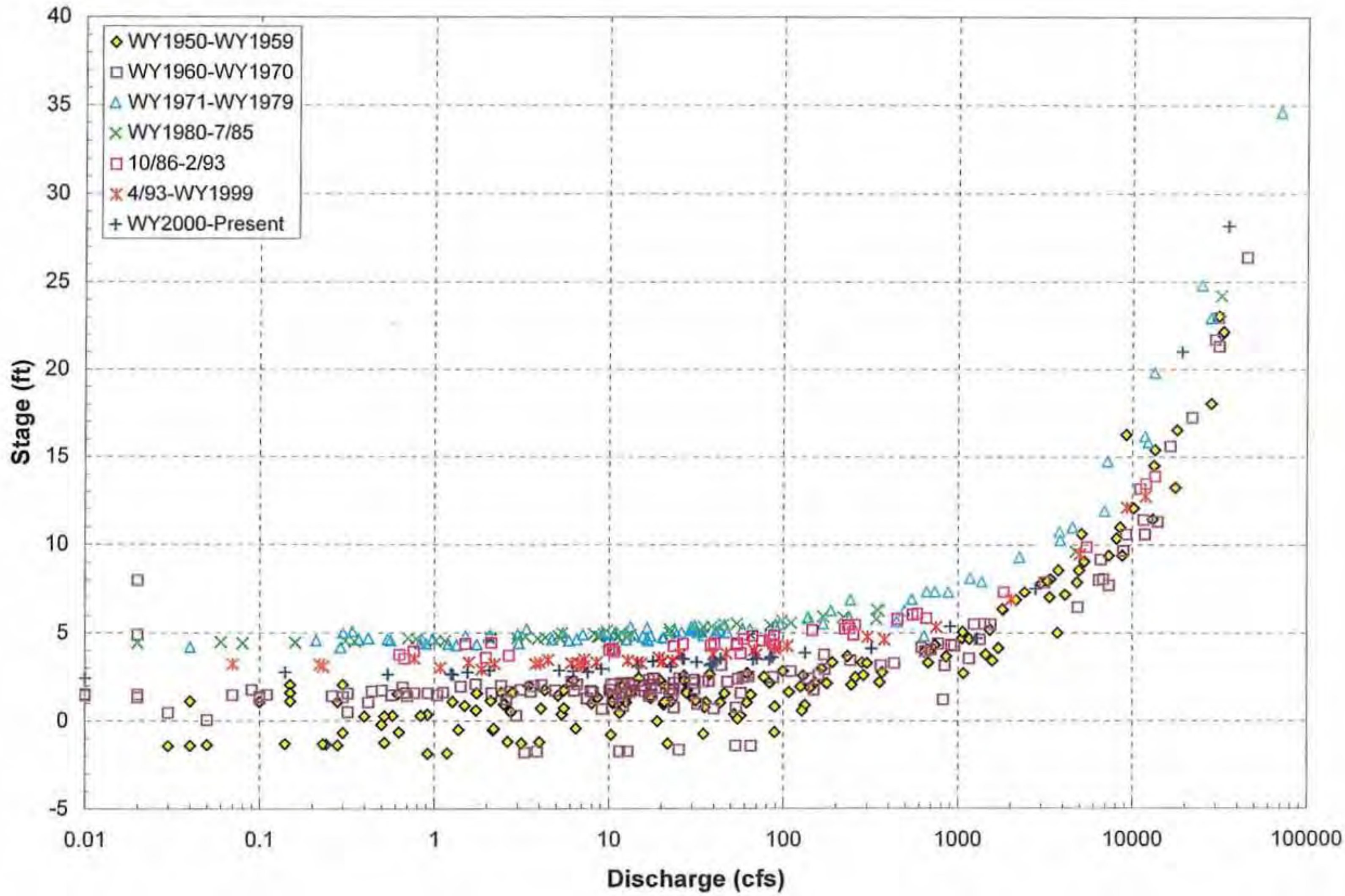


Figure 2.35. Stage-discharge rating curves from 1950 to 2005 for the USGS gage on the North Sulphur River, near Cooper, Texas (USGS Gage No. 07343000).

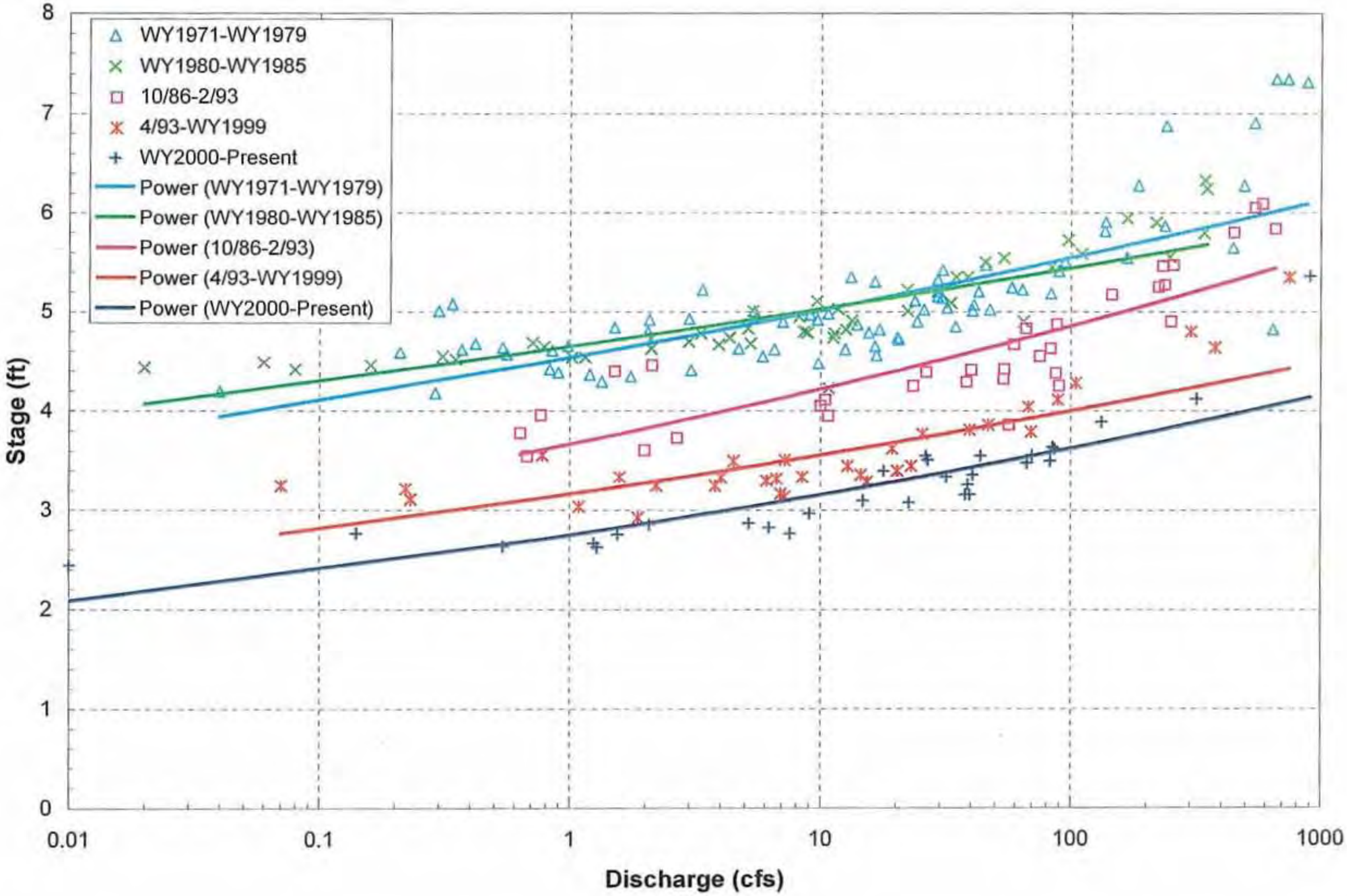


Figure 2.36. Stage-discharge rating curves from 1971 to 2005 for the USGS gage on the North Sulphur River, near Cooper, Texas (USGS Gage No. 07343000).

Gober Chalk (Figure A.4). Shale clasts in the coarse sand to fine gravel range are transported as bed material in dune-like features (**Figure A.47**).

During the two field visits to the NSR and its tributaries a number of bed-material samples were collected (**Figure 2.37**). Field observations indicated that in the upstream areas of the NSR as well as the tributaries, the bed material was predominantly composed of shale clasts (**Figure A.48**). Farther downstream at the Bralley Pool Creek confluence, the bed material contains less shale pieces and more non-shale material as a result of downstream transport, weathering and slaking of the shale clasts (**Figure A.49**). At the FM 904 bridge, the bed material is primarily composed of non-shale clasts (**Figure A.50**).

Samples collected in the NSR and the tributaries were provided to the Kleinfelder soils laboratory in McKinney, Texas. Dry and slaked gradations were developed for each of the samples (Appendix B). Dry (dry-sieved field samples) gradations for the NSR bed-material samples are shown on **Figure 2.38**. The median (D_{50}) sizes of the samples range from 1.7 to 3.7 mm (coarse sand to fine gravel), and the D_{84} sizes range from 4.5 to 13.2 mm (fine to medium gravel). Silt-clay contents (<0.075 mm) are less then 5 percent of the samples. Wet (slaked) gradations for the same bed-material samples are shown on **Figure 2.39**. Median sizes range from <0.075 to 2.0 mm and the D_{84} sizes range from <0.075 to 4.7 mm. Silt-clay contents range from 10 to 90 percent of the samples. Comparison of the dry and wet gradations for sample NSR4 (**Figure 2.40**) demonstrates the effects of slaking on the size distribution of the materials available for transport, and confirms the necessity of taking the transformation of bed material into bed material and wash load into account in any computation of sediment transport. The dry D_{50} is 2.7 mm (fine gravel), the wet D_{50} is 0.7 mm (coarse sand) and the silt-clay content increases from 4 to 45 percent.

Dry- and wet-sieved gradation curves for the tributary samples (Baker, Merrill, and Bralley Pool Creeks) are shown in **Figure 2.41**. Dry-sieved D_{50} values range from 2.5 to 3.7 mm, and wet-sieved values range from <0.075 to 1.5 mm. Silt-clay contents for the dry-sieved samples are about 3 percent, and range from 24 to 66 percent for the wet-sieved samples.

The transformation of the slaked bed material from silt-clay dominated in the upstream reaches to non-shale sand-sized material in the downstream reaches is summarized in **Figure 2.42**. In the upstream reach (NSR2) the silt-clay content is greater than 80 percent. At the Bralley Pool confluence (NSR0), the silt-clay content reduces to about 30 percent, at the FM 904 bridge (NSR8) it is about 12 percent, and at the USGS gage near Cooper (NSR1), it is reduced to about 10 percent. Conversely, the non-shale component varies from less than 20 percent upstream to about 90 percent downstream.

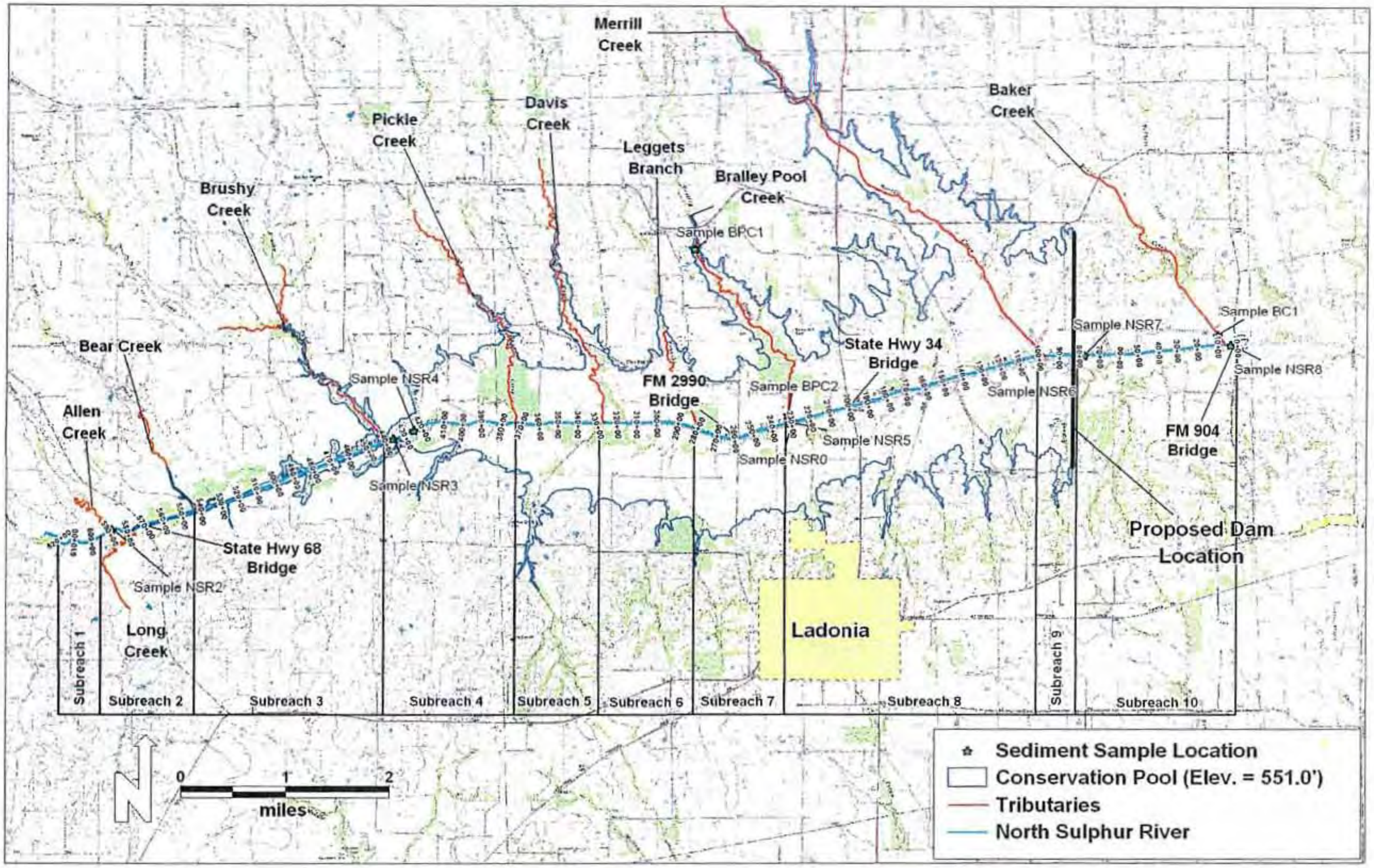
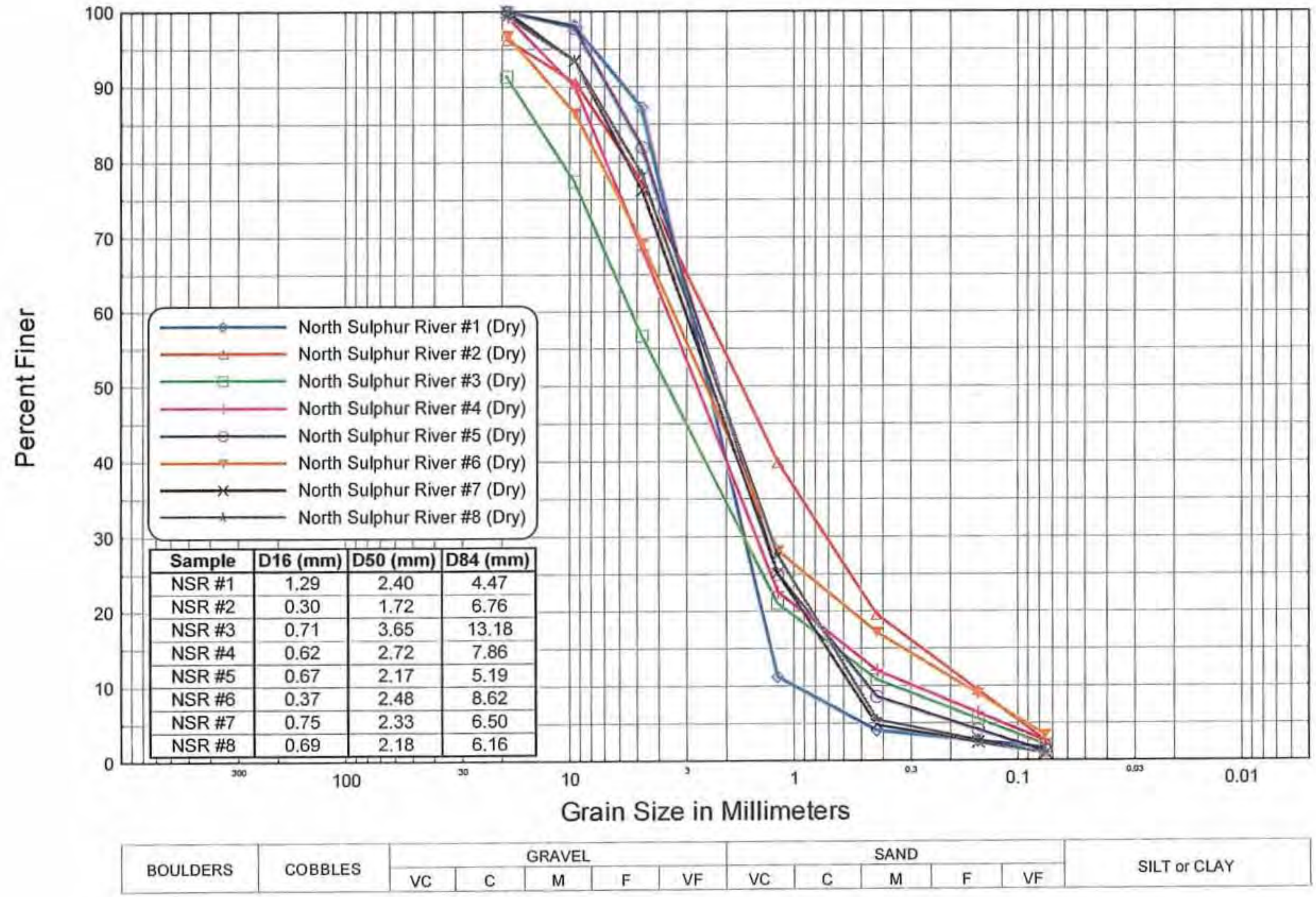
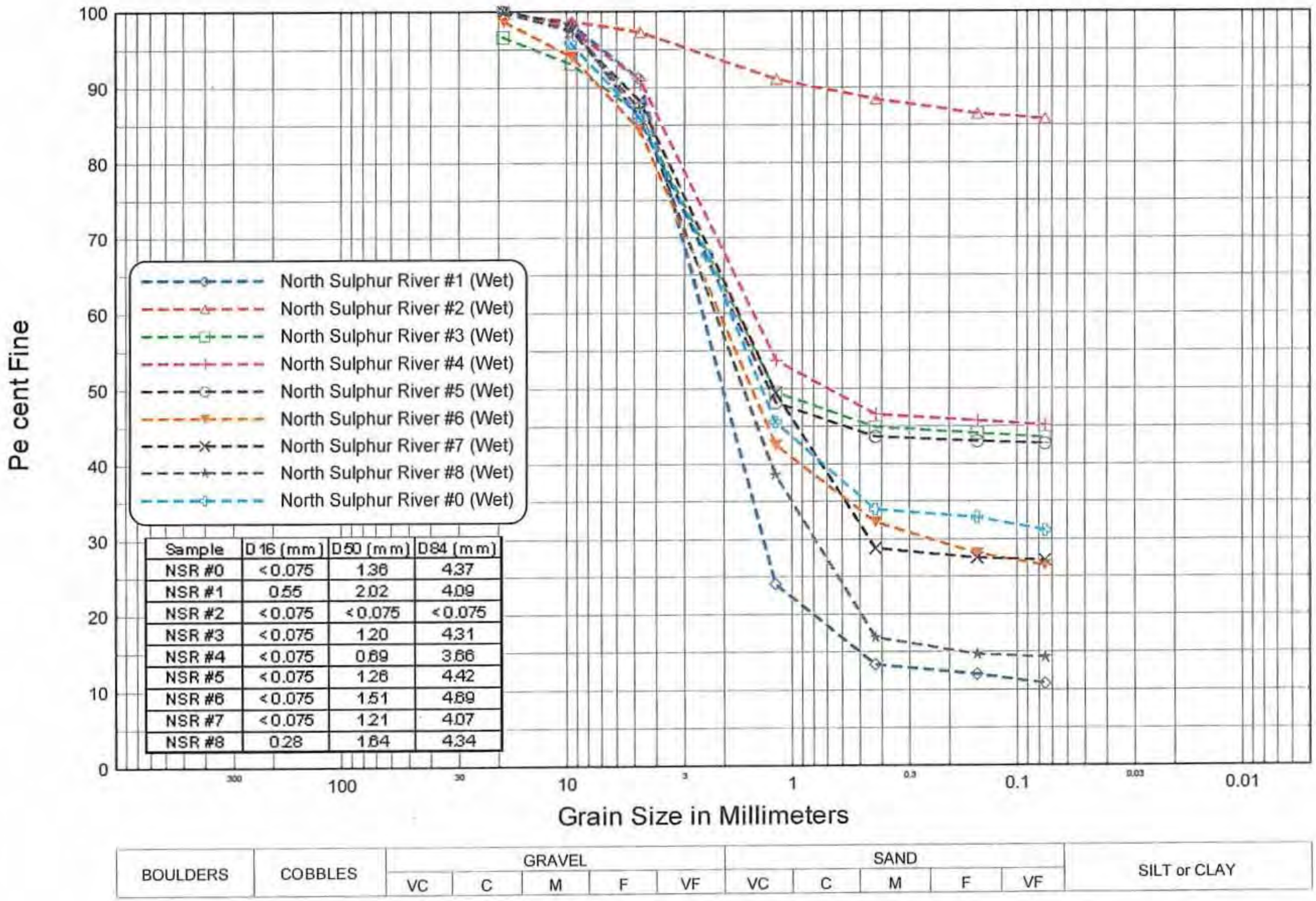


Figure 2.37. Map showing the locations of the bed-material samples in the North Sulphur River and the tributaries.



Sample	D16 (mm)	D50 (mm)	D84 (mm)
NSR #1	1.29	2.40	4.47
NSR #2	0.30	1.72	6.76
NSR #3	0.71	3.65	13.18
NSR #4	0.62	2.72	7.86
NSR #5	0.67	2.17	5.19
NSR #6	0.37	2.48	8.62
NSR #7	0.75	2.33	6.50
NSR #8	0.69	2.18	6.16

Figure 2.38. Dry-sieved gradation curves for the bed-material samples collected in the North Sulphur River.



Sample	D16 (mm)	D50 (mm)	D84 (mm)
NSR #0	<0.075	1.36	4.37
NSR #1	0.55	2.02	4.09
NSR #2	<0.075	<0.075	<0.075
NSR #3	<0.075	1.20	4.31
NSR #4	<0.075	0.69	3.66
NSR #5	<0.075	1.26	4.42
NSR #6	<0.075	1.51	4.69
NSR #7	<0.075	1.21	4.07
NSR #8	0.28	1.64	4.34

Figure 2.39. Wet-sieved gradations for the bed-material samples collected in the North Sulphur River.

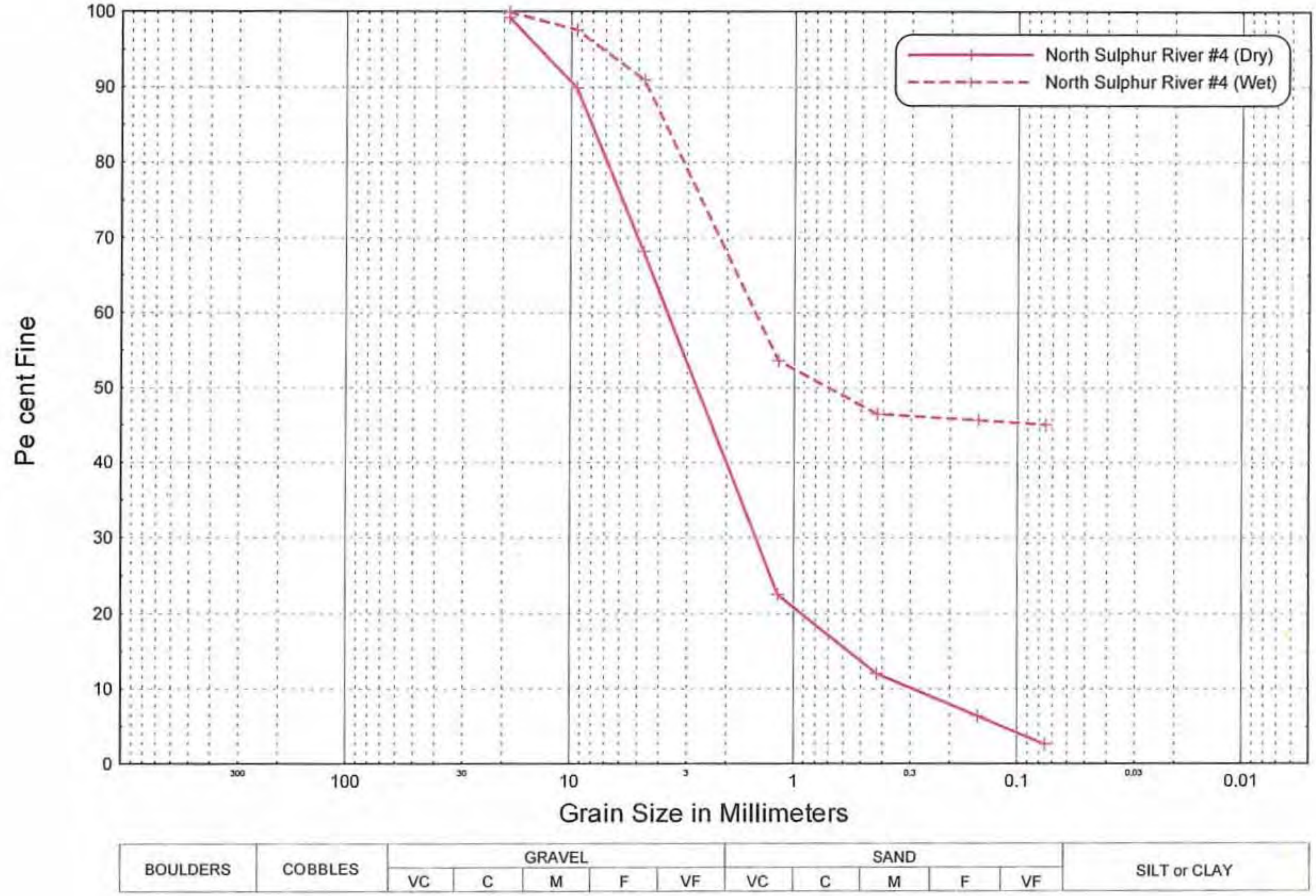


Figure 2.40. Comparison of dry- and wet-sieved gradations for sample NSR4 located about 1,500 feet downstream of the Brushy Creek confluence.

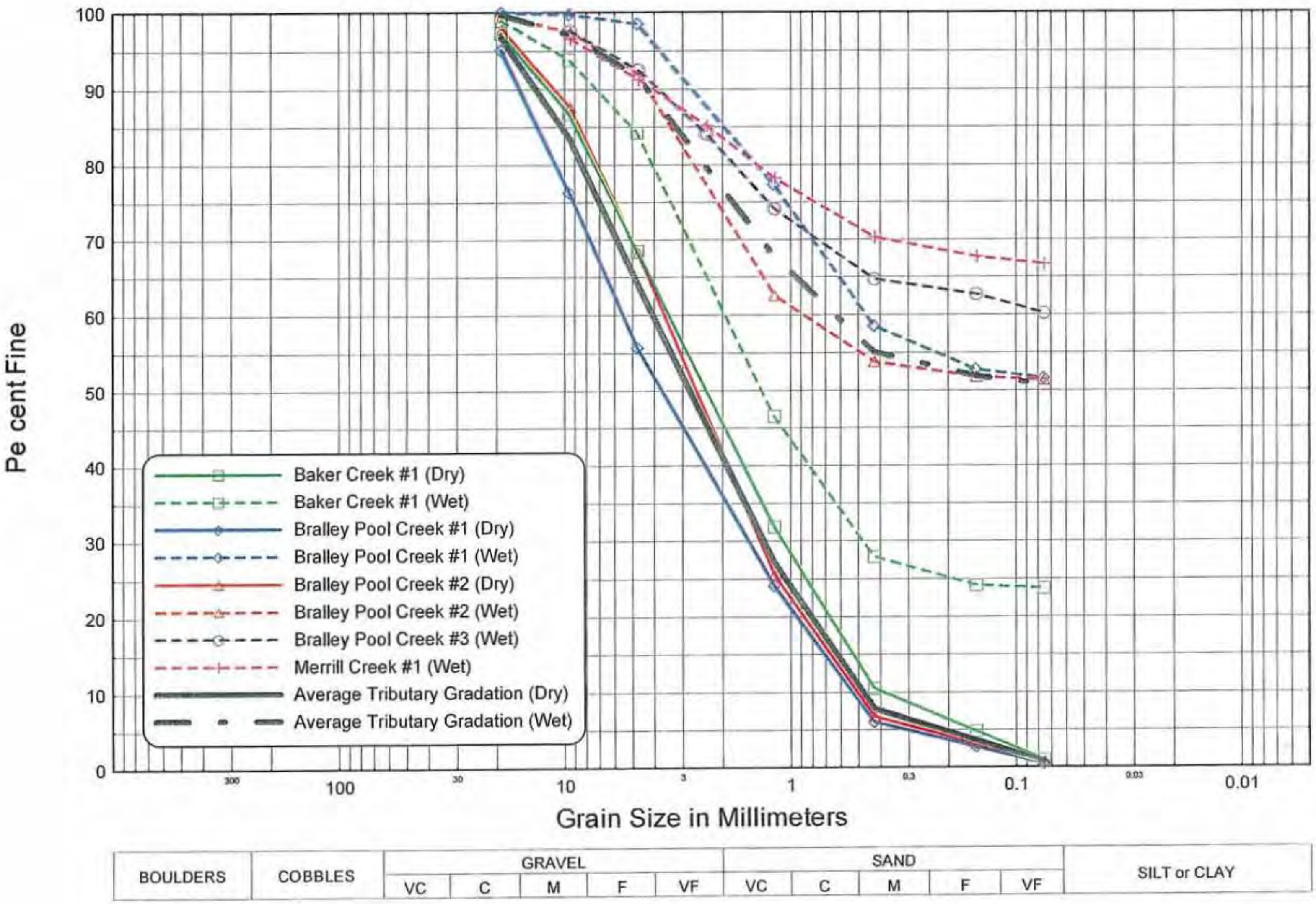


Figure 2.41. Dry- and wet-sieved gradation curves for the tributary samples.

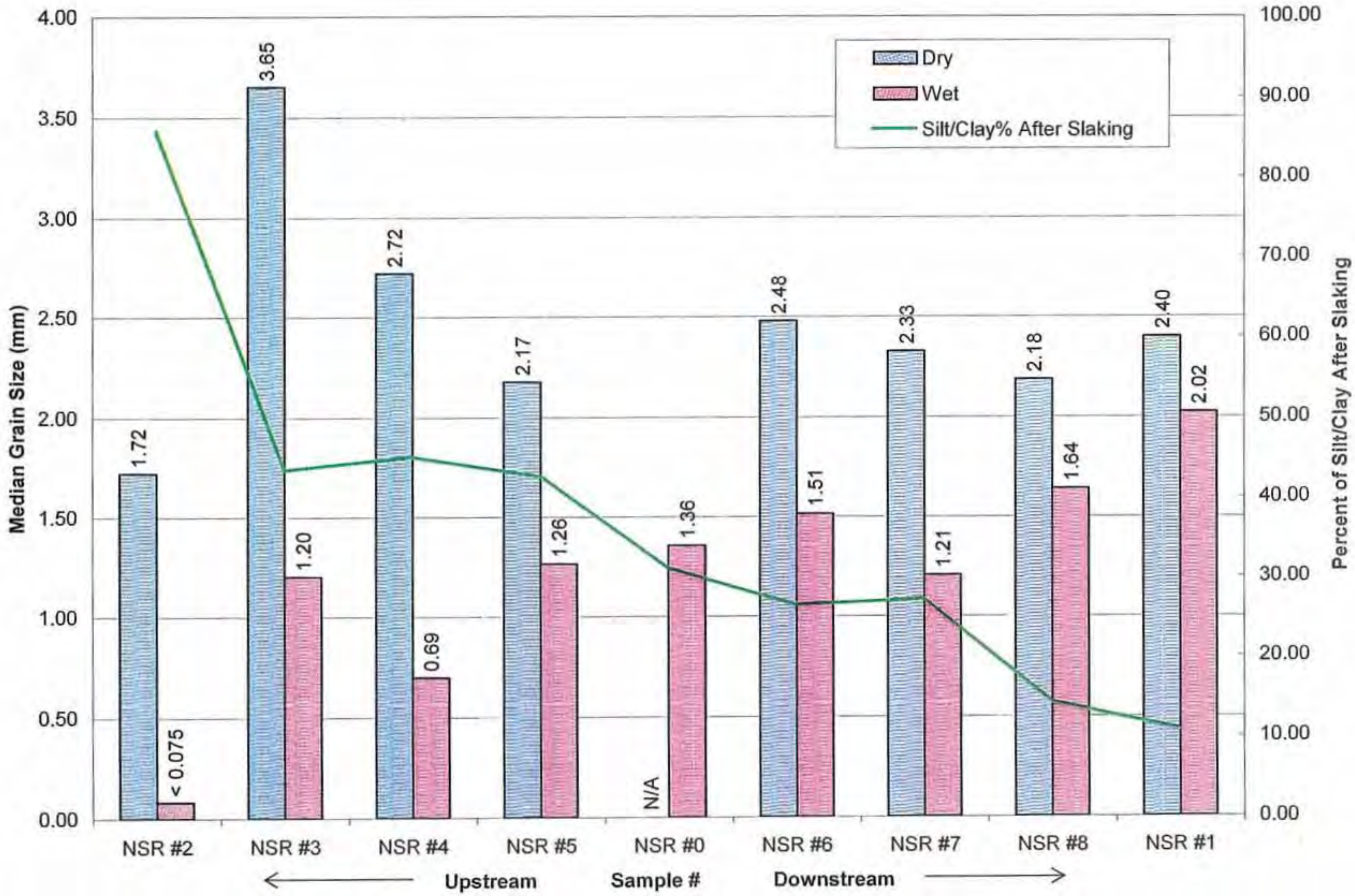


Figure 2.42. Changes in the median sizes of the dry- and wet-sieved bed-material samples of the North Sulphur River as well as the silt-clay content of the samples following slaking arranged from upstream to downstream.

3. HYDROLOGY

An evaluation of the hydrologic data and information in the vicinity of the proposed Lake Ralph Hall project was conducted to evaluate the existing hydrologic conditions in the NSR watershed. The evaluation included a review of the measured flow data, regional regression relationships, and previously-developed hydrologic models (HEC-1; RJ Brandes Co., 2004), development of revised hydrologic models (HEC-1), and an analysis of the peak flood frequencies and flow durations. The results of the hydrologic analysis were used to conduct the hydraulic and sediment-transport analyses.

3.1. USGS Gage near Cooper, Texas

Measured flow data were obtained for the USGS North Sulphur River near Cooper, Texas, gage (USGS Gage No. 07343000), which is located about 19.5 river miles downstream from the proposed dam site and has a drainage area of about 276 square miles. Available data at the gage include mean daily flow and peak flood data that extend from Water Year (WY) 1950 to WY2004. Mean daily flow-duration and peak flood-frequency analyses were performed at the gage to provide a basis of reference for the hydrologic analysis at the proposed dam location.

3.1.1. Annual Flow Volume and Mean Daily Flow-duration Analysis

Annual water volumes were computed for the period of record using the measured mean daily flows (**Figure 3.1**), and indicate that the annual volume ranges from 25,200 to 397,000 ac-ft, with an average volume of about 191,000 ac-ft. The mean daily flow-duration curve (**Figure 3.2**) indicates that the median flow is 12 cfs, that the flow exceeds 1.0 cfs 75 percent of the time, exceeds 316 cfs 10 percent of the time, and exceeds 5,830 cfs about 1 percent of the time.

3.1.2. Peak Flood Frequency Analysis

Measured flood peaks during the gage period of record have ranged from 5,600 cfs in 1996 to 90,600 cfs in 1972 (**Figure 3.3**). Using these flood peaks, a flood-frequency curve was developed using the U.S. Army Corps of Engineers HEC-FFA computer program (USACE, 1992), which is based on the procedures outlined in Water Resource Council (WRC) Bulletin 17B (WRC, 1981) with a generalized skew coefficient of -0.28 (**Figure 3.4**). At the Cooper gage, the computed frequency curve indicates that the 2-year peak flow is about 34,800 cfs, the 10-year peak flow is about 60,700 cfs, and the 100-year peak flow is about 84,100 cfs (**Table 3.1**).

3.1.3. Annual Flow Frequency

Because the NSR is an ephemeral stream, an evaluation of the number of times per year that the river is dry was carried out to assess the effects of wetting and drying on slaking of the shale bed and banks, and the breakdown of the bed material (discussed in Chapter 2). The evaluation was conducted by determining the number of times that a specified flow rate of <1 cfs occurred at the gage each year in the period of record. The results from the analysis indicate that, on average, about six periods occur throughout the year when the flow is less than 1 cfs (Figure 2.3), and, therefore, the bed is essentially dry. Since the location of the proposed dam has a significantly smaller drainage area, it is likely that the very low discharges measured at the gage are representative of conditions within the project reach as well.

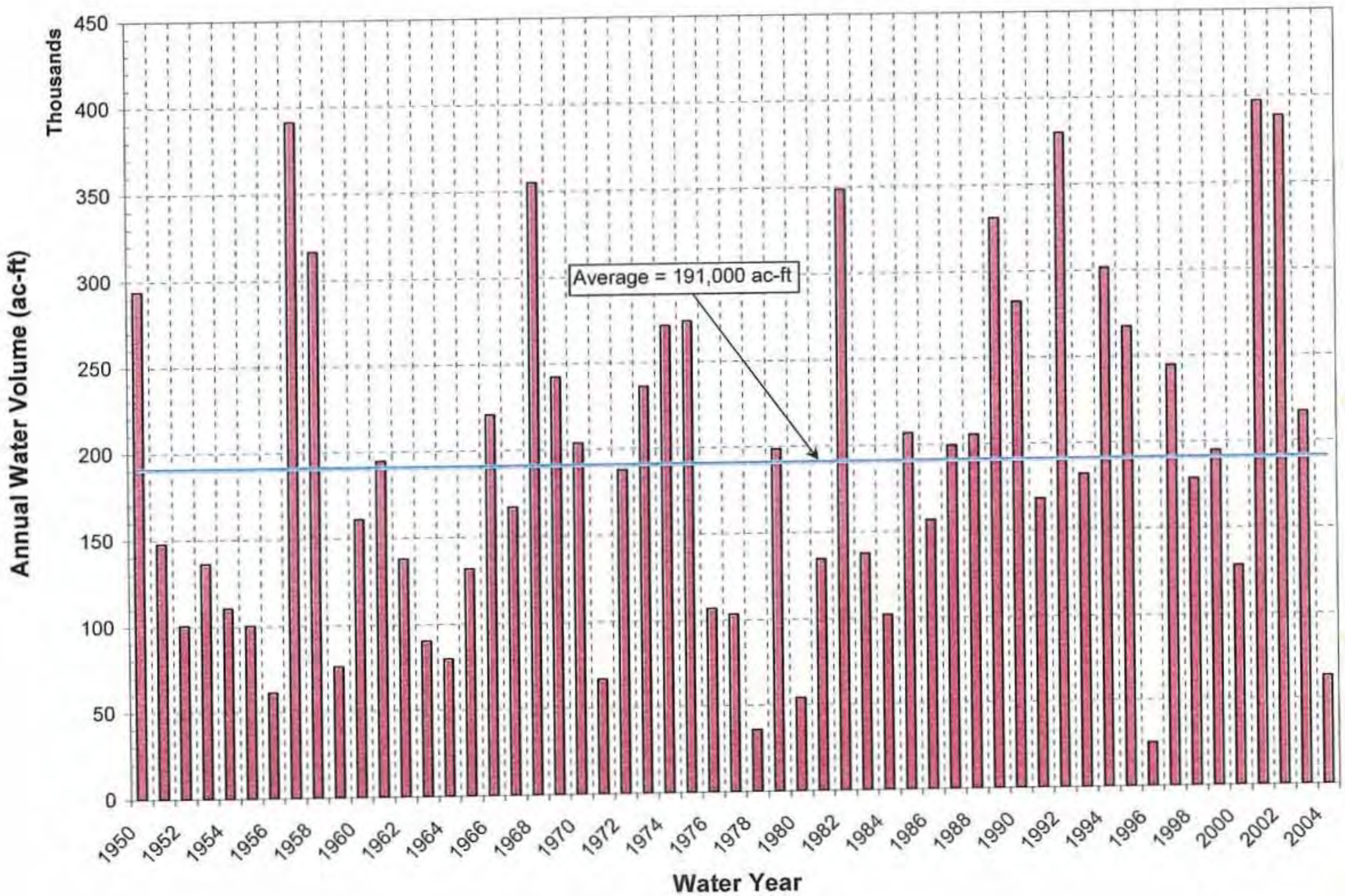


Figure 3.1. Mean annual flow volumes between 1950 and 2004 at the North Sulphur River near Cooper, Texas, gage.

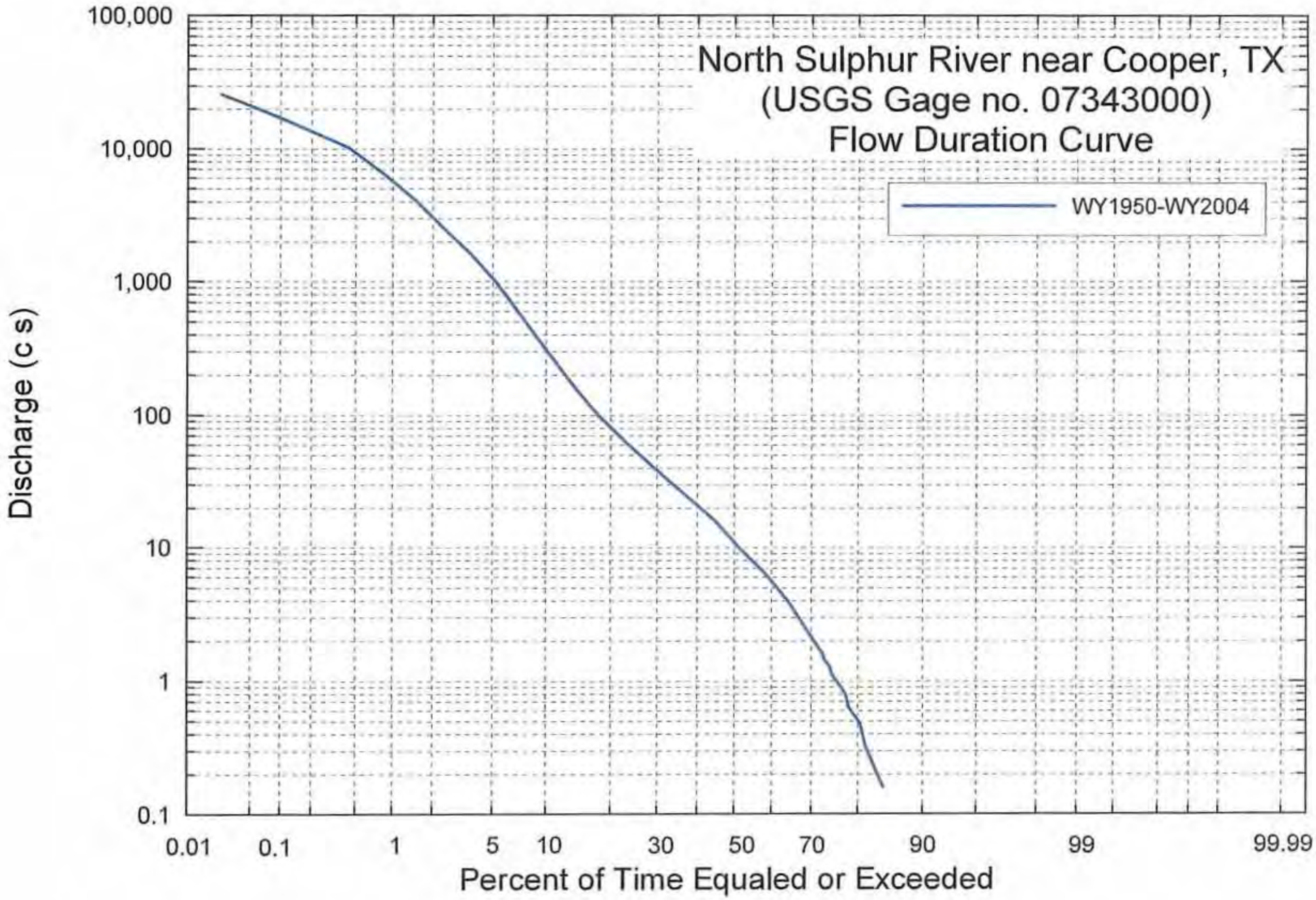


Figure 3.2. Computed flow-duration curve at the North Sulphur River near Cooper, Texas, gage.

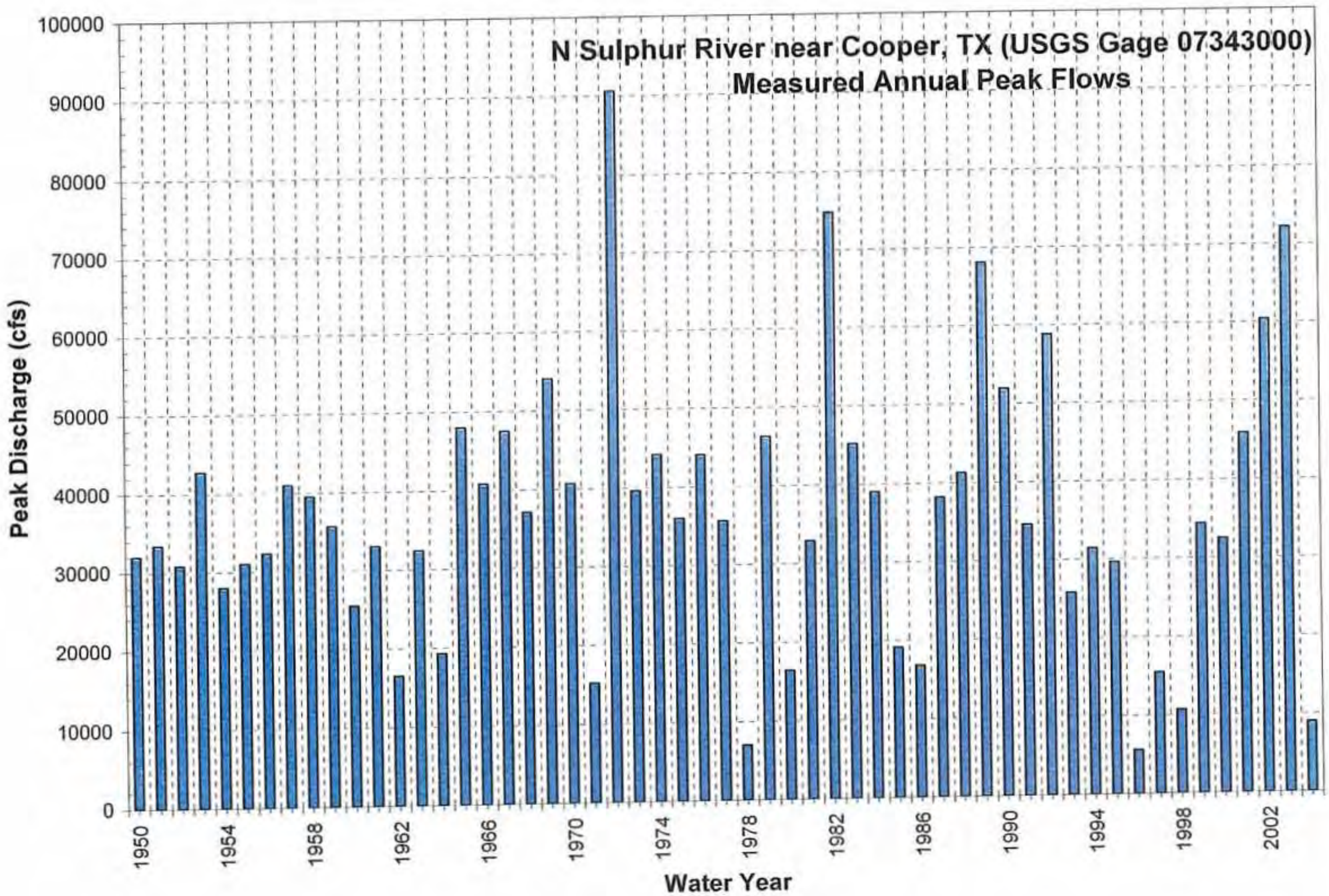


Figure 3.3. Measured annual peak flows (1950-2004) at the North Sulphur River near Cooper, Texas, gage.

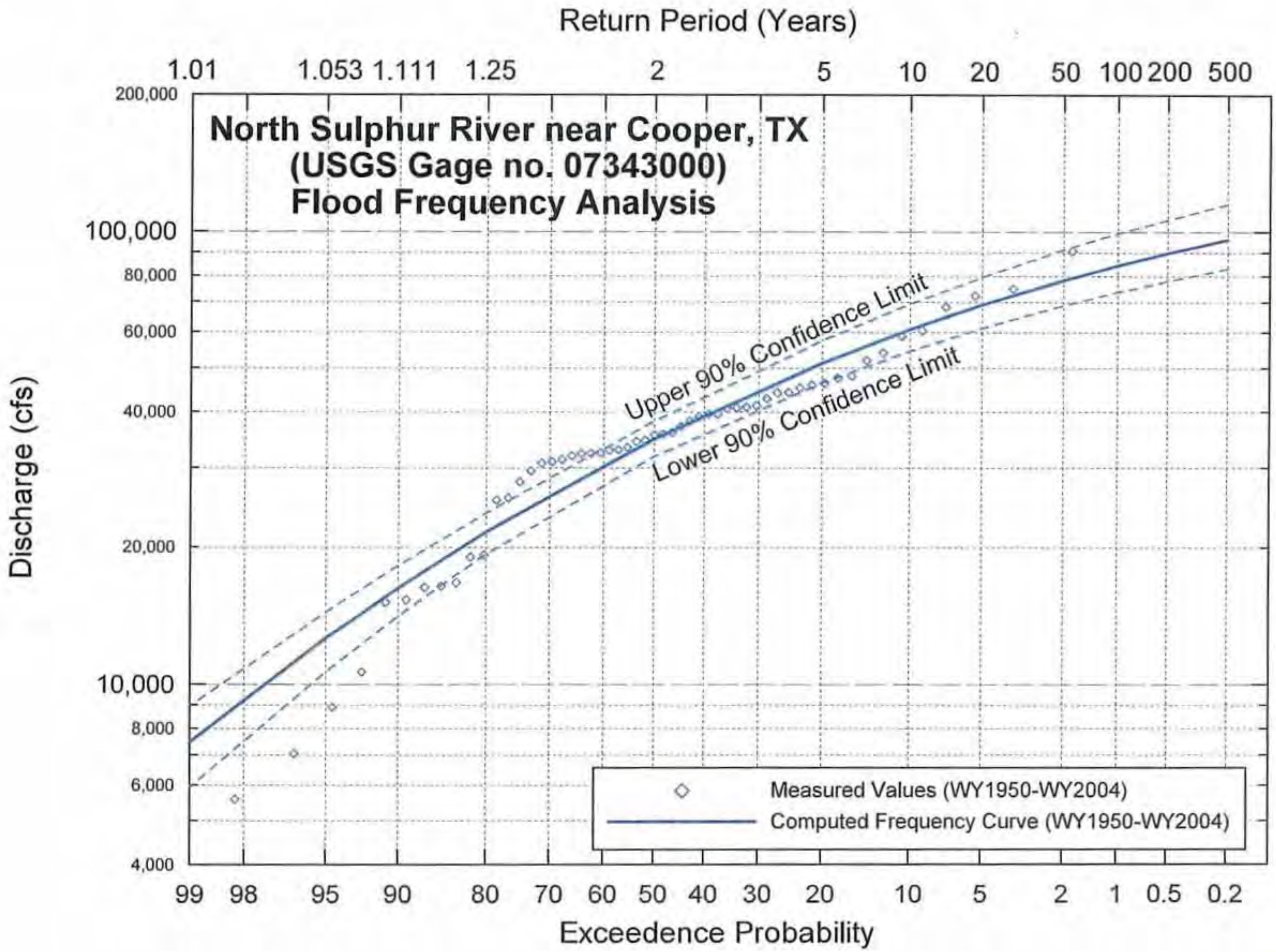


Figure 3.4. Computed peak flow-frequency curve at the North Sulphur River near Cooper, Texas, gage.

Table 3.1.	Summary of results from the flood-frequency analysis at the Cooper gage.	
Flow (cfs)	Return Interval (yrs)	Exceedence Percent
21,700	1.25	80
27,300	1.5	66.7
34,800	2	50
51,200	5	20
60,700	10	10
68,800	20	5
78,000	50	2
84,100	100	1

3.2. Hydrology at the Proposed Dam Site

The contributing drainage basin at the proposed dam is about 100 square miles, significantly less than the drainage basin area at the USGS gage. It was, therefore, necessary to evaluate the peak flow frequency and annual flow volumes at the dam location to provide input to the hydraulic model and sediment-transport analysis of annual sediment yield to the dam site.

3.2.1. Regional Regressional Relationships

A series of regional regression equations for estimation of peak streamflow frequency for ungaged natural basins in Texas was developed by Asquith and Slade (1997). The regression equations are based on measured peak flow data (up to 1993) and estimated frequency curves for 559 stations in Texas with natural (unregulated and rural) basins. The State of Texas was subdivided into 11 separate regions, and equations were developed for the 2-, 5-, 10-, 25-, 50-, and 100-year events in each of the regions based on the drainage area, the stream slope, and a basin area shape factor.

The Lake Ralph Hall dam is located in Region 7, and the equations for this region for basins with drainage areas greater than 32 square miles are:

$$Q_2=129\ A^{0.578}\ SL^{0.364} \tag{3.1}$$

$$Q_5=133\ A^{0.605}\ SL^{.578} \tag{3.2}$$

$$Q_{10}=178\ A^{0.644}\ SL^{0.699}\ SH^{-0.239} \tag{3.3}$$

$$Q_{25}=219\ A^{0.651}\ SL^{0.776}\ SH^{-0.267} \tag{3.4}$$

$$Q_{50}=261\ A^{0.653}\ SL^{0.817}\ SH^{-0.291} \tag{3.5}$$

$$Q_{100}=313\ A^{0.654}\ SL^{0.849}\ SH^{-0.316} \tag{3.6}$$

where Q_2, Q_5, Q_{10}, Q_{25}, Q_{50}, Q_{100} = the peak flows for the 2-, 5-, 10-, 25-, 50-, and 100-year events,

A = the contributing drainage area in square miles,

SL = the stream slope in feet per mile, and

SH = the basin shape factor (ratio of length of longest stream channel in basin squared to contributing drainage area).

The Cooper gage site is located in Region 10, and the regression equations for basins with drainage areas greater than 32 square miles are given by:

$$Q_2=16.9\ A^{0.798}\ SL^{0.777} \quad (3.7)$$

$$Q_5=33.0\ A^{0.790}\ SL^{.795} \quad (3.8)$$

$$Q_{10}=51.3\ A^{0.775}\ SL^{0.785} \quad (3.9)$$

$$Q_{25}=87.9A^{0.752}\ SL^{0.760} \quad (3.10$$

$$Q_{50}=129A^{0.733}\ SL^{0.735} \quad (3.11)$$

$$Q_{100}=187\ A^{0.713}\ SL^{0.708} \quad (3.12)$$

Using the Region 10 equations and the measured area, slope, and basin shape factors for the contributing basin to the USGS gage results in significant underestimation of peak values (**Figure 3.5**), and the regression equations were therefore not used to estimate the peak flow values at the Lake Ralph Hall dam site. The reason for the underestimation of peak flows using the regression equations is not clear, but could be a result of local climate and soil conditions (Harmel et al., 2006), and the incised nature of the channels that affect the time of concentration and thus increase the flood peaks.

3.2.2. Hydrologic (HEC-1) Models

3.2.2.1. RJ Brandes Company Model

The magnitude and duration of flood flows of various recurrence intervals in the vicinity of the proposed dam were previously evaluated by RJ Brandes Company (RJ Brandes Co., 2004) to design the dam and spillway. The evaluation was carried out using the Corps of Engineers HEC-1 computer software (USACE, 1990). HEC-1 simulates the surface runoff response of a river basin to precipitation by representing the basin as a system of interconnected hydrologic and hydraulic components. The precipitation-runoff response of the watershed is simulated by performing mathematical computations for four hydrologic and hydraulic processes:

a) Precipitation
b) Infiltration/interception
c) Transformation of precipitation excess to subbasin outflow
d) Hydrograph routing

The RJ Brandes Co. model for existing conditions used three subwatersheds and two connecting stream channels for the 100-square-mile basin that contributes runoff to the proposed dam location. Precipitation input was based on the 24-hour rainfall duration as prescribed in the U.S. Weather Bureau's Technical Paper Number 40 (Hershfield, 1961). As described in the Brandes report (RJ Brandes Co., 2004), no area reduction factor was applied to the precipitation depths, which likely results in higher than actual rainfall intensities, and therefore, the model results would be expected to be conservatively high. The infiltration (movement of water into the soil) and interception (surface storage in topographic depressions and vegetation) of the precipitation were simulated using Soil Conservation Service (SCS, U.S. Department of Agriculture) curve numbers, which are empirical parameters that describe the

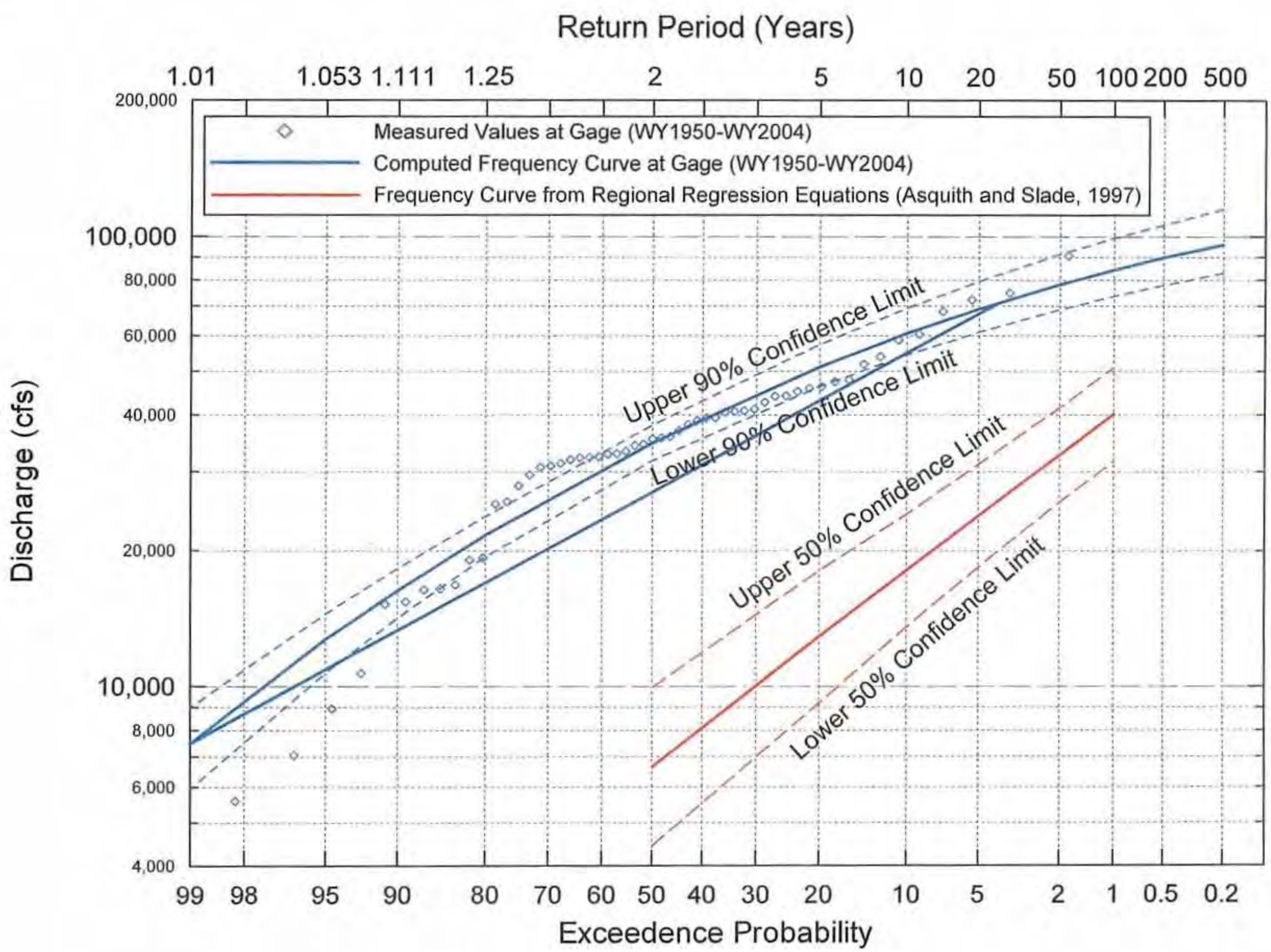


Figure 3.5. Computed frequency curves at the Cooper gage using the measured peak flow values (HEC-FFA) and using the regional regression equations for Region 10 (Asquith and Slade, 1997).

drainage characteristics of soil based on typical soil cover, land-use and antecedent runoff conditions (ARC). A curve number of 70 was assigned to each of the three subbasins in this model based on normal antecedent runoff conditions and soil types and conditions throughout the watershed. The Snyder unit hydrograph method was used to transform the excess rainfall (i.e., precipitation remaining after infiltration and interception) to subbasin flow. The lag time (the time between the center of mass of rainfall excess and the peak of the unit hydrograph at the point of interest) was developed using procedures outlined in SCS Technical Release 55 (NRCS, 1986), and ranged from 1.14 to 3.44 for the three subbasins, while a Snyder peaking coefficient of 0.55 was used for the entire watershed, consistent with Corps of Engineers studies for nearby lakes located in the Sulphur River Basin (Lakes Jim Chapman and Wright Patman) (RJ Brandes Co., 2004). The computed subbasin hydrographs were routed through the connections and main channels using the Modified Puls method using a volume-discharge rating curve that was based on results from a one-dimensional (1-D) hydraulic step-backwater model.

The Brandes model indicates that the peak of the 100-year event at the location of the proposed dam under existing conditions is about 36,300 cfs. A modified model was developed for with-dam conditions that included a fourth subbasin that represented the reservoir surface area and modified basin parameters to account for the effects of the reservoir. The model indicated that the 100-year peak flow would increase to about 46,200 cfs due to the increased flow that result from rainfall falling directly onto the reservoir.

3.2.3. Modified HEC-1 Models for Contributing Watershed to Proposed Dam Site

Since the RJ Brandes Co. HEC-1 model was developed primarily to evaluate the 100-year event and the Probable Maximum Storm (PMS) to design the dam and spillways, a separate series of models were developed for this study for the more frequent storms. Precipitation input to the models was based on the 24-hour duration rainfall depths for the 2- through 100-year events (Hershfield, 1961). Consistent with the RJ Brandes Co. model, no area reduction factor was applied to the precipitation-duration input with the expectation that the models will predict conservatively high results. Except for the SCS curve numbers, all input and basin parameters used in the Brandes models were adopted for the additional models. To determine the appropriate SCS curve numbers, an evaluation of the antecedent runoff conditions was carried out using daily precipitation data from the National Weather Surface (NWS, National Oceanic and Atmospheric Administration) weather station gage at Honey Grove, Texas (Figure 1.1). Assessment of the number of days with heavy rainfall (i.e., greater than 0.1 inches) that precede the measured peak at the USGS gage (**Figure 3.6**) indicates that heavy rainfall typically occurs for about two days prior to the flood peak, which suggests that wet antecedent runoff conditions should be considered in the rainfall-runoff calculations for the more frequent events. The assessment also indicated a slight trend toward more normal antecedent runoff conditions for the less frequent events. Selected curve numbers for the revised models, therefore, ranged from 85 for the 2-year event to 72 for the 100-year event.

The frequency curve that was developed from the computed peaks at the proposed dam location (**Figure 3.7**) is generally parallel to the computed frequency curve at the USGS gage near Cooper, Texas, and is similar to the curve that is based on the unit discharge (discharge per unit area of basin) at the dam location using an area exponent of 0.8. (An area exponent of 0.8 was selected based on previous experience with rivers in the Southwest.) A summary of the computed peak flows at the location of the dam is provided in **Table 3.2**. The frequency curves that were developed using the regional regression equations and from the HEC-1 models with normal antecedent runoff conditions significantly underpredict the peak discharges, especially at the more frequent events.

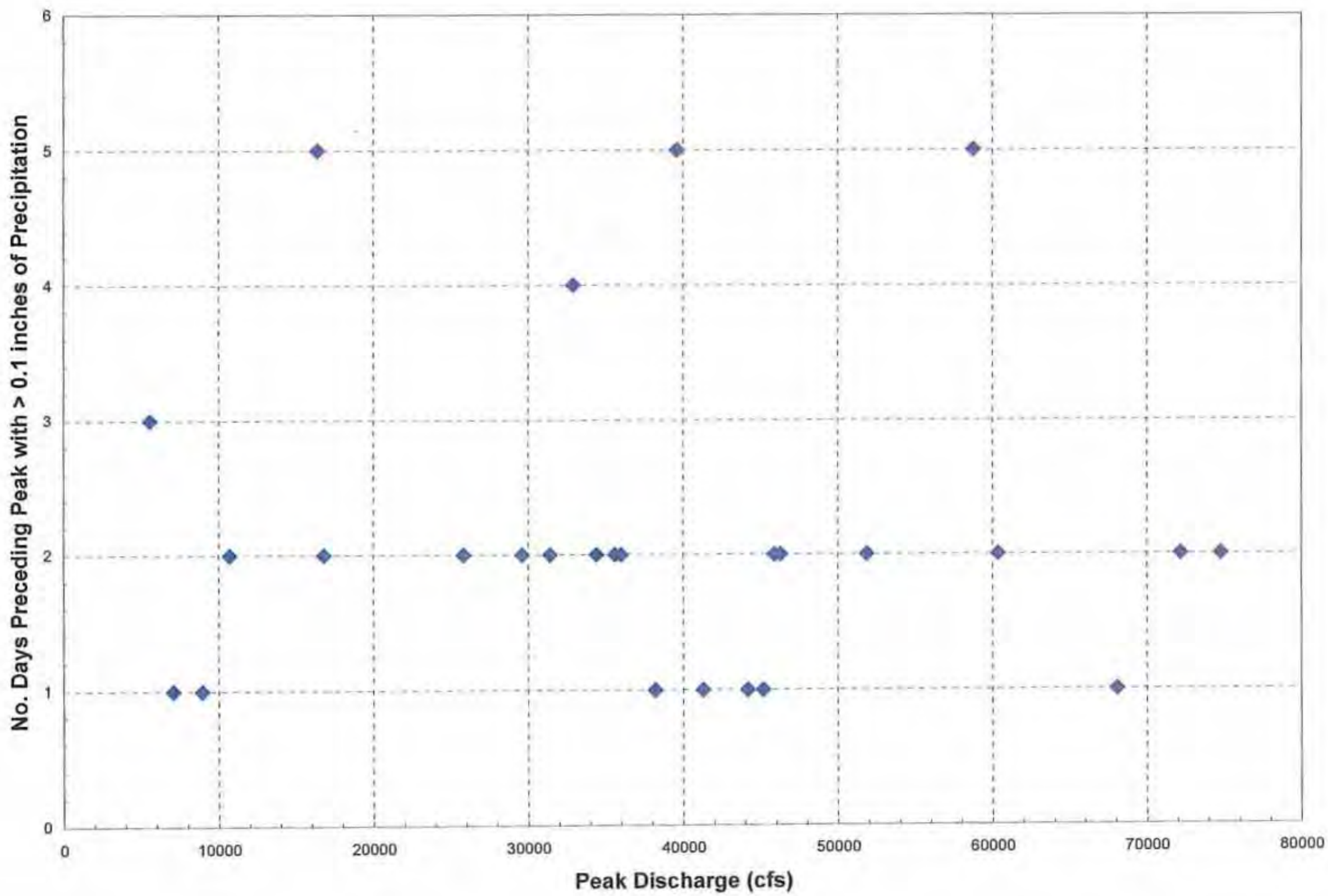


Figure 3.6. Number of days preceding measured peak discharges at the USGS gage near Cooper with rainfall exceeding 0.1 inches at the Honey Grove weather station.

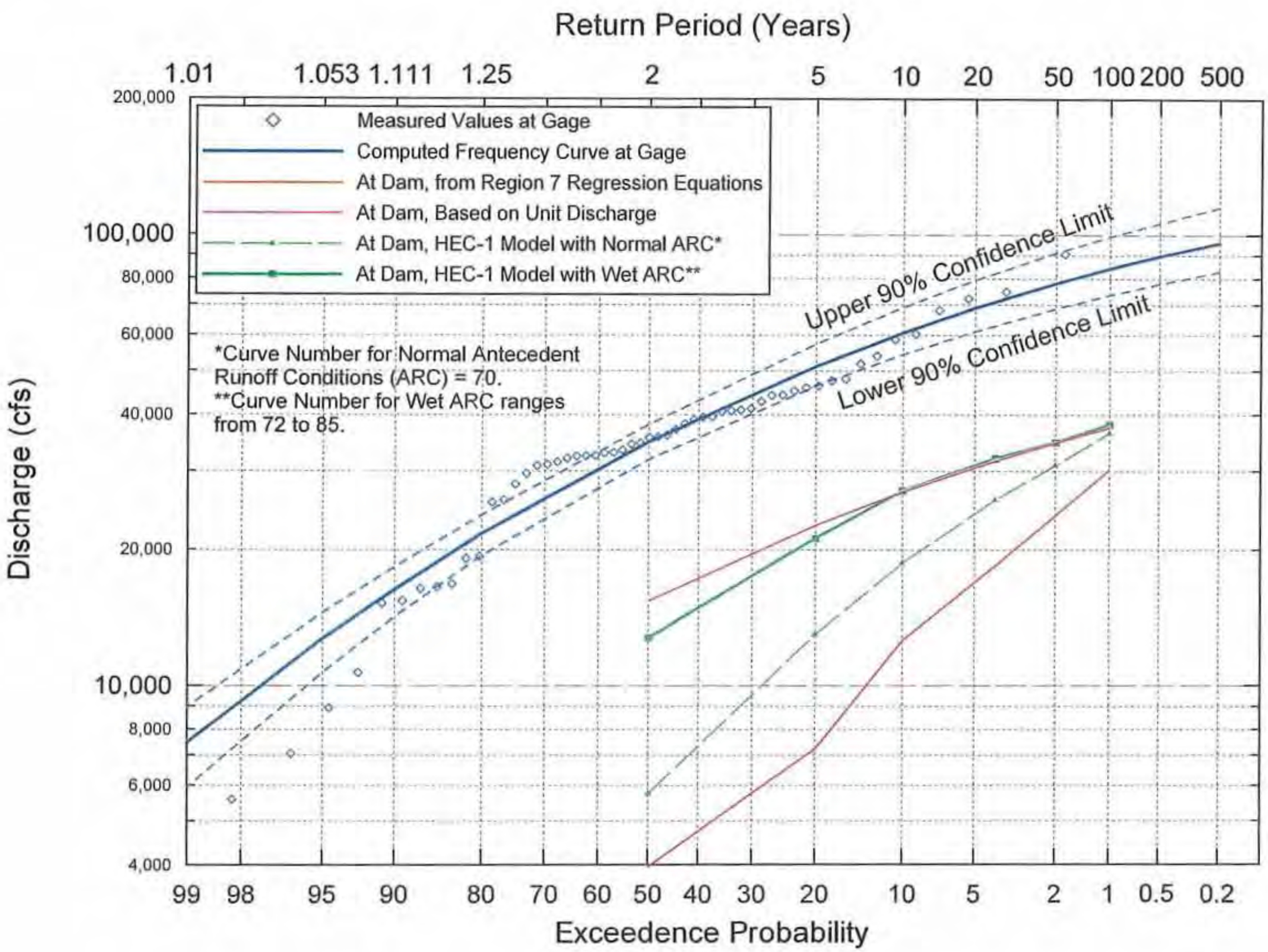


Figure 3.7. Flood-frequency curves based on the computed peak discharges from the MEI HEC-1 models (using wet antecedent runoff conditions), from the HEC-1 models using normal antecedent runoff conditions, from the regional regression equations, and based on the unit discharge at the Cooper gage location. Also shown is the computed frequency curve at the USGS gage near Cooper, Texas.

Table 3.2. Summary of computed peak flows at the proposed dam from the HEC-1 model with wet antecedent runoff conditions (ARC).		
Flow (cfs)	Return Interval (yrs)	Exceedence Percent
12,700	2	50
21,100	5	20
27,000	10	10
31,900	25	4
34,600	50	2
37,900	100	1

3.2.4. HEC-1 Models for Tributary Basins

HEC-1 models were developed for each of the nine larger tributaries located upstream of the dam. These include Merrill Creek, Bralley Pool Creek, Leggets Branch, Davis Creek, Pickle Creek. Brushy Creek, Bear Creek, Long Creek, and Allen Creek. An additional model was developed for Baker Creek to evaluate a representative tributary below the proposed dam site. A model for Pot Creek (tributary to Brushy Creek) was developed to complete the Brushy Creek model. Basin areas were computed using a USGS Digital Elevation Model (DEM), (September 2001), and included up to seven subbasins for the overall tributary basins (**Figure 3.8**). (Basin parameters are summarized in Table 5.1.) Precipitation input and the SCS curve numbers for wet antecedent runoff conditions that were developed for the overall basin were applied to the tributary models. The lag time was developed using procedures outlined in SCS Technical Release 55 (NRCS, 1986) and information from the hydraulic models that were developed for each of the tributaries (Chapter 4). Computed lag times ranged from 0.84 hours in the smallest subbasin to 2.5 hours in the largest subbasin. Consistent with the overall reservoir model, a Snyder peaking coefficient of 0.55 was used for the each of the subbasins, and the computed subbasin hydrographs were routed through the connections and main channels using the Modified Puls method with a volume-discharge rating curve that was developed from the hydraulic models (Chapter 4).

Because the primary tributaries do not include all of the subbasins that contribute water and sediment to the proposed dam location (Figure 3.8), data from the tributary models were used to develop regression equations that relate peak discharge or storm runoff volume to contributing drainage area (**Figures 3.9 and 3.10**). The regression equations are believed to adequately relate peak flow and runoff volume to drainage area because the square of the Pearson product moment correlation coefficient (R^2 value) ranges from 0.97 to 0.98 for the peak flow equations, and range from 0.99 to nearly 1.00 for the storage volume equations. The regression equations were used to estimate the peak flow and runoff volume from the tributary basins that were not specifically modeled under existing conditions, and for all of the tributaries under with-dam conditions.

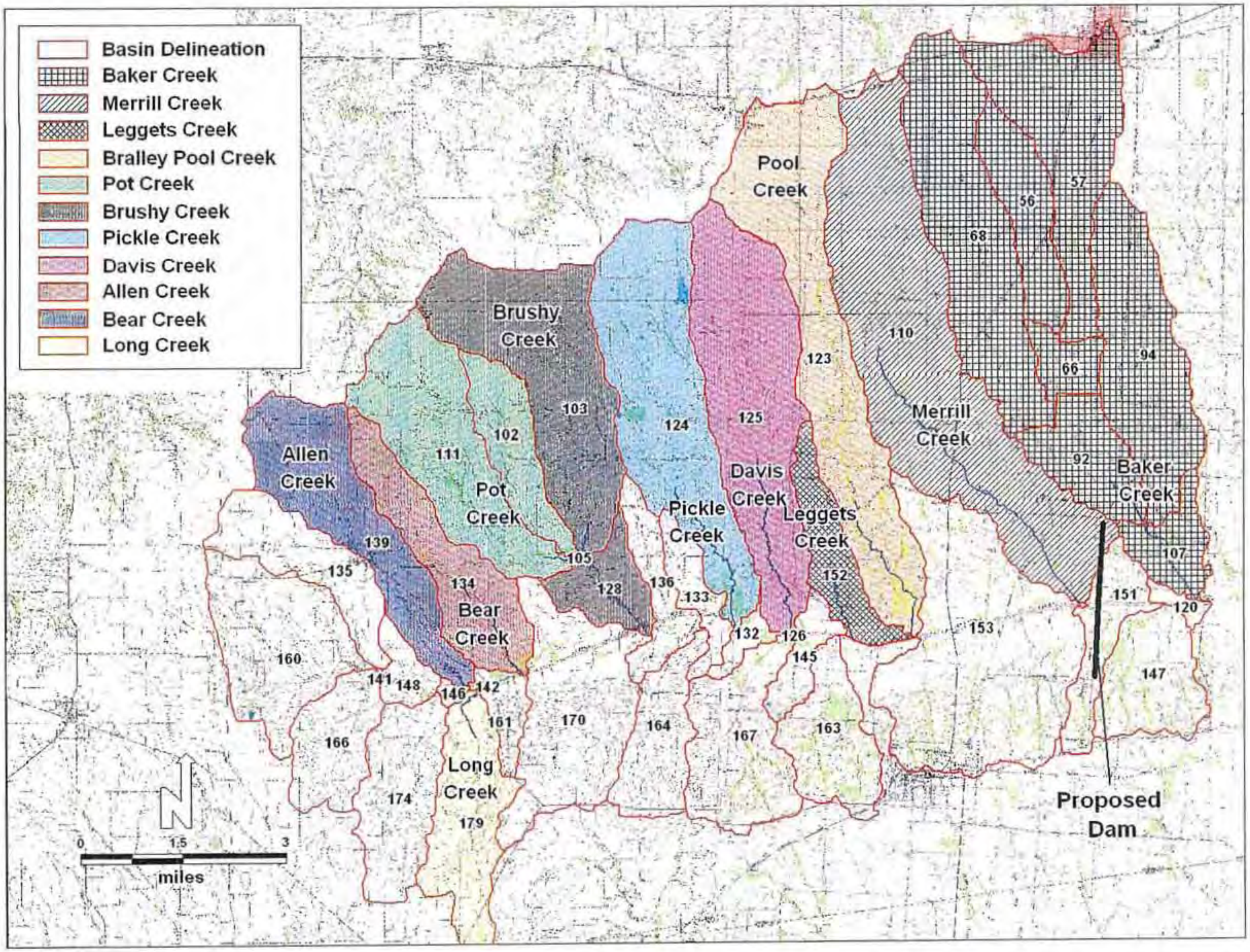


Figure 3.8. Delineated subbasins for the contributing watershed to the proposed Lake Ralph Hall dam site near Ladonia, Texas, and the primary tributary basins that were modeled using HEC-1.

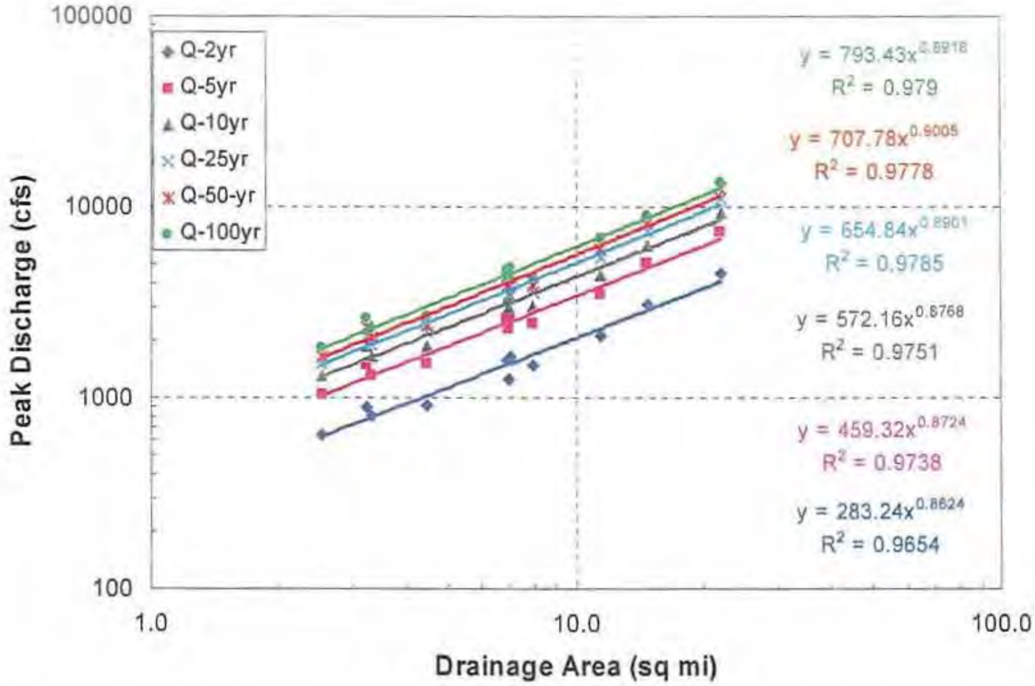


Figure 3.9. Computed peak discharges from the HEC-1 models for the primary tributaries as a function of contributing drainage area, and the resulting regression equations.

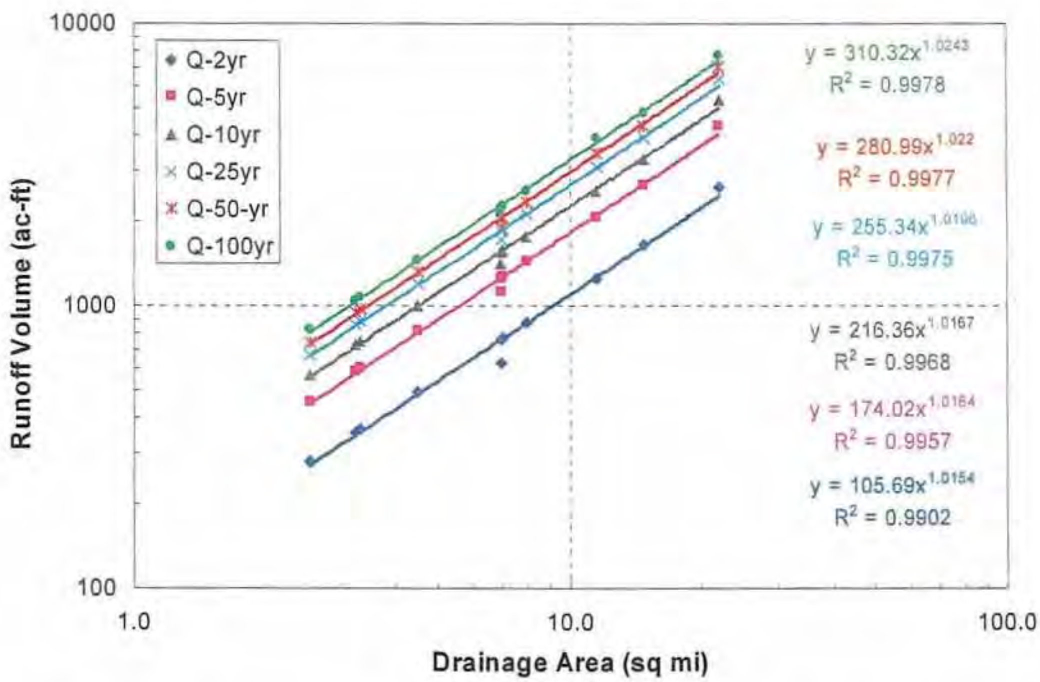


Figure 3.10. Computed storm volumes from the HEC-1 models for the primary tributaries as a function of contributing drainage area, and the resulting regression equations.

3.2.5. With-Dam Downstream Impacts

The effects of the proposed Ralph Hall Dam on downstream flows in the North Sulphur River will likely be significant due to the detention storage capacity available in the reservoir. Based on previous work (RJ Brandes Company, 2004), the 100-year peak discharge in the river below the dam will be reduced from about 38,000 cfs (Figure 3.7) to less than 10,000 cfs, with corresponding reductions in average velocities in the river channel from about 6 feet per second (fps) down to about 4 fps. The more frequent flood events also will be significantly reduced in terms of their peak discharge since runoff volumes for these lesser magnitude storms will be considerably less and subject to greater attenuation in the reservoir. When the reservoir is below its conservation storage capacity, inflows to the reservoir from smaller storms are likely to be entirely contained and stored, with no outflows passed downstream. The floodwater detention storage capabilities of the reservoir should result in significantly less erosion of the downstream channel below the dam.

Provisions are being incorporated into the operating plan for the reservoir to provide for the passage of sufficient low flows to maintain a proposed wetlands restoration project along approximately 14,000 feet of an abandoned segment of the original river channel within the southern floodplain of the river. Excess flows from this segment will be discharged back into the existing river channel approximately three miles below the dam; however, these flows are expected to be minimal. Because of the ephemeral natural of the existing river downstream of the proposed dam site, only very limited aquatic biological resources and habitat exist along the river channel, thus there is no great necessity for the passing substantial flows through the reservoir for environmental purposes.

4. HYDRAULICS

Hydraulic models were developed to quantify the hydraulic conditions (i.e., velocity, depth, water-surface elevation) within the project reach of the NSR and major tributaries over a range of flows up to and including the 100-year peak flow. The analysis was conducted using the U.S. Army Corps of Engineers 1-D HEC-RAS step-backwater program, Version 3.1.3 (USACE, 2005). A single model was developed for the mainstem NSR, and separate models were developed for each of the primary tributaries.

4.1. Hydraulic Model for the North Sulphur River

The HEC-RAS model for the mainstem NSR extends upstream from about 100 feet below the FM 904 bridge for a distance of about 11.8 miles to about 1 mile above SH 68, and includes 101 cross sections at an average spacing of 620 feet. The model geometry was based on cross sections that were cut from the Digital Terrain Model (DTM) of the project reach that was developed by CP&Y using aerial photography from February 2002. The cross sections were placed at representative locations along the channel, or at locations where hydraulic controls (i.e., bridges or other constrictions), extended across the entire main channel. HEC-RAS accounts for energy losses that result from roughness along the channel bed and banks with a roughness coefficient, or Manning's *n*-value. A vertical variation in *n*-values was used to account for the reduced roughness at higher flow depths that are typical in large channels such as the NSR. On the basis of field observations and previous experience with similar incised channels, the selected roughness values ranged from 0.040 at very low flows to about 0.022 at the 100-year peak flow in the main channel, and ranged from 0.048 to 0.070 in the overbanks. For the NSR model, the overbanks are defined as the region outside of the bank stations, which are established at the change in roughness that occurs at the boundary between the edge of vegetation and the exposed channel bed, and therefore do not typically coincide with the topographic top-of-bank. A normal-depth downstream boundary condition with a slope of 0.2 percent was used in the model based on the existing slope of the channel bed at the downstream limit of the model. All bridges were coded into the model using the most recent bridge design plans.

The model was run over a range of flows from 20 cfs (at downstream limit of model) to the 100-year event, with a flow distribution based on the MEI HEC-1 model (using wet antecedent runoff conditions) for mainstem flows at the dam and the unit discharge (discharge per unit drainage area) to estimate contribution from major tributaries since the individual tributary peak flows are likely not coincident with the mainstem peak flows.

4.1.1. Model Calibration

The model was calibrated to high-water marks that were identified and measured during the December 2005, field visit. Since the measured peak flows in 2004 and 2005 were relatively small (less than 8,950 cfs), and the measured high-water marks were between 9 and 13 feet above the channel bed, it was assumed that the field-observed high-water marks were associated with the 2002 and 2003 annual peaks. The 2002 and 2003 measured peak flows at the USGS Cooper gage were 60,400 and 72,200 cfs (corresponding to the 10- and 25-year peak flows, Figure 3.6), respectively, and were adjusted to flows at the dam and throughout the model reach based on the computed unit discharge. Using the roughness values and boundary conditions described above, the model calibrates well with the measured high-water marks (**Figure 4.1**).

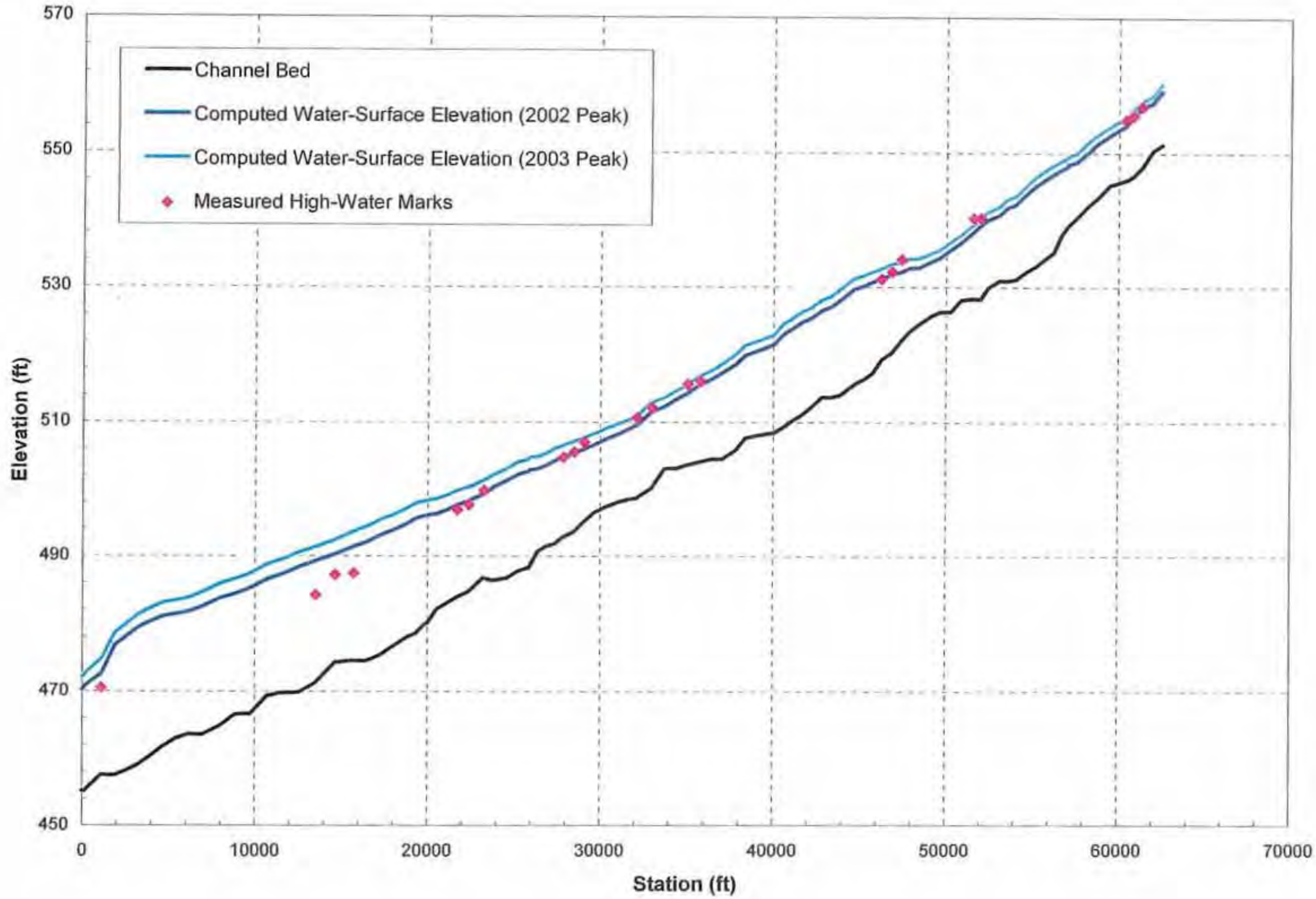


Figure 4.1. Measured high-water marks and computed water-surface elevations for the 2002 (Q=60,400 cfs) and the 2003 (72,200 cfs) flood peaks.

4.1.2. Reach-averaged Hydraulics

Subreach-averaged hydraulic conditions were evaluated by subdividing the model of the NSR into 10 subreaches, based on the locations of significant hydraulic controls and the location of the major tributaries (**Table 4.1, Figure 4.2**.) The average main channel velocity, hydraulic depth, effective (active channel) width, and total shear stress were computed for each of the subreaches over the range of modeled discharges (**Figures 4.3 through 4.6**). As expected, a general increasing trend of velocity, depth, and width occur in the downstream direction due to the effects of tributary inflows. Main channel velocities range from 4.8 to 9.7 fps at the 2-year peak discharge, and range from 5.6 to 11.6 fps at the 100-year peak discharge. Hydraulic depths in the main channel do not extend to the top of the channel over the range of modeled discharges, ranging from 3.7 to 10.1 feet at the 2-year event, and from 6.5 to 17.2 feet at the 100-year event. The effective widths are also limited to the main channel, ranging from 87 feet to about 190 feet at the 2-year peak discharge, and from about 130 feet to about 250 feet at the 100-year peak discharge. To evaluate the potential for the river to entrain bed material and to adjust the channel geometry (refer to Section 2.2.2), the total stream power (i.e., the amount of energy dissipated per unit length along the channel boundary) and unit stream power (stream power per unit width) were computed over the range of modeled flows (**Figures 4.7 and 4.8**). The total stream power averages about 600 lb/s at the 2-year event, ranging from 113 lb/s in the upstream subreaches to 2,460 lb/s in the downstream subreach, and averages about 1,560 lb/s at the 100-year peak discharge, ranging from 177 to 6720 lb/s. Unit stream powers also show a general increasing trend in the downstream direction, ranging from 0.9 to 15.9 lb/ft-s (average of 4.2 lb/ft-s) at the 2-year peak discharge, and from 0.8 to 26.9 lb/ft-s (average of 8.1 lb/ft-s) at the 100-year peak discharge. High unit-stream power in Subreach 5 corresponds to the high vertical, eroding banks in the subreach and indicates that further widening can be expected.

A critical grain-size analysis was carried out on a subreach-averaged basis to determine the size of sediment that can be mobilized at various discharges. The results indicate that the 2-year peak discharge will mobilize sediment sizes ranging from 48 to 68 mm, while the 100-year peak discharge will mobilize sediment sizes ranging from 74 to 120 mm (**Figure 4.9**). These results are consistent with evidence of transport of the gravel- and cobble-sized material that was observed at some locations along the channel bed.

Table 4.1. Summary of subreach delineations used for the reach-averaged hydraulic calculations.

Subreach	Description	Upstream Station (ft)	Downstream Station (ft)	Subreach Length (ft)
1	Upstream	61,966	59,106	2,861
2	Allen Creek to Bear Creek	59,106	54,342	4,764
3	Bear Creek to Brushy Creek	54,342	44,264	10,078
4	Brushy Creek to Pickle Creek	44,264	37,423	6,840
5	Pickle Creek to Davis Creek	37,423	32,513	4,910
6	Davis Creek to Leggets Branch	32,513	28,138	4,375
7	Leggets Branch to Bralley Pool Creek	28,138	22,786	5,352
8	Bralley Pool Creek to Merrill Creek	22,786	10,214	12,572
9	Merrill Creek to proposed dam location	10,214	7966	9,588
10	Proposed dam location to FM 904	7966	6	620

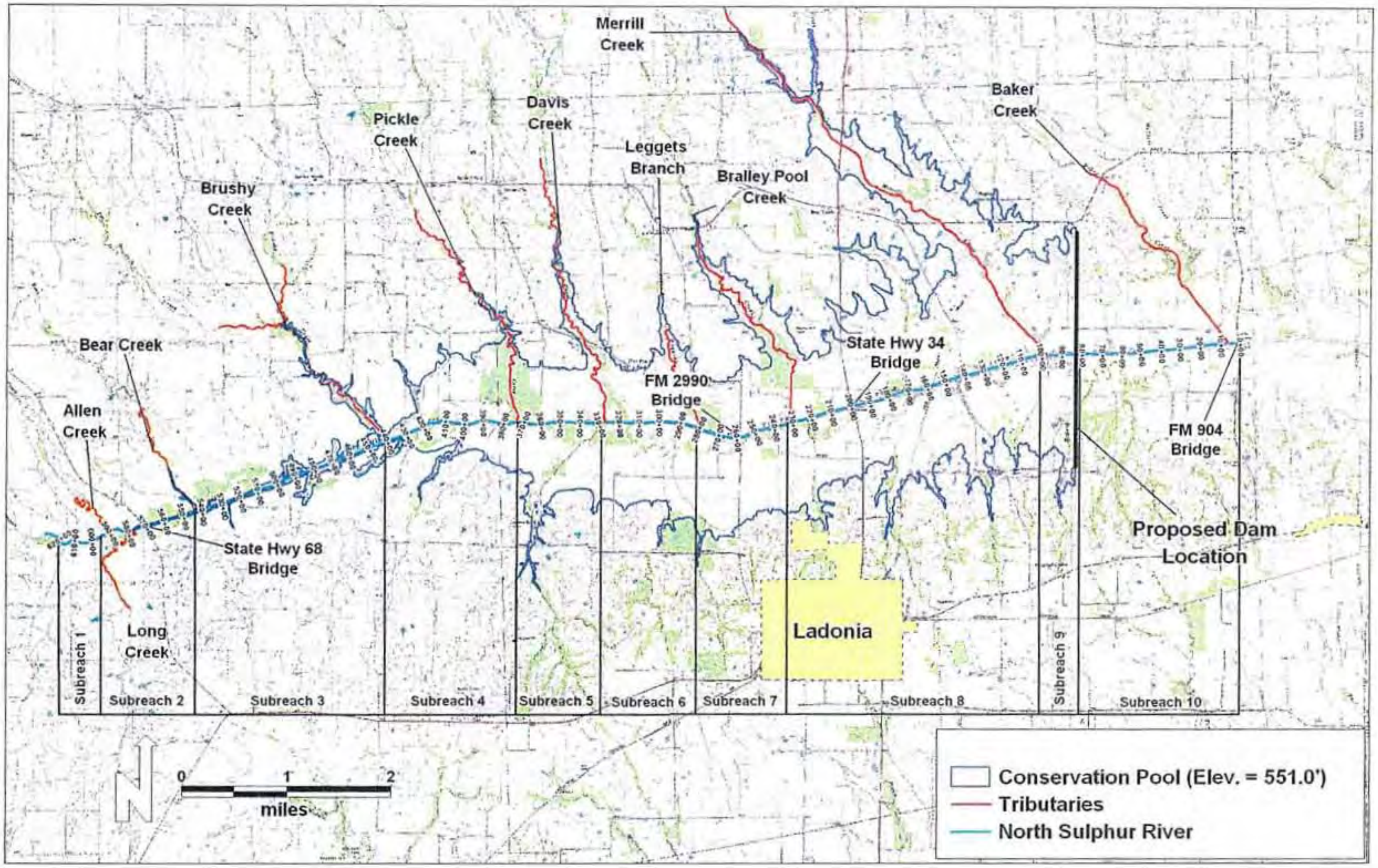


Figure 4.2. Subreach breakdown for the subreach-averaged hydraulic computations.

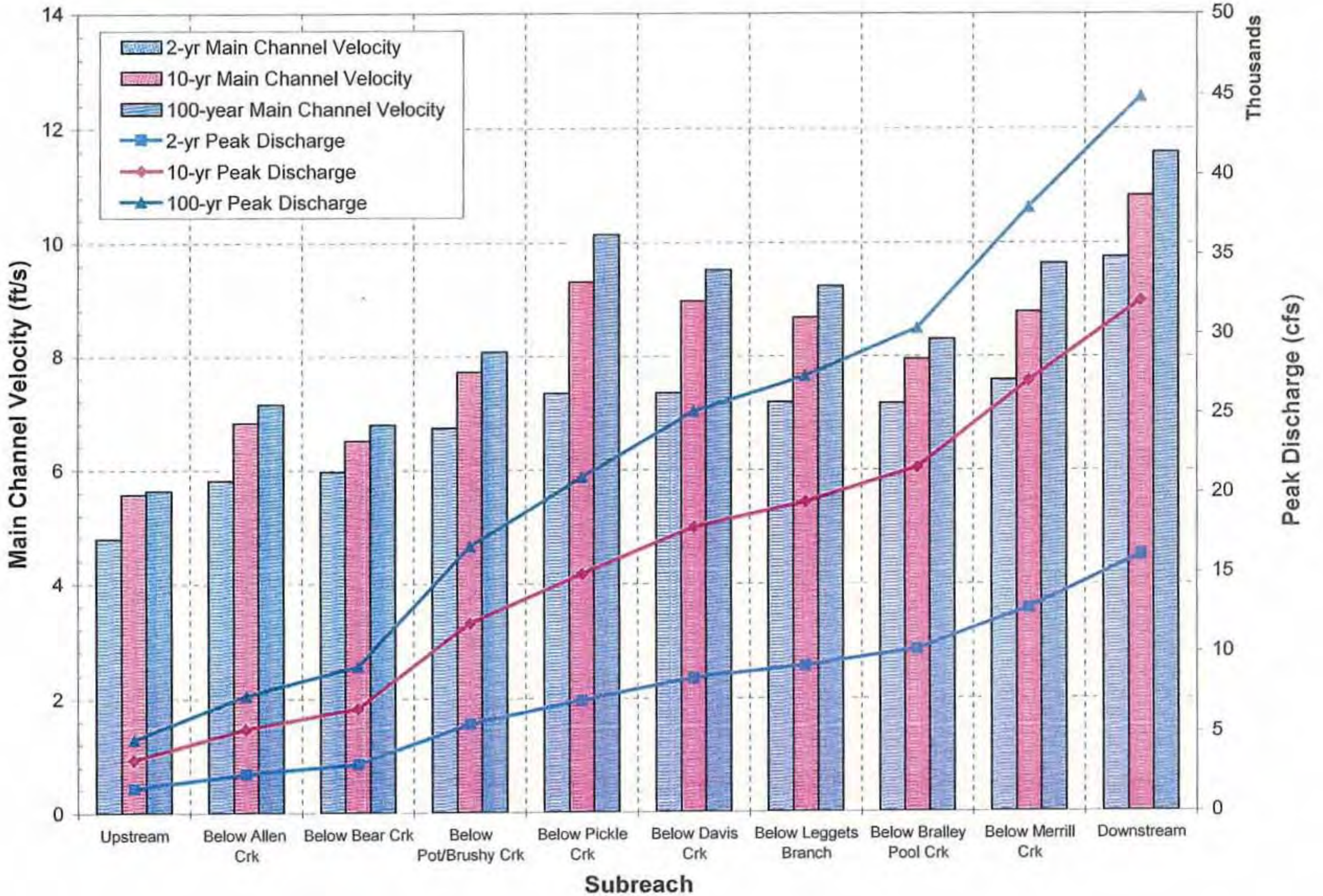


Figure 4.3. Subreach-averaged main channel velocities (bar graphs) for each of the subreaches for the 2-, 10-, and 100-year peak flows. Also shown are the peak discharges (curves).

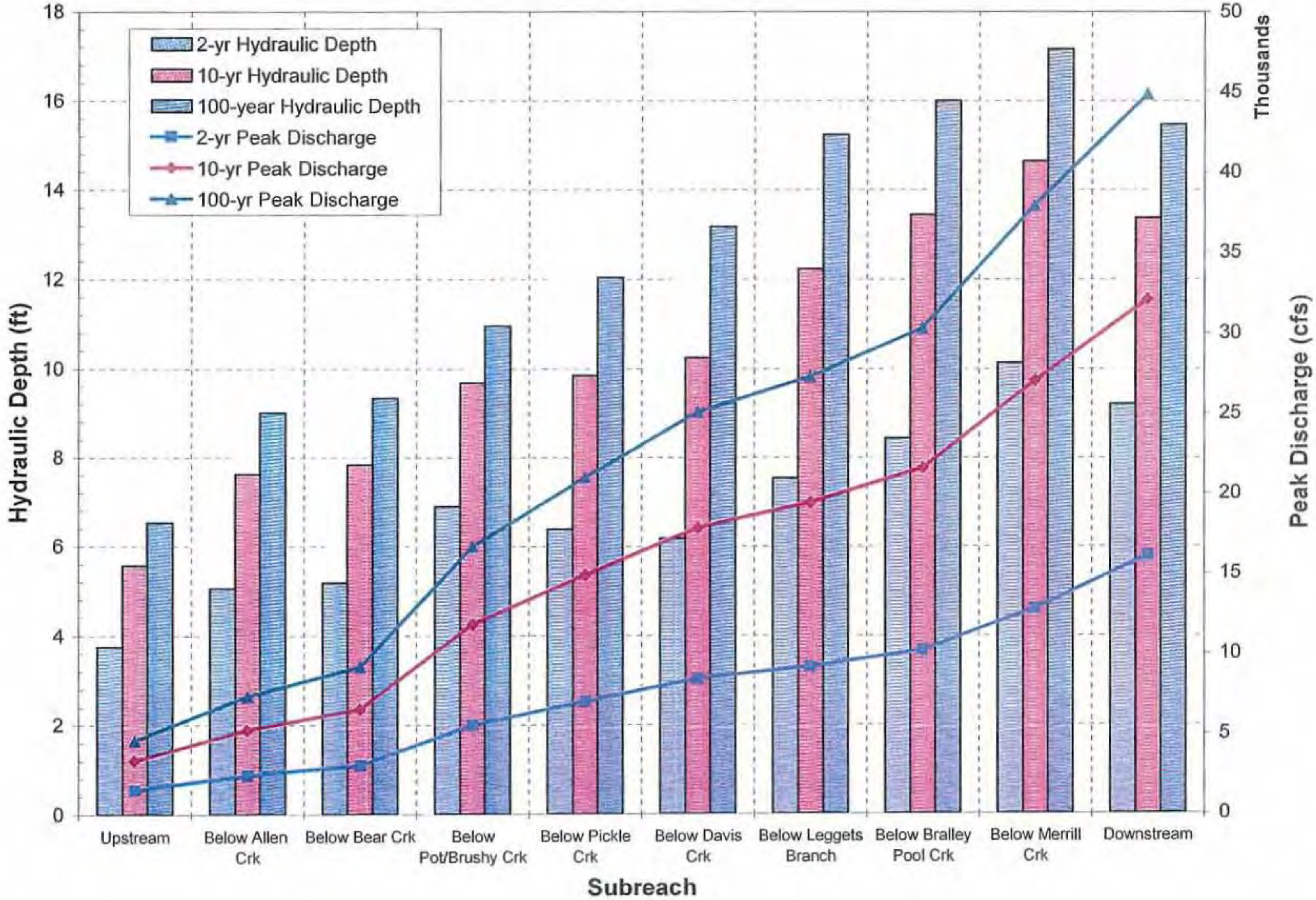


Figure 4.4. Subreach-averaged hydraulic depth (bar graphs) for each of the subreaches for the 2-, 10-, and 100-year peak flows. Also shown are the peak discharges (curves).

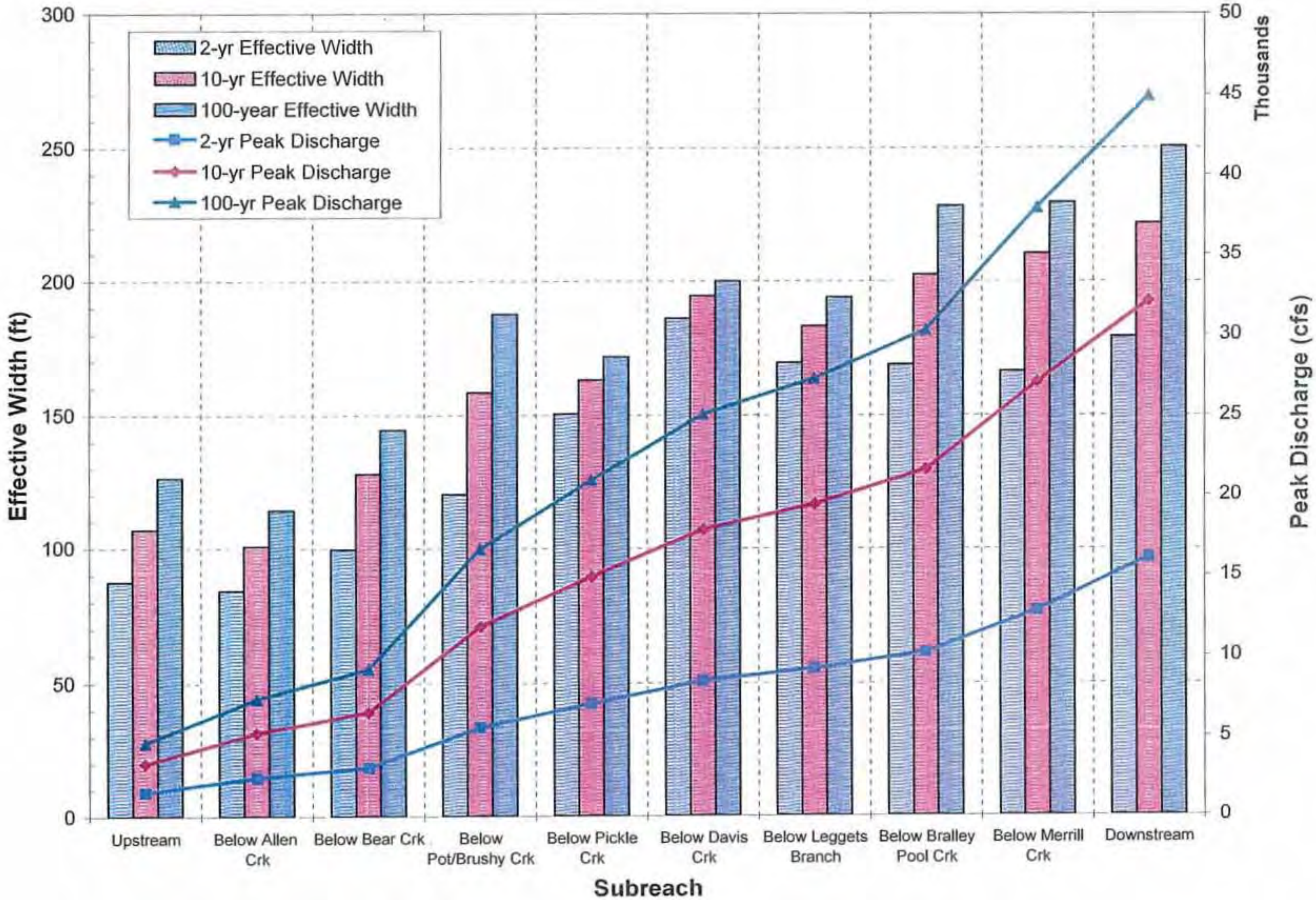


Figure 4.5. Subreach-averaged effective width (bar graphs) for each of the subreaches for the 2-, 10-, and 100-year peak flows. Also shown are the peak discharges (curves).

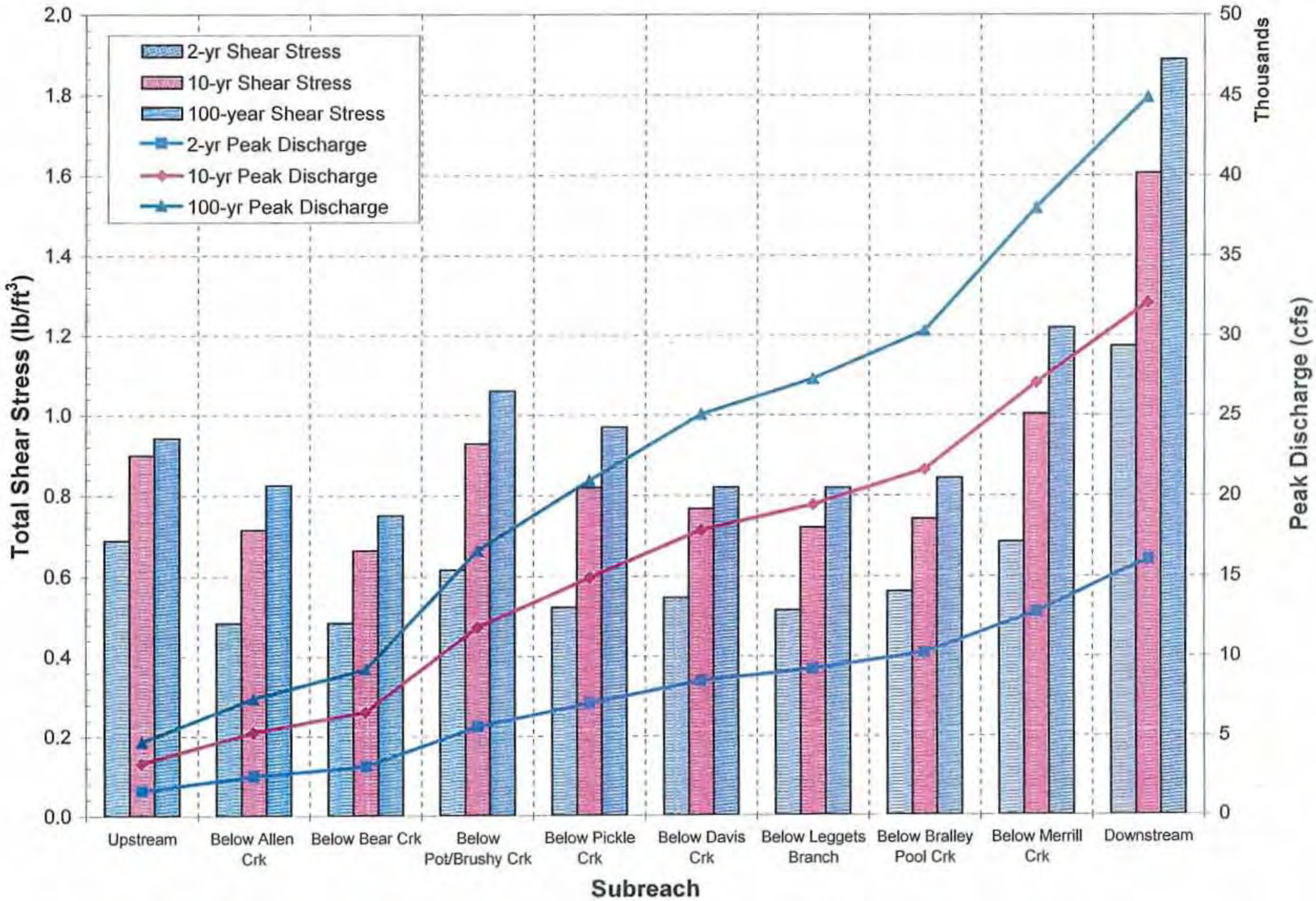


Figure 4.6. Subreach-averaged total shear stress (bar graphs) for each of the subreaches for the 2-, 10-, and 100-year peak flows. Also shown are the peak discharges (curves).

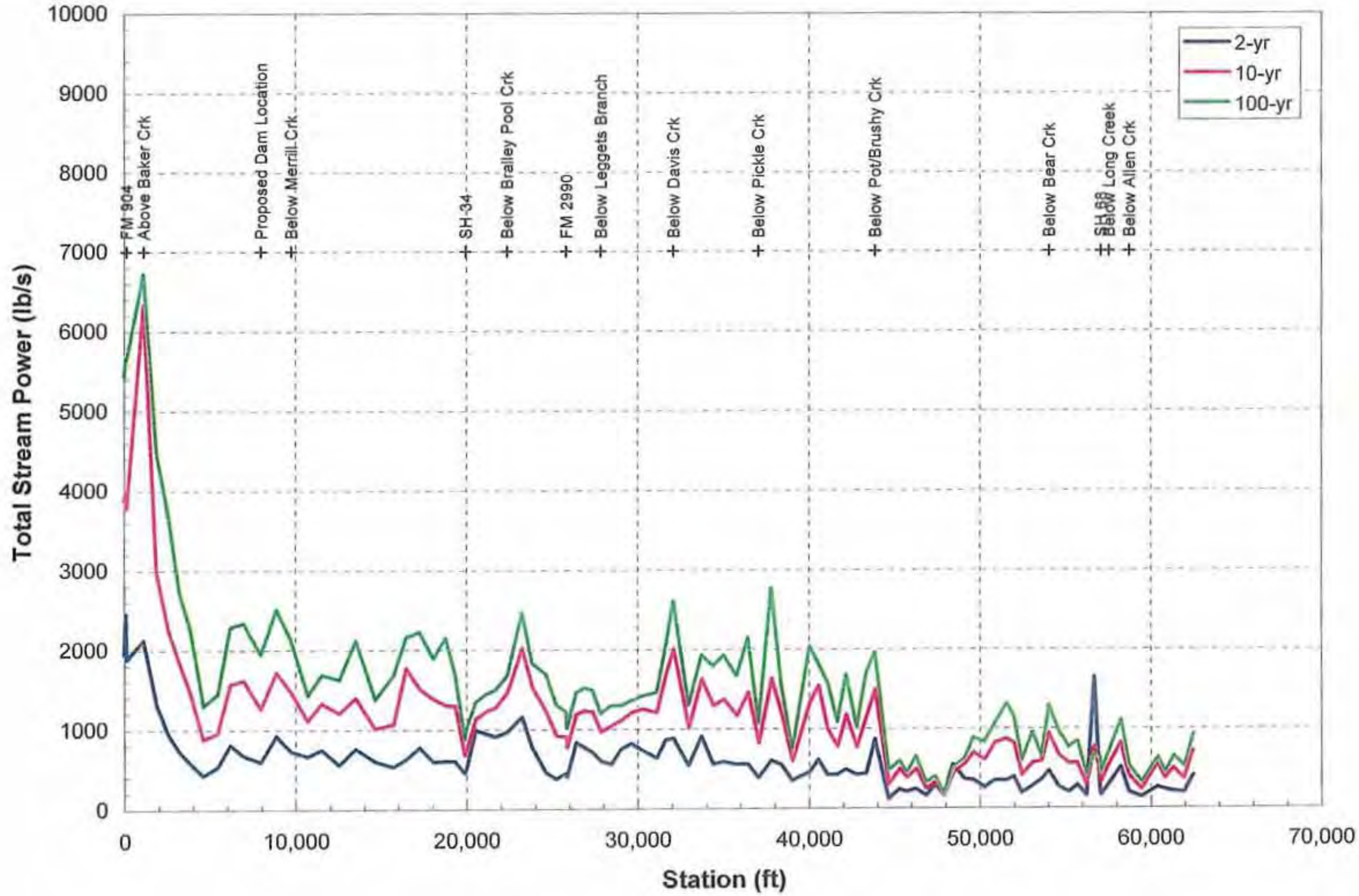


Figure 4.7. Computed total stream power along the modeled reach of the North Sulphur River for the 2-, 10-, and 100-year peak discharges.

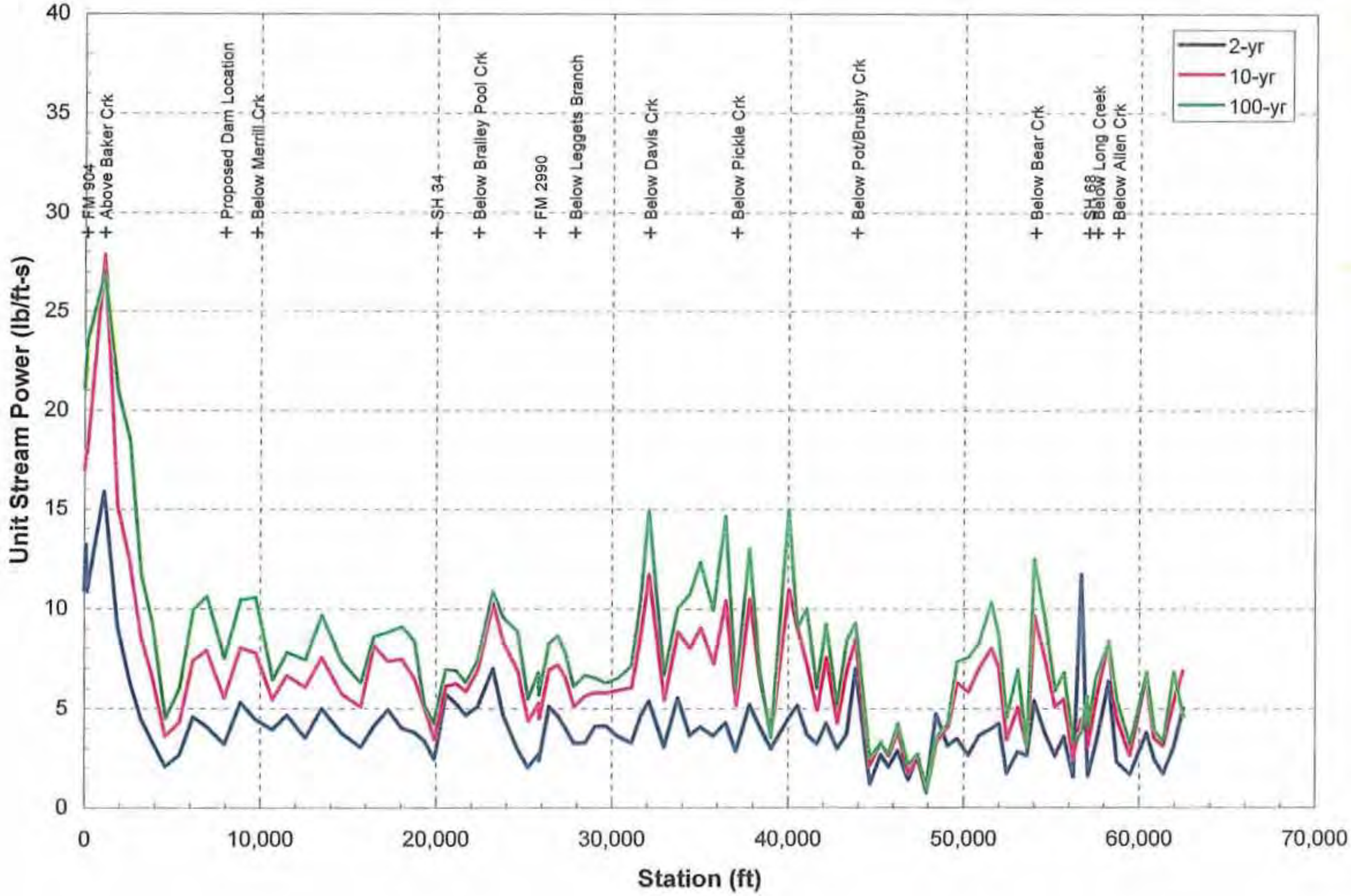


Figure 4.8. Computed unit stream power along the modeled reach of the North Sulphur River for the 2-, 10-, and 100-year peak discharges.

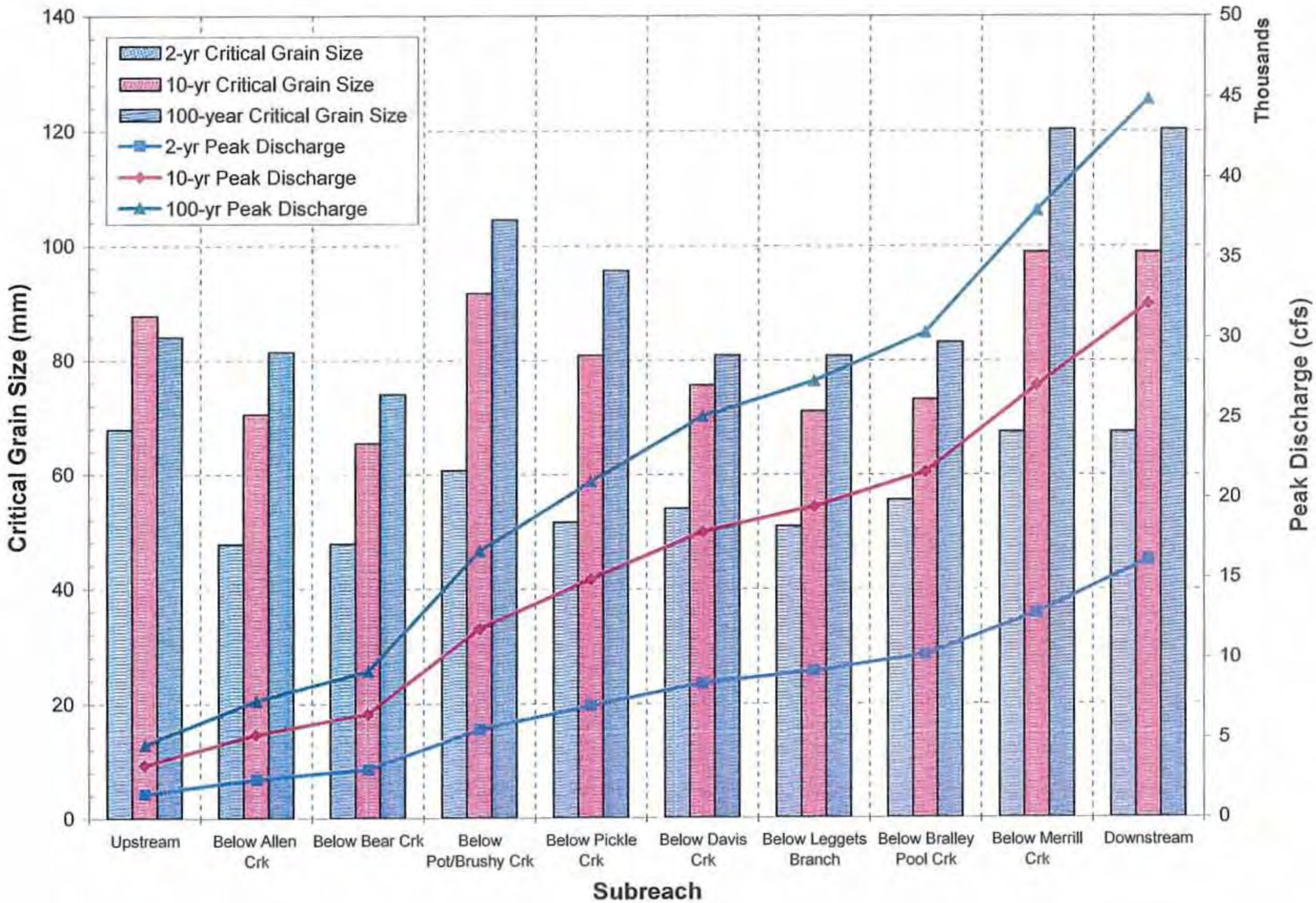


Figure 4.9. Subreach-averaged critical grain size (bar graphs) for each of the subreaches for the 2-, 10-, and 100-year peak flows. Also shown are the peak discharges (curves).

4.2. Hydraulic Models for the Primary Tributaries

Hydraulic models were developed for each of the primary tributaries in the project reach to estimate the hydraulic conditions to provide input to the sediment-transport calculations (Chapter 5). The modeled tributaries included Merrill Creek, Bralley Pool Creek, Leggets Branch, Davis Creek, Pickle Creek. Brushy Creek, Bear Creek, Long Creek, and Allen Creek. A model for Baker Creek was also developed to estimate hydraulic conditions in a typical tributary downstream from the location of the proposed dam. The model geometry was based on cross sections that were cut from the DTM developed by CP&Y. The length of the modeled reach and the number of sections in the model depended on the available topography covered by the DTM (**Table 4.2**). Model lengths ranged from 3,640 feet in Allen Creek to about 27,700 feet in Merrill Creek, and the number of sections in the models ranged from 20 in Allen Creek to 116 in Bralley Pool Creek, with average cross-sectional spacing ranging from 58 to 269 feet. In some cases, two models were developed for the individual tributaries, including one model for the portion of the tributary near the upstream limit of the DTM coverage and a separate model for the portion of the tributary near the confluence with the NSR. Consistent with the model for the mainstem, a vertical variation in the roughness was applied, resulting in Manning's *n*-values that ranged from 0.027 to 0.040 in the main channel, and from 0.049 to 0.070 in the overbanks. Downstream boundary conditions were based on normal depth conditions with a starting energy slope set equal to the average bed slope near the confluence with the mainstem NSR, and did not include backwater effects from the mainstem since the timing of the peaks in the tributaries is likely to be different than in the mainstem.

Table 4.2. Summary of hydraulic model information for the primary tributaries in the project reach.

Tributary	Model Length (ft)	Number of Cross Sections	Average Cross Section Spacing (ft)
Allen Creek	3,640	20	85/107*
Long Creek	7,032	22	58/64*
Bear Creek	7,338	46	154
Pot Creek	3,699	32	113
Brushy Creek	13,977	22	85/202*
Pickle Creek	15,963	24	99/117*
Davis Creek	17,301	95	182
Leggets Branch	6,886	28	254/269*
Bralley Pool Creek	21,393	116	182
Merrill Creek	27,729	106	258
Baker Creek	15,435	70	218

*Modeled upstream and downstream portion of tributary

Each of the tributary models was run over a range of flows up to and including the 100-year peak flow, based on results from the hydrologic (HEC-1) models.

4.2.1. Reach-averaged Hydraulics

Reach-averaged hydraulic conditions in the tributaries were computed using the results from the HEC-RAS models, and dividing each of the tributaries into up- and downstream (and in some cases, middle) subreaches. The reach-averaged discharge, velocity, depth, and topwidth for the main channel at the 2-, 10-, and 100-year peak discharges are summarized in **Table 4.3**. At the 2-year event, a maximum reach-averaged velocity of 11.1 fps is indicated in Pickle Creek, and a maximum depth of 7.6 feet and maximum topwidth of 105 feet occur in Baker Creek. Maximum main-channel velocities, depths, and topwidths occur in the same tributaries at the 10- and 100-year events due to the relatively steep nature of Pickle Creek and the significant drainage area that contributes flow to Baker Creek. At the 10-year event, reach-averaged results indicate that velocities are as much as 15 fps (Pickle Creek), while maximum hydraulic depths are about 12 feet (Baker Creek), and topwidths exceed 130 feet (Baker Creek). Results for the 100-year event suggest maximum main channel velocities exceed 16 fps in Pickle Creek, maximum depths approach 15 feet in Baker Creek, and maximum topwidths exceed 140 feet in Baker Creek.

Location	2-year Peak Discharge					10-year Peak Discharge					100-year Peak Discharge				
	Total Flow (cfs)	Main Channel Flow (cfs)	Main Channel Velocity (ft/s)	Main Channel Hydraulic Depth (ft)	Main Channel Top Width (ft)	Total Flow (cfs)	Main Channel Flow (cfs)	Main Channel Velocity (ft/s)	Main Channel Hydraulic Depth (ft)	Main Channel Top Width (ft)	Total Flow (cfs)	Main Channel Flow (cfs)	Main Channel Velocity (ft/s)	Main Channel Hydraulic Depth (ft)	Main Channel Top Width (ft)
Allen-DS	927	875	6.4	3.4	41.0	1,882	1,730	8.7	5.0	41.0	2,655	2,397	9.8	6.0	41.0
Allen-Mid	927	820	6.7	3.6	33.8	1,882	1,570	9.1	5.1	33.8	2,655	2,133	10.1	6.2	33.8
Allen-US	927	798	6.2	4.1	32.5	1,882	1,519	8.2	5.9	32.5	2,655	2,068	9.2	7.1	32.5
Long-DS	904	742	7.8	5.1	19.2	1,858	1,404	10.2	7.3	19.2	2,609	1,884	11.4	8.7	19.2
Long-Mid	904	820	7.9	4.8	21.8	1,858	1,563	9.8	7.3	21.8	2,609	2,122	11.2	8.7	21.8
Long-US	904	497	8.7	6.7	8.9	1,858	870	11.0	9.0	8.9	2,609	1,154	12.9	10.1	8.9
Bear-DS	811	765	7.1	3.5	30.7	1,646	1,483	9.5	5.2	30.7	2,315	2,020	10.5	6.3	30.7
Bear-Mid	811	747	6.8	3.7	30.6	1,646	1,448	9.1	5.2	30.6	2,315	1,975	10.2	6.4	30.6
Bear-US	811	648	7.0	4.1	24.1	1,646	1,203	8.8	6.0	24.1	2,315	1,612	9.7	7.2	24.1
Pot-DS	1,663	1,275	7.8	4.8	34.6	3,364	2,300	9.6	6.8	34.6	4,762	3,050	10.7	8.0	34.6
Pot-Mid	1,201	945	7.4	4.4	29.8	2,417	1,700	9.0	6.4	29.8	3,411	2,241	9.5	8.0	29.8
Pot-US	1,201	1,074	8.0	4.7	28.6	2,417	2,008	10.2	6.8	28.6	3,411	2,712	11.3	8.3	28.6
Brushy-US	1,696	1,411	8.8	5.9	28.2	3,408	2,660	11.4	8.6	28.2	4,799	3,623	12.9	10.2	28.2
Brushy-DS	3,093	2,837	8.0	5.7	63.4	6,364	5,581	10.1	8.9	63.4	9,043	7,744	11.2	11.1	63.4
Pickle-DS	1,592	1,385	9.3	5.6	27.5	3,320	2,686	11.5	8.7	27.5	4,715	3,656	12.7	10.6	27.5
Pickle-Mid	1,592	1,478	11.1	5.1	26.1	3,320	2,891	14.8	7.5	26.1	4,715	3,966	16.5	9.2	26.1
Pickle-US	1,592	1,012	9.8	6.9	14.9	3,320	1,806	12.0	10.1	14.9	4,715	2,377	13.2	12.1	14.9
Davis-DS	1,266	1,059	7.1	4.0	38.4	2,948	2,313	9.9	6.3	38.4	4,257	3,241	11.2	7.8	38.4
Davis-Mid	1,266	949	5.0	5.4	32.7	2,948	2,072	7.4	7.9	32.7	4,257	2,887	8.7	9.5	32.7
Davis-US	1,266	1,092	8.2	4.6	30.2	2,948	2,328	10.5	7.5	30.2	4,257	3,188	11.3	9.4	30.2
Leggetts-DS	648	645	7.1	3.1	29.5	1,304	1,281	8.8	4.5	32.9	1,838	1,779	9.7	5.5	34.1
Leggetts-US	648	483	5.3	2.8	33.8	1,304	790	6.4	3.7	33.8	1,838	1,017	7.1	4.3	33.8
Bralley-DS	1,482	1,400	7.6	4.6	40.5	3,052	2,758	10.0	6.9	40.5	4,328	3,809	11.2	8.4	40.5
Bralley-Mid	1,482	1,224	7.6	4.8	35.6	3,052	2,360	9.7	7.1	35.6	4,328	3,211	10.7	8.6	35.6
Bralley-US	1,482	1,268	9.3	5.5	24.9	3,052	2,329	10.6	7.9	24.9	4,328	2,983	10.9	9.8	24.9
Merrill-DS	2,123	1,990	7.8	5.2	49.8	4,459	3,998	10.3	7.9	49.8	6,795	5,915	12.0	10.0	49.8
Merrill-Mid	2,123	1,758	7.7	5.4	44.3	4,459	3,484	10.0	8.3	44.3	6,795	5,112	11.5	10.5	44.3
Merrill-US	2,123	1,686	8.4	5.5	37.5	4,459	3,146	10.2	7.8	37.5	6,795	4,219	11.4	9.7	37.5
Baker-DS	4,538	4,538	6.3	7.6	94.8	9,424	9,184	6.9	12.1	111.3	13,427	12,682	7.7	14.8	113.6
Baker-US	4,538	4,482	6.0	7.2	105.0	9,424	9,233	6.9	10.2	132.7	13,427	13,030	7.5	12.4	143.9

Table 4.3. Summary of reach-averaged hydraulic conditions for the 2-, 10-, and 100-year events for subreaches of the primary tributaries.

5. SEDIMENT TRANSPORT

The sediment-transport analysis was conducted under existing and with-dam (i.e., project) conditions to evaluate potential sediment loading to the proposed reservoir and to determine the effects of the dam on downstream channel conditions. Sediment transport is typically evaluated with two components:

1. Wash load: The portion of the sediment load that is primarily fine material and is not found in significant amounts in the bed material.
2. Bed-material load: The portion of the sediment load that is transported along the channel bed and makes up the material that is found in appreciable quantities on the channel bed.

The wash-load component of the overall sediment load includes the fine sediments that are delivered to the channel from the watershed (watershed sediment yield) and the fine material that is eroded from the bed and banks. Typically, the wash load is not morphologically significant. The bed-material load is typically made up of coarser material that is eroded from the bed and banks, and is considered to be morphologically significant. In the project reach of the NSR, the bed and lower banks are composed primarily of shale that, when entrained by the flow, enters the system as coarse bed-material load and breaks down into fine wash load as it is transported downstream due to cycles of wetting and drying that cause slaking (Chapter 2; Allen et al., 2002).

5.1. Watershed Sediment Yield

Evaluation of the watershed sediment yield requires an assessment of the sediment sources in the watershed, the cover (vegetation type and density) and management practices, and the types of erosion (sheet, rill, ephemeral gulley) that are prevalent. This information, combined with hydrologic information, can then be used to estimate the watershed sediment yield using empirically derived relationships. For this study, the Modified Universal Soil Loss Equation (MUSLE) was used to estimate the sheet-and-rill sediment yield to the location of the proposed dam under existing, and with-dam, conditions. Estimates of the ephemeral gulley sediment yield were developed from the soil erosion literature (Laflen et al., 1986).

5.1.1. Soil Characteristics, Cover and Management Practices

The types of soil in the NSR watershed were identified using maps from the Soil Survey of Fannin County, Texas (NRCS, 2001) and the NRCS online Web Soil Survey. In general, the watershed includes:

1. Clayey and loamy, slightly acid to moderately alkaline soils on uplands,
2. Loamy, very strongly acid to neutral soils on terraces,
3. Loamy and clayey, moderately acid to neutral soils on uplands, and
4. Clayey and loamy, moderately alkaline soils on floodplains.

Each of these soil types is relatively erodible due to the loamy properties. Specific soil types and their physical properties are outlined in the soil survey and can be found on the online Web Soil Survey.

Based on information from the Texas State Soil and Water Conservation Board (TSSWCB, 1997), land in Fannin County is primarily used for pasture, crops, and range. Under the assumption that the project watershed has a similar distribution of land-use practices to Fannin County, in general, about 42 percent of the watershed is used for pasture, 26 percent is used for cropping, 24 percent is rangeland, and only 2 percent is forested. These values are consistent with recent assessments of land use (written comm., Loretta Mokry, Alan Plummer and Associates, April 2006) that indicate about 21 percent of the project watershed is currently being used for cropping, and estimated rates of cropland loss of about 0.5 percent per year (personal comm., Randy Moore, NRCS, 1996). Because land used for crops typically has relatively low ground cover (especially during the non-growing season when the soil is essentially bare), and there is a significant amount of cropland in the watershed, the potential for surface erosion is relatively high in the Texas Blackland Prairie region (Harmel et al., 2006).

5.1.2. Modified Universal Soil Loss Equation

The Modified Universal Soil Loss Equation (MUSLE) was developed to estimate sediment yields from watersheds based on single storms. The equation, as presented by Williams and Berndt (1972), differs from the original Universal Soil Loss Equation by inclusion of a runoff factor in place of a rainfall energy factor. Since it directly considers the runoff associated with individual storms, it is more applicable to the ephemeral streams within the project watershed where runoff and sediment delivery to the channel system is primarily the result of rainfall. The MUSLE is given by:

$$Y_S = \alpha(Vq_p)^{\beta} KLSCP \tag{5.1}$$

where Y_s = sediment yield for the storm in tons,
K = soil erodibility factor,
LS = topographic factor representing the combination of slope length and slope gradient,
C = cover and management factor,
P = erosion-control practice factor,
V = runoff volume for the storm in ac-ft, and
q_p = peak discharge of the storm in cfs.

Values for α and β can be derived through calibration when sufficient data are available. The most commonly used values for α and β are 95 and 0.56, respectively, and were derived from data in experimental watersheds in Texas and Nebraska. Although the MUSLE was originally developed to represent the total watershed sediment yield, the equation likely accounts for only the fine sediment (wash load) yield for the project watershed. The bed-material component of the total sediment load is discussed later in this chapter.

The soil erodibility factor (K) was obtained from the Fannin County Soil Survey maps and tables (NRCS, 2001), which delineate the specific soil types and summarize the K-factors for each soil type. Area-weighted K-factors were computed for each subbasin (Figure 3.8) in the project watershed under existing and with-dam conditions, and are summarized in **Table 5.1**.

The basin shape and topography factor (LS) is computed as:

$$LS = \left(\frac{\lambda}{72.6}\right)^n \left(0.065 + 0.0454S + 0.0065S^2\right) \tag{5.2}$$

Table 5.1. Summary of MUSLE factors for the individual subbasins in the project watershed.														
Basin	ID	Existing Conditions						With Dam Conditions						
		Area (sq mi)	Slope (%)	Average Length (ft)	K	n	LS	Area (sq mi)	Slope (%)	Average Length (ft)	K	n	LS
Baker1	107	1.30	1.24	1167	0.38	0.3	0.30	1.30	1.24	1167	0.38	0.3	0.30
Baker2	92	2.08	1.10	1616	0.33	0.3	0.31	2.08	1.10	1616	0.33	0.3	0.31
Baker3 (Mclure)	94	4.76	0.79	2183	0.32	0.3	0.29	4.76	0.79	2183	0.32	0.3	0.29
Baker4 (Moss)	68	6.19	0.65	2297	0.32	0.3	0.27	6.19	0.65	2297	0.32	0.3	0.27
Baker5	66	0.85	0.80	1051	0.32	0.3	0.24	0.85	0.80	1051	0.32	0.3	0.24
Baker6	56	2.22	0.51	1363	0.32	0.3	0.22	2.22	0.51	1363	0.32	0.3	0.22
Baker7	57	4.68	0.52	2002	0.32	0.3	0.24	4.68	0.52	2002	0.32	0.3	0.24
DS1	147	2.41	1.37	2062	0.38	0.3	0.38	2.41	1.37	2062	0.38	0.3	0.38
HB	151	1.68	1.20	1124	0.38	0.3	0.29	1.68	1.20	1124	0.38	0.3	0.29
Merrill	110	11.49	0.82	3057	0.34	0.3	0.33	8.65	0.68	3057	0.32	0.3	0.30
LRH1	153	8.59	1.28	4541	0.36	0.3	0.46	3.97	1.46	4541	0.36	0.3	0.50
Bralley Pool	123	7.95	0.66	2125	0.33	0.3	0.27	7.05	0.55	2125	0.32	0.3	0.25
Leggetts	152	2.53	0.87	1402	0.35	0.3	0.27	1.32	0.89	1402	0.32	0.3	0.27
LRH2	163	2.41	1.76	1816	0.33	0.3	0.43	1.90	1.95	1816	0.33	0.3	0.47
LRH3	145	0.88	1.12	2472	0.36	0.3	0.36	0.24	2.02	2472	0.36	0.3	0.53
Davis	125	6.99	0.70	2457	0.34	0.3	0.29	6.32	0.64	2457	0.32	0.3	0.28
LRH4A	126	0.00	1.00	759	0.32	0.3	0.24	0.00	1.00	759	0.32	0.3	0.24
LRH5	132	0.42	0.84	623	0.32	0.3	0.21	0.07	1.11	623	0.32	0.3	0.24
LRH4	167	3.39	1.62	2259	0.33	0.3	0.44	2.90	1.66	2259	0.33	0.3	0.44
LRH6	133	1.03	0.74	1090	0.33	0.3	0.23	0.51	0.86	1090	0.33	0.3	0.24
Pickle	124	6.93	0.72	2434	0.33	0.3	0.29	6.37	0.70	2434	0.32	0.3	0.29
LRH8	136	1.38	0.83	1187	0.33	0.3	0.25	0.77	0.76	1187	0.33	0.3	0.24
LRH7	164	2.10	1.29	1502	0.32	0.3	0.33	1.97	1.24	1502	0.32	0.3	0.33
LRH9	170	4.73	1.11	2373	0.34	0.3	0.35	3.90	1.14	2373	0.34	0.3	0.36
Brushy1	128	1.44	0.88	1398	0.33	0.3	0.27	1.02	0.70	1398	0.33	0.3	0.24
Brushy2	103	6.37	0.63	2787	0.34	0.3	0.29	6.36	0.62	2787	0.34	0.3	0.29
Pot1	105	0.02	1.00	1827	0.32	0.3	0.31	0.00	1.00	1827	0.32	0.3	0.31
Pot2	102	2.01	0.76	1327	0.32	0.3	0.25	2.01	0.76	1327	0.32	0.3	0.25
Pot3	111	4.98	0.63	2345	0.33	0.3	0.27	4.97	0.63	2345	0.33	0.3	0.27
LRH9A	161	0.73	1.53	1025	0.32	0.3	0.33	0.68	1.40	1025	0.32	0.3	0.31
Bear	134	3.32	0.76	1569	0.34	0.3	0.26	3.29	0.74	1569	0.34	0.3	0.26
LRH10	142	0.02	1.00	2133	0.32	0.3	0.32	0.00	1.00	2133	0.32	0.3	0.32
Long	179	3.23	1.40	1849	0.32	0.3	0.37	3.22	1.40	1849	0.32	0.3	0.37
LRH11	146	0.11	0.97	13236	0.32	0.3	0.55	0.11	0.97	13236	0.32	0.3	0.55
Allen	139	4.49	0.54	1870	0.32	0.3	0.24	4.48	0.54	1870	0.32	0.3	0.24
LRH12	148	0.69	0.95	897	0.32	0.3	0.24	0.69	0.94	897	0.32	0.3	0.24
LRH13	174	2.93	1.28	1977	0.32	0.3	0.36	2.93	1.28	1977	0.32	0.3	0.36
LRH14A	141	0.02	1.00	2193	0.32	0.3	0.32	0.02	1.00	2193	0.32	0.3	0.32
LRH15	135	2.52	0.66	1446	0.32	0.3	0.24	2.52	0.66	1446	0.32	0.3	0.24
LRH14	166	2.03	1.07	1939	0.32	0.3	0.32	2.03	1.07	1939	0.32	0.3	0.32
LRH16	160	4.03	1.03	2798	0.32	0.3	0.35	4.03	1.03	2798	0.32	0.3	0.35

where λ = slope length (distance from the point of overland flow origin to the point where the water enters a well-defined channel),
S = percent slope, and n is an exponent depending upon the slope.
The exponent n is given by: n=0.3 for slope <= 3 percent
n=0.4 for slope <= 4 percent
n=0.5 for slope >= 5 percent

The slope length and the basin slope were measured from the DTM developed by CP&Y for each of the subbasins (Figure 3.8) that make up the contributing watershed to the dam site.

The measured values, the exponent (n) and the resulting LS factors are summarized for existing and with-dam conditions in Table 5.1.

The cover and management factor (C) is based on the vegetation type, height and percentage of ground cover, and is derived from SCS Agriculture Handbook Number 537 (1978). For the project watershed, a composite C-factor of 0.17 was computed for the overall watershed assuming that the percentage of cropland, rangeland, pastureland, and forestland is similar to the values reported by TSSWCB (1997) based on the individual land-use C-factors presented in **Table 5.2**.

Table 5.2. Summary of selected C-factors for the individual land-use types in the project watershed.

Type of Land Use	Percent (%)	C-factor
Forest	24.0	0.01
Row Crops	3.8	0.20
Close Crops	22.3	0.17
Pasture	41.9	0.17
Rangeland	24.0	0.17
Not Identified/Misc	5.8	0.20
Composite C-Value:		**0.17**

The erosion-control practice factor (P) accounts for the effect of conservation practices such as contouring, strip cropping, and terracing on erosion. It is defined as the ratio of soil loss using one of these practices to the loss using straight row farming up and down the slope. To be conservative, a P-factor of 1.0 was selected for the MUSLE calculations in this study, even though there are significant erosion-control measures in the basin.

Processes of erosion and sedimentation are cumulative over the long term, so it is necessary to evaluate sediment transport not only for a specific flood event, but also for the intervening smaller flows. For purposes of analyzing the long-term erosion potential, the representative annual event can be more accurately defined by considering individual storm events independently and weighting the effect of each based on their probability of occurrence. This is accomplished by integrating the flow-duration curve over discrete intervals resulting in the following equation (Mussetter et al., 1994):

$$Y_m=0.015Y_{100}+0.015Y_{50}+0.04Y_{25}+0.08Y_{10}+0.2Y_5+0.4Y_2 \qquad (5.3)$$

where Y_m = magnitude of the average annual event (i.e., sediment yield) and
Y_i = magnitude of the event for the 2-, 5-, 10-, 25-, 50-, and 100-year return period storms.

Watershed sheet-and-rill sediment yields were computed from each subbasin (Figure 3.8) for the 2- through 100-year storm events using the MUSLE with the above factors and the results from the hydrologic analysis (peak flow and storm volume) that were developed using the HEC-1 models or from the rating curves (Chapter 3). Annual sediment yields were computed using Equation 5.3. Although previous studies (Smith et al., 1984) have indicated that the sediment yields predicted by the MUSLE are reasonable for Blackland Prairie soils, computed annual sediment yields using the identified parameters in the MUSLE (Table 5.1) are about 37 percent of observed rates of sheet-and-rill erosion in the Blackland Prairie region that are about 2.0 t/ac/yr (Alan Plummer and Associates, 2005). Therefore, the value of the alpha coefficient (95) in the MUSLE was adjusted by a factor of 2.7, resulting in a new alpha coefficient of about 257 that is similar to values successfully used in other areas with high erosion rates (Mussetter et al., 1994).

5.1.3. Ephemeral Gully Erosion

The overall watershed sediment yield includes not only the portion that is accounted for by the MUSLE calculations (sheet-and-rill erosion), but also the portion of fine sediments that are eroded by ephemeral gullies. Ephemeral gullies are defined as small channels that form in croplands or nonvegetated, exposed soils at locations where the rills join and the macrotopography allows for concentrated flow. Ephemeral gullies are formed by the shearing forces of concentrated flow, and are typically removed (filled) on an annual basis through tilling and other crop-related practices (Laflen et al., 1986).

Initial estimates of sediment yield from ephemeral gulley erosion were computed using the SCS Ephemeral Gulley Erosion Model (Woodward, 1999), and indicated that a maximum annual detachment rate of about 0.4 t/ac would result from ephemeral gulley erosion within the project watershed. This estimate is believed to somewhat under-predict the actual sediment load that results from ephemeral gullies (pers. comm., Randy Moore, NRCS, 2006). The soil erosion literature indicates that ephemeral gullies may produce as much as 1.5 times the amount of sediment that is predicted by the Universal Soil Loss Equation (USLE), but typically the range is a factor of 0.25 to 1.0 (Laflen et al., 1986). Therefore, the amount of fine sediment volume that is eroded from ephemeral gullies was estimated as 1.0 times the sheet-and-rill erosion predicted by the MUSLE for the portion of land used for cropping, where ephemeral gullies form. The resulting ephemeral gulley erosion rates are 0.26 times the MUSLE (sheet-and-rill) erosion rates since approximately 26 percent of the project watershed is cropland.

5.1.4. Sediment Delivery Ratios

The portion of the gross sheet-and-rill erosion that is delivered to an outlet in a channel depends on the drainage area, watershed slope, drainage density, and runoff (Gottschalk, 1964). The sediment delivery ratio (SDR) expresses the percentage of on-site eroded material that reaches a designated downstream location. Renfro (1975) developed an equation for the SDR in the Blackland Prairie using measured gross erosion rates and watershed sediment yields:

$$\log_{10}SDR=1.8768-0.141911(\log_{10}(10(A))) \tag{5.4}$$

where SDR = sediment delivery ratio percentage, and
A = drainage area in square miles.

Compared with other relationships and estimates of the SDR (Shen and Julien, 1993; Alan Plummer and Associates, 2005), the relationship presented in Equation 5.4 produces the largest SDR values, and was therefore adopted to conservatively estimate the fine sediment yield resulting from sheet-and-rill erosion in this study.

The concept of the SDR also applies to sediment yields resulting from ephemeral gullies, but the percentage of eroded material is typically higher than for sheet-and-rill erosion because the fine material eroded from gullies is transported as suspended load by concentrated flow. Previous work indicates that the SDR for ephemeral gulley erosion in the Blackland Prairie should be about 0.67 (Alan Plummer and Associates, 2005). To compute the SDR for sediments eroded in ephemeral gullies, the relationship between SDR and drainage area provided by Shen and Julien (1993) was adopted, and the coefficient was adjusted to compute a basin area-weighted SDR of 0.67. The resulting equation for estimating the SDR for ephemeral gullies is given by:

$$SDR_{ege}=0.43(A)^{-0.31} \tag{5.5}$$

where SDR_{ege} = sediment delivery ratio for ephemeral gulley erosion, and
A = basin area in square miles.

5.1.5. Existing Conditions Watershed Sediment Yield

Gross sheet-and-rill and gross ephemeral gulley erosion volumes were computed for each sub-basin (Figure 3.8) for the 2-, 5-, 10-, 25-, 50-, and 100-year events, and annual sediment yields were computed using Equation 5.3. Details of the computations are provided in **Appendix** E. The computed SDR values were then applied to the annual gross sheet-and-rill and gross ephemeral gulley erosion volumes to obtain the overall sediment yield from the project watershed. The results indicate that about 81,000 t/yr of fine sediment will be eroded from the watershed upstream from the location of the proposed dam (**Figure 5.1**).

5.1.6. With-Dam Sediment Yield

The estimates of the sheet-and-rill and ephemeral gulley erosion were revised to include the effects of the reservoir on watershed sediment yield. The basin parameters (i.e., basin area, watershed slope and slope length) that were used as input to the existing conditions sheet-and-rill (MUSLE) calculations were adjusted using the reservoir area at a conservation pool elevation of 551 feet (Appendix E). Compared to existing conditions, the basins located partially or entirely within the conservation pool have reduced basin areas and slopes, but the slope lengths are similar (Table 5.1). The procedures for estimation of sediment yield from ephemeral gulley erosion and application of the SDR under existing conditions were used for with-dam conditions, and indicate that the total watershed sediment yield would be reduced from 81,000 tons under existing conditions to about 69,000 tons under project conditions (Figure 5.1).

5.1.7. Worst-Case Sediment Yield

To determine the worst-case sediment yield from the project watershed, an estimate of the sheet-and-rill erosion was developed by assuming that the entire watershed was composed of cropland, and by using the highest measured annual sheet-and-rill erosion rates in the Blackland Prairie (Greiner, 1982). The measured annual sheet-and-rill erosion rates (3.74 t/ac) were applied uniformly over the watershed and the methods for estimating ephemeral gulley erosion and the SDR described above were incorporated into the computations for worst-case watershed sediment yields (Appendix E). The results indicate that the worst-case annual sediment yield delivered to the dam site would be about 147,000 tons under existing conditions, and about 90,000 tons under with-dam conditions (**Figure 5.2**).

5.2. Total Sediment Yield

The total sediment yield to the proposed dam location includes the watershed sediment yield (transported as wash load) and the sediment yield that results from erosion of the bed and banks of the mainstem NSR and its tributaries (channel sediment yield). In systems such as the NSR that have a sediment supply that is less than the hydraulic capacity of the channel to

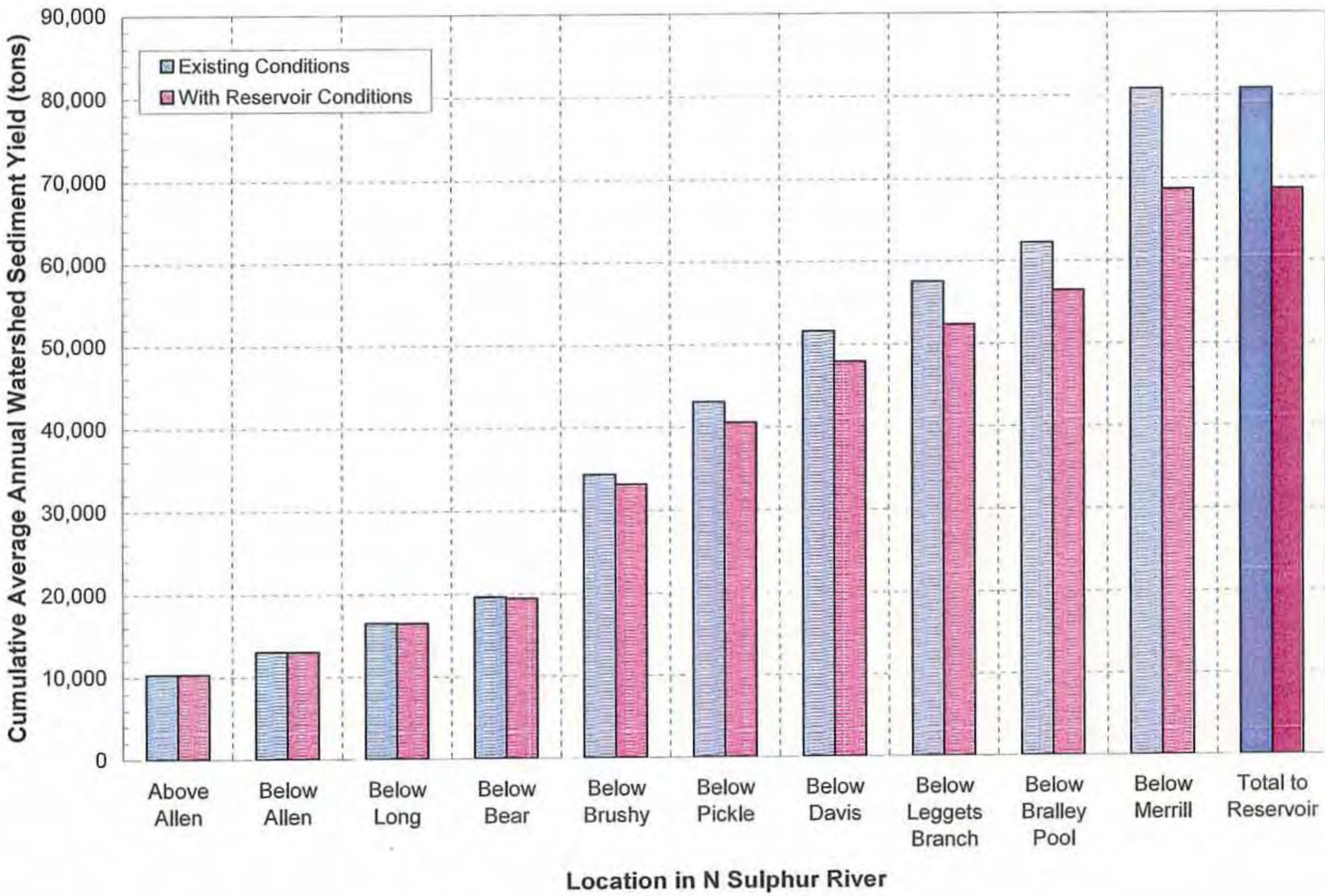


Figure 5.1. Cumulative total watershed sediment yield from the contributing basin at various locations within the project reach under existing and with-dam conditions.

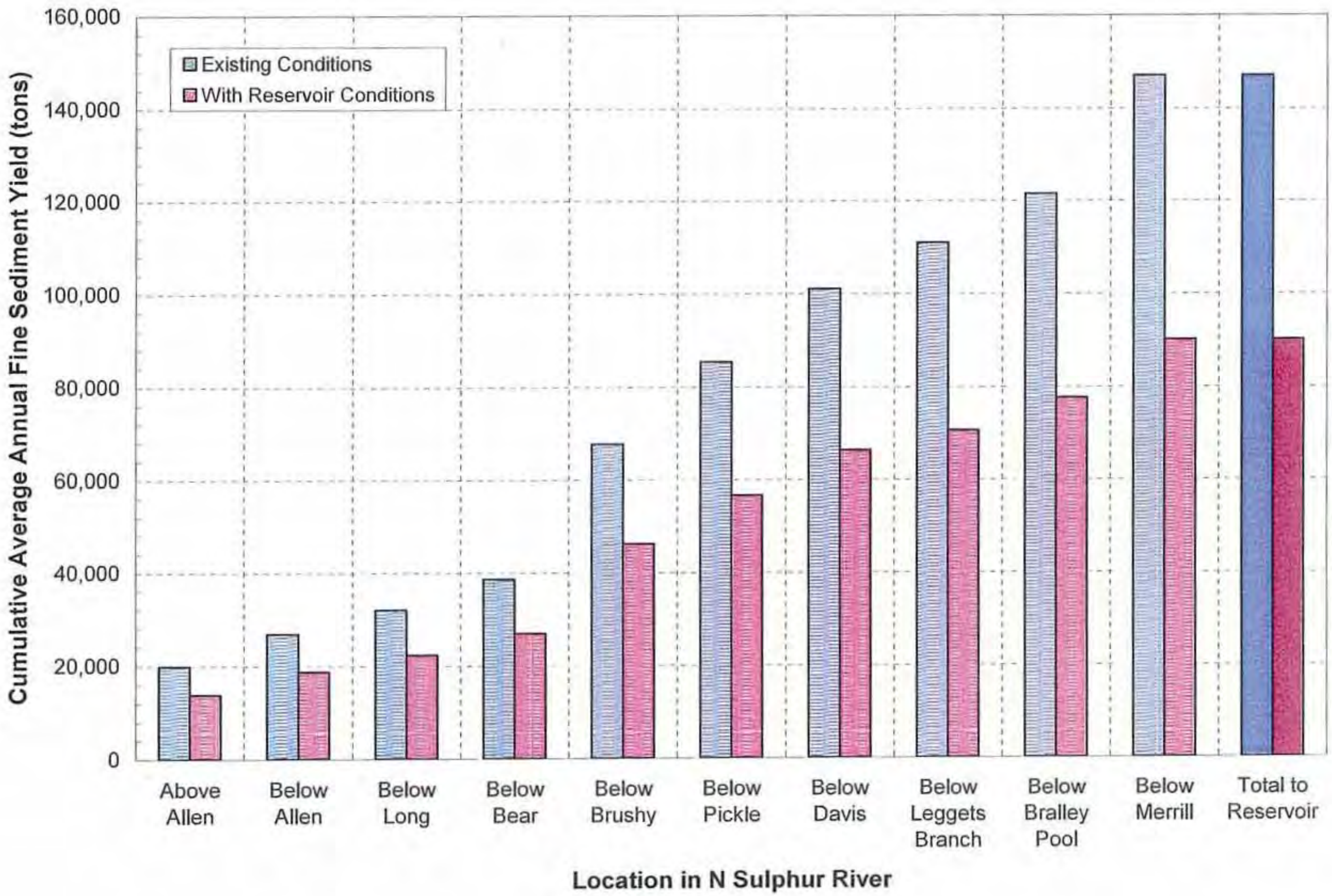


Figure 5.2. Cumulative total watershed sediment yield from the contributing basin at various locations within the project reach for the worst-case watershed sediment yields under existing and with-dam conditions.

convey sediment load (i.e., supply-limited conditions), the channel geometry typically responds through bed incision and/or erosion of the banks. In systems that have a sediment supply that exceeds the hydraulic capacity (i.e., capacity-limited conditions), net aggradation due to sediment deposition is typically expected. For the project reach of the NSR, a sediment-routing analysis was conducted to evaluate the expected total sediment yields by accounting for the observed supply-limited conditions. To evaluate the total sediment yield under capacity-limited conditions (worst-case sediment loading), a sediment-continuity analysis was performed. These two analyses are discussed in the following sections.

5.2.1. Bed-material Capacity Calculations

An estimation of the bed-material transport capacity was necessary to verify the assumption of supply-limited conditions for the sediment-routing analysis and to perform the sediment-continuity analysis for capacity-limited (worst-case) conditions. The bed-material capacity of each subreach in the mainstem NSR (Table 4.1; Figure 4.2) and in the tributaries was computed using the Meyer-Peter and Müller bed-load transport equation (Meyer-Peter and Müller, 1948) and Einstein's depth-integration of the suspended-bed sediment discharge (Einstein, 1950). Input for the capacity calculations included representative sediment gradations and specific gravities of the shale-derived material, and hydraulic data from the HEC-RAS model. The representative gradation for the mainstem shale material was based on the average of the dry sieve gradations from Samples NSR2 through NSR8 (Figure 2.38), and the representative tributary gradation was based on the average dry sieve gradations from Samples BP1, BP2, and BC1 (Figure 2.41). Specific gravity values for the mainstem were based on average bulk specific gravities of Samples NSR2 through NSR8, and an average bulk specific gravity from Samples BP1, BP2, and BC1 was used for the tributaries. The D_{16} (size with 16-percent passing), D_{50} (median diameter), D_{84} (size with 84-percent passing), and the specific gravity values are summarized in **Table 5.3**.

Bed-material transport capacity rating curves were developed for each subreach in the mainstem and the downstream subreaches in the primary tributaries by computing the bed-material load for a range of discharges using the reach-averaged hydraulic data presented in Chapter 4. The computed rating curves for the mainstem and the primary tributaries are presented in **Figures 5.3 and 5.4**. The rating curves were then integrated over the subreach hydrographs (Chapter 3) for the 2- through 100-year events and average annual bed-material capacities were computed using Equation 5.3.

5.2.2. Sediment-Routing Analysis

A sediment-routing analysis was performed using the subreaches developed for the reach-averaged hydraulic computations (Table 4.1 and Figure 4.2), estimated upstream and tributary sediment supplies, and estimated annual bed-and-bank erosion rates. The sediment-routing analysis was performed on an **annual** basis using the steps outlined in **Figure 5.5** as follows:

1. The upstream sediment supply was estimated by multiplying the computed hydraulic capacity of the upstream subreach (Subreach 1) by the percent area of the channel bed in Subreach 1 that is composed of depositional bars. The percent area with depositional bars is believed to represent the portion of the capacity that is supplied to the subreach (Struiksma, 1999), and was measured by delineating vegetated or topographically discernible bars using digital orthophotographic images taken in February 2002.

Table 5.3. Summary of gradation information used in the sediment-transport analysis.										
Subreach	Bed material capacity calculations					Shale break down to sand/washload Calculations				
	Sample	D16 (mm)	D50 (mm)	D84 (mm)	S.G.	Sample	D16 (mm)	D50 (mm)	D84 (mm)	% Sand
1	Average of All Mainstem (Dry) Sample Gradations	0.58	2.38	7.52	2.50	NSR #2 (Wet)	0.30	1.72	6.76	14%
2		0.58	2.38	7.52	2.50	NSR #2 (Wet)	0.30	1.72	6.76	14%
3		0.58	2.38	7.52	2.50	NSR #2 (Wet)	0.30	1.72	6.76	14%
4		0.58	2.38	7.52	2.50	Avg NSR#3,#4 (Wet)	0.67	3.19	10.52	56%
5		0.58	2.38	7.52	2.50	Avg NSR#3,#4 (Wet)	0.67	3.19	10.52	56%
6		0.58	2.38	7.52	2.50	Avg NSR#5,#6 (Wet)	0.52	2.33	6.91	65%
7		0.58	2.38	7.52	2.50	Avg NSR#5,#6 (Wet)	0.52	2.33	6.91	65%
8		0.58	2.38	7.52	2.50	Avg NSR#5,#6 (Wet)	0.52	2.33	6.91	65%
9		0.58	2.38	7.52	2.50	NSR #7 (Wet)	0.75	2.33	6.50	73%
10		0.58	2.38	7.52	2.50	NSR #8 (Wet)	0.69	2.18	6.16	86%
Allen	Average of All Tribubutary (Dry) Sample Gradations	0.65	2.79	9.75	2.53	Average of All Tribubutary (Wet) Sample Gradations	0.65	2.79	9.75	49%
Long		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Bear		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Pot		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Brushy		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Pickle		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Davis		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Leggetts		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Bralley Poc		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Merrill		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%
Baker		0.65	2.79	9.75	2.53		0.65	2.79	9.75	49%

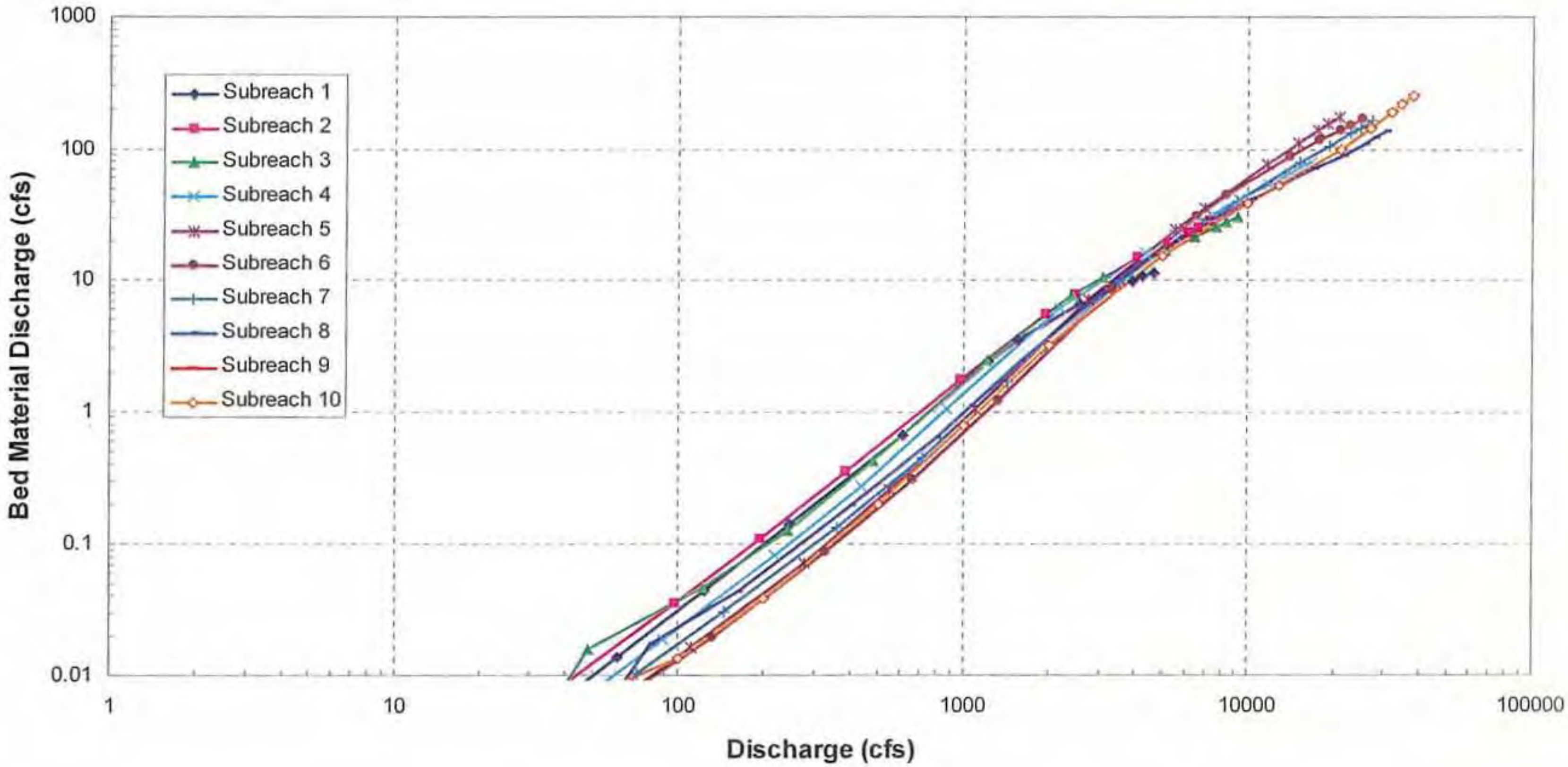


Figure 5.3. Computed bed-material transport capacity rating curves for the 10 subreaches in the mainstem North Sulphur River using MPM-Einstein equation.

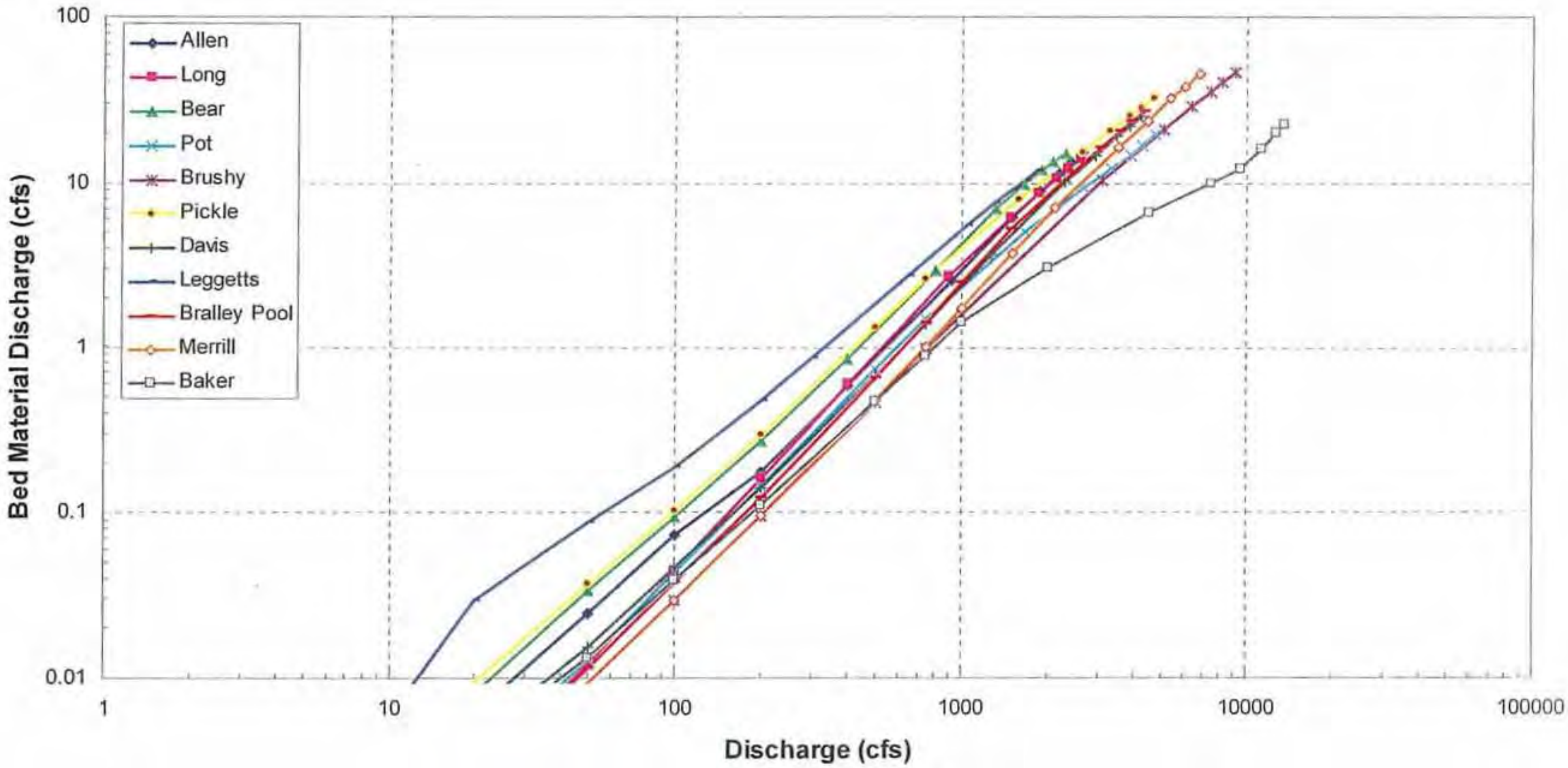
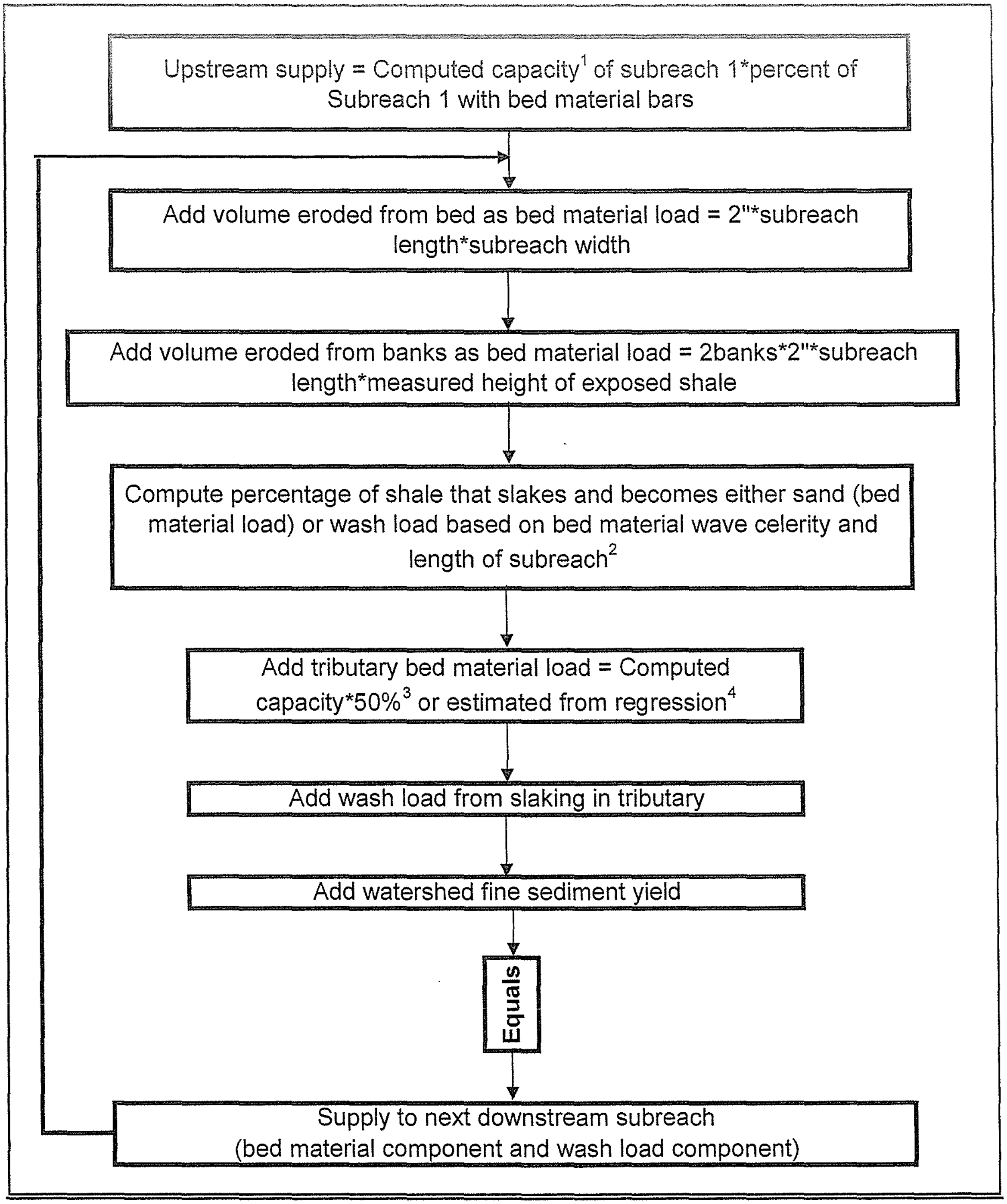


Figure 5.4. Computed bed-material transport capacity rating curves for the primary tributaries to the North Sulphur River using MPM-Einstein equation.



[1]Computed capacity based on representative dry gradation.

[2]Portion of shale that does not slake to wash load or sand remains shale bed material load.

[3]For primary tributaries: 50% is average of measured percentage of tributary bed that had bed material bars.

[4]For secondary tributaries: estimated from regression on computed primary tributary load as function of drainage area.

Figure 5.5. Flow chart for the sediment-routing computations.

2. The annual volume of material eroded from the shale bed was estimated using an annual degradation volume of 2 inches (Allen et al., 2002) distributed uniformly over the bed area of the subreach.

3. The annual volume of material eroded from the shale banks was estimated using an annual bank retreat of 2 inches (i.e., 4 inches of channel widening) (Allen et al., 2002), the measured (field-observed) height of the exposed shale, and the length of the subreach.

4. The percentage of shale that slakes and becomes either sand (bed-material load) or wash load was estimated using the celerity of the bed-material wave, the length of the subreach, and the time period over which the material is transported. The bed-material wave celerity (C_n) was computed from Li et al. (1988):

$$C_n=2.33(a(b-c)V^bd^{(c-1)})^{0.96} \tag{5.6}$$

where V and d = the main channel velocity and depth of flow, respectively (obtained from the reach-averaged hydraulic calculations), and
a, b, and c are solved by performing a multiple regression analysis on the relation:

$$q_s=aV^bd^c \tag{5.7}$$

where q_s = the unit bed-material sediment-transport rate computed using the total rate in Step 1 and the reach-averaged topwidth.

The distance that a shale particle travels before breaking down was estimated by multiplying the computed bed-material wave celerity by the time that the particle is subjected to transport by flowing water. This was estimated as the average time period for which flow is greater than 100 cfs at the gage times two cycles. Based on the unit discharge, a discharge of 100 cfs at the gage would represent about 40 cfs at the dam site, and was selected because the computed rating curves (Figure 5.3) indicate that flows below this discharge do not transport appreciable amounts of bed material. Two cycles were selected because previous work indicates that it requires two wetting-drying cycles for a particle of the Taylor/Ozan shale to lose about 50 percent of its weight (Allen et al., 2002).

After computing the percentage of shale bed material that breaks down within the subreach, the volume of remaining sand was computed using the percent of sand material based on the wet sieve analysis of the representative subreach sample (Figure 2.39, Table 5.3). It should be noted that the computations in this step do not affect the total sediment load estimates, since the overall volume of material is the same whether it is transported as wash load or bed-material load.

5. The bed-material load from the primary tributaries entering the subreach was estimated by multiplying the computed hydraulic capacity of the tributary by the percent area of the tributary that is composed of depositional bars. Similar to the upstream supply in Step 1, the percent area was measured by delineating vegetated or topographically-discernible bars using digital orthophotographic images taken in February 2002. However, because the orthophoto was not of sufficient resolution to delineate the bars in the smaller tributaries, the percent area calculations were conducted for the five largest tributaries (Brushy Creek, Davis Creek, Bralley Pool Creek, Merrill Creek and Baker Creek), and

resulted in an average of 27.6-percent area with depositional bars. To be conservative, the computed average was replaced with a value of 50 percent for each of the tributaries. The bed-material load from the smaller, un-named tributaries was estimated by performing a linear regression on the computed bed material load from the primary tributaries as a function of basin area, and resulted in the following regression equation:

6.

$$G_b = 351.42 * A + 54.174 \tag{5.8}$$

where G_b = tributary bed-material load in tons and
A = basin area in square miles.

The coefficient of determination (R^2) value for the regression was 0.91, indicating that tributary drainage area is a reasonable predictor for tributary bed material load.

7. The portion of the tributary bed-material load that breaks down into sand or wash-load material was not computed, since the time period over which it is subjected to transport by flowing water is not known. This simplification does not affect the overall amount of sediment supplied by the tributaries, because the volume of material is the same whether it is transported as wash load or bed-material load.

8. The bed-material and wash-load components of the total sediment load in Steps 1 through 6 are added to the computed watershed fine sediment yield from the subbasins that contribute to the subreach (Section 5.1). This total sediment load represents the supply to the next downstream subreach, and replaces Step 1 for all of the subreaches below Subreach 1.

The sediment-routing analysis was carried out under existing conditions (pre-project) to provide the best estimate of the total annual sediment yield, and indicates that about 174,000 t/yr (86 ac-ft/yr) will be transported to the proposed location of the dam. To verify the assumption that the system is supply-limited, the annual sediment loads at the downstream limit of each of the subreaches were compared to the computed hydraulic capacities, which indicated that supply-limited conditions occur throughout the project reach (i.e., the annual sediment loads were less than the computed capacities). An additional verification of this approach was carried out by comparing the measured percent area with depositional bars to the ratio of the computed sediment load to the hydraulic capacity as a percentage for each subreach (**Figure 5.6**). The comparison indicates that the ratio of the computed sediment load to the hydraulic capacity is generally greater than the percent area composed of depositional bars, and, therefore, the estimates are conservatively high.

The sediment routing was also carried out for with-dam conditions. This analysis assumed that the bed-material transport capacity is negligible within the reservoir, so bed-material transport occurs only in Subreaches 1 (upstream from the reservoir) and 10 (downstream from the reservoir). All sand material that results from slaking was deposited in the subreach. For the tributary sediment loading, the bed-material sediment loads computed for existing conditions were converted to wash load or deposited sands, since the bed material would be transported to the conservation pool, where it would be subjected to wetting and drying cycles as the pool elevation fluctuates. The results indicate that the total sediment load deposited upstream from the dam will be about 104,000 t/yr (~51 ac-ft/yr), of which about 20,000 t/yr is sand and the remainder is fine material.

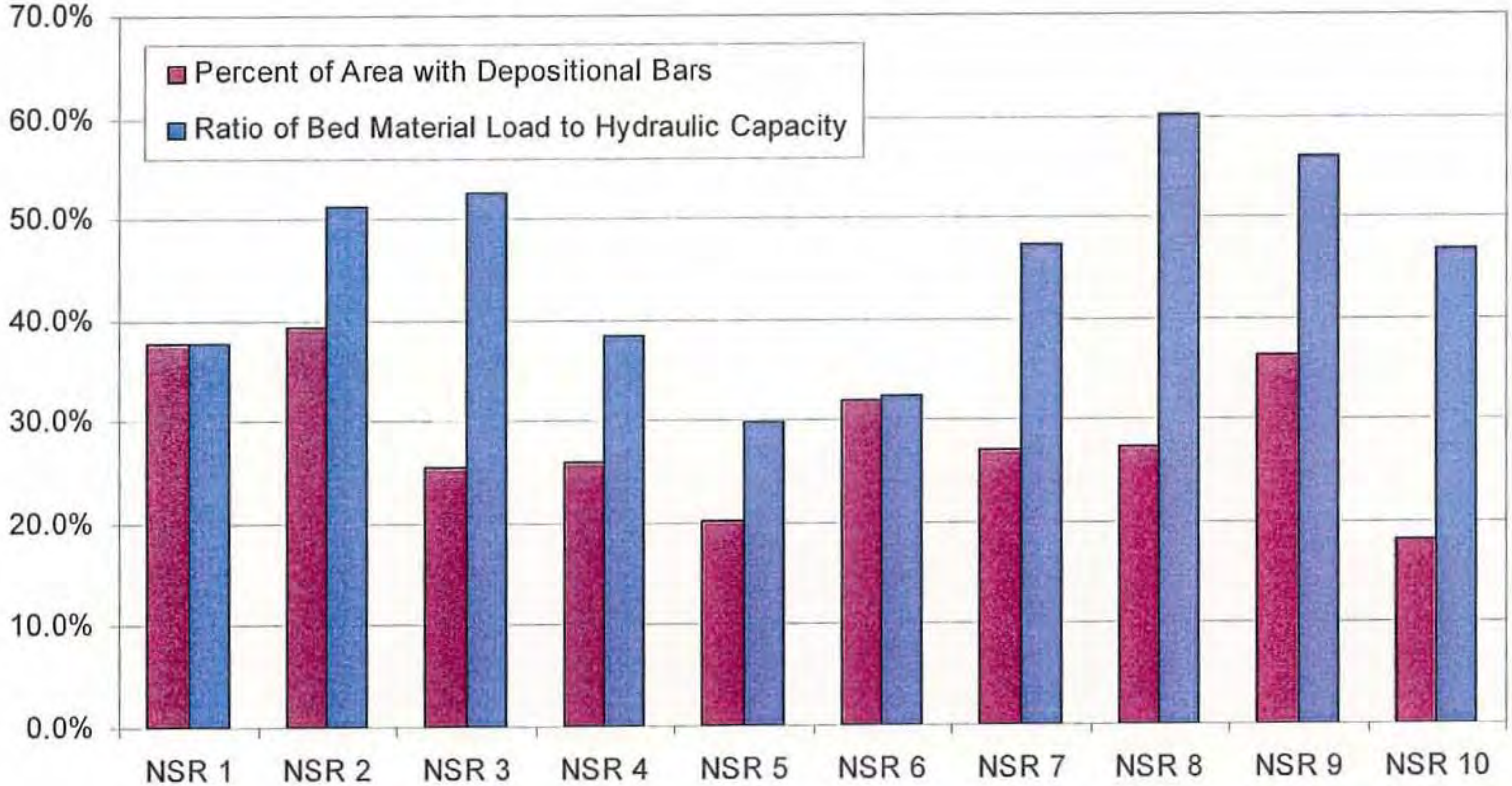


Figure 5.6. Measured percent area composed of depositional bars and the ratio (as percentage) of computed annual bed-material load to the hydraulic capacity for each subreach from the sediment routing analysis.

5.2.3. Capacity-Limited Sediment-Continuity Analysis

Worst-case channel sediment yields would result if the amount of bed-material supply to the system exceeds the hydraulic capacity. Although capacity-limited conditions are clearly not representative of the NSR, a sediment-continuity analysis was performed to evaluate maximum channel sediment yields. The sediment-continuity analysis involves comparing the upstream and tributary supply to a given subreach with the computed hydraulic capacity. If the supply exceeds the capacity, deposition occurs and the supply to the next downstream subreach is limited by the capacity of the current subreach. If the capacity exceeds the supply, degradation is indicated and the deficit is balanced through erosion of the channel bed and banks. The analysis was carried out using the bed-material capacity rating curves for the mainstem NSR and for the primary tributaries (Figures 5.3 and 5.4), and the wash-load component was accounted for by adding the watershed fine sediment yields (Section 5.1.5) and the amount of wash load that would result from breakdown of the shale material. To be conservative, it was assumed that the upstream bed-material supply for each subreach completely breaks down to wash load (or sand), and that erosion of the channel bed and banks balances the reduction in sediment load, thereby maintaining a sediment load that equals the hydraulic capacity at the downstream limit of the subreach Results from the analysis indicate that the total volume delivered to the proposed dam site could be about 373,000 t/yr (184 ac-ft/yr) if there was an unlimited supply of bed material to the system. Aggradation/degradation depths for each subreach were computed by dividing computed volume of deposited or eroded sediments by the bed area of each subreach. The results indicate that net aggradation rates of less than 0.2 in./yr would occur with an unlimited supply of bed material. Net aggradation in the project reach is clearly not representative of observed conditions, indicating the assumption of unlimited bed-material supply overestimates actual supply rates (**Figure 5.7**).

The sediment-continuity analysis was also carried out for with-dam conditions to evaluate worst-case channel sediment yields. Consistent with the sediment-routing analysis for with-dam conditions, it was assumed that the bed-material capacity is negligible within the reservoir, so bed-material transport occurs only in Subreaches 1 (upstream from the reservoir) and 10 (downstream from the reservoir). All sand material that results from break down of the shale was deposited in the subreach. For the tributary sediment loading, the bed-material sediment loads computed for existing conditions were converted to wash load or deposited sand, since the bed material would be transported to the conservation pool, where it would be subjected to wetting and drying cycles as the pool elevation fluctuates. The results indicate that the total sediment load deposited upstream from the dam will be about 128,000 t/yr (63.3 ac-ft/yr), of which about 20,000 t/yr is sand and the remainder is fine material.

5.2.4. Worst-Case Watershed Sediment Yield and Summary of Total Sediment Yields

The worst-case watershed fine sediment yields were incorporated into the sediment-routing and sediment-continuity analyses to determine the impacts of extreme watershed erosion on the total sediment loading to the proposed dam site. The analysis was carried out for existing and with-dam conditions, and indicates that for the best-estimate of the channel yield (bed-material supply limitations), the worst-case watershed sediment yield increases the total sediment yield to the dam by a factor of 1.4 under existing conditions and 1.2 under with-dam conditions. For the worst-case channel yield (unlimited bed-material supply), the worst-case watershed sediment yield increases the total sediment yield to the dam by a factor of 1.2 under existing and with-dam conditions (**Figure 5.8**).

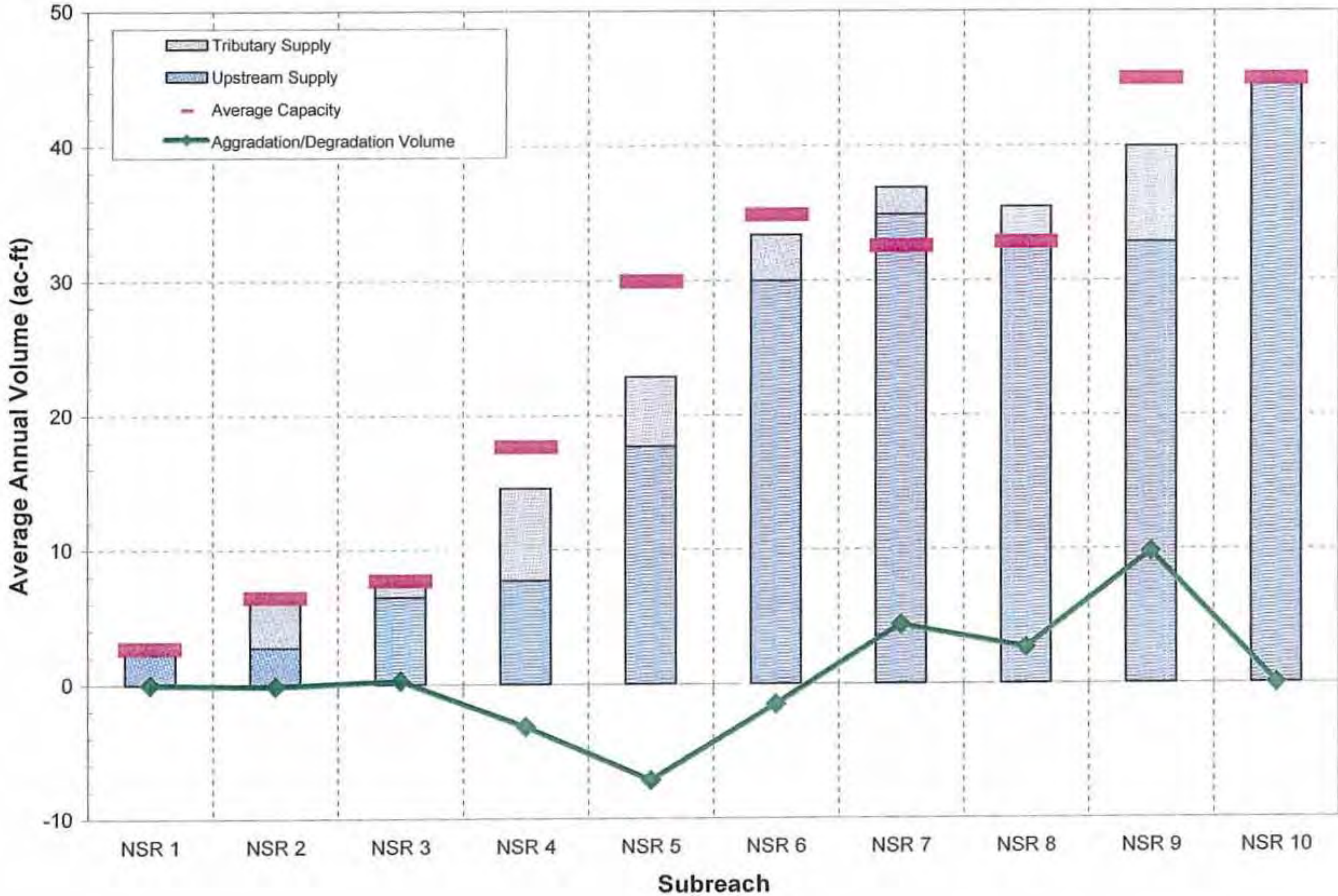


Figure 5.7. Computed upstream and tributary bed-material supply, hydraulic capacity, and the resulting aggradation/degradation volume for the sediment-continuity analysis (worst-case channel sediment yield with no supply limitations).

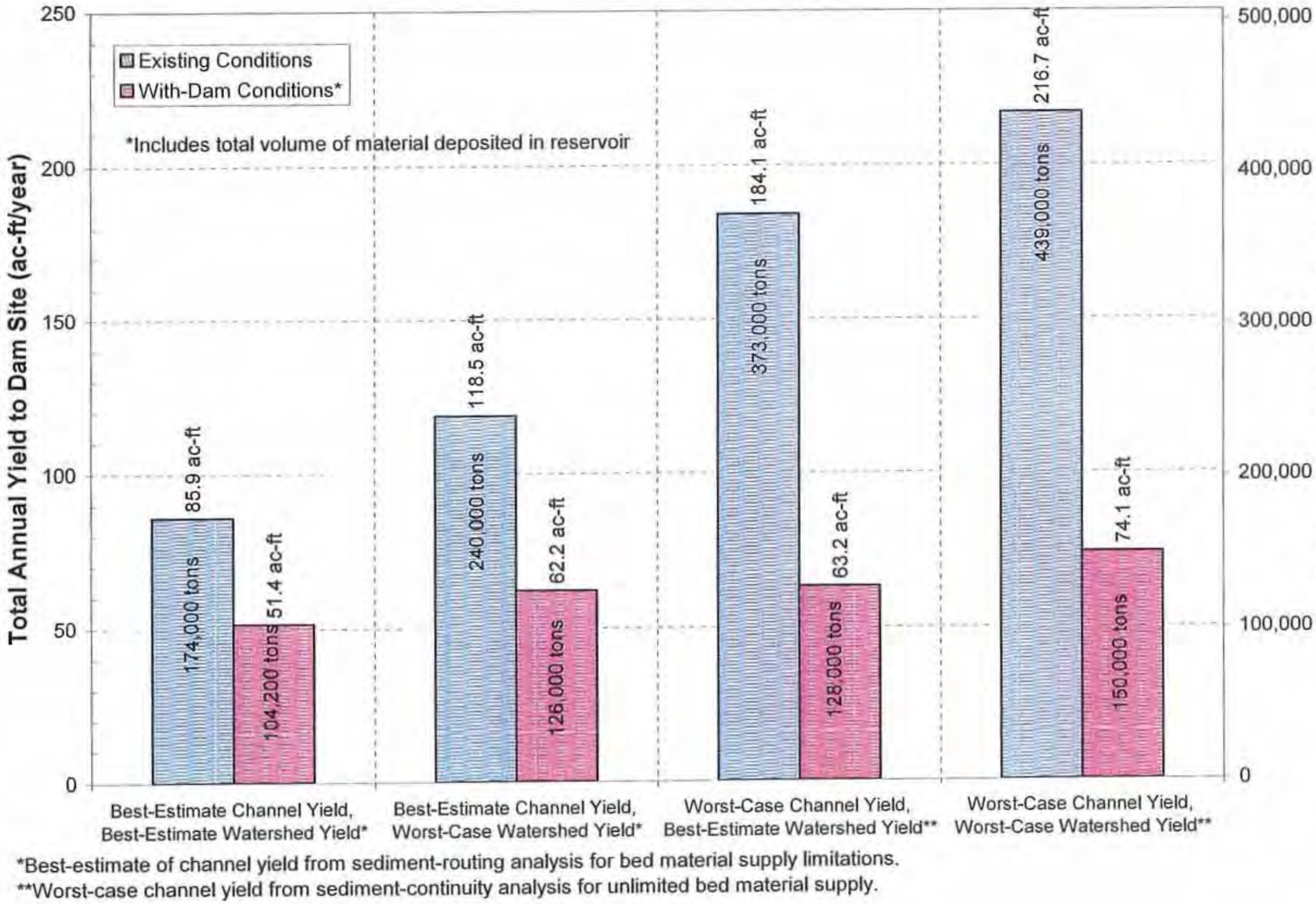


Figure 5.8. Summary of total sediment yield to the proposed location of the dam for the best-estimates and worst-case watershed and channel sediment yields under existing and with-dam conditions.

5.2.5. Summary

In summary, under conservative assumptions regarding existing conditions in the watershed, and assuming that the channel is supply limited, which is the most appropriate assumption based on the observed geomorphic conditions, the best estimate of annual sediment yield to the dam site under pre-project (without-dam) conditions is 85.9 ac-ft (174,000 tons). With the reservoir in place, the contributing watershed area is reduced as is the length of channel that is supplying sediment, and therefore, the annual sediment yield to the reservoir reduces to 51.4 ac-ft (104,000 tons). Therefore, the best conservative estimate of sediment delivery to the 160,235 ac-ft reservoir over the project life of 50 years is about 2,570 ac-ft which represents a loss of reservoir storage of approximately 1.6 percent over the project life. Under the assumptions of the worst case, and highly improbable, watershed (100 percent of the watershed under cultivation with no soil conservation measures) and channel sediment yields (transport capacity limited assumption) the estimated annual yield to the dam site is 217 ac-ft (439,000 tons). With the reservoir in place, this reduces to an annual yield of 74 ac-ft (150,000 tons). Therefore, the worst-case estimate of sediment delivery to the 160,235 ac-ft reservoir over the 50-year project life is about 3,700 ac-ft, which represents a loss of reservoir storage capacity of approximately 2.3 percent.

To put the estimated annual sediment yields at the dam site into perspective, a review was conducted of other sediment yield studies in the Blackland Prairie region of Texas (**Table 5.4**).

Table 5.4. Comparison of estimated sediment yields from Texas Blackland Prairie watersheds.

Source	Annual Sediment Yield at Dam Site (t/yr)	Unit Annual Sediment Yield (t/sq mi)	Unit Annual Sediment Yield (t/ac)	Annual Sediment Yield (ac-ft)
MEI best estimate	174,000	1,740	2.7	86
MEI worst-case estimate	439,000	4,390	6.9	217
Alan Plummer and Associates (2005) reservoir surveys	100,000	1,000	1.6	49
Greiner (1982) sheet, rill, gully and channel erosion	105,600	1,056	1.7	51
Simon et al. (2004) Blackland ecoregion analysis	25th Perc. 25,500 50th Perc. 179,000 75th Perc. 375,300	255 1,790 3,753	0.4 2.8 5.9	13 88 188
Coonrod et al. (1998) suspended sediment yields in Texas watersheds	104,900	1,049	1.6	52
Texas Dept. Water Resources (1979) maximum suspended sediment load, Sulphur River at Talco, Texas (1968)	264,200	2,642	4.2	130
NRCS, Birket (1994) Mill Creek sediment analysis	108,220	1,082	1.7	53

With the exception of the Simon et al. (2004) ecoregion analysis 50th and 75th percentile values that were based on only six data points, and the highest suspended sediment value measured at the Talco gage (TDWB, 1974), when there was likely a much higher channel erosion component, the conservatively based MEI best estimate of annual sediment yield is significantly higher than other reported data for the Texas Blackland region. The MEI worst-case estimate significantly exceeds any measured or estimated values, and can be, therefore, considered to represent an upper limit that would encompass all likely sediment sources in the watershed.

One of the concerns about the Lake Ralph Hall project is the potential downstream effects of the dam on channel conditions and channel capacity. Potential problems could include sediment accumulation in the bed of the channel since operation of the reservoir will affect the magnitude and frequency of flows in the downstream channel, but will not affect sediment supply from the watershed tributaries and channel sources downstream of the dam. Field and helicopter reconnaissance of the NSR from its confluence with the South Sulphur River to the headwaters indicates that the channel of the NSR is deeply incised for its entire length, and that the bed of the channel is composed of shale bedrock. Locally, near the mouths of some of the large tributaries downstream of the dam site (e.g., Hickory and Big Sandy Creeks) there are alternate bars in the bed of the channel, but these reflect local sediment supply and do not extend downstream for any distance. Under existing conditions, the best estimate of the annual total sediment yield to the dam site is about 174,000 tons (Figure 5.7), but only about 25 percent is composed of bed material, the remainder being wash load. Therefore, construction of the dam will reduce the morphologically-significant sediment yield to the channel downstream of the dam by about 25 percent. Since the sediment-transport capacity greatly exceeds the sediment supply, this level of reduction in supply will have an insignificant effect on downstream channel morphology.

Based on the geologic map (Figure 2.2), and field observations, the characteristics of the shale exposed in the mainstem and tributaries downstream of the dam site are similar to those upstream of the site, and therefore, it can be assumed that the sediment characteristics are also similar. This being the case, the bulk of the sediments being delivered to the NSR by the tributaries downstream of the dam will be composed of shale clasts that break down into wash-load size materials as they are exposed to transport and weathering processes (slaking). Furthermore, the NSR is a supply-limited system that has the capacity to transport considerably more bed material than is currently being supplied to the channel. Consequently, it is unlikely that significant amounts of sediment will accumulate in the bed of the NSR downstream of the dam. If sediment accumulation does occur it is highly unlikely that there will be significant loss of channel capacity since flows far greater than the 100-year flood peak can be conveyed in-bank.

6. SEDIMENT MANAGEMENT

Although estimated sediment yields to the Lake Ralph Hall reservoir are relatively low, the sediment yields could be further reduced by implementation of soil conservation measures on the watershed and by reducing the exposure of shale in the mainstem of the NSR and the tributaries between the upstream end of the reservoir and the Roxton/Gober Chalk outcrop (Figure 2.2).

6.1. Watershed Sediment Reduction

The percentage of the NSR watershed area under cultivation has reduced from about 75 percent in the late 1920s (Williams, 1928) to about 26 percent presently (TSSWCB, 1997; Loretta Mokry, pers. comm., 2006), and the percentage in cropland is reducing at a rate of about 0.5 percent per year (Randy Moore, NRCS Fannin Co., pers. comm., 2006). Data from Reisel, Texas in the Blackland Prairie have shown that net soil losses with conservation management range from 0.2 to 1.0 t/ac/yr on cultivated soils (Harmel et al., 2006). In contrast, under native meadow grasses net soil losses are as low as 0.05 t/ac/yr (Richardson, 1993). Therefore conversion of cropland to native grassland could reduce the net soil loss by factors of 4 to 20.

Review of the aerial photography of the NSR watershed indicates that significant areas of the watershed have been improved with soil conservation measures including contour cultivation and terracing in the past. Field observation indicates that many of the measures have not been maintained. Therefore, sediment yields from the watershed, especially in those areas still under cultivation, could be reduced by maintaining the soil conservation structures. Additionally, a number of SCS floodwater retarding structures (FWRS) have been built within the watershed. A number of the structures have been breached as a result of baselevel-lowering-induced channel erosion, and others appear to have lost much of their storage capacity due to sedimentation. Replacement and rehabilitation of the FWRS will reduce sediment yield from the watershed. A relatively high number of gulleys were observed in areas adjacent to the incised tributary channels, especially on the south side of the watershed. Gully stabilization measures, including installation of gully plugs to store sediment on the gulley floors, revegetation, and construction of water diversion structures around the head of the gulleys to reduce erosion would reduce sediment yields from this source.

Riparian tree and shrub buffers are located along many of the channel segments in the NSR tributaries, and these tend to trap sheet-and-rill erosion-derived sediments and prevent them being delivered to the channel system. Further, the presence of a robust riparian buffer tends to increase the stability of the upper banks, both as a result of root reinforcement and development of positive matric suction when the soils are wet (Simon et al., 1999). Therefore, re-establishment of a riparian buffer zone along channel segments that have been cleared of woody vegetation is likely to reduce sediment yield to the channels.

6.2. Channel Sediment Reduction

Erosion of the shale exposed in the bed and banks of the NSR and its incised tributaries is due primarily to weathering processes (slaking) that are controlled by the frequency of wetting and drying cycles (Allen et al., 2002). As shown in the sediment-transport calculations, removal of long segments of the channel due to reservoir construction reduces the volume of channel-derived sediments by about 40 percent. Further reduction in the shale-derived channel sediment yield could be achieved by preventing further weathering of the shale. This could be achieved by inundating the currently exposed shale outcrop on a year-round basis by constructing a number of small in-channel check structures that pond water. The extent of the exposed shale upstream of the reservoir boundaries is determined by the distance between the elevation of the top of the conservation pool and the in-channel outcrop of the Roxton/Gober Chalk (Table 2.1). The HEC-RAS models of the tributaries indicate that they easily contain the 100-year peak flow within-bank, and therefore, construction of in-channel check structures is not likely to cause out-of-bank flooding. Spacing and sizing of the check structures for the individual tributaries can be done with the HEC-RAS models (Appendix D).

A number of concrete box culverts have been constructed at road crossings on the incised tributaries to the NSR and these structures provide a measure of grade control in the channels. However, downstream erosion has caused damage to many of the structures and these will need to be maintained if they are to provide grade control in the future. A concrete box culvert at the FM 2990 crossing of Leggetts Branch that has prevented a significant amount of degradation from progressing upstream will be inundated by the reservoir, but the box culvert crossing of FM 1550 is upstream of the reservoir and will provide grade control for the upstream channel (Figure 2.25). Similarly, the box culvert at the FM 2990 crossing of Davis Creek will be inundated but the FM 1550 crossing will provide grade control provided that the structure is maintained (Figure 2.26). The box culvert at the FM 1550 crossing of Pickle Creek is also providing grade control and it too must be maintained (Figure 2.27). The H-pile and concrete beam grade-control structure below the FM 1550 bridge on Brushy Creek (Figure A.22) appears to be a successful structure, and similar structures may need to be constructed downstream of many of the other bridge and culvert crossings in the watershed.

7. SUMMARY AND CONCLUSIONS

7.1. Summary

The Upper Trinity Regional Water District (UTRWD) is proposing to build a 160,235-ac-ft water supply reservoir, Lake Ralph Hall, on the NSR about 3.5 miles north of Ladonia in Fannin County, Texas (Figure 1.1). Fannin County is located within the Texas Blackland Prairie physiographic area (NRCS, 2001). The NSR and its tributaries, within the boundaries of the proposed reservoir, as well as upstream and downstream, are deeply incised and eroding. Current conditions are the result of channelization and straightening of the sinuous, meandering river and the lower reaches of its tributaries to prevent frequent overbank flooding on the NSR floodplain in the late 1920s (Williams, 1928; Avery, 1974). Prior to channelization, the NSR was a sinuous (1.7) meandering stream with a slope of about 4.3 ft/mi. In the vicinity of the proposed dam site, the natural channel was about 48 feet wide and 6 feet deep and had a hydraulic capacity of between 700 and 1,000 cfs. The channelized and straightened channel had a top width of 16 to 30 feet, and a depth of 9 to 12 feet with a slope of 6.5 ft/mi (Avery, 1974; Chiang, Patel & Yerby, Inc., 2004; AR Consultants, Inc., 2005) and a hydraulic capacity of about 700 cfs. Currently, at the proposed dam site the NSR is 300 feet wide and about 40 feet deep, the bed and lower portions of the banks of the channel are composed of erodible shale (Ozan Formation), and the channel contains flows well in excess of the 100-year flood peak (38,000 cfs). Between the late 1920s and the present about 28M tons of sediment have been eroded from the mainstem NSR and its tributaries upstream of the proposed dam site. At the time of the channelization in the late 1920s about 75 percent of the watershed was under cultivation (Williams, 1928), and consequently soil erosion rates were probably very high (up to 16 t/ac/yr) (Baird, 1948, 1964), which may have contributed to loss of channel capacity and increased frequency of overbank flooding that occasioned the channelization. Currently about 21 percent of the watershed that contributes water and sediment to the proposed reservoir is cultivated (Texas State Soil and Water Conservation Board, 1997).

The primary objectives of this geomorphic and sedimentation study of the Lake Ralph Hall project were:

1. Quantification of the sediment delivery to the reservoir site for the 50-year project life under pre- and post-project conditions,
2. Evaluation of the downstream effects of the dam on channel conditions and flow capacity, and
3. Assessment of the potential for reducing or managing the upstream sediment supply to the reservoir.
4. Assessment of future conditions in the North Sulphur River and tributaries upstream of the dam site in the absence of the project.

Future loss of reservoir capacity due to sedimentation is the primary issue of concern for this investigation of the Lake Ralph Hall project and, therefore, estimates of sediment yield from the 100-square-mile watershed upstream of the proposed dam were required. Potential sources of sediment identified included channel erosion in the mainstem NSR and the incised tributaries (bed and banks) and watershed erosion (sheet, rill, ephemeral gully). Hydrologic analyses of the gage record at the USGS North Sulphur River near Cooper gage (USGS Gage No. 07343000) and HEC-1 models were used to estimate peak flow frequencies (Figures 3.7, 3.9), mean daily durations and flow volumes (Figure 3.10) for the dam site and the tributaries. One–dimensional HEC-RAS models were developed for the mainstem and for the major tributaries based on the 2-foot contour interval Digital Terrain Model (DTM) provided by CP&Y, and the models were calibrated to field-measured high-water marks for the 2002 (10-year event) and 2003 (25-year event) peak flows. Reach-averaged hydraulic output (effective width, hydraulic depth and average velocity) from the HEC-RAS models was used to compute sediment transport.

Field observations of the NSR and its tributaries indicated that in common with other incised streams, the morphological adjustments of the river and the larger tributaries can be described by a geomorphic model of incised channel evolution (Schumm et al., 1984; Simon and Hupp, 1986; Simon, 1989). A channel evolution model (NSRCEM) was developed for the NSR and its tributaries (Figure 2.19). The model varies substantially from those developed for alluvial streams (Figure 2.4) in that it does not predict an equilibrium end point because both vertical and lateral erosion of the exposed shale outcrop is controlled by wetting and drying cycles (Tinkler and Parish, 1989; Allen et al., 2002) and not hydraulic processes. There is little doubt that following channelization in the late 1920s the NSR incised and widened (Avery, 1974) and followed the typical channel evolution sequence while the channel boundary materials were composed of alluvium (Types I through V). However, exposure of the shale added a significant complicating factor to the evolution of the channel. Based on the flow record at the USGS gage on the NSR near Cooper, there are an average of six wetting and drying cycles per year (Figure 2.3). Flow events in the channel remove the weathering products and re-initiate vertical and lateral erosion into the shale. As a rule, lateral erosion rates exceed vertical erosion rates in bedrock and result in the formation of gravel-covered strath surfaces that become terraces when vertical erosion of the bed occurs (Leopold et al., 1964; Schumm, 1977) (Type VI). Deep-seated slump failures of the overlying alluvium bury the strath surfaces (Type VII) and prevent lateral erosion of the shale. Resulting channel narrowing may actually accelerate erosion of the shale exposed in the bed, which in turn leads to undercutting of the erosion-resistant, root-reinforced alluvium, thereby leading to re-exposure of the shale in the toe of the banks and ongoing lateral retreat of the shale (Type VIII). It is likely that over time the incision into the shale will induce further mass failure of the alluvial valley fill and a Type VII condition will be reestablished at a lower bed elevation and there will be additional channel widening. The NSRCEM applies equally to the larger tributaries that have eroded into the shale.

Between the FM 904 Bridge and the upstream end of the watershed, the NSR was subdivided into 10 subreaches (Table 2.2). Based on the NSRCEM, Subreaches 1 through 3 were classified as Type VI, Subreach 4 was classified as Type VII, Subreaches 5 through 8 were classified as Type VIII, and Subreaches 9 and 10 were classified as Type VII. Similar sequences are present in the larger tributaries. Incision in the headwaters of the NSR and the major north-side tributaries has been limited by outcrop of reasonably erosion resistant Roxton/Gober Chalk (Figure 2.2). Currently, the incised channel has the ability to convey in excess of the 100-year flood in-bank (Figures 2.5 through 2.18), the bed of the river is composed of shale, and therefore, the current supply of sediment to the channel is far less than the transport capacity.

The primary sources of bed-material-sized sediment are the exposed shale outcrops in the bed and banks of the river and the tributaries. Based on studies of the erosion of the shale (Allen et al., 2002; Crawford, in prep) and the results of analysis of stage-discharge rating curves for the Cooper gage (Figure 2.36) and comparative bridge profiles (Figure 2.34), erosion rates for shale exposed in the bed and banks of the channel are on the order of 2 to 4 in./year, respectively. Transport and slaking of the shale clasts results in a temporal and spatial transformation of initially gravel-sized material, which is transported as bed material, to silt-clay-sized wash load

(Figure 2.40) that has little or no morphological significance. At the upstream end of the NSR about 80 percent of the bed material that forms a thin veneer over in-situ shale slakes to silt-clay-sized material, whereas in the downstream reaches only about 10 percent of the bed-material slakes (Figure 2.42). Based on a supply-limited model of sediment-transport capacity, calibrated to the area of the bed covered by depositional bars, and incorporating the transformation of the bed material to wash load, the best estimate of sediment yield from channel sources to the dam site under pre-project conditions is 93,100 t/yr. Based on a somewhat unrealistic transport capacity-limited model, the worst-case estimate of sediment yield from channel sources to the dam site is 292,000 t/yr. With the dam in place, the best-case estimate of annual sediment yield from channel sources to the reservoir is 35,600 tons, and the worst–case estimate is 59,600 tons. The reduced amount of sediment is because the reservoir inundates a high proportion of the contributing channel area and eliminates it as a contributing source.

Estimates of the sheet-and-rill erosion on the watershed were developed with the Modified Universal Soil Equation (MUSLE) with appropriate parameters based on the subbasin topography and soil types (clays and loams) determined from the Soil Survey of Fannin County (NRCS, 2001). Application of the MUSLE with the appropriate parameters underestimated reported gross sheet-and-rill erosion rates on the Blackland Prairie soils (2 t/ac/yr), and therefore the alpha coefficient for the MUSLE was increased by a factor of 2.7. Ephemeral gully erosion for the cropland portions of the watershed was estimated to be equivalent to the sheet-and-rill gross erosion rates on the basis of the soil erosion literature (Laflen et al., 1986). Sediment delivery ratios (SDR) for the sheet-and-rill erosion were estimated with Equation 5.4 (Renfro, 1975) that yields the highest SDR values. For the ephemeral gully erosion the SDR was estimated to be 0.67 (Alan Plummer and Associates, 2005). Worst-case watershed sediment yields were estimated with an assumption of 100-percent cropping in the watershed with a gross erosion rate of 3.74 t/ac/yr (Richardson, 1993). The best conservative estimate of the current annual watershed sediment yield at the dam site is about 81,000 t/yr which reduces to about 69,000 t/yr with the reservoir in place. Under worst-case conditions the existing annual watershed sediment yield to the dam site is about 147,000 t/yr, and this reduces to about 90,000 t/yr with the reservoir in place. When placed in the context of reported sediment yields in the Blackland Prairie (Table 5.4), these estimates are very conservative especially because a 100 percent trap efficiency has been assumed for the reservoir.

Although estimated sediment yields to the Lake Ralph Hall reservoir are relatively low, the sediment yields could be further reduced by implementation of soil conservation measures on the watershed and by reducing the exposure of shale in the mainstem of the NSR and the tributaries between the upstream end of the conservation pool and the Roxton/Gober Chalk outcrop (Figure 2.2).

The potential downstream effects of the Lake Ralph Hall project on channel conditions and channel capacity are a concern. Potential problems could include sediment accumulation in the bed of the channel since operation of the reservoir will affect the magnitude and frequency of flows in the downstream channel, but will not affect sediment supply from the watershed, tributary and channel sources below the dam. Field and helicopter reconnaissance of the NSR from its confluence with the South Sulphur River to the headwaters indicates that the channel of the NSR is deeply incised for its entire length, and that the bed of the channel is composed of shale bedrock. Since the rates of bedrock erosion are controlled by the number of wetting and drying cycles (Allen et al., 2002), and not by hydraulic processes, the upstream dam is unlikely to have any effects on bedrock erosion rates. On an average annual basis, the shale will continue to erode vertically at a rate of about 2 inches per year and laterally at a rate of about 4 inches per year. Locally, near the mouths of some of the large tributaries downstream of the dam site (e.g., Hickory and Big Sandy Creeks) there are alternate bars in the bed of the channel, but these reflect local sediment supply and do not extend downstream for any distance. Under existing conditions, the best estimate of the annual total sediment yield to the dam site is about 174,000 tons (Figure 5.8), but only about 25 percent is composed of bed material, the remainder being wash load. Therefore, construction of the dam will reduce the morphologically-significant sediment yield to the channel downstream of the dam by about 25 percent, which will have an insignificant effect on the channel morphology in this sediment supply-limited system.

Based on the geologic map (Figure 2.2), and field observations, the characteristics of the shale exposed in the mainstem NSR and tributaries downstream of the dam site are similar to those upstream of the site, and therefore, it can be assumed that the sediment characteristics are also similar. This being the case, the bulk of the sediments being delivered to the NSR by the tributaries downstream of the dam will be composed of shale clasts that break down into wash-load size materials as they are exposed to transport and weathering processes (slaking). Furthermore, the NSR is a supply-limited system that has the capacity to transport considerably more bed material than is currently being supplied to the channel. Consequently, it is unlikely that significant amounts of sediment will accumulate in the bed of the river downstream of the dam. If sediment accumulation does occur it is highly unlikely that there will be significant loss of channel capacity. Even with the loss of channel capacity, flows far greater than the 100-year flood peak can be conveyed in-bank.

7.2. Conclusions

The geomorphic, hydrologic, hydraulic and sediment-transport studies conducted for this investigation of the Lake Ralph Hall project allow the following to be concluded:

1. Channelization-induced degradation and widening of the NSR and its principal tributaries upstream of the dam site has resulted in the erosion of about 28M tons of sediment since the late 1920s. Current channel erosion rates are controlled by slaking rates of the exposed shale and not by hydraulic processes and are, therefore, less than historic rates.
2. The conservative estimate of total annual sediment yield to the dam site under pre-project conditions is 86 ac-ft (174,000 tons). With the reservoir in place, the contributing watershed area is reduced, as is the length of channel that is supplying sediment, and therefore, the total annual sediment yield to the reservoir reduces to 51 ac-ft (104,000 tons). Therefore, estimated sediment delivery to the 160,235-ac-ft reservoir over a 50-year period, assuming 100 percent trap efficiency, is about 2,570 ac-ft, which represents a loss of reservoir storage capacity of approximately 1.6 percent.
3. Under the assumptions of the worst-case watershed (100 percent of the watershed under cultivation with no soil conservation measures) and channel sediment yields (transport capacity limited assumption) the estimated total annual sediment yield to the dam site is 217 ac-ft (439,000 tons). With the reservoir in place, the worst-case reduces to an annual sediment yield to the reservoir of 74 ac-ft (150,000 tons). Under these circumstances, estimated sediment delivery to the 160,235 ac-ft reservoir over a 50-year period, assuming 100 percent trap efficiency, is about 3,700 ac-ft, which represents a loss of reservoir storage capacity of approximately 2.3 percent.
4. In the absence of the Lake Ralph Hall project there will be continued erosion of the NSR and its tributaries. On average, where shale is exposed in the bed and banks of the channels, the channel depth will increase by about 8 feet and the channel bottom widths will increase by about 16 feet over a 50-year period. Increased channel depths are also

likely to cause further mass failure of the alluvial portions of the banks, thereby increasing channel top widths, as well.

5. No adverse downstream impacts on channel morphology or capacity are expected as a result of sediment trapping in the reservoir, or operation of the reservoir.

6. Watershed sediment yields could be reduced by implementation of best soil conservation management practices, reduction in the area under cultivation and re-establishment of riparian buffer areas along the channel margins where they have been cleared.

7. Channel sediment yields between the elevation of the top of the conservation pool and the downstream extent of the Roxton/Gober Chalk could be reduced by construction of in-channel structures that pond water and prevent weathering of the shale outcrop. Given the existing hydraulic capacity of the channels there is little likelihood that the in-channel structures would cause out-of-bank flooding.

8. REFERENCES

Alan Plummer and Associates, 2005. Lake Sedimentation Issues. In Texas Water Development Board, *Dredging vs New Reservoirs*, TWDB Contract No. 2004-483-534, in conjunction with Allen, P.M. and Dunbar, J.A., pp. 2-1 - 2-25.

Allen, P.M., Arnold, J.G., and Skipwith, W., 2002. Erodibility of Urban Bedrock and Alluvial Channels, North Texas. Journal of the American Water Resources Association, v. 38, no. 5, October, pp. 1477-1492.

AR Consultants, Inc., 2005. Archaeology and Quaternary Geology at Lake Ralph Hall, Fannin County, Texas. Texas Antiquities Permit No. 3693, prepared for Upper Trinity Regional Water District, Lewisville, Texas, September, 110 p.

Asquith, W.H. and Slade, R.M., Jr., 1997. Regional Equations for Estimation of Peak-Streamflow Frequency for Natural Basins in Texas. U.S. Geological Survey, Water-Resources Investigations Report 96-4307, prepared in cooperation with the Texas Dept. of Transportation.

Avery, J., 1974. Letter from J. Avery to U.S. Army Corps of Engineers, Tulsa District, December 26.

Baird, R.W., 1948. Runoff and soil conservation practices. Agric. Eng., pp. 216-217.

Baird, R.W., 1964. Sediment yields from Blackland watersheds. Trans. ASAE 7, pp. 454-465.

Barnes, H.H., 1967. Roughness characteristics of natural channels. U.S. Geological Survey Water-Supply Paper 1849.

Bircket, M., 1994. Trinity River Watershed (WF-03) Chambers Creek Subwatershed, Mill Creek Tributary. Supplemental Work Plan and Agreement No. VII, prepared by U.S. Dept. of Agriculture, Natural Resources Conservation Service, Temple, Texas, December, 88 p.

Chiang, Patel & Yerby, Inc., 2004. Geological Characteristics of Proposed Lake Ralph Hall. Draft Report, Prepared for Upper Trinity Regional Water District, Project No. UTR9836, February.

Coonrod, J.E.A., Holley, E.R., and Maidment, D.R., 1998. Suspended sediment yield in Texas watersheds. Center for Research in Water Resources, Technical Report CRWR 270.

Crawford, C., in preparation. Methods: Slake Durability Analysis. Draft M.S. Thesis, Baylor University.

Einstein, H.A., 1950. The bedload function for sediment transportation in open channel flows. U.S. Soil Conservation Service, Tech. Bull. No. 1026.

Frye, J.C. and Leonard, A.B. 1963. Pleistocene Geology of the Red River Basin in Texas. Bureau of Economic Geology Report of Investigations 49, University of Texas, Austin.

Gellis, A.C., Hereford, R., Schumm, S.A., and Hayes, B.R., 1995. Channel evolution and hydrologic variations in Colorado River Basin: Factors influencing sediment and salt loads. Journal of Hydrology 124, pp. 317-344

Gottschalk, L.C., 1964. Sedimentation, Part I, Reservoir Sedimentation. In Chow, V.T. (ed), *Handbook of Applied Hydrology*, Sec. 17-1, McGraw-Hill, New York.

Greiner, J., 1982. Erosion and Sedimentation by Water in Texas: Average Annual Rates Estimated in 1979. Texas Water Development Board Report 268, Austin, Texas.

Happp, S.C., Rittenhouse, G., and Dobson, G.C., 1940. Some principles of accelerated stream and valley sedimentation. U.S. Dept. of Agriculture Tech. Bull 695, 133 p.

Harmel, R.D., Richardson, C.W., King, K.W., Allen, P.M., 2006. Runoff and soil loss relationships for the Texas Blackland Prairies ecoregion. Journal of Hydrology (accepted for publication).

Harvey, M.D. and Watson, C.C., 1986. Fluvial processes and morphological thresholds in incised channel restoration. Water Resources Bulletin, v. 22, no. 3, pp. 359-368. Reprinted in, Jackson, W.L. (ed), *Engineering Considerations in Small Stream Management*, AWRA Monograph Series No. 5.

Hershfield, D.M., 1961 . Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and return periods from 1 to 100 years. Technical Paper No. 40, U.S. Dept. of Commerce, Washington, D.C., 61 p.

Howard, A.D., 1998. Long profile development of bedrock channels: Interaction of weathering, mass wasting, bed erosion, and sediment transport. In Tinkler, K.J. and Wohl, E.E. (eds), *Rivers over Rock: Fluvial Processes in Bedrock Channels*, American Geophysical Union, Geophysical Monograph 107, pp. 297-319.

Ireland, H.A., Sharpe, C.F.S., and Eargle, D.H., 1939. Principles of gully erosion in the Piedmont of South Carolina. U.S. Dept. of Agriculture Technical Bulletin 633.

Kleinfelder, 2005. Preliminary Subsurface Exploration, Ralph Hall Dam, Fannin County, Texas. Prepared for Chiang, Patel & Yerby, Inc., Dallas Texas, Project No. 53882, June, 30 p.

Laflen, J.M., Watson, D.A., and Franti, T.G., 1986. Ephemeral Gully Erosion. In Proceedings of the Fourth Federal Interagency Sedimentation Conference, Volume I. Subcommittee on Sedimentation of the Interagency Advisory Committee on Water Data, pp. 3-29-3-37.

Leopold, L.B., Wolman, M.G., and Miller, J.P., 1964. *Fluvial Processes in Geomorphology.* Freeman Co., San Francisco, California, and London, 522 p.

Li, R.M., Mussetter, R.A., and Grindeland, T.R., 1988. Sediment-Routing Model, HEC2SR in Twelve Selected Computer Stream Sedimentation Models Developed in the United States, Subcommittee on Sedimentation, Interagency Advisory Committee on Water Data, Dr. Shou-shan Fan, Federal Energy Regulatory Commission.

Little, W.C., Thorne, C.R., and Murphey, J.B., 1981. Mass bank failure analysis of selected Yazoo Basin streams. Transactions of the American Society of Agricultural Engineers, no. 25, pp. 1321-1328.

Lohnes, R. and Handy, R.L., 1968. Slope Angles in Friable Loess. Journal of Geology 76, pp. 247-258.

Meyer-Peter, E. and Müller, R., 1948. Formulas for bed load transport. In Proceedings of the 2nd Congress of the International Association for Hydraulic Research, Stockholm, 2: Paper No. 2, pp. 39-64.

Mussetter, R. A., Lagasse, P.F., and Harvey, M.D., 1994. *Erosion and Sediment Design Guide*. Prepared for Albuquerque Metropolitan Arroyo and Flood Control Authority.

National Resources Conservation Service, 1986. Urban Hydrology for Small Watersheds. Technical Release 55, USDA, June, 160 p.

National Resources Conservation Service , 2001. Soil Survey of Fannin County, Texas. USDA, in cooperation with the Texas Agricultural Experiment Station, Texas State Soil and Water Conservation Board, and U.S. Forest Service.

Prosser, I.P. Hughes, A.O., and Rutherfurd, I.D., 2000. Bank erosion of an incised upland channel by subaerial processes, Tasmania, Australia. Earth Surface Processes and Landforms 25, pp. 1085-1101.

Rainey, M., 1974. The Quaternary Stratigraphy of the North Sulphur River. Unpublished MA Thesis, Southern Methodist University, Dallas, Texas.

Renfro, G.W., 1975. Use of erosion equations and sediment delivery ratios for predicting sediment yield. In Present and Prospective Technology for Predicting Sediment Yields and Sources, Agricultural Resources Services, ARS-S-40, U.S. Dept. of Agriculture, Washington, D.C., pp. 33-45.

Richardson, C.W., 1993. Disappearing Land: Erosion in the Blacklands. In Sharless, M.R. and Yelderman, J.C. (eds), *The Texas Blackland Prairie, Land, History, Culture*, Baylor University Press, Waco, Texas.

RJ Brandes Company, 2004. Hydrologic and Hydraulic Studies of Lake Ralph Hall. Prepared for Upper Trinity Regional Water District, Lewisville, Texas, April.

Schumm, S.A., 1977. *The Fluvial System*. John Wiley & Sons, New York, New York, 338 p.

Schumm, S.A., 1999. Causes and Controls of Channel Incision. In Darby, S.E. and Simons, A. (eds). *Incised River Channels: Processes, Forms, Engineering, and Management,* John Wiley & Sons, New York, Chapter 2.

Schumm, S.A., Harvey, M.D. and Watson, C.C. (eds), 1984. *Incised Channels: Morphology Dynamics and Control.* Water Resources Pub., Littleton, Colorado, 200 p.

Shen, H.W. and Julien, P.Y., 1993. Erosion and sediment transport. In Maidment, D.R. (ed), *Handbook of Hydrology*, Chapter 12, McGraw-Hill, New York.

Simon, A., 1989. A model of channel response in disturbed alluvial channels. Earth Surface Processes and Landforms, 14, 1, pp. 11-26.

Simon, A. and Darby, S., 1999. The Nature and Significance of Incised River Channels. In Darby, S.E. and Simons, A. (eds), 1999. *Incised River Channels: Processes, Forms, Engineering, and Management,* John Wiley & Sons, New York, Chapter 1.

Simon, A. and Hupp, C.R., 1986. Channel evolution in modified Tennessee channels. In Proceedings of the Fourth Federal Interagency Sedimentation Conference, v. 2, U.S. Government Printing Office, Washington, D.C. pp. 5-71 – 5-82.

Simon, A., Dickerson, W., and Heins, A., 2004. Suspended-sediment transport rates at the 1.5-year recurrence interval of ecoregions of the United States: transport conditions at the bankfull and effective discharge? Geomorphology 58, pp. 243-262.

Simon, A., Rinaldi, M., and Hadish, G., 1996. Channel evolution in the loess area of the midwestern United States. In Proceedings of the Sixth Federal Interagency Sediment Conference, US Government Printing Office, Washington, DC, pp. III-86 to III-93.

Simon, A., Curini, A., Darby, S.E., and Langendoen, E.J., 1999. Streambank Mechanics and the Role of Bank and Near-bank Processes in Incised Channels. In Darby, S.E. and Simon, A (eds), *Incised River Channels: Processes, Forms, Engineering, and Management*, Wiley & Sons, West Sussex, England, pp. 123-152.

Sklar, L. and Dietrich, W.E., 1998. River longitudinal profiles and bedrock incision models: Stream power and the influence of sediment supply. In Tinkler, K.J. and Wohl, E.E. (eds), *Rivers over Rock: Fluvial Processes in Bedrock Channels*, American Geophysical Union, Geophysical Monograph 107, pp. 237-260.

Slaughter, B.H. and Hoover, B.T., 1963. Sulphur River Formation and the Pleistocene Mammals of the Ben Franklin Local Fauna. Journal of Graduate Research Center, South Methodist University, 31(3), pp. 132-138.

Slaughter, B.H. and Hoover, B.T., 1965. An Antler Artifact from the Late Pleistocene of Northeast Texas. American Antiquity 30, pp. 351-352.

Smith, S.J., Williams, J.R., Menzel, R.G., and Coleman, G.A., 1984. Prediction of Sediment Yield from Southern Plans Grasslands with the Modified Universal Soil Loss Equation. Journal of Range Management, v. 37, no. 4, July, pp. 295-297.

Soil Conservation Service, 1978. *Predictions of Rainfall Erosion Losses*. SCS Agriculture Handbook Number 537.

Stock, J.D., Montgomery, D.R., Collins, B.D., Dietrich, W.E., and Sklar, L., 2005. Field measurements of incision rates following bedrock exposure: Implication for process controls on the long profiles of valleys cut by rivers and debris flows. GSA Bulletin, v. 117, no. 11/12, January/February, pp. 174-194.

Struiksmas, N., 1999. Mathematical modeling of bedload over nonerodible layers. IAHR Symp. River, Coastal and Estuarine Morphodynamics, Genova, Italy, September 6-10, v. 1, pp. 89-98.

Texas Bureau of Economic Geology, 1966. Geologic Atlas of Texas, Texarkana Sheet. Scale 1:250,000, second printing 1979.

Texas Bureau of Economic Geology, 1967. Geologic Atlas of Texas, Sherman Sheet. Scale 1:250,000, revised 1991.

Texas Dept. of Water Resources, 1979. Suspended Sediment Load of Texas Streams. Compilation Report, October 1971 through September 1975, Report No. 233, May, 11 p.

Texas State Soil & Water Conservation Board, 1997. Soil and Water Conservation District Fact Sheet. Fannin County SWCD #520, Bonham, Texas.

Texas Water Development Board, 1974. Suspended sediment load of Texas streams, October 1965 through September 1971. TWDB Report 184, Austin, Texas.

Thorne, C.R., 1988. Analysis of bank stability in the DEC watersheds, Mississippi. Final report to the U.S. Army European Research Office, Contract No. DAJA45-87-C-0021, Dept. of Geography, Queen Mary College, London.

Thorne, C.R., 1999. Bank Processes and Channel Evolution in the Incised River of North-Central Mississippi. In Darby, S.E. and Simons, A. (eds), 1999. *Incised River Channels: Processes, Forms, Engineering, and Management,* John Wiley & Sons, New York, Chapter 5.

Tinkler, K.J. and Parish, J., 1998. Recent adjustments to the long profile of Cooksville Creek, an urbanized bedrock channel in Mississauga, Ontario. In Tinkler, K.J. and Wohl, E.E. (eds), *Rivers over Rock: Fluvial Processes in Bedrock Channels*, American Geophysical Union, Geophysical Monograph 107, pp. 167-187.

Trimble, S.W., 1974. Man-induced soil erosion on the southern Piedmont 1700-1970. Soil Conservation Society, Ankemy, Iowa, 180 p.

U.S. Army Corps of Engineers, 1990. HEC-1, Flood Hydrograph Package, Version 4.0, Hydrologic Engineering Center, Davis, California.

U.S. Army Corps of Engineers, 1992. HEC-FFA, Flood Frequency Analysis, User's Manual, Hydrologic Engineering Center, Davis, California.

U.S. Army Corps of Engineers, 2005. HEC-RAS, River Analysis System, Users Manual, Version 3.1.3, Hydrologic Engineering Center, Davis, California.

Water Resources Council. 1981. Guidelines for Determining Flood Flow Frequency. Bulletin No. 17B of the Hydrology Committee.

Watson, C.C., Harvey, M.D., and Garbrecht, J., 1986. Geomorphic-hydraulic simulation of channel evolution. Proceedings of the Fourth Federal Interagency Sedimentation Conference, v. 2, pp. 5.21-5.30.

Watson, C.C., Harvey, M.D., Biedenharn, D.S., and Combs, P., 1988. Geotechnical and hydraulic stability numbers for channel rehabilitation: Part I, The Approach. In Abt, S.R. and Gessler, J. (eds), ASCE, Hyd. Div., 1988 National Conference, pp. 120-125.

Williams, E.F., 1928. State Reclamation Engineer Report 40, April 13.

Williams, J.R. and Berndt, H.D., 1972. Sediment yields computed with Universal Equation. Journal of Hydraulics, ASCE, v. 102, no. HY9, pp. 1241-1253.

Woodward, D.E., 1999. Method to predict cropland ephemeral gully erosion. Published by Elsevier Science B.V.

APPENDICES
(see enclosed CD)

APPENDIX A
Photographs of North Sulphur River and Tributaries

APPENDIX B
Sediment Sample Analysis Report (Kleinfelder, 2006)

APPENDIX C
HEC-1 Models for the Mainstem North Sulphur River and the Primary Tributaries

APPENDIX D
HEC-RAS Models for the Mainstem North Sulphur River and the Primary Tributaries

APPENDIX E
Fine Sediment Yield Calculations (Best Estimate and Worst Case; Existing Conditions)

APPENDIX F

Responses to Peer Review Prepared by Drs. Craig MacRae and Peter Allen

Mussetter
Engineering
Inc.

Consultants in
Water Resource Engineering &
Engineering Geomorphology

October 23, 2006

Mr. John Levitt, P.E.
Chiang Patel & Yerby, Inc.
1820 Regal Row, Suite 200
Dallas, Texas 75235

Re: Responses to Peer Review Prepared by Drs. Craig MacRae and Peter Allen

Dear Mr. Levitt:

We have reviewed the Peer Review comments on our Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Lake Ralph Hall Dam Project that were prepared by Dr. Craig MacRae of Aquafor Beech Limited, and Dr. Peter Allen of Baylor University. Our responses are keyed to the comments in their review, and are provided below.

2.0. FUNCTIONALITY OF REPORT

1. Explanation expanded in Section 3.1.3., p.3.1. In the absence of any other data, we assumed that the wetting–drying cycles recorded at the Cooper gage could be applied to the main stem North Sulphur River and tributaries. Figure 2.3 has been simplified to better reflect wet-dry cycles at the Cooper gage.

2. All calculations for watershed and channel sediment yields have been provided in Appendix E.

3. Gully erosion certainly exists in the watershed upstream of the dam, but was not quantified. Given the conservative nature of our sediment yield estimates, we believe that sediment yield as a result of gullying in the watershed upstream of the dam site is probably accounted for.

4. The locations of the Roxton/Gober Chalk outcrop were mapped in the field and are shown on Figure 2.2.

5. Review of the Harmel et al. (2006) and Baird and Richardson (1970) work does provide some insight into the under prediction of the USGS regression relations, but does not account for the magnitude of the difference. It is also likely that the incised nature of the mainstem and tributaries affects the time of concentration and thus increases the flood peaks.

1730 S. College Avenue, Suite 100 • Fort Collins, CO 80525
970-224-4612 • fax 970-472-6062
www.mussei.com

6a. An Executive Summary has been added to the report.

6b. Table 5.1 contains the drainage area and slope data and has been cross referenced with Figure 3.8 on page 3.13.

6c. Figure 2.37 has been cross referenced in the report.

6d. Geologic units of interest to the study are shown on Figure 2.1.

6e. All cross sections have been plotted at same scale and have been replaced in the text.

7. Figures 2.5 through 2.18 have been left in the text.

8. The discussion in Section 2.2.2.1 has been expanded to differentiate between alluvial and bedrock controlled channels. The effects of the bedrock control are clearly shown in Figure 2.19 and are discussed in the accompanying text. The lower erodibility of the Taylor materials in comparison to the overlying alluvial materials is responsible for the "funnel" shape of the cross sections in the bedrock controlled reaches.

3.0 TECHNICAL DISCUSSION

3.1. Sediment Yield

1. Table 5.4 was added to reflect the range of measured sediment yields in the Blackland Prairie region, and to put the computed estimated values in this report into perspective. We concur that the range of MEI estimated yields are higher than those reported in the literature, and support the conservative nature of the MEI analysis.

2. Watershed sediment yields for all of the tributaries upstream of the dam site were computed, but bed-material loads were only computed for the larger tributaries for which HEC-RAS models were developed. Regression relations between computed bed-material yield and drainage area were developed for the modeled tributaries that incorporated one south-side tributary, Long Creek. The regression equation (5.8) with an R^2 value of 0.9 was used to estimate bed material yields for the smaller un-modeled tributaries (Section 5.2.2.).

3.2. Rates of Degradation

We concur with the discussion of the mechanisms for, and rates of degradation of the exposed bedrock.

3.3. Erosion Hazard for the South and North Slope Tributaries

1. We agree that there are mass failures of the banks on the south side tributaries, but similar mass failures were also observed on the north side

tributaries. In general, the south side tributaries are smaller than the north side tributaries and therefore the sediment yields are likely to be lower. With the exception of Long Creek, the smaller south side tributaries were not hydraulically modeled, but bed material yields were computed with a regression equation developed from estimated yields from the north side tributaries (Section 5.2.2). The estimated bed material yield for Long Creek was very similar to those from the north side tributaries. In the absence of further data to the contrary, we have not differentiated between the north and south side tributary sediment yields.

2. Grading and revegetation of the banks of all the tributaries will reduce the potential for reservoir-related erosion and this is addressed in Section 6.1.

3.4. Downstream Impacts

Currently, only about 25 percent of the estimated total sediment load delivered to the dam site from upstream is composed of bed material. The remainder is wash load and this fraction has little or no morphological significance on the downstream channel morphology. While reductions in sediment delivery downstream of the dam can cause channel changes, the nature and magnitude of the channel changes depends on the relative magnitude of the change in effective sediment supply and the nature of the channel bed material downstream of the dam (Williams, G.P. and Wolman, M.G., 1984. Downstream Effects of Dams on Alluvial Rivers. USGS Professional Paper 1286). Given that the bed of the North Sulphur River downstream of the dam is composed of bedrock, it is apparent that the transport capacity of the river greatly exceeds the bed material supply under existing conditions. Therefore, a 25 percent reduction in the bed material supply is unlikely to have any significant effects on downstream channel erosion rates, especially since erosion rates in the bedrock are primarily controlled by weathering rates and not hydraulic processes. Reduction in the bed-load supply could reduce the covered area of the bed for some distance downstream of the dam and this could locally increase the rate of bedrock weathering. However, field observation did not indicate that the sediment veneer over the shale had an observable impact on the shale weathering rate.

We agree that the observations of the lack of effect of the dam on the alluvial channel of the South Sulphur River are probably inapplicable to the North Sulphur River given the great differences in the boundary materials.

3.5. MUSLE

Local experience in the Blackland Prairie region (R. Moore, NRCS, pers. Communication, 2006) indicates that the MUSLE model does not estimate ephemeral gully erosion very well in that region. Consequently, to preserve the conservative nature of the watershed sediment yield estimates, we conducted a separate calculation for ephemeral gully erosion rates.

4.0. SUPPLEMENTARY INFORMATION

The supplemental morphometric relations developed from field measurements of the tributaries and from plotted cross sections of the main stem in the report do confirm the general relations between channel width and depth and support the channel evolution models discussed in the report. However, without considerably more investigation and a larger data set, the height-width relations should be used with caution to predict future channel widths in the tributaries in the absence of the project and with the project in place. Where the channels have incised into the shale future channel widening is governed by the rate of lateral erosion of the shale, and field observations suggest that top bank erosion and channel widening in the overlying alluvium is limited by the relatively erosion resistant shale toe, which results in a funnel-shaped cross section. Over a long enough period of time the width-depth relations will apply, but until the temporal lag factor has been identified, the relations should be used for predictive purposes with caution.

Sincerely,

MUSSETTER ENGINEERING, INC.

Michael D. Harvey, Ph.D., P.G.
Principal Geomorphologist

MDH/bbv
Enclosure

APPENDIX G

Review of Mussetter Engineering, Inc.'s Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Lake Ralph Hall Project

By Aquafor Beech Limited in Association with Dr. Peter Allen

Aquafor Beech

October 16, 2006

Chiang, Patel & Yerby, Inc.
1820 Regal Row
Suite 200
Dallas, Texas
75235

Attn: Mr. John Levitt, P.E.

Re: Review of Draft Report Entitled "Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Lake Ralph Hall Dam Project" prepared by Mussetter Engineering Incorporated.

Dear Mr. Levitt,

Please find attached a review of the above mentioned Report prepared by Drs. MacRae and Allen.

The North Sulphur River is one of the most interesting Rivers we have had the opportunity to study. Given the complex nature of the River's morphology we found the Report by Mussetter to be well written and a reasonable interpretation of the fluvial system.

It was a pleasure to have worked with you on this project.

Yours truly,

Craig MacRae
Senior Associate
Aquafor Beech Limited
920 Princess Street
Kingston, Ontario
K7L-1H1

Review of
Draft Report Entitled:

"Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Ralph Hall Lake Dam Project"

Authored By:

Mussetter Engineering Incorporated

Submitted To:

Chiang, Patel & Yerby Incorporated.

Prepared By

Aquafor Beech Limited

In Association With

Dr. Peter Allen

For

Chiang, Patel & Yerby Incorporated.

October 16, 2006

Project No.: 64670

4.0 SUPPLEMENTARY INFORMATION

Data characterizing hydraulic geometry attributes of the tributary channels were collected during the field reconnaissance to supplement data collected for the main stem of the North Sulphur River as presented in the Report. The supplementary information may be used to:

1. Document evidence of the magnitude of downcutting at selected locations;
2. Provide insight into models of channel evolution; and,
3. Provide a basis for estimation of sediment loadings from the tributary channels to the North Sulphur River.

Supplementary data includes the following measurements:

a) The top width of the floodplain channel (TW);
b) The height to Top of Bank of the floodplain channel (H);
c) The bottom width of the floodplain channel (B);
d) The width of the active channel at the depth of the dominant discharge (W_{BFL});
e) The depth of the dominant discharge (d_{BLF});
f) The bottom width of the active channel (W_{BED});
g) The slope of the bank (S_b) recorded for the left and right banks of the active and floodplain channels.

Distinct differences in channel form were observed based on the degree of incision into bedrock as noted in the Report. Consequently the channels can grouped into two main classes:

A) Those channels worn into alluvial materials; and,
B) Those channels that have downcut through the alluvium into the underlying bedrock, represented primarily by the Taylor Group of materials.

The later group was further differentiated into:

a. Those channels having contacted the Taylor materials but the depth of incision, for at least one bank, was $d<0.3d_{BFL}$ (where d_{BFL} represents the depth of flow for the dominant discharge). These channels are referred to herein as Rock Bed (RB-Type) channels; and,
b. Those channels where the depth of downcutting into the Taylor Group of materials is $d=0.3d_{BFL}$ for both banks. These channels are referred to herein as Bedrock Controlled (RC-Type).

Definition sketches for the above parameters are provided in Figures 4.1 and 4.2 for channels worn into alluvium and Taylor materials respectively.

In addition to the above observations documenting downcutting were noted along with bank material type and evidence of bank failure through slaking or slumping.

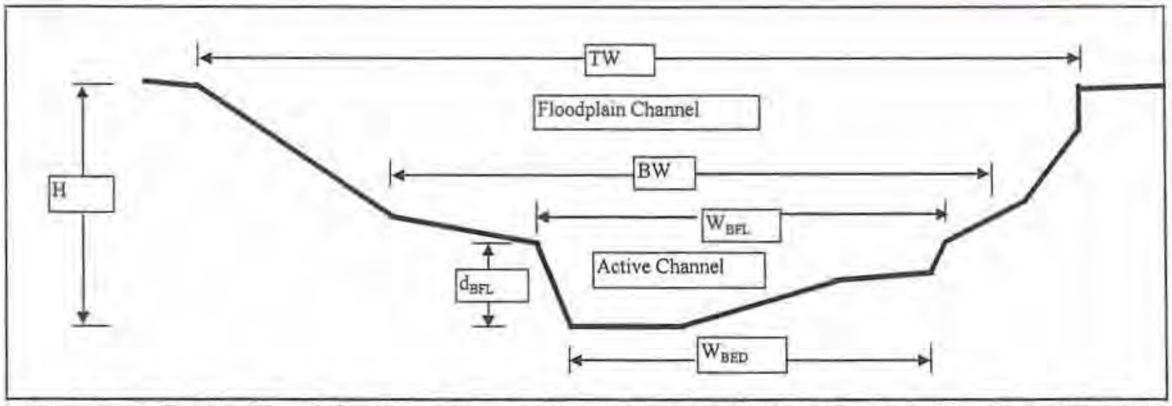


Fig. 4.1. Definition Sketch for Cross-Section Dimensions in Tributary Channels Worn into Alluvial Materials (AL-Type channels).

Supplementary observations were recorded at 23 selected locations as indicated in Table 4.1. One of the survey sites was recorded on the main stem of the North Sulphur River (Site2). One other survey site was located on a remnant of the pre-channelized main stem of the North Sulphur River (Site 1). The remaining survey sites were located on tributaries of the North Sulphur River. Sites 3 through 12 and Sites 17 and 18 are located on the north side of the River while Sites 13 through 16 are located on the south side.

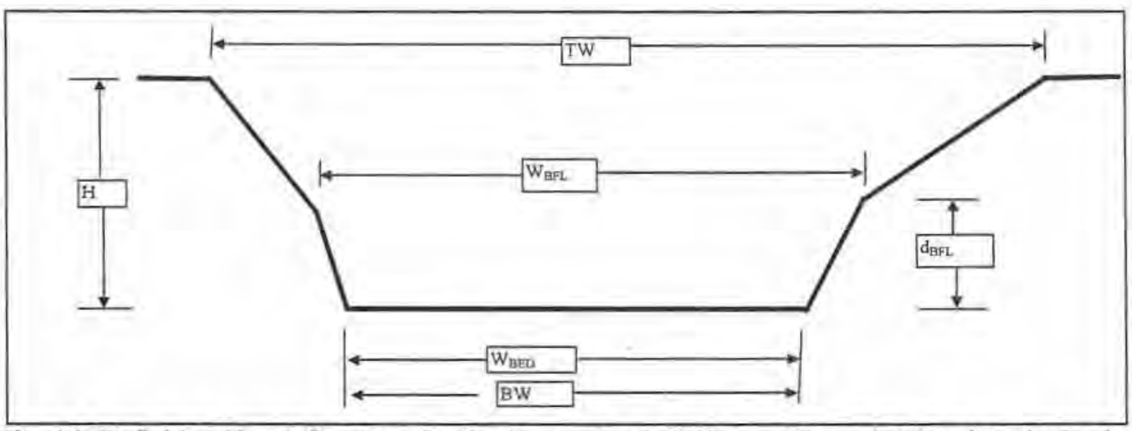


Fig. 4.2. Definition Sketch for Cross-Section Parameters in Tributary Channels Worn into the Taylor Group (RB-Type and RC-Type channels).

The hydraulic geometry relationships for the tributary channels and Site 1 are provided in Figures 4.3, 4.4 and 4.5. The relationship between Height to Top of Bank (H) and Bottom Width (BW) is illustrated in Figure 4.3. The square of the coefficient of calibration (R^2=0.37) for the fitted line is relatively poor. However relationships between H and channel Top Width (TW, Fig. 4.4) and Top Width with Bottom Width (Fig. 4.5) were found to be R^2=0.67 (fair) and R^2=0.73 (good) respectively.

Table 4.1. Summary of Supplementary Field Survey Site Location and Channel Type

Site ID	Location			Channel Type				
	Latitude	Longitude	Description	Main	Tributary	Classification AL	Classification RB	Classification RC
1	33°27.257'	095°56.463'	Pre-channelization remnant of N. Sulphur R. south of current main channel d/s of culvert under Highway 34	?		?		
2	33°27.390'	095°56.531'	N. Sulphur R. u/s of Highway 34 Bridge	?				?
3	33°27.517'	095°56.544'	Small unnamed tributary north of N. Sulphur R. d/s of culvert under Highway 34		?	?		
4	33°27.524'	095°56.555'	Small unnamed tributary north of N. Sulphur R. u/s of culvert under Highway 34		?	?		
5			Merrill Creek 609 ft d/s of County Rd 1550 Bridge		?			?
6	33°28.888'	095°56.463'	Bralley Ck. 90 ft d/s of County Rd. 1550		?			?
7a	33°29.259'	095°58.014'	Davis Ck. 203 ft d/s of County Rd. 1550		?			?
7b			Davis Ck. 150 ft d/s of County Rd. 1550		?			?
7c			Davis Ck. 50 ft d/s of County Rd. 1550		?			?
8a	33°29.240'	095°58.692'	Leggets Br. 220 ft d/s of County Rd. 1550		?	?		
8b			Leggets Br. 250 ft d/s of County Rd. 1550		?	?		
9	33°29.265'	095°59.735'	Davis Ck. 200 ft d/s of County Rd. 1550		?		?	
10a	33°29.314'	095°59.738'	Davis Ck. tributary 30 ft u/s of culvert under County Rd. 1550		?	?		
10b			Davis Ck. tributary 20 ft d/s of culvert under County Rd. 1550		?		?	
11a	33°29.288'	096°00.777'	Pickle Ck. 400 ft d/s of County Rd. 1550		?	?		
11b			Pickle Ck. u/s of culvert		?	?		
12			Bushy Ck. d/s of County Rd. 1550		?		?	
13	33°26.421'	095°56.161'	Unnamed tributary 60 ft. u/s of County Rd. 3540 east of Highway 34		?	?		
14			Unnamed tributary 40 ft d/s of culvert under County Rd. 3640 east of Highway 34		?	?		
15	33°26.867'	095°55.296'	Unnamed tributary d/s of County Rd. 3640 near Pleasant Grove Cemetery		?		?	
16	33°26.867'	095°55.296'	Hedrick Br. 300 ft u/s of County Rd. 3640		?	?		
17			Pot Ck 60 ft d/s of County Rd 3330		?			?
18	33°28.614'	095°03.518'	Pot Ck 180 ft u/s of Gober Outcrop		?			?

Figures 4.6 and 4.7 illustrate relationships for cross-section parameters for the active channel for all channel Types. The relationships as represented using the square of the coefficient of calibration ranged from fair for Bankfull Width (W_{BFL}) as a function of the depth of the dominant discharge (d_{BFL}, Fig. 4.6) to very good for W_{BFL} as a function of Bed Width (W_{BED}, Fig. 4.7). The inclusion of all channel Types explains some of the scatter observed in the plots because of differences in the width:depth (W_{BFL}/d_{BFL}) ratios. The average values for the width:depth ratios were W_{BFL}/d_{BFL} =5.7, 7.9 and 11.1 ft for AL-Type, RB-Type and RC-Type respectively. Consequently it is possible that stronger relationships could be developed based on channel Type given a larger data base.

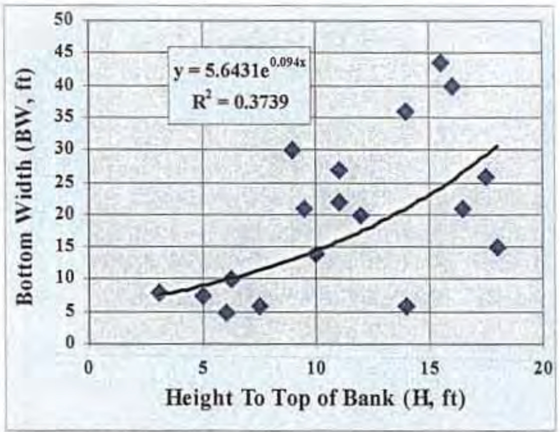


Fig. 4.3. Relationship Between Bottom Width and Height to Top of Bank For AL-Type Channels Tributary to the North Sulphur

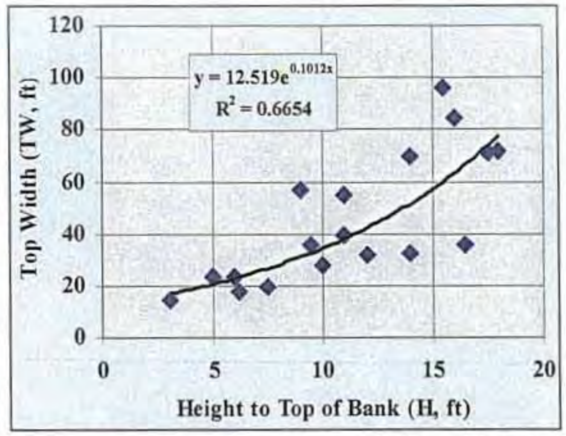


Fig. 4.4. Relationship Between Top Width and Height to Top of Bank for AL-Type Channels Tributary to River. the North Sulphur River.

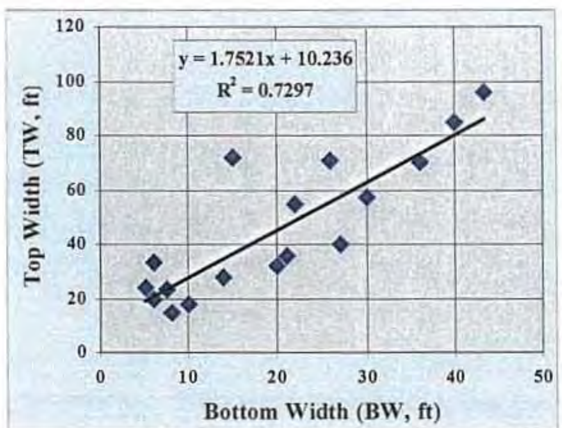


Fig. 4.5. Relationship Between Top Width and Bottom Width for AL-Type Channels Tributary to the North Sulphur River.

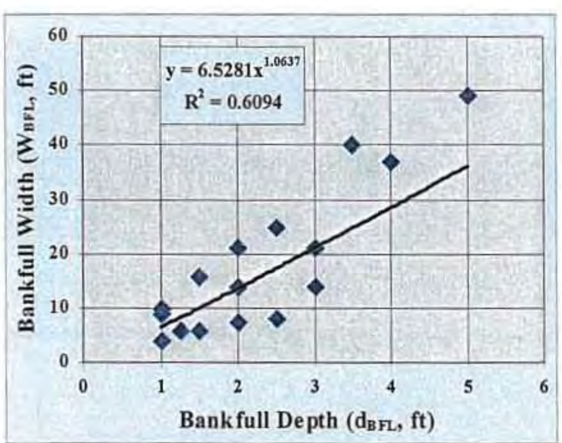


Fig. 4.6. Relationship Between Bankfull Depth And Bankfull Width For the Active Channels Tributary to the North Sulphur River

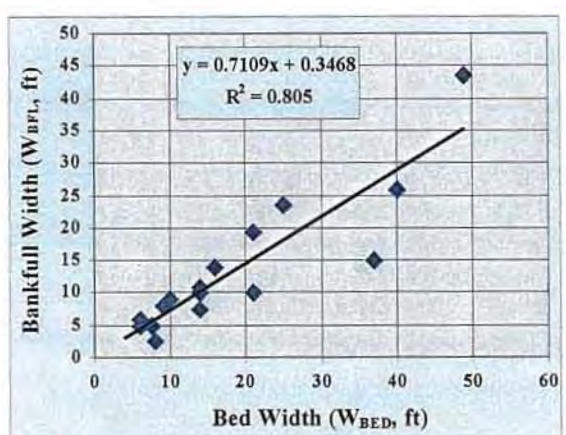


Fig. 4.7. Relationship Between Bed Width And Bankfull Width For the Active Channels Tributary to the North Sulphur River.

The relationship between the floodplain channel and active channel as represented by Top Width (TW) as a function of bankfull width (W_{BFL}) was found to be fair (Fig. 4.8).

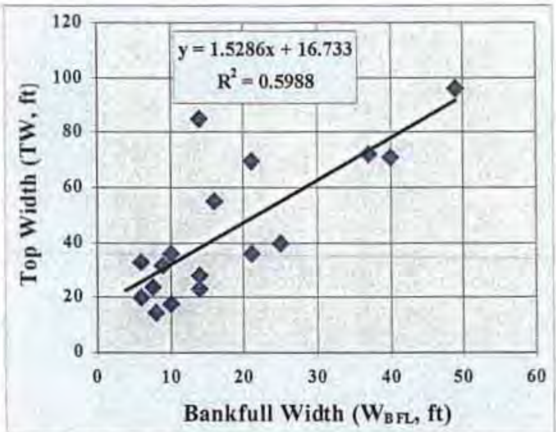


Fig. 4.8. Top Width of the Floodplain Channel as a Function of Bankfull Width of the Active Channel For Tributaries of the North Sulphur River.

The tributary observations represent small scale channel systems relative to the main stem of the North Sulphur River. Combining the data sets broadens the spatial scale for examination of possible relationships. Potential relationships as presented in Figures 4.9 through 4.11 were very good to excellent. The data for the main stem of the North Sulphur River were obtained from Site 2 reported above and Figures 2.5 through 2.18 of the Mussetter Report.

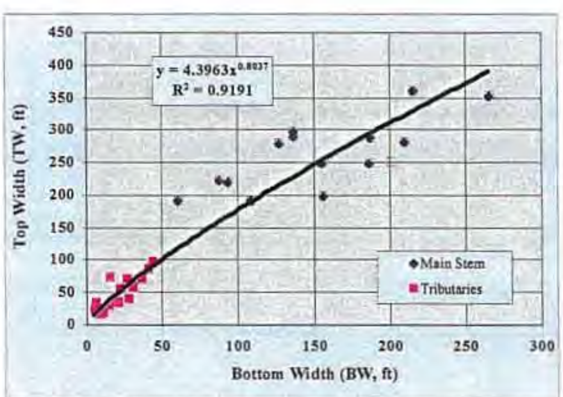


Fig. 4.9. Top Width of the Floodplain Channel as a Function of Bottom Width for the North Sulphur River Upstream of the Proposed Dam.

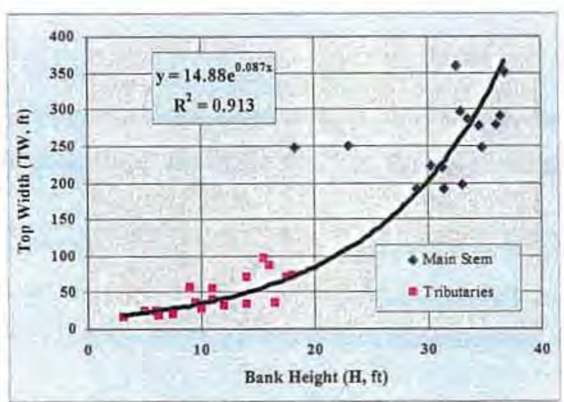


Fig. 4.10. Top Width of the Floodplain Channel as a Function of Bank Height for the North Sulphur River Upstream of the Proposed Lake Ralph Hall Dam

The relationships illustrated in Figs. 4.9 through 4.11 suggest that the North Sulphur River is behaving in a predictable and consistent manner. Further, it is possible to predict the evolution of channel morphology in the tributary channels based on behavior of the main stem of the North Sulphur River. Consequently the channel evolution models applied to the main stem of the River are applicable to the tributary channels.

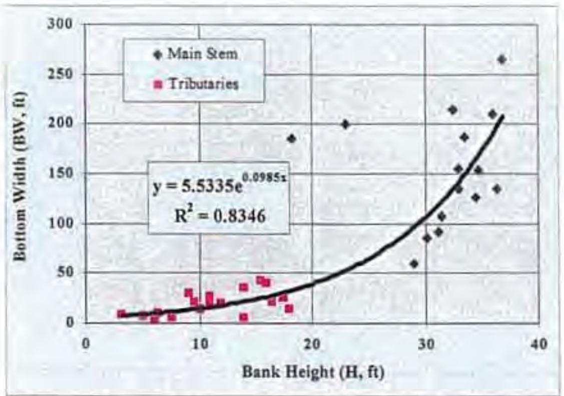


Fig. 4.11. Bottom Width as a Function of Bank Height for the North Sulphur River and Tributary Channels Upstream of the Proposed Lake Ralph Hall Dam.

Some of the scatter observed in the relationships presented above is related to differences in the resistance of the boundary materials, stratification, groundwater characteristics, aspect, livestock access and the type, density and distribution of riparian vegetation among other factors. Differences in boundary material resistance may be approximated by the slope of the banks of the channel as presented in Figures 4.12, 4.13 and 4.14.

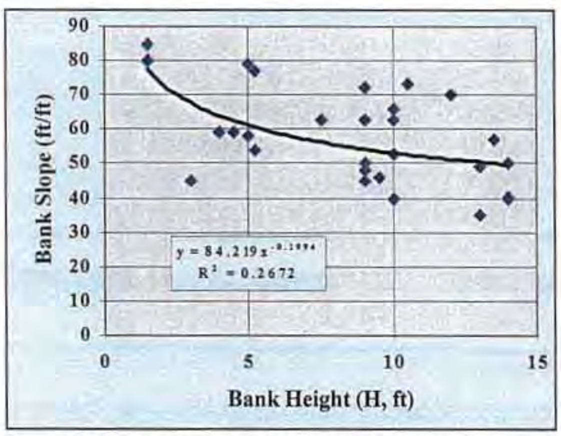


Fig. 4.12. Bank Slope As A Function Of Bank Height For AL-Type Tributary Channels To The North Sulphur River Upstream Of The Proposed Lake Ralph Hall Dam.

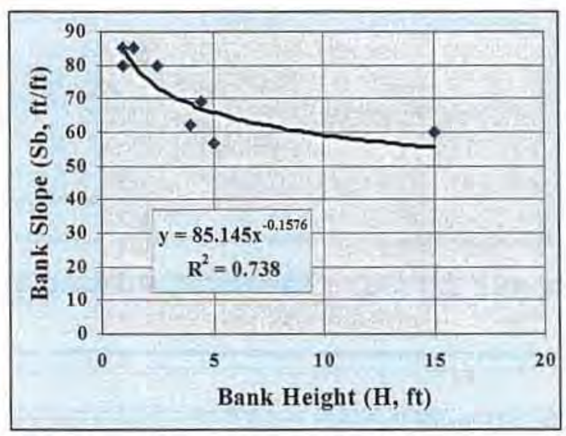


Fig. 4.13. Bank Slope As A Function Of Bank Height For RB- And RC-Type Tributary Channels To The North Sulphur River Upstream Of The Proposed Lake Ralph Hall Dam.

The relationships in Figs. 4.12 through 4.14 suggest that bank slope decreases with bank height, which is also a surrogate variable for decreasing effectiveness of root binding associated with riparian vegetation. Although the number of observations is limited the data provides a means to approximate the change in bank slope with degradation.

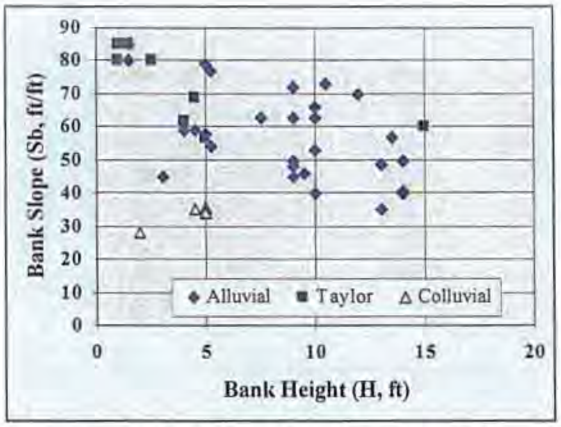


Fig. 4.14. Comparison of Bank Slope For AL-Type and RB- and RC-Type Channels Worn Into the Taylor Group of Materials and Colluvial Deposits.

The relationships presented above may be used to predict changes in channel form through time knowing the rate of downcutting. Downcutting may be approximated as noted in the Report using historic cross-section and sediment yields as well as literature values noted in this review. Supplementary data from which rates of downcutting in the tributary channels may be derived was also collected during the field reconnaissance as reported below.

Site 3: The depth of downcutting at Site 3, a small unnamed tributary crossing under Highway 34 north of the main stem of the North Sulphur River was estimated to be 7 feet below the culvert invert Fig. 4.15. Gabions and a concrete splash pad have been installed beneath the original culvert invert with the top of the new splash pad 5 feet below the original invert elevation. The channel has subsequently downcut and estimated 2 feet below the invert of the new splash pad. Knowing the dates of construction of the original culvert and new splash pad would allow for estimation of downcutting over this interval. A second rate may be obtained over the interval between construction of the new splash pad and the current date.

The incision of the channel downstream of the culvert is contrasted by a scour pool of approximately 0.5 feet in depth upstream of the culvert (Site 4).

Site 7b: Post-settlement alluvial deposits were observed in the upper bank region of Davis Creek at Site 7b. These deposits represent the probable elevation of the thalweg at the time of channelization of the main stem of the North Sulphur River. The deposits were observed 6 feet below the top of bank, 18 feet above the current channel thalweg representing a probable depth of incision of 12 feet.

Site 7c: Concrete from the old bridge buried in granular material was exposed in the bank of the active channel. The stratum containing the concrete was approximately 1.5 feet thick. The channel has downcut through this unit and 3 feet into the underlying Taylor material. If the date of replacement of the bridge on County Road 1550 is known then the rate of downcutting can be approximated.

Site 10b: The thalweg of Davis Creek immediately downstream of the culvert at County Road 1550 is approximately 10 feet below the culvert invert. The channel has cut through 8 feet of alluvium and 2 feet into the underlying Taylor shale.

Site 11b: The thalweg of Pickle Creek 48 feet downstream of culvert under County Road 1550 is 5 feet below the invert of the culvert.

5.0 CONCLUSIONS

In summary the Mussetter report was found to be an insightful, comprehensive, and well documented overview of past and current erosion processes as well as future erosion potential and morphologic evolution of the upper North Sulphur River (NSR). This represented a unique challenge because the River does not conform to the majority of Blackland Prairie channel systems. Indeed the North Sulphur River is infamous in North Texas for the dramatic stream erosion/degradation that has occurred over the past 75 years due to past channelization projects. Deciphering its history is difficult owing to disaggregated data and often poor historical records. However this Report provides a fair, descriptive story of the erosion history and quantitative assessment of erosion potential for the upper North Sulphur River watershed.

A summary of major points discussed in Section 3.0 Technical Review is provided below.

Section 3.1: Given the rates of erosion in the Blackland in reservoir studies, gage data, and assumptions provided in the text, and our limited field observations in the North Sulphur River Basin, the cited 1630 tons per square mile per year appears to be a reasonable estimate of future erosion rates in the basin. The worse case scenario cited by the Mussetter report of 4280 tons per square mile appears to be a reasonable upper limit for erosion in this watershed given the cited information and past studies by the authors.

More extensive quantification of the tributary contribution to the estimate of sediment yield would help substantiate the values in the Report. Our preliminary field assessment indicates that there is a predictable continuum from the tributaries to the main channel which could be useful in this explanation of the future impacts of tributary erosion in the watershed.

Section 3.2: Slaking is the limiting factor in controlling the rate of future degradation in this climatic regime of seasonal wetting and drying and shale bed material. It follows that mitigation measures for the control of erosion must prevent exposure of the shales to the elements. This may be achieved either through burial or submergence of the Taylor materials.

Much of the work done by Dr. Allen and others on sedimentary rock incision supports the rates of degradation cited by the Mussetter Report; in the range of 1-3 inches per year.

Section 3.3: Mitigation measures such as grading and planting the banks along the shoreline of the proposed lake and other forms of toe control could significantly reduce the sediment yield associated with mass failure of the banks and reduce the sediment loads entering the proposed reservoir. There are numerous reservoirs already built which are in such shale terrain that could be studied and used to assess the magnitude of this potential problem prior to design.

When estimating sediment yields to the North Sulphur River it may be advisable to differentiate between the north and south side tributaries.

Section 3.4: More explanation of the potential effects on the receiving channel downstream of the project is warranted.

Section 3.5: If the MULSE model is typically thought to include ephemeral gully erosion 5.1.3 then the procedure used in the report would tend to be conservative or overestimate sediment yield.

6.0 CITED REFERENCES:

Simon, A. Dickerson, W., Heins, A.(2004). Suspended-sediment transport rates at the 1.5 year recurrence interval for ecoregions of the United States: transport conditions at the bankfull and effective discharge?. Geomorphology, 58 pp. 243-262.

Biedenharn, D.S. and Watson, C.C. (1997) Geomorphic assessment of incised channel systems. (In) Proceedings of the Conference on Management of Landscapes Disturbed by Channel Incision (eds.) Wang, S.S.Y., Langendoen, E., and Shields, F.D. Jr., Oxford Mississippi., pp. 753-758.

Greiner, J.H. Jr. (1982) Erosion and sedimentation by water in Texas. Report 268, Texas Department of Water resources, Austin, Texas, 145p.

Coonrod, J.E.A., Holley, E.R., and Maidment, D.R. (1998). Suspended sediment yield in Texas watersheds. Center for Research in Water Resources, Technical Report, CRWR 270.

Alan Plummer and Associates, Allen, P.M., and Dunbar, J. (2005). Dredging versus new reservoirs. Report to Texas Water Development Board. TWDB Contract #2004-483-524. misc. pages.

Schumm, S.A., Harvey, M.D., and Watson, C.C. 1984. *Incised Channels: Morphology, Dynamics and Control.* Water resources Publications, Littleton CO. 200 p.

Harmel, R.D., Richardson, C.W., King, K.W., Allen, P.M. (2006) Runoff and soil loss relationships for the Texas Blackland Prairies ecoregion. (Accepted for publication, Journal of Hydrology).

Texas Water Development Board (1974) Suspended sediment load of Texas Streams. October 1965-September 1971. TWDB Report 184., Austin, Texas, 119p.

Baird, R.W. and Richardson, C.W. (1970). Effects of Conservation Practices on Strom Runoff in the Texas Blackland Prairie. USDA, Tech. Bulletin, No. 1406. 31p.

Appendix D

Hydrologic and Hydraulic Studies

D-1: Evaluation of Hydrologic Modeling

D-2: Hydrologic and Hydraulic Studies for Lake Ralph Hall

D-3: RiverWare Modeling

D-1: Evaluation of Hydrologic Modeling

DiNatale
WATER CONSULTANTS

Matt Barkley
Michael Baker International
810 Hester's Crossing, Suite 163
Round Rock, TX 78681

June 3, 2016

Chandler Peter
US Army Corps of Engineers
Regulatory Division, Fort Worth District
819 Taylor St.
Fort Worth, TX 76102

Mr. Barkley and Mr. Peter,

Please find attached our draft-final report on our evaluation of the hydrologic modeling of the Sulphur River used for the Lake Ralph Hall EIS. This draft addresses your comments to our April 4, 2016 and May 24, 2016 drafts. Pursuant to the review process we have discussed, I submit this draft for your review. If you have no further questions, please forward this draft to the Upper Trinity Regional Water District.

Thank you,

Matt Bliss
Project Manager

303 709 7044 dinatalewater.com
2919 VALMONT ROAD SUITE 204 BOULDER CO 80301

Evaluation of Hydrologic Modeling in Support of the Lake Ralph Hall Environmental Impact Statement

Prepared for the U.S. Army Corps of Engineers
Draft-final for Corps, Third Party Contractor and Upper Trinity Regional Water District Review
June 3, 2016

Contents

List of Figures:

Table of Tables:

1.0 Introduction

The United States Army Corps of Engineers (Corps) is developing an environmental impact statement (EIS) related to the proposed construction of Lake Ralph Hall near Ladonia, Texas. The applicant for the project is the Upper Trinity Reginal Water District (hereafter Applicant or Upper Trinity). During the course of the EIS, Upper Trinity has used hydrologic models to assess stream impacts to the Sulphur River. DiNatale Water was contracted as the Corps' third-party contractor to evaluate the adequacy of the hydrologic modeling for the purposes of the EIS, verify the modeling performed by Upper Trinity, and perform additional modeling as necessary.

The Corps' regulatory requirements associated with the EIS require an analysis of the impacts to aquatic resources caused by the proposed project. The Corps has identified the following aquatic resources that could potentially be impacted by the proposed project:

- Geomorphology and sediment transport
- In-channel pools and puddles that support benthic organisms and fish
- Floodplain resources
- Water quality and temperature
- Groundwater

Hydrologic modeling can be used to assist in quantitatively assessing the impacts to each of these resources by simulating a current conditions baseline scenario and a with-project scenario. The differences in hydrology between the baseline and with-project scenarios allow the resources to be evaluated under both conditions and any changes from the baseline are attributed to the project. The Corps' requirements also include an evaluation of cumulative impacts from other projects or reasonably foreseeable future actions. To assess cumulative impacts in locations where multiple projects are being considered or where land and water uses are projected to change significantly within the planning horizon timeframe, the Corps may simulate future hydrologic conditions to assess the likely future impacts attributable to the project. In this instance, the Corps determined that future hydrologic conditions were not necessary to adequately evaluate the aquatic impacts of Lake Ralph Hall, and therefore only current conditions hydrologic scenarios were used. A more in-depth discussion of this determination is included in Section 5.

Upper Trinity has used two different models to evaluate the flows below the proposed dam. The first is the State of Texas' Water Availability Model that uses the Water Rights Analysis Package modeling platform (WAM/WRAP) developed for the Sulphur River basin. The second is a RiverWare model developed by the Corps for a larger Red River Basin modeling effort (the Sulphur River is tributary to the Red River). The Corps also provided a HEC-RAS model that was developed by the Corps for the Sulphur River basin. The models have different characteristics and were built for different purposes. DiNatale Water evaluated the models in terms of the adequacy of assessing the impacts to the aquatic resources described above.

For the Lake Ralph Hall EIS, the Corps developed approaches to evaluation of each of the aquatic resources identified above. The hydrology for in-channel pools and puddles that support benthic organisms and fish was evaluated more in depth by the Applicant. This report evaluates the Applicant's analysis and provides recommendations on its use (Section 4.1). Detailed hydrology for floodplain resources was evaluated in this report using a Corps-developed HEC-RAS model for the Sulphur River Basin (Section 4.2). Geomorphology, water quality and temperature, and groundwater resources will be evaluated using a qualitative approach. These approaches do not require detailed hydrologic modeling for input, but we discuss the potential supporting role existing modeling can provide for these resources and identify modifications necessary if it is later determined that more refined evaluation is required for any of these resources (Sections 4.3, 4.4 and 4.5).

2.0 Site and Hydrology

The proposed Lake Ralph Hall is located in Fannin County, Texas near the town of Ladonia, northeast of the Dallas/Fort Worth metropolitan area. The proposed Lake Ralph Hall has a conservation pool capacity of approximately 160,000 AF and a maximum capacity of approximately 180,000 AF and a maximum surface area of 8,500 acres. The reservoir will be located on the existing channel of the North Sulphur River. Upper Trinity proposes to pump water directly from the reservoir through a new pipeline south and westward and will connect with an existing pipeline for delivery to the Upper Trinity service area. Upper Trinity anticipates pumping a maximum of 45,000 AF per year with a maximum diversion rate of 205 cubic feet per second (cfs) from Lake Ralph Hall. The WAM/WRAP hydrologic modeling used in support of the Lake Ralph Water right indicates the annual yield may drop to as low as 16,800 AF per year through the design drought of the 1950's.

At the location of the proposed dam for Lake Ralph Hall, the North Sulphur River resembles a deep canal (Figure 1). Prior to the 1930's, the bottomland of the North Sulphur River was a swamp and marsh area. In the late 1920's, local residents sponsored a channelization project and dug a straight canal through the bottomland to drain the area and open up large amounts of land for agriculture (TCEQ Proposed Order, undated). This canal is the current day course of the North Sulphur River. The channelized section of stream extends east to near Talco, some 40 miles from the Lake Ralph Hall site. The channelized section of the river is clearly visible from areal imagery to the confluence with the South Sulphur

River and to near the State Highway 37 bridge. Over the past 80 years, the North Sulphur River has eroded the canal and has cut down through layers of claystone and widened. The canal today is approximately 60 feet deep and 200 feet wide near the Lake Ralph Hall site.

The hydrology of the North Sulphur River is highly variable and flashy. The river will often have no flow, or very little flow. During a rain event, however, flows increase very rapidly and to flow rates of several thousand cfs. After a rain event, flows recede typically within a day or two to near zero flow again. After these large events, some small ponds and puddles form in the bottom of the river channel and may be able to sustain benthic organisms and fish between larger flow events. There is one stream gage on the North Sulphur River, located near the town of Cooper (USGS gage 07343000, N Sulphur Rv nr Cooper, TX, hereafter the “Cooper gage”). There are other downstream gages on the Sulphur River near Talco (USGS gage 07343200, Sulphur Rv nr Talco, TX, herafter the “Talco gage”) and Sulphur River near Dalby Springs (USGS gage 07343450, Sulphur Rv at IH 30 nr Dalby Springs, TX, hereafter the “Dalby Springs gage”). Figure 2 shows a typical hydrograph storm events at the Cooper gage and follows these same storm events to the downstream Talco and Dalby Springs gages. The catchment basin above the Cooper gage is 311 square miles. The Lake Ralph Hall site is a subset of this basin with a catchment area of 101 square miles. Figure 3 shows the average annual rainfall totals in northeast Texas and the approximate locations of Lake Ralph Hall and the Cooper Gage. Although the Sulfur River and tributaries flow through two different types of land resource areas which are characterized by different soils, the Lake Ralph Hall site and the Cooper Gage lie within the same Blackland Prairie area with predominately clay and silty clay soils which help encourage agricultural land use above the confluence of the North and South Sulfur Rivers. Other than the trend of decreasing precipitation moving west in the basin, there are no distinguishing factors for the basin above the Lake Ralph Hall site that would indicate different runoff per unit area than at the Cooper Gage as a whole.

Figure 1. Photo of the North Sulphur River channel at the State Highway 34 bridge

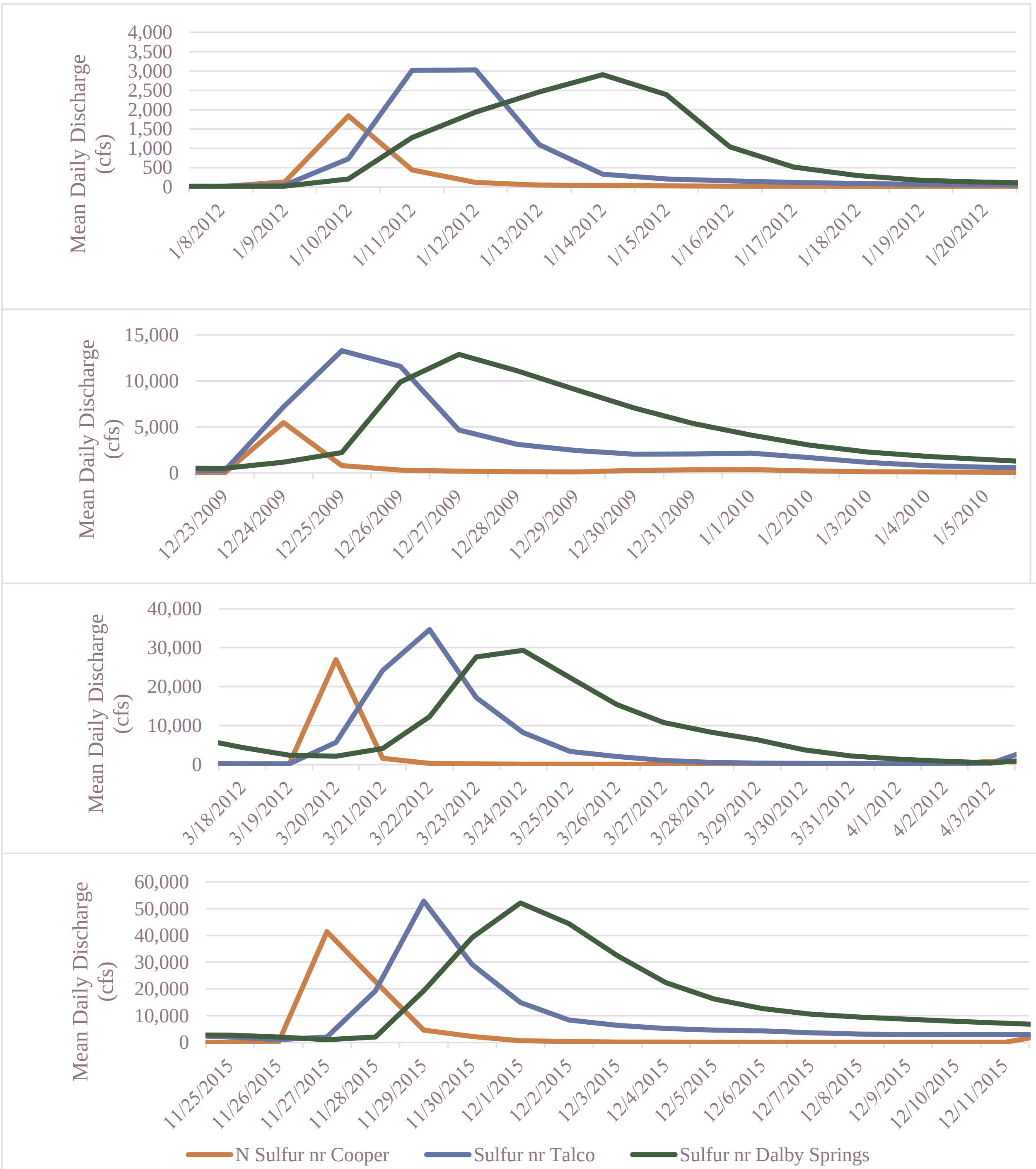


Figure 2. Hydrographs for each gage showing different levels of flow from different storm events.

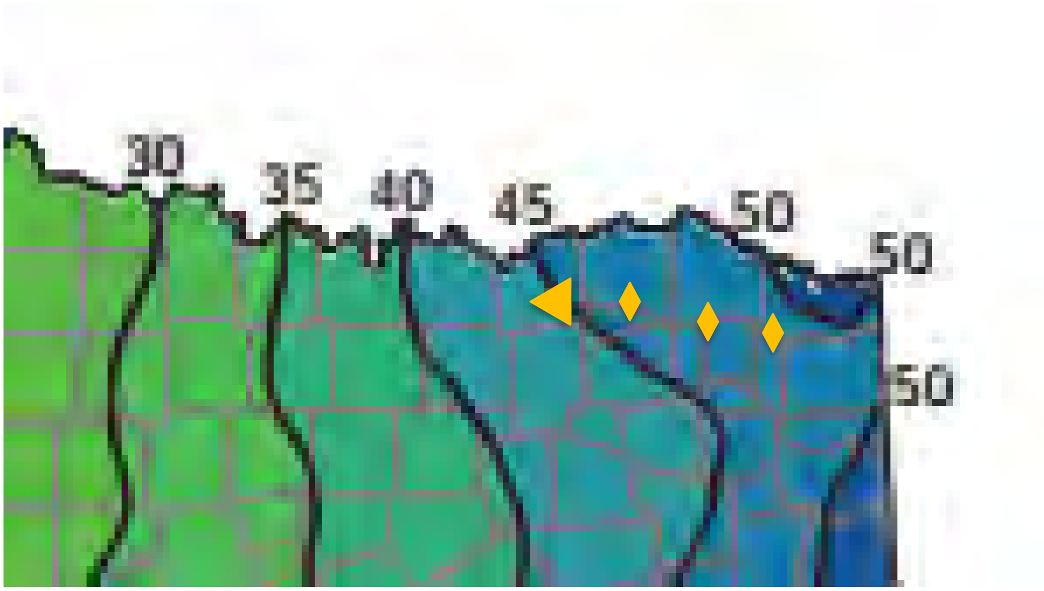


Figure 3. Average annual precipitation (1981 to 2010, values in inches) in northeast Texas. Lake Ralph Hall approximate location shown as a triangle, Cooper gage, Talco gage and the Dalby Springs gage approximate locations shown as a diamonds. Precipitation map source: 2012 Texas State Water Plan.

3.0 Hydrologic Models Utilized

Several hydrologic models have been used to-date for analysis of various aspects of the proposed project. Often models are constructed for one particular intended use, and the model results do not directly apply to evaluation of aquatic impacts. DiNatale Water evaluated the adequacy of the models for the purposes of evaluating the aquatic impacts for the EIS. For this project, we evaluated three model platforms 1) Sulphur River Water Rights Analysis Package and Water availability model (WRAP/WAM), 2) Corps RiverWare model of the Red River with Applicant modifications, and 3) a Corps HEC-RAS model.

3.1 Sulphur River WAM/WRAP

The Texas Commission on Environmental Quality (TCEQ) has developed several hydrologic water availability models for different river basins throughout Texas. The Water Rights Analysis Package (WRAP) is the computer program or modeling platform. Each river basin's model has its own set of input files that describe the hydrology, water rights, demands and other features of the basin. These inputs files are referred to as the Water Availability Model (WAM).

The water availability models are used by the TCEQ to evaluate whether water will be available to a proposed use under various assumptions. The Sulphur River WAM model simulates the North Sulphur River, South Sulphur River, Sulphur River mainstem, White

Oak Creek and the watershed above Wright Patman Lake. The simulation utilizes historical hydrology as flow inputs, but can be configured to include current demands (WAM Run Number 8), or can include full authorization of all water rights in the basin (WAM Run Number 3). The simulation allocates flow to the various water rights according to demand for water and priority of the water right. The TCEQ uses information from the full authorization model run to evaluate the reliability of a proposed water right under future conditions with other conservative assumptions about return flows and water reuse. This model run is useful in determining the future reliability of a water right, but is not necessarily representative of how streamflows will be affected under current water uses.

3.1.1 CORPS' EVALUATION OF WAM MODELS

The Corps recently completed an investigation into the utility of the publically available version of the Sulphur River WAM model developed by the TCEQ as compared to a model modified or developed for a specific project (Corps 2016). The report included a case-study evaluation of the Sulphur WAM model. One of the key conclusions in the report was that while WAM modeling is appropriate for its original intent – water rights administration and reliability analyses, the WAM modeling may not be appropriate for other resources that the Corps evaluates through the EIS process. The report pointed to several reasons why the WAM model may not accurately portray actual stream conditions to a level needed for the Corps analysis. The current conditions WAM model run (Run 8) is better suited to evaluate the impacts to streamflows that would be caused by a proposed project, but utilizes the highest demands from the past 10 years, sets agricultural return flows to zero and uses the lowest return flows in the past 10 years. These assumptions may over-predict diversions and under-predict streamflows under average current conditions.

The Corps 2016 report also compared several historical stream gage and historical reservoir storage levels against the WAM current conditions run. The quality of calibration varied between different locations in the basin. However, at the North Sulphur near Cooper gage (control point B10), the WAM flows matched gaged flows almost exactly. This is somewhat expected due to the minimal water resources development upstream of the Cooper gage. Therefore, use of the current conditions WAM flows on the North Sulphur River above the Cooper gage will avoid many of the potential shortcomings identified in the Corps 2016 report. Use of WAM flows at downstream locations, including the Sulphur River below the confluence with the South Sulphur River, is not recommended due to the relatively poor calibration depicted in the report near Lake Jim Chapman.

The Corps 2016 report also evaluated the WAM models from a temporal and spatial perspective. The WAM models use a monthly time step. This time step is appropriate for water rights reliability and yield analyses, but is inadequate for some of the resources being evaluated for the Lake Ralph Hall EIS. For example, floodplain resources are impacted by peak flow events when the river overtops its banks. This type of event would not be captured in monthly flow volumes due to the averaging of flows over the entire month. Water quality factors may not be accurately represented with a monthly time step, especially in the North Sulphur River. As depicted in Figure 2, the flashy nature of the flows is not captured in a monthly time step, and a small number of large flow events followed by no flow would likely have very different water quality effects than a monthly-averaged flow rate over the entire month. Additional information and recommendations are presented in Section 4 related to specific resources.

The Sulphur River WAM model uses hydrology from 1940 to 1996. This timeframe is reasonable because it captures periods with low, average and high flow events, including the 1950's drought which had been the drought of record throughout Texas. In some areas of Texas, the 2011 drought was more severe than the 1950's drought and established a new drought of record. During a project meeting, the Applicant stated that the 2011 drought was not as severe as the 1950's drought in the Sulphur Basin. Streamflow records at the North Sulphur near Cooper gage confirm this for the Lake Ralph Hall drainage basin, with the cumulative deficit (compared to average) from 1951 to 1957 larger than the cumulative deficit from 2010 to 2014. Although the more recent 2010 to 2014 drought appears to have been more intense than the 1950's drought, it was shorter in duration. Therefore, it is reasonable that the yield analysis performed using the WAM model and the 1940 to 1996 study period is valid in light of the more recent 2011 drought.

From a spatial perspective, the WAM model reasonably includes areas that could be affected by the proposed project in the Sulphur River Basin. To our knowledge, no analysis has been done about the potential impacts to the receiving basin, which is not included in the Sulphur WAM modeling. Water introduced to the Trinity River Basin from Lake Ralph Hall will be consumed by Upper Trinity customers through first use and reuse of the water. If additional analysis of the effects of this inflow water to the basin is needed, the Sulphur River WAM would not be the appropriate tool. The Trinity River WAM may provide some insight, but our assumption is that potential impacts to the Trinity River basin would involve water quality or reservoir operations in the receiving lakes that would be better addressed through reservoir and water quality analysis techniques better suited to evaluate those resources.

3.1.2 VERIFICATION OF WAM MODELS USED FOR LAKE RALPH HALL EIS

Upper Trinity provided DiNatale Water with the WAM model files that were used in the water right application for Lake Ralph Hall. The models provided by Upper Trinity include one version with Lake Ralph Hall operable, and one with Lake Ralph Hall disabled. DiNatale Water executed the models and was able to replicate the model results provided by Upper Trinity. We also compared the model inputs and model results to the publically available Sulphur River Basin WAM files available on the TCEQ website. The Upper Trinity version of the model had refined the area near Lake Ralph Hall considerably to include the details of the basin above and below the project site. We compared the hydrologic inflows above Lake Ralph Hall to the TCEQ version and found them to be identical. We compared other model inputs and outputs, including inflows at other locations, demands, and simulated stream flows at gaged locations and concluded that the WAM models used by Upper Trinity in its evaluation were reasonable adaptations of the publically available version.

The WAM model operates on a monthly time-step. This time-step is useful for determining the yield of a project and reservoir operations and for water supply planning purposes. The WAM model also adheres to the Texas water rights system where upstream junior water rights must pass water to downstream senior water rights when the downstream senior rights are not fully satisfied. However, given the flashy nature of streamflows in the North Sulphur River (Figure 2), monthly flows will not adequately capture the peak flows and long periods of low or no flow that are common to this river basin. For example, the second flow event shown in Figure 2 is one of two high flow events of that month. The peak flow lasts for two or three days, peaking at 5,470 cfs before flows return to near zero. When summarized on a monthly basis, the daily average flow for the entire month is 375 cfs.

One of the primary advantages to the Sulphur WAM modeling over the Corps' Red River RiverWare model of the basin is WAM's simulation of water rights in the Sulphur Basin. In an Applicant report (Brandes 2015), a comparison of releases from Lake Ralph Hall in the WAM model and the RiverWare model indicate higher releases in the WAM modeling than in RiverWare due to the draw from downstream senior water rights. Simulation of senior water rights in the WAM model simulates times when water would be bypassed at Lake Ralph Hall to downstream senior water users. This is particularly important to understanding the impacts of Lake Ralph Hall during low flow times, as downstream seniors would only call water past Lake Ralph Hall during times of shortage. The WAM modeling will show water bypassed at Lake Ralph Hall during some low flow periods where the Corps' RiverWare model – as currently configured – will not.

Although the WAM model will show bypasses to downstream senior water right at Lake Ralph Hall during low flow periods, the WAM modeling may over-predict the amount of water bypassed. The WAM documentation for the Sulphur River (Brandes 1999) indicated that the modeling of Lake Wright Patman included a seasonal conservation pool target. In months where the target storage level increases, an immediate demand for upstream water to satisfy the senior Wright Patman water right is simulated and water may be bypassed from upstream junior water rights, such as Lake Ralph Hall. With regard to Senate Bill 1 water availability analyses, this is the correct interpretation of strict administration of the prior appropriation doctrine. However, Brandes (1999) states that this situation is "somewhat artificial and not likely to happen under current reservoir operating procedures and water rights administration policies."

Therefore, while the WAM model results correctly simulates bypassing water to downstream senior water rights, it may over-predict streamflows below Lake Ralph Hall at times when the water is being passed downstream to Wright Patman. The potential impacts of this operation on aquatic resource evaluations are discussed in more detail below in Section 4.

3.1.3 CONCLUSIONS ON USE OF WAM FOR THE LAKE RALPH HALL EIS

The Corps' regulatory framework requires evaluation of impacts to the aquatic habitat resources that often require understanding of daily flow rates. Use of a monthly-averaged flow rate to evaluate these types of aquatic resources will not provide a correct evaluation of such resources. The Applicant's analysis of aquatic impacts to benthic organisms and fish in the puddles and pools below the dam used monthly flow values by determining a monthly flow threshold so that WAM monthly modeling results could be used. The aquatic impacts to floodplain resources require daily flow rates, and therefore the WAM model results would not be appropriate to use to evaluate these impacts. For the Lake Ralph Hall EIS, stream morphology, water quality and temperature, and groundwater resources are being evaluated qualitatively so detailed hydrologic modeling is not required. However, more detailed refined and quantitative analyses of stream morphology and water quality and temperature resources would likely require a daily time step and WAM would not be appropriate to support evaluation of these resources quantitatively. Monthly modeling results from WAM would likely be appropriate for groundwater resource evaluation given the typically longer time-scales associated with groundwater flow.

The Corps evaluation of the publically available WAM models identified certain assumptions in the WAM modeling related to the seasonal conservation pool in Wright Patman and the underlying assumptions used in the current conditions WAM model run 8 can introduce inaccuracies to simulated streamflows for both the current conditions baseline and with-project model runs. These potential inaccuracies were considered when evaluating impacts to various resources, and is discussed further in Section 4.

The monthly WAM model is an appropriate model to evaluate the reliability and yield of Lake Ralph Hall. Several conservative assumptions related to use of water by other water rights holders in the Sulphur River Basin, return flows from such uses, and strict administration of senior water rights at Lake Ralph Patman are all used in the analysis of firm yield and in the project's ability to meet the overall project purpose and need.

Despite its shortcomings for evaluation of some aquatic impacts for Lake Ralph Hall related primarily to the monthly time step, the WAM model can be used to inform other modeling efforts that are better suited for evaluating those impacts. For example, WAM modeling can be used to evaluate issues related to the Texas water rights system. The model includes extensive data-collection and documentation associated with its development for the Sulphur River that could be relied upon for more detailed analysis or to support conclusions from the less sophisticated evaluations.

3.2 Red River Basin RiverWare

The Corps developed a river network model for the Red River Basin using the RiverWare modeling platform. RiverWare is a modeling platform developed at the Center for Advanced Decision Support for Water and Environmental Systems (CADSWES), located at the University of Colorado, Boulder, and funded primarily by the United States Bureau of Reclamation, Tennessee Valley Authority and the Corps. RiverWare models are able to simulate complex river and reservoir networks. One of RiverWare's most useful features is its user-developed policy rules. These rules allow nearly unlimited flexibility to develop and simulate different operating policies and protocols.

The Corps' Red River Basin RiverWare model includes the Sulphur River and North Sulphur River because these rivers are tributary to Lake Wright Patman (a Corps reservoir), and ultimately, tributary to the Red River. The model was developed to evaluate different operations for the Corps, including flood control in the Red River Basin. The model is a daily model that includes Lake Ralph Hall, but does not include any simulated diversions to Upper Trinity from the reservoir and simply spills any water over an uncontrolled spillway when full. While RiverWare is capable of simulating water rights priority, the Corps model did not include this feature in its Red River model, and Lake Ralph Hall does not pass water to downstream senior water rights as currently configured in the RiverWare model.

This model was modified by the Applicant (Brandes 2015) to include the Upper Trinity diversions at Lake Ralph Hall in order to produce a with-project RiverWare model. The Applicant also developed a without-project model that disabled Lake Ralph Hall rather than keeping the uncontrolled spillway used in the Corps version. Using the modified RiverWare models, the Applicant evaluated the effects of the reservoir on the flows at the Cooper and Talco gages, and in-channel pools and puddles that support benthic organisms and fish. Additional information on the Brandes 2015 analysis is presented in Sections 4.1 and 4.2.

3.2.1 VERIFICATION OF RIVERWARE MODELING

DiNatale Water reviewed the original Corps RiverWare model and the modified version used by the Applicant. We found the modifications to include Lake Ralph Hall to be appropriate. The modeled diversions from the reservoir were based on the same logic as in the Applicants' WAM modeling, diverting up to 45,000 AFY at a maximum rate of 205 cfs from Lake Ralph Hall to Upper Trinity, and reducing annual diversions to 16,800 AFY whenever the reservoir storage level fell below 27,500 AF.

The hydrologic inputs above Lake Ralph Hall in the RiverWare models are set to 37% of the Cooper Gage amount during periods when the Cooper gage was operational (beginning October 1949). The drainage area above Lake Ralph Hall is approximately 32.5% of the drainage area above the Cooper gage, based on data provided by the Applicant. It is not clear why 37% was used in the Corps models, but was potentially an approximation made by the original RiverWare modelers who may not have had detailed information on the Lake Ralph Hall site. In contrast, the WAM modeling modified for Lake Ralph Hall sets inflows to 32.5% of the Cooper Gage flow. Based on this difference, the RiverWare models have approximately 9,000 AFY more flow entering Lake Ralph Hall than the WAM models. This difference would have little effect on the Corps' flood control analysis, but could have important implications for the EIS analysis. Section 4.1 discusses this aspect relative to the evaluation of the impacts to benthic organisms and fish. If a quantitative approach is used in the future for stream morphology or water quality and temperature, the RiverWare modeled inflows should be adjusted downward to 32.5% of the Cooper gage flow to more accurately simulate flows downstream of Lake Ralph Hall.

The RiverWare model run without Lake Ralph Hall matches the historical gage flow at the Cooper gage almost perfectly. As in the WAM modeling, this is not a surprising result given the limited water resources development on the North Sulphur River. Calibration at the Talco gage is also good, although simulated flows are lower than observed flows by about 10%. This difference is likely attributable to the RiverWare operations at Lake Jim Chapman. We did not evaluate this further because the evaluation of benthic organisms and fish is focused on the pools that form on the North Sulphur River and refinement of the Lake Jim Chapman operations does not impact flows on the North Sulphur River. The quantitative hydrologic evaluation relative to floodplain resources assessed historical high flow events on the Sulphur River, so discrepancies in the RiverWare results would not impact the results. If a quantitative evaluation of stream morphology or water quality and temperature is used in the future that extends to the Sulphur River below the confluence with the South Sulphur River, the RiverWare operations at Lake Jim Chapman should be refined.

The Corps' RiverWare model does not incorporate the Texas water rights system, although RiverWare has the ability to simulate water rights through its water rights package. The Corps model developers presumably determined that the impact of the water rights administration was not relevant to the flood control and reservoir operations it evaluated with the model. This is a common modeling practice, as model developers will make certain assumptions based on the objectives of the modeling project. It is entirely plausible that the impacts of the Sulphur River water rights are not relevant to the flood control and reservoir operations objectives of the study for which the Corps model was developed, and disregarding the water rights was a reasonable assumption for that purpose. However, for the Lake Ralph Hall EIS, the water rights administration must be evaluated more closely for evaluation of aquatic impacts due to the impacts of water rights administration during periods of low flow.

In comparison, the WAM modeling includes water rights. Upstream junior water rights (e.g. Lake Ralph Hall) must pass water to downstream senior water rights if the downstream rights are not satisfied. The WAM modeling results show some water passed through Lake Ralph Hall to the more senior Wright Patman Lake. However, as discussed in Section 3.1.1 and 3.1.2, this operation may be overstated due to assumptions about return flows and demands in the current conditions WAM model run and historical operation of Lake Wright Patman. Given this understanding of how each model operates, the models provide an upper and lower limit to the flows that would be passed through Lake Ralph Hall to downstream rights: WAM's strict administration of water rights represents the most bypasses to downstream rights, and the absence of water rights in the RiverWare model represents the least amount of bypasses.

3.2.2 CONCLUSIONS ON USE OF RIVERWARE MODELS

The current configuration of the Corps RiverWare model and the version modified by the Applicant are not appropriate to support a detailed quantification of the aquatic impacts from Lake Ralph Hall to the benthic organisms and fish or the floodplain resources. However, the current models can assist in the evaluation of the impacts to benthic organisms and fish by providing an upper limit on the amount of low flows that would be passed through Lake Ralph Hall to downstream senior water rights (see Section 4.1). The RiverWare model uses a daily time step and, if needed, could be used to evaluate aquatic impacts with more precision than the monthly WAM model.

There are several possible methods of using the RiverWare and WAM models in coordination to obtain data needed for the impacts analysis if needed. The most involved process would be to reconfigure the RiverWare model to simulate water rights and would also require calibration of other major operations in the basin (e.g. Lake Jim Chapman and Lake Wright Patman). Documentation and inputs from the WAM modeling can be used to guide these modifications and calibration efforts. Alternatively, the monthly WAM model results can be used to inform the daily RiverWare model by evaluating times when the two model results diverge. For example, the WAM model monthly flows will indicate times when flows should have been passed to downstream senior water rights when the RiverWare model will show water stored at Lake Ralph Hall. A closer investigation of such instances or minor adjustments to the RiverWare model could provide additional modeling data that could be used in specific resource evaluations that may be sufficient for the purposes of the EIS without the time or expense a full reconfiguration and recalibration of the RiverWare model.

3.3 Sulphur River HEC-RAS Model

A HEC-RAS model of the Sulfur River Basin developed by the Corps was provided to DiNatale Water that included unsteady flow simulations of calculated probable maximum floods. The model includes multiple geometries with various proposed reservoirs in the basin, but does not include the proposed Lake Ralph Hall. DiNatale Water reviewed the cross-sections in the HEC-RAS model to confirm the location of the channelized river from other reports and found the general region of channelization to match the cross-sections. To verify the model, DiNatale Water compared gage heights at the Cooper Gage on the North Sulphur to the model's water depth at the Cooper Gage at matching steady state flows. The flows and gage heights predicted in the HEC-RAS model using steady state conditions matched reasonably well with the historical gage data (Figure 4). The HEC-RAS model was used to evaluate the potential impacts to floodplain resources as described in Section 4.2.

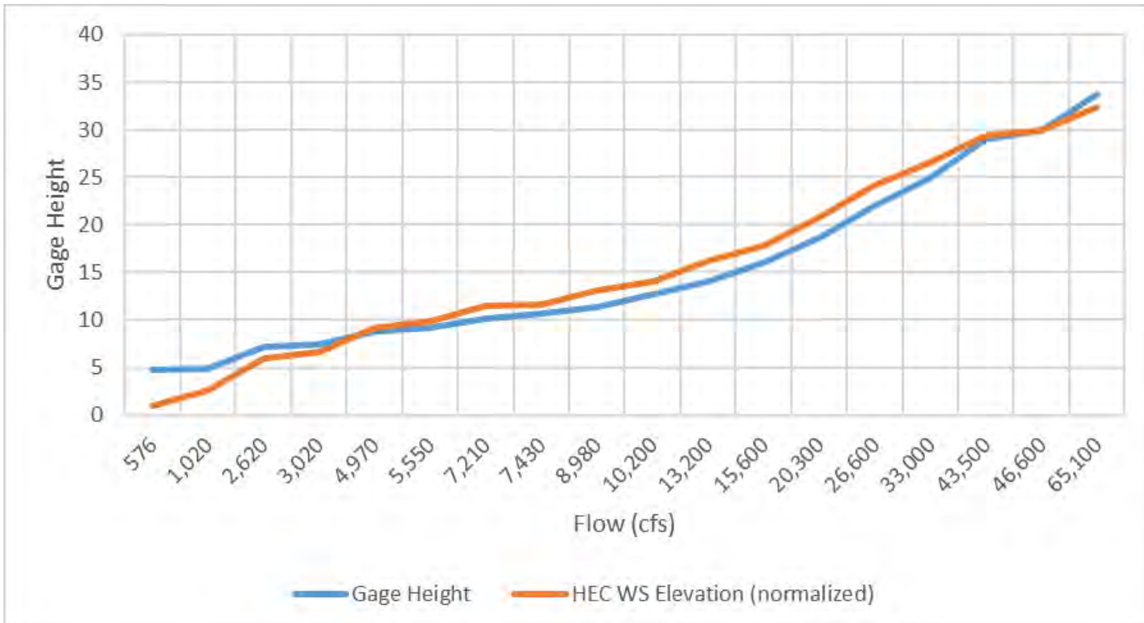


Figure 4. Comparison of USGS gage rating (Gage Height) compared to normalized HEC-RAS modeled water surface elevation at the Cooper gage. Gage height in feet.

4.0 Evaluation of Modeling for Aquatic Resources

Under NEPA and the 404(b)(1) Guidelines, the Corps is responsible for evaluating the impacts to various aquatic resources of a proposed project and its alternatives in order to determine the least environmentally damaging practicable alternative (LEDPA). For the Lake Ralph Hall project, the Corps identified the following aquatic resources for evaluation: benthic organisms and fish, floodplain resources, geomorphology and sediment transport, water quality and groundwater. Section 4.1 and 4.2 discuss the ability of the models to provide quantitative hydrologic data used to support assessment and conclusions of the impacts analysis for benthic organisms and fish, and floodplain resources. Sections 4.3, 4.4 and 4.5 identify modifications necessary if it is later determined that quantitative hydrologic data is required to support more a refined assessment of geomorphology and sediment transport, water quality and temperature, or groundwater resources.

4.1 In-channel pools and puddles that support benthic and fish communities

During the TCEQ water use permit process for Lake Ralph Hall, benthic communities were identified in the basin within in-channel pools and puddles in the North Sulphur riverbed. Cooperating agencies have also identified potential fisheries that may occur within these features. These pools and puddles form after flow events and can sustain the benthic organisms and potentially fish until the next flow event. Testimony from Dr. Norman Jones on behalf of the National Wildlife Federation during the water use permit process indicates that the total volume of such pools in the 20-mile reach between the proposed dam site and the Cooper Gage is 166 AF. Dr. Jones added a 5% channel loss factor to arrive at approximately 175 AF needed to fill the pools below the dam and above the Cooper Gage.

The Applicant analyzed impact of the reservoir on filling of the pools and puddles using the WAM model and the RiverWare model (Brandes 2015). This analysis was performed on a monthly time step with WAM results and the daily RiverWare results were aggregated to monthly flow volumes. The results were summarized with the percent of time the pools would be filled at various locations downstream from the dam. The results between the RiverWare model and the WAM model were very similar except for just downstream of the dam site, where the RiverWare model indicated up to a 13.5% decrease in the amount of time the pools would fill. The Brandes 2015 report attributed this difference to the lack of bypassed flows for downstream senior water rights in the RiverWare modeling.

As described in Section 3.2.1, the RiverWare and WAM results appear to provide the upper and lower ends of the range of flows expected below Lake Ralph Hall. The RiverWare model tends to have less flow because no water is passed for downstream water rights. The WAM modeling tends to have higher flows because of its strict adherence to downstream water rights and other conservative modeling assumptions. When both models are used on a monthly basis as in Brandes (2015), the actual impact based on the monthly flow analysis is between the impact predicted by WAM and by RiverWare. Table 1 is a replica of Attachment E of Brandes (2015) with additional annotation explaining the differences in the ‘Deviation From Without LRH Case’ column comparison between the models.

The analysis of filling the pools and puddles by Brandes (2015) is based on monthly flow volumes. A daily analysis would allow a more detailed analysis that would demonstrate how quickly the pools and puddles dry out due to evaporation, and if all the pools are filled or just a portion of the pools. For example, the monthly analysis shows a flow volume of at least 175 fills all the pools one time in a month. However, if the monthly flow is comprised

Table 1. Replica of Brandes (2015) Attachment E monthly analysis with additional explanation.

STATION NO.	WATER COURSE	LOCATION DESCRIPTION	DRAINAGE AREA sq. mi.	DISTANCE ABOVE N SULPHUR GAGE miles	VOLUME REQUIRED TO FILL ALL D/S POOLS ac-ft	POOL VOLUME IN EACH D/S REACH ac-ft	% OF TIME POOLS ARE FILLED: Without Lake Ralph Hall	With Lake Ralph Hall	Deviation From Without LRH Case
FROM RIVERWARE MODEL (06-26-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	--	--	--	--
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	92.7%	83.6%	-9.1%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	86.7%	73.2%	-13.5%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	85.8%	82.0%	-3.8%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	86.7%	86.3%	-0.4%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	85.8%	85.4%	-0.4%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	85.4%	83.6%	-1.8%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	89.8%	89.6%	-0.1%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	85.1%	82.7%	-2.3%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	--	--	--	--
FROM WAM (04-06-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	--	--	--	--
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	90.8%	90.2%	-0.6%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	84.8%	83.5%	-1.3%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	83.9%	83.8%	-0.1%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	85.4%	85.4%	0.0%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	83.9%	83.9%	0.0%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	83.3%	83.2%	-0.1%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	88.6%	88.6%	0.0%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	83.2%	83.0%	-0.1%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	--	--	--	--

Higher impacts below dam site shown with RiverWare model

Actual monthly impacts within the range between the two models

Lower impacts below dam site shown with WAM model

of two flow events of 87 AF each separated by a few weeks, it is possible that the first event would fill the uppermost pools, and then after evaporation reduces the amount of water in those pools, the second event would refill the upper pools. In that scenario, the lower pools would not be refilled unless enough inflow from contributing basins below Lake Ralph Hall were sufficient.

Use of a daily model will provide more detailed information on the impacts. However, based on the review of the pools analysis performed by Dr. Jones during the water permit process (TCEQ proposed order, undated), our inspection of the channel and other reports describing the condition of the channel, it appears that the monthly analysis will adequately represent stream impacts to the benthic communities in the ponds and puddles below the dam site for the purposes of the EIS. In the event that benthic resource specialists require more detailed data that can be provided by a daily model, modifications to the RiverWare model or a daily disaggregation of monthly WAM output would be appropriate. Such changes may not require a full reconfiguration of the RiverWare model, but could utilize WAM model outflows to guide a daily analysis for times when water should be bypassed at Lake Ralph Hall. In addition, Upper Trinity has proposed restoration of approximately three miles of original North Sulphur River stream channel below the dam site. The design is not yet complete on this mitigation system and could utilize a recirculating pump, but under some scenarios, water may be released out of the mitigation reach into the channelized portion of the existing river bed. These types of flows from the mitigation reach may assist in maintaining or filling of pools and puddles during dry times. More detailed analysis involving operational considerations may be needed to allow a determination of the value of such inputs associated with this proposed mitigation strategy. However, such conditions are not part of the impact assessment since USACE cannot use compensatory mitigation features, and their influence and benefits, in its impact analysis to determine the LEDPA.

Table 1 shows that the there is almost no difference between the with and without Lake Ralph Hall model runs for both the RiverWare and WAM models by the Cooper gage. It follows that downstream of the Cooper gage, there would be no impact of Lake Ralph Hall on filling puddles and pools below the gage because the increased drainage area below the Cooper gage is sufficient to fill the pools during rain events even if no flow passes the dam site.

4.2 Floodplain Resources

Downstream of the confluence with of the North and South Sulphur Rivers, the Sulphur River is not channelized as on the North Sulphur River, although some channelized portions are visible from aerial imagery. Riparian habitat is more established downstream of the confluence and the river meanders along its course rather than flowing through straight reaches of the channelized portions. High flows often provide benefits to the riparian habitat in the floodplain when flows go out of the banks of the river and infiltrate into the surrounding areas. Brandes (2015) notes that the flood stage flows in the North Sulphur River rarely if ever exceed the deep incised channel. However, no analysis was presented for more downstream locations where the channelized nature of the river changes to a more typical riverine system. We are not aware of other studies of floodplain impacts for the Lake Ralph Hall EIS that may include this more downstream reach, so we present a brief evaluation of floodplain impacts in this section.

The Corps' HEC-RAS model (Section 3.3) was utilized to assess the impacts to floodplain resources at more downstream locations. As received, the model included transient simulations for various flood control scenarios that were typically very high flow events only, meant for flood control and facility sizing events, and for a number of proposed reservoir sites. We took a simplified approach of using the HEC-RAS model to evaluate the river stage at several locations using historical gaged flows and evaluating river stage at steady state at that flow rate. We used the basin geometry (cross-sections) containing only existing reservoirs for the analysis.

This analysis requires daily flows, as monthly flow volumes averaged over a month will not represent the level of peak flow seen in the basin. In a more detailed analysis of the floodplain impacts, RiverWare model outputs could be used as inputs to the HEC-RAS model. However, since the RiverWare model is not currently configured to simulate Lake Ralph Hall operations, we took a simplified approach of using historical gage data for a baseline set, and we applied an adjustment to the gage flow to represent the maximum potential impact of Lake Ralph Hall (i.e. assume Lake Ralph Hall diverts and stores the entire inflow to the lake). This approach is a conservative approach because it assumes the maximum impact at Lake Ralph Hall. Therefore, the impacts computed under this approach will yield higher impacts than a more detailed approach using simulated outflows from Lake Ralph Hall.

Historical gaged flows from the Cooper Gage, Talco Gage, and Dalby Springs Gage were used in this analysis. Several flow events with varying levels of flow were selected and tracked through all three gages upstream to downstream. The historical flow rates at the three gages were simulated in the HEC-RAS model to determine river stage. The flow was then adjusted to assume Lake Ralph Hall stored the entire inflow to the lake during the flow event, and the adjusted flow was simulated in HEC-RAS to determine the river stage decline due to Lake Ralph Hall. Gaged flows at each location were adjusted by computing the total volume of the flow event and proportionally removing 33% of the volume of water observed at the Cooper Gage (33% was rounded from the 32.5% drainage area ratio of Lake Ralph Hall to the Cooper Gage). This approach maintained the flow routing and attenuation patterns from the upper reaches to the lower gages. The HEC-RAS model output provides a river stage, but also plots the inundated area of the cross section being evaluated. These visual depictions show the floodplain impacts laterally from the river, providing more information than the river stage only.

Four separate rainfall events were selected to evaluate Lake Ralph Hall's impacts to floodplain resources. The events were chosen based on frequency of the flow event, with the lowest flow expected to occur several times per year, the next highest flow expected to occur about once a year, the next highest expected once every few years, and the highest flow event expected to occur about once every 20 years. Table 2 shows the events, the gaged peak daily flow, the total flow volume of the event and the adjustments made for the without Lake Ralph Hall scenario.

Hydrographs for the January 8, 2012 event are shown in Figure 5**Error! Reference source not found.**. Cross-sections for the Talco and Dalby Springs gages for the without and with Lake Ralph Hall cases are shown in Figure 6 and Figure 7. Cross-Section and water surface of January 8, 2012 rain event at the Dalby Springs Gage for with and without Lake Ralph Hall scenarios., respectively. Figure 8, Figure 9, and Figure 10 are the corollary figures for the December 23, 2009 rain event. Figure 11, Figure 12, and Figure 13 are the corollary figures for the March 9, 2012 rain event. Figure 14, Figure 15, and Figure 16 are the corollary figures for the November 27, 2015 rain event.

The figures show how quickly the impact of Lake Ralph Hall is attenuated at downstream locations. Due to the large difference in contributing drainage area downstream of Cooper gage, a 33% decrease in flow at the Cooper gage due to Lake Ralph Hall has little effect on maximum river stage downstream following a rain event. During larger storms, while the magnitude of the flow decrease is significantly larger due to water going into storage at Lake Ralph Hall, there is still little effect on maximum river stage below the confluence of the North and South Sulphur Rivers. In all four rain events evaluated, the river stage changes at the Talco Gage are small, and become even smaller at downstream locations. The cross-section figures contain both the without Lake Ralph Hall baseline river stage and the with Lake Ralph Hall river stage. The decreases in river stage with Lake Ralph Hall are nearly imperceptible on the cross sections, and the lateral extent of the flow is nearly identical to the without Lake Ralph Hall scenarios. Table 3 shows the changes in river stage at the peak daily flow rates.

Based on the results of the HEC-RAS analysis and the estimates for streamflow reduction due to Lake Ralph Hall, there are no significant floodplain impacts due to the proposed Lake Ralph Hall because the river stage and lateral extent of flows changes very little downstream of the channelized section of the river. As with other resources, additional detail could be obtained by using daily modeling results rather than using the approach applied in this analysis. However, this analysis used conservative assumptions about the amount of water in storage at Lake Ralph Hall at the time of the flow event, so the additional precision gained through daily modeling will not yield differing hydrology results.

Table 2. Rain events used to evaluate flood plain resource impacts of Lake Ralph Hall.

		Without Lake Ralph Hall Observed Flow						With Lake Ralph Hall Adjusted Flow					
		Gaged Peak Daily Flow (cfs)			Gaged Total Event Flow Volume (AF)			Adjusted Peak Daily Flow (cfs)			Adjusted Total Event Flow Volume (AF)		
Beginning Date	**Frequency**	**Cooper**	**Talco**	**Dalby Springs**	**Cooper**	**Talco**	**Dalby Springs**	**Cooper**	**Talco**	**Dalby Springs**	**Cooper**	**Talco**	**Dalby Springs**
January 8, 2012	Several times per year	1,840	3,030	2,910	5,109	17,302	26,452	1,565	2,930	2,835	3,406	15,599	24,748
December 23, 2009	a few times per year	5,470	13,300	12,900	10,850	72,774	109,864	3,645	12,850	12,575	7,233	69,157	106,248
March 19, 2012	once every few years	26,900	34,600	29,300	56,450	186,684	242,162	18,975	33,200	28,435	37,633	167,868	223,345
November 27, 2015	once every 20 years	41,400	52,800	52,100	140,945	294,803	585,183	33,000	50,635	50,685	93,964	247,821	538,202

**Without Lake Ralph Hall adjusted flow computed by removing 33% of Cooper Gage flow volume from downstream gages, proportional to each day's flow volume during the flow event*

Table 3. Water surface elevation, in feet, with and without Lake Ralph Hall at key gages.

Beginning Date	**Frequency**	**Cooper without Lake Ralph Hall**	**Cooper without Lake Ralph Hall**	**Difference**	**Talco without Lake Ralph Hall**	**Talco without Lake Ralph Hall**	**Difference**	**Dalby Springs without Lake Ralph Hall**	**Dalby Springs without Lake Ralph Hall**	**Difference**
January 8, 2012	Several times per year	376.84	376.22	0.62	294.16	293.98	0.18	244.50	244.30	0.20
December 23, 2009	a few times per year	381.97	379.78	2.19	301.14	301.02	0.12	253.98	253.88	0.10
March 19, 2012	once every few years	396.56	392.26	4.30	303.76	303.64	0.12	257.00	256.89	0.11
November 27, 2015	once every 20 years	401.18	398.78	2.40	305.20	305.04	0.16	259.45	259.33	0.12

**Lake Ralph hall river stage computed using HEC-RAS model using maximum flow shown in Table 2*

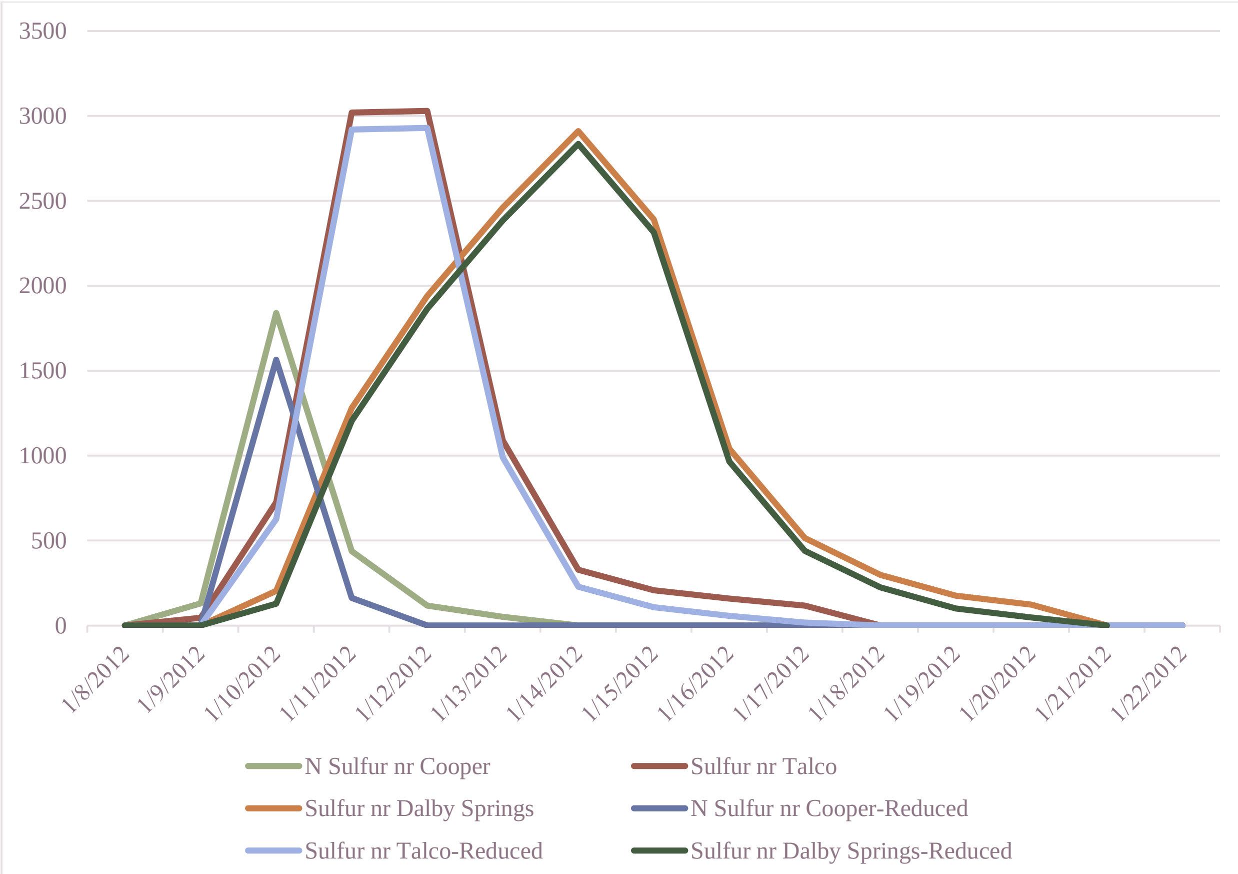


Figure 5. Hydrographs from gages for rain event beginning January 8, 2012 for with and without Lake Ralph Hall scenarios.

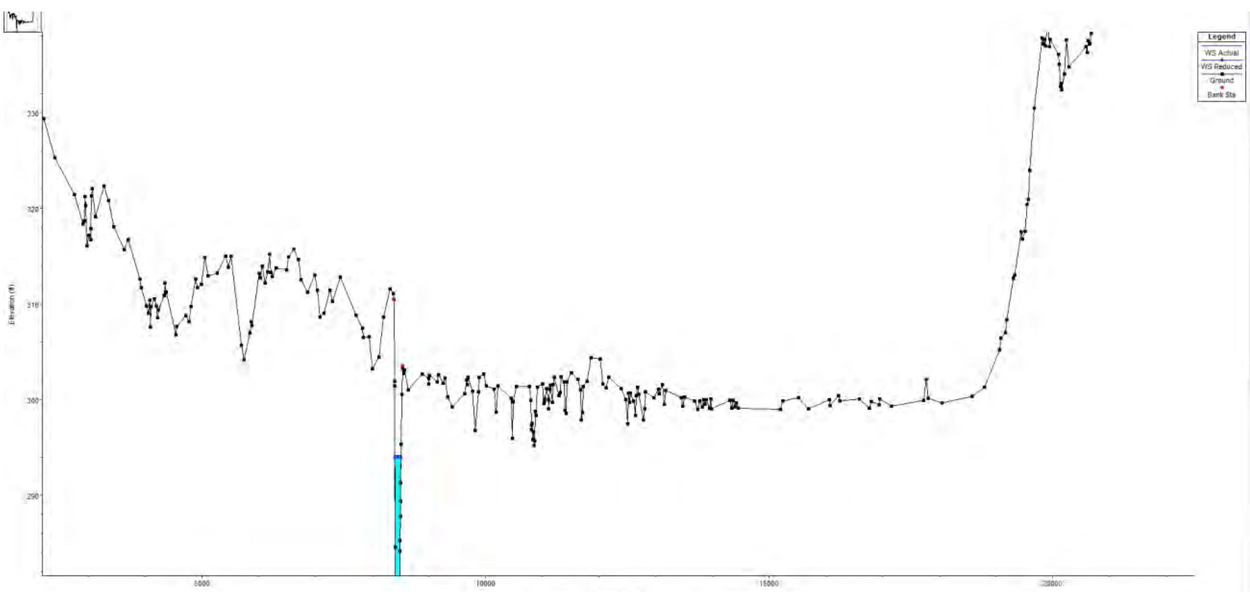


Figure 6. Cross-Section and water surface of January 8, 2012 rain event at the Talco Gage for with and without Lake Ralph Hall scenarios.

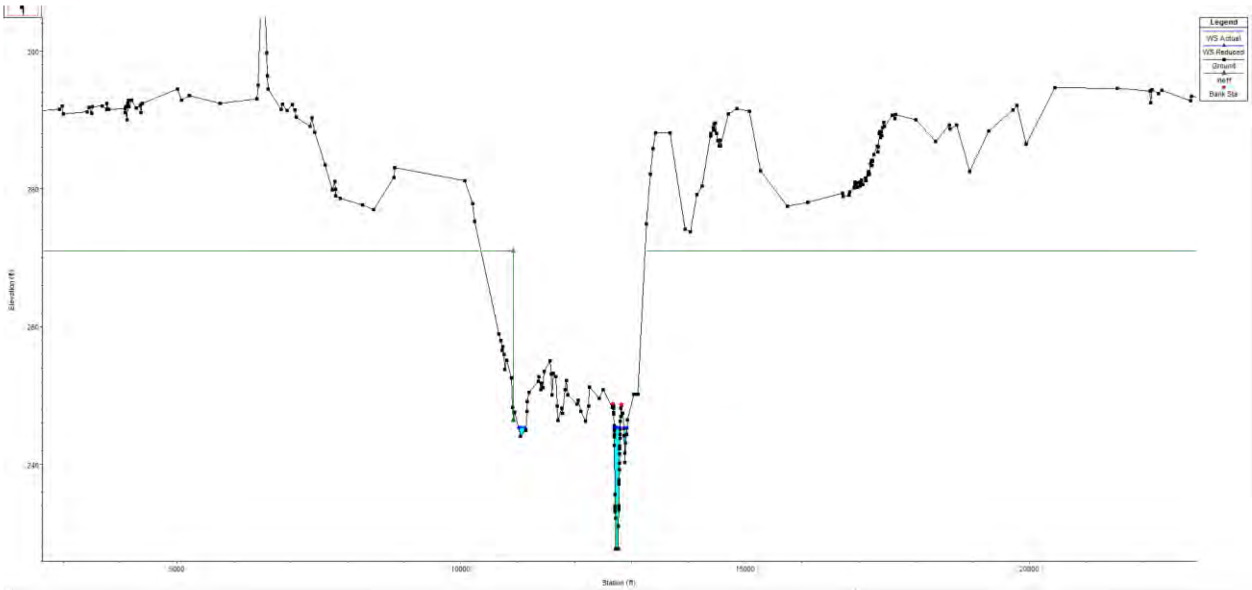


Figure 7. Cross-Section and water surface of January 8, 2012 rain event at the Dalby Springs Gage for with and without Lake Ralph Hall scenarios.

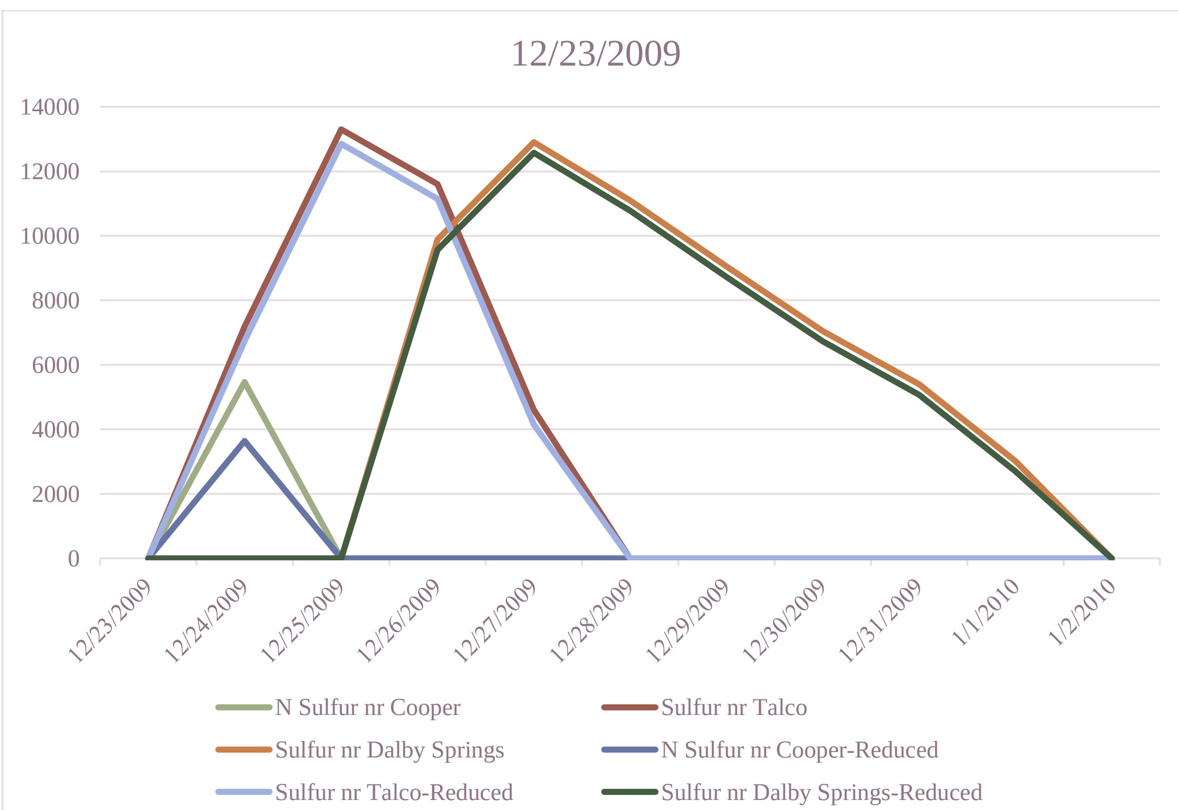


Figure 8. Hydrographs from gages for rain event beginning December 23, 2009 for with and without Lake Ralph Hall scenarios.

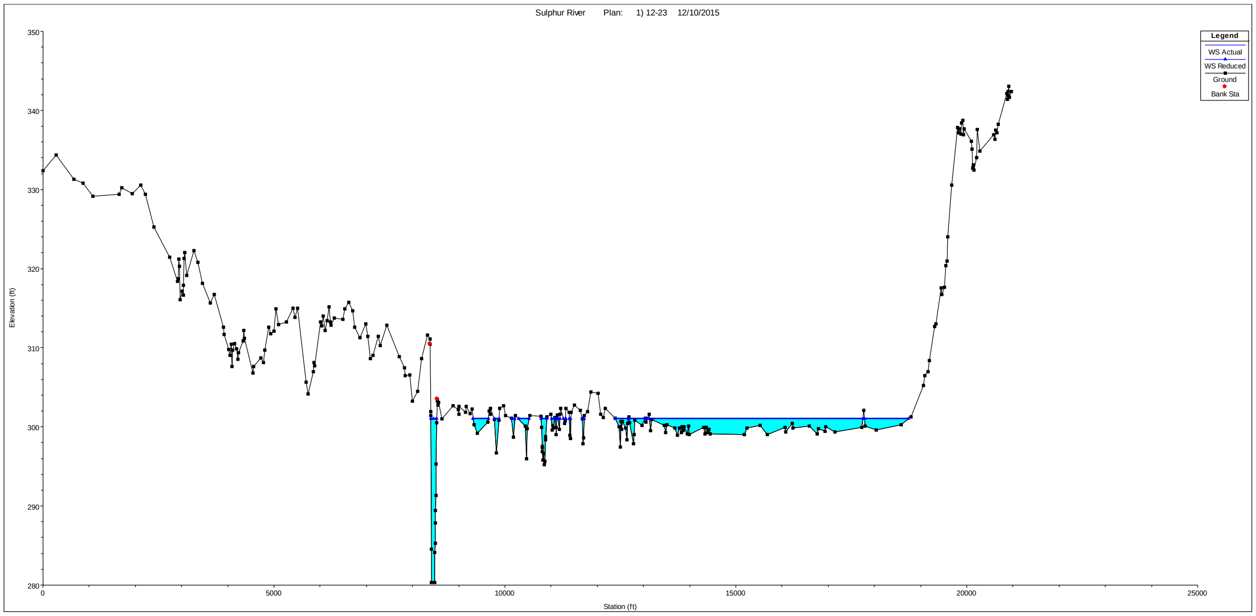


Figure 9. Cross-Section and water surface of December 23, 2009 rain event at the Talco Gage for with and without Lake Ralph Hall scenarios.

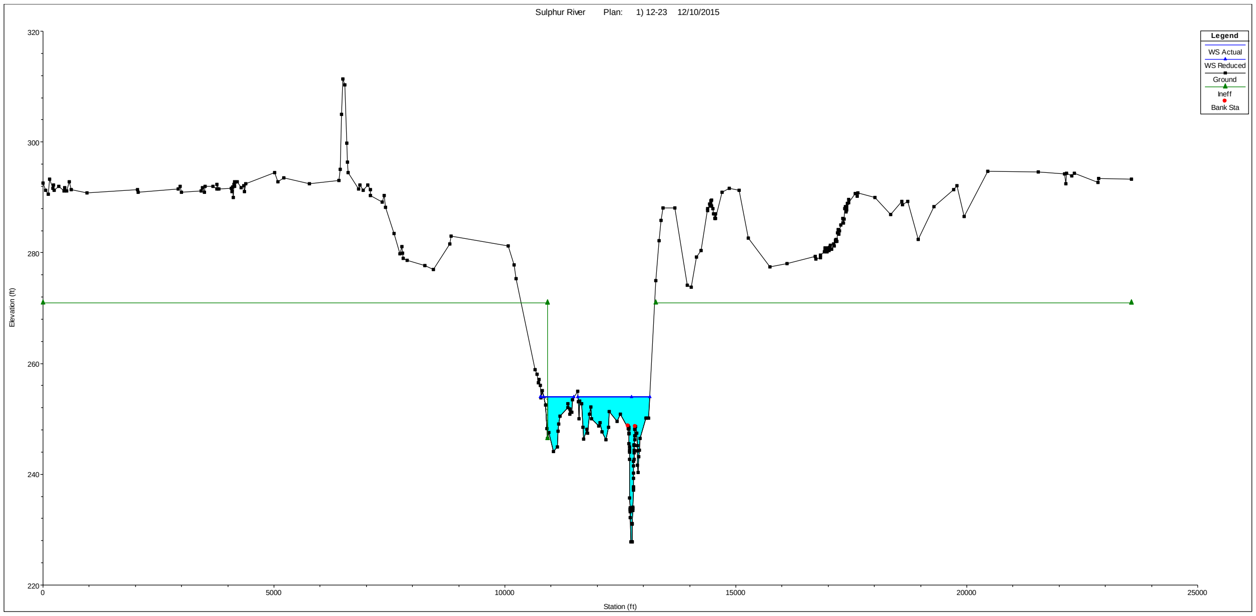


Figure 10. Cross-Section and water surface of December 23, 2009 rain event at the Dalby Springs Gage for with and without Lake Ralph Hall scenarios.

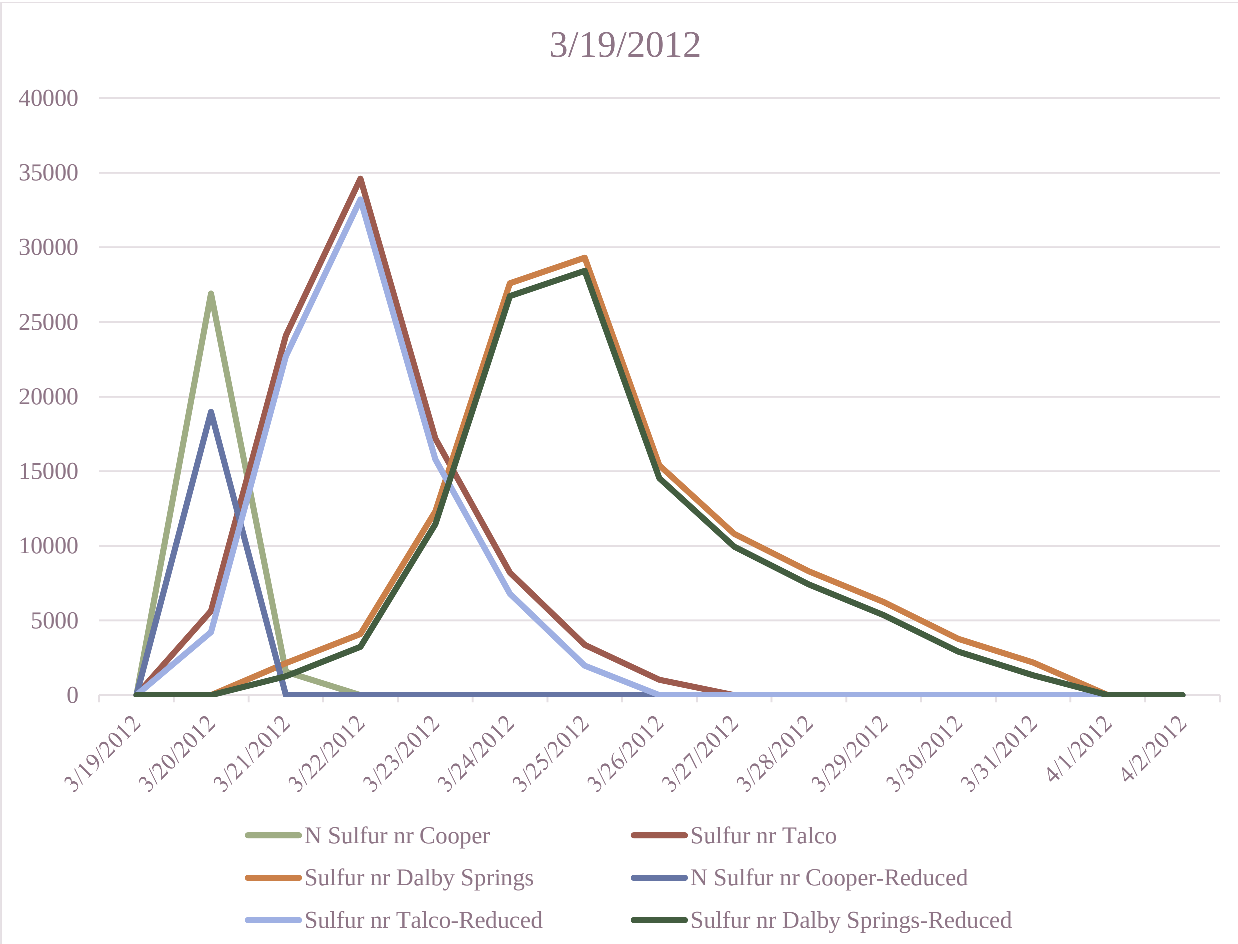


Figure 11. Hydrographs from gages for rain event beginning March 19, 2012 for with and without Lake Ralph Hall scenarios.

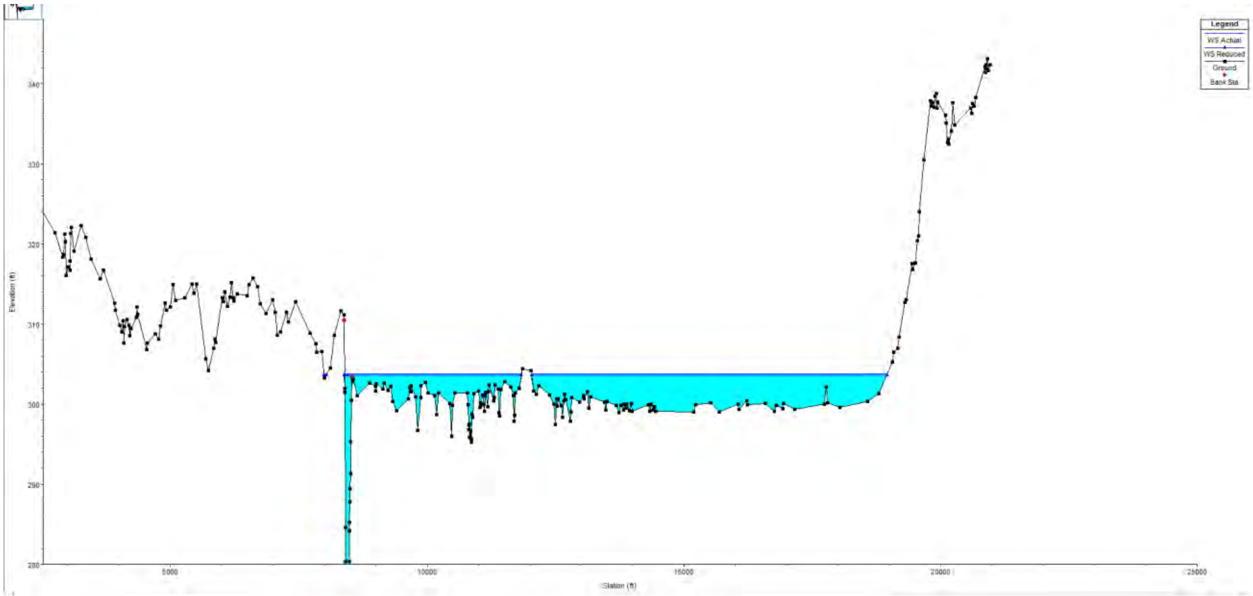


Figure 12. Cross-Section and water surface of March 19, 2012 rain event at the Talco Gage for with and without Lake Ralph Hall scenarios.

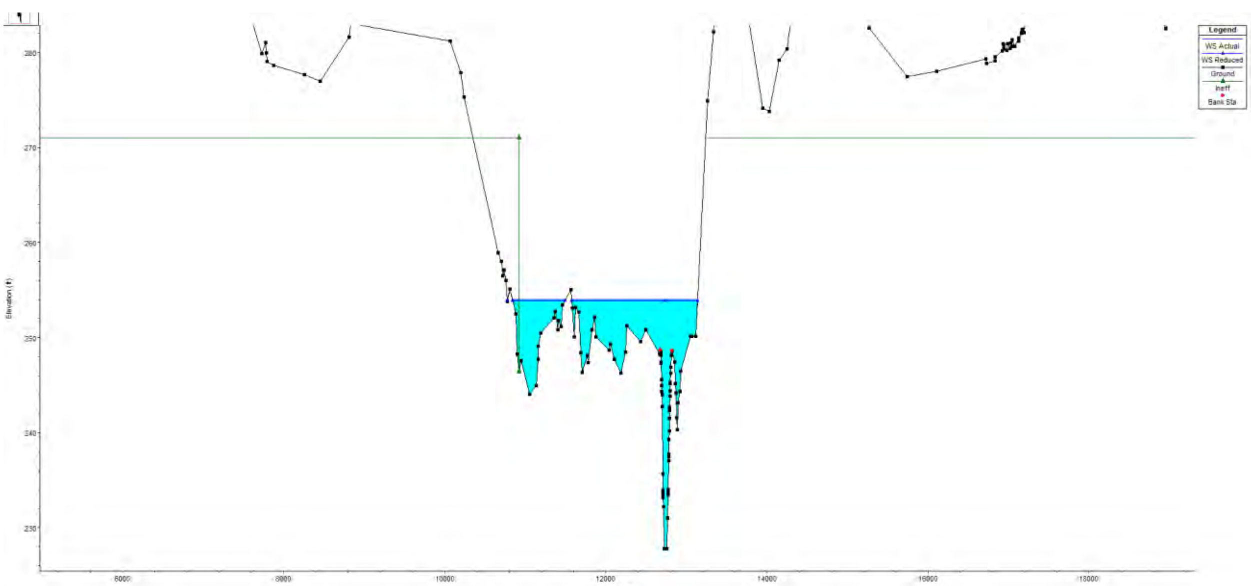


Figure 13. Cross-Section and water surface of March 19, 2012 rain event at the Dalby Springs Gage for with and without Lake Ralph Hall scenarios.

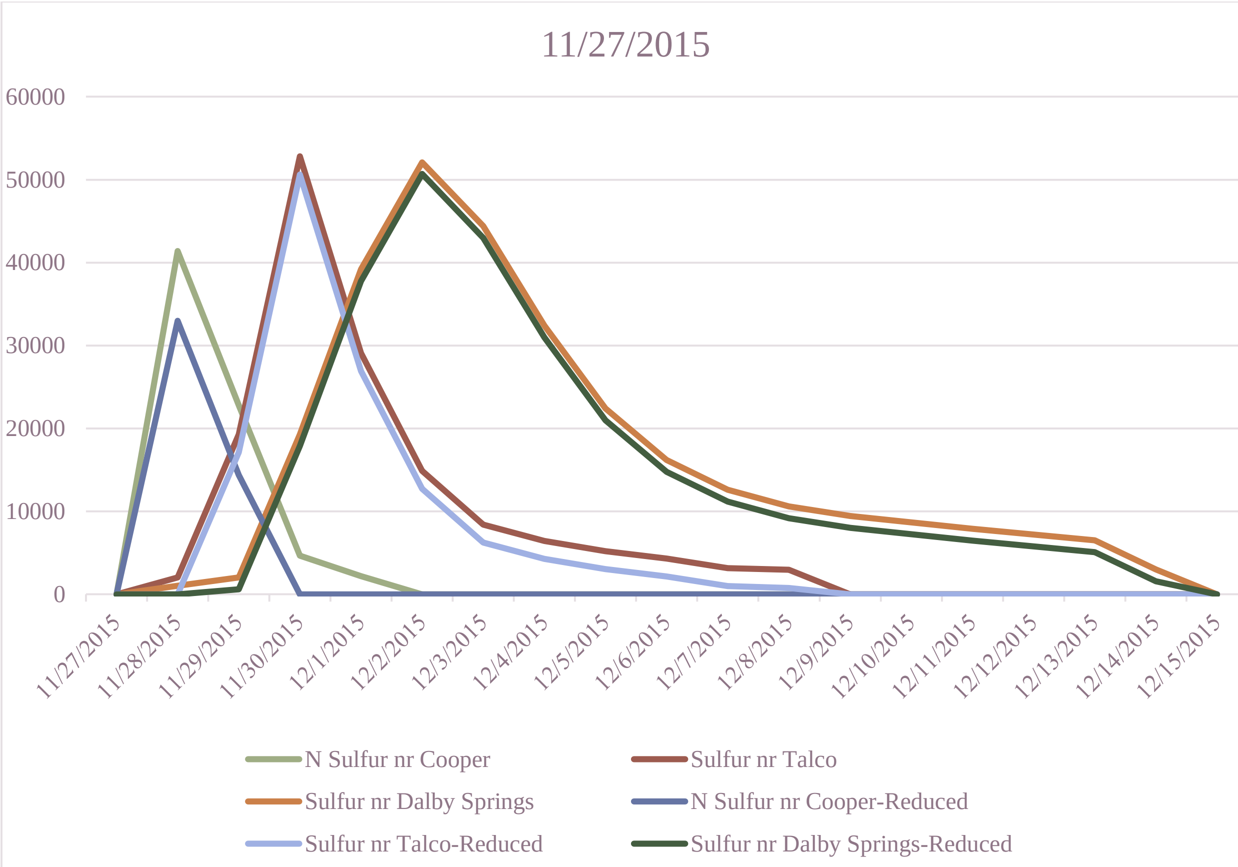


Figure 14. Hydrographs from gages for rain event beginning November 27, 2015 for with and without Lake Ralph Hall scenarios.

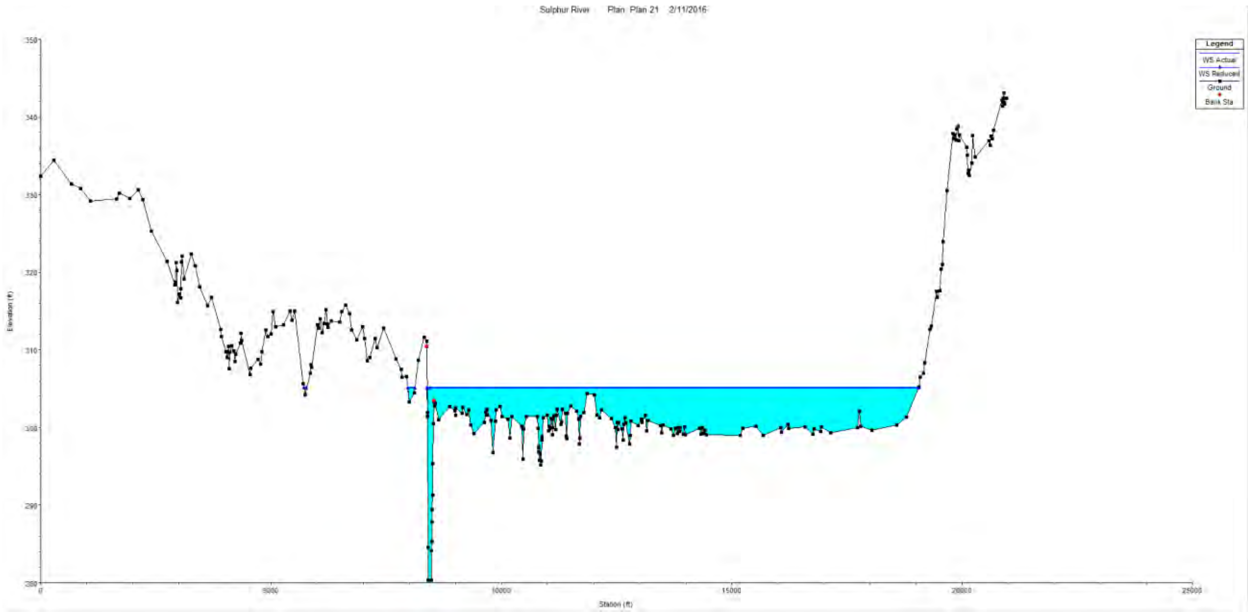


Figure 15. Cross-Section and water surface of November 27, 2015 rain event at the Talco Gage for with and without Lake Ralph Hall scenarios.

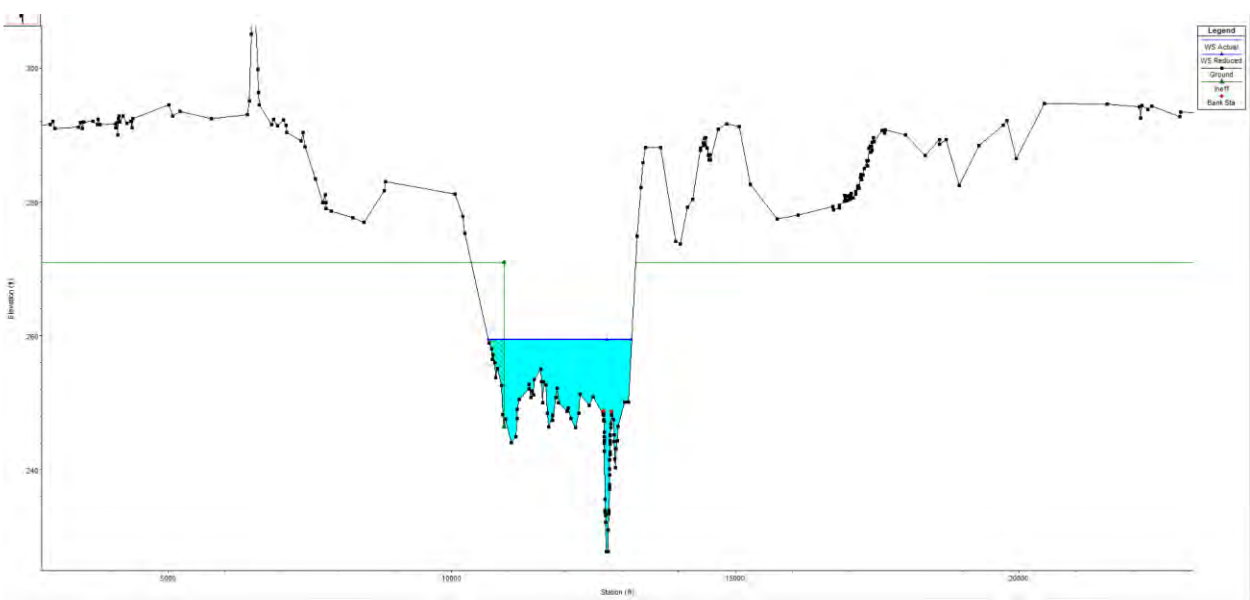


Figure 16. Cross-Section and water surface of November 27, 2015 rain event at the Dalby Springs Gage for with and without Lake Ralph Hall scenarios.

4.3 Geomorphology and Sediment Transport

The proposed approach to assess the potential impact of the Lake Ralph Hall project on stream morphology is to use the Texas Rapid Assessment Method (TXRAM) to assess the current morphological condition of the stream. TXRAM does not include an intensive, quantitative functional assessment nor does it focus on specific ecological functions. TXRAM was chosen because it is an accepted method for assessment of stream integrity and health in Texas and can be used to assess stream impacts, including the comparison of stream alternatives (Corps 2010). TXRAM requires an analysis dependent on field observations, photos, and aerial assessments and the assessment would be based on existing field data. Two components within the TXRAM analysis consider flow, specifically flow regime or a ranking of the stream flow conditions, and channel flow status, which accounts for the movement of water through a reach. The primary method used to obtain information to complete the TXRAM analysis for all of the required components is through field assessments. The method is a general conditional assessment which allows for the inference of resource function and condition but does not require quantitative hydrologic data. The Corps has stated that the degraded condition of the channel as well as the unique properties associated with the channel's substrate do not warrant a more intensive review and detailed hydrologic analysis.

The TXRAM approach being pursued by the Corps does not require detailed quantitative information for a with-project scenario. Current modeling using the WAM model results or the RiverWare results are not adequate to support an intensive quantitative functional assessment of the geomorphology and sediment transport impacts assessment. The WAM modeling uses a monthly time step which does not represent peak flows and rapid recession of flows common to this basin. These types of peak flows are important to geomorphology and sediment transport evaluation. The RiverWare modeling provides daily flow values, but the model inputs would need to be adjusted as recommended in Section 3.2.2 and water rights should be added to the model if detailed quantitative hydrologic data is desired for a more detailed approach to geomorphology and sediment transport.

However, both the WAM and RiverWare models may be able to support qualitative conclusions in the TXRAM approach. For example, the WAM model as configured by the Applicant has multiple sub-basins below Lake Ralph Hall and may contain useful information on soil composition and drainage areas that contribute to the North Sulphur reach at several locations below the dam site. Basic mass balance analysis of the inflows into Lake Ralph Hall, diversions and evaporation rates from the WAM and RiverWare models may also help provide a qualitative picture of with-project streamflows.

We also reviewed existing reports prepared for Upper Trinity on hydraulics and hydrology and fluvial geomorphology (Upper Trinity 2004, Upper Trinity 2006). Both reports included analyses of the area upstream of the Lake Ralph Hall dam site and downstream on the North Sulphur River for 100 feet below the dam, which does not cover the extent of the potential impacts on the North Sulphur River and Sulphur River downstream of the dam site.

4.4 Water quality and Temperature

The water quality and temperature assessment is using a qualitative approach and will include analysis of water quality stored in Lake Ralph Hall as well as impacts to stream reaches below the dam. By nature, a qualitative approach does not require an intensive quantitative model, therefore there is no need to perform additional hydrologic modeling to support the qualitative water quality assessment. This seems to be an appropriate approach based on the degraded condition of the existing river channel downstream of the dam site. Existing monthly model results may provide sufficient information to support the in-lake water quality and temperature analysis.

If the Corps subsequently determines that a more detailed quantitative analysis of water quality impacts is required, the most rigorous method would be reconfiguration and calibration of the RiverWare model as described in 3.2.2. However, as described for the other resources, some daily data can be more readily determined from existing modeling in order to reduce the time and expense of a full RiverWare model update. Daily information for Lake Ralph Hall inflows can be estimated as 32.5% of the Cooper gage flows. Diversions from the lake to Upper Trinity can be estimated as a constant daily rate as simulated in the WAM modeling (or based on some other pattern based on Upper Trinity's demand for water). Daily outflows would require somewhat more analysis that would be determined based on the specific needs for a quantitative water quality analysis.

4.5 Groundwater

Groundwater aquifers in the region are much deeper than the river channel. The river channel at the Lake Ralph Hall site is comprised primarily of clay that impedes vertical flow to lower aquifers. The lack of connection even to local shallow aquifers is apparent by the lack of stream baseflow in the North Sulphur River during the periods of low precipitation. Therefore, there is limited potential for hydrologic interrelationships between the river and the groundwater system.

Downstream locations closer to Lake Wright Patman may have increased groundwater interaction where the river overlies the Carrizo-Wilcox aquifer, downstream of the channelized portion of the North Sulphur River. However, at these downstream locations, the differences in flow due to Lake Ralph Hall are minimal in terms changes in river stage (see Section 4.3) and therefore changes to the surface water-groundwater interaction would be small or negligible.

If the Corps determines that more detailed quantitative evaluation of the groundwater impacts are necessary, the monthly WAM model results would be suitable for simulating the surface water component of the evaluation. Groundwater time-scales are typically much longer than surface water systems, so a monthly time step would be appropriate. Due to the minimal interaction between the surface water and groundwater system near the dam site, the WAM model's conservative assumptions that tend to overstate the amount of water bypassed to downstream water rights would provide a scenario where the upper limit of impacts to groundwater resources could be determined.

5.0 Cumulative Impacts

As part of its responsibilities to disclose impacts of a proposed projects to the public, the Corps must consider the cumulative impacts of other known and reasonably foreseeable future actions that may also impact the project area. In project areas where land and water uses are rapidly changing, or where other projects are proposed, the Corps may require a future conditions baseline scenario for evaluating impacts. A future condition baseline helps determine which impacts to a project area are attributable to the proposed project, and which are attributable to the reasonably foreseeable future actions.

For the Sulphur River Basin, the Corps determined that a future conditions baseline modeling scenario is not required at this time. This decision was based on minimal expected changes to the Sulphur River basin in terms of development and land and water use which would modify hydrology in the foreseeable future that are in addition to the proposed Lake Ralph Hall project. Several other reservoir sites have been proposed in the Sulphur River Basin, including Marvin Nichols. An organization called the Sulphur Basin Group, which is a consortium of parties interested in water development in the Sulphur River basin, evaluated the Marvin Nichols and other dam sites for yield and reliability using the WAM model (SBG 2015). Recently the Marvin Nichols reservoir was removed from state planning documents and would not be constructed prior to 2070. Due to more than 50 years between the current evaluation for Lake Ralph Hall and potential future construction, Marvin Nichols was not considered a reasonably foreseeable future action. Construction of Marvin Nichols would require a Corps permit, and the impacts of Lake Ralph Hall (if permitted and constructed), would be considered in the impact analysis of Marvin Nichols.

Regional water providers are also evaluating re-allocation of storage in lake Wright Patman. This proposed project would not affect the Lake Ralph Hall evaluation because changes to storage levels in Lake Wright Patman due to the reallocation will impact the inundation area and flows downstream of Lake Wright Patman. In Section 4.2, we demonstrated that changes to the flow due to Lake Ralph Hall do not have a significant impact on river stage or floodplain resources at downstream locations, such as Lake Wright Patman. Therefore, the outcome of the Wright Patman reallocation are irrelevant to the impacts analysis of Lake Ralph Hall.

In summary, the projected consistent land and water use in the Sulphur River Basin and lack of other reasonably foreseeable future actions in the region support the use of an impacts analysis that relies on a current conditions baseline and comparing to a with-project future scenario.

6.0 Conclusions

The purpose of this report was to determine the adequacy of existing hydrologic modeling to support the evaluation of the impacts to the aquatic resources caused by the proposed Lake Ralph Hall in the Sulphur River Basin. Existing modeling tools include Texas' WAM models, as modified by Upper Trinity to simulate with and without Lake Ralph Hall conditions, the Corps' RiverWare model of the Red River basin that includes the Sulphur and North Sulphur Rivers as tributaries to the Red River, and the Corps' HEC-RAS model of the project area. The Corps took a more robust approach to evaluate impacts to benthic organisms and fish, and this report included a quantitative analysis of hydrologic impacts to floodplain resources. The Corps has taken a more qualitative approach to evaluating other resources including geomorphology and sediment transport, water quality and temperature and groundwater impacts. This report identified potential supporting uses of the models for these approaches and recommended approaches to modifying the existing modeling in the event additional detailed analysis is used in the future.

6.1 Conclusions Related to Available Models (Section 3)

The WAM models of the Sulphur River utilize a monthly time step that is appropriate for water rights administration purposes and yield and reliability analyses, but is not appropriate for evaluating impacts that require daily resolution of flow. In the North Sulphur River, this point is important due to the flashy nature of the river system, where flows can fluctuate between no flow and several thousand cfs within a few days. The WAM model current conditions run uses some conservative assumptions on demands and return flows that may not accurately represent streamflow during average years.

The current configuration of the RiverWare model is not appropriate for supporting determinations of aquatic impacts of Lake Ralph Hall because of the lack of detailed operations at Lake Ralph Hall, including bypasses to downstream junior water rights. The

RiverWare model uses a daily time step and, if needed, could be used to evaluate aquatic impacts with more precision than the monthly WAM model with appropriate modifications.

Although neither the WAM nor the RiverWare model were configured for the purposes of the EIS evaluations, there are several possible methods of using the models as-is or with minor modifications to better support resource analyses. The WAM and RiverWare models simulate opposite tendencies with respect to the amount of water passed downstream at Lake Ralph Hall. The WAM model passes flows downstream to meet the demands of senior water rights. This is an important feature to include in resource analysis, however, the conservative demand and reuse assumptions in WAM, and the manner in which Lake Wright Patman is simulated may overstate these bypassed flows. The RiverWare model, on the other hand, never passes water downstream to senior rights. Therefore, the streamflows and resource impacts can therefore be predicted to occur within the range of hydrologic impacts predicted by the two models in situations where monthly flows provide a sufficient level of detail.

For the evaluation of the impacts to benthic organisms and fish, the current configuration of the RiverWare model in combination with the monthly WAM results provides a sufficient level of detail that does not require model changes at this time. If additional detailed assessments, refinements or modifications are made in the future for any of the resources, the RiverWare model's naturalized flows at Lake Ralph Hall should be reduced to 32.5% of Cooper gage. If additional detail for impacts analysis at the Talco Gage or downstream are needed, additional calibration of Lake Jim Chapman should be part of the that RiverWare model refinement.

6.2 Conclusions Related to Use of Output for Evaluation of Resource Impacts (Section 4)

The Applicant evaluated the use of WAM and RiverWare to inform impact assessments to the benthic and fish communities in the pools and puddles in the North Sulphur River channel. Daily flows from the RiverWare model were summed to monthly values by the Applicant to compare the results from the two models. Use of the daily data could provide more detailed information relative to resource impacts than monthly flows. However, based on a comparison of the bias inherent in the WAM monthly model and the RiverWare daily model, the hydrologic impacts to the benthic organisms and fish are within the range of impacts simulated by the two models. Hydrologic impacts are shown on Table 1 for both models. Based on these results, the results of the existing WAM model and RiverWare model are adequate to represent hydrologic impacts to the benthic communities and fish in the North Sulphur River for the purposes of the EIS and a more detailed daily model is not required.

Floodplain resources downstream of the Lake Ralph Hall dam site had not been previously quantified. We utilized the Corps HEC-RAS model of the Sulphur River basin to analyze streamflow and lateral extent of flows at the Talco and Dalby Springs gages several miles downstream of the proposed Lake Ralph Hall where the Sulphur River is no longer channelized. The analysis was done for a variety of flow events, ranging from a frequency of a few times per year to a one-in-twenty year event. Flows were estimated for a scenario with Lake Ralph Hall and compared to the historical river stage and lateral extent of flow. The results showed very small differences between the scenarios with and without the Lake Ralph Hall project due to the increasing contributing drainage area and flow to the river further downstream of the site. The analysis showed the impacts to floodplain resources due to Lake Ralph Hall are negligible downstream of the channelized portion of the river.

Geomorphology and sediment transport are being generally assessed using the TXRAM method in light of additional data that will not require detailed quantitative hydrologic data. If more detailed quantitative analysis of this resource category is required in the future, the current modeling configuration of WAM would not be suitable due to the monthly time step. If refined as described above, the RiverWare model could be used to inform quantitative assessments for this resource category. Both the WAM and RiverWare in their current configurations could be used to support the qualitative conclusions by utilizing model inputs, documentation and mass balance to support the qualitative findings from TXRAM.

Water quality and temperature are being evaluated by the Corps using a qualitative approach in coordination with the Texas Commission on Environmental Quality 401 certification agency. Modeling results from the WAM model and the RiverWare model can provide bounds on a range of likely flow conditions that could impact water quality and can support the qualitative conclusions of the analysis, if needed. It is likely that any additional quantitative detail for water quality would require daily resolution of flows, making modifications to the RiverWare model the preferred approach if the Corps determines a more detailed quantitative approach is warranted at some point in the future.

Due to the limited connection between surface water and deeper groundwater aquifers, hydrologic data is not required for assessment to groundwater resources near the project site. The lower reaches of the Sulphur River overlay the Carrizo-Wilcox aquifer. Due to the small changes in river stage at downstream locations, the effects on the Carrizo-Wilcox aquifer are likely very small or negligible. If a more detailed quantitative analysis were used in the future, these effects could be quantified using a groundwater model and monthly WAM output due to the longer time scales associated with groundwater flow.

7.0 References

Robert J. Brandes Company, 1999. Final Report: Water Availability Modeling for the Sulphur River Basin. Prepared for the Texas Natural Resource Conservation Commission. June 1999.

Robert J. Brandes Consulting, 2015. Memorandum to Ed Motley, CH2M-Hill. Subject: Lake Ralph Hall RiverWare Modeling. June 29, 2015.

Sulphur Basin Group (SBG), 2015. Memorandum to SRBA. Subject: Sulphur River Basin Feasibility Study SBG15119. June 4, 2015.

Upper Trinity Regional Water District, 2006. Prepared by Mussetter Engineering, Inc. Geomorphic and Sedimentation Evaluation of North Sulphur River and Tributaries for the Lake Ralph Hall Project. October 23, 2006.

Upper Trinity Regional Water District, 2004. Prepared by R.J. Brandes. Hydrologic and Hydraulic Studies of Lake Ralph Hall Report. April 27, 2004.

U.S. Army Corps of Engineers, 2010. The Texas Rapid Assessment Method (TXRAM) Wetland and Streams Modules – Final Draft for Public Review. U.S Army Corps of Engineers – Regulatory Branch. October 2010.

U.S. Army Corps of Engineers, 2016. Evaluation and Documentation of the State of Texas Water Availability Model and Water Rights Analysis Package. Final Summary Report. Prepared by CDM Smith. January 2016.

D-2: Hydrologic and Hydraulic Studies for Lake Ralph Hall

ATTACHMENT 2
HYDROLOGIC AND HYDRAULIC STUDIES OF LAKE RALPH HALL REPORT

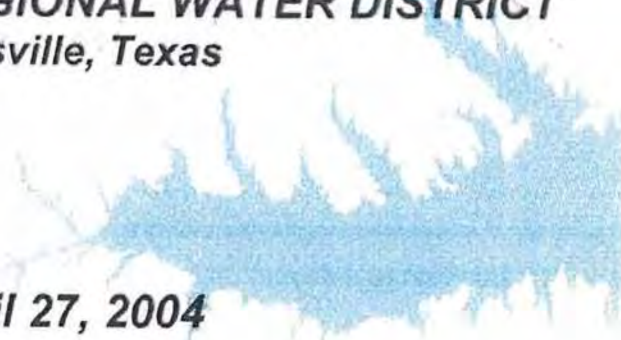
PREPARED BY
R.J. BRANDES COMPANY

UPPER TRINITY
REGIONAL WATER DISTRICT

HYDROLOGIC AND HYDRAULIC STUDIES OF LAKE RALPH HALL

prepared for

UPPER TRINITY REGIONAL WATER DISTRICT
Lewisville, Texas



April 27, 2004

prepared by

R. J. BRANDES COMPANY
Austin, Texas

HYDROLOGIC AND HYDRAULIC STUDIES OF LAKE RALPH HALL

prepared for

UPPER TRINITY REGIONAL WATER DISTRICT

UPPER TRINITY REGIONAL WATER DISTRICT
Lewisville, Texas

April 27, 2004

STATE OF TEXAS
ROBERT J. BRANDES
39120
REGISTERED
PROFESSIONAL ENGINEER

Robert J. Brandes
04-30-04

prepared by

R. J. BRANDES COMPANY
Austin, Texas

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

HYDROLOGIC AND HYDRAULIC STUDIES OF LAKE RALPH HALL

1.0 PROJECT DESCRIPTION

Lake Ralph Hall (the "Project") is being proposed on the North Sulphur River in the Sulphur River Basin in Fannin County, Texas by the Upper Trinity Regional Water District ("UTRWD") for the primary purpose of creating and developing a municipal water supply reservoir. Water from the Project is to be used to meet future water demands within that portion of Fannin County that lies in the Sulphur River Basin and within the service area of the UTRWD in the Trinity River Basin. The use of water from the proposed reservoir in the Trinity River Basin will involve an interbasin transfer across the boundary between the Sulphur and Trinity Basins.

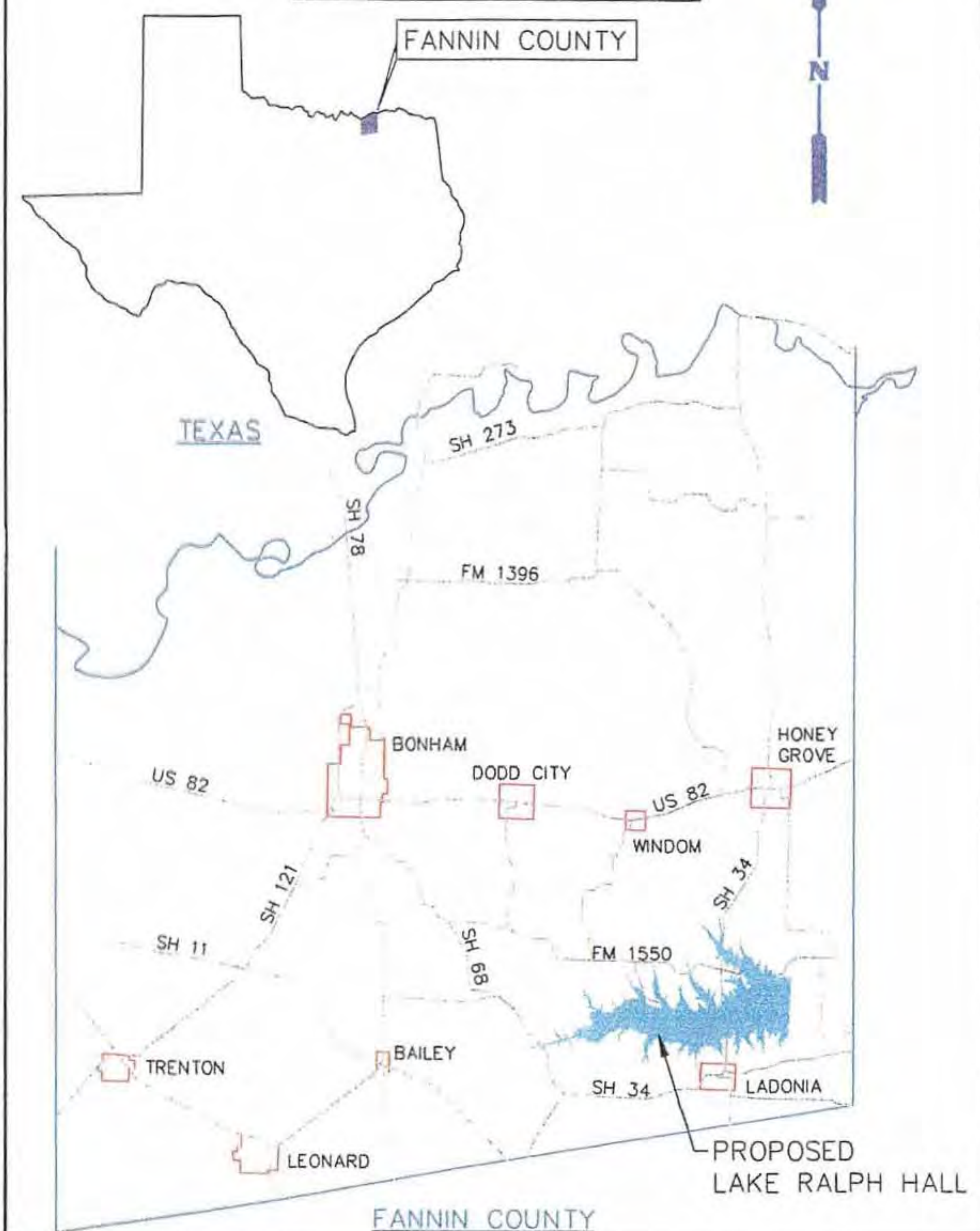
As proposed, Lake Ralph Hall will have a maximum conservation storage capacity of 160,235 acre-feet (at an elevation of 551.0 feet above mean sea level), and at that capacity, the surface area of the reservoir will cover approximately 7,605 acres (or about 11.9 square miles). The maximum depth of the reservoir at the dam will be approximately 90 feet. The firm yield of the Project is estimated to be approximately 32,940 acre-feet/year; however, annual withdrawals from the reservoir may be as much as 45,000 acre-feet/year as the Project is operated in a systems mode with other UTRWD sources of water in order to maximize UTRWD's overall available water supply.

Ralph Hall Dam is to be located on the North Sulphur River approximately 22.5 miles southeast from the city of Bonham, the county seat of Fannin County. Figure 1-1 presents a map of Fannin County that shows the location of the dam and the associated reservoir. An enlarged map of the reservoir area and the boundary of the reservoir is presented in Figure 1-2. The closest city to the Project is Ladonia, which is located approximately 3.5 miles southwest of the dam. The basin boundary of the North Sulphur River upstream of Ralph Hall Dam is delineated on the map of the region in Figure 1-3, along with sub-basin boundaries used in the hydrologic analyses. The total area of this watershed above the dam site is approximately 64,600 acres, or about 100.9 square miles. As shown on the map, the area surrounding and upstream of Lake Ralph Hall is rural and generally undeveloped and used primarily for agriculture, both farming and ranching.

The reach of the North Sulphur River where the Project is to be located is unique because of the river's deep, incised and eroded channel that lies within a fairly broad, flat floodplain. While the depth and width of the river channel vary in the vicinity of the proposed Project, at the proposed dam site it is a steep-walled, deep gorge approximately 40 feet deep and 300 feet wide, with the capacity to fully contain and convey the 100-year flood. The existing river channel has been formed over the years by extensive erosion of a relatively small man-made drainage ditch that was constructed in the late 1920's and early 1930's along the valley of the North Sulphur River to protect and drain agricultural fields. With the impoundment of Lake Ralph Hall, the ongoing erosional processes in the river channel within the reservoir and for some distance downstream will be curtailed.

The proposed structure for Ralph Hall Dam will consist of an earth-filled embankment across the valley of the North Sulphur River with a concrete uncontrolled principal spillway located within the existing channel of the river and a concrete ogee-type emergency spillway located within the

FIGURE 1-1
FANNIN COUNTY MAP
SHOWING PROJECT LOCATION



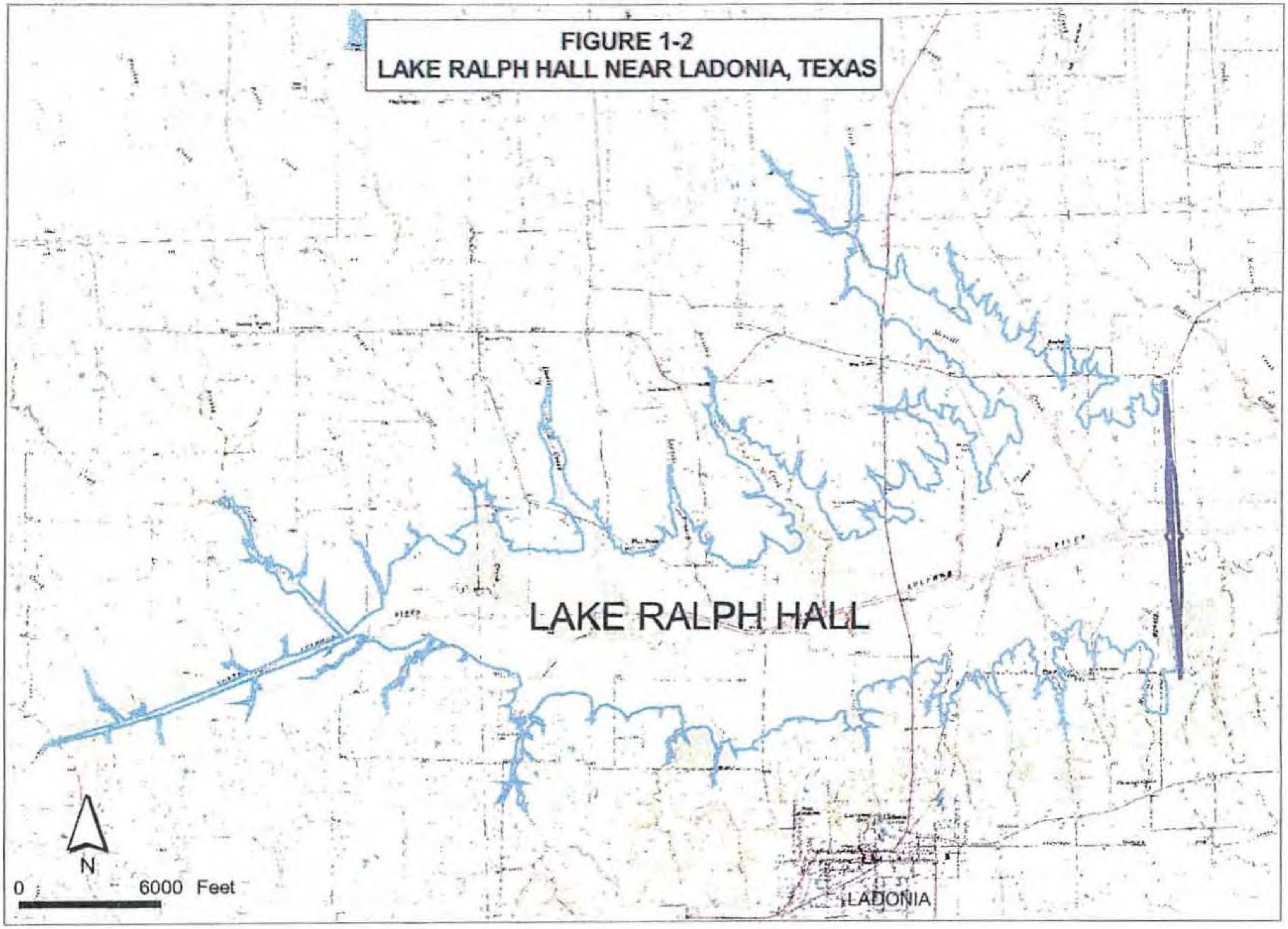


FIGURE 1-2
LAKE RALPH HALL NEAR LADONIA, TEXAS
LAKE RALPH HALL
N
0
6000 Feet
LADONIA

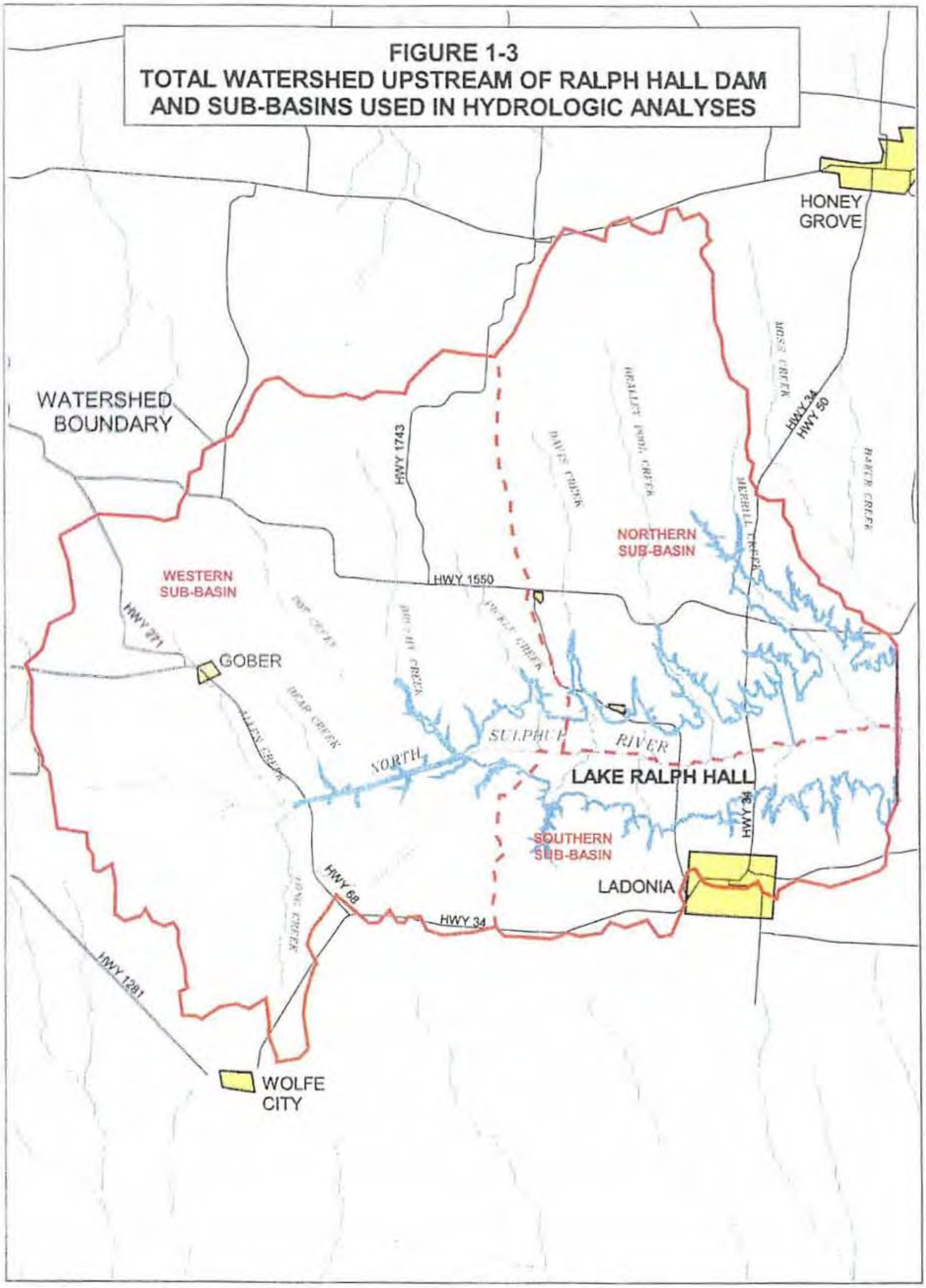


FIGURE 1-3
TOTAL WATERSHED UPSTREAM OF RALPH HALL DAM
AND SUB-BASINS USED IN HYDROLOGIC ANALYSES
HONEY GROVE
WATERSHED BOUNDARY
HWY 1743
HWY 34
HWY 50
NORTHERN SUB-BASIN
WESTERN SUB-BASIN
HWY 1550
HWY 271
GOBER
NORTH SULPHUR RIVER
LAKE RALPH HALL
HWY 34
SOUTHERN SUB-BASIN
LADONIA
HWY 68
HWY 34
HWY 1281
WOLFE CITY

embankment on the northern floodplain of the river. The top of the dam (embankment) will be at an elevation of 562.0 feet above mean sea level ("msl") and will tie in to existing natural ground on both ends of the structure.

The principal spillway, which is configured as a five-cycle, 300-foot wide trapezoidal labyrinth weir with a total crest length of 827 feet and a crest elevation of 552.0 feet msl, is designed to pass the 100-year flood with only about 3.1 feet of rise in the level of the reservoir above the top of the conservation pool. The downstream end of the center cycle of the labyrinth weir will be lowered by 1.0 foot (to elevation 551.0 feet msl) to provide an 80-foot long service spillway for the dam with the capacity to pass small flood flows (up to approximately the two-year flood). With its crest at elevation 551.0 feet msl, this service spillway will control the maximum level of the conservation pool of the reservoir. An additional low-flow pipe outlet with a gate tower also is to be installed to provide a means for passing low river flows through the dam when the normal overflows through the service spillway are not sufficient to satisfy downstream flow requirements. The low-flow pipe outlet also may be used to provide flows into an abandoned segment of the old river channel downstream of the dam that is being considered for restoration as part of the Project for environmental mitigation purposes.

The crest of the emergency spillway is to be set at an elevation of 554.1 feet msl, i.e., the maximum level of the reservoir during passage of the 100-year flood, and this spillway, combined with the principal spillway, is designed to safely pass the probable maximum flood with approximately 2.0 feet of freeboard. Downstream of the dam, a set of training berms are to be constructed to direct overflows from the emergency spillway across the northern floodplain toward the existing channel of the river.

2.0 RIVER HYDROLOGY

Flows in the North Sulphur River primarily are runoff-driven, although spring discharges do occur for sustained periods following rainfall events. During prolonged dry periods of several months, conditions of no flow persist along substantial reaches of the channel of the North Sulphur River.

There is one streamflow gage located on the North Sulphur River that can be used to characterize and evaluate historical river flow conditions. This gage is operated by the U. S. Geological Survey ("USGS"), and it is referred to as the "North Sulphur River near Cooper, TX" gage (No. 07343000). Mean daily streamflow records from this gage are available since October, 1949. The gage is located approximately 20 river miles downstream of the Ralph Hall Dam site. The total drainage area upstream of this gage covers 276 square miles, which is approximately 175 square miles more than the drainage area above the dam site. The drainage area above the dam site represents 36.6 percent of the total drainage area above the gage.

The mean daily flow in the North Sulphur River at the gage for the period from October, 1950 through September, 2001 is reported by the USGS to have been 261 cubic feet per second ("cfs"), which is equivalent to a mean annual flow of approximately 188,900 acre-feet per year. The median flow of the river for this same period was only 11 cfs, which indicates that the flow in the river has been low much of the time and that significant flood events periodically have occurred and caused the historical mean flow of the river to be relatively high. Statistical analyses of the historical daily flows at this gage indicate that the flow has been zero at least ten percent of the time, and that it has exceeded only 306 cfs approximately ten percent of the time.

Historical monthly flows measured at the gage on the North Sulphur River are plotted on the graph in Figure 2-1. As shown, the monthly river flows have varied considerably obviously in response to rainfall conditions in the basin. Some months the flows have been almost zero, whereas in other months significant flood flows have occurred. These historical monthly flows have provided a substantial part of the hydrologic record that has been used to develop the inflows to Lake Ralph Hall for purposes of evaluating the yield of the reservoir.

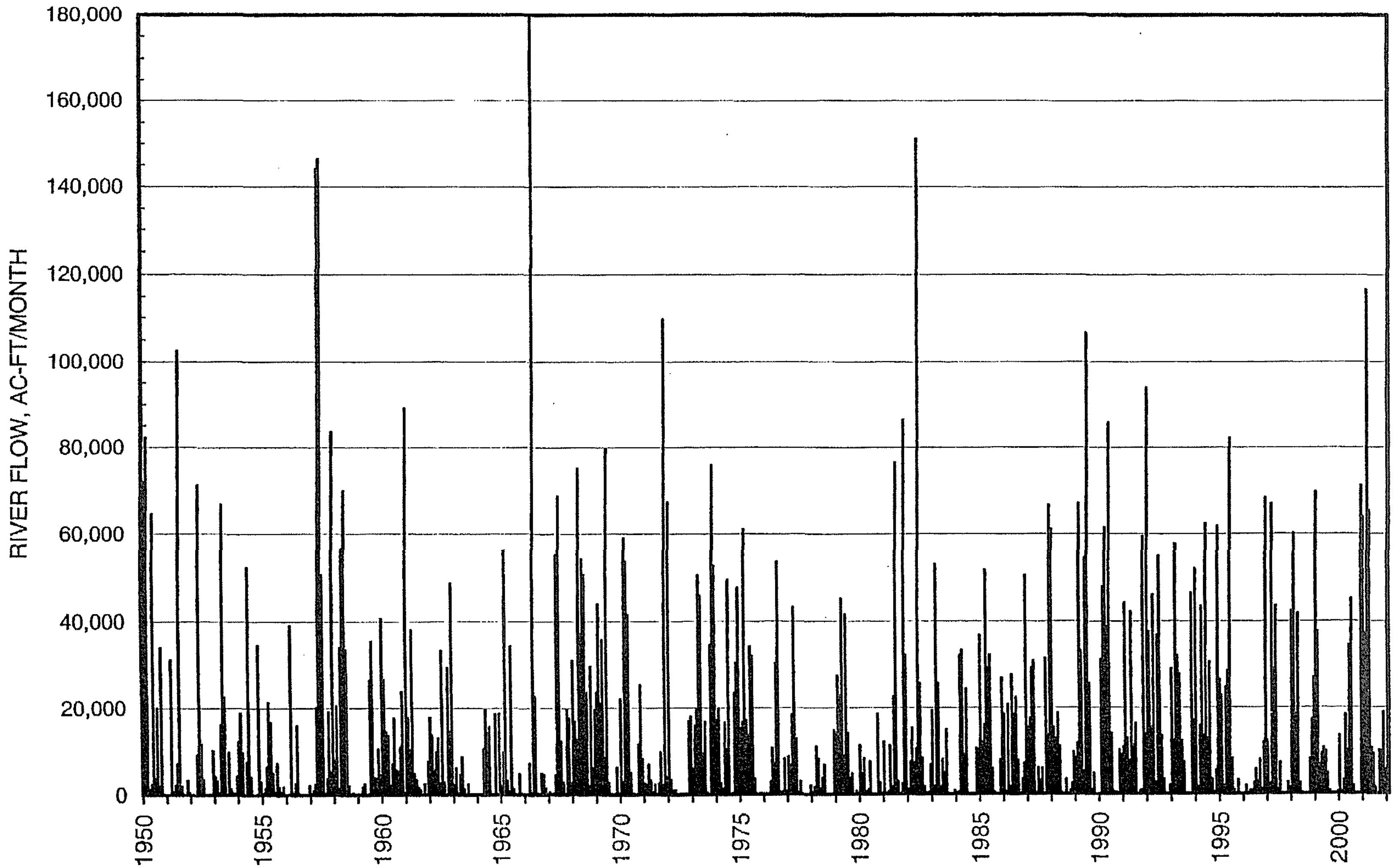
An important aspect of the hydrologic conditions that have occurred historically on the North Sulphur River relates to certain minimum flows that may be necessary to protect the existing aquatic ecosystem of the river. Even though the gage records indicate that river flows have been zero for extended periods of time suggesting that viable communities of aquatic organisms are not likely to have been sustained continuously over time along the river, the construction of Ralph Hall Dam and the operation of Lake Ralph Hall will likely require that certain quantities of river flow be passed through the reservoir, but not released from reservoir storage, in order to protect downstream aquatic resources.

The amounts of these required environmental flows for the Project will be finally determined based on results from in-depth field and analytical studies and future discussions with State regulatory agencies. However, in the mean time, it is considered prudent to include some level of environmental flow requirements in the analysis of the firm yield of Lake Ralph Hall. For this purpose, the default methodology of the Texas Commission on Environmental Quality ("TCEQ"), referred to as the Lyons Method, for establishing preliminary minimum environmental flows in Texas streams has been applied. This method basically assumes that 40 percent of the median daily flow for each of the months of October through February and 60 percent of the median daily flow for each of the months of March through September are adequate to protect existing riverine aquatic resources. Notwithstanding that historical flows in the North Sulphur River often have been less than these levels of flow and, in fact, some times have been zero for extended periods, the Lyons Method has been used to establish preliminary estimates of the required minimum environmental flows for the sole purpose of determining the yield of Lake Ralph Hall.

Results from applying the Lyons Method to the historical flows of the North Sulphur River are summarized in Table 2-1. In this table, the historical median daily flows in the river at the "North Sulphur River near Cooper, TX" gage are listed for each month of the year based on October, 1949 through September, 2002 daily flow records. The corresponding median monthly flows at the dam site are estimated by applying the drainage area ratio for the dam site relative to the gage (0.366). The Lyons monthly flow factors (40% or 60%) then are applied to the monthly median daily flows at the dam site to establish the corresponding preliminary estimates of the required monthly minimum environmental flows for the North Sulphur River at the dam site.

It is the practice of the TCEQ that the minimum environmental flows for a particular stream reach should not be less than the minimum flow that is necessary for application of the State's water quality standards in that particular reach. In this case, the minimum flow required for application of the State's water quality standards in this reach of the North Sulphur River as

FIGURE 2-1
HISTORICAL MONTHLY NORTH SULPHUR RIVER FLOWS AT GAGE NO. 07343000



established by the TCEQ is 0.1 cfs, or approximately 6.0 acre-feet/month. An adjustment for this minimum flow condition is required for the month of August (when the Lyons minimum environmental flow in Table 2-1 is indicated to be less than 0.1 cfs). With this adjustment, the preliminary minimum environmental flows at the dam site that have been used for purposes of the Project yield analyses have been determined and are listed in the far right column of Table 2-1. As shown, these flows range from 0.1 cfs (6 acre-feet/month) during August and September when zero or low river flows often occur up to 7.9 cfs (486 acre-feet/month) during March in the spring when storms typically produce higher flows in the river. The values of the preliminary monthly minimum environmental flows listed in Table 2-1 have been specified as environmental flow requirements in the yield analyses for Lake Ralph Hall, and these are the quantities of river flow that have been passed through the reservoir for environmental purposes, limited to the available inflows to the reservoir.

TABLE 2-1
ANALYSIS OF LYONS ENVIRONMENTAL FLOW REQUIREMENTS

Drainage Area at Ralph Hall Dam Site:	100.9	square miles
Drainage Area at Gage No. 07343000	276.0	square miles
Ratio of Dam-to-Gage Drainage Areas:	0.366	
TCEQ Minimum Flow for Water Quality:	0.1	cfs (7Q2 Flow)
TCEQ Minimum Flow for Water Quality:	6	ac-ft/month

MONTH	MEDIAN * FLOW AT GAGE cfs	MEDIAN FLOW AT DAM SITE cfs	LYONS % OF MEDIAN FLOW	LYONS MINIUMUM ENVIRON. FLOWS AT DAM SITE cfs	ac-ft	PRELIMINARY MINIMUM ENVIRON. FLOWS AT DAM SITE cfs	ac-ft
JAN	26.0	9.5	40%	3.8	211	3.8	211
FEB	40.0	14.6	40%	5.8	325	5.8	325
MAR	36.0	13.2	60%	7.9	486	7.9	486
APR	28.0	10.2	60%	6.1	365	6.1	365
MAY	24.0	8.8	60%	5.3	324	5.3	324
JUN	11.0	4.0	60%	2.4	144	2.4	144
JUL	1.6	0.6	60%	0.4	22	0.4	22
AUG	0.2	0.1	60%	<0.1	3	0.1	6
SEP	0.5	0.2	60%	0.1	7	0.1	7
OCT	1.6	0.6	40%	0.2	14	0.2	14
NOV	9.3	3.4	40%	1.4	81	1.4	81
DEC	20.0	7.3	40%	2.9	180	2.9	180
						Total =	2,164

* Based on 1949-2002 mean daily flow records.

3.0 PROJECT YIELD

The firm annual yield of Lake Ralph Hall has been evaluated using the TCEQ's current version of the Water Availability Model ("WAM") for the Sulphur River Basin. For these analyses, the Run 3 data set, which assumes full utilization of all water rights in the basin and no return flows from municipal and industrial wastewater treatment plants, has been applied. This is the data set that the TCEQ normally would use for evaluating water availability for applications for new or amended surface water appropriations. Lake Ralph Hall has been incorporated into the Sulphur Basin WAM by establishing a new control point on the North Sulphur River at the location of the proposed dam site and assigning appropriate watershed parameters to the control point for the upstream drainage area, i.e., drainage area equal to 100.9 square miles, curve number equal to 70 and mean annual rainfall equal to 43.0 inches. Elevation-area-capacity relationships and the corresponding conservation storage capacity for the proposed reservoir, as determined from a two-foot contour map of the reservoir site prepared specifically for the Project, also were specified in the WAM data file. The elevation-area-capacity relationships for the proposed reservoir are plotted on the graphs in Figure 3-1.

The modified WAM with Lake Ralph Hall included has been operated for a range of maximum conservation storage capacities to develop a relationship between reservoir storage and firm annual yield for Lake Ralph Hall. For these simulations, the maximum elevation of the conservation pool of the reservoir has been assumed to range between elevation 545 feet msl and elevation 552 feet msl, and the corresponding maximum conservation storage capacities have been used in the firm yield analyses. For each maximum conservation storage capacity, iterative simulations with assumed annual demands on the reservoir have been made with the WAM until the firm yield has been determined. For all of these simulations, a municipal-type monthly demand distribution has been used. The resulting yield-versus-conservation storage capacity relationship is plotted on the graph in Figure 3-2.

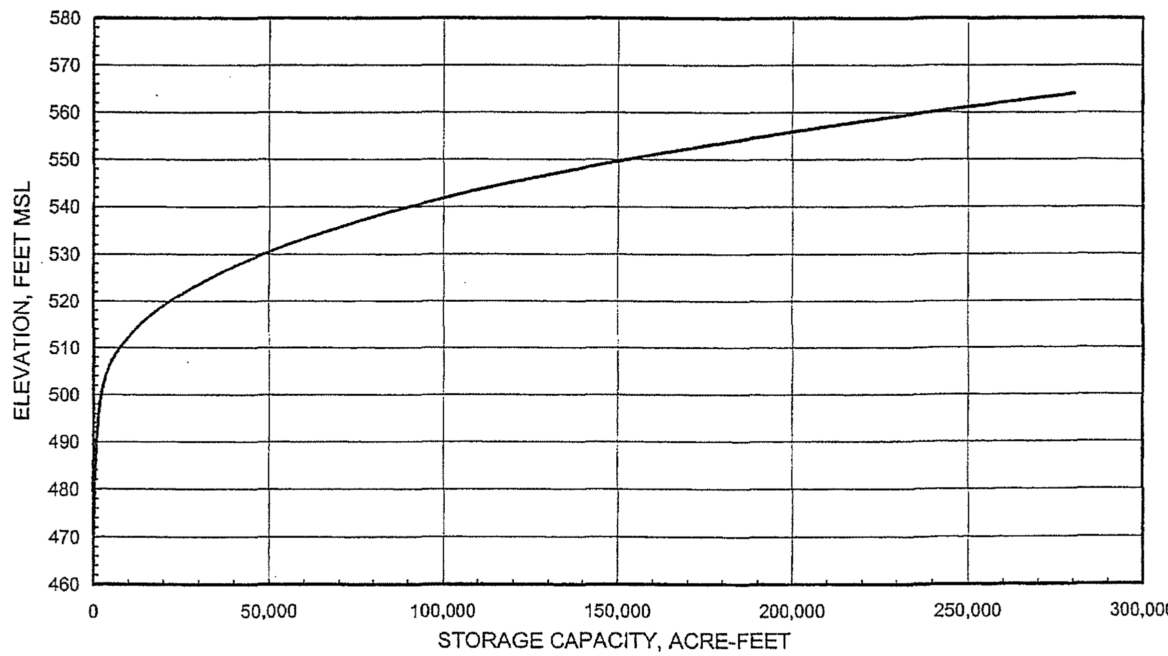
The determination of the final configuration and size of the proposed Lake Ralph Hall and Ralph Hall Dam has involved consideration of the firm yield results depicted in the above graph in conjunction with results from the analysis of the ability of the reservoir to safely pass various design floods as described later in this report. The adopted designs for the service, principal and emergency spillways for the dam correspond to a maximum conservation pool level of 551.0 feet msl and a maximum conservation storage capacity of 160,235 acre-feet. As shown on the graph in Figure 3-2, the resulting firm annual yield for this size reservoir based on the WAM simulations is 32,940 acre-feet/year, and this is the yield that has been used by the UTRWD for purposes of water supply planning relative to Lake Ralph Hall.

4.0 FLOOD MODELING

For analyzing the flood operation aspects of Lake Ralph Hall and Ralph Hall Dam, several different hydrologic and hydraulic models have been developed to represent conditions at the reservoir site. For simulating flood flow hydraulics along the existing channel and floodplain of the North Sulphur River in the vicinity of the reservoir, the Corps of Engineers' HEC-RAS River Analysis System program has been applied. For simulating stormwater runoff hydrographs for the drainage area upstream of the reservoir in response to specified rainfall events and for routing these flood flow hydrographs down the river in the vicinity of the reservoir under existing conditions and through the reservoir under conditions with the Project in place, the Corp of Engineers' HEC-1 Flood Hydrograph Package program has been used.

FIGURE 3-1
LAKE RALPH HALL ELEVATION-STORAGE-SURFACE AREA RELATIONSHIPS

STORAGE CAPACITY VERSUS ELEVATION



SURFACE AREA VERSUS ELEVATION

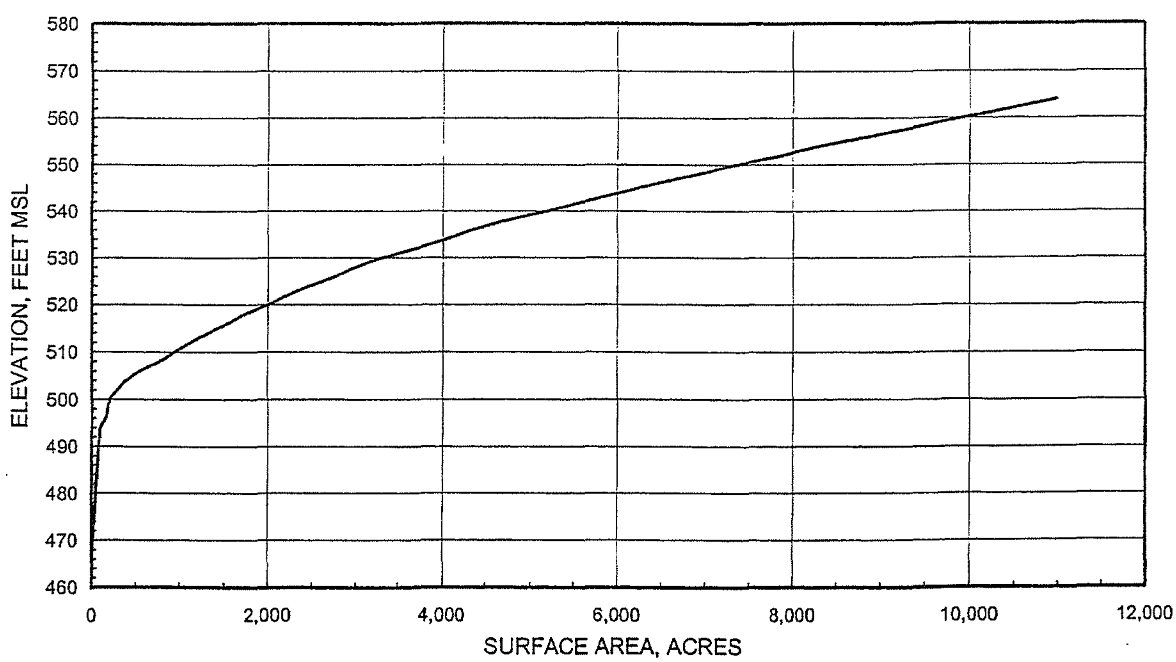
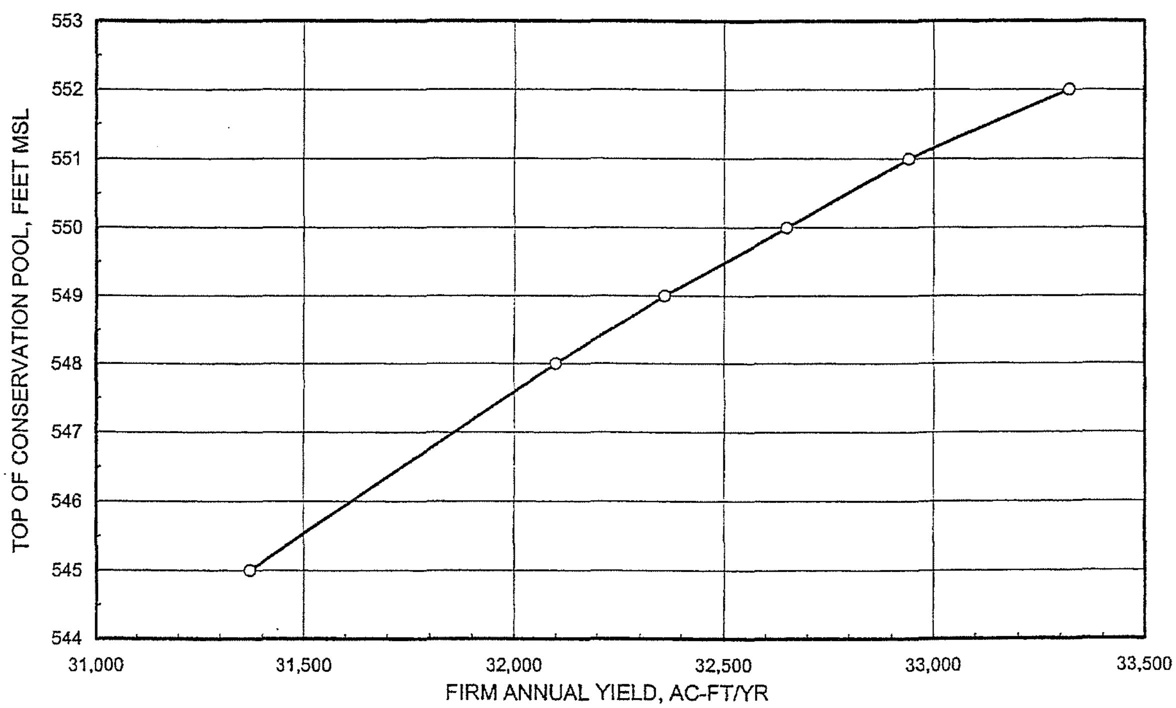


FIGURE 3-2
VARIATION OF PROJECT YIELD WITH CONSERVATION STORAGE CAPACITY



Together, these modeling tools provide the means for evaluating the behavior of the reservoir and the operation of the proposed spillways under different design flood conditions and for assessing the impacts of the Project with respect to flooding along the river both downstream and upstream of the dam.

4.1 HEC-RAS Model

The computational sections used to construct the HEC-RAS model of the reach of the North Sulphur River in the vicinity of the proposed reservoir are delineated on the map of the area in Figure 4-1. There are 32 sections included in the model to describe the geometric configuration and hydraulic roughness condition of the river channel and floodplain through this reach of the river. For each of these sections, geometric data describing the cross-sectional shape of the section have been developed from the two-foot contour map of the reservoir site that was prepared specifically for the Project. These data have been extended to include the higher floodplain areas using available USGS topographic maps of the area.

Manning's "n" roughness coefficients have been assigned to different segments of each of the HEC-RAS computational sections based on inspection of aerial photography of the reservoir area to identify general land use types and vegetation coverage and field observations of actual channel and overbank roughness conditions. Generally, the assigned values of the Manning's

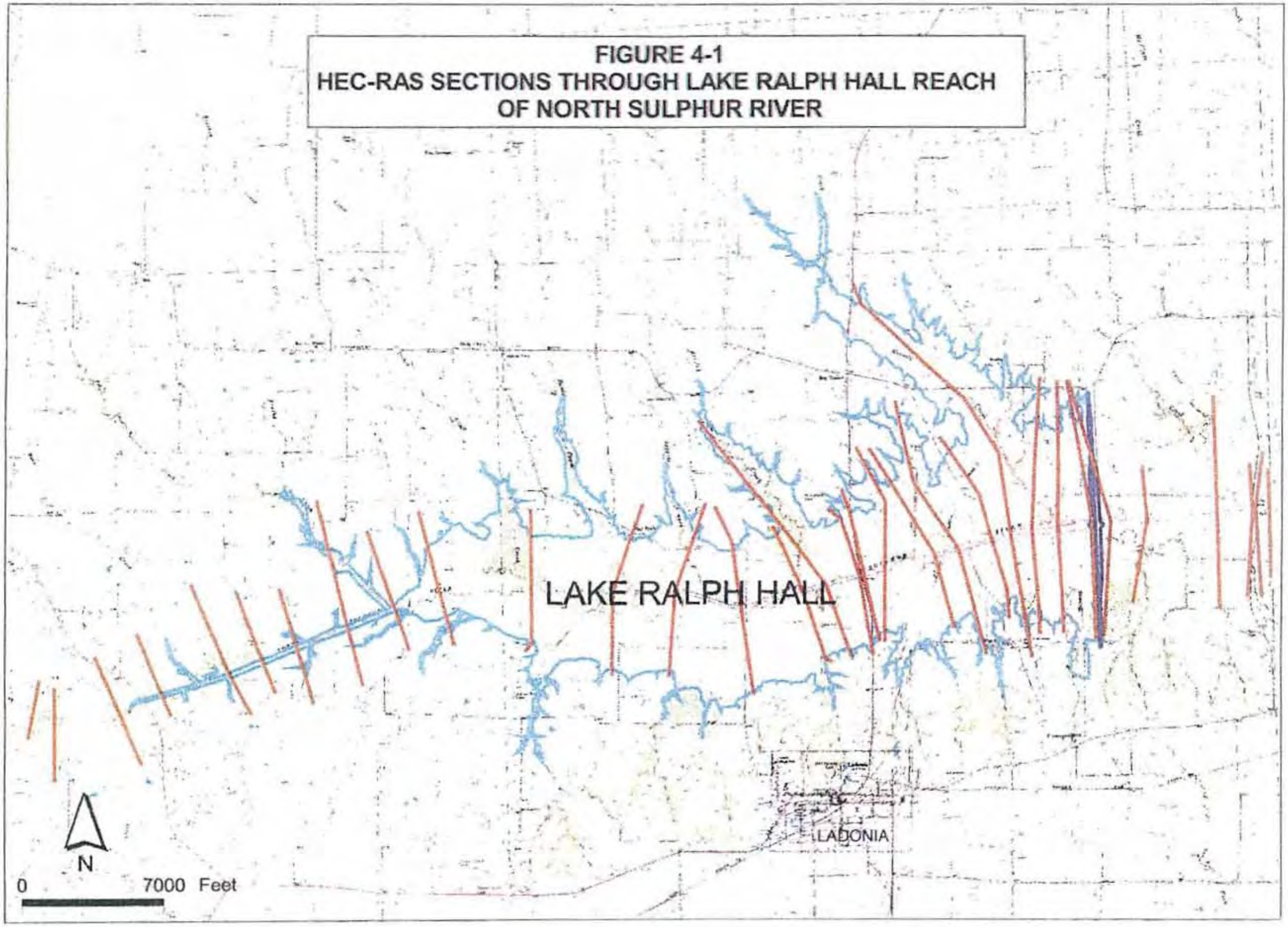
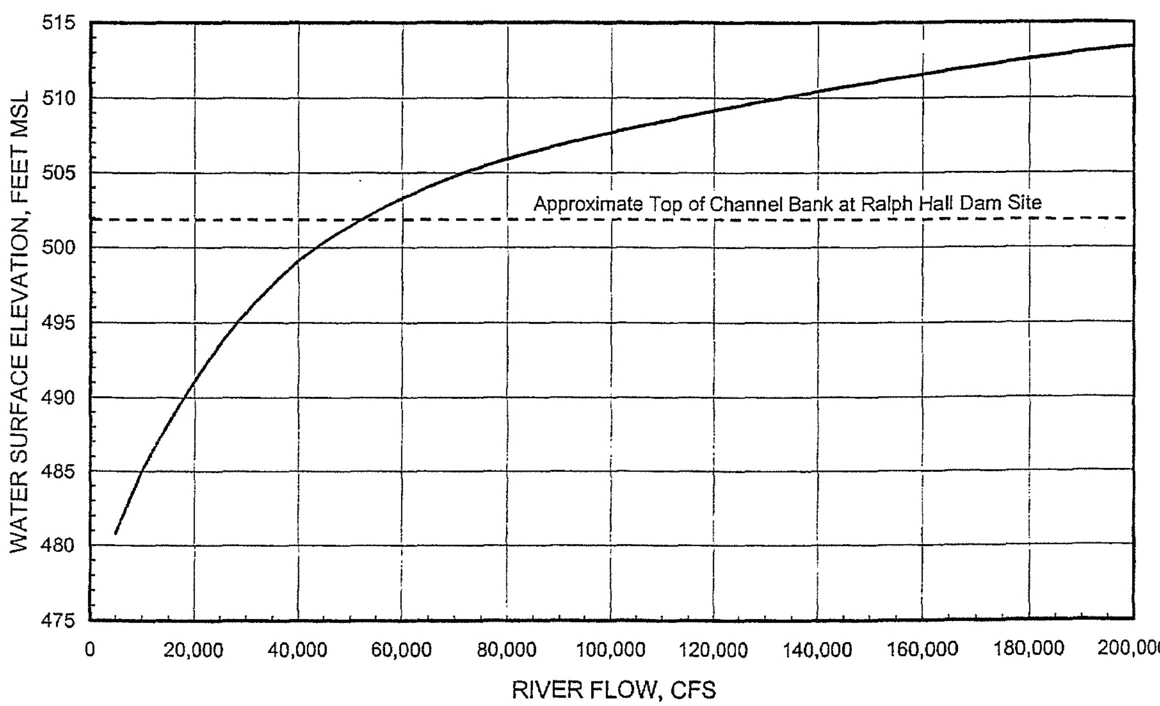


FIGURE 4-1
HEC-RAS SECTIONS THROUGH LAKE RALPH HALL REACH
OF NORTH SULPHUR RIVER
LAKE RALPH HALL
LADONIA
N
0
7000 Feet

"n" parameters used in the HEC-RAS model are on the order of 0.05 for the river channel and in the range of 0.07 for the overbank and floodplain areas.

The HEC-RAS model has been used primarily to investigate flood levels along the river in the vicinity of the reservoir under existing conditions for different levels of river flood flow to establish the flood-carrying capacity of the existing channel and to determine tailwater conditions at the dam site. The graph in Figure 4-2 shows the variation of the water level in the river at the proposed dam site with flow as simulated with the HEC-RAS model. As indicated, river flows on the order of 50,000 cfs begin to overtop of the existing channel banks and cause inundation of the floodplain. HEC-RAS simulations also have been made to establish the storage-versus-discharge relationships for the river that have been used for Modified Puls flood routing in the HEC-1 model.

FIGURE 4-2
WATER SURFACE ELEVATION VERSUS FLOW IN NORTH SULPHUR RIVER
AT PROPOSED RALPH HALL DAM SITE AS SIMULATED WITH HEC-RAS MODEL



4.2 HEC-1 Models

As noted above, two different HEC-1 flood routing models have been developed for the Project. One reflects existing channel and floodplain conditions along the river in the vicinity of Lake Ralph Hall (referred herein as the "existing conditions" HEC-1 model), and the other represents conditions with the proposed reservoir in place with its associated spillways in operation (referred herein as the "reservoir conditions" HEC-1 model).

Both HEC-1 models include the same representation of the upstream watershed that contributes runoff and flood flows to the reach of the river that is to be occupied by the reservoir (as in the case of the existing conditions model) or actually occupied by the reservoir (as in the case of the reservoir conditions model). The existing conditions HEC-1 model routes flood flows from the upstream watershed through this river reach using the Modified Puls method with appropriate storage-versus-discharge relationships derived from the HEC-RAS hydraulic model. The reservoir conditions HEC-1 model routes flood flows from the upstream watershed and rainfall that falls directly on the reservoir surface through the reservoir itself using the level-pool routing procedure in the HEC-1 program. Both models produce a flood flow hydrograph immediately downstream of the proposed dam site. These are the hydrographs that have been compared to evaluate the impacts of the proposed Project on downstream flooding conditions.

For structuring the runoff component of the HEC-1 models, the total Lake Ralph Hall watershed has been divided into three sub-basins to facilitate the description of actual rainfall-runoff processes and the overall hydrologic behavior of the watershed. These sub-basins are referred to as the Western Sub-Basin, the Southern Sub-Basin and the Northern Sub-Basin, and their boundaries are delineated on the map of the region in Figure 1-3. For the existing conditions model, the area of three sub-basins includes the surface area that is to be inundated by Lake Ralph Hall at its normal maximum pool level, i.e., at elevation 551.0 feet msl. For the reservoir conditions model, the area of each of the three sub-basins is reduced by an amount equal to the actual surface area that is to be inundated by Lake Ralph Hall, and a separate (fourth) sub-basin is included in the reservoir conditions model to represent the entire surface area of the reservoir at its normal maximum pool level, i.e., 11.9 square miles.

To model runoff from the Lake Ralph Hall watershed for specified amounts and patterns of rainfall, various hydrologic parameters have been determined and specified as input data to the HEC-1 models. To account for infiltration losses and surface retention within the watershed, the "curve number" method developed by the U. S. Soil Conservation Service ("SCS") has been applied. Soil types and conditions throughout the watershed have been examined using GIS techniques with the SCS's digitized soil classification data base (STATSGO), and this information has been combined with electronic land use data and digital elevation data from the Texas Natural Resources Information System to establish the appropriate runoff curve numbers for each of the sub-basins. For normal antecedent moisture conditions, the resulting curve number values for all the sub-basins have been determined to be approximately 70, and this is the value that has been used each of the sub-basins in the HEC-1 models for all rainfall events except the probable maximum flood. For the PMF, the curve number has been adjusted to reflect wet antecedent moisture conditions, and the adopted value that has been used is 85. For modeling the runoff associated with rainfall directly on the reservoir surface, a curve number value of 100 has been used.

To translate the specified rainfall distribution for a particular storm event to a runoff hydrograph with the HEC-1 model, several different unit hydrograph techniques have been considered, including the SCS dimensionless unit hydrograph approach and the Snyder unit hydrograph method. The Snyder unit hydrograph method was previously used by the Corps of Engineers for developing flood inflow hydrographs for Lakes Jim Chapman and Wright Patman, both of which are located in the Sulphur Basin; therefore, to facilitate comparison and validation of the unit hydrograph parameters for Lake Ralph Hall, the Snyder method also has been adopted for simulating flood inflows to Lake Ralph hall. Values of the required Snyder coefficients for each of the three sub-basins in the Lake Ralph Hall watershed have been established through a process involving detailed analysis of runoff travel times (times of concentration) and watershed characteristics for each of the three sub-basins and consideration of specific Snyder coefficient information for other watersheds in the region. Particular relevance has been given to the parameters developed by the Corps for Lake Jim Chapman, since its watershed is immediately adjacent to the Lake Ralph Hall watershed. The following Snyder coefficients were developed and used by the Corps for Lake Jim Chapman: Ct = 2.5 and Cp640 = 350 (Cp = 0.55).

The time of concentration ("tc") of each of the three sub-basins in the Lake Ralph Hall watershed has been estimated using the SCS procedures outlined in the SCS Technical Release 55 report titled "Urban Hydrology for Small Watersheds" (1986). In accordance with this method, travel time calculations have been made for conditions of sheet flow, shallow concentrated flow and channel flow for each of the sub-basins, and these results have been combined with the wave propagation time for the reservoir to estimate the total time of concentration and SCS lag time (0.6 x tc) for each of the sub-basins. Appendix A of this report contains the spreadsheet calculations that were performed in applying the SCS TR-55 method for estimating the time of concentration for each of the sub-basins, assuming that the proposed reservoir is in place. The resulting time of concentration values, the corresponding SCS lag times and the corresponding Snyder Ct values, based on the standard Snyder equation for lag time ("tp"), are summarized in the following table.

TABLE 4-1
RUNOFF TRAVEL TIME PARAMETERS FOR LAKE RALPH HALL
BASED ON SCS TR-55 METHOD

SUB-BASIN	TIME OF CONCENTRATION (hours)	SCS LAG TIME (hours)	SNYDER Ct COEF
Western	5.34	3.20	1.99
Southern	1.50	0.90	1.34
Northern	4.09	2.45	1.85

The runoff travel time parameters for the Lake Ralph Hall sub-basins also have been derived based on the Snyder Ct value of 2.5 that was adopted and used by the Corps for determining flood inflow hydrographs for Lake Jim Chapman. Using this coefficient value with the standard Snyder tp equation for lag time, the resulting lag times and times of concentrations for the three sub-basins in the Lake Ralph Hall watershed have been determined and are summarized below in Table 4-2.

TABLE 4-2
RUNOFF TRAVEL TIME PARAMETERS FOR LAKE RALPH HALL
BASED ON SNYDER STANDARD LAG TIME EQUATION AND Ct = 2.5

SUB-BASIN	TIME OF CONCENTRATION (hours)	SCS LAG TIME (hours)	SNYDER Ct COEF
Western	6.70	4.02	2.5
Southern	2.78	1.67	2.5
Northern	5.52	3.31	2.5

As noted, the travel time parameters based on the Snyder lag time equation and the Corps' Lake Jim Chapman Ct value are slightly higher than those that were derived based on application of the SCS TR-55 method, but they generally are of the same magnitude. Since Corps guidelines regarding the selection of watershed runoff parameters and other applications of the Snyder unit hydrograph method suggest that values of the Snyder coefficients should be generally consistent within a given region, the final values of the time of concentration and the SCS lag time that have been adopted for simulating flood inflow hydrographs for Lake Ralph Hall have been established based on approximate averages of the values presented in Tables 4-1 and 4-2. The adopted travel time parameters for the Lake Ralph Hall sub-basins with the reservoir in place are listed in the following table.

TABLE 4-3
ADOPTED RUNOFF TRAVEL TIME PARAMETERS FOR LAKE RALPH HALL

SUB-BASIN	TIME OF CONCENTRATION (hours)	SCS LAG TIME (hours)	SNYDER Ct COEF
Western	6.00	3.60	2.24
Southern	2.00	1.20	1.80
Northern	5.00	3.00	2.27

The above travel time parameters for the Lake Ralph Hall sub-basins are specifically applicable to the condition with the proposed reservoir in place. The travel time for flood wave propagation through the reservoir was included in the derivation of the SCS times of concentration. Hence, in order to derive appropriate travel time parameters for the three sub-basins for existing watershed conditions without the reservoir in place, the effects of the flood wave propagation travel time have been removed. As noted in the TR-55 spreadsheet calculations for time of concentration that are included in Appendix A, the travel times associated with flood wave propagation through the reservoir for the three sub-basins were determined to be 0.26 hours for the Western Sub-Basin and 0.10 hours for the Southern and Northern Sub-Basins. Applying these corrections to the adopted travel time parameters for conditions with the reservoir in place that are presented in Table 4-3 produces the corresponding travel time parameters for existing conditions without the reservoir in place as listed in Table 4-4.

TABLE 4-4
ADOPTED EXISTING CONDITIONS RUNOFF TRAVEL TIME PARAMETERS

SUB-BASIN	TIME OF CONCENTRATION (hours)	SCS LAG TIME (hours)	SNYDER Ct COEF
Western	5.74	3.44	2.14
Southern	1.90	1.14	1.71
Northern	4.90	2.94	2.22

As noted previously, the value of the Snyder Cp coefficient that was derived and used by the Corps for Lake Jim Chapman was 0.55, which is equivalent to a Cp640 value of 350. Since this parameter is particularly related to basin storage characteristics that generally tend to be regionally similar and consistent, the same value used by the Corps for Lake Jim Chapman has been adopted for application to all of the Lake Ralph Hall sub-basins.

4.3 Rainfall Data

Rainfall amounts and patterns have been specified in the HEC-1 models using different procedures depending on the magnitude of storm event being simulated and the purpose for which the models were being operated relative to the overall dam and spillway design process. In accordance with SCS and Corps guidelines for simulating runoff from watersheds associated with reservoirs the size of Lake Ralph Hall and for designing these types of structures, the 24-hour rainfall duration has been adopted and used for evaluating alternative spillway designs. Flood inflow hydrographs for the 100-year rainfall event and the probable maximum storm have been simulated with the HEC-1 models and used in the analyses for designing the principal and emergency spillways, respectively. More frequent storm events on the order of the one-year and two-year storms have been used for evaluating the service spillway (low-flow outlet).

Rainfall data for storm magnitudes equal to or less than the 100-year storm have been obtained for the Lake Ralph Hall site from the U. S. Weather Bureau's Technical Paper No. 40[1]. These 24-hour rainfall amounts for different storm return periods are listed in Table 4-5. These data represent historical rainfall conditions at the Lake Ralph Hall site.

TABLE 4-5
24-HOUR RAINFALL AMOUNTS FOR DIFFERENT RETURN PERIODS
AT THE LAKE RALPH HALL SITE

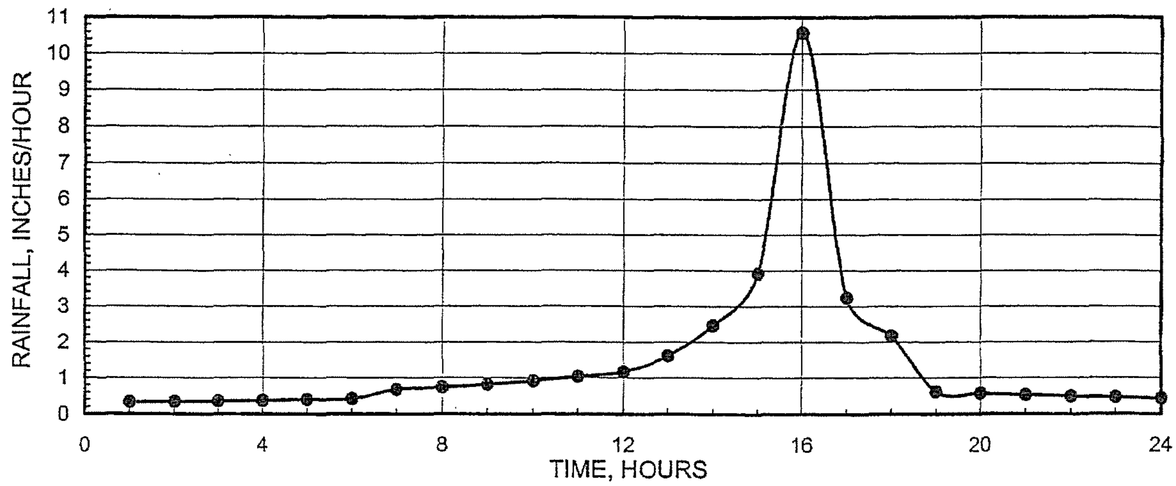
1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
3.34	4.08	5.50	6.47	7.61	8.56	9.62

For modeling storm magnitudes equal to or less than the 100-year event, the total 24-hour rainfall amounts have been distributed over the 24-hour duration of the storms using the "balanced storm" method (PH Card) included in the HEC-1 program. In applying this method, rainfall depths for durations of 5, 15 and 60 minutes and 2, 3, 6, 12 and 24 hours have been specified in the HEC-1 data files for each of the storms analyzed. The HEC-1 program automatically constructs a rainfall pattern that positions the higher rainfall intensities during the central part of the storm duration. In effect, this approach produces a temporal rainfall pattern for a given return period that includes in a single storm event all of the rainfall intensities ranging from the 5-minute intensity up to the 24-hour intensity for the same return period, which is very likely less frequent (more extreme) than the return period of the storm actually being analyzed.

For modeling the probable maximum storm ("PMS") event, the procedures outlined in the Corps Hydrologic Engineering Center's "HMR52 Probable Maximum Storm Users Manual" (1983) and included in the HMR-52 computer program (as modified, 1988) have been applied to develop the PMS rainfall characteristics for the Lake Ralph Hall site. The basin boundaries for the watershed upstream of the Ralph Hall Dam have been digitized and used as input to the HMR-52 program along with the basin size and the orientation of the PMS relative to the basin. Rainfall depth-area-duration data for the Lake Ralph Hall watershed have been compiled from the HMR-51 joint report of the Corps and the U. S. Department of Commerce[2]. The HMR-52 program has been operated to generate the 72-hour PMS rainfall pattern for the Lake Ralph Hall site, and in accordance with Corps guidelines, the most severe second-day rainfall distribution has been adopted for simulating the PMF inflows to Lake Ralph Hall. This 24-hour PMS rainfall pattern is plotted on the graph in Figure 4-3 in terms of one-hour rainfall amounts. As shown, the most intense rainfall occurs at hour 16 of the 24-hour period with a maximum of 10.58 inches falling in one hour. The total rainfall for the 24-hour PMS is 34.7 inches.

[1] Hershfield, D.M.; "Rainfall Frequency Atlas for the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years"; U. S. Department of Commerce, Weather Bureau; Washington, D.C.; 1961.
[2] Schreiner, L.D. and J. T. Riedel; "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian"; Hydrometeorological Branch, Office of Hydrology, National Weather Service, U.S. Department of Commerce and Corps of Engineers, U.S. Department of the Army; Washington, D.C.; 1978.

FIGURE 4-3
PATTERN OF HOURLY RAINFALL FOR 24-HOUR PROBABLE MAXIMUM STORM



4.4 Flood Hydrographs

The HEC-1 models have been operated to simulate flood flow hydrographs at the Ralph Hall Dam site under existing conditions without Lake Ralph Hall in place and under reservoir conditions with Lake Ralph Hall in place (but without the flood being routed through the reservoir and proposed spillways). These results are plotted on the graph in Figure 4-4 for the 100-year storm event and in Figure 4-5 for the PMS.

The effect of rainfall directly on the surface of the reservoir is readily apparent on these graphs. With the reservoir in place, the peak flow due to rainfall directly on the reservoir surface occurs before the peak flow due to runoff from the upstream watershed; consequently, the hydrographs exhibits two peaks. Furthermore, the peak flow for the 100-year flood at the dam site is increased from 36,312 cfs under existing conditions to 46,219 cfs with the reservoir in place because of the additional volume of flow produced with rainfall directly on the reservoir. Similarly, for the PMS, the peak flood flow is increased from 176,482 cfs to 206,719 cfs. As discussed in the next section, the combined effects of the reservoir and the proposed spillways substantially reduce the peak outflows from the dam as a result of the temporary storage of a significant portion of the flood inflows as surcharge above the conservation pool.

5.0 DAM AND SPILLWAY DESIGN

For the proposed height of Ralph Hall Dam (approximately 100 feet) and the proposed maximum conservation storage capacity (160,235 acre-feet), the dam safety rules of the TCEQ (Texas Administrative Code, Chapter 299) stipulate that the proposed facility is classified as a "Large" dam and reservoir, which means that the structure must be designed to safely pass the probable maximum flood ("PMF"). Pursuant to this requirement, a system of spillways has been configured and sized for Ralph Hall Dam such that the PMF for the region can be passed through the reservoir without overtopping of the dam structure.

FIGURE 4-4
COMPARISON OF INFLOW HYDROGRAPHS FOR 100-YEAR FLOOD
UNDER CONDITIONS WITHOUT AND WITH RALPH HALL RESERVOIR

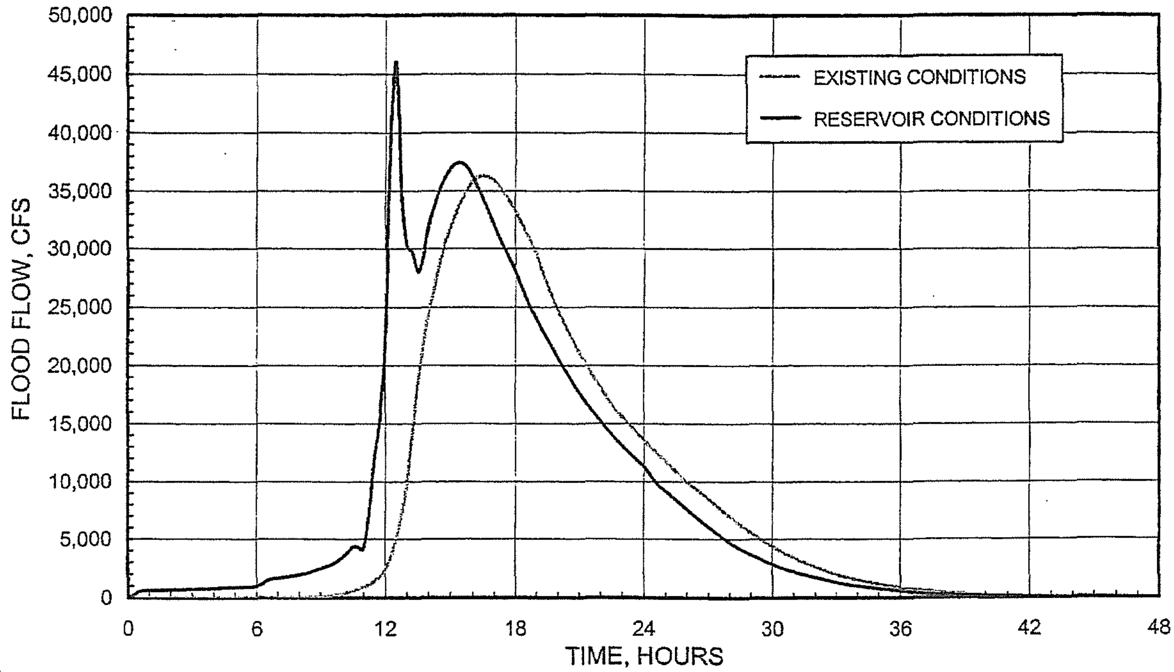
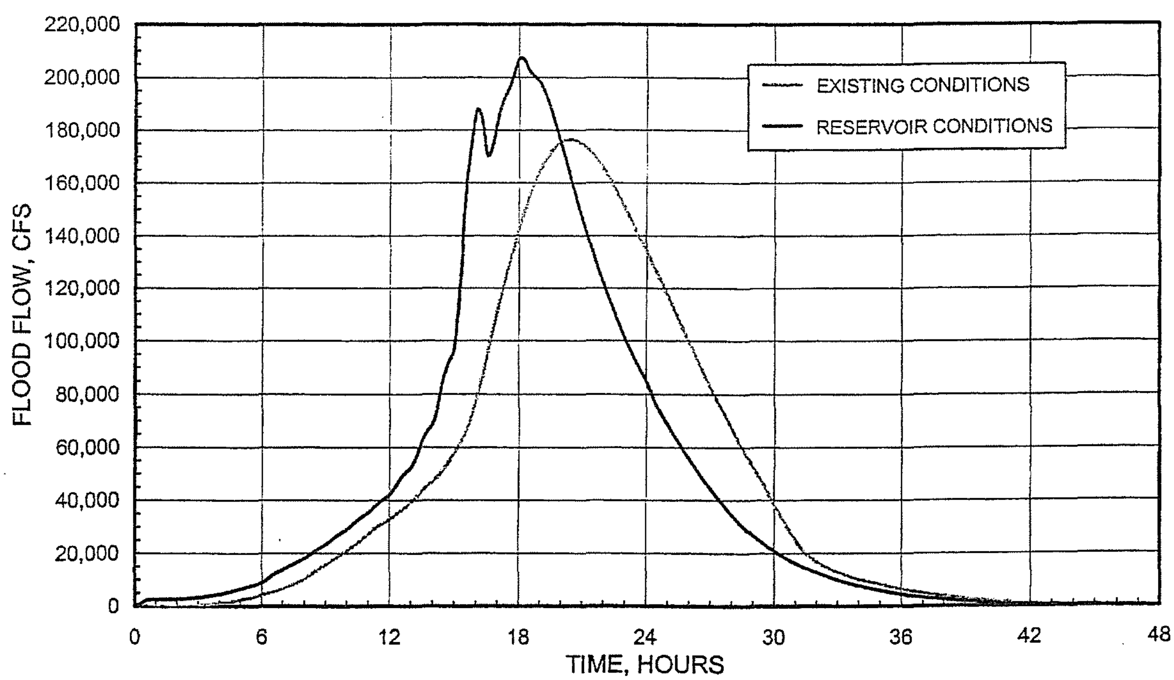


FIGURE 4-5
COMPARISON OF INFLOW HYDROGRAPHS FOR PROBABLE MAXIMUM FLOOD
UNDER CONDITIONS WITHOUT AND WITH RALPH HALL RESERVOIR



Analyses using the HEC-1 reservoir conditions model with various sizes, shapes, configurations, and combinations of principal and emergency spillways have been undertaken to establish spillway designs that satisfy the following specific design criteria for the dam.

1) Maximum water surface elevation of the reservoir under PMF design conditions no higher than 560.0 feet msl.
2) Normal maximum operating level of the conservation pool of the reservoir at or above elevation 548.0 feet msl in order to provide acceptable Project yield.
3) Peak outflows from the dam no greater than corresponding peak river flows under existing conditions for similar magnitude storm events.
4) General reduction in peak river flows downstream of the dam to reduce erosion of the existing river channel.
5) Principal spillway capacity adequate to safely pass the 100-year flood, with no flow through the emergency spillway.
6) Principal spillway located within the existing river channel with the spillway design discharge confined to the existing river channel downstream and with an appropriate stilling basin to dissipate outflow energy to acceptable levels.
7) Emergency spillway capacity adequate to safely pass the PMF with at least 2.0 feet of freeboard below the top of the dam structure.
8) Emergency spillway either incorporated into the principal spillway or located separately within the dam on the floodplain of river in a manner that minimizes downstream flooding and erosion impacts.
9) To the extent possible, entirely uncontrolled (ungated) spillways to minimize requirements for onsite operation and monitoring of the dam.

5.1 Dam Structure

The proposed structure for Ralph Hall Dam will consist of an earth-filled embankment with an impervious core. A conceptual drawing showing a typical section of the dam structure is contained in Appendix B as Figure B-1. As shown, the upstream face of the embankment will be constructed with a 3:1 slope (horizontal-to-vertical) and will be protected from wave erosion with a rock riprap blanket. The downstream face will be constructed with a 4:1 slope to improve stability and to facilitate maintenance and mowing activities. The overall top width of the embankment will be 20 feet. Internal drains will be provided to remove any seepage that may accumulate within the downstream slope of the embankment.

5.2 Principal Spillway

As noted earlier, the existing river channel at and below the proposed dam site has the capacity to fully contain and convey the 100-year flood flow. Hence, it is desirable to align the principal spillway with the existing channel of the river in order to be able to discharge outflows from the dam for all flood events up to and including the 100-year flood directly into the existing river channel. This type of spillway configuration has been investigated, and it has been determined that a simple uncontrolled linear ogee crest cannot be used because of the significant crest length required to pass flows on the order of the maximum 100-year flood flow within the maximum head limitations imposed by the design criteria. For example, to pass 30,000 cfs with a maximum head of 4.0 feet requires a crest length on the order of 1,000 feet, which is too long for an effective flow transition from the spillway to the 300-foot wide existing river channel. To align the principal spillway with the river channel requires a spillway width that is generally consistent with the width of the existing channel.

Recognizing this limitation, an alternative design involving the use of a labyrinth weir has been investigated for the principal spillway. For analyzing this type of weir, the design procedures and criteria developed by Tullis, et al[3] at Utah State University have been applied. Numerous combinations of the parameters defining the shape, height, width and number of cycles for trapezoidal labyrinth weirs have been analyzed, and a final design has been adopted that satisfies the specific Tullis design criteria for these types of weirs, as well as the specific design criteria for Ralph Hall Dam. The spreadsheet calculations summarizing the design analyses for the adopted labyrinth weir configuration are presented in Table 5-1. This design provides for a five-cycle trapezoidal labyrinth weir with a cycle width of 60 feet (total spillway width of 300 feet), a weir depth of 70 feet (perpendicular to the axis of the dam), and a wall height for the weir of 10 feet (above a flat approach apron). The total crest length of this labyrinth weir is 827 feet, with the crest of the weir set at elevation 552.0 feet msl, one foot above the top of the conservation pool of Lake Ralph Hall. This one foot of depth in the reservoir provides approximately 7,000 acre-feet of detention storage capacity that is effective in reducing the peak outflow from the reservoir for the 100-year design flood, which, in turn, reduces the required length and discharge capacity of the principal spillway. As indicated by the discharge rating in Table 5-1, the outflow ranges up to almost 45,000 cfs with 8.0 feet of head, i.e., 560.0 feet msl reservoir level.

The discharge rating curve for the principal spillway in Table 5-1 has been incorporated into the reservoir conditions HEC-1 model, and the model has been operated to simulate the behavior of the reservoir and spillway for the 100-year flood. The simulated outflow hydrograph for the 100-year flood is plotted on the graph in Figure 5-1, along with the corresponding hydrograph from the existing conditions HEC-1 model. As shown, the detention storage effects of the reservoir, particularly with the crest of the principal spillway set one foot above the top of the conservation pool, are significant and result in the 100-year peak flow at the dam site being reduced from 36,312 cfs under existing conditions down to only 7,993 cfs with the dam and spillway in place. It is likely that this substantial reduction in peak flood flows downstream of the dam will significantly reduce the potential for erosion of the river channel. For the 100-year flood, the average velocity in the river channel as simulated with the HEC-RAS model is reduced from approximately 6.0 feet per second ("fps") downs to about 4.0 fps as a result of the reservoir.

With the adopted principal spillway in place, the water surface of the reservoir as simulated with the HEC-1 model for the 100-year flood temporarily rises approximately 3.1 feet above the top of the conservation pool to elevation 554.1 feet msl. In accordance with the design criteria for the dam, this is the elevation that has been used to establish the elevation of the crest of the emergency spillway. With this configuration, all reservoir inflows associated with flood

[3] Tullis, J. P., N. Amanian and D. Waldron; "Design of Labyrinth Spillways"; American Society of Civil Engineers, Journal of Hydraulic Engineering, Vol. 121, No. 3; March, 1995.

TABLE 5-1
DESIGN CALCULATIONS FOR DEVELOPING DISCHARGE RATING FOR TRAPEZOIDAL LABYRINTH WEIR

RESERVOIR WATER SURFACE ELEV. feet msl	H U/S HEAD feet	a APEX HALF LENGTH feet	W CYCLE WIDTH feet	2a/W APEX RATIO < 0.08	B SIDE-WALL LENGTH feet	S DEPTH OF WEIR feet	L CYCLE CREST LENGTH feet	M =L/W RATIO 2 ? M ? 10	α WALL ANGLE > 6° degrees	Ld DISTURB LENGTH feet	Ld/B DISTURB LENGTH RATIO ? 0.3	P U/S WALL HEIGHT feet	H/P RATIO < 0.9	W/P RATIO > 2.5	Ct DISCHG COEF α (23.3)	Ct DISCH COEF α (90°)	ε EFFI-CACY	Q DISCHG PER CYCLE cfs	N NO. OF CYCLES	LT TOTAL CREST LENGTH feet	TOTAL DISCHG FOR WEIR cfs
552.0	0.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	0.0	0.00	10	0	6.0	0.49	0.49	2.76	0	5	827	0
552.5	0.5	4	60	0.13	74.7	70.0	165.4	2.8	17.4	1.2	0.02	10	0.05	6.0	0.54	0.56	2.69	170	5	827	850
553.0	1.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	2.5	0.03	10	0.1	6.0	0.58	0.61	2.61	513	5	827	2,565
554.0	2.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	4.9	0.07	10	0.2	6.0	0.61	0.69	2.44	1,532	5	827	7,658
554.1	2.1	4	60	0.13	74.7	70.0	165.4	2.8	17.4	5.2	0.07	10	0.21	6.0	0.61	0.70	2.42	1,650	5	827	8,251
554.5	2.5	4	60	0.13	74.7	70.0	165.4	2.8	17.4	6.2	0.08	10	0.25	6.0	0.61	0.72	2.35	2,143	5	827	10,716
555.0	3.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	7.4	0.10	10	0.3	6.0	0.61	0.74	2.27	2,787	5	827	13,936
556.0	4.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	9.9	0.13	10	0.4	6.0	0.58	0.76	2.11	4,096	5	827	20,478
557.0	5.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	12.3	0.16	10	0.5	6.0	0.54	0.76	1.96	5,358	5	827	26,792
558.0	6.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	14.8	0.20	10	0.6	6.0	0.50	0.76	1.84	6,560	5	827	32,799
559.0	7.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	17.2	0.23	10	0.7	6.0	0.47	0.75	1.73	7,741	5	827	38,707
560.0	8.0	4	60	0.13	74.7	70.0	165.4	2.8	17.4	19.7	0.26	10	0.8	6.0	0.45	0.76	1.63	8,959	5	827	44,794

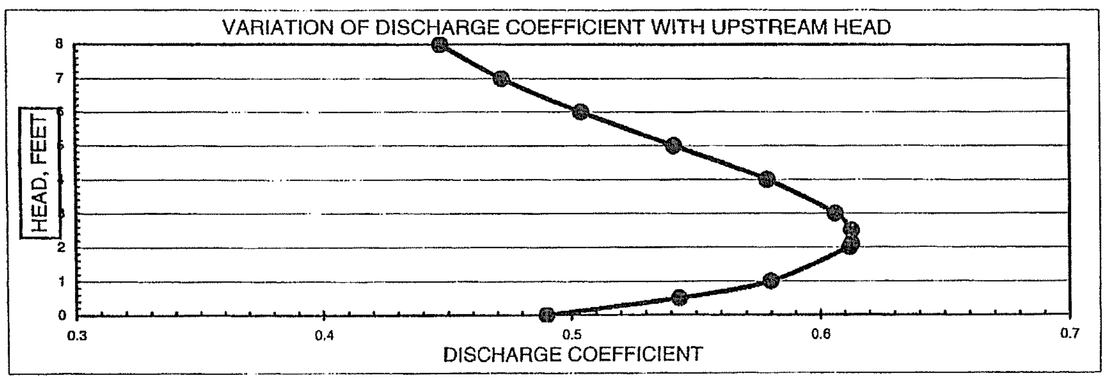
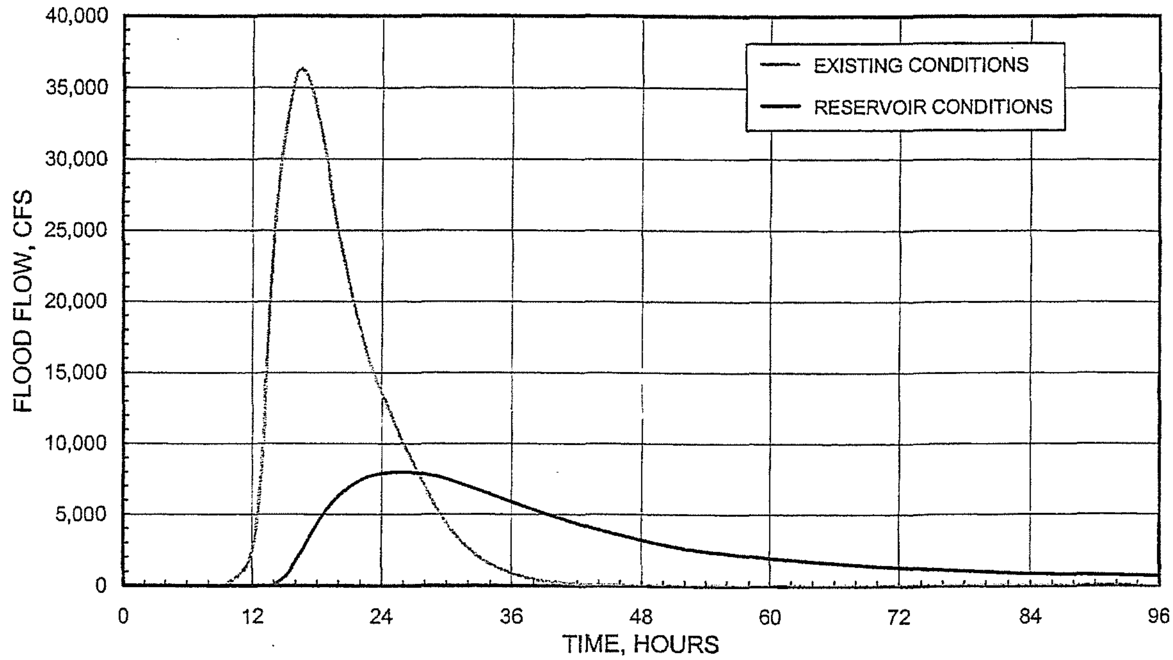


FIGURE 5-1
COMPARISON OF OUTFLOW HYDROGRAPH FROM PRINCIPAL SPILLWAY WITH EXISTING RIVER HYDROGRAPH FOR 100-YEAR FLOOD



magnitudes up to and including the 100-year flood will be passed through the reservoir and discharged entirely through the principal spillway. Reservoir inflows from larger storms will be discharged through both the principal spillway and the emergency spillway.

Conceptual drawings showing the primary features and general dimensions of the principal spillway are included in Appendix B. The plan view in Figure B-2 shows a segment of the dam embankment, the five-cycle trapezoidal labyrinth weir that serves as the primary flow control structure, the discharge chute that provides the transition section between the weir and the stilling basin, and the stilling basin (U.S. Bureau of Reclamation Type II) where energy associated with the high velocity chute flows is dissipated prior to the flows being discharged downstream. As shown, approximately 400 feet of rock riprap is provided downstream of the stilling basin to protect the natural river channel. A cross section view of these same features is presented on the drawing in Figure B-3.

5.3 Service Spillway

As shown on the plan view of the principal spillway in Figure B-2 in Appendix B, the crest of the downstream end of the center cycle of the labyrinth weir is to be lowered one foot to elevation 551.0 feet msl (the top of the conservation pool) to provide a service spillway for the dam. This service spillway section is to have a total length of 80.0 feet (36.0 feet on each wall of the central weir plus 8.0 feet at the end of the central weir). With normal inflows to the reservoir, the service spillway will limit the normal maximum level of the reservoir to the top of the conservation pool. Simulations with the reservoir conditions HEC-1 model with this service spillway included indicate that the spillway, with one foot of head, will be able to pass the peak flow for approximately the two-year flood event.

5.4 Low-Flow Outlet

A low-flow pipe outlet with a gate tower is to be installed as part of the dam to provide a means for passing river flows through the reservoir when the normal overflows through the service spillway are not sufficient to satisfy downstream flow requirements. This pipe outlet will have the capacity to discharge sufficient flow as may be required to satisfy downstream minimum environmental flows and/or flows for downstream senior water rights, and it will discharge directly into the stilling basin below the principal spillway to allow the flows to pass downstream in the river. A separate pipe with a control valve may be incorporated into the low-flow pipe outlet to provide a mechanism for passing reservoir inflows into an abandoned segment of the old river channel immediately downstream of the dam that is being considered for restoration as part of the Project for environmental mitigation purposes.

5.5 Emergency Spillway

The emergency spillway for the dam has been designed to provide the additional outflow capacity, above that provided by the principal spillway, necessary to safely pass the PMF without causing the maximum level of the reservoir to exceed elevation 560.0 feet msl. The adopted design consists of a concrete ogee spillway within the northern embankment of the dam with a crest elevation of 554.1 feet msl, i.e., the 100-year flood level. To pass the PMF, the required length of the ogee crest of the emergency spillway has been determined to be 1,550 feet. The calculations for the discharge rating of this spillway are summarized in Table 5-2.

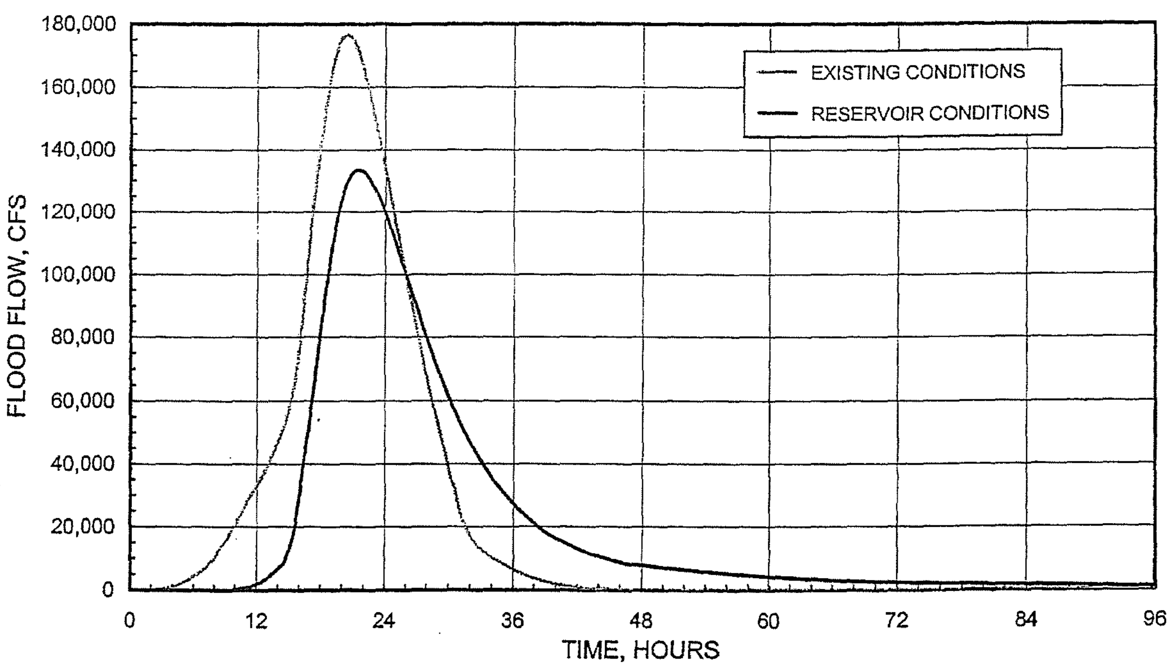
TABLE 5-2
DISCHARGE RATING CALCULATIONS FOR EMERGENCY SPILLWAY

RESERVOIR WATER SURFACE ELEVATION feet msl	HEAD ABOVE SPILLWAY CREST feet	HEAD-TO MAXIMUM HEAD RATIO	OGEE DISCHARGE COEF.	LENGTH OF WEIR feet	DISCHARGE OVER WEIR cfs
554.1	0.0	0.00	3.00	1,550	0
554.5	0.4	0.07	3.00	1,550	1,176
555.0	0.9	0.15	3.05	1,550	4,036
556.0	1.9	0.32	3.37	1,550	13,680
557.0	2.9	0.49	3.59	1,550	27,480
558.0	3.9	0.66	3.77	1,550	45,006
559.0	4.9	0.83	3.90	1,550	65,568
560.0	5.9	1.00	4.00	1,550	88,853

With this emergency spillway incorporated into the reservoir conditions HEC-1 model, along with the principal spillway, the PMF has been simulated and routed through the reservoir and spillway system. The resulting outflow hydrograph is plotted on the graph in Figure 5-2 along with the corresponding hydrograph at the dam site for existing conditions without the reservoir in place. As shown, the combined effects of the outflow control provided by the principal and emergency spillways and the corresponding detention storage provided by the reservoir cause the peak flow of the PMF to be reduced from 176,482 cfs under existing conditions down to 133,571 cfs with Lake Ralph Hall in place. By design, the maximum water surface elevation of the reservoir during passage of the PMF as simulated with the HEC-1 model is 560.0 feet msl, which is 2.0 feet below the proposed top of the dam.

The location and general layout of the emergency spillway within the northern embankment of the dam are shown on the drawing in Figure B-4 in Appendix B. As indicated, the spillway is located where natural ground elevations are not substantially lower than the top of the embankment, thus minimizing the spillway height and stilling basin requirements. Training berms are to be constructed downstream of the spillway to direct floodwaters discharged from the spillway toward the existing river channel. Some grading of the area within the training berms may be required to provide a more uniform flow transition to the river channel, and this grading will be finalized as part of the development of the material balance for the Project. Details and dimensions of the various features of the emergency spillway are shown on the plan view drawing in Figure B-5 and the section view drawing in Figure B-6. As with the principal spillway, the emergency spillway includes a stilling basin (U.S. Bureau of Reclamation Type III) immediately below the ogee weir and rock riprap for 150 feet downstream of the stilling basin to protect the natural ground from erosion by the spillway discharges. Figure B-7 shows plan and profile views of the entire embankment of the dam with the different spillways identified.

FIGURE 5-2
COMPARISON OF OUTFLOW HYDROGRAPH FROM RALPH HALL DAM
WITH EXISTING RIVER HYDROGRAPH FOR PROBABLE MAXIMUM FLOOD



APPENDIX A

TIME OF CONCENTRATION CALCULATIONS FOR LAKE RALPH HALL SUB-BASINS USING SCS TR-55 METHODS

TIME OF CONCENTRATION CALCULATIONS BASED ON SCS TR55 METHOD

(adjusted based on experience and engineering judgement)

PROJECT: **Lake Ralph Hall** WATERSHED: **1 - Western Drainage Area**

SUMMARY OF TIME OF CONCENTRATION CALCULATIONS

SHEET FLOW	0.75	hours
SHALLOW FLOW	1.03	hours
CHANNEL FLOW	3.56	hours
RESERVOIR	0.26	hours
Total Tc	5.34	hours
SCS LAG TIME	3.20	hours

SHEET FLOW:

MANNING'S "N" CALCULATION

Undeveloped Land Use	n value	% Land use	Inc n
Conc.,gravvel,asphalt,bare soil	0.011	**0**	0.000
Grass Short Prairie	0.150	**10**	0.015
Grass dense	0.240	**40**	0.096
Grass bermuda	0.410	**10**	0.041
Woods Light Underbrush	0.400	**30**	0.120
Woods Dense Underbrush	0.800	**10**	0.080
		100	0.352

COMPUTED WEIGHTED "N" VALUE	0.352		
FLOW PATH LENGTH	**300**	feet	MAX 300'
2 YR 24 HOUR PRECIP	**4.1**	inches	FROM TP40
SLOPE	**0.01000**	feet/foot	
COMPUTED TRAVEL TIME	0.91	hours	USE **0.75**

SHALLOW CONCENTRATED FLOW:

1=PAVED; 2=UNPAVED	**2**	
FLOW PATH LENGTH	**5,300**	feet
SLOPE	**0.00792**	feet/foot
VELOCITY FROM FIGURE 3.1=	1.4	feet/sec
COMPUTED TRAVEL TIME	1.03	

PROJECT: **Lake Ralph Hall** WATERSHED: **1 - Western Drainage Area**

CHANNEL FLOW:

REACH 1

BOTTOM WIDTH	**20**	feet
DEPTH	**3**	feet
TOPWIDTH	**35**	feet
CALCULATED SIDE SLOPE (X:1)	2.50	feet/foot
CALCULATED CROSS-SECTION ARE	82.5	sq. feet
CALCULATED WETTED PERIMETER	36.2	feet
CHANNEL SLOPE	**0.00617**	feet/foot
MANNINGS N	**0.085**	
COMPUTED VELOCITY	2.4	feet/sec
CHANNEL LENGTH	**8,100**	feet
COMPUTED TRAVEL TIME	0.94	hours

REACH 2

BOTTOM WIDTH	**30**	feet
DEPTH	**4**	feet
TOPWIDTH	**50**	feet
CALCULATED SIDE SLOPE (X:1)	2.50	feet/foot
CALCULATED CROSS-SECTION ARE	160	sq. feet
CALCULATED WETTED PERIMETER	51.5	feet
CHANNEL SLOPE	**0.00495**	feet/foot
MANNINGS N	**0.080**	
COMPUTED VELOCITY	2.8	feet/sec
CHANNEL LENGTH	**10,100**	feet
COMPUTED TRAVEL TIME	1.01	hours

REACH 3

BOTTOM WIDTH	**50**	feet
DEPTH	**5**	feet
TOPWIDTH	**60**	feet
CALCULATED SIDE SLOPE (X:1)	1.00	feet/foot
CALCULATED CROSS-SECTION ARE	275	sq. feet
CALCULATED WETTED PERIMETER	64.1	feet
CHANNEL SLOPE	**0.00370**	feet/foot
MANNINGS N	**0.075**	
COMPUTED VELOCITY	3.2	feet/sec
CHANNEL LENGTH	**8,100**	feet
COMPUTED TRAVEL TIME	0.71	hours

PROJECT:	**Lake Ralph Hall**	WATERSHED:	**1 - Western Drainage Area**

REACH 4

HEC-RAS COMPUTED VELOCITY	**4.3**	feet/sec	(HEC-RAS, Q=120,000 (
CHANNEL LENGTH	**14,000**	feet	
COMPUTED TRAVEL TIME	0.90	hours	

RESERVOIR:

AVE. RESERVOIR DEPTH OVER REA(	**45**	feet	~ (550-465)/2
GRAVITY ACCELERATION	**32.2**	feet/sec-sec	
WAVE CELERITY	38.1	feet/sec	c =(g*Davg)^0.5
LENGTH	**35,000**	feet	
COMPUTED TRAVEL TIME	0.26	hours	

TIME OF CONCENTRATION CALCULATIONS BASED ON SCS TR55 METHOD

(adjusted based on experience and engineering judgement)

PROJECT: **Lake Ralph Hall** WATERSHED: **2 - Southern Drainage Area**

SUMMARY OF TIME OF CONCENTRATION CALCULATIONS

SHEET FLOW	0.75	hours
SHALLOW FLOW	0.17	hours
CHANNEL FLOW	0.58	hours
RESERVOIR	0.10	hours
Total Tc	1.50	hours
SCS LAG TIME	0.90	hours

SHEET FLOW:

MANNING'S "N" CALCULATION

Undeveloped Land Use	n value	% Land use	Inc n
Conc.,gravvel,asphalt,bare soil	0.011	**0**	0.000
Grass Short Prairie	0.150	**10**	0.015
Grass dense	0.240	**40**	0.096
Grass bermuda	0.410	**10**	0.041
Woods Light Underbrush	0.400	**30**	0.120
Woods Dense Underbrush	0.800	**10**	0.080
		100	0.352

COMPUTED WEIGHTED "N" VALUE	0.352		
FLOW PATH LENGTH	**300**	feet	MAX 300'
2 YR 24 HOUR PRECIP	**4.1**	inches	FROM TP40
SLOPE	**0.00500**	feet/foot	
COMPUTED TRAVEL TIME	1.20	hours USE:	**0.75**

SHALLOW CONCENTRATED FLOW:

1=PAVED; 2=UNPAVED	**2**	
FLOW PATH LENGTH	**1,400**	feet
SLOPE	**0.02140**	feet/foot
VELOCITY FROM FIGURE 3.1=	2.4	feet/sec
COMPUTED TRAVEL TIME	0.17	

PROJECT: **Lake Ralph Hall** WATERSHED: **2 - Southern Drainage Area**

CHANNEL FLOW:

REACH 1

BOTTOM WIDTH	**20**	feet
DEPTH	**3**	feet
TOPWIDTH	**35**	feet
CALCULATED SIDE SLOPE (X:1)	2.50	feet/foot
CALCULATED CROSS-SECTION AREA	83	sq. feet
CALCULATED WETTED PERIMETER	36.2	feet
CHANNEL SLOPE	**0.0161**	feet/foot
MANNINGS N	**0.075**	
COMPUTED VELOCITY	4.4	feet/sec
CHANNEL LENGTH	**6,100**	feet
COMPUTED TRAVEL TIME	0.39	hours

REACH 2

BOTTOM WIDTH	**70**	feet
DEPTH	**8**	feet
TOPWIDTH	**100**	feet
CALCULATED SIDE SLOPE (X:1)	1.88	feet/foot
CALCULATED CROSS-SECTION AREA	680	sq. feet
CALCULATED WETTED PERIMETER	104.0	feet
CHANNEL SLOPE	**0.00500**	feet/foot
MANNINGS N	**0.065**	
COMPUTED VELOCITY	5.7	feet/sec
CHANNEL LENGTH	**4,000**	feet
COMPUTED TRAVEL TIME	0.20	hours

RESERVOIR:

AVE. RESERVOIR DEPTH	**55**	feet	
GRAVITY ACCELERATION	**32.2**	feet/sec-sec	
WAVE CELERITY	42.1	feet/sec	c =(g*Davg)^0.5
LENGTH	**15,000**	feet	
COMPUTED TRAVEL TIME	0.10	hours	

TIME OF CONCENTRATION CALCULATIONS BASED ON SCS TR55 METHOD

(adjusted based on experience and engineering judgement)

PROJECT: **Lake Ralph Hall** WATERSHED: **3 - Northern Drainage Area**

SUMMARY OF TIME OF CONCENTRATION CALCULATIONS

SHEET FLOW	0.75	hours
SHALLOW FLOW	0.35	hours
CHANNEL FLOW	2.99	hours
RESERVOIR	0.10	hours
Total Tc	4.09	hours
SCS LAG TIME	2.45	hours

SHEET FLOW:

MANNING'S "N" CALCULATION

Undeveloped Land Use	n value	% Land use	Inc n
Conc.,gravvel,asphalt,bare soil	0.011	**0**	0.000
Grass Short Prairie	0.150	**10**	0.015
Grass dense	0.240	**40**	0.096
Grass bermuda	0.410	**10**	0.041
Woods Light Underbrush	0.400	**30**	0.120
Woods Dense Underbrush	0.800	**10**	0.080
		100	0.352

COMPUTED WEIGHTED "N" VALUE	0.352			
FLOW PATH LENGTH	**300**	feet		MAX 300'
2 YR 24 HOUR PRECIP	**4.1**	inches		FROM TP40
SLOPE	**0.01670**	feet/foot		
COMPUTED TRAVEL TIME	0.74	hours	USE:	**0.75**

SHALLOW CONCENTRATED FLOW:

1=PAVED; 2=UNPAVED	**2**	
FLOW PATH LENGTH	**2,000**	feet
SLOPE	**0.01000**	feet/foot
VELOCITY FROM FIGURE 3.1=	1.6	feet/sec
COMPUTED TRAVEL TIME	0.35	

PROJECT: **Lake Ralph Hall** WATERSHED: **3 - Northern Drainage Area**

CHANNEL FLOW:

REACH 1

BOTTOM WIDTH	**35**	feet
DEPTH	**4**	feet
TOPWIDTH	**45**	feet
CALCULATED SIDE SLOPE (X:1)	1.25	feet/foot
CALCULATED CROSS-SECTION AREA	160	sq. feet
CALCULATED WETTED PERIMETER	47.8	feet
CHANNEL SLOPE	**0.0038**	feet/foot
MANNINGS N	**0.075**	
COMPUTED VELOCITY	2.7	feet/sec
CHANNEL LENGTH	**18,300**	feet
COMPUTED TRAVEL TIME	1.86	hours

REACH 2

BOTTOM WIDTH	**45**	feet
DEPTH	**5**	feet
TOPWIDTH	**55**	feet
CALCULATED SIDE SLOPE (X:1)	1.00	feet/foot
CALCULATED CROSS-SECTION AREA	250	sq. feet
CALCULATED WETTED PERIMETER	59.1	feet
CHANNEL SLOPE	**0.00448**	feet/foot
MANNINGS N	**0.065**	
COMPUTED VELOCITY	4.0	feet/sec
CHANNEL LENGTH	**16,400**	feet
COMPUTED TRAVEL TIME	1.14	hours

RESERVOIR:

AVE. RESERVOIR DEPTH	**55**	feet	
GRAVITY ACCELERATION	**32.2**	feet/sec-sec	
WAVE CELERITY	42.1	feet/sec	c =(g*Davg)^0.5
LENGTH	**15,000**	feet	
COMPUTED TRAVEL TIME	0.10	hours	

TIME OF CONCENTRATION CALCULATIONS BASED ON HEC-RAS VELOCITIES

(adjusted based on experience and engineering judgement)

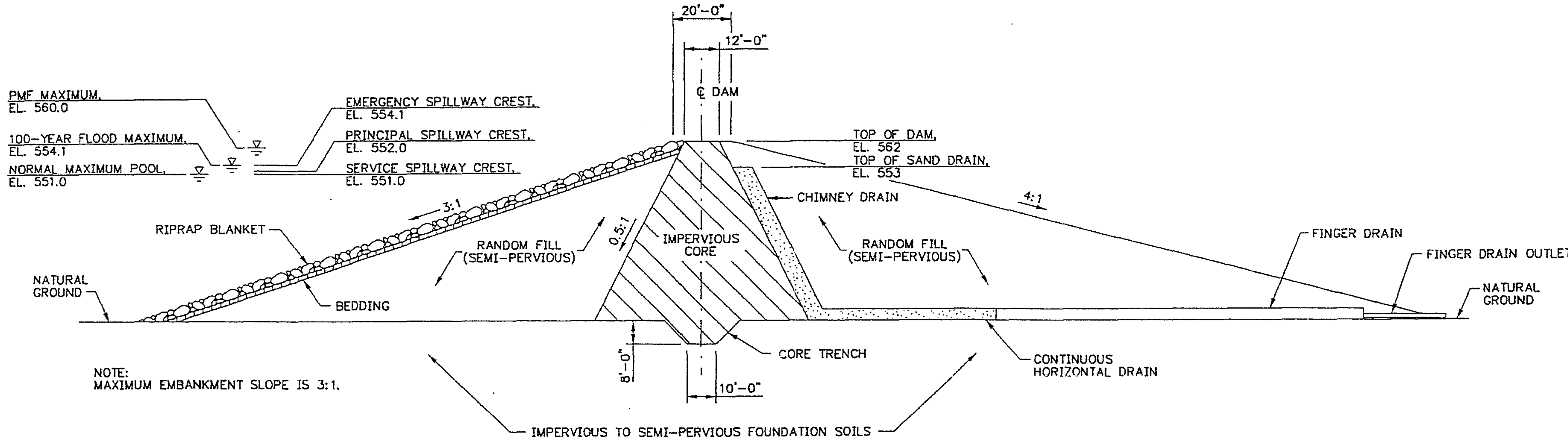
PROJECT: **Lake Ralph Hall** WATERSHED: **4 - Reservoir**

RESERVOIR:

AVE. RESERVOIR DEPTH OVER REACH	**45**	feet	~ (550-465)/2
GRAVITY ACCELERATION	**32.2**	feet/sec-sec	
WAVE CELERITY	38.1	feet/sec	c =(g*Davg)^0.5
LENGTH	**35,000**	feet	
COMPUTED TRAVEL TIME	0.26	hours	

APPENDIX B

CONCEPTUAL DRAWINGS OF RALPH HALL DAM AND SPILLWAYS



TYPICAL DAM EMBANKMENT SECTION

0 20 40
SCALE IN FEET

R. J. BRANDES COMPANY

Chiang, Patel & Yerby, Inc.

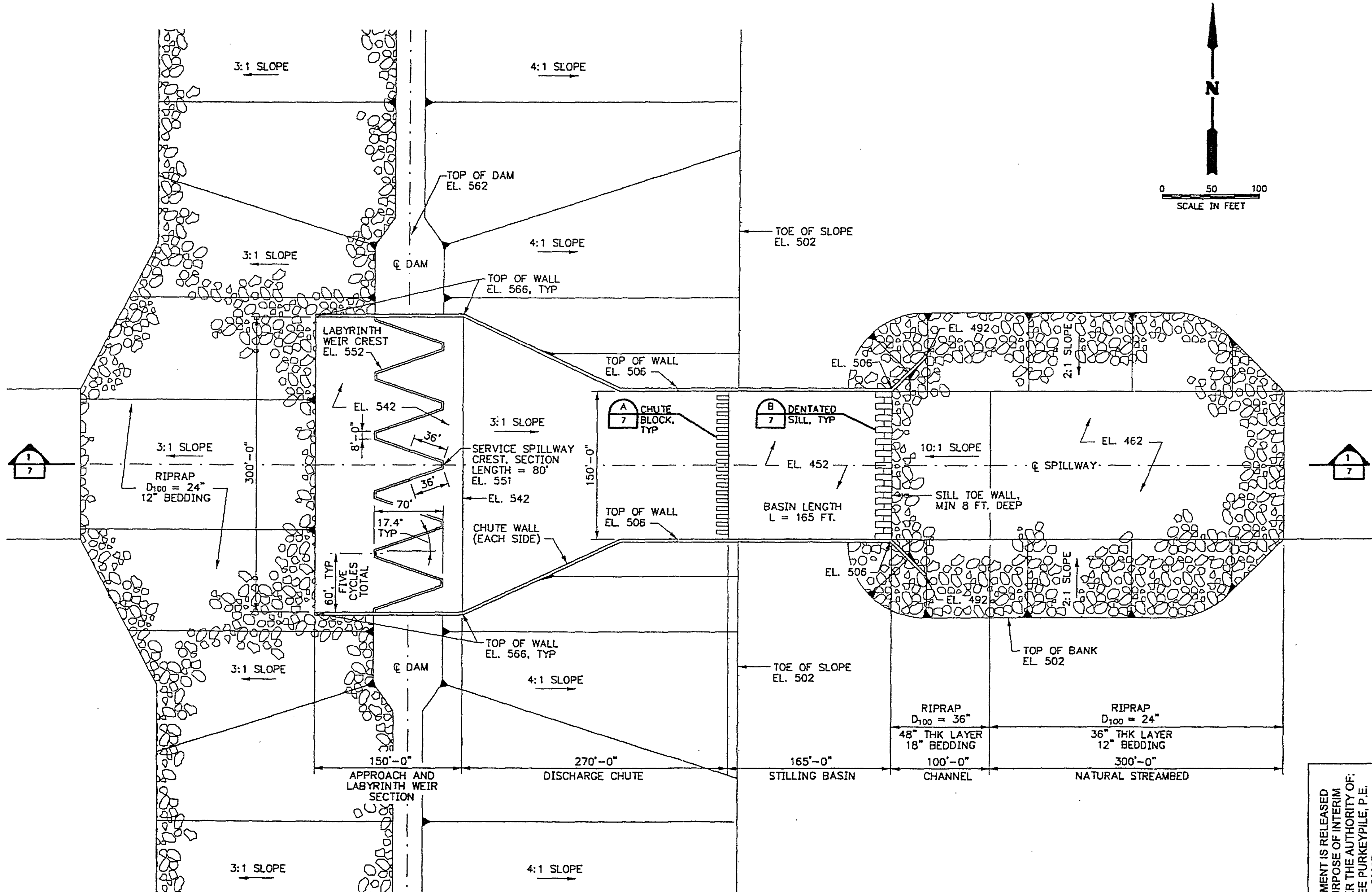
APRIL 29 2004

APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT
WATER RIGHTS PERMIT APPLICATION

TYPICAL DAM EMBANKMENT - SECTION

THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027
R.J. BRANDES COMPANY.
IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

FIGURE
B-1



THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027
R.J. BRANDES COMPANY.
IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

R. J. BRANDES COMPANY

Chiang, Patel & Yerby, Inc.

APRIL 29, 2004

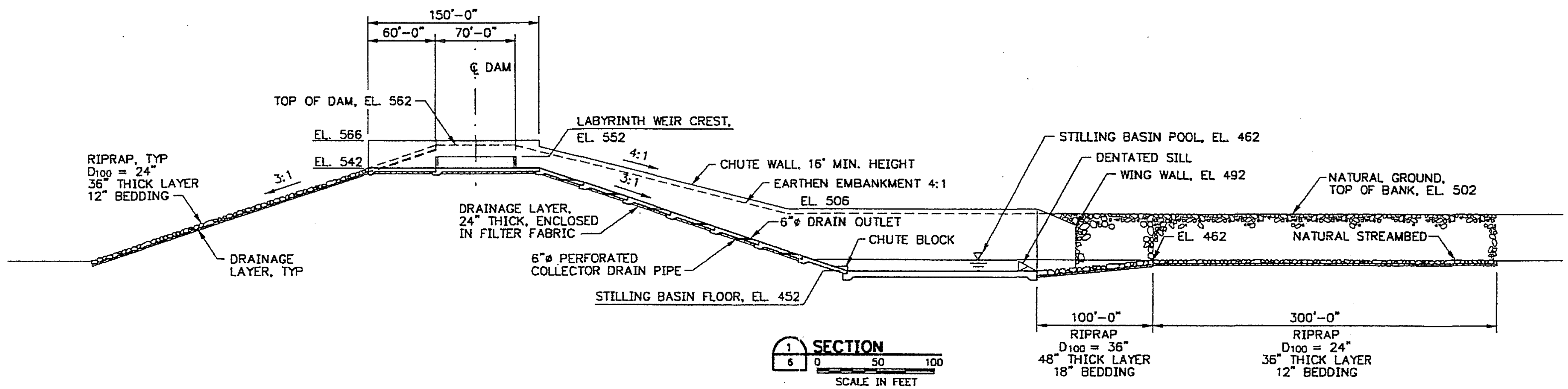
APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT

WATER RIGHTS PERMIT APPLICATION

RESERVOIR PRINCIPAL SPILLWAY - PLAN

FIGURE

B–2

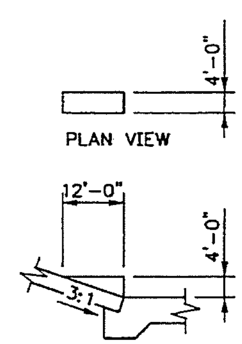


1/6 SECTION

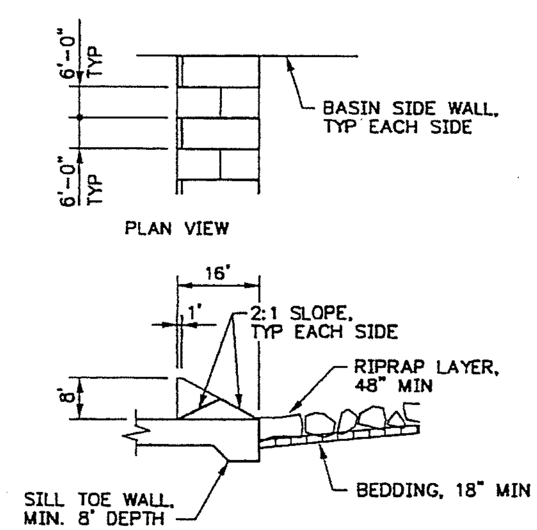
0 50 100
SCALE IN FEET

NOTES:

1. SPACE CHUTE BLOCKS AT 8'-0" O.C. FOR ENTIRE WIDTH OF STILLING BASIN.
2. PROVIDE 5' GAP EACH SIDE OF BASIN.



A/6 CHUTE BLOCK DETAIL
NTS



B/6 DENTATED SILL DETAIL
NTS

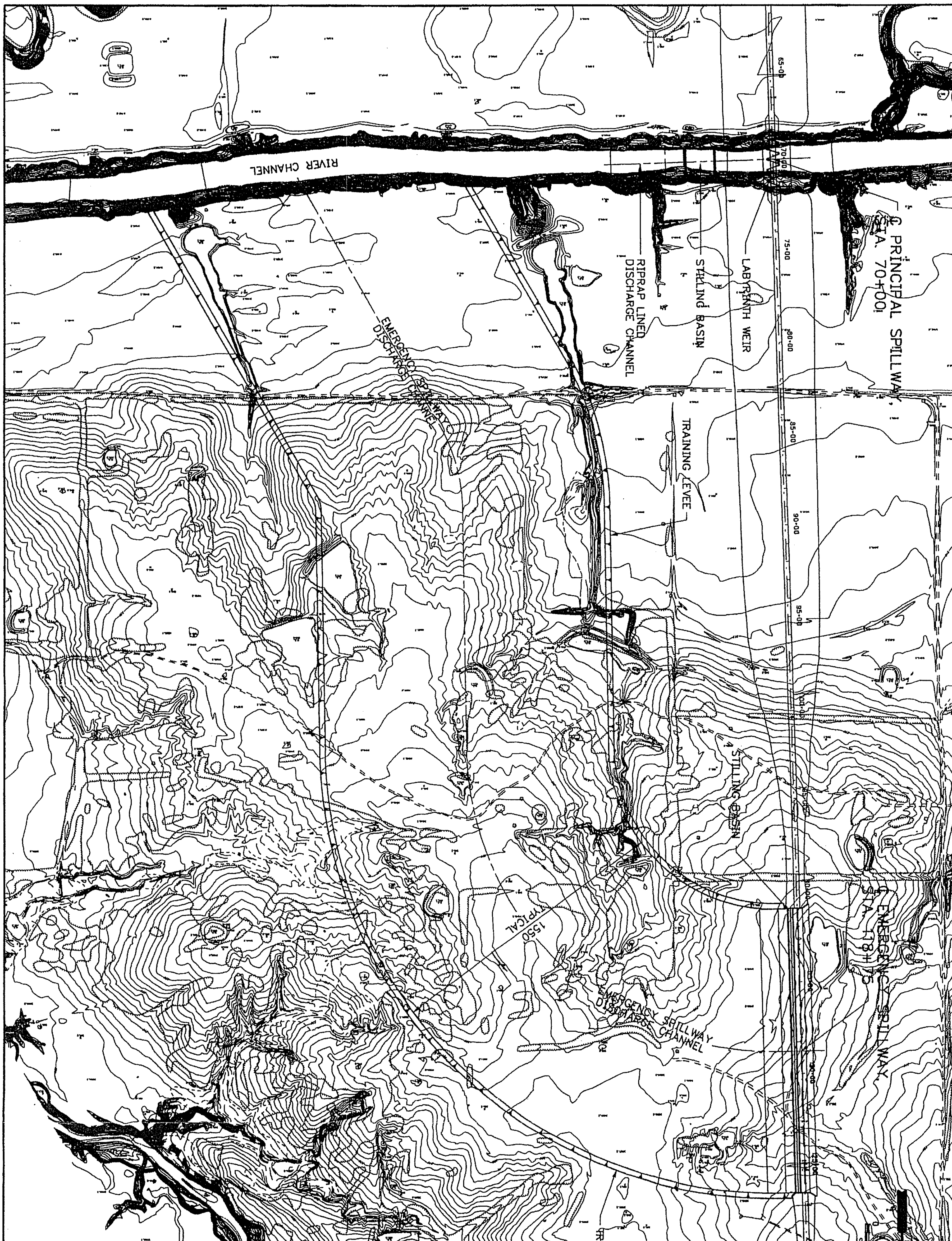
THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027
R.J. BRANDES COMPANY.
IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

R. J. BRANDES COMPANY
Chiang, Patel & Yerby, Inc.
APRIL 29, 2004

APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT
WATER RIGHTS PERMIT APPLICATION

RESERVOIR PRINCIPAL SPILLWAY - SECTION

FIGURE
B-3



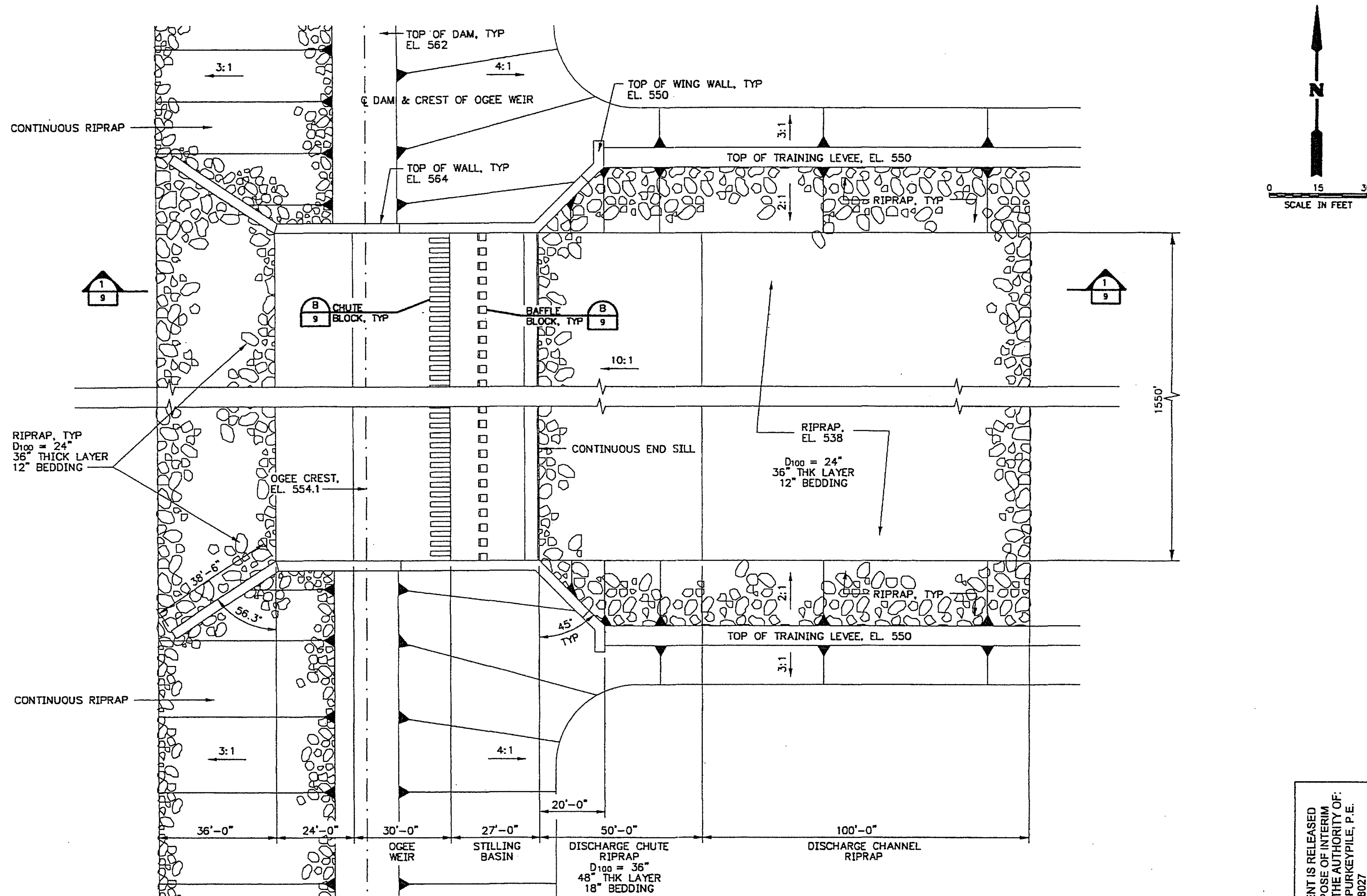
THIS DOCUMENT IS RELEASED
FOR THE PURPOSE OF INTERIM
REVIEW UNDER THE AUTHORITY OF:
RICHARD DEE PURKEYPILE, P.E.
68027
R.J. BRANDES COMPANY.
IT IS NOT TO BE USED FOR BIDDING
OR CONSTRUCTION PURPOSES.

FIGURE B-4

APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT
WATER RIGHTS PERMIT APPLICATION

EMERGENCY SPILLWAY - LOCATION MAP

R. J. BRANDES COMPANY
Chiang, Patel & Yerby, Inc.
APRIL 29, 2004



THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027

R.J. BRANDES COMPANY.

IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

R. J. BRANDES COMPANY

Chiang, Patel & Yerby, Inc.

APRIL 29, 2004

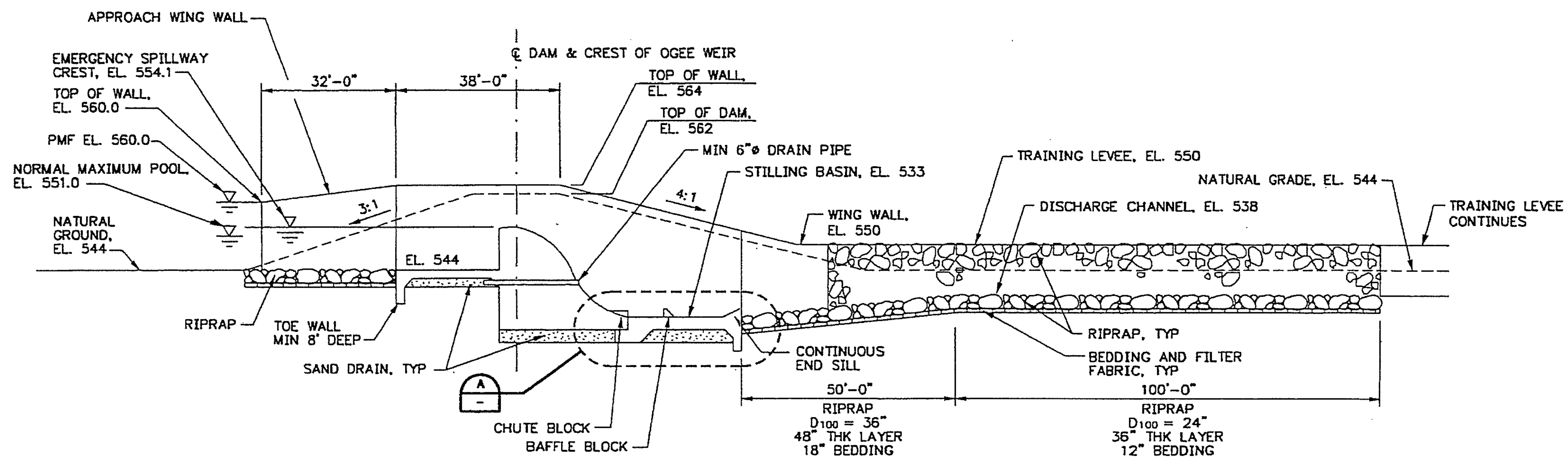
APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT

WATER RIGHTS PERMIT APPLICATION

EMERGENCY SPILLWAY - PLAN

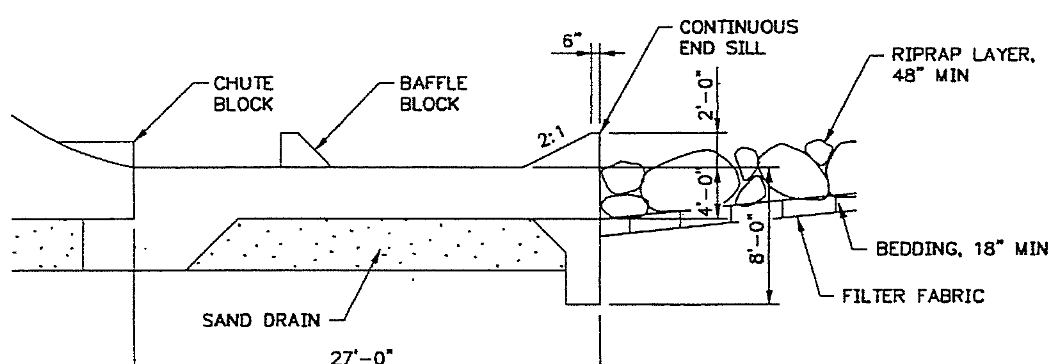
FIGURE

B-5

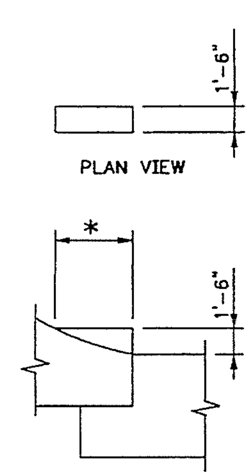


1 / 8 **SECTION**

0 15 30
SCALE IN FEET



A / - **DETAIL**
NTS



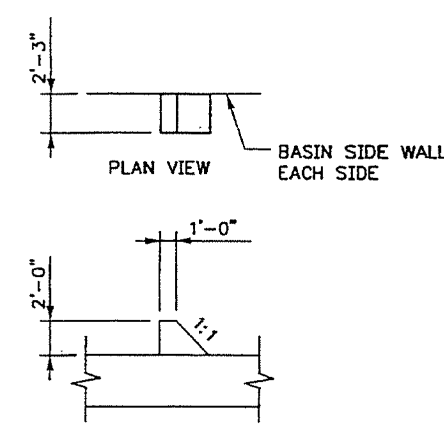
NOTES:

1. * DIMENSION TO BE DETERMINED BY CONTOUR OF WEIR.

2. SPACE CHUTE BLOCKS AT 3'-0" O.C. FOR ENTIRE WIDTH OF STILLING BASIN.

3. PROVIDE 1'-6" GAP EACH SIDE OF BASIN.

B / 8 **CHUTE BLOCK DETAIL**
NTS



NOTE:

SPACE BAFFLE BLOCKS AT 2'-3" O.C. FOR ENTIRE WIDTH OF STILLING BASIN.

C / 8 **BAFFLE BLOCK DETAIL**
NTS

THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027
R.J. BRANDES COMPANY.
IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

R. J. BRANDES COMPANY

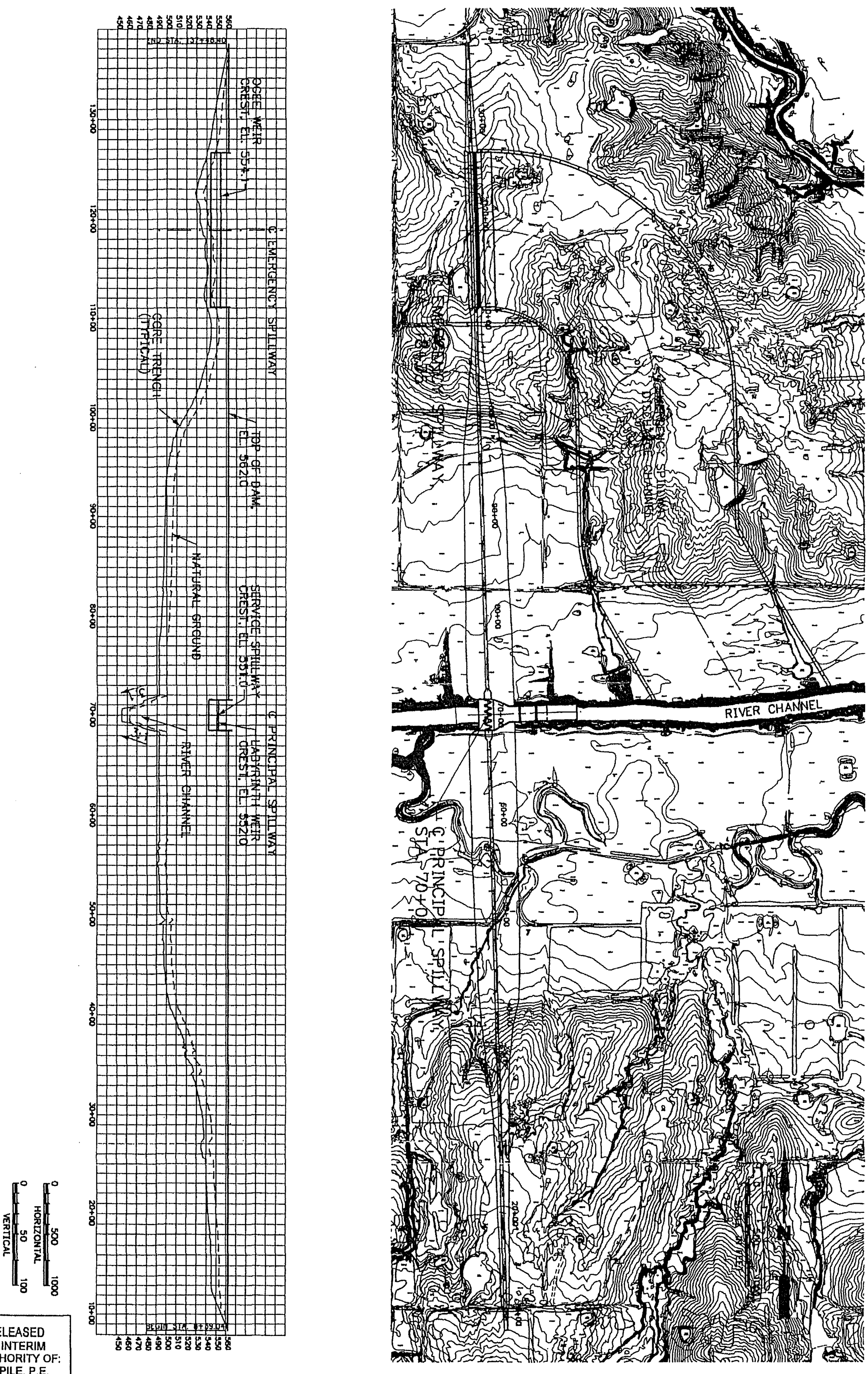
Chiang, Patel & Yerby, Inc.

APRIL 29, 2004

APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT
WATER RIGHTS PERMIT APPLICATION

EMERGENCY SPILLWAY - SECTION & DETAILS

FIGURE
B-6



THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW UNDER THE AUTHORITY OF: RICHARD DEE PURKEYPILE, P.E. 68027 R.J. BRANDES COMPANY. IT IS NOT TO BE USED FOR BIDDING OR CONSTRUCTION PURPOSES.

FIGURE B-7

APPLICANT: UPPER TRINITY REGIOANL WATER DISTRICT
WATER RIGHTS PERMIT APPLICATION

EMBANKMENT PLAN AND PROFILE

R. J. BRANDES COMPANY
Chiang, Patel & Yerby, Inc.
APRIL 29, 2004

D-3: RiverWare Modeling

Robert J. Brandes Consulting

MEMORANDUM

To: Ed Motley
CH2M-Hill

From: Bob Brandes
Kirk Kennedy

Subject: Lake Ralph Hall
RiverWare Modeling

Date: June 29, 2015

As directed by the Upper Trinity Regional Water District (UTRWD), we have responded to the request from Corps of Engineers Fort Worth Office (Corps) to operate the Corps' daily RiverWare model of the Sulphur, Cypress and Red River Basins under conditions without and with the Lake Ralph Hall Project. From the modeling results, we have extracted daily river flows at locations along the North Sulphur and Sulphur Rivers where computational nodes exist in the model, and we have analyzed these flows with regard to frequency of occurrence and the frequency of filling river channel pools along the segment of the North Sulphur River from the proposed Lake Ralph Hall dam site downstream to the USGS streamflow gage near Cooper (Gage No. 07343000). We also have extracted and analyzed the daily storage and diversions for Lake Ralph Hall as simulated with the RiverWare model.

The version of the RiverWare model provided by the Corps included the physical representation of Lake Ralph Hall, but it did not have any diversions specified for withdrawing water from the reservoir as proposed by the UTRWD. We incorporated the same diversion routine that was used in the WAM for the previous analyses of the impacts of Lake Ralph Hall on monthly river flows that were conducted in July of 2014. This routine allows monthly diversions equivalent to 45,000 acre-feet per year to be made from Lake Ralph Hall provided the beginning-of-month storage in the reservoir exceeds 27,500 acre-feet, with the monthly diversions reduced to the equivalent of 16,800 acre-feet per year when the storage falls below 27,500 acre-feet. As originally modeled with the WAM, this operating procedure was designed to protect a firm annual yield of 16,800 acre-feet for Lake Ralph Hall while allowing overdrafting of the reservoir up to the full authorized diversion amount of 45,000 acre-feet per year when adequate stored water is available in the reservoir.

The period of record for the hydrologic conditions simulated with the daily RiverWare model is 1938 through 2014, which encompasses the monthly hydrologic conditions simulated with the WAM that extend from 1940 through 1996. While the source and derivation of the monthly naturalized flows used in the WAM are well documented, we do not have information regarding the procedures used to develop the daily flows that are input into the RiverWare model; however, as will be demonstrated, it is apparent that historical flow data for the North Sulphur River from the USGS streamflow gage near Cooper, to the extent they are available, have been used for representing flow conditions in the RiverWare model for at least the upper segment of the North

6000 Maurys Trail
Austin, Texas 78730
robert.brandes@atkinsglobal.com
Office Phone: 512/342-3233
Mobile Phone: 512/461-1477

Sulphur River. As we have discussed before, the RiverWare model does not apply the prior appropriation doctrine for allocating available streamflows among existing water rights in the Sulphur Basin, so in the model no streamflows are ever required to be passed downstream during water shortage periods by the more junior water rights to satisfy the demands of the more senior water rights. Furthermore, it appears that the only demands associated with existing water rights in the entire Sulphur Basin that are included in the RiverWare model are those for Lake Chapman and Lake Wright Patman; all other water rights are not represented. The WAM includes all existing water rights in the Sulphur Basin, with total authorized diversions of about 500,000 acre-feet per year, and allocates water to these water rights in order of seniority as required under Texas state law; so in the WAM, Lake Ralph Hall, with its relatively junior priority, must pass inflows downstream whenever senior water rights are not fully satisfied. These differences in the models regarding how streamflow allocations are made to existing water rights are reflected in their respective simulated river flows.

Since the WAM uses a monthly time step for performing water availability simulations, the underlying purpose for applying the daily RiverWare model was to be able to evaluate daily flow variations under conditions without and with Lake Ralph Hall. Therefore, the first set of results presented herein consists of plots of simulated daily flows, expressed in cubic feet per second (cfs), at USGS Gage No. 07343000 on the North Sulphur River near Cooper (see Attachment A) and at Gage No. 07343200 on the Sulphur River near Talco (see Attachment B). These depictions of daily flows illustrate conditions on the eroded and degraded segment of the North Sulphur River, as well as on the more natural segment of the Sulphur River below the confluence with the South Sulphur River and also below the infamous log jam. Graphs of daily flows covering one calendar year each are presented for 1956, 1980, 1992 and 2011, with two graphs with maximum flow scales of 500 cfs and 5,000 cfs provided for each year. The selected years are characterized by periods of extremely low flows (1956 and 2011), varying flows (1980), and very high flows (1992). As expected, these plots of daily flows without and with Lake Ralph Hall indicate some reduction in peak flows for individual flood events as a result of the reservoir, with these reductions more pronounced at the upper gage on the North Sulphur River. The peak flow reductions are less pronounced at the lower gage on the Sulphur River, as would be expected with the increased tributary inflows from the intervening watershed. Since the major reductions in peak flows are limited to the eroded and degraded channel of the North Sulphur River where overbanking of adjacent floodplain areas typically does not occur, the impacts of these reduced peak flows are not likely to be significant.

We have also compiled the daily simulated flows from the RiverWare model into monthly values to better provide meaningful comparisons of conditions without and with Lake Ralph Hall and to facilitate comparisons with the results from the WAM. Attachment C contains a group of plots and tables illustrating these comparisons for locations along the North Sulphur and Sulphur Rivers where the RiverWare model has computational nodes.

The first two plots on pages 1 and 2 of Attachment C show the storage in Lake Ralph Hall and the diversions from the reservoir as simulated with the RiverWare model and with the WAM. As illustrated, the simulated storage in the reservoir is considerably greater for the RiverWare model, with substantially more spills from the reservoir downstream into the North Sulphur River. As shown on the graph on page 2, during these higher storage periods, more water is able to be diverted from the reservoir since the criterion for making diversions up to the fully authorized amount of 45,000 acre-feet/year is satisfied more often.

The disparity between the storage results for Lake Ralph Hall from the RiverWare model and the WAM leads to questions as to the source and magnitude of the inflows to the reservoir as simulated with the two models, notwithstanding the fact that the RiverWare model ignores water rights and does not require junior water rights to pass flows to downstream senior water rights during times of water shortage. It is assumed that both models utilize historical flow data from the gage on the North Sulphur River near Cooper as the underlying basis for their specified river flow inputs for this segment of the overall river system network. This has been confirmed by comparing the simulated flows in the river at this gage location without Lake Ralph Hall in operation. As shown on the graph on page 3 of Attachment C, the monthly flow values from the two models at the gage location and the corresponding measured monthly flows at the gage are essentially the same over the common period of the model simulations when the gage was in operation (which began in October 1949). This analysis rules out the possibility that different sources of flow data were used for the upper segment of the North Sulphur River in the two models. However, when this same comparison is made of the simulated inflows to Lake Ralph Hall approximately 20 miles upstream from the gage, differences are noted between the two models. The graph on page 4 of Attachment C indicates that the simulated inflows to Lake Ralph Hall for the RiverWare model generally are higher than those for the WAM. This graph also indicates that apparently different base flows were used in the models prior to the existence of the gage in 1949, possibly due to the application of different data fill-in techniques. The graph on page 5 of Attachment C presents a time-series plot of the cumulative inflows to Lake Ralph Hall as simulated with the two models for the common period when the gage was in operation beginning in 1950, and it further illustrates the differences in these two sets of inflows, with the total cumulative deviation over 50 years approaching about 500,000 acre-feet. The differences in the inflows to Lake Ralph Hall during the period when gage flow records are available may be due to the fact that the RiverWare model uses a daily time step, with various flow routing parameters and lag coefficients to account for the movement of water downstream, whereas the WAM uses a monthly time step with no time adjustments other than those reflected in the flow data themselves. In any event, these differences in the inflows to Lake Ralph Hall between the two models are worthy of note, and they are likely reflected in the simulated flows downstream and must be considered when evaluating results.

A plot of the monthly simulated outflows from Lake Ralph Hall for the two models is presented on the graph on page 6 of Attachment C, again illustrating the significant spills from the reservoir as simulated with the RiverWare model. Inflows periodically passed downstream for satisfying the demands of senior water rights also are indicated on this plot by the WAM flows during dry periods. Monthly flows from the RiverWare model at the location of the first tributary downstream of Lake Ralph Hall (Baker Creek), which enters the North Sulphur River approximately one mile below the dam, are plotted with two different scales on the graphs on pages 7 and 8 of Attachment C for conditions without and with Lake Ralph Hall. Both plots illustrate the obvious; more flow is in the river downstream without Lake Ralph Hall than with it. The graph on page 9 of Attachment C depicts similar results at the location of the gage on the North Sulphur River near Cooper, but it is interesting to compare the flow magnitudes in this graph with those in the graph on page 8, both of which are plotted at the same flow scale. This comparison clearly illustrates the significant effect of flows that enter the river downstream of Lake Ralph Hall from tributaries, even with the reservoir in operation.

Finally, the tables on pages 10 through 13 present statistical results for the simulated monthly flows from the RiverWare model and from the WAM. Flows corresponding to specific percentiles and exceedance frequencies are indicated for the RiverWare model and the WAM and for conditions without and with Lake Ralph Hall in operation. These values are presented at locations where the RiverWare model has computational nodes, plus one additional location at the confluence of Baker Creek with the North Sulphur River. These locations can be identified on the map of the Sulphur River Basin in Attachment D, and they include upstream of Lake Ralph Hall for the inflow to the reservoir, below Lake Ralph Hall immediately downstream of Baker Creek (Catchment 3 on the map), at the North Sulphur River gage near Cooper (Gage No. 07343000 on the map), at the proposed site for the Parkhouse 2 Reservoir on the North Sulphur River (immediately below Catchment 14 on the map), at the Sulphur River gage near Talco (Gage No. 07343200 on the map), and at the proposed site of the Marvin Nichols Reservoir on the Sulphur River (immediately below Catchment 18 on the map). Flows from the RiverWare model at the Baker Creek location have been derived by adding to the simulated outflows from Lake Ralph Hall the incremental inflow from the watershed between the reservoir and Baker Creek, including Baker Creek. This incremental inflow was calculated by applying a drainage area ratio to the total simulated incremental inflow from the watershed between the reservoir and the North Sulphur River gage near Cooper. Comparisons of statistical results are presented for flows from the RiverWare model and from the WAM with Lake Ralph Hall (page 10) and without Lake Ralph Hall (page 11), for flows from the RiverWare model with and without Lake Ralph Hall (page 12), and for flows from the WAM with and without Lake Ralph Hall (page 13). As shown on each table, for flows at the Baker Creek location and at the North Sulphur River gage near Cooper, the exceedance frequencies have been determined for a flow of 175 acre-feet/month, which is the flow volume determined by Dr. Norman Johns of the National Wildlife Foundation as that needed to completely fill all of the downstream pools in the channel of the North Sulphur River from Baker Creek to the gage on the river near Cooper. While these exceedance frequencies provide some insight as to the effects of using the different models and the impacts of Lake Ralph Hall itself, a more in-depth analysis of downstream pool filling is discussed below.

Attachment E presents a summary of the results from the downstream pool filling analyses performed using monthly flows simulated with the RiverWare model by applying the same procedures previously employed (April 2015) for analyzing pool filling with WAM flows at the same locations. These previous results from analyzing the WAM flows also are included at the bottom of this table for reference purposes. This table presents the % of Time Pools Are Filled, on a monthly basis, under conditions without and with Lake Ralph Hall in operation for each of the reaches between tributaries for the segment of the North Sulphur River from the Lake Ralph Hall dam down to the North Sulphur River streamflow gage near Cooper. These values were derived by analyzing the monthly flows as simulated with the RiverWare model and the WAM at each of these locations to determine if they are sufficient to fill the pools in each of the downstream reaches based on Dr. Johns' pool volume estimate of 175 acre-feet for the total dam-to-gage reach. The intervening values of the flow volume required for filling the pools in each of the reaches were derived by making proportional adjustments of the 175 acre-foot value based on river channel distance below the dam. This assumes that the total pool volume is linearly distributed along this segment of the river channel. As shown in the table, and as expected, the values of Volume Required to Fill All Downstream Pools decrease with distance below the Lake Ralph Hall dam since the volume of pools decreases. The value of the % of Time Pools Are Filled at a particular location reflects the use of river flows to fill upstream pools, increases in river flows in the downstream direction with added tributary inflows, and the different pool volumes as they vary by reach. The monthly river flows from the RiverWare model at each of these locations were derived using the same approach described above for determining the river flows at the Baker

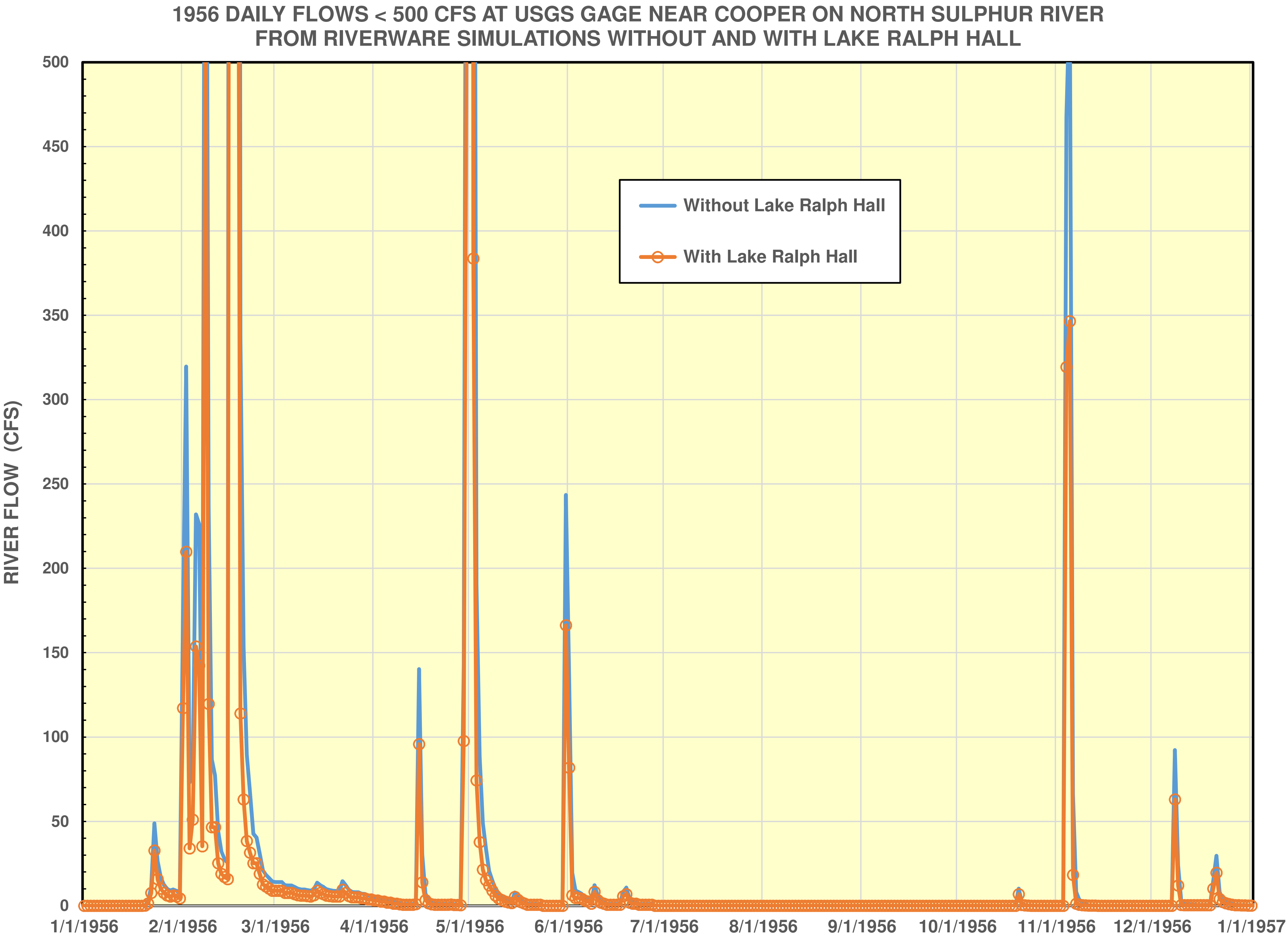
Creek location based on the simulated RiverWare flows at the dam and at the downstream gage near Cooper. As noted, the maximum reduction in the % of Time Pools Are Filled from the Without Lake Ralph Hall case to the With Lake Ralph Hall case for the RiverWare results is 13.5%, with the second largest reduction equal to 9.7%. For the WAM flows, these maximum reductions are 0.6% and 1.3%, respectively. As expected, both of these sets of higher reductions occur in reaches of the river closest to Lake Ralph Hall. Beginning at a point about half way down the river between Lake Ralph Hall and the gage, the reductions are substantially less, generally at levels considered to be within the simulation accuracy of the models considering the sources and accuracy of data and the simulation procedures used in the models. Over the entire segment of the North Sulphur River from Lake Ralph Hall down to the gage, the reach length-weighted average reduction in the % of Time Pools Are Filled from the Without Lake Ralph Hall case to the With Lake Ralph Hall case is -5.9% for the RiverWare flows and -0.5% for WAM flows.

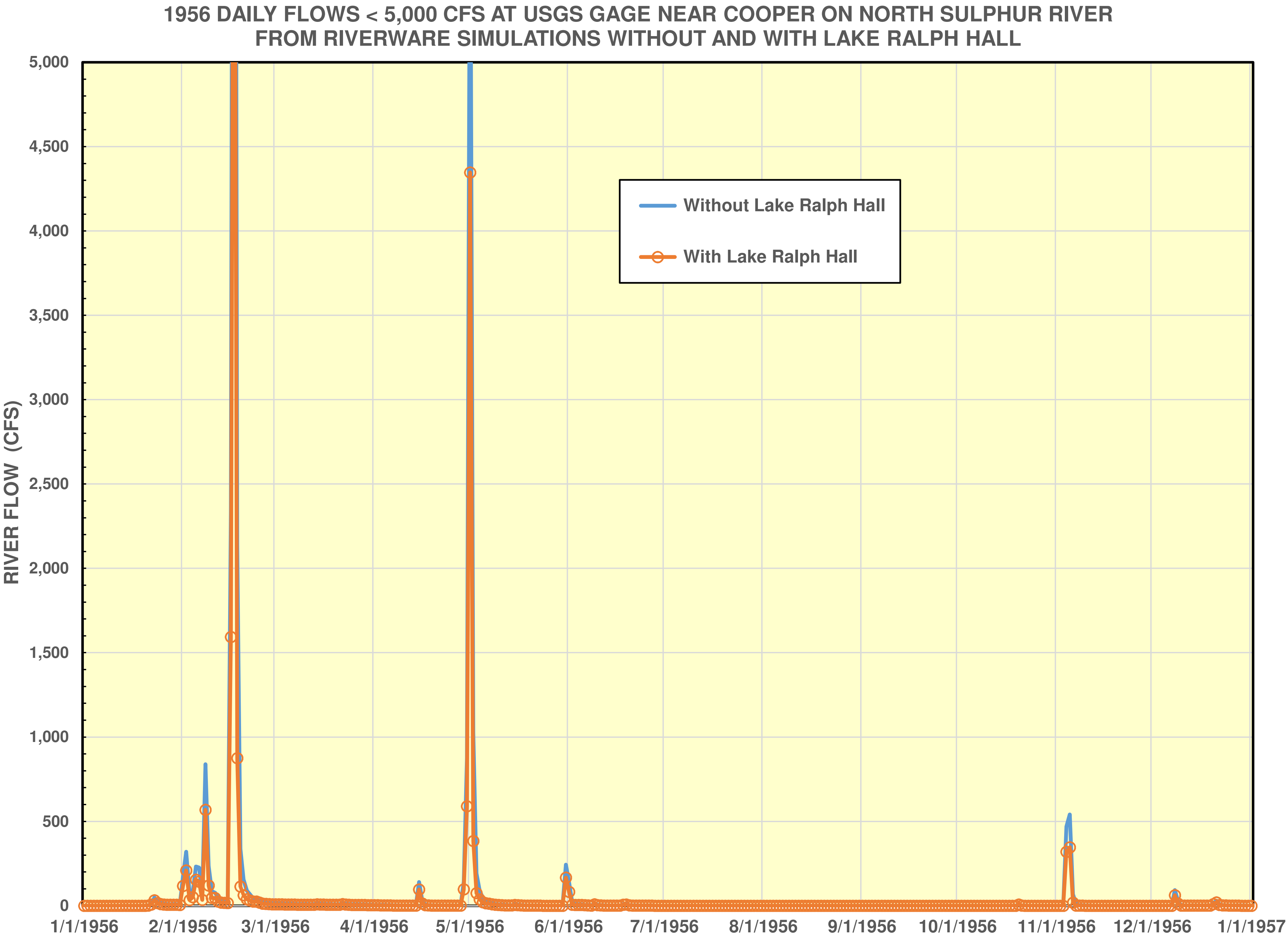
While the RiverWare model does provide daily simulations of flows in the North Sulphur and Sulphur Rivers, it is apparent from comparisons of these flows under conditions without and with Lake Ralph Hall that the daily variations themselves really do not tell us much more, if anything, about the effects of Lake Ralph Hall than monthly flow values. From the graphs of daily flows in Attachments A and B, it is shown that flood hydrographs occur at generally the same frequency and duration without or with Lake Ralph Hall. It is only the peaks of these hydrographs that are somewhat reduced due to the effects of Lake Ralph Hall, and peak flood flows in the North Sulphur River, unless they are associated with significant flood events on the order of the 25-year flood or greater, do not produce overbanking conditions that normally might be considered important from an aquatic ecological perspective. The incised channel of the North Sulphur River upstream of and for some distance downstream of the gage near Cooper simply is too deep to allow overtopping by the vast majority of flood events and too steep-walled to support and maintain typical lower floodplain conditions. Farther downstream, as inflows continue to enter the North Sulphur River and the Sulphur River below the confluence with the South Sulphur River, the reduction of river flows caused by Lake Ralph Hall becomes relatively less significant, to the point that the reservoir likely has minimal impact on instream and floodplain conditions.

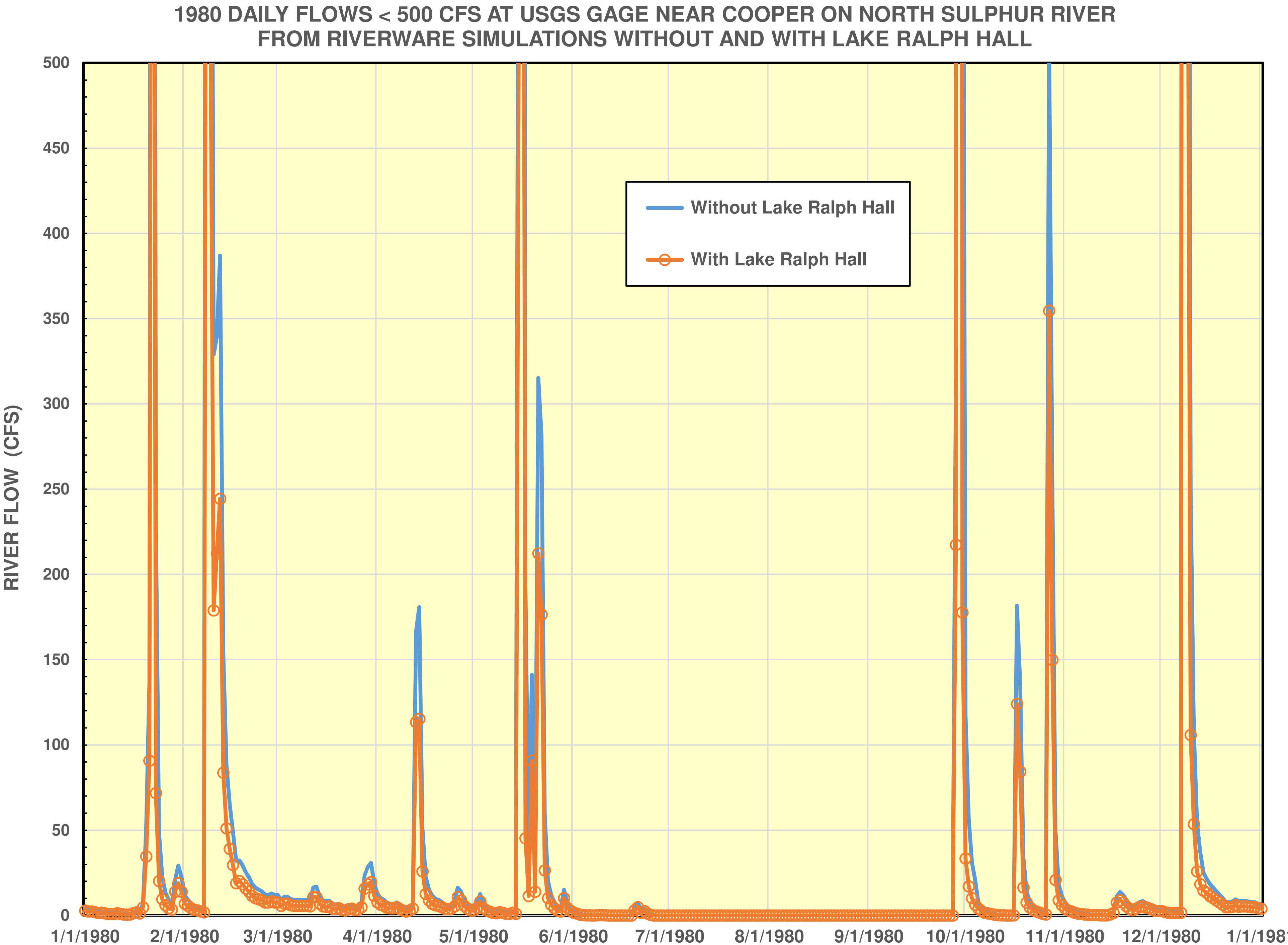
When considering the results from the RiverWare model of the Sulphur, Cypress and Red River Basins, it also is important to note that some of the deficiencies of the model could be relevant with respect to evaluating the impacts of Lake Ralph Hall. The exclusion of existing water rights from the model and the prior appropriation doctrine precludes any passing of inflows through the reservoir to satisfy the demands of downstream senior water rights. These additional flows in the river, which the WAM does model, could serve to supplement tributary inflows for filling channel pools and supporting aquatic life downstream of the reservoir. While typically the passing of flows for satisfying senior water rights only occurs during extremely dry periods when a "call" is made by the downstream senior water rights, it is not something that would never occur as the RiverWare model assumes. With the construction and operation of Lake Ralph Hall, it is very likely that owners of existing downstream water rights, especially those with large irrigation rights located near or below the confluence of the North and South Sulphur Rivers, as well as Lake Wright Patman located farther downstream on the Sulphur River, will closely monitor their available water supplies from the river and will certainly issue a call for Lake Ralph Hall to pass inflows to meet their needs if they believe Lake Ralph Hall is depriving them of flows to which they are entitled. In this regard, the WAM probably provides a better estimate of low flow conditions in the North Sulphur River with Lake Ralph Hall in operation than the daily RiverWare model does. Another point to note relates to the higher level of inflows to Lake Ralph Hall that the RiverWare model produces. It is not clear as to why this occurs, but it definitely affects the operation of the reservoir and may artificially increase the frequency of flood spills from the reservoir that flow into the river downstream.

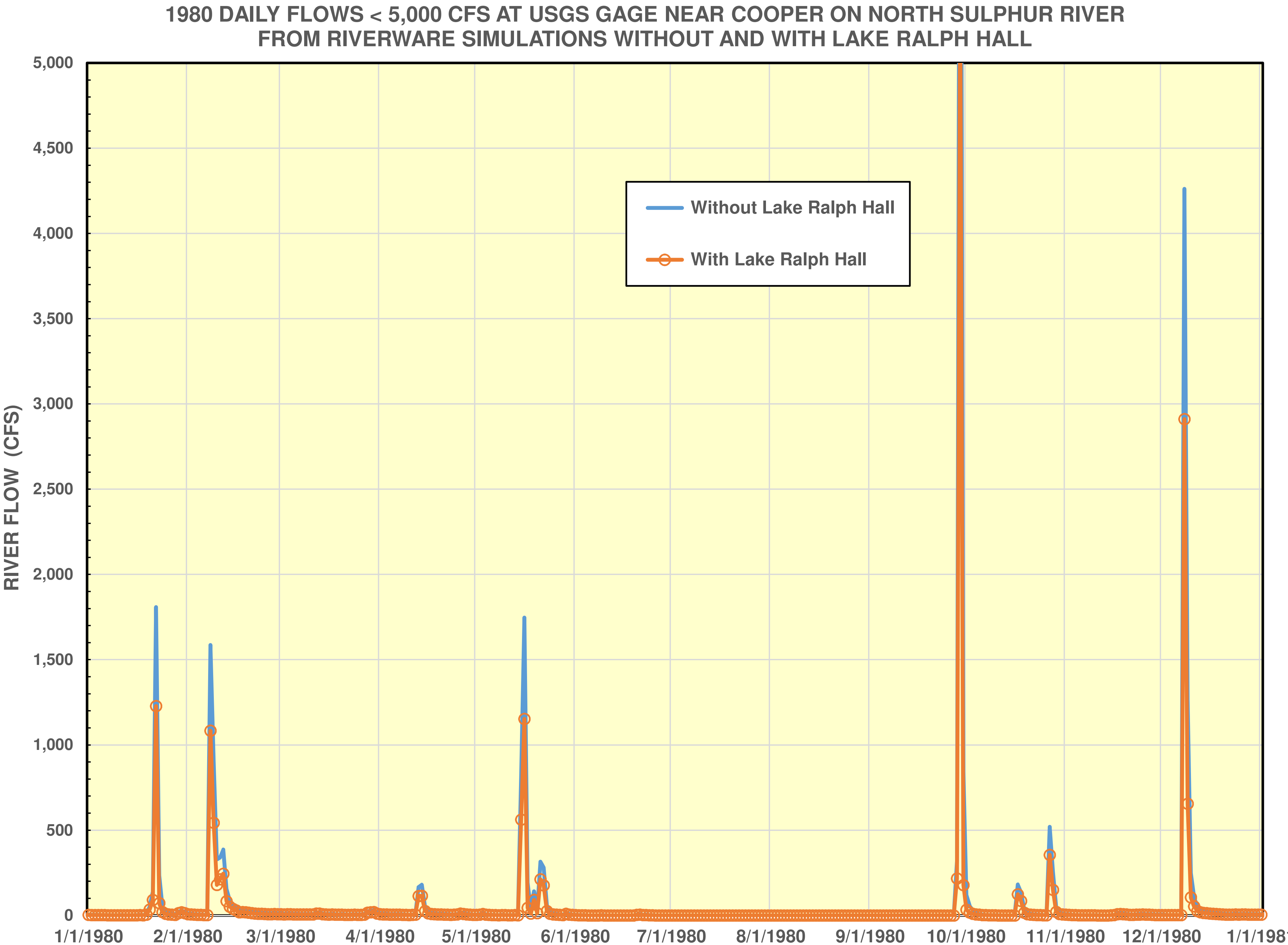
In summary, the application of the daily RiverWare model for analyzing the effects of Lake Ralph Hall on downstream river flows is considered to have been a worthwhile effort. It has provided a better understanding of the significance of daily variations in river flows and how Lake Ralph Hall might affect those flow variations and flood hydrographs, information that may be useful for further evaluating the impacts of Lake Ralph Hall. In the end, however, it remains that the place where Lake Ralph Hall will likely have its most significant effect on the flow regime of the North Sulphur and Sulphur Rivers is still the segment immediately downstream of the reservoir that is characterized by an eroded and degraded channel devoid of significant aquatic life such that reductions in river flows caused by the reservoir are not likely to result in noticeable environmental impacts. Even then, the UTRWD is proposing to develop and construct the mitigation area on the south floodplain of the North Sulphur River below the reservoir by restoring the configuration of approximately 14,000 feet of the abandoned river channel, planting native vegetation and trees, and stocking the restored pools and channel with fish and aquatic species that typically inhabited the historical river system.

If you have any questions regarding the material presented herein or if you want to discuss these results further, please contact us at your convenience. Also, we are in the process of assembling the RiverWare results files and the various spreadsheets used in analyzing and presenting the results for delivery to the Corps.

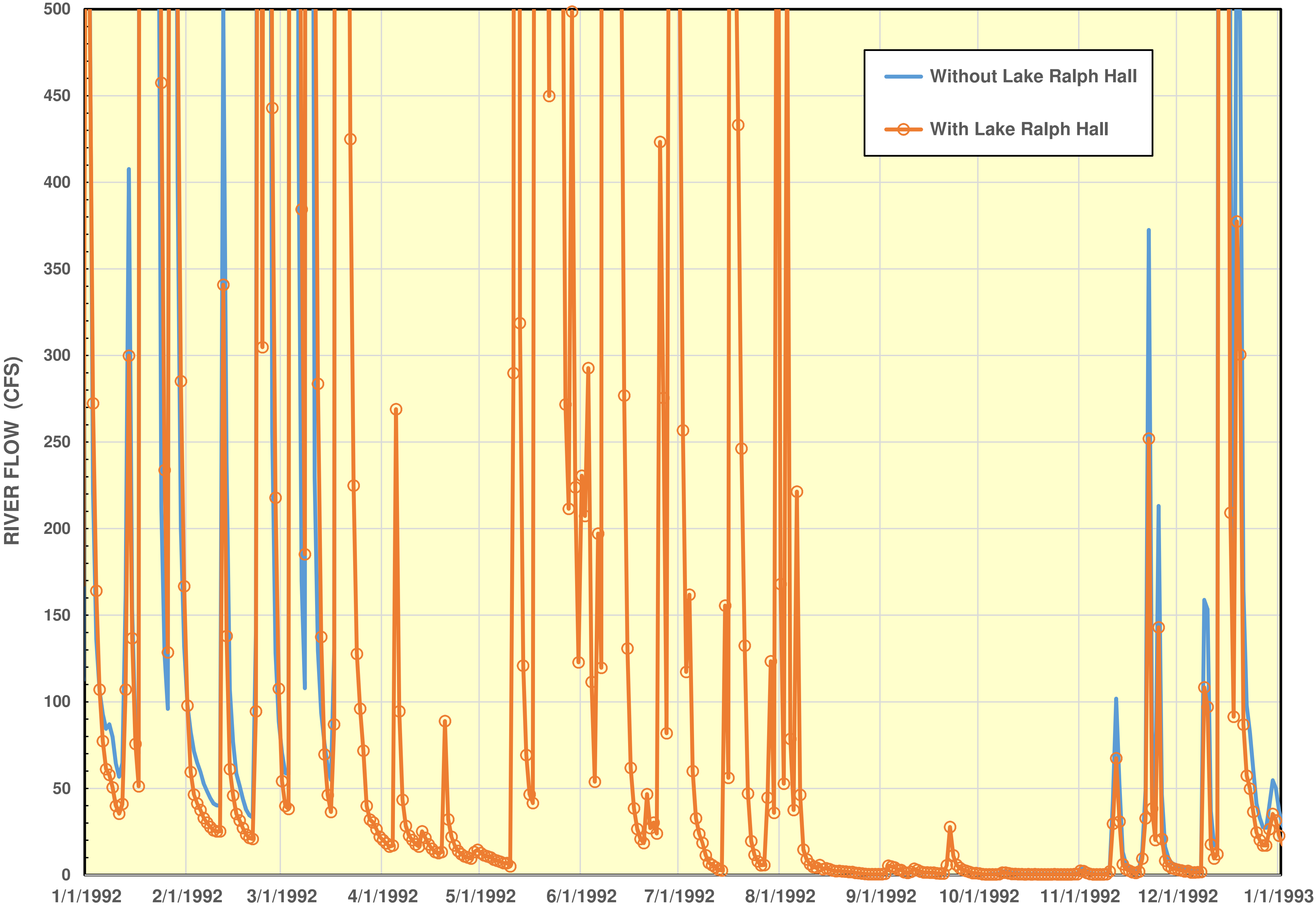

1956 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957

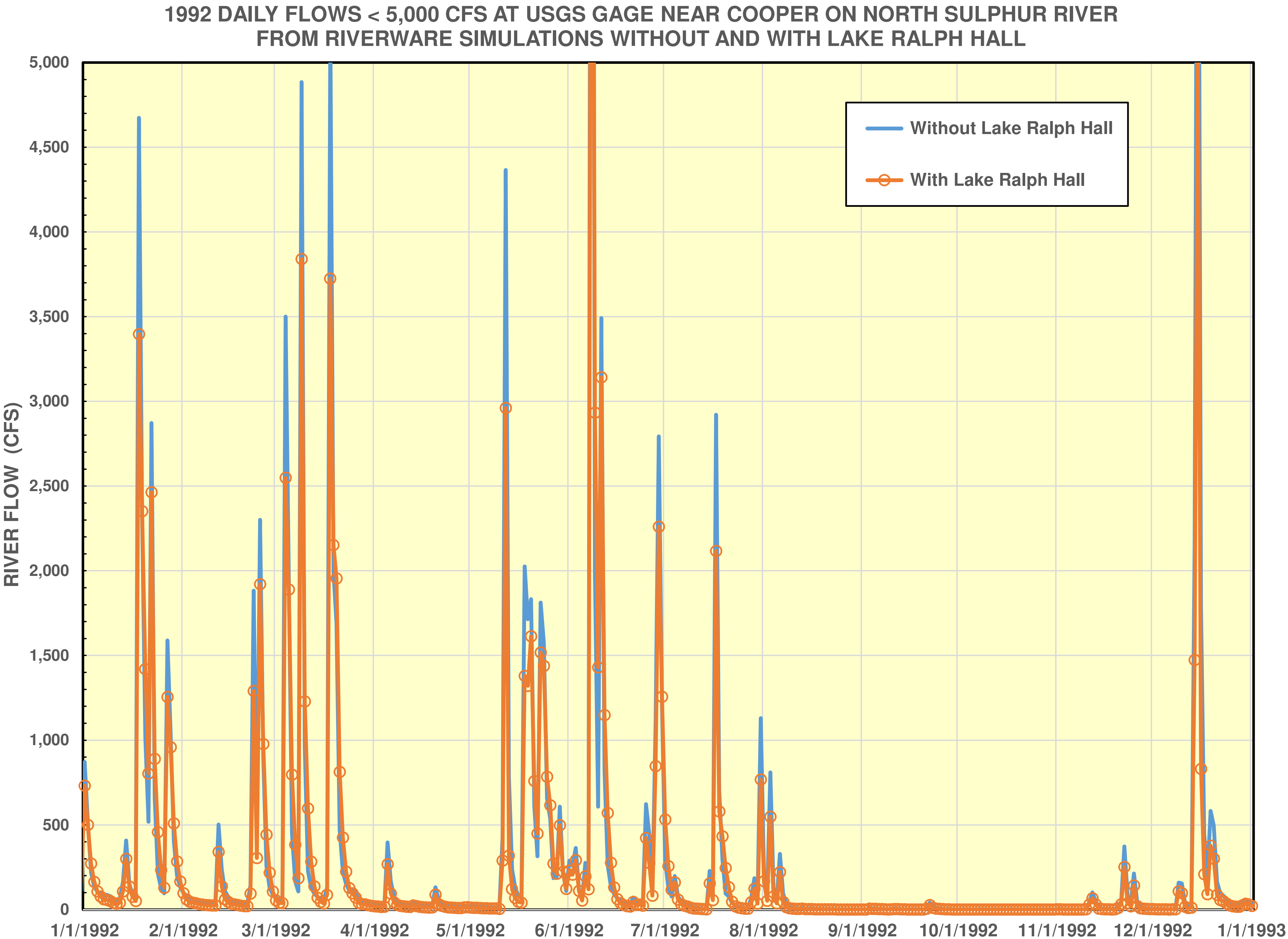

1956 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957
Without Lake Ralph Hall
With Lake Ralph Hall


1980 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall

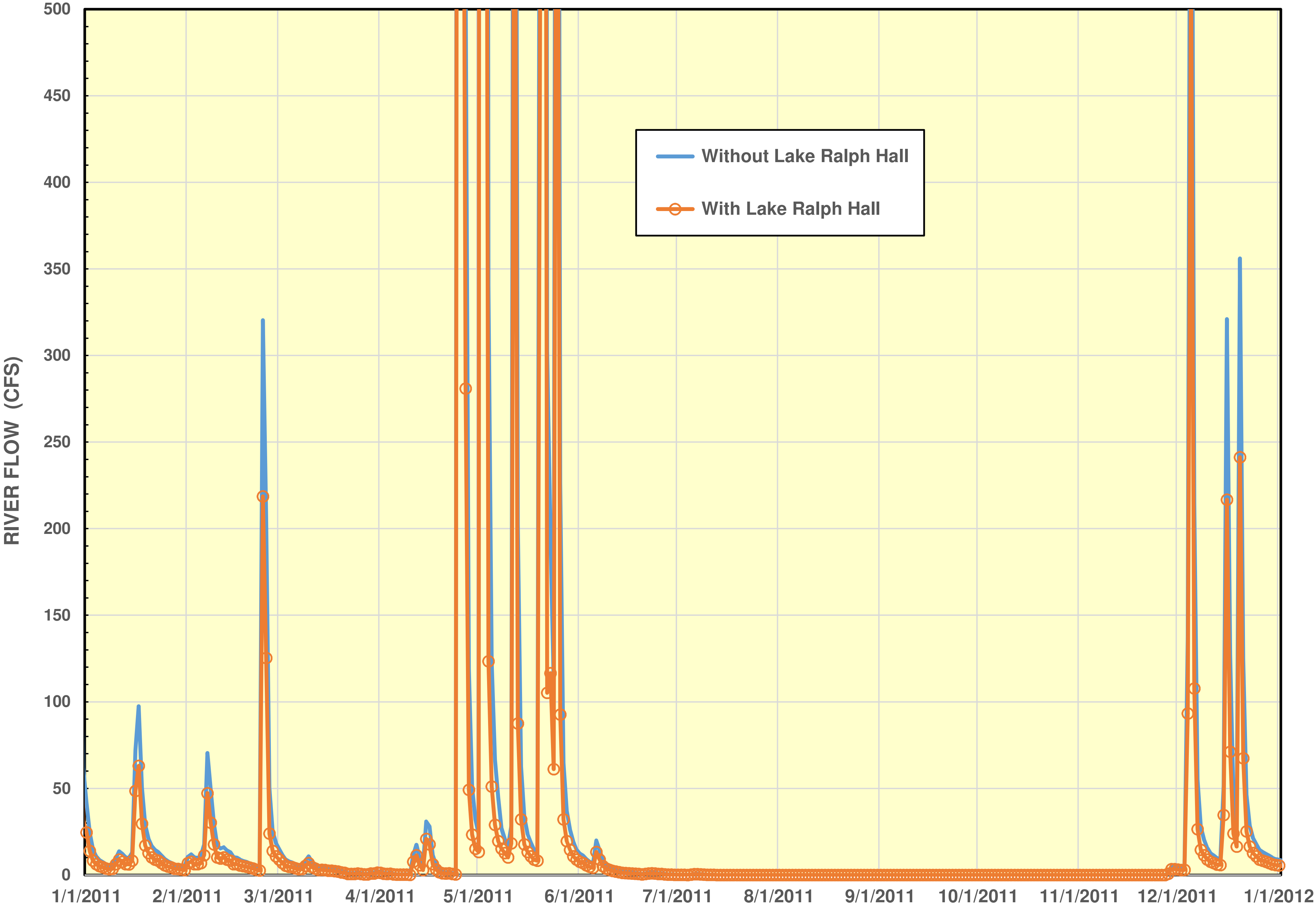

1980 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall

1992 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL

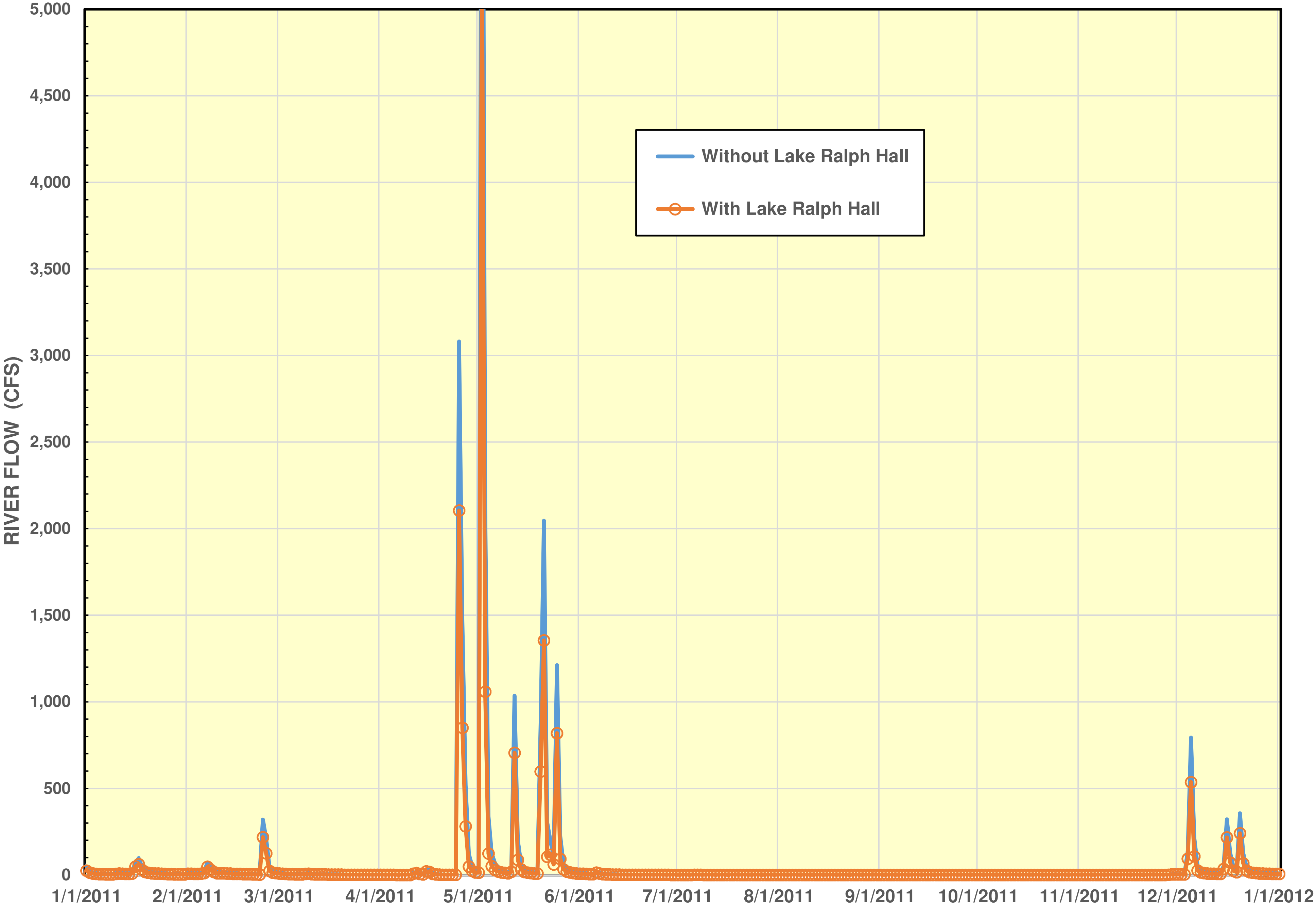


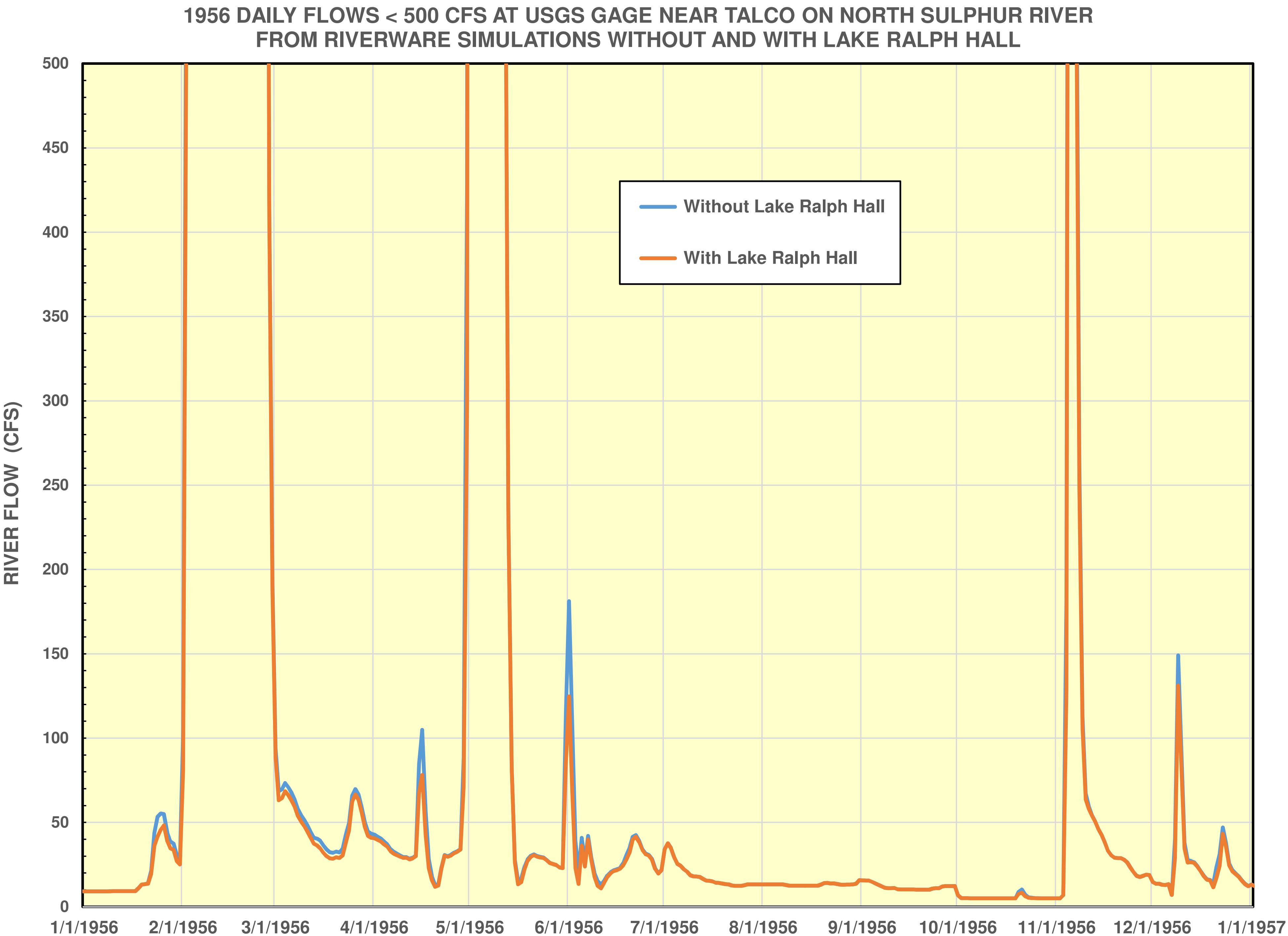

1992 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall

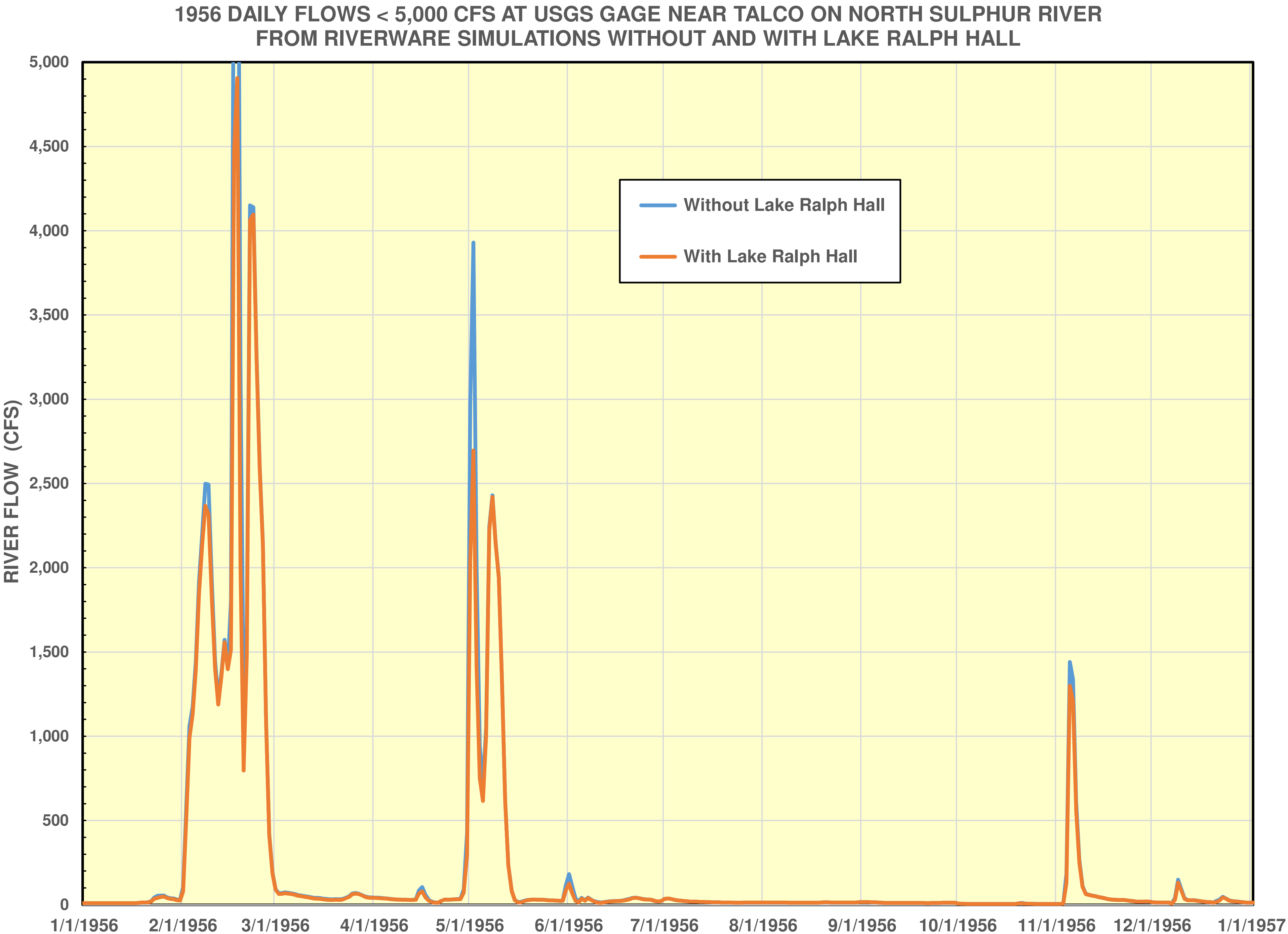
2011 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL

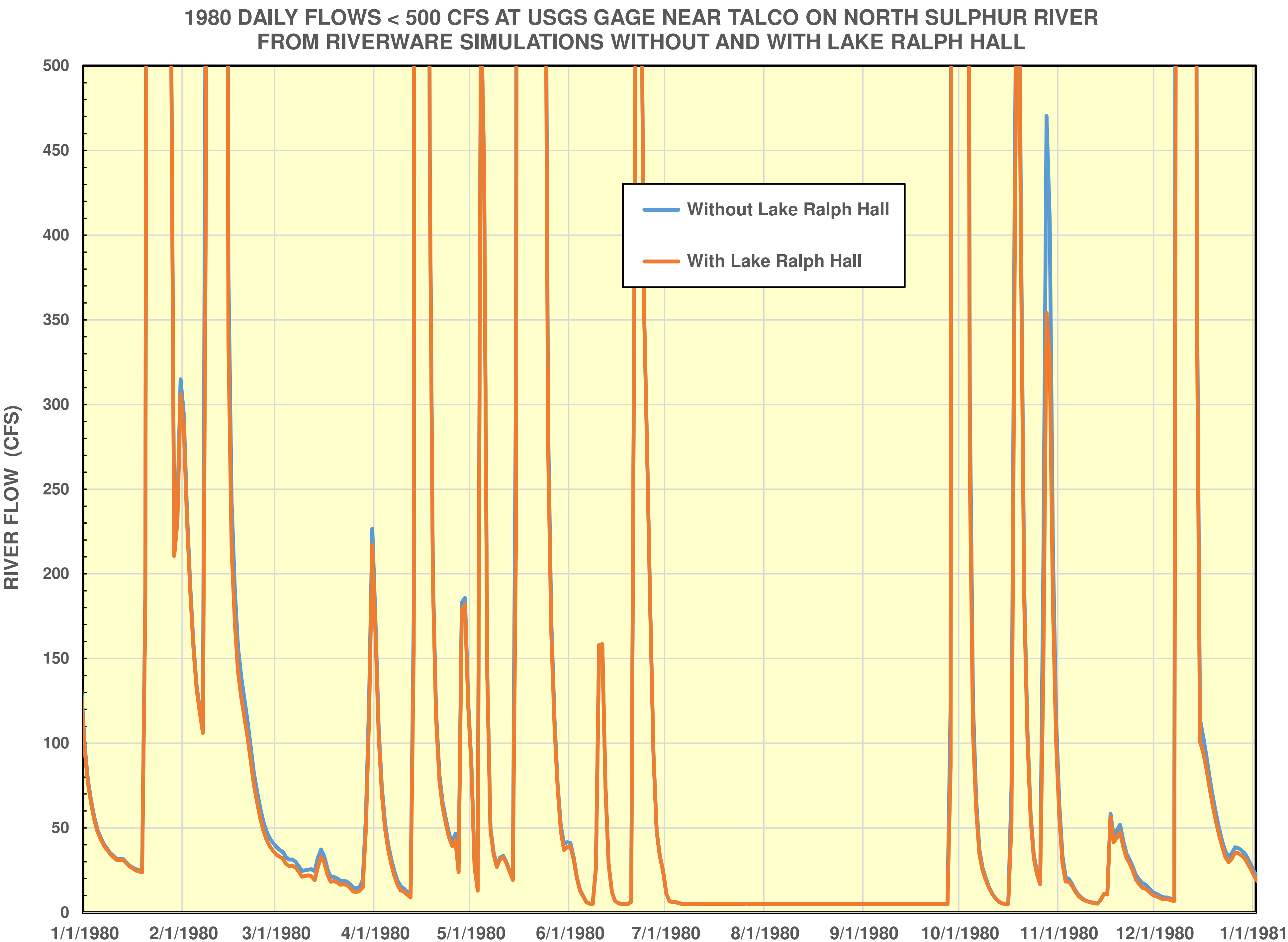


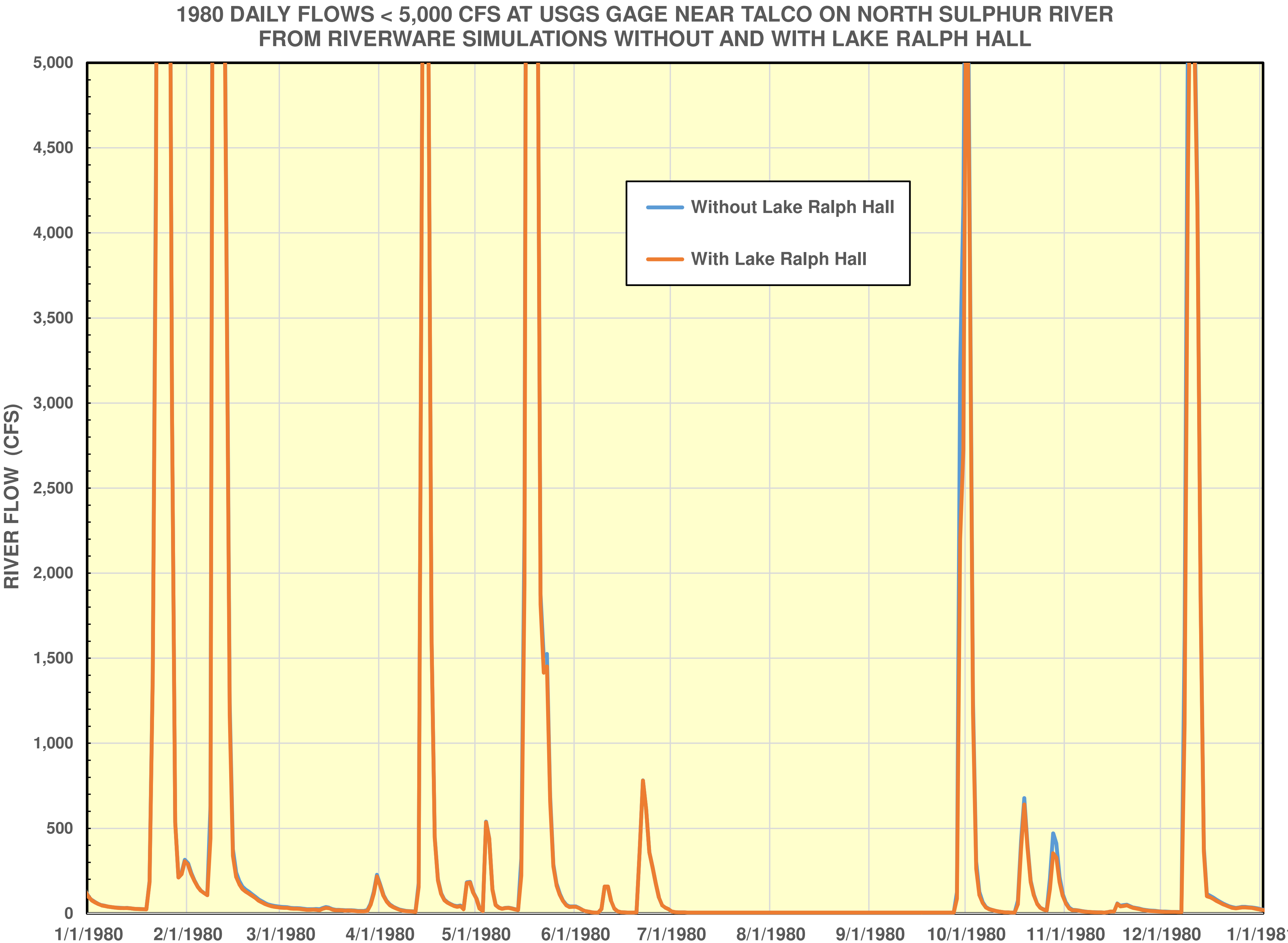
2011 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL

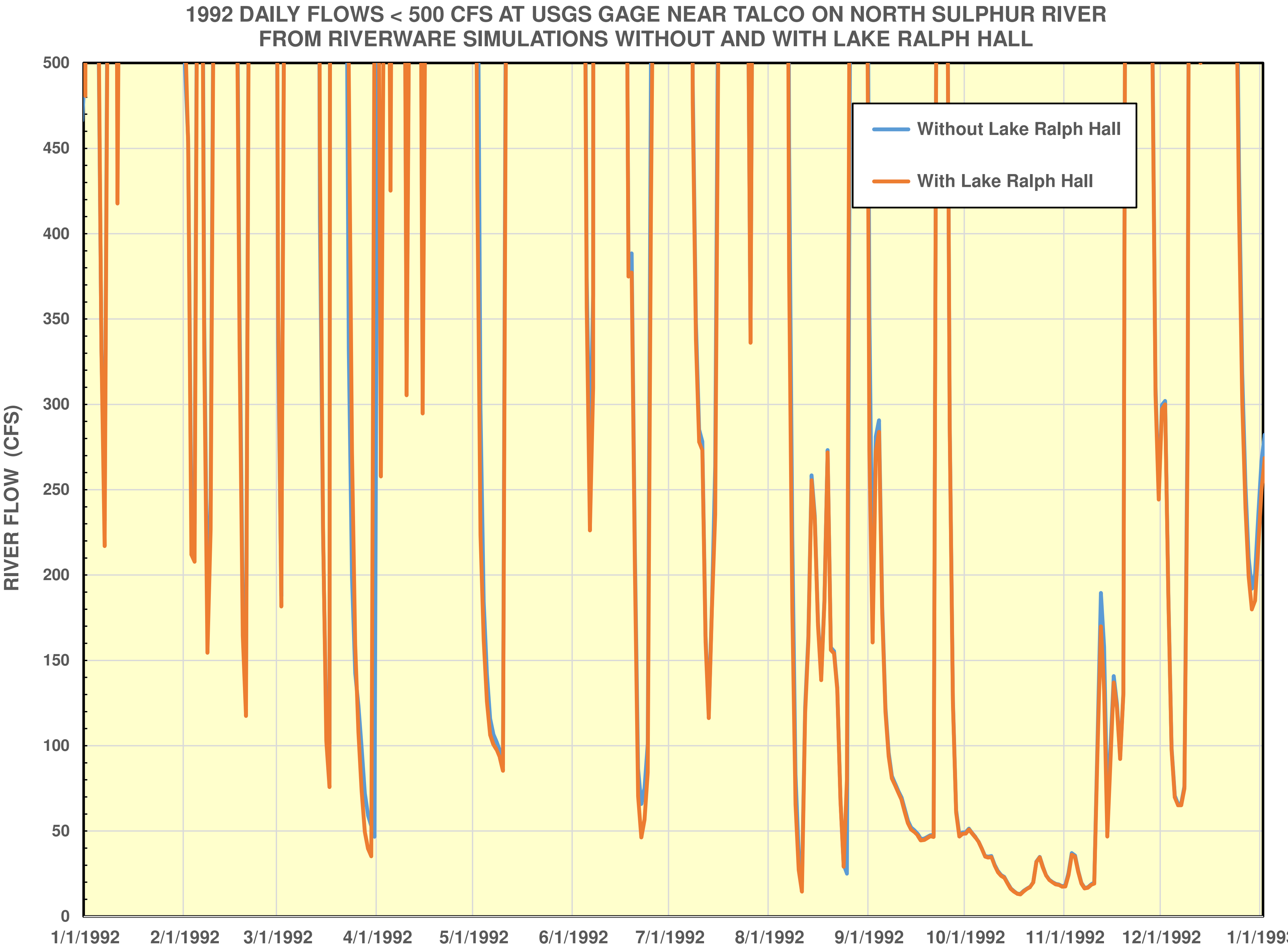


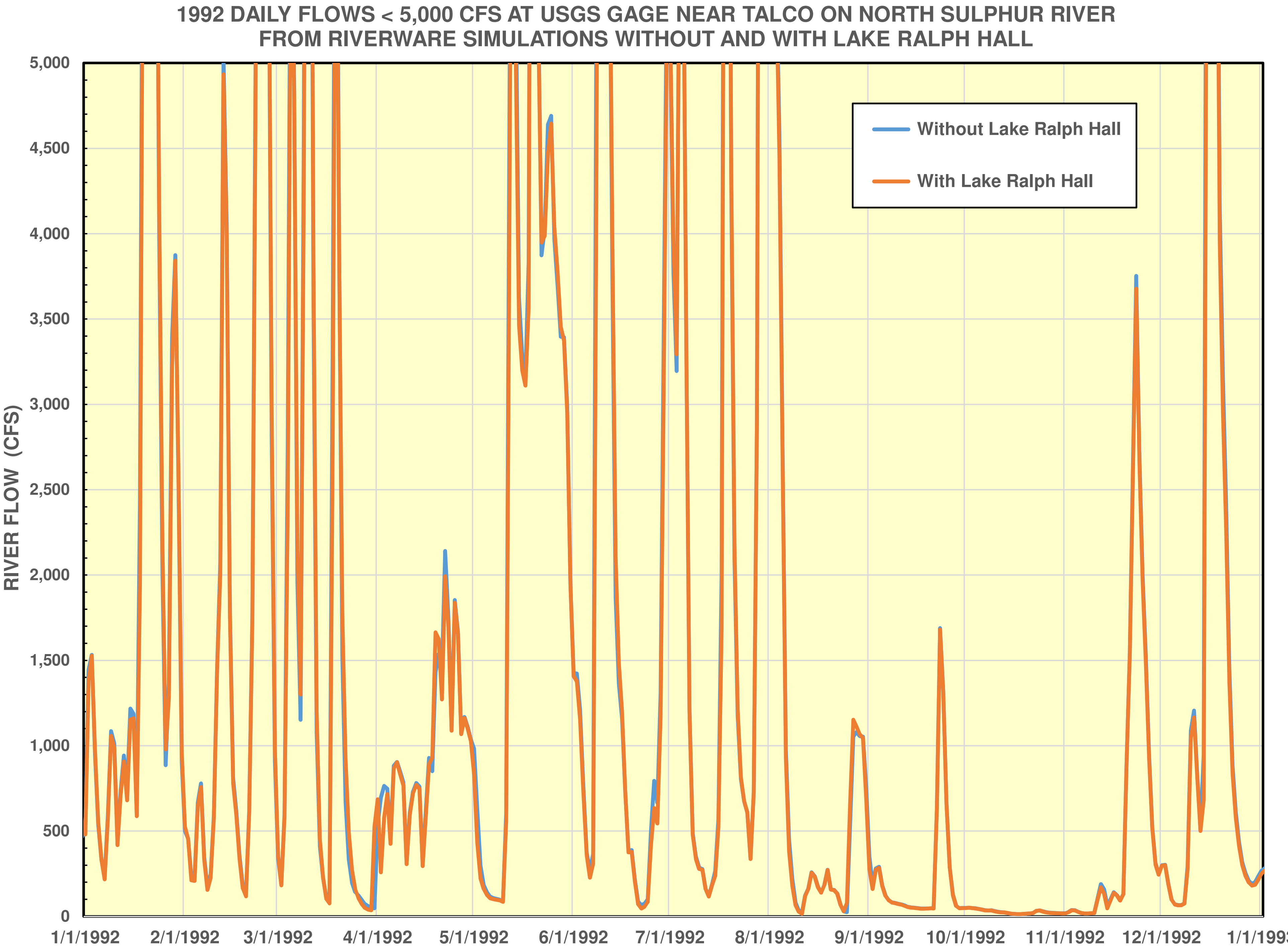

1956 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957

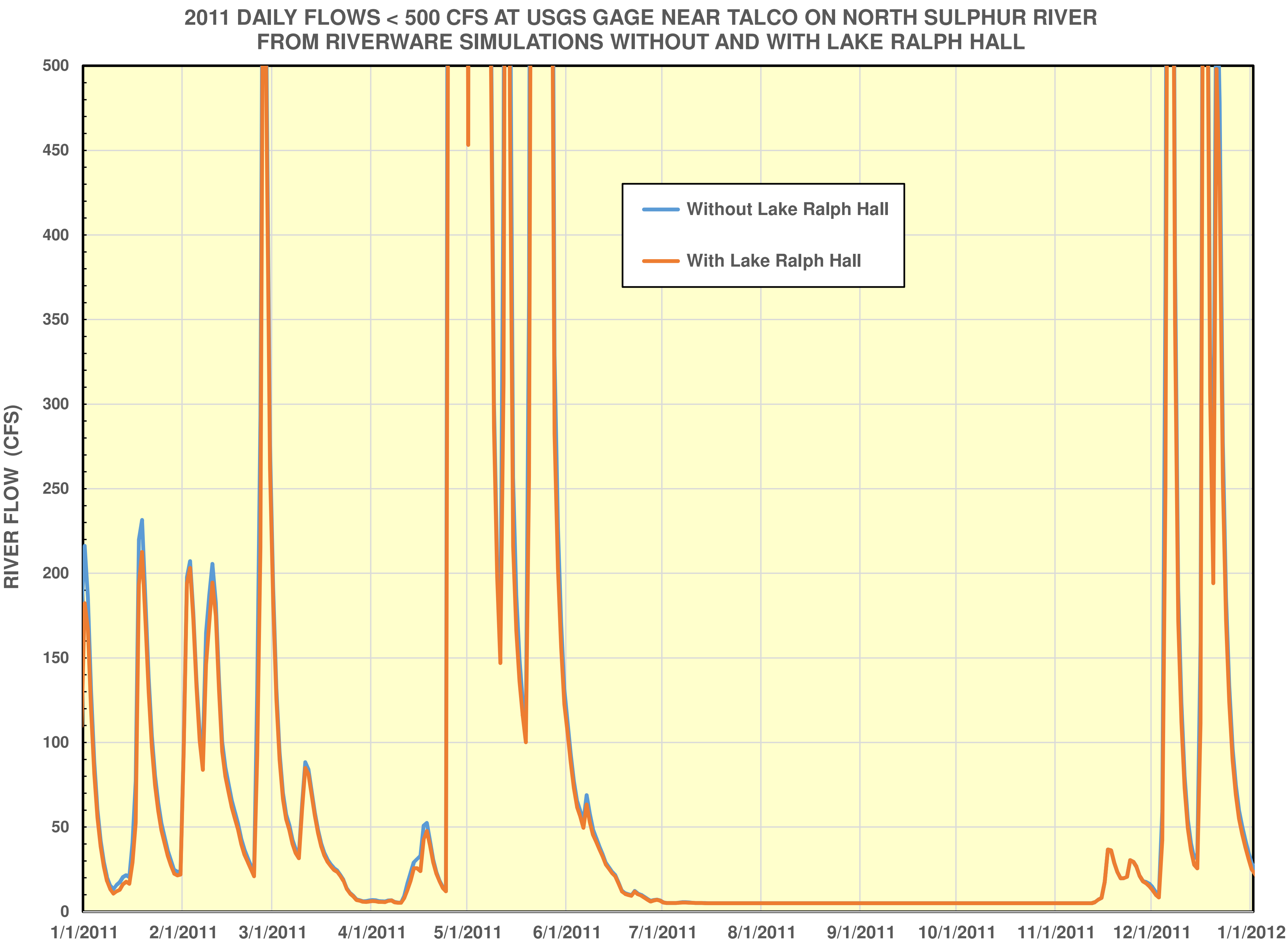

1956 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957

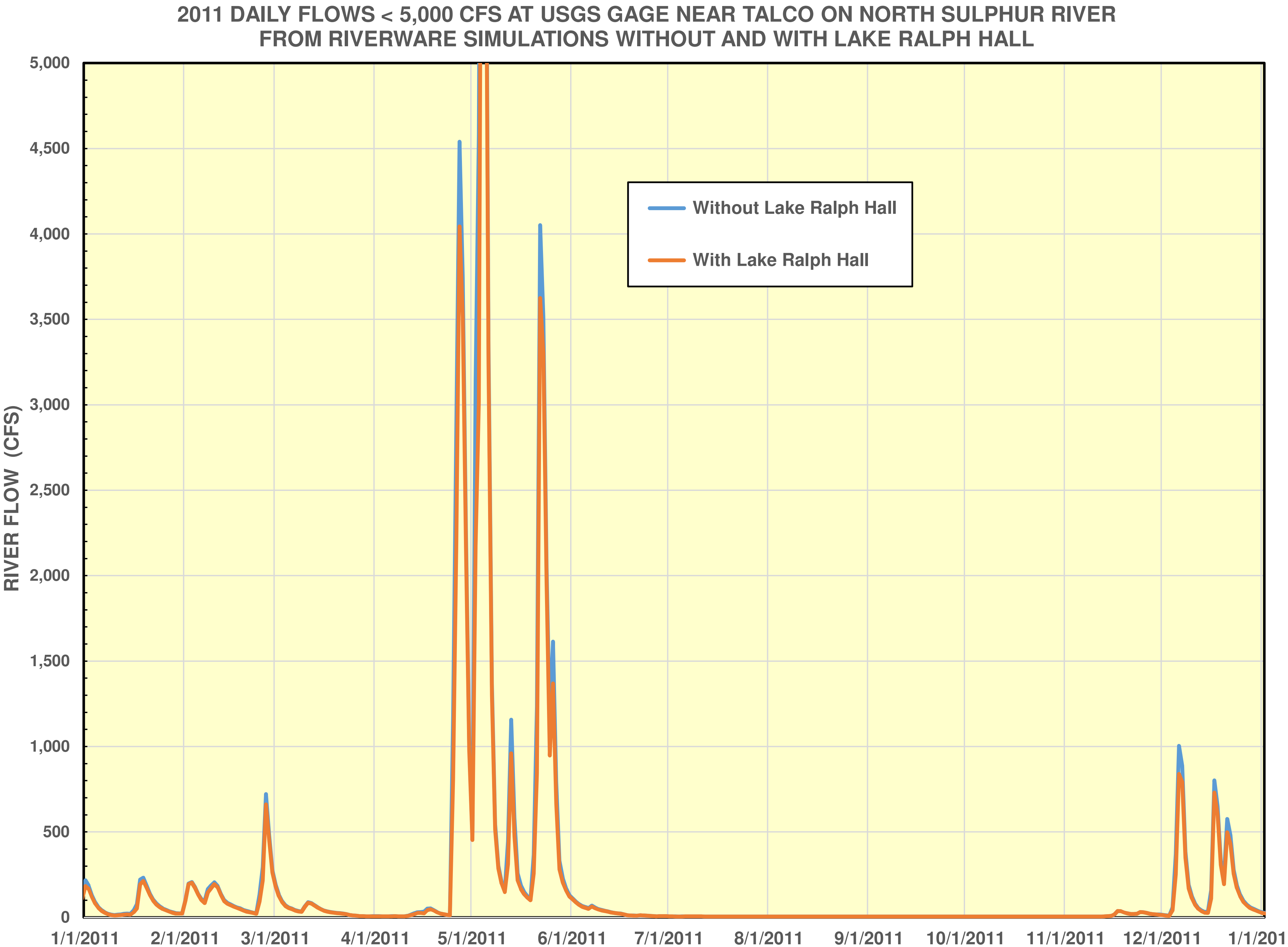

1980 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall


1980 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall

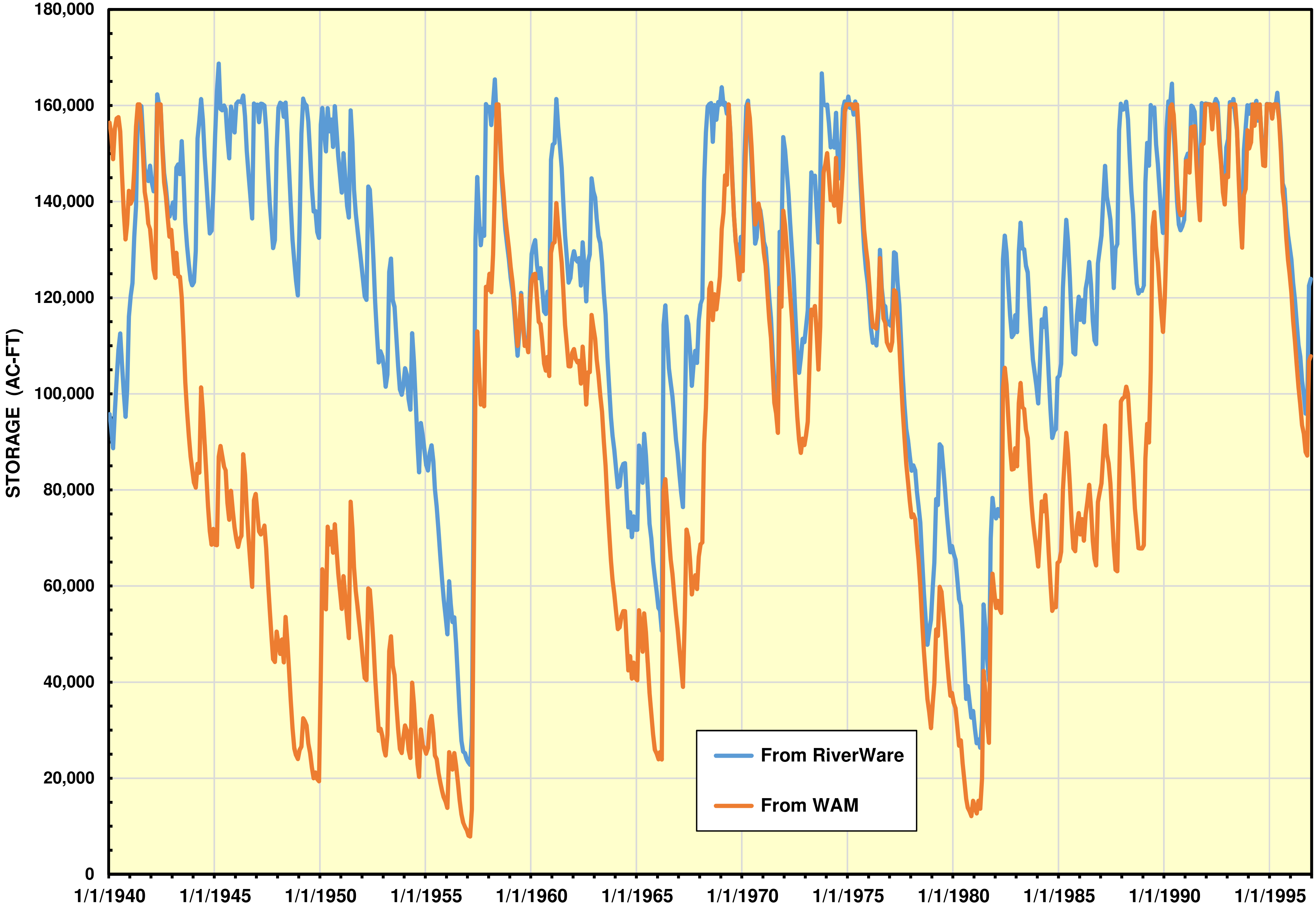

1992 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
0
50
100
150
200
250
300
350
400
450
500
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall


1992 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall

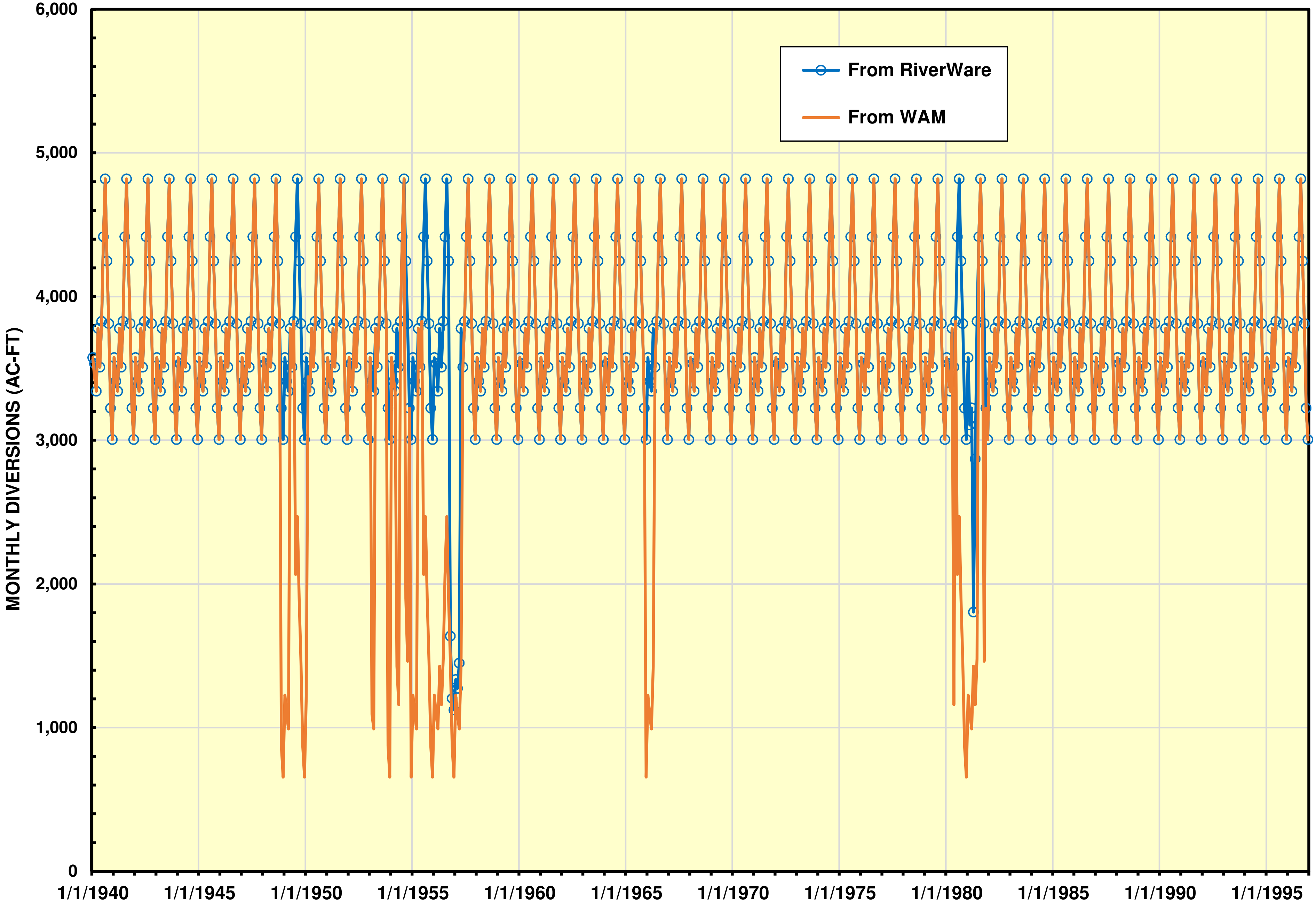

2011 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
0
50
100
150
200
250
300
350
400
450
500
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012
Without Lake Ralph Hall
With Lake Ralph Hall


2011 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012
Without Lake Ralph Hall
With Lake Ralph Hall

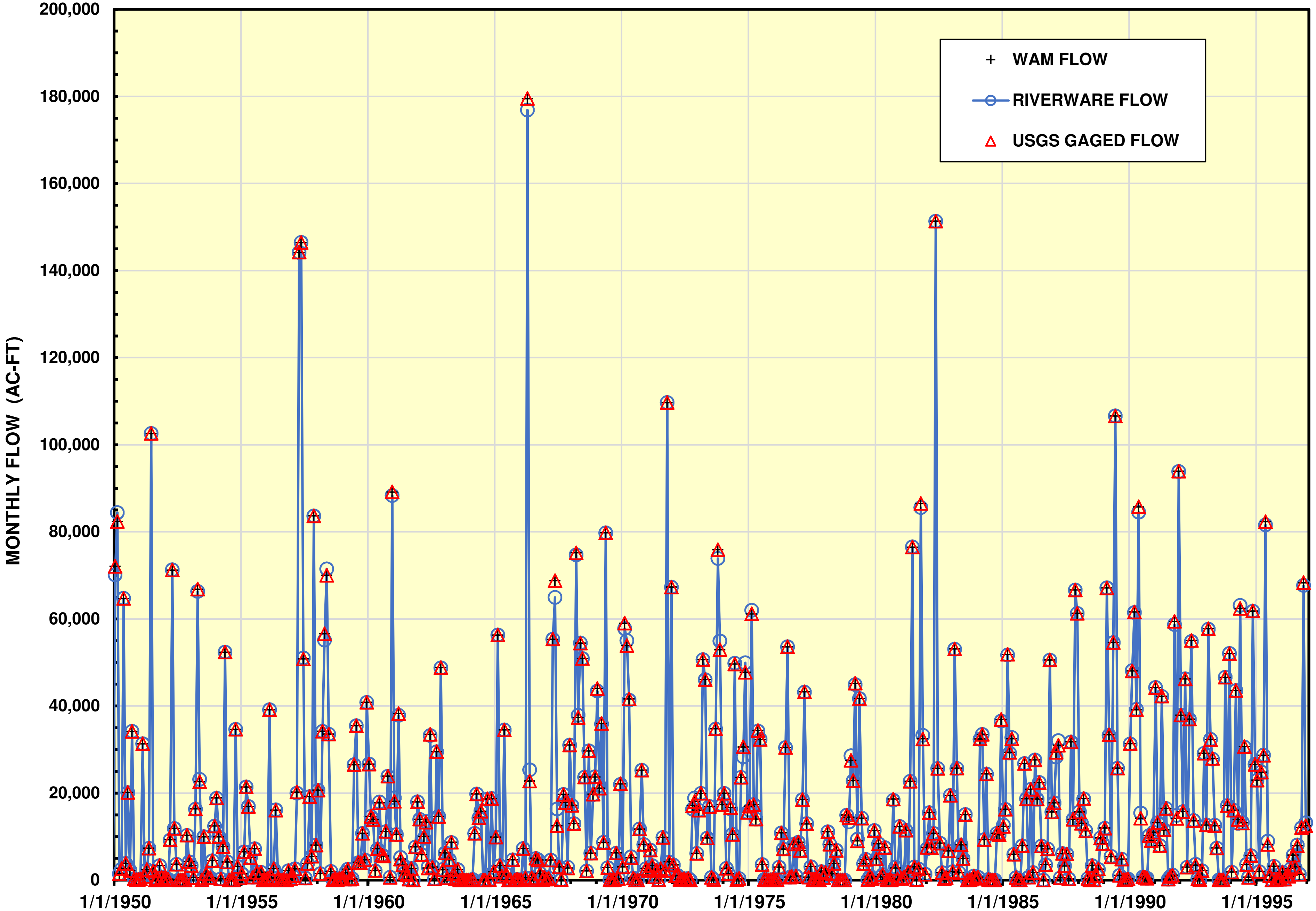
MONTHLY STORAGE IN LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



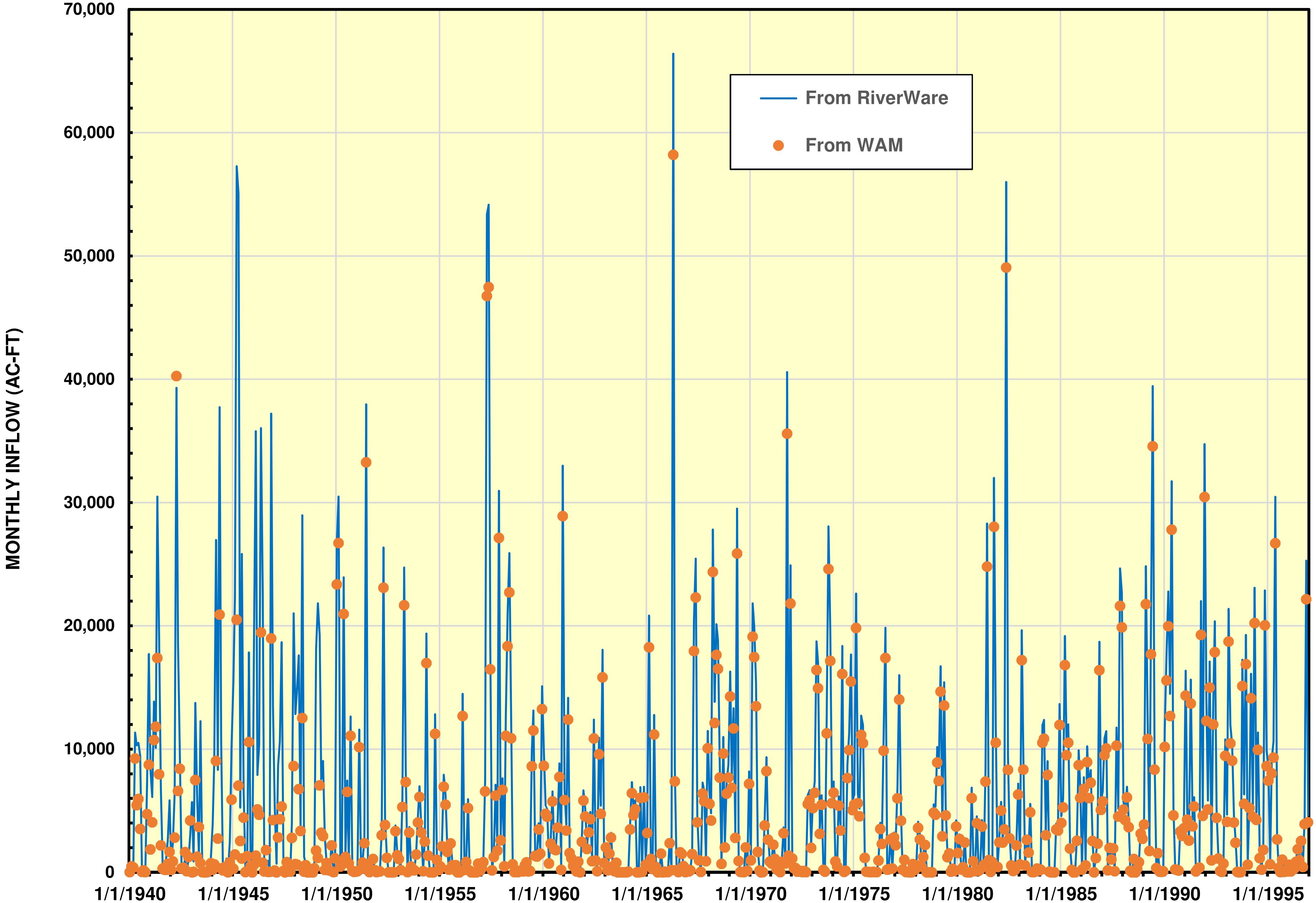
MONTHLY DIVERSIONS FROM LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



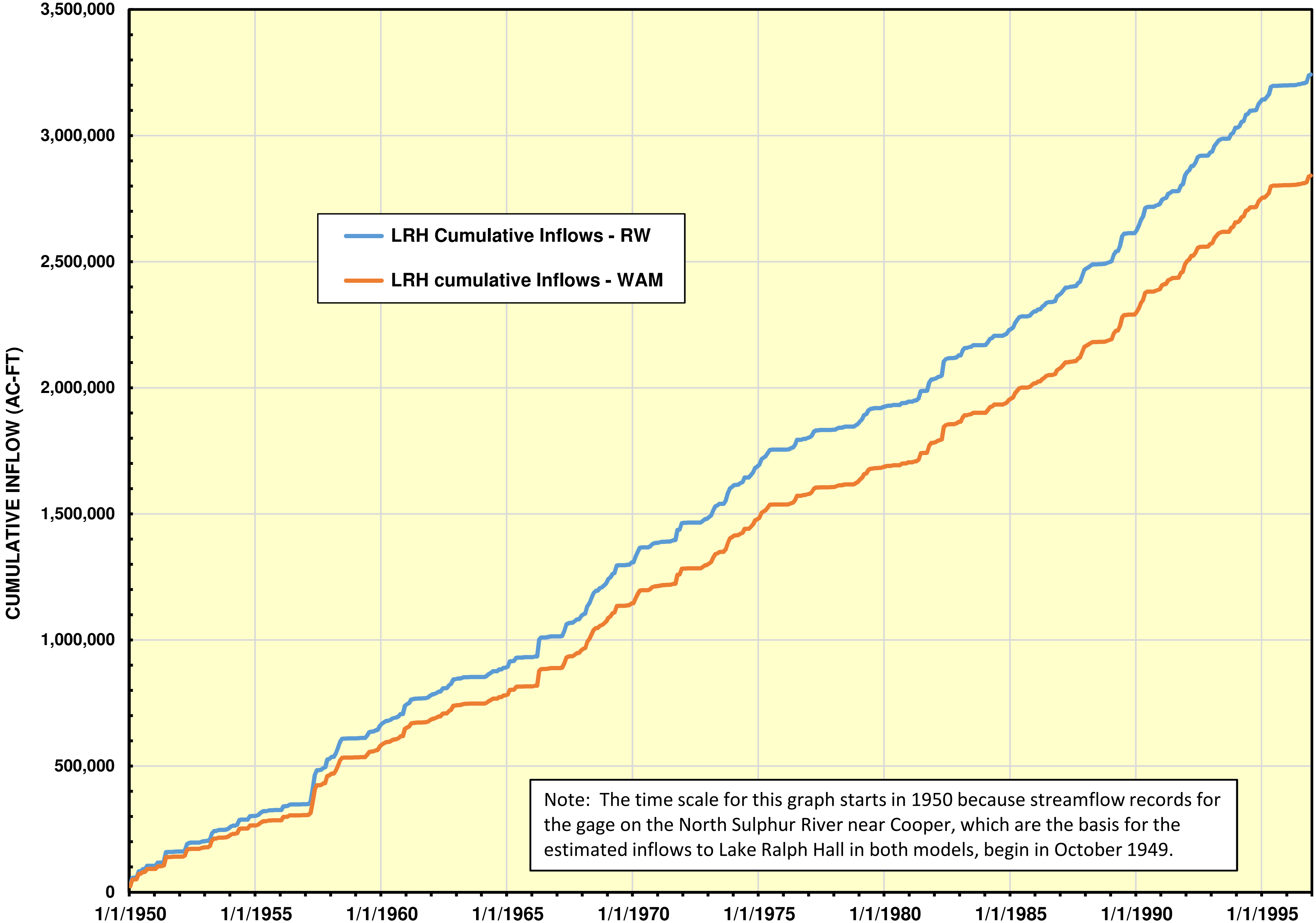
MONTHLY FLOWS AT NORTH SULPHUR RIVER GAGE NEAR COOPER FROM RIVERWARE AND WAM WITHOUT LAKE RALPH HALL AND FROM USGS GAGE



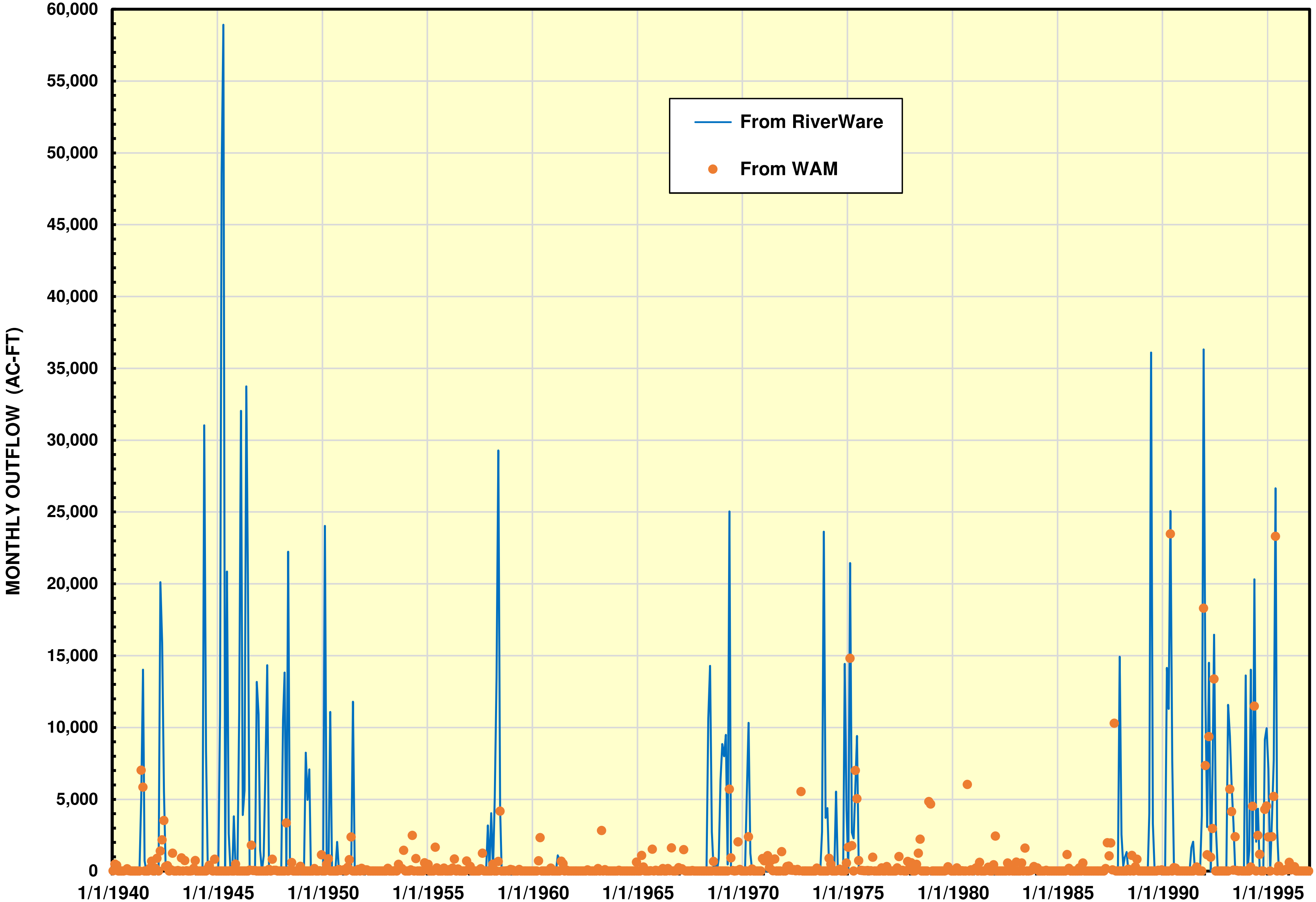
MONTHLY INFLOWS TO LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



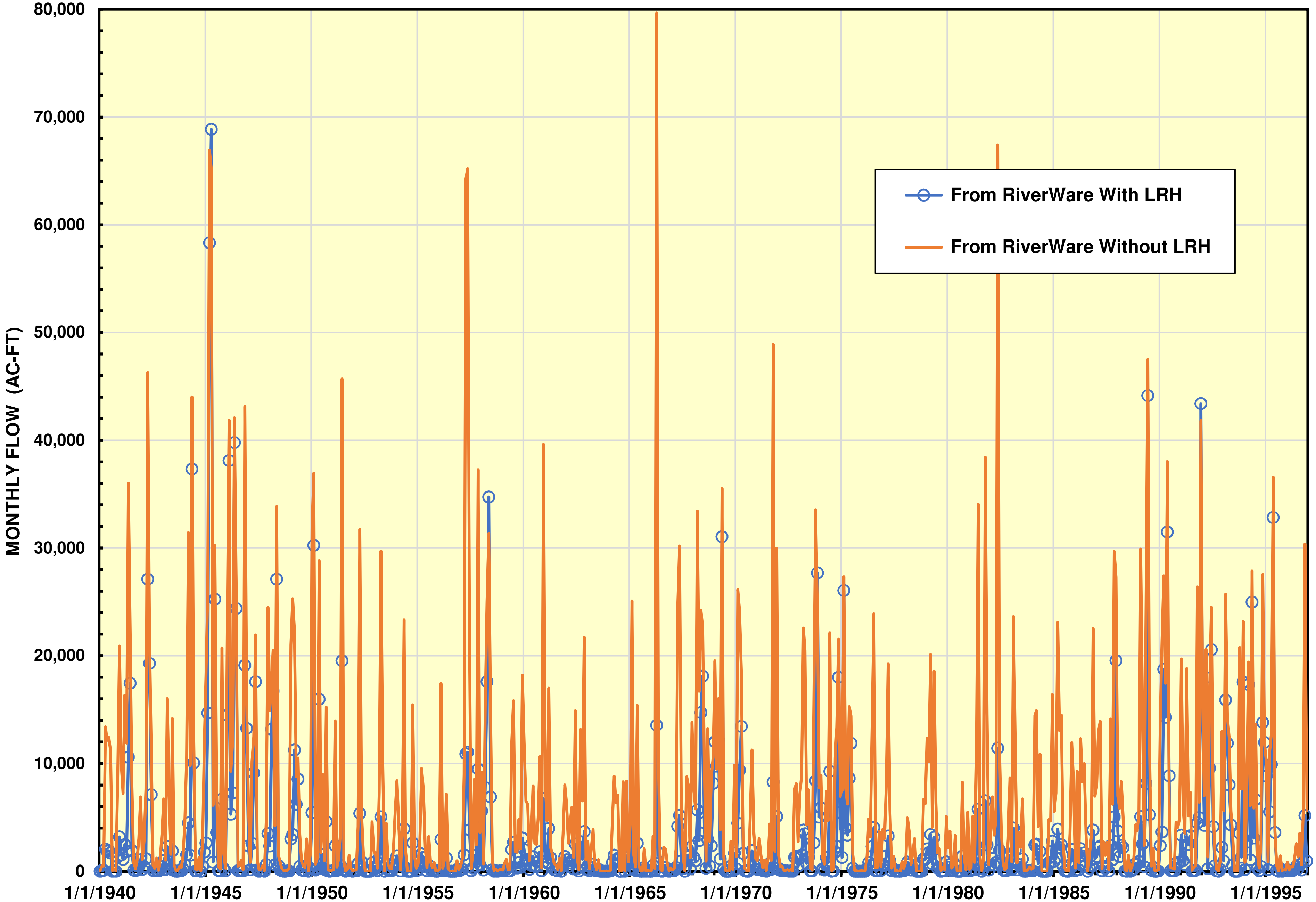
CUMULATIVE MONTHLY INFLOWS TO LAKE RALPH HALL BEGINNING IN 1950 FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



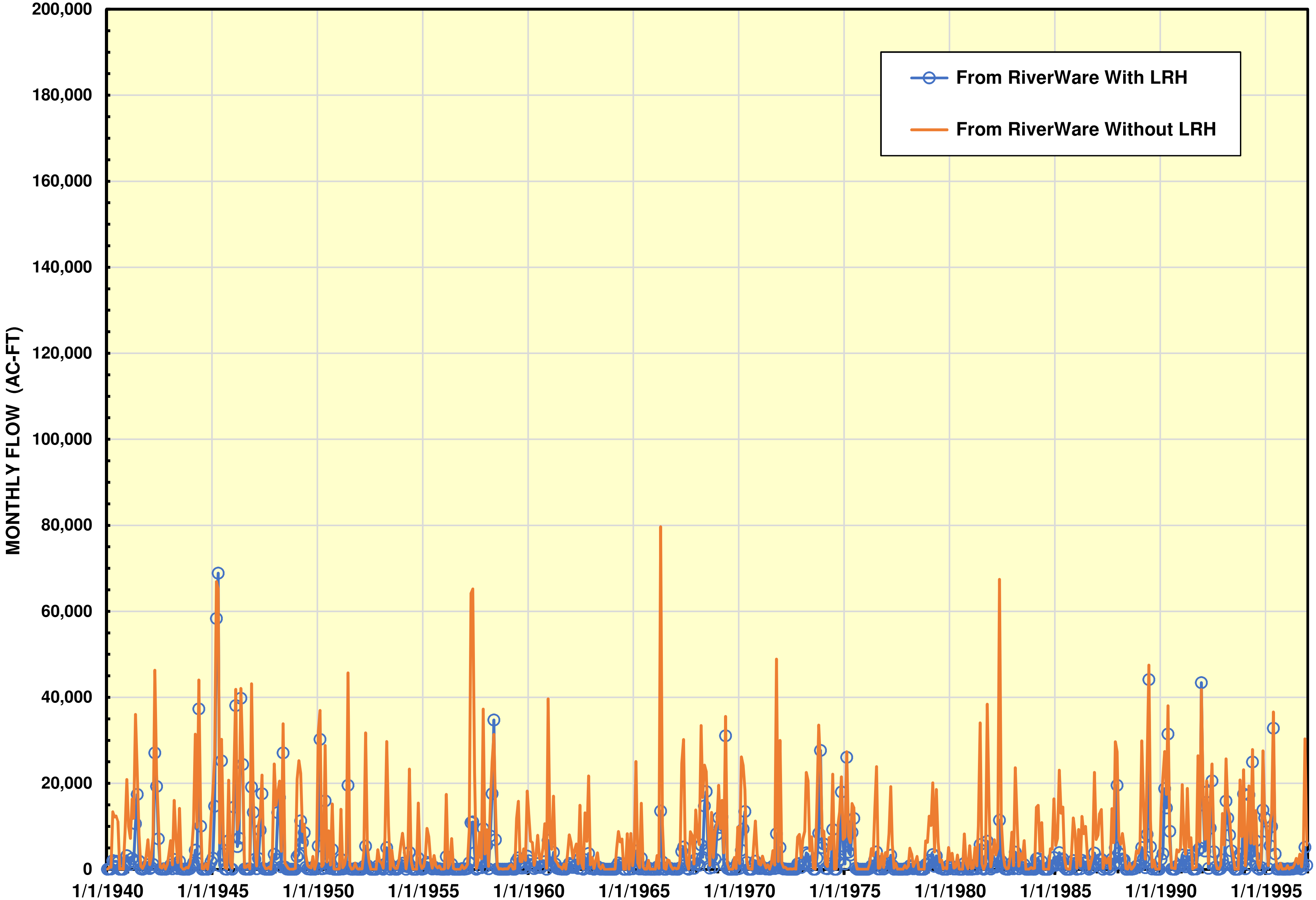
MONTHLY OUTFLOWS FROM LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



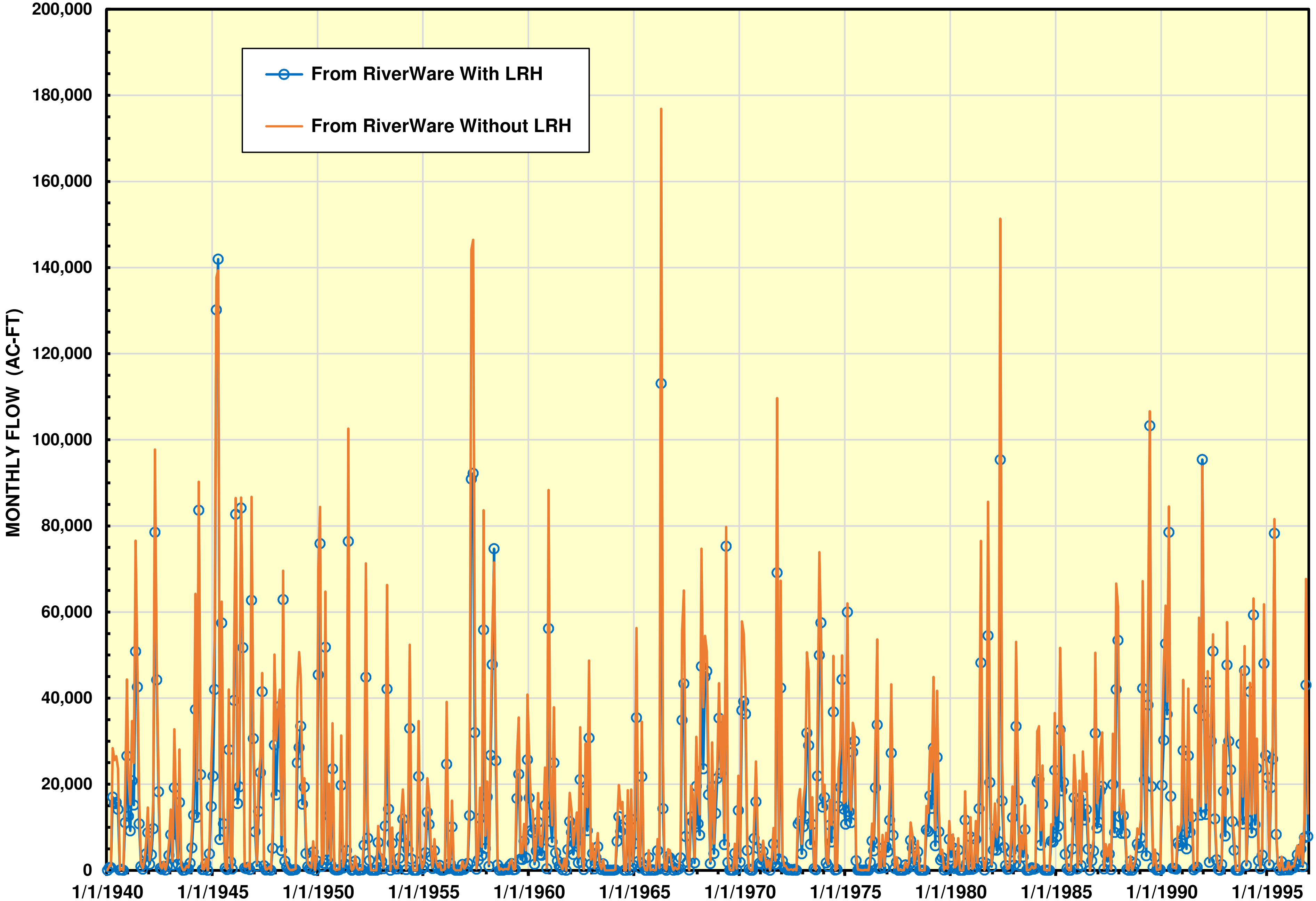
MONTHLY FLOWS BELOW LAKE RALPH HALL DAM SITE AT BAKER CREEK CONFLUENCE FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



MONTHLY FLOWS BELOW LAKE RALPH HALL DAM SITE AT BAKER CREEK CONFLUENCE FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



MONTHLY FLOWS AT NORTH SULPHUR RIVER GAGE NEAR COOPER FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE AND FROM WAM WITH LAKE RALPH HALL

	RiverWare	WAM
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	62.2%	73.0%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	82.1%	83.8%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
%	%	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon
1.0%	99.0%	0	0	0	0	0	2	1	3	308	208	308	284
2.0%	98.0%	0	0	0	0	0	3	5	4	316	310	341	416
3.0%	97.0%	0	0	0	0	0	4	11	10	343	378	369	472
4.0%	96.0%	3	2	1	3	4	9	30	23	350	384	442	509
5.0%	95.0%	5	4	1	5	9	16	38	34	394	423	527	590
7.0%	93.0%	13	8	3	9	22	28	63	57	455	473	720	751
10.0%	90.0%	27	17	5	19	45	54	114	121	658	587	1,046	1,180
15.0%	85.0%	76	48	14	47	115	149	288	364	1,051	1,053	1,740	1,919
16.2%	83.8%	90	57	18	53	147	175	329	425	1,151	1,201	2,172	2,389
17.9%	82.1%	111	78	21	66	175	235	420	503	1,462	1,278	2,801	3,199
20.0%	80.0%	137	105	26	93	217	290	510	677	1,727	1,539	3,657	3,713
25.0%	75.0%	239	181	46	148	385	531	985	1,057	3,086	2,708	5,774	5,251
27.0%	73.0%	281	210	54	175	450	612	1,151	1,365	3,871	3,706	6,747	6,034
30.0%	70.0%	427	294	74	216	622	801	1,495	1,925	4,750	4,630	8,313	8,534
35.0%	65.0%	719	558	136	279	1,133	1,417	2,494	3,058	7,525	6,802	13,183	10,734
37.8%	62.2%	900	665	175	347	1,462	1,721	2,971	3,867	10,190	8,458	17,103	13,954
40.0%	60.0%	1,006	775	200	399	1,653	2,002	3,481	4,583	12,496	9,491	19,602	15,409
45.0%	55.0%	1,407	1,082	289	580	2,401	2,687	5,245	5,949	18,340	12,596	28,830	23,245
50.0%	50.0%	2,282	1,564	464	703	3,858	3,686	8,023	9,206	26,824	18,267	40,908	32,715
55.0%	45.0%	3,045	2,332	623	873	5,163	5,292	10,668	11,533	37,805	24,879	53,370	42,984
60.0%	40.0%	4,134	2,999	883	1,045	7,131	6,710	14,234	14,376	47,497	33,221	71,843	54,994
65.0%	35.0%	5,321	3,984	1,211	1,241	9,225	8,393	18,076	18,587	61,125	45,782	88,631	73,743
70.0%	30.0%	6,622	4,888	1,521	1,470	11,757	10,596	23,588	22,868	79,418	65,486	103,849	92,557
75.0%	25.0%	8,405	6,029	2,217	1,824	14,846	12,991	28,116	29,924	98,188	79,181	130,400	127,491
80.0%	20.0%	10,811	7,705	3,078	2,418	19,379	17,072	35,927	36,748	123,556	104,573	171,682	151,680
85.0%	15.0%	13,673	10,382	4,480	3,096	25,781	22,466	46,575	45,590	155,803	135,489	208,709	190,183
90.0%	10.0%	18,784	14,228	8,361	4,370	35,820	30,500	62,134	58,028	198,349	175,216	255,076	243,622
93.0%	7.0%	21,825	17,406	11,975	5,443	43,397	36,793	76,704	78,355	257,081	216,641	322,727	306,866
95.0%	5.0%	24,891	19,863	15,947	6,296	49,700	43,180	89,430	92,857	290,876	284,076	382,976	375,193
96.0%	4.0%	26,864	21,407	17,862	6,954	54,159	45,865	96,410	95,949	323,213	314,282	421,932	418,985
97.0%	3.0%	30,469	22,901	19,541	8,289	61,368	50,686	105,126	103,312	345,471	343,599	432,516	458,729
98.0%	2.0%	35,099	26,692	27,108	11,373	77,062	57,164	122,428	121,197	379,523	377,268	480,264	501,764
99.0%	1.0%	39,638	33,484	35,168	13,319	91,093	79,347	147,879	151,390	431,441	445,099	562,465	569,985
99.1%	0.9%	40,419	34,369	36,952	14,273	92,034	81,036	148,070	154,008	445,392	451,806	583,688	574,870
99.99%	0.01%	65,795	57,578	68,143	30,362	141,161	119,938	208,524	211,279	606,742	673,524	733,092	877,480

STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE AND FROM WAM WITHOUT LAKE RALPH HALL

	RiverWare	WAM
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	79.6%	77.4%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	85.5%	83.9%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	FLOW AT LAKE RALPH HALL DAM SITE		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
%	%	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon
1.0%	99.0%	0	0	0	0	0	2	1	3	308	208	308	284
2.0%	98.0%	0	0	0	0	0	3	7	4	317	310	344	416
3.0%	97.0%	0	0	0	0	0	4	13	10	346	378	392	472
4.0%	96.0%	3	2	3	3	7	9	37	23	369	384	472	509
5.0%	95.0%	5	4	6	5	14	16	55	34	411	423	534	590
7.0%	93.0%	13	8	16	11	36	28	83	57	496	473	774	751
10.0%	90.0%	27	17	33	21	73	55	150	121	694	587	1,142	1,198
14.5%	85.5%	67	42	81	53	175	134	359	337	1,134	983	1,757	1,863
15.0%	85.0%	76	48	91	60	200	150	381	374	1,182	1,053	1,913	1,939
16.1%	83.9%	88	56	106	69	235	175	430	421	1,299	1,201	2,370	2,453
20.0%	80.0%	137	105	163	131	360	327	731	691	2,019	1,604	3,845	3,812
20.5%	79.6%	147	106	175	133	370	331	760	727	2,118	1,642	4,269	3,941
22.6%	77.4%	196	140	233	175	508	438	894	907	2,767	2,185	5,275	4,384
25.0%	75.0%	239	181	283	226	637	560	1,297	1,068	3,486	2,907	6,486	5,462
30.0%	70.0%	427	294	503	368	1,007	911	2,139	1,993	5,794	4,761	9,477	8,559
35.0%	65.0%	719	558	859	697	1,864	1,724	3,194	3,424	8,666	7,289	14,329	11,054
40.0%	60.0%	1,006	775	1,213	967	2,662	2,390	4,504	4,838	14,348	9,807	21,706	16,383
45.0%	55.0%	1,407	1,082	1,654	1,351	3,702	3,337	6,918	6,546	21,168	14,049	30,418	25,207
50.0%	50.0%	2,282	1,564	2,748	1,953	6,103	4,819	10,317	10,683	29,881	20,578	41,964	33,876
55.0%	45.0%	3,045	2,332	3,674	2,912	8,216	7,193	13,709	14,082	41,520	27,605	56,561	45,630
60.0%	40.0%	4,134	2,999	4,974	3,745	11,140	9,241	18,641	17,926	53,220	36,086	77,273	59,338
65.0%	35.0%	5,321	3,984	6,475	4,977	14,611	12,279	23,018	22,405	65,830	49,758	94,761	77,924
70.0%	30.0%	6,622	4,888	7,932	6,104	17,763	15,061	29,660	27,658	85,531	68,570	111,283	96,196
75.0%	25.0%	8,405	6,029	10,144	7,529	22,106	18,597	35,934	35,918	106,032	87,441	140,059	132,052
80.0%	20.0%	10,811	7,705	12,957	9,622	28,326	23,757	44,314	42,962	131,134	113,998	181,748	160,522
90.0%	10.0%	18,784	14,228	22,501	17,768	49,903	43,878	74,562	71,524	214,631	188,588	268,410	255,851
93.0%	7.0%	21,825	17,406	25,778	21,736	55,542	53,675	90,564	93,102	269,188	234,764	333,275	323,591
95.0%	5.0%	24,891	19,863	29,967	24,804	66,111	61,264	103,576	110,149	308,811	301,091	399,997	390,320
96.0%	4.0%	26,864	21,407	31,828	26,733	70,919	65,990	116,735	113,552	342,029	326,063	433,457	433,702
97.0%	3.0%	30,469	22,901	36,303	28,598	80,704	70,599	124,159	126,166	361,655	368,055	447,459	475,112
98.0%	2.0%	35,099	26,692	41,839	33,332	86,632	82,322	137,801	142,550	401,174	394,235	499,927	515,625
99.0%	1.0%	39,638	33,484	47,723	41,814	107,136	103,241	164,893	184,164	433,424	463,329	596,116	598,896
99.9%	0.1%	60,174	51,960	71,297	64,886	159,440	160,240	234,060	255,580	636,248	714,960	747,687	828,098
99.99%	0.01%	65,795	57,578	78,816	71,901	175,146	177,515	240,444	260,229	654,534	722,475	770,216	925,058

ATTACHMENT C

STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE WITH AND WITHOUT LAKE RALPH HALL

	With LRH	Without LRH
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	62.2%	79.6%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	82.1%	85.5%

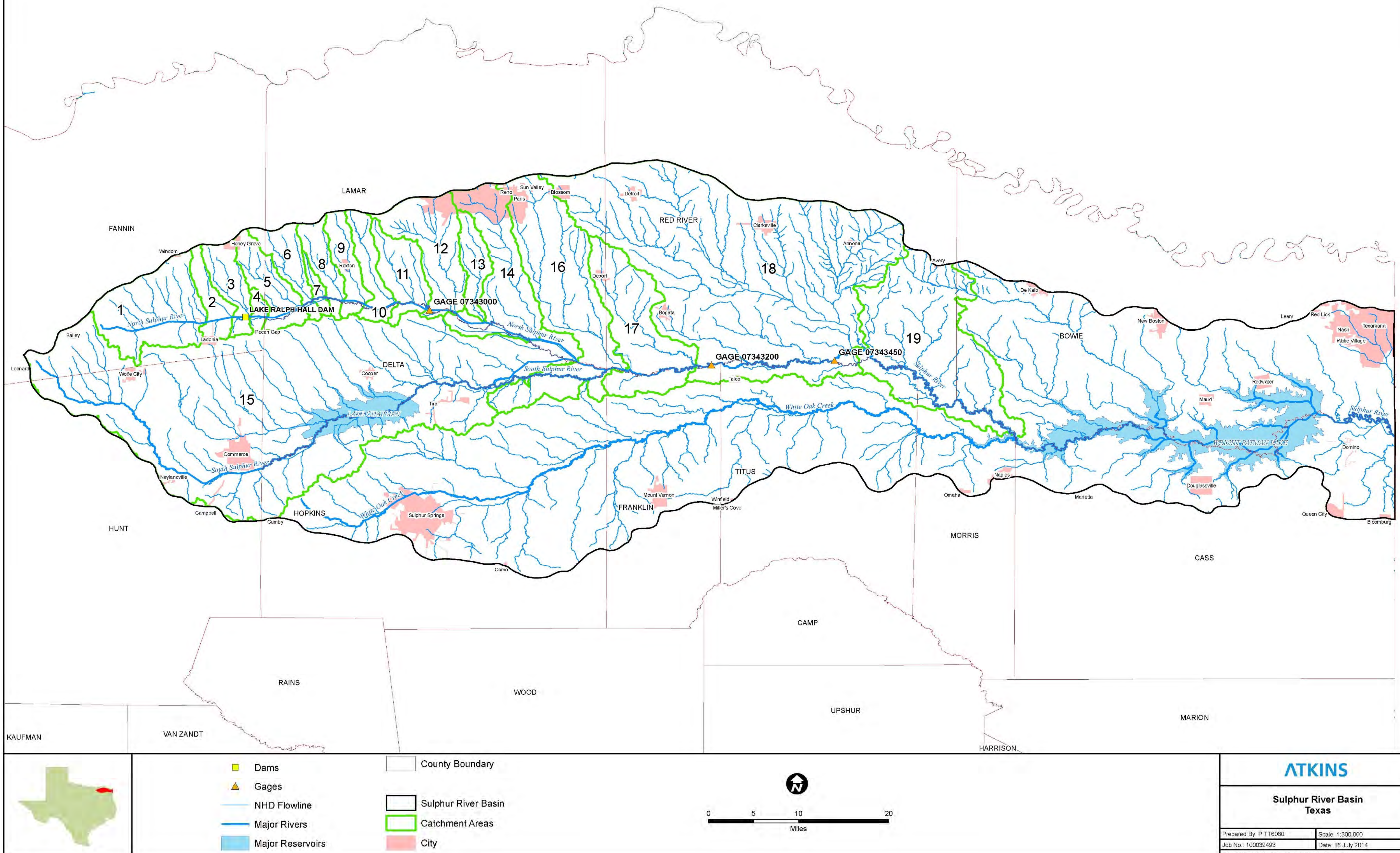
PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
		With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH
%	%	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon
1.0%	99.0%	0	0	0	0	0	0	1	1	308	308	308	308
2.0%	98.0%	0	0	0	0	0	0	5	7	316	317	341	344
3.0%	97.0%	0	0	0	0	0	0	11	13	343	346	369	392
4.0%	96.0%	3	3	1	3	4	7	30	37	350	369	442	472
5.0%	95.0%	5	5	1	6	9	14	38	55	394	411	527	534
7.0%	93.0%	13	13	3	16	22	36	63	83	455	496	720	774
10.0%	90.0%	27	27	5	33	45	73	114	150	658	694	1,046	1,142
14.5%	85.5%	67	67	13	81	106	175	283	359	1,008	1,136	1,661	1,759
15.0%	85.0%	76	76	14	91	115	200	288	381	1,051	1,182	1,740	1,913
18.0%	82.1%	111	111	21	134	175	281	420	528	1,463	1,599	2,802	3,012
20.0%	80.0%	137	137	26	163	217	360	510	731	1,727	2,019	3,657	3,845
20.5%	79.6%	147	147	27	175	228	370	550	760	1,835	2,118	3,788	4,269
25.0%	75.0%	239	239	46	283	385	637	985	1,297	3,086	3,486	5,774	6,486
30.0%	70.0%	427	427	74	503	622	1,007	1,495	2,139	4,750	5,794	8,313	9,477
35.0%	65.0%	719	719	136	859	1,133	1,864	2,494	3,194	7,525	8,666	13,183	14,329
37.8%	62.2%	901	901	175	1,072	1,464	2,331	2,974	3,938	10,207	11,672	17,118	18,683
40.0%	60.0%	1,006	1,006	200	1,213	1,653	2,662	3,481	4,504	12,496	14,348	19,602	21,706
45.0%	55.0%	1,407	1,407	289	1,654	2,401	3,702	5,245	6,918	18,340	21,168	28,830	30,418
50.0%	50.0%	2,282	2,282	464	2,748	3,858	6,103	8,023	10,317	26,824	29,881	40,908	41,964
55.0%	45.0%	3,045	3,045	623	3,674	5,163	8,216	10,668	13,709	37,805	41,520	53,370	56,561
60.0%	40.0%	4,134	4,134	883	4,974	7,131	11,140	14,234	18,641	47,497	53,220	71,843	77,273
65.0%	35.0%	5,321	5,321	1,211	6,475	9,225	14,611	18,076	23,018	61,125	65,830	88,631	94,761
70.0%	30.0%	6,622	6,622	1,521	7,932	11,757	17,763	23,588	29,660	79,418	85,531	103,849	111,283
75.0%	25.0%	8,405	8,405	2,217	10,144	14,846	22,106	28,116	35,934	98,188	106,032	130,400	140,059
80.0%	20.0%	10,811	10,811	3,078	12,957	19,379	28,326	35,927	44,314	123,556	131,134	171,682	181,748
85.0%	15.0%	13,673	13,673	4,480	16,198	25,781	35,713	46,575	58,546	155,803	163,200	208,709	218,084
90.0%	10.0%	18,784	18,784	8,361	22,501	35,820	49,903	62,134	74,562	198,349	214,631	255,076	268,410
93.0%	7.0%	21,825	21,825	11,975	25,778	43,397	55,542	76,704	90,564	257,081	269,188	322,727	333,275
95.0%	5.0%	24,891	24,891	15,947	29,967	49,700	66,111	89,430	103,576	290,876	308,811	382,976	399,997
96.0%	4.0%	26,864	26,864	17,862	31,828	54,159	70,919	96,410	116,735	323,213	342,029	421,932	433,457
97.0%	3.0%	30,469	30,469	19,541	36,303	61,368	80,704	105,126	124,159	345,471	361,655	432,516	447,459
98.0%	2.0%	35,099	35,099	27,108	41,839	77,062	86,632	122,428	137,801	379,523	401,174	480,264	499,927
99.0%	1.0%	39,638	39,638	35,168	47,723	91,093	107,136	147,879	164,893	431,441	433,424	562,465	596,116
99.9%	0.1%	60,174	60,174	61,662	71,297	133,926	159,440	194,211	234,060	597,068	636,248	725,870	747,687
99.99%	0.01%	65,795	65,795	68,143	78,816	141,161	175,146	208,524	240,444	606,742	654,534	733,092	770,216

STATISTICAL ANALYSIS OF FLOWS FROM WAM WITH AND WITHOUT LAKE RALPH HALL

	With LRH	Without LRH
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	73.0%	77.4%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	83.8%	83.9%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
		With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH
%	%	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon
1.0%	99.0%	0	0	0	0	2	2	3	3	208	208	284	284
2.0%	98.0%	0	0	0	0	3	3	4	4	310	310	416	416
3.0%	97.0%	0	0	0	0	4	4	10	10	378	378	472	472
4.0%	96.0%	2	2	3	3	9	9	23	23	384	384	509	509
5.0%	95.0%	4	4	5	5	16	16	34	34	423	423	590	590
7.0%	93.0%	8	8	9	11	28	28	57	57	473	473	751	751
10.0%	90.0%	17	17	19	21	54	55	121	121	587	587	1,180	1,198
15.0%	85.0%	48	48	47	60	149	150	364	374	1,053	1,053	1,919	1,939
16.1%	83.9%	56	56	53	69	167	175	421	421	1,190	1,201	2,329	2,453
16.2%	83.8%	57	57	53	72	175	180	425	425	1,201	1,206	2,389	2,506
20.0%	80.0%	105	105	93	131	290	327	677	691	1,539	1,604	3,713	3,812
22.6%	77.4%	140	140	113	175	381	437	874	906	2,070	2,182	4,275	4,377
25.0%	75.0%	181	181	148	226	531	560	1,057	1,068	2,708	2,907	5,251	5,462
27.0%	73.0%	210	210	175	262	612	651	1,365	1,393	3,706	4,016	6,034	6,184
30.0%	70.0%	294	294	216	368	801	911	1,925	1,993	4,630	4,761	8,534	8,559
35.0%	65.0%	558	558	279	697	1,417	1,724	3,058	3,424	6,802	7,289	10,734	11,054
40.0%	60.0%	775	775	399	967	2,002	2,390	4,583	4,838	9,491	9,807	15,409	16,383
45.0%	55.0%	1,082	1,082	580	1,351	2,687	3,337	5,949	6,546	12,596	14,049	23,245	25,207
50.0%	50.0%	1,564	1,564	703	1,953	3,686	4,819	9,206	10,683	18,267	20,578	32,715	33,876
55.0%	45.0%	2,332	2,332	873	2,912	5,292	7,193	11,533	14,082	24,879	27,605	42,984	45,630
60.0%	40.0%	2,999	2,999	1,045	3,745	6,710	9,241	14,376	17,926	33,221	36,086	54,994	59,338
65.0%	35.0%	3,984	3,984	1,241	4,977	8,393	12,279	18,587	22,405	45,782	49,758	73,743	77,924
70.0%	30.0%	4,888	4,888	1,470	6,104	10,596	15,061	22,868	27,658	65,486	68,570	92,557	96,196
75.0%	25.0%	6,029	6,029	1,824	7,529	12,991	18,597	29,924	35,918	79,181	87,441	127,491	132,052
80.0%	20.0%	7,705	7,705	2,418	9,622	17,072	23,757	36,748	42,962	104,573	113,998	151,680	160,522
90.0%	10.0%	14,228	14,228	4,370	17,768	30,500	43,878	58,028	71,524	175,216	188,588	243,622	255,851
93.0%	7.0%	17,406	17,406	5,443	21,736	36,793	53,675	78,355	93,102	216,641	234,764	306,866	323,591
95.0%	5.0%	19,863	19,863	6,296	24,804	43,180	61,264	92,857	110,149	284,076	301,091	375,193	390,320
96.0%	4.0%	21,407	21,407	6,954	26,733	45,865	65,990	95,949	113,552	314,282	326,063	418,985	433,702
97.0%	3.0%	22,901	22,901	8,289	28,598	50,686	70,599	103,312	126,166	343,599	368,055	458,729	475,112
98.0%	2.0%	26,692	26,692	11,373	33,332	57,164	82,322	121,197	142,550	377,268	394,235	501,764	515,625
99.0%	1.0%	33,484	33,484	13,319	41,814	79,347	103,241	151,390	184,164	445,099	463,329	569,985	598,896
99.9%	0.1%	51,960	51,960	30,086	64,886	108,282	160,240	207,607	255,580	666,987	714,960	779,543	828,098
99.99%	0.01%	57,578	57,578	30,362	71,901	119,938	177,515	211,279	260,229	673,524	722,475	877,480	925,058

ATTACHMENT D



ATTACHMENT E

SUMMARY OF ANALYSIS OF FILLING RIVER CHANNEL POOLS DOWNSTREAM OF LAKE RALPH HALL WITH NORTH SULPHUR RIVER FLOWS SIMULATED WITH RIVERWARE MODEL AND WITH WAM FOR CONDITIONS WITHOUT AND WITH LAKE RALPH HALL PROJECT

STATION NO.	WATER COURSE	LOCATION DESCRIPTION	DRAINAGE AREA sq. mi.	DISTANCE ABOVE N SULPHUR GAGE miles	VOLUME REQUIRED TO FILL ALL D/S POOLS ac-ft	POOL VOLUME IN EACH D/S REACH ac-ft	% OF TIME POOLS ARE FILLED		
							Without Lake Ralph Hall	With Lake Ralph Hall	Deviation From Without LRH Case
FROM RIVERWARE MODEL (06-26-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	- -	- -	- -	- -
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	92.7%	83.6%	-9.1%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	86.7%	73.2%	-13.5%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	85.8%	82.0%	-3.8%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	86.7%	86.3%	-0.4%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	85.8%	85.4%	-0.4%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	85.4%	83.6%	-1.8%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	89.8%	89.6%	-0.1%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	85.1%	82.7%	-2.3%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	- -	- -	- -	- -
FROM WAM (04-06-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	- -	- -	- -	- -
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	90.8%	90.2%	-0.6%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	84.8%	83.5%	-1.3%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	83.9%	83.8%	-0.1%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	85.4%	85.4%	0.0%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	83.9%	83.9%	0.0%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	83.3%	83.2%	-0.1%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	88.6%	88.6%	0.0%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	83.2%	83.0%	-0.1%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	- -	- -	- -	- -

MEMORANDUM

To: Ed Motley
CH2M-Hill

From: Bob Brandes
Kirk Kennedy

Subject: Lake Ralph Hall
RiverWare Modeling

Date: June 29, 2015

As directed by the Upper Trinity Regional Water District (UTRWD), we have responded to the request from Corps of Engineers Fort Worth Office (Corps) to operate the Corps' daily RiverWare model of the Sulphur, Cypress and Red River Basins under conditions without and with the Lake Ralph Hall Project. From the modeling results, we have extracted daily river flows at locations along the North Sulphur and Sulphur Rivers where computational nodes exist in the model, and we have analyzed these flows with regard to frequency of occurrence and the frequency of filling river channel pools along the segment of the North Sulphur River from the proposed Lake Ralph Hall dam site downstream to the USGS streamflow gage near Cooper (Gage No. 07343000). We also have extracted and analyzed the daily storage and diversions for Lake Ralph Hall as simulated with the RiverWare model.

The version of the RiverWare model provided by the Corps included the physical representation of Lake Ralph Hall, but it did not have any diversions specified for withdrawing water from the reservoir as proposed by the UTRWD. We incorporated the same diversion routine that was used in the WAM for the previous analyses of the impacts of Lake Ralph Hall on monthly river flows that were conducted in July of 2014. This routine allows monthly diversions equivalent to 45,000 acre-feet per year to be made from Lake Ralph Hall provided the beginning-of-month storage in the reservoir exceeds 27,500 acre-feet, with the monthly diversions reduced to the equivalent of 16,800 acre-feet per year when the storage falls below 27,500 acre-feet. As originally modeled with the WAM, this operating procedure was designed to protect a firm annual yield of 16,800 acre-feet for Lake Ralph Hall while allowing overdrafting of the reservoir up to the full authorized diversion amount of 45,000 acre-feet per year when adequate stored water is available in the reservoir.

The period of record for the hydrologic conditions simulated with the daily RiverWare model is 1938 through 2014, which encompasses the monthly hydrologic conditions simulated with the WAM that extend from 1940 through 1996. While the source and derivation of the monthly naturalized flows used in the WAM are well documented, we do not have information regarding the procedures used to develop the daily flows that are input into the RiverWare model; however, as will be demonstrated, it is apparent that historical flow data for the North Sulphur River from the USGS streamflow gage near Cooper, to the extent they are available, have been used for representing flow conditions in the RiverWare model for at least the upper segment of the North

6000 Maurys Trail
Austin, Texas 78730
robert.brandes@atkinsglobal.com
Office Phone: 512/342-3233
Mobile Phone: 512/461-1477

Sulphur River. As we have discussed before, the RiverWare model does not apply the prior appropriation doctrine for allocating available streamflows among existing water rights in the Sulphur Basin, so in the model no streamflows are ever required to be passed downstream during water shortage periods by the more junior water rights to satisfy the demands of the more senior water rights. Furthermore, it appears that the only demands associated with existing water rights in the entire Sulphur Basin that are included in the RiverWare model are those for Lake Chapman and Lake Wright Patman; all other water rights are not represented. The WAM includes all existing water rights in the Sulphur Basin, with total authorized diversions of about 500,000 acre-feet per year, and allocates water to these water rights in order of seniority as required under Texas state law; so in the WAM, Lake Ralph Hall, with its relatively junior priority, must pass inflows downstream whenever senior water rights are not fully satisfied. These differences in the models regarding how streamflow allocations are made to existing water rights are reflected in their respective simulated river flows.

Since the WAM uses a monthly time step for performing water availability simulations, the underlying purpose for applying the daily RiverWare model was to be able to evaluate daily flow variations under conditions without and with Lake Ralph Hall. Therefore, the first set of results presented herein consists of plots of simulated daily flows, expressed in cubic feet per second (cfs), at USGS Gage No. 07343000 on the North Sulphur River near Cooper (see Attachment A) and at Gage No. 07343200 on the Sulphur River near Talco (see Attachment B). These depictions of daily flows illustrate conditions on the eroded and degraded segment of the North Sulphur River, as well as on the more natural segment of the Sulphur River below the confluence with the South Sulphur River and also below the infamous log jam. Graphs of daily flows covering one calendar year each are presented for 1956, 1980, 1992 and 2011, with two graphs with maximum flow scales of 500 cfs and 5,000 cfs provided for each year. The selected years are characterized by periods of extremely low flows (1956 and 2011), varying flows (1980), and very high flows (1992). As expected, these plots of daily flows without and with Lake Ralph Hall indicate some reduction in peak flows for individual flood events as a result of the reservoir, with these reductions more pronounced at the upper gage on the North Sulphur River. The peak flow reductions are less pronounced at the lower gage on the Sulphur River, as would be expected with the increased tributary inflows from the intervening watershed. Since the major reductions in peak flows are limited to the eroded and degraded channel of the North Sulphur River where overbanking of adjacent floodplain areas typically does not occur, the impacts of these reduced peak flows are not likely to be significant.

We have also compiled the daily simulated flows from the RiverWare model into monthly values to better provide meaningful comparisons of conditions without and with Lake Ralph Hall and to facilitate comparisons with the results from the WAM. Attachment C contains a group of plots and tables illustrating these comparisons for locations along the North Sulphur and Sulphur Rivers where the RiverWare model has computational nodes.

The first two plots on pages 1 and 2 of Attachment C show the storage in Lake Ralph Hall and the diversions from the reservoir as simulated with the RiverWare model and with the WAM. As illustrated, the simulated storage in the reservoir is considerably greater for the RiverWare model, with substantially more spills from the reservoir downstream into the North Sulphur River. As shown on the graph on page 2, during these higher storage periods, more water is able to be diverted from the reservoir since the criterion for making diversions up to the fully authorized amount of 45,000 acre-feet/year is satisfied more often.

The disparity between the storage results for Lake Ralph Hall from the RiverWare model and the WAM leads to questions as to the source and magnitude of the inflows to the reservoir as simulated with the two models, notwithstanding the fact that the RiverWare model ignores water rights and does not require junior water rights to pass flows to downstream senior water rights during times of water shortage. It is assumed that both models utilize historical flow data from the gage on the North Sulphur River near Cooper as the underlying basis for their specified river flow inputs for this segment of the overall river system network. This has been confirmed by comparing the simulated flows in the river at this gage location without Lake Ralph Hall in operation. As shown on the graph on page 3 of Attachment C, the monthly flow values from the two models at the gage location and the corresponding measured monthly flows at the gage are essentially the same over the common period of the model simulations when the gage was in operation (which began in October 1949). This analysis rules out the possibility that different sources of flow data were used for the upper segment of the North Sulphur River in the two models. However, when this same comparison is made of the simulated inflows to Lake Ralph Hall approximately 20 miles upstream from the gage, differences are noted between the two models. The graph on page 4 of Attachment C indicates that the simulated inflows to Lake Ralph Hall for the RiverWare model generally are higher than those for the WAM. This graph also indicates that apparently different base flows were used in the models prior to the existence of the gage in 1949, possibly due to the application of different data fill-in techniques. The graph on page 5 of Attachment C presents a time-series plot of the cumulative inflows to Lake Ralph Hall as simulated with the two models for the common period when the gage was in operation beginning in 1950, and it further illustrates the differences in these two sets of inflows, with the total cumulative deviation over 50 years approaching about 500,000 acre-feet. The differences in the inflows to Lake Ralph Hall during the period when gage flow records are available may be due to the fact that the RiverWare model uses a daily time step, with various flow routing parameters and lag coefficients to account for the movement of water downstream, whereas the WAM uses a monthly time step with no time adjustments other than those reflected in the flow data themselves. In any event, these differences in the inflows to Lake Ralph Hall between the two models are worthy of note, and they are likely reflected in the simulated flows downstream and must be considered when evaluating results.

A plot of the monthly simulated outflows from Lake Ralph Hall for the two models is presented on the graph on page 6 of Attachment C, again illustrating the significant spills from the reservoir as simulated with the RiverWare model. Inflows periodically passed downstream for satisfying the demands of senior water rights also are indicated on this plot by the WAM flows during dry periods. Monthly flows from the RiverWare model at the location of the first tributary downstream of Lake Ralph Hall (Baker Creek), which enters the North Sulphur River approximately one mile below the dam, are plotted with two different scales on the graphs on pages 7 and 8 of Attachment C for conditions without and with Lake Ralph Hall. Both plots illustrate the obvious; more flow is in the river downstream without Lake Ralph Hall than with it. The graph on page 9 of Attachment C depicts similar results at the location of the gage on the North Sulphur River near Cooper, but it is interesting to compare the flow magnitudes in this graph with those in the graph on page 8, both of which are plotted at the same flow scale. This comparison clearly illustrates the significant effect of flows that enter the river downstream of Lake Ralph Hall from tributaries, even with the reservoir in operation.

Finally, the tables on pages 10 through 13 present statistical results for the simulated monthly flows from the RiverWare model and from the WAM. Flows corresponding to specific percentiles and exceedance frequencies are indicated for the RiverWare model and the WAM and for conditions without and with Lake Ralph Hall in operation. These values are presented at locations where the RiverWare model has computational nodes, plus one additional location at the confluence of Baker Creek with the North Sulphur River. These locations can be identified on the map of the Sulphur River Basin in Attachment D, and they include upstream of Lake Ralph Hall for the inflow to the reservoir, below Lake Ralph Hall immediately downstream of Baker Creek (Catchment 3 on the map), at the North Sulphur River gage near Cooper (Gage No. 07343000 on the map), at the proposed site for the Parkhouse 2 Reservoir on the North Sulphur River (immediately below Catchment 14 on the map), at the Sulphur River gage near Talco (Gage No. 07343200 on the map), and at the proposed site of the Marvin Nichols Reservoir on the Sulphur River (immediately below Catchment 18 on the map). Flows from the RiverWare model at the Baker Creek location have been derived by adding to the simulated outflows from Lake Ralph Hall the incremental inflow from the watershed between the reservoir and Baker Creek, including Baker Creek. This incremental inflow was calculated by applying a drainage area ratio to the total simulated incremental inflow from the watershed between the reservoir and the North Sulphur River gage near Cooper. Comparisons of statistical results are presented for flows from the RiverWare model and from the WAM with Lake Ralph Hall (page 10) and without Lake Ralph Hall (page 11), for flows from the RiverWare model with and without Lake Ralph Hall (page 12), and for flows from the WAM with and without Lake Ralph Hall (page 13). As shown on each table, for flows at the Baker Creek location and at the North Sulphur River gage near Cooper, the exceedance frequencies have been determined for a flow of 175 acre-feet/month, which is the flow volume determined by Dr. Norman Johns of the National Wildlife Foundation as that needed to completely fill all of the downstream pools in the channel of the North Sulphur River from Baker Creek to the gage on the river near Cooper. While these exceedance frequencies provide some insight as to the effects of using the different models and the impacts of Lake Ralph Hall itself, a more in-depth analysis of downstream pool filling is discussed below.

Attachment E presents a summary of the results from the downstream pool filling analyses performed using monthly flows simulated with the RiverWare model by applying the same procedures previously employed (April 2015) for analyzing pool filling with WAM flows at the same locations. These previous results from analyzing the WAM flows also are included at the bottom of this table for reference purposes. This table presents the % of Time Pools Are Filled, on a monthly basis, under conditions without and with Lake Ralph Hall in operation for each of the reaches between tributaries for the segment of the North Sulphur River from the Lake Ralph Hall dam down to the North Sulphur River streamflow gage near Cooper. These values were derived by analyzing the monthly flows as simulated with the RiverWare model and the WAM at each of these locations to determine if they are sufficient to fill the pools in each of the downstream reaches based on Dr. Johns' pool volume estimate of 175 acre-feet for the total dam-to-gage reach. The intervening values of the flow volume required for filling the pools in each of the reaches were derived by making proportional adjustments of the 175 acre-foot value based on river channel distance below the dam. This assumes that the total pool volume is linearly distributed along this segment of the river channel. As shown in the table, and as expected, the values of Volume Required to Fill All Downstream Pools decrease with distance below the Lake Ralph Hall dam since the volume of pools decreases. The value of the % of Time Pools Are Filled at a particular location reflects the use of river flows to fill upstream pools, increases in river flows in the downstream direction with added tributary inflows, and the different pool volumes as they vary by reach. The monthly river flows from the RiverWare model at each of these locations were derived using the same approach described above for determining the river flows at the Baker

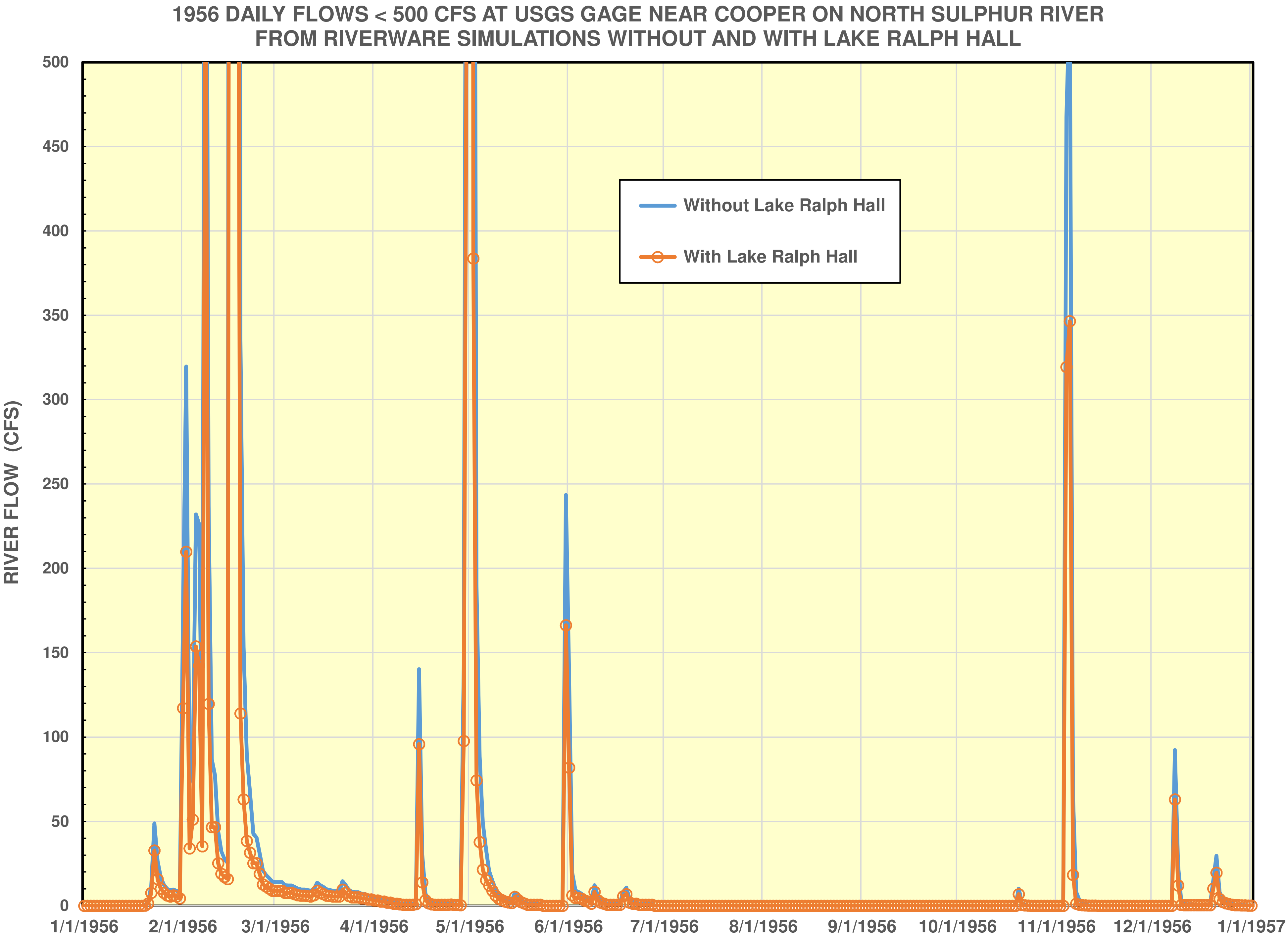
Creek location based on the simulated RiverWare flows at the dam and at the downstream gage near Cooper. As noted, the maximum reduction in the % of Time Pools Are Filled from the Without Lake Ralph Hall case to the With Lake Ralph Hall case for the RiverWare results is 13.5%, with the second largest reduction equal to 9.7%. For the WAM flows, these maximum reductions are 0.6% and 1.3%, respectively. As expected, both of these sets of higher reductions occur in reaches of the river closest to Lake Ralph Hall. Beginning at a point about half way down the river between Lake Ralph Hall and the gage, the reductions are substantially less, generally at levels considered to be within the simulation accuracy of the models considering the sources and accuracy of data and the simulation procedures used in the models. Over the entire segment of the North Sulphur River from Lake Ralph Hall down to the gage, the reach length-weighted average reduction in the % of Time Pools Are Filled from the Without Lake Ralph Hall case to the With Lake Ralph Hall case is -5.9% for the RiverWare flows and -0.5% for WAM flows.

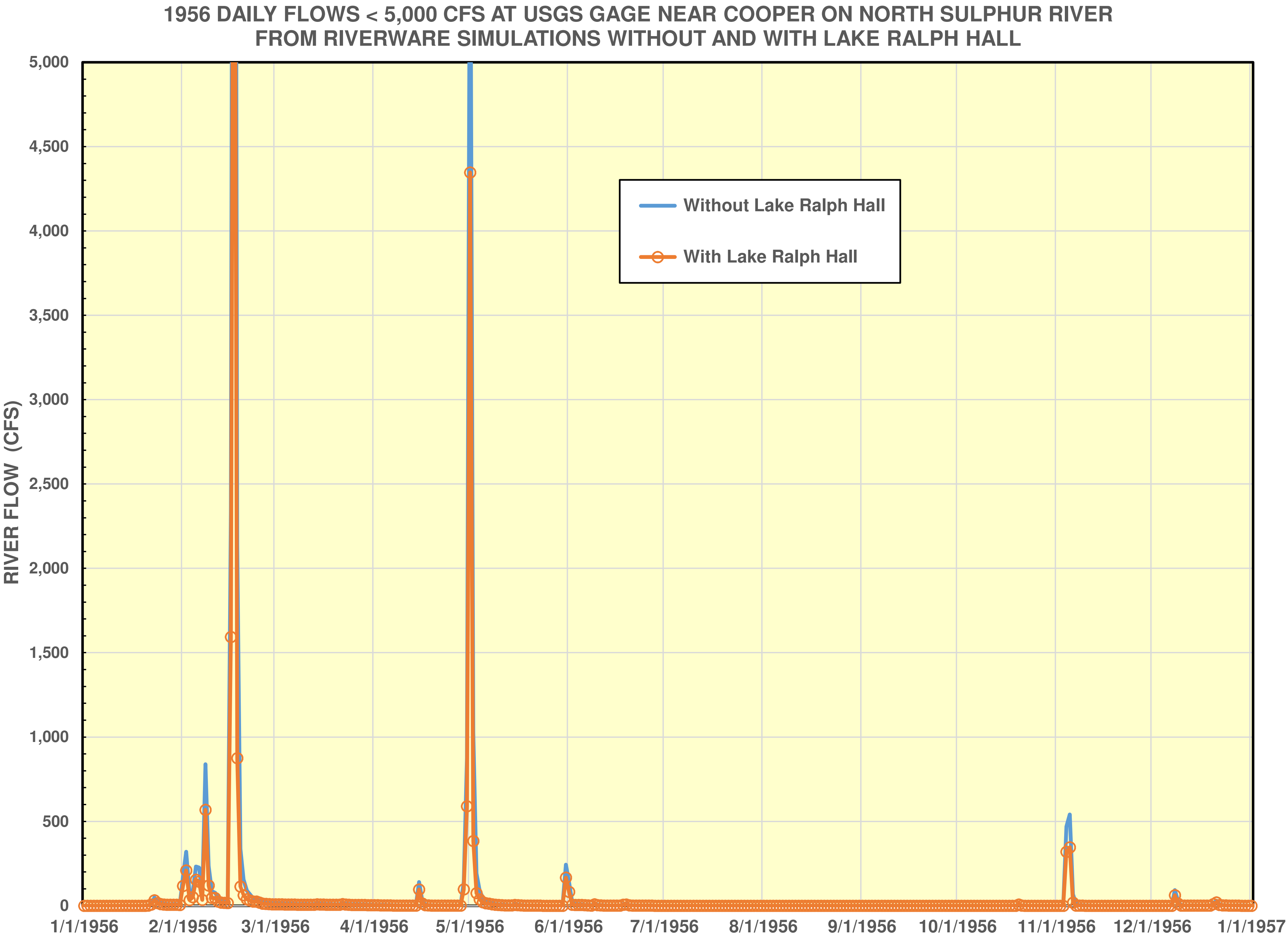
While the RiverWare model does provide daily simulations of flows in the North Sulphur and Sulphur Rivers, it is apparent from comparisons of these flows under conditions without and with Lake Ralph Hall that the daily variations themselves really do not tell us much more, if anything, about the effects of Lake Ralph Hall than monthly flow values. From the graphs of daily flows in Attachments A and B, it is shown that flood hydrographs occur at generally the same frequency and duration without or with Lake Ralph Hall. It is only the peaks of these hydrographs that are somewhat reduced due to the effects of Lake Ralph Hall, and peak flood flows in the North Sulphur River, unless they are associated with significant flood events on the order of the 25-year flood or greater, do not produce overbanking conditions that normally might be considered important from an aquatic ecological perspective. The incised channel of the North Sulphur River upstream of and for some distance downstream of the gage near Cooper simply is too deep to allow overtopping by the vast majority of flood events and too steep-walled to support and maintain typical lower floodplain conditions. Farther downstream, as inflows continue to enter the North Sulphur River and the Sulphur River below the confluence with the South Sulphur River, the reduction of river flows caused by Lake Ralph Hall becomes relatively less significant, to the point that the reservoir likely has minimal impact on instream and floodplain conditions.

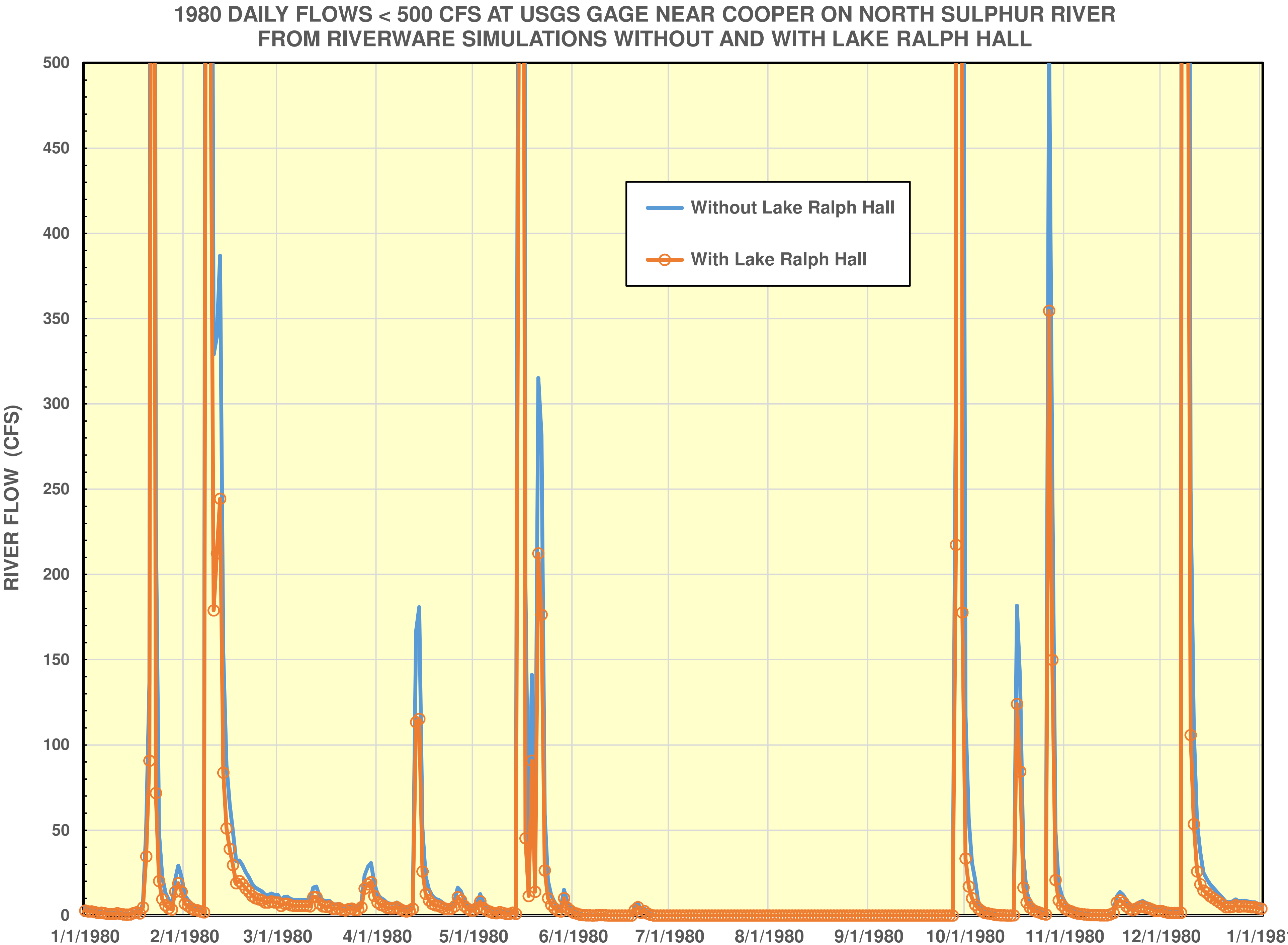
When considering the results from the RiverWare model of the Sulphur, Cypress and Red River Basins, it also is important to note that some of the deficiencies of the model could be relevant with respect to evaluating the impacts of Lake Ralph Hall. The exclusion of existing water rights from the model and the prior appropriation doctrine precludes any passing of inflows through the reservoir to satisfy the demands of downstream senior water rights. These additional flows in the river, which the WAM does model, could serve to supplement tributary inflows for filling channel pools and supporting aquatic life downstream of the reservoir. While typically the passing of flows for satisfying senior water rights only occurs during extremely dry periods when a "call" is made by the downstream senior water rights, it is not something that would never occur as the RiverWare model assumes. With the construction and operation of Lake Ralph Hall, it is very likely that owners of existing downstream water rights, especially those with large irrigation rights located near or below the confluence of the North and South Sulphur Rivers, as well as Lake Wright Patman located farther downstream on the Sulphur River, will closely monitor their available water supplies from the river and will certainly issue a call for Lake Ralph Hall to pass inflows to meet their needs if they believe Lake Ralph Hall is depriving them of flows to which they are entitled. In this regard, the WAM probably provides a better estimate of low flow conditions in the North Sulphur River with Lake Ralph Hall in operation than the daily RiverWare model does. Another point to note relates to the higher level of inflows to Lake Ralph Hall that the RiverWare model produces. It is not clear as to why this occurs, but it definitely affects the operation of the reservoir and may artificially increase the frequency of flood spills from the reservoir that flow into the river downstream.

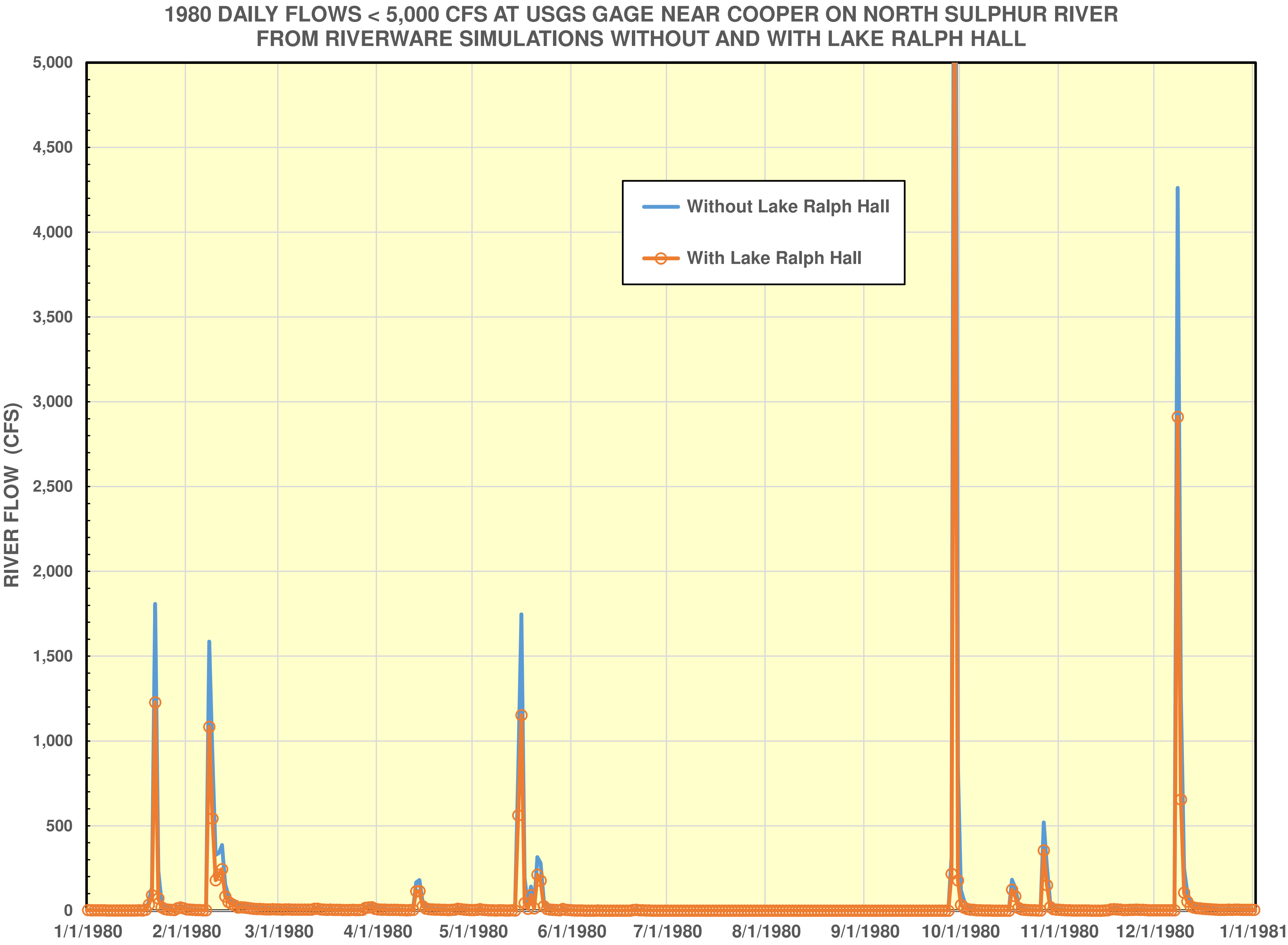
In summary, the application of the daily RiverWare model for analyzing the effects of Lake Ralph Hall on downstream river flows is considered to have been a worthwhile effort. It has provided a better understanding of the significance of daily variations in river flows and how Lake Ralph Hall might affect those flow variations and flood hydrographs, information that may be useful for further evaluating the impacts of Lake Ralph Hall. In the end, however, it remains that the place where Lake Ralph Hall will likely have its most significant effect on the flow regime of the North Sulphur and Sulphur Rivers is still the segment immediately downstream of the reservoir that is characterized by an eroded and degraded channel devoid of significant aquatic life such that reductions in river flows caused by the reservoir are not likely to result in noticeable environmental impacts. Even then, the UTRWD is proposing to develop and construct the mitigation area on the south floodplain of the North Sulphur River below the reservoir by restoring the configuration of approximately 14,000 feet of the abandoned river channel, planting native vegetation and trees, and stocking the restored pools and channel with fish and aquatic species that typically inhabited the historical river system.

If you have any questions regarding the material presented herein or if you want to discuss these results further, please contact us at your convenience. Also, we are in the process of assembling the RiverWare results files and the various spreadsheets used in analyzing and presenting the results for delivery to the Corps.

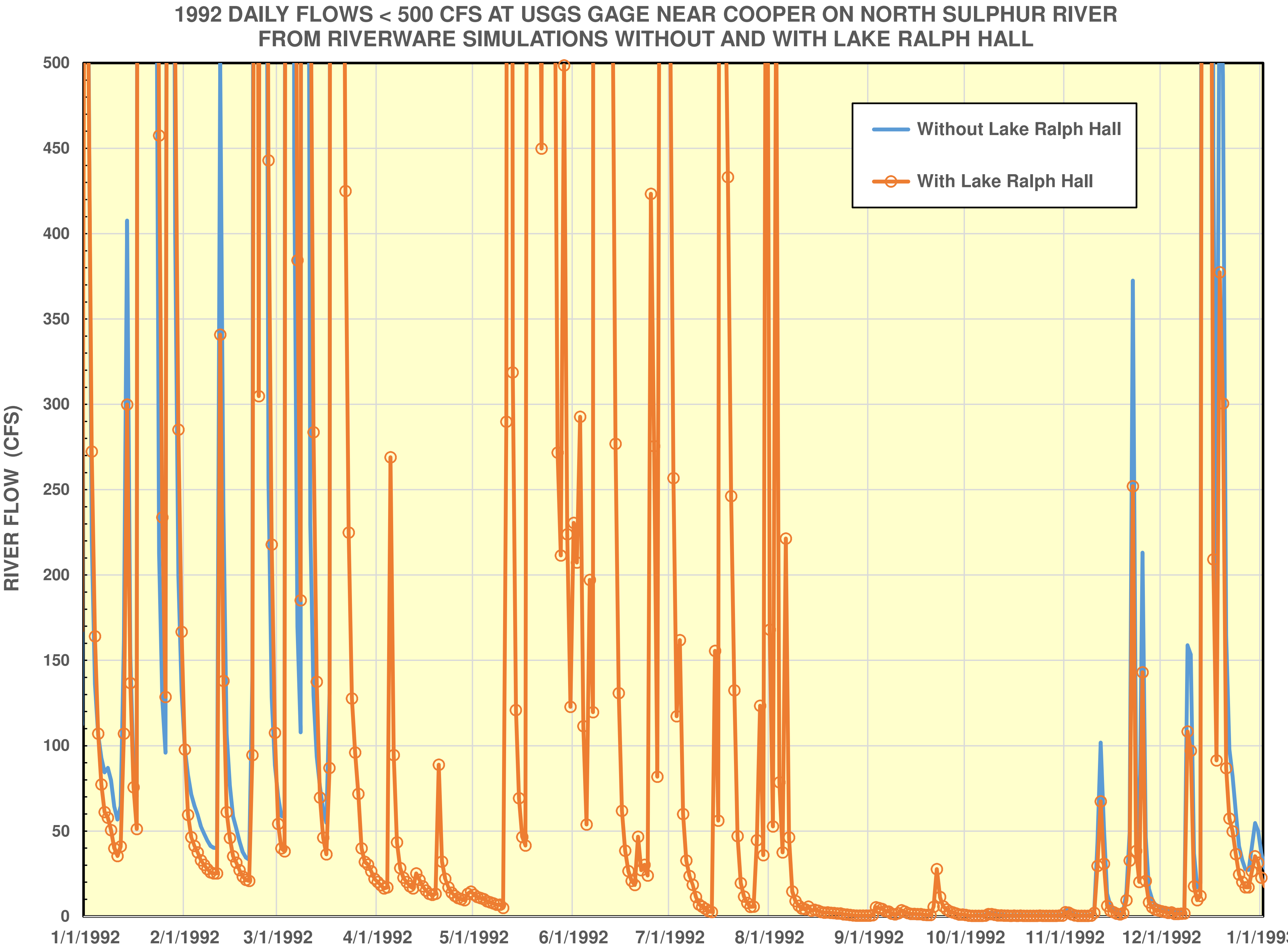

1956 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957
Without Lake Ralph Hall
With Lake Ralph Hall

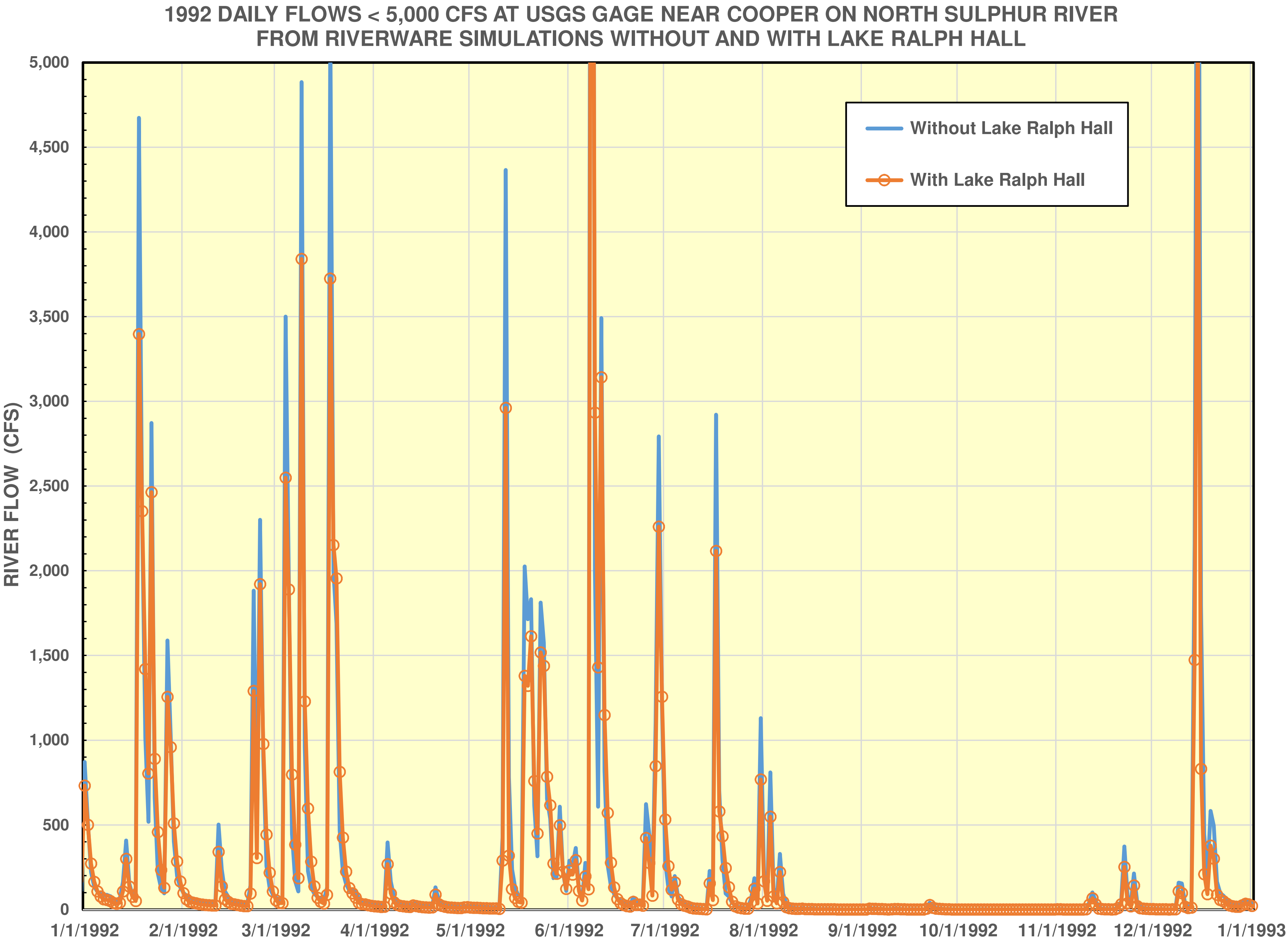

1956 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957
Without Lake Ralph Hall
With Lake Ralph Hall

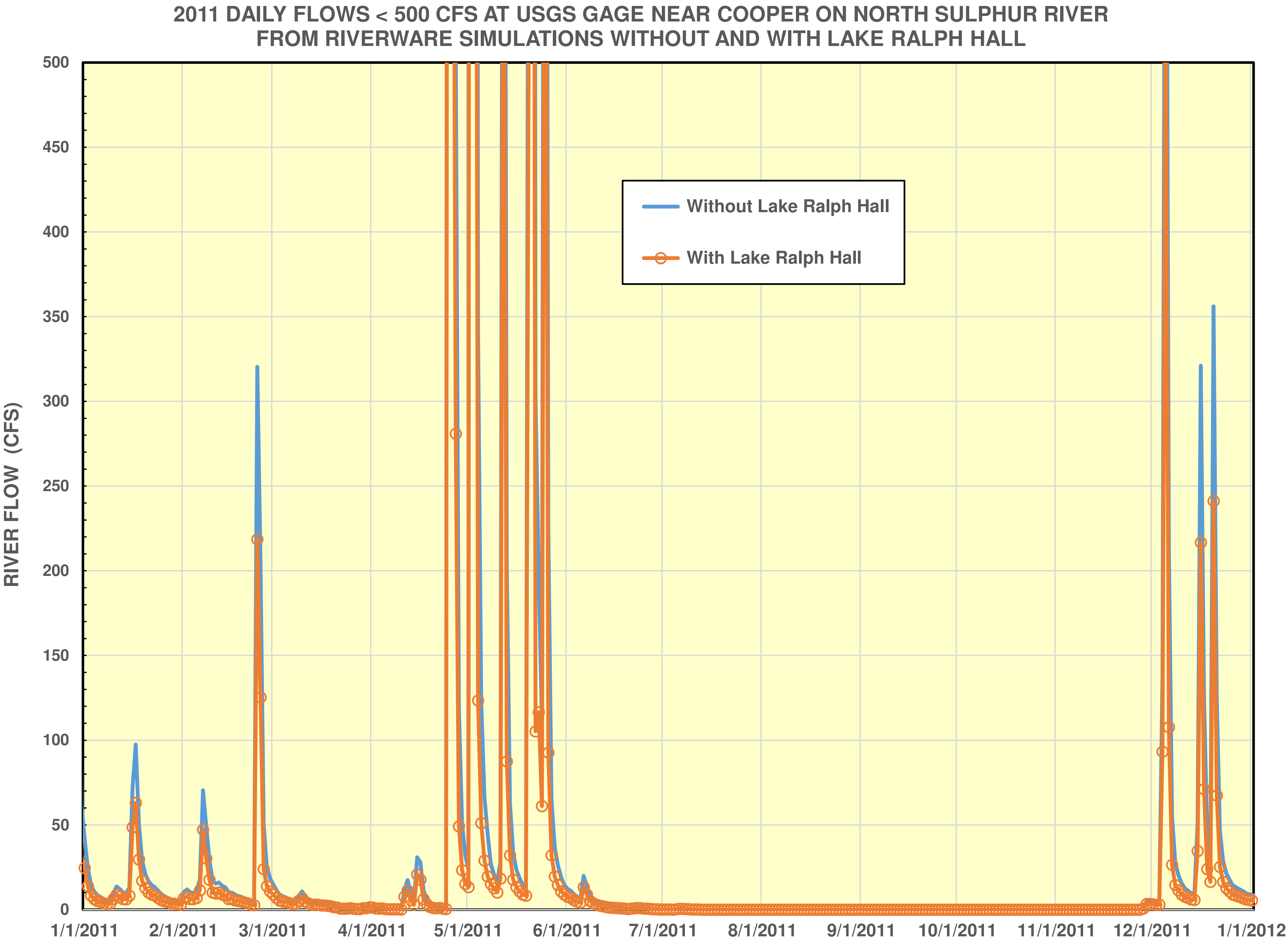

1980 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
0
50
100
150
200
250
300
350
400
450
500
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall

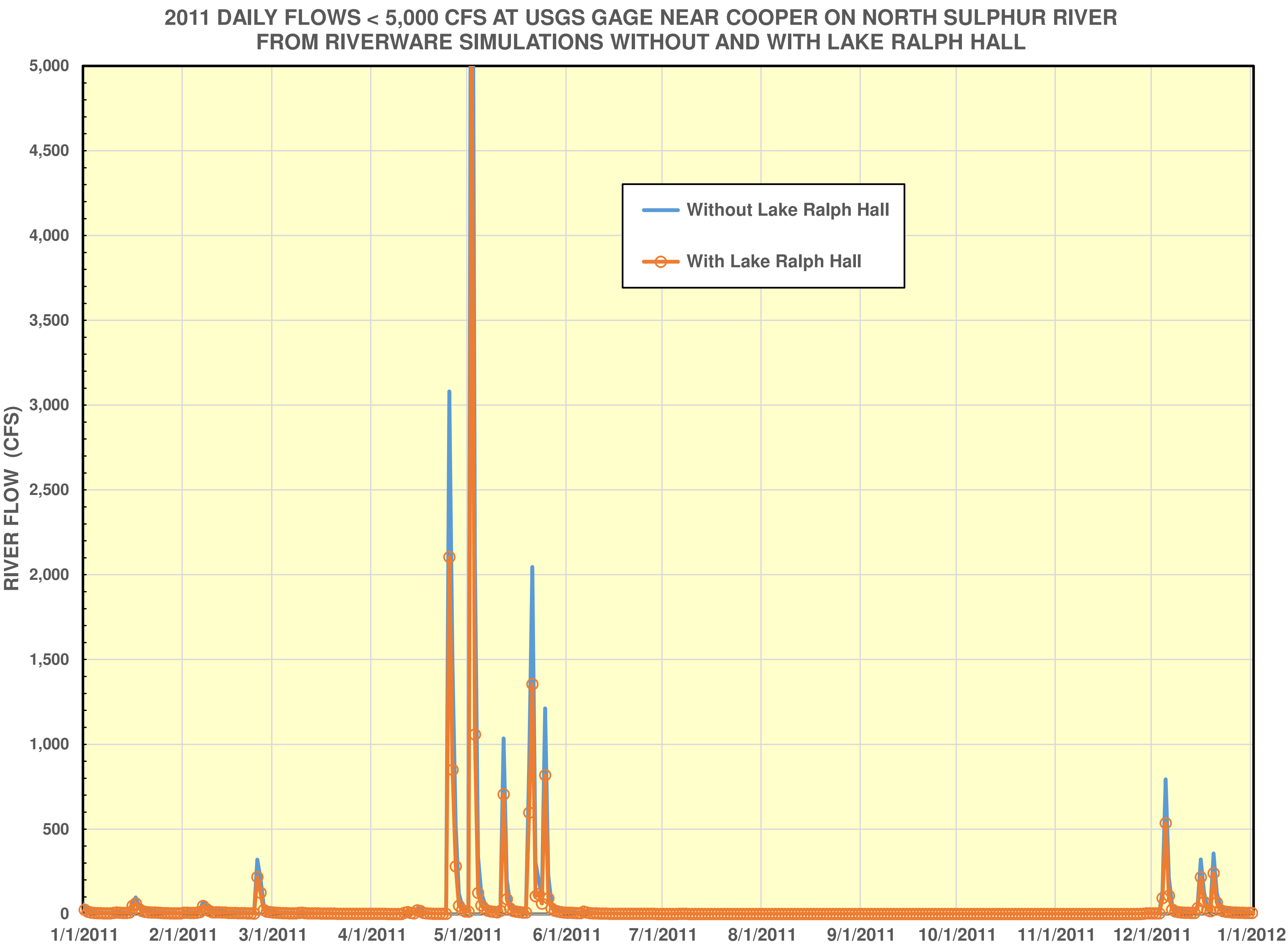


1980 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981

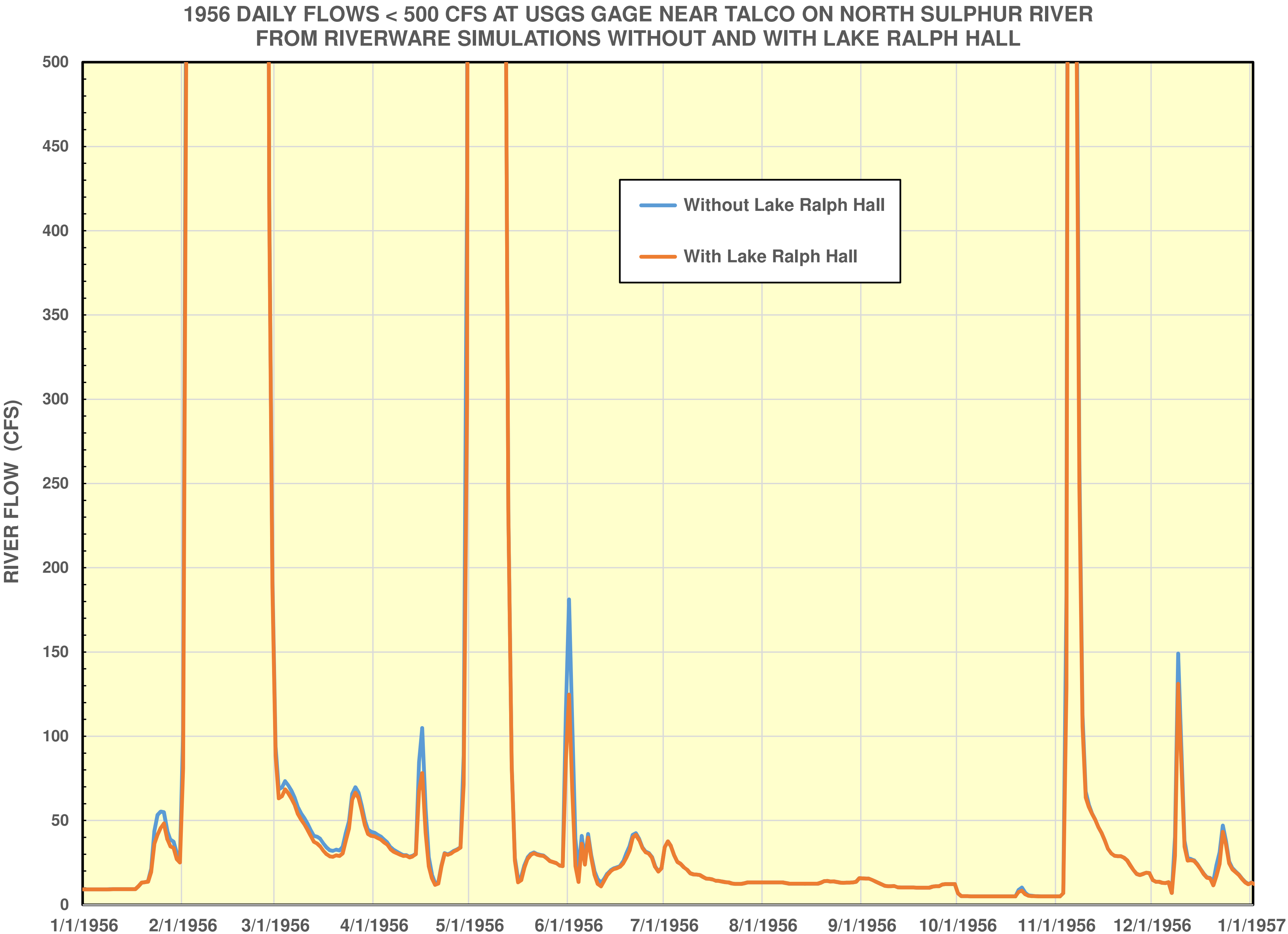

1992 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall


1992 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall

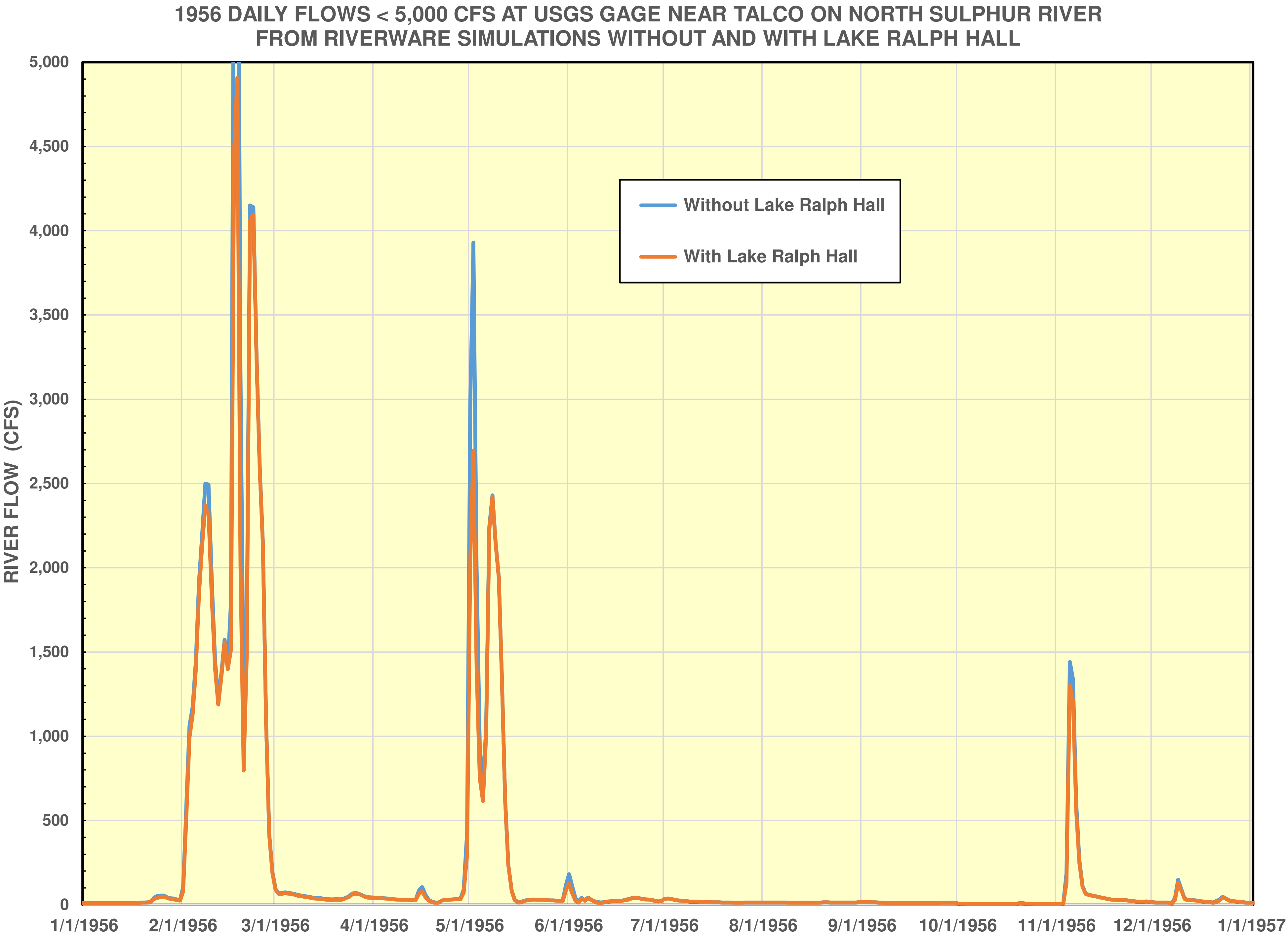

2011 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012

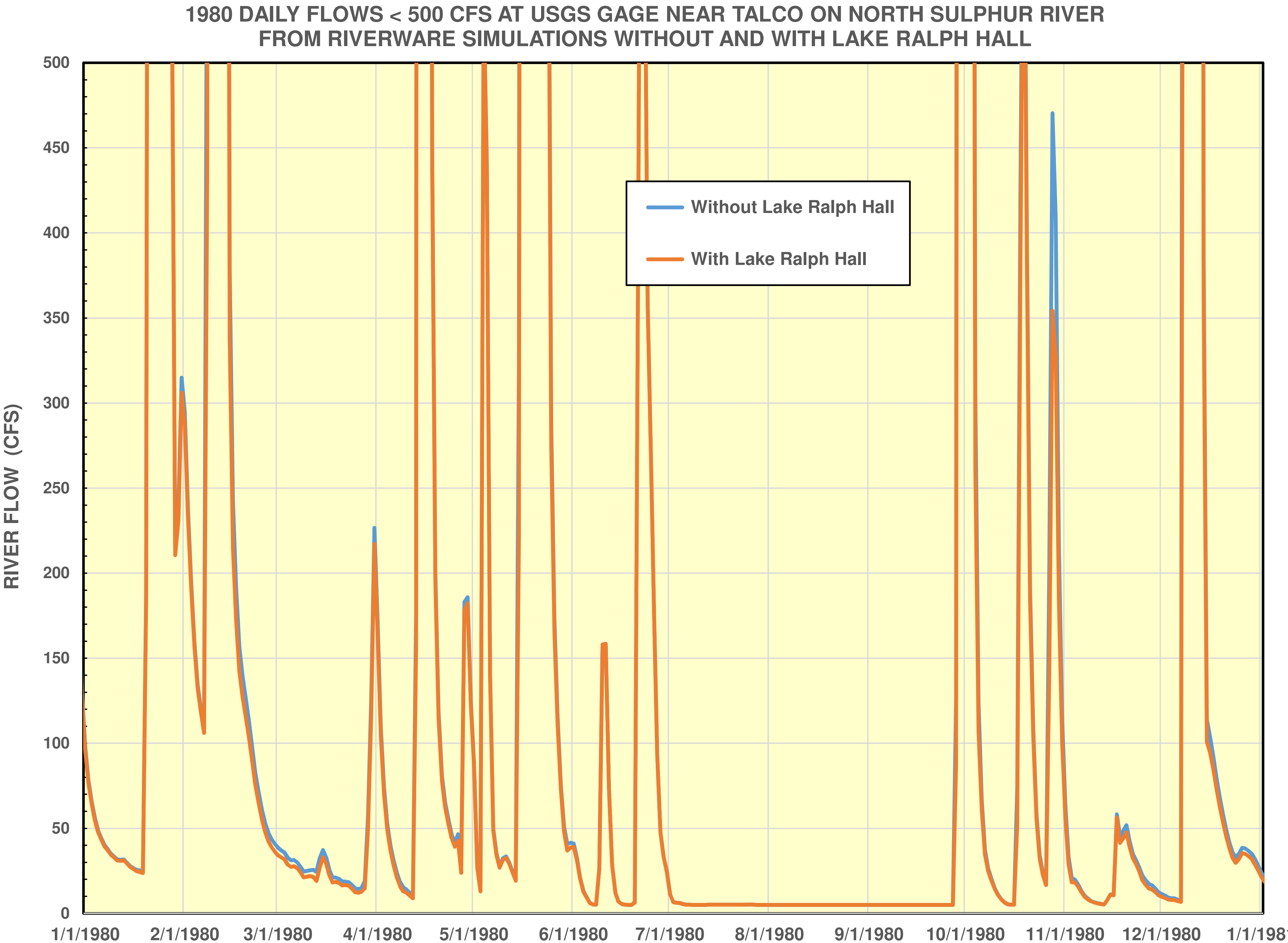


2011 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR COOPER ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012
Without Lake Ralph Hall
With Lake Ralph Hall

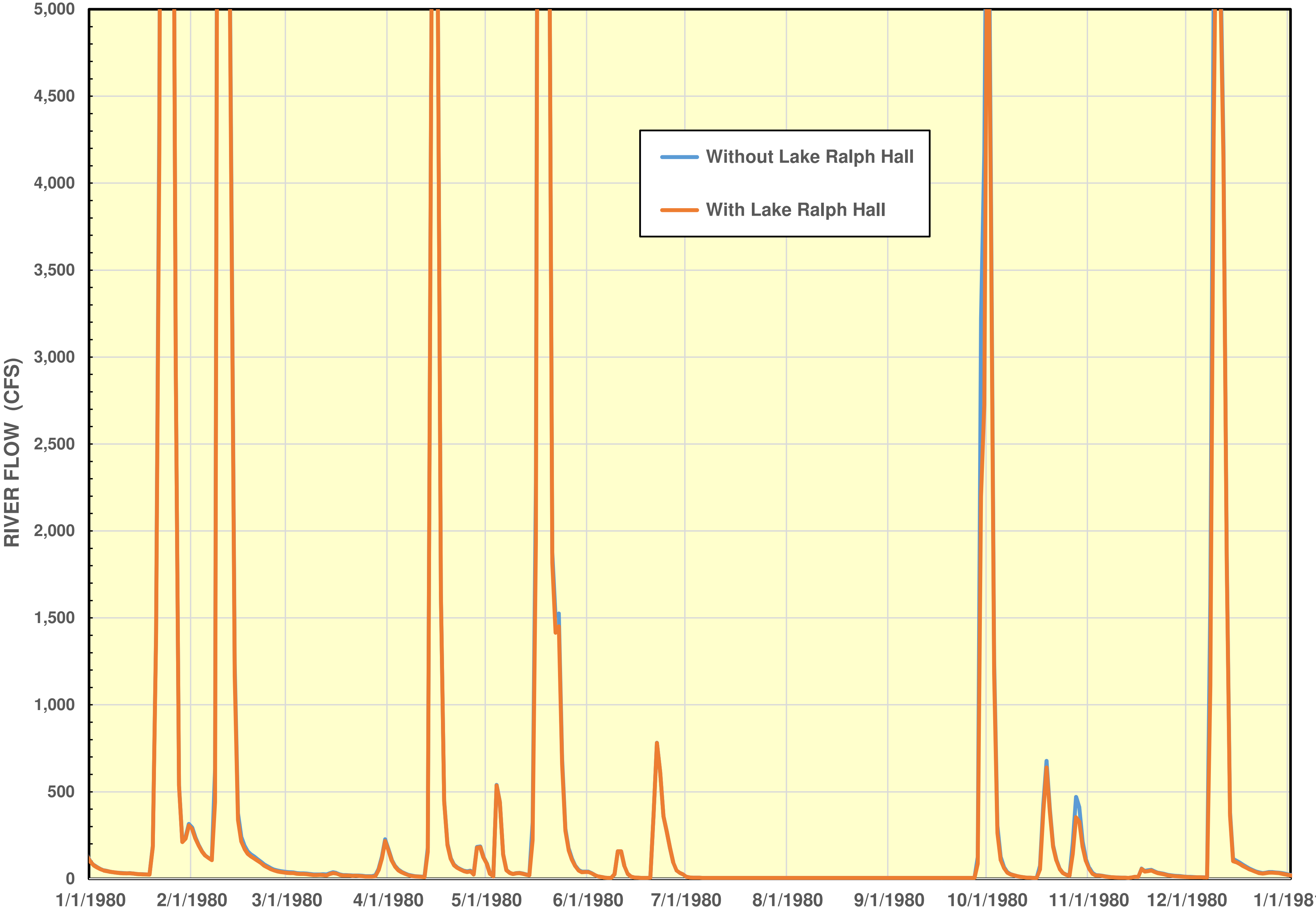


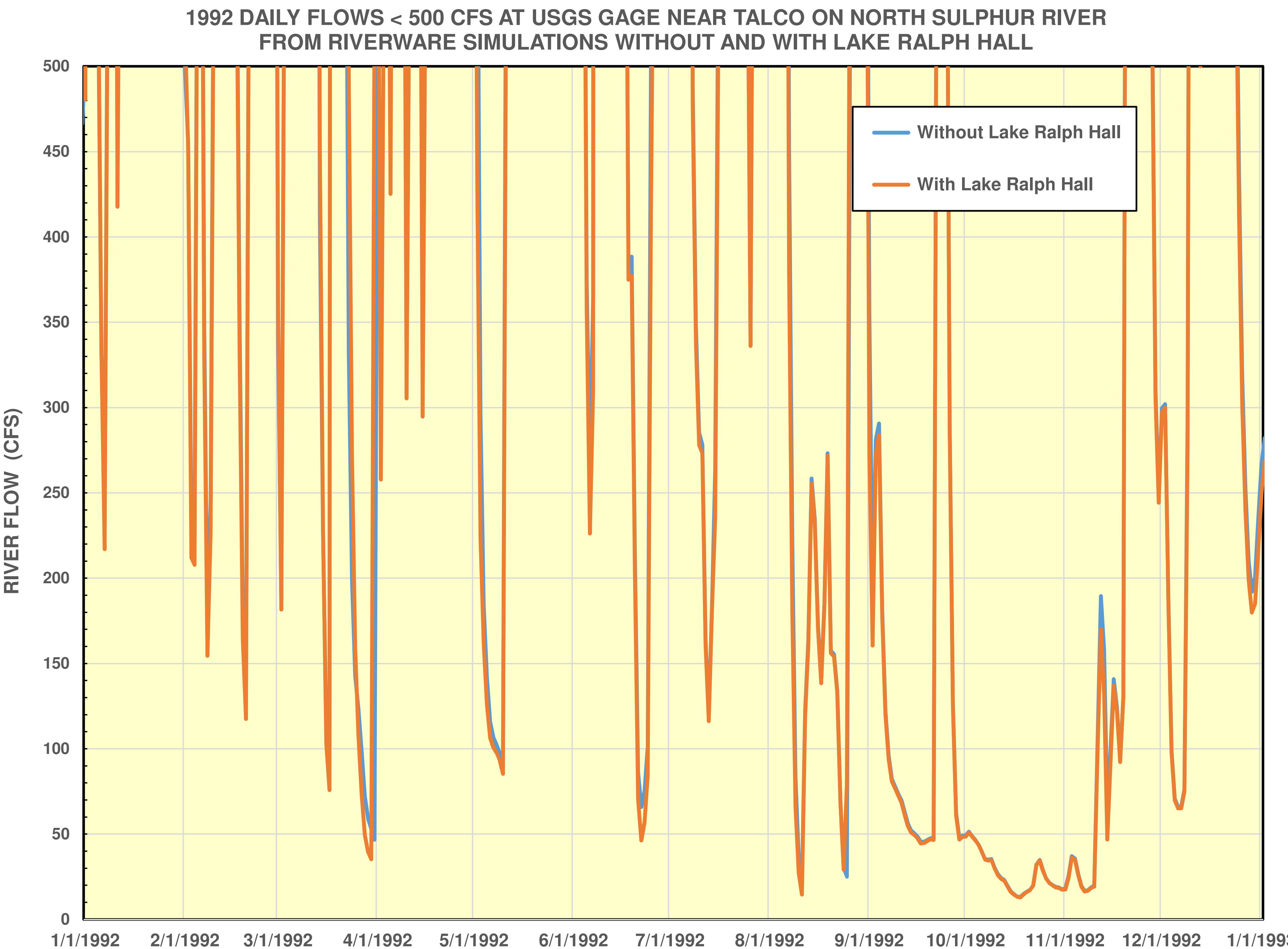
1956 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957

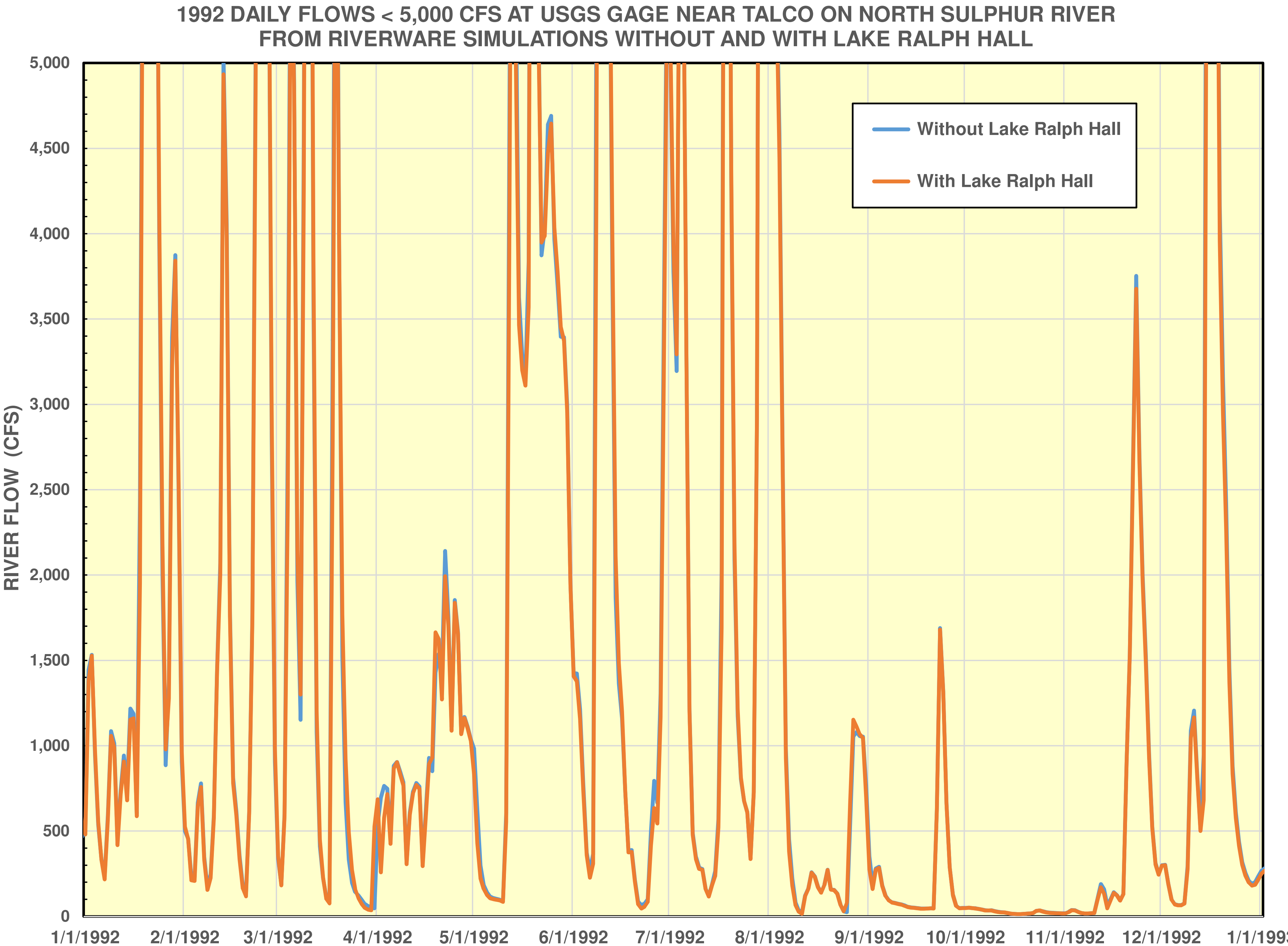

1956 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1956
2/1/1956
3/1/1956
4/1/1956
5/1/1956
6/1/1956
7/1/1956
8/1/1956
9/1/1956
10/1/1956
11/1/1956
12/1/1956
1/1/1957
Without Lake Ralph Hall
With Lake Ralph Hall

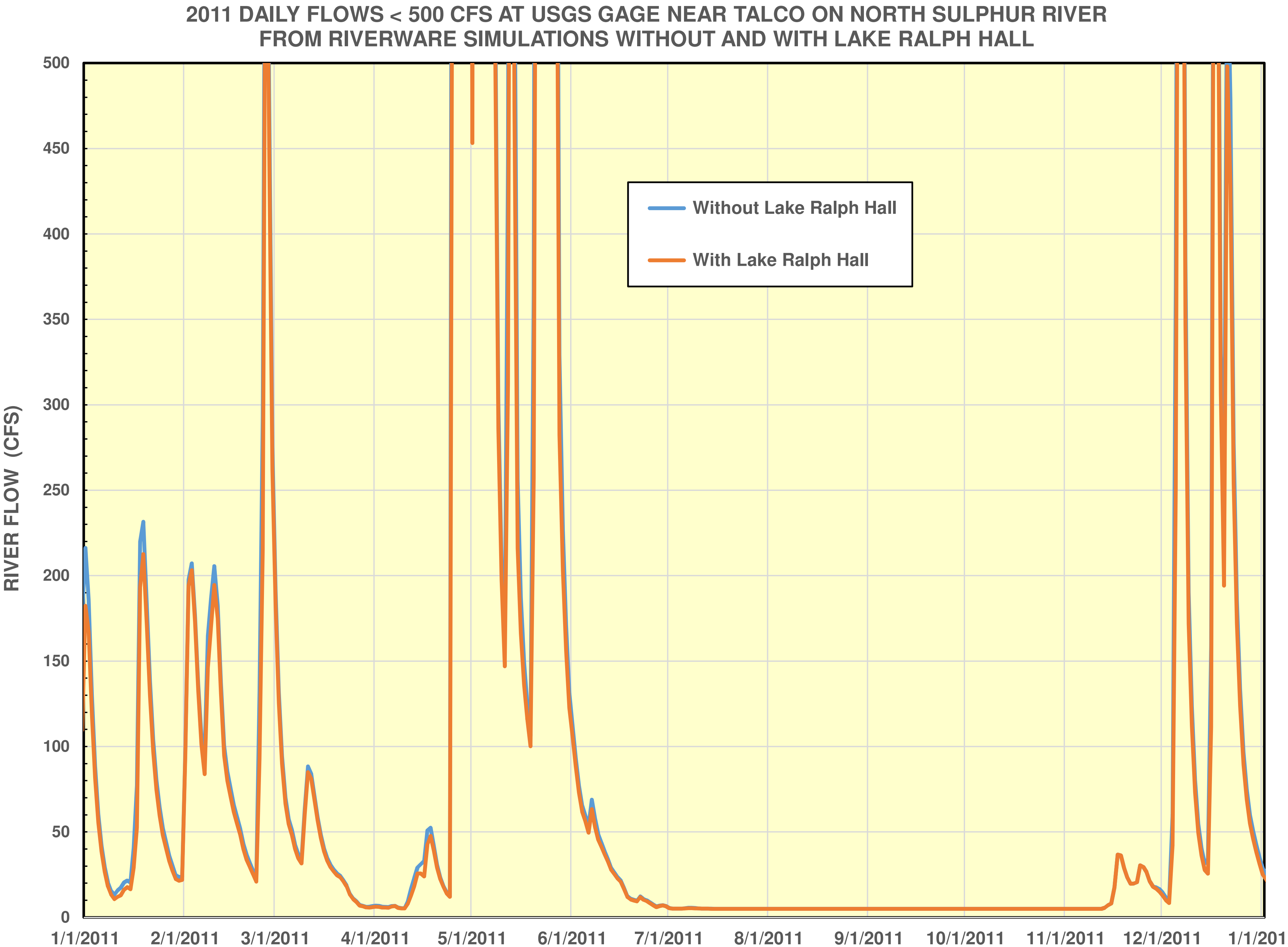

1980 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
0
50
100
150
200
250
300
350
400
450
500
1/1/1980
2/1/1980
3/1/1980
4/1/1980
5/1/1980
6/1/1980
7/1/1980
8/1/1980
9/1/1980
10/1/1980
11/1/1980
12/1/1980
1/1/1981
Without Lake Ralph Hall
With Lake Ralph Hall

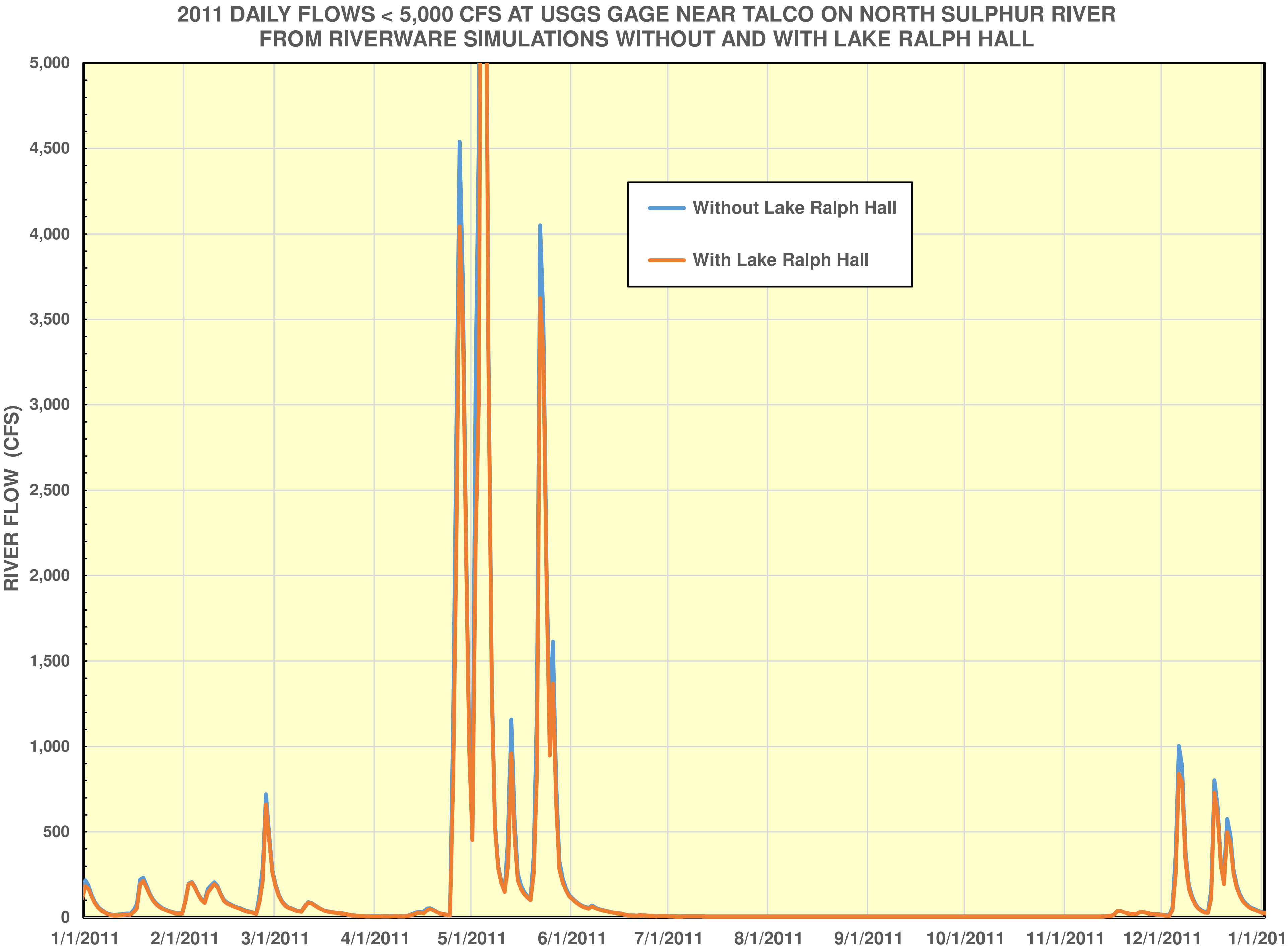
1980 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL




1992 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall

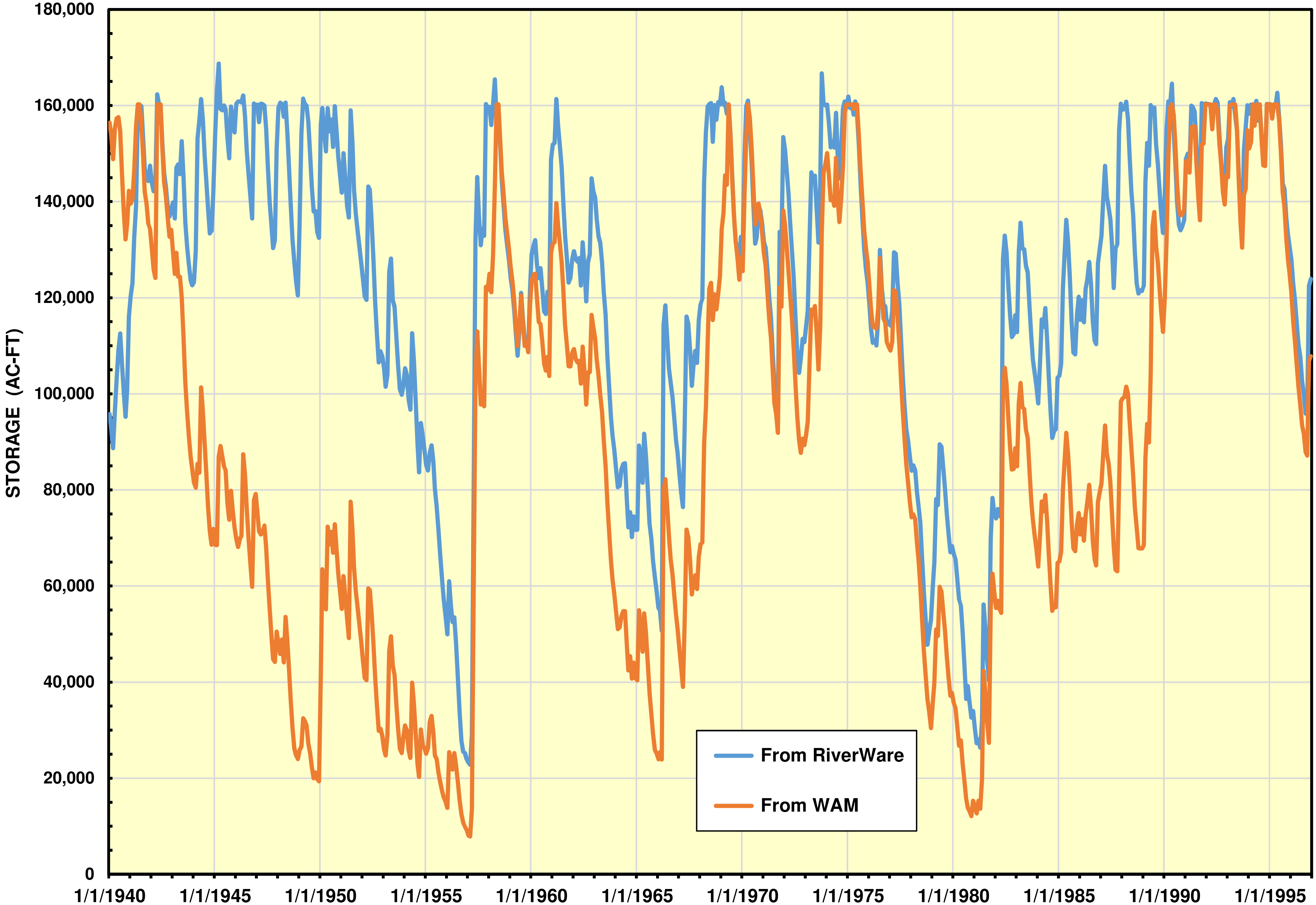

1992 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/1992
2/1/1992
3/1/1992
4/1/1992
5/1/1992
6/1/1992
7/1/1992
8/1/1992
9/1/1992
10/1/1992
11/1/1992
12/1/1992
1/1/1993
Without Lake Ralph Hall
With Lake Ralph Hall


2011 DAILY FLOWS < 500 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
500
450
400
350
300
250
200
150
100
50
0
Without Lake Ralph Hall
With Lake Ralph Hall
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012

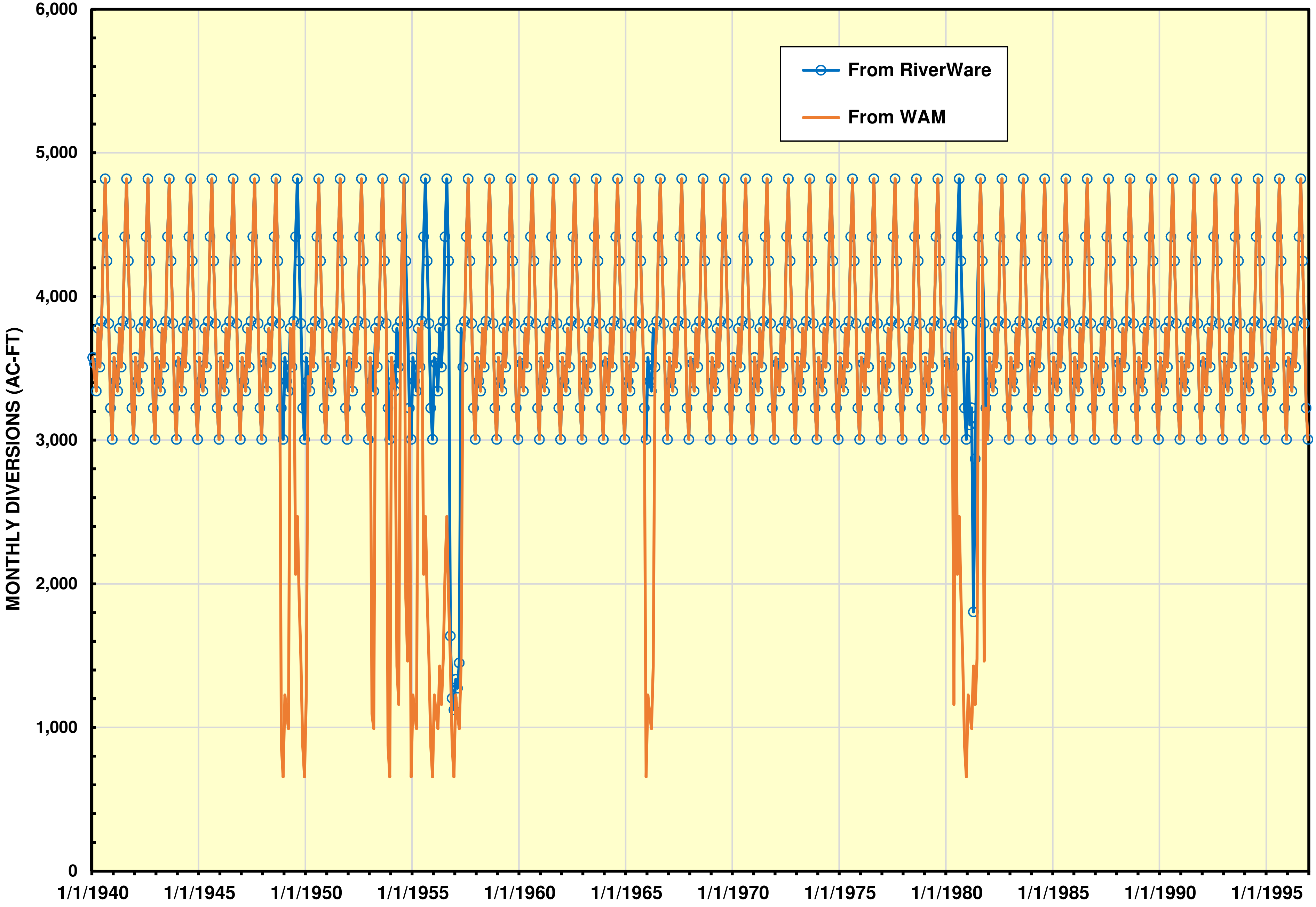


2011 DAILY FLOWS < 5,000 CFS AT USGS GAGE NEAR TALCO ON NORTH SULPHUR RIVER
FROM RIVERWARE SIMULATIONS WITHOUT AND WITH LAKE RALPH HALL
RIVER FLOW (CFS)
5,000
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
1/1/2011
2/1/2011
3/1/2011
4/1/2011
5/1/2011
6/1/2011
7/1/2011
8/1/2011
9/1/2011
10/1/2011
11/1/2011
12/1/2011
1/1/2012
Without Lake Ralph Hall
With Lake Ralph Hall

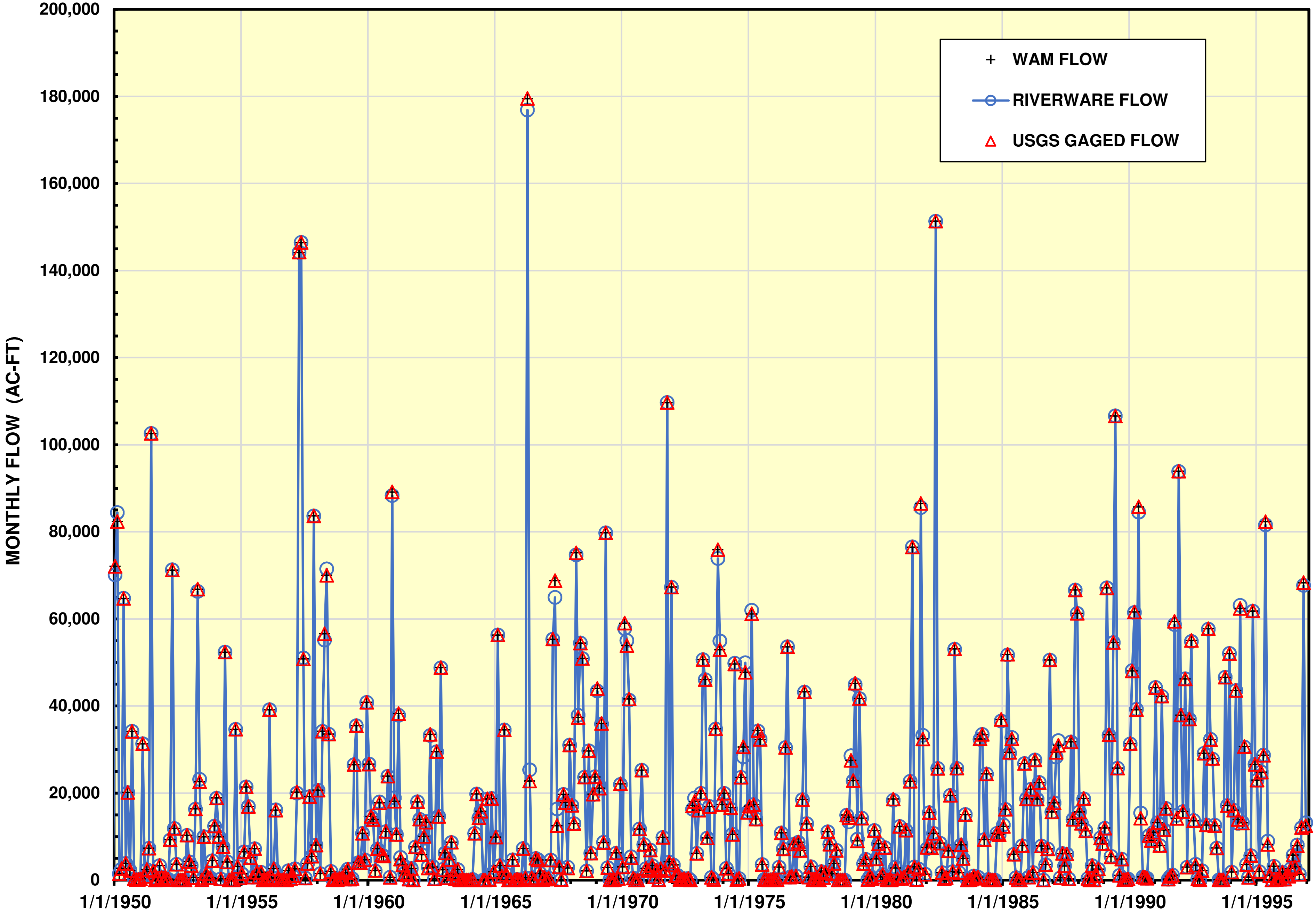
MONTHLY STORAGE IN LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL

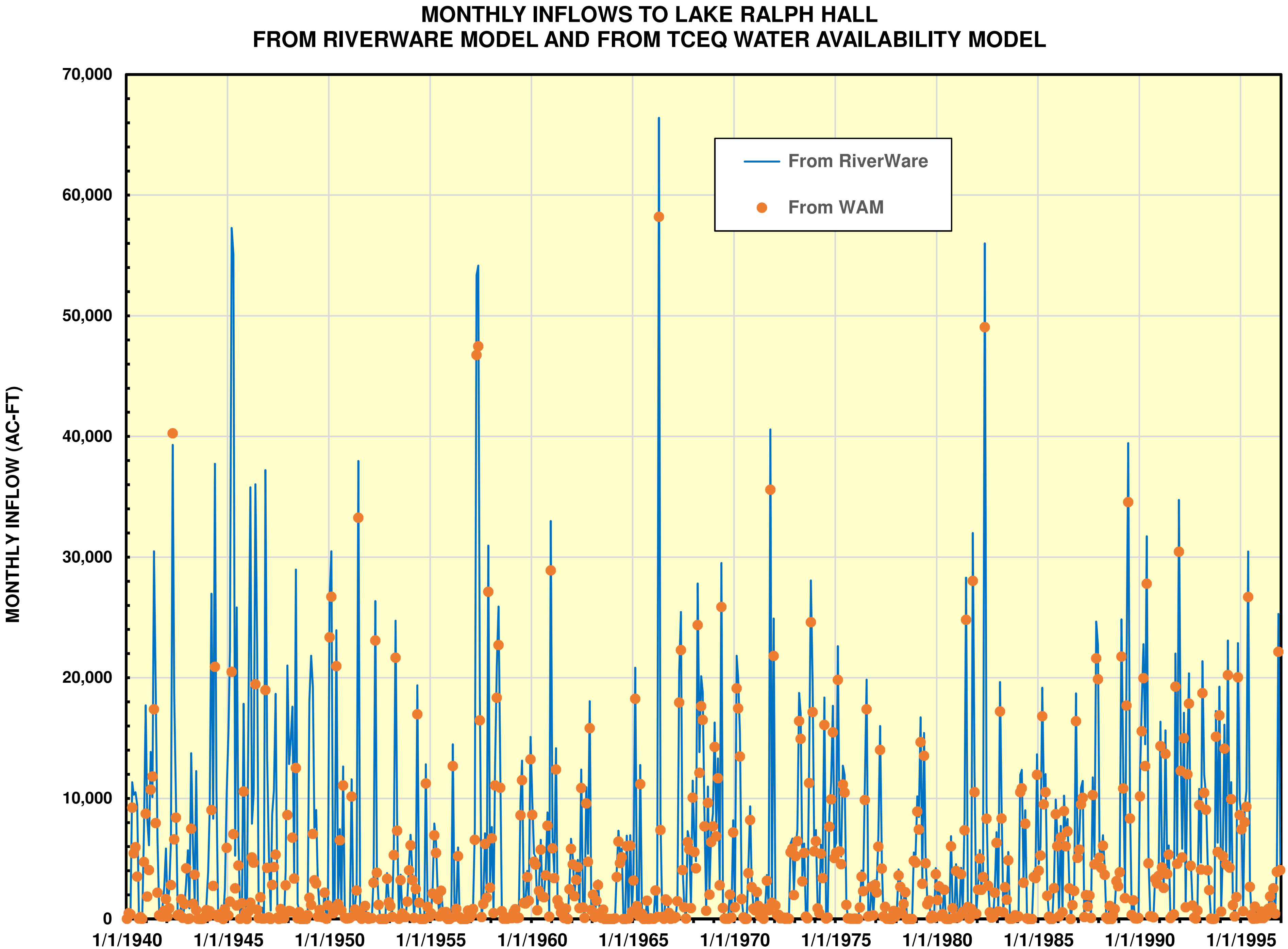


MONTHLY DIVERSIONS FROM LAKE RALPH HALL FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL

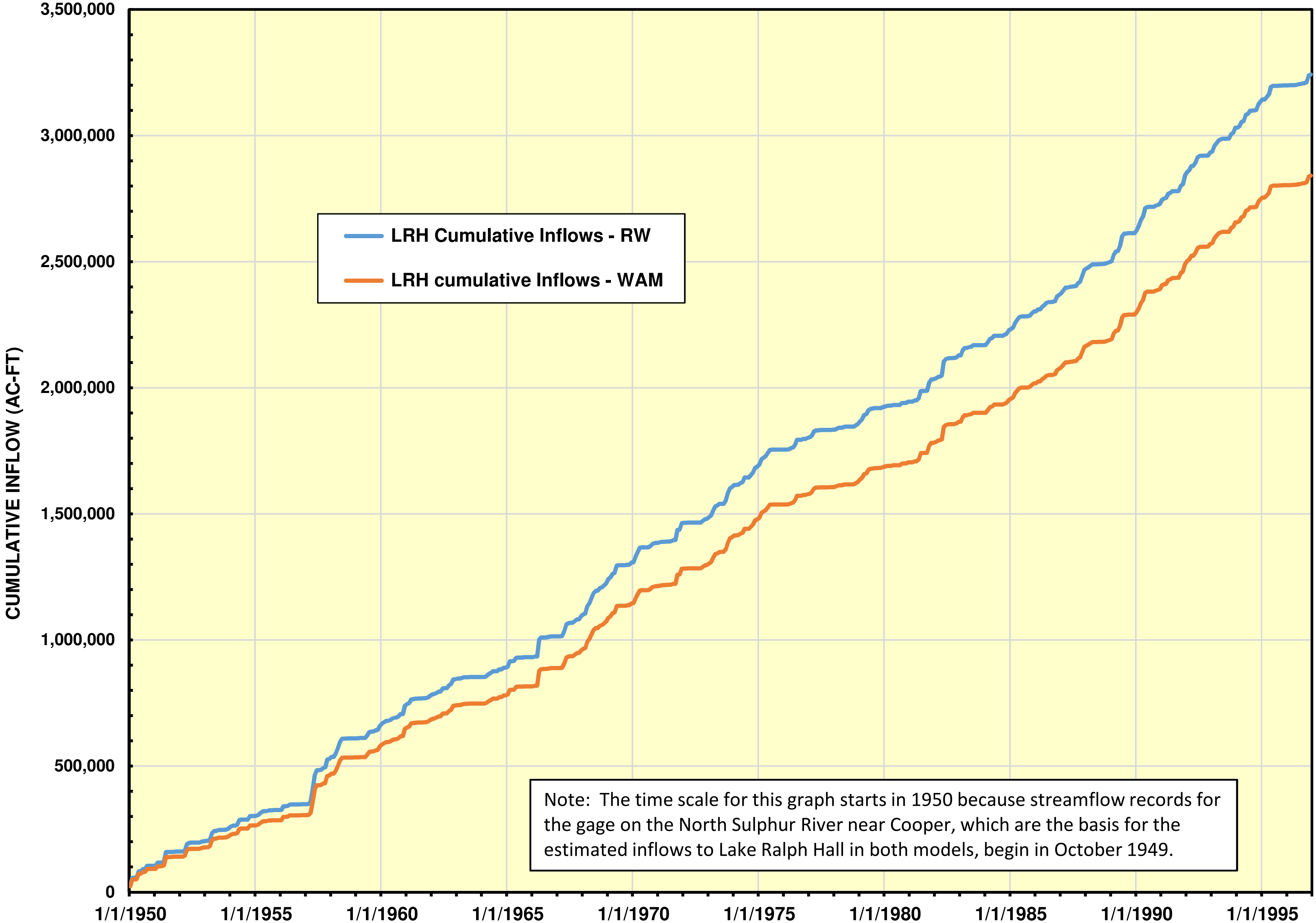


MONTHLY FLOWS AT NORTH SULPHUR RIVER GAGE NEAR COOPER FROM RIVERWARE AND WAM WITHOUT LAKE RALPH HALL AND FROM USGS GAGE

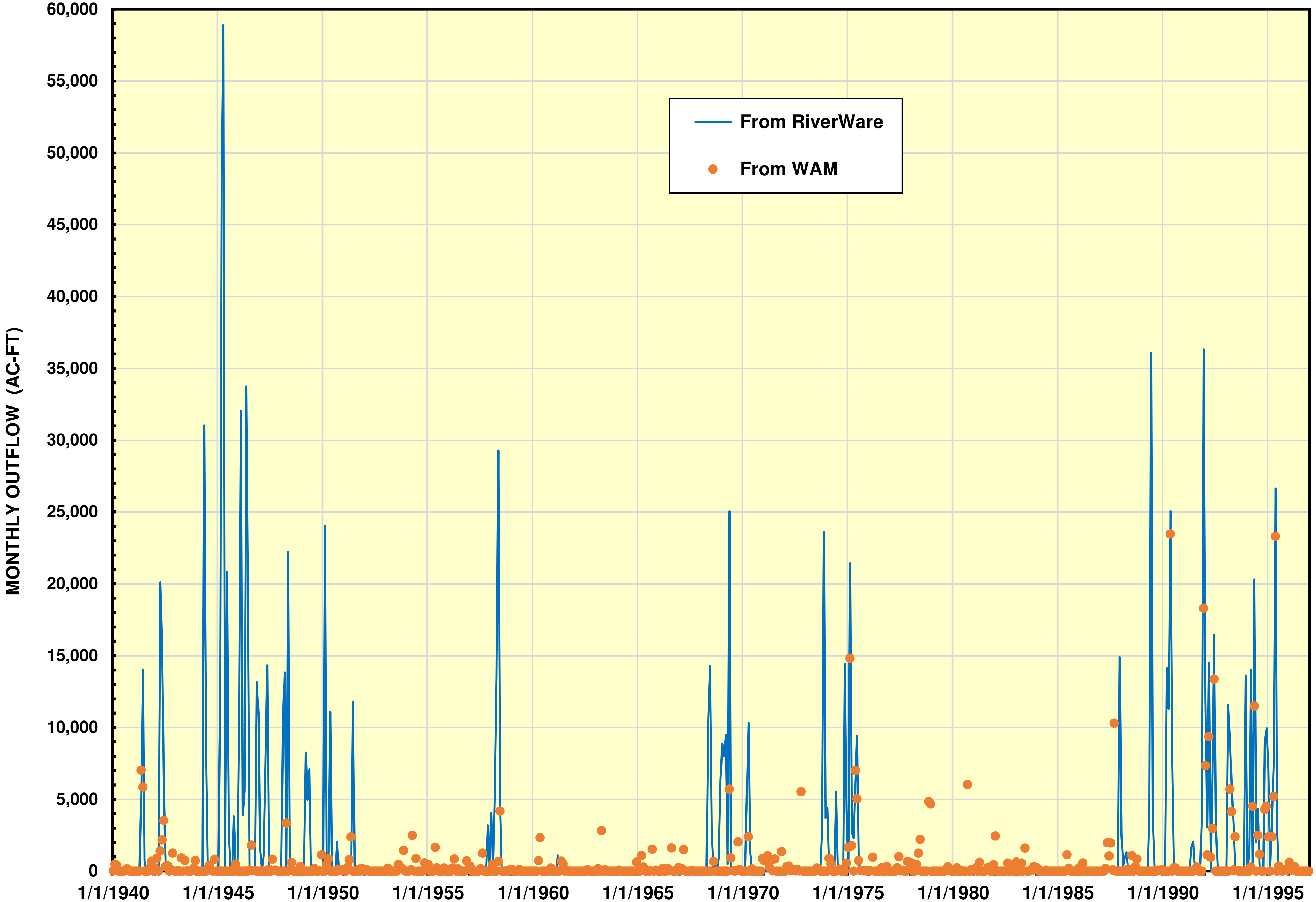



MONTHLY INFLOWS TO LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL
MONTHLY INFLOW (AC-FT)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
1/1/1940
1/1/1945
1/1/1950
1/1/1955
1/1/1960
1/1/1965
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
From RiverWare
From WAM

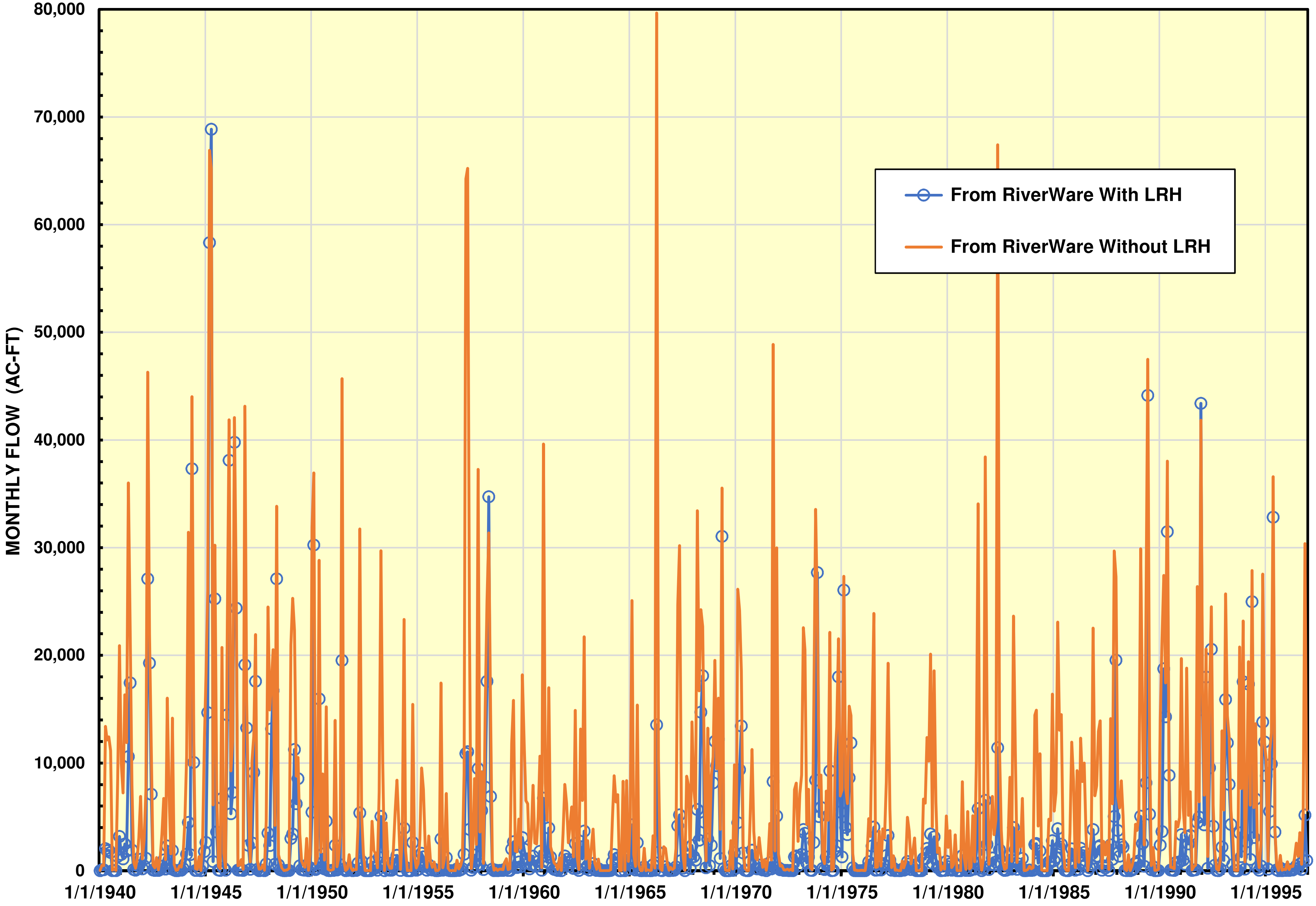
CUMULATIVE MONTHLY INFLOWS TO LAKE RALPH HALL BEGINNING IN 1950 FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



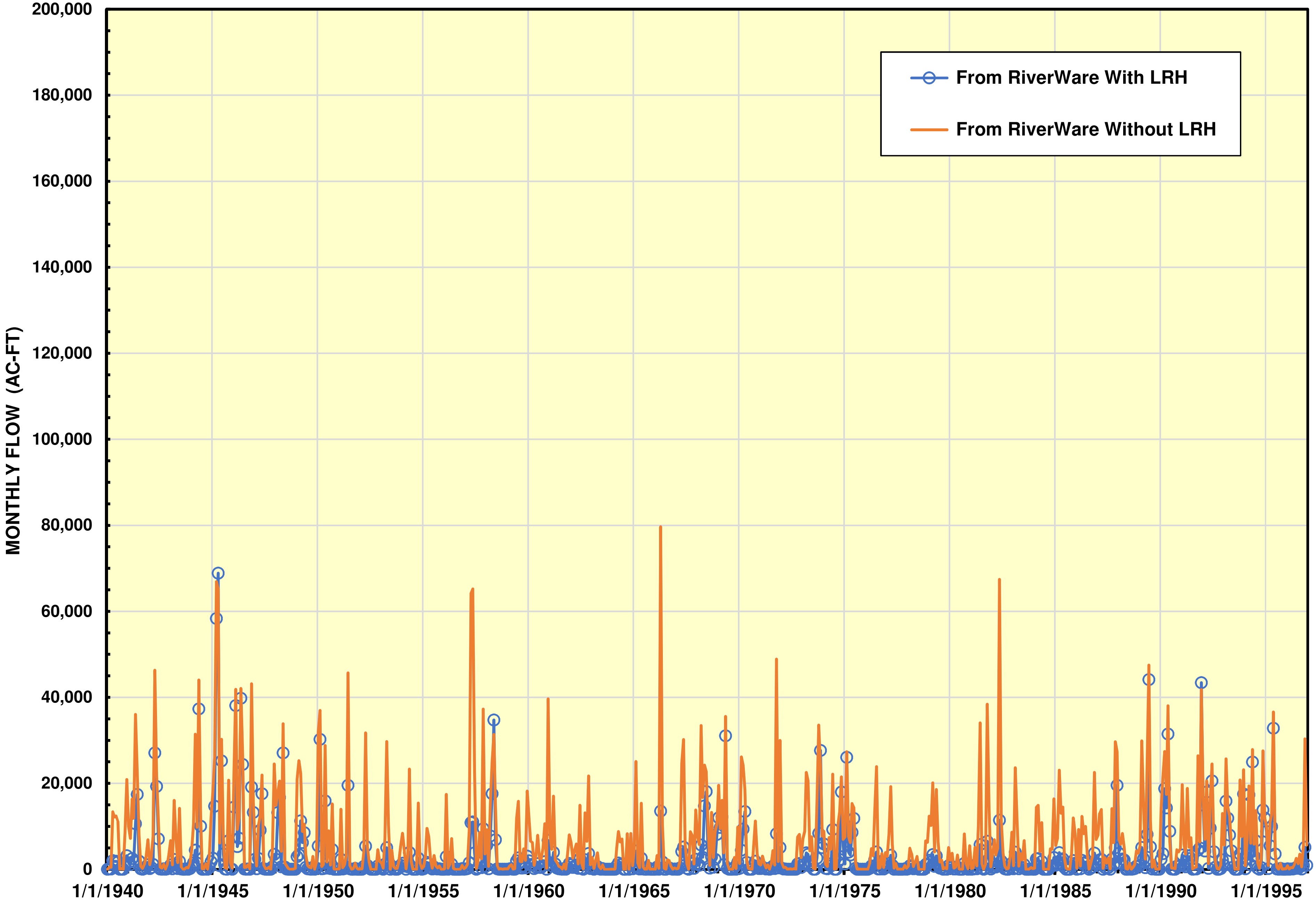
MONTHLY OUTFLOWS FROM LAKE RALPH HALL
FROM RIVERWARE MODEL AND FROM TCEQ WATER AVAILABILITY MODEL



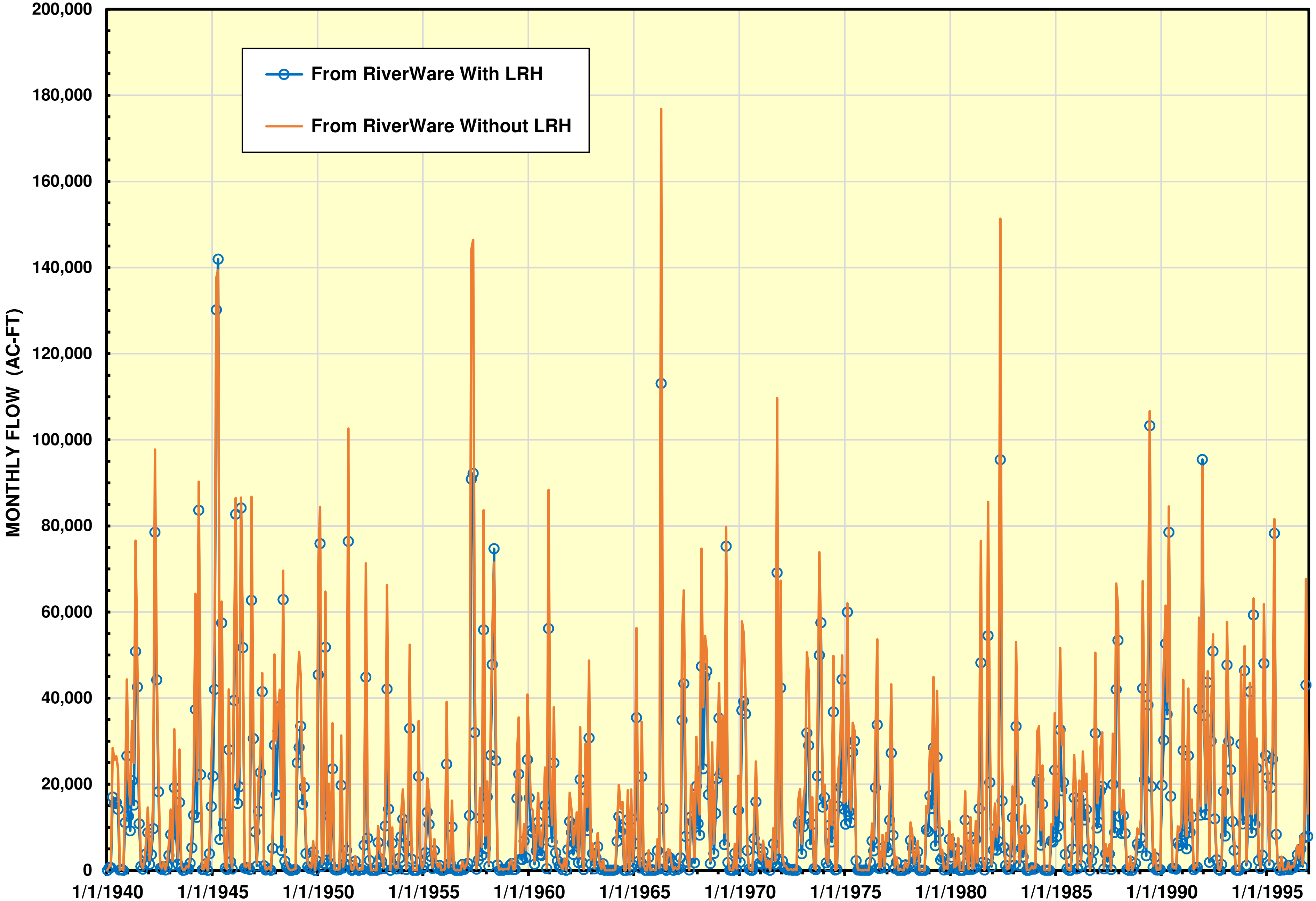
MONTHLY FLOWS BELOW LAKE RALPH HALL DAM SITE AT BAKER CREEK CONFLUENCE FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



MONTHLY FLOWS BELOW LAKE RALPH HALL DAM SITE AT BAKER CREEK CONFLUENCE FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



MONTHLY FLOWS AT NORTH SULPHUR RIVER GAGE NEAR COOPER FROM RIVERWARE MODEL WITH AND WITHOUT LAKE RALPH HALL



ATTACHMENT C

STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE AND FROM WAM WITH LAKE RALPH HALL

	RiverWare	WAM
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	62.2%	73.0%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	82.1%	83.8%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
%	%	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon
1.0%	99.0%	0	0	0	0	0	2	1	3	308	208	308	284
2.0%	98.0%	0	0	0	0	0	3	5	4	316	310	341	416
3.0%	97.0%	0	0	0	0	0	4	11	10	343	378	369	472
4.0%	96.0%	3	2	1	3	4	9	30	23	350	384	442	509
5.0%	95.0%	5	4	1	5	9	16	38	34	394	423	527	590
7.0%	93.0%	13	8	3	9	22	28	63	57	455	473	720	751
10.0%	90.0%	27	17	5	19	45	54	114	121	658	587	1,046	1,180
15.0%	85.0%	76	48	14	47	115	149	288	364	1,051	1,053	1,740	1,919
16.2%	83.8%	90	57	18	53	147	175	329	425	1,151	1,201	2,172	2,389
17.9%	82.1%	111	78	21	66	175	235	420	503	1,462	1,278	2,801	3,199
20.0%	80.0%	137	105	26	93	217	290	510	677	1,727	1,539	3,657	3,713
25.0%	75.0%	239	181	46	148	385	531	985	1,057	3,086	2,708	5,774	5,251
27.0%	73.0%	281	210	54	175	450	612	1,151	1,365	3,871	3,706	6,747	6,034
30.0%	70.0%	427	294	74	216	622	801	1,495	1,925	4,750	4,630	8,313	8,534
35.0%	65.0%	719	558	136	279	1,133	1,417	2,494	3,058	7,525	6,802	13,183	10,734
37.8%	62.2%	900	665	175	347	1,462	1,721	2,971	3,867	10,190	8,458	17,103	13,954
40.0%	60.0%	1,006	775	200	399	1,653	2,002	3,481	4,583	12,496	9,491	19,602	15,409
45.0%	55.0%	1,407	1,082	289	580	2,401	2,687	5,245	5,949	18,340	12,596	28,830	23,245
50.0%	50.0%	2,282	1,564	464	703	3,858	3,686	8,023	9,206	26,824	18,267	40,908	32,715
55.0%	45.0%	3,045	2,332	623	873	5,163	5,292	10,668	11,533	37,805	24,879	53,370	42,984
60.0%	40.0%	4,134	2,999	883	1,045	7,131	6,710	14,234	14,376	47,497	33,221	71,843	54,994
65.0%	35.0%	5,321	3,984	1,211	1,241	9,225	8,393	18,076	18,587	61,125	45,782	88,631	73,743
70.0%	30.0%	6,622	4,888	1,521	1,470	11,757	10,596	23,588	22,868	79,418	65,486	103,849	92,557
75.0%	25.0%	8,405	6,029	2,217	1,824	14,846	12,991	28,116	29,924	98,188	79,181	130,400	127,491
80.0%	20.0%	10,811	7,705	3,078	2,418	19,379	17,072	35,927	36,748	123,556	104,573	171,682	151,680
85.0%	15.0%	13,673	10,382	4,480	3,096	25,781	22,466	46,575	45,590	155,803	135,489	208,709	190,183
90.0%	10.0%	18,784	14,228	8,361	4,370	35,820	30,500	62,134	58,028	198,349	175,216	255,076	243,622
93.0%	7.0%	21,825	17,406	11,975	5,443	43,397	36,793	76,704	78,355	257,081	216,641	322,727	306,866
95.0%	5.0%	24,891	19,863	15,947	6,296	49,700	43,180	89,430	92,857	290,876	284,076	382,976	375,193
96.0%	4.0%	26,864	21,407	17,862	6,954	54,159	45,865	96,410	95,949	323,213	314,282	421,932	418,985
97.0%	3.0%	30,469	22,901	19,541	8,289	61,368	50,686	105,126	103,312	345,471	343,599	432,516	458,729
98.0%	2.0%	35,099	26,692	27,108	11,373	77,062	57,164	122,428	121,197	379,523	377,268	480,264	501,764
99.0%	1.0%	39,638	33,484	35,168	13,319	91,093	79,347	147,879	151,390	431,441	445,099	562,465	569,985
99.1%	0.9%	40,419	34,369	36,952	14,273	92,034	81,036	148,070	154,008	445,392	451,806	583,688	574,870
99.99%	0.01%	65,795	57,578	68,143	30,362	141,161	119,938	208,524	211,279	606,742	673,524	733,092	877,480

STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE AND FROM WAM WITHOUT LAKE RALPH HALL

	RiverWare	WAM
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	79.6%	77.4%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	85.5%	83.9%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	FLOW AT LAKE RALPH HALL DAM SITE		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
%	%	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon	From RiverWare ac-ft/mon	From WAM ac-ft/mon
1.0%	99.0%	0	0	0	0	0	2	1	3	308	208	308	284
2.0%	98.0%	0	0	0	0	0	3	7	4	317	310	344	416
3.0%	97.0%	0	0	0	0	0	4	13	10	346	378	392	472
4.0%	96.0%	3	2	3	3	7	9	37	23	369	384	472	509
5.0%	95.0%	5	4	6	5	14	16	55	34	411	423	534	590
7.0%	93.0%	13	8	16	11	36	28	83	57	496	473	774	751
10.0%	90.0%	27	17	33	21	73	55	150	121	694	587	1,142	1,198
14.5%	85.5%	67	42	81	53	175	134	359	337	1,134	983	1,757	1,863
15.0%	85.0%	76	48	91	60	200	150	381	374	1,182	1,053	1,913	1,939
16.1%	83.9%	88	56	106	69	235	175	430	421	1,299	1,201	2,370	2,453
20.0%	80.0%	137	105	163	131	360	327	731	691	2,019	1,604	3,845	3,812
20.5%	79.6%	147	106	175	133	370	331	760	727	2,118	1,642	4,269	3,941
22.6%	77.4%	196	140	233	175	508	438	894	907	2,767	2,185	5,275	4,384
25.0%	75.0%	239	181	283	226	637	560	1,297	1,068	3,486	2,907	6,486	5,462
30.0%	70.0%	427	294	503	368	1,007	911	2,139	1,993	5,794	4,761	9,477	8,559
35.0%	65.0%	719	558	859	697	1,864	1,724	3,194	3,424	8,666	7,289	14,329	11,054
40.0%	60.0%	1,006	775	1,213	967	2,662	2,390	4,504	4,838	14,348	9,807	21,706	16,383
45.0%	55.0%	1,407	1,082	1,654	1,351	3,702	3,337	6,918	6,546	21,168	14,049	30,418	25,207
50.0%	50.0%	2,282	1,564	2,748	1,953	6,103	4,819	10,317	10,683	29,881	20,578	41,964	33,876
55.0%	45.0%	3,045	2,332	3,674	2,912	8,216	7,193	13,709	14,082	41,520	27,605	56,561	45,630
60.0%	40.0%	4,134	2,999	4,974	3,745	11,140	9,241	18,641	17,926	53,220	36,086	77,273	59,338
65.0%	35.0%	5,321	3,984	6,475	4,977	14,611	12,279	23,018	22,405	65,830	49,758	94,761	77,924
70.0%	30.0%	6,622	4,888	7,932	6,104	17,763	15,061	29,660	27,658	85,531	68,570	111,283	96,196
75.0%	25.0%	8,405	6,029	10,144	7,529	22,106	18,597	35,934	35,918	106,032	87,441	140,059	132,052
80.0%	20.0%	10,811	7,705	12,957	9,622	28,326	23,757	44,314	42,962	131,134	113,998	181,748	160,522
90.0%	10.0%	18,784	14,228	22,501	17,768	49,903	43,878	74,562	71,524	214,631	188,588	268,410	255,851
93.0%	7.0%	21,825	17,406	25,778	21,736	55,542	53,675	90,564	93,102	269,188	234,764	333,275	323,591
95.0%	5.0%	24,891	19,863	29,967	24,804	66,111	61,264	103,576	110,149	308,811	301,091	399,997	390,320
96.0%	4.0%	26,864	21,407	31,828	26,733	70,919	65,990	116,735	113,552	342,029	326,063	433,457	433,702
97.0%	3.0%	30,469	22,901	36,303	28,598	80,704	70,599	124,159	126,166	361,655	368,055	447,459	475,112
98.0%	2.0%	35,099	26,692	41,839	33,332	86,632	82,322	137,801	142,550	401,174	394,235	499,927	515,625
99.0%	1.0%	39,638	33,484	47,723	41,814	107,136	103,241	164,893	184,164	433,424	463,329	596,116	598,896
99.9%	0.1%	60,174	51,960	71,297	64,886	159,440	160,240	234,060	255,580	636,248	714,960	747,687	828,098
99.99%	0.01%	65,795	57,578	78,816	71,901	175,146	177,515	240,444	260,229	654,534	722,475	770,216	925,058

STATISTICAL ANALYSIS OF FLOWS FROM RIVERWARE WITH AND WITHOUT LAKE RALPH HALL

	With LRH	Without LRH
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	62.2%	79.6%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	82.1%	85.5%

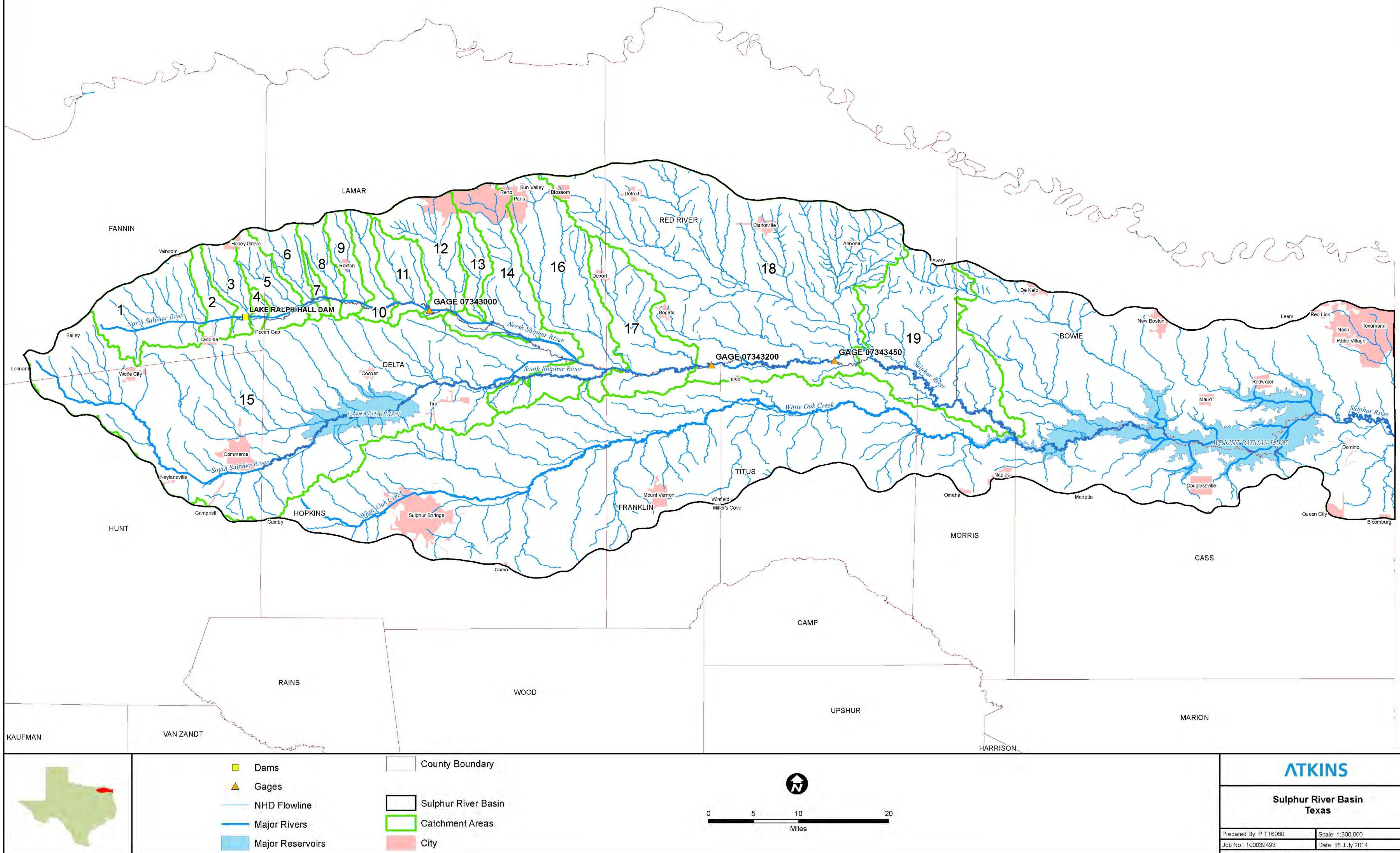
PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
%	%	With LRH ac-ft/mon	Without LRH ac-ft/mon	With LRH ac-ft/mon	Without LRH ac-ft/mon	With LRH ac-ft/mon	Without LRH ac-ft/mon	With LRH ac-ft/mon	Without LRH ac-ft/mon	With LRH ac-ft/mon	Without LRH ac-ft/mon	With LRH ac-ft/mon	Without LRH ac-ft/mon
1.0%	99.0%	0	0	0	0	0	0	1	1	308	308	308	308
2.0%	98.0%	0	0	0	0	0	0	5	7	316	317	341	344
3.0%	97.0%	0	0	0	0	0	0	11	13	343	346	369	392
4.0%	96.0%	3	3	1	3	4	7	30	37	350	369	442	472
5.0%	95.0%	5	5	1	6	9	14	38	55	394	411	527	534
7.0%	93.0%	13	13	3	16	22	36	63	83	455	496	720	774
10.0%	90.0%	27	27	5	33	45	73	114	150	658	694	1,046	1,142
14.5%	85.5%	67	67	13	81	106	175	283	359	1,008	1,136	1,661	1,759
15.0%	85.0%	76	76	14	91	115	200	288	381	1,051	1,182	1,740	1,913
18.0%	82.1%	111	111	21	134	175	281	420	528	1,463	1,599	2,802	3,012
20.0%	80.0%	137	137	26	163	217	360	510	731	1,727	2,019	3,657	3,845
20.5%	79.6%	147	147	27	175	228	370	550	760	1,835	2,118	3,788	4,269
25.0%	75.0%	239	239	46	283	385	637	985	1,297	3,086	3,486	5,774	6,486
30.0%	70.0%	427	427	74	503	622	1,007	1,495	2,139	4,750	5,794	8,313	9,477
35.0%	65.0%	719	719	136	859	1,133	1,864	2,494	3,194	7,525	8,666	13,183	14,329
37.8%	62.2%	901	901	175	1,072	1,464	2,331	2,974	3,938	10,207	11,672	17,118	18,683
40.0%	60.0%	1,006	1,006	200	1,213	1,653	2,662	3,481	4,504	12,496	14,348	19,602	21,706
45.0%	55.0%	1,407	1,407	289	1,654	2,401	3,702	5,245	6,918	18,340	21,168	28,830	30,418
50.0%	50.0%	2,282	2,282	464	2,748	3,858	6,103	8,023	10,317	26,824	29,881	40,908	41,964
55.0%	45.0%	3,045	3,045	623	3,674	5,163	8,216	10,668	13,709	37,805	41,520	53,370	56,561
60.0%	40.0%	4,134	4,134	883	4,974	7,131	11,140	14,234	18,641	47,497	53,220	71,843	77,273
65.0%	35.0%	5,321	5,321	1,211	6,475	9,225	14,611	18,076	23,018	61,125	65,830	88,631	94,761
70.0%	30.0%	6,622	6,622	1,521	7,932	11,757	17,763	23,588	29,660	79,418	85,531	103,849	111,283
75.0%	25.0%	8,405	8,405	2,217	10,144	14,846	22,106	28,116	35,934	98,188	106,032	130,400	140,059
80.0%	20.0%	10,811	10,811	3,078	12,957	19,379	28,326	35,927	44,314	123,556	131,134	171,682	181,748
85.0%	15.0%	13,673	13,673	4,480	16,198	25,781	35,713	46,575	58,546	155,803	163,200	208,709	218,084
90.0%	10.0%	18,784	18,784	8,361	22,501	35,820	49,903	62,134	74,562	198,349	214,631	255,076	268,410
93.0%	7.0%	21,825	21,825	11,975	25,778	43,397	55,542	76,704	90,564	257,081	269,188	322,727	333,275
95.0%	5.0%	24,891	24,891	15,947	29,967	49,700	66,111	89,430	103,576	290,876	308,811	382,976	399,997
96.0%	4.0%	26,864	26,864	17,862	31,828	54,159	70,919	96,410	116,735	323,213	342,029	421,932	433,457
97.0%	3.0%	30,469	30,469	19,541	36,303	61,368	80,704	105,126	124,159	345,471	361,655	432,516	447,459
98.0%	2.0%	35,099	35,099	27,108	41,839	77,062	86,632	122,428	137,801	379,523	401,174	480,264	499,927
99.0%	1.0%	39,638	39,638	35,168	47,723	91,093	107,136	147,879	164,893	431,441	433,424	562,465	596,116
99.9%	0.1%	60,174	60,174	61,662	71,297	133,926	159,440	194,211	234,060	597,068	636,248	725,870	747,687
99.99%	0.01%	65,795	65,795	68,143	78,816	141,161	175,146	208,524	240,444	606,742	654,534	733,092	770,216

STATISTICAL ANALYSIS OF FLOWS FROM WAM WITH AND WITHOUT LAKE RALPH HALL

	With LRH	Without LRH
Probability That Monthly Flow below Lake Ralph Hall Dam at Bakers Creek Exceeds Channel Pool Volume of 175 ac-ft:	73.0%	77.4%
Probability That Monthly Flow at North Sulphur River Gage near Cooper Exceeds Channel Pool Volume of 175 ac-ft:	83.8%	83.9%

PER-CENTILE	EXCEED-ENCE PROBA-BILITY	INFLOW TO LAKE RALPH HALL		FLOW BELOW LAKE RALPH HALL AT BAKERS CREEK Map Catchment 3		FLOW AT N SULPHUR RIVER COOPER GAGE Map Gage 07343000		FLOW AT PARKHOUSE 2 DAM SITE Map Catchment 14		FLOW AT N SULPHUR RIVER TALCO GAGE Map Gage 07343200		FLOW AT MARVIN NICHOLS DAM SITE Map Catchment 18	
		With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH	With LRH	Without LRH
%	%	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon	ac-ft/mon
1.0%	99.0%	0	0	0	0	2	2	3	3	208	208	284	284
2.0%	98.0%	0	0	0	0	3	3	4	4	310	310	416	416
3.0%	97.0%	0	0	0	0	4	4	10	10	378	378	472	472
4.0%	96.0%	2	2	3	3	9	9	23	23	384	384	509	509
5.0%	95.0%	4	4	5	5	16	16	34	34	423	423	590	590
7.0%	93.0%	8	8	9	11	28	28	57	57	473	473	751	751
10.0%	90.0%	17	17	19	21	54	55	121	121	587	587	1,180	1,198
15.0%	85.0%	48	48	47	60	149	150	364	374	1,053	1,053	1,919	1,939
16.1%	83.9%	56	56	53	69	167	175	421	421	1,190	1,201	2,329	2,453
16.2%	83.8%	57	57	53	72	175	180	425	425	1,201	1,206	2,389	2,506
20.0%	80.0%	105	105	93	131	290	327	677	691	1,539	1,604	3,713	3,812
22.6%	77.4%	140	140	113	175	381	437	874	906	2,070	2,182	4,275	4,377
25.0%	75.0%	181	181	148	226	531	560	1,057	1,068	2,708	2,907	5,251	5,462
27.0%	73.0%	210	210	175	262	612	651	1,365	1,393	3,706	4,016	6,034	6,184
30.0%	70.0%	294	294	216	368	801	911	1,925	1,993	4,630	4,761	8,534	8,559
35.0%	65.0%	558	558	279	697	1,417	1,724	3,058	3,424	6,802	7,289	10,734	11,054
40.0%	60.0%	775	775	399	967	2,002	2,390	4,583	4,838	9,491	9,807	15,409	16,383
45.0%	55.0%	1,082	1,082	580	1,351	2,687	3,337	5,949	6,546	12,596	14,049	23,245	25,207
50.0%	50.0%	1,564	1,564	703	1,953	3,686	4,819	9,206	10,683	18,267	20,578	32,715	33,876
55.0%	45.0%	2,332	2,332	873	2,912	5,292	7,193	11,533	14,082	24,879	27,605	42,984	45,630
60.0%	40.0%	2,999	2,999	1,045	3,745	6,710	9,241	14,376	17,926	33,221	36,086	54,994	59,338
65.0%	35.0%	3,984	3,984	1,241	4,977	8,393	12,279	18,587	22,405	45,782	49,758	73,743	77,924
70.0%	30.0%	4,888	4,888	1,470	6,104	10,596	15,061	22,868	27,658	65,486	68,570	92,557	96,196
75.0%	25.0%	6,029	6,029	1,824	7,529	12,991	18,597	29,924	35,918	79,181	87,441	127,491	132,052
80.0%	20.0%	7,705	7,705	2,418	9,622	17,072	23,757	36,748	42,962	104,573	113,998	151,680	160,522
90.0%	10.0%	14,228	14,228	4,370	17,768	30,500	43,878	58,028	71,524	175,216	188,588	243,622	255,851
93.0%	7.0%	17,406	17,406	5,443	21,736	36,793	53,675	78,355	93,102	216,641	234,764	306,866	323,591
95.0%	5.0%	19,863	19,863	6,296	24,804	43,180	61,264	92,857	110,149	284,076	301,091	375,193	390,320
96.0%	4.0%	21,407	21,407	6,954	26,733	45,865	65,990	95,949	113,552	314,282	326,063	418,985	433,702
97.0%	3.0%	22,901	22,901	8,289	28,598	50,686	70,599	103,312	126,166	343,599	368,055	458,729	475,112
98.0%	2.0%	26,692	26,692	11,373	33,332	57,164	82,322	121,197	142,550	377,268	394,235	501,764	515,625
99.0%	1.0%	33,484	33,484	13,319	41,814	79,347	103,241	151,390	184,164	445,099	463,329	569,985	598,896
99.9%	0.1%	51,960	51,960	30,086	64,886	108,282	160,240	207,607	255,580	666,987	714,960	779,543	828,098
99.99%	0.01%	57,578	57,578	30,362	71,901	119,938	177,515	211,279	260,229	673,524	722,475	877,480	925,058

ATTACHMENT D



ATTACHMENT E

SUMMARY OF ANALYSIS OF FILLING RIVER CHANNEL POOLS DOWNSTREAM OF LAKE RALPH HALL WITH NORTH SULPHUR RIVER FLOWS SIMULATED WITH RIVERWARE MODEL AND WITH WAM FOR CONDITIONS WITHOUT AND WITH LAKE RALPH HALL PROJECT

STATION NO.	WATER COURSE	LOCATION DESCRIPTION	DRAINAGE AREA sq. mi.	DISTANCE ABOVE N SULPHUR GAGE miles	VOLUME REQUIRED TO FILL ALL D/S POOLS ac-ft	POOL VOLUME IN EACH D/S REACH ac-ft	% OF TIME POOLS ARE FILLED		
							Without Lake Ralph Hall	With Lake Ralph Hall	Deviation From Without LRH Case
FROM RIVERWARE MODEL (06-26-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	- -	- -	- -	- -
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	92.7%	83.6%	-9.1%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	86.7%	73.2%	-13.5%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	85.8%	82.0%	-3.8%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	86.7%	86.3%	-0.4%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	85.8%	85.4%	-0.4%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	85.4%	83.6%	-1.8%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	89.8%	89.6%	-0.1%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	85.1%	82.7%	-2.3%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	- -	- -	- -	- -
FROM WAM (04-06-15)									
LRH	North Sulphur R.	Lake Ralph Hall Dam Site	100.9	20.00	175.0	- -	- -	- -	- -
3	North Sulphur R.	Downstream of mouth of Baker Ck.	126.1	18.13	175.0	17.8	90.8%	90.2%	-0.6%
4	North Sulphur R.	Downstream of mouth of Bledsoe Ck.	132.1	16.29	157.2	46.4	84.8%	83.5%	-1.3%
5	North Sulphur R.	Downstream of mouth of Wafer Ck.	165.7	11.48	110.8	27.9	83.9%	83.8%	-0.1%
6	North Sulphur R.	Downstream of mouth of Ghost Ck.	191.8	8.59	82.9	11.2	85.4%	85.4%	0.0%
7	North Sulphur R.	Downstream of mouth of Morrison Ck.	198.3	7.42	71.7	6.0	83.9%	83.9%	0.0%
8	North Sulphur R.	Downstream of mouth of Rowdy Ck.	220.2	6.81	65.7	21.6	83.3%	83.2%	-0.1%
9	North Sulphur R.	Downstream of mouth of Cane Ck.	244.9	4.57	44.1	5.5	88.6%	88.6%	0.0%
10	North Sulphur R.	Downstream of mouth of Maxwell Ck.	270.8	4.00	38.6	38.6	83.2%	83.0%	-0.1%
B10	North Sulphur R.	USGS Gage 7343000 near Cooper	311.3	0.00	0.0	- -	- -	- -	- -