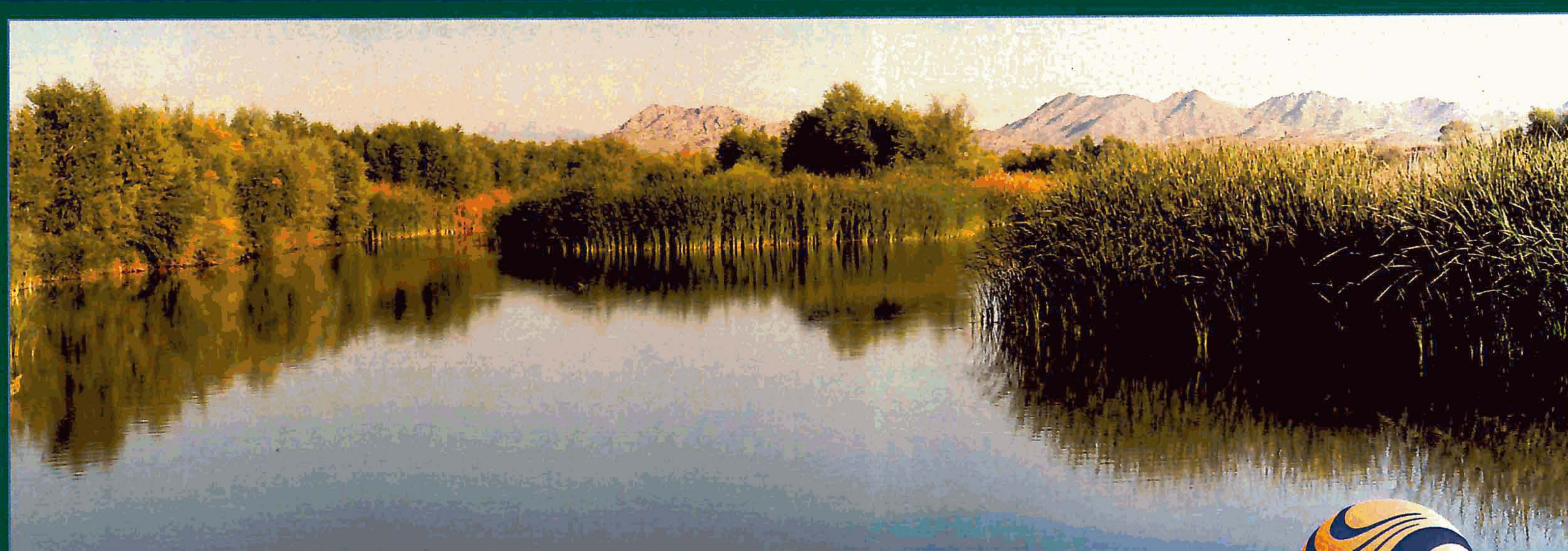
Flood Control District of Maricopa County

ATTACHMENT 4- EL RIO SEDIMENTATION ANALYSIS-

Book 2 of 2- El Rio Alternative Sedimentation Analysis

El Rio Watercourse Master Plan and Area Drainage Master Plan

Contract FCD 2001C024
Stantec Project No. 82000240



November 2005
Revised March 2006

Stantec

FINAL REPORT

El Rio Watercourse Master Plan

EL RIO SEDIMENTATION ANALYSIS

Book 2-El Rio Alternative Sedimentation Analysis

PREPARED FOR:

FLOOD CONTROL DISTRICT
of
Maricopa County

PREPARED BY:

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Stantec

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ATTACHMENTS

INTRODUCTION

PURPOSE

The sedimentation analyses of a portion of the Gila River from the confluence with the Agua Fria River to the State Route 85 Bridge (SR 85) are performed in support of the El Rio Watercourse Master Plan and Area Drainage Master Plan (El Rio WMP). This report presents the sedimentation analyses of elements of the alternative that is selected for that reach of the river. Those elements are:

1. Levees
2. Vegetation enhancement
3. Sand and gravel mining pits/recreation lakes
4. Buckeye Water Conservation and Drainage District (BWCDD) diversion structure

In addition to the alternative elements that are considered for the El Rio WMP, there are modifications to the river that are planned as part of the King Ranch development and Cotton Lane Bridge improvements. Those projects are outside the scope of the El Rio WMP project and are being analyzed by others. However, those projects are to be implemented in the near future; therefore, those projects are incorporated into all of the analyses of the El Rio WMP alternative.

Each alternative element is evaluated in regard to potential sedimentation impacts to the El Rio reach of the Gila River and to the sediment leaving the downstream limit (the SR 85 Bridge) of the study area. Each alternative element is evaluated to assess if it produces an adverse in-situ impact, for example at a bridge, or changes the sediment balance of the river thus adversely impacting the Gila River downstream of the SR 85 Bridge. The results of the sedimentation analyses are usually presented in tabular and/or graphical form with accompanying explanation.

Sedimentation embodies the processes of sediment transport, erosion (scour), deposition (fill), entrainment, and the compaction of sediment deposits (American Society of Civil Engineers, 1975). For the purpose of this report, sedimentation is limited to the processes of sediment transport, erosion (local scour and riverbed degradation), and deposition (local fill and riverbed aggradation). Where appropriate in this report, the specific sedimentation process being investigated or reported is identified.

ORGANIZATION OF REPORT

The main body of the report is in text with tables of model results and selected graphical representation of model results. The computer models that are used for these analyses produce extensive output. Often the most effective way of reviewing and presenting those results is by the use of graphs. Many graphs were developed and the presentation of all the graphs in the report would impair the readability of the report. For that reason, many graphs are provided in attachments with only selected graphs in the report. The reader is encouraged to consult the attachments for a more complete understanding of the discussion and results in the report.

The HEC-6T and HEC-RAS input files and selected digital output files of computer models that are used in the sedimentation analyses are provided on CD in attachments.

HEC-6T MODEL VIEWER

HEC-6T produces copious output, which can be very informative in interpreting the results and, more importantly, very useful in understanding the behavior of the river and the sedimentation process. The report, herein, presents a selected amount of model results as tables and figures in the text of the report. Those are provided to illustrate and support the report. In addition, Stantec developed an HEC-6T Model Viewer that facilitates viewing model output. The reader is encouraged to use the Model Viewer on the CD of Attachment 1. The viewer provides a graphical view of the report results and may aid in an overall understanding of the model results. Instructions for using the HEC-6T Model Viewer are contained in a READ.ME file on that CD.

ANALYSIS METHODOLOGY

It is unlikely that the various elements of the recommended alternative of the El Rio WMP will be implemented as a single project over a short time period. Rather, it is envisioned that elements of the alternatives will be implemented on a basis that is set by development needs and funding availability. For example, the sand and gravel mining pits will be developed as the commercial need for rock products is realized. Similarly, the levees will probably be constructed in a logical progression as development is undertaken in the adjacent floodplain. Therefore, certain elements of the alternative are analyzed as separate, independent components and the incremental impact assessed. In the case of the gravel pits, three sites are identified. The impact of each pit is analyzed independently. The impact of all three pits at near ultimate size is analyzed for the cumulative impact.

In the case of the levee, a logical progression of the construction of the levee cannot be anticipated. Therefore, the levee is treated in the analyses as if it is constructed as a unit and the entire levee is incorporated in each alternative analysis.

The King Ranch/Cotton Lane Bridge channelization and modifications to the river are considered to occur in the near future. Therefore, the proposed King Ranch/Cotton Lane Bridge improvements are incorporated in each analysis. The King Ranch/Cotton Lane Bridge channelization and river modifications, as represented by WEST Consultants in May 2005, are used.

AUTHORITY FOR STUDY

Pursuant to Arizona revised Statues 48-3609.01 the Flood Control District of Maricopa County (FCDMC) is authorized to conduct watercourse master plans for river reaches within Maricopa County. Stantec Consulting Inc. was awarded the contact (FCD 2001C024).

ALTERNATIVES ANALYZED FOR SEDIMENTATION IMPACTS

DESCRIPTION OF THE RECOMMENDED EL RIO ALTERNATIVE

The proposed condition for the El Rio reach of the Gila River consists of a combination of soft structural elements along with landscape treatment and a non-structural alternative. The soft structural alternative is a combination of levees and bank protection that closely follows the 100-year floodway alignment. A non-structural alternative is applied to river segments located along the south bank where erosion resistant geologic formations occur and bank protection is not required.

Resource enhancements are those elements of the recommended plan that enhance the existing biological resources. Biological resource enhancements include development of higher quality habitat, such as native cottonwood and willow, through conversion of less desirable habitat, such as tamarisk. Resource enhancement may also include removal of non-native species by replacing them with open water, wetland marsh, and native riparian vegetation.

In addition to the recommended levees, bank protection and resource enhancements, the El Rio WMP includes the assessment of two sand and gravel pits and a combined irrigation diversion gravel pit/recreation lake at the BWCDD intake to the BWCDD (Buckeye) Canal. As an aspect of the El Rio WMP, it is expected that those pits may be converted or operated as recreational lakes during and/or after sand and gravel extraction is terminated.

A final aspect of the El Rio sedimentation study is the proposed plan by others to channelize the Gila River as part of the King Ranch development and in conjunction with the Cotton Lane Bridge by Maricopa County. Although the planning of those projects is performed by others and they are not a part of the El Rio WMP, nonetheless, those plans are incorporated into the sedimentation analyses for the El Rio WMP as if they are "existing" conditions.

The recommended El Rio alternative consists of numerous individual elements but only those elements that potentially may impact the sedimentation of the Gila River are considered in the sedimentation analysis. The elements that have the potential to impact the sedimentation of the river are:

- Levees and bank protection that encroach into the floodplain and potentially affect the hydraulics of flow (velocity and depth).
- Sand and gravel mining pits and recreational lakes that result in changes to topography and the creation of open water.
- Resource enhancement that can alter the distribution of water and sediment, or expose the land surface to accelerated erosion.

Flood Control District of Maricopa County
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EL RIO WATERCOURSE MASTER PLAN
FCD 2001C024

Consultants:

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Legend:

- BANK STATION
- CROSS SECTION
- PROPOSED KING RANCH BANK PROTECTION/BANK ARMORING
- PROPOSED LEVEE ALIGNMENT (BY OTHERS)
- PROPOSED LEVEE ALIGNMENT
- KING RANCH CHANNEL
- CENTERLINE

100-YEAR FLOOD ZONE

- A
- AE
- AH
- FLOODWAY

Notes:

Title:

FIGURE 1
EL RIO WMP ALTERNATIVE ELEMENTS FOR THE SEDIMENTATION ANALYSIS

Produced by: BGS

Date: 2/12/2006

Each element that is analyzed for sedimentation impacts is shown in Figure 1 and is briefly described in the following:

LEVEES

Levees are proposed (see the Alternative Evaluation Report) that generally follow the floodway delineation on the north side of the river and short segments along the south side. The complete levee configuration is included in all sedimentation analyses.

SAND AND GRAVEL MINING PITS/RECREATIONAL LAKES

Sand and gravel pits in the El Rio study area are identified as:

- the BWCDD Lake upstream of the confluence with the Agua Fria River,
- the Tuthill Pit within a half mile downstream of the Tuthill Bridge, and
- a zone of several pits in the right overbank (ROB) from 1 to 3 miles upstream of the SR 85 Bridge. It is assumed that those pits eventually merge into one contiguous pit and that is the ultimate condition that is analyzed.

BWCDD DIVERSION STRUCTURE

The BWCDD Lake will be developed by the excavation of a sand and gravel pit and a downstream diversion structure to divert irrigation water into the BWCDD Main Canal. The diversion structure will be an earthen embankment that would be overtopped and probably washed out during any sustained floods. For the purpose of reconstructing the earthen diversion after passage of floods, a structural foundation will be provided upon which to build the earthen embankment. The foundation will be an erosion resistant pad constructed of concrete or cement stabilized alluvium. Therefore, the foundation of the diversion structure will act as a grade control structure.

The impact of the BWCDD diversion structure and pit on the sedimentation of the pit and of the river downstream of the pit is analyzed. The cumulative impact of that structure along with the other downstream pits is evaluated.

RESOURCE ENHANCEMENT

Resource enhancement is the selective removal and replacement of existing vegetation with alternative plant species. Although resource enhancement in the El Rio WMP takes several forms, the only aspect that is evaluated for sedimentation impacts is vegetation enhancement. Vegetation enhancement does not include regrading or large-scale earth moving, the topography remains essentially unchanged. Vegetation enhancement is being considered for the reach of river from the confluence with Waterman Wash to the SR 85 Bridge. There are small, isolated tracts of land that are proposed for vegetation enhancement on the left overbank of the river. Those tracts are too small and isolated to result in meaningful in-situ or downstream sedimentation impacts. However, the right overbank (ROB) comprises a large area of the El Rio WMP study area and that area is generally covered by thick stands of tamarisk. Therefore, vegetation enhancement of the ROB could result in significant in-situ and/or downstream sedimentation impacts.

Vegetation enhancement could take many forms. Factors to be considered in regard to sedimentation impacts are:

- extent of area undergoing treatment at any point in time
- the timing and duration of treatment
- the impact of the treatment on flow hydraulics
- the resistance to flow in the treated area during the time that vegetation is disturbed until the replacement vegetation can stabilize the soil
- the resistance to flow in the treated area after the replacement vegetation matures

There are an infinite number of combinations of these factors. Some vegetation enhancement treatments can be evaluated intuitively regarding sedimentation impacts. For example, a small, isolated plot of land for which the treatment practice would not appreciably affect the hydraulics of flow would not have adverse sedimentation impact. Alternatively, large-scale vegetation removal would expose unprotected soil to overtopping flood flows thus potentially resulting in soil erosion. Such large-scale vegetation enhancement practices need to be assessed in regard to in-situ and potentially adverse downstream sedimentation impacts. Several scenarios of large-scale vegetation enhancement on the ROB were assessed for sedimentation impacts.

KING RANCH/COTTON LANE BRIDGE CHANNELIZATION

The channelization and other modifications to the river in the King Ranch region are not alternatives for the El Rio WMP, nor are they existing conditions. However, the channelization and modifications to the river are incorporated into every sedimentation analysis; that is, they are treated as existing conditions. The information was obtained from WEST Consultants, Inc. as supplied during May 2005.

SEDIMENTATION CONCERNS TO BE ASSESSED

Sedimentation impacts from each alternative element can be in-situ (local or in the immediate vicinity) and/or downstream of the site. Both situations must be adequately addressed to avoid adverse environmental impact. Potential sedimentation concerns are assessed by comparing measures of sedimentation, such as sediment transport rates, volumes of sediment, erosion and scour depths for the river without the alternative (the base condition) to a model of the river with the element of the alternative. Each alternative element presents unique sedimentation concerns; therefore, the method to assess the sedimentation impact varies by element. The following is a discussion of the sedimentation concerns for each element.

LEVEES

The levees are set at the floodway limit and as such the levees will have very little impact on flow hydraulics (velocity and depth). Therefore, the sedimentation impact of the levee will be very small in regard to the river as a whole. However, the levees may produce flow conditions or be subjected to flows that would jeopardize the levee from scour. Part of the design of a levee includes toe-down for scour protection. The estimated toe-down depth for the levees is presented in the Scour Analysis section of this report.

VEGETATION ENHANCEMENT

The sedimentation impacts of vegetation enhancement are very complex. They are a function of both the manner in which the vegetation enhancement is carried out and the flow hydraulics including the sediment load. The vegetation enhancement factors that may affect sedimentation are:

- areal extent of the treatment
- orientation relative to flow paths
- effect on flow resistance which may vary from an initial low resistance to higher resistance as vegetation matures
- increase in conveyance resulting in increased flow velocity in the area of the vegetation enhancement

The hydraulic factors that may affect sedimentation are:

- the occurrence of floods
- the magnitude and duration of the flood
- the capacity of the vegetation enhanced area to capture a larger portion of the flood discharge

- the sediment load

- the distribution of the sediment load that would be diverted into the vegetation enhanced area

- the sediment transport capacity through the vegetation enhancement area

The temporal and spatial factors to be considered when evaluating vegetation enhancement, along with the uncertainty and 3-dimensional flow hydraulics, makes a sediment analysis difficult to perform and interpret. Since the El Rio WMP is a planning study, only generalized conditions can be considered. As specific vegetation enhancement options are considered, appropriate flow hydraulics and sedimentation analyses will need to be performed.

SAND AND GRAVEL PITS/RECREATIONAL LAKES

The pits and lakes will serve as sediment traps. The consequences are that if the pits/lakes capture the flood flows, they will trap sediment and the river downstream will experience "clear water" scour. Adverse impacts downstream could accumulate depending on the magnitude of the pits, the trap efficiencies, the number of pits in operation and the timing of floods. In-situ impacts would include the potential for filling the pits with sediment and the advance of the downstream pit wall or the headcut of the upstream pit wall endangering structures such as bridges or contributing to local scour depth at levees. Downstream impacts could include accelerated degradation of the river channel.

BWCDD DIVERSION STRUCTURE

The foundation of the diversion structure will be an erosion resistant structure that intersects the riverbed. As such, it will serve as a grade control structure. During flood flows when the earthen berm is washed out, the foundation would cause some local scour downstream of the structure. That is assessed in the Scour Analysis section of this report. More importantly, the pit upstream of the structure will trap sediment resulting in depleted sediment loads past the pit and foundation. The foundation will serve as a hard point limiting advance of the pit wall downstream. Downstream of the foundation, the "clear water" may have excess sediment transport capacity leading to degradation of the riverbed downstream of the diversion and may contribute to local scour at the foundation, of the diversion structure.

BASE HYDRAULIC AND SEDIMENTATION MODELS

BASE HYDRAULIC (HEC-RAS) MODELS

Hydraulic models using HEC-RAS were developed to assess various configurations of vegetation enhancement. The hydraulic parameters of velocity, depth and percent discharge through the vegetation enhancement area were compared between the base condition (no vegetation enhancement) and the vegetation enhanced condition. The base model for comparison was taken directly from the existing conditions base hydraulic model as described in the Existing Condition Hydrology and Hydraulics Memorandum. This model contained geometry through the King Ranch portion of the model (RS 188.69 to 194.20) that was updated in 2004. The hydraulic model contained bank stations that were set to FEMA floodway limits, whereas the sediment model bank stations are set at the active channel limits. In order to make the hydraulic model overbanks comparable to the overbanks of the sediment model, the bank stations were adjusted along the entire model length. For the most part, this created a narrower channel and wider overbanks than are represented in the original hydraulic model.

BASE SEDIMENT TRANSPORT MODELS

Sediment transport models using HEC-6T were developed to investigate sedimentation impacts that may result from elements of the alternative. The model 'base.t5' described in the Existing Conditions Sedimentation Analysis was used as the base condition for HEC-6T comparisons. That model includes topography from RS 188.69 to 194.20 that was updated in 2004 for the King Ranch project, and bank stations which were set at the active channel limits. In order to provide meaningful comparisons for various alternative models, the base model was run for four different hydrologic events; flows to simulate the 1993 and the 1980 floods, a sequence of flows simulating the March 1978 through February 1980 floods, and the full sequence of flood hydrology developed in the Existing Conditions Sedimentation Analysis.

ANALYSIS OF LEVEES

Levees are recommended in the El Rio WMP for much of the right bank of the river and several portions of the left bank. The King Ranch/Cotton Lane Bridge modifications that are proposed by others include additional levees and bank protection on both banks of the river (see Figure 1 for location of the levees and bank protection). For the most part, the alignment of those flood protection works is at the floodway line. The existing condition HEC-6T model is modified to include the recommended El Rio WMP levees and the King Ranch/Cotton Lane Bridge levees, bank protection and channelization.

The levees are analyzed using that HEC-6T model for the following hydrologic sequence of discharges:

1. The 1993 flood consisting of the 24 days with discharges greater than 35,000 cfs. That is the historic flood with the greatest volume of streamflow through the study area.

2. The 1980 flood consisting of 11 days with discharges greater than 35,000 cfs. That is the largest recorded flood.

3. A sequence of floods, represented by the period from 1978 through 1980, consisting of 27 days with discharges greater than 35,000 cfs. That hydrologic event represents a sequence of closely spaced large floods.

4. The full hydrologic sequence that was used in the El Rio Existing Condition Sedimentation Analysis. That hydrology is based on historic flows from 1921 through 2004, plus a 100-year flood at the end of that sequence.

The levees in the El Rio WMP study area, being at the floodway line, will have little, if any, measurable affect on the hydraulics of flow (velocity and depth) in the channel of the river as defined by the HEC-6T bank stations (see Figure 1). The channelization, levees and bank protection for the King Ranch development project similarly result in little affect on flow hydraulics. The construction of the river for the Cotton Lane Bridge will result in some additional bed scour over the existing condition, but that analysis is not within the scope of the El Rio WMP and is to be performed by others. A measure of the net impact of the proposed levees, bank protection and channelization on the sedimentation of the river is the sediment load passing the SR 85 Bridge. The total sediment loads passing the SR 85 Bridge for the four modeled hydrologic sequences for the existing condition and the with levee condition are:

1. For the modeled portion of 1993 flood, the sediment load for the existing condition is 96.2 million tons, and for the with levee condition it is 96.7 million tons.

2. For the 1980 flood, the sediment load for the existing condition is 80.7 million tons, and for the with levee condition it is 80.4 million tons.

3. For the floods between 1978 and 1980, the sediment load for the existing condition is 138.0 million tons, and for the with levee condition it is 138.6 million tons.

4. For the full hydrologic sequence, the sediment load for the existing condition is 349.9 million tons.

The levees and King Ranch/Cotton Lane Bridge modifications to the river will have no measurable impact on the sediment balance of the river. The four HEC-6T models of the levee condition are provided on CD in Attachment 1.

ANALYSIS OF VEGETATION ENHANCEMENT

DESCRIPTION OF VEGETATION ENHANCEMENT

Vegetation enhancement is the selective removal of vegetation and replacement by other vegetation species. For example, removal of tamarisk and replacement by cottonwood/willow. Vegetation enhancement is not the same as vegetation clearing where existing vegetation is cleared by mechanical or other method without replacement by alternative vegetation. However, vegetation enhancement requires an interim period between the removal of existing vegetation and the development of well established replacement vegetation. During that time, the soil is exposed and subject to erosion by floods.

Vegetation enhancement is a viable alternative for the reach of the Gila River from near Tuthill Bridge (RS 188) to the SR 85 Bridge (RS 180). That reach of river has a wide right overbank (ROB) (as defined for sediment and hydraulic modeling) ranging from about 2,000 feet to more than 9,000 feet. That area is covered to a large extent by dense tamarisk (see Figure 1). Because the soil in the ROB is a very fine sandy silt (see soil gradation for sample 6 in Figure 34 of the the Existing Conditions Sedimentation Analysis), that soil would erode and could produce high rates of sediment transport if exposed and subjected to high flow velocities. Portions of the ROB have secondary flow channels (braids) that convey significant quantities of flood discharges. The large and sustained flood of 1993 demonstrates that overbank areas vegetated by tamarisk are able to maintain the vegetation throughout the flood thus retaining the fine soil and keeping it from eroding and being flushed downstream. That action on the overbanks is contrasted to the main channel of the river that is generally deeper, less densely vegetated, has coarser bed material and which conveys a large portion of flood discharges. During the onset of flood discharges, the main channel widens quickly through bank erosion, which removes vegetation with the bank retreat thus providing an efficient channel with a large flood conveyance capacity. During floods, the channel carries 50 to as much as 100 percent of the total flow.

A condition to be assessed is the potential to erode large quantities of the fine soil in the ROB during a flood(s) if portions of the ROB were temporarily cleared for vegetation enhancement. The consequence of such an occurrence would be the creation of radically eroded areas in the ROB with the possible deposit of massive quantities of sediment downstream of the El Rio study area. Portions of those sediments could be trapped in the thick tamarisk floodplain downstream of the SR 85 Bridge although large quantities would flush through the river to Painted Rock Reservoir. The consequence of such an event would be an adverse impact to the river in the El Rio study area and downstream. The potential sedimentation of the river due to vegetation enhancement was investigated by the use of both the HEC-6T and the HEC-RAS models.

SEDIMENTATION ANALYSIS USING HEC-6T

A sedimentation analysis of vegetation enhancement was attempted by modifying the existing condition HEC-6T model. The major modifications to the model were:

1. The bed material size gradation was changed to represent the fine material in the ROB, and

2. It was assumed that vegetation would be removed in continuous corridors through the ROB and the flow resistance was reduced to 0.04 in those assumed corridors.

Various methods were attempted to model those two conditions in the HEC-6T model. Those efforts were not successful because:

1. The 3-dimensional nature of this sediment transport phenomenon exceeds the capability of the 1-dimensional limits of HEC-6T.

2. Modeling the two bed material size distributions, that is, the coarser bed material in the channel and the very fine bed material in the ROB, required the use of the split-flow (island) option. Use of that option requires the assumption that flow remains separated throughout the modeled reach. Although hydraulically successful, the hydraulic results were unreasonable in restricting the flow of water from exchanging between the channel and the ROB. Secondly, the ROB, even in the cleared corridor, became a depositional zone. That occurred because coarser bed material from upstream of the flow split would enter the secondary channel in the same proportion (concentration) as in the main channel. The lessened transport capacity in the secondary channel resulted in local deposition of coarse bed material, reduced water conveyance capacity and reduced sediment transport capacity and, therefore, sediment deposition. Due to the inability to reasonably model the sedimentation of the ROB using HEC-6T, an alternative analysis technique was attempted using HEC-RAS.

HYDRAULIC ANALYSIS USING HEC-RAS

Hydraulic models of the El Rio study reach were developed to investigate flow hydraulics on the ROB while that area was subjected to vegetation enhancement practices. The assumptions of such modeling are:

- A corridor of vegetation would be temporarily cleared of native vegetation in preparation for revegetation.

- During that interim period a flood would occur and a portion of the flood discharge would pass through the cleared corridor.

- The flow resistance in the cleared corridor is reduced to represent flow resistance after removal of tamarisk. A Manning n value of 0.04 is assumed for the areas undergoing vegetation enhancement. That value is based on the assumption that although larger vegetation, such as tamarisk, is removed, smaller vegetation ground cover (bushes, grass and forbs) will remain, Additionally, the land surface in areas undergoing vegetation enhancement will be irregular (hummocky) resulting in form resistance.

The HEC-RAS model that was developed by Stantec for the purpose of investigating the hydraulic performance of the preferred alternative was used (see the Alternative Evaluation Report for a discussion of that model). That model was then modified to represent various conditions of vegetation enhancement in the lower portion (between Tuthill Bridge and the SR 85 Bridge) of the El Rio study reach. The following HEC-RAS models were developed:

1. Base Condition

    - existing condition without vegetation enhancement

- bank stations were reassigned for each cross section in the HEC-RAS model to agree with bank stations in the existing condition HEC-6T model

2. 200-foot Wide Vegetation Enhanced Corridor

    - levees as recommended by the El Rio WMP
    - channelization as proposed for the King Ranch land development project
    - the Cotton Lane Bridge
    - A 200-foot wide corridor for vegetation enhancement within the ROB extending from RS 186.78 to RS 180.04.
    - The "n" value in the 200-foot wide corridor was set to 0.04.

(The source of hydraulic information for the King Ranch area and the Cotton Lane Bridge was provided by WEST Consultants, Inc., dated May 2005).

3. 500-foot Wide Vegetation Enhanced Corridor

    - The same as 200-foot condition model with a 500-foot wide corridor for vegetation enhancement within the ROB extending from RS 186.78 to RS 180.04.
    - The "n" value in the 500-foot wide corridor was set to 0.04.

4. Full Vegetation Enhanced Corridor

    - The same as the previous two models with the entire ROB subjected to vegetation enhancement from RS 186.78 to RS 180.04.
    - The "n" value in the ROB was set to 0.04.

Digital files of the HEC-RAS models that were used for this hydraulic analysis are provided in Attachment 2.

The results of those four models were compared to investigate the hydraulic effects of vegetation enhancement in the El Rio study area. Each model was run with four discharges:

10-year at 46,000 cfs

20-year at 68,000 cfs

100-year at 210,000 cfs

500-year at 270,000 cfs

The hydraulic characteristics of interest are:

percent flow in the ROB

maximum velocity in the ROB

maximum depth in the ROB

The value of each of the three hydraulic parameters for each of the four discharges and each of the four modeled conditions were plotted versus river station from RS 180.04 to RS 186.78. A set of those graphs for the 500-foot wide corridor is provided in Figures 2 through 4, for percent discharge, maximum velocity and maximum depth, respectively. A full set of all 12 graphs is provided in Attachment 3. Notice that the patterns are similar on each graph and that there is little difference between the 10- and 20-year graphs and the 100- and 500-year graphs.

It is unlikely that the flow hydraulics would fluctuate from one station to the next (about 500 feet apart) as they appear in Figures 2 through 4. The hydraulic parameters probably vary more gradually as influenced by conditions in the river over a longer reach. For that reason, and to aid in the interpretation of the graphs, 5-point moving average graphs were prepared to correspond to Figures 2 through 4 and those are presented in Figures 5 through 7, respectively. A full set of all 12 moving-average graphs is provided in Attachment 4. Notice that, in general, the graphed lines follow consistent patterns regardless of flood frequency, and that the 10- and 20-year flood and the 100- and 500-year graphs are very similar with relatively small differences in magnitudes.

Percent Flow in ROB- Figure 8 is a composite of the 5-point moving average data graphs from Attachment 4 for the 10-year flood for each of the conditions being considered. The following are noted about the percent of flow in the ROB during the 10-year flood:

1. The percentages range from near zero to a maximum of near 50 percent.
2. Over most of that 6.5 mile reach, there is relatively little difference in the percentage of flow in the ROB when comparing the four different conditions.
3. Maximum deviations occur at the upper end (RS 186.2), the lower end (RS 180.3) and near the middle (RS 182.7).
4. Maximum deviations are less than 20 percent.

Figure 9 is a composite of the 5-point moving average data graphs from Attachment 4 for the 100-year flood for each of the conditions being considered. The following are noted about the percent of flow in the ROB during the 100-year flood:

1. The percentage of flow ranges from less than 5 percent to a maximum greater than 50 percent.

2. Full vegetation removal results in greater percent of flow in the ROB for most of the reach.

3. There is often little (less than 10 percent) difference in percent of flow for the base, 200-foot and 500-foot conditions.

4. The maximum deviations are less than 20 percent.

5. The trends and magnitudes are similar in both Figures 8 and 9.

Figure 2
Overbank Hydraulics- 500' ROB Vegetation Removal, Percent of Flow in ROB

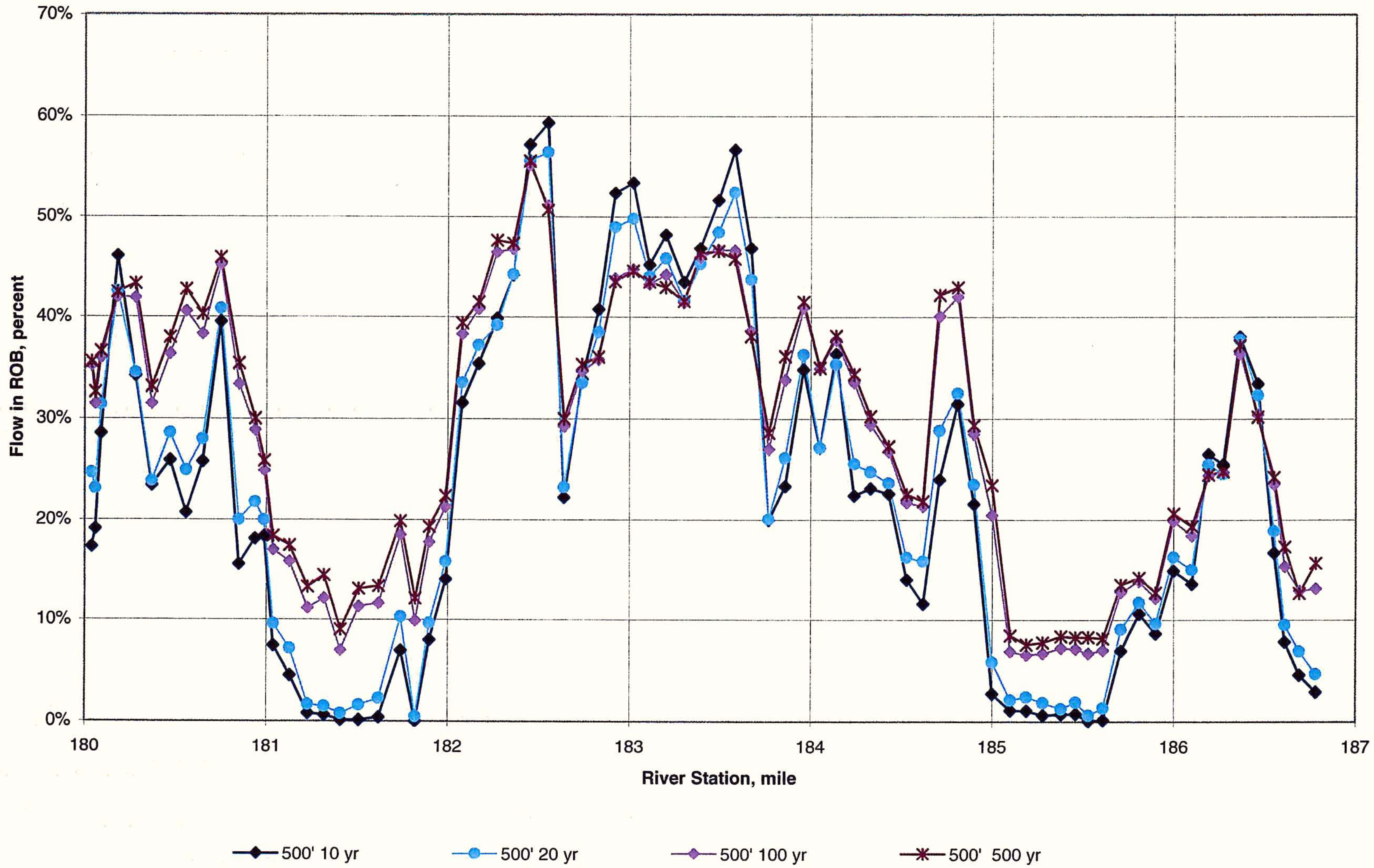


Figure 3

Overbank Hydraulics- 500' ROB Vegetation Removal, Maximum Velocity in ROB

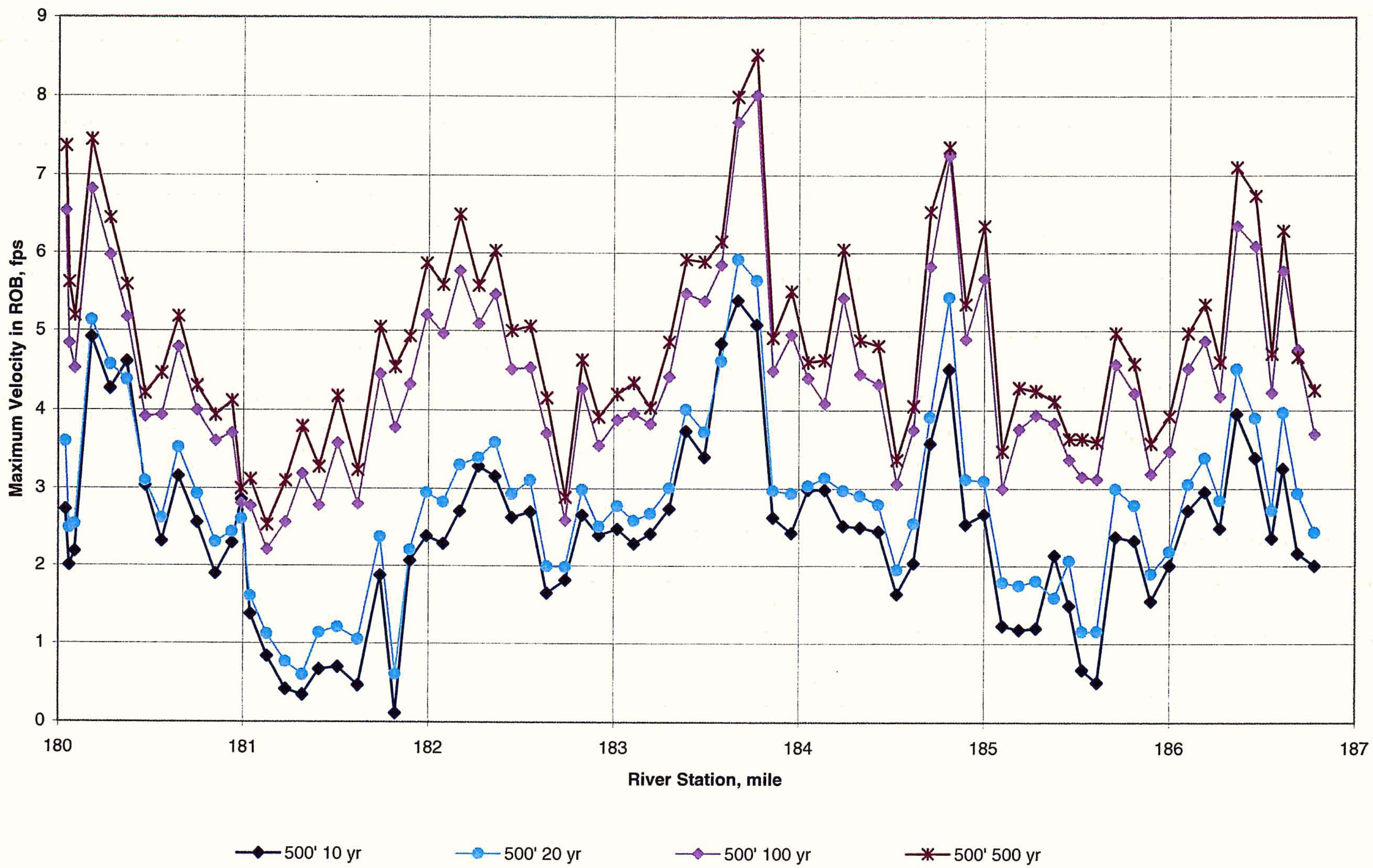


Figure 4
Overbank Hydraulics- 500' ROB Vegetation Removal, Maximum Depth in ROB

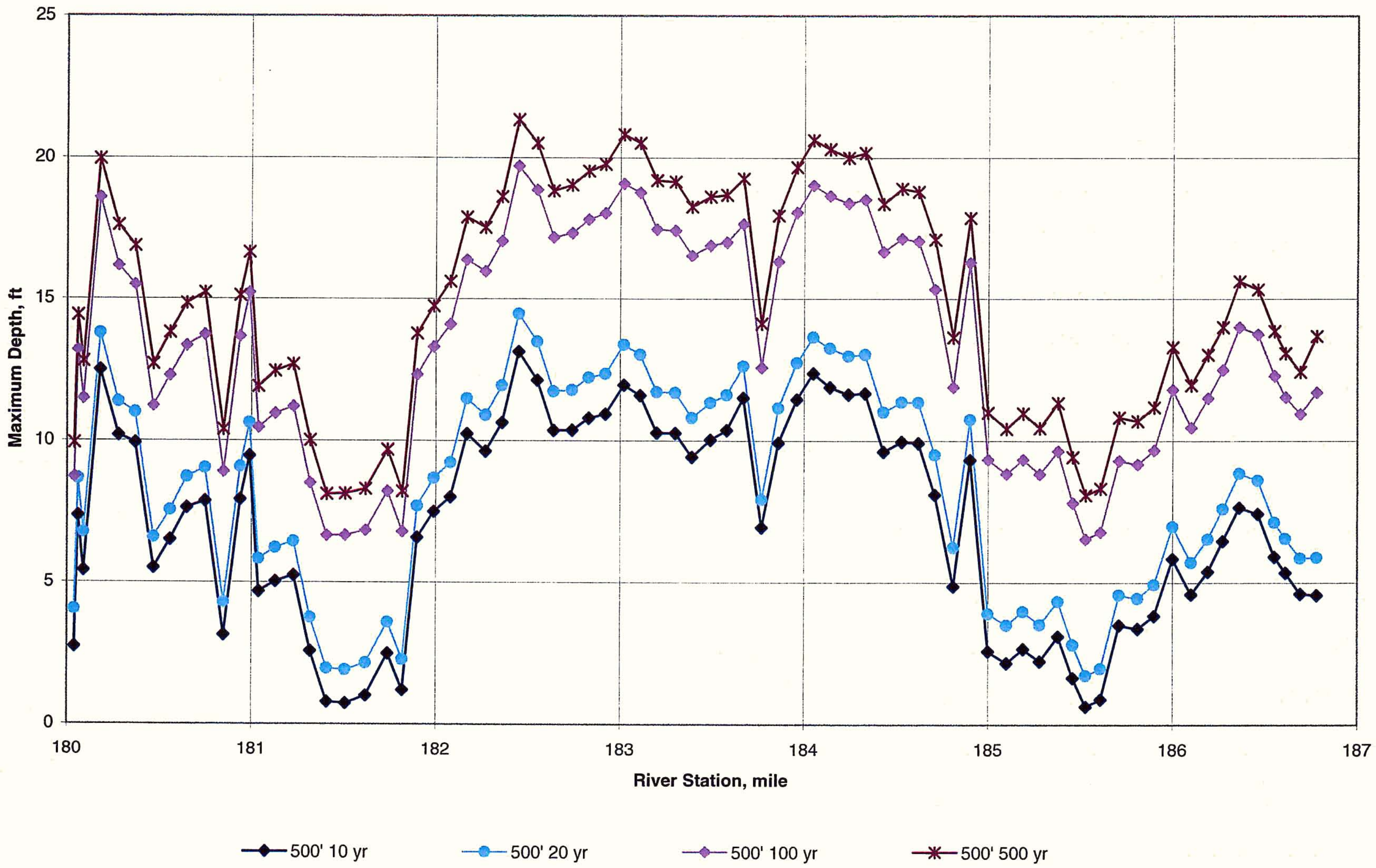


Figure 5
Overbank Hydraulics- 500' ROB Vegetation Removal, Moving Average-Percent of Flow in ROB

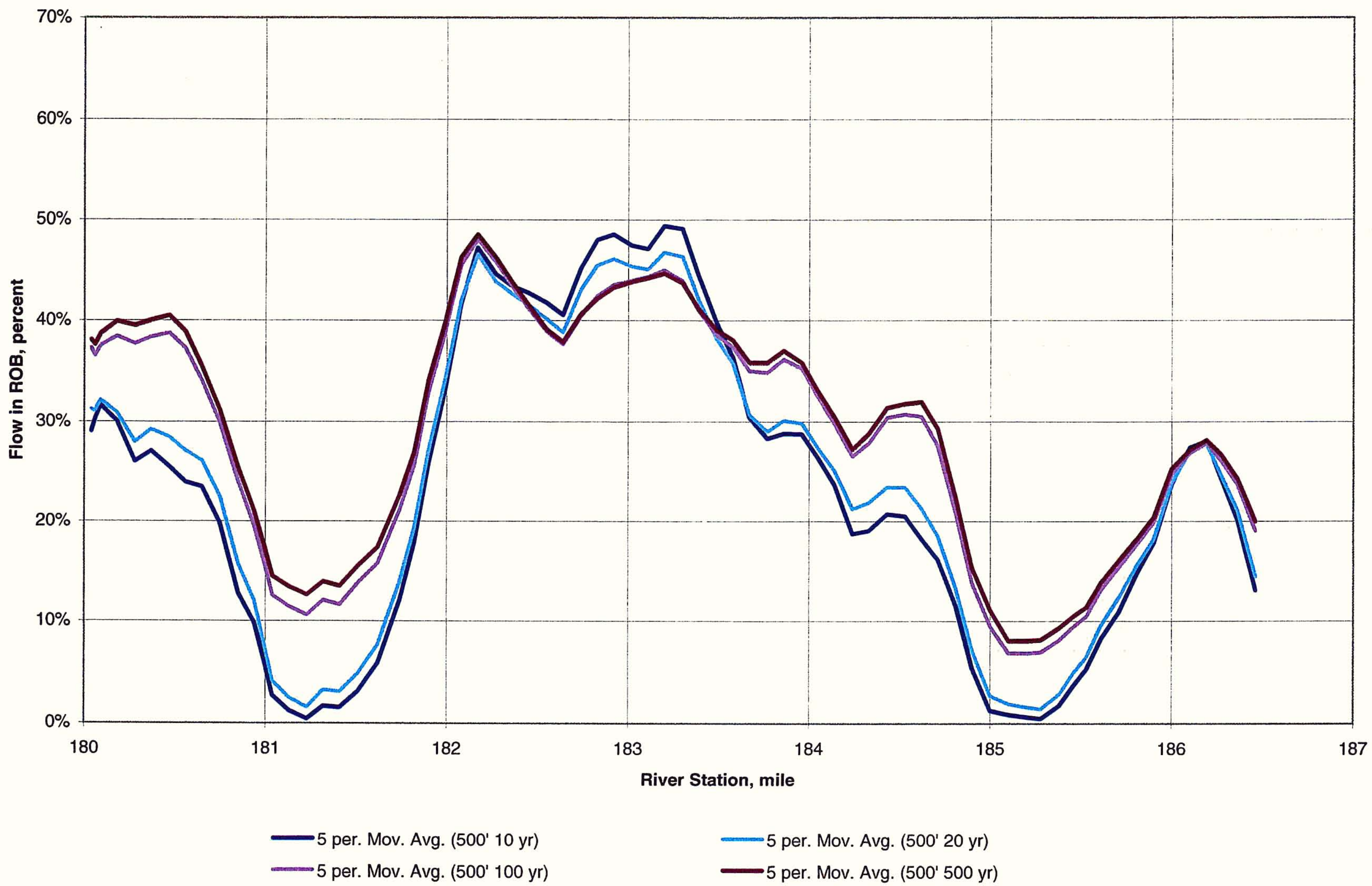


Figure 6
Overbank Hydraulics- 500' ROB Vegetation Removal, Moving Average-Maximum Velocity in ROB

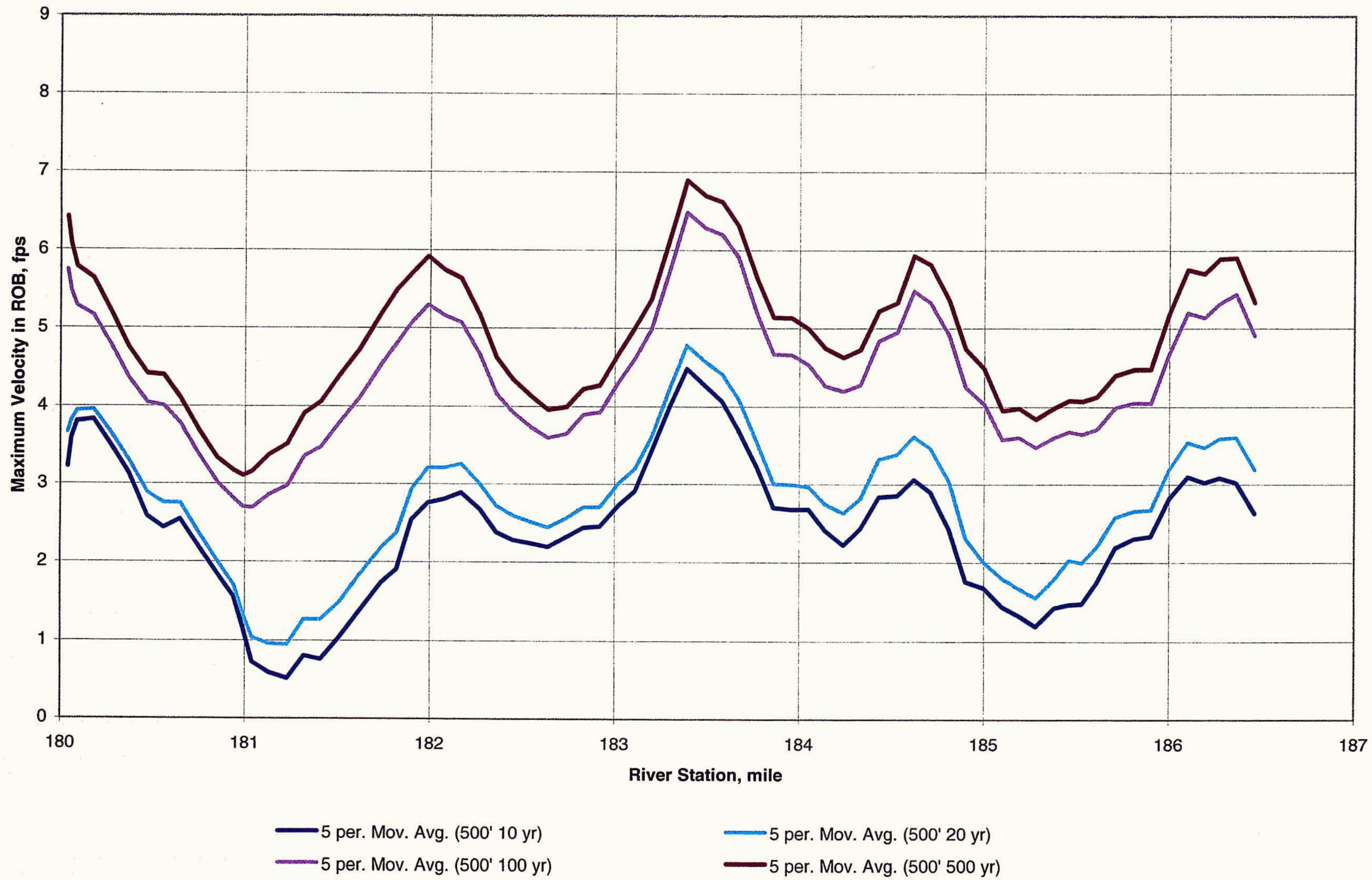


Figure 7
Overbank Hydraulics- 500' ROB Vegetation Removal, Moving Average-Maximum Depth in ROB

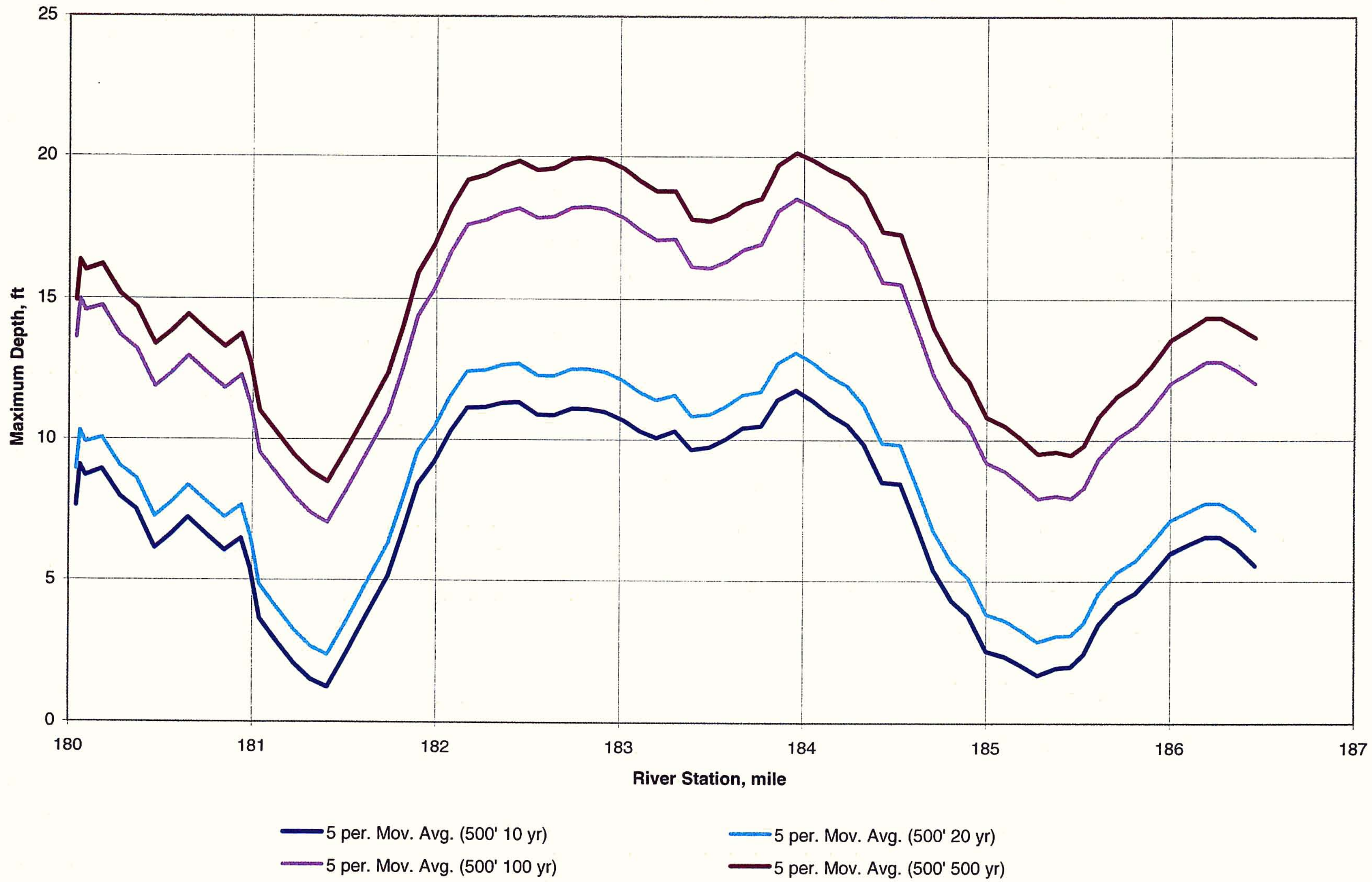


Figure 8
Overbank Hydraulics- 10 yr Comparison, Moving Average-Percent of Flow in ROB

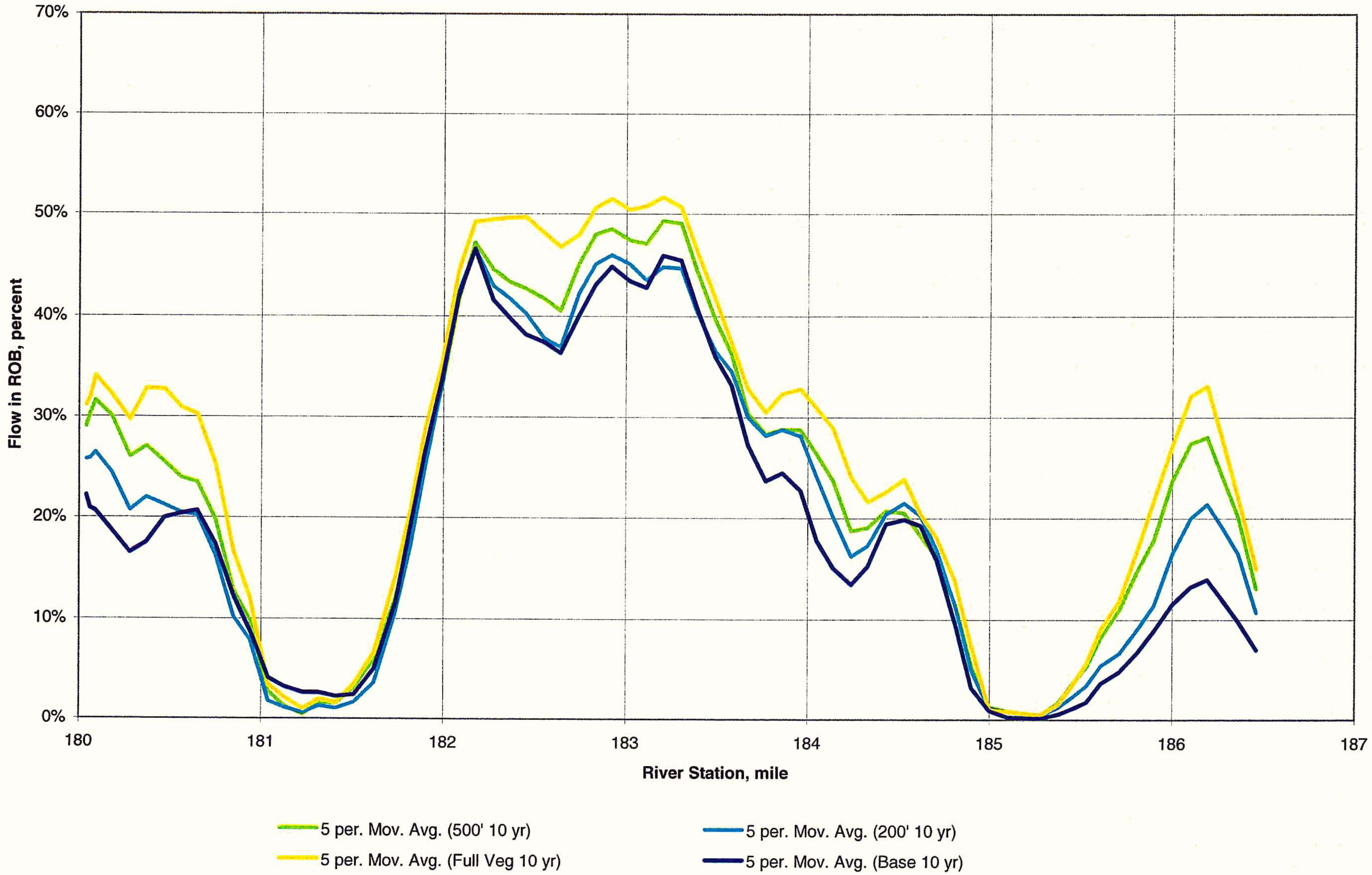
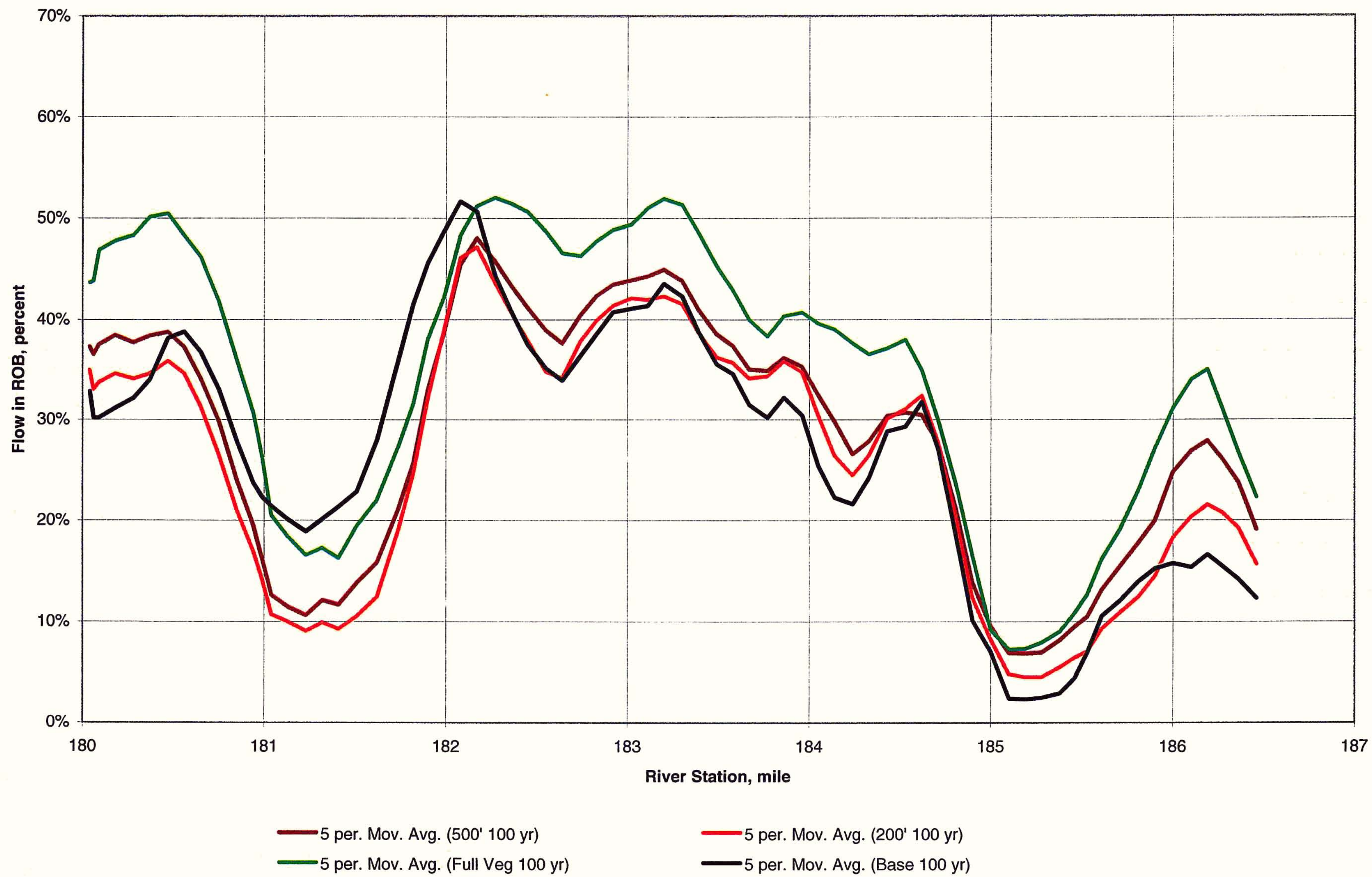


Figure 9

Overbank Hydraulics- 100 yr Comparison, Moving Average-Percent of Flow in ROB



Maximum Velocity in ROB- Figure 10 is a composite of the 5-point moving average data graphs from Attachment 4 for the 10-year flood for each of the conditions being considered. The following are noted about the maximum velocity in the ROB during the 10-year flood:

1. The velocity ranges from less than 0.5 feet per second to less than 4.5 feet per second.

2. The maximum velocities are very similar for much of the river regardless of vegetation enhancement condition.

3. The deviation in velocity from one condition to another is typically small with maximum deviations less than 2 feet per second.

Figure 11 is a composite of the 5-point moving average data graphs from Attachment 4 for the 100-year flood for each of the conditions being considered. The following are noted about the maximum velocity in the ROB during the 100-year flood:

1. The velocity ranges from about 1 foot per second to less than 6.5 feet per second.

2. The maximum velocity can vary by as much as 3 feet per second due to vegetation enhancement condition.

3. The trends of maximum velocity are similar in Figures 10 and 11.

Maximum Depth in ROB- Figure 12 is a composite of the 5-point moving average data graphs from Attachment 4 for the 10-year flood for each of the conditions being considered. The following are noted about the maximum depth in the ROB during the 10-year flood:

1. The maximum depth is minimally affected by the vegetation enhancement condition.

2. The maximum depth varies appreciably throughout the 6.5 mile reach, ranging from 12 feet to barely more than 1 foot.

Figure 13 is a composite of the 5-point moving average data graphs from Attachment 4 for the 100-year flood for each of the conditions being considered. The following are noted about the maximum depth in the ROB during the 100-year flood:

1. The maximum depth is minimally affected by the vegetation enhancement condition.

2. The maximum depth varies appreciably throughout the 6.5 mile reach, ranging from about 18 feet to about 6 feet.

3. The trends of maximum depth are similar in Figures 12 and 13.

Figure 10
Overbank Hydraulics- 10 yr Comparison, Moving Average-Maximum Velocity in ROB

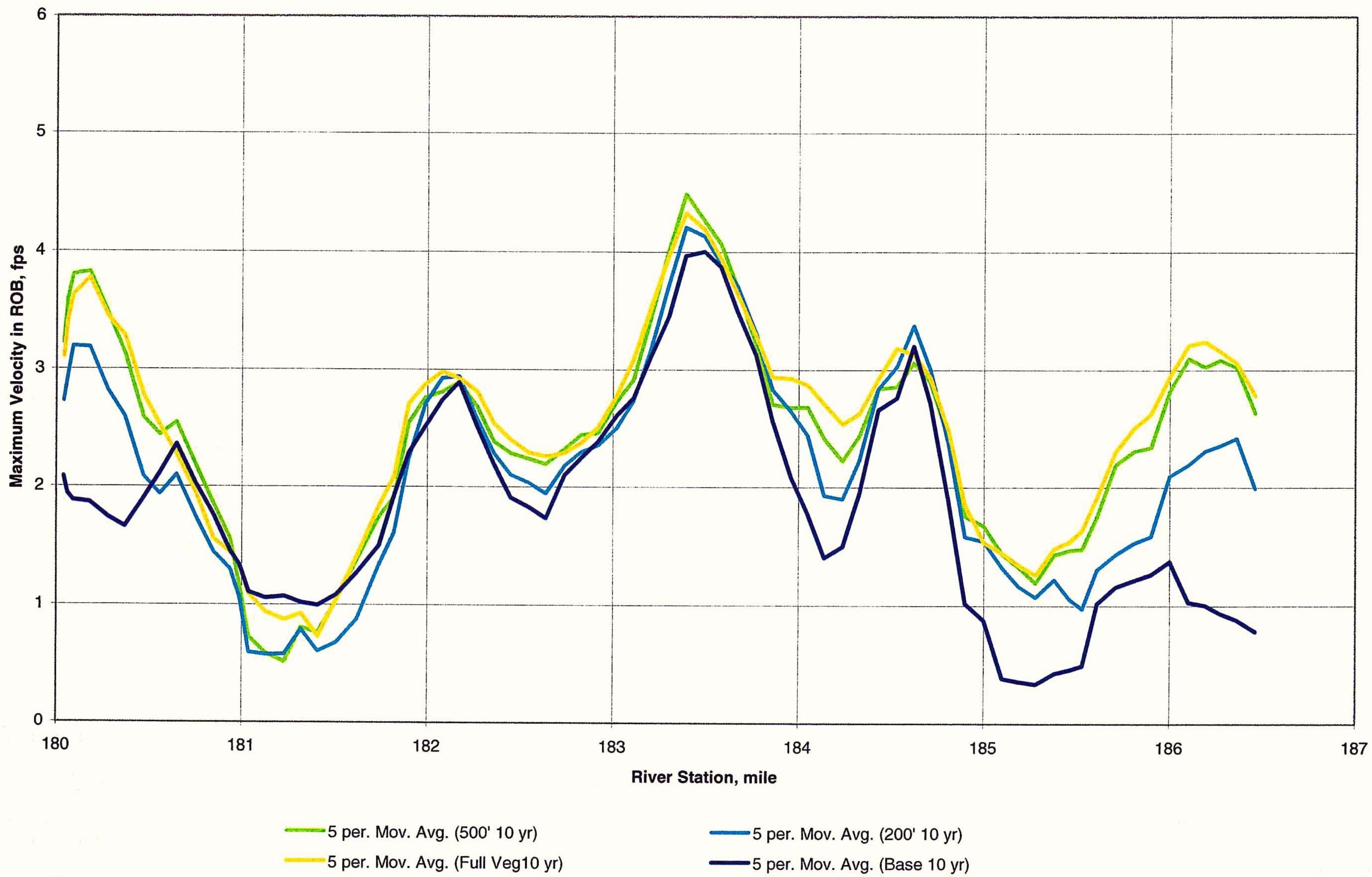


Figure 11
Overbank Hydraulics- 100 yr Comparison, Moving Average-Maximum Velocity in ROB

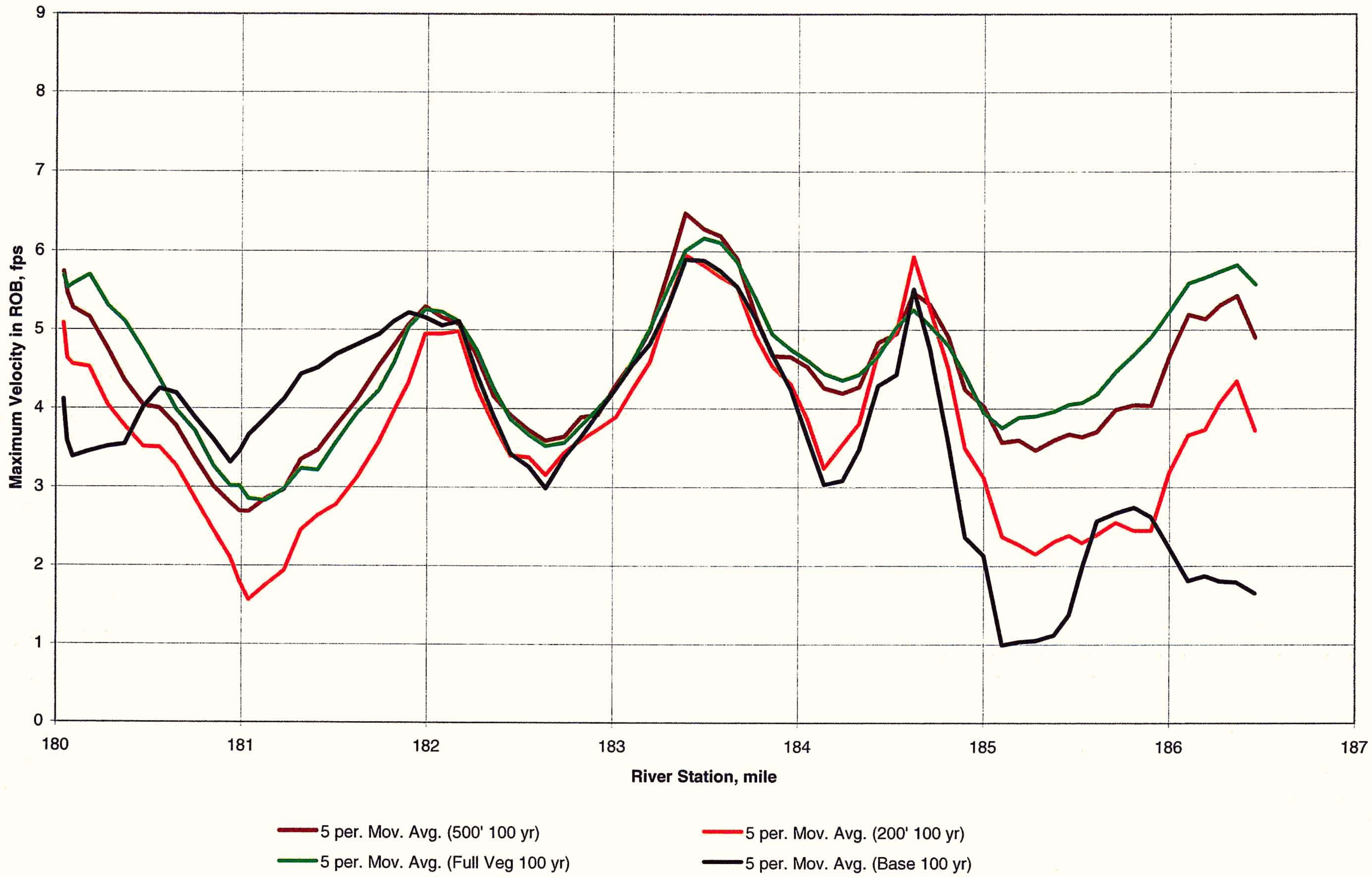


Figure 12
Overbank Hydraulics- 10 yr Comparison, Moving Average-Maximum Depth in ROB

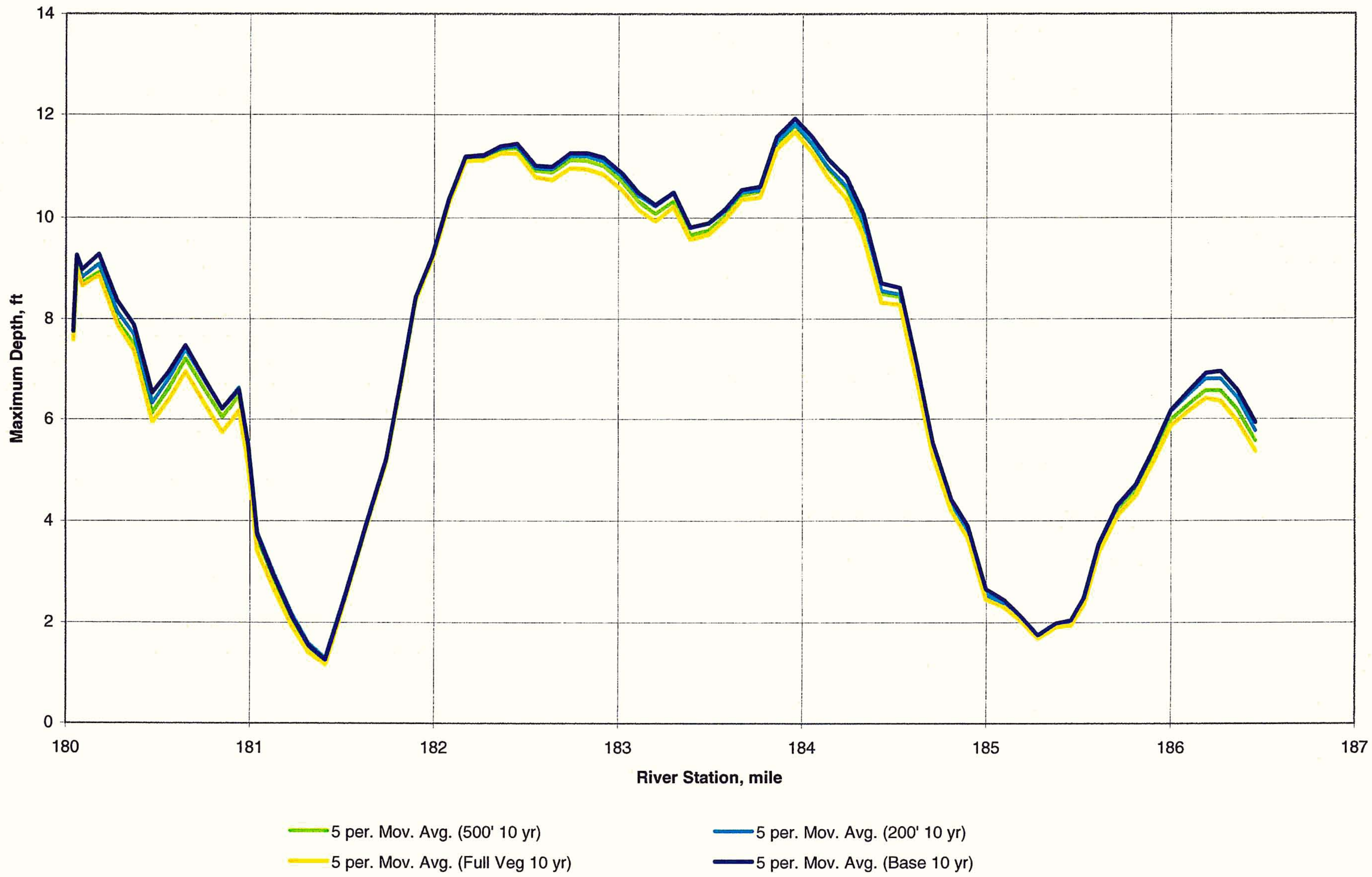
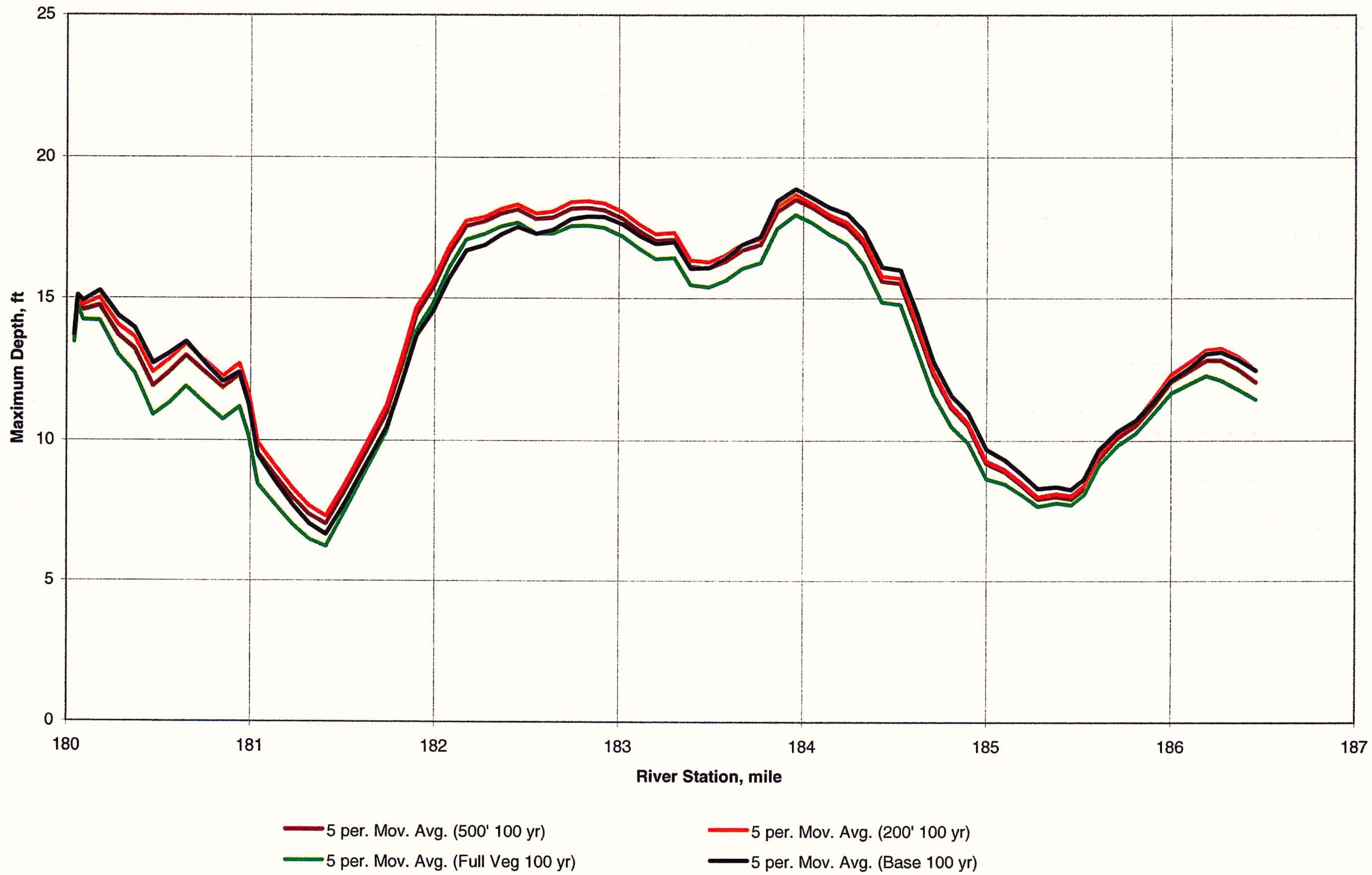


Figure 13
Overbank Hydraulics- 100 yr Comparison, Moving Average-Maximum Depth in ROB



Figures 12 and 13 present interesting and potentially useful results. Those figures indicate that vegetation enhancement, even during the interim transition period of vegetation clearing, provides very little change in flow depth and therefore nearly similar water surface elevations compared to each other or to the existing condition. This analysis shows that for the El Rio study area from RS 180.04 to RS 186.78, that vegetation enhancement will have little effect on the water surface elevation for frequent floods such as the 10-year, or less frequent floods such as the 100-year, or for large floods such as the 500-year (see the graphs in Attachment 4 for 500-year data graphs).

Because of the importance of flow depth (and therefore water surface elevation) in regard to assessing the effectiveness of alternatives for the El Rio WMP, the maximum flow depths were plotted at each modeling station and compared in Figures 14 and 15 for the 10-year flood and 100-year flood, respectively. Those two figures confirm the findings of Figures 12 and 13. The deviations in maximum flow depth in comparing one condition to another are essentially zero for the 10-year flood and are usually less than 1 foot for the 100-year flood.

CONCLUSIONS OF THE VEGETATION ENHANCEMENT ANALYSIS

1. The hydraulic analysis of flow over the ROB reveals that there is little hydraulic advantage to performing vegetation enhancement in the Gila River between RS 180.04 and RS 186.78. Regardless of the magnitude of vegetation enhancement, there is no appreciable lowering of the water surface for floods. Even completely clearing the ROB of large vegetation (tamarisk) results in little lowering of the water surface during floods.

2. Inspection of maximum velocity and maximum depth graphs (see Figures 10 and 12 for the 10-year flood or Figures 11 and 13 for the 100-year flood) reveal that the right overbank has highly nonuniform hydraulic conditions. For the 100-year flood, the hydraulics in the ROB can be velocities of a few feet per second with depths of about 6 feet (RS 181-182) to high velocities of more than 6 feet per second and depths of 19 feet (RS 183-184). Those hydraulics would result in very irregular sediment transport capacities for flow over the ROB. These irregularities would make sediment transport modeling very difficult, as the ROB would shift from reaches of high transport capacity to reaches of low sediment transport. Secondly, inspection of the percent flow in ROB graphs (Figures 8 and 9) show that the water conveyance on the ROB varies appreciably. Flow is continually being exchanged between the ROB and main channel in that reach of the river. That is attributed to the braided channel segments that traverse the reach. Braids in the ROB are periodically diverting flow from the main channel into the ROB and then returning it to the channel. Such dramatically non-1-dimensional flow makes sediment transport modeling with a 1-dimensional model such as HEC-6T rather tenuous.

3. The results of the hydraulic analysis of vegetation enhancement on the ROB using HEC-RAS provides better insight to the hydraulics and sedimentation than does HEC-6T modeling.

Figure 14
Overbank Hydraulics- 10 yr Comparison, Maximum Depth in ROB

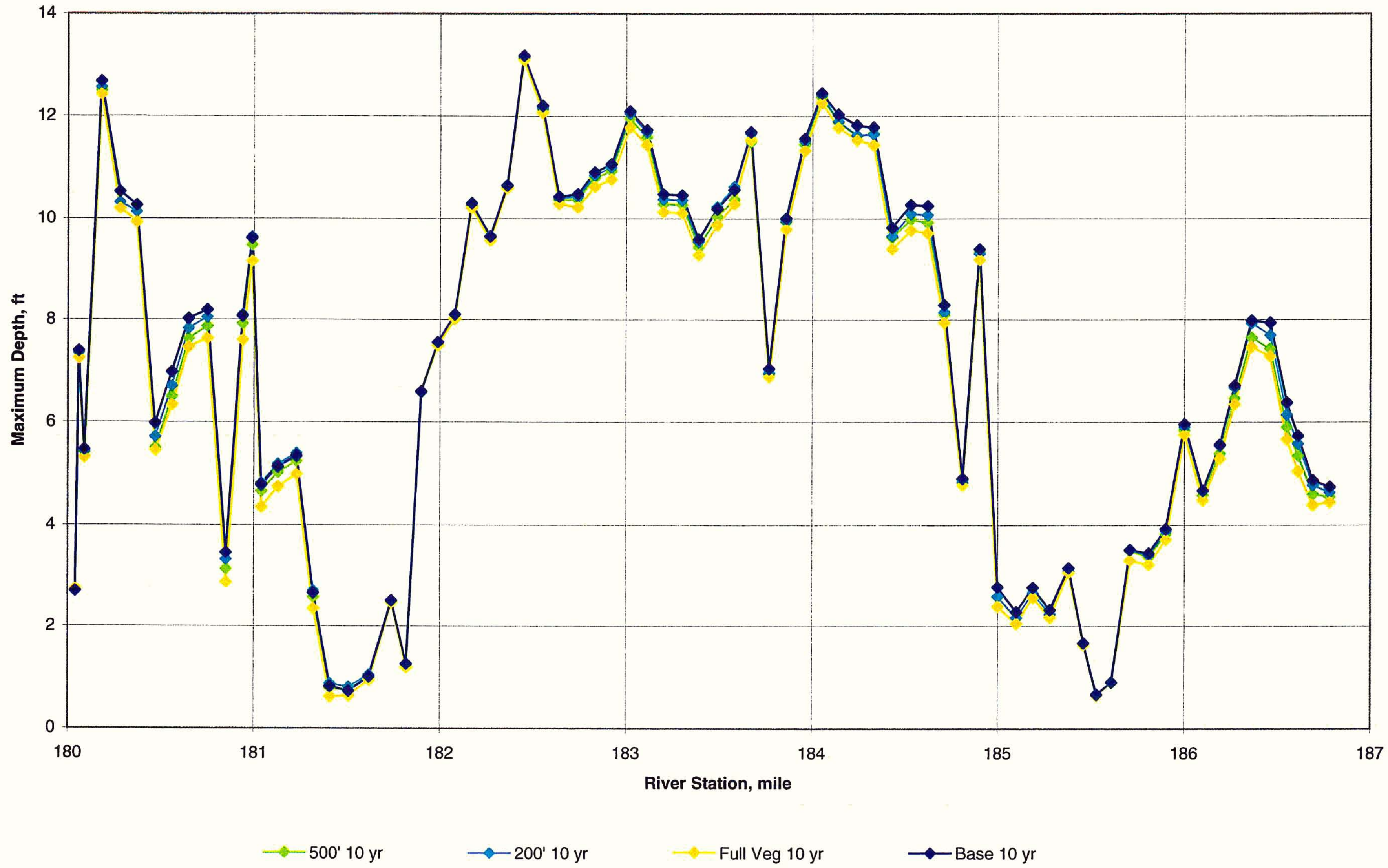
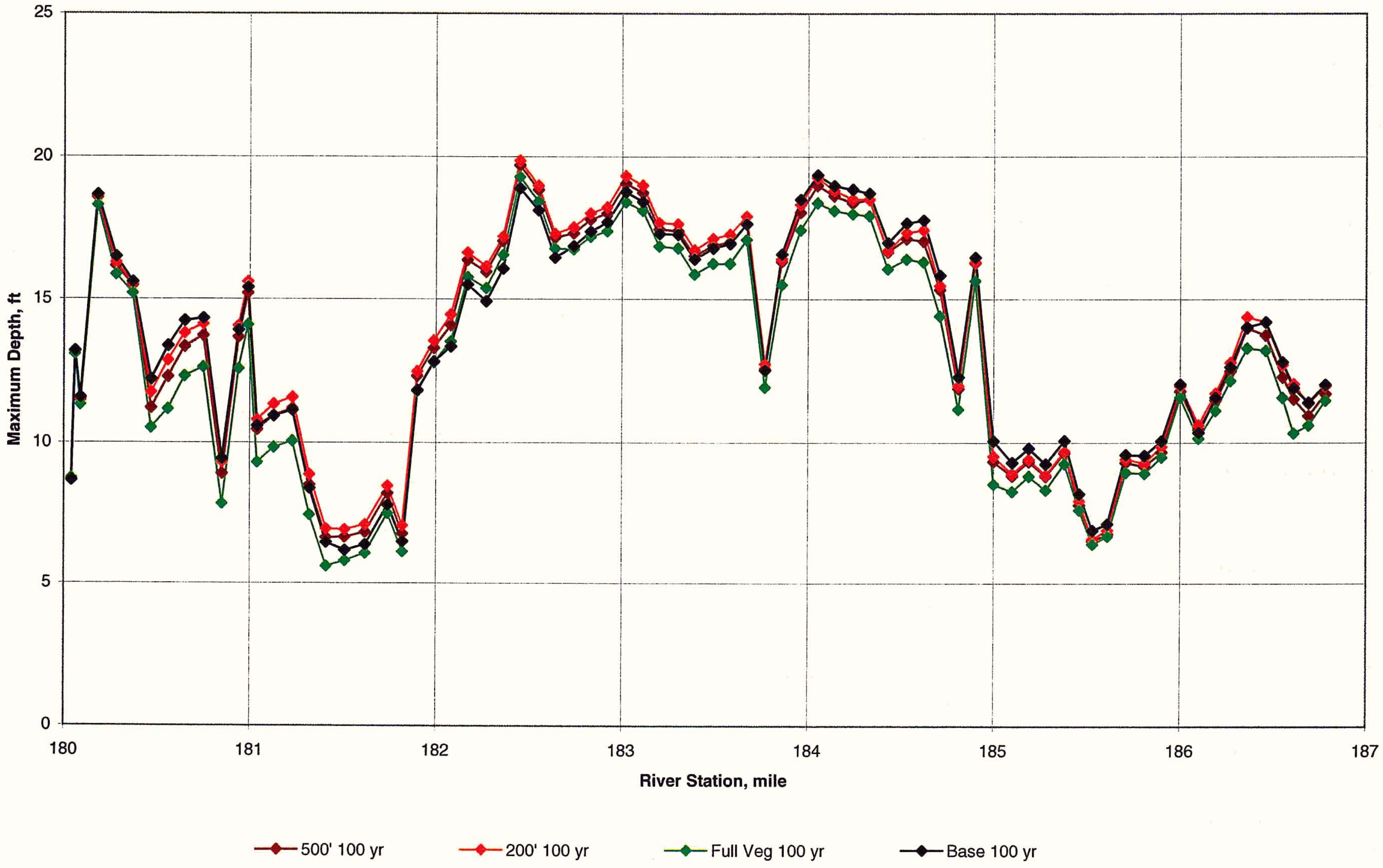


Figure 15
Overbank Hydraulics- 100 yr Comparison, Maximum Depth in ROB



4. Vegetation enhancement of the ROB of the El Rio study area from RS 186.78 to 180.04 offers little hydraulic benefit over the existing conditions without vegetation enhancement.

5. Vegetation enhancement in continuous corridors on the ROB in that reach of the river is to be avoided and could result in severe erosion where flow velocities and depths are large.

ANALYSIS OF GRAVEL PITS & RECREATION LAKES

GENERAL DISCUSSION AND METHOD OF ANALYSIS

Three locations were identified for sand and gravel mining. Those are identified and located as:

BWCDD Lake between RS 196.04 to RS 196.32

Tuthill Pit Between RS 187.45 to RS 187.73

Buckeye Lake between RS 181.13 to RS 183.02

It is not possible at this time to know the precise location, configuration or operational plan of any of the pits/lakes. The locations are estimated based on currently available information and are shown in Figure 1. The configuration of each pit was assumed.

BWCDD Lake would be constructed and operated as part of the BWCDD diversion into the BWCDD (Buckeye) Canal. That pit would result in an expansion of the main channel of the river by excavation into the left overbank of the river.

The BWCDD Lake configuration is dictated to some extent by geologic features and by the location of the BWCDD diversion structure and Buckeye Canal. The configuration of the Tuthill Pit is assumed to be rectangular. It is proposed in the floodway of the river about a half-mile downstream of Tuthill Bridge. The Buckeye Lake configuration is based on the assumption that the several adjacent pits that exist or are proposed for that area will eventually be enlarged to a single pit or that the individual pits function as a single large pit in regard to sedimentation. The Buckeye Lake is located on the right overbank (ROB) of the river. Table 1 provides information on the assumed geometry of each pit and the HEC-6T cross sections that were modified to represent that geometry. The HEC-6T bank stations for the BWCDD Lake and the Buckeye Lake were moved so as to contain those pits. The Tuthill Pit is contained within the bank stations of the base levee modally and did not need to be moved.

Table 1

Assumed Pit / Lake Configurations for HEC-6T Models

Pit / Lake (1)	Modified Cross Sections (2)	Top Width Feet (3)	Length Feet (4)	Depth Feet (5)	Comment (6)
BWCDD Lake	196.04 – 196.23	1,700	2,000	22	Moved bank stations to include lake.
Tuthill Pit	187.45 – 187.73	1,500	1,500	25	Pit is located between the existing bank stations
Buckeye Lake	181.13 – 183.02	2,000	10,000	50	Moved bank stations to include lake.

For the BWCDD Lake, it is assumed that the pit will be operated with relatively small berms to protect the pit from unusual streamflow and more frequent floods (less than 35,000 cfs). Similarly for both the Tuthill Pit and the Buckeye Lake, it is assumed that the operational pits will be protected by berms or even by levees to prevent uncontrolled streamflow from entering the pits. However, the effectiveness of those berms/levees to restrict larger floods from inundating the pits is uncertain. Performing a sedimentation analysis with the assumption that the pits are protected from inundation would not be particularly useful for the El Rio WMP. The results of such an analysis would possibly show some locally induced scour of the river as the floodflows are constricted around the pits, and such scour could be seriously detrimental to structures such as bridges and levees, and could adversely impact adjacent lands. More importantly, if the pits/lakes are not active features of the river, then they will have no adverse impact in regard to sedimentation beyond local flow encroachment induced scour. However, without adequate details of the pit protective works, such an analysis is beyond the scope of the El Rio WMP. It is assumed that such analyses will be performed as individual pit operators make application for sand and gravel extraction.

The pits/lakes are analyzed without any flow protection berms or levees, in fact, for purposes of HEC-6T modeling, the pits are included within the bank stations of the river models. That approach is valid for two conditions; one being that regardless of attempts to protect the pits from flood inundation, the pits are in the active channel of the river during floods; and two, the pits are maintained as open water features after the sand and gravel extraction is finished. In both cases it is valid to include the pits/lakes within the active flow and sediment transport portion of the channel. All of the HEC-6T models of the pits/lakes are based on the assumption that the pits are not protected from floodflows and that the pits are within the bank stations (erodible limits) of the river.

The pits are analyzed for the following hydrologic sequence of discharges:

1. The 1993 flood consisting of the 24 days with discharges greater than 35,000 cfs. That is the historic flood with the greatest volume of streamflow through the study area.

2. The 1980 flood consisting of 11 days with discharges greater than 35,000 cfs. That is the largest recorded flood.

3. A sequence of floods, represented by the period from 1978 through 1980, consisting of 27 days with discharges greater than 35,000 cfs. That hydrologic event represents a sequence of closely spaced large floods.

4. The full hydrologic sequence that was used in the El Rio Existing Condition Sedimentation Analysis. That hydrology is based on historic flows from 1921 through 2004, plus a 100-year flood at the end of that sequence.

The hydrologic input is as described for those events in the El Rio Sedimentation Analysis – El Rio Existing Condition Sedimentation Analysis report.

Each pit was analyzed individually then the combined impact of all three pits was analyzed. All analyses were performed by modifying the existing condition HEC-6T model. That model was modified to include the levees and the King Ranch/Cotton Lane Bridge improvements in addition to modifications that were necessary to incorporate each pit into the HEC-6T models. The digital HEC-6T files for each model are provided in Attachment 1.

The results of the HEC-6T model results are presented in selected tables and graphs of model output. Additional graphics of model output are provided in attachments. The model results are presented in the following manner: First, the response of each pit (without the presence of the other pits) to each of the four hydrologic sequences is evaluated. The time to fill the pit, local upstream and downstream scour, and volume of sediment trapped by the pit are discussed. Second, the response of the river downstream of the pit is compared to the response without the pit (the existing condition). Results are presented in terms of sediment load. Changes in bed elevation were also inspected but those are averages for the cross section, therefore they do not represent the magnitudes of actual bed elevation changes that can be expected. Because of this, average bed change elevation is rather meaningless in this case and is not reported. A key location to assess the impacts of the pits is at the downstream boundary of the El Rio WMP study area, the SR 85 Bridge.

INDIVIDUAL & CUMULATIVE IMPACTS AT THE PITS

<u>BWCDD Lake</u> - The filling of the BWCDD Lake by the four hydrologic sequences are illustrated in Figures 16 through 19. Figure 16 shows that the pit essentially fills in the first five days of the 1993 flood and that the pit traps about 2 million cubic yards of sediment. Figure 17 shows the pit filling with about 2 million cubic yards of sediment in the first four days of the 1980 flood. Figure 18 shows that it takes about the first six days of the 1978–1980 hydrologic sequence to trap about 1.8 million cubic yards of sediment. Interestingly, that long duration event ends with the large 1980 flood (the last 11 days in

Figure 18). During the 1980 flood, some of the trapped sediment in the pit is eroded from the pit. Figure 19 shows that with relatively moderate floods, it takes eight days to trap about 1.7 million cubic yards of sediment. After the eighth day, there is a general erosion of sediment from the basin. An explanation for the erosion of sediment from the pit after it is filled with sediment will be discussed in a later section.

The time to fill the pit with sediment and the volume of sediment retained by the BWCDD Lake for each of the four hydrologic sequences is shown in Table 2. The pit fills in the first four days of the 1980 flood which is the shortest fill time for the four hydrologic sequences and that is reasonable since the 1980 flood was the largest in terms of peak discharge, therefore it carried the highest sediment load for those four days compared to any other flood. The BWCDD Lake fills for all hydrologic events being evaluated, and the sediment volume is about 2 million cubic yards.

Figure 16
BWCDD Lake - 1993 Event at cross section 196.04 (BWCDD Lake)

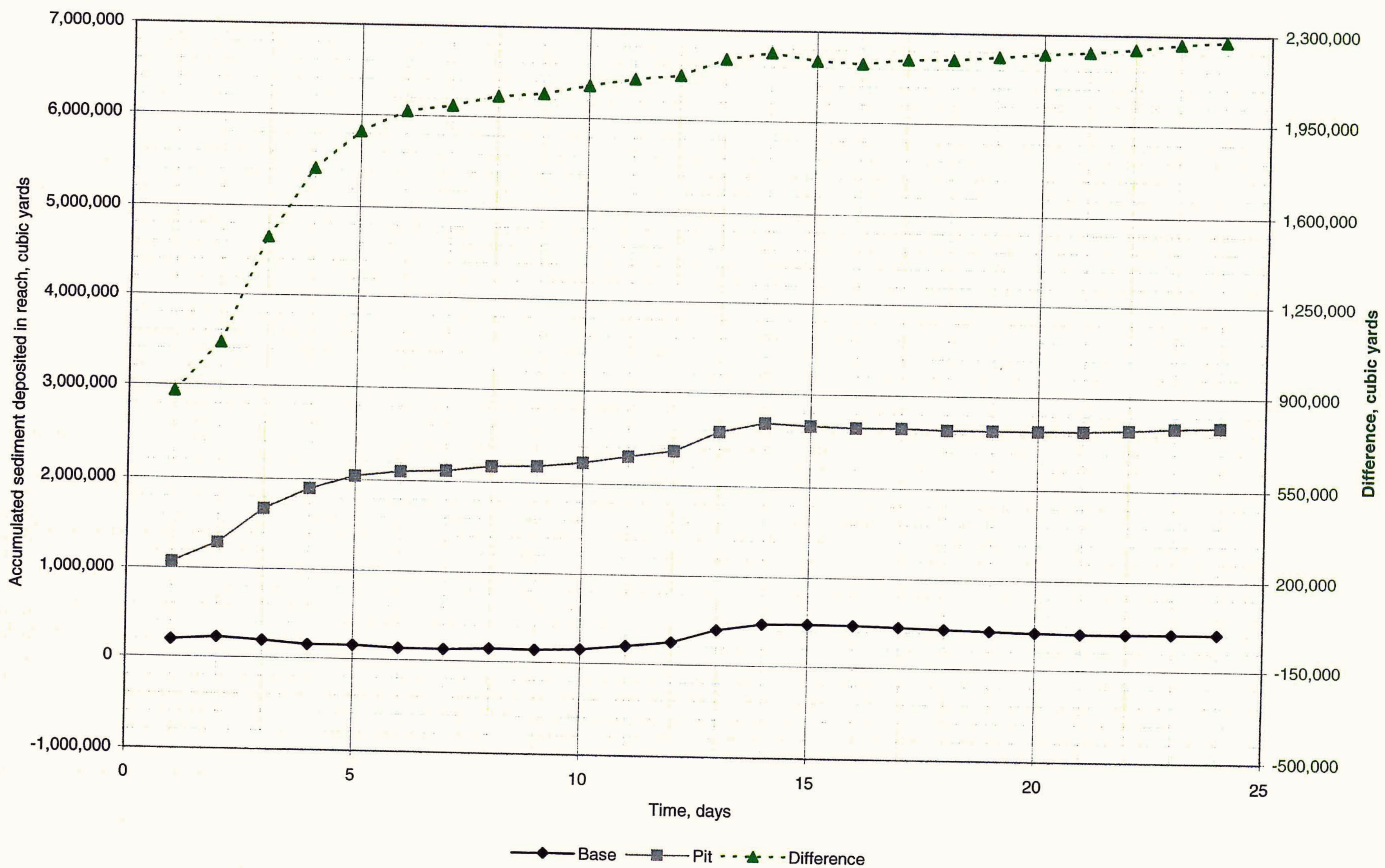


Figure 17
BWCDD Lake - 1980 Event at cross section 196.04 (BWCDD Lake)

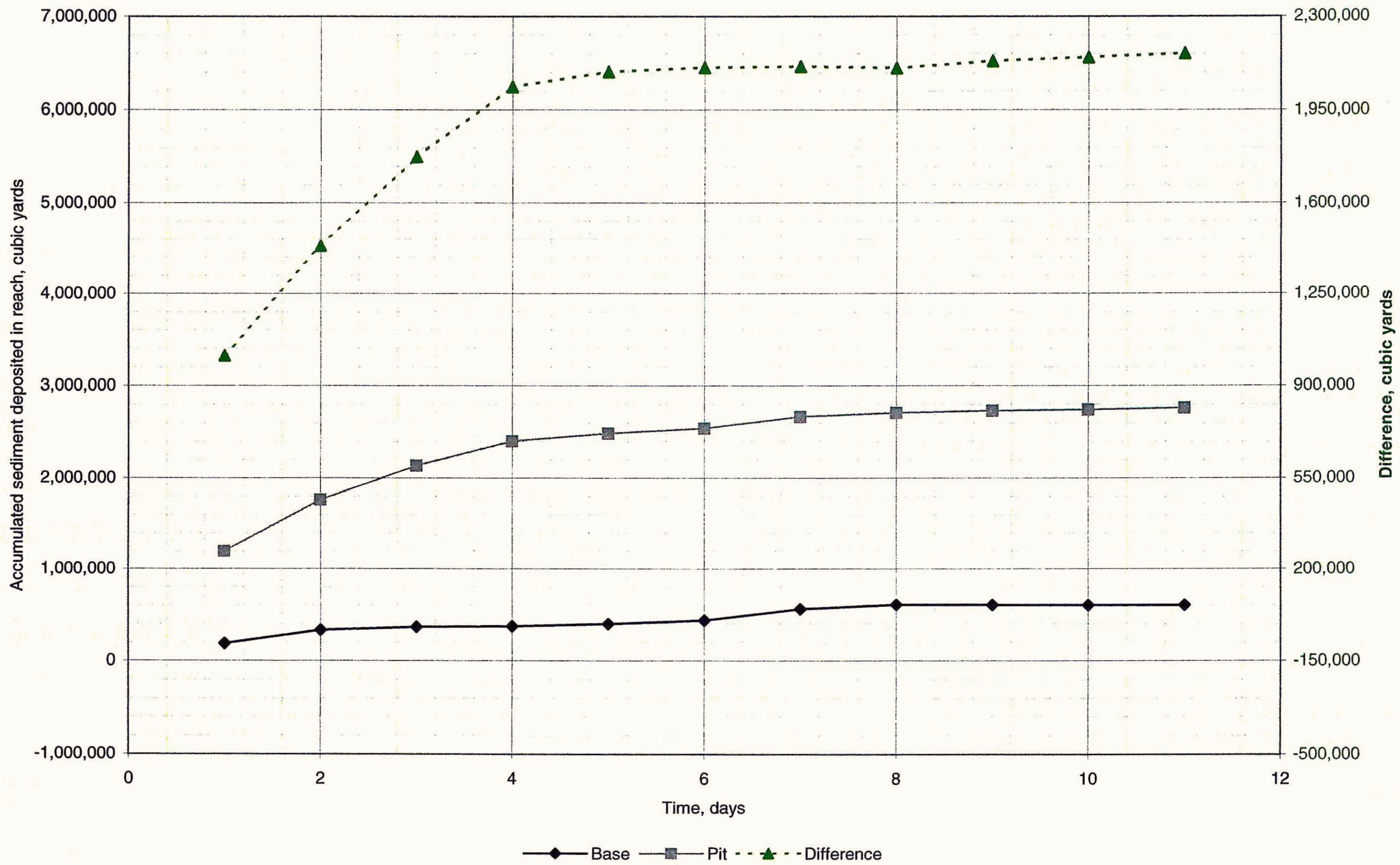


Figure 18
BWCDD Lake - 1978-1980 Events at cross section 196.04 (BWCDD Lake)

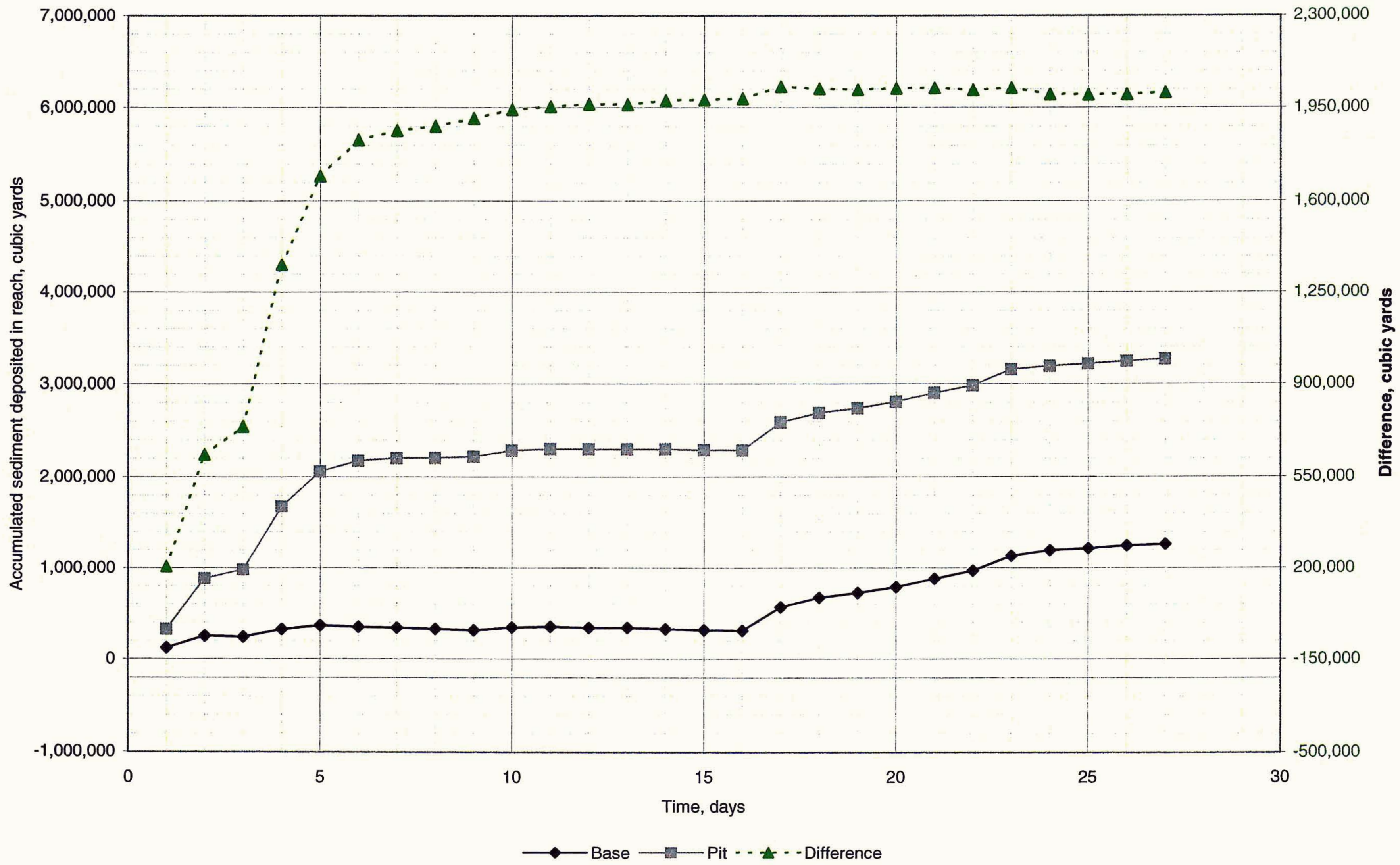


Figure 19
BWCDD Lake - Full Hydrology at cross section 196.04 (BWCDD Lake)

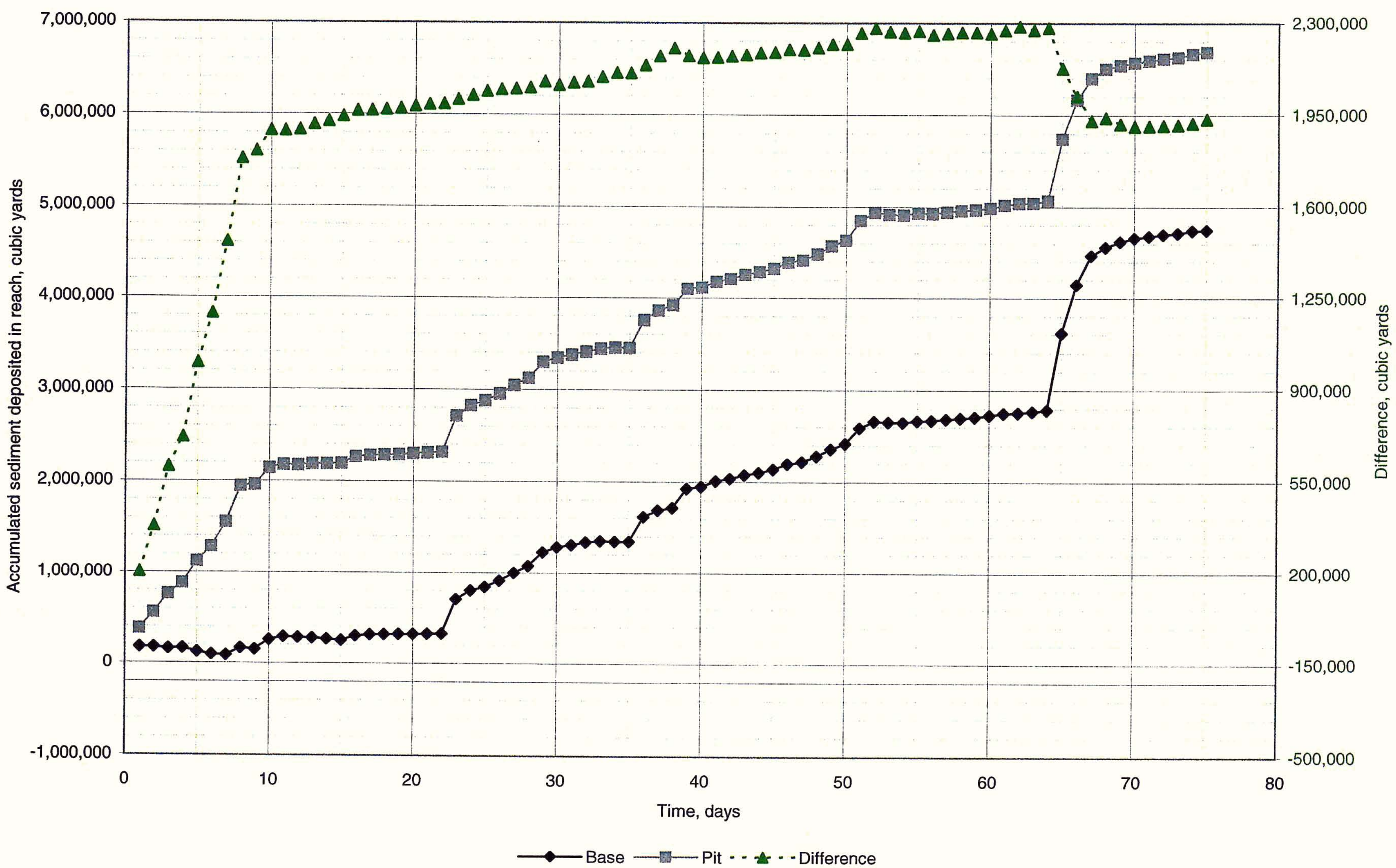


Table 2
Response of Pits to Hydrologic Sequences

		BWCDD Lake		Tuthill Pit		Buckeye Lake	
Flood	Flood Duration days	Fill Time days	Volume million cubic yards	Fill Time days	Volume million cubic yards	Fill Time days	Volume million cubic yards
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1993	24	5	2.0	9	2.0	DNF	15.9
1980	11	4	2.0	6	2.0	DNF	10.6
1978-1980	27	6	1.8	10	2.1	DNF	18.6
Full Record	75	8	1.7	10	2.0	67	33.8

Note: *DNF – Does Not Fill*

The model results for the BWCDD Lake indicate that there is local scour (lowered bed elevation) both immediately upstream and a short distance downstream of the pit. It is noted that the modeling of the BWCDD Lake includes an erosion resistant foundation for the diversion structure. The depth of sediment is set to zero at that section (RS 195.75). The downstream scour is reported at the next downstream section (RS 195.66). The lowered bed elevation upstream of the pit is due to local scour at that severe discontinuity in the bed profile as the river "enters" the pit. There is also lowered bed elevation downstream of the diversion dam foundation that is due to accelerated scour from the "clear water" discharges of water exiting the pit. Although the HEC-6T model correctly represents those areas of erosion, the magnitudes of erosion are unreliable for those local scour areas.

Figure 19 illustrates that for the full hydrologic sequence the pit fills in the first eight days. From day nine through 64 there is a gradual depletion (erosion) of the sediment volume in the pit. From day 65 through 70 there is a dramatic depletion of the sediment in the pit. This is explained as follows: The construction of the pit is by excavation of the coarser sand, gravel and even some cobble from the bed of the river. The pit is subsequently filled with sediment (the filling time being a function of the magnitude and duration of flood discharges; see Table 2). However, the sediment that fills the pit is much finer than the parent bed material. That is because the majority of the inflowing sediment load is a finer particle size than the river bed material. Therefore, the sand in the pit will be susceptible to erosion when flood discharges are larger than the discharges during pit filling. Accelerated erosion of the pit material will occur during exceptionally large floods. Therefore, the period from day nine through day 64 represents a gradual erosion of the finer sediment in the pit, but the accelerated erosion starting on day 65 is due to the 100-year flood that is modeled into the hydrologic sequence starting on that day.

Tuthill Pit – The filling of the Tuthill Pit by the four hydrologic sequences are illustrated in Figures 20 through 23. Figure 20 shows that the pit essentially fills in the first nine days of the 1993 flood and that it traps about 2 million cubic yards of sediment. (Note that this analysis does not include the upstream BWCDD Lake.) Figure 21 shows the pit filling with about 2 million cubic yards of sediment in the first six days of the 1980 flood. Figure 22, shows that it takes about the first 10 days of the 1978-1980 hydrologic sequence to trap about 2.1 million cubic yards of sediment. The last 11 days in Figure 22 represents the 1980 flood. Again, just as was illustrated for the BWCDD Lake (Figure 18), the large discharge of the 1980 flood results in some accelerated local erosion of the finer sediment deposited in the pit. Figure 23 shows that with relatively moderate floods, it takes 10 days to trap about 2 million cubic yards of sediment. After the tenth day, there is a general erosion of the finer sediment from the basin, and, as with the BWCDD Lake (Figure 19), there is accelerated erosion of the pit sediment during an exceptionally large flood.

The time to fill the pit with sediment and the volume of sediment retained by the Tuthill Pit for each of the four hydrologic events is shown in Table 2. The pit fills in the first six days of the 1980 flood which is the shortest fill time for the four events, and that is reasonable since the 1980 flood was the largest in terms of peak discharge, therefore it carried the highest sediment load for those six days compared to any other flood. The Tuthill Pit fills for all hydrologic events being evaluated, and the sediment volume is about 2 million cubic yards.

The model results for the Tuthill Pit indicate that there is local scour (lowered bed elevation) both immediately upstream and downstream of the pit. The lowered bed elevation upstream of the pit is due to local scour at that severe bed profile discontinuity as the river "enters" the pit. The lowered bed elevation downstream of the pit is due to accelerated scour from the "clear water" discharges of water exiting the pit. Although the HEC-6T model is correctly representing those areas of erosion, the magnitudes of erosion are unreliable for those local scour areas.

Figure 23 illustrates that the pit, once filled with sediment of smaller particle size than the parent bed material, is susceptible to accelerated erosion during exceptionally large floods. That condition was discussed for the BWCDD Lake.

Buckeye Lake – The Buckeye Lake represents the impact of several pits that are being considered for that area and the configuration of that pit assumes that those pits are eventually enlarged until they merge or hydraulically function as a single large pit. This pit evaluates the potential impacts of large scale sand and gravel extraction from the Gila River in the El Rio study area. That large pit has the capacity to cause a huge volume of sediment to be trapped in the pit. That pit only fills during the full hydrologic sequence of 75 days that is modeled. Figures 24 through 27 illustrate the filling of the pit (exclusive of either the BWCDD Lake or the Tuthill Pit) during the four hydrologic sequences that are modeled.

Figure 20
Tuthill Pit - 1993 Event at cross section 187.45 (Tuthill Pit)

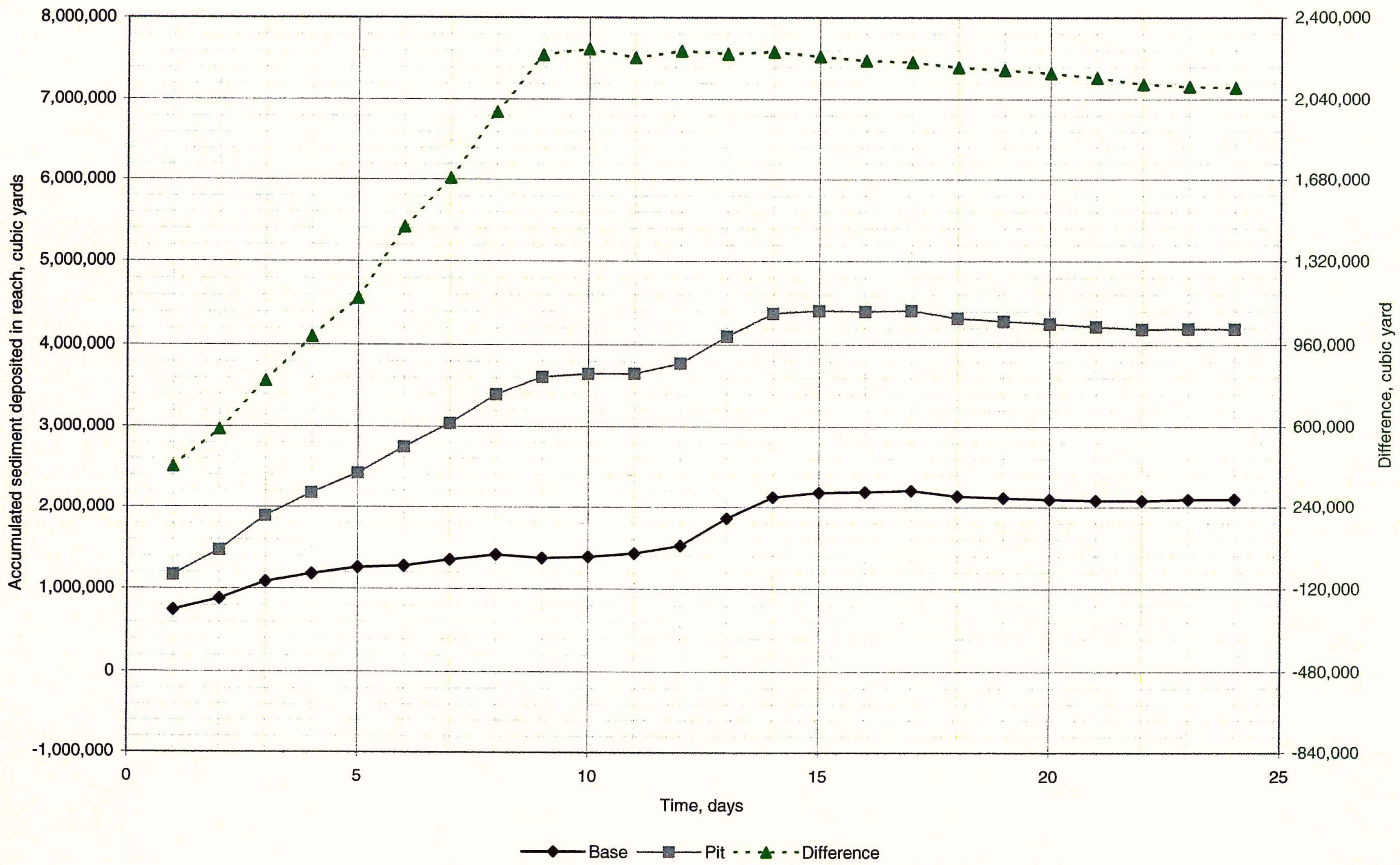


Figure 21
Tuthill Pit - 1980 Event at cross section 187.45 (Tuthill Pit)

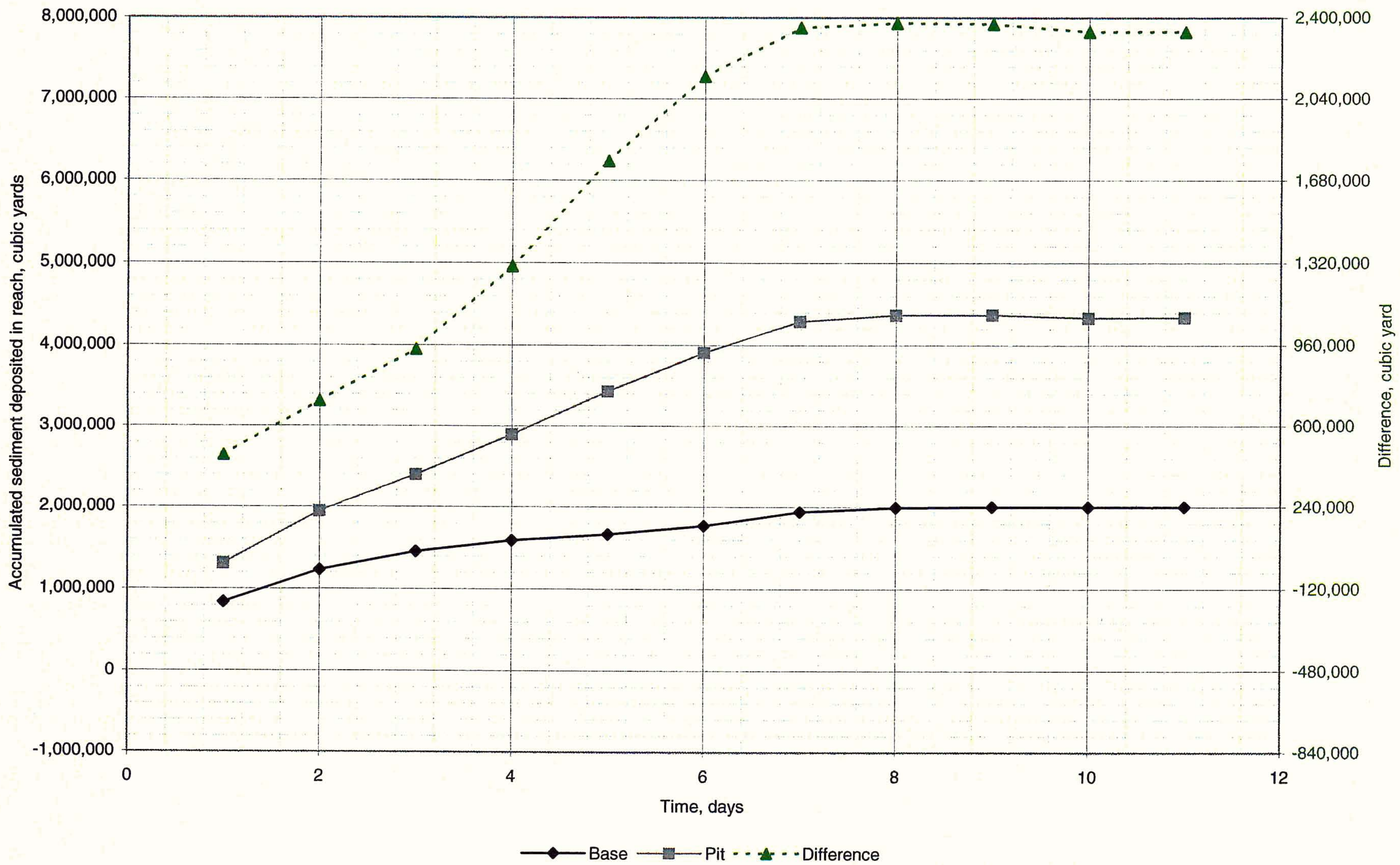


Figure 22
Tuthill Pit - 1978-1980 Events at cross section 187.45 (Tuthill Pit)

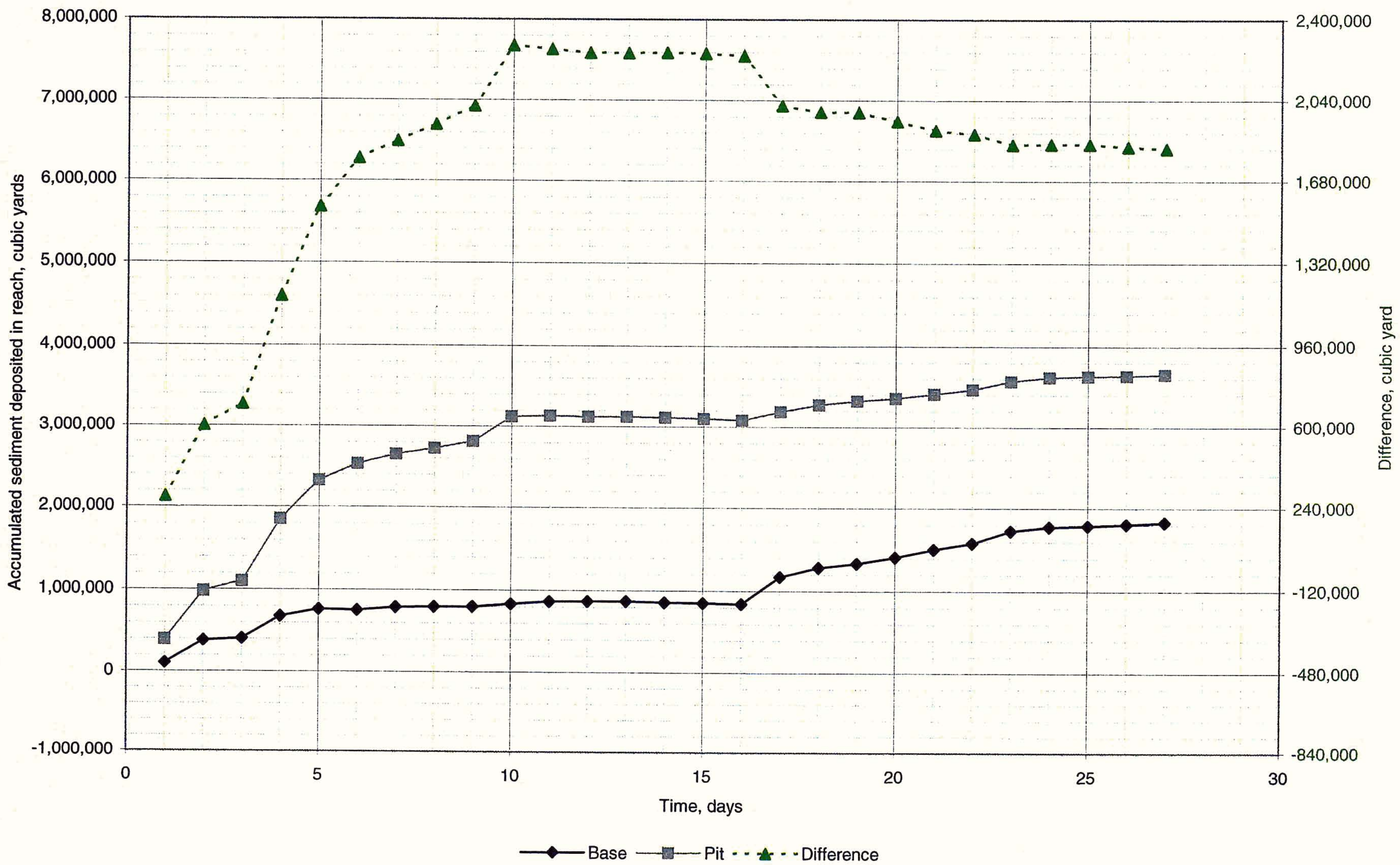


Figure 23
Tuthill Pit - Full Hydrology at cross section 187.45 (Tuthill Pit)

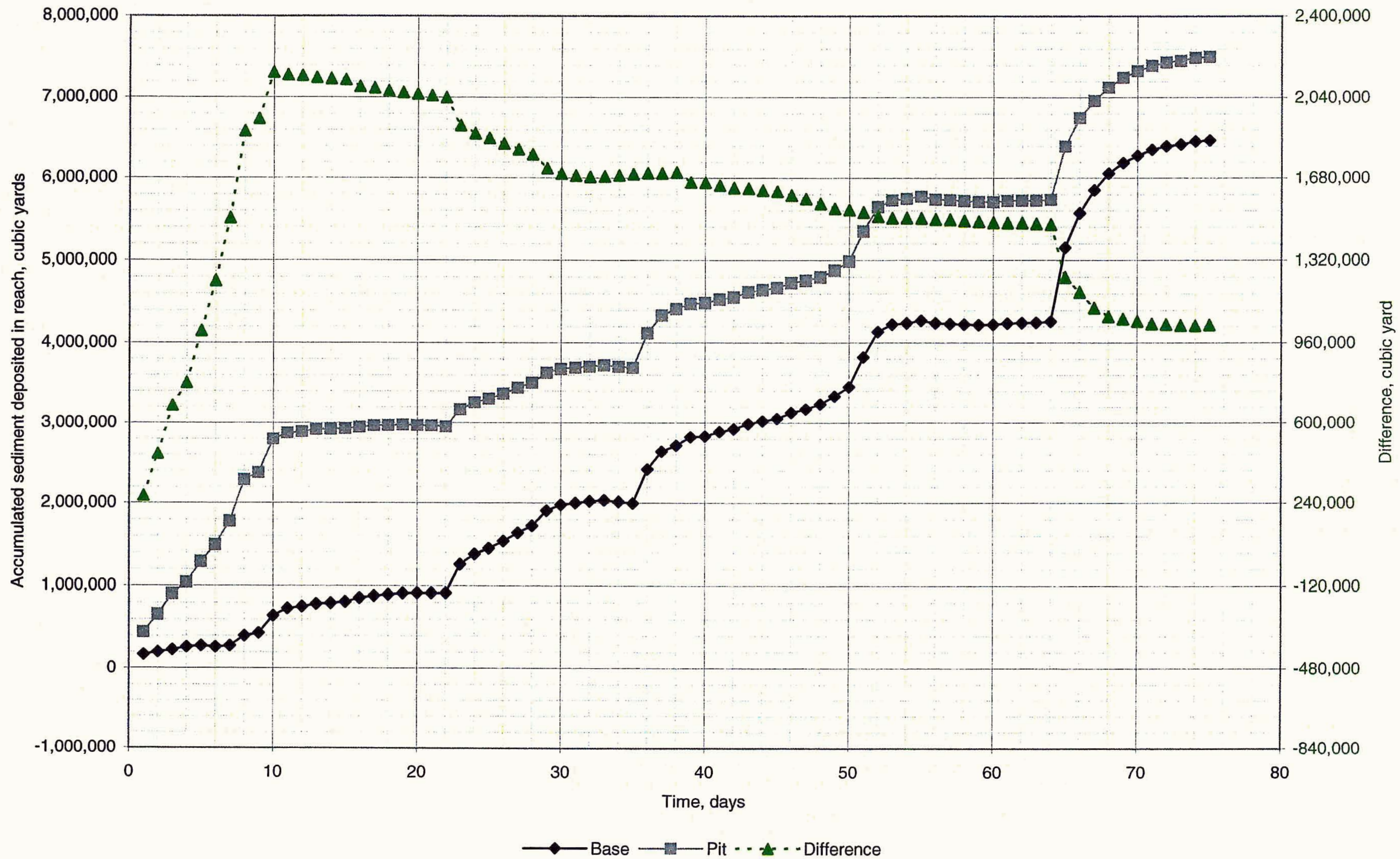


Figure 24
Buckeye Lake - 1993 Event at cross section 181.13 (Buckeye Lake)

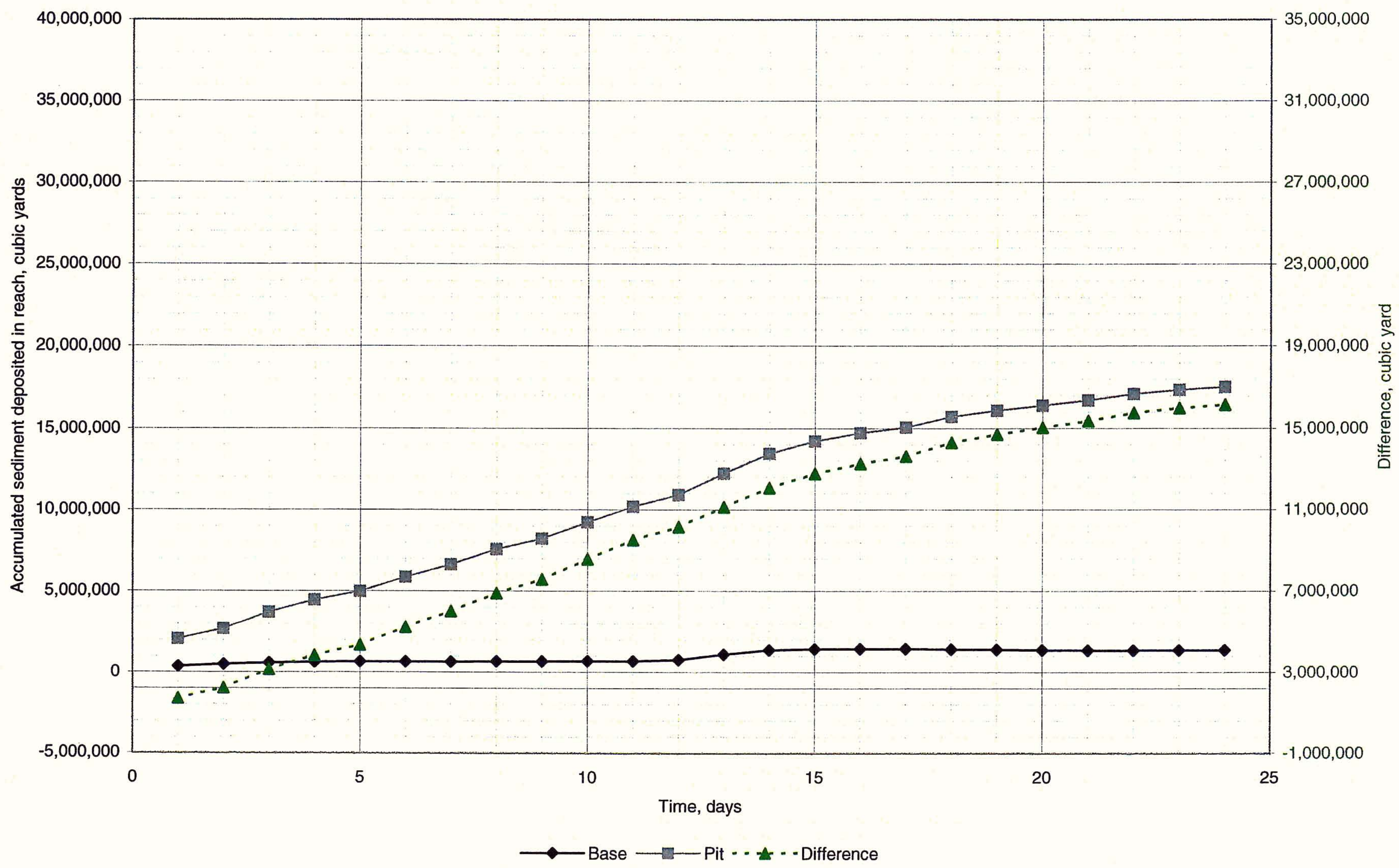


Figure 25
Buckeye Lake - 1980 Event at cross section 181.13 (Buckeye Lake)

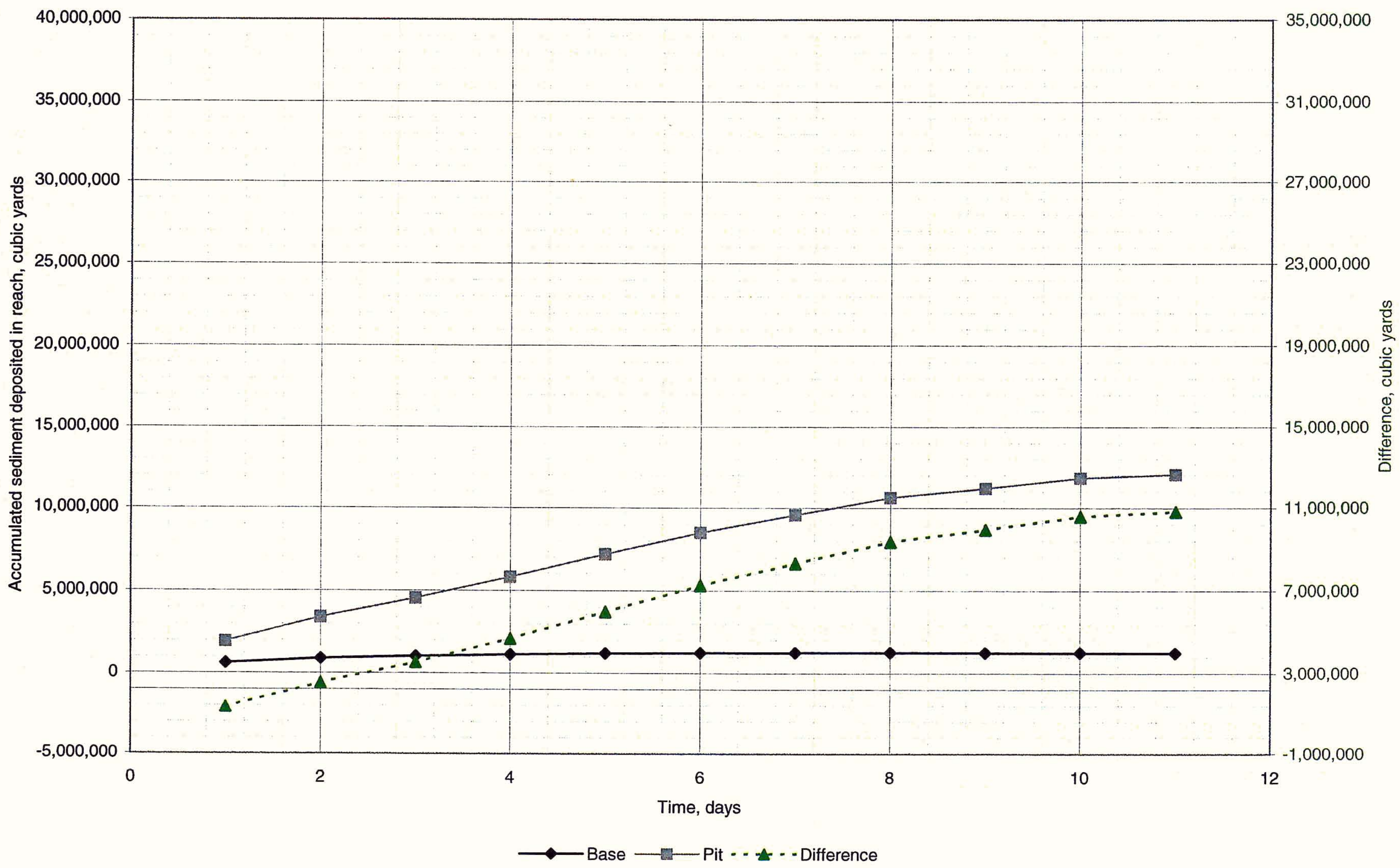


Figure 26
Buckeye Lake - 1978-1980 Events at cross section 181.13 (Buckeye Lake)

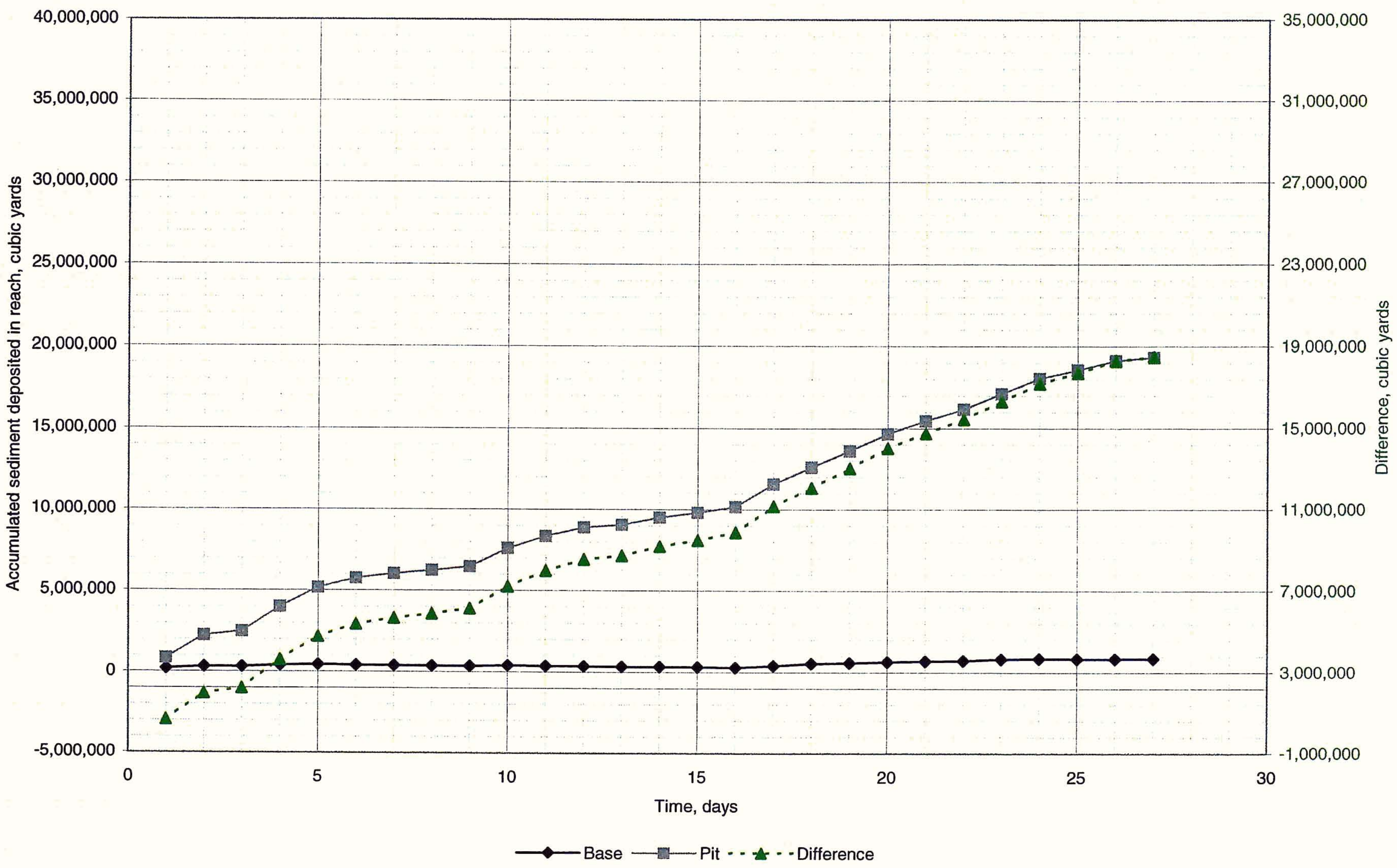
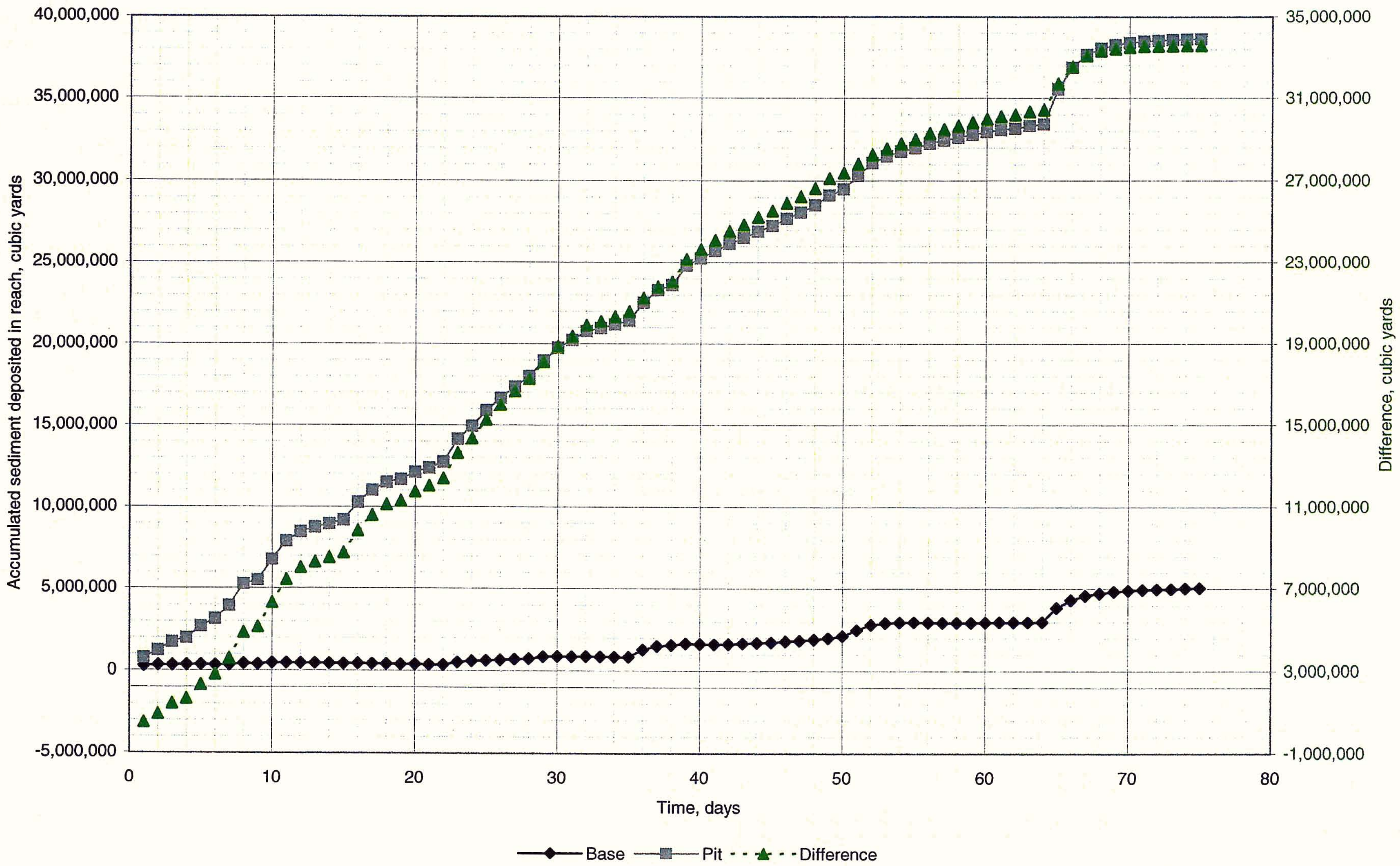


Figure 27
Buckeye Lake - Full Hydrology at cross section 181.13 (Buckeye Lake)



The volume of sediment trapped by that pit for each of the four hydrologic sequence of floods is shown in Table 2. The long duration 1993 flood results in about 15.9 million cubic yards of sediment (mostly sand) being trapped. The short duration but larger peak discharge flood of 1980 results in about 10.6 million cubic yards being trapped. The sequence of floods during 1978 through 1980 results in about 18.6 million cubic yards of sediment being trapped. The full hydrologic sequence of 75 flood days including a 100-year flood as was used for the existing condition analysis, results in about 33.8 million cubic yards being trapped, which fills the pit.

The model illustrates that there would be local scour immediately upstream of the pit. That local scour is headcutting as the pit wall is scoured and the pit wall advances in an upstream direction. The HEC-6T model cannot accurately model that headcutting but the model is correctly indicating the increased bed scour upstream due to that streambed discontinuity. There is also increased streambed scour that is induced by the trapping of sediment in the pit. The HEC-6T model indicates that the cross sectional average streambed degradation at the SR 85 Bridge due to the trapping of sediment in the pit is about 3 feet. The maximum streambed degradation at the SR 85 Bridge would be greater than 3 feet. A more detailed analysis is required to estimate the maximum scour that would be experienced at the SR 85 Bridge, or elsewhere, due to the sediment trapping in Buckeye Lake.

<u>Combination of Three Pits</u> – The impact of the three pits at their full ultimate size was modeled. The distance between the BWCDD Lake and the Tuthill Pit is about 8.2 miles. The distance between the Tuthill Pit and the Buckeye Lake is about 4.5 miles. The distance from the downstream end of Buckeye Lake to the SR 85 Bridge is about 1.1 miles. The BWCDD Lake and the Tuthill Pit fill with sediment for all four hydrologic sequences. The Buckeye Lake traps about the same volume of sediment with or without the presence of the two upstream pits.

There is a small increase in streambed degradation between the Tuthill Pit and the Buckeye Lake due to the combined sediment trapping in both the BWCDD Lake and the Tuthill Pit. There is a similar small increase in streambed degradation downstream of the Buckeye Lake.

IMPACTS OF THE PITS AT THE SR 85 BRIDGE

Table 3 lists the changes in sediment load at the SR 85 Bridge for each of the pits individually and for the combined effect of all three pits. The values shown in Table 3 are the differences in sediment load passing the SR 85 Bridge for the with pit(s) model minus the sediment load for the existing condition model. Notice that with the BWCDD Lake (column 2 of Table 3) there is an increase in sediment load passing the SR 85 Bridge for the three hydrologic sequences other than the 1980 flood. During the initial filling of the pit, there is a net reduction in sediment load past the SR 85 Bridge. This can be observed in all the graphs in Attachment 5. If the duration of the flood or sequence of floods increases, then there is a net increase in the sediment load past the SR 85 Bridge as compared to the model without pit condition. It is observed that both the BWCDD Lake and the Tuthill Pit can increase the long-term sediment yield past the SR 85 Bridge.

Table 3
Change in Sediment Load at the SR85 Bridge

	Change in sediment load, in tons			
Flood	**w/BWCDD Lake**	**w/Tuthill Pit**	**w/Buckeye Lake**	**w/all three**
(1)	(2)	(3)	(4)	(5)
1993	215,000	102,000	- 8,027,000	- 8,014,000
1980	- 345,000	- 493,000	- 5,444,000	- 5,421,000
1978-1980	547,000	456,000	- 11,632,000	- 11,734,000
Full Record	1,532,000	1,194,000	- 28,274,000	- 28,865,000

Note: *A negative sign means that there was less sediment load passing SR85 Bridge for the with pit condition when compared tothe existing (without pit) condition.*

Similar behavior occurs with the Tuthill Pit (column 3 of Table 3). For long duration floods or a sequence of floods that cause those pits to fill early in the hydrologic sequence, there is an increase in sediment load downstream of each pit as those pits experience accelerated erosion of fine sediments as they are scoured during the passage of subsequent floods. That phenomenon was previously explained.

The large Buckeye Lake is a short distance (only about 1.1 miles) upstream of the SR 85 Bridge. That pit does not fill with sediment except for a very long sequence of floods. Therefore, that pit results in reduced sediment loads passing the SR 85 Bridge. The presence of the two smaller pits upstream of the large pit, have little impact on the sediment load passing the SR 85 Bridge.

EFFECT OF UNMODELED LOW FLOWS

It is assumed that very low discharges (less than the 5-year flood of 17,000 cfs at the confluence with Waterman Wash) will be diverted around the pits and therefore, there will be no sediment trapping in the pits for those frequent discharges. Therefore, the pits will have no impact on the river and no impact on the sediment load passing the SR 85 Bridge for low flows. At some threshold of discharge, which cannot be determined without detailed information on the configuration of the pits, the flow will enter the pits and they will function to trap sediment. Typical floods of less than 10- to 25-year frequency are relatively small floods within the context of this analysis and those floods are typically of short duration, therefore those floods, even if captured by the pits may have relatively small impact on the river. But, such an analysis was not performed. The only floods that are considered in this analysis are relatively large floods and the long-term sequence of floods exceeding 35,000 cfs. Analyses of any pit or combination of pits that are planned to be excavated and operated will need to assess the impacts at and near the pit, and the impacts to the entire river system.

HEC-6T PIT MODEL VERIFICATION

The performance of the HEC-6T models to reasonably model the sedimentation process of the pits/lakes was checked by the use of a procedure to estimate sediment deposition in a settling basin by Pemberton and Lara (1971). An HEC-6T model of the Tuthill Pit was run for a steady discharge of 100,000 cfs. The deposition volume in the pit at the end of one day as reported by the HEC-6T model is 0.43 million cubic yards. Using the same discharge and sediment loads by size fraction from that HEC-6T model with the procedure by Pemberton and Lara results in a deposition volume of 0.37 million cubic yards. Considering the difference in methodologies and assumptions (for example, Pemberton and Lara method applies the Einstein equation while the El Rio HEC-6T model uses the Yang sediment transport relation) these two methods yield surprisingly close agreement. Calculations for the verification are provided in Attachment 6.

CONCLUSIONS OF THE PIT/LAKE ANALYSIS

1. During large floods, such as those of 1980 and 1993, both the BWCDD Lake and the Tuthill Pit will fill with sediment in less than a week after the onset of the flood. The volume of the modeled pit is about 2 million cubic yards each.

2. The pits/lakes will be susceptible to upstream headcutting during floods. A thorough analysis is not undertaken and would require details of the pit construction.

3. The pits/lakes will contribute to downstream scour during floods due to "clear water" releases from the pits/lakes. A thorough analysis is not undertaken and would require details of the pit construction.

4. A large pit at Buckeye Lake will require a long sequence of floods to fill with sediment.

5. A large pit, such as the modeled Buckeye Lake, would result in significantly reduced sediment loads past the SR 85 Bridge during large floods. The Gila River downstream of such a large pit would experience long-term degradation of 3 feet or more.

6. Pits that fill with sediment can result in increased long-term downstream sediment loading during exceptionally large floods.

7. Pits/lakes of the size modeled for the BWCDD Lake and the Tuthill Pit have little impact on each other when there is sufficient distance between them (in the condition modeled, the distance between the two pits is 8.2 miles).

SCOUR ANALYSES

GENERAL

Scour, as presented herein, is a lowering of the channel bed due to erosion. Structures placed in a watercourse must be designed with consideration of existing scour potential as well as potential scour due to the imposition of the structure. Scour is estimated for the study reach of the El Rio WMP in order to determine appropriate toe-down depths and associated planning level cost estimates for the proposed levee/bank protection and BWCDD diversion structure.

METHODOLOGY & TOTAL SCOUR COMPONENTS

The total scour that can be expected to occur is the sum of individual scour components. Scour components typically considered are:

- Long-term degradation,
- General scour,
- Local scour,
- Bend scour (when not considered as part of local scour),
- Bedform movement, and
- Low-flow incisement.

Methodologies and procedures for estimating each component of scour are provided in the Maricopa County Drainage Design Manual, Volume II, Hydraulics (Flood Control District of Maricopa County, 2003) and are discussed in the following sections.

Hydraulic parameters used in the scour calculations are taken from the recommended alternative HEC-RAS model for the 100-year flood. That model contains three plans that represent a phased implementation of the proposed master plan elements. Because the sequence of implementation, both in terms of the individual elements and their lateral extent, is uncertain, the scour estimates are based on the hydraulic parameters of any of the three plans that yield the maximum scour depth.

Bed material size gradation used in the local scour calculations are shown in Figure 34 of the the Existing Conditions Sedimentation Analysis. Discussion of the bed material characterization and sampling approach is provided in the Data Collection section of that report.

Long-term Degradation - Long-term degradation is a general, progressive lowering of the channel bed over the length of a watercourse. It is generally considered to be a result of a "system-wide" change in the morphology of the watercourse or watershed. Examples of events that could result in long-term degradation are the construction of a dam or the urbanization of the watershed. Evaluation of the magnitude of long-term degradation can be accomplished by inspection of historical data or the application of equilibrium slope equations.

Equations for estimating equilibrium slope are recommended by Pemberton and Lara (1984). However, application of those equations to long-term degradation requires the identification of a downstream control point where the bed elevation is not expected to change. The closest downstream control point to the El Rio WMP study limit is Gillespie Dam, a distance of approximately 14 miles. The sediment deposition upstream of Gillespie Dam since 1921 and the breach of that dam in 1993 make the estimation of long-term degradation in the study area unreliable.

Historical data available for evaluation of long-term degradation is documented in the El Rio WMP Lateral Migration Analysis Report. Data considered in the evaluation consists of topographic mapping for the period of 1948 to 1998. The results of that evaluation indicate the study reach experiences episodes of aggradation and degradation. Based on the time period analyzed, it is likely that the aggradation-degradation cycles are tied to the now emphemeral nature of the river due to the control, particularly of the larger floods, by upstream reservoirs. Although pre-dam topographic mapping is not available, it is likely, given the length of time that the major dams have been closed, that the river has already adjusted for long-term degradation. Inspection of historic streambed gradients does not indicate a trend for long-term degradation.

General Scour - General scour occurs during a flood and/or during a series of floods that are expected to occur during the design life of a structure. General scour occurs across the entire width of the channel, but not necessarily uniformly. General scour for the study reach is estimated using the existing condition sedimentation model results, in particular, the maximum bed elevation change. For master planning purposes, general scour is assumed to occur uniformly across the width of the channel.

A special case of general scour is contraction scour. Contraction scour occurs due to an increase in velocities and shear stress due to natural, abrupt changes in channel width or encroachment into the watercourse, such as at bridge crossings. Establishment of the proposed levee/bank protection alignment was set with smooth transitions to limit adverse scour conditions. However, where the proposed levee ties into the existing bridge crossings, the hydraulic conditions may be sufficient to result in contraction scour. Procedures for estimating contraction scour are provided in *Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges* (Federal Highway WA, 2001), herein referred to as HEC-18. The procedure requires hydraulic parameters at the approach section and the contracted section and the selection of the live bed or clear water equations. Live bed contraction scour occurs when there is sediment transport from the approach section into the contracted section. The live bed condition is assumed for all bridge crossings. Results of the calculation will vary depending on the selection of the approach section.

Local Scour - Local scour is caused by flow irregularities due to bends or restrictions along the bank or by structures in the watercourse. Establishment of the proposed levee/bank protection alignment was set such that conditions that could cause local scour are minimized. However, local scour will occur where tributary flow enters the Gila River through the levee, at the bridge abutments and at the proposed BWCDD diversion structure. Since the nature and magnitude of the majority of any tributary inflow is unknown at this time, that specific local scour condition must be addressed at the design level. Local scour will also result at the bridge abutments and at the proposed BWCDD diversion structure.

Bridge abutment scour is a special case of local scour. Two equations for estimating abutment scour are presented in HEC-18. Both equations require hydraulic data at the section immediately upstream of the bridge. Both equations yield conservative results. For this analysis, the HIRE equation is used.

The proposed BWCDD diversion structure is primarily a solid foundation with a fuse plug embankment. Part of the structure will be a concrete (or similar material) spillway approximately 6 feet in height. The purpose of the spillway is to allow more frequent flows to pass the diversion structure without failure of the fuse plug. There are two approaches for estimating local scour downstream of a hydraulic structure presented in the draft Hydraulics Manual. The first approach is a set of equations for free fall conditions presented by Pemberton and Lara (1984). The second approach is for a submerged structure that was derived by Simons, Li & Associates (1986) through a physical model study.

Bend Scour - Bend scour for sand-bed watercourses can be estimated using an empirical equation developed by Zeller (1981). That equation requires the hydraulic parameters immediately upstream of the bend and an estimation of the angle of curvature. In the study reach, the only location that is subject to bend scour is in the vicinity of the Tuthill Bridge. In this area, the south bank of the river is on the outside of the bend where the scour would occur. Results of the El Rio WMP Lateral Migration Analysis indicate that the south bank is relatively stable. Therefore, levee/bank protection is only provided on the north bank, which is on the inside of the bend and not subject to bend scour.

Bedform Movement - Bedforms are a result of the interaction of hydraulic forces (boundary shear stress) and the bed sediment. Typically, bedforms consist of alternating "mounds" and "troughs" that move longitudinally along the watercourse. The type and magnitude of the bedform is a function of the flow regime. During upper regime flow, Froude Number (F_r) greater than 0.7, conditions are often sufficient to results in antidune formations. Antidune height will typically be greater than dune height which forms in lower regime flow, F_r less than 0.7. During upper regime flow, the water surface is in phase with the bed surface (standing wave) except when an antidune breaks (breaking wave). Standing waves are illustrated in an aerial photograph of the Gila River upstream of Tuthill Bridge taken on 9 January 1993, as shown in Figure 28.

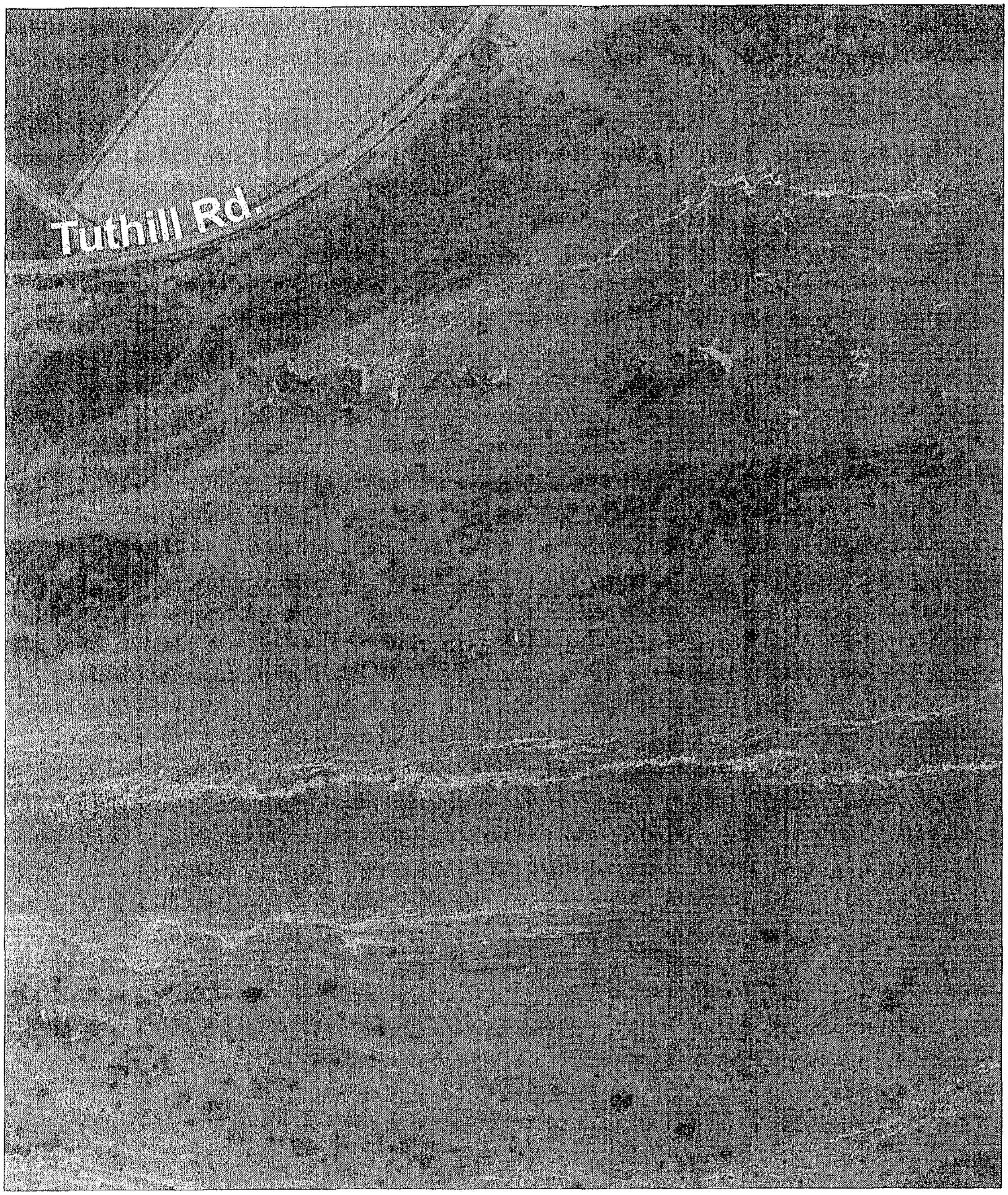


Figure 28 Standing waves in the Gila River

Bedform movement as a component of total scour can be estimated using the empirical equation presented in the draft Hydraulics Manual. Hydraulic parameters required are the hydraulic depth and Froude Number. Inspection of the hydraulic model results for the average F_r for the channel would suggest that upper regime flow does not occur, as F_r is generally less than 0.5. However, based on review of aerial photography from several flood events, the presence of standing waves occurs throughout the study reach but is typically confined to a narrow corridor within the main channel. Therefore, in order to estimate antidune height, a F_r of 0.8 is assumed.

Low-flow Incisement - Numerous field visits to the study reach including visits after the winter 2005 flooding were conducted. At no location was evidence of low-flow incisement observed. Therefore, low-flow incisement as a scour component is not considered for the study reach.

TOTAL SCOUR ESTIMATION

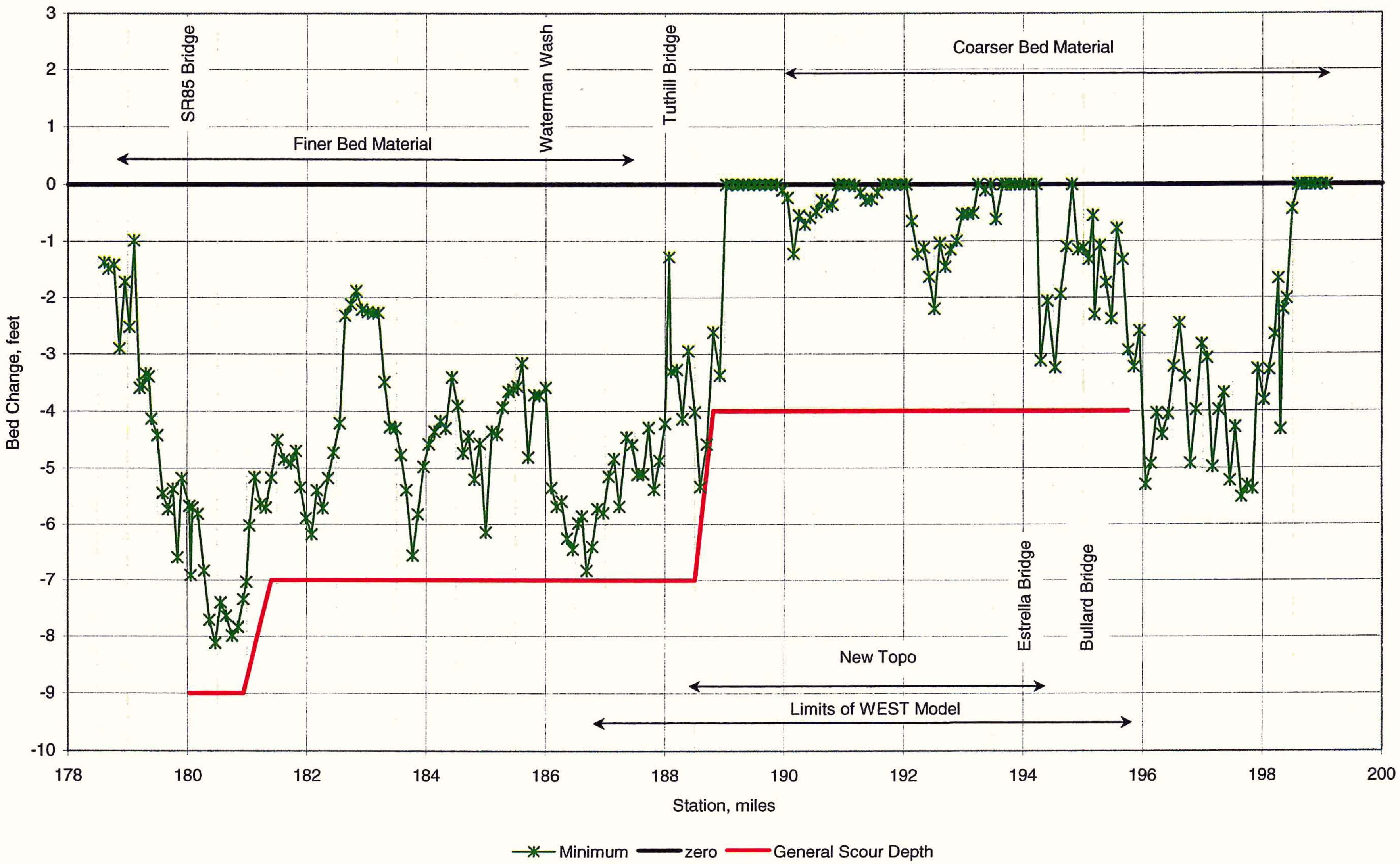
The total scour depths that can reasonably be expected to occur during the design life of the proposed levee/bank protection and BWCDD diversion structure is the sum of general, local, bend and bedform movement scour components. Calculations for each scour component are prepared at each cross section and presented in the following sections.

General Scour - Estimation of general scour for the study reach is based on interpretation of the existing condition sediment model results, in particular the maximum bed elevation change depicted in Figure 29. Inspection of that figure suggests that the magnitude of general scour that can reasonably be expected to occur ranges from 4 to 9 feet. The distribution of general scour along the study reach is assumed to occur uniformly across three sub-reaches. The first sub-reach extends from the SR 85 Bridge to river station 180.90 and the scour depth is set to 9 feet. The second sub-reach extends from river station 181.41 to 188.50 and the scour depth is set to 7 feet. The third reach extends from 188.81 to 195.75 and the scour depth is set to 4 feet. General scour in the transition zones of the sub-reaches is a linear interpolation of the depths for the bounding sub-reaches. The general scour estimated at each cross section is listed in column 8 of Table 4.

Contraction scour is a special case of general scour and for the proposed conditions is assumed to occur only at the bridge crossings. Figures illustrating the contraction conditions at each bridge along with the hydraulic data and contraction scour calculations are provided in Attachment 7 and are summarized in column 9 of Table 4. The contraction scour estimated at the SR 85, Tuthill Road, Estrella Parkway and Bullard Avenue Bridges is 3.7, 8.1, 3.9 and 1.4 feet, respectively. The contraction scour estimated is the maximum depth that is assumed to occur at the bridge. However, scour due to the contraction will likely occur along the length between the approach section and the bridge. The rate of change in scour depth through this reach is assumed to be linear.

Local Scour - The local scour conditions that are anticipated to impact the proposed levee/bank protection are limited to the bridge abutments. Abutment scour calculations are provided in Attachment 7 and are summarized in column 10 of Table 4. Abutment scour calculations are performed for each bridge, although, based on the proposed alignment true abutment scour conditions that could impact the integrity of the levee/bank protection would not exist. Furthermore, abutment scour can be arrested with appropriate countermeasures. However, for planning purposes, given that the levee alignment could change, abutment scour is included in the estimation of total scour.

Figure 29
General Scour Depth



The abutment scour estimated at the SR 85, Tuthill Road, Estrella Parkway and Bullard Avenue Bridges is 41.0, 29.0, 35.0 and 40.8 feet, respectively. The zone of influence of abutment scour upstream of the bridge in regard to the proposed levee/bank protection is treated in the same fashion as the contraction scour.

For the proposed BWCDD diversion structure, the depth of the scour hole that can reasonably be expected to occur is estimated using a set of equations presented by Pemberton and Lara (1984) and the equations derived by Simons, Li and Associates (1986). Estimates of the scour depth using each equation are provided in Attachment 7. The magnitude of scour depth estimated from each equation ranges from 5.5 feet to 55.6 feet. The depth of scour estimated using the Simons, Li and Associates equation is 11.8 feet. This equation is considered appropriate for submerged conditions. The equations presented by Pemberton and Lara are for free fall conditions. Because the majority of the proposed diversion structure would be constructed as a fuse plug that would be expected to wash out at a discharge much less than the 100-year discharge, free fall conditions are not anticipated. Therefore, the scour depth estimated using the Simons, Li and Associates equation of 11.8 feet is adopted.

Bedform Movement - The depth of scour associated with bedforms is taken as one-half the bedform amplitude. The amplitude for anti-dune bedform is calculated at each cross section using the hydraulic depth in the main channel and an assumption of 0.8 for F_r. The corresponding scour depth is listed in column 11 of Table 4. The maximum scour depth due to bedform movement is 5.1 feet occurring at Tuthill Road Bridge (both immediately upstream and downstream). The minimum scour depth is 1.9 feet. The average scour depth is 3.0 feet.

Total Scour - Total scour is estimated as the sum of the individual scour components with the addition of a factor of safety multiplier of 1.3. The total scour that can reasonably be expected is estimated at each cross section and is listed in column 12 of Table 4. The average total scour depth is approximately 17 feet. The maximum total scour depths occur at the bridges. The magnitude of scour at the bridges is driven by inclusion of the abutment scour (local scour). Without consideration of abutment scour the average total scour depth is approximately 12 feet.

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
178.610	157,311	17.6	9.8	4.8	3,315	0.00069					
178.680	147,889	17.7	10.0	4.3	3,401	0.00062					
178.770	174,161	15.5	10.7	4.9	3,305	0.00059					
178.860	180,837	15.3	10.0	5.3	3,439	0.00105					
178.950	197,202	16.7	9.7	5.4	3,774	0.00101					
179.030	189,246	14.0	9.3	5.1	4,029	0.00098					
179.110	197,677	12.3	9.2	5.4	4,037	0.00104					
179.200	173,409	13.9	8.8	5.0	3,986	0.00092					
179.250	182,424	11.7	7.8	6.0	3,881	0.00155					
179.300	157,003	11.6	8.3	5.4	3,541	0.00104					
179.350	136,174	11.5	8.1	5.2	3,264	0.00098					
179.400	153,801	13.7	8.2	6.1	3,059	0.00135					
179.500	159,555	11.0	7.2	7.3	3,046	0.00174					
179.590	193,475	12.2	6.9	8.0	3,536	0.00210					
179.680	180,166	13.2	6.8	8.4	3,178	0.00243					
179.760	181,995	13.0	6.9	7.9	3,333	0.00230					
179.840	183,904	12.3	6.2	9.1	3,286	0.00341					
179.910	184,460	12.1	8.1	6.9	3,315	0.00187					
180.010	210,000	15.1	7.2	8.3	3,607	0.00394					
180.025	**SR 85 Bridge**										
180.040	210,000	16.0	8.1	7.3	3,610	0.00347	9.0	3.7	41.0	2.3	72.8
180.060	204,300	13.2	10.1	6.3	3,265	0.00115	9.0	3.6	39.7	2.8	71.6
180.090	198,020	13.3	10.7	5.8	3,208	0.00105	9.0	3.4	37.6	3.0	69.0
180.180	166,466	18.5	8.7	7.5	2,544	0.00177	9.0	2.9	31.6	2.5	59.7
180.280	136,308	16.2	8.1	8.1	2,075	0.00174	9.0	2.2	24.9	2.3	49.9
180.370	172,066	15.3	9.2	12.7	1,468	0.00367	9.0	1.7	18.8	2.6	41.8
180.470	144,712	15.1	10.8	9.4	1,424	0.00190	9.0	1.1	12.1	3.0	32.8
180.560	131,337	14.8	10.7	7.3	1,689	0.00178	9.0	0.5	6.0	3.0	24.2
180.650	128,459	15.1	11.2	7.0	1,637	0.00100	9.0	0.0	0.0	3.2	15.8
180.750	109,299	14.5	11.5	6.0	1,591	0.00080	9.0	0.0	0.0	3.2	15.9

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
180.850	140,616	15.0	11.1	6.7	1,881	0.00086	9.0	0.0	0.0	3.1	15.8
180.940	153,513	15.6	11.3	5.7	2,387	0.00112	9.0	0.0	0.0	3.2	15.8
180.990	162,859	16.0	11.7	5.7	2,441	0.00054	9.0	0.0	0.0	3.3	16.0
181.040	187,334	15.1	11.8	5.4	2,967	0.00049	8.6	0.0	0.0	3.3	15.5
181.130	183,876	15.2	12.2	4.7	3,252	0.00040	8.2	0.0	0.0	3.4	15.1
181.230	190,973	16.0	11.7	5.0	3,328	0.00048	7.8	0.0	0.0	3.3	14.4
181.320	190,355	15.1	10.3	5.7	3,413	0.00078	7.4	0.0	0.0	2.9	13.4
181.410	200,609	15.3	10.2	6.7	3,398	0.00107	7.0	0.0	0.0	2.9	12.8
181.510	197,224	16.1	10.5	5.8	3,381	0.00075	7.0	0.0	0.0	2.9	12.9
181.620	196,391	14.5	10.3	6.1	3,257	0.00085	7.0	0.0	0.0	2.9	12.9
181.740	192,752	13.2	9.7	6.8	3,108	0.00160	7.0	0.0	0.0	2.7	12.7
181.820	198,119	13.9	10.3	7.5	2,717	0.00127	7.0	0.0	0.0	2.9	12.9
181.900	194,017	14.8	10.1	7.4	2,687	0.00131	7.0	0.0	0.0	2.9	12.8
181.990	197,479	15.9	10.9	6.6	2,748	0.00276	7.0	0.0	0.0	3.1	13.1
182.080	184,844	14.7	11.2	5.3	3,114	0.00107	7.0	0.0	0.0	3.2	13.2
182.170	204,581	16.9	10.9	5.9	3,300	0.00097	7.0	0.0	0.0	3.1	13.1
182.270	194,580	16.4	10.2	5.4	3,508	0.00105	7.0	0.0	0.0	2.9	12.8
182.360	204,029	17.3	11.0	5.4	3,443	0.00082	7.0	0.0	0.0	3.1	13.1
182.450	198,392	20.0	12.3	4.6	3,597	0.00065	7.0	0.0	0.0	3.5	13.6
182.550	209,336	19.2	12.4	4.3	4,298	0.00056	7.0	0.0	0.0	3.5	13.6
182.640	207,202	17.6	12.0	4.5	4,645	0.00072	7.0	0.0	0.0	3.4	13.5
182.740	195,812	17.7	11.8	4.2	4,566	0.00052	7.0	0.0	0.0	3.3	13.4
182.830	194,387	18.2	11.3	4.1	4,642	0.00042	7.0	0.0	0.0	3.2	13.2
182.920	191,635	18.4	11.4	4.0	4,533	0.00056	7.0	0.0	0.0	3.2	13.3
183.020	191,084	19.4	11.3	4.3	4,282	0.00069	7.0	0.0	0.0	3.2	13.2
183.110	189,198	19.1	11.1	4.6	4,193	0.00068	7.0	0.0	0.0	3.1	13.2
183.200	194,379	17.8	11.7	4.5	4,034	0.00078	7.0	0.0	0.0	3.3	13.4
183.300	203,038	17.8	11.3	5.0	3,895	0.00100	7.0	0.0	0.0	3.2	13.2
183.390	202,590	16.9	10.8	5.3	3,843	0.00092	7.0	0.0	0.0	3.0	13.1
183.490	204,893	17.3	9.9	5.6	4,003	0.00137	7.0	0.0	0.0	2.8	12.7

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
183.580	210,000	17.3	8.9	6.6	3,883	0.00183	7.0	0.0	0.0	2.5	12.4
183.670	210,000	18.1	8.2	7.1	3,797	0.00236	7.0	0.0	0.0	2.3	12.1
183.770	210,000	13.3	7.5	8.3	3,466	0.00341	7.0	0.0	0.0	2.1	11.8
183.860	209,997	16.4	8.2	6.9	3,876	0.00217	7.0	0.0	0.0	2.3	12.1
183.960	208,944	18.1	9.9	5.8	3,794	0.00110	7.0	0.0	0.0	2.8	12.7
184.050	210,000	19.4	9.2	6.3	3,919	0.00192	7.0	0.0	0.0	2.6	12.5
184.140	210,000	19.4	10.3	5.6	3,997	0.00117	7.0	0.0	0.0	2.9	12.9
184.240	210,000	19.4	10.7	5.6	3,804	0.00106	7.0	0.0	0.0	3.0	13.0
184.330	209,998	19.5	10.9	5.4	3,865	0.00096	7.0	0.0	0.0	3.1	13.1
184.430	210,000	17.5	11.3	4.7	4,194	0.00090	7.0	0.0	0.0	3.2	13.2
184.530	209,997	17.9	10.8	4.2	4,938	0.00074	7.0	0.0	0.0	3.0	13.0
184.620	210,000	18.0	9.8	4.4	5,236	0.00072	7.0	0.0	0.0	2.8	12.7
184.710	210,000	16.9	8.9	4.9	5,332	0.00106	7.0	0.0	0.0	2.5	12.3
184.810	210,000	18.0	7.7	5.3	5,626	0.00158	7.0	0.0	0.0	2.2	11.9
184.900	210,000	17.3	7.5	4.8	5,856	0.00253	7.0	0.0	0.0	2.1	11.8
185.000	210,000	17.9	7.2	5.6	5,280	0.00185	7.0	0.0	0.0	2.0	11.7
185.100	210,000	16.7	8.8	4.7	5,169	0.00078	7.0	0.0	0.0	2.5	12.3
185.190	210,000	16.1	9.0	5.2	4,508	0.00107	7.0	0.0	0.0	2.5	12.4
185.280	210,000	14.4	7.3	6.6	4,486	0.00158	7.0	0.0	0.0	2.0	11.8
185.380	209,044	13.7	7.4	7.1	4,230	0.00181	7.0	0.0	0.0	2.1	11.8
185.460	210,000	13.3	6.9	7.2	4,587	0.00163	7.0	0.0	0.0	1.9	11.6
185.530	210,000	13.1	6.7	7.2	4,837	0.00157	7.0	0.0	0.0	1.9	11.5
185.610	209,424	13.9	7.7	6.9	4,303	0.00131	7.0	0.0	0.0	2.2	11.9
185.710	210,000	13.4	8.0	6.2	4,369	0.00194	7.0	0.0	0.0	2.3	12.0
185.810	210,000	16.5	8.3	5.7	4,544	0.00120	7.0	0.0	0.0	2.3	12.1
185.900	210,000	15.4	9.2	5.3	4,396	0.00089	7.0	0.0	0.0	2.6	12.4
186.000	210,000	15.3	9.7	5.1	4,276	0.00087	7.0	0.0	0.0	2.7	12.6
186.100	210,000	15.1	8.5	6.0	4,175	0.00165	7.0	0.0	0.0	2.4	12.2
186.190	227,000	13.4	9.0	6.3	4,143	0.00167	7.0	0.0	0.0	2.5	12.4
186.270	227,000	13.3	9.4	6.2	4,110	0.00167	7.0	0.0	0.0	2.6	12.5

Table 4
Estimation of Total Scour for the Recommended Alternative

River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
	100-Year Hydraulic Data						Scour Components				
186.360	227,000	14.7	9.2	6.4	4,094	0.00188	7.0	0.0	0.0	2.6	12.5
186.460	227,000	14.9	9.8	6.3	3,981	0.00163	7.0	0.0	0.0	2.8	12.7
186.550	226,999	15.6	9.6	6.7	3,823	0.00192	7.0	0.0	0.0	2.7	12.6
186.610	227,000	16.2	8.4	8.3	3,494	0.00309	7.0	0.0	0.0	2.4	12.2
186.690	226,968	16.3	8.5	8.0	3,371	0.00290	7.0	0.0	0.0	2.4	12.2
186.780	226,777	16.2	9.5	7.6	3,224	0.00167	7.0	0.0	0.0	2.7	12.6
186.870	226,429	16.0	9.7	7.4	3,244	0.00128	7.0	0.0	0.0	2.7	12.6
186.970	227,000	17.5	9.8	7.4	3,266	0.00164	7.0	0.0	0.0	2.8	12.7
187.060	227,000	18.4	11.2	6.7	3,183	0.00127	7.0	0.0	0.0	3.2	13.2
187.150	224,619	17.7	11.7	7.4	2,813	0.00136	7.0	0.0	0.0	3.3	13.4
187.240	222,601	17.6	11.8	8.5	2,406	0.00147	7.0	0.0	0.0	3.3	13.4
187.360	221,931	18.7	12.1	7.3	2,723	0.00098	7.0	0.0	0.0	3.4	13.5
187.450	204,722	17.6	11.5	6.8	2,756	0.00094	7.0	0.0	0.0	3.2	13.3
187.540	225,405	17.3	11.4	8.1	2,700	0.00116	7.0	0.0	0.0	3.2	13.3
187.640	219,724	17.7	12.3	8.1	2,444	0.00105	7.0	0.0	0.0	3.5	13.6
187.730	223,984	18.8	11.7	8.8	2,406	0.00128	7.0	0.0	0.0	3.3	13.4
187.820	227,000	19.2	12.2	9.3	2,186	0.00134	7.0	0.0	0.0	3.4	13.5
187.910	227,000	18.7	12.5	9.5	2,080	0.00143	7.0	0.0	0.0	3.5	13.7
188.000	226,998	17.5	12.9	9.7	1,975	0.00140	7.0	0.0	0.0	3.6	13.8
188.040	227,000	21.5	18.0	7.5	1,751	0.00060	7.0	0.0	14.5	5.1	34.5
188.055	**Tuthill Road Bridge**										
188.070	227,000	21.6	18.0	7.5	1,751	0.00059	7.0	8.1	29.0	5.1	63.9
188.100	227,000	20.8	14.9	7.9	2,031	0.00089	7.0	7.3	26.3	4.2	58.3
188.200	224,899	18.0	14.7	6.3	2,532	0.00084	7.0	4.8	17.2	4.1	43.1
188.290	226,362	18.1	13.6	5.4	3,217	0.00047	7.0	2.5	9.1	3.8	29.1
188.390	226,510	17.6	14.2	4.9	3,478	0.00155	7.0	0.0	0.0	4.0	14.3
188.500	227,000	18.1	12.7	4.9	3,912	0.00073	7.0	0.0	0.0	3.6	13.7
188.590	227,000	17.7	12.1	4.8	4,178	0.00127	6.0	0.0	0.0	3.4	12.2
188.690	227,000	28.9	15.9	4.2	4,336	0.00055	5.0	0.0	0.0	4.5	12.3
188.810	227,000	27.5	15.8	3.9	4,534	0.00027	4.0	0.0	0.0	4.4	11.0

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
189.020	219,549	24.8	13.5	3.9	4,482	0.00034	4.0	0.0	0.0	3.8	10.1
189.110	215,934	22.8	13.8	3.9	4,189	0.00080	4.0	0.0	0.0	3.9	10.2
189.210	211,626	23.6	13.1	4.1	4,152	0.00081	4.0	0.0	0.0	3.7	10.0
189.300	219,810	24.4	13.2	4.2	4,155	0.00047	4.0	0.0	0.0	3.7	10.0
189.390	219,536	26.0	13.6	4.1	4,169	0.00039	4.0	0.0	0.0	3.8	10.2
189.480	220,938	23.3	13.6	3.9	4,338	0.00041	4.0	0.0	0.0	3.8	10.2
189.580	221,385	23.4	12.9	4.0	4,501	0.00043	4.0	0.0	0.0	3.6	9.9
189.670	224,848	18.8	12.7	4.1	4,616	0.00049	4.0	0.0	0.0	3.6	9.8
189.770	225,437	21.0	12.8	4.1	4,453	0.00049	4.0	0.0	0.0	3.6	9.9
189.870	227,000	19.0	11.9	4.6	4,390	0.00060	4.0	0.0	0.0	3.3	9.6
189.960	227,000	24.7	12.2	4.9	4,155	0.00063	4.0	0.0	0.0	3.4	9.7
190.050	227,000	24.4	12.2	5.0	4,252	0.00072	4.0	0.0	0.0	3.4	9.7
190.150	227,000	17.1	11.1	5.6	4,320	0.00106	4.0	0.0	0.0	3.1	9.3
190.240	227,000	17.1	11.9	5.1	4,494	0.00093	4.0	0.0	0.0	3.3	9.5
190.340	227,000	17.2	11.6	5.2	4,545	0.00082	4.0	0.0	0.0	3.3	9.4
190.430	227,000	16.7	11.7	5.5	3,624	0.00081	4.0	0.0	0.0	3.3	9.5
190.530	227,000	18.4	11.4	5.4	3,790	0.00082	4.0	0.0	0.0	3.2	9.4
190.620	227,000	17.5	10.9	5.4	3,902	0.00077	4.0	0.0	0.0	3.1	9.2
190.720	227,000	18.9	10.4	5.5	4,068	0.00072	4.0	0.0	0.0	2.9	9.0
190.810	227,000	17.7	10.1	5.6	4,114	0.00074	4.0	0.0	0.0	2.8	8.9
190.910	226,999	18.0	10.0	5.6	4,075	0.00083	4.0	0.0	0.0	2.8	8.9
191.000	226,905	16.7	9.9	5.4	4,298	0.00070	4.0	0.0	0.0	2.8	8.8
191.100	222,902	15.8	10.1	5.3	4,243	0.00066	4.0	0.0	0.0	2.8	8.9
191.190	221,342	17.3	9.6	5.3	4,359	0.00081	4.0	0.0	0.0	2.7	8.7
191.290	224,070	16.8	9.4	5.4	4,500	0.00080	4.0	0.0	0.0	2.6	8.6
191.380	225,197	16.2	9.0	5.4	4,617	0.00112	4.0	0.0	0.0	2.5	8.5
191.480	223,543	17.4	9.0	5.5	4,526	0.00080	4.0	0.0	0.0	2.5	8.5
191.570	210,623	18.1	9.2	6.2	3,722	0.00094	4.0	0.0	0.0	2.6	8.6
191.670	210,855	16.5	9.4	6.8	3,333	0.00110	4.0	0.0	0.0	2.6	8.6
191.760	218,513	15.0	9.6	7.2	3,145	0.00112	4.0	0.0	0.0	2.7	8.7

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
191.860	Downstream limit of King Ranch development (R.S. 191.76)										
191.950											
192.040											
192.140											
192.230											
192.330											
192.380											
192.390	**Proposed Cotton Lane Bridge**										
192.410											
192.520											
192.610											
192.700											
192.790											
192.890											
192.980											
193.070											
193.160	Upstream limit of King Rach development (R.S. 193.25)										
193.250	227,000	19.4	11.4	7.5	2,637	0.00118	4.0	0.0	0.0	3.2	9.4
193.340	227,000	19.5	10.8	6.9	3,026	0.00082	4.0	0.0	0.0	3.0	9.2
193.430	227,000	21.7	10.1	6.8	3,279	0.00088	4.0	0.0	0.0	2.8	8.9
193.530	227,000	22.5	10.1	7.1	3,172	0.00095	4.0	0.0	0.0	2.9	8.9
193.620	227,000	23.7	9.8	7.4	3,149	0.00109	4.0	0.0	0.0	2.8	8.8
193.730	226,311	20.4	9.6	7.0	3,394	0.00099	4.0	0.0	0.0	2.7	8.7
193.790	226,713	20.4	9.8	6.9	3,523	0.00097	4.0	0.0	0.0	2.8	8.8
193.870	226,454	20.6	9.9	7.3	3,446	0.00112	4.0	0.0	0.0	2.8	8.8
193.940	226,992	18.4	10.0	7.8	3,385	0.00123	4.0	0.0	0.0	2.8	8.9
194.020	226,378	18.6	9.9	8.7	3,051	0.00151	4.0	0.0	0.0	2.8	8.8
194.100	227,000	20.8	9.9	9.3	2,624	0.00167	4.0	0.0	0.0	2.8	8.8
194.200	227,000	22.4	10.5	10.2	2,117	0.00187	4.0	0.0	17.5	3.0	31.8
194.205	**Estrella Parkway Bridge**										

Table 4
Estimation of Total Scour for the Recommended Alternative

	100-Year Hydraulic Data						Scour Components				
River Station miles (1)	Discharge in Channel cfs (2)	Max. Channel Depth feet (3)	Channel Hydraulic Depth feet (4)	Channel Velocity fps (5)	Channel Top Width feet (6)	Energy Slope feet/feet (7)	General feet (8)	Contraction feet (9)	Local feet (10)	Anti-Dune feet (11)	Total Scour (12)
194.210	227,000	22.6	10.7	10.0	2,121	0.00173	4.0	3.9	35.0	3.0	59.7
194.290	217,940	10.7	8.2	10.5	2,575	0.00334	4.0	2.9	26.3	2.3	46.1
194.400	215,011	13.0	10.3	7.8	2,708	0.00126	4.0	1.6	14.2	2.9	29.5
194.530	210,970	14.7	10.1	6.9	3,052	0.00098	4.0	0.0	0.0	2.9	8.9
194.620	206,206	14.4	10.5	6.8	2,913	0.00094	4.0	0.0	0.0	2.9	9.0
194.720	212,466	15.2	10.9	7.1	2,810	0.00116	4.0	0.0	0.0	3.1	9.2
194.810	214,416	15.4	11.5	7.0	2,714	0.00090	4.0	0.0	0.0	3.2	9.4
194.910	214,269	14.9	10.6	8.5	2,457	0.00153	4.0	0.0	0.0	3.0	9.1
195.000	211,292	15.4	11.2	9.2	2,130	0.00164	4.0	0.0	0.0	3.1	9.3
195.090	219,703	14.2	11.7	10.7	1,808	0.00198	4.0	0.0	0.0	3.3	9.5
195.130	222,266	13.7	13.0	10.0	1,768	0.00153	4.0	0.0	20.4	3.7	36.5
195.145	Bullard Avenue Bridge										
195.160	221,969	13.9	13.2	9.8	1,769	0.00144	4.0	1.4	40.8	3.7	64.9
195.190	213,056	13.4	11.6	10.4	1,826	0.00191	4.0	1.3	36.9	3.3	59.0
195.280	223,044	14.0	12.3	8.9	2,117	0.00127	4.0	0.9	25.0	3.5	43.3
195.380	218,946	15.7	11.8	8.1	2,377	0.00117	4.0	0.4	11.8	3.3	25.5
195.470	222,800	14.6	11.3	8.6	2,405	0.00168	4.0	0.0	0.0	3.2	9.3
195.560	227,000	15.8	12.0	9.6	2,032	0.00153	4.0	0.0	11.8	3.4	24.9

CONCLUSIONS

1. The levees, as proposed, do not produce adverse sedimentation impacts either in the El Rio study reach or past the SR 85 Bridge

2. The toe-down scour depths for the levee are estimated based on estimates of all components of total scour.

3. There is little hydraulic advantage to performing vegetation enhancement in the Gila River between RS 180.04 and RS 186.78. There is no appreciable lowering of the flood water surface elevation as a result of vegetation management.

4. Vegetation enhancement in continuous corridors on the right overbank (ROB) in the Gila River between RS 180.04 and RS 186.78 is to be avoided and could result in severe erosion and excess sediment loads past the SR 85 Bridge.

5. Sand and gravel pits and recreation ponds can fill with sediment during a large flood such as the 1980 and 1993 floods.

6. Pits can contribute to upstream headcut scour and downstream scour during floods.

7. Large pits can result in long-term streambed degradation downstream of the pit. Average degradation of 3 feet can be expected with larger degradation in the thalweg and low-flow channels.

8. Once pits/lakes fill with sediment, they are susceptible to accelerated scour during exceptionally large floods and can produce excess sediment loads downstream of the pit/lake during floods.

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Attachment 1:

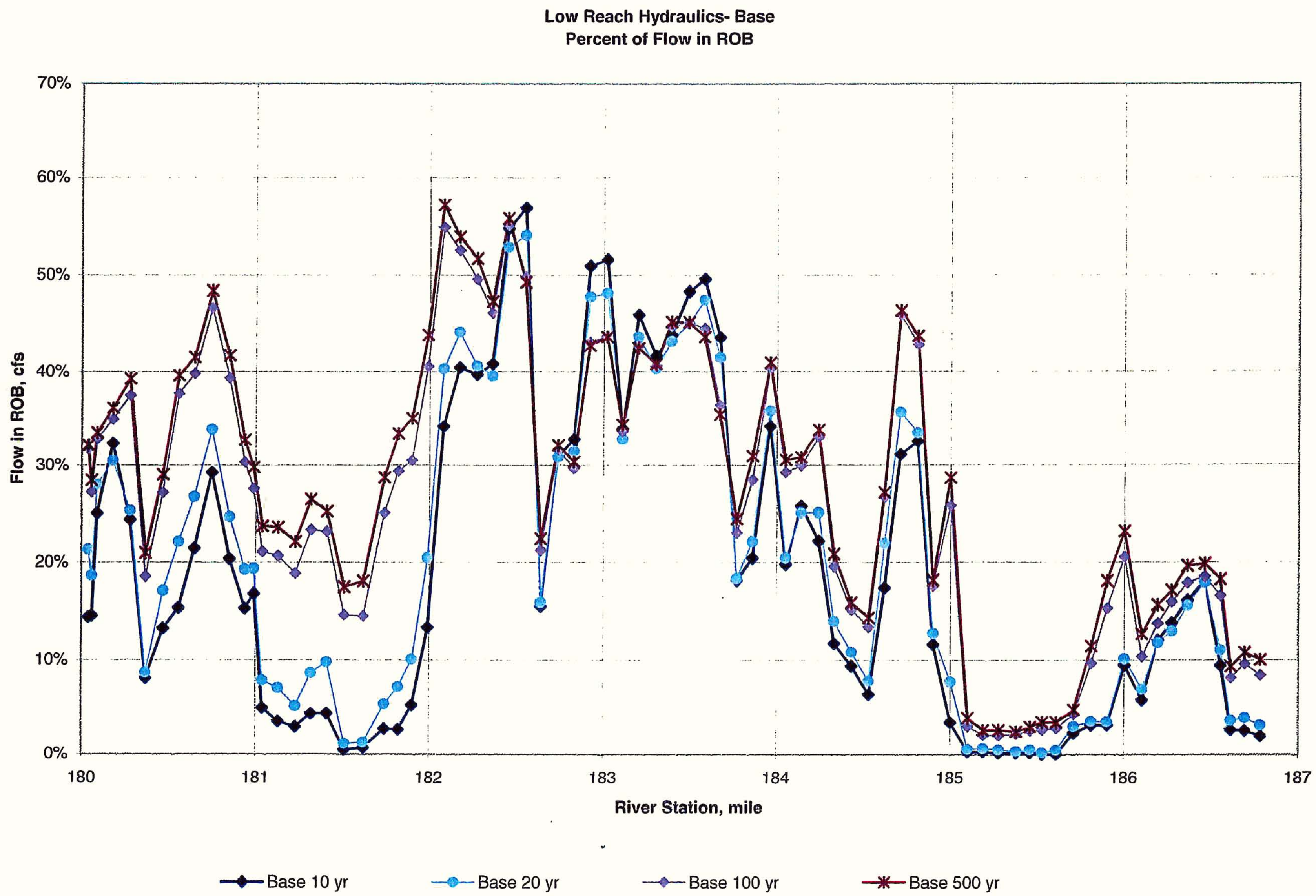
CD Of HEC-6T Models

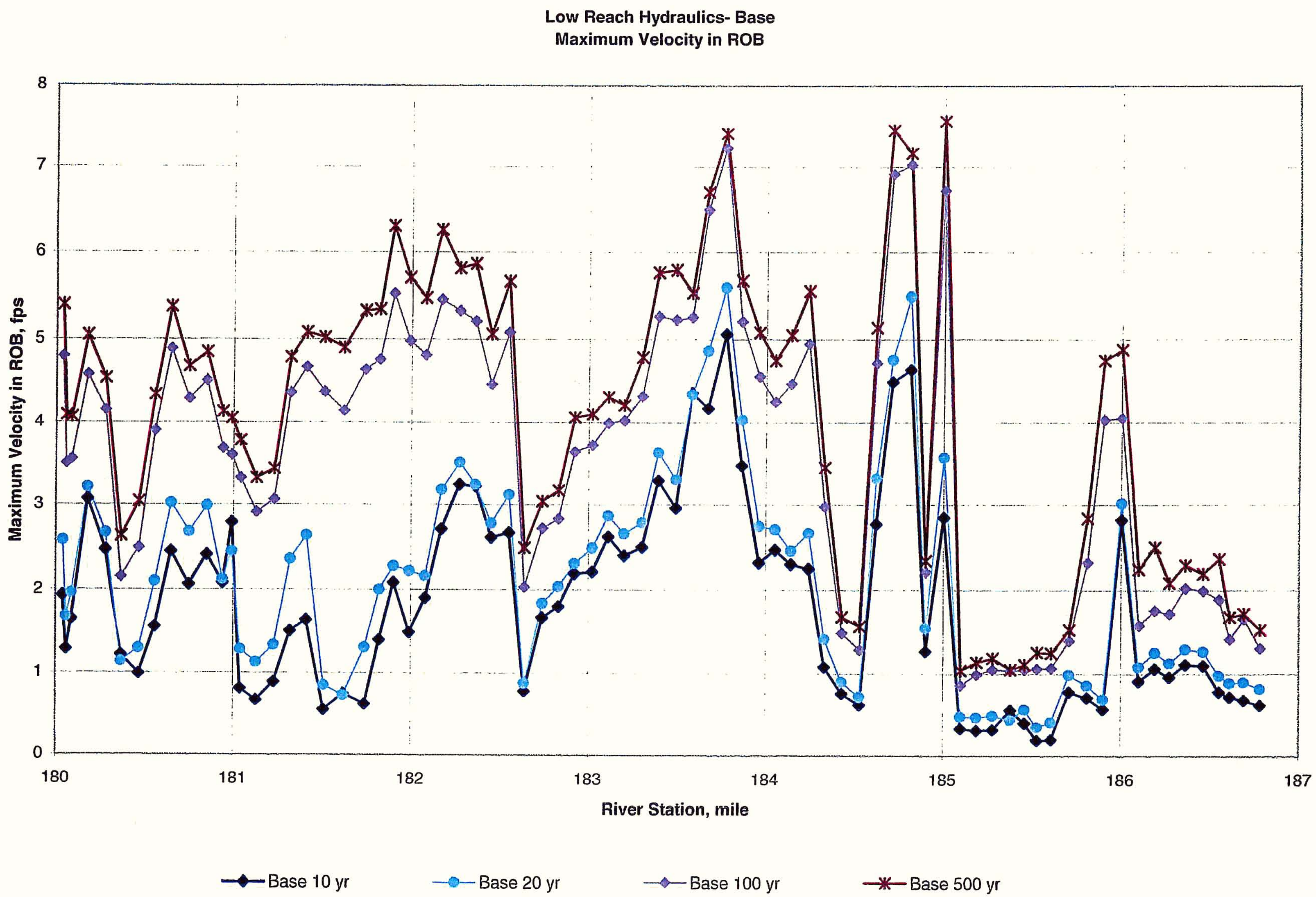
Attachment 2:

CD Of HEC-RAS Models

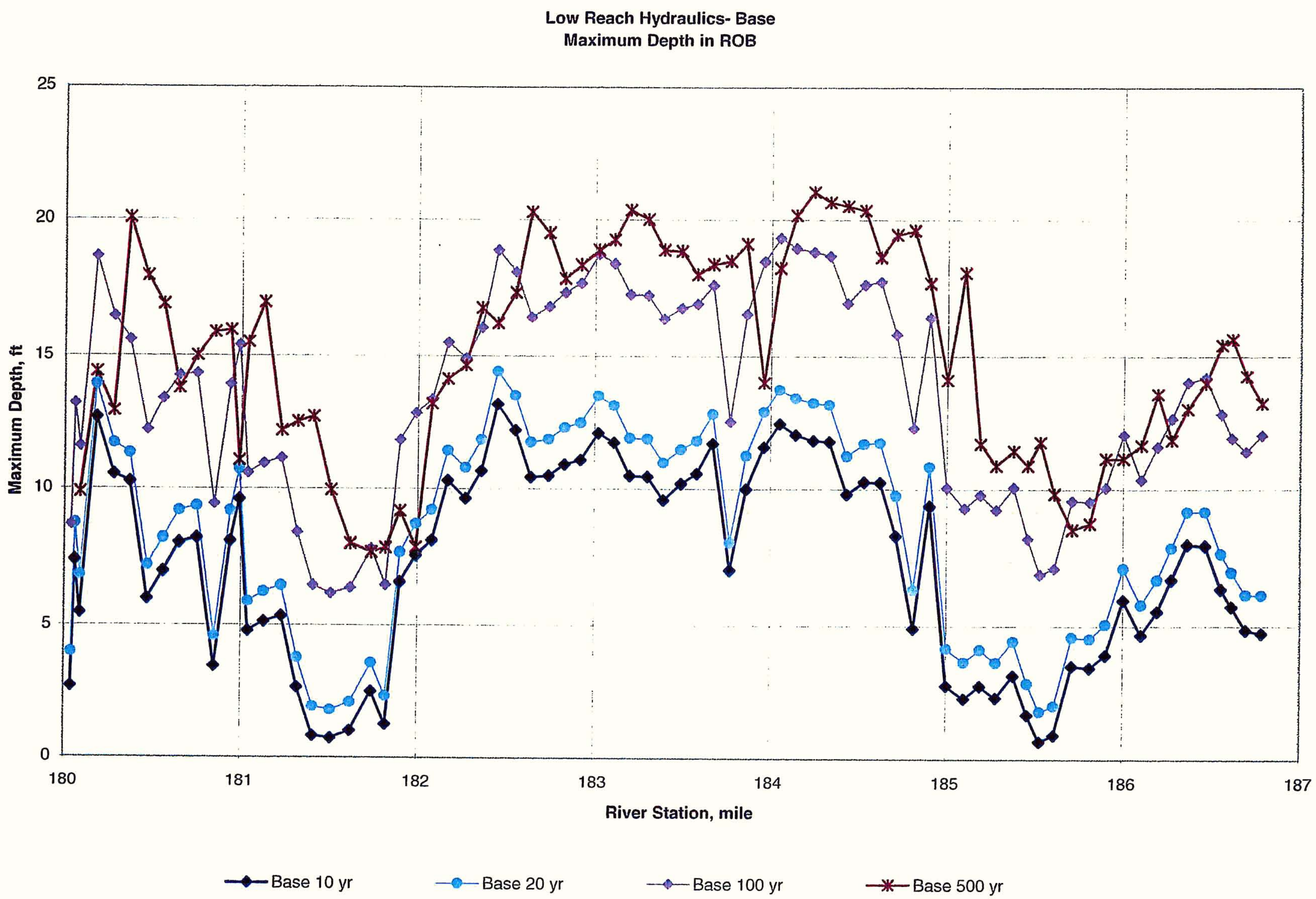
Attachment 3:

Graphs Of HEC-RAS Output For Vegetation Enhancement Analysis

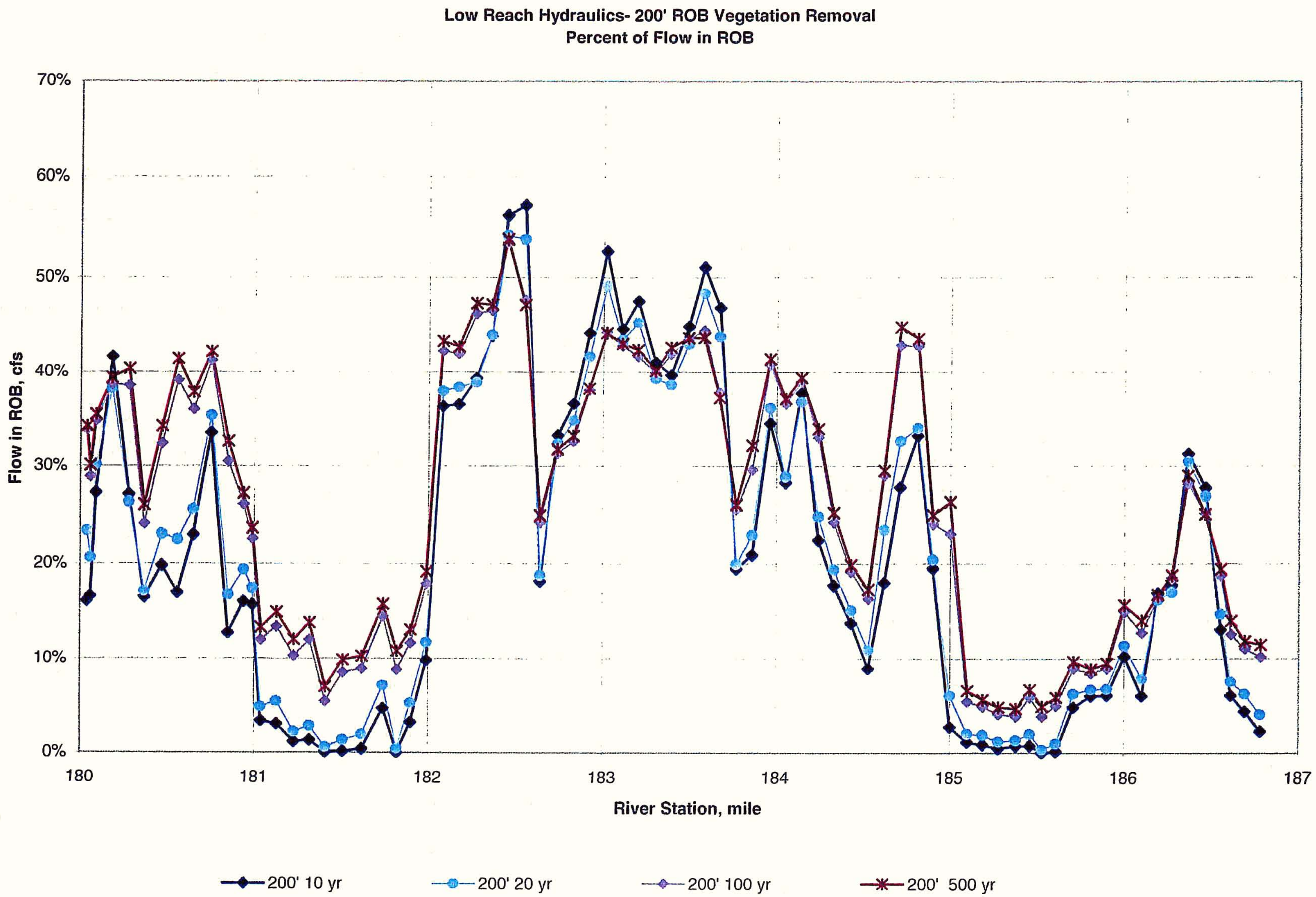

Low Reach Hydraulics- Base
Percent of Flow in ROB
70%
60%
50%
40%
30%
20%
10%
0%
Flow in ROB, cfs
180
181
182
183
184
185
186
187
River Station, mile
Base 10 yr
Base 20 yr
Base 100 yr
Base 500 yr



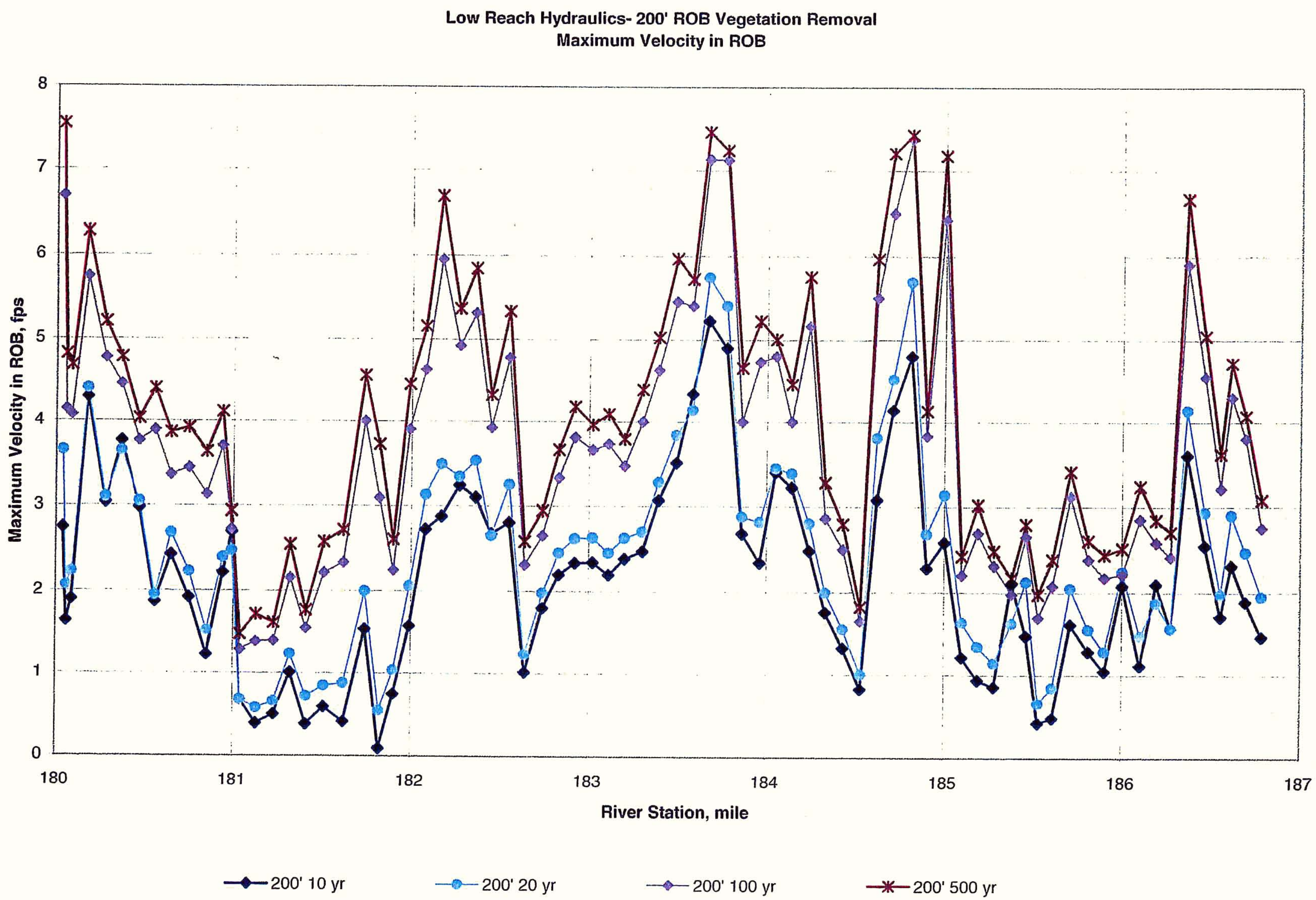
Low Reach Hydraulics- Base
Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
180
181
182
183
184
185
186
187
River Station, mile
Base 10 yr
Base 20 yr
Base 100 yr
Base 500 yr

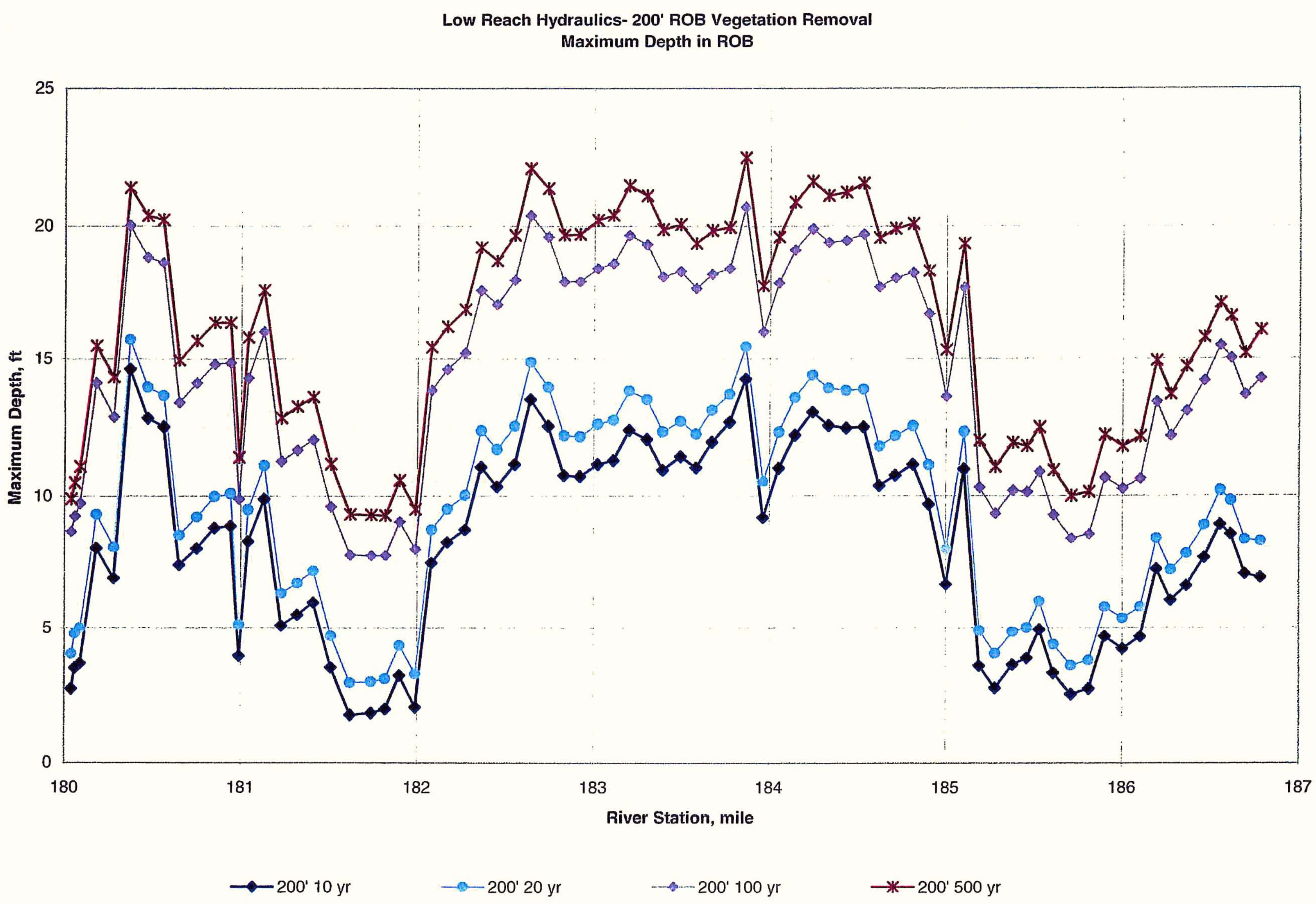


Low Reach Hydraulics- Base
Maximum Depth in ROB
25
20
15
10
5
0
Maximum Depth, ft
180
181
182
183
184
185
186
187
River Station, mile
Base 10 yr
Base 20 yr
Base 100 yr
Base 500 yr



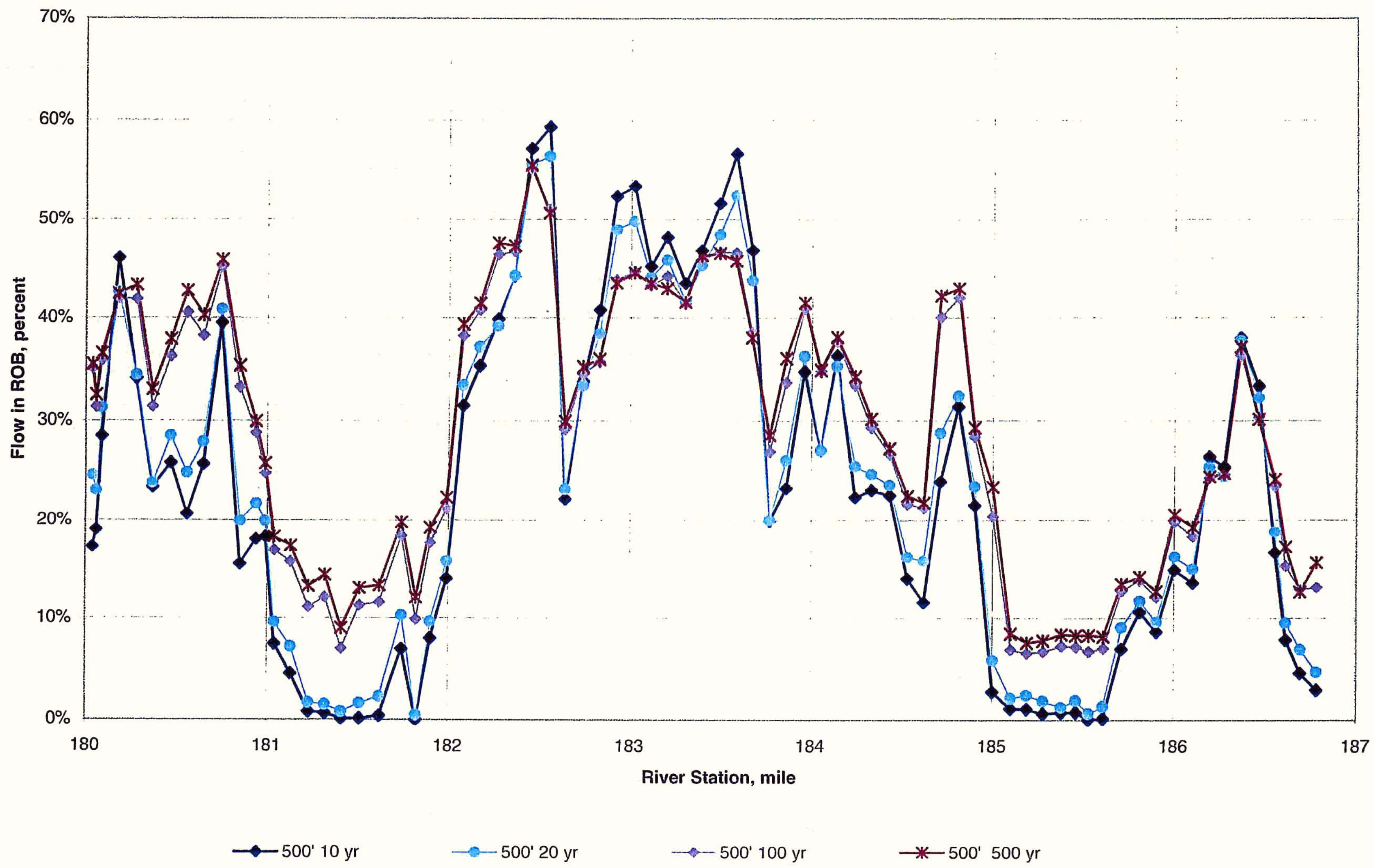
Low Reach Hydraulics- 200' ROB Vegetation Removal
Percent of Flow in ROB
70%
60%
50%
40%
30%
20%
10%
0%
Flow in ROB, cfs
180
181
182
183
184
185
186
187
River Station, mile
200' 10 yr
200' 20 yr
200' 100 yr
200' 500 yr

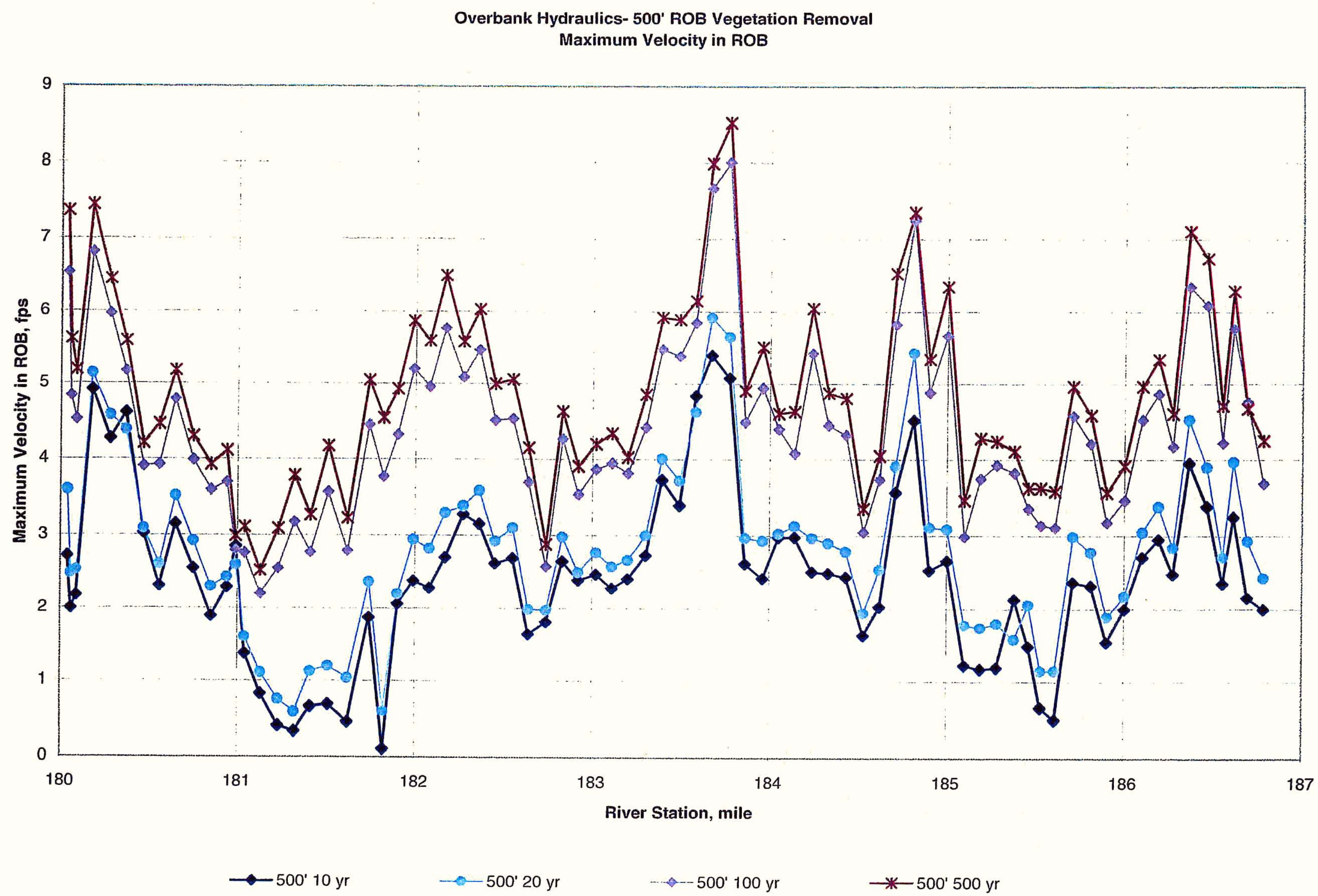

Low Reach Hydraulics- 200' ROB Vegetation Removal
Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
180
181
182
183
184
185
186
187
River Station, mile
200' 10 yr
200' 20 yr
200' 100 yr
200' 500 yr

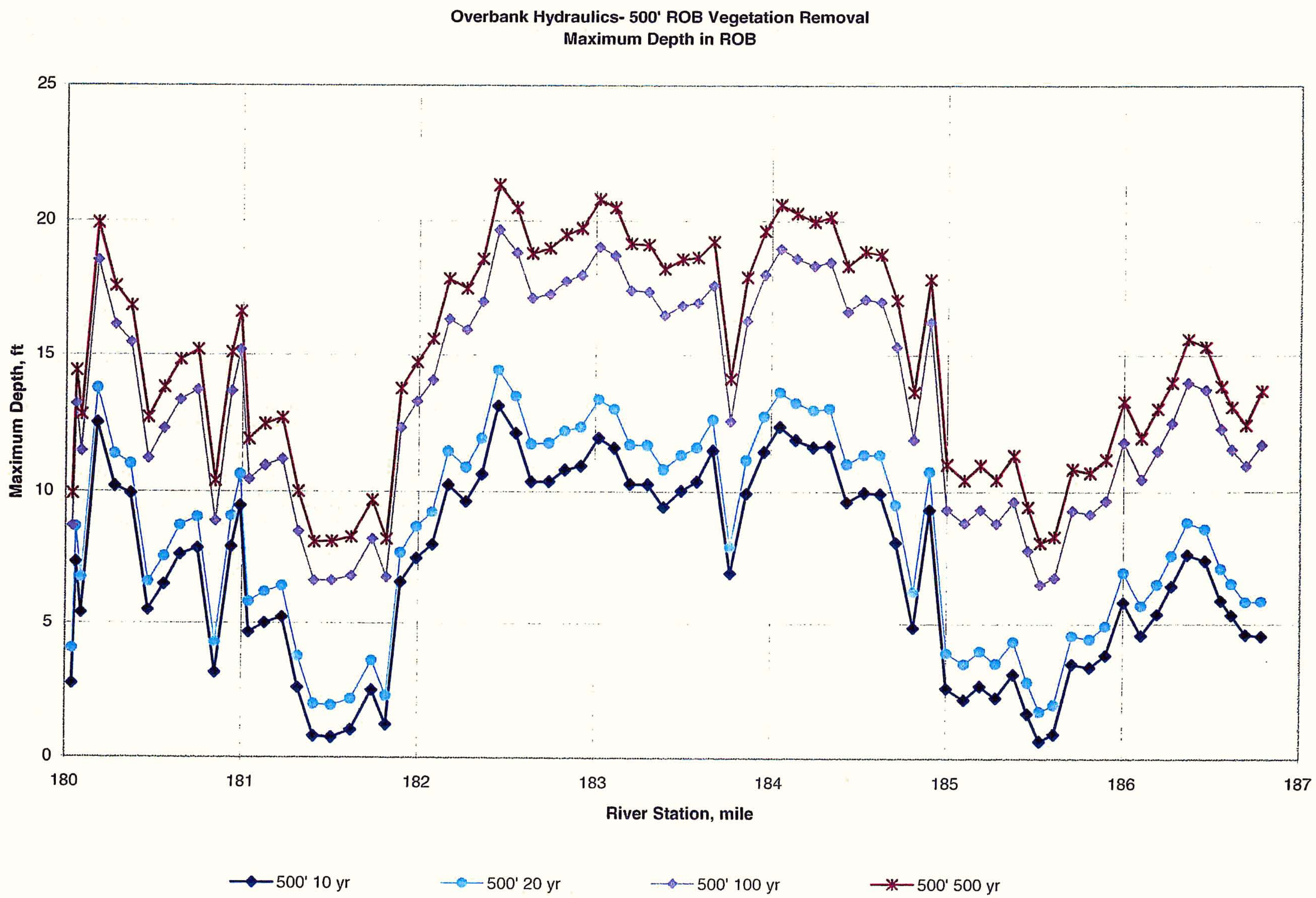


Low Reach Hydraulics- 200' ROB Vegetation Removal
Maximum Depth in ROB
Maximum Depth, ft
0
5
10
15
20
25
180
181
182
183
184
185
186
187
River Station, mile
200' 10 yr
200' 20 yr
200' 100 yr
200' 500 yr

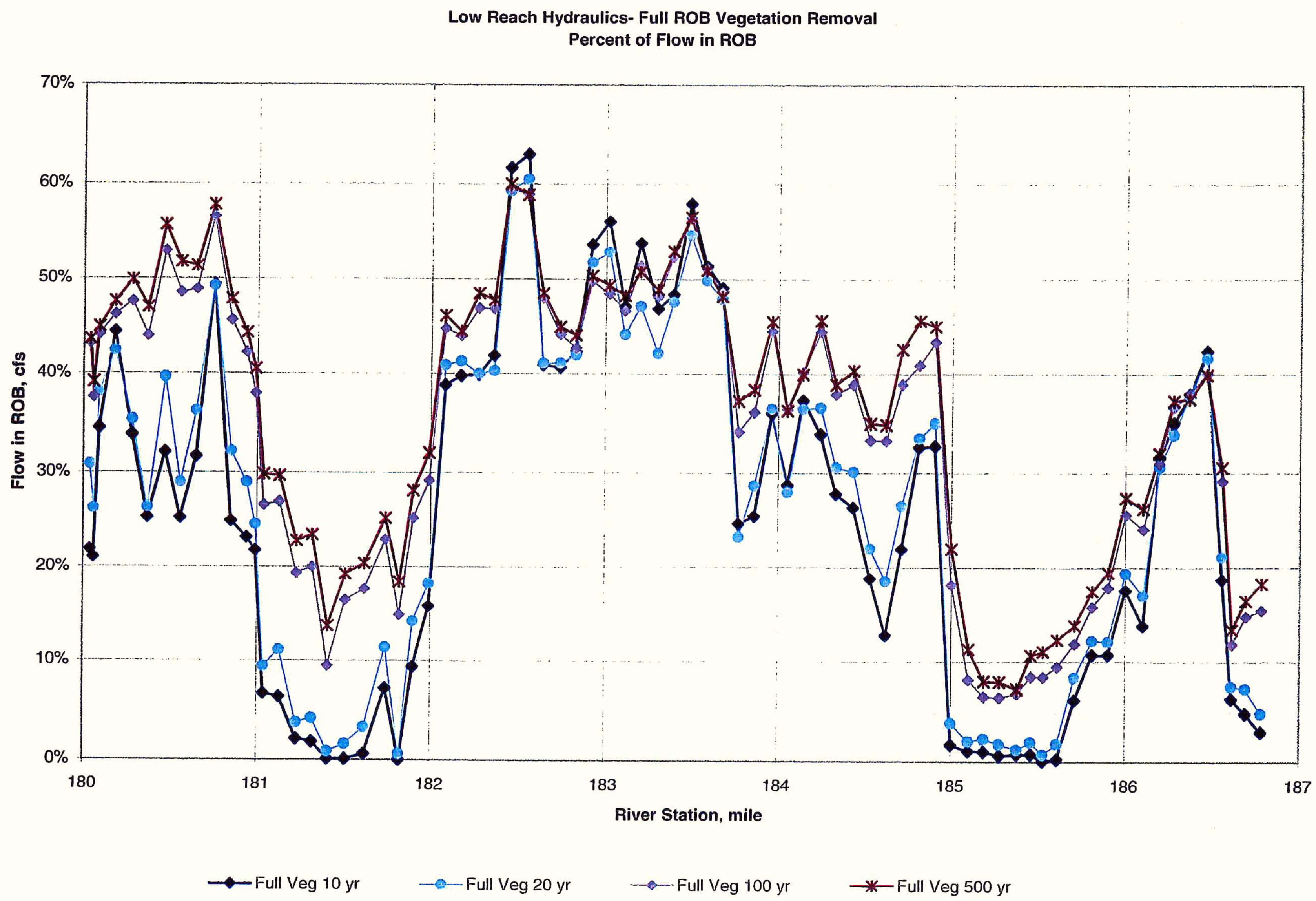
Overbank Hydraulics- 500' ROB Vegetation Removal
Percent of Flow in ROB

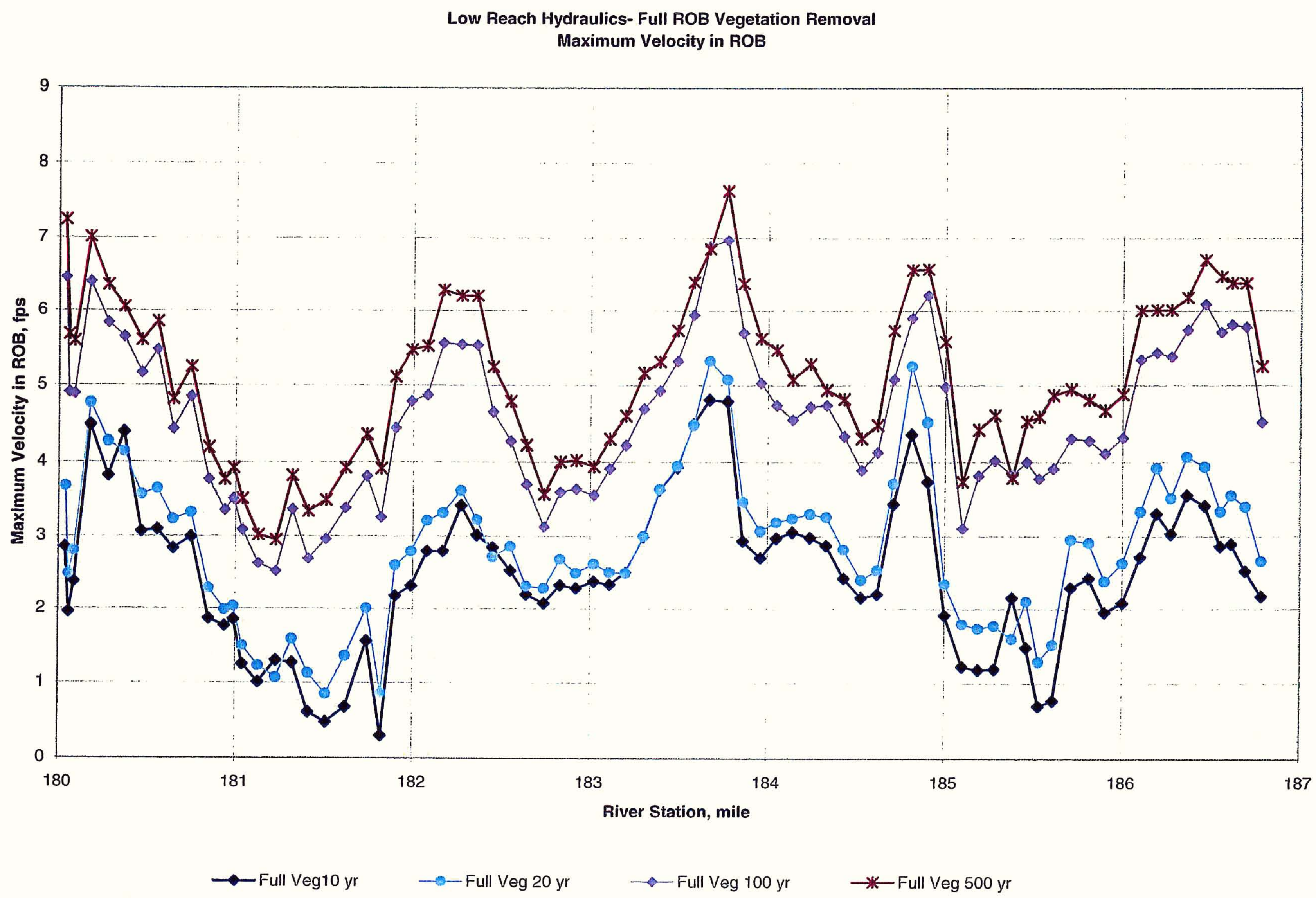


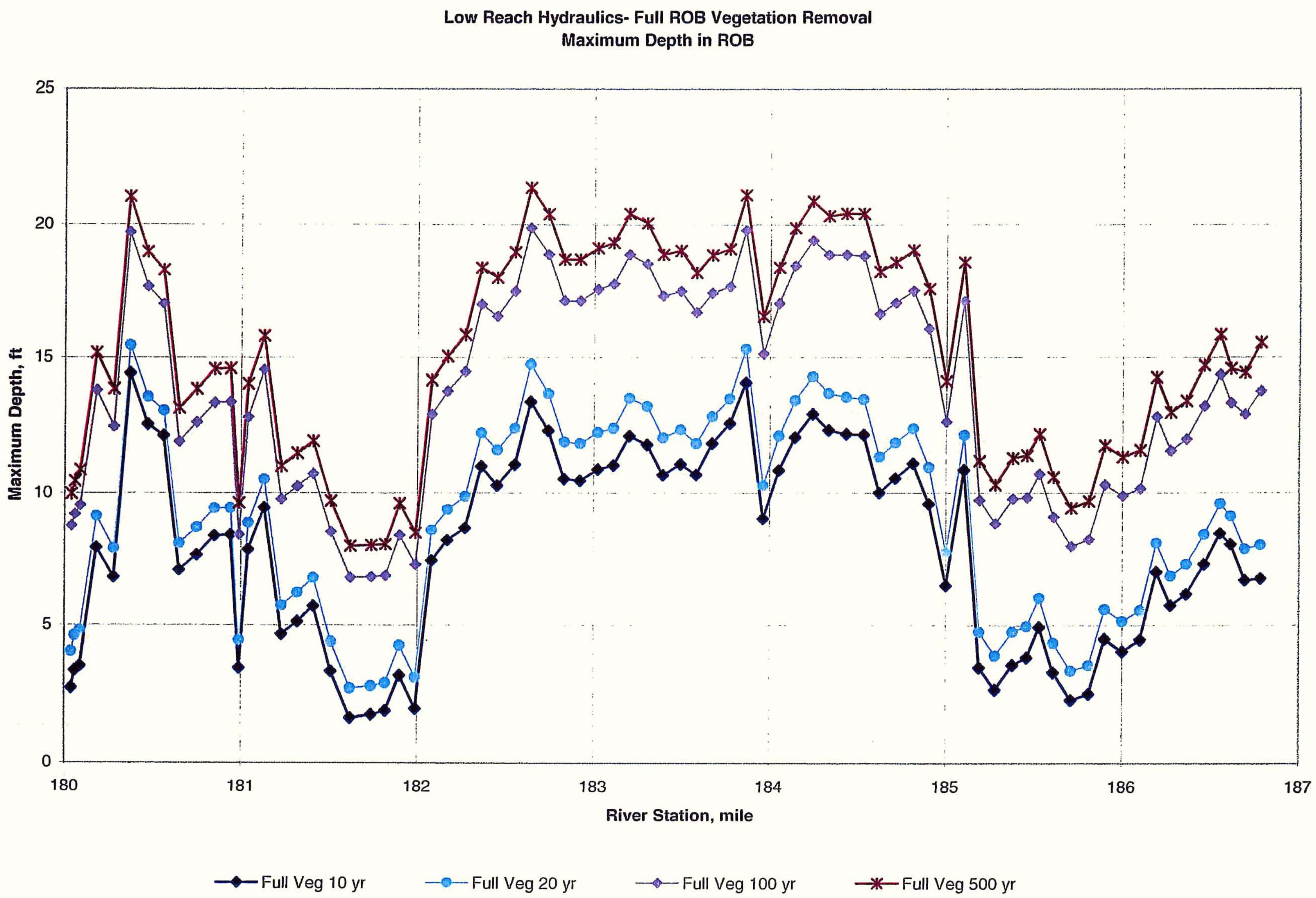

Overbank Hydraulics- 500' ROB Vegetation Removal
Maximum Velocity in ROB
Maximum Velocity in ROB, fps
River Station, mile
9
8
7
6
5
4
3
2
1
0
180
181
182
183
184
185
186
187
500' 10 yr
500' 20 yr
500' 100 yr
500' 500 yr



Overbank Hydraulics- 500' ROB Vegetation Removal
Maximum Depth in ROB
Maximum Depth, ft
River Station, mile
0
5
10
15
20
25
180
181
182
183
184
185
186
187
500' 10 yr
500' 20 yr
500' 100 yr
500' 500 yr


Low Reach Hydraulics- Full ROB Vegetation Removal
Percent of Flow in ROB
Flow in ROB, cfs
70%
60%
50%
40%
30%
20%
10%
0%
180
181
182
183
184
185
186
187
River Station, mile
Full Veg 10 yr
Full Veg 20 yr
Full Veg 100 yr
Full Veg 500 yr

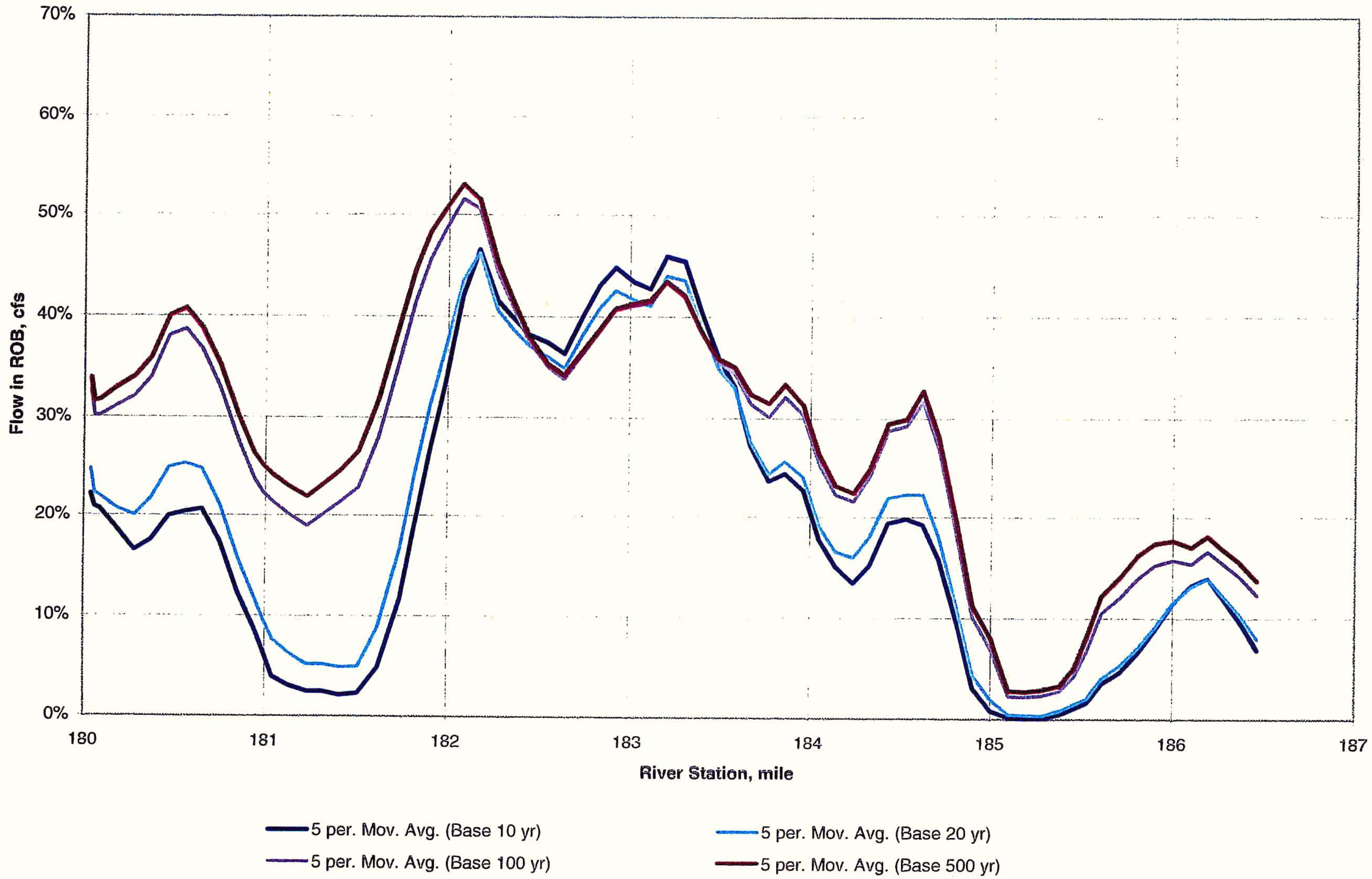

Low Reach Hydraulics- Full ROB Vegetation Removal
Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
9
180
181
182
183
184
185
186
187
River Station, mile
Full Veg10 yr
Full Veg 20 yr
Full Veg 100 yr
Full Veg 500 yr


Low Reach Hydraulics- Full ROB Vegetation Removal
Maximum Depth in ROB
25
20
15
10
5
0
Maximum Depth, ft
180
181
182
183
184
185
186
187
River Station, mile
Full Veg 10 yr
Full Veg 20 yr
Full Veg 100 yr
Full Veg 500 yr

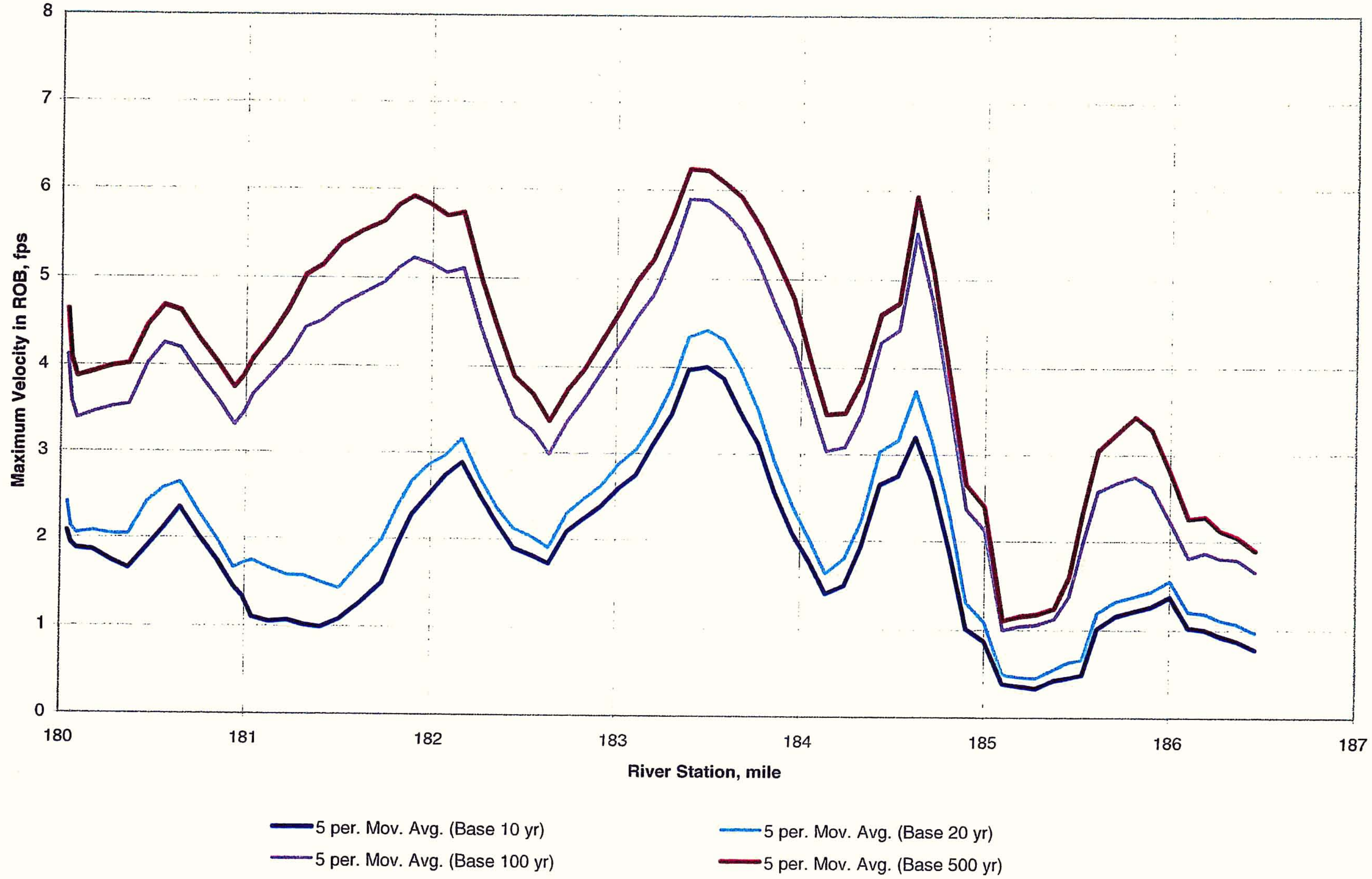
Attachment 4:

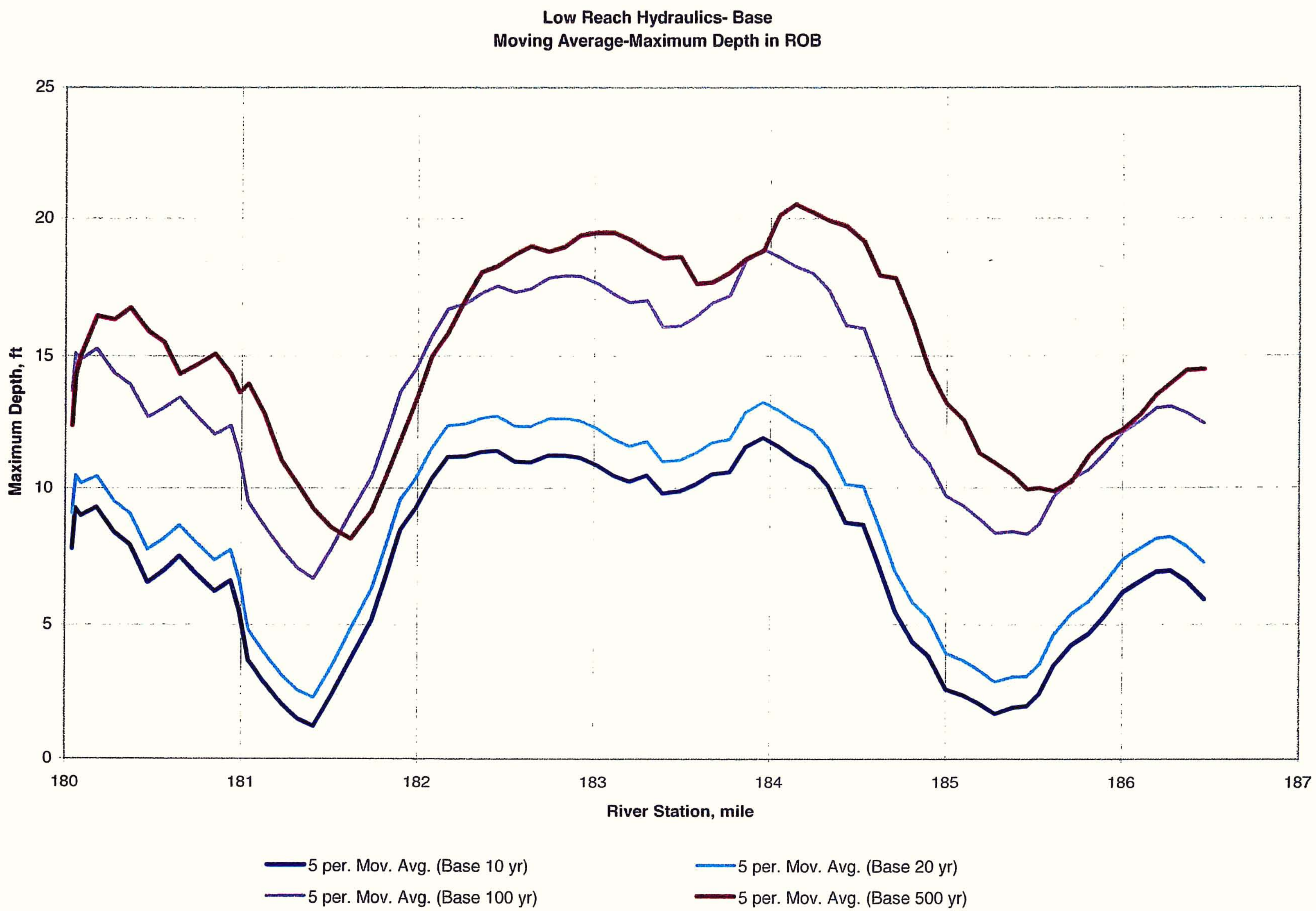
Moving Average Graphs Of HEC-RAS Output For Vegetation Enhancement Analysis

Low Reach Hydraulics- Base
Moving Average-Percent of Flow in ROB

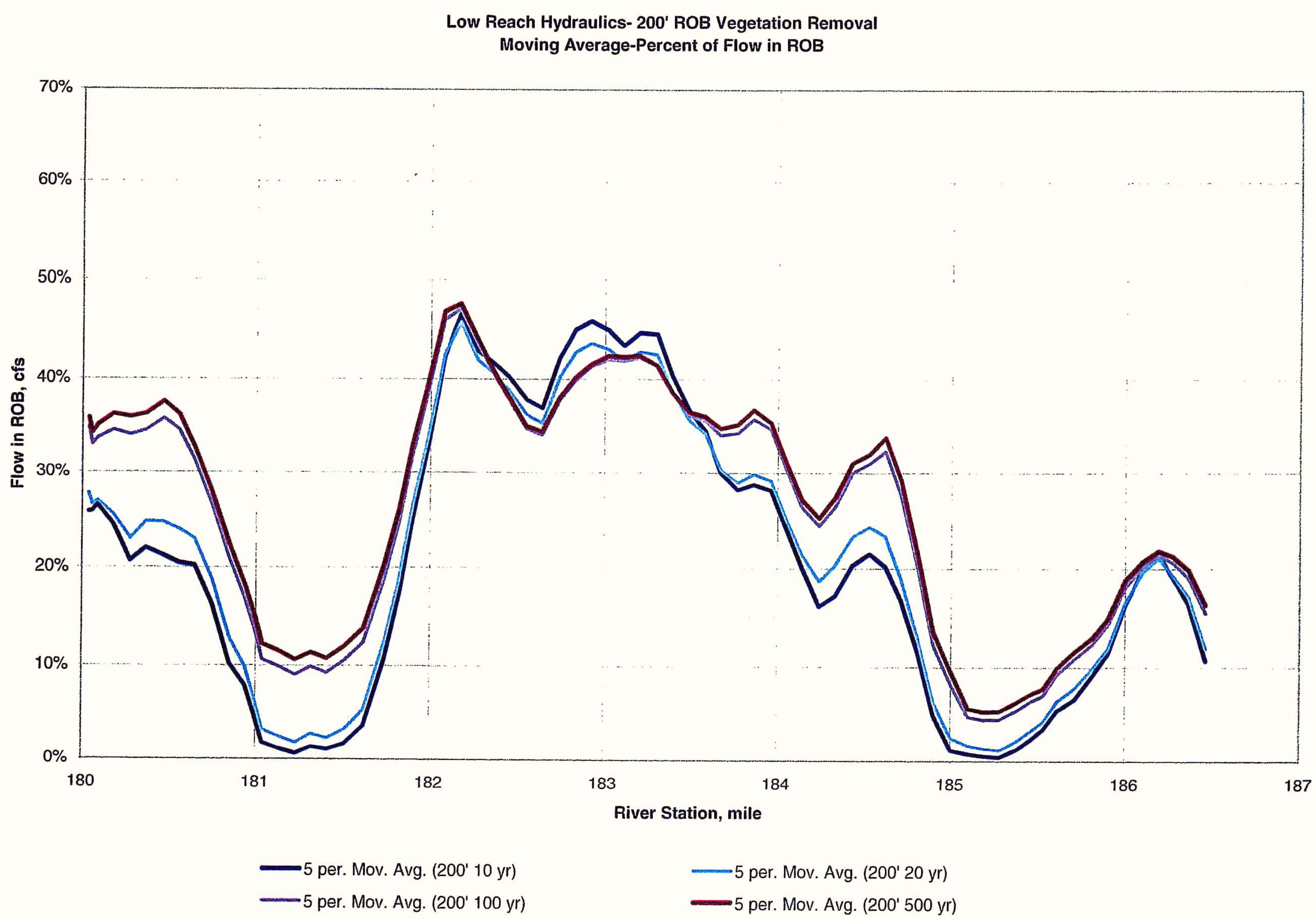


Low Reach Hydraulics- Base
Moving Average-Maximum Velocity in ROB

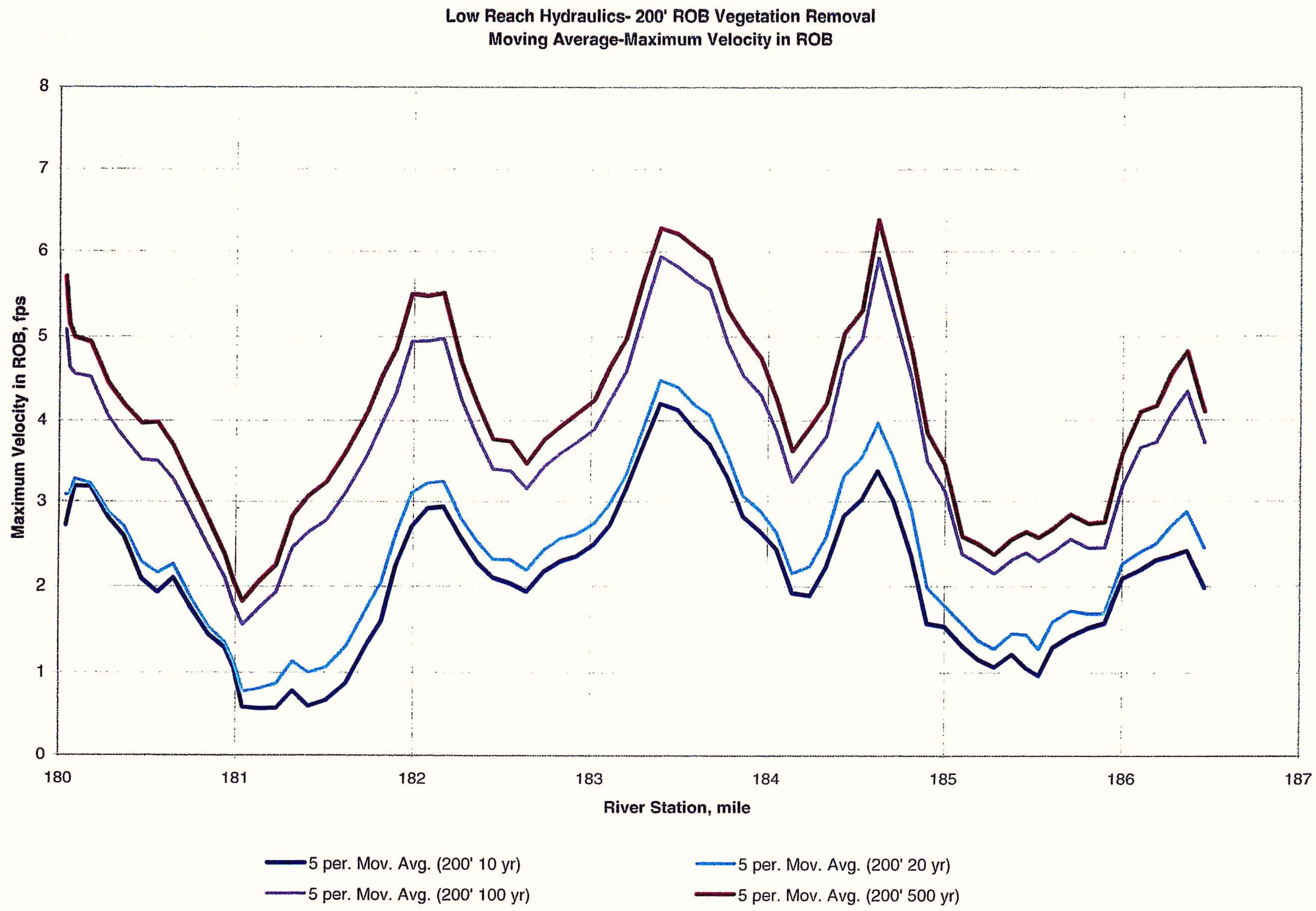




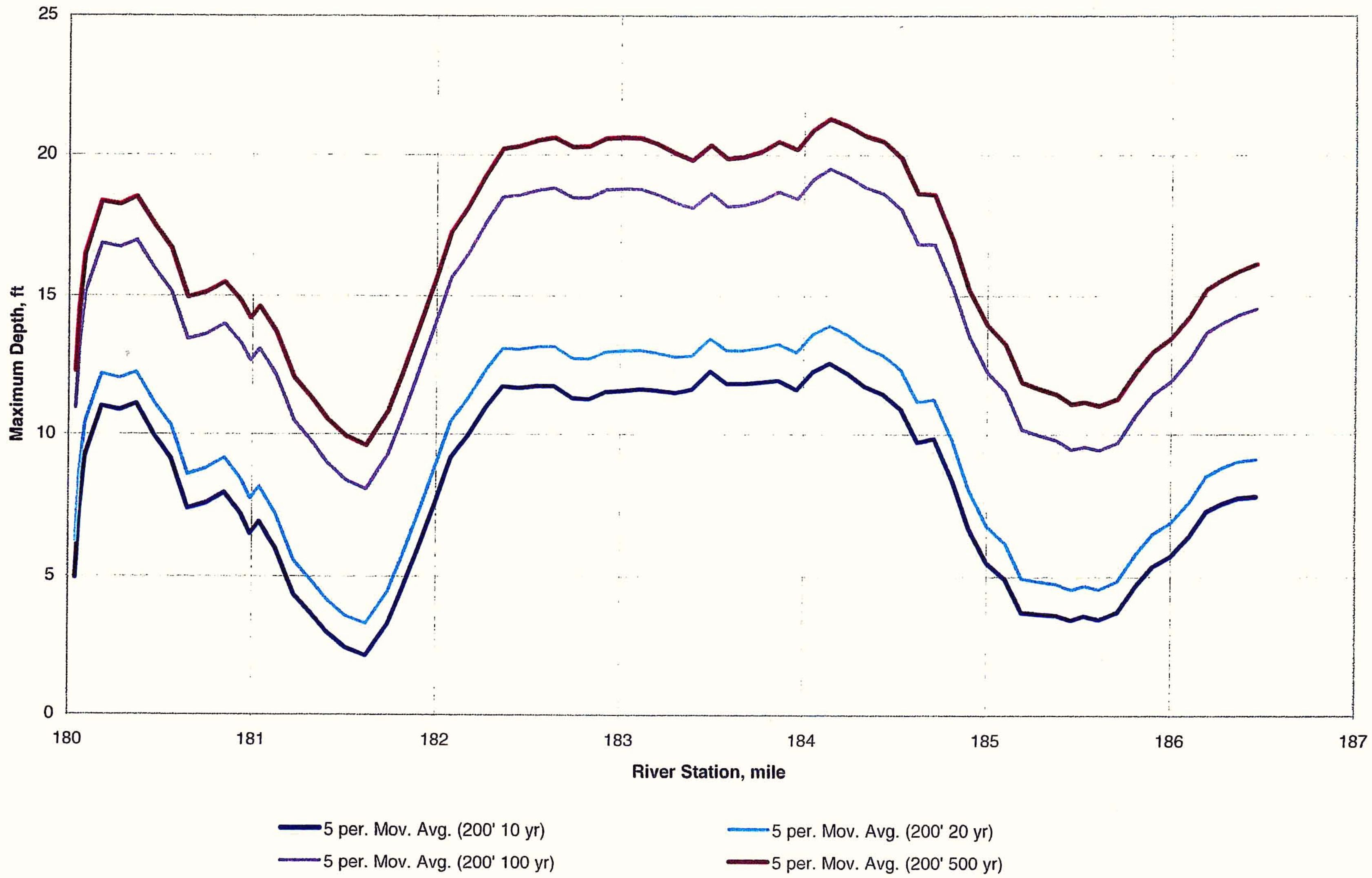
Low Reach Hydraulics- Base
Moving Average-Maximum Depth in ROB
25
20
15
10
5
0
Maximum Depth, ft
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (Base 10 yr)
5 per. Mov. Avg. (Base 20 yr)
5 per. Mov. Avg. (Base 100 yr)
5 per. Mov. Avg. (Base 500 yr)



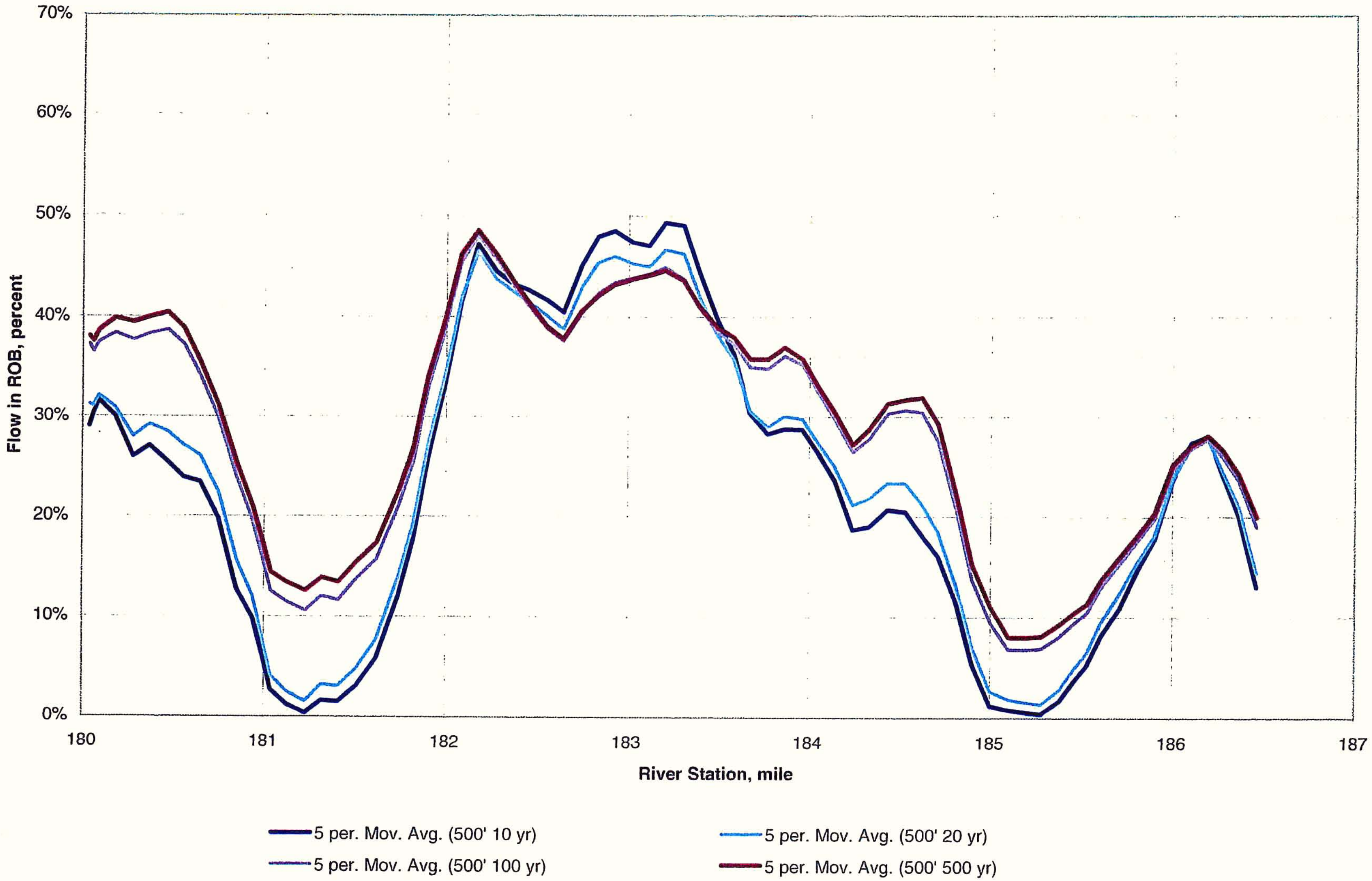
Low Reach Hydraulics- 200' ROB Vegetation Removal
Moving Average-Percent of Flow in ROB
70%
60%
50%
40%
30%
20%
10%
0%
Flow in ROB, cfs
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (200' 10 yr)
5 per. Mov. Avg. (200' 20 yr)
5 per. Mov. Avg. (200' 100 yr)
5 per. Mov. Avg. (200' 500 yr)

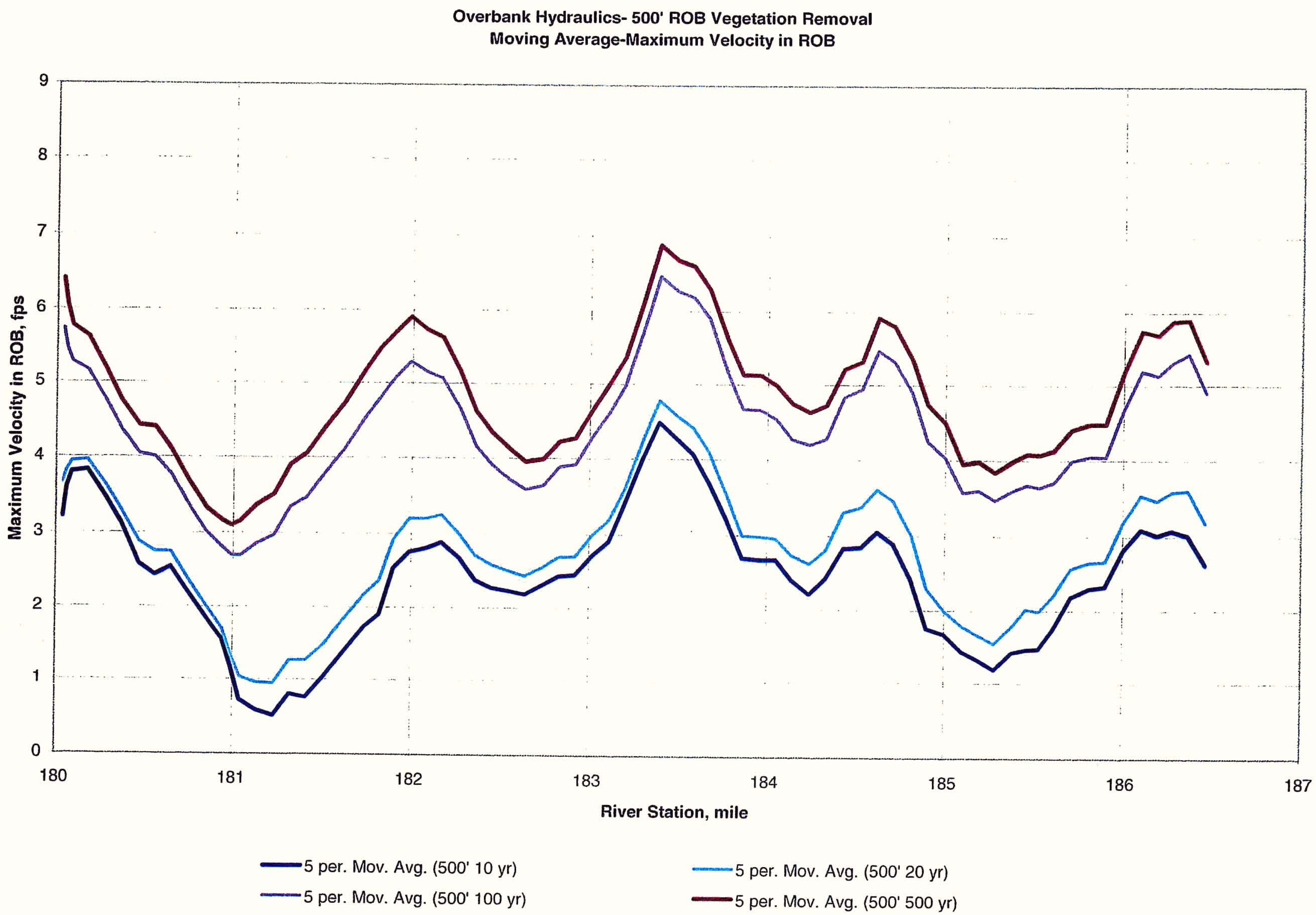

Low Reach Hydraulics- 200' ROB Vegetation Removal
Moving Average-Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (200' 10 yr)
5 per. Mov. Avg. (200' 20 yr)
5 per. Mov. Avg. (200' 100 yr)
5 per. Mov. Avg. (200' 500 yr)

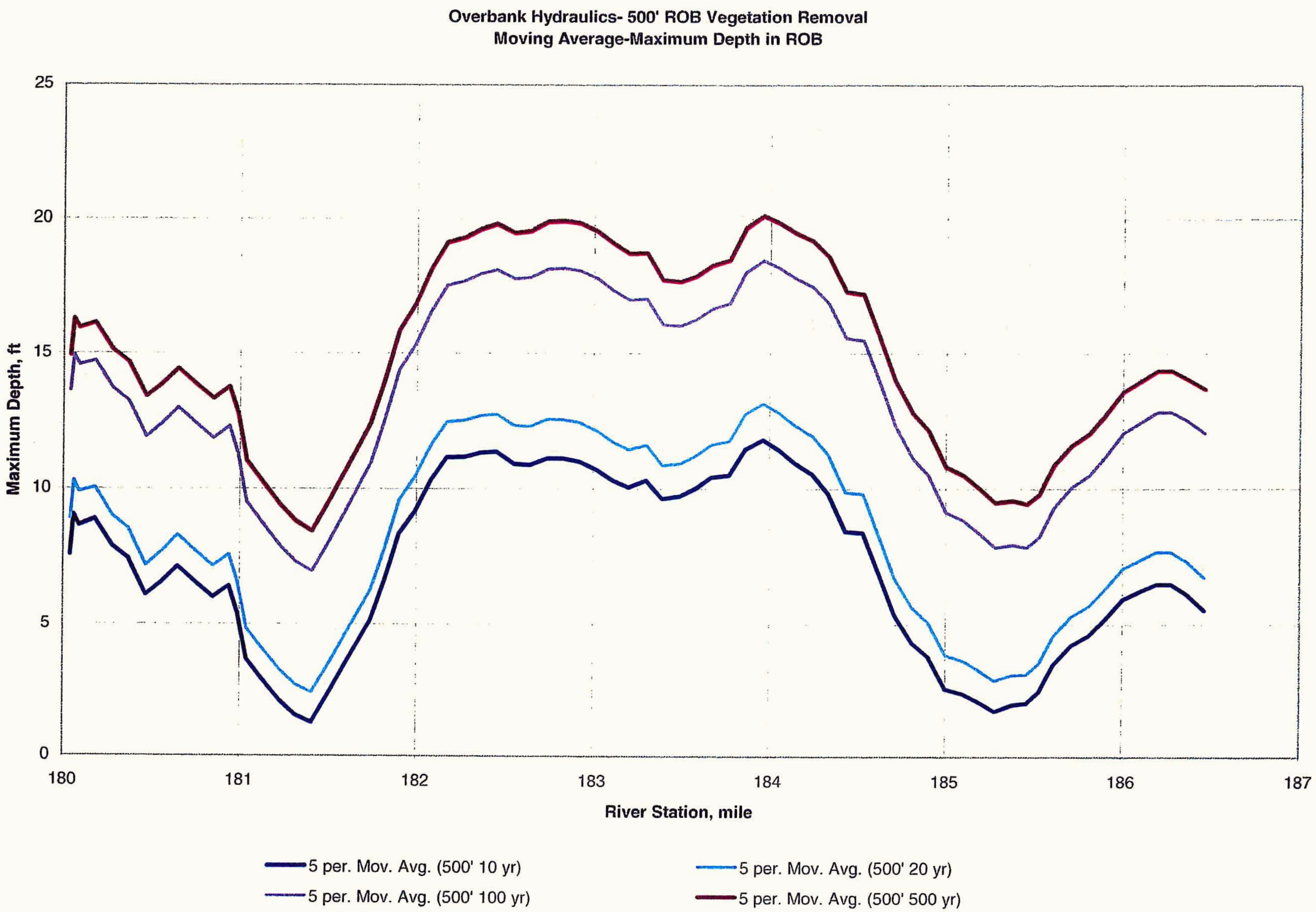
Low Reach Hydraulics- 200' ROB Vegetation Removal
Moving Average-Maximum Depth in ROB



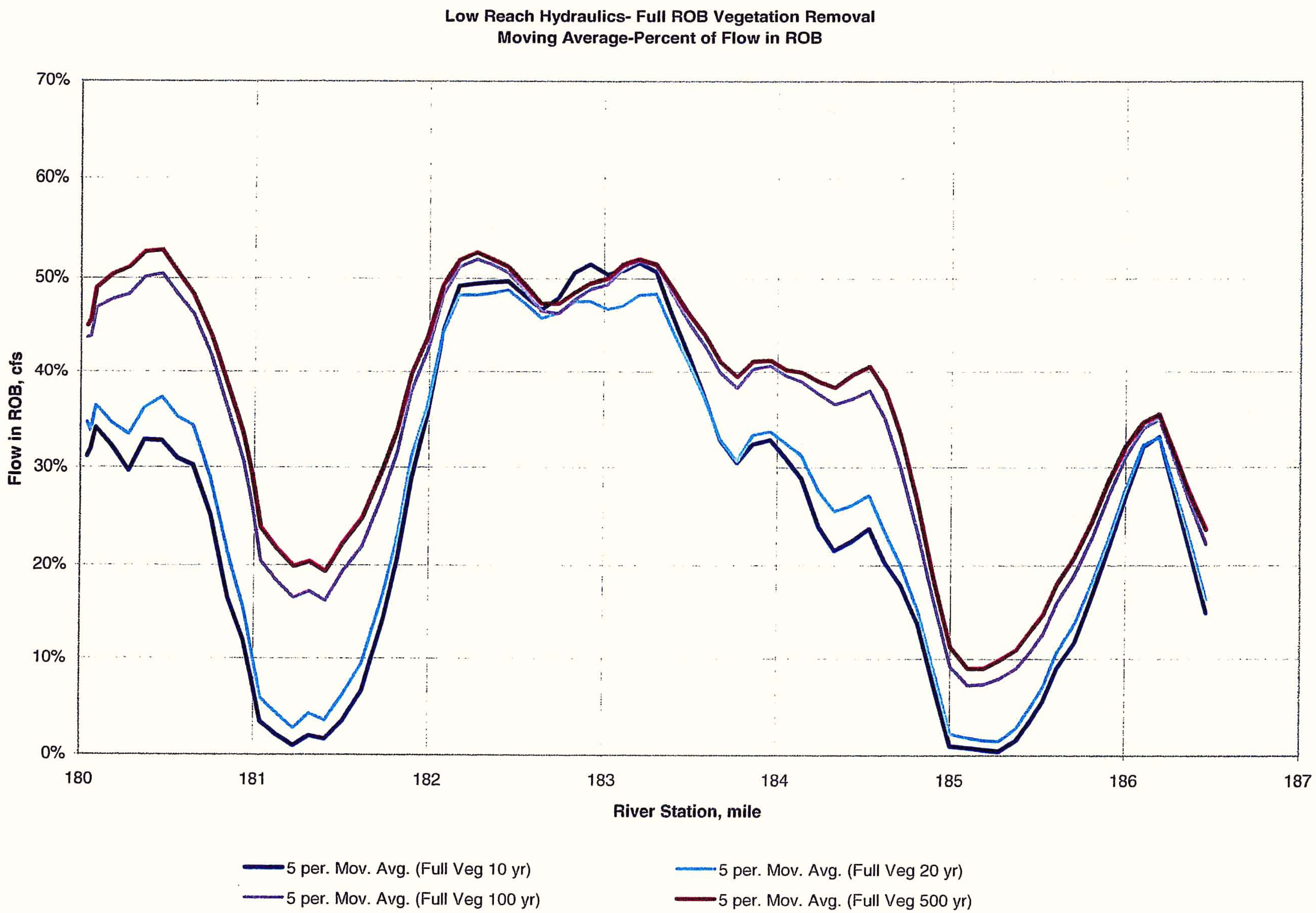
Overbank Hydraulics- 500' ROB Vegetation Removal
Moving Average-Percent of Flow in ROB



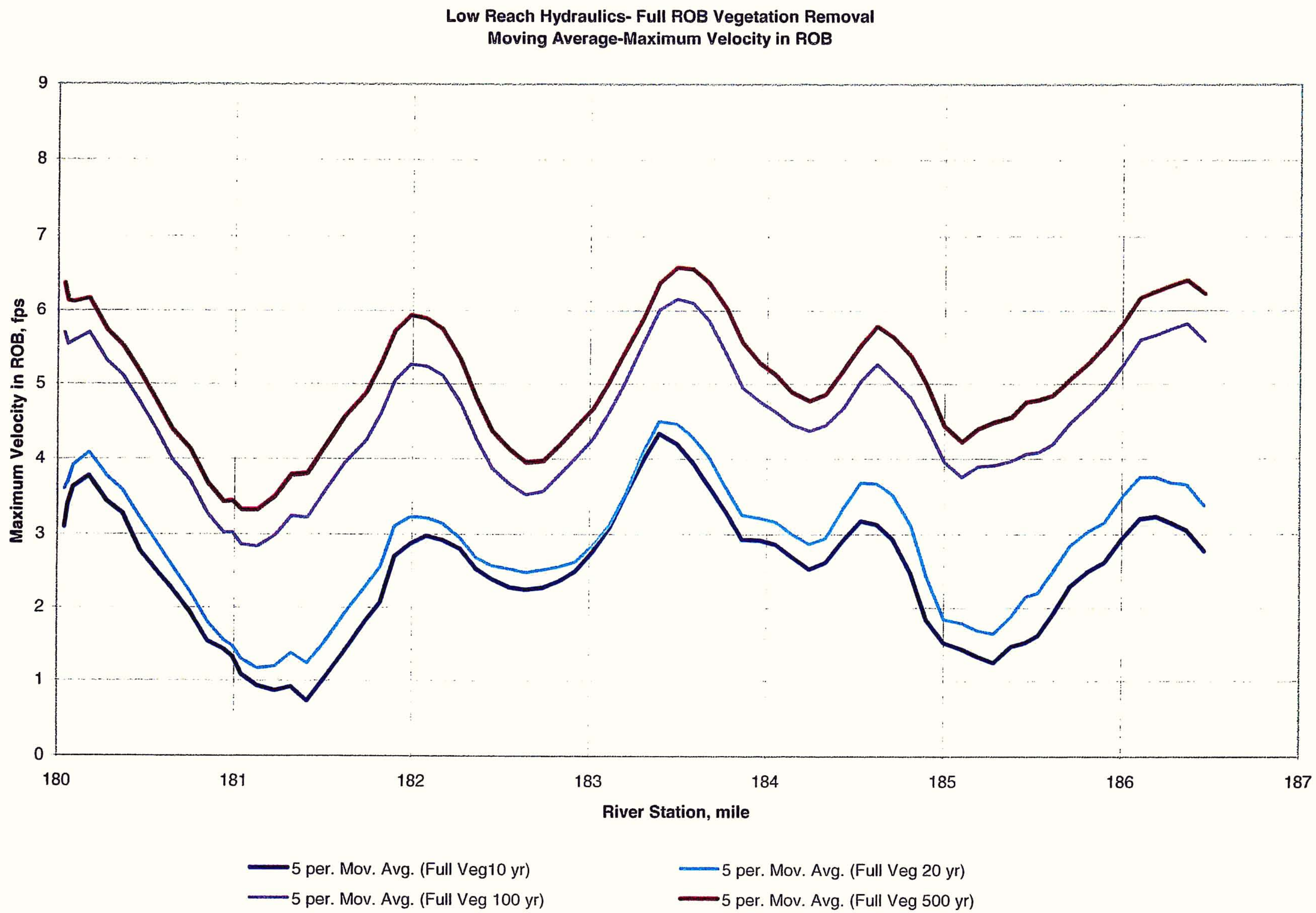

Overbank Hydraulics- 500' ROB Vegetation Removal
Moving Average-Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
9
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (500' 10 yr)
5 per. Mov. Avg. (500' 20 yr)
5 per. Mov. Avg. (500' 100 yr)
5 per. Mov. Avg. (500' 500 yr)

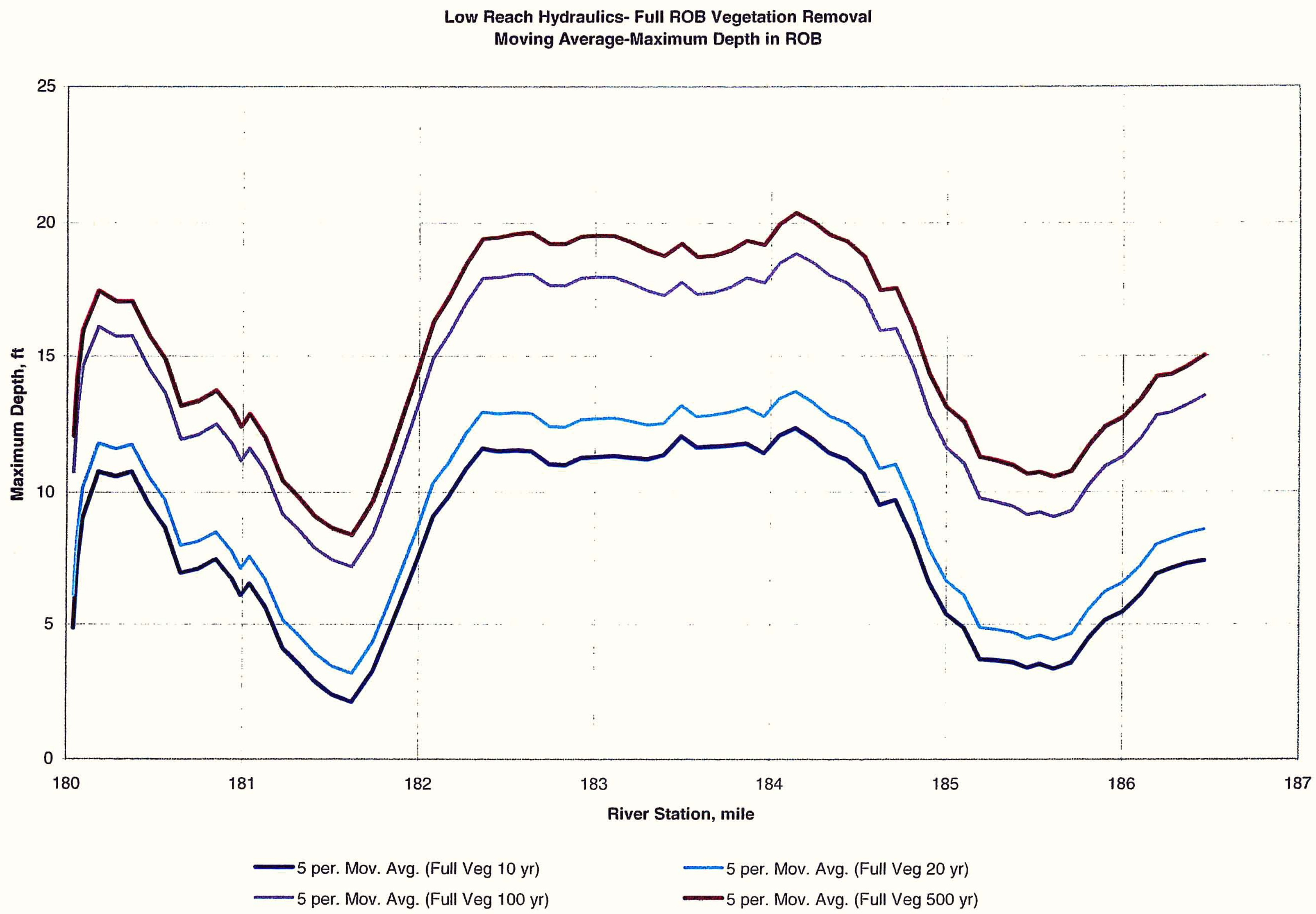


Overbank Hydraulics- 500' ROB Vegetation Removal
Moving Average-Maximum Depth in ROB
25
20
15
10
5
0
Maximum Depth, ft
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (500' 10 yr)
5 per. Mov. Avg. (500' 20 yr)
5 per. Mov. Avg. (500' 100 yr)
5 per. Mov. Avg. (500' 500 yr)



Low Reach Hydraulics- Full ROB Vegetation Removal
Moving Average-Percent of Flow in ROB
70%
60%
50%
40%
30%
20%
10%
0%
Flow in ROB, cfs
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (Full Veg 10 yr)
5 per. Mov. Avg. (Full Veg 20 yr)
5 per. Mov. Avg. (Full Veg 100 yr)
5 per. Mov. Avg. (Full Veg 500 yr)


Low Reach Hydraulics- Full ROB Vegetation Removal
Moving Average-Maximum Velocity in ROB
Maximum Velocity in ROB, fps
0
1
2
3
4
5
6
7
8
9
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (Full Veg10 yr)
5 per. Mov. Avg. (Full Veg 20 yr)
5 per. Mov. Avg. (Full Veg 100 yr)
5 per. Mov. Avg. (Full Veg 500 yr)

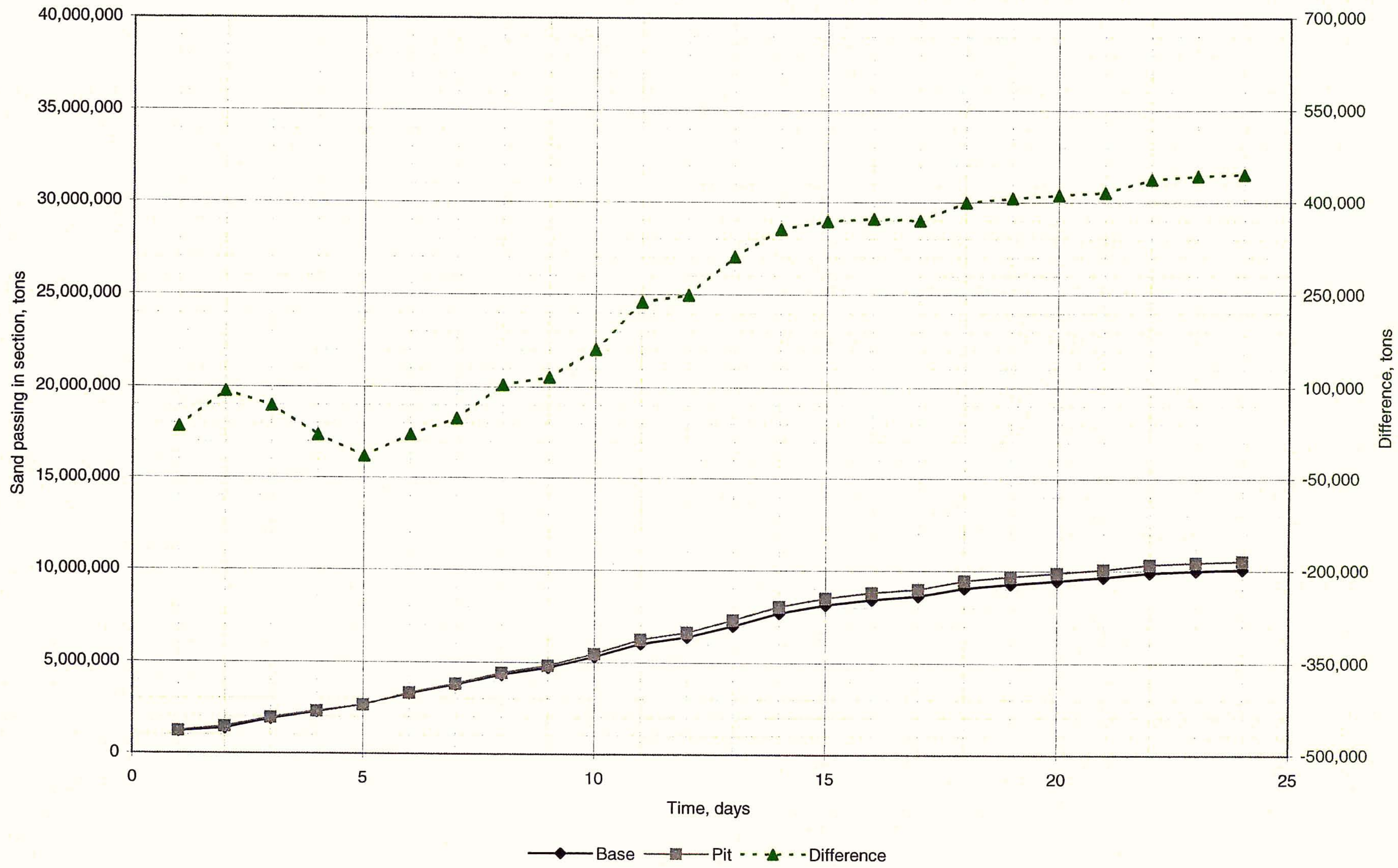


Low Reach Hydraulics- Full ROB Vegetation Removal
Moving Average-Maximum Depth in ROB
Maximum Depth, ft
25
20
15
10
5
0
180
181
182
183
184
185
186
187
River Station, mile
5 per. Mov. Avg. (Full Veg 10 yr)
5 per. Mov. Avg. (Full Veg 20 yr)
5 per. Mov. Avg. (Full Veg 100 yr)
5 per. Mov. Avg. (Full Veg 500 yr)

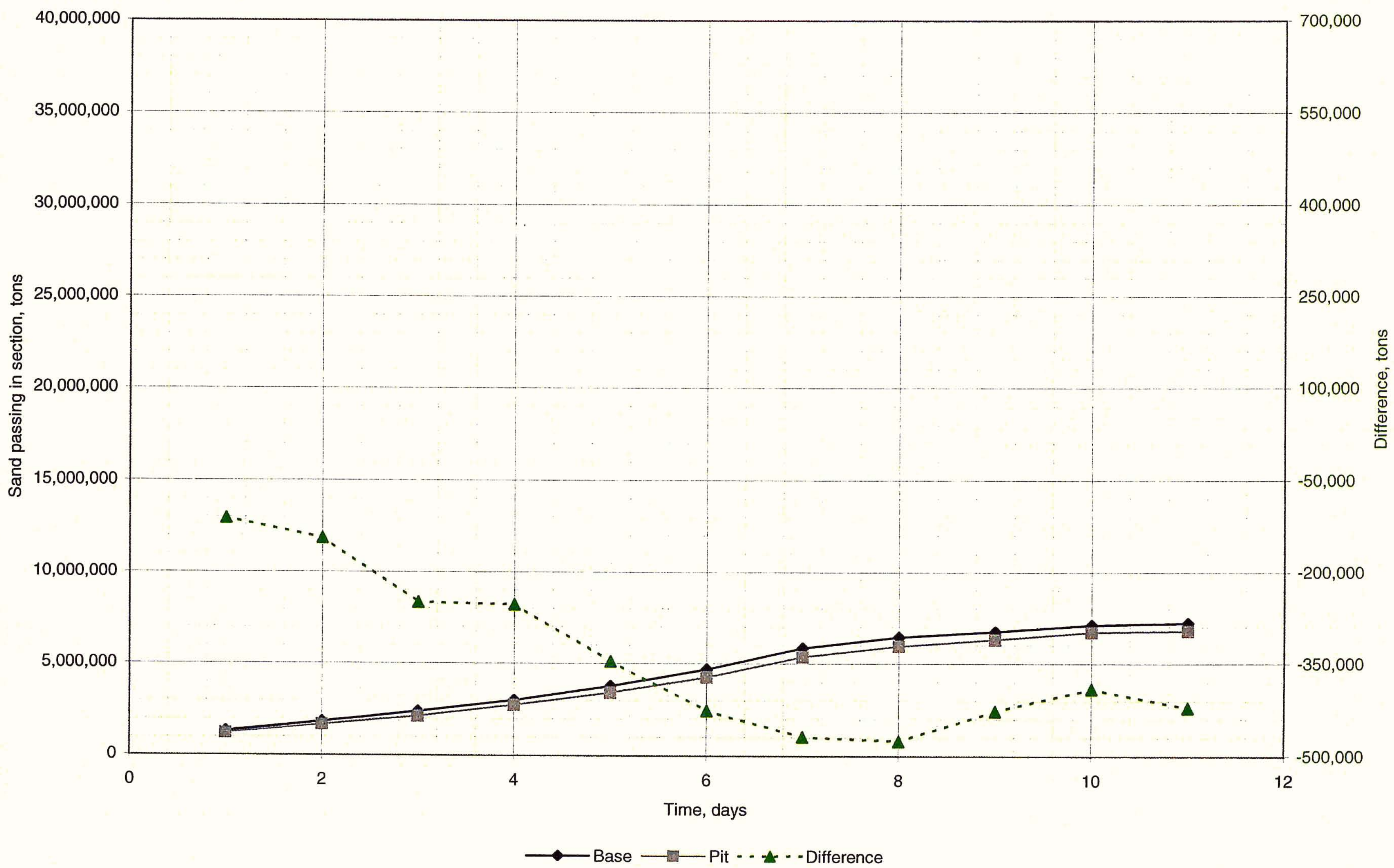
Attachment 5:

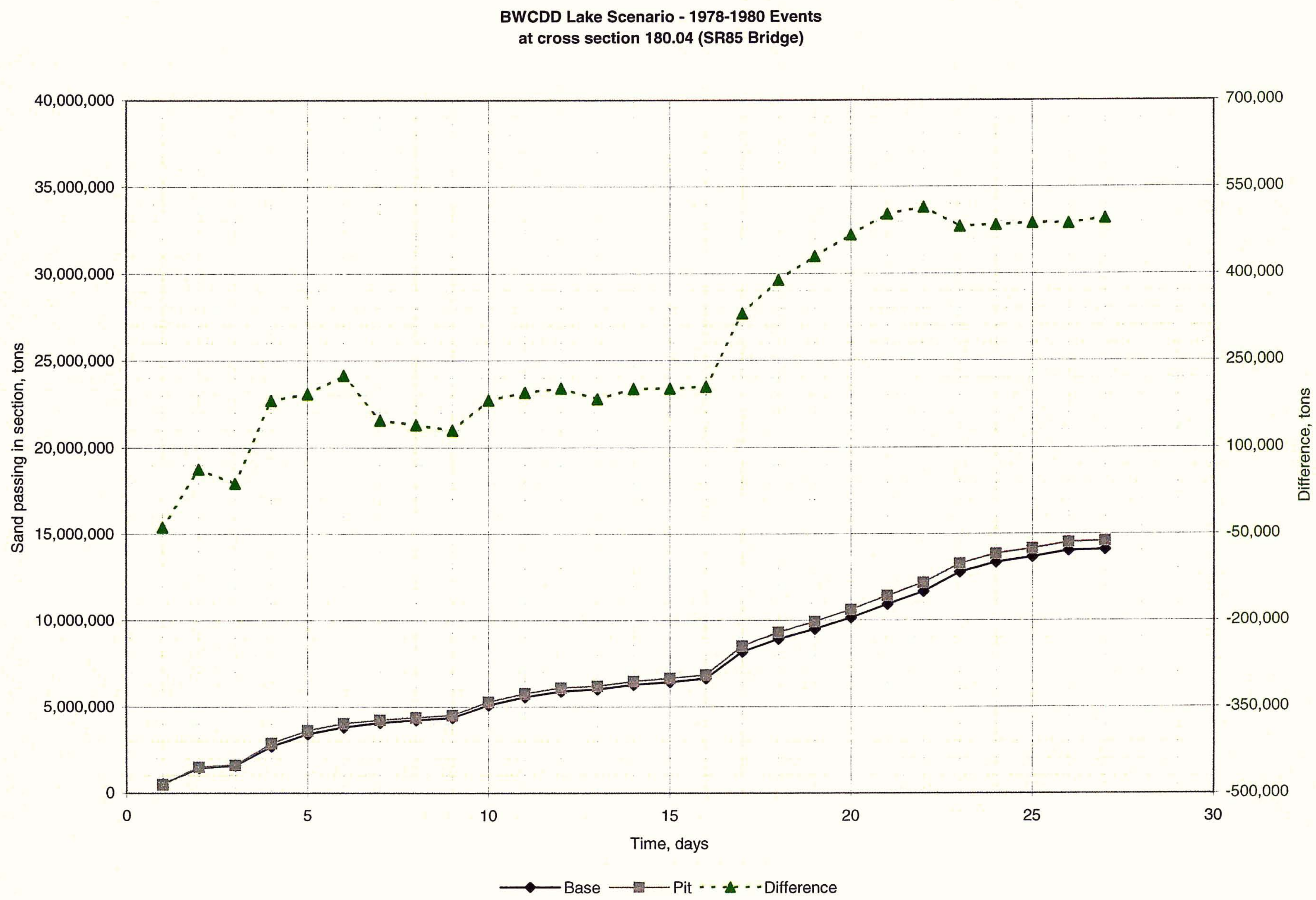
Sediment Loads Passing The SR85 Bridge From The BWCDD Lake Models

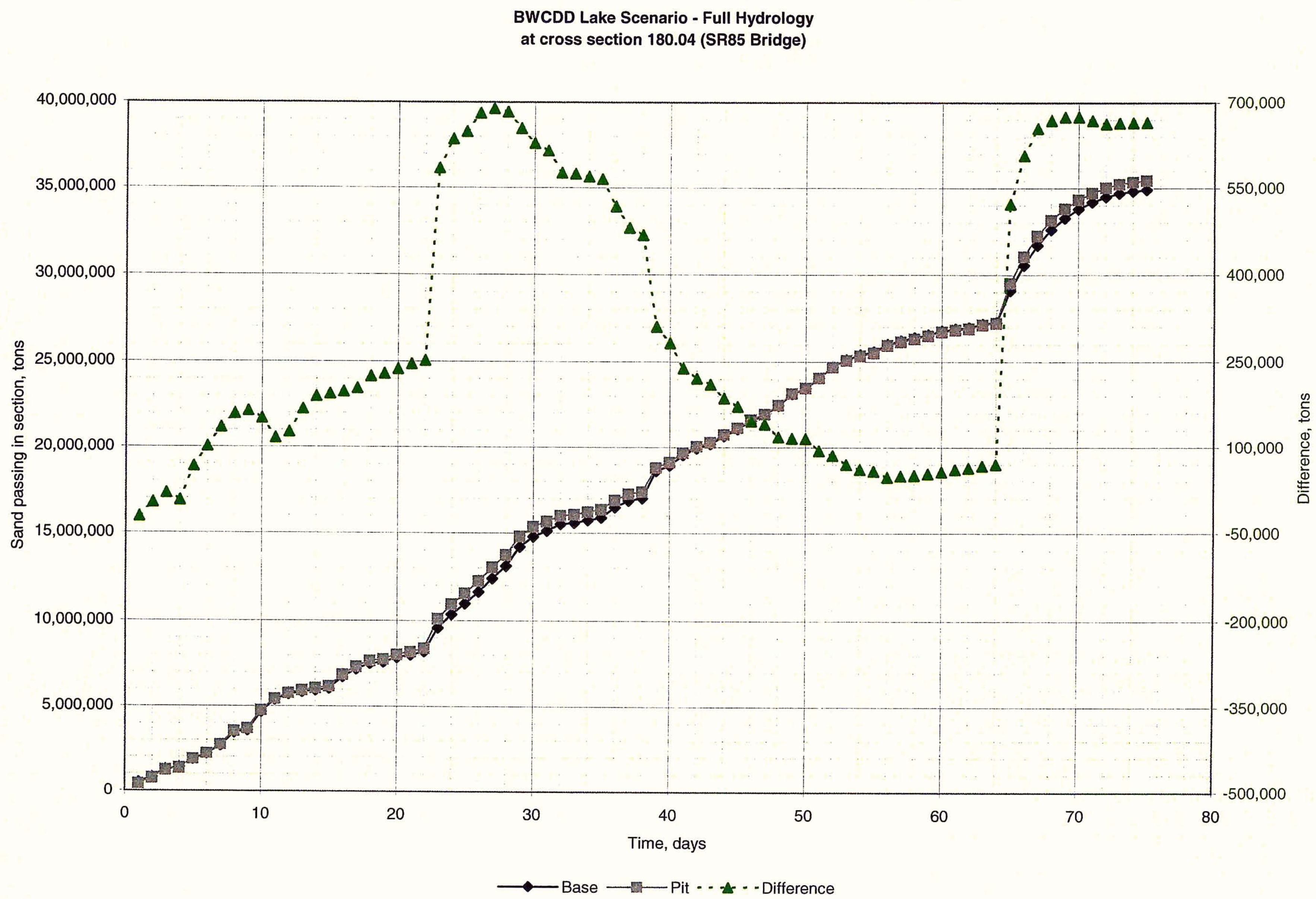
BWCDD Lake Scenario - 1993 Event
at cross section 180.04 (SR85 Bridge)



BWCDD Lake Scenario - 1980 Event
at cross section 180.04 (SR85 Bridge)




BWCDD Lake Scenario - 1978-1980 Events
at cross section 180.04 (SR85 Bridge)
Sand passing in section, tons
40,000,000
35,000,000
30,000,000
25,000,000
20,000,000
15,000,000
10,000,000
5,000,000
0
Difference, tons
700,000
550,000
400,000
250,000
100,000
-50,000
-200,000
-350,000
-500,000
0
5
10
15
20
25
30
Time, days
Base
Pit
Difference


BWCDD Lake Scenario - Full Hydrology
at cross section 180.04 (SR85 Bridge)
Sand passing in section, tons
40,000,000
35,000,000
30,000,000
25,000,000
20,000,000
15,000,000
10,000,000
5,000,000
0
Difference, tons
700,000
550,000
400,000
250,000
100,000
-50,000
-200,000
-350,000
-500,000
0
10
20
30
40
50
60
70
80
Time, days
Base
Pit
Difference

Attachment 6:

Pit Sedimentation Verification Using Pemberton And Lara (1971)

Pemberton and Lara Check

ID Number	Classification	Geometric Mean D	Fall Velocity V_s	Basin Length ℓ	Average Flow Velocity V	Water Depth d	$1.055\ell Vs/(Vd)$	$e^{1.055\ell Vs/(Vd)}$	Percent of material deposited over total basin length P	Load	Load for 1 day	Load for 1 day	Deposit	Deposit
		mm	ft/sec	ft	ft/sec	ft				tons/day	tons	ac-ft	ac-ft	yd^3
(1)	(2)	(3)	(4)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	Very Fine Sand	0.088	0.022	1,500	1.96	35.58	0.4992	2	39.30	397,037	397,037	196	77.0	124,283
2	Fine Sand	0.177	0.066	1,500	1.96	35.58	1.4977	4	77.64	98,649	98,649	49	37.8	61,001
3	Medium Sand	0.354	0.145	1,500	1.96	35.58	3.2904	27	96.28	17,076	17,076	8	8.1	13,095
4	Coarse Sand	0.707	0.26	1,500	1.96	35.58	5.9000	365	99.73	114,349	114,349	56	56.3	90,829
5	Very Coarse Sand	1.414	0.4	1,500	1.96	35.58	9.0770	8,752	99.99	118,752	118,752	59	58.6	94,575
6	Very Fine Gravel	2.828	0.6	1,500	1.96	35.58	13.6155	8.E+05	100.00	43,002	43,002	21	21.2	34,251
7	Fine Gravel	5.657	0.84	1,500	1.96	35.58	19.0617	2.E+08	100.00	2,390	2,390	1	1.2	1,904
8	Medium Gravel	11.314	1.2	1,500	1.96	35.58	27.2310	7.E+11	100.00	637	637	0	0.3	507
9	Coarse Gravel	22.627	1.7	1,500	1.96	35.58	38.5772	6.E+16	100.00	1,522	1,522	1	0.8	1,212
10	Very Coarse Gravel	45.255	2.4	1,500	1.96	35.58	54.4619	4.E+23	100.00	504	504	0	0.2	401
11	Small Cobbles	90.5	3.3	1,500	1.96	35.58	74.8851	3.E+32	100.00	157	157	0	0.1	125
Total													262	422,182

(1), (2) and (3) from HEC-6T manual, page F-13.
(4) from Pemberton and Lara publication, Figure 2 @ 68 °F.
(8) Obtained from HEC-6T model tut-con.T5 at the beginning of the model in the channel at cross section 187.64.
(9) At the beginning of the HEC-6T model Tut-con.T5 difference between the water surface elevation (865.98) and the original thalweg elevation (830.40) at cross section 187.73.
(12) Equation (5) from Pemberton and Lara publication, page 3.
(13) from HEC-6T model part.T5 at the end of the model at cross section 187.91 from SB-1 table.
(15) Assumed unit weight of deposited sand = 93 lb/ft^3 (from HEC-6T manual).

Results from Tuthill Pit HEC-6T model after 1 day

Section	Sediment passing section, tons				Sediment deposited in reach, in cubic yards				
	Total	Sand	Silt	Clay	Total	Accumulated	Sand	Silt	Clay
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
187.82	8,432,351	549,827	2,627,508	5,255,017	-103,603	259,654	-103,603	0	0
187.73	7,954,528	72,003	2,627,508	5,255,017	380,584	640,238	380,585	0	0
187.64	7,935,256	52,731	2,627,508	5,255,017	15,350	655,588	15,350	0	0
187.54	7,925,731	43,206	2,627,508	5,255,017	7,587	663,175	7,587	0	0
187.45	7,918,326	35,801	2,627,508	5,255,017	5,898	669,073	5,898	0	0
187.36	8,007,255	124,730	2,627,508	5,255,017	-70,832	598,241	-70,832	0	0

409,419 Total of cross sections 187.73 - 187.45

* These are results from the HEC-6T model tut-con.t5. This model includes a constant flow rate of 100,00 cfs for 10 days.

Attachment 7:

Scour Analysis Figures And Calculations

Stantec

Estimation of contraction and abutment scour
- Reference FHWA HEC-18

Contraction Scour
- Assume live-bed equation

$$Y_s = Y_1 \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{K_1} - Y_0$$

Where: Y_s ≡ Scour depth, in feet
Y_1 ≡ Average channel flow depth of the approach section, in feet
Q_1 ≡ Flow in channel of approach section, in cfs
** W_1 ≡ Channel bottom width of approach section, in feet
* K_1 ≡ Degree of sediment transport
Q_2 ≡ Flow in channel of contract section, in cfs
** W_2 ≡ Channel bottom width of contracted section, in feet
Y_0 ≡ Average flow depth of contracted channel, in feet

* K_1 is a function of the shear velocity and the fall velocity of the bed material based on the d_{50}. The d_{50} of the bed material is estimated from Figure 34 of the existing condition sediment report. K_1 is determined using the hydraulic design utility in HEC-RAS

** Bottom width is difficult to estimate consistently, therefore the top width reported in HEC-RAS is used.

Designed by: MCG 28 Oct 05 Checked by: SRM 11/17/2005

Abutment Scour
- Use the HIRE Equation

$$Y_s = 4Y_1 \left(\frac{K_1}{0.55}\right) K_2 F_r^{0.33}$$

Where: Y_s ≡ Scour depth, in feet
Y_1* ≡ Flow depth at toe of abutment taken at section just u/s of bridge, in feet
K_1 ≡ Abutment shape factor (See Table 7.1 of HEC-18)
K_2 ≡ Correction for angle of attack, θ ($K_2 = (\theta/90)^{0.13}$)
F_r* ≡ Froude number at toe of abutment $= V_1/\sqrt{gY_1}$, where V_1 is taken from the closest station to the toe station from flow distribution table for section just u/s of bridge

* Because of uncertainties associated with the topographic mapping, specifically, the presence of water in the river during the mapping, and the lack of detailed survey at the bridges, Y_1 is estimated as the average channel for the cross section just upstream of the bridge.

- The HIRE equation is used primarily because of the difficulties in estimating some of the parameters of the Froehlich equation

Designed by: Checked by:

Bullard Ave Bridge

Contraction condition illustrated in following figure. Hydraulic data is summarized in the following tables.

Contraction Scour

- River station 195.47 is selected as the approach section

Table 1
$$\left\{\begin{array}{l} Y_1 = 11.24\ ft \\ Q_1 = 221{,}529\ cfs \\ W_1 = 2{,}405\ ft \\ d_{50} = 0.45\ mm\ (\text{sample } \#3 \,\&\, 4) \\ K_1 = 0.69 \end{array}\right.$$

$$\left.\begin{array}{l} Q_2 = 227{,}000\ cfs^{*} \\ W_2 = 1{,}713\ ft \\ Y_0 = 13.06\ ft \end{array}\right\}$$
Table 3

$$Y_s = Y_1 \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{K_1} - Y_0$$

$$= 11.24 \left(\frac{221{,}000}{221{,}529}\right)^{6/7} \left(\frac{2{,}405}{1{,}713}\right)^{0.69} - 13.06$$

$$\underline{Y_s = 1.4'}$$

* The full discharge is assumed; flow will be contained in the main channel until Vinyard Rd. overtops. The model does not treat the road as a levee.

Abutment Scour

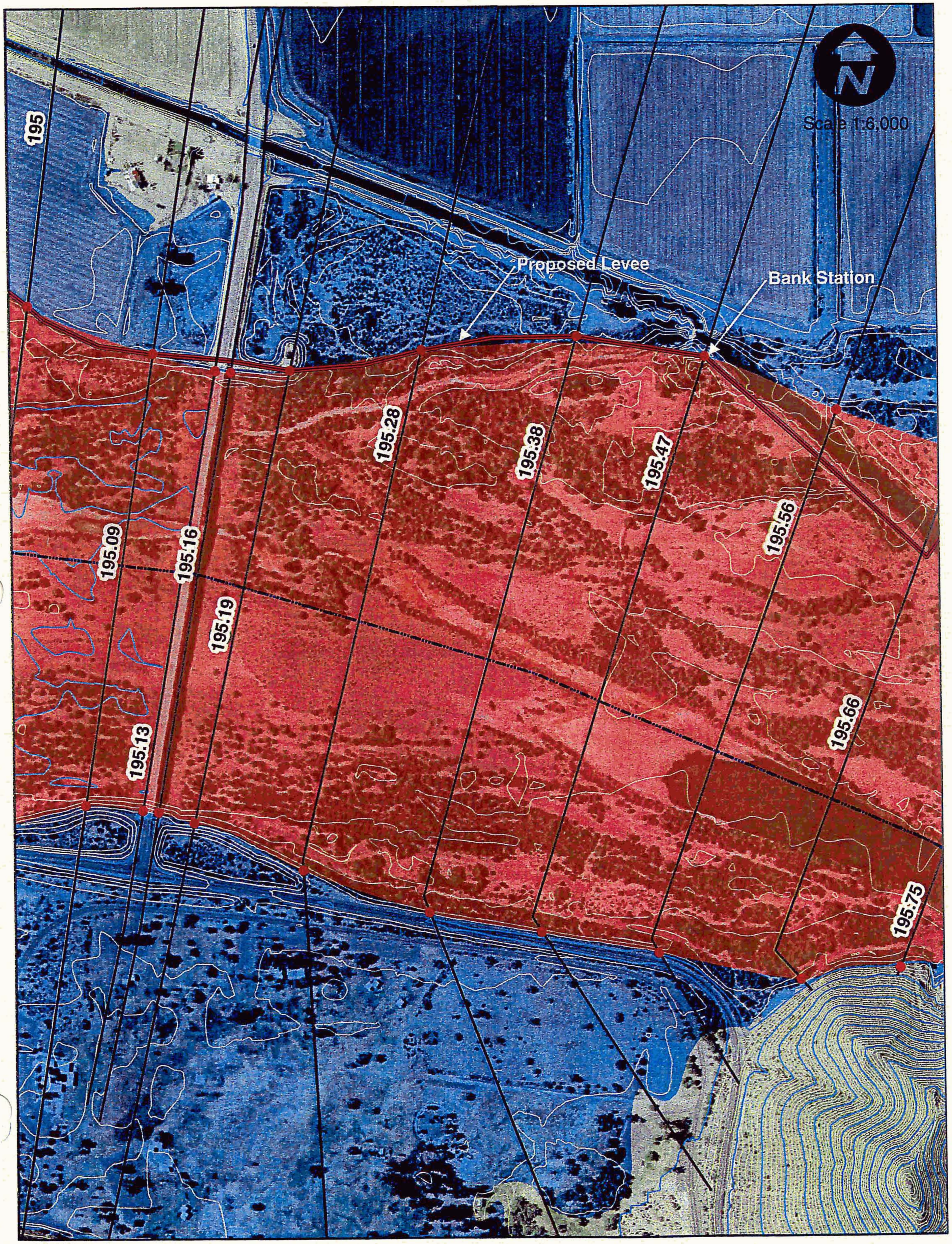
Table 2 $\begin{cases} Y_1 = 13.18^{ft} \\ K_1 = 0.55 \text{ (assume spill through abutment)} \\ K_2 = 1 \\ V_1 = 9.5^{fps} \end{cases}$

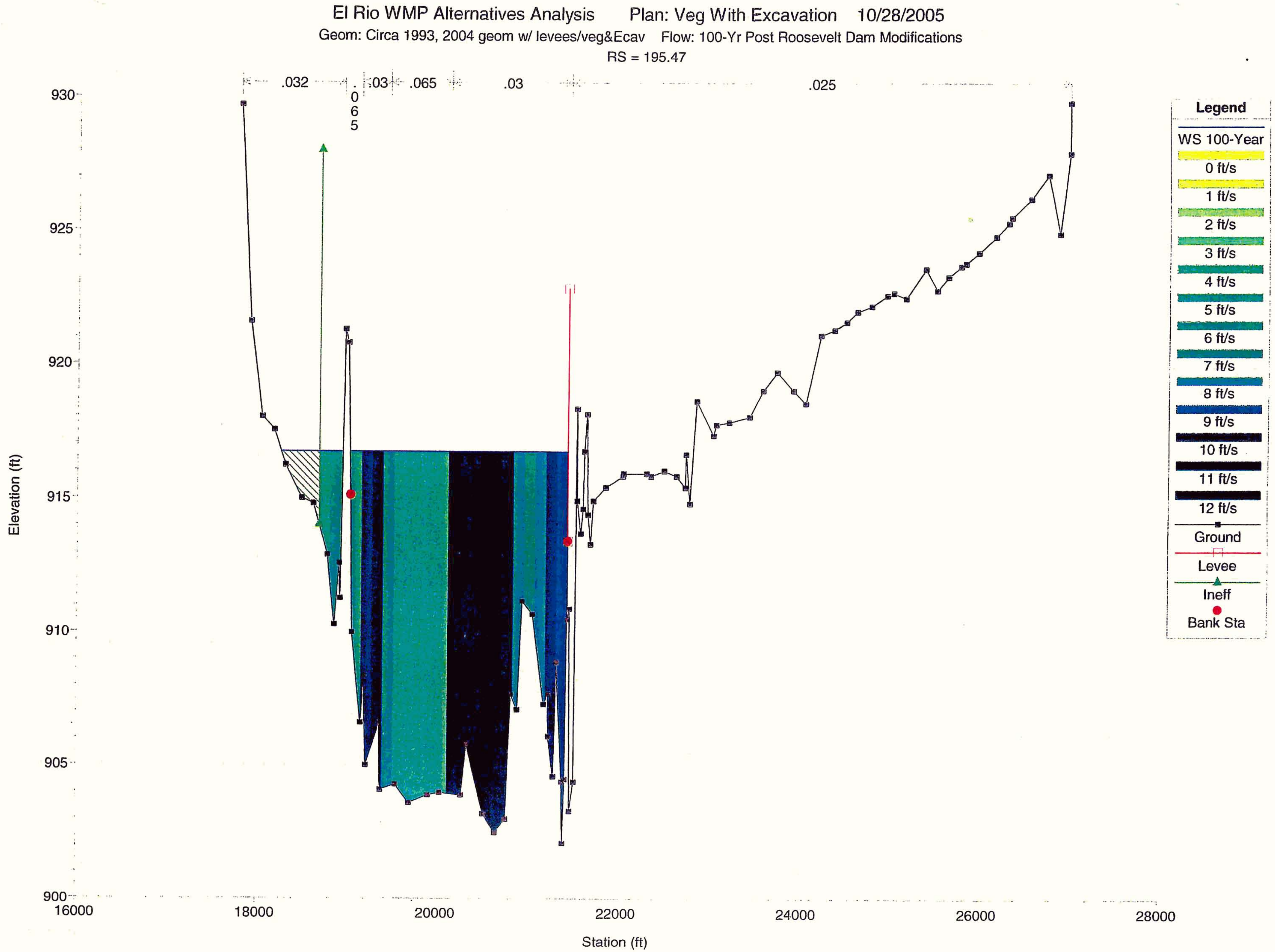
$$Y_s = 4Y_1\left(\frac{K_1}{0.55}\right)K_2 Fr^{0.33}$$

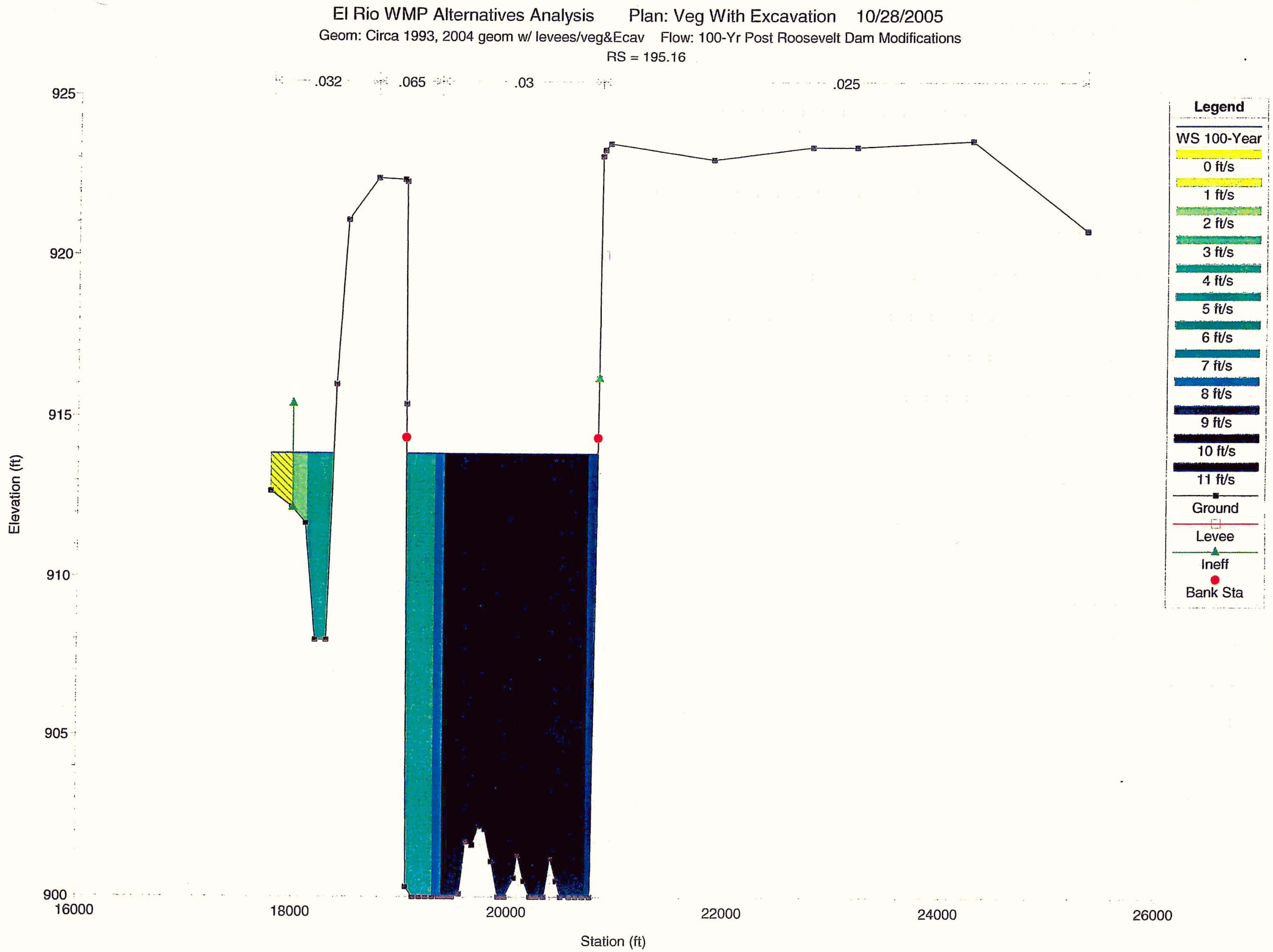
$$= (4)(13.18)\left(\frac{0.55}{0.55}\right)(1)\left(\frac{9.5}{\sqrt{(32.2)(13.18)}}\right)^{0.33}$$

$\underline{Y_s = 40.8^{ft}}$

Bullard Ave Bridge




El Rio WMP Alternatives Analysis Plan: Veg With Excavation 10/28/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 195.47
.032
.065
.03
.065
.03
.025
Elevation (ft)
930
925
920
915
910
905
900
16000
18000
20000
22000
24000
26000
28000
Station (ft)
Legend
WS 100-Year
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
10 ft/s
11 ft/s
12 ft/s
Ground
Levee
Ineff
Bank Sta



El Rio WMP Alternatives Analysis Plan: Veg With Excavation 10/28/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 195.16
.032
.065
.03
.025
925
920
915
910
905
900
Elevation (ft)
16000
18000
20000
22000
24000
26000
Station (ft)
Legend
WS 100-Year
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
10 ft/s
11 ft/s
Ground
Levee
Ineff
Bank Sta

Plan: Excav Gila River El Rio WMP RS: 195.47 Profile: 100-Year

Table 1

E.G. Elev (ft)	917.72	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.03	Wt. n-Val.	0.032	0.037	
W.S. Elev (ft)	916.70	Reach Len. (ft)	485.00	472.11	530.00
Crit W.S. (ft)	912.00	Flow Area (sq ft)	1100.83	27022.93	
E.G. Slope (ft/ft)	0.001675	Area (sq ft)	1688.12	27022.93	
Q Total (cfs)	227000.00	Flow (cfs)	5471.43	221528.60	
Top Width (ft)	3095.98	Top Width (ft)	691.07	2404.90	
Vel Total (ft/s)	8.07	Avg. Vel. (ft/s)	4.97	8.20	
Max Chl Dpth (ft)	14.60	Hydr. Depth (ft)	4.09	11.24	
Conv. Total (cfs)	5546668.0	Conv. (cfs)	133692.6	5412976.0	
Length Wtd. (ft)	472.52	Wetted Per. (ft)	270.37	2413.18	
Min Ch El (ft)	902.10	Shear (lb/sq ft)	0.43	1.17	
Alpha	1.02	Stream Power (lb/ft s)	2.12	9.60	
Frctn Loss (ft)	0.65	Cum Volume (acre-ft)	2145.98	71008.43	9471.52
C & E Loss (ft)	0.04	Cum SA (acres)	723.36	7046.75	1687.73

Plan: Excav Gila River El Rio WMP RS: 195.16 Profile: 100-Year

Table 2

E.G. Elev (ft)	915.24	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.37	Wt. n-Val.	0.032	0.033	
W.S. Elev (ft)	913.88	Reach Len. (ft)	1.00	1.00	1.00
Crit W.S. (ft)	908.68	Flow Area (sq ft)	1382.59	23325.58	
E.G. Slope (ft/ft)	0.001440	Area (sq ft)	1672.46	23325.58	
Q Total (cfs)	227000.00	Flow (cfs)	5794.27	221205.70	
Top Width (ft)	2347.63	Top Width (ft)	578.53	1769.10	
Vel Total (ft/s)	9.19	Avg. Vel. (ft/s)	4.19	9.48	
Max Chl Dpth (ft)	13.88	Hydr. Depth (ft)	3.67	13.18	
Conv. Total (cfs)	5982625.0	Conv. (cfs)	152709.0	5829916.0	
Length Wtd. (ft)	1.00	Wetted Per. (ft)	376.88	1775.36	
Min Ch El (ft)	900.00	Shear (lb/sq ft)	0.33	1.18	
Alpha	1.04	Stream Power (lb/ft s)	1.38	11.20	
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	1986.43	70025.49	9471.52
C & E Loss (ft)	0.01	Cum SA (acres)	660.01	6963.98	1687.73

Plan: Excav Gila River El Rio WMP RS: 195.145 BR U Profile: 100-Year

Table 3

E.G. Elev (ft)	915.23	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.47	Wt. n-Val.	0.032	0.033	
W.S. Elev (ft)	913.76	Reach Len. (ft)	62.20	62.20	62.20
Crit W.S. (ft)	908.88	Flow Area (sq ft)	1337.65	22370.89	
E.G. Slope (ft/ft)	0.002053	Area (sq ft)	1603.36	22370.89	
Q Total (cfs)	227000.00	Flow (cfs)	6562.51	220437.50	
Top Width (ft)	2289.80	Top Width (ft)	577.26	1712.54	
Vel Total (ft/s)	9.57	Avg. Vel. (ft/s)	4.91	9.85	
Max Chl Dpth (ft)	13.76	Hydr. Depth (ft)	3.56	13.06	
Conv. Total (cfs)	5010521.0	Conv. (cfs)	144852.9	4865668.0	
Length Wtd. (ft)	62.20	Wetted Per. (ft)	375.60	2090.34	
Min Ch El (ft)	900.00	Shear (lb/sq ft)	0.46	1.37	
Alpha	1.04	Stream Power (lb/ft s)	2.24	13.51	
Frctn Loss (ft)	0.13	Cum Volume (acre-ft)	1986.39	70024.97	9471.52
C & E Loss (ft)	0.00	Cum SA (acres)	659.99	6963.94	1687.73

Plan: Excav Gila River El Rio WMP RS: 195.47 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	18172.80	18299.60	0.00	8.68	41.23	0.00	0.21	0.00
2	LOB	18299.60	18426.40	0.00	110.16	126.80	0.00	0.87	0.00
3	LOB	18426.40	18553.20	0.00	206.22	126.80	0.00	1.63	0.00
4	LOB	18553.20	18680.00	0.00	262.21	126.81	0.00	2.07	0.00
5	LOB	18680.00	18796.70	1659.56	396.25	116.72	0.73	3.40	4.19
6	LOB	18796.70	18913.40	3476.10	617.74	116.83	1.53	5.29	5.63
7	LOB	18913.40	19030.10	335.77	86.84	36.82	0.15	2.41	3.87
8	Chan	19030.10	19150.34	4467.25	924.69	120.93	1.97	7.69	4.83
9	Chan	19150.34	19270.59	11933.46	1256.80	120.51	5.26	10.45	9.50
10	Chan	19270.59	19390.83	12881.23	1315.46	120.43	5.67	10.94	9.79
11	Chan	19390.83	19511.08	11277.92	1503.55	120.25	4.97	12.50	7.50
12	Chan	19511.08	19631.32	7494.44	1510.60	120.25	3.30	12.56	4.96
13	Chan	19631.32	19751.57	7938.38	1563.67	120.25	3.50	13.00	5.08
14	Chan	19751.57	19871.81	7859.33	1554.30	120.24	3.46	12.93	5.06
15	Chan	19871.81	19992.06	7707.44	1536.22	120.25	3.40	12.78	5.02
16	Chan	19992.06	20112.30	7644.62	1528.68	120.24	3.37	12.71	5.00
17	Chan	20112.30	20232.55	15876.57	1533.92	120.25	6.99	12.76	10.35
18	Chan	20232.55	20352.79	14958.93	1438.18	120.28	6.59	11.96	10.40
19	Chan	20352.79	20473.04	15195.54	1451.69	120.26	6.69	12.07	10.47
20	Chan	20473.04	20593.29	18379.97	1627.22	120.25	8.10	13.53	11.30
21	Chan	20593.29	20713.53	19569.23	1689.56	120.25	8.62	14.05	11.58
22	Chan	20713.53	20833.78	14864.54	1433.53	120.44	6.55	11.92	10.37
23	Chan	20833.78	20954.02	7658.07	962.83	120.42	3.37	8.01	7.95
24	Chan	20954.02	21074.27	4574.06	706.36	120.25	2.02	5.87	6.48
25	Chan	21074.27	21194.51	7872.61	978.49	120.28	3.47	8.14	8.05
26	Chan	21194.51	21314.76	11835.14	1252.77	121.03	5.21	10.42	9.45
27	Chan	21314.76	21435.00	11539.84	1254.42	126.13	5.08	10.43	9.20

Plan: Excav Gila River El Rio WMP RS: 195.145 BR U Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	17788.00	17889.00	0.00	265.71	203.06	0.00	2.63	0.00
2	LOB	17889.00	17990.00	0.00	0.00	0.00	0.00		0.00
[illegible]	LOB	17990.00	18121.27	724.02	244.08	131.28	0.32	1.86	2.97
4	LOB	18121.27	18252.54	3429.83	620.76	131.34	1.51	4.73	5.53
5	LOB	18252.54	18383.81	2408.65	472.80	112.99	1.06	4.19	5.09
6	LOB	18383.81	18515.09						
7	LOB	18515.09	18646.36						
8	LOB	18646.36	18777.63						
9	LOB	18777.63	18908.90						
10	LOB	18908.90	19040.17						
11	Chan	19040.17	19128.74	4936.56	1024.26	103.47	2.17	11.95	4.82
12	Chan	19128.74	19217.31	6473.29	1190.03	100.26	2.85	13.76	5.44
13	Chan	19217.31	19305.88	2179.80	443.67	43.55	0.96	5.25	4.91
14	Chan	19305.88	19394.45	9222.79	1163.35	112.08	4.06	13.76	7.93
15	Chan	19394.45	19483.02	15844.43	1218.40	88.57	6.98	13.76	13.00
16	Chan	19483.02	19571.58	12372.68	1154.12	112.09	5.45	13.65	10.72
17	Chan	19571.58	19660.15	10647.85	1041.82	108.70	4.69	12.32	10.22
18	Chan	19660.15	19748.72	10051.04	1002.70	107.71	4.43	11.86	10.02
19	Chan	19748.72	19837.29	12325.06	1047.96	88.58	5.43	11.83	11.76
20	Chan	19837.29	19925.86	11786.82	1112.18	109.89	5.19	13.15	10.60
21	Chan	19925.86	20014.43	12442.80	1158.02	112.08	5.48	13.69	10.74
22	Chan	20014.43	20103.00	11663.91	1103.58	109.49	5.14	13.05	10.57
23	Chan	20103.00	20191.57	14617.14	1160.89	88.58	6.44	13.11	12.59
24	Chan	20191.57	20280.14	12503.42	1161.39	112.08	5.51	13.73	10.77
25	Chan	20280.14	20368.71	12363.58	1153.61	112.09	5.45	13.64	10.72
26	Chan	20368.71	20457.28	12519.75	1106.73	99.16	5.52	12.88	11.31
27	Chan	20457.28	20545.84	14014.51	1190.61	100.50	6.17	13.65	11.77
[illegible]	Chan	20545.84	20634.41	12538.54	1163.35	112.08	5.52	13.76	10.78
[illegible]	Chan	20634.41	20722.98	12538.88	1163.37	112.08	5.52	13.76	10.78
30	Chan	20722.98	20811.55	9394.70	891.18	88.76	4.14	10.29	10.54

Plan: Excav Gila River El Rio WMP RS: 195.16 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	17788.00	17889.00	0.00	131.84	102.18	0.00	1.31	0.00
2	LOB	17889.00	17990.00	0.00	158.03	101.00	0.00	1.56	0.00
[illegible]	LOB	17990.00	18121.27	676.03	259.78	131.28	0.30	1.98	2.60
4	LOB	18121.27	18252.54	3009.21	636.45	131.34	1.33	4.85	4.73
5	LOB	18252.54	18383.81	2109.02	486.36	114.26	0.93	4.27	4.34
6	LOB	18383.81	18515.09						
7	LOB	18515.09	18646.36						
8	LOB	18646.36	18777.63						
9	LOB	18777.63	18908.90						
10	LOB	18908.90	19040.17						
11	Chan	19040.17	19128.74	4689.39	1061.41	91.87	2.07	12.09	4.42
12	Chan	19128.74	19217.31	6135.23	1228.99	88.57	2.70	13.88	4.99
13	Chan	19217.31	19305.88	6135.10	1228.96	88.57	2.70	13.88	4.99
14	Chan	19305.88	19394.45	10193.98	1228.96	88.57	4.49	13.88	8.29
15	Chan	19394.45	19483.02	13293.01	1228.99	88.57	5.86	13.88	10.82
16	Chan	19483.02	19571.58	13126.05	1219.73	88.58	5.78	13.77	10.76
17	Chan	19571.58	19660.15	11059.50	1100.63	88.58	4.87	12.43	10.05
18	Chan	19660.15	19748.72	10380.99	1059.55	88.57	4.57	11.96	9.80
19	Chan	19748.72	19837.29	10364.52	1058.55	88.58	4.57	11.95	9.79
20	Chan	19837.29	19925.86	12305.05	1173.39	88.58	5.42	13.25	10.49
21	Chan	19925.86	20014.43	13196.56	1223.64	88.57	5.81	13.82	10.78
22	Chan	20014.43	20103.00	12141.51	1164.00	88.58	5.35	13.14	10.43
23	Chan	20103.00	20191.57	12272.07	1171.48	88.58	5.41	13.23	10.48
24	Chan	20191.57	20280.14	13257.45	1227.01	88.57	5.84	13.85	10.80
25	Chan	20280.14	20368.71	13116.94	1219.22	88.57	5.78	13.77	10.76
26	Chan	20368.71	20457.28	11947.08	1152.79	88.58	5.26	13.02	10.36
27	Chan	20457.28	20545.84	13108.80	1218.75	88.57	5.77	13.76	10.76
[illegible]	Chan	20545.84	20634.41	13292.71	1228.96	88.57	5.86	13.88	10.82
[illegible]	Chan	20634.41	20722.98	13293.01	1228.99	88.57	5.86	13.88	10.82
30	Chan	20722.98	20811.55	7896.79	901.57	89.16	3.48	10.36	8.76

Estrella Parkway Bridge

Contraction condition illustrated in the following figure. Hydraulic data is summarized in the following tables.

Contraction Scour

- River station 194.53 is selected as the approach section.

Table 4

$Y_1 = 10.14$ ft
$Q_1 = 210,282$ cfs
$W_1 = 3,052$ ft
$d_{50} = 0.76$ mm (Samples 3, 4 + 11)
$K_1 = 0.69$

$Q_2 = 227,000$ cfs
$W_2 = 2,034$ ft
$Y_0 = 10.41$ ft

Table 6

$$Y_s = Y_1 \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{K_1} - Y_0$$

$$= 10.14 \left(\frac{227,000}{210,282}\right)^{6/7} \left(\frac{3,052}{2,034}\right)^{0.69} - 10.41$$

$Y_s = 3.9'$

Abutment Scour

Table 5

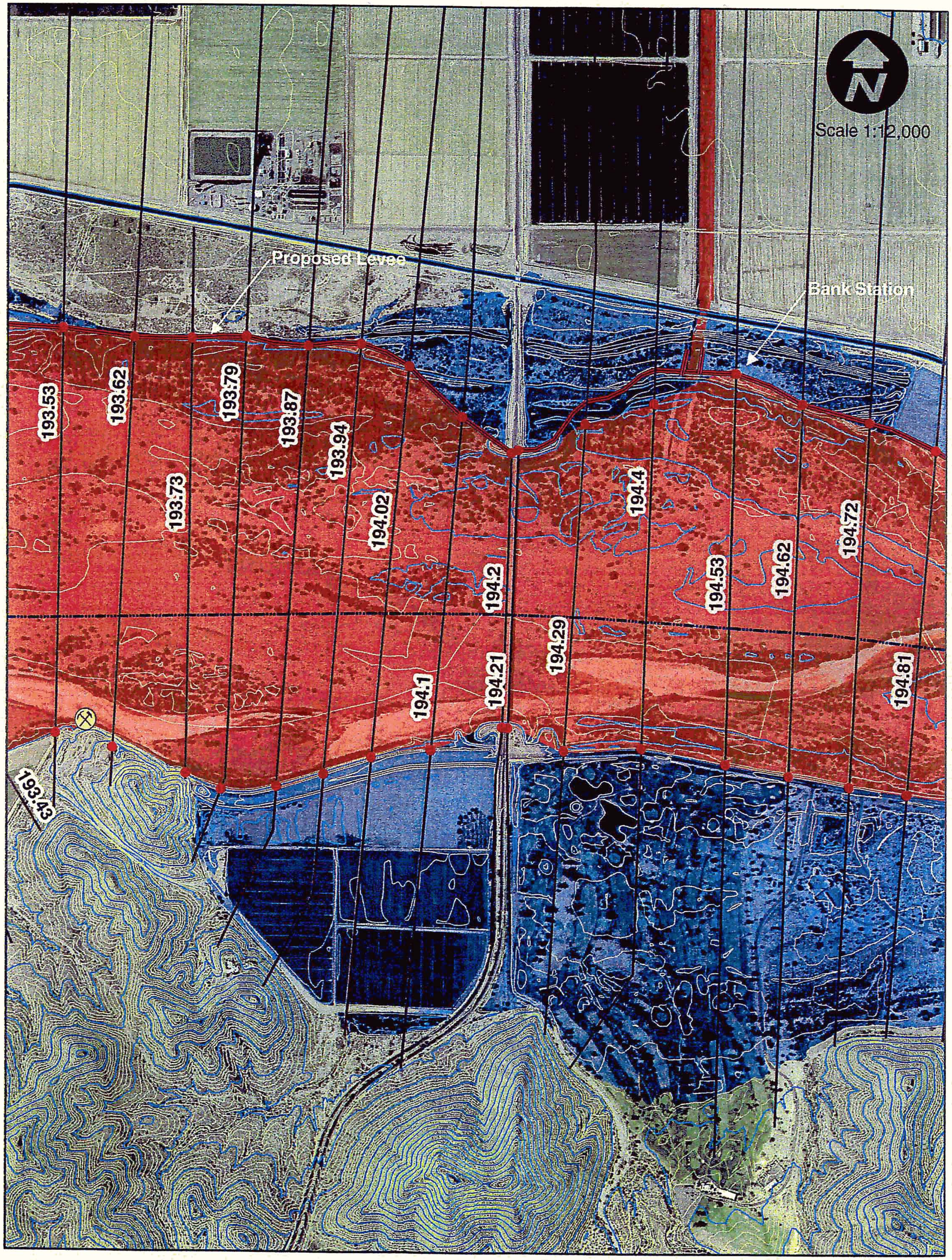
$$\begin{cases} y_1 = 10.72^{ft} \\ K_1 = 0.55 \text{ (Spill through)} \\ K_2 = 1.0 \\ V_1 = 10.0^{fps} \end{cases}$$

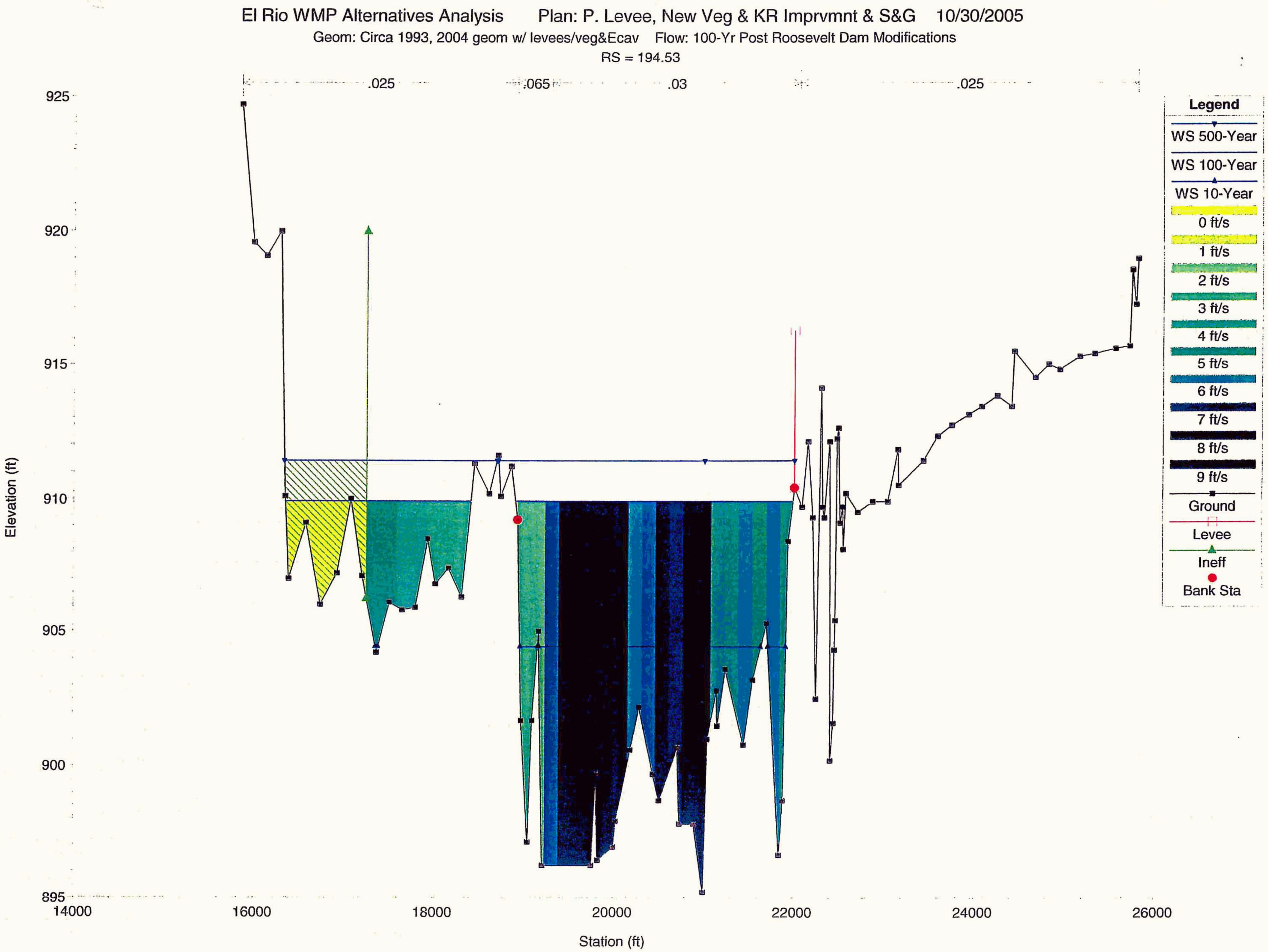
$$Y_s = 4y_1 \left(\frac{K_1}{0.55}\right) K_2 F_r^{0.33}$$

$$= 4(10.72)\left(\frac{0.55}{0.55}\right)(1)\left(\frac{10.0}{\sqrt{(32.2)(10.72)}}\right)^{0.33}$$

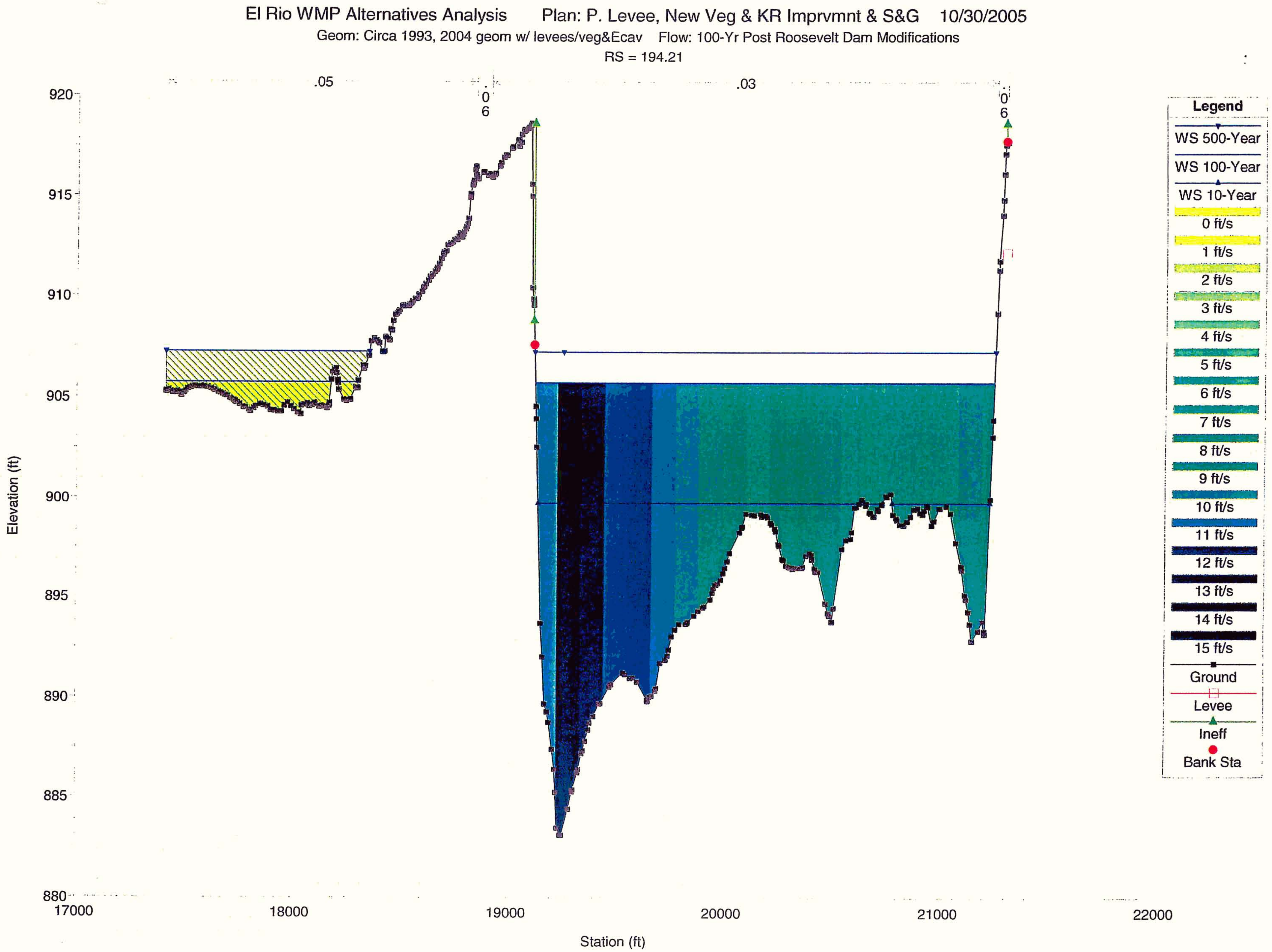
$$\underline{Y_s = 35.0^{ft}}$$

Estrella Pkwy Bridge

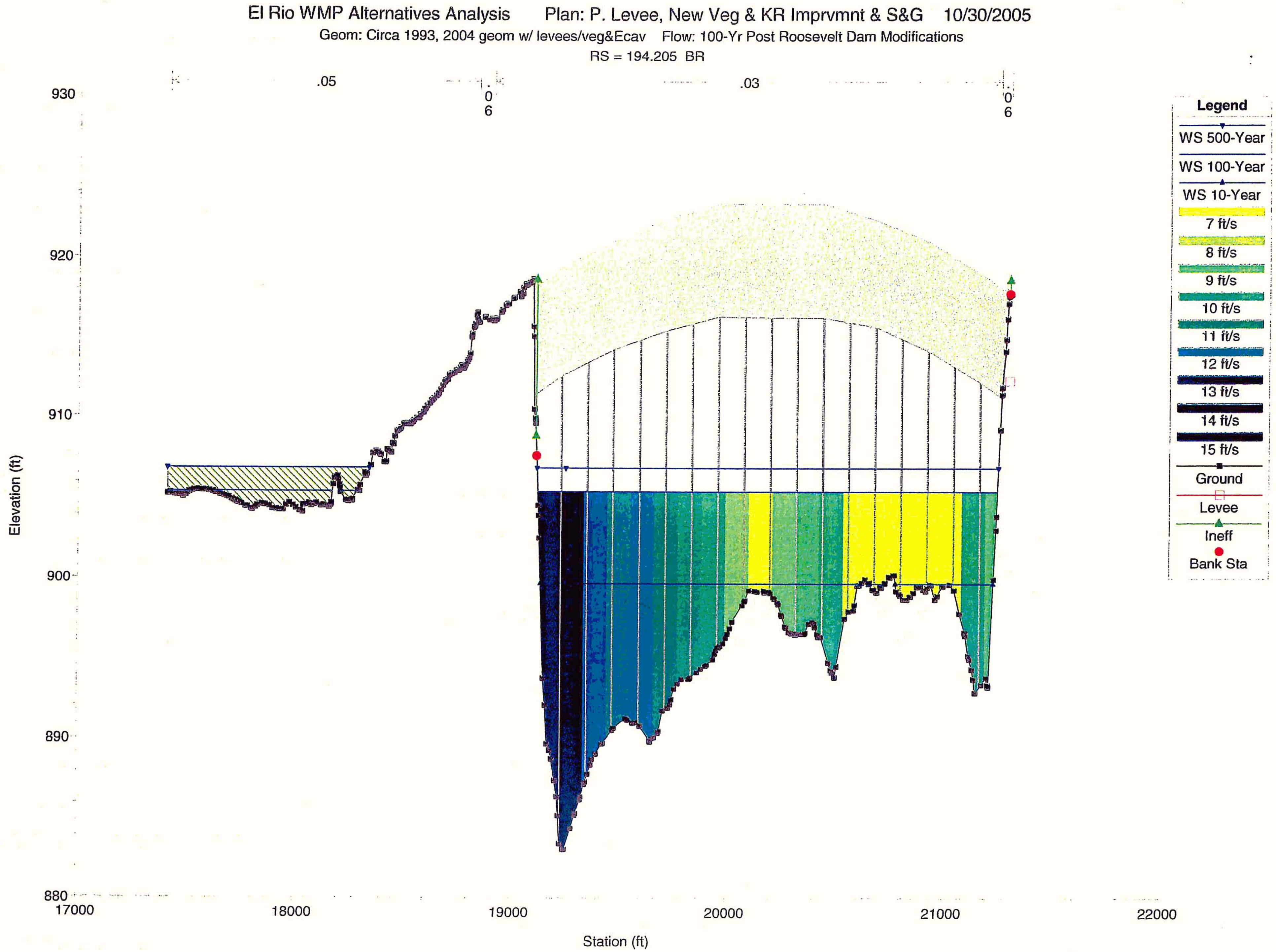




El Rio WMP Alternatives Analysis Plan: P. Levee, New Veg & KR Imprvmnt & S&G 10/30/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 194.53
.025
.065
.03
.025
Legend
WS 500-Year
WS 100-Year
WS 10-Year
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
925
920
915
910
905
900
895
Station (ft)
14000
16000
18000
20000
22000
24000
26000



El Rio WMP Alternatives Analysis   Plan: P. Levee, New Veg & KR Imprvmnt & S&G   10/30/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav   Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 194.21
.05
.03
Elevation (ft)
Station (ft)
920
915
910
905
900
895
890
885
880
17000
18000
19000
20000
21000
22000
Legend
WS 500-Year
WS 100-Year
WS 10-Year
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
10 ft/s
11 ft/s
12 ft/s
13 ft/s
14 ft/s
15 ft/s
Ground
Levee
Ineff
Bank Sta


El Rio WMP Alternatives Analysis Plan: P. Levee, New Veg & KR Imprvmnt & S&G 10/30/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 194.205 BR
.05
.03
Elevation (ft)
Station (ft)
930
920
910
900
890
880
17000
18000
19000
20000
21000
22000
Legend
WS 500-Year
WS 100-Year
WS 10-Year
7 ft/s
8 ft/s
9 ft/s
10 ft/s
11 ft/s
12 ft/s
13 ft/s
14 ft/s
15 ft/s
Ground
Levee
Ineff
Bank Sta

Plan: Excav Gila River El Rio WMP RS: 194.53 Profile: 100-Year

Table 4

E.G. Elev (ft)	910.60	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.68	Wt. n-Val.	0.025	0.032	
W.S. Elev (ft)	909.92	Reach Len. (ft)	475.00	661.08	630.00
Crit W.S. (ft)	905.21	Flow Area (sq ft)	3986.28	30953.19	
E.G. Slope (ft/ft)	0.000967	Area (sq ft)	5922.05	30953.19	
Q Total (cfs)	227000.00	Flow (cfs)	16718.36	210281.70	
Top Width (ft)	5129.90	Top Width (ft)	2078.36	3051.54	
Vel Total (ft/s)	6.50	Avg. Vel. (ft/s)	4.19	6.79	
Max Chl Dpth (ft)	14.72	Hydr. Depth (ft)	3.36	10.14	
Conv. Total (cfs)	7300549.0	Conv. (cfs)	537679.2	6762870.0	
Length Wtd. (ft)	650.15	Wetted Per. (ft)	1185.57	3056.13	
Min Ch El (ft)	895.20	Shear (lb/sq ft)	0.20	0.61	
Alpha	1.04	Stream Power (lb/ft s)	0.85	4.15	
Frctn Loss (ft)	0.71	Cum Volume (acre-ft)	1610.84	67956.12	9471.52
C & E Loss (ft)	0.05	Cum SA (acres)	543.12	6775.39	1687.73

Plan: Excav Gila River El Rio WMP RS: 194.21 Profile: 100-Year

Table 5

E.G. Elev (ft)	907.28	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.55	Wt. n-Val.		0.030	
W.S. Elev (ft)	905.73	Reach Len. (ft)	0.01	0.01	0.01
Crit W.S. (ft)	902.05	Flow Area (sq ft)		22731.65	
E.G. Slope (ft/ft)	0.001727	Area (sq ft)	728.63	22731.65	
Q Total (cfs)	227000.00	Flow (cfs)		227000.00	
Top Width (ft)	2974.36	Top Width (ft)	853.81	2120.55	
Vel Total (ft/s)	9.99	Avg. Vel. (ft/s)		9.99	
Max Chl Dpth (ft)	22.63	Hydr. Depth (ft)		10.72	
Conv. Total (cfs)	5463073.0	Conv. (cfs)		5463073.0	
Length Wtd. (ft)	0.01	Wetted Per. (ft)		2126.83	
Min Ch El (ft)	883.10	Shear (lb/sq ft)		1.15	
Alpha	1.00	Stream Power (lb/ft s)		11.50	
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	1452.88	66925.19	9444.31
C & E Loss (ft)	0.07	Cum SA (acres)	474.50	6668.24	1681.96

Plan: Excav Gila River El Rio WMP RS: 194.205 BR U Profile: 100-Year

Table 6

E.G. Elev (ft)	907.21	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.78	Wt. n-Val.		0.030	
W.S. Elev (ft)	905.42	Reach Len. (ft)	34.52	34.52	34.52
Crit W.S. (ft)	902.27	Flow Area (sq ft)		21178.28	
E.G. Slope (ft/ft)	0.002569	Area (sq ft)		21178.28	
Q Total (cfs)	227000.00	Flow (cfs)		227000.00	
Top Width (ft)	2033.72	Top Width (ft)		2033.72	
Vel Total (ft/s)	10.72	Avg. Vel. (ft/s)		10.72	
Max Chl Dpth (ft)	22.32	Hydr. Depth (ft)		10.41	
Conv. Total (cfs)	4478791.0	Conv. (cfs)		4478791.0	
Length Wtd. (ft)	34.52	Wetted Per. (ft)		2400.47	
Min Ch El (ft)	883.10	Shear (lb/sq ft)		1.41	
Alpha	1.00	Stream Power (lb/ft s)		15.17	
Frctn Loss (ft)	0.09	Cum Volume (acre-ft)	1452.88	66925.18	9444.31
C & E Loss (ft)	0.00	Cum SA (acres)	474.50	6668.24	1681.96

Plan: Excav Gila River El Rio WMP RS: 194.53 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	16132.48	16411.86	0.00	125.35	63.19	0.00	1.99	0.00
2	LOB	16411.86	16691.24	0.00	505.65	279.41	0.00	1.81	0.00
[illegible]	LOB	16691.24	16970.62	0.00	883.01	279.40	0.00	3.16	0.00
4	LOB	16970.62	17250.00	0.00	421.76	271.16	0.00	1.56	0.00
5	LOB	17250.00	17585.78	7477.81	1524.07	335.81	3.29	4.54	4.91
6	LOB	17585.78	17921.56	5045.68	1203.62	335.80	2.22	3.58	4.19
7	LOB	17921.56	18257.34	3053.38	890.44	335.80	1.35	2.65	3.43
8	LOB	18257.34	18593.12	1137.63	358.91	152.30	0.50	2.36	3.17
9	LOB	18593.12	18928.90	3.86	9.25	25.85	0.00	0.36	0.42
10	Chan	18928.90	19082.35	3953.27	1354.53	154.37	1.74	8.83	2.92
11	Chan	19082.35	19235.80	3799.25	1322.76	154.41	1.67	8.62	2.87
12	Chan	19235.80	19389.25	14222.18	2104.67	153.45	6.27	13.72	6.76
13	Chan	19389.25	19542.70	17926.04	2104.70	153.45	7.90	13.72	8.52
14	Chan	19542.70	19696.15	17925.82	2104.67	153.45	7.90	13.72	8.52
15	Chan	19696.15	19849.60	16058.23	1972.21	153.84	7.07	12.85	8.14
16	Chan	19849.60	20003.05	16871.23	2029.51	153.46	7.43	13.23	8.31
17	Chan	20003.05	20156.50	12645.25	1707.23	153.48	5.57	11.13	7.41
18	Chan	20156.50	20309.95	8251.01	1321.38	153.47	3.63	8.61	6.24
19	Chan	20309.95	20463.40	9446.98	1433.16	153.47	4.16	9.34	6.59
20	Chan	20463.40	20616.85	11940.91	1649.42	153.46	5.26	10.75	7.24
21	Chan	20616.85	20770.30	11196.72	1587.58	153.61	4.93	10.35	7.05
22	Chan	20770.30	20923.75	14691.62	1867.88	153.46	6.47	12.17	7.87
23	Chan	20923.75	21077.20	14391.52	1847.15	153.93	6.34	12.04	7.79
24	Chan	21077.20	21230.65	6675.86	1164.10	153.62	2.94	7.59	5.73
25	Chan	21230.65	21384.10	6197.08	1112.83	153.47	2.73	7.25	5.57
26	Chan	21384.10	21537.55	7673.83	1265.16	153.49	3.38	8.24	6.07
27	Chan	21537.55	21691.00	4151.70	875.08	153.46	1.83	5.70	4.74
[illegible]	Chan	21691.00	21844.45	8666.76	1361.74	153.70	3.82	8.87	6.36
[illegible]	Chan	21844.45	21997.90	3596.38	767.41	137.09	1.58	5.64	4.69

Plan: Excav Gila River El Rio WMP RS: 194.21 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	17414.00	17583.88	0.00	64.75	170.35	0.00	0.38	0.00
2	LOB	17583.88	17753.76	0.00	97.82	169.90	0.00	0.58	0.00
[illegible]	LOB	17753.76	17923.64	0.00	210.83	169.89	0.00	1.24	0.00
4	LOB	17923.64	18093.52	0.00	209.78	169.92	0.00	1.23	0.00
5	LOB	18093.52	18263.40	0.00	129.64	138.31	0.00	0.94	0.00
6	LOB	18263.40	18433.28	0.00	15.81	36.11	0.00	0.44	0.00
7	LOB	18433.28	18603.16						
8	LOB	18603.16	18773.04						
9	LOB	18773.04	18942.92						
10	LOB	18942.92	19112.80						
11	LOB	19112.80	19116.12						
12	Chan	19116.12	19225.04	15106.86	1414.84	106.78	6.66	13.64	10.68
13	Chan	19225.04	19333.96	33509.98	2302.52	109.20	14.76	21.14	14.55
14	Chan	19333.96	19442.88	23299.54	1849.91	108.98	10.26	16.98	12.59
15	Chan	19442.88	19551.80	18739.32	1622.99	108.93	8.26	14.90	11.55
16	Chan	19551.80	19660.73	19253.27	1649.60	108.94	8.48	15.14	11.67
17	Chan	19660.73	19769.65	17029.17	1532.81	109.00	7.50	14.07	11.11
18	Chan	19769.65	19878.57	12921.82	1298.54	108.93	5.69	11.92	9.95
19	Chan	19878.57	19987.49	10711.74	1160.38	108.94	4.72	10.65	9.23
20	Chan	19987.49	20096.41	6775.37	881.62	108.97	2.98	8.09	7.69
21	Chan	20096.41	20205.34	4810.04	717.71	108.93	2.12	6.59	6.70
22	Chan	20205.34	20314.26	7033.09	901.58	108.96	3.10	8.28	7.80
23	Chan	20314.26	20423.18	8115.87	982.43	108.95	3.58	9.02	8.26
24	Chan	20423.18	20532.10	10555.97	1150.90	109.10	4.65	10.57	9.17
25	Chan	20532.10	20641.02	5433.73	772.46	109.03	2.39	7.09	7.03
26	Chan	20641.02	20749.94	4335.02	674.35	108.95	1.91	6.19	6.43
27	Chan	20749.94	20858.87	4807.66	717.63	108.98	2.12	6.59	6.70
[illegible]	Chan	20858.87	20967.79	4649.23	703.30	108.96	2.05	6.46	6.61
[illegible]	Chan	20967.79	21076.71	4903.04	726.16	108.99	2.16	6.67	6.75
30	Chan	21076.71	21185.63	11620.04	1219.14	109.10	5.12	11.19	9.53
31	Chan	21185.63	21294.55	3389.27	452.79	58.22	1.49	8.05	7.49

Plan: Excav Gila River El Rio WMP RS: 194.205 BR U Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	Chan	19116.12	19225.04	18210.77	1383.09	105.88	8.02	13.44	13.17
2	Chan	19225.04	19333.96	30837.27	2162.07	146.81	13.58	20.80	14.26
3	Chan	19333.96	19442.88	21898.44	1726.77	139.85	9.65	16.62	12.68
4	Chan	19442.88	19551.80	18146.41	1515.11	133.69	7.99	14.58	11.98
5	Chan	19551.80	19660.73	18783.49	1543.52	132.98	8.27	14.85	12.17
6	Chan	19660.73	19769.65	16705.13	1431.17	131.26	7.36	13.77	11.67
7	Chan	19769.65	19878.57	12843.96	1206.97	127.17	5.66	11.61	10.64
8	Chan	19878.57	19987.49	10835.18	1077.68	123.64	4.77	10.37	10.05
9	Chan	19987.49	20096.41	6990.89	813.13	117.97	3.08	7.82	8.60
10	Chan	20096.41	20205.34	4882.53	652.53	116.61	2.15	6.28	7.48
11	Chan	20205.34	20314.26	7656.93	854.11	116.38	3.37	7.96	8.96
12	Chan	20314.26	20423.18	8729.19	917.98	114.50	3.85	8.70	9.51
13	Chan	20423.18	20532.10	10771.95	1070.80	122.75	4.75	10.30	10.06
14	Chan	20532.10	20641.02	5418.19	700.89	119.27	2.39	6.74	7.73
15	Chan	20641.02	20749.94	4367.25	609.80	116.37	1.92	5.87	7.16
16	Chan	20749.94	20858.87	4855.44	651.40	117.08	2.14	6.27	7.45
17	Chan	20858.87	20967.79	4730.25	639.30	116.18	2.08	6.15	7.40
18	Chan	20967.79	21076.71	5015.95	662.20	116.18	2.21	6.37	7.57
19	Chan	21076.71	21185.63	11316.83	1124.11	128.71	4.99	10.82	10.07
20	Chan	21185.63	21294.55	4003.97	435.66	57.19	1.76	7.89	9.19

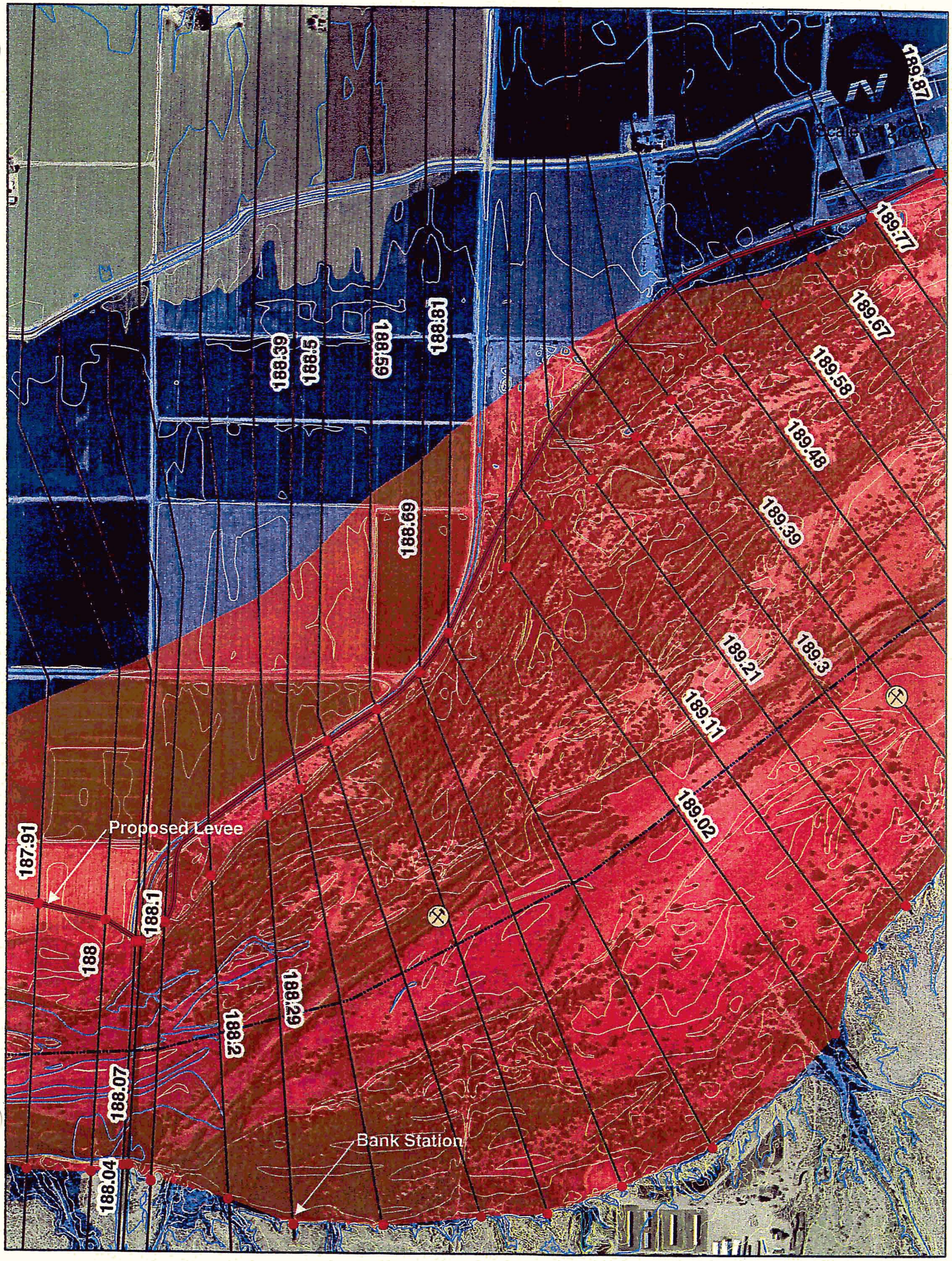
Tuthill Rd Bridge

Contraction condition illustrated in the following figure. Hydraulic data is summarized in the following tables.

Existing scour conditions are estimated in the "Bridge Scour Investigation and Design of Corrective Measures" (Baker, 1998). River station 188.39 is used as the approach section. The potential scour depths at the bridge are estimated to be 8.1 feet for contraction and 28 feet for left abutment scour and 29 feet for right abutment scour.

Comparing the hydraulic parameters used in the existing condition analysis to the parameters for proposed conditions, the proposed condition potential scour depths are similar, less in fact, than existing. Therefore, the existing condition scour depths will be used.

Tuthill Rd Bridge



State Route 85 Bridge

Contraction conditions illustrated in the following figure. Hydraulic data is summarized in the following tables.

Contraction Scour

- River station 180.65 is used as the approach section

Channel:

Table 7

$Y_1 = 8.6^{ft}$

$Q_1 = 72,476^{cfs}$

$W_1 = 1,637^{ft}$

$d_{50} = 0.50^{mm}$ (Samples 1 & 2)

$K_1 = 0.69$

$Q_2^* = 85,726^{cfs}$

$W_2^* = 902^{ft}$

$Y_0^* = 11.3^{ft}$

$$Y_s^{ch} = Y_1 \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{K_1} - Y_0$$

$$= 8.6 \left(\frac{85,726}{72,476}\right)^{6/7} \left(\frac{1,637}{902}\right)^{0.69} - 11.3$$

$\underline{Y_s^{ch} = 3.7'}$

* Estimated from flow distribution tables (Table 12), station 19,607.39 – 20,509.79 due to location of channel bank stations for FEMA & proposed condition modeling purposes compared to hydraulic conditions for scour estimations

Right OB

Table 7
$y_1 = 6.2^{ft}$
$Q_1 = 132{,}300^{cfs}$
$W_1 = 4{,}740^{ft}$
$d_{50} = 0.50^{mm}$ (Samples 1 & 2)
$K_1 = 0.64$

$Q_2^* = 107{,}268^{cfs}$
$W_2^* = 2{,}346^{ft}$
$y_0^* = 7.0^{ft}$

$$y_s = 6.2\left(\frac{107{,}268}{132{,}300}\right)^{6/7}\left(\frac{4{,}740}{2{,}346}\right)^{0.64} - 7.0$$

$$\underline{y_s^{ROB} = 1.1^{ft}}$$

* Flow distribution results station 20,509.79 – 22,856.02 from Table 12.

Abutment Scour

Left (Channel ~ left abutment is in main channel)
$y_1 = 12.8^{ft}$
$K_1 = 0.55$ (Spill through)
$K_2 = 1.0$
$V_1 = 10.4^{fps}$

$$y_s = 4y_1\left(\frac{K_1}{0.55}\right)K_2\,Fr^{0.33}$$

$$= (4)(12.8)\left(\frac{0.55}{0.55}\right)(1)\left(\frac{10.4}{\sqrt{(32.2)(12.8)}}\right)^{0.33}$$

$$\underline{y_s^{L} = 41.0^{ft}}$$

Designed by: Checked by:

Stantec

Right

$Y_1 = 6.2^{ft}$

$K_1 = 0.55$

$K_2 = 1.0$

$V_1 = 6.8^{fps}$

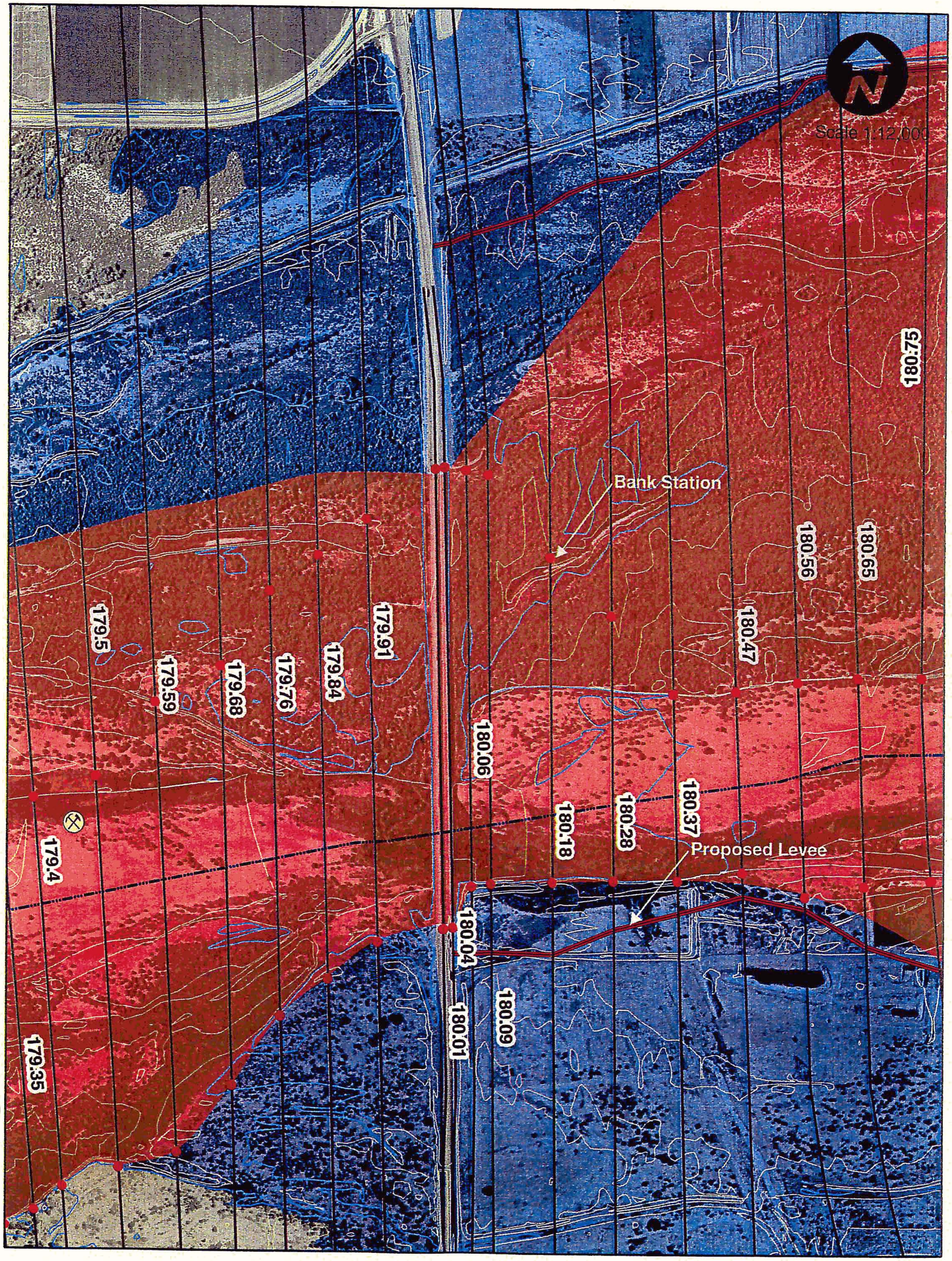
$$Y_s = (4)(6.2)\left(\frac{6.8}{\sqrt{(32.2)(6.2)}}\right)^{0.33}$$

$$\underline{Y_s = 19.5^{ft}}$$

Designed by:

Checked by:

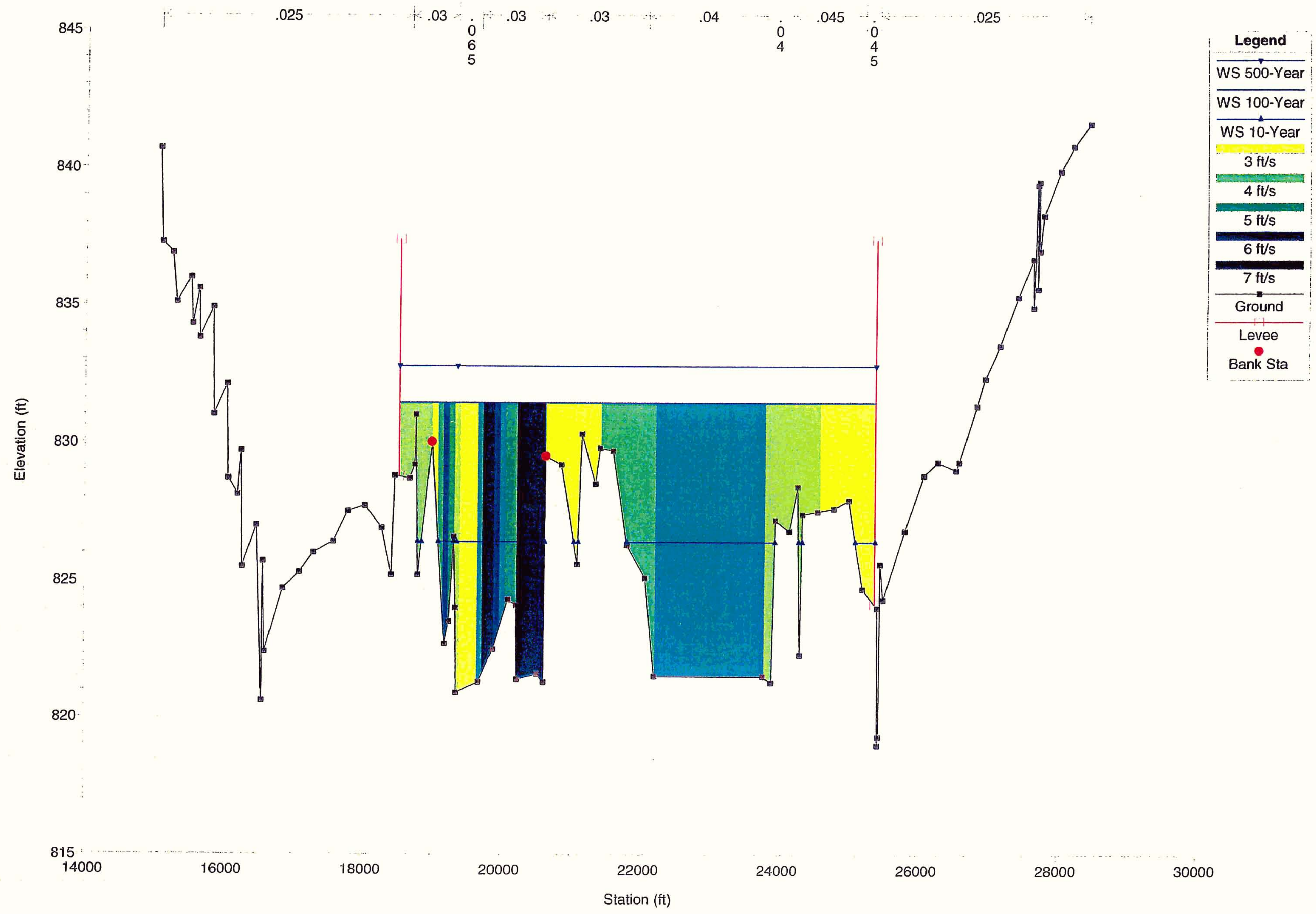
State Route 85 Bridge



El Rio WMP Alternatives Analysis Plan: P. Levee, New Veg & KR Imprvmnt & S&G 10/30/2005

Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications

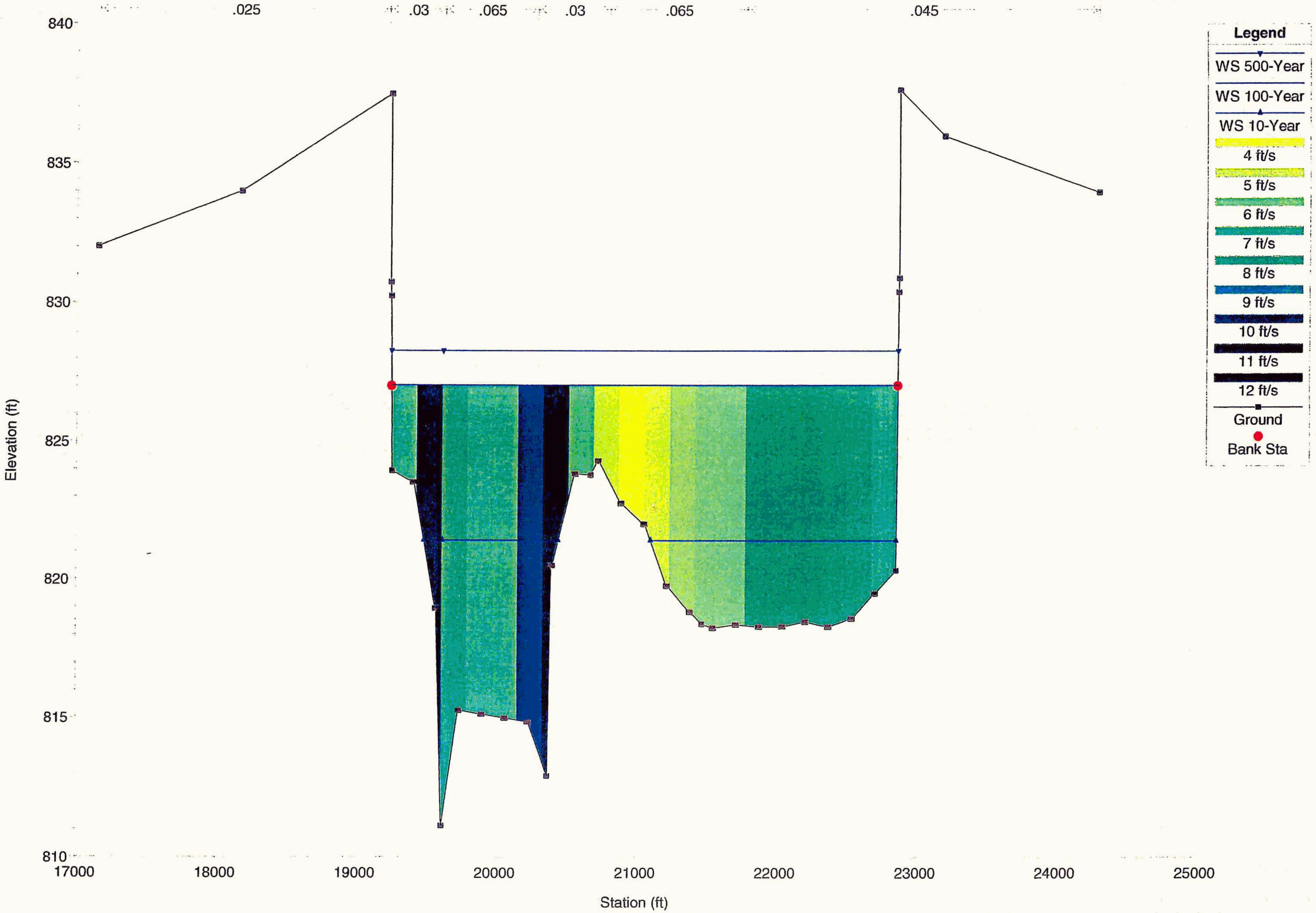
RS = 180.65

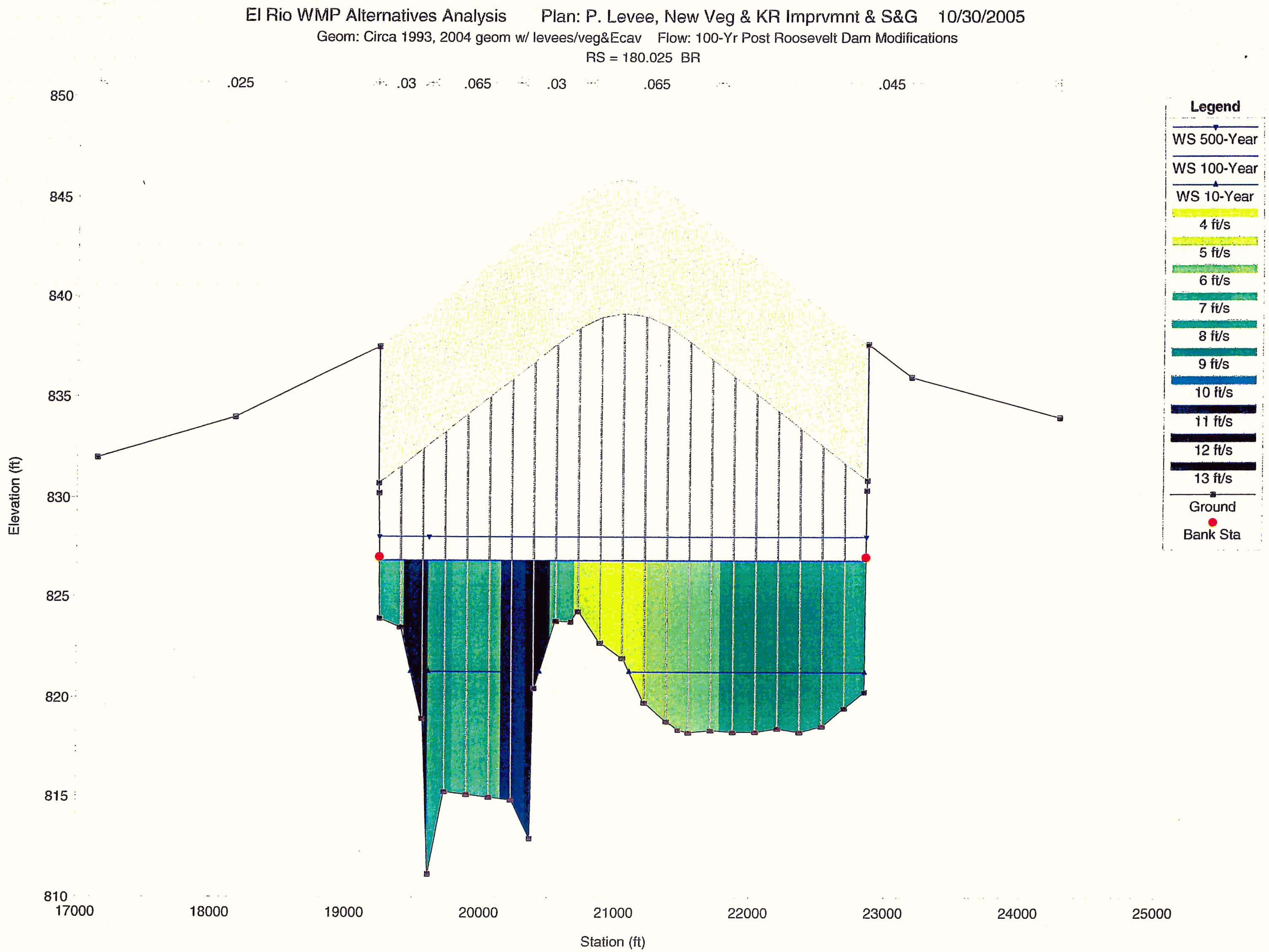


El Rio WMP Alternatives Analysis Plan: P. Levee, New Veg & KR Imprvmnt & S&G 10/30/2005

Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications

RS = 180.04




El Rio WMP Alternatives Analysis Plan: P. Levee, New Veg & KR Imprvmnt & S&G 10/30/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 180.025 BR
.025
.03
.065
.03
.065
.045
Legend
WS 500-Year
WS 100-Year
WS 10-Year
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
10 ft/s
11 ft/s
12 ft/s
13 ft/s
Ground
Bank Sta
Elevation (ft)
850
845
840
835
830
825
820
815
810
17000
18000
19000
20000
21000
22000
23000
24000
25000
Station (ft)

Plan: Excav Gila River El Rio WMP RS: 180.65 Profile: 100-Year Table 7

E.G. Elev (ft)	831.77	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.34	Wt. n-Val.	0.029	0.038	0.039
W.S. Elev (ft)	831.43	Reach Len. (ft)	465.00	483.21	485.00
Crit W.S. (ft)	827.04	Flow Area (sq ft)	1477.95	14081.65	29458.85
E.G. Slope (ft/ft)	0.001001	Area (sq ft)	1477.95	14081.65	29458.85
Q Total (cfs)	210000.00	Flow (cfs)	5224.45	72475.94	132299.60
Top Width (ft)	6846.43	Top Width (ft)	469.62	1637.10	4739.71
Vel Total (ft/s)	4.66	Avg. Vel. (ft/s)	3.53	5.15	4.49
Max Chl Dpth (ft)	12.43	Hydr. Depth (ft)	3.15	8.60	6.22
Conv. Total (cfs)	6638137.0	Conv. (cfs)	165145.8	2290977.0	4182014.0
Length Wtd. (ft)	484.06	Wetted Per. (ft)	472.90	1639.97	4748.61
Min Ch El (ft)	820.90	Shear (lb/sq ft)	0.20	0.54	0.39
Alpha	1.02	Stream Power (lb/ft s)	0.69	2.76	1.74
Frctn Loss (ft)	0.63	Cum Volume (acre-ft)	519.20	6344.20	3947.89
C & E Loss (ft)	0.05	Cum SA (acres)	223.47	752.04	803.39

Plan: Excav Gila River El Rio WMP RS: 180.04 Profile: 100-Year Table 8

E.G. Elev (ft)	827.87	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.82	Wt. n-Val.	0.000	0.050	0.000
W.S. Elev (ft)	827.05	Reach Len. (ft)	1.00	1.00	1.00
Crit W.S. (ft)	823.51	Flow Area (sq ft)	0.00	28952.93	0.00
E.G. Slope (ft/ft)	0.003473	Area (sq ft)	0.00	28952.93	0.00
Q Total (cfs)	210000.00	Flow (cfs)	0.00	210000.00	0.00
Top Width (ft)	3609.69	Top Width (ft)	0.05	3609.59	0.05
Vel Total (ft/s)	7.25	Avg. Vel. (ft/s)	0.17	7.25	0.11
Max Chl Dpth (ft)	15.93	Hydr. Depth (ft)	0.02	8.02	0.02
Conv. Total (cfs)	3563588.0	Conv. (cfs)	0.0	3563588.0	0.0
Length Wtd. (ft)	1.00	Wetted Per. (ft)	0.06	3614.81	0.06
Min Ch El (ft)	811.12	Shear (lb/sq ft)		1.74	
Alpha	1.00	Stream Power (lb/ft s)		12.59	
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	426.89	5009.43	2258.60
C & E Loss (ft)	0.04	Cum SA (acres)	199.03	597.78	545.16

Plan: Excav Gila River El Rio WMP RS: 180.025 BR U Profile: 100-Year Table 9

E.G. Elev (ft)	827.82	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.95	Wt. n-Val.		0.050	
W.S. Elev (ft)	826.87	Reach Len. (ft)	116.00	116.00	116.00
Crit W.S. (ft)	823.75	Flow Area (sq ft)		26869.28	
E.G. Slope (ft/ft)	0.004700	Area (sq ft)		26869.28	
Q Total (cfs)	210000.00	Flow (cfs)		210000.00	
Top Width (ft)	3420.17	Top Width (ft)		3420.17	
Vel Total (ft/s)	7.82	Avg. Vel. (ft/s)		7.82	
Max Chl Dpth (ft)	15.75	Hydr. Depth (ft)		7.86	
Conv. Total (cfs)	3063171.0	Conv. (cfs)		3063171.0	
Length Wtd. (ft)	116.00	Wetted Per. (ft)		3747.69	
Min Ch El (ft)	811.12	Shear (lb/sq ft)		2.10	
Alpha	1.00	Stream Power (lb/ft s)		16.44	
Frctn Loss (ft)	0.57	Cum Volume (acre-ft)	426.89	5008.79	2258.60
C & E Loss (ft)	0.08	Cum SA (acres)	199.03	597.70	545.16

Table 10

Plan: Excav Gila River El Rio WMP RS: 180.65 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	LOB	18515.48	18985.10	5224.45	1477.95	472.90	2.49	3.15	3.53
[illegible]	Chan	18985.10	19066.96	694.52	242.04	81.91	0.33	2.96	2.87
[illegible]	Chan	19066.96	19148.81	2267.09	492.22	81.91	1.08	6.01	4.61
4	Chan	19148.81	19230.66	3870.30	678.37	81.88	1.84	8.29	5.71
5	Chan	19230.66	19312.52	2538.43	526.88	81.96	1.21	6.44	4.82
6	Chan	19312.52	19394.38	2513.81	751.87	82.33	1.20	9.19	3.34
7	Chan	19394.38	19476.23	2615.90	852.73	81.86	1.25	10.42	3.07
8	Chan	19476.23	19558.08	2572.98	844.29	81.85	1.23	10.31	3.05
9	Chan	19558.08	19639.94	2530.46	835.90	81.86	1.20	10.21	3.03
10	Chan	19639.94	19721.79	4407.87	820.23	81.86	2.10	10.02	5.37
11	Chan	19721.79	19803.65	4914.09	782.75	81.85	2.34	9.56	6.28
12	Chan	19803.65	19885.50	4511.48	743.63	81.86	2.15	9.08	6.07
13	Chan	19885.50	19967.36	4020.96	694.01	81.86	1.91	8.48	5.79
14	Chan	19967.36	20049.21	3501.05	638.68	81.86	1.67	7.80	5.48
15	Chan	20049.21	20131.07	3083.03	591.77	81.86	1.47	7.23	5.21
16	Chan	20131.07	20212.92	3115.60	595.96	82.01	1.48	7.28	5.23
17	Chan	20212.92	20294.78	5237.11	814.41	82.15	2.49	9.95	6.43
18	Chan	20294.78	20376.63	5247.96	814.25	81.86	2.50	9.95	6.45
19	Chan	20376.63	20458.49	5198.21	809.60	81.85	2.48	9.89	6.42
20	Chan	20458.49	20540.34	5171.01	807.06	81.86	2.46	9.86	6.41
21	Chan	20540.34	20622.20	4464.08	745.01	83.55	2.13	9.10	5.99
22	ROB	20622.20	21412.15	6705.27	2181.86	790.18	3.19	2.76	3.07
23	ROB	21412.15	22202.10	15238.71	3677.71	790.03	7.26	4.66	4.14
24	ROB	22202.10	22992.05	42287.68	7827.38	789.95	20.14	9.91	5.40
25	ROB	22992.05	23782.01	42292.22	7827.89	789.95	20.14	9.91	5.40
26	ROB	23782.01	24571.96	14148.47	4118.08	791.21	6.74	5.21	3.44
27	ROB	24571.96	25361.91	11627.24	3825.93	797.29	5.54	4.84	3.04

Table 11

Plan: Excav Gila River El Rio WMP RS: 180.04 Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	Chan	19246.43	19426.91	3742.94	600.21	181.42	1.78	3.33	6.24
2	Chan	19426.91	19607.39	13542.26	1297.54	181.14	6.45	7.19	10.44
[illegible]	Chan	19607.39	19787.87	18610.31	2386.61	180.68	8.86	13.22	7.80
4	Chan	19787.87	19968.35	14494.81	2146.22	180.48	6.90	11.89	6.75
5	Chan	19968.35	20148.83	14808.05	2173.93	180.48	7.05	12.05	6.81
6	Chan	20148.83	20329.31	21798.48	2267.99	180.49	10.38	12.57	9.61
7	Chan	20329.31	20509.79	15109.73	1387.74	181.82	7.20	7.69	10.89
8	Chan	20509.79	20690.27	3729.43	597.68	180.49	1.78	3.31	6.24
9	Chan	20690.27	20870.75	2682.67	602.48	180.49	1.28	3.34	4.45
10	Chan	20870.75	21051.22	3006.45	835.18	180.48	1.43	4.63	3.60
11	Chan	21051.22	21231.70	4979.91	1130.56	180.49	2.37	6.26	4.40
12	Chan	21231.70	21412.18	7281.52	1419.98	180.48	3.47	7.87	5.13
13	Chan	21412.18	21592.66	8531.28	1561.55	180.48	4.06	8.65	5.46
14	Chan	21592.66	21773.14	8654.23	1566.52	180.48	4.12	8.68	5.52
15	Chan	21773.14	21953.62	12470.39	1572.72	180.48	5.94	8.71	7.93
16	Chan	21953.62	22134.10	12422.07	1569.07	180.48	5.92	8.69	7.92
17	Chan	22134.10	22314.58	12193.86	1551.71	180.48	5.81	8.60	7.86
18	Chan	22314.58	22495.06	12299.14	1559.73	180.48	5.86	8.64	7.89
19	Chan	22495.06	22675.54	11107.15	1467.20	180.48	5.29	8.13	7.57
20	Chan	22675.54	22856.02	8535.31	1258.33	182.50	4.06	6.97	6.78

Chan

ROB

Note: Flow velocity for abutment scour is the max. velocity in the subsection nearest to the abutment

Hydraulic depth for the two subsections is the weighted average

Table 12

Plan: Excav Gila River El Rio WMP RS: 180.025 BR U Profile: 100-Year

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth(ft)	Velocity (ft/s)
1	Chan	19246.43	19426.91	3656.12	538.45	178.85	1.74	3.14	6.79
2	Chan	19426.91	19607.39	13349.47	1194.61	187.95	6.36	6.97	11.17
3	Chan	19607.39	19787.87	18818.85	2250.64	194.86	8.96	13.12	8.36
4	Chan	19787.87	19968.35	14300.20	2008.99	194.95	6.81	11.72	7.12
5	Chan	19968.35	20148.83	14601.37	2035.43	195.23	6.95	11.87	7.17
6	Chan	20148.83	20329.31	22527.41	2128.24	195.52	10.73	12.41	10.59
7	Chan	20329.31	20509.79	15478.98	1298.84	185.54	7.37	7.57	11.92
8	Chan	20509.79	20690.27	3677.30	538.74	177.56	1.75	3.14	6.83
9	Chan	20690.27	20870.75	2633.91	547.78	176.61	1.25	3.19	4.81
10	Chan	20870.75	21051.22	2857.03	738.29	178.73	1.36	4.46	3.87
11	Chan	21051.22	21231.70	4742.80	1019.73	187.35	2.26	6.06	4.65
12	Chan	21231.70	21412.18	7251.81	1316.14	187.53	3.45	7.68	5.51
13	Chan	21412.18	21592.66	8513.18	1452.58	188.67	4.05	8.47	5.86
14	Chan	21592.66	21773.14	8667.55	1458.63	188.42	4.13	8.51	5.94
15	Chan	21773.14	21953.62	12463.60	1464.11	188.58	5.94	8.54	8.51
16	Chan	21953.62	22134.10	12413.88	1460.55	188.57	5.91	8.52	8.50
17	Chan	22134.10	22314.58	12205.13	1444.72	188.23	5.81	8.42	8.45
18	Chan	22314.58	22495.06	12279.78	1451.12	188.58	5.85	8.46	8.46
19	Chan	22495.06	22675.54	11060.50	1361.19	188.01	5.27	7.94	8.13
20	Chan	22675.54	22856.02	8501.17	1160.53	187.95	4.05	6.78	7.33

Note: Flow through the two subsections of the bridge is the sum of incremental discharges from station 19,607.39 – 20,509.79

Flow depth for the two subsections at the bridge is the weighted average hydraulic depth from station 19,607.39 – 20,509.79.

CHAPTER 5

GENERAL SCOUR

5.1 INTRODUCTION

General scour is the general decrease in the elevation of the bed across the bridge opening. It does not include localized scour at the foundations (local scour) or the long-term changes in the stream bed elevation (aggradation or degradation). General scour may not have a uniform depth across the bridge opening. General scour can be cyclic, that is, there can be an increase and decrease of the stream bed elevation (cutting and filling) during the passage of a flood.

The most common general scour is contraction scour. There are several cases and flow conditions for contraction scour. Typically, contraction scour occurs where the bridge opening is smaller than the flow area of the upstream channel and/or floodplain. Other general scour conditions can result from erosion related to planform characteristics of the stream, flow around a bend, variable downstream control, or other changes that decrease the bed elevation at the bridge. In this chapter, methods and equations will be presented to estimate general scour.

5.2 CONTRACTION SCOUR

5.2.1 Contraction Scour Conditions

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). In the case of **live-bed scour**, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For **live-bed** scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For **clear-water** scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. Normally, for both live-bed and clear-water scour the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in.

Clear-water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material.

There are four conditions (cases) of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations: (1) **live-bed** scour, and (2) **clear-water** scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in the Appendix C. This equation is:

$$V_c = K_u \, y^{1/6} \, D^{1/3} \tag{5.1}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
y	=	Average depth of flow upstream of the bridge, m (ft)
D	=	Particle size for V_c, m (ft)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, m (ft)
K_u	=	6.19 SI units
K_u	=	11.17 English units

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 0.3 m (1 ft) of the stream bed.

Live-bed contraction scour depths may be limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge cross-section. Under these conditions, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

5.2.2 Contraction Scour Cases

Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (Figure 5.1);

b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment (Figure 5.2); or

c. Abutments are set back from the stream channel (Figure 5.3).

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river (Figures 5.4 and 5.5).

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour) (Figure 5.6).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1) (Figure 5.7).

Notes:

1. **Cases 1, 2, and 4** may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity at the approach section for the D_{50} of the bed material using the equation given above and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V_*) in the contracted section to the fall velocity (ω) of the D_{50} of the bed material being transported from the upstream reach (see the definition of V_* in the live-bed contraction scour equation). If the ratio is much larger than 2, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

2. **Case 1c is very complex.** The depth of contraction scour depends on factors such as (1) how far back from the bank line the abutment is set, (2) the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), (3) whether the stream is narrower or wider at the bridge than at the upstream section, (4) the magnitude of the overbank flow that is returned to the bridge opening, and (5) the distribution of the flow in the bridge section, and (6) other factors.

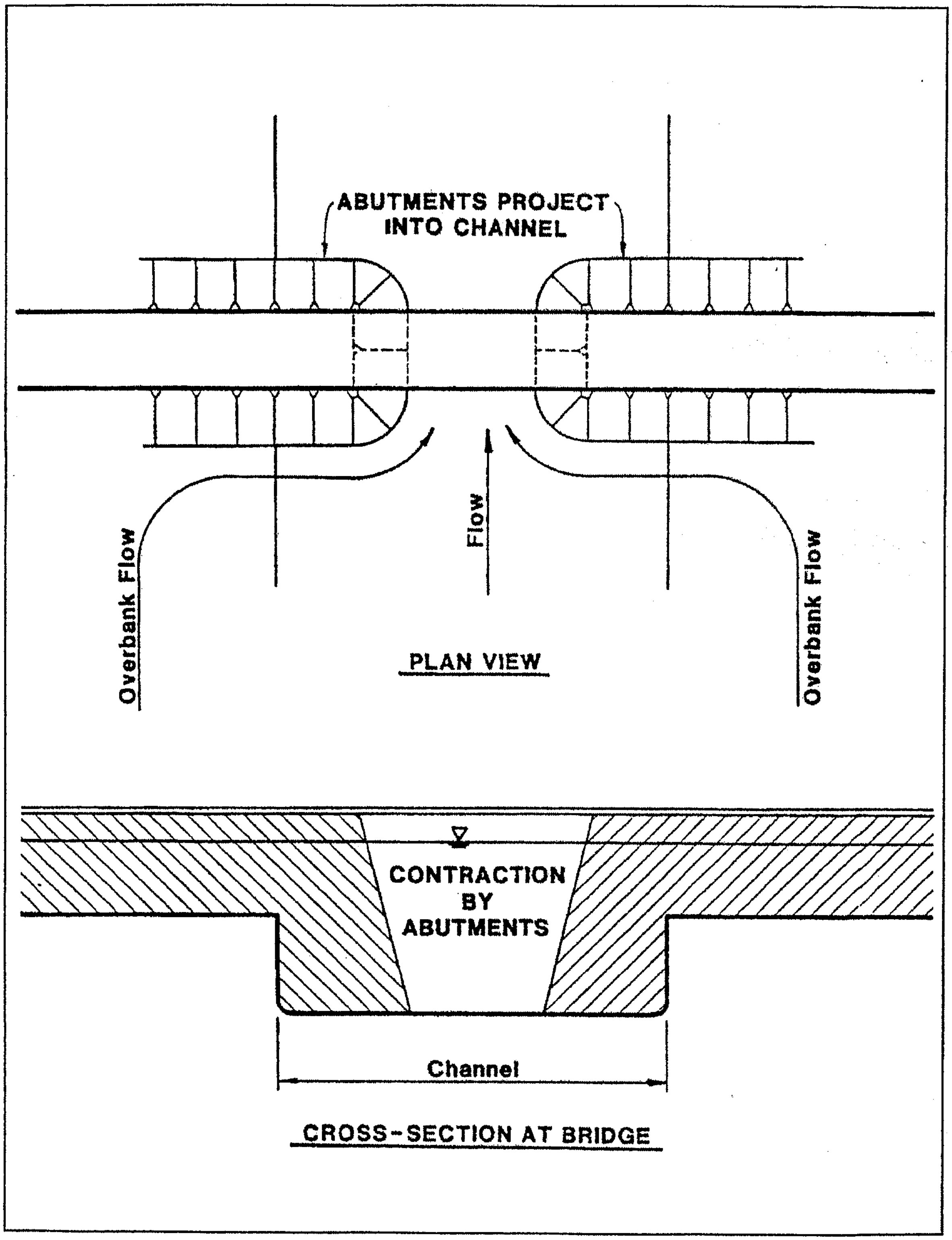


Figure 5.1. Case 1A: Abutments project into channel.

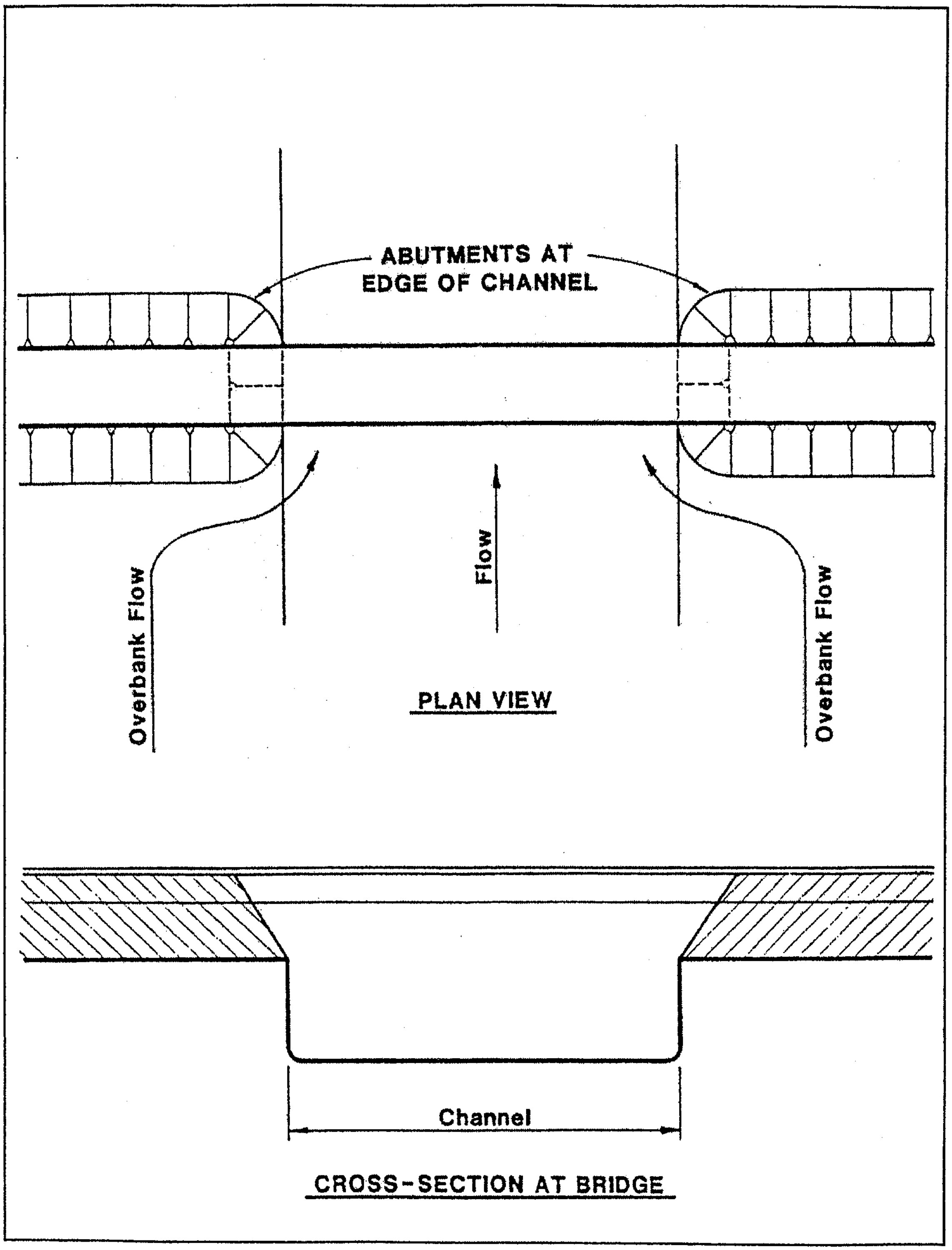


Figure 5.2. Case 1B: Abutments at edge of channel.

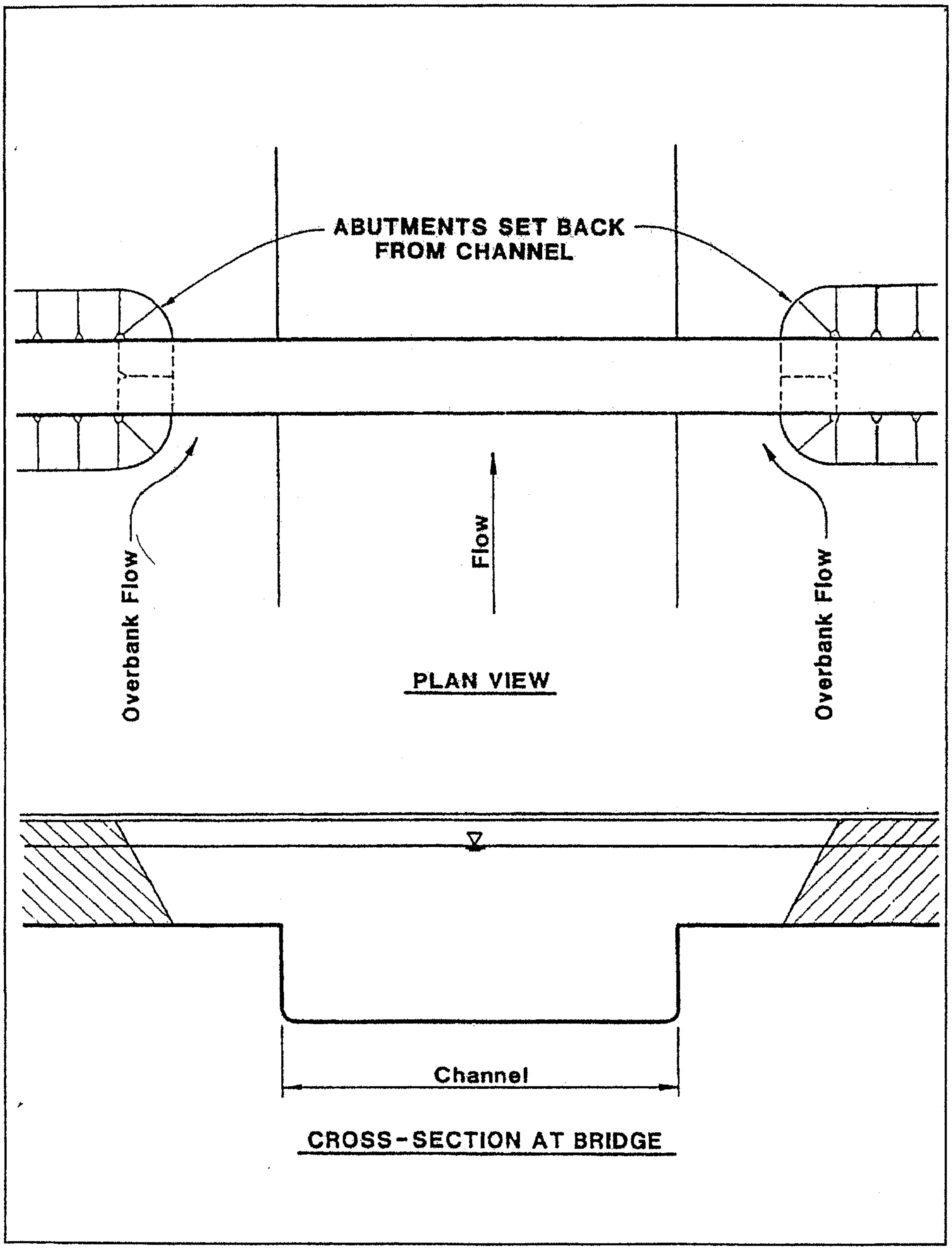


Figure 5.3. Case 1C: Abutments set back from channel.

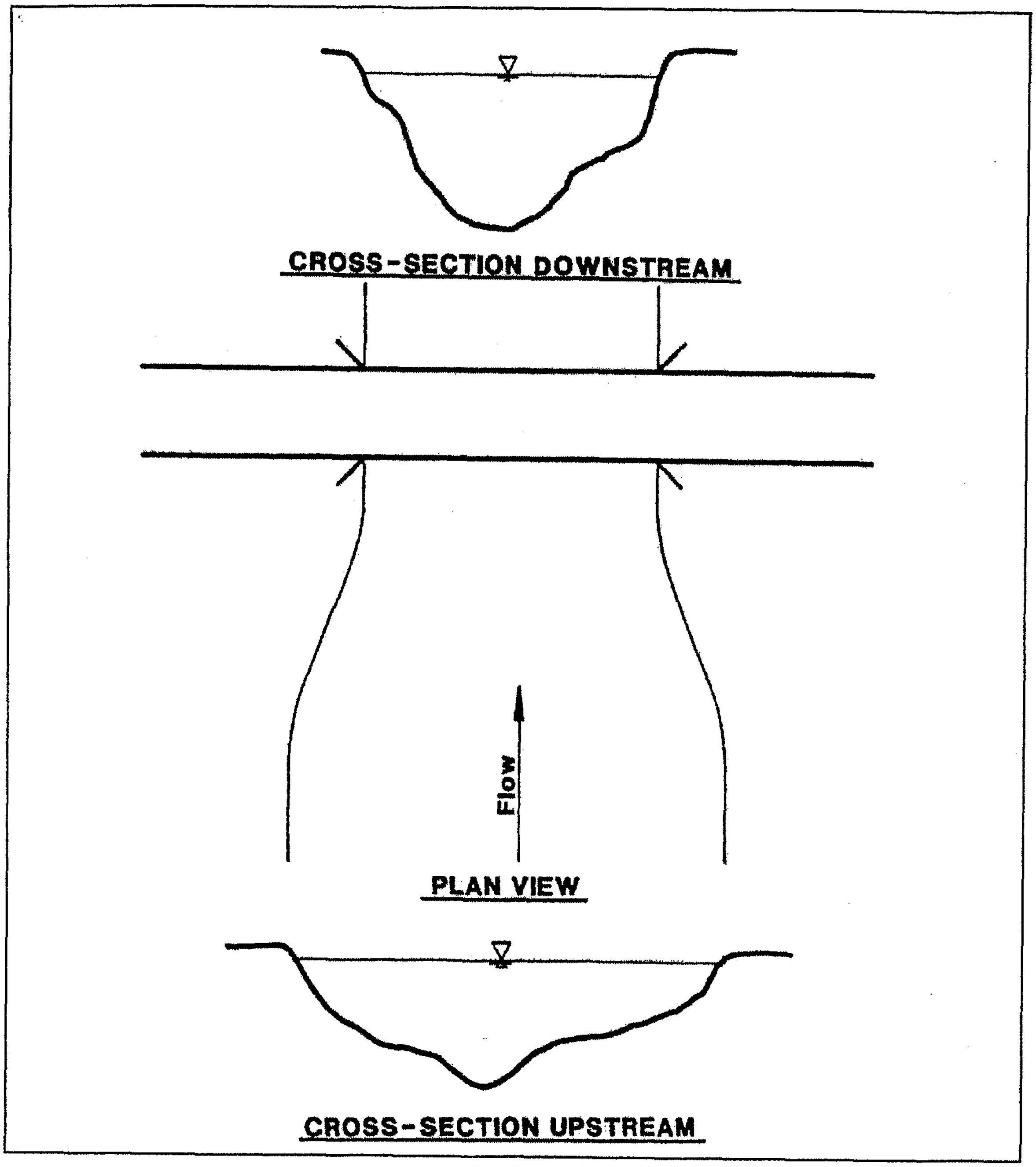


Figure 5.4. Case 2A: River narrows.

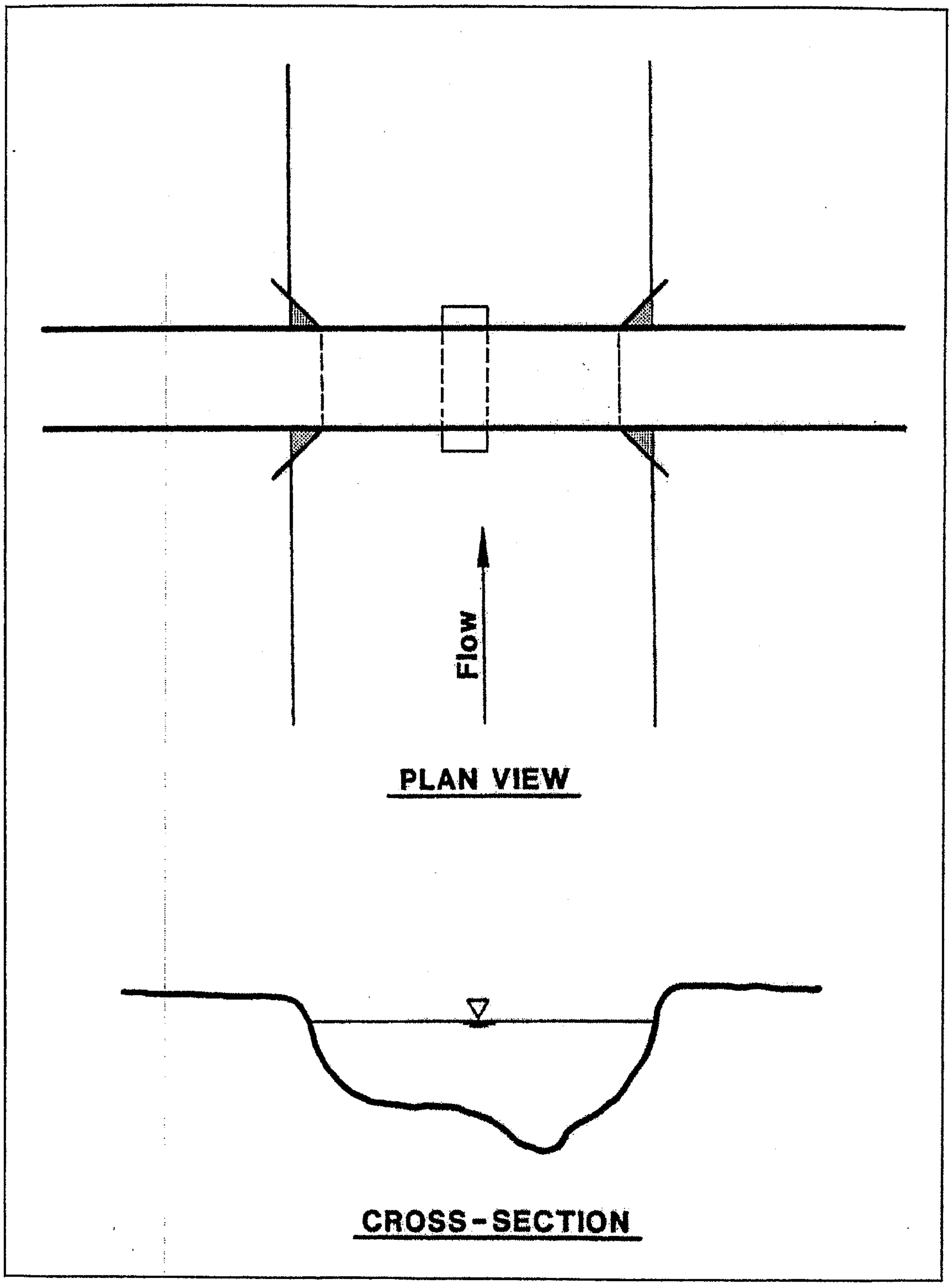


Figure 5.5. Case 2B: Bridge abutments and/or piers constrict flow.

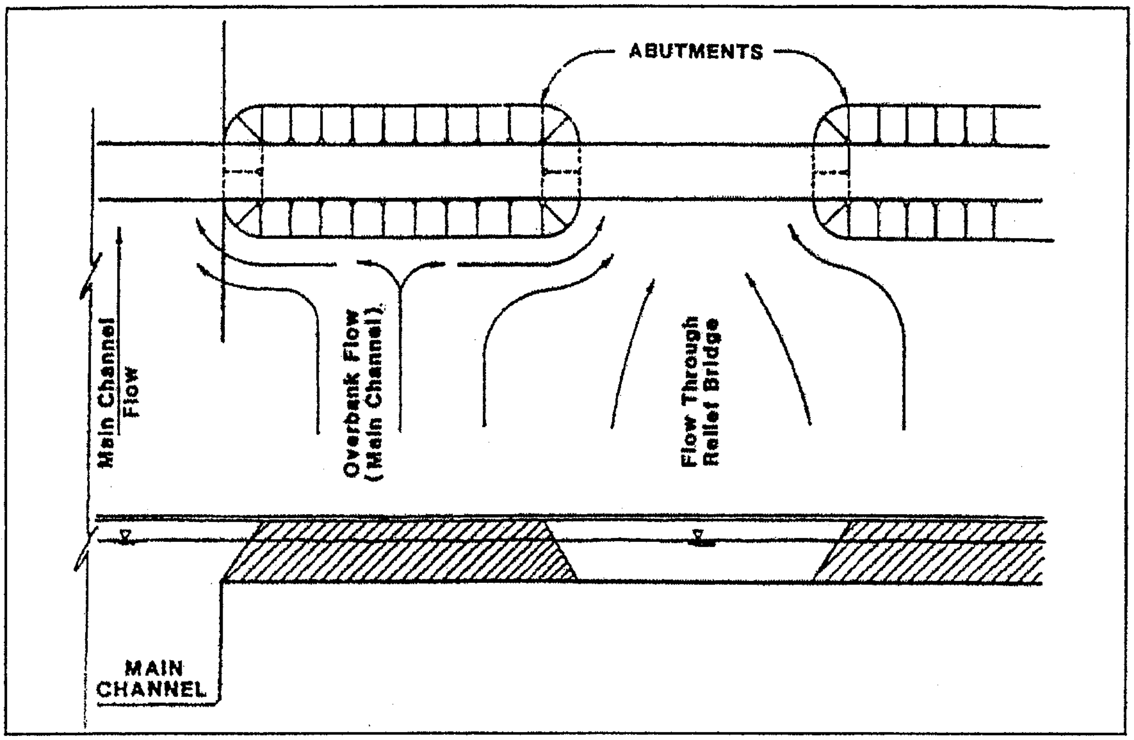


Figure 5.6. Case 3: Relief bridge over floodplain.

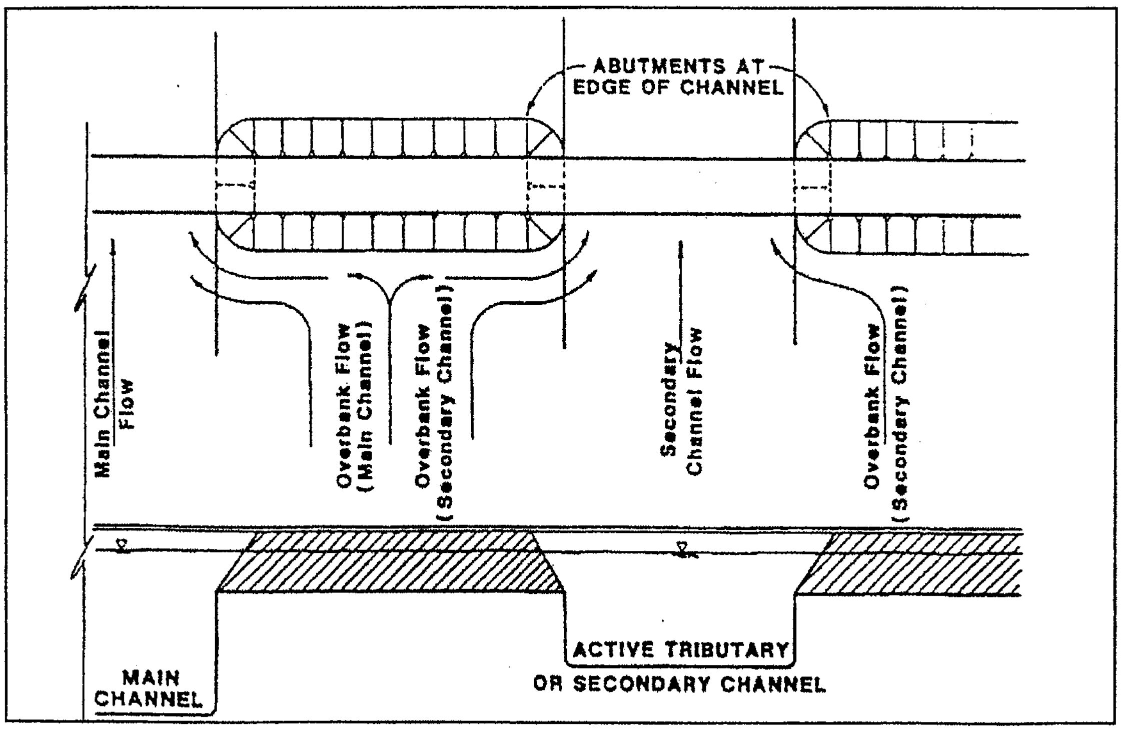


Figure 5.7. Case 4: Relief bridge over secondary stream.

The main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.

WSPRO[15] or HEC-RAS[16,17] can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening. However, the distribution of flow needs to be done with care. Studies by Chang[41] and Sturm [42] have shown that conveyance calculations do not properly account for the flow distribution under the bridge.

If the abutment is set back only a small distance from the bank (less than 3 to 5 times the average depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Consideration should be given to using a guide bank and/or protecting the bank and bed under the bridge in the overflow area with rock riprap. See HEC-23[7] for guidance on designing rock riprap.

3. **Case 3** may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.

4. **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from case 1, but analysis is required to determine the floodplain discharge associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO[15] or HEC-RAS.[16, 17]

5.3 LIVE-BED CONTRACTION SCOUR

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section.[43] The original equation is given in Appendix C. The modification is to eliminate the ratio of Manning's n (see the following Note #3). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad (5.2)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \qquad (5.3)$$

where:

y_1	=	Average depth in the upstream main channel, m (ft)
y_2	=	Average depth in the contracted section, m (ft)
y_o	=	Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1	=	Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2	=	Flow in the contracted channel, m^3/s (ft^3/s)

W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)

W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)

k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (gy_1\, S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)

ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 5.8) For fall velocity in English units (ft/s) multiply ω in m/s by 3.28

g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)

S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

τ_o = Shear stress on the bed, Pa (N/m^2) (lb/ft^2)

ρ = Density of water (1000 kg/m^3) (1.94 $slugs/ft^3$)

Notes:

1. Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. **It is not the total flow for Case 1c**. For Case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.

2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

3. The Manning's n ratio is eliminated in Laursen live-bed equation to obtain Equation 5.2 (Appendix C).This was done for the following reasons. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planing out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case, whereas in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal. Consequently, the n value ratio is not recommended or presented in Equation 5.2.

4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the topwidth of the main channel to define these widths. Whether topwidth or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

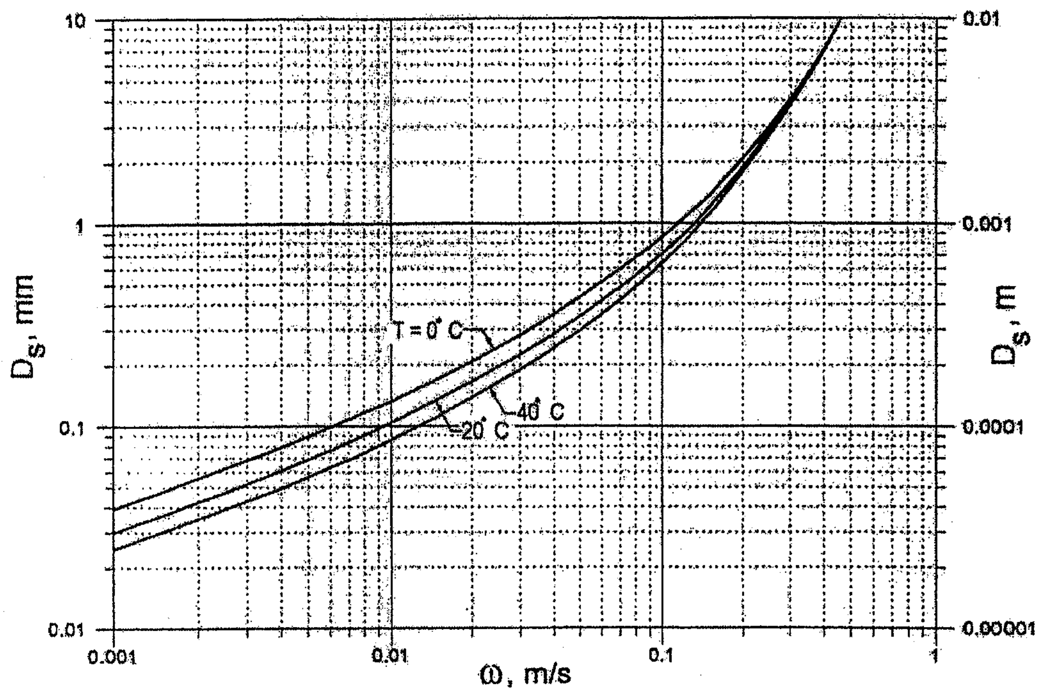


Figure 5.8. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

5. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.

6. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

7. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.

8. **Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation, and that the smaller calculated scour depth be used.**

5.4 CLEAR-WATER CONTRACTION SCOUR

The recommended clear-water contraction scour equation is based on a development suggested by Laursen[44] (presented in the Appendix C). The equation is:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \tag{5.4}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \tag{5.5}$$

where:

y_2	=	Average equilibrium depth in the contracted section after contraction scour, m (ft)
Q	=	Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)
D_m	=	Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)
D_{50}	=	Median diameter of bed material, m (ft)
W	=	Bottom width of the contracted section less pier widths, m (ft)
y_o	=	Average existing depth in the contracted section, m (ft)
K_u	=	0.025 SI units
K_u	=	0.0077 English units

Equation 5.4 is a rearranged version of 5.1.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50}. For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

5.5 CONTRACTION SCOUR WITH BACKWATER

The **live-bed** contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. With live-bed scour the equation computes a depth after the long contraction where the sediment transport into the downstream reach is equal to the sediment transport out. The **clear-water** contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that flow goes from one uniform flow condition to another. Both equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the energy at the same section before (1) and after (2) the contraction scour.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

CHAPTER 7

EVALUATING LOCAL SCOUR AT ABUTMENTS

7.1 GENERAL

Scour occurs at abutments when the abutment and embankment obstruct the flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented:(69)

- Overtopping of abutments or approach embankments
- Lateral channel migration or stream widening processes
- Contraction scour
- Local scour at one or both abutments

Abutment damage is often caused by a combination of these factors. Where abutments are set back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths of as much as four times the approach flow depth on the floodplain. As a general rule, the abutments most vulnerable to damage are those located at or near the channel banks.

The flow obstructed by the abutment and approach highway embankment forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and a vertical wake vortex at the downstream end of the abutment (Figure 7.1).

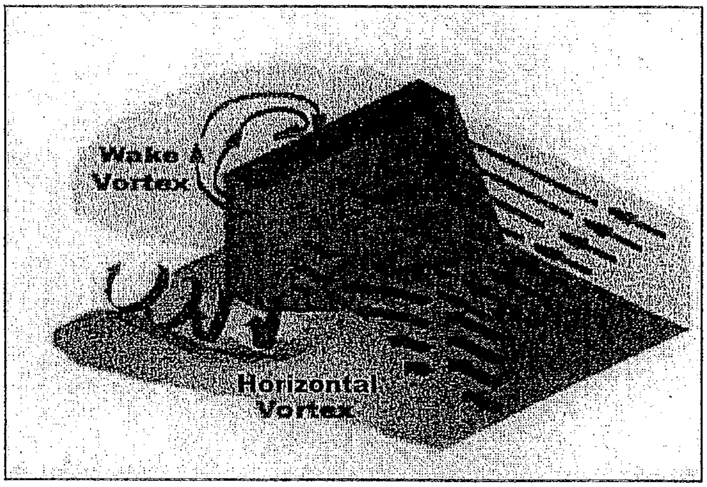


Figure 7.1. Schematic representation of abutment scour.

The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called horseshoe) vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex has not been conducted. An example of abutment and approach erosion of a bridge due to the action of the horizontal and wake vortex is shown in Figure 7.2.

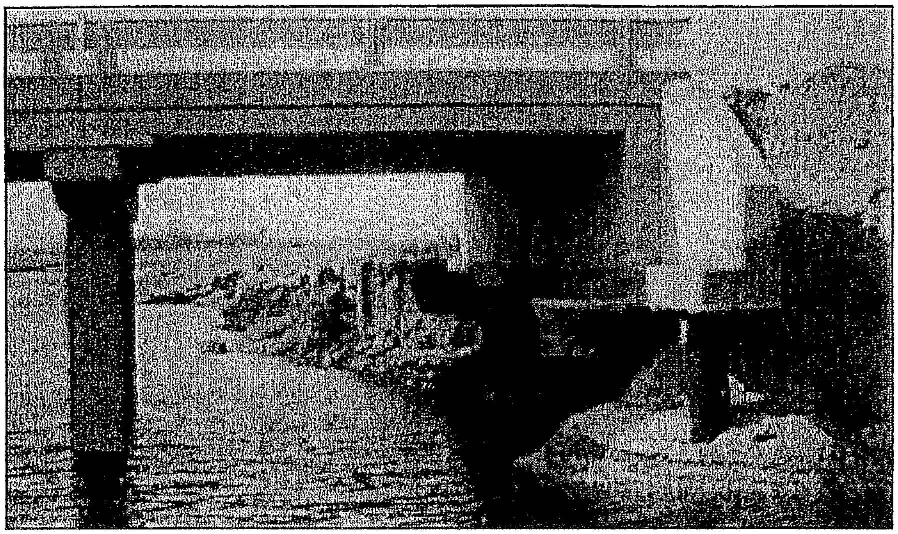


Figure 7.2. Scour of bridge abutment and approach embankment.

The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and highway embankment and subsequent contraction and turbulence of the flow at the abutments. There are other conditions that develop during major floods, particularly on wide floodplains, that are more difficult to foresee but that need to be considered in the hydraulic analysis and design of the substructure:[69]

- Gravel pits on the floodplain upstream of a structure can capture the flow and divert the main channel flow out of its normal banks into the gravel pit. This can result in an adverse angle of attack of the flow on the downstream highway with subsequent breaching of the embankment and/ or failure of the abutment.
- Levees can become weakened and fail with resultant adverse flow conditions at the bridge abutment.
- Debris can become lodged at piers and abutments and on the bridge superstructure, modifying flow conditions and creating adverse angles of attack of the flow on bridge piers and abutments.

7.2 ABUTMENT SCOUR EQUATIONS

7.2.1 Overview

Equations for predicting abutment scour depths such as Liu et al., Laursen, Froehlich, and Melville are based entirely on laboratory data.[70,48,71,72] The problem is that little field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data.[70] Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment.[48] Froehlich's equations were derived from dimensional analysis and regression analysis of the available laboratory data.[71] Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.[72]

Until recently, the equations in the literature were developed using the abutment and roadway approach length as one of the variables. This approach results in excessively conservative estimates of scour depth. Richardson and Richardson pointed this out in a discussion of Melville's (1992) paper:[73, 72]

> "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case."

Figure 7.3. illustrates the difference. Thus, equations for predicting abutment scour would be more applicable to field conditions if they included the discharge intercepted by the embankment rather than embankment length. Sturm[42,74] concluded that a discharge distribution factor is the appropriate variable to use on local scour depth rather than abutment length.

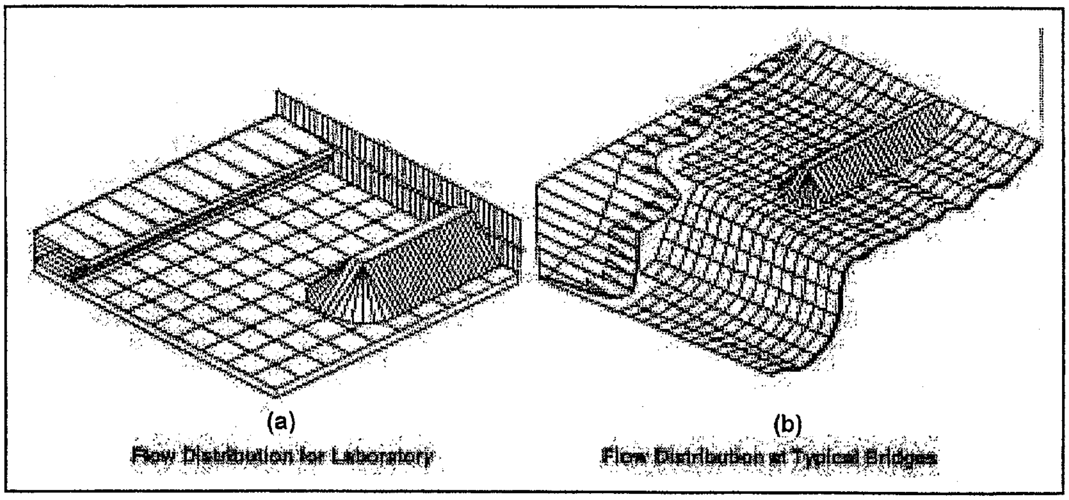


Figure 7.3. Comparison of (a) laboratory flow characteristics to (b) field flow conditions.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case. Richardson and Richardson noted that abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment.[73] In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Most of the early laboratory research failed to replicate these field conditions.

Recent research sponsored by the National Cooperative Highway Research Program of the Transportation Research Board has developed an equation to determine abutment scour that includes the discharge intercepted by an abutment and its approach rather than abutment and approach length.[75] The equation and method are presented in Appendix E. In addition, Maryland State Highway Administration has developed a method to determine scour depths at abutments, which is presented in Appendix F.[41,76] Both methods are under development and show promise of improving abutment scour calculations. They should be used with caution, and use of engineering judgment is needed for application at this time.

Abutment foundations should be designed to be safe from long-term degradation, lateral migration, and contraction scour; and protected from local horizontal and wake vortex scour with riprap and/or guidebanks, dikes, or revetments protected with riprap. The two equations provided in this chapter should be used as guides in the design.

7.2.2 Abutment Scour Parameter Determination

Many of the abutment scour prediction equations presented in the literature use the length of an abutment (embankment) projected normal to flow as an independent variable. In practice, the length of embankment projected normal to flow that is used in these relationships is determined from the results of 1-dimensional hydraulic models such as WSPRO[15] or HEC-RAS.[16,17] These models assume an average velocity over the entire cross section (Figure 7.3a). In reality, conveyance and associated velocity and flow depth at the outer extremes of a floodplain are much less, particularly in wide and shallow heavily vegetated floodplains (Figure 7.3b). This flow is typically referred to as "ineffective" flow. When applying abutment scour equations that use the length of embankment projected normal to flow, it is imperative that the length used be the length of embankment blocking "live" flow.

The length of embankment blocking "live" flow can be determined from a graph of conveyance versus distance across a representative cross-section upstream of the bridge (Figure 7.4). If a relatively large portion of a cross-section is required to convey a known amount of discharge in the floodplain, then the length of embankment blocking this flow should probably not be included when determining the length of embankment for use in the abutment scour prediction relationship. Alternately, if the flow in a significant portion of the cross-section has low velocity and/or is shallow, then the length of embankment blocking this flow should probably not be used either. Both WSPRO[15] and HEC-RAS[16,17] can easily compute conveyance versus distance across a cross section.

For example, Figure 7.4 shows the plan view of an embankment blocking three equal conveyance tubes on the right floodplain at a bridge. Since the right conveyance tube occupies the majority of floodplain but conveys only one-third of the floodplain flow, it should not be included in the "live" flow area for determining L'. In this case the length of embankment, L', blocking the "live" flow is approximately the length of the two inner conveyance tubes. In the event that the conveyance versus distance graph does not show a conclusive break point between "live" flow and ineffective flow, an alternative procedure is to estimate L' as the width of the conveyance tube directly upstream of the abutment times the total number of conveyance tubes (including fractional portions) obstructed by the embankment. This length is more representative of the uniform flow conditions in the laboratory experiments used to develop abutment scour equations.

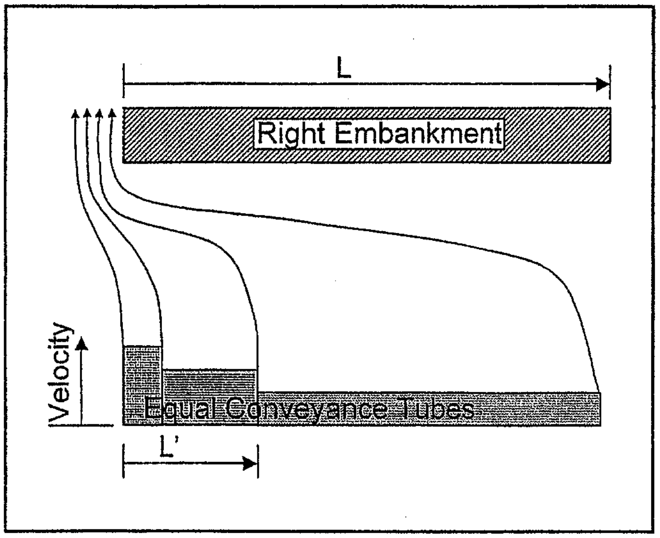


Figure 7.4. Determination of length of embankment blocking live flow for abutment scour estimation.

7.3 ABUTMENT SITE CONDITIONS

Abutments can be set back from the natural stream bank, placed at the bankline or, in some cases, actually set into the channel itself. Common designs include stub abutments placed on spill-through slopes, and vertical wall abutments, with or without wingwalls. Scour at abutments can be live-bed or clear-water scour. The bridge and approach road can cross the stream and floodplain at a skew angle and this will have an effect on flow conditions at the abutment. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

7.4 ABUTMENT SKEW

The skew angle for an abutment (embankment) is depicted in Figure 7.5. For an abutment angled downstream, the scour depth is decreased, whereas the scour depth is increased for an abutment angled upstream. An equation and guidance for adjusting abutment scour depth for embankment skew are given in Section 7.7.1.

7.5 ABUTMENT SHAPE

There are three general shapes of abutments: (1) spill-through abutments, (2) vertical walls without wing walls, and (3) vertical-wall abutments with wing walls (Figure 7.6). These shapes have varying angles to the flow. As shown in Table 7.1, depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments. Similarly, scour at vertical wall abutments with wingwalls is reduced to 82 percent of the scour of vertical wall abutments without wingwalls.

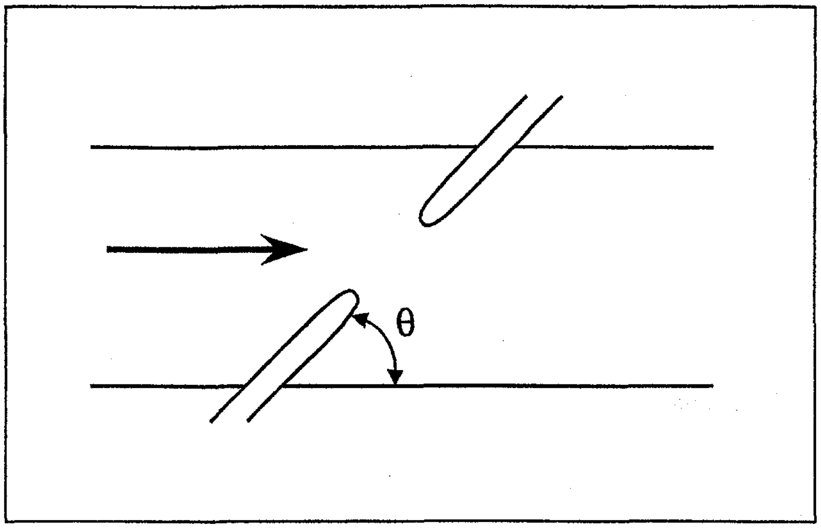


Figure 7.5. Orientation of embankment angle, θ, to the flow.

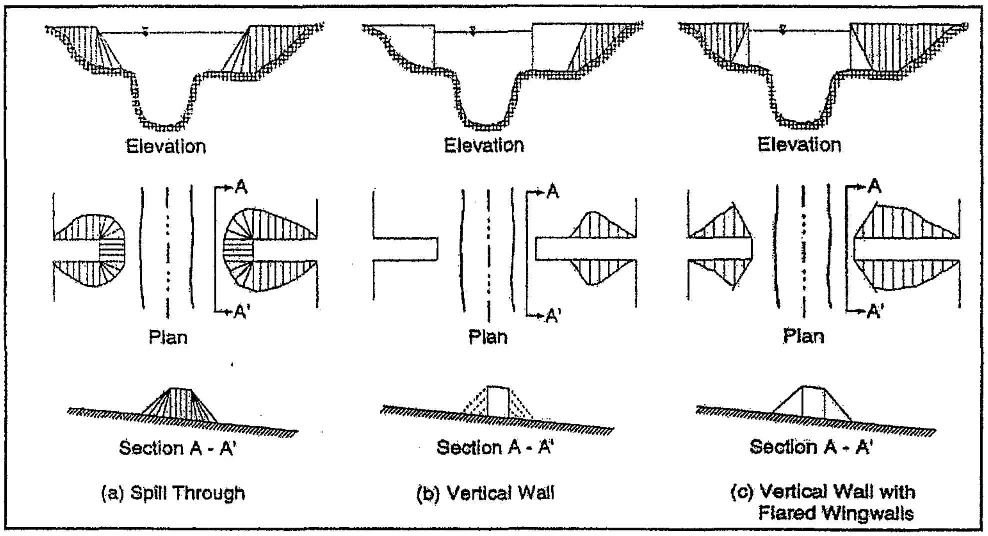


Figure 7.6. Abutment shape.

Table 7.1. Abutment Shape Coefficients.

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

7.6 DESIGNING FOR SCOUR AT ABUTMENTS

The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Available technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an abutment. **Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when they are protected with rock riprap and/or with a guide bank placed upstream of the abutment designed in accordance with guidelines in HEC-23.[7] Cost will be the deciding factor.**

Based on lessons learned from field evaluations of damaged abutments, consideration should be given to designing deep foundations (piles and shafts) to support both vertical wall abutments and stub abutments on spill-through slopes for the condition where the approach embankment is breached and all supporting soil around the abutment (including the spill through slope) has been removed (see Figure 7.2). Piling for abutments should be driven below the elevation of the long-term degradation and contraction scour. The potential for lateral channel instability should also be considered when designing abutment foundation depths. Some State DOTs evaluate the abutment for scour in a manner similar to that of a pier.

On wide floodplains or on floodplains with complex conditions which could affect future flood flows (confluences, adverse meander patterns and bends, gravel mining pits, ponding of the flow, levee systems, etc.) additional scour countermeasures such as guidebanks, dikes or revetments should be evaluated for inclusion with the initial bridge construction. The intent here is to establish a control to maintain a favorable approach flow condition at the abutment even though upstream conditions may change.

The potential for lateral channel migration, long-term degradation and contraction scour should be considered in setting abutment foundation depths near the main channel. It is recommended that the abutment scour equations presented in this chapter be used to develop insight as to the scour potential at an abutment.

Where spread footings are placed on erodible soil, the preferred approach is to place the footing below the elevation of total scour. If this is not practicable, a second approach is to place the top of footings below the depth of the sum of contraction scour and long-term degradation and to provide scour countermeasures. For spread footings on erodible soil, it becomes especially important to protect adjacent embankment slopes with riprap or other appropriate scour countermeasures. The toe or apron of the riprap serves as the base for the slope protection and must be carefully designed to resist scour while maintaining the support for the slope protection.

In summary, as a minimum, abutment foundations should be designed assuming no ground support (lateral or vertical) as a result of soil loss from long-term degradation, stream instability, and contraction scour. The abutment should be protected from local scour using riprap and/or guide banks. Guidelines for the design of riprap and guide banks are given in HEC-23.[7] To protect the abutment and approach roadway from scour by the wake vortex several DOTs use a 15-meter (50-ft) guide bank extending from the downstream corner of the abutment. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures.

In the following sections, two equations are presented for use in estimating scour depths as a guide in designing abutment foundations. The methods can be used for either **clear-water or live-bed** scour.

7.7 LIVE-BED SCOUR AT ABUTMENTS

As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap and/or guide banks, Froehlich's[70] live-bed scour equation or the HIRE[22] equation can be used.

7.7.1 Froehlich's Live-Bed Abutment Scour Equation

Froehlich[71] analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \qquad (7.1)$$

where:

K_1	=	Coefficient for abutment shape (Table 7.1)
K_2	=	Coefficient for angle of embankment to flow
K_2	=	$(\theta/90)^{0.13}$ (see Figure 7.4 for definition of θ)
		$\theta<90^0$ if embankment points downstream
		$\theta>90^0$ if embankment points upstream
L'	=	Length of active flow obstructed by the embankment, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)
Fr	=	Froude Number of approach flow upstream of the abutment
	=	$V_e/(gy_a)^{1/2}$
V_e	=	Q_e/A_e, m/s (ft/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
y_a	=	Average depth of flow on the floodplain (A_e/L), m (ft)
L	=	Length of embankment projected normal to the flow, m (ft)
y_s	=	Scour depth, m (ft)

It should be noted that Equation 7.1 is not consistent with the fact that as L' tends to 0, y_s also tends to 0. The 1 was added to the equation so as to envelope 98 percent of the data. See Section 7.2.2 and Figure 7.4 for guidance on estimating L'.

7.7.2 HIRE Live-Bed Abutment Scour Equation

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE) can also be used for estimating abutment scour.[22] This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The modified equation, referred to herein as the HIRE equation, is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4\ Fr^{0.33}\ \frac{K_1}{0.55}\ K_2 \tag{7.2}$$

where:

y_s	=	Scour depth, m (ft)
y_1	=	Depth of flow at the abutment on the overbank or in the main channel, m (ft)
Fr	=	Froude Number based on the velocity and depth adjacent to and upstream of the abutment
K_1	=	Abutment shape coefficient (from Table 7.1)
K_2	=	Coefficient for skew angle of abutment to flow calculated as for Froehlich's equation (Section 7.7.1)

7.8 CLEAR-WATER SCOUR AT AN ABUTMENT

Equations 7.1 and 7.2 are recommended for both live-bed and clear-water abutment scour conditions. If a method other than Froehlich's equation is used, it is suggested that scour for both the clear water and live bed condition be computed (see Appendix E and Appendix F). Engineering judgment should then be used to select the most appropriate scour depth.

7.9 ABUTMENT SCOUR EXAMPLE PROBLEMS (SI)

7.9.1 Example Problem 1 (SI)

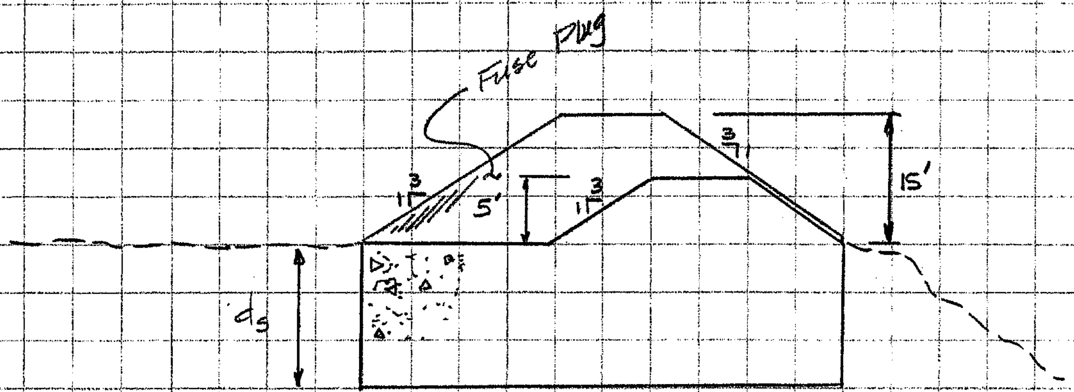
Determine abutment scour depth for the following conditions to aid in scour evaluation and design of countermeasures. The right abutment is at the bankline with 3.00 m of overbank flow width. The left abutment projects into the channel 61.96 m. Each of these lengths represents the full length of obstruction of active flow. The projection on the left side is the result of stream erosion and widening. The right channel bank is 0.61 m high and the embankment extends back 3.00 m to a 3 m high bank. The bridge and approach are oriented at a 10° angle upstream to the flow from the right side.

Given:

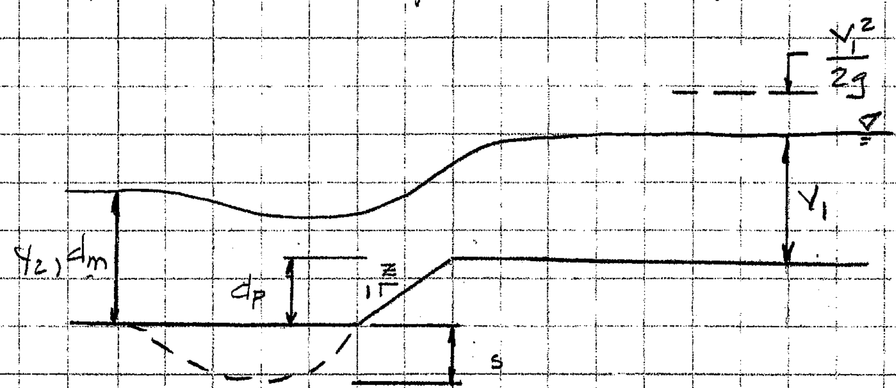
Upstream channel depth = 2.62 m
Discharge = 773.05 m^3/s
Bridge is vertical wall with wingwalls

Original (unscoured) depth of flow at bridge is estimated as 2.16 m

Typical Section



Representation for scour estimation purposes



Equations for estimation of scour depth

- Simons, Li & Associates, Inc. 1986 from "Project Report, Hydraulic Model Study of Local Scour Downstream of Rigid Grade-Control Structures, Pima County Dept. of Transportation, Tucson AZ

$$d_s = C q^{0.667} \left(\frac{Y_1 + \frac{V_1^2}{2g} - Y_2}{Y_2} * 100 \right)^{P_1} \left(\frac{Y_2 - d_p}{Y_1} * 100 \right)^{P_2}$$

Where :

Z	C	P_1	P_2
0	0.151	0.411	-0.118
1	0.483	0.158	-0.134
3	0.011	0.989	0.161

Appropriate for

- $25 \text{ cfs/ft} < q < 400 \text{ cfs/ft}$
- $3\% < \frac{Y_1 + \frac{V_1^2}{2g} - Y_2}{Y_2} * 100 < 96\%$
- $\frac{Y_2 - d_p}{Y_1} * 100 > 15\%$

- USBR, 1984, "Computing Degradation and Local Scour (Pemberton & Lara), Denver Colorado

  - Three equations are presented

Eqn. 34 : $d_s = \frac{K H^{0.2} q^{0.57}}{D_{90}^{0.32}} - d_m$ (Schoklitsh eqn)

Where : K = 3.15
H = Water surface differential across drop
d_m = Downstream mean water depth

Designed by: Checked by:

Eqn 35 : $d_s = K H_T^{0.225} q^{0.54} - d_m$ (Veronese Eqn)

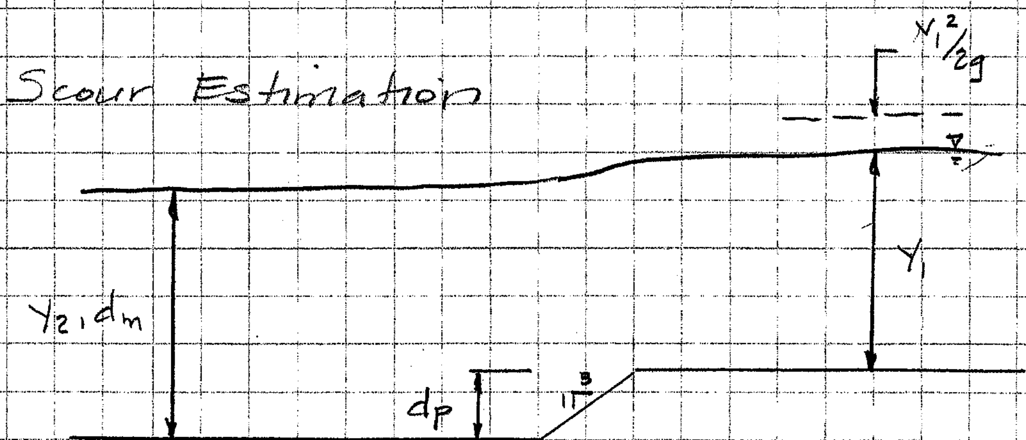
Where : $K = 1.32$
H_T = Head from upstream reservoir to tailwater
d_m = Downstream mean water depth

Eqn 36 : $d_s = K \left(\frac{q^{0.82}}{D_{85}^{0.23}} \right) \left(\frac{d_m}{q^{2/3}} \right)^{0.93} - d_m$ (Zimmerman + Maniak)

Where : $K = 1.95$
d_m = Downstream mean water depth

It is recommended that these three equations be averaged.

Scour Estimation



$Y_1 = 15.5'$
$V_1 = 11.8$ fps
$Y_2 = 15.0'$
$q = 227{,}000 / 1{,}540 = 175$ cfs/ft
$d_p = 5.0'$

$\frac{V_1^2}{2g} = 2.2'$

- Simons, Li & Associates Eqn.

$$d_s = (0.011)(175^{0.667})\left(\frac{15.5 + 2.2 - 15.0}{15.0} * 100\right)^{0.989}\left(\frac{15.0 - 5}{15.5} * 100\right)^{0.1}$$

$d_s = 11.8'$

- USBR Eqn's

$$d_s = \frac{3.15\, H^{0.2} q^{0.57}}{D_{90}^{0.32}} - d_m$$

$H = 0.5'$
D_{90} = ranges from 0.45 mm to 25 mm
$d_m = 11.6$ ft

$$= \frac{(3.15)(0.5^{0.2})(175^{0.57})}{0.45^{0.32}} - 11.6$$

$d_s = 55.6$ ft
(If $D_{90} = 25$ mm, $d_s = 7.0'$)

$$d_s = 1.32\, H_T^{0.225} q^{0.54} - d_m$$

$$= (1.32)(0.5^{0.225})(175^{0.54}) - 11.6$$

$d_s = 6.7$ ft

Designed by: Checked by:

Stantec

$$d_s = 1.95\left(\frac{q^{0.82}}{D_{85}^{0.23}}\right)\left(\frac{d_m}{q^{2/3}}\right)^{0.93} - d_m$$

$D_{85} = 0.38^{mm}$ to 18^{mm}

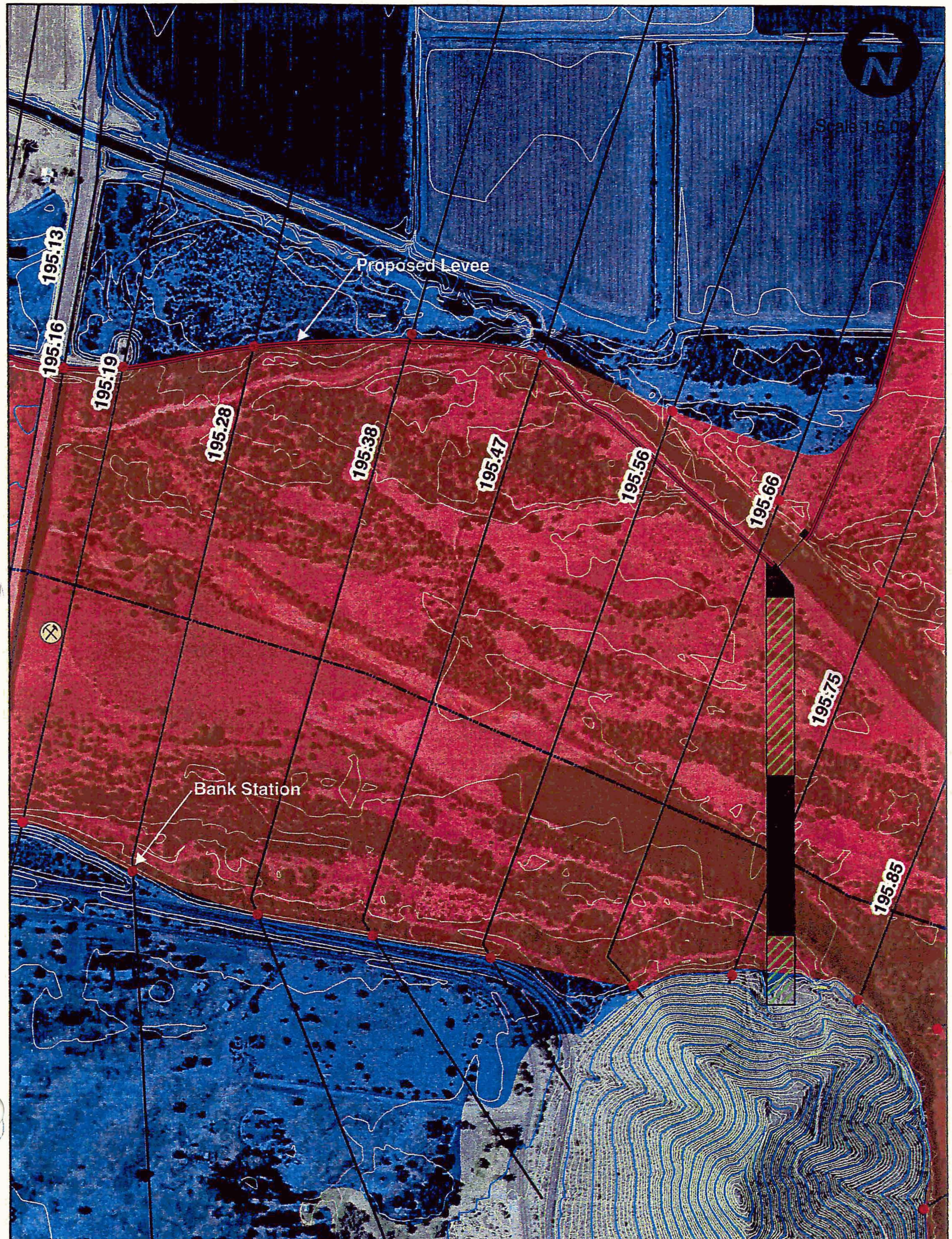
$$= 1.95\left(\frac{175^{0.82}}{0.38^{0.23}}\right)\left(\frac{11.6}{175^{2/3}}\right)^{0.93} - 11.6$$

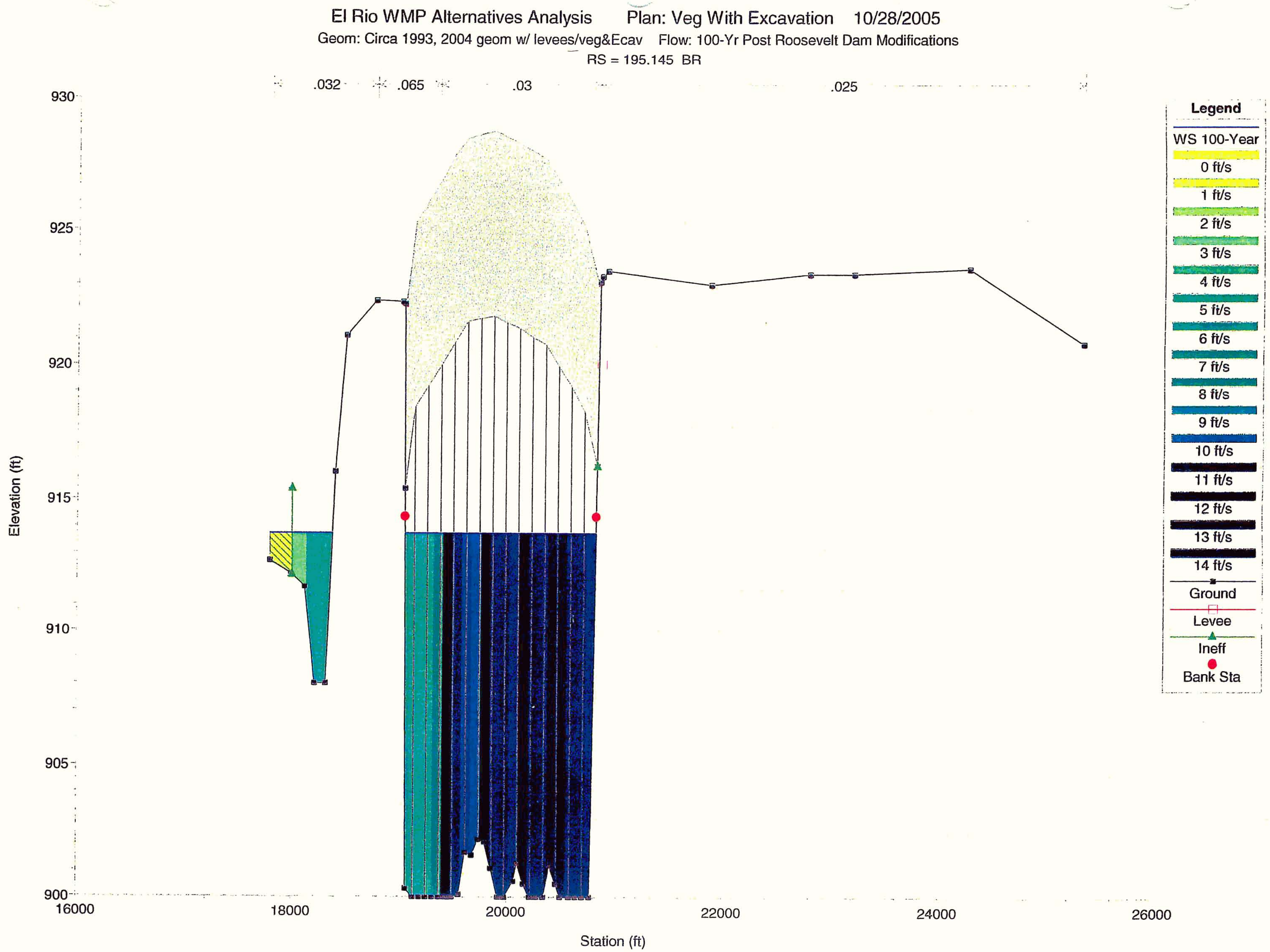
$\underline{d_s = 55.3^{ft}}$

(If $D_{85} = 18^{mm}$, $d_s = 16'$)

Designed by: Checked by:

BWCDD Diversion Structure




El Rio WMP Alternatives Analysis Plan: Veg With Excavation 10/28/2005
Geom: Circa 1993, 2004 geom w/ levees/veg&Ecav Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 195.145 BR
.032
.065
.03
.025
Elevation (ft)
930
925
920
915
910
905
900
16000
18000
20000
22000
24000
26000
Station (ft)
Legend
WS 100-Year
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
7 ft/s
8 ft/s
9 ft/s
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11 ft/s
12 ft/s
13 ft/s
14 ft/s
Ground
Levee
Ineff
Bank Sta