FLOODPLAIN AND DRAINAGE ASSESSMENT REPORT

S.H. 96A (4th St.) Bridge over the Arkansas River
Pueblo, Colorado

Prepared for

Colorado Department of Transportation
Region 2

and

Figg Bridge Engineers, Inc.

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1. INTRODUCTION

1.1 Project Review

This report presents the conceptual drainage and hydraulic studies for the proposed replacement or rehabilitation of State Highway 96A (SH96A) or 4th Street bridge over the Arkansas River and Union Pacific (UP) and Burlington Northern Santa Fe (BNSF) rail yards. The studies were performed by Ayres Associates for the Colorado Department of Transportation (CDOT).

An analysis of the existing local drainage facilities was performed using the rational method and information obtained from the City of Pueblo (COP) on existing storm sewer systems both east and west of the bridge. Precipitation values were obtained using a combination of "The Storm Drainage Design Criteria and Drainage Policies for City of Pueblo, Colorado" (COP 1997) and "Drainage Design Manual" (CDOT 1995). Other hydrologic information was obtained from the above two sources or the Urban Drainage and Flood Control District's (UDFCD) "Urban Storm Drainage Criteria Manual" (UDFCD 2001). This analysis was used to formulate a conceptual design for any additional drainage facilities that may be necessary. Additional facilities include detention, inlets, and storm sewer connections and upgrades resulting from the increased impervious area caused by the bridge widening.

The Arkansas River at the project reach drains approximately 4,790 square miles of a basin ranging in elevation from the continental divide at 14,433 ft to 4,600 ft at the bridge location. Pueblo Reservoir, completed in 1976, reduced the peak flood discharges upstream of Wild Horse-Dry Creek to a maximum of 6,000 cubic feet per second (cfs). Riverine and rail yard hydraulic modeling was performed using HEC-RAS, a 1-dimensional steady-state river analysis system (HEC 2001). Additionally, the Arkansas River in the immediate vicinity of the bridge was modeled with RMA-2v, a 2-dimensional depth-average hydrodynamic model (WES 1996). Flood discharges were determined using the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) and checked by four other methods. Scour and stream stability analyses were performed in accordance with Federal Highway Administration (FHWA) Hydraulic Engineering Circulars Nos. 18, 20, and 23 (FHWA 2001). The scour and hydraulic analyses will provide information for evaluating pier placement alternatives. Additionally, the scour analyses provide insight into the required depth of bridge pier foundations.

1.2 Data Acquisition

Ayres Associates personnel conducted a site visit on May 2-3, 2001, to gather field notes, take photographs, and become more familiar with the drainage and hydraulics associated with the site (see **Appendix A** for field notes).

Abel Engineering provided the results of their survey from August 2001 consisting of topographic information of the channel for approximately 1,000 feet upstream and downstream of the road crossing. In addition, the survey contained cross sections of the rail yard upstream and downstream of the existing bridge. This information was used as the hydraulic model geometry in the vicinity of the bridge.

United States Geological Survey (USGS) 7.5 minute quadrangle maps in conjunction with field investigation were used to delineate drainage basins. Figg Bridge Engineers provided an aerial photograph that was used to create the 2-dimensional hydraulic model and to assess potential options for local drainage solutions. Roadway alignment and cross section information was provided by PBS&J, while bridge sketches and conceptual designs were provided by Figg Bridge Engineers.

Existing local drainage information was collected during the May 2-3, 2001 field visit and from information provided by the City of Pueblo's Drainage Engineer, Dennis Meroney. This information included a statement of assumed design storm frequency and plan views of both systems.

2. LOCAL DRAINAGE

2.1 Project Description

The project limits for the SH96A (4th Street) Bridge Project are located within the NW ¼ of Section 36, Township 20, Range 65 West and the NE ¼ of Section 35, Township 20, Range 65 West. The project is located in the southwest side of the City of Pueblo and is partially situated within the Central Business District (CBD) as delineated by the City of Pueblo. A project vicinity map is shown below in **Figure 2.1**.

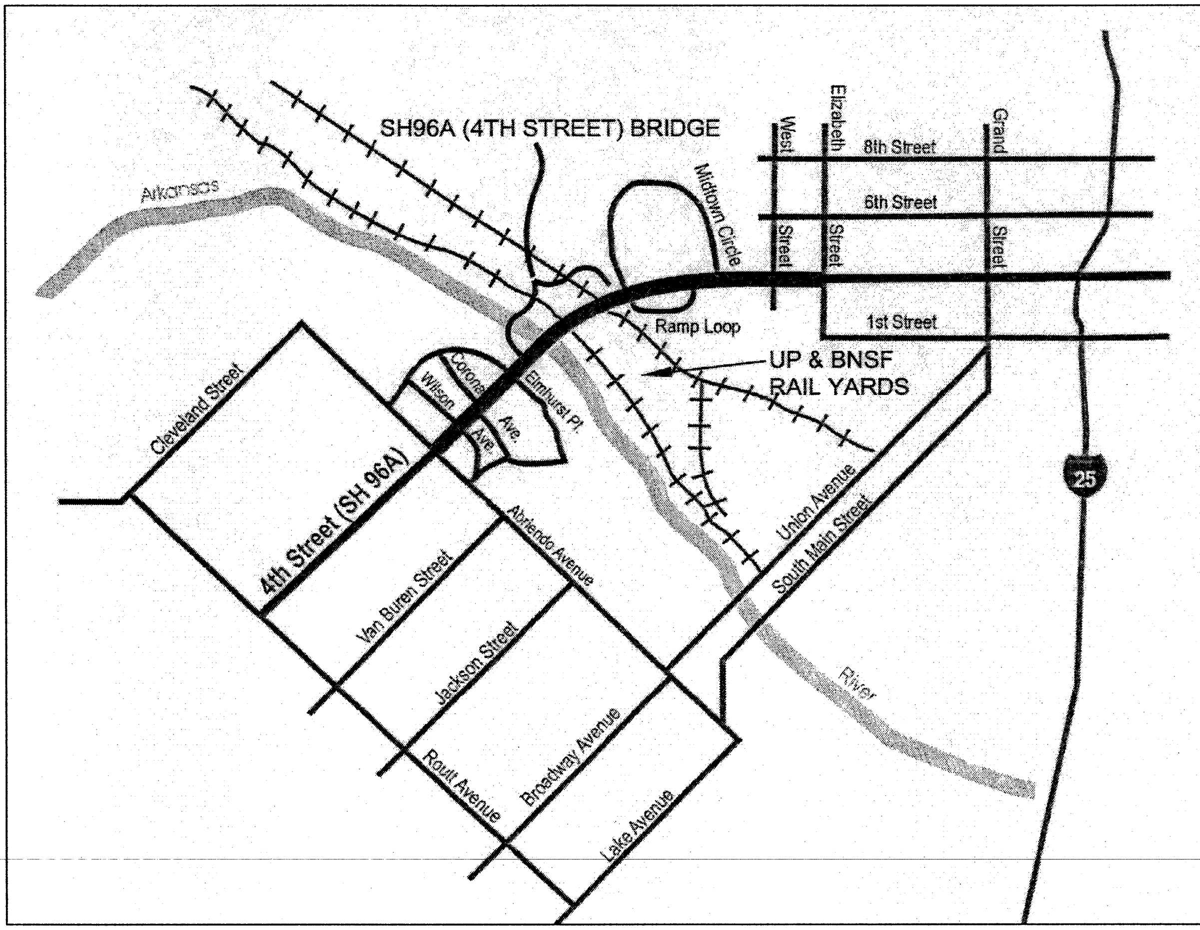


Figure 2.1. Location map.

The existing bridge spanning both the Arkansas River channel and the UP and BNSF rail yards is approximately 1,074 ft long and includes a total of six piers (one in the river and five in the rail yard area). This bridge project proposes to either replace or rehabilitate the existing bridge to a total width of 104 ft to handle the increase in average daily traffic since the existing bridge was constructed in the 1950s. This section presents conceptual findings of ways to mitigate the increase in runoff. To accomplish this task, runoff values were calculated for existing and proposed conditions to determine the increase that the proposed bridge will cause. Design runoff values for the storm sewer systems located to the east and west of the bridge were not known. These values were estimated using the rational method. Hydrologic design information was not available for either the west side or east side storm drains.

2.2 Drainage Areas

The 4th Street bridge and areas east of the bridge are located in the CBD and therefore currently drain into the CBD storm sewer system that begins with inlets and a 27 inch reinforced concrete pipe (RCP) at Midtown Circle Drive and 4th Street. The existing bridge has a four-lane cross section and is approximately 1,074 feet long and 68 feet wide. Its proposed replacement or rehabilitated size will have a 104 feet wide four-lane cross section. This proposed cross section includes 10 feet wide shoulders and a 10 feet wide pedestrian/bikepath on either edge.

Drainage of the 32.60 acre basin west of the bridge collects into a storm sewer system that discharges directly to the Arkansas River under the existing bridge. This area is not located in the CBD and therefore, falls under different criteria than the bridge and eastern areas.

2.3 Precipitation and Land Use

The "Storm Drainage Design Criteria and Drainage Policies for City of Pueblo, Colorado" (COP 1997) were used for precipitation criteria. This manual provided intensities for the 5-, 10-, 25-, and 100-year frequency storms from 5 to 60 minute durations. The computed times of concentration are 18.62 minutes for the west side basin and 5 minutes for the bridge deck. The intensities, durations and frequencies for 5 and 18.6 minute times of concentration in this study are presented in **Table 2.1**. The Rational Method has been used for all local drainage runoff calculations mentioned in this report.

Table 2.1. Intensities, Durations, and Frequencies.

Storm Frequency	Duration	Intensity (in/hr)
5-year	5 minute	5.28
5-year	18.6 minute	3.10
25-year	5 minute	7.20
25-year	18.6 minute	4.20
100-year	5 minute	9.24
100-year	18.6 minute	5.40

Existing and proposed ground cover for 4th Street, including the bridge, is pavement and thus both conditions will have the same runoff coefficient (C). This is considered to be 100 percent impervious and has a C coefficient of 0.88, 0.92, and 0.93 for the 5-, 25-, and 100-year storm events respectively. Land use for areas located in the basin west of the bridge were determined by percentage of each use type determined through field investigation records and aerial photography. The runoff coefficients calculated for each land use type are shown in **Appendix B**.

2.4 Criteria

This drainage study was performed in accordance with the CDOT "Drainage Design Manual" except where the City of Pueblo storm drainage policies were more restrictive.

Specifically, where CDOT requires the use of a 5-year frequency minor storm, the City of Pueblo requires the use of a 25-year frequency minor storm in the CBD. This changes the requirement for storm sewer design, allowable gutter flow, and gutter spread width allowances. In all cases, the criteria set forth by CDOT for the 5-year storm have been met in addition to meeting the 25-year minor storm criteria set forth by Pueblo for the CBD. Both criteria agree on the use of a 100-year frequency major storm.

For the specified 45 mph design speed, CDOT criteria specifies that the minor storm gutter flow spread width can extend through the shoulder and into 4 feet of one driving lane. Pueblo criteria for spread width in the CBD specifies that 10 feet of one driving lane must be left free of inundation in the 25-year event. These spread width criterion influence whether there is a need for inlets and their spacing along the bridge and roadway.

2.5 Existing Conditions

Basins on the east and west sides of the bridge are currently drained by storm sewer systems with assumed 25- and 5-year storm capacities respectively. The existing systems are described below.

2.5.1 East of Bridge

The system east of the bridge was designed by Sellards & Grigg and constructed in 1979. It was designed for the CBD standard 25-year storm capacity. Hydrologic design information for this system, however, was not available from Sellards & Grigg or the City of Pueblo. For this study, existing flows to the initial inlets at Midtown Circle Drive and 4th Street were estimated using the rational method. These inlets currently collect minor storm flows from the 4th Street Bridge, areas between the bridge and the inlets, and excess flows from the basin west of the bridge. Its capacity was assumed equal to the computed 42.8 cfs peak runoff resulting from the 25-year storm for the areas draining to the inlet under existing conditions. The peak discharge rate reaching Midtown Circle Drive during the 100-year storm was calculated as 80.9 cfs and is used as the allowable 100-year peak discharge from the eastern project limit.

2.5.2 West of Bridge

It is not known by Ayres Associates when the west side system was constructed or what its design capacity is. Through field investigation, Ayres Associates has estimated that this storm sewer drains approximately 32.6 acres west of the bridge. Pursuant to conversations with Dennis Meroney of the COP, it has been assumed that this storm sewer was designed for a 5-year event. During further stages of the project the capacity of this storm sewer should be checked to insure that the existing conditions have been modeled accurately.

Collection for this system begins with inlets at the intersection of Abriendo Avenue and 4th Street and ends with inlets approximately 50 feet west of the west bridge abutment. Discharge from this system flows directly into the Arkansas River through two 36-inch RCP pipes. The outlet end for this system contains no erosion protection and flows down a vertical masonry wall to the floodplain below. Extensive erosion is occurring both above and below the masonry wall as a result of the unprotected outlet. As a result, sections of the storm sewer pipe have broken off and fallen into the floodplain as shown in **Figure 2.2.**

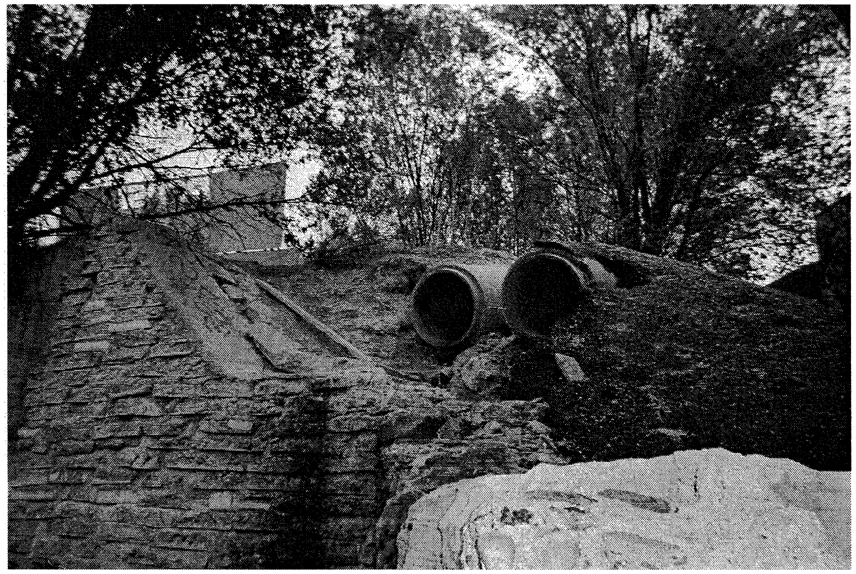


Figure 2.2. Broken sections of storm sewer pipe.

2.6 Proposed Conditions

The center line of the proposed alignment for the new 4th Street Bridge (if the replacement option is chosen) is located approximately 100 feet north of the existing center line alignment. It is approximately the same length (+12 feet) as the existing bridge, but it will create more runoff due to its 36 feet increase in width. This increase in runoff will be adequately compensated for if the west basin storm sewer capacity is increased to the 25-year peak discharge as a part of this project. If, however, the west basin storm sewer is reconstructed to a capacity less than the 25-year peak discharge, detention may be required. Detention volumes presented in this report were calculated to account for the possibility that the west basin storm sewer will be reconstructed to carry only the 5-year peak discharge, which is assumed to be the current storm sewer capacity.

2.6.1 West Side Basin

As mentioned earlier, the west basin 36-inch double RCP storm sewer is in disrepair and should be increased in size to carry the 25-year storm. Accordingly, the existing west side storm sewer underneath 4th Street that currently connects to the double 36-inch pipe storm sewer will be removed and replaced within the limits of the street reconstruction. This section and appurtenant inlets should be designed to increase the capacity from the assumed 5-year storm to the 25-year storm in order to remove additional flow from the street upslope from the bridge. As a result, flow onto the bridge will be minimized and the need for inlets on the bridge to satisfy the multiple spread width criteria will be avoided.

When the west basin storm sewer is reconstructed within the project limits, energy dissipation will be required at the outlet near the west bridge abutment to prevent erosion from occurring and adding sediment to the Arkansas River. This is a sensitive issue because of the proposed Legacy Project and the future recreational use in this reach of the Arkansas River. Consequently, the energy dissipator will need to fit in to the natural environment or be buried. Phase II of the Environmental Protection Agency's National Pollution Discharge Elimination Standards (NPDES) will cover the Pueblo Area beginning in

2003. It is not clear at this point what level of stormwater quality treatment will be required under Phase II at this location. The issue will need to be resolved at a later project phase.

One option for providing energy dissipation includes installing a storm sewer pipe (buried or on the surface) down the west abutment slope and daylighting it into a concrete dissipation basin. Concrete hanging baffle basins are very effective at dissipating energy in these types of situations. The disadvantage of this method is that the basin would be fairly large and visible considering its location in the floodplain. The concrete basin would then discharge into a grass swale to carry to flow to the Arkansas River. Grassed swales are considered by the UDFCD as a BMP for stormwater quality and may satisfy additional requirements that will be in place by the time this project is constructed. In addition, this is a low maintenance method of treating stormwater.

Another option is to construct an aesthetically sensitive cascading open channel drop, combined with a stilling basin to dissipate the energy prior to flowing into the river. This would require a riprap basin at the bottom of the slope and a cross drainage structure (bridge or culvert) for the trail, but may look more natural considering the future planned use of the area. This solution would also include a grassed swale to the Arkansas River.

2.6.2 Mitigating Runoff Increases

If the west side storm sewer is reconstructed with a 25-year capacity, then detention will not be required to mitigate runoff increases. A discharge summary table that supports this conclusion is presented in **Table 2.2**.

Table 2.2. Summary Table for Discharges at East 4th Street Bridge Abutment.

Frequency Storm Event	Condition		
	Existing (cfs)	Proposed – 25-Year West Storm Sewer Capacity (cfs)	Proposed – 5-Year West Storm Sewer Capacity (cfs)
25-year	42.8	18.0	49.5
100-year	80.9	58.1	89.6

If detention is required, a proposed location for the detention pond is in the middle of the Midtown Mall loop access road. It has been conceptually sized to hold excess 25- and 100-year storm runoff, limiting the peak discharge rate to historic levels for areas extending from the west bank of the Arkansas River to Midtown Circle Drive. Detention capacities were determined for the 25- and 100-year storms using the FAA method taken from the UDFCD Volume 2 Criteria Manual. Calculations were made at 5-minute intervals for a period of two hours.

Calculated detention volumes use existing runoff calculations, which assume that the existing west basin storm sewer is sized for the 5-year event. Under this assumption, the computed peak discharge rates entering the existing storm sewer at 4th Street and Midtown Circle Drive are 11.4 cfs and 80.9 cfs for the 25- and 100-year storms, respectively. Detention volumes have been calculated to keep the peak discharge rates at or below these values under proposed conditions. Runoff up to the 25-year event will be collected at inlets and completely routed through the detention pond. Much of the 100-year flow will bypass the inlets. Total flow rates allowed in the storm sewer at 4th Street and Midtown Circle Drive will include discharge from the detention pond and any remaining flow entering the inlets from the street. The estimated detention volumes necessary for the 25-year and 100-year storms are 0.09 acre-feet and 0.11 acre-feet respectively.

The detention pond outlet will consist of a weir box with a small orifice inlet and a crest set at the 100-year water surface elevation. The orifice will be sized to carry the peak 25-year outflow, while the weir box will be sized to handle the peak 100-year outflow. The outlet pipe will connect into the existing manhole at 4th Street and Midtown Circle Drive. A schematic of the proposed storm drainage facilities, including the detention pond, is shown in **Figure 2.3**.

2.6.3 Gutter Flow and Spread Width

According to CDOT, the maximum spread width for the 5-year storm on the proposed bridge is 14 feet on either side. Using an equation shown in UDFCD Storm Drainage Criteria Manual (as referenced in CDOT Drainage Design Manual) for triangular gutters, an average longitudinal slope of 2.1 percent and a street cross section slope of 2 percent produces a spread width of 11 ft on either side. Pueblo criteria for the 25-year storm allows a spread width equal to the shoulder width plus the entire first driving lane. Calculated width for the 25-year storm is equal to 12.5 ft and therefore less than the 22 feet allowed. Finally, the 100-year CDOT criteria states that street flow should be limited to no greater than 6 inches over the crown, whereas we have calculated that the spread width will not reach the crown. The runoff and spread width values are provided in **Table 2.3**.

Table 2.3. Runoff and Spread Width Calculations.

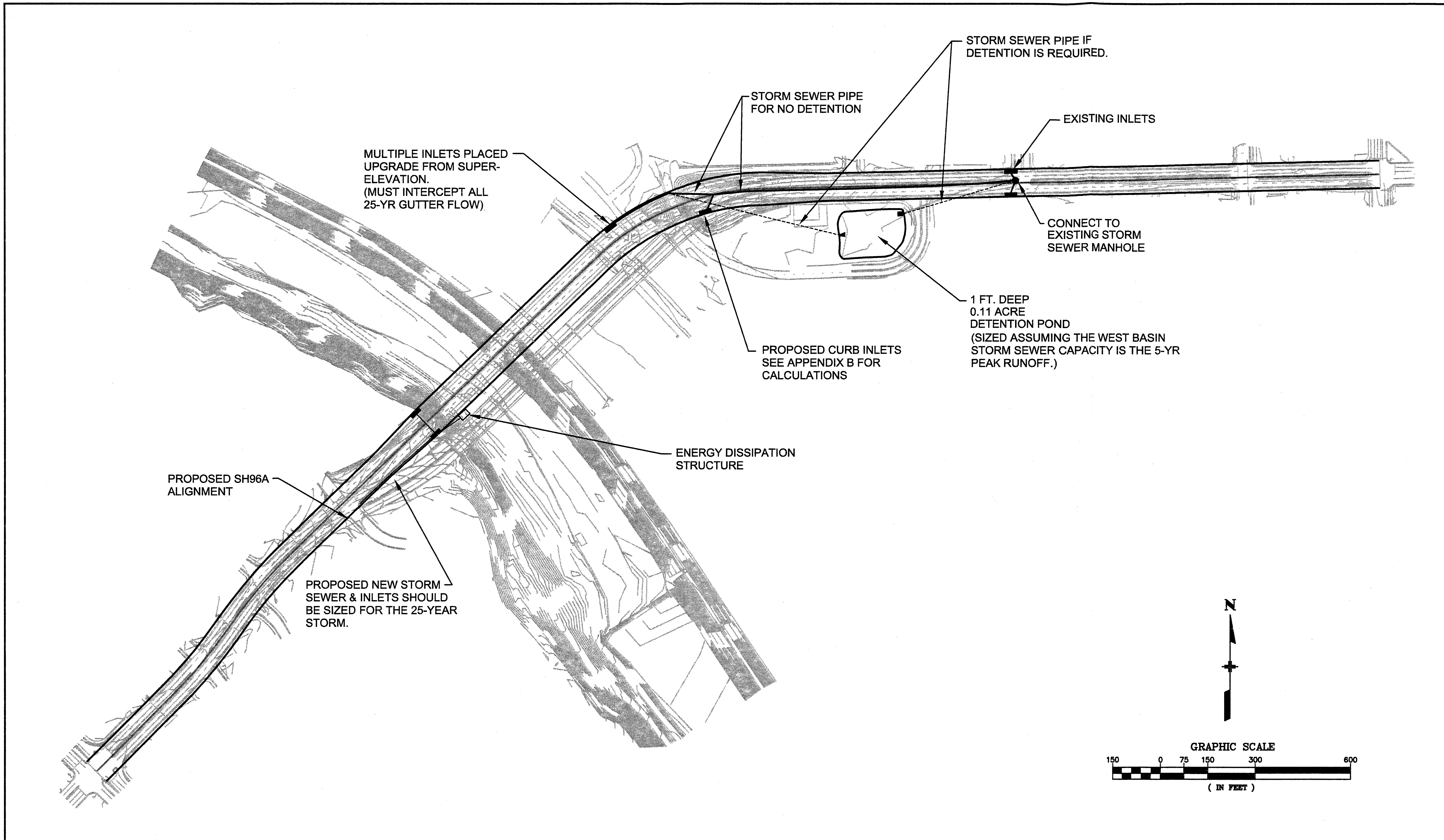
Flow Event	Runoff for Proposed Bridge (cfs)	Spread Width		
		Proposed Bridge (ft)	CDOT Criteria (ft)	Pueblo Criteria (ft)
5-year	12.65	12.3	14.0	N/A
25-year	18.03	14.1	N/A	22.0
100-year	58.07*	21.9	50.0	50.0

*Total is runoff for 100-year storm plus difference b/w West side 100-year and 25-year runoffs

2.6.4 Bridge Deck Drainage and Inlets

It has been established that environmental concerns prohibit the release of bridge runoff to the Arkansas River floodplain or channel for this project. Furthermore, the runoff cannot be released onto the rail yard as this would aggravate an existing drainage problem in that area. All runoff on the bridge, therefore, must be conveyed to the east end of the bridge, either by surface flow in the gutters or in a bridge deck drainage system.

The CDOT "Drainage Design Manual" and "Bridge Design Manual" both direct the designer to the FHWA publication HEC-21 "Design of Bridge Deck Drainage" for design standards and procedures. The use of inlets on bridges is discouraged, unless absolutely necessary, for several reasons in FHWA HEC-21. These reasons include maintenance; safety, and freezing concerns. For instance, HEC-21 states, "an ideal solution is no inlets. The fewer inlets, the easier to maintain them--clogged inlets are a widespread maintenance problem." Drainage figures and calculations supporting the conceptual drainage design are shown in Appendix B.



NOTES:	SCALE: 1"=300'	DESIGN: JTU DRAWING: JER	AYRES ASSOCIATES

FIGURE 2.3
PROPOSED DRAINAGE FACILITIES SCHEMATIC

Table 2.3 shows that inlets are not needed in order to meet the spread width criteria on the bridge. However, the need for inlets on the bridge may be dictated by other factors including, but not limited to:

1. Superelevation of the curve on the east end of the bridge that would cause flow in the north gutter to cross traffic
2. Need to remove flow prior to crossing bridge joints
3. Bridge icing considerations

A system of at least 4 inlets (if the Neenah R-3922 inlet is used) should be placed in the north gutter upstream of the superelevation to intercept the 25-year storm event. This will prevent gutter flow from crossing traffic lanes during the minor storm as defined by the City of Pueblo.

Also, a system of inlets in the south gutter may be required upstream of the eastern bridge joint. These inlets will prevent water from crossing the joint in the minor storm. The curb height will prevent gutter flow from spilling out onto the abutment slopes, with or without these inlets.

In addition to the inlets mentioned above, it may be beneficial to place other inlets along the length of the bridge, if they can be shown to increase motorist safety during local flooding or winter icing conditions. It should be noted, however, that any runoff collected by inlets will have to be conveyed within a bridge deck drainage system to the east end of the bridge. This leads to increased complexity in the design of the bridge, as well as increased maintenance costs.

Depending on the numbers and placement of inlets on the bridge, additional inlets may be required along 4th Street east of the bridge. These would be curb opening inlets designed according to City of Pueblo standards.

2.7 Stormwater Pollution Prevention

Pollution prevention measures must be provided for stormwater discharges during construction (temporary) and for permanent stormwater discharges from the proposed system. In addition, pollution prevention measures must be taken in the river during pier construction. Permanent pollution prevention measures may include the proposed west-side energy dissipation structure (hanging baffle dissipator or cascade chute and riprap basin) used for erosion control. In addition, the proposed grass swale downstream of the energy dissipator will filter the stormwater discharge prior to entering the Arkansas River.

Temporary pollution prevention measures related to construction onsite will include placement of silt fences and hay bales to prevent sediment from entering any stormwater facility or natural drainageway. The extent of required temporary pollution prevention measures in the river depends on whether the 1 or 2 pier option is pursued. The long span options (one pier in the Arkansas River floodplain) may only require placement of silt fence around the construction area while the pier is erected. The moderate span options (two piers in the floodplain) may require more extensive protection because of the placement of the proposed east pier in the low flow channel. Most likely this alternative will require greater care including the use of turbidity barriers.

3. ARKANSAS RIVER HYDRAULIC ANALYSES

One- and two-dimensional hydraulic analyses were performed on a reach of the Arkansas River including the 4th Street Bridge to assess the potential impacts of the proposed SH96A (4th Street Bridge) project. The 1-dimensional model was performed using the U.S. Army Corps of Engineers (USACE) HEC-RAS program while the 2-dimensional model utilized RMA2v with the SMS pre- and post-processor.

3.1 Channel Description

The modeled reach of the Arkansas River includes a 2000-foot length of the river, approximately centered on the existing bridge. The Arkansas River floodplain in the subject reach was channelized by the construction of a floodwall in 1923 following the devastating 1921 Pueblo flood. The concrete-lined floodwall forms the left limit of the floodplain and protects the City of Pueblo from flooding. The right limit of the Arkansas River floodplain through the study reach is comprised of a natural bluff that runs parallel to the low flow channel. Between these two constraints the floodplain cross section has relatively flat cross slopes. A typical cross section is shown below in **Figure 3.1.**

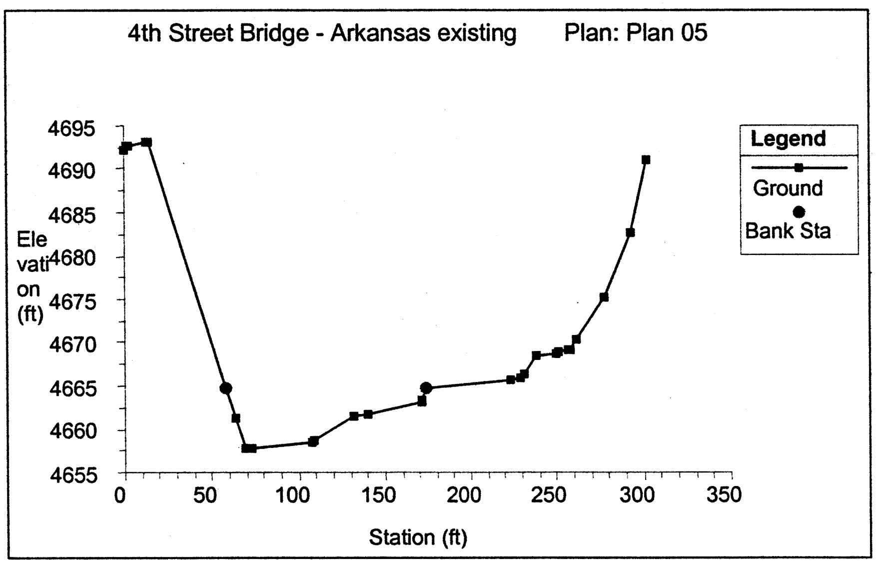


Figure 3.1. Typical Arkansas River cross section at SH96A.

According to the FIS, the floodplain contained by the floodwall is capable of carrying a discharge of over 100,000 cfs. Pueblo Reservoir's construction in the 1970's, however, decreased the flood flows in the channel to a point where the capacity is not likely to be exceeded. Information from the FIS and from the Bureau of Reclamation suggests that the capacity of the floodwall far exceeds the capacity required for the 100- or 500-year flood, when the effects of Pueblo Dam are considered.

The low flow channel bed consists mainly of gravel and cobble material. Vegetation in the floodplain is characterized by grasses near the channel while weeds and bare soil cover the right overbank area downstream of the bridge. Upstream of the bridge, willows and small trees line the right overbank causing an increase in roughness.

3.2 Historical Hydraulic Studies

The Arkansas River in the vicinity of the SH96A crossing has not been the subject of any known detailed hydraulic or hydrologic studies and is plotted as a Zone A or Approximate Floodplain by FEMA.

3.3 Hydrologic Analysis

A riverine hydraulic analysis requires hydrologic parameters for model input. The steady state one-dimensional and two-dimensional simulations performed in this case required discharge flow rates for the 100- and 500-year storms.

According to a discharge summary table provided by the USBR the frequency event controlled by Pueblo dam depends on the hydrologic method used. All but two of the seven methods however, support that the reservoir releases a maximum of 6000 cfs until at least the 500-year event. A table of discharges for each of the methods used by the USBR is presented in Appendix B. Wild Horse-Dry Creek, therefore, provides the main component of peak flows in the Arkansas through the project reach since the 100- and 500-year flows listed in the FIS for Wild Horse Creek are 19,500 cfs and 39,500 cfs respectively. These are much greater than the maximum release from Pueblo Reservoir and correspond with the 20,000 cfs and 40,000 cfs flows listed for the Arkansas River in the FEMA FIS. The USACE, in cooperation with the US Bureau of Reclamation, is in the process of revising the hydrologic model of the entire Arkansas basin, however, that effort is not anticipated to be complete until at least 2003. Therefore, riverine flood discharges were determined from the FEMA FIS and the table of expected discharges from Pueblo Reservoir provided by the USBR.

A figure illustrating the Wild Horse - Dry Creek watershed is provided in Appendix B. The flood frequency relationship reported in the FEMA FIS for Wild Horse-Dry Creek was checked using four methods including:

- Colorado Department of Natural Resources Technical Manual 1, "Manual for Estimating Flood Characteristics of Natural-Flow Streams in Colorado" (McCain and Jarrett 1976)
- USGS Water Resources Investigations Report 99-4190, "Analysis of the Magnitude and Frequency of Floods in Colorado"
- USGS Water Resources Investigations Report 87-4094, "Techniques for Estimating Regional Flood Characteristics of Small Rural Watersheds in the Plains Region of Eastern Colorado"
- NRCS TR-55, "Urban Hydrology for Small Watersheds"

The first three of these methods are regional regression equations that use the drainage basin area to compute various recurrence interval peak discharges. Equations and calculations for each of these methods are presented in Appendix B.

The total drainage area was computed by delineating the drainage boundaries on USGS quadrangle maps. The maps were registered digitally and a basin area of 87.3 square miles was determined using the CAD package Microstation. Technical Manual 1 values differed only slightly from the FEMA FIS discharges while the other three methods were above and below the FEMA FIS values as shown in **Figure 3.2**. It appears that a rough average curve between all five of these methods would follow the FIS relationship relatively well and therefore support the use of FEMA FIS discharges.

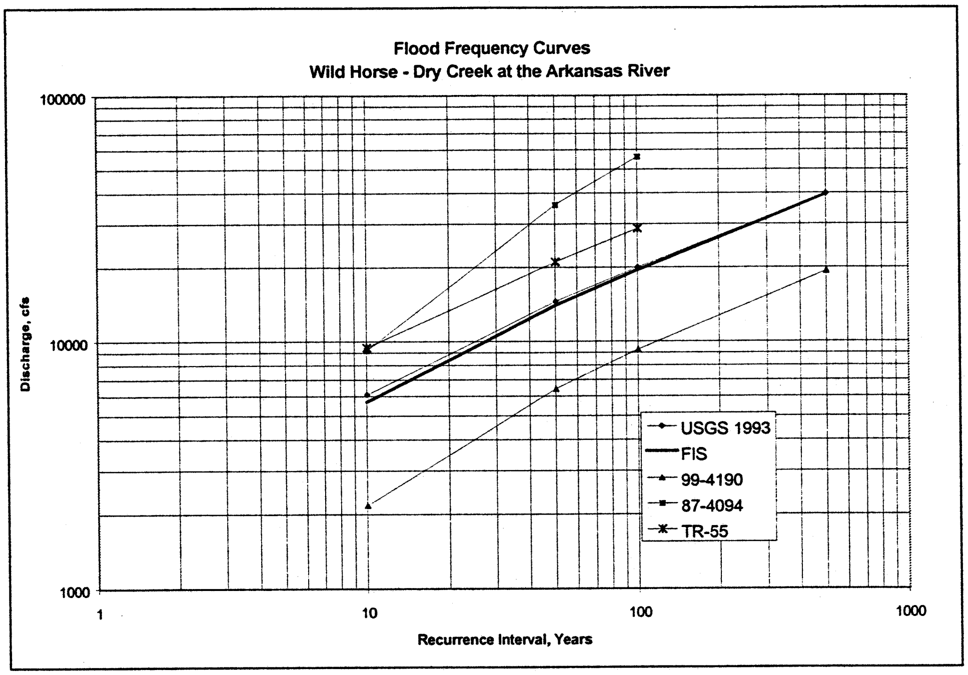


Figure 3.2. Regional regression equation FIS comparison.

Different flow criteria were used in this study for bridge hydraulic impacts and for scour analysis. The FEMA Arkansas River FIS flows were used without modification for determining the impact of any proposed bridge configurations on the 100-year water surface elevations. Although the floodplain through the project reach is mapped as Approximate Zone A, this determination was performed to assure consistency with the current FEMA and FHWA regulations.

The 6,000 cfs maximum release from Pueblo Dam was added to the 100-year and 500-year FEMA FIS Arkansas River discharges to obtain stream flow input values for the 2-dimensional models of the Arkansas River used in scour analysis and floodwall impact assessment. The resulting values represent conservative maximum discharges for each return interval.

The discharge used for the rail yard pier scour analysis was 19,750 cfs, 50 percent of the total 500-year peak flow in Wild Horse - Dry Creek. This discharge in the rail yard represents a conservative flow estimate for scour analysis purposes. This value will be refined (and possibly reduced) by a more detailed analysis conducted in later stages of the project.

3.4 Criteria

This bridge hydraulic analysis determines the impact of proposed construction on the existing floodplain, scour depths, and provides information for evaluation of pier placement alternatives.

All FEMA floodway and floodplain criteria must be met. Where FEMA floodways have been established, no net rise in the 100-year water surface elevation may result from the proposed bridge. The Arkansas River floodplain currently has no regulatory floodway through the project reach. The proposed bridge must cause no more than 1 foot of total water surface rise compared with natural conditions. Natural conditions are defined as the hydraulic conditions that would exist if no bridges or piers were present in the floodplain.

Criteria related to bridge scour are derived from FHWA and CDOT policy. Bridges should be designed to withstand the scour from a 100-year flood (or a smaller flood if it produces deeper scour) with all appropriate structural and geotechnical safety factors fully satisfied. Furthermore, bridges should be designed to withstand the scour from a superflood (usually the 500-year flood) with safety factors greater than or equal to 1.0.

3.5 Steady-State Hydraulic Simulation Model: HEC-RAS

The riverine hydraulic analysis was performed using HEC-RAS, a 1-dimensional steady-state hydraulic simulation program developed and maintained by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (HEC 2001). Given a steady-state discharge, HEC-RAS solves the energy and continuity equations for reaches and networks of waterways. In addition to the Arkansas River model, a steady-state hydraulic model of the rail yard was used for assessing the 500-year scour potential at piers located in the rail yard.

3.5.1 Model Development

The upstream model limit was set approximately 1,000 feet upstream of the existing 4th Street Bridge and outside of the upper limit of any bridge contraction effects. The downstream model limit was set at 800 feet downstream of the Historic Arkansas River of Pueblo (HARP) diversion structure to assure that a reasonable variation in tailwater depths would have a minimal impact on model results at the bridge, and to allow the model to calculate velocities at the toe of the diversion structure for future stability analyses. Cross sections were developed in Autocad from topographic mapping supplied by Abel Engineering. The survey data provided included channel bathymetry for the low flow channel. Cross sections were cut at locations to represent changes in roughness, channel width, depth, and variations in overbank configuration likely to impact hydraulic properties at the bridge.

Reach lengths between cross sections and overbank elevations were obtained from the project topographic mapping in Autocad. Channel lengths were measured along the channel thalweg, and overbank reach lengths were measured from the appropriate overbank center of conveyance at each cross section. Channel and overbank roughness estimates were based on field investigation and photographs taken on the May 2-3 site visit. Roughness values for the main channel were set at 0.03, while the overbanks included roughness values for the concrete floodwall of 0.013, 0.035 in grassy areas, and 0.05 in willow and tree covered areas upstream of the bridge. The existing bridge over the Arkansas River was modeled based on the project survey data, CDOT construction plans for the bridge and photographs from the site visit. This data indicated that the existing bridge has a 202 foot span on the west side of the river and a 303 feet span extending into the rail yard on the east side. The one pier located in the Arkansas River floodplain was modeled as shown on the CDOT plans, a tapered pier with an 8 foot bottom width and 5 foot top width. This bent was aligned parallel to the predominant flow path. Deck structure width was not a consideration because flow stays below the existing low chord during all model runs. The proposed bents were modeled as 5 foot wide columns with 15 foot wide footings extending 3 feet above existing ground. Two proposed pier configurations were modeled, one having a single pier at approximately the same cross section station as the existing pier, and the other with two piers separated by a 142 foot span.

The rail yard model was developed in order to obtain an estimate of the velocity for pier scour computations discussed later in this report. As a conservative expediency, bridge piers were omitted from this model.

The 100- and 500-year riverine floods were modeled in the Arkansas River model, while only the 500-year flood was modeled for the rail yard model. For all models the downstream boundary water surface elevation was set using normal depth and a representative friction slope.

3.5.2 Model Results

The HEC-RAS model led to the conclusion that neither pier configuration will affect the 100-year Arkansas River Water Surface Profile by a significant amount. The greatest increase occurred with the two-pier configuration and was only a 0.78 foot rise when compared to the natural conditions (no bridge) model. Summary tables of the 1-dimensional hydraulic analysis results at the upstream bridge face and at the approach section are given in **Tables 3.1 and 3.2** respectively. **Figure 3.3** presents a water surface profile plot of the models (see **Appendix C** for detailed hydraulic output).

Table 3.1. Summary of Hydraulic Results at Upstream Bridge Face.

Conditions	Q = 20,000 cfs			
	WSEL (ft-NAVD)	Channel Velocity (ft/s)	Δ From Natural (ft)	Δ From Existing (ft)
Natural Conditions (no bridge)	4671.83	13.1	--	--
Existing Conditions	4672.06	12.8	0.23	--
1 Pier Option	4672.22	12.5	0.39	0.15
2 Pier Option	4672.61	12.0	0.78	0.38

Table 3.2. Summary of Hydraulic Results at Upstream Approach Section.

Conditions	Q = 20,000 cfs			
	WSEL (ft-NAVD)	Channel Velocity (ft/s)	Δ From Natural (ft)	Δ From Existing (ft)
Natural Conditions (no bridge)	4672.73	11.3	--	--
Existing Conditions	4672.86	11.2	0.13	--
1 Pier Option	4672.98	11.0	0.25	0.12
2 Pier Option	4673.24	10.7	0.51	0.38

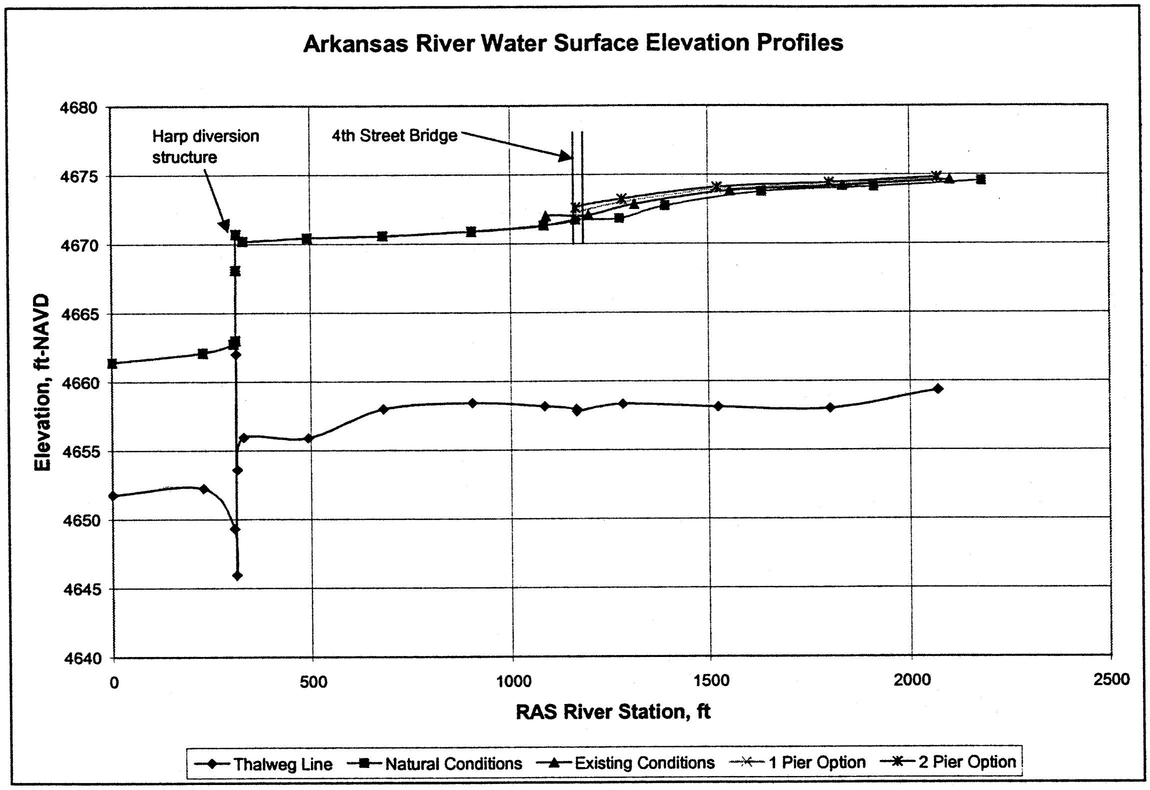


Figure 3.3. Water surface profile.

4. SCOUR ANALYSIS

As water flows around a pier or abutment or through a constriction, the erosive action causes scour of the bed and banks. The Federal Highway Administration report FHWA NHI 01-001 (HEC-18, FHWA 2001) describes total scour at highway crossings by adding three components:

1. Long-term aggradation and degradation of the river bed
2. General Scour at the bridge
   a. Contraction scour
   b. Other general scour
3. Local scour at the piers or abutments

These three components are assumed to occur independent of and additive to each other. This assumption leads to conservative scour depth estimates.

4.1 Arkansas River Scour Analysis and Floodwall Impact Assessment

An RMA-2v 2-dimensional model of the Arkansas River was developed to assess the impacts of the proposed bridge alternatives on the existing concrete floodwall revetment and to predict scour depths at the bridge.

RMA-2v is a 2-dimensional depth-average velocity finite-element hydrodynamic model maintained by the United States Army Corps of Engineers Waterways Experiment Station (WES) (WES 1996). Ayres Associates has enhanced RMA-2v to account for pier drag, equivalent roughness, weir flow, and pressure flow conditions. RMA-2v solves the depth-averaged 2-dimensional equations of motion using a Finite Element Method solver. RMA-2v requires a geometric representation of the modeled region and boundary conditions (stage or flow) at all open boundaries of the model.

The geometric representation used by RMA-2v is a Finite Element Mesh (FEM or mesh). The mesh is defined by points located in space and connected into planar triangular or rectangular elements. Each of these elements is assigned a material type corresponding to the roughness characteristics of the area bounded by that element. The model study reach encompasses the Arkansas River floodplain inundated by the 500-year riverine flood event and extends from 900 feet downstream of the proposed 4th Street alignment to 1,200 feet upstream of the proposed bridge site. The models also include artificially low entrance and exit regions to enhance model stability. These regions are placed far from the bridge and do not affect the results at the bridge.

The model geometry was developed in the SMS preprocessor to RMA-2v using aerial photography of the area and was highly refined in the bridge vicinity to accurately resolve detailed hydraulic conditions at the bridge site. Model elevations were assigned from the 2001 survey provided to Ayres Associates by Abel Engineering. Bridge pier locations for the long span (one channel pier) and moderate span (two channel piers) alternatives were directly incorporated into the model geometry. They were located in the channel using the August 2001 span layout provided to Ayres Associates by Figg Bridge Engineering. **Figure 4.1** presents the FEM used for the 100-year event. The 1- and 2-pier alternatives vary from each other only in the location and number of the piers in the channel.

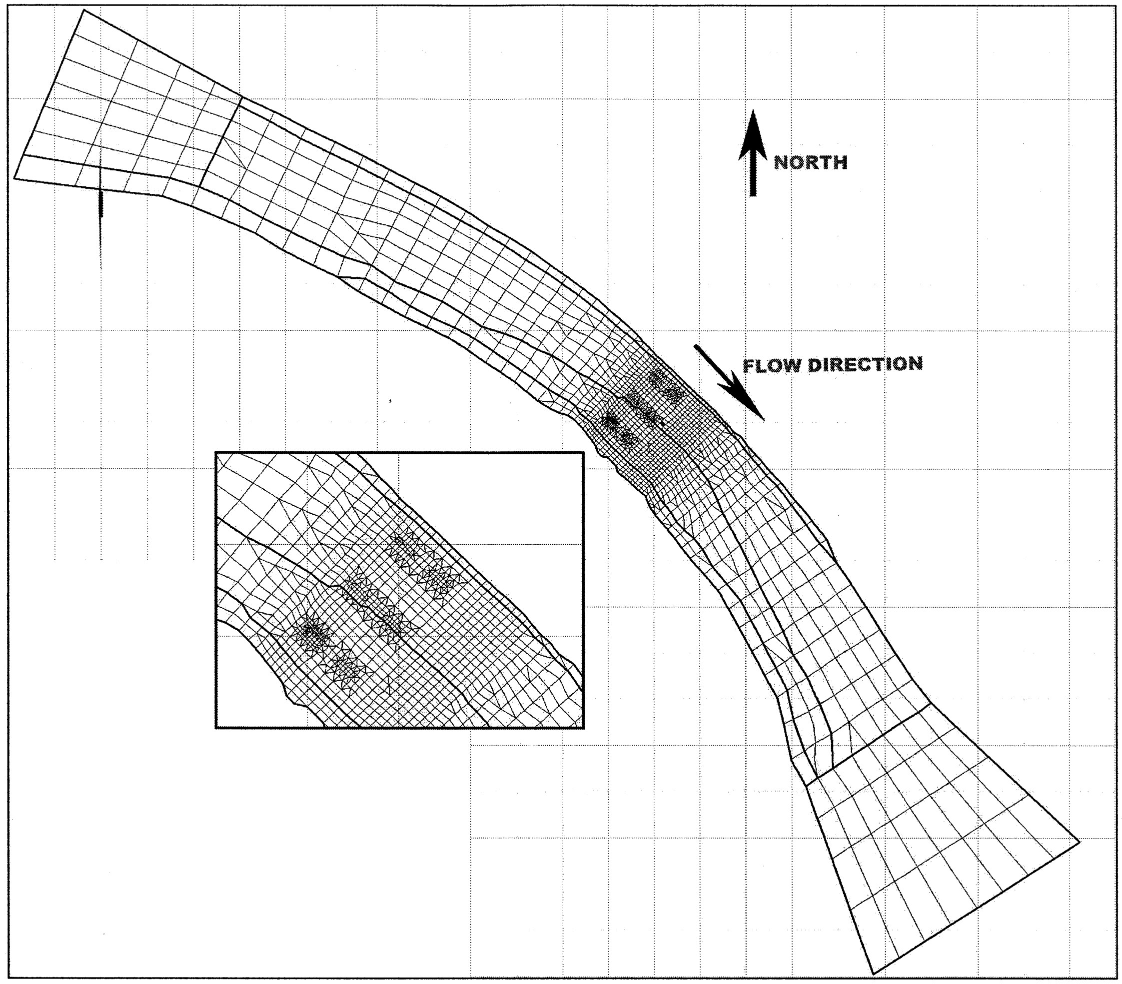


Figure 4.1. 100-year FEM.

Initial Manning roughness coefficients (Manning's n) for the channel and inundated overbank regions were assigned based on aerial photography, field observation, ground photographs, and tabulated values. The channel roughness values were adjusted to calibrate the 2-dimensional predicted water surface elevations (WSELs) at the bridge site to the water surface elevations predicted by the HEC-RAS 1-dimensional model of the Arkansas River for the same discharge rates.

One Hundred and 500-year riverine flood flows were simulated for both the long-span and moderate span alternatives. The downstream water surface boundary condition was based on the predicted HEC-RAS WSEL immediately upstream of the Pueblo Diversion for the West Plains Energy power plant. The discharge values used for the upstream flow boundary condition represent a conservative maximum discharge for the 100- and 500-year riverine flood events and were developed by superimposing the maximum regulated discharge from Pueblo Dam with the Arkansas River FEMA FIS discharges. **Table 4.1** presents the boundary conditions used for the 100- and 500-year scour analysis. Note that since some of the 500-year flow in Wild Horse-Dry Creek will be trapped behind the floodwall, it is conservative to apply the full 500-year flow to the Arkansas River as we have done.

Table 4.1. Boundary Conditions for 2-Dimensional Scour Analysis.		
Event	Q cfs	Downstream Water Surface Elevation (ft-NAVD)
100-year scour	26,000	4672.2
500-year scour	46,000	4676.3

Table 4.2 presents the hydraulic properties at the bridge for the 100-year and 500-year scour flows, respectively.

Table 4.2. Hydraulic Properties at Upstream Bridge Face.				
Variable	Alternative			
	Long-span		Moderate-span	
	100-year	500-year	100-year	500-year
Discharge (cfs)	26,000	46,000	26,000	46,000
Max WSEL (ft-NAVD)	4676.2	4681.0	4676.2	4681.2
Average Floodplain Velocity (fps)	11.2	13.7	10.76	13.1
Maximum Local Velocity at Wall (fps)	13.0	17.1	15.9	19.7

Note that the moderate span option subjects the floodwall to 24-30 percent greater local velocities than the long span option. **Figure 4.2** and **Figure 4.3** present velocity contour plots of the 100-year scour discharge for the long-span and moderate span alternatives, respectively.

The floodwall on the left bank is subject to contraction scour, bendway scour, flow impingement scour, and is affected by scour hole overlap from Pier AR-2 in the moderate span alternative. Contraction scour is general bed lowering associated with flow acceleration through a constriction. Bendway scour is bed lowering on the outside of a bend associated with increased velocities and shear stresses on the outside of a bend. Local scour is a reduction in the bed level associated with flow redirection, plunging flow, and vortices produced by a blockage to flow such as a pier or abutment. Flow impingement scour is a form of local scour associated with high-velocity flow impingement on a wall or structure parallel to the general flow path. The bendway scour is not sensitive to the bridge alternative chosen and was therefore not computed for this study.

Impingement scour depths were predicted using techniques outlined in HEC-23 (FHWA 2001). Note that impingement scour depths do not account for local velocity effects. Potential local and contraction scour depths were predicted using the techniques outlined in HEC-18, 4th edition (FHWA 2001). **Table 4.3** presents predicted bridge pier scour depths. **Figures 4.4 and 4.5** present the potential scour depths for the 100-year and 500-year scour flows, respectively. Impingement scour depths are presented in **Table 4.4**. The potential scour depths extend into the underlying claystone shale material. These scour depths may therefore be reduced to reflect the resistance to erosion of this underlying shale. Any reduction would require examination and approval by a qualified geotechnical engineer with knowledge of the properties of the material.

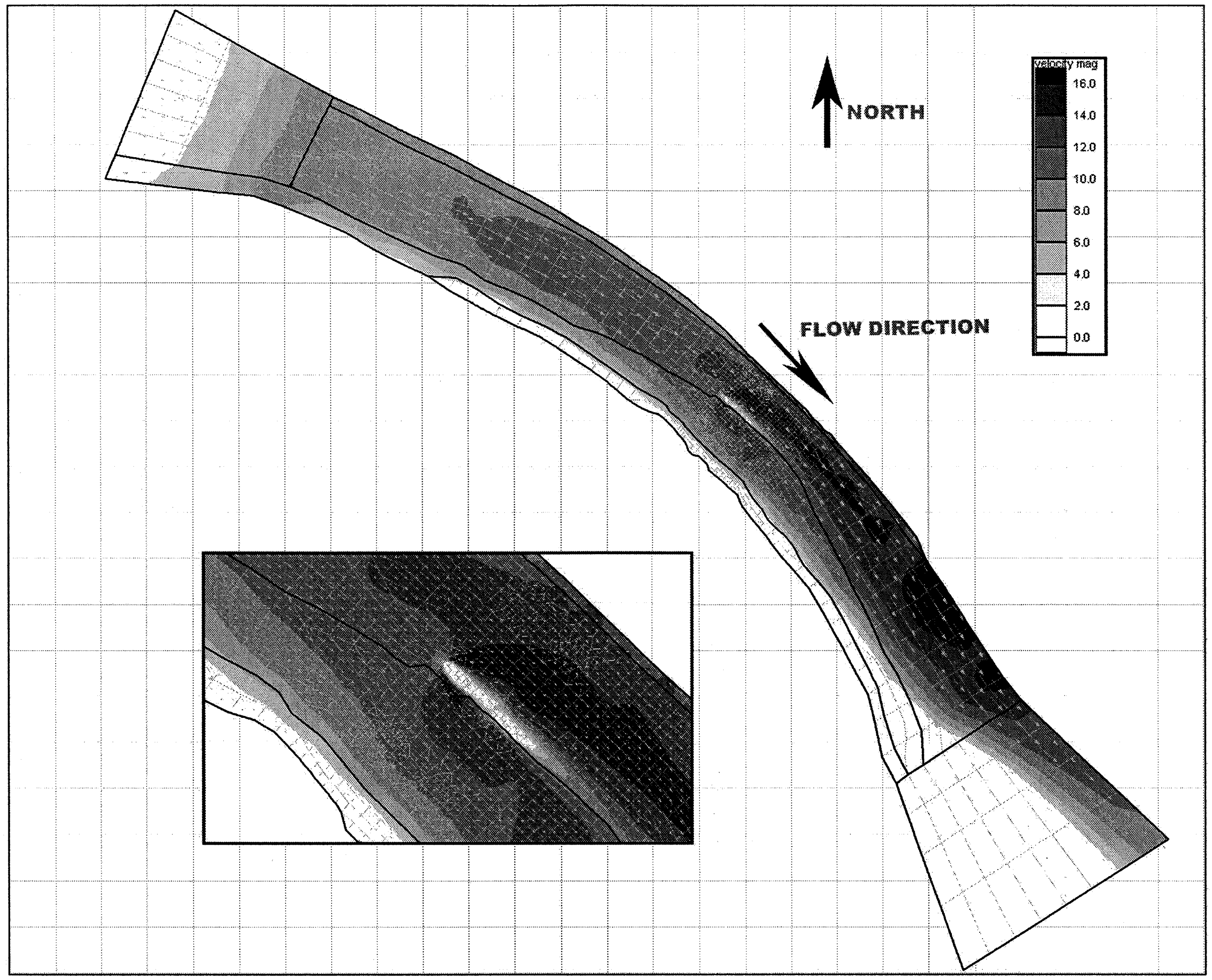


Figure 4.2. Long-span alternative; 100-year scour flow velocity contour plot.

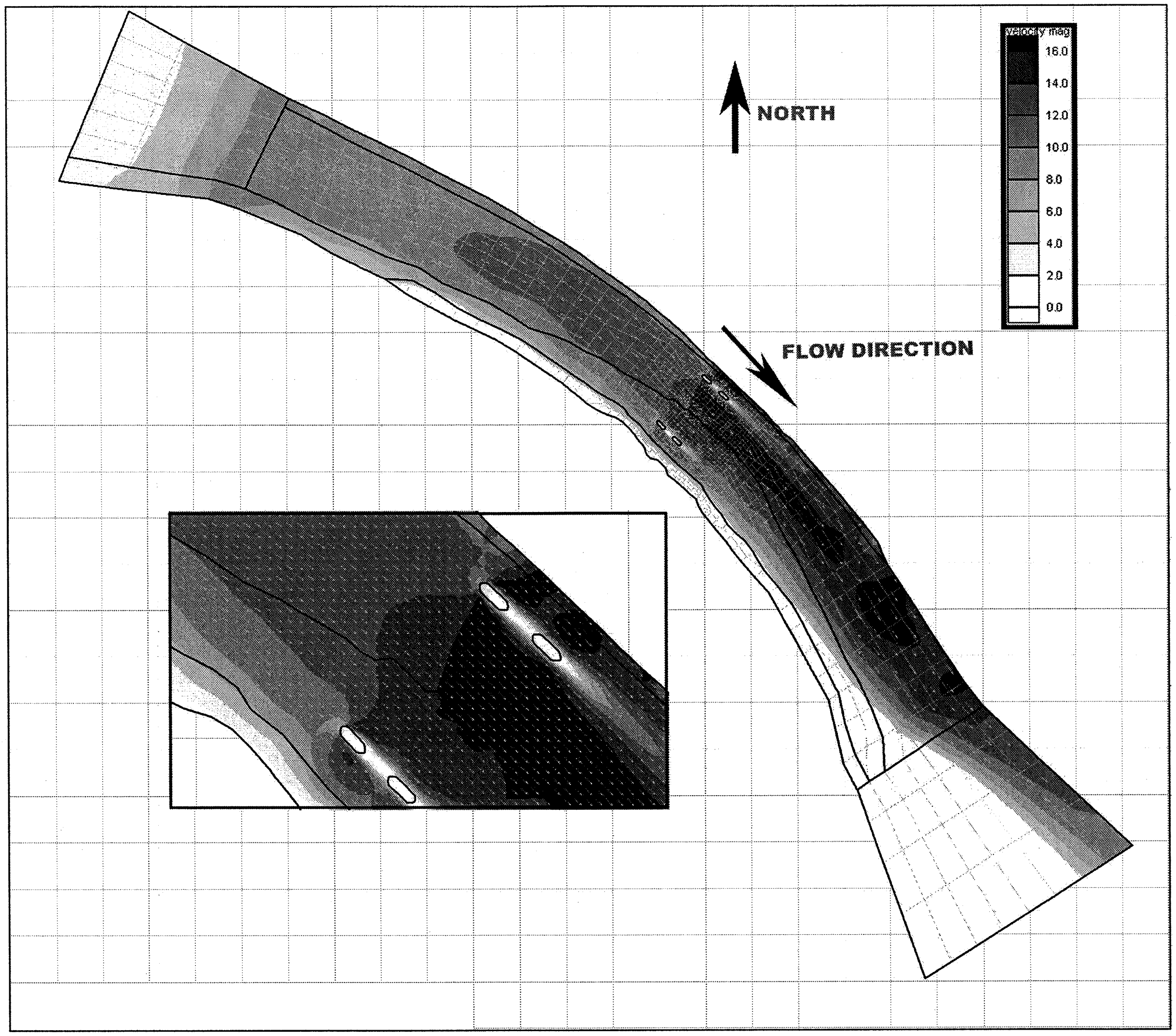


Figure 4.3. Moderate span alternative 100-year scour flow velocity contours.

The moderate span alternative subjects the floodwall to up to 30 percent greater flow velocities, local scour overlap from the pier AR-2, and up to a 34 percent increase in impinging flow scour depths compared to the long span alternative. Whether the increased flow velocity and potential scour depth constitute unacceptable impacts on the existing floodwall depends on the erodibility of the underlying claystone shale layer.

Table 4.3. Scour Depth Summary at Bridge Piers.

Pier		100-year event				500-year Event			
	Ground Elevation (ft-NAVD)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft-NAVD)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft-NAVD)
AR-2	4658.1	1.2	21.5	22.7	4635.4	1.7	23.4	25.1	4633.0
AR-1a	4662.5	1.8	21.9	23.7	4638.8	3.0	23.6	26.6	4635.9
AR-1	4665.7	1.2	21.5	22.7	4643.0	1.7	23.4	25.1	4640.6

Table 4.4. Scour Depth Summary at Wall.

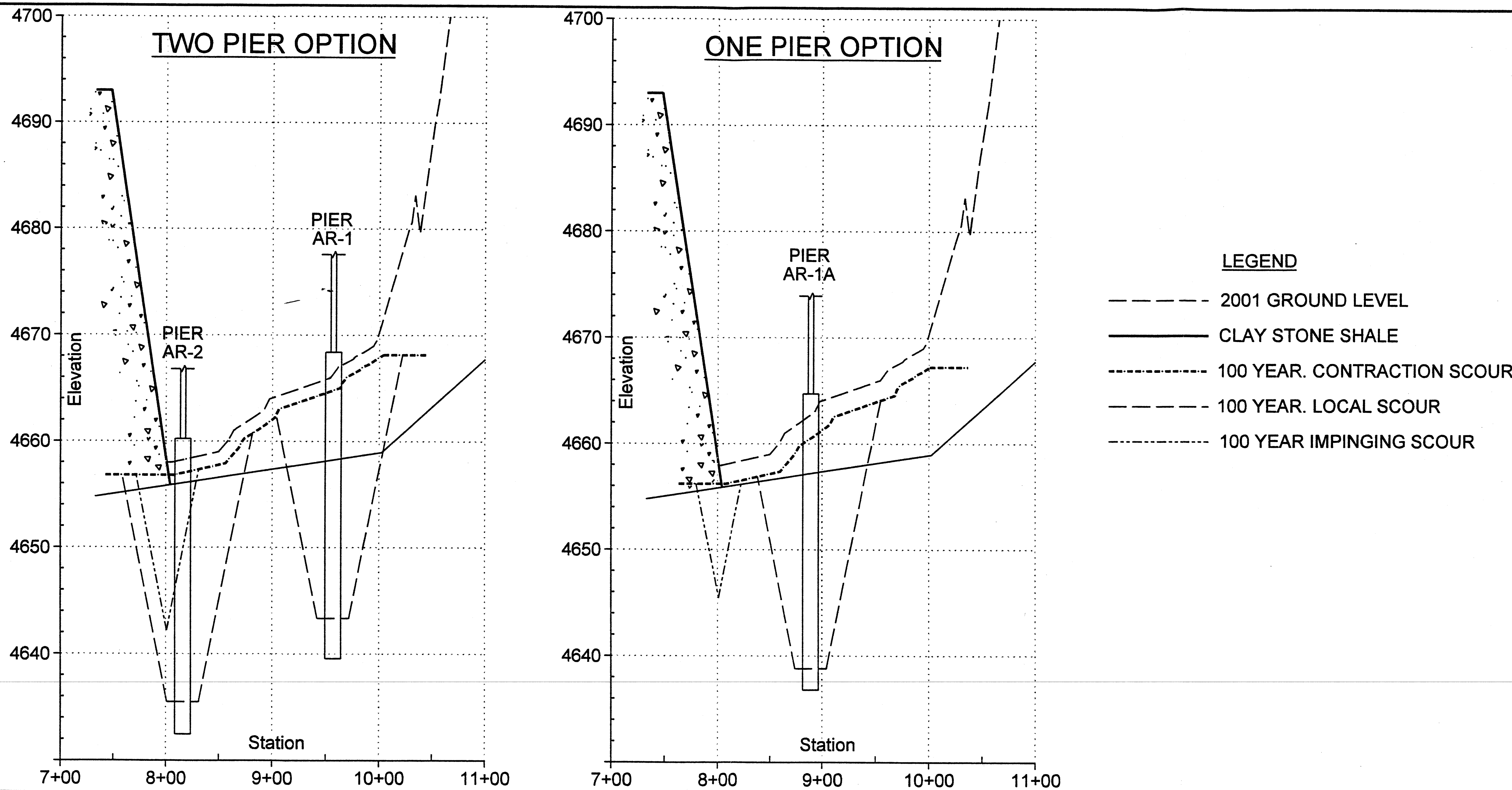
Alternative		Ground Elevation (ft-NAVD)	Contraction Scour (ft)	Impinging-Flow Scour (ft)	Total Scour (ft)	Scour Elevation (ft-NAVD)
Long-Span	100-year	4658.0	1.8	10.7	12.5	4645.5
	500-year	4658.0	3.0	15.3	18.3	4639.7
Moderate-Span	100-year	4658.0	1.2	14.6	15.8	4642.2
	500-year	4658.0	1.7	20.5	22.2	4635.8

According to the 1923 bridge and floodwall plans, the floodwall is cast into the shale layer at the bridge site. If this material is resistant to scour under the high-velocity flow conditions predicted at the bridge crossing, then both the long-span and the moderate-span alternatives will produce similar scour impacts on the existing floodwall. However, if the claystone shale material is significantly erodible under flood-flow conditions, then the moderate span alternative would produce more severe hydraulic and scour conditions at the floodwall and could therefore not be recommended without addressing potential impacts to the floodwall.

A brief investigation into the erodibility of the bedrock using the "Erodibility Index Method" (Annandale 1999) indicates that accounting for the bedrock material properties may reduce the predicted scour by only 30 percent. This reduction is not enough to negate the possible negative impacts of the moderate-span option.

4.2 Diversion Structure Scour Analysis

Scour at the diversion structure for the 100-year flood was computed using the USBR equation outlined in HEC-23 (FHWA 2001) for vertical drops. Hydraulic inputs to the USBR equation were taken from the previously discussed HEC-RAS model. Using the USBR equation, the computed post scour elevation at the downstream toe of the diversion is 4642.7 feet, which represents an average scour depth of about 7.4 feet. It appears that a portion of the predicted scour has already occurred at the downstream toe of the structure, because there is a hole with the thalweg elevation only about 3 feet above the post-scour elevation. We conclude that the diversion structure would probably not fail in a 100-year flood event. If the diversion were to fail, however, the 100-year storm duration in the Wild Horse-Dry Creek drainage basin (assuming a NRCS type II storm) would probably not be long enough to allow the resulting headcut to move the 800 feet upstream to 4th Street. Consequently, the diversion structure scour is not expected to increase the scour depths at the 4th Street Bridge piers.



NOTES:

CROSS SECTIONS PLOTTED
LOOKING DOWNSTREAM

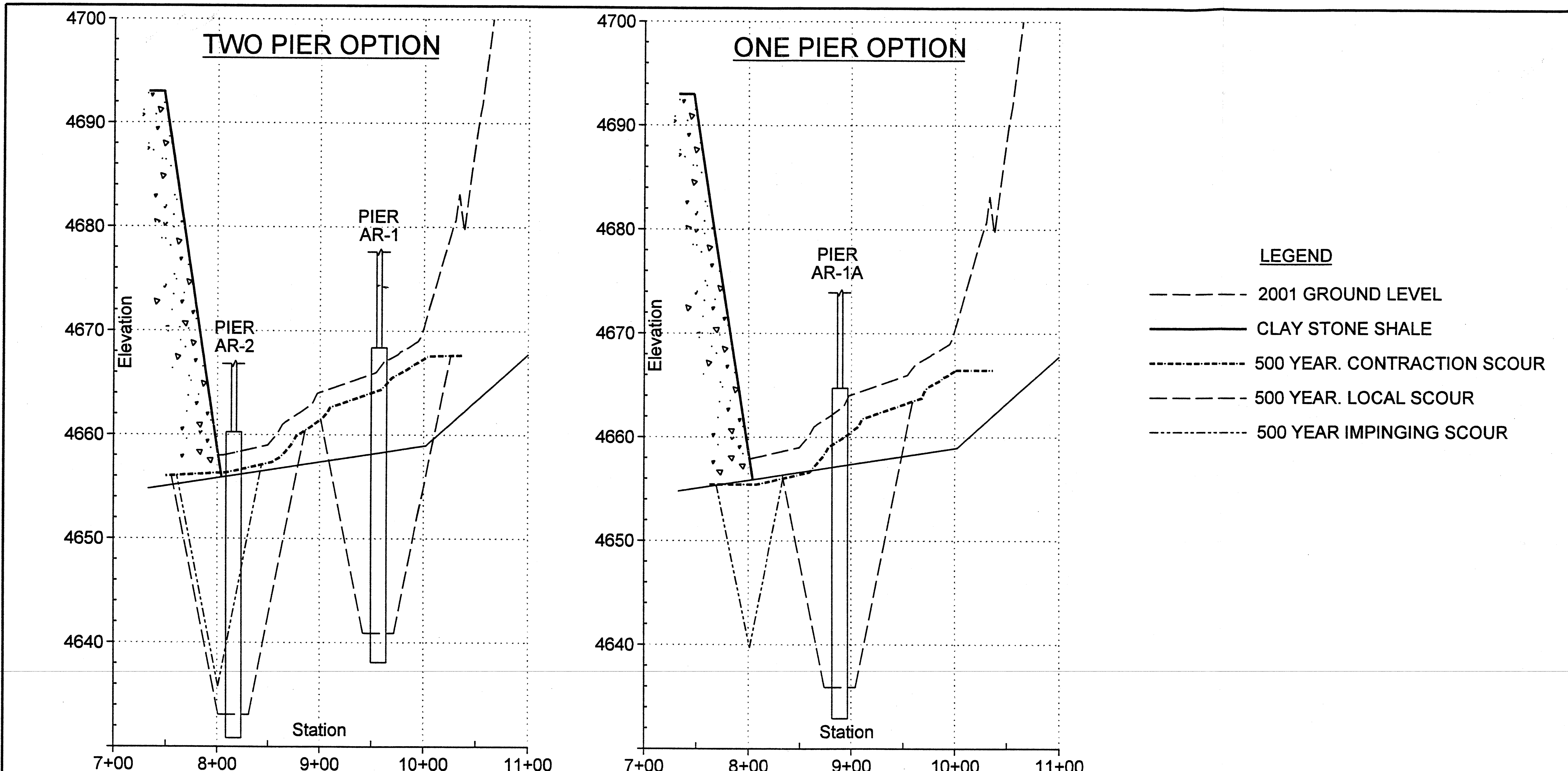
SCALE:

HORIZONTAL: 1"=100'
VERTICAL: 1"=10'

DESIGN: WMdeR
DRAWING: JER

AYRES ASSOCIATES

FIGURE 4.4
100-YEAR. SCOUR DEPTHS FOR PROPOSED
4TH STREET CROSSING OVER ARKANSAS RIVER



NOTES:

CROSS SECTIONS PLOTTED
LOOKING DOWNSTREAM

SCALE:

HORIZONTAL: 1"=100'
VERTICAL: 1"=10'

DESIGN: WMdeR
DRAWING: JER

AYRES ASSOCIATES

FIGURE 4.5
500-YEAR. SCOUR DEPTHS FOR PROPOSED
4TH STREET CROSSING OVER ARKANSAS RIVER

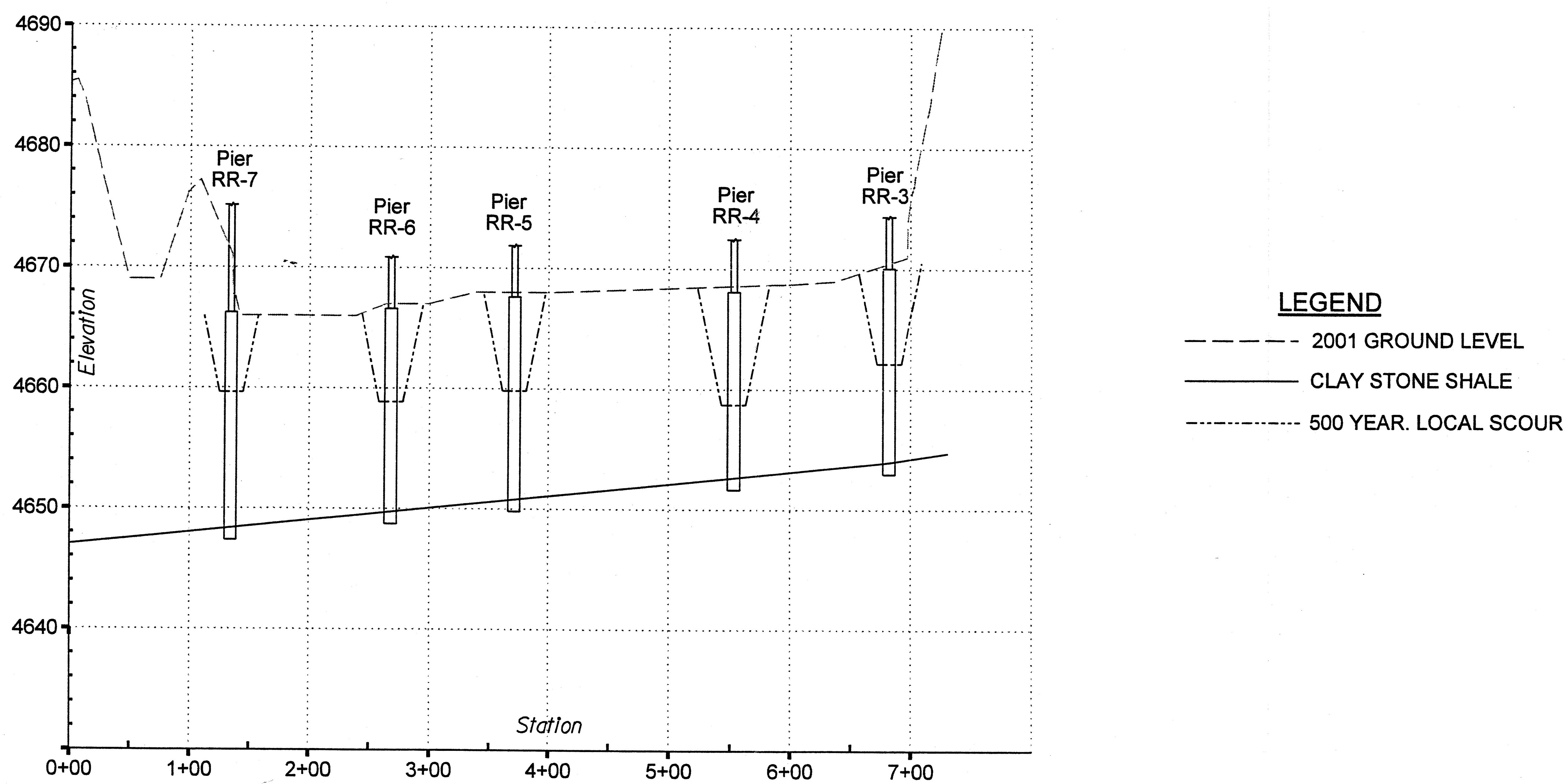
4.3 Rail Yard Scour Analysis

During the 100-year event, flow from Wild Horse – Dry Creek is completely contained by the existing east-bank levee north of the 11th Street Bridge. Thus, all of the flow continues into the Arkansas River, and the 4th Street Bridge piers in the rail yard north of the floodwall are not subjected to scouring flows. However during the 500-year event, flow is forced around the levee. A significant portion of the total discharge is diverted around the levee and into the rail yard. The rail yard north of the floodwall is not subject to aggradation or degradation, and contraction scour would not be expected in the rail yard beneath the 4th Street Bridge. For this reason only the local scour at each pier was computed for the proposed piers within the rail yard. The CSU Equation, as presented in HEC-18, 4th edition, is used for predicting local scour at piers (FHWA 2001).

A 500-year discharge of approximately 19,750 cfs flowing through the rail yard was incorporated into a HEC-RAS model to determine the effective velocity at each pier and the hydraulic depths associated with this flow. This discharge was conservatively estimated as 50 percent of the total 500-year peak flow in Wild Horse – Dry Creek. The appropriate velocity and depth were then used in the above equation to calculate the potential scour depths at each pier in the rail yard. The results are provided in **Table 4.5** and illustrated in **Figure 4.6**.

Table 4.5. Local Scour Through Rail Yard.

Pier Number	Approx. Ground Elevation (ft-NAVD)	Velocity (ft/s)	Flow Depth (ft)	Local Scour Depth (ft)	Resulting Elevation (ft-NAVD)
RR 3	4669.0	3.7	2.7	6.9	4662.1
RR 4	4666.0	5.8	4.7	7.9	4658.1
RR 5	4668.0	6.3	5.3	8.2	4659.8
RR 6	4668.0	7.0	6.3	9.1	4658.9
RR 7	4669.0	7.8	7.3	9.4	4659.6



NOTES:

CROSS SECTION PLOTTED LOOKING DOWNSTREAM

SCALE:

HORIZONTAL: 1"=100'
VERTICAL: 1"=10'

DESIGN: JJE
DRAWING: JER

AYRES ASSOCIATES

FIGURE 4.6
SCOUR DEPTHS FOR PROPOSED 4TH STREET BRIDGE OVER BNSF & UP RAIL YARDS

5. REFERENCES

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APPENDIX A
Site Visit Notes

Site Visit Notes

Project: 32-0444 4th Street Bridge Replacement in Pueblo
Site Visit Dates: 5/02/01 and 5/03/01
Site Visit Participants: John Hunt, Jason Ullmann

Meeting with Dennis Maroney, City Drainage Engineer

- 4th Street is classified by the City as a Prinipal Arterial.
- He gave us copies of the drainage system layout maps.
- The drainage area to the inlets and pipes on 4th street just west of the bridge has its westerly limit at Abriendo. The ground does get higher further west but there seems to be a rise at the east edge of the Abriendo intersection. Also, there are inlets that drain to a 15-inch storm drain that runs along the east edge of Abriendo carrying flow to the northwest, away from 4th street.
- He provided us with asbuilt maps of the storm drain that takes drainage from 4th street east of Midtown mall.
- We purchased a copy of the City's Drainage Criteria Manual.
- We borrowed a copy of the Corp report on the hydrology and hydraulics for Wild Horse/ Dry Creek and Goodnight Arroyo.
- Stormwater Quality: He said that Pueblo is a Phase 2 city, which means that by 2003 they will have to implement the same BMP's that Phase 1 cities must now use.
- He also indicated that CDOT is already held to the NPDES BMP requirements. These requirements regulate both the construction period (temporary BMP's) and the post-construction period (permanent BMP's). Therefore, in following the CDOT requirements, we'll be satisfying the NPDES Phase 2 requirements.
- Storm Drainage Design Criteria: Look at both CDOT and City criteria and follow the most restrictive.

Investigation of Drainage Patterns West of Bridge

- We walked the apparent drainage area west of the bridge in order to delineate the basin.
- We confirmed that Abriendo is probably the upper edge of the basin.
- Camera 1 Photo 39: Inlet at Abriendo and Carlisle, center median. This inlet is connected to the storm drain in Abriendo, drains away from 4th street.
- Cam1 Photo 38: Looking at the diversion structure d/s from the bridge. Photographer is standing on top of right bluff.
- Cam 1 Photo 37: On right bluff, just north of the fishing dock parking lot, looking at a stormwater inlet that carries flow to the river south of 4th street.
- Cam 1 Photo 36, 35: On the fishing dock, looking u/s at the bridge.
- Cam 1 Photo 34, 33: On the fishing dock, looking d/s at the diversion structure.
- Cam1 Photo 32: 18-inch cmp pipe entrance. This pipe flows to the river on the north side of the bridge. The entrance is located at the northeast corner of the 4th st. Elmhurst intersection.
- Cam 1 Photo 31, 30: Looking at the large (about 10 feet long) sump inlet on the north curb of 4th st. Just west of the bridge. This inlet has dual RCP's coming in from

another inlet opposite this one at the center median. This inlet also has dual exit RCP's coming out the back and to the outfall just north of the west abutment.

- Cam 1 Photo 29: Looking at rubble-stabilized area on top of the west bluff, just north of the bridge. Surface wash from the area behind the north curb.
- Cam 1 Photo 28: Looking at the outlet end of the small pipe draining a sump at the angle point in the north curb line just west of the bridge. Outlet is in the same location shown in Photo 29.
- Cam 1 Photo 27: Looking downstream along the downstream end of the 4th street local storm drain. The dual RCP's are carrying flow from the inlet in photo 31, 30 to a free overfall.
- Cam 1 Photo 26: Looking at outlet end of storm drain.
- Cam 1 Photo 25: Looking down at the drop from the storm drain outlet.
- Cam 1 Photo 24, 23: Continuation, drop from storm drain outlet.
- Cam 1 Photo 22: Looking up at apparent bridge deck drainage downspouts. These are visible at various points along the north cell, fifth cell and south cell. If water can get to these downspouts, they drop water onto floodplain, river channel, and railyard.
- Cam 1 Photo 21: Standing under bridge just west of water pier, looking at the storm drain outfall on the north side of the west abutment.
- Cam 1 Photo 20: Looking at the outlet end of the pipe whose inlet is shown on Photo 32.
- Cam 1 Photo 19: Looking at the 3-ft curb inlet on the west side of Corona St. just south of 4th St. An exit pipe from this inlet leads out toward the inlet shown in Photo 18.
- Cam 1 Photo 18: Looking at the 6-ft curb inlet on the east side of Corona just south of 4th St. An exit pipe flows out from this inlet toward the bridge.
- Cam 1 Photo 17: Looking at the 10-ft (approx) median curb inlet just west of the bridge. This inlet sends flow in dual RCP's to the inlet shown in Photo 31, 30.

Investigation of Drainage Patterns East of Bridge

- Cam 1 Photo 16: Looking at the apparent collection point for surface drainage from the paved area on the west side of the Midtown Mall. The collection point is in the foreground, at the southwest corner of the paved parking area. From here, the flow spills over into the rail yard on the north side of the old abutment. Flow goes around the abutment and along the east edge of the railyard to a pond further south.
- The paved parking area of the mall has area drain inlets throughout the area, providing evidence of the storm drainage system that we should try to find information on.

Investigation of Arkansas River and Rail Yard Hydraulics

- Cam 1 Photo 15: Looking downstream of the railroad bridge which is downstream of Main St. Bridge, and looking at I-25 & Santa Fe bridges
- Cam 1 Photo 14: Scourhole on upstream side at the nose of the center pier of the railroad bridge
- Cam 1 Photo 13: Looking upstream at Main and Union bridges from left bank flood wall

- Cam 1 Photo 12: Looking upstream at Main and Union bridges from left bank flood wall
- Cam 1 Photo 11: Looking downstream from Main Street Bridge – Panorama under bridge
- Cam 1 Photo 10: Looking downstream from Main Street Bridge – Panorama under bridge
- Cam 1 Photo 9: Looking downstream from Main Street Bridge – Panorama under bridge
- Cam 1 Photo 8: Standing on left bank flood wall looking across the river, under Main St. Bridge
- Cam 1 Photo 7: Standing on left bank flood wall looking across the river, under Main St. Bridge
- Cam 1 Photo 6: Standing on left bank flood wall looking across the river, under Main St. Bridge
- Cam 1 Photo 5: Standing on left bank flood wall looking across the river, under Main St. Bridge
- Cam 1 Photo 4: Storm drain outfall downstream face of right abutment
- Cam 1 Photo 3: Down stream of Main St. Bridge, looking at railroad branch between River Bridge , that goes towards Santa Fe (NE)

*This branch shows the supposed preferred flow path for flows behind the left flood wall.

- Cam 1 Photo 2: Union St. Bridge looking at storm drain outfll, downstream face right abutment
- Cam 1 Photo 1: Standing on left flood wall looking across river on a perpendicular line from the power plant
- Cam 2 Photo 39: Standing on left flood wall looking across river at approx. Arkansas River sec. 8
- Cam 2 Photo 38: Standing on left flood wall looking across river at approx. Arkansas River sec. 8 (AR8)
- Cam 2 Photo 37: Extension of AR8 into rail yard
- Cam 2 Photo 36: looking downstream from AR8
- Cam 2 Photo 35: Standing on left flood wall looking at cross sections of AR6 & AR7
- Cam 2 Photo 34: Standing on left flood wall looking at cross section of rail yard 9
- Cam 2 Photo 33: Looking downstream at cross section rail yard 9
- Cam 2 Photo 32: Standing at left flood wall looking at cross section of AR5
- Cam 2 Photo 31: Standing at left flood wall looking at cross section of AR5
- Cam 2 Photo 30: Looking cross the railyard at RY8
- Cam 2 Photo 29: Standing on left flood wall looking cross river downstream face existing bridge (AR4)
- Cam 2 Photo 28: Looking cross rail yard downstream face of existing bridge (RY 7)
- Cam 2 Photo 27: Looking towards the Power Plant from flood wall at downstream of existing bridge
- Cam 2 Photo 26: Standing on left floodwall looking across the river at upstream face of existing bridge (AR3)
- Cam 2 Photo 25: Looking across rail yard at RY 6
- Cam 2 Photo 24: Looking upstream along rail yard from the bridge
- Cam 2 Photo 23: Looking upstream along river from the bridge
- Cam 2 Photo 22: Looking downstream from approximate location of AR2
- Cam 2 Photo 21: Looking down stream of rail yard for approximate location of RY5
- Cam 2 Photo 20: Looking across the river at cross section AR1

- Cam 2 Photo 19: Looking downstream from approximate location of AR1
- Cam 2 Photo 18: Looking across rail yard at approx. location of RY4
- Cam 2 Photo 17: Looking downstream standing on left flood wall from location opposite stadium (RY3)
- Cam 2 Photo 16: Looking downstream along rail yard from approx. location of RY3
- Cam 2 Photo 15: Standing on flood wall, looking upstream along Arkansas River flood plain at point where water first impinges on flood wall
- Cam 2 Photo 14: Standing at the same point looking across the river
- Cam 2 Photo 13: Standing at the same point looking downstream along the river
- Cam 2 Photo 12: At the same point looking across the rail yard (approx. location of RY2)
- Cam 2 Photo 11: Looking across the rail yard at cross section RY1
- Cam 2 Photo 10: Looking across the river flood plain slightly downstream at approx. location of RY1
- Cam 2 Photo 9: Looking at upstream side of street bridge over Wild Horse Creek along levee
- Cam 2 Photo 8: Looking upstream along Wild Horse Creek Levee

Bed Samples for Riverine Scour Analyses

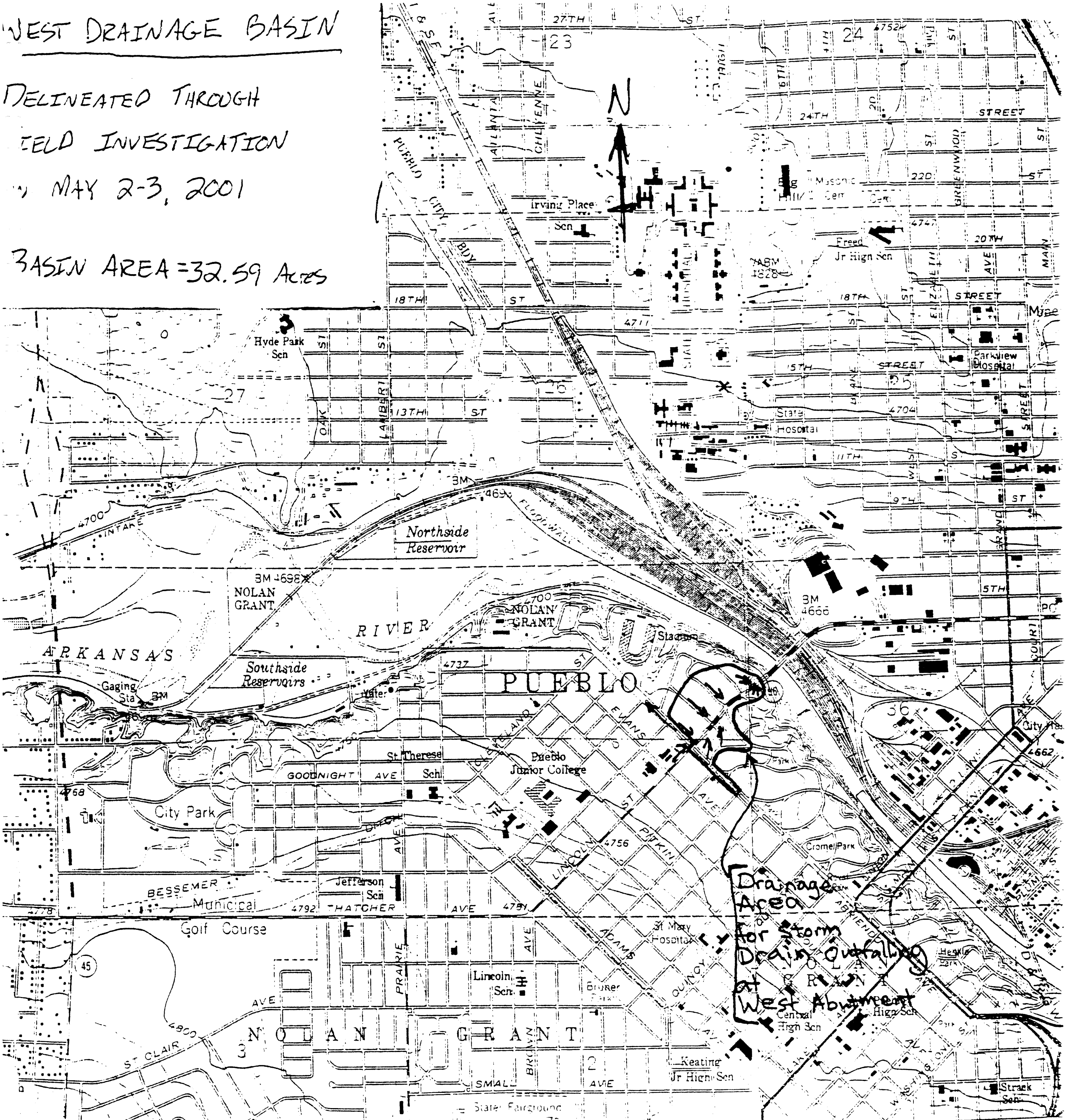
- Bed Sample #1: 700' upstream of bridge on right side of gravel bar in the middle of the river
- Bed Sample #2: Near the right bank of channel, just downstream of existing bridge pier

APPENDIX B
Hydrology Calculations and Supporting Information

WEST DRAINAGE BASIN

DELINEATED THROUGH
FIELD INVESTIGATION
ON MAY 2-3, 2001

BASIN AREA = 32.59 Acres



Scale 1" = 2000'

Mapped, edited, and published by the

ROAD CLASSIFICATION

Primary highway, hard surface		Light-duty road, hard or improved surface	
Secondary highway, hard surface		Unimproved road	

Interstate Route U S Route State Route

Control by USGS and NOS/NOAA

Topography by photogrammetric methods from aerial photographs taken 1954 and 1957. Field checked

Polyconic projection. 1927 North American datum
10,000-foot grid based on Colorado coordinate system
1000-metre Universal Transverse Mercator grid ticks, zone 13, shown in blue

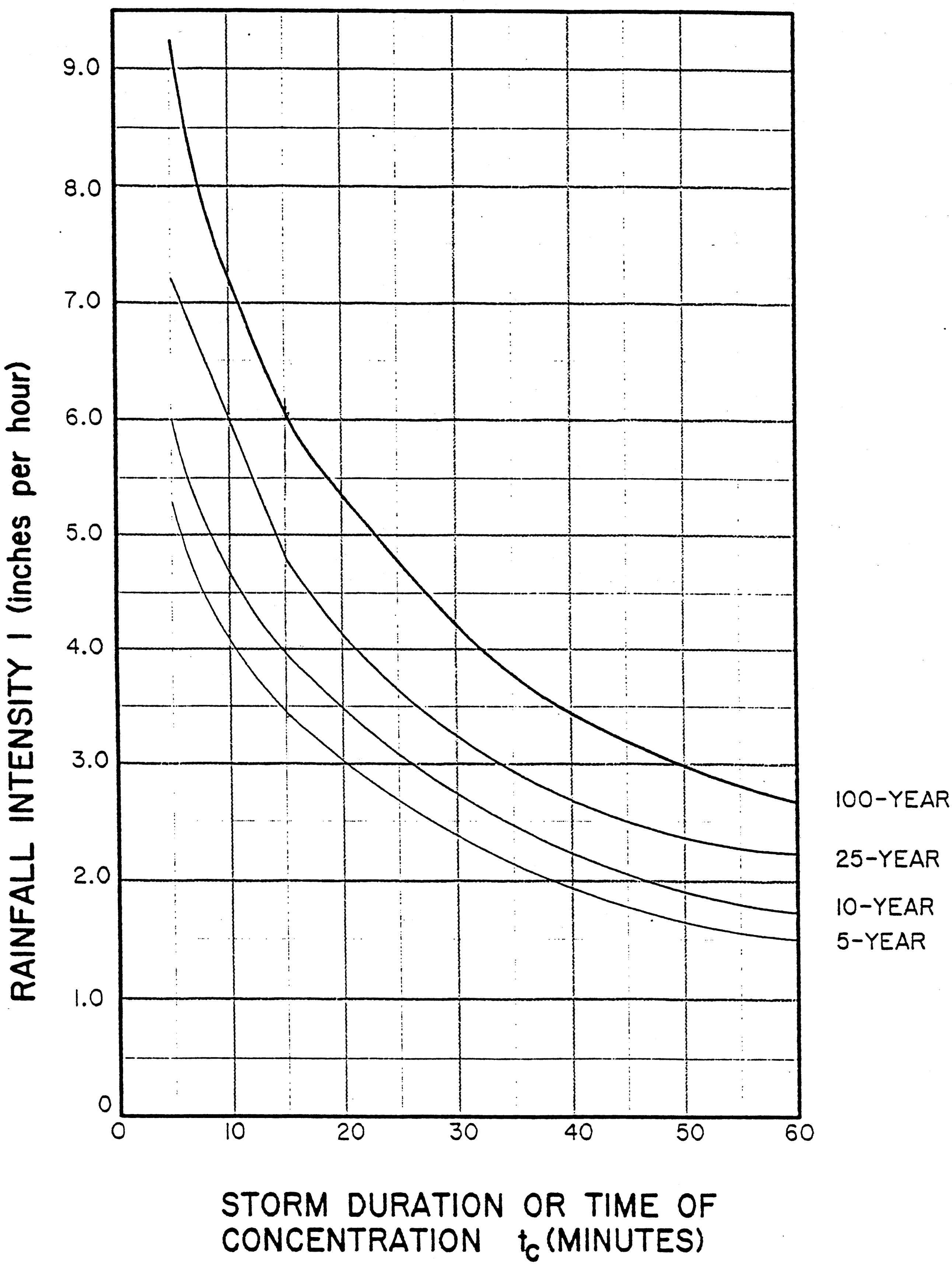
Red tint indicates areas in which only landmark

COLORADO

QUADRANGLE LOCATION

NORTHWEST PUEBLO, COLO.

TIME-INTENSITY-FREQUENCY CURVES



CITY OF PUEBLO, COLORADO

DATE: 6/97

EXISTING CONDITIONS PEAK RUNOFF CALCULATIONS FOR DRAINAGE BASIN WEST OF 4th STREET BRIDGE

(Basin Map Shown Later in Appendix B)

Q=CIA
C = coefficient of runoff
I = intensity
A = area

Areas of Various Use Types in the Basin

Business	Area =	437935	ft^2 =	10.05 acres
Residential	Area =	940908	ft^2 =	21.60 acres
4th Street Paved	Area =	41054	ft^2 =	0.94 acres
Total West Basin	**Area =**	**1419898**	**ft^2 =**	**32.60 acres**

Composite Runoff Coefficients - C

Use Type	Area	5-yr Storm (C)	C*A	25-yr Storm (C)	C*A	100-yr Storm (C)	C*A
Neighborhood Business Area =	10.05	0.65	6.53	0.75	7.52	0.81	8.17
Single Family Multi-Unit Detached =	21.60	0.5	10.80	0.58	12.42	0.63	13.50
4th Street Paved =	0.94	0.95	0.90	1.00	0.94	1.00	0.94
Totals =	32.60	-	18.23	-	20.88	-	22.61
Composite Runoff Coefficients =		**0.56**		**0.64**		**0.69**	

Existing Time of Concentration

Total Length - L_t (ft) =	1551.32
Basin slope (%) =	0.9
Overland Length - L_{ov} (ft) =	70
Overland Slope - S_{ov} (%) =	0.9
Remaining Length - L_t (ft) =	1481.32

Assuming fallow or minimum tillage cultivation:
V (ft/sec) = 1.6 (Appendix A-5)

Overland Flow Time (t_i) = $\frac{1.8*(1.1-C_5)*SQRT(L_{ov})}{(S_{ov})^{\wedge}(1/3)}$ = **8.43 minutes**

Remaining Travel Time (t_t) = $L_t/60V$= **15.43 minutes**

Time of Concentration (t_c)= t_i+t_t = **23.86 minutes**

t_{check} (min) per UDFCD = $(L_t/180)+10$ = **18.62 minutes**

*Note: Since the t_{check} is less than t_c, t_{check} will be used

For 5-Year Storm
I (in/hr) = 3.1
Q (cfs) = 56.99

For 25-Year Storm
I (in/hr) = 4.20
Q (cfs) = 88.42

For 100-Year Storm
I (in/hr) = 5.40
Q (cfs) = 123.12

Peak Discharge Flowing Across 4th Street Bridge From West Basin

* Assuming the existing storm sewer design event = 5 years

Q_{bridge} (cfs) = Q_{100} - Q_{25} = **66.13**

PROPOSED CONDITIONS PEAK RUNOFF CALCULATIONS FOR DRAINAGE BASIN WEST OF 4th STREET BRIDGE

25-YEAR WEST BASIN STORM SEWER CAPACITY

(Basin Map Shown Later in Appendix B)

Q=CIA
C = coefficient of runoff
I = intensity
A = area

Areas of Various Use Types in the Basin

Business	Area =	418669	ft^2 =	9.61 acres
Residential	Area =	940908	ft^2 =	21.60 acres
4th Street Paved	Area =	60320	ft^2 =	1.38 acres
Total West Basin	**Area =**	**1419898**	**ft^2 =**	**32.60 acres**

Composite Runoff Coefficients - C

Use Type	Area	5-yr Storm (C)	C*A	25-yr Storm (C)	C*A	100-yr Storm (C)	C*A
Neighborhood Business Area =	9.61	0.65	6.25	0.75	7.18	0.81	7.81
Single Family Multi-Unit Detached =	21.60	0.5	10.80	0.58	12.42	0.63	13.50
4th Street Paved =	1.38	0.95	1.32	1.00	1.38	1.00	1.38
Totals =	32.60	-	18.36	-	20.99	-	22.69
Composite Runoff Coefficients =		**0.56**		**0.64**		**0.70**	

Existing Time of Concentration

Total Length - L_t (ft) =	1551.32
Basin slope (%) =	0.9
Overland Length - L_{ov} (ft) =	70
Overland Slope - S_{ov} (%) =	0.9
Remaining Length - L_t (ft) =	1481.32

Assuming fallow or minimum tillage cultivation:

V (ft/sec) = 1.6 (Appendix A-5 UDFCD)

Overland Flow Time (t_i) = $\frac{1.8*(1.1-C_5)*SQRT(L_{ov})}{(S_{ov})\text{^}(1/3)}$ = **8.37** **minutes**

Remaining Travel Time (t_t) = $L_t/60V$= **15.43** **minutes**

Time of Concentration (t_c)= t_i+t_t = **23.80** **minutes**

t_{check} (min) per UDFCD = $(L_t/180)+10$ = **18.62** **minutes**

*Note: Since the t_{check} is less than t_c, t_{check} will be used

Rational Method Peak Discharges

Change in Flow Caused by Proposed Bridge & Drainage Facilities

5-Year Storm
I (in/hr) = 3.1
Q (cfs) = 57.40 — **0.41 cfs**

25-Year Storm
I (in/hr) = 4.20
Q (cfs) = 88.89 — **0.47 cfs**

100-Year Storm
I (in/hr) = 5.40
Q (cfs) = 123.57 — **0.45 cfs**

Peak Discharge Flowing Across 4th Street Bridge From West Basin

* Assuming the proposed storm sewer design event = 25 years

Q_{bridge} (cfs) = Q_{100} - Q_{25} = 34.68 — **-31.45 cfs**

BRIDGE DECK PEAK RUNOFF CALCULATIONS FOR EXISTING AND PROPOSED BRIDGE CONDITONS
25-YEAR WEST SIDE STORM SEWER CAPACITY

Q=CIA
C = coefficient of runoff
I = intensity
A = area

Existing Bridge	Area =	74,900	ft^2 =	1.72 acres
Proposed Bridge	Area =	118,560	ft^2 =	2.72 acres

Area	Area	5-yr Storm (C)	C*A	25-yr Storm (C)	C*A	100-yr Storm (C)	C*A
Existing Bridge Impervious Area	1.72	0.88	1.51	0.92	1.58	0.93	1.60
Proposed Bridge Impervious Area	2.72	0.88	2.40	0.92	2.50	0.93	2.53

t_c = t_i+t_t Assume min. urbanized t_c = 5 minutes

Flows at East Bridge Abutment - Used to Determine Detention Volume & Spread Widths

EXISTING BRIDGE

For 5-Year Storm			
I (in/hr) =	5.28		
Q (cfs) =	7.99		
For 25-Year Storm		Runoff from Basin West of Bridge	25-year Total runoff on bridge
I (in/hr) =	7.2		
Q (cfs) =	11.39	31.43	42.82
For 100-Year Storm		Runoff from Basin West of Bridge	100-year Total runoff on bridge
I (in/hr) =	9.24		
Q (cfs) =	14.78	66.13	80.91

PROPOSED BRIDGE

For 5-Year Storm			
I (in/hr) =	5.28		
Q (cfs) =	12.65		
For 25-Year Storm			
I (in/hr) =	7.2		
Q (cfs) =	18.03		
For 100-Year Storm		Runoff from Basin West of Bridge	100-year Total runoff on bridge
I (in/hr) =	9.24		
Q (cfs) =	23.39	34.68	58.07

Therefore, because more flow is being removed prior to crossing the bridge (25-year vs. 5-year) the flow at the eastern abutment is actually less than existing and no detention or mitigation should be required. Any piping placed on the bridge for mitigation of other drainage issues (superelevation and bridge joints) can be directly connected to the storm sewer and the project will actually decrease the flow into the inlets at 4th Street and Midtown Circle Drive.

PROPOSED CONDITIONS PEAK RUNOFF CALCULATIONS FOR DRAINAGE BASIN WEST OF 4th STREET BRIDGE

5-YEAR WEST BASIN STORM SEWER CAPACITY

(Basin Map Shown Later in Appendix B)

Q=CIA
C = coefficient of runoff
I = intensity
A = area

Areas of Various Use Types in the Basin

Business	Area =	418669	ft^2 =	9.61 acres
Residential	Area =	940908	ft^2 =	21.60 acres
4th Street Paved	Area =	60320	ft^2 =	1.38 acres
Total West Basin	**Area =**	**1419898**	**ft^2 =**	**32.60 acres**

Composite Runoff Coefficients - C

Use Type	Area	5-yr Storm (C)	C*A	25-yr Storm (C)	C*A	100-yr Storm (C)	C*A
Neighborhood Business Area =	9.61	0.65	6.25	0.75	7.18	0.81	7.81
Single Family Multi-Unit Detached =	21.60	0.5	10.80	0.58	12.42	0.63	13.50
4th Street Paved =	1.38	0.95	1.32	1.00	1.38	1.00	1.38
Totals =	32.60	-	18.36	-	20.99	-	22.69
Composite Runoff Coefficients =		**0.56**		**0.64**		**0.70**	

Existing Time of Concentration

Total Length - L_t (ft) = 1551.32
Basin slope (%) = 0.9
Overland Length - L_{ov} (ft) = 70
Overland Slope - S_{ov} (%) = 0.9
Remaining Length - L_t (ft) = 1481.32
Assuming fallow or minimum tillage cultivation:
V (ft/sec) = 1.6 (Appendix A-5, UDFCD)

Overland Flow Time (t_i) = $\frac{1.8*(1.1-C_5)*SQRT(L_{ov})}{(S_{ov})^{\wedge}(1/3)}$ = **8.37** **minutes**

Remaining Travel Time (t_t) = $L_t/60V$= **15.43** **minutes**

Time of Concentration (t_c)= t_i+t_t = **23.80** **minutes**

t_{check} (min) per UDFCD = $(L_t/180)+10$ = **18.62** **minutes**

*Note: Since the t_{check} is less than t_c, t_{check} will be used

Rational Method Peak Discharges

		Change in Flow Caused by Proposed Bridge & Drainage Facilities
5-Year Storm		
I (in/hr) =	3.1	
Q (cfs) =	**57.40**	**0.41 cfs**
25-Year Storm		
I (in/hr) =	4.20	
Q (cfs) =	**88.89**	**0.47 cfs**
100-Year Storm		
I (in/hr) =	5.40	
Q (cfs) =	**123.57**	**0.45 cfs**

100-year Peak Discharge Flowing Across 4th Street Bridge From West Basin

* Assuming the proposed storm sewer design event = 5 years

Q_{bridge} (cfs) = Q_{100} - Q_{25} = **66.17** **0.04 cfs**

BRIDGE DECK PEAK RUNOFF CALCULATIONS FOR EXISTING AND PROPOSED BRIDGE CONDITONS:
5-YEAR WEST SIDE STORM SEWER CAPACITY

Q=CIA
C = coefficient of runoff
I = intensity
A = area

Existing Bridge	Area =	74,900	ft^2 =	1.72 acres
Proposed Bridge	Area =	118,560	ft^2 =	2.72 acres

Area	Area	5-yr Storm (C)	C*A	25-yr Storm (C)	C*A	100-yr Storm (C)	C*A
Existing Bridge Impervious Area	1.72	0.88	1.51	0.92	1.58	0.93	1.60
Proposed Bridge Impervious Area	2.72	0.88	2.40	0.92	2.50	0.93	2.53

t_c = t_i+t_t Assume min. urbanized t_c = 5 minutes

Flows at East Bridge Abutment - Used to Determine Detention Volume & Spread Widths

EXISTING BRIDGE

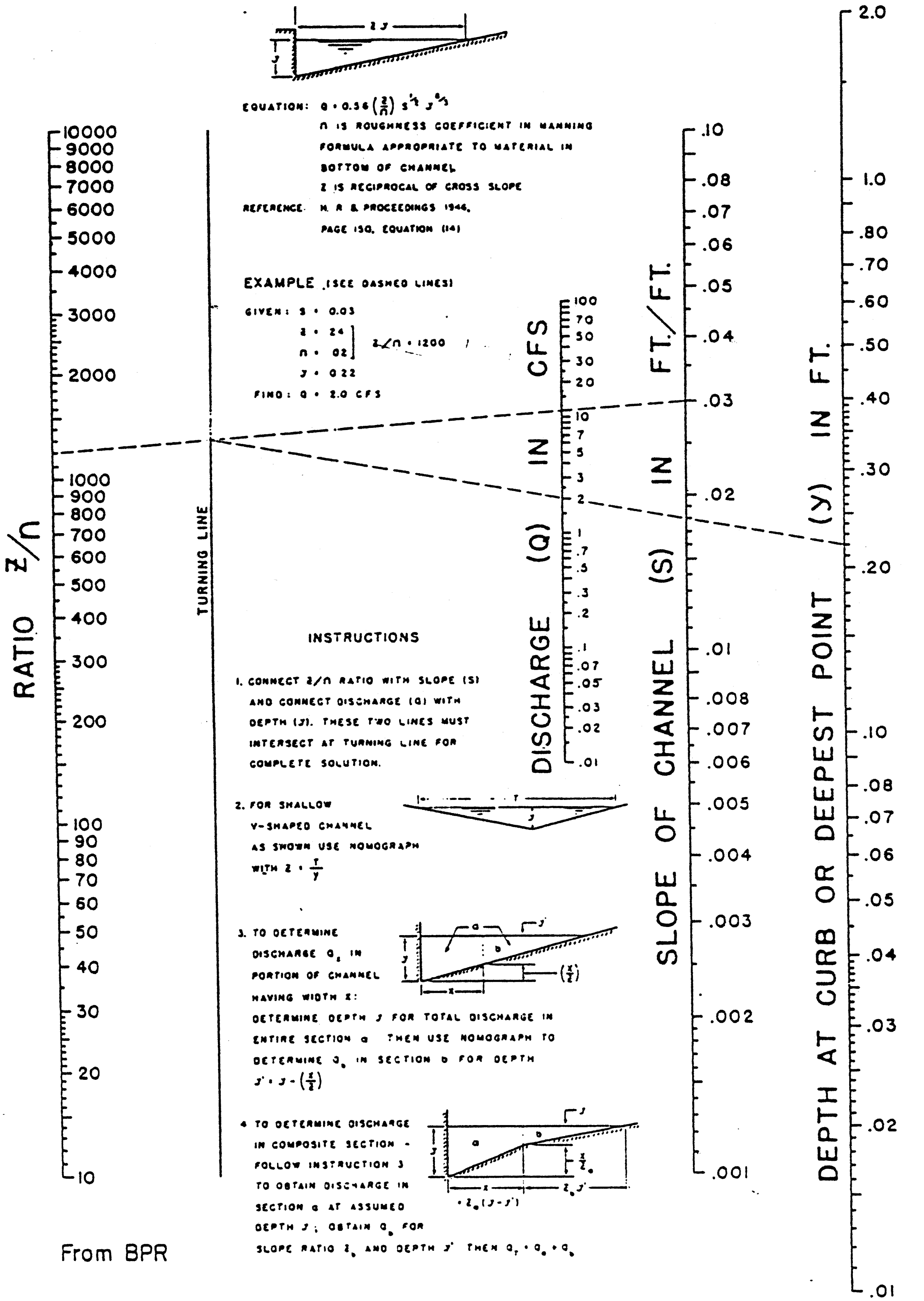
For 5-Year Storm			
I (in/hr) =	5.28		
Q (cfs) =	7.99		
For 25-Year Storm		Runoff from Basin West of Bridge	25-year Total runoff on bridge
I (in/hr) =	7.2		
Q (cfs) =	11.39	31.43	42.82
For 100-Year Storm		Runoff from Basin West of Bridge	100-year Total runoff on bridge
I (in/hr) =	9.24		
Q (cfs) =	14.78	66.13	80.91

PROPOSED BRIDGE

For 5-Year Storm			
I (in/hr) =	5.28		
Q (cfs) =	12.65		
For 25-Year Storm		Runoff from Basin West of Bridge	25-year Total runoff on bridge
I (in/hr) =	7.2		
Q (cfs) =	18.03	31.49	49.52
For 100-Year Storm		Runoff from Basin West of Bridge	100-year Total runoff on bridge
I (in/hr) =	9.24		
Q (cfs) =	23.39	66.17	89.56

Therefore, the flow at the east abutment is greater than existing by 8.56 cfs and must be mitigated through detention.

NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTERS



From BPR

Spread Width Flow - 25-year capacity west storm sewer

Bridge Characteristics (Assuming triangular gutter)

n =	0.015
S_x(ft/ft) =	0.02
Gutter slope (ft/ft) =	0.021

* using nomograph for triangular gutters A-7 in City of Pueblo Criteria Manual

z = 50 z/n = 3333

Event	Total Runoff (cfs)	Single Gutter (cfs)	Depth, y (ft)	Spread Width, T (ft)
5-year	12.65	6.32	0.25	12.34
25-year	18.03	9.01	0.28	14.09
100-year	58.07	29.04	0.44	21.85

* meets 25-yr City of Pueblo Criteria

Spread Width Flow - 5-year capacity west storm sewer

Bridge Characteristics (Assuming triangular gutter)

n =	0.015
S_x(ft/ft) =	0.02
Gutter slope (ft/ft) =	0.021

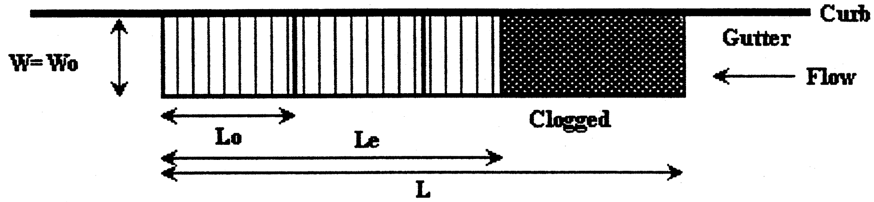
* using nomograph for triangular gutters A-7 in City of Pueblo Criteria Manual

z = 50 z/n = 3333

Event	Total Runoff (cfs)	Single Gutter (cfs)	Depth, y (ft)	Spread Width, T (ft)
5-year	12.65	6.32	0.25	12.34
25-year	49.52	24.76	0.41	20.58
100-year	89.56	44.78	0.51	25.71

GRATE INLET ON A GRADE

Project: SH96A (4th St.) Bridge over the Arkansas Rivei
Inlet ID: Representative Inlet - Neenah R-3922



Design Information (Input)

Design Discharge on the Street (from *Street Hy*)	Q_o =	**9.0**	**cfs**
Type of Grate	Type =	30-Degree Bar	
Length of a Unit Grate	L_o =	1.02	ft
Width of a Unit Grate	W_o =	1.96	ft
Clogging Factor for a Unit Grate	C_o =	0.50	
Water Depth for Design Condition	Y_d =	2.99	inches
Number of Grates	No =	1	

Analysis (Calculated)

Total Length of Grate Inlet	L =	1.02	ft
Ratio of Gutter Flow to Design Flow Eo (from *Street Hy*)	E_o =	0.37	
Equivalent Slope Se (from *Street Hy*)	S_e =	0.0200	ft/ft
Flow Velocity Vs (from *Street Hy*)	V_s =	5.80	fps
Spash-over Velocity: Check Against Flow Velocity	V_o is:	< Vs	
Under No-Clogging Condition			
Interception Rate of Gutter Flow	R_f =	0.71	
Effective Length of Grate Inlet	L =	1.02	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	R_x =	0.01	
Interception Capacity	Q_i =	2.4	cfs
Under Clogging Condition			
Interception Rate of Gutter Flow	R_f =	0.71	
Clogging Coefficient for Multiple-unit Grate Inlet	Coef =	1.00	
Clogging Factor for Multiple-unit Grate Inlet	Clog =	0.04	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L_e =	0.98	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	R_x =	0.01	
Actual Interception Capacity	Q_a =	**2.4**	**cfs**
Carry-Over Flow = Q_o-Q_a =	Q_b =	**6.6**	**cfs**
Capture Percentage = Q_a/Q_o =	C% =	**26.61**	**%**

25-YR DETENTION SIZING FOR PROPOSED CONDITIONS
ASSUMING WEST SIDE STORM SEWER CAPACITY = 5-YR

* Calculated using the FAA method taken from the UDFCD 2001 Volume 2 Criteria manual

C = 0.92
Area (acres) = 2.72
T (min) = 120
Tc (min) = 5
Existing Peak Runoff = Max Allowable Release Rate (cfs) = 11.39

Time (min)	Rainfall Intensity (in/hr)	Inflow Volume (cft)	Adjustment Factor	Average Outflow (cfs)	Vo (cft)	Vs (cft)	Vs (Acre-ft)
5	7.20	5405	1.00	11.39	3417	1988	0.05
10	6.00	9009	0.75	8.54	5126	3883	0.09
15	4.80	10810	0.67	7.59	6834	3976	0.09
20	4.10	12312	0.63	7.12	8543	3769	0.09
25	3.60	13513	0.60	6.83	10251	3262	0.07
30	3.24	14594	0.58	6.64	11960	2634	0.06
35	3.00	15765	0.57	6.51	13668	2097	0.05
40	2.80	16816	0.56	6.41	15377	1440	0.03
45	2.70	18242	0.56	6.33	17085	1157	0.03
50	2.50	18768	0.55	6.26	18794	-26	0.00
55	2.40	19819	0.55	6.21	20502	-683	-0.02
60	2.23	20089	0.54	6.17	22211	-2121	-0.05
65	2.13	20834	0.54	6.13	23919	-3085	-0.07
70	2.03	21327	0.54	6.10	25628	-4301	-0.10
75	1.93	21787	0.53	6.07	27336	-5549	-0.13
80	1.85	22218	0.53	6.05	29045	-6826	-0.16
85	1.77	22625	0.53	6.03	30753	-8128	-0.19
90	1.70	23009	0.53	6.01	32462	-9452	-0.22
95	1.64	23374	0.53	5.99	34170	-10796	-0.25
100	1.58	23720	0.53	5.98	35879	-12158	-0.28
105	1.53	24051	0.52	5.97	37587	-13536	-0.31
110	1.48	24368	0.52	5.95	39296	-14928	-0.34
115	1.43	24671	0.52	5.94	41004	-16333	-0.37
120	1.39	24962	0.52	5.93	42713	-17750	-0.41

100-YR DETENTION SIZING FOR PROPOSED CONDITIONS
ASSUMING WEST SIDE STORM SEWER CAPACITY = 5-YR

* Calculated using the FAA method taken from the UDFCD 2001 Volume 2 Criteria manual

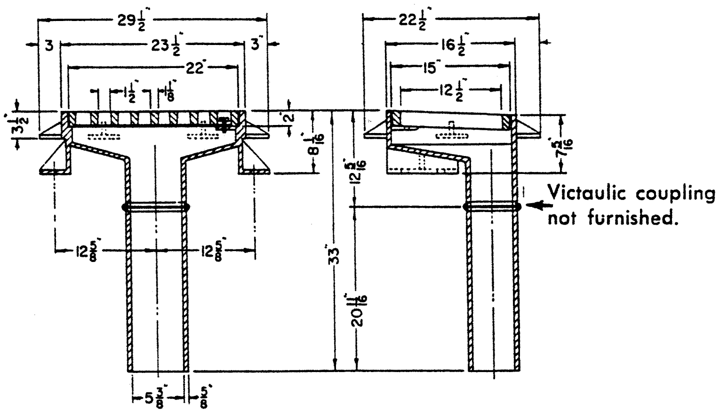
C = 0.93
Area (acres) = 2.72
T (min) = 120
Tc (min) = 5
Existing Peak Runoff = Max Allowable Release Rate(cfs) = 14.78

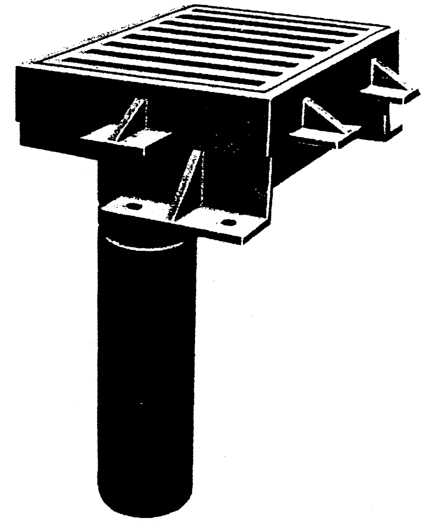
Time (min)	Rainfall Intensity (in/hr)	Inflow Volume (cft)	Adjustment Factor	Average Outflow (cfs)	Vo (cft)	Vs (cft)	Vs (Acre-ft)
5	9.24	6937	1.00	14.78	4434	2503	0.06
10	7.20	10810	0.75	11.09	6651	4159	0.10
15	6.08	13693	0.67	9.85	8868	4825	0.11
20	5.30	15915	0.63	9.24	11085	4830	0.11
25	4.70	17642	0.60	8.87	13302	4340	0.10
30	4.22	19008	0.58	8.62	15519	3489	0.08
35	3.00	15765	0.57	8.45	17736	-1971	-0.05
40	2.80	16816	0.56	8.31	19953	-3137	-0.07
45	2.70	18242	0.56	8.21	22170	-3928	-0.09
50	2.50	18768	0.55	8.13	24387	-5619	-0.13
55	2.40	19819	0.55	8.06	26604	-6785	-0.16
60	2.67	24053	0.54	8.01	28821	-4768	-0.11
65	2.56	24945	0.54	7.96	31038	-6093	-0.14
70	2.43	25535	0.54	7.92	33255	-7720	-0.18
75	2.32	26086	0.53	7.88	35472	-9386	-0.22
80	2.21	26602	0.53	7.85	37689	-11087	-0.25
85	2.12	27089	0.53	7.82	39906	-12817	-0.29
90	2.04	27549	0.53	7.80	42123	-14574	-0.33
95	1.96	27985	0.53	7.78	44340	-16355	-0.38
100	1.89	28401	0.53	7.76	46557	-18156	-0.42
105	1.83	28797	0.52	7.74	48774	-19977	-0.46
110	1.77	29176	0.52	7.73	50991	-21815	-0.50
115	1.71	29539	0.52	7.71	53208	-23669	-0.54
120	1.66	29887	0.52	7.70	55425	-25538	-0.59

R-3921-A Bridge Drain Frame and Bolted Grate

Heavy Duty

Total Weight 330 Pounds

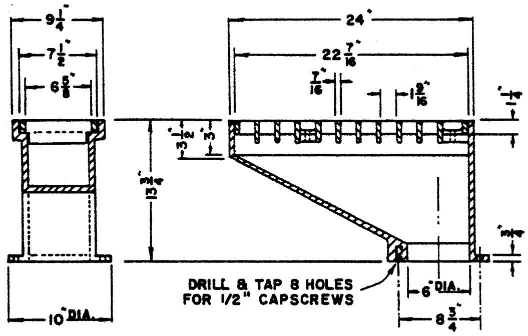


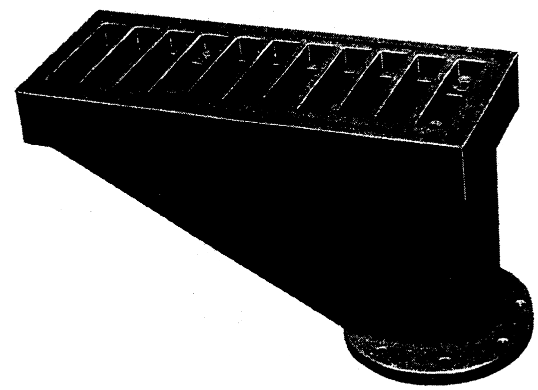


R-3921-D Bridge Drain Frame and Bolted Grate

Heavy Duty

Total Weight 115 Pounds

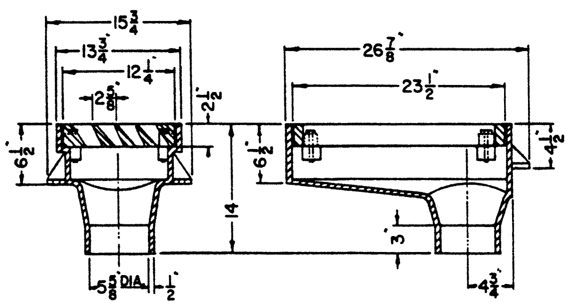


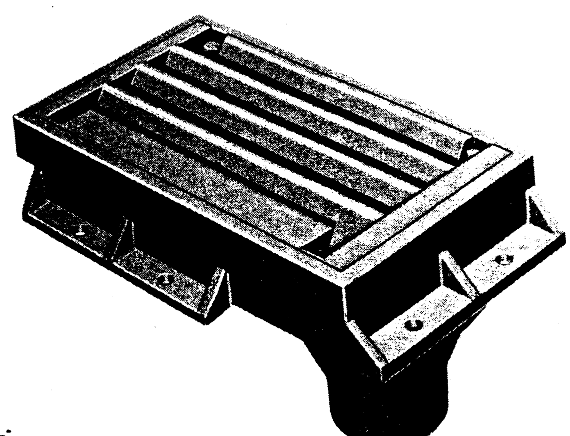


R-3922 Bridge Drain Frame and Bolted Grate

Heavy Duty

Total Weight 245 Pounds





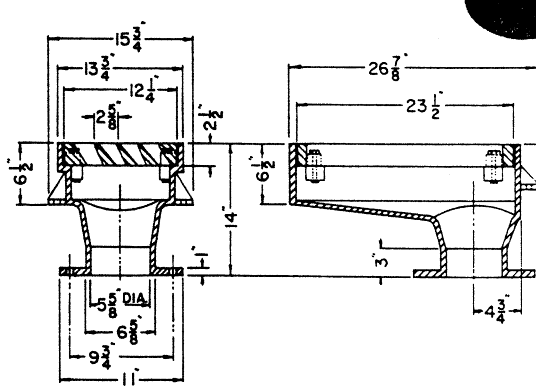
R-3922-A

Heavy Duty

Total Weight 255 Pounds

Same as R-3922 except with bolting flange.

Downspout furnished by others.



HEC-21: INLET CONSIDERATIONS

unsightly. Pipes affixed to exterior surfaces of structures, running at odd angles, can present an unpleasant silhouette and detract from a bridge's architectural aesthetics. To avoid this, pipes can be run in slots up the backs of the columns or can be hidden behind decorative pilasters. However, encased piping poses serious maintenance considerations and is not typically used in Northern States due to potential freezing damage.

1.2.5 Minimization of Maintenance

An ideal solution is no inlets. The fewer inlets, the easier to maintain them—clogged inlets are a widespread maintenance problem. The drainage design engineer should first consider whether or not bridge drains are essential. If drains are required, the system design should provide means for convenient maintenance.

1.2.6 Bicycle Safety

The design engineer should also consider the hazards that inlets themselves present to cyclists. Grates with bars parallel to the centerline may be unsafe for bicyclists. Remedy this by putting crossbars or vanes at right angles to the flow or using a reticuline composite grate. The safety remedy, however, does reduce the efficiency of the inlet to admit water. If bicyclists are not allowed, then parallel bar grates without crossbars are the most efficient hydraulic solution.

1.3 SYSTEMS

The bridge deck drainage system includes the bridge deck itself, bridge gutters, inlets, pipes, downspouts, and bridge end collectors. The details of this system are typically handled by the bridge engineer and coordinated with the hydraulic engineer. Coordination of efforts is essential in designing the various components of the system to meet the objectives described in the previous section.

1.3.1 Deck and Gutters

The bridge deck and gutters are surfaces that initially receive precipitation and debris. If grades, super-elevations, and cross-slopes are properly designed, water and debris are efficiently conveyed to the inlets or bridge end collectors. Bridge deck designs with zero grades or sag vertical curves are poor hydraulic designs and can cause water problems. Super-elevation transitions through a zero grade causes water problems as well.

1.3.2 Hardware—Inlets, Pipes, and Downspouts

From the deck and gutters, water and debris flow to the inlets, through pipes and downspouts, and to the outfall. Various grate and inlet box designs are available to discourage clogging. Collector pipes ~~and downspouts~~ with a minimum of T-connections and bends help prevent clogging mid-system. Collector pipes need sufficient slope to sustain self-cleansing velocities. Open chutes are not recommended for downdrains because of difficulties in maintaining chutes and capturing, and then containing the flow. Inlets, and associated hardware, should be called for only when necessary. Super-elevated bridge decks only need inlets on the low side, if any.

(Clean copy of this page w/ the 1.2.5 & 1.2.6 highlighted not 1.3.2

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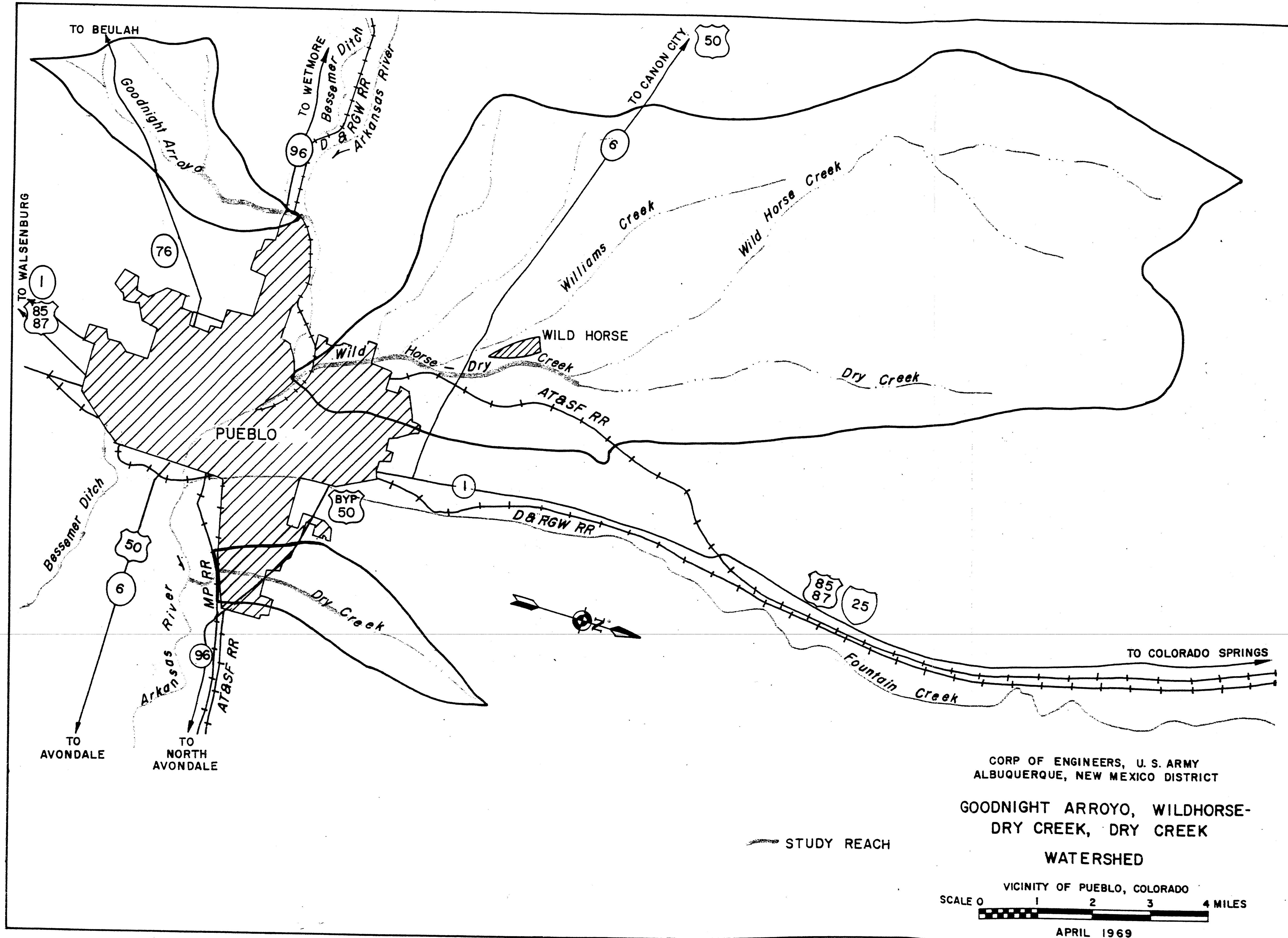


PLATE I

WILD HORSE-DRY CREEK DISCHARGE DATA

Pueblo, CO FIS (1986) Discharge Data

Recurrence Interval	Peak Discharge		
10	Q_{10} =	5700	cfs
50	Q_{50} =	14000	cfs
100	Q_{100} =	19500	cfs
500	Q_{500} =	39500	cfs

Regional Regression Discharges for the Wild Horse - Dry Creek Drainage

USGS 1993

Recurrence Interval	Peak Discharge *											
10	Q_{10} =	6101	cfs	A =	87.3	mi^2	Δ Elev. =	1320	ft			
50	Q_{50} =	14502	cfs	S_b =	62.0	ft/mi	Dist. =	21.31	mi =	112498	ft
100	Q_{100} =	19989	cfs	Rf =	1302	ft					
500	Q_{500} =	39784	cfs								

* Source: Nationwide Summary of U.S. Geological Survey Regional Regression Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites, 1993. (page 43)

Note: The area and distance were obtained by digitizing the drainage basin area, and the longest flow line through the basin off of a 1:250,000 scale contour map into MicroStation. The MicroStation file is named "w.h.basin".

USGS 99-4190, PLAINS REGION

Recurrence Interval	Peak Discharge		
10	Q_{10} =	2179	cfs
50	Q_{50} =	6435	cfs
100	Q_{100} =	9289	cfs
500	Q_{500} =	19314	cfs

USGS Water-Resources Investigations Report 87-4094

Recurrence Interval	log		Peak Discharge		
10	log Q_{10} =	3.96	Q_{10} =	9195	cfs
50	log Q_{50} =	4.55	Q_{50} =	35834	cfs
100	log Q_{100} =	4.75	Q_{100} =	56106	cfs
500	log Q_{500} =		Q_{500} =		cfs

TR-55

Estimating Runoff

Q = runoff (in)
P = rainfall (in) (from NOAA Atlas 2)
S = potential maximum retention after runoff begins (in)
Cn = curve number

Assume soil type = B
Cover type = brush
Hydrologic condition = poor
Cn = 67 (from Table 2-2c)
Estimated Impervious Area = 10499269 ft^2 0.38 mi^2
% Impervious Area = 0.43
Unadjusted Cn = 67
S = 1000/Cn - 10
S (in) = 4.93

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)}$$

Recurrance Interval	Rainfall 24 hr. (in)	Runoff (Q) (in)
10	3	0.5850
50	4	1.1448
100	4.5	1.4638

Time of Concentration and Travel Time

Tt (hr) = travel time
Tc = time of concentration
L (ft) = flow length
V (ft/s) = average velocity
P2 (in)= 2-yr, 24 hour rainfall
s (ft/ft) = slope of hydraulic grade line (land slope)
n = Manning's roughness coefficient
r = hydraulic radius (ft) and is equal to a/p_w

a (ft^2) = cross sectional flow area
p_w (ft) = wetted perimeter

Sheet Flow

n = 0.13 (Range (natural) Table 3-1)
P_2 (in) = 2
s (ft/ft) = 0.040
L (ft) = 200

$$Tt = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}k}$$

Tt (hr) = 0.2431

Shallow Concentration Flow

L (ft) = 200

V (ft/s) = 1.63 (from fig. 3-1)

$$Tt = \frac{L}{3600V}$$

Tt (hr) 0.034

Channel Flow

L (ft) = 112498

V (ft/s) = 20 (from April 1969 Pueblo Flood Plain Information, p. 45, Table 10)

$$Tt = \frac{L}{3600V}$$

Tt (hr) 1.562

Tc (hr) = Tt(sheet flow) + Tt(shallow concentration) + Tt (channel flow)

Tc (hr) = 1.84

Peak Discharge Computations

Rainfall Distribution Type = II

Cn = 67

I_a (in) = 0.985

Peak Discharge = q_p (cfs)= q_u*A*Q*F_p

Recurrance Interval (years)	24 hr. Rainfall (in)	I_a/P	q_u (csm/in) Table 4-II	Runoff (Q) (in)	F_p (pond and swamp adjuster)	Peak Discharge (q_p) (cfs)
10	3	0.33	185	0.58	1	9449
50	4	0.25	210	1.14	1	20989
100	4.5	0.22	225	1.46	1	28755

Pueblo Dam Maximum Flood Discharge (cfs) - Using Various Flood Frequency Studies

Frequency (years)	Projected Site Specific Data	1997 Hydrographs Modified Using Paleo Rain Storm	1997 Hydrographs Modified Using Paleo Rain-on-Snow	1921 Flood Modified Using Paleo Rain Storm	GEV Balanced Hydrographs	GEV Adjusted to Wakeby Volume	1997 Hydrographs
10	6,000	6,000	6,000	6,000	6,000	6,000	6,000
12	6,000	6,000	6,000	6,000	6,000	6,000	6,000
15	6,000	6,000	6,000	6,000	6,000	6,000	6,000
20	6,000	6,000	6,000	6,000	6,000	6,000	6,000
30	6,000	6,000	6,000	6,000	6,000	6,000	6,000
50	6,000	6,000	6,000	6,000	6,000	6,000	6,000
100	6,000	6,000	6,000	6,000	16,583	6,000	6,000
200		6,000	6,000	6,000	23,053	8,607	6,000
500		6,000	6,000	6,000	33,171	21,146	6,000
1,000		14,294	6,000	6,000	41,601	27,316	6,000
2,000		30,650	6,000	6,000	51,040	32,961	6,000
5,000		52,836	6,863	6,000	64,534	41,496	7,814
10,000	25,275	69,256	9,515	7,369	75,250	49,379	11,560
20,000		84,383	14,494	21,272	86,819	58,115	18,880
50,000		102,424	20,916	52,114	102,535	71,918	30,642
100,000		115,044	25,616	82,032	115,291	83,762	42,034
200,000		126,376	30,064	116,201	128,413	96,862	57,105
500,000		140,901	35,224	168,872	147,299	116,700	86,317
1,000,000		150,608	38,948	211,063	162,046	133,837	120,432

Note: None of the methods shown are approved for use. A flood frequency analysis by the Bureau of Reclamation is scheduled for completion in 2003.
The minimum flow of 6,000 cfs is actually the maximum controlled discharge based on safe downstream channel capacity. Actual flows could be lower.

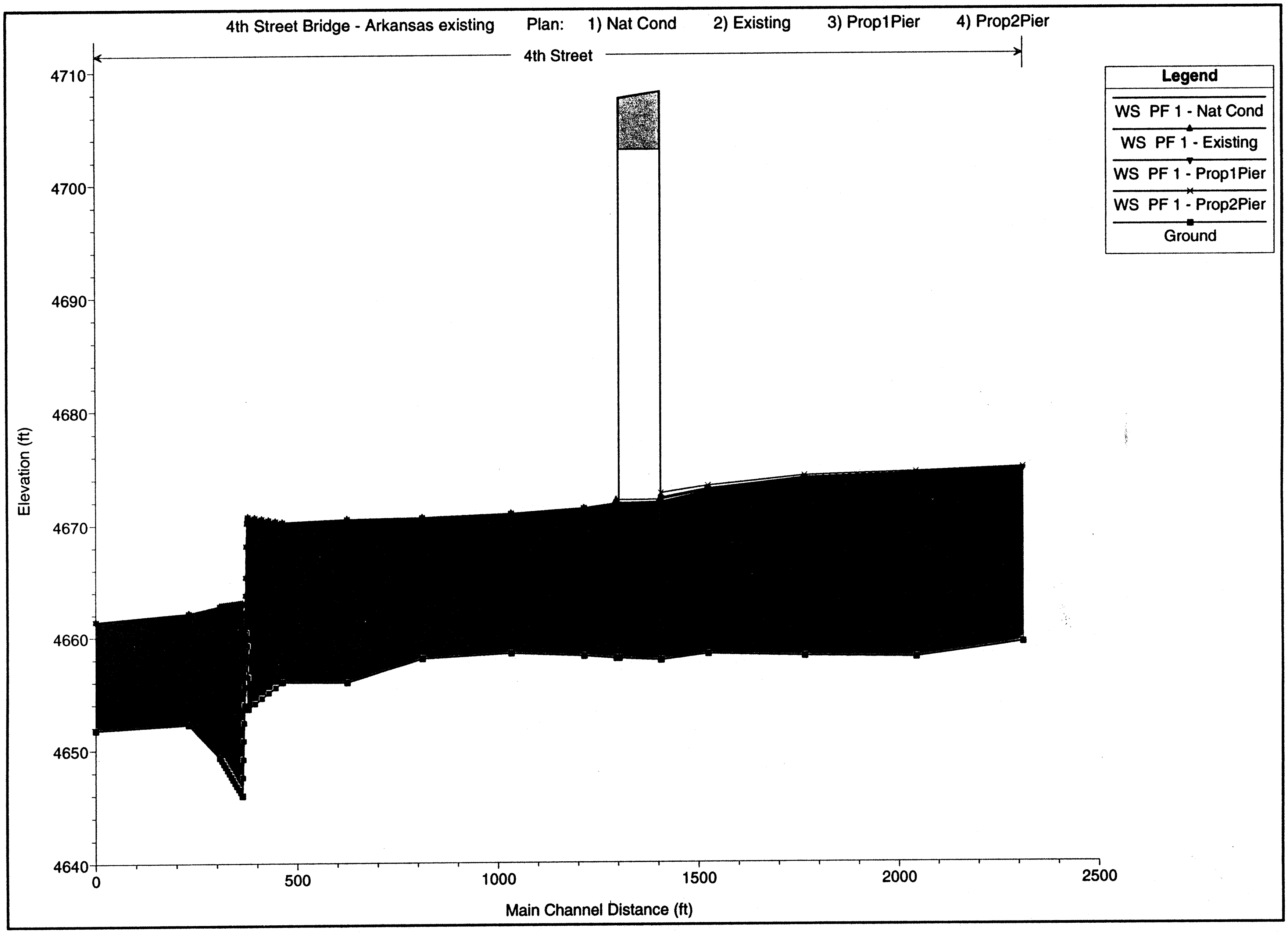
APPENDIX C
Hydraulic Modeling Information and Results

HEC-RAS River: Arkansas Reach: 4th Street Profile: PF 1

Reach	River Sta	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
4th Street	8	Nat Cond	20000.00	4659.35	4674.55	4669.83	4675.67	0.001159	8.47	2412.38	240.06	0.44
4th Street	8	Existing	20000.00	4659.35	4674.63	4669.83	4675.72	0.001134	8.42	2430.34	240.90	0.44
4th Street	8	Prop1Pier	20000.00	4659.35	4674.69	4669.83	4675.77	0.001114	8.37	2444.36	241.56	0.43
4th Street	8	Prop2Pier	20000.00	4659.35	4674.83	4669.83	4675.88	0.001068	8.26	2478.44	243.16	0.43
4th Street	7.5	Nat Cond	20000.00	4658.02	4674.12		4675.33	0.001242	8.85	2306.68	221.06	0.46
4th Street	7.5	Existing	20000.00	4658.02	4674.20		4675.40	0.001210	8.78	2325.81	221.70	0.45
4th Street	7.5	Prop1Pier	20000.00	4658.02	4674.27		4675.45	0.001187	8.73	2340.55	222.18	0.45
4th Street	7.5	Prop2Pier	20000.00	4658.02	4674.43		4675.58	0.001132	8.60	2376.56	223.37	0.44
4th Street	7	Nat Cond	20000.00	4658.13	4673.76		4674.99	0.001253	8.98	2361.00	244.60	0.46
4th Street	7	Existing	20000.00	4658.13	4673.84		4675.06	0.001220	8.91	2382.52	245.18	0.46
4th Street	7	Prop1Pier	20000.00	4658.13	4673.92		4675.12	0.001193	8.85	2401.46	245.69	0.45
4th Street	7	Prop2Pier	20000.00	4658.13	4674.11		4675.26	0.001129	8.70	2447.40	246.93	0.44
4th Street	6.5	Nat Cond	20000.00	4658.34	4672.73		4674.55	0.002043	11.30	2045.73	224.75	0.59
4th Street	6.5	Existing	20000.00	4658.34	4672.86		4674.64	0.001957	11.15	2076.16	225.32	0.57
4th Street	6.5	Prop1Pier	20000.00	4658.34	4672.98		4674.71	0.001887	11.02	2101.83	225.80	0.56
4th Street	6.5	Prop2Pier	20000.00	4658.34	4673.24		4674.88	0.001736	10.73	2162.51	226.93	0.54
4th Street	6.25	Nat Cond	20000.00	4657.80	4671.83		4674.23	0.002767	13.08	1840.57	218.85	0.68
4th Street	6.25	Existing	20000.00	4657.80	4672.06		4674.33	0.002570	12.76	1889.53	219.97	0.66
4th Street	6.25	Prop1Pier	20000.00	4657.80	4672.22	4670.10	4674.41	0.002436	12.54	1925.68	220.78	0.64
4th Street	6.25	Prop2Pier	20000.00	4657.80	4672.61	4670.10	4674.62	0.002152	12.04	2011.98	222.72	0.61
4th Street	6	Nat Cond	20000.00	4657.97	4671.74		4673.84	0.002592	12.16	1852.97	232.24	0.65
4th Street	6	Existing	20000.00	4657.97	4672.02	4669.98	4673.97	0.002350	11.78	1916.42	233.50	0.62
4th Street	6	Prop1Pier	20000.00	4657.97	4671.74		4673.84	0.002592	12.16	1852.97	232.24	0.65
4th Street	6	Prop2Pier	20000.00	4657.97	4671.74		4673.84	0.002592	12.16	1852.97	232.24	0.65
4th Street	5	Nat Cond	20000.00	4658.17	4671.32		4673.60	0.002715	13.05	1814.98	226.43	0.67
4th Street	5	Existing	20000.00	4658.17	4671.32		4673.60	0.002715	13.05	1814.98	226.43	0.67
4th Street	5	Prop1Pier	20000.00	4658.17	4671.32		4673.60	0.002715	13.05	1814.98	226.43	0.67
4th Street	5	Prop2Pier	20000.00	4658.17	4671.32		4673.60	0.002715	13.05	1814.98	226.43	0.67
4th Street	4.5	Nat Cond	20000.00	4658.43	4670.89		4673.07	0.002789	12.33	1798.88	226.69	0.67
4th Street	4.5	Existing	20000.00	4658.43	4670.89		4673.07	0.002789	12.33	1798.88	226.69	0.67
4th Street	4.5	Prop1Pier	20000.00	4658.43	4670.89		4673.07	0.002789	12.33	1798.88	226.69	0.67
4th Street	4.5	Prop2Pier	20000.00	4658.43	4670.89		4673.07	0.002789	12.33	1798.88	226.69	0.67
4th Street	4	Nat Cond	20000.00	4658.00	4670.57		4672.40	0.002368	10.98	1895.99	223.75	0.62
4th Street	4	Existing	20000.00	4658.00	4670.57		4672.40	0.002368	10.98	1895.99	223.75	0.62
4th Street	4	Prop1Pier	20000.00	4658.00	4670.57		4672.40	0.002368	10.98	1895.99	223.75	0.62
4th Street	4	Prop2Pier	20000.00	4658.00	4670.57		4672.40	0.002368	10.98	1895.99	223.75	0.62
4th Street	3.5	Nat Cond	20000.00	4655.91	4670.44		4671.90	0.001784	9.73	2083.66	230.19	0.54
4th Street	3.5	Existing	20000.00	4655.91	4670.44		4671.90	0.001784	9.73	2083.66	230.19	0.54
4th Street	3.5	Prop1Pier	20000.00	4655.91	4670.44		4671.90	0.001784	9.73	2083.66	230.19	0.54
4th Street	3.5	Prop2Pier	20000.00	4655.91	4670.44		4671.90	0.001784	9.73	2083.66	230.19	0.54
4th Street	3.25	Nat Cond	20000.00	4655.96	4670.20		4671.60	0.001730	9.52	2129.84	240.43	0.53
4th Street	3.25	Existing	20000.00	4655.96	4670.20		4671.60	0.001730	9.52	2129.84	240.43	0.53
4th Street	3.25	Prop1Pier	20000.00	4655.96	4670.20		4671.60	0.001730	9.52	2129.84	240.43	0.53
4th Street	3.25	Prop2Pier	20000.00	4655.96	4670.20		4671.60	0.001730	9.52	2129.84	240.43	0.53
4th Street	3	Nat Cond	20000.00	4653.61	4670.70		4671.24	0.000470	5.81	3456.84	316.95	0.28
4th Street	3	Existing	20000.00	4653.61	4670.70		4671.24	0.000470	5.81	3456.84	316.95	0.28
4th Street	3	Prop1Pier	20000.00	4653.61	4670.70		4671.24	0.000470	5.81	3456.84	316.95	0.28
4th Street	3	Prop2Pier	20000.00	4653.61	4670.70		4671.24	0.000470	5.81	3456.84	316.95	0.28
4th Street	2.5	Nat Cond	20000.00	4662.01	4668.11	4668.11	4671.01	0.005853	12.06	1568.32	293.44	0.89
4th Street	2.5	Existing	20000.00	4662.01	4668.11	4668.11	4671.01	0.005853	12.06	1568.32	293.44	0.89
4th Street	2.5	Prop1Pier	20000.00	4662.01	4668.11	4668.11	4671.01	0.005853	12.06	1568.32	293.44	0.89
4th Street	2.5	Prop2Pier	20000.00	4662.01	4668.11	4668.11	4671.01	0.005853	12.06	1568.32	293.44	0.89
4th Street	2	Nat Cond	20000.00	4645.95	4663.00		4663.52	0.000466	5.83	3456.68	285.05	0.29
4th Street	2	Existing	20000.00	4645.95	4662.98		4663.51	0.000468	5.83	3453.34	285.01	0.29
4th Street	2	Prop1Pier	20000.00	4645.95	4663.00		4663.52	0.000466	5.83	3456.68	285.05	0.29
4th Street	2	Prop2Pier	20000.00	4645.95	4663.00		4663.52	0.000466	5.83	3456.68	285.05	0.29
4th Street	1.75	Nat Cond	20000.00	4649.31	4662.74		4663.41	0.000682	6.56	3071.43	276.68	0.34
4th Street	1.75	Existing	20000.00	4649.31	4662.74		4663.41	0.000682	6.56	3071.43	276.68	0.34
4th Street	1.75	Prop1Pier	20000.00	4649.31	4662.74		4663.41	0.000682	6.56	3071.43	276.68	0.34
4th Street	1.75	Prop2Pier	20000.00	4649.31	4662.74		4663.41	0.000682	6.56	3071.43	276.68	0.34
4th Street	1.5	Nat Cond	20000.00	4652.23	4662.09		4663.28	0.001659	8.77	2300.43	264.20	0.51

HEC-RAS River: Arkansas Reach: 4th Street Profile: PF 1 (Continued)

Reach	River Sta	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
4th Street	1.5	Existing	20000.00	4652.23	4662.09		4663.28	0.001659	8.77	2300.43	264.20	0.51
4th Street	1.5	Prop1Pier	20000.00	4652.23	4662.09		4663.28	0.001659	8.77	2300.43	264.20	0.51
4th Street	1.5	Prop2Pier	20000.00	4652.23	4662.09		4663.28	0.001659	8.77	2300.43	264.20	0.51
4th Street	1	Nat Cond	20000.00	4651.76	4661.39	4658.57	4662.83	0.002041	9.69	2109.28	246.07	0.57
4th Street	1	Existing	20000.00	4651.76	4661.39	4658.57	4662.83	0.002041	9.69	2109.28	246.07	0.57
4th Street	1	Prop1Pier	20000.00	4651.76	4661.39	4658.57	4662.83	0.002041	9.69	2109.28	246.07	0.57
4th Street	1	Prop2Pier	20000.00	4651.76	4661.39	4658.57	4662.83	0.002041	9.69	2109.28	246.07	0.57


4th Street Bridge - Arkansas existing Plan: 1) Nat Cond 2) Existing 3) Prop1Pier 4) Prop2Pier
4th Street
Legend
WS PF 1 - Nat Cond
WS PF 1 - Existing
WS PF 1 - Prop1Pier
WS PF 1 - Prop2Pier
Ground
Elevation (ft)
4710
4700
4690
4680
4670
4660
4650
4640
0
500
1000
1500
2000
2500
Main Channel Distance (ft)

2DM 100-Year 1-Pier Option

b_1ark12.flo

GEOMETRY & SOLUTION FILES bark1.bin b_1ark12.sol

0.000

LINE	NAME	Q,cfs	QX	QY	AREA,sf	LENGTH,ft
1	us appr	-26104.	-21335.	4768.	2810.	257.
2	us cont	-26035.	-14351.	11685.	2330.	229.

2DM 100-Year 2Pier Option

b_1ark22.flo

GEOMETRY & SOLUTION FILES bark2.bin b_1ark22.sol

0.000

LINE	NAME	Q,cfs	QX	QY	AREA,sf	LENGTH,ft
1	us appr	-26101.	-21331.	4770.	2866.	257.
2	us cont	-26074.	-13877.	12197.	2421.	229.

2DM 500-Year 1-Pier Option

a_5ark14.flo

GEOMETRY & SOLUTION FILES aark1.bin a_5ark14.sol

0.000

LINE	NAME	Q,cfs	QX	QY	AREA,sf	LENGTH,ft
1	u/s cont	-46018.	-24848.	21171.	3360.	245.
2	u/s appr	-46066.	-37469.	8596.	4009.	257.

2DM SCC-Year EVENT 2-Pier OPTION

a_5ark24.flo

GEOMETRY & SOLUTION FILES aark2.bin a_5ark24.sol

0.000

LINE	NAME	Q,cfs	QX	QY	AREA,sf	LENGTH,ft
1	u/s cont	-46091.	-24159.	21932.	3530.	245.
2	u/s appr	-46054.	-37428.	8626.	4110.	257.

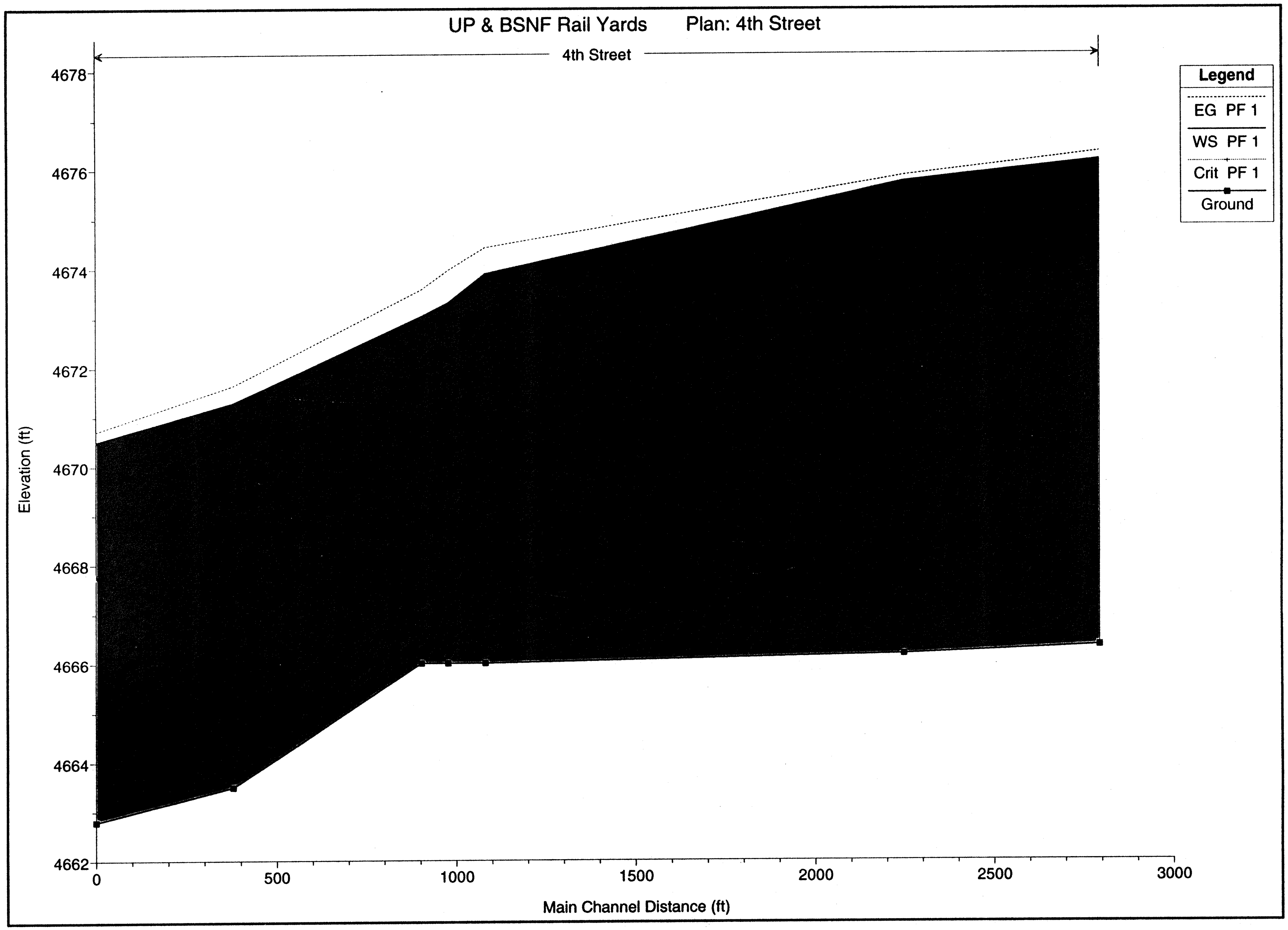
Flow Distribution Data UP & BSNF Rail Yards 4th Street RS: 3 Profile: PF 1

Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	% Conv.	Hydr D. (ft)	Velocity (ft/s)
LB 99.36	130.73	166.67	42.62	16.37	0.84	3.39	3.91
130.73	**162.1**	**1777.83**	**228.81**	**31.38**	**9**	**7.29**	**7.77**
162.1	193.47	1778.46	228.82	31.37	9	7.29	7.77
193.47	224.84	1778.46	228.82	31.37	9	7.29	7.77
224.84	**256.21**	**1570.86**	**212.45**	**31.39**	**7.95**	**6.77**	**7.39**
256.21	287.57	1390.98	197.45	31.37	7.04	6.29	7.04
287.57	318.94	1250.02	185.21	31.38	6.33	5.9	6.75
318.94	350.31	1052.29	167.02	31.37	5.33	5.32	6.3
350.31	**381.68**	**1042.54**	**166.08**	**31.37**	**5.28**	**5.29**	**6.28**
381.68	413.05	1029.83	164.86	31.37	5.21	5.26	6.25
413.05	444.42	992.31	161.23	31.37	5.02	5.14	6.15
444.42	475.79	954.66	157.53	31.37	4.83	5.02	6.06
475.79	507.16	917.59	153.84	31.37	4.65	4.9	5.96
507.16	**538.53**	**881.19**	**150.14**	**31.37**	**4.46**	**4.79**	**5.87**
538.53	569.9	845.29	146.44	31.37	4.28	4.67	5.77
569.9	601.26	809.05	142.64	31.37	4.1	4.55	5.67
601.26	632.63	748.13	136.11	31.37	3.79	4.34	5.5
632.63	**664**	**537.26**	**111.6**	**31.39**	**2.72**	**3.56**	**4.81**
664	695.37	226.59	60.65	24.95	1.15	2.65	3.74

[shaded] = Cross sections used for local scour computaions through rail yard

HEC-RAS River: UP & BSNF Rail Yards Reach: 4th Street Profile: PF 1

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
4th Street	5	19750	4666.34	4676.16		4676.32	0.001302	3.19	6200.07	1202.21	0.25
4th Street	4	19750	4666.17	4675.73		4675.85	0.000587	2.75	7210.19	965.29	0.18
4th Street	3.5	19750	4666	4673.88		4674.41	0.00368	5.85	3376.92	571.44	0.42
4th Street	3	19750	4666	4673.29		4673.95	0.005171	6.49	3042.34	568.72	0.49
4th Street	2	19750	4666	4673.02		4673.56	0.004392	5.87	3361.89	649.94	0.46
4th Street	1	19750	4663.49	4671.28		4671.63	0.002962	4.76	4160.62	824.1	0.37
4th Street	0	19750	4662.79	4670.5	4667.76	4670.71	0.001862	3.67	5383.24	1108.62	0.29


UP & BSNF Rail Yards Plan: 4th Street
4th Street
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
4678
4676
4674
4672
4670
4668
4666
4664
4662
0
500
1000
1500
2000
2500
3000
Main Channel Distance (ft)

APPENDIX D
Scour Calculations

EARTH ENGINEERING CONSULTANTS, INC.

SUMMARY OF GRADATION TEST RESULTS

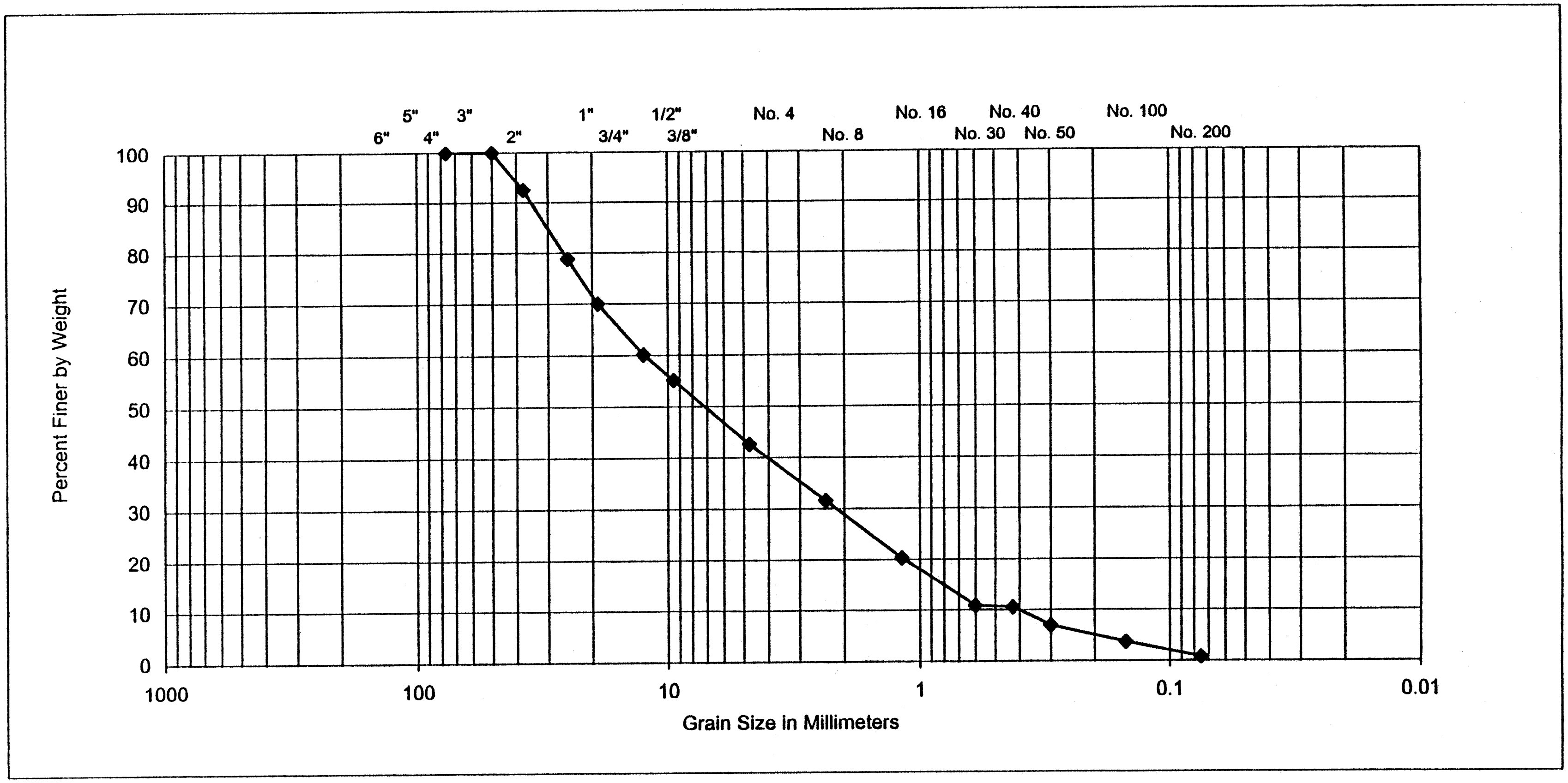
GRADATION OF AGGREGATE (ASTM C-136)	
SIEVE SIZE	PERCENT PASSING
1 1/2"	93%
1"	79%
3/4"	70%
1/2"	60%
3/8"	55%
No. 4	43%
No. 8	32%
No. 16	20%
No. 30	11%
No. 40	11%
No. 50	7%
No. 100	4%
No. 200	0.8%

Project: Ayres & Associates
Project Number: 1015015E
Sample Number: Arkansas River, 4th Street Bridge
Date: August 2001

EEC

EARTH ENGINEERING CONSULTANTS, INC.

Summary of Washed Sieve Analysis Tests (ASTM C-117 & C-136)



Cobble	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Project: Ayres & Associates
Project Number: 1015015E
Sample Number: Arkansas River, 4th Street Bridge
Date: August 2001

EEC

EARTH ENGINEERING CONSULTANTS, INC.

SUMMARY OF GRADATION TEST RESULTS

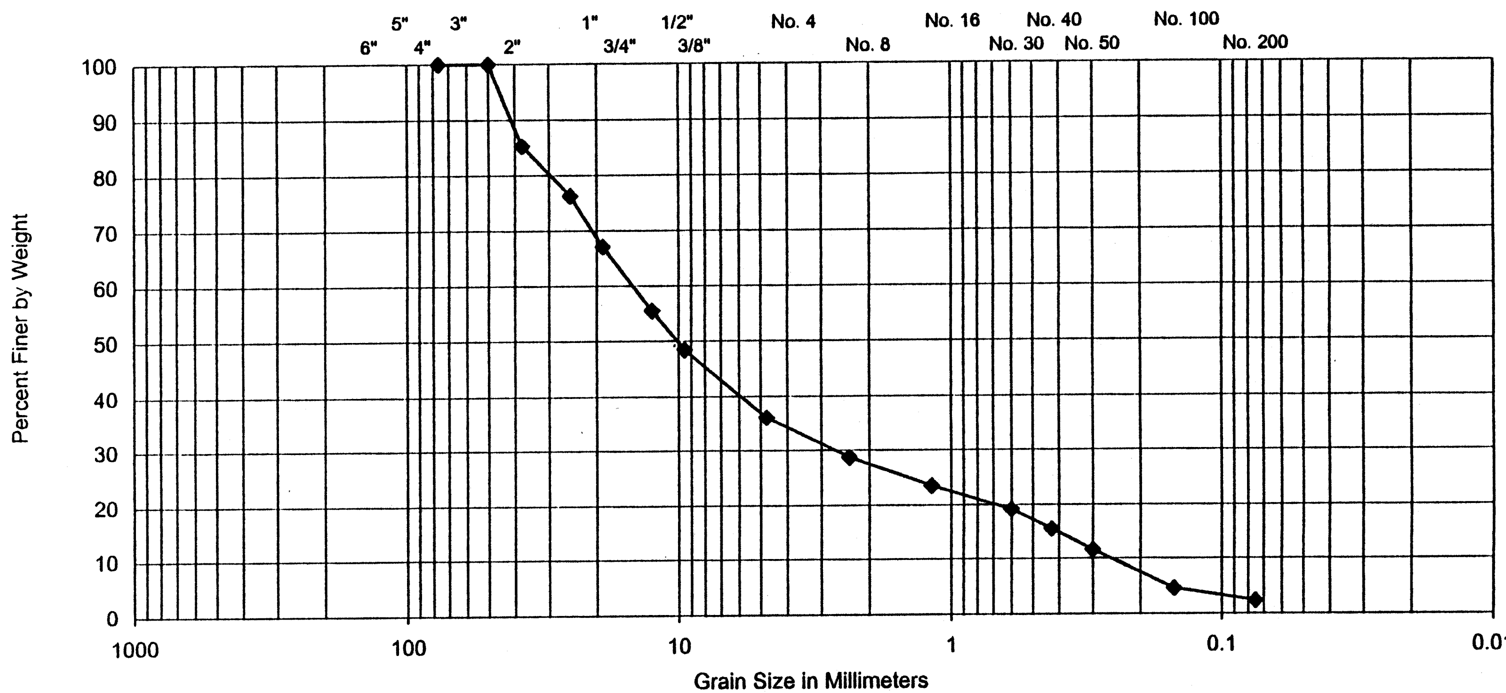
GRADATION OF AGGREGATE (ASTM C-136)	
SIEVE SIZE	PERCENT PASSING
1 1/2"	85%
1"	76%
3/4"	67%
1/2"	56%
3/8"	48%
No. 4	36%
No. 8	29%
No. 16	24%
No. 30	19%
No. 40	15%
No. 50	12%
No. 100	5%
No. 200	2.4%

Project: Ayres & Associates
Project Number: 1015015E
Sample Number: Arkansas River, Upstream of 4th Street Bridge
Date: August 2001

EEC

EARTH ENGINEERING CONSULTANTS, INC.

Summary of Washed Sieve Analysis Tests (ASTM C-117 & C-136)



Cobble	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Project: Ayres & Associates
Project Number: 1015015E
Sample Number: Arkansas River, Upstream of 4th Street Bridg
Date: August 2001

EEC

100-YEAR RIVERINE SCOUR SUMMARY
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

Pier/Bent	Groundline Elevation (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft)
AR-2	4658.1	1.2	21.5	22.7	4635.4
AR-1	4665.7	1.2	21.5	22.7	4643.0

500-YEAR RIVERINE SCOUR SUMMARY
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

Pier/Bent	Groundline Elevation (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft)
AR-2	4658.1	1.7	23.4	25.1	4633.0
AR-1	4665.7	1.7	23.4	25.1	4640.6

NOTES:
These tables present potential scour depths for the associated hydraulic events. If a soil horizon exists beneath the bridge which is resistant to scour, the predicted scour depths could be reduced to reflect the competence of the material. This reduction would require examination and approval by a qualified geotechnical engineer with knowledge of the properties of the material.

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[illegible]	Date:	9/26/01

SCOUR MODE COMPUTATION
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following computations are made using Laursen's Equation (Equation 15 in HEC-18):

$$V_c = 11.16 \times Y_1^{1/6} \times D_{50}^{1/3}$$

100-YEAR RIVERINE DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	2,866
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	257
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	11.15
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022966
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	4.741
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	26,000.0
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	9.07
MAIN CHANNEL SCOUR MODE	=	**LIVE-BED**

500-YEAR RIVERINE DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	4,110
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	257
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	15.99
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022966
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	5.04
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	46,000
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	11.19
MAIN CHANNEL SCOUR MODE	=	**LIVE-BED**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[illegible]	Date:	9/26/01

CONTRACTION SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following computations are made using the HEC-18 equation for Live Bed Contraction Scour:

$$Y_s=Y_2-Y_0$$

$$Y_2=((Q_2/Q_1)^{6/7}((W_1/W_2)^{k_1}))*Y_1$$

100-YEAR RIVERINE DISCHARGE

LIVE-BED CONTRACTION SCOUR COMPUTATIONS

ENERGY SLOPE	=	8.62E-04
ω FALL VELOCITY (fps)	=	1.31
AVERAGE UPSTREAM CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	11.15
V. SHEAR VELOCITY IN UPSTREAM SECTION (fps)	=	0.56
V*/w	=	0.42
k_1 SEE PAGE 30 IN HEC-18	=	0.59
DISCHARGE IN UPSTREAM CHANNEL (cfs), Q_1	=	26,000
DISCHARGE IN CONTRACTED SECTION (cfs), Q_2	=	26,000
WIDTH OF UPSTREAM CHANNEL SECTION (ft), W_1	=	257
WIDTH OF MAIN CHANNEL CONTRACTED SECTION (ft), W_2	=	220
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022965879
COMPUTED WATER DEPTH OF CONTRACTED SECTION (ft), Y_2	=	12.22
AVERAGE WATER DEPTH AT BRIDGE(ft), Y_0	=	11.00
AVERAGE SCOUR DEPTH AT CONTRACTED SECTION, Y_S	=	1.22

Calc. By:	WMdeR	Date:	9/20/01
Check By:	JWR	Date:	9/26/01

CONTRACTION SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following computations are made using the HEC-18 equation for Live Bed Contraction Scour:

$$Y_s = Y_2 - Y_0$$

$$Y_2 = ((Q_2/Q_1)^{6/7}((W_1/W_2)^{k_1}))*Y_1$$

500-YEAR RIVERINE DISCHARGE
LIVE-BED CONTRACTION SCOUR COMPUTATIONS

ENERGY SLOPE	=	8.11E-04
w FALL VELOCITY	=	1.31
AVERAGE UPSTREAM CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	15.99
V. SHEAR VELOCITY IN UPSTREAM SECTION	=	0.65
V*/w	=	0.49
k_1 SEE PAGE 30 IN HEC-18	=	0.59
DISCHARGE IN UPSTREAM CHANNEL (cfs), Q_1	=	46,000
DISCHARGE IN CONTRACTED SECTION (cfs), Q_2	=	46,000
WIDTH OF UPSTREAM CHANNEL SECTION (ft), W_1	=	257
WIDTH OF MAIN CHANNEL CONTRACTED SECTION (ft), W_2	=	230
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022965879
COMPUTED WATER DEPTH OF CONTRACTED SECTION (ft), Y_2	=	17.07
AVERAGE WATER DEPTH AT BRIDGE(ft), Y_0	=	15.35
AVERAGE SCOUR DEPTH AT CONTRACTED SECTION, Y_S	=	1.73

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

ALONG-WALL SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-23

HYDRAULIC VARIABLES USED IN IMPINGING-FLOW SCOUR	100-year	500-year
y_1: Average Upstream Flow Depth in the Main Channel (ft)	10.50	14.30
θ : Impinging Flow Angle (degrees)	5.00	5.00
v_1: Average Upstream Flow Velocity in Main Channel (fps)	15.90	19.70
g: Acceleration Due to Gravity (ftpsqsec)	32.20	32.20
F: Upstream Froude Number	0.86	0.92
$Y_{simpinging}$: Equilibrium Depth of Scour (ft)	14.56	20.53
Y_s Total: Total Scour at Wall (ft)	**14.6**	**20.5**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

100-YEAR RIVERINE DISCHARGE
LOCAL PIER SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-18 for Pier Scour:

SCOUR ANALYSIS FOR Q_{100} - CASE 1 (WITHOUT DEBRIS)

HYDRAULIC VARIABLES USED IN CSU EQUATION

PIER COMPONENT

	AR-2	AR-1
V_1: VELOCITY (fps)	12.30	12.30
Y_1: DEPTH (ft)	14.4	14.4
ATTACK ANGLE, Degrees	0	0
h_1: PIER STEM HEIGHT ABOVE BED (ft)	3.0	3.0
INDIVIDUAL PIER WIDTH (ft)	5.00	5.00
a: PIER WIDTH (ft)	5.00	5.00
L: PIER LENGTH (ft)	40.00	40.00
f: PIER SETBACK FROM EDGE OF PILE CAP (ft)	5.00	5.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	C	C
K_1: SHAPE COEFFICIENT	1.00	1.00
K_2: ANGLE COEFFICIENT	1.00	1.00
K_3: BED COND. COEFFICIENT	1.10	1.10
K_{hpier}: WEIGHTING FACTOR FOR PIER SCOUR	0.17	0.17
FROUDE NUMBER, Fr	0.57	0.57
LOCAL SCOUR DEPTH (ft), Ys_{pier}	**2.12**	**2.12**

CAP COMPONENT

Bent Number	AR-2	AR-1
V_2: VELOCITY (fps)	11.46	11.46
Y_2: DEPTH (ft)	15.5	15.5
ATTACK ANGLE, Degrees	0	0
h_{opc}: PRE-SCOUR PILE CAP BOTTOM HEIGHT ABOVE BED (ft)	3.0	3.0
D50, ft	0.022966	0.022966
Ks, ft	0.045932	0.045932
Vc critical transport velocity, fps	4.741500	4.741500
Y_f: distance from bed to top of footing, ft	4.1	4.1
V_f: average velocity in the flow zone below the top of the footing, ft/sec	9.6	9.6
a_{pc}: PILE CAP WIDTH (ft)	15.00	15.00
L_{pc}: PILE CAP LENGTH (ft)	60.00	60.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	c	c
K_1: SHAPE COEFFICIENT	1.00	1.00
K_2: ANGLE COEFFICIENT	1.00	1.00
K_3: BED COND. COEFFICIENT	1.10	1.10
K_w: WIDE PIER ADJUSTMENT FACTOR	1.00	1.00
FROUDE NUMBER, Fr	0.84	0.84
LOCAL SCOUR DEPTH (ft), Y_{spc}	**19.36**	**19.36**

	AR-2	AR-1
TOTAL SCOUR DEPTH = $Y_{spier} + Y_{spc} + Y_{spg}$	**21.5**	**21.5**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

500-YEAR RIVERINE DISCHARGE
LOCAL PIER SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
TWO PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-18 for Pier Scour:

SCOUR ANALYSIS FOR Q_{100} - CASE 1 (WITHOUT DEBRIS)

HYDRAULIC VARIABLES USED IN CSU EQUATION

PIER COMPONENT

	AR-2	AR-1
V_1: VELOCITY (fps)	15.00	15.00
Y_1: DEPTH (ft)	18	18
ATTACK ANGLE, Degrees	0	0
h_1: PIER STEM HEIGHT ABOVE BED (ft)	3.0	3.0
INDIVIDUAL PIER WIDTH (ft)	5.00	5.00
a: PIER WIDTH (ft)	5.00	5.00
L: PIER LENGTH (ft)	40.00	40.00
f: PIER SETBACK FROM EDGE OF PILE CAP (ft)	5.00	5.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	C	C
K_1: SHAPE COEFFICIENT	1.00	1.00
K_2: ANGLE COEFFICIENT	1.00	1.00
K_3: BED COND. COEFFICIENT	1.10	1.10
K_{hpier}: WEIGHTING FACTOR FOR PIER SCOUR	0.17	0.17
FROUDE NUMBER, Fr	0.62	0.62
LOCAL SCOUR DEPTH (ft), Ys_{pier}	**2.38**	**2.38**

CAP COMPONENT

Bent Number	AR-2	AR-1
V_2: VELOCITY (fps)	14.07	14.07
Y_2: DEPTH (ft)	19.2	19.2
ATTACK ANGLE, Degrees	0	0
h_{opc}: PRE-SCOUR PILE CAP BOTTOM HEIGHT ABOVE BED (ft)	3.0	3.0
D50, ft	0.022966	0.022966
Ks, ft	0.045932	0.045932
Vc critical transport velocity, fps	5.04	5.04
Y_f: distance from bed to top of footing, ft	4.2	4.2
V_f: average velocity in the flow zone below the top of the footing, ft/sec	11.5	11.5
a_{pc}: PILE CAP WIDTH (ft)	15.00	15.00
L_{pc}: PILE CAP LENGTH (ft)	60.00	60.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	c	c
K_1: SHAPE COEFFICIENT	1.00	1.00
K_2: ANGLE COEFFICIENT	1.00	1.00
K_3: BED COND. COEFFICIENT	1.10	1.10
K_w: WIDE PIER ADJUSTMENT FACTOR	1.00	1.00
FROUDE NUMBER, Fr	0.99	0.99
LOCAL SCOUR DEPTH (ft), Y_{spc}	**21.05**	**21.05**

TOTAL SCOUR DEPTH = $Y_{spier} + Y_{spc} + Y_{spg}$	**23.4**	**23.4**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

100-YEAR RIVERINE SCOUR SUMMARY
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

Pier/Bent	Groundline Elevation (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft)
AR-1A	4662.5	1.8	21.9	23.7	4638.8

500-YEAR RIVERINE SCOUR SUMMARY
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

Pier/Bent	Groundline Elevation (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft)
AR-1A	4662.5	3.0	23.6	26.6	4635.9

NOTES:
These tables present potential scour depths for the associated hydraulic events. If a soil horizon exists beneath the bridge which is resistant to scour, the predicted scour depths could be reduced to reflect the competence of the material. This reduction would require examination and approval by a qualified geotechnical engineer with knowledge of the properties of the material.

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[illegible]	Date:	9/26/01

SCOUR MODE COMPUTATION
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following computations are made using Laursen's Equation (Equation 15 in HEC-18):

$$V_c = 11.16 \times Y_1^{1/6} \times D_{50}^{1/3}$$

100-YEAR RIVERINE DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	2,810
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	257
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	10.93
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022966
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	4.726
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	26,000.0
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	9.25
MAIN CHANNEL SCOUR MODE	=	**LIVE-BED**

500-YEAR RIVERINE DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	4,009
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	257
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	15.60
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022966
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	5.01
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	46,000
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	11.47
MAIN CHANNEL SCOUR MODE	=	**LIVE-BED**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[illegible]	Date:	9/26/01

CONTRACTION SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following computations are made using the HEC-18 equation for Live Bed Contraction Scour:

$$Y_s = Y_2 - Y_0$$

$$Y_2 = ((Q_2/Q_1)^{6/7}((W_1/W_2)^{k_1}))*Y_1$$

100-YEAR RIVERINE DISCHARGE

LIVE-BED CONTRACTION SCOUR COMPUTATIONS

ENERGY SLOPE	=	9.20E-04
ω FALL VELOCITY (fps)	=	1.31
AVERAGE UPSTREAM CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	10.93
V· SHEAR VELOCITY IN UPSTREAM SECTION (fps)	=	0.57
V*/w	=	0.43
k_1 SEE PAGE 30 IN HEC-18	=	0.59
DISCHARGE IN UPSTREAM CHANNEL (cfs), Q_1	=	26,000
DISCHARGE IN CONTRACTED SECTION (cfs), Q_2	=	26,000
WIDTH OF UPSTREAM CHANNEL SECTION (ft), W_1	=	257
WIDTH OF MAIN CHANNEL CONTRACTED SECTION (ft), W_2	=	220
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022965879
COMPUTED WATER DEPTH OF CONTRACTED SECTION (ft), Y_2	=	11.98
AVERAGE WATER DEPTH AT BRIDGE(ft), Y_0	=	10.17
AVERAGE SCOUR DEPTH AT CONTRACTED SECTION, Y_S	=	1.81

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

CONTRACTION SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following computations are made using the HEC-18 equation for Live Bed Contraction Scour:

$$Y_s=Y_2-Y_0$$

$$Y_2=((Q_2/Q_1)^{6/7}((W_1/W_2)^{k_1}))*Y_1$$

500-YEAR RIVERINE DISCHARGE
LIVE-BED CONTRACTION SCOUR COMPUTATIONS

ENERGY SLOPE	=	8.81E-04
w FALL VELOCITY	=	1.31
AVERAGE UPSTREAM CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	15.60
V· SHEAR VELOCITY IN UPSTREAM SECTION	=	0.67
V*/w	=	0.51
k_1 SEE PAGE 30 IN HEC-18	=	0.64
DISCHARGE IN UPSTREAM CHANNEL (cfs), Q_1	=	46,000
DISCHARGE IN CONTRACTED SECTION (cfs), Q_2	=	46,000
WIDTH OF UPSTREAM CHANNEL SECTION (ft), W_1	=	257
WIDTH OF MAIN CHANNEL CONTRACTED SECTION (ft), W_2	=	230
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.022965879
COMPUTED WATER DEPTH OF CONTRACTED SECTION (ft), Y_2	=	16.75
AVERAGE WATER DEPTH AT BRIDGE(ft), Y_0	=	13.71
AVERAGE SCOUR DEPTH AT CONTRACTED SECTION, Y_S	=	3.03

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/21/01

ALONG-WALL SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-23

HYDRAULIC VARIABLES USED IN IMPINGING-FLOW SCOUR	100-year	500-year
y_1: Average Upstream Flow Depth in the Main Channel (ft)	11.50	15.50
θ : Impinging Flow Angle (degrees)	0.00	0.00
v_1: Average Upstream Flow Velocity in Main Channel (fps)	12.90	17.00
g: Acceleration Due to Gravity (ftpsqsec)	32.20	32.20
F: Upstream Froude Number	0.67	0.76
$Y_{simpinging}$: Equilibrium Depth of Scour (ft)	10.67	15.26
Y_s Total: Total Scour at Wall (ft)	**10.7**	**15.3**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[illegible]	Date:	9/26/01

100-YEAR RIVERINE DISCHARGE
LOCAL PIER SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-18 for Pier Scour:

SCOUR ANALYSIS FOR Q_{100} - CASE 1 (WITHOUT DEBRIS)

HYDRAULIC VARIABLES USED IN CSU EQUATION

PIER COMPONENT

	Main Channel
V_1: VELOCITY (fps)	12.80
Y_1: DEPTH (ft)	15.4
ATTACK ANGLE, Degrees	0
h_1: PIER STEM HEIGHT ABOVE BED (ft)	3.0
INDIVIDUAL PIER WIDTH (ft)	5.00
a: PIER WIDTH (ft)	5.00
L: PIER LENGTH (ft)	40.00
f: PIER SETBACK FROM EDGE OF PILE CAP (ft)	5.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	C
K_1: SHAPE COEFFICIENT	1.00
K_2: ANGLE COEFFICIENT	1.00
K_3: BED COND. COEFFICIENT	1.10
K_{hpier}: WEIGHTING FACTOR FOR PIER SCOUR	0.17
FROUDE NUMBER, Fr	0.57
LOCAL SCOUR DEPTH (ft), Ys_{pier}	**2.18**

CAP COMPONENT

Bent Number	Main Channel
V_2: VELOCITY (fps)	11.95
Y_2: DEPTH (ft)	16.5
ATTACK ANGLE, Degrees	0
h_{opc}: PRE-SCOUR PILE CAP BOTTOM HEIGHT ABOVE BED (ft)	3.0
D50, ft	0.022966
Ks, ft	0.045932
Vc critical transport velocity, fps	4.725931
Y_f: distance from bed to top of footing, ft	4.1
V_f: average velocity in the flow zone below the top of the footing, ft/sec	9.9
a_{pc}: PILE CAP WIDTH (ft)	15.00
L_{pc}: PILE CAP LENGTH (ft)	60.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	c
K_1: SHAPE COEFFICIENT	1.00
K_2: ANGLE COEFFICIENT	1.00
K_3: BED COND. COEFFICIENT	1.10
K_w: WIDE PIER ADJUSTMENT FACTOR	1.00
FROUDE NUMBER, Fr	0.87
LOCAL SCOUR DEPTH (ft), Y_{spc}	**19.68**

TOTAL SCOUR DEPTH = $Y_{spier} + Y_{spc} + Y_{spg}$	**21.9**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/26/01

100-YEAR RIVERINE DISCHARGE
LOCAL PIER SCOUR COMPUTATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO
ONE PIER OPTION

SEPTEMBER 2001

The following calculations are made using the methods outlined in HEC-18 for Pier Scour:

SCOUR ANALYSIS FOR Q_{100} - CASE 1 (WITHOUT DEBRIS)

HYDRAULIC VARIABLES USED IN CSU EQUATION

PIER COMPONENT

	Main Channel
V_1: VELOCITY (fps)	15.20
Y_1: DEPTH (ft)	20.2
ATTACK ANGLE, Degrees	0
h_1: PIER STEM HEIGHT ABOVE BED (ft)	3.0
INDIVIDUAL PIER WIDTH (ft)	5.00
a: PIER WIDTH (ft)	5.00
L: PIER LENGTH (ft)	40.00
f: PIER SETBACK FROM EDGE OF PILE CAP (ft)	5.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	C
K_1: SHAPE COEFFICIENT	1.00
K_2: ANGLE COEFFICIENT	1.00
K_3: BED COND. COEFFICIENT	1.10
K_{hpier}: WEIGHTING FACTOR FOR PIER SCOUR	0.17
FROUDE NUMBER, Fr	0.60
LOCAL SCOUR DEPTH (ft), Ys_{pier}	**2.43**

CAP COMPONENT

Bent Number	Main Channel
V_2: VELOCITY (fps)	14.34
Y_2: DEPTH (ft)	21.4
ATTACK ANGLE, Degrees	0
h_{opc}: PRE-SCOUR PILE CAP BOTTOM HEIGHT ABOVE BED (ft)	3.0
D50, ft	0.022966
Ks, ft	0.045932
Vc critical transport velocity, fps	5.01
Y_f: distance from bed to top of footing, ft	4.2
V_f: average velocity in the flow zone below the top of the footing, ft/sec	11.6
a_{pc}: PILE CAP WIDTH (ft)	15.00
L_{pc}: PILE CAP LENGTH (ft)	60.00
PIER SHAPE (S=SQUARE, C = CIRCULAR)	c
K_1: SHAPE COEFFICIENT	1.00
K_2: ANGLE COEFFICIENT	1.00
K_3: BED COND. COEFFICIENT	1.10
K_w: WIDE PIER ADJUSTMENT FACTOR	1.00
FROUDE NUMBER, Fr	1.00
LOCAL SCOUR DEPTH (ft), Y_{spc}	**21.13**

TOTAL SCOUR DEPTH = $Y_{spier} + Y_{spc} + Y_{spg}$	**23.6**

Calc. By:	WMdeR	Date:	9/20/01
Check By:	[signature]	Date:	9/21/01

500-Year Discharge
LOCAL PIER SCOUR COMPUTATIONS
4th Street Bridge - Rail Yard
Pueblo, Colorado

The following calculations are made using Equation 6.3 in HEC-18 for Pier Scour:

$$Y_s = (2 * K_1 * K_2 * K_3 * (Y_1/a)^{0.35} * Fr^{0.43}) * a$$

SCOUR ANALYSIS FOR Q_{500}

HYDRAULIC VARIABLES USED IN CSU EQUATION

STATION PIER/BENT NUMBER	**RR 3**	**RR 4**	**RR 5**	**RR 6**	**RR 7**
V_1: VELOCITY (fps)	4.81	5.87	6.28	7.39	7.77
Y_1: DEPTH (ft)	3.56	4.79	5.29	6.77	7.29
ATTACK ANGLE, Degrees	0.00	0.00	0.00	0.00	0.00
a: PIER WIDTH (ft)	5.00	5.00	5.00	5.00	5.00
L: PIER LENGTH (ft)	---	---	---	---	---
K_1: SHAPE COEFFICIENT	1.00	1.00	1.00	1.00	1.00
K_2: ANGLE COEFFICIENT	1.00	1.00	1.00	1.00	1.00
K_3: BED COND. COEFFICIENT	1.10	1.10	1.10	1.10	1.10
FROUDE NUMBER, Fr	0.45	0.47	0.48	0.50	0.51
LOCAL SCOUR DEPTH (ft), Y_S	6.92	7.85	8.19	9.08	9.37
MAX SCOUR DEPTH ($a*K_1*K_2^{1.538}*2.4$), Y_{sm}	12.00	12.00	12.00	12.00	12.00
CONTROLLING LOCAL SCOUR DEPTH (ft), Y_s	**6.92**	**7.85**	**8.19**	**9.08**	**9.37**

Calc. By:	JJE	Date:	9/25/2001
Check By:	[signature]	Date:	9/25/2001

100-YEAR RIVERINE DISCHARGE
HARP DIVERSION STRUCTURE SCOUR CALCULATIONS
FOR PROPOSED 4th ST BRIDGE OVER ARKANSAS RIVER
PUEBLO, COLORADO

November 2001

The following calculations are made using the methods outlined in HEC-23 for Vertical Drop Scour:

VARIABLES USED IN USBR VERTICAL DROP SCOUR EQUATION

q: DISCHARGE PER UNIT WIDTH, (cfs/ft)	68.26	
Ht: TOTAL DROP IN HEAD - FROM U.S. TO D.S. EGL (ft)	7.5	
dm, Yd: TAILWATER DEPTH (ft)	12.9	average
Ku: English = 1.32, SI = 1.9	1.32	
SCOUR DEPTH DOWNSTREAM OF DIVERSION (ft), ds =	**7.4**	average

DOWNSTREAM POST SCOUR BED ELEVATION (ft) =	**4642.7**

EXISTING BED ELEVATION (ft) =	**4645.8**

Therefore, all but 3.1 ft of the 7.4 ft of scour has already occurred at the left end of the diversion structure.
Conclusion, this structure could probably withstand the 100-year scour.