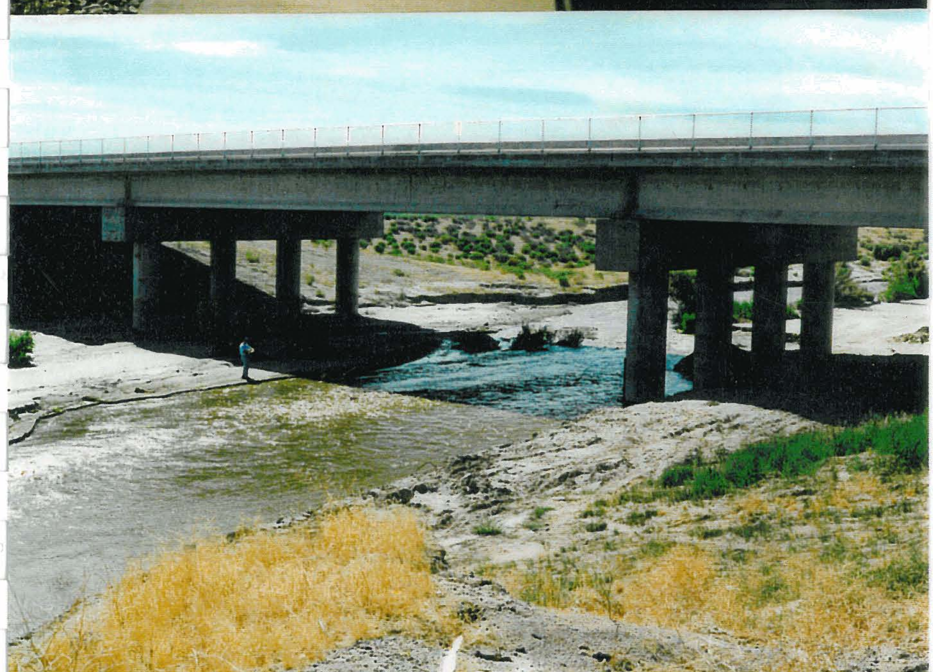

Final
BRIDGE SCOUR ASSESSMENT REPORTS
for 16 Maricopa County Bridges

VOLUME II
Structure Numbers
9427, 9588, 9999, 8038
7818, 9154, 9142, 7553

July, 1996

MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION (MCDOT)
Work Order No. 80407

Registered Professional Engineer (Structural)
CERTIFICATE NO.
6161
JERRY A.
CANNON
Date signed 7/9/96
ARIZONA, U.S.A.

Cannon & Associates, Inc.
Consulting Engineers

Final
BRIDGE SCOUR ASSESSMENT REPORTS
for 16 Maricopa County Bridges
CA 94046-1

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July, 1996

Prepared for
MARICOPA COUNTY
DEPARTMENT OF
TRANSPORTATION (MCDOT)
2910 West Durango Street
Phoenix, Arizona 85009
602-506-8600

Prepared by
Cannon & Associates, Inc.
Consulting Engineers
2701 North 16th Street, Suite 122
Phoenix, Arizona 85016
602-230-0563

Registered Professional Engineer (Structural)
CERTIFICATE NO.
6161
JERRY A.
CANNON
Date signed 7/9/96
ARIZONA, U.S.A.

MCDOT W.O. No. 80407
Maricopa County Department of Transportation
BRIDGE SCOUR ASSESSMENT

VOLUME I: FINAL BRIDGE SCOUR ASSESSMENT REPORTS
for Structure Numbers 9691, 8981, 9301, 9859, 9145, 7819, 8028, 8639

VOLUME II: FINAL BRIDGE SCOUR ASSESSMENT REPORTS
for Structure Numbers 9427, 9588, 9999, 8038, 7818, 9154, 9142, 7553

VOLUME III: TECHNICAL APPENDIX
Final Bridge Scour Hydraulic Calculations

VOLUME IV: TECHNICAL APPENDIX
Final Bridge Scour Structural Calculations

VOLUME V: TECHNICAL APPENDIX
Final Geotechnical Analysis

Cannon & Associates, Inc.
Consulting Engineers

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Maricopa County
Department of Transportation
Work Order No. 80407
Date: July 9, 1996

EXECUTIVE SUMMARY

The following is a Summary Listing of the 16 Bridges that were evaluated for Scour and the Scour Assessment Results:

Bridge	Structure No.	Location of Bridge	Footing Type	Scour Assessment
1	9691	Bell Road Bridge over Agua Fria River	Drilled Shaft	Scour Stable
2	8981	Olive Avenue Bridge over Agua Fria River	Drilled Shaft	Scour Stable
3	9301	Glendale Avenue Bridge over Agua Fria River	Spread Footings	Scour Critical
4	9859	Camelback Road Bridge over Agua Fria River	Drilled Shaft	Scour Stable
5	9145	Indian School Road Bridge over Agua Fria River	Spread Footings and Drilled Shafts	Scour Critical
6	7819	Maricopa County Highway 85 Bridge over Agua Fria River	Steel Pile	Scour Critical
7	8028	New River Road Bridge over New River	Spread Footings	Scour Critical
8	8639	I-17 Frontage Road Bridge over New River	Spread Footings	Scour Stable
9	9427	Peoria Avenue Bridge over New River	Spread Footings	Scour Stable
10	9588	Olive Avenue Bridge over New River	Spread Footings	Scour Stable
11	9999	Old US 80 Bridge over Hassayampa River	Drilled Shaft	Scour Critical
12	8038	Rittenhouse Road Bridge over Queen Creek	Steel Pile	Scour Critical
13	7818	Hawes Road Bridge over Queen Creek	Drilled Shaft	Scour Stable
14	9154	Power Road Bridge over Queen Creek	Steel Pile	Scour Critical
15	9142	Higley Road Bridge over Queen Creek	Steel Pile	Scour Stable
16	7553	Deer Valley Road Bridge over unnamed wash	Drilled Shaft	Scour Critical

The Federal Highway Administration (FHWA) has directed that all existing bridges over waterways be evaluated for the risk of failure from scour during a superflood on the order of magnitude of a 500-year flood. The Maricopa County Department of Transportation (MCDOT) owns approximately 111 bridges over waterways. In April 1995, MCDOT retained Cannon & Associates, Inc. Consulting Engineers as Prime Consultant to direct an interdisciplinary team of structural, hydraulic, and geotechnical engineers to evaluate 16 of these bridges to determine their vulnerability to scour. The study team includes:

Cannon & Associates, Inc. Consulting Engineers
Prime Consultant and Structural Engineer

Morrison-Maierle/CSSA
Hydraulic Engineer

AGRA Earth & Environmental
Geotechnical Engineer

Urban Engineering
Field Surveys

The procedures used for evaluating the bridges were developed in accordance with FHWA recommendations and guidelines included in Technical Advisory T 5140.23, October 28, 1991 and FHWA Hydraulic Engineering Circulars 18 and 20 (HEC-18 and HEC-20).

The evaluation discharge is the lesser of the 500-year discharge or the discharge that just reaches the low chord elevation of the bridge. The purpose of the study is to evaluate for scour and to classify the bridges as follows:

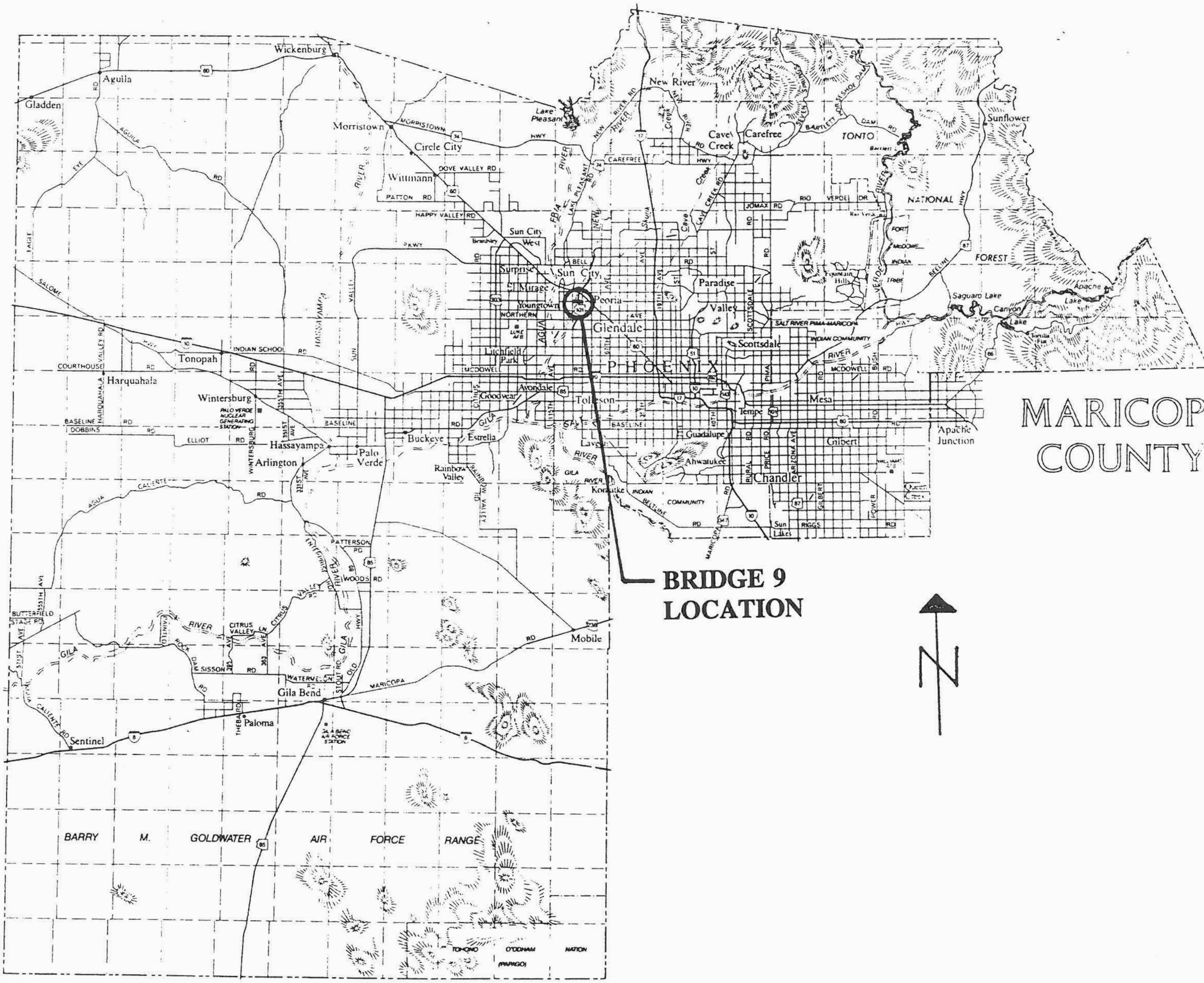
<u>Scour Stable</u>: Scour stable bridges are considered safe from catastrophic failure due to scour or erosion associated with a determinant discharge referred to as the evaluation discharge.

<u>Scour Critical</u>: Scour critical bridges are considered to be at risk of catastrophic failure due to scour or erosion produced by the evaluation discharge.

This report incorporates the findings of a preliminary scour assessment based on historical records, aerial photographs, site inspections, as-built plans, reports, and other available information.

BRIDGE 9

PEORIA AVENUE BRIDGE OVER NEW RIVER



Location Map


New River
Peoria Ave.
Peoria Ave. Bridge #9427
over New River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994
Figure 2

BRIDGE 9: PEORIA AVENUE BRIDGE OVER NEW RIVER (Structure #9427)

Assessment: Scour Stable

LOCATION: The Peoria Avenue Bridge at New River lies on the section line between Sections 21 and 28 of T3N, RIE, Gila and Salt River Baseline and Meridian, on Peoria Avenue between 95th Avenue and 99th Avenue. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a four-span, precast concrete I-girder bridge with a total length of 298'-6" center-to-center of abutment bearings and a skew of 25 degrees to the right. (See Location Plan, Figure 3.) The flow rate used for design was 38,000 cubic feet per second (cfs), corresponding to a 50-year flood. The bridge was designed by Karan Engineering Corp. in 1968 and built in 1972 as MCHD Project No. S-382(2).

The abutments consist of reinforced concrete walls on spread footings founded approximately 5' below existing grade. The footing measures 84' long, 12.5' wide and 2.5' high. Short wingwalls parallel to the roadway centerline extend from the ends of the abutment wall.

The piers also consist of reinforced concrete walls on spread footings which are founded approximately 5' below existing grade. Pier walls are 1'-6" wide, 77' long, and have rounded noses.

EXISTING SCOUR PROTECTION: Scour protection at the bridge consists of a grouted riprap sill across the entire bottom of the channel, extending approximately 36' upstream and downstream from the faces of the piers, measured along the pier axis. According to as-built plans at the Flood Control District of Maricopa County (FCDMC), the sill is 15" thick and is keyed 15' into the stream bed at both the downstream and upstream ends. In addition, the channel banks are lined with soil-cement upstream and downstream of the bridge, thereby providing scour protection for the abutments.

OTHER SITE CHARACTERISTICS: New River has been channelized into a nearly-rectangular, flat-bottomed section for most of its length between Grand Avenue and Olive Avenue. The sides have been lined with soil-cement at approximately 1/4:1 slopes, and the invert has been lowered approximately 6' from its original grade across the entire width of the channel. The bottom width of the channel increases gradually from upstream to downstream through the bridge section.

STREAM FORM: Because New River has been channelized, it does not have a natural stream form, although the apparent low flow channel can be characterized as straight to braided. (See Figure 4.) Occasional low sand bars form along the direction of flow. Channel sediments generally become finer towards the banks.

LAND USE: Land use in the vicinity of the bridge is primarily commercial and medium to high density residential, with some undeveloped land along the Agua Fria Freeway to the east. It is assumed that urbanization will increase, which may have an impact on flows of low return frequency, but not on major floods.

?

SURFACE SOILS: Surface soils consist primarily of sand, gravel and cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 8 mm. The armoring potential of the river bed is estimated to be moderate.

SLOPE: The estimated slope of New River in the vicinity of the Peoria Avenue Bridge, as measured from U.S. Geological Survey (USGS) topographic maps, is 0.0028 ft/ft, or approximately 15' per mile. Not correct since the bank has been

VEGETATION: Vegetation on the channel bottom is very sparse and consists primarily of small bushes and low grasses. There is no vegetation on the soil-cement banks.

STREAM STABILITY: Lateral stability of the stream is maintained by the soil-cement banks. The low-flow channel, however, is free to meander across the width of the channel.

Vertical stability in the river is controlled by concrete grade control structure approximately 800' upstream of the bridge. Inspection of this structure showed some erosion of the channel on the downstream side. At the bridge, the channel grade is controlled by the grouted riprap sill described previously. The sill is in good condition, with a very slight amount of channel erosion below the downstream edge.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in New River at Peoria Avenue come from discharges from New River Dam on New River near Jomax Road, approximately 10 miles upstream from the bridge, from tributaries such as Skunk Creek and from regional detention basins and drainage channels. Smaller flows come from off-site developed and undeveloped lands between the upstream dams and the bridge.

Available plans, flow records, and hydrologic models provided the following information:

1. The flow and flood frequency used in design are 38,000 cfs and 50 years, respectively, according to the construction documents.

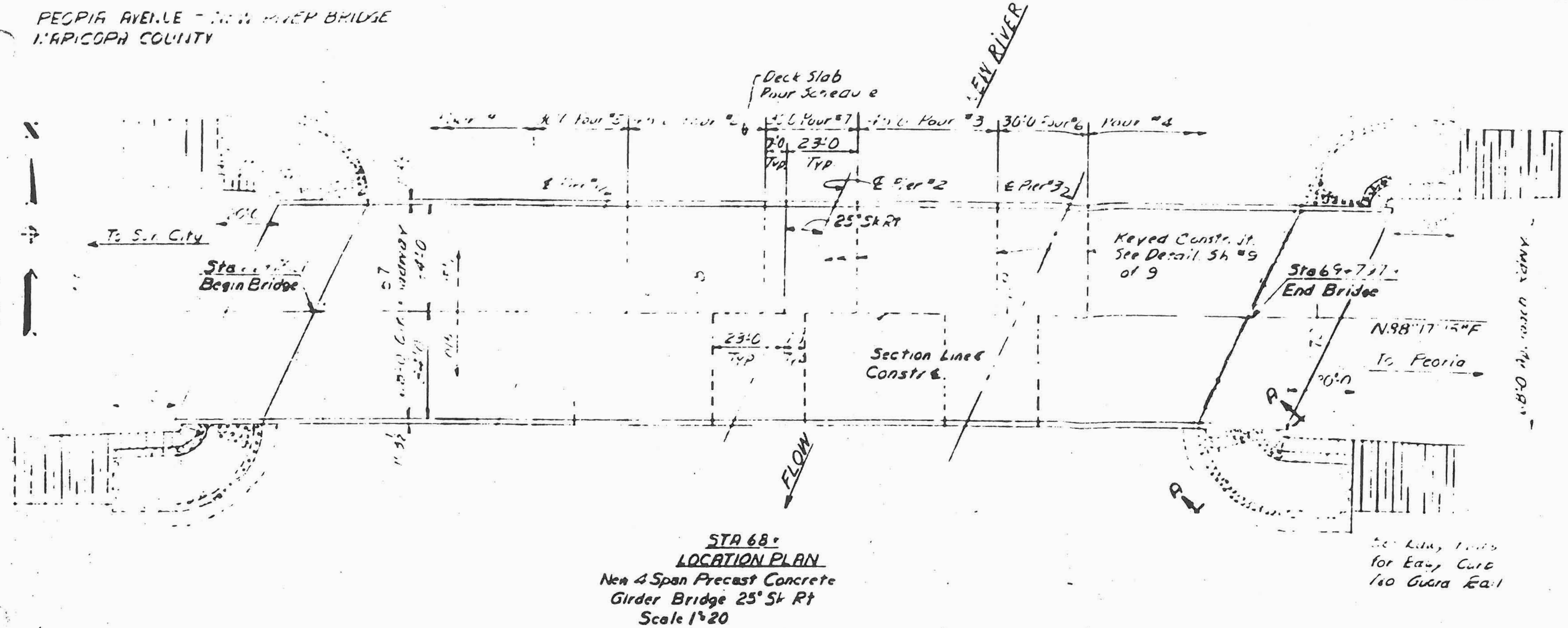
2. USGS data show that the largest recorded flood between 1961 and the present was 19,800 cfs on December 19, 1967, as measured at the flow gauge on the Glendale Avenue Bridge over New River, approximately 3 miles downstream from the Peoria Avenue Bridge.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 41,000 cfs.

4. The Federal Emergency Management Agency (FEMA) flood insurance study of 1993 estimates the 500-year flood (superflood) at 75,000 cfs.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided

PEORIA AVENUE - NEW RIVER BRIDGE
MARICOPA COUNTY

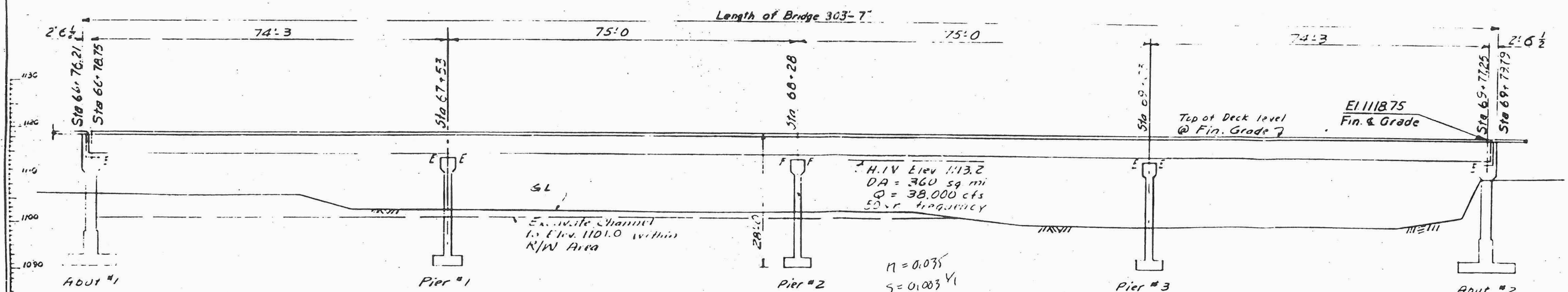


STA 68+
LOCATION PLAN
New 4 Span Precast Concrete
Girder Bridge 25° Sk Rt
Scale 1"=20'

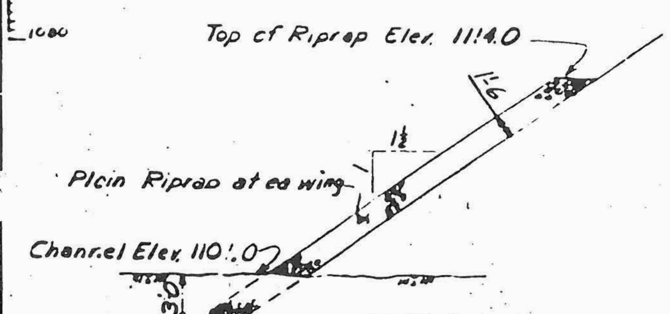
GENERAL NOTES:
Construction - Standard Specs Arizona Highway Dept
Edition of 1960, revised to date
Design - AASHO Specs for Highway Bridges 1965
Loading Class - HS 20-44
Composite Design - Dead Load carried by girders
Stresses - Class A Concrete fc = 1000 psi
Class D Concrete fc = 1200 psi n = 10
Class S Concrete f'c = See Plans; n = 6
Reinforcing Steel fs = 20,000 psi
Structural Steel fs = 20,000 psi ASTM A 36

Prestressing Steel Min. Ultimate Strength
High Strength Strands:
7/16 Strands Ult Strength = 31,000 # A = .117 in²
1/2 Strands Ult Strength = 41,300 # A = .153 in²
All concrete shall be Class A except deck slab shall be Class D and prestressed girders shall be Class S.
Reinforcing steel shall be intermediate grade and shall conform to ASTM Specs. A 615 Grade 40.
Structural steel shall conform to ASTM A 36

All dimensions for reinforcing steel shall be to center of bars except when noted otherwise on plans
Dimensions shall not be scaled from drawings.
Type A Joint Filler is Chemically or Bituminous Treated Preformed Expansion Joint Filler for Concrete AASHO M 213.



SECTION ON ℄ BRIDGE
Scale 1"=10'



SECTION A-A
Scale ⅛"=1'-0

APPROXIMATE QUANTITIES

Item	Struct. Excav CY	Spec. Backfill CY	Conc. C.Y. Class A	Conc. C.Y. Class D	Reinf Steel Lbs	Precast Girders ea.	Metal Hdrl. Lin. Ft.
2 Abutments	2,660	2,060	738.52		90,460		
3 Piers	870		404.89		29,240		
Deck			113.11	439.44	175,370	44	689.17
2 Appro. Slabs			149.60		13,380		
TOTALS	3,530	2,060	1406.12	439.44	308,450	44	689.17

Plain Riprap 340 C.Y.

KARAN ENGINEERING CORP
STRUCTURAL & CIVIL ENGINEERS
PHOENIX ARIZONA

Design	N.F.K	2-68
Drawn	J.A.N	2-68
Checked	N.F.K	2-68

Sheet No. 2 of 9

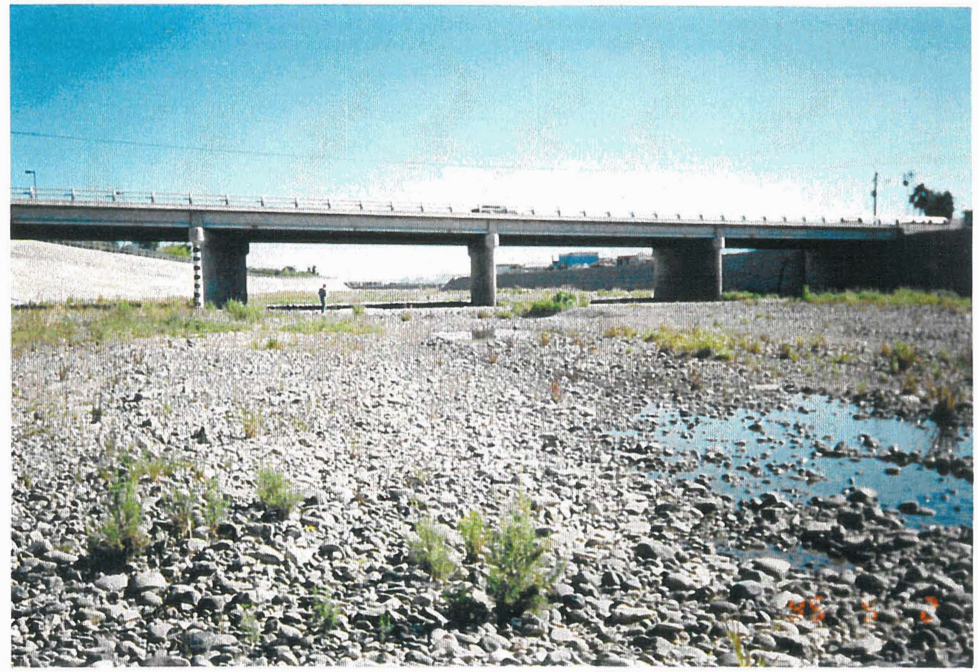
MARICOPA COUNTY HIGHWAY DEPARTMENT
COUNTY of MARICOPA
STA 68+
NEW RIVER BRIDGE
LOCATION DETAILS

Bridge No
Drawing No

Figure 3

PEORIA AVENUE

Water Course	New River
Stream Form	Straight to braided
Sinuosity	Not applicable
General Channelization	Sides of channel US and DS of bridge lined with soil cement forming trapezoidal section with 0.25:1 sideslopes.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.004603
Channel Contraction/Expansion	Narrower at US grade control, slowly expands toward/through bridge section.
Primary Surface Sediment Type	sand/gravel/cobbles
D50 Size	8 MM (estimate)
Armoring Potential	Moderate
Channel Vegetation	
Type/Size	Small bush, low grasses.
Density/Occurrence	Very sparse.
Relative Age	Young
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Soil cement banks.
Vertical	Grouted rip-rap sill at bridge extending approximately 36 ft. US/DS; grade control structure approximately 800 ft. US.
Sediment Deposits & Bars	Occasional low bars forming along channel flow direction. Channel sediments generally accumulate and become finer toward banks.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Land use commercial, medium to high density residential with some undeveloped land along Agua Fria Freeway; general assumption is for increasing urbanization.
Sand & Gravel Extraction	Evidence of former instream gravel operation approximately 1500 ft. DS.
Freeway Construction	No, but general roadway improvements are likely in vicinity.
Dams	New River dam several miles US near Jomax Road.
Drainage Channels	Small diameter storm drain outfalls on both sides of channel near bridge.



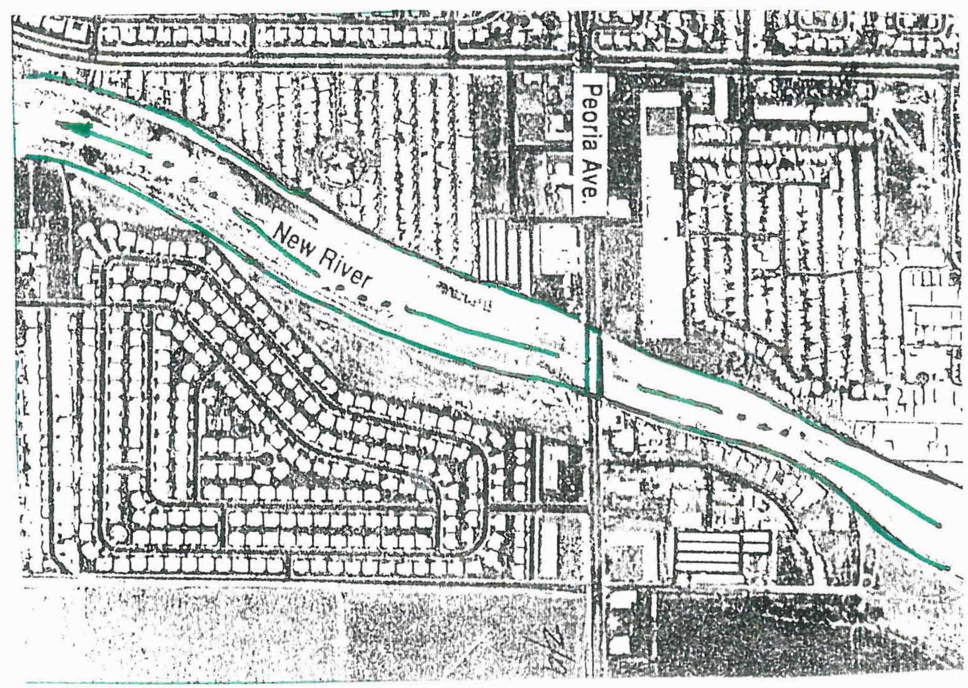


Figure 4

based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1100.69	13,500	Q_{10}
1105.17	38,000	Q_{Design}
1105.63	41,000	Q_{100}
1110.06	75,000	Q_{500}
1113.89	110,700	Low Chord

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 75,000 cfs.

SCOUR CALCULATIONS: Scour at the piers and abutments of the Peoria Avenue Bridge is considered to effectively prevented by the grouted riprap sill across the bottom of the channel and by the soil-cement banks.

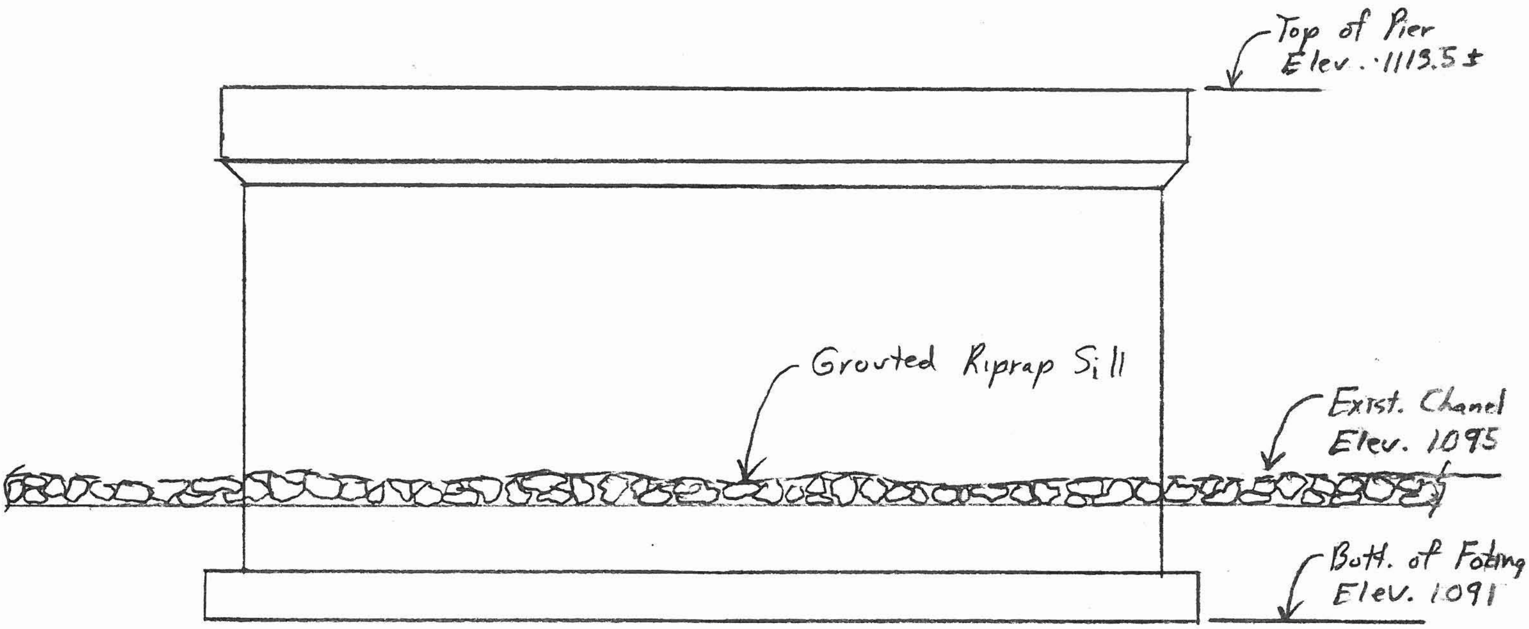
CONCLUSIONS: A grouted riprap sill has been placed across the bottom of the channel, and the banks are protected with soil cement. As long as these protective measures are in place, the bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: The condition of the sill and banks should be monitored frequently and any erosion be promptly repaired.

PEORIA AVENUE BRIDGE OVER NEW RIVER

HYDRAULIC DATA (Per MM/CSSA)

Q_{500} = 75,000 cfs
H.W. Elev = 1110.06
Total Scour = 0'



PIER ELEVATION

Figure 5

Photo 1: View looking toward upstream face of structure from approximately mid-channel slightly downstream of grade control structure. Note that the relatively coarse nature of typical channel sediments may have significant potential for armoring channel during low to moderate flows. Also note sparse grasses typical of this reach increase in density slightly toward channel embankments.

Photo 2: View looking downstream from top of right embankment near structure. Note relatively clear, coarse low flow channel. Also note sparsely grassed, sandier channel toward embankments.

Photo 3: View looking across upstream face from right embankment. Note grouted rip-rap channel bottom extending approximately 30 feet upstream and downstream of structure. Also note absence of significant debris in bridge waterway.

Photo 4: View of grouted rip-rap and typical channel material downstream of structure. Concrete is somewhat eroded and slight scour has occurred at interface.

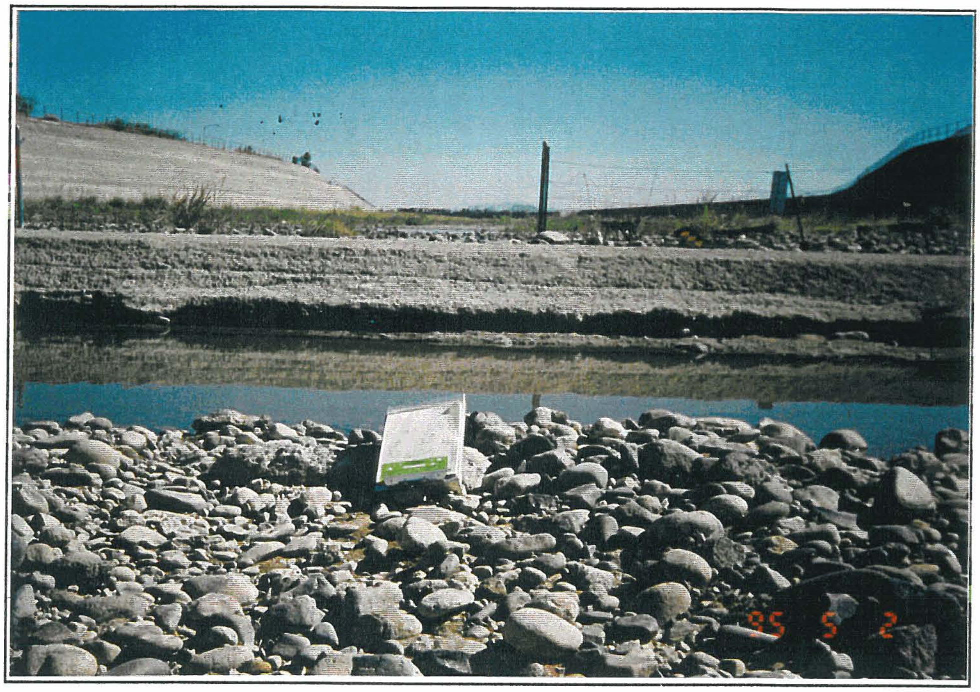
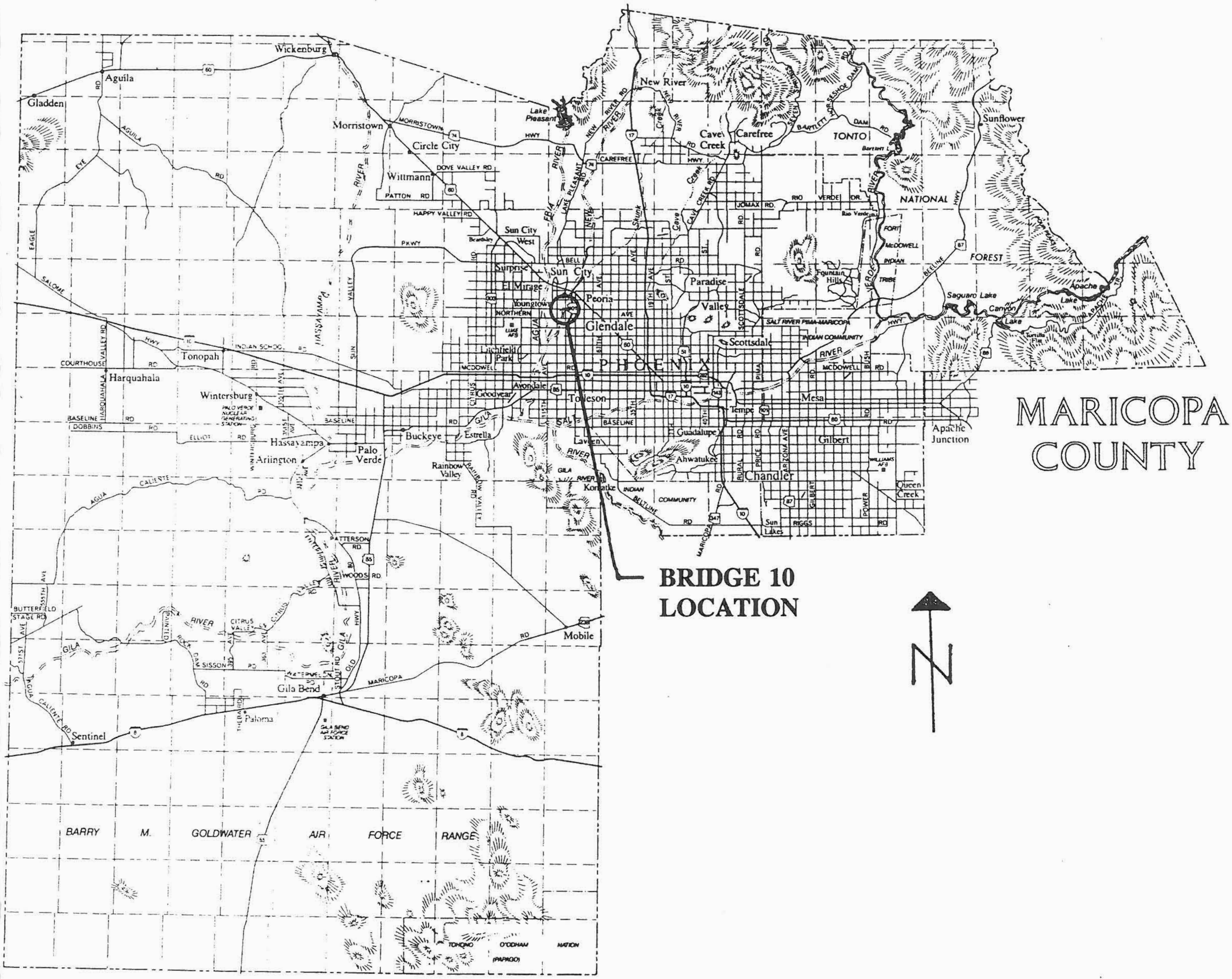


Photo 5: View looking toward grade control structure upstream of bridge. Note 2-3 foot erosion of channel material along downstream side of structure.

Photo 6: View looking at upstream edge of east side pier. Note sediment deposition along east side of solid pier.

BRIDGE 10

OLIVE AVENUE BRIDGE OVER NEW RIVER



Location Map

Figure 1


New River
Olive Ave.
Olive Ave. Bridge #9588
over New River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994
Figure 2

BRIDGE 10: OLIVE AVENUE BRIDGE OVER NEW RIVER (Structure #9588)

Assessment: Scour Stable

LOCATION: The Olive Avenue Bridge at New River lies on the section line between Sections 28 and 33 of T3N, R1E, Gila and Salt River Baseline and Meridian, on Olive Avenue between 95th Avenue and 99th Avenue. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a four-span, precast concrete I-girder bridge with an total length of 300' center-to-center of abutment bearings and a skew of 20 degrees to the right. (See Location Plan, Figure 3). "As-built" plan show that the flow rate used for design was 19,000 cubic feet per second (cfs), corresponding to a 14 year flood. The plan also notes that the 100 year flow was 55,000 cfs. The bridge was designed by Earle V. Miller Engineers in 1975 and built in 1976 as Maricopa County Highway Department (MCHD) Project No. M-502-7(1).

The abutments consist of five 3' diameter reinforced concrete columns on spread footings founded approximately 14' below existing grade. Short wingwalls parallel to the roadway centerline extend from the ends of the abutment wall.

The piers also consist of five 3' diameter reinforced concrete walls on spread footings which are founded approximately 14' below existing grade. The clear distance between the circular columns is 17.35'.

EXISTING SCOUR PROTECTION: Scour protection at the bridge consists of a reinforced concrete sill across the entire bottom of the channel, extending approximately 29' upstream and 25' downstream of the faces of the piers, measured along the pier axis. According to as-built plans at the Flood Control District of Maricopa County (FCDMC), the sill is 8" thick and is keyed into the stream bed at a 1:1 slope to a depth of 15' on the upstream side of the bridge and 20' on the downstream side. In addition, the channel banks are lined with soil-cement upstream and downstream of the bridge, thereby providing scour protection for the abutments. The space between the columns at the abutments has been filled with reinforced concrete to create a smooth inclined surface matching the slope of the channel banks.

A review of bridge inspection reports showed that there was no mention of scour at the bridge. Considering that the river has been channelized, the absence of scour problems in the reports is not unexpected.

The only scour observed during the site inspection was minor erosion of the soil-cement bank at the upstream side of the east abutment.

STREAM FORM: New River has been channelized into a nearly-rectangular, flat-bottomed section for most of its length between Grand Avenue and Olive Avenue. The sides have been lined with soil-cement at slopes ranging from 1/4:1 to 1:1 and the invert has been lowered approximately 6' from its original grade across the entire width of the channel. The bottom width of the channel is contracted at the bridge site, increasing gradually in the downstream direction. Approximately 1/4 mile downstream of the bridge, the channel divides around an "island" of natural ground. The "island" is approximately 6' higher than the channel invert around it.

The channel banks make a sharp bend to the left (in the direction of flow) at the Olive Avenue Bridge. The bend is approximately 45 degrees at the east abutment, but only about 10 degrees at the west abutment. Flows in the river therefore have a tendency to cross the bridge from east to west, with the formation of a stagnant area downstream of the bridge on the east side.

Because New River has been channelized, it does not have a natural stream form, although the apparent low flow channel can be characterized as braided to meandering upstream of the bridge and braided to straight downstream of the bridge. (See Figure 4.) Sand bars upstream of the bridge consist of low point, alternate and poorly developed middle bars; downstream of the bridge, occasional low elongated bars form in the direction of flow.

LAND USE: Land use in the vicinity of the bridge is primarily commercial and medium to high density residential, with some agricultural land downstream of the bridge on the left bank of the river. It is assumed that urbanization will increase, which may increase flow rates for low-frequency return periods but not for major floods.

SURFACE SOILS: Surface soils consist primarily of sand, gravel and cobbles. There is a considerable amount of sand accumulated in the lower part of the concrete sill. Silt deposits are common downstream of the east abutment. The armoring potential of the river bed is estimated to be moderate.

SLOPE: The slope of New River in the vicinity of the Olive Avenue Bridge is 0.0028 ft/ft, or approximately 15' per mile, as estimated from U.S. Geological Survey (USGS) topographic maps.

VEGETATION: Vegetation on the channel bottom is very sparse and consists primarily of small bushes and low grasses. There is no vegetation on the soil-cement banks. There is a moderately dense stand of grasses approximately 3' high growing downstream of the concrete sill.

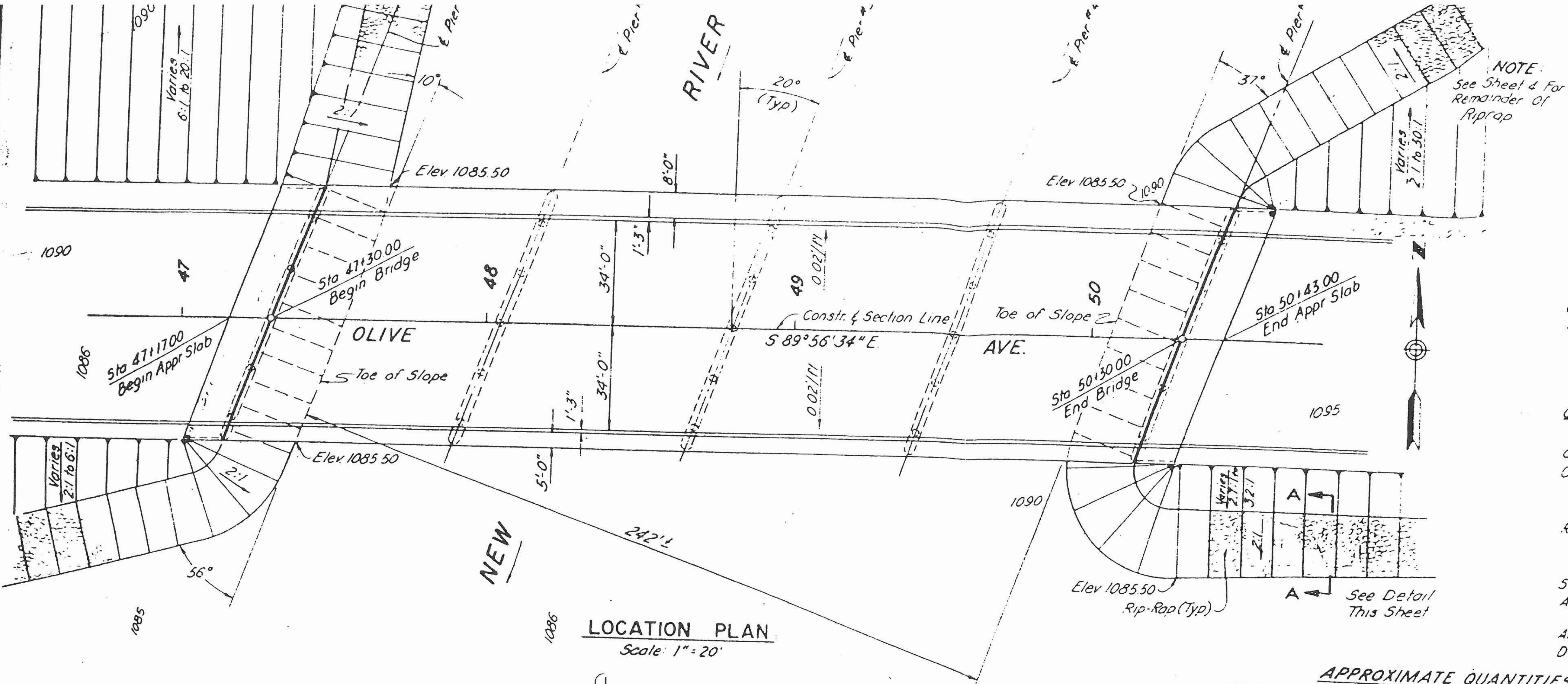
STREAM STABILITY: Lateral stability of the stream is maintained by the soil-cement banks. The low-flow channel, however, is free to meander across the width of the channel.

There appears to be a concrete grade control structure approximately 2000' upstream of the bridge. At the bridge, the channel grade is controlled by the reinforced concrete sill described previously. The sill is in good condition, with no erosion of the channel at the downstream edge.

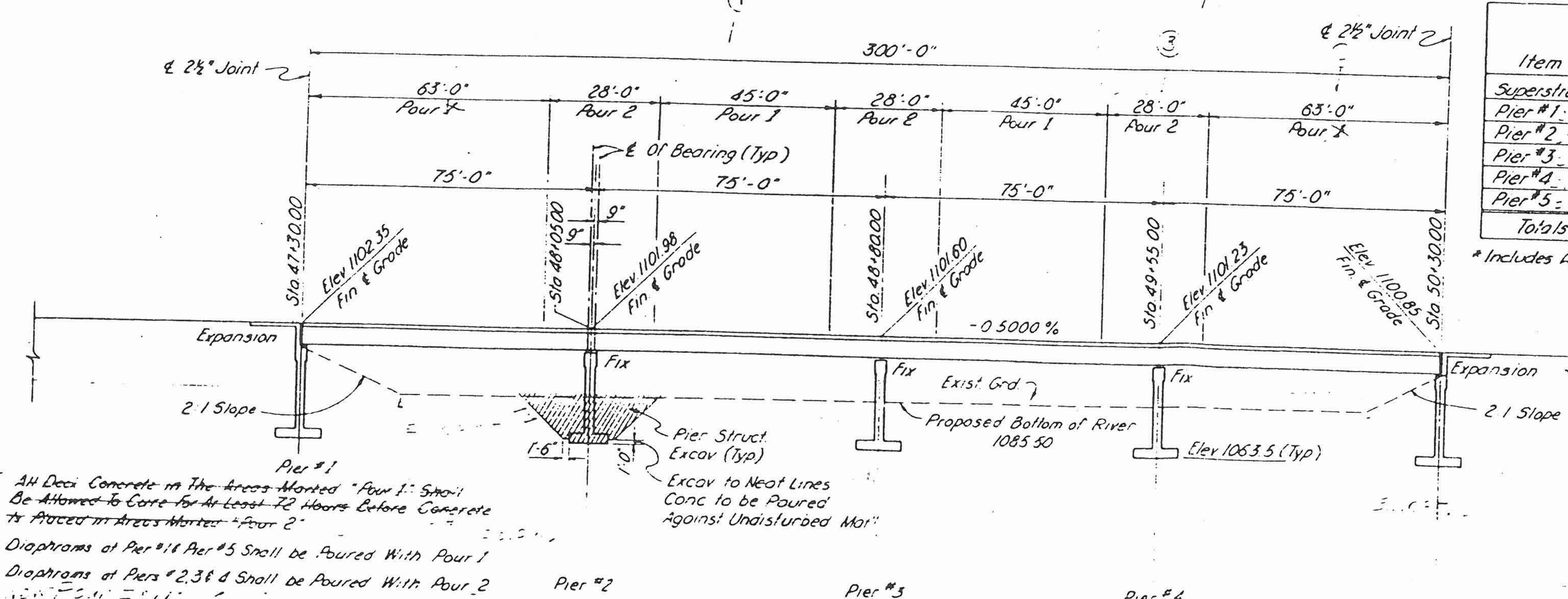
CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in New River at Olive Avenue come from discharges from New River Dam on New River near Jomax Road, approximately 11 miles upstream from the bridge, from tributaries such as Skunk Creek and from regional detention basins and drainage channels. Smaller flows come from off-site developed and undeveloped lands between the upstream dams and the bridge.

REGION	STATE	PROJECT NO	SHEET	OF
4	ARIZ	M 502-7(1)	5	10

EARLE V. MILLER ENGINEERS
DIV OF INTERNATIONAL ENGINEERING CO INC
5-5-75 DATE — FOR THE CONSULTING ENG
DESIGN R.A. Hedrick — DRAWN S Lewis — CHECKED



LOCATION PLAN
Scale 1" = 20'



NOTE ~~All Deck Concrete in The Areas Marked "Pour 1" Shall Be Allowed To Cure For At Least 72 Hours Before Concrete Is Placed in Areas Marked "Pour 2"~~

Diaphrams at Pier #1 & Pier #5 Shall be Poured With Pour 1

Diaphrams at Piers #2,3 & 4 Shall be Poured With Pour 2

CONCRETE SLAB POURING SEQUENCE & SECTION ON CONSTRUCTION ℄
Scale 1" = 20'

GENERAL NOTES

CONSTRUCTION - Standard Specifications, Arizona Highway Department Edition of 1969; Revised to Date

DESIGN - AASHTO Specifications for Highway Bridges Revised to Date

DEAD LOAD - Dead Load Includes Allowance of 35 Pounds Per Square Foot for Future Wearing Surface

LOADING CLASS - HS20-44

COMPOSITE DESIGN: Dead Load By Girders Only

STRESSES
Class D Concrete f'c = 3000 psi, fc = 1200 psi
Class S Concrete f'c = 5000 psi, and 4000 psi
Structural Steel fs = 20,000 psi
Prestressing Steel - ½" 7 Wire Strand (A = .153 Sq In) f's = 270,000 psi P = 41,300 Lbs

All Exposed Corners Shall Be Chamfered ¾" Except As Noted

CONCRETE CLASS DESIGNATIONS

CLASS D - Slabs, Wingwalls, Backwalls, Footings & Diaphragms
CLASS S - S: f'c = 5000 psi Precast Girders
S: f'c = 4000 psi Columns, Column Caps

Reinforcing Steel Shall Conform to ASTM Spec A615 Grade 40 For Bar Sizes #4 And Smaller (fs = 20,000 psi) Grade 60 For Bar Sizes #5 And Larger (fs = 24,000 psi)

Structural Steel Shall Conform to ASTM Spec A36

All Dimensions For Reinforcing Steel Shall Be To Center Of Bars Unless Noted Otherwise

All Reinforcing Steel Shall Have 2" Clear Cover Unless Noted Otherwise

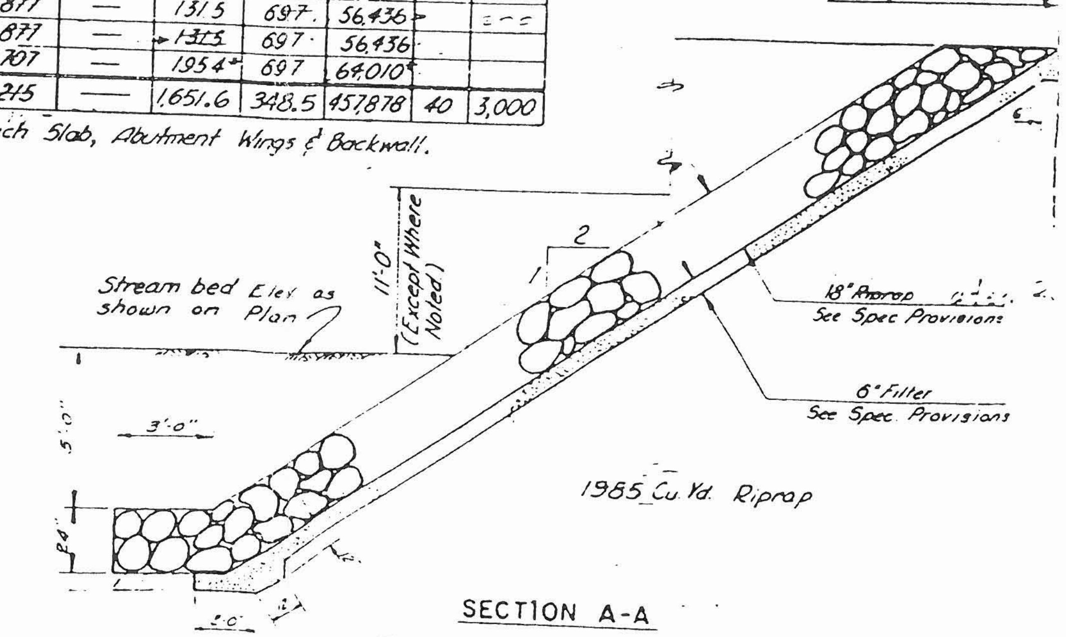
Dimensions Shall Not Be Scaled From Drawings

NOTE:
Provisions Have Been Made For Future Expansion Of The Bridge In Either Direction The Pier Columns Have Been Designed For This Future Load At Piers Nos 1 & 5 The Expansion Joints At These Locations Should Remain If Future Spans Are Added With The Future Spans Being Simple Spans

APPROXIMATE QUANTITIES

Item	Struct. Excav C.Y.	Class A Conc C.Y.	Class D Conc C.Y.	Class S Conc C.Y.	Reinf Steel Lbs	Precast Girders No	Precast Girders LF
Superstruct.		—	866.3	—	160,550	40	3,000
Pier #1	1,877	—	195.4*	69.7	64,010*		
Pier #2	1,877	—	131.5	69.7	56,436		
Pier #3	1,877	—	131.5	69.7	56,436		
Pier #4	1,877	—	131.5	69.7	56,436		
Pier #5	3,707	—	195.4*	69.7	64,010*		
Totals	11,215	—	1,651.6	348.5	457,878	40	3,000

* Includes Approach Slab, Abutment Wings & Backwall.



SECTION A-A

RIPRAP DETAIL
No Scale

OLIVE AVENUE BRIDGE

Figure 3

OLIVE AVENUE (SN 9588)

Water Course	New River
Stream Form	(US) Braided to meandering (DS) Braided to straight
Sinuosity	(US) 1.06
General Channelization	Sides of channel US and DS of bridge lined with soil cement forming trapezoidal section with 0.25:1 sideslopes.
Channel Slope	Stepped at bridge.
Estimated Channel Slope (ft/ft)	0.002439
Channel Contraction/Expansion	Wider US; contracted at bridge; slow expansion DS.
Primary Surface Sediment Type	sand/gravel/cobbles
D50 Size	
Armoring Potential	Moderate (locally)
Channel Vegetation	
Type/Size	Small bush to 3 ft., grasses.
Density/Occurrence	Moderately dense stand immediately DS of concrete sill; otherwise vegetation sparse.
Relative Age	Young
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Soil cement banks.
Vertical	Concrete sill extending approximately 29 ft. US and 25 ft. DS of structure; possible grade control structure approximately 2000 ft. US.
Sediment Deposits & Bars	(US) Low point, alternate and poorly developed middle bars. (DS) Occasional low elongate bars forming along channel flow direction. Sand accumulation in dropped section of concrete sill.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	US east abutment showing some erosion due to flow angle.
Land Use	
Urbanization of Upstream Watershed	Land use commercial, high density residential and some agricultural in DS LOB. General assumption is for increasing urbanization.
Sand & Gravel Extraction	Possible former gravel operations US and DS; elongate "island" in mid-channel forces flow toward banks.
Freeway Construction	No, but general roadway improvements are likely in vicinity.
Dams	New River dam several miles US near Jomax Road.
Drainage Channels	Small diameter storm drain outfalls on both sides of channel near bridge.

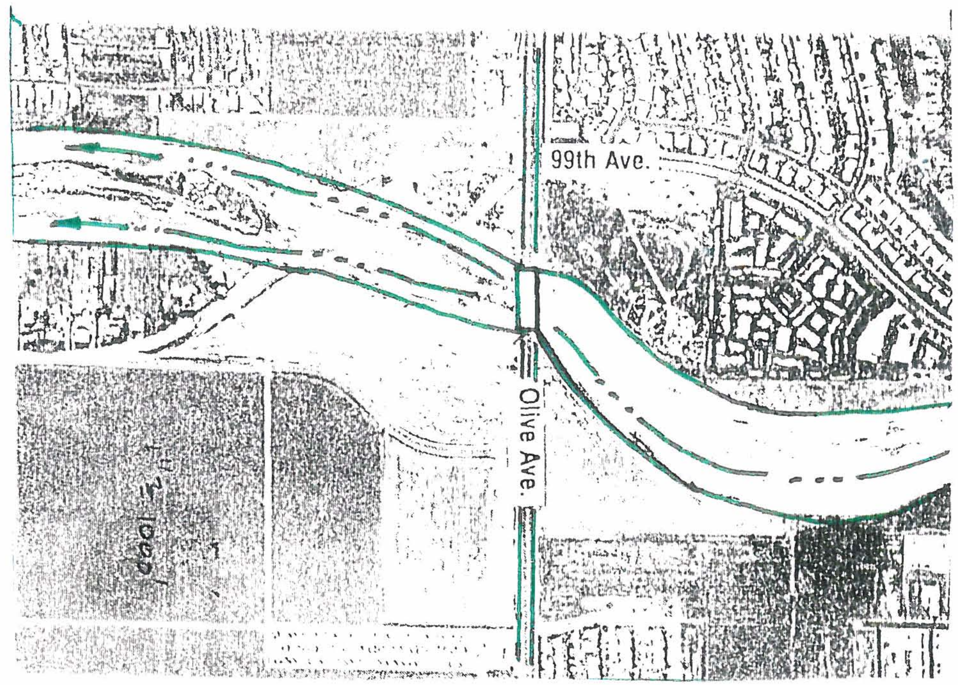


Figure 4

Available plans, flow records, and hydrologic models provided the following information:

1. The design flow and design flood frequency are 19,000 cfs and 14 years, respectively, according to the construction documents.

2. USGS data show that the largest recorded flood between 1961 and the present was 19,800 cfs on December 19, 1967, as measured at the flow gauge on the Glendale Avenue Bridge over New River, approximately 2 miles downstream from the Olive Avenue Bridge.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 41,000 cfs.

4. The Federal Emergency Management Agency (FEMA) flood insurance study of 1993 estimates the 500-year flood (superflood) at 75,000 cfs.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100 year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1084.42	13,500	Q_{10}
1086.20	20,000	Assumed
1090.72	41,000	Q_{100}
1094.69	64,300	Low Chord
1096.30	75,000	Q_{500}

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 64,300 cfs.

SCOUR CALCULATIONS: Scour at the piers and abutments of the Olive Avenue Bridge is considered to be effectively prevented by the reinforced concrete sill across the bottom of the channel and the soil-cement banks.

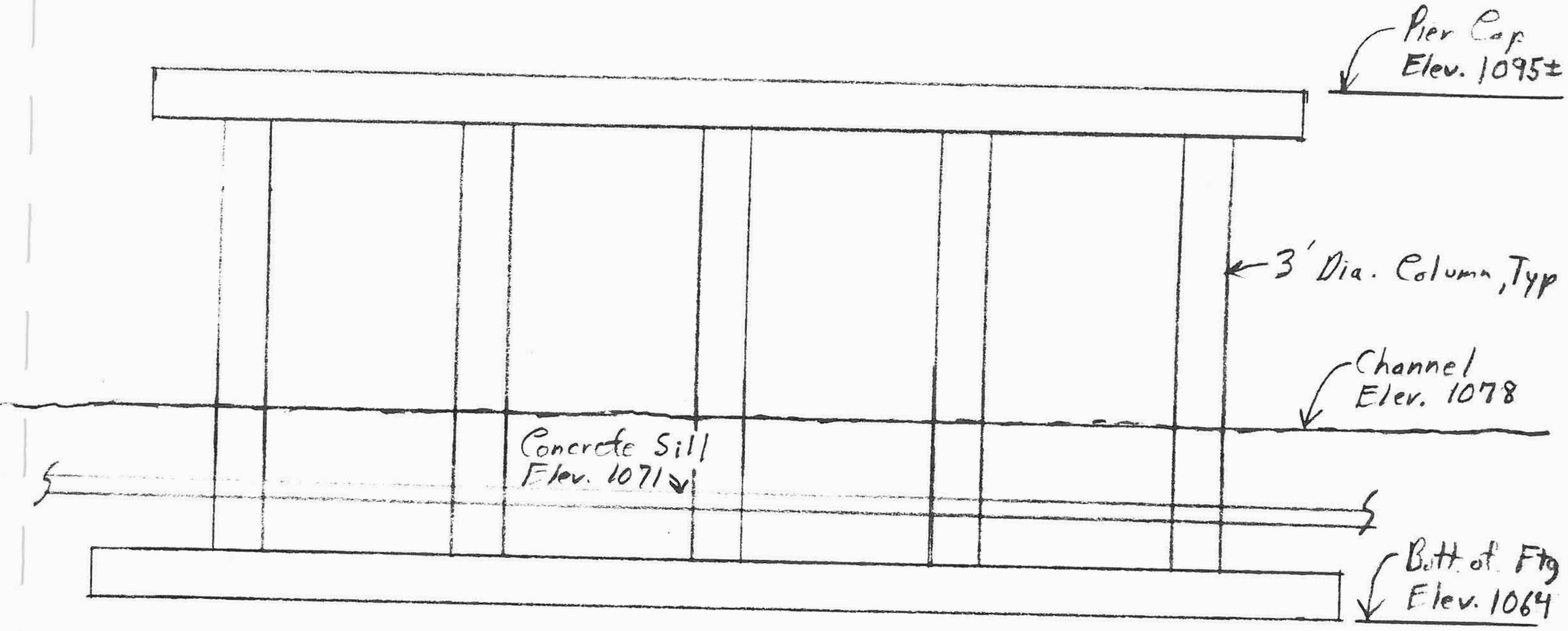
CONCLUSIONS: A grouted riprap sill has been placed across the bottom of the channel, and the banks are protected with soil cement. As long as these protective measures are in place, the bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: The condition of the sill and banks should be monitored frequently and any erosion be promptly repaired.

OLIVE AVENUE OVER NEW RIVER

HYDRAULIC DATA: (Per MM/CSSA)

$Q_{Low\ Chord}$ = 64,300 CFS
H. W. Elev = 1094.69
Total Scour = 0'



PIER ELEVATION

Figure 5

Photo 1: View looking upstream from alongside east abutment. Note wide, sparsely vegetated main channel constrained by levee embankments. Also note that main channel surface sediments are relatively coarse with a substantial percentage of cobble-sized material.

Photo 2: View looking downstream from west embankment near structure. Note continuation of channelization downstream of structure In background, note island remaining after excavating channel along both embankments. Also note growth of channel grasses is more substantial downstream of structure.

Photo 3: View looking across upstream face of bridge from alongside east abutment. Note sloping upstream section of concrete apron which extends approximately 30 feet upstream and downstream of bridge faces. Also note low flow notch toward center of channel.

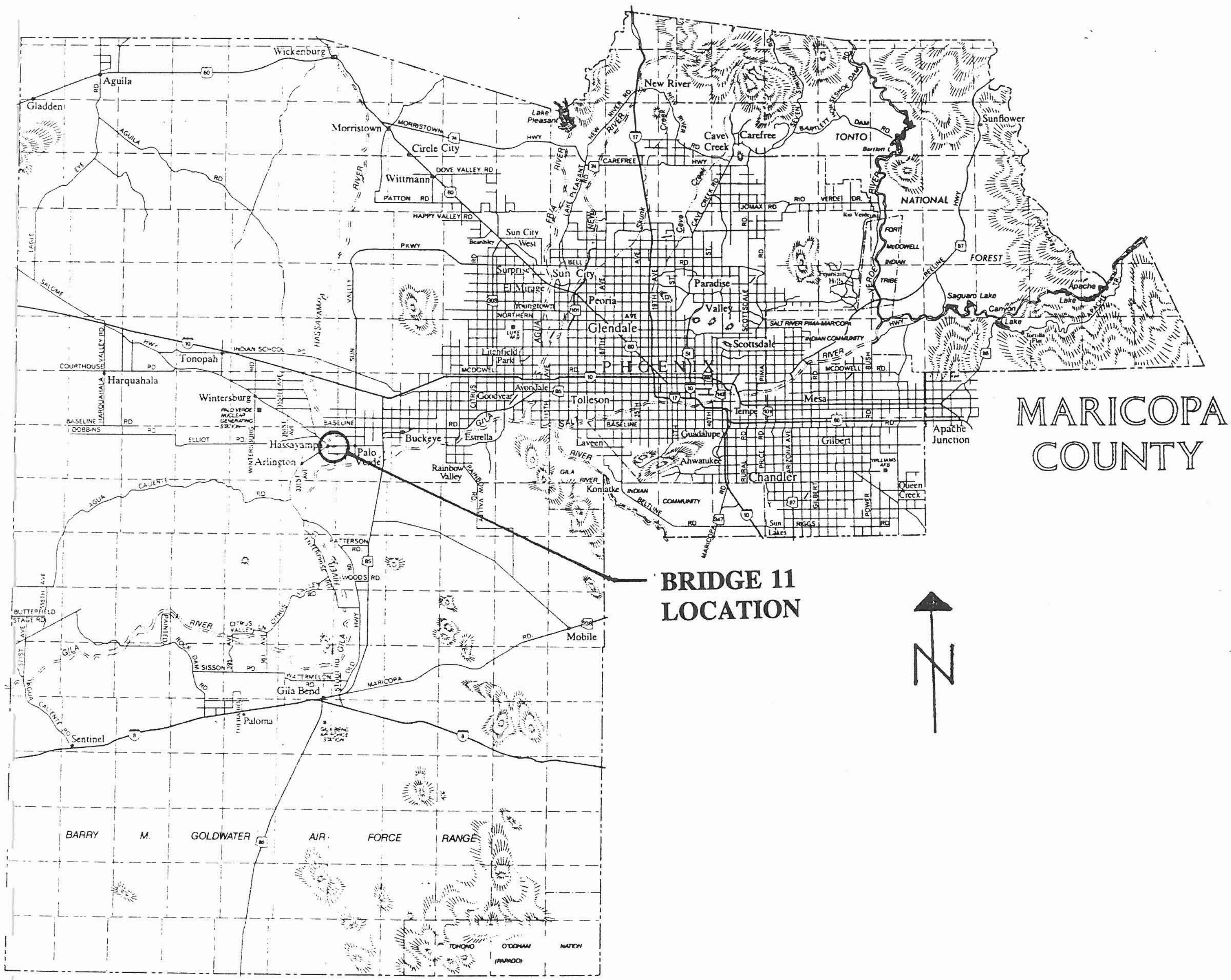
Photo 4: View looking west across upstream face of structure. Note local accumulation of very coarse channel material along upstream edge of concrete apron.

Photo 5: View of upstream side of east abutment. Note erosion of embankment toe occurring near change in channel direction at upstream face of bridge.

Photo 6: View of downstream channel from embankment. Note island in center of main channel. Vertical relief is approximately 8 feet above main channel. Flow forced to east and west around island.

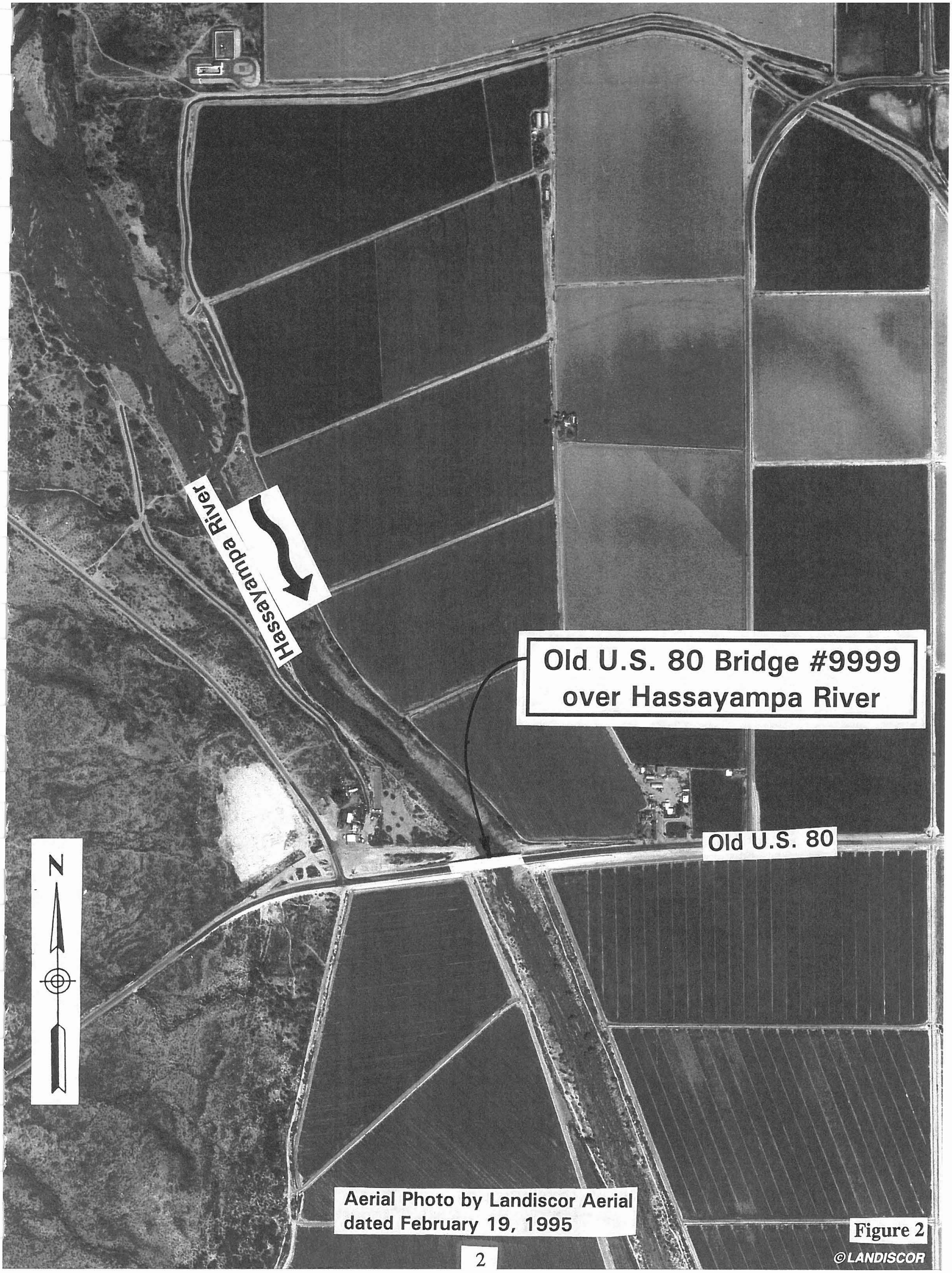
BRIDGE 11

OLD US HIGHWAY 80 BRIDGE OVER HASSAYAMPA RIVER



Location Map

Figure 1


Hassayampa River
Old U.S. 80 Bridge #9999
over Hassayampa River
Old U.S. 80
N
Aerial Photo by Landiscor Aerial
dated February 19, 1995
Figure 2
©LANDISCOR

BRIDGE 11: OLD U.S. 80 HIGHWAY BRIDGE OVER HASSAYAMPA RIVER

(Structure #9999)
Assessment: Scour Critical

LOCATION: The Old US Highway 80 Bridge at the Hassayampa River is located in Section 13 of T1S, R5W, Gila and Salt River Baseline and Meridian, approximately 5 miles west of the Town of Buckeye, Arizona, on Old US Highway 80. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a 4-span, precast concrete I-girder bridge with a total length of 483' (center-to-center of abutment bearings) and a skew of 15 degrees to the left. (See Location Plan, Figure 3.) The bridge was designed in 1987 by Royden Engineering Co. and built in 1993 as MCDOT Project No. 68399. It replaced an existing concrete slab bridge.

The abutments consist of a cap beam and backwall on four 5'-6" diameter drilled shafts founded approximately 50' below grade. Wingwalls at each abutment extend to the end of the roadway approach slab.

The piers consist of a cap beam on four 5'-6" diameter drilled shafts founded approximately 50' below grade.

The flow rate in the Hassayampa River used for design of the bridge was 36,800 cubic feet per second (cfs), corresponding to a flood frequency of 50 years. The bridge was designed such that this flow rate would pass through Spans 2 through 4, with Span 1 reserved as an overflow channel for floods exceeding the 50-year event. According to the design plans, the channel under Span 1 was to have been lined with gunite. However, this lining was deleted by a change order during construction.

EXISTING SCOUR PROTECTION: Scour protection at the bridge consists of a 5' thick layer of 12" diameter dumped riprap around Piers 2 and 3, extending 10' past the end of the drilled shafts in the longitudinal direction and approximately 7' in the transverse direction. No structural riprap has been provided at the abutments or along the channel banks. Bridge inspection reports showed that there was no mention of scour at the bridge. Considering that the bridge was constructed in 1993 and that there have been no significant flows in the Hassayampa River since that date, the absence of scour problems in the reports is not unexpected.

The only scour observed during the site inspection was some incision of side slopes that were shown on the plans as being uniformly graded at 5:1 at Pier 1 and 8:1 at the east abutment. Since the low flow channel partially crosses Pier 2, there may be some local scour at the pier columns; however, this could not be verified during the site inspection due to the turbidity of the water.

STREAM FORM: The general stream form of the Hassayampa River at Old US Highway 80 can be described as straight, although upstream of the bridge the river exhibits some of the characteristics of a braided stream, with low to moderate point, alternate and middle bars. (See

Figure 4.) Downstream of the bridge, the river is generally straight, with slight point and alternating bars. This slight difference in stream characteristics may be due to different bank characteristics on each side of the bridge. Downstream of the bridge, the river banks are composed of earthen levees, giving the appearance of a channelized river with a trapezoidal section. Upstream of the bridge, the east bank has an earthen levee extending approximately one mile upstream to the wasteway of the Buckeye Canal while the west bank on the upstream side of the bridge appears to be in a natural or near-natural state, with steeply cut banks in some areas.

LAND USE: The predominant land use in the vicinity of the bridge is agricultural. Residential or commercial development consists small settlements such as Palo Verde, approximately 3 miles east of the bridge site, or isolated farm houses, stores and gas stations. It is not expected that the level of development upstream of the bridge will increase such that flows in the river will be significantly affected.

SURFACE SOILS: The surface soils of the river bed are primarily silts and sands with an estimated median diameter (D_{50}) of 0.30 mm. Although there are some cobbles in the surface soils, the potential for armoring during high flows is considered to be low.

SLOPE: The slope of the Hassayampa River in the vicinity of the Old US 80 Bridge was estimated from U.S. Geological Survey (USGS) topographic maps as 0.0036 ft/ft, or approximately 19' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the Hassayampa River includes trees such as desert willow, cottonwood, palo verde, mesquite and ironwood; bushes and shrubs such as desert broom, creosote, ephedra, brittle bush and sage; and grasses such as Johnson grass and wild oats. The larger trees occur sparsely on sand bars and on the banks, brush and shrubs on river banks at low to moderate density, and grasses on the banks at low to moderate density.

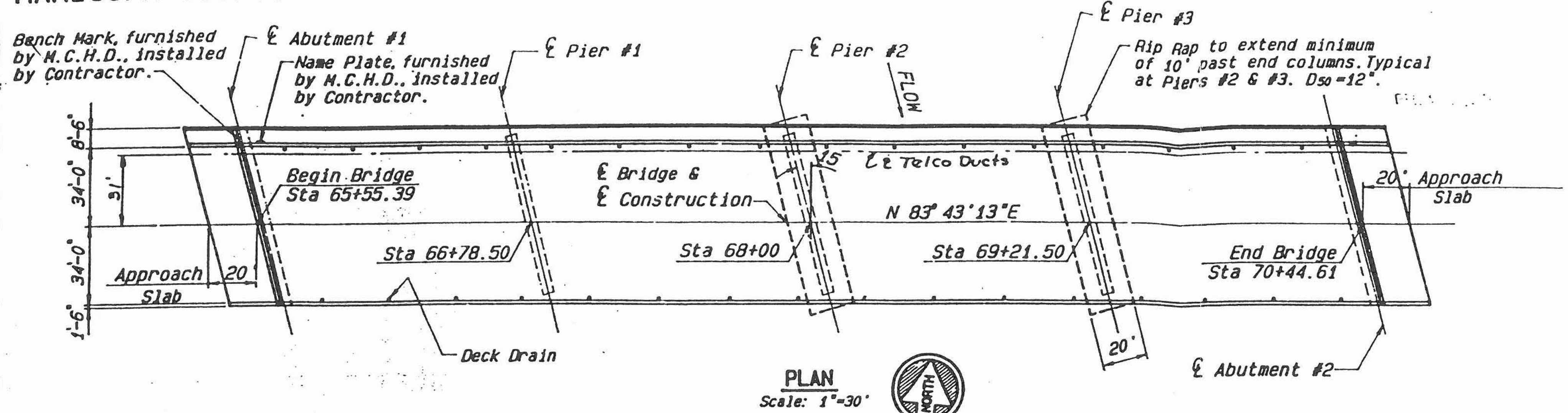
STREAM STABILITY: Lateral stability of the stream is maintained by non-structural (unlined) earthen levees upstream and downstream of the bridge, as described previously. Flows in excess of the design flow (Q_{50} = 36,800 cfs) would overtop the natural banks on the west side of the river, as anticipated by the overflow area provided between the west abutment and Pier 1. Lateral migration of the Hassayampa River that could potentially affect the bridge is considered to be minor.

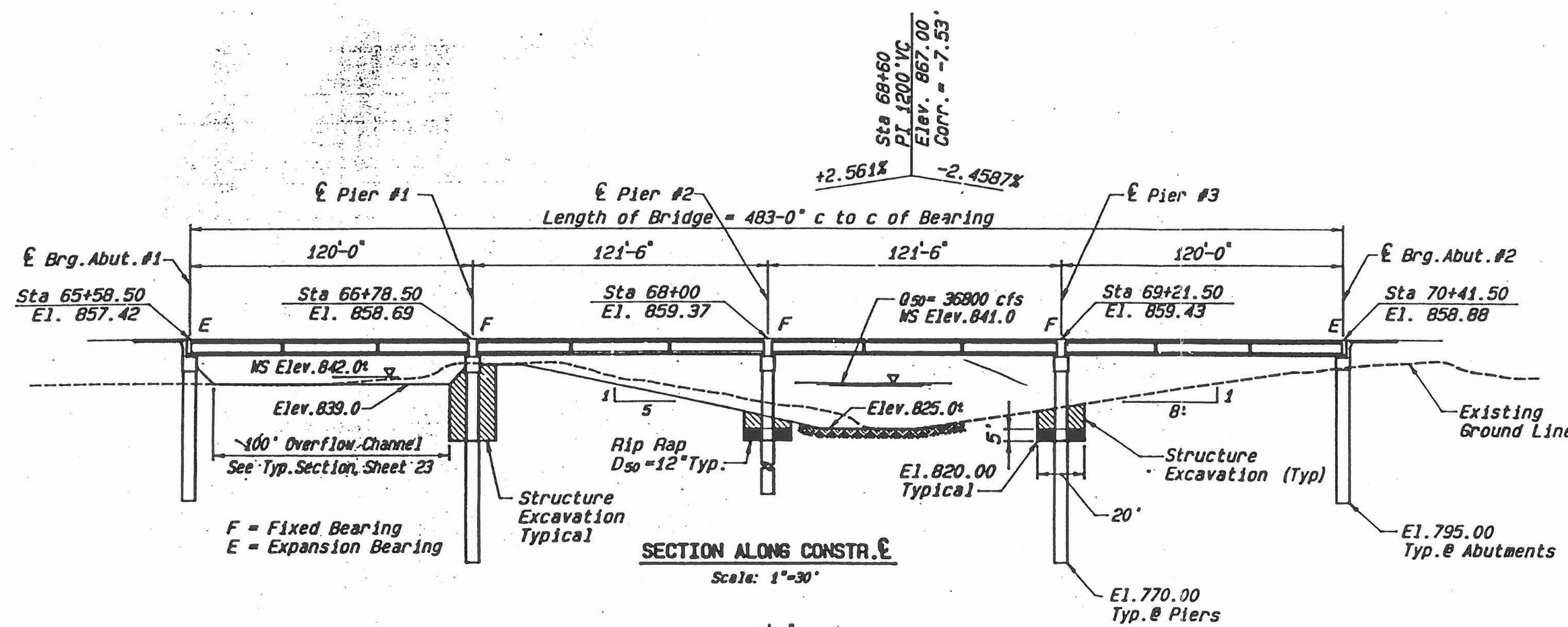
There are no upstream or downstream controls on the vertical stability of the river at the bridge site. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade, although there is no measurable degradation observed at the site.

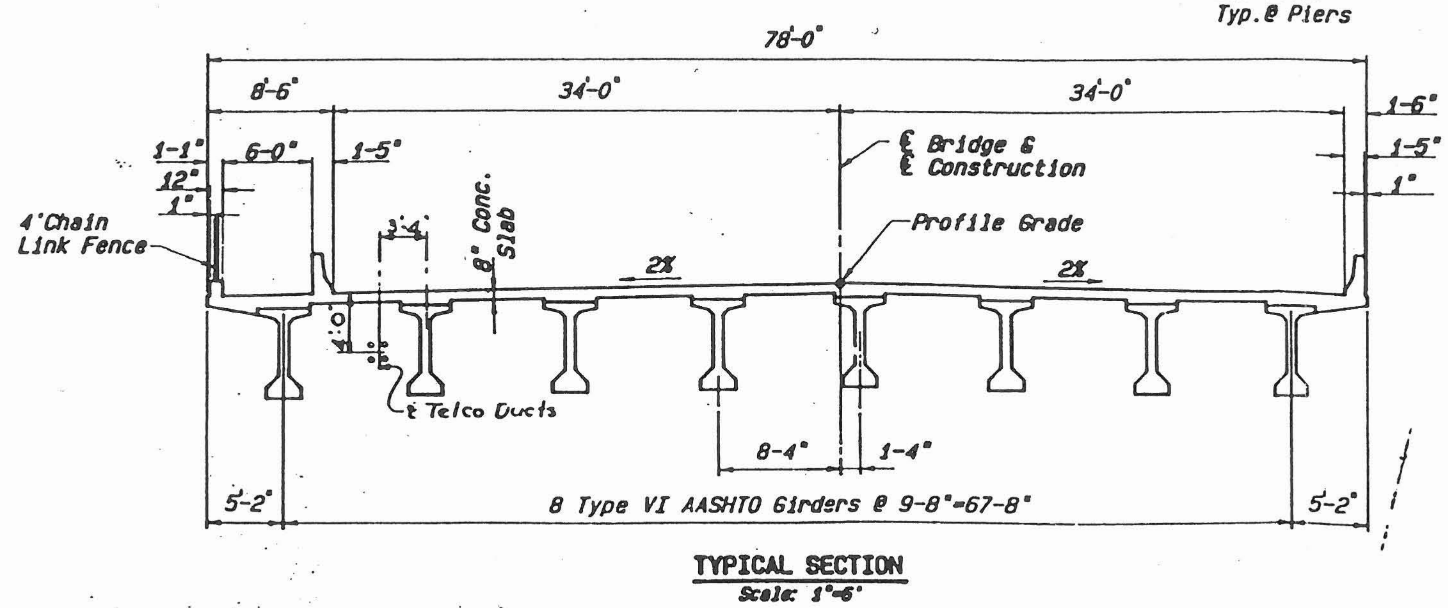
CURRENT HYDROLOGY AND FLOW ANALYSIS: Flow in the Hassayampa River comes from uncontrolled runoff from the upstream watershed. There are no dams on the river to store peak flows and reduce their magnitude.

The Old US 80 Bridge at the Hassayampa River is approximately one mile downstream of the wasteway of the Buckeye Canal. Water wasted from the canal flows down the river past the

OLD U.S.80 – SALOME HWY. TO 309th AVE.
MARICOPA COUNTY







GENERAL NOTES

1. Construction--Uniform Standard Specification for Public Works Construction by the Maricopa Association of Governments, 1979 Edition, as amended by Maricopa County Highway Department and revisions to date.
2. Design--"Standard Specification for Highway Bridges" by AASHTO, 1983 Edition, revised to date.
3. Loading Class--HS 20-44. Dead load includes allowance for 25 psf future wearing surface.
4. Composite Design--Dead load carried as simple span; live load carried through continuous spans. Simple beam design for ultimate load.
5. All concrete shall be M.A.G. Class "A" except for prestressed girders and bridge deck. Bridge deck shall be M.A.G. Class "AA". Class "A", f'c = 3000 psi; Class "AA", f'c = 4000 psi.
6. Reinforced concrete design as per Strength Design Method.
7. Reinforcing steel shall conform to ASTM spec.A615, Grade 60, fy = 60,000 psi.
8. All dimensions for reinforcing steel shall be center to center of bars unless noted otherwise. Bending diagram dimensions shall be out to out.
9. All reinforcing bars shall have 2" clear cover unless noted otherwise.
10. Chamfer all exposed edges of concrete 3/4" unless noted otherwise.
11. All structural steel shall conform to ASTM A-36. All exposed structural steel shall have one prime coat of red lead paint and two coats of aluminum paint. Paint shall conform to MAG specification 790, unless noted otherwise.
12. All welding for structural steel shall be in accordance with the American Welding Society, (AWS) Structural Welding Code AWS D1-1-80.
13. Drilled Caisson Loads:
    Pier DL 858 Kips
    TL 1237 Kips
    Abutment DL 482 Kips
    TL 678 Kips

APPROXIMATE QUANTITIES

ITEM	TYPE VI P.S. CONC. GIRDERS	M.A.G. CLASS "AA" CONC.	M.A.G. CLASS "A" CONC.	REINF. STEEL	DRILLED CAISSON 5-6" DIAM.	CHAIN LINK FENCE
	Each	C.Y.	C.Y.	Lbs.	Lin. Ft.	Lin. Ft.
Superstructure	32	1,352		185,726		530
Abutment No.1			* 337	* 43,089	197	
Piers No.1, 2 & 3			* 1,097	* 173,986	600	
Abutment No.2			* 341	* 43,636	204	
Approach Slabs			111	17,870		
Barriers & Curb			116	23,106		
TOTALS	32	1,352	* 2,002	* 487,413	1,001	530

* Includes Drilled Caissons

263-1100

GENERAL PLAN & ELEVATION

COUNTY	STATE	PROJECT NO.	SHEET	OF	AS BUILT
MARICOPA	ARIZ.	68399	24	40	

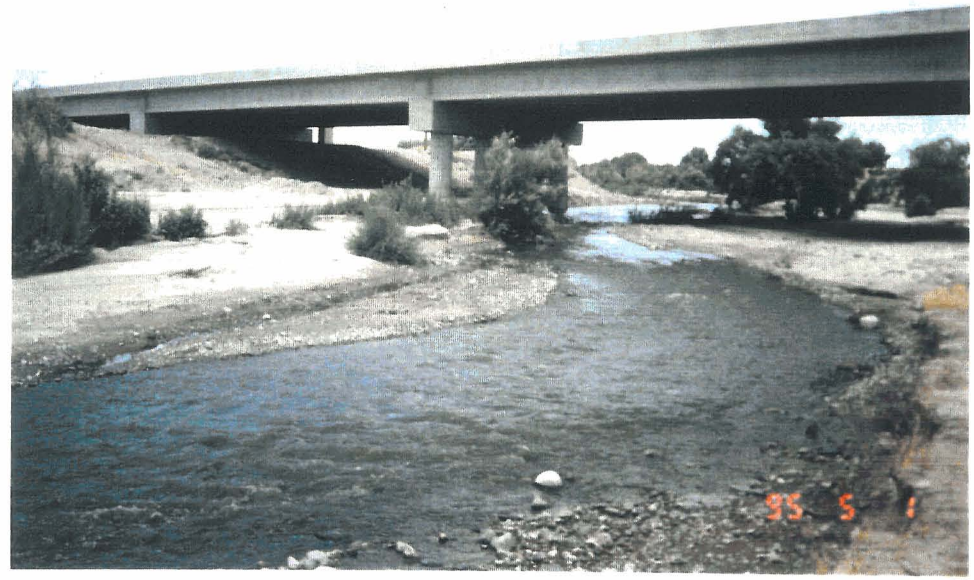
ROYDEN ENGINEERING CO.
3035 W. INDIAN SCHOOL RD.
PHOENIX, ARIZONA 85017
TEL. 602 279-3541

DESIGN	ASD
DRAWN	RAB
CHECK	ASD
DATE	1/14/87
JOB NO.	85039

Figure 3

OLD U.S. HIGHWAY 80 (SN 9999)

Water Course	Hassayampa
Stream Form	Straight (US) Braided Characteristics (DS) Straight
Sinuosity	Not applicable
General Channelization	Non-structural (agricultural), trapezoidal, earthen levees.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.002897
Channel Contraction/Expansion	Main channel slightly wider US
Primary Surface Sediment Type	silt/sand
D50 Size	0.30 MM (estimate)
Armoring Potential	Low
Channel Vegetation	
Type/Size	Desert Willow to 15 ft., Cottonwood to 15 ft., Palo Verde, Mesquite to 9 ft., Ironwood; Broom to 6 ft., Creosote, Ephedra, Brittle Bush, Sage; dry grasses.
Density/Occurrence	Larger trees occur sparsely on bars and on banks; smaller brush occurs mainly on bank sideslopes with low to moderate density; dry grasses occur on bank sideslopes with low to moderate density.
Relative Age	
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Non-structural, earthen levees.
Vertical	No
Sediment Deposits & Bars	US: Low to moderate point, alternate and middle bars forming in braided condition. DS: Slight point and alternating bars forming in a generally straight condition.
Evidence of Degradation	Steep cut banks may indicate potential for incision, otherwise none.
Evidence of Aggradation	No
Evidence of Scour	
Pier	Primary low flow crosses pier 2 indicating potential for scour, none otherwise.
Abutment	None
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agriculturalor very low density residential.
Sand & Gravel Extraction	No
Freeway Construction	None locally.
Dams	None locally.
Drainage Channels	Side channel from west may contribute small to moderate flow; possible irrigation inflows US/DS.



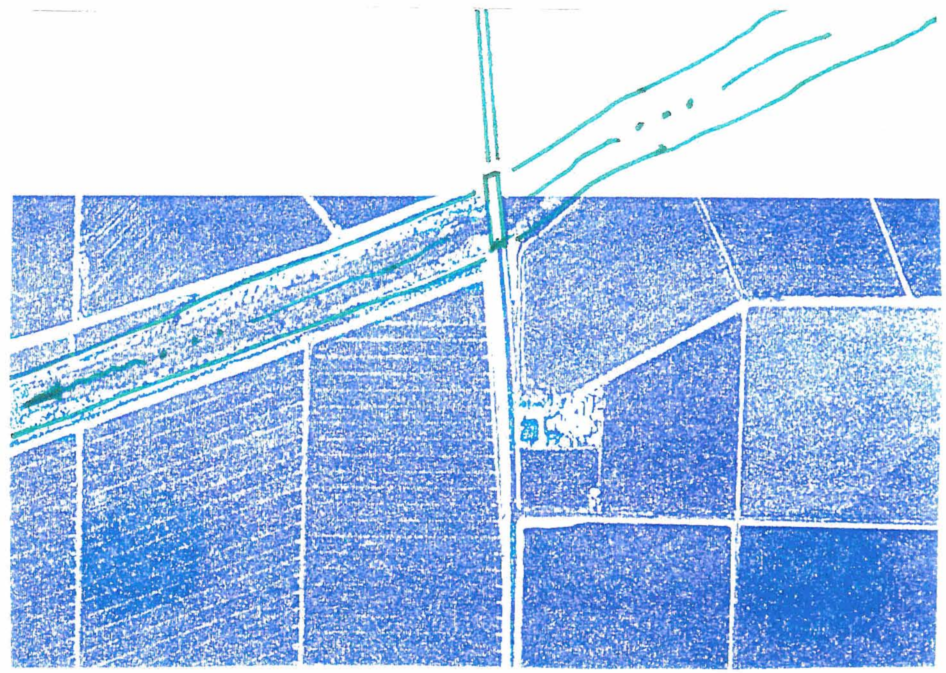


Figure 4

bridge on an almost continuous basis, except during canal dry-up periods. At the time of the site visit (May 1, 1995), water flowing against a staff gauge on the upstream column of Pier 2 measured 6.0' (bottom elevation of the stream unknown). There were signs that flows reached levels several feet higher, as indicated by splash marks on the pier columns above the 12.0' level of the staff gauge.

Available plans, flow records, and hydrologic models provided the following information:

1. The design flow and design flood frequency are 36,800 cfs and 50 years, respectively, according to the construction documents.

2. USGS data show that the largest recorded flood between 1961 and the present was 39,000 cfs on September 9, 1970, as measured at the flow gauge near Arlington, Arizona.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 73,500 cfs.

4. There was no published information providing an estimate of the 500-year flood (superflood). USGS regression equations for ungaged watersheds were used to estimate a 500-year flow of 125,700 cfs.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
842.25	36,800	Q_{50}
842.88	40,000	-
847.92	73,500	Q_{100}
849.70	89,100	Low Chord
853.19	125,700	Q_{500}

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 89,100 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the critical flood (Q_{LC}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Old US Highway 80 Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and a summary of drilled shaft embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{LC} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

	Q = 73,500 cfs (Q_{100})	Q = 89,100 cfs (Q_{LC})
1. Scour at Piers		
Contraction Scour, ft	0.0	0.0
Local Scour, ft	33.6	34.6
General Scour, ft	4.0	4.0
Total Scour, ft	**37.6**	**38.6**

2. Scour at Abutments - Negligible (not tabulated)

Table 3. Summary of Drilled Shaft Embedment

	Q = 73,500 cfs (Q_{100})	Q = 89,100 cfs (Q_{LC})
1. Embedment at Piers		
Channel Elevation	825.8	825.8
Total Scour, ft	37.6	38.6
Bottom of Scour Hole Elev.	789.5	787.2
Drilled Shaft Tip Elev.	770.0	770.0
Embedment Remaining, ft	**19.5**	**17.2**

~~Enough depth left in the Embedment @ Piers~~
~~why is the~~ bridge scour critical?

STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{LC} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads and stream flow forces. The structural capacity of the concrete columns and drilled shafts, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was not warranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, the Old US 80 Bridge at the Hassayampa River does not have sufficient structural capacity to resist the loads resulting from flows up to and including 89,100 cfs, i.e., the low chord flow rate. The bridge is scour *critical*.

why?

DEFICIENCIES AND COUNTERMEASURES:

Scour-related deficiencies include the following:

a. Insufficient embedment of the drilled shafts at the piers to resist the low chord flow.

Countermeasures to remedy scour-related deficiencies include the following:

a. Install scour monitoring devices and close the bridge to traffic if scour reaches a predetermined critical depth;

b. Construct a continuous concrete or grouted riprap sill across the width of the channel with the sill keyed deeply into the channel bed at the upstream and downstream ends;

c. Encase the piers in a reinforced concrete beam supported on drilled shaft foundations (underpinning);

d. Remove the bridge and construct a new bridge on deeper foundations.

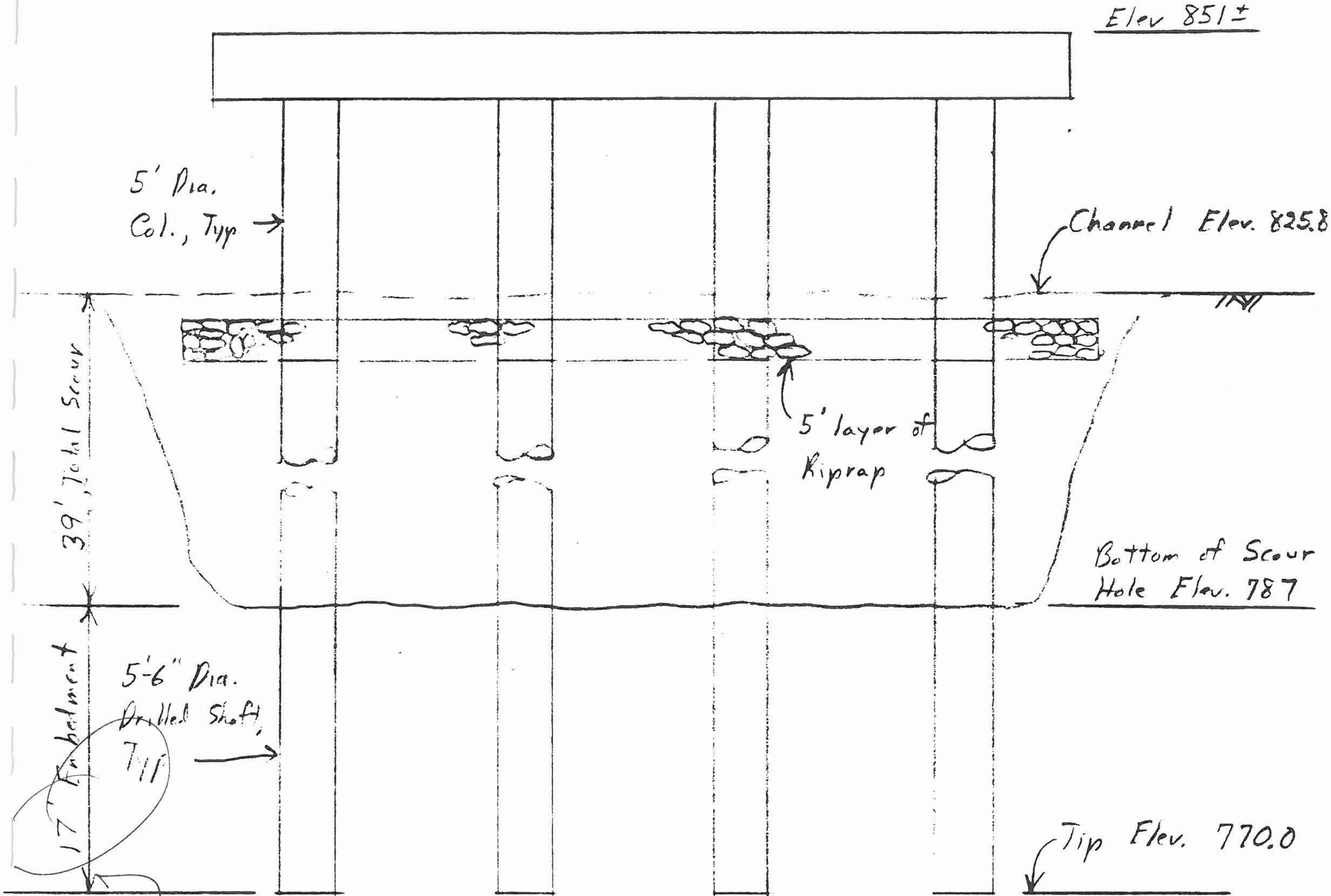
~~why.~~ OK!

structurally not safe

OLD US HIGHWAY BRIDGE AT HASSAYAMPA RIVER

Hydraulic Data (Per MM/CSSA)

$Q_{Low\ Chord}$ = 89,100 cfs
H.W. Elev. = 849.7
Total Scour = 39'



PIER ELEVATION

should have adequate depth left in embedment!!!

Figure 5

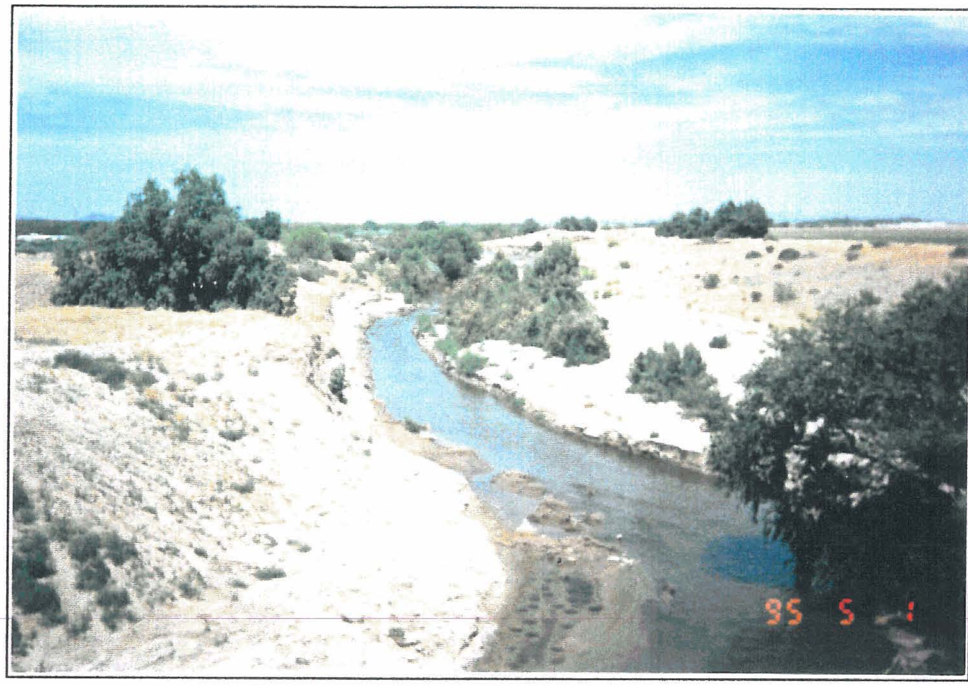


Photo 1: View looking approximately northwest at upstream main channel from bridge deck. Note clean sandy primary flow channels divided by sandy bars exhibiting relatively well established vegetation. Also note that the main channel approaches the structure at an angle somewhat west of north.

Photo 2: View looking toward upstream face of bridge from east levee embankment. Note trend of levee embankments and main channel is northwest upstream relative to structure and approximately south downstream of structure. Also note undercutting of west bank of upstream main channel by first order channel.

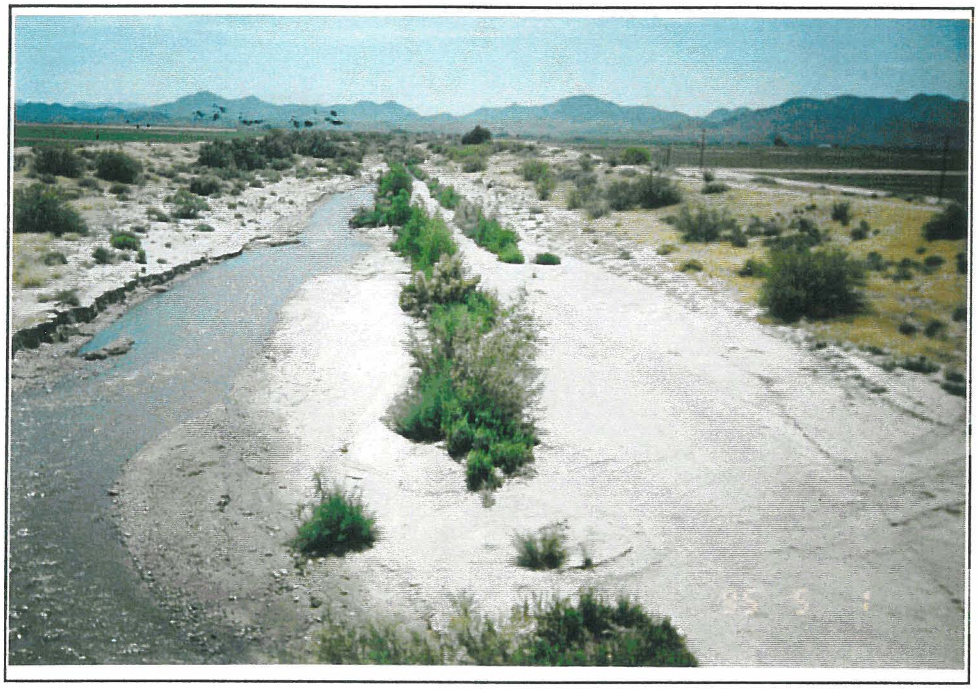


Photo 3: View looking downstream from bridge deck. Note approximately trapezoidal section formed by the levee embankments. Also note relatively high percentage of medium to coarse sand comprising the surface sediments in the main channel and that low flow stream is undercutting sandy east bank. Vegetation consists of sparse to moderately dense dry grasses on embankments with occasional shrubs and larger trees on bars and banks. Note that overbanks consist primarily of cultivated fields.

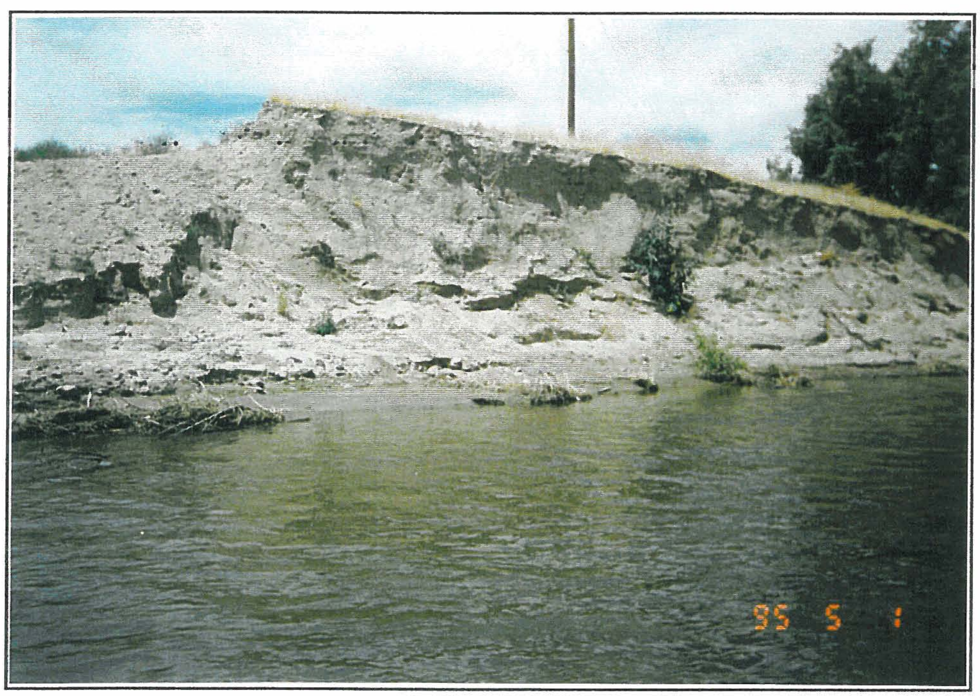
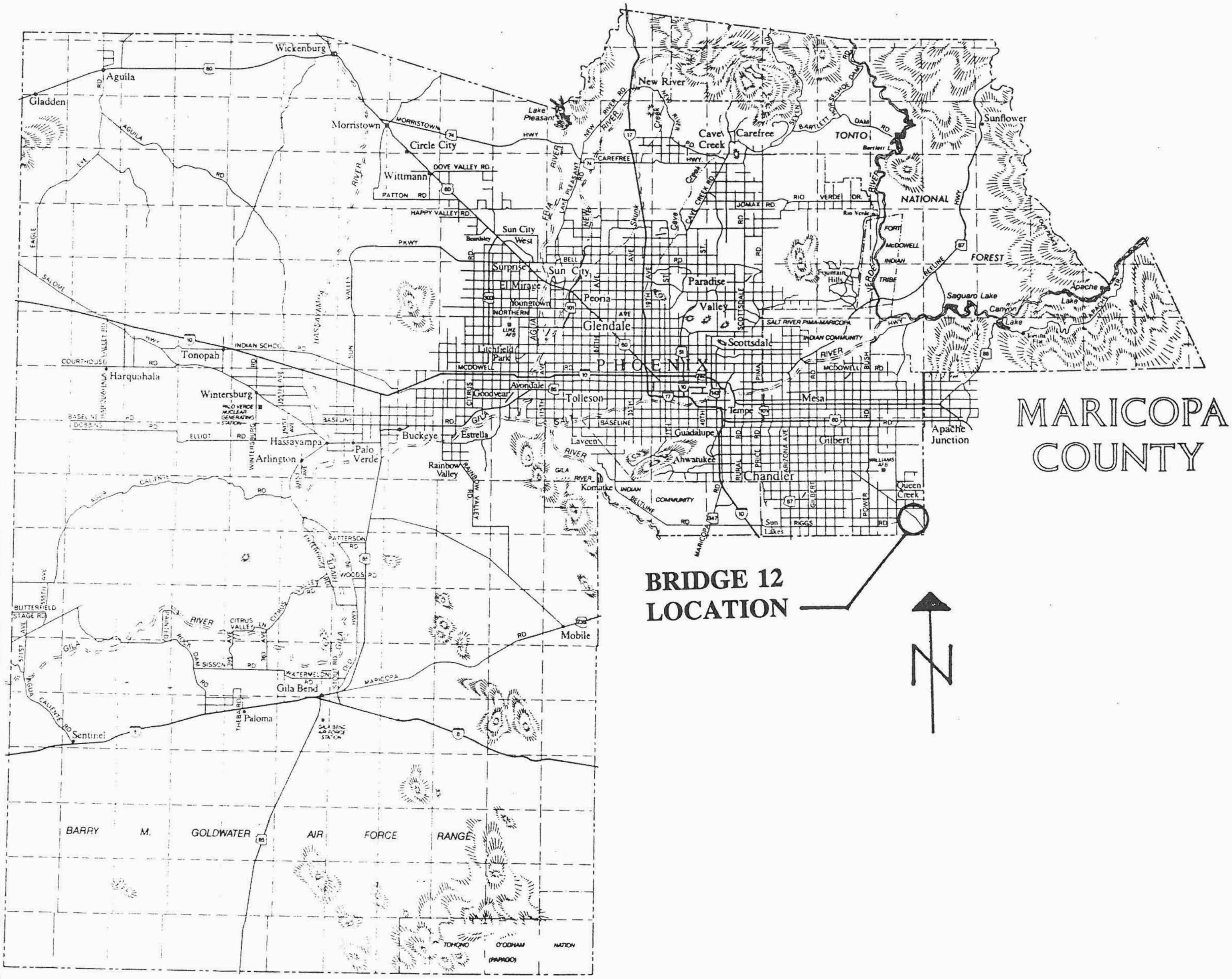


Photo 4: View of west bank of main channel upstream of structure. Note steep cut bank alongside low flow channel slightly upstream structure.

Photo 5: View looking upstream of structure alongside low flow channel. Note significant proportion of medium to coarse sand comprising channel surface sediments and its potential erodability under high flow conditions.

BRIDGE 12

RITTENHOUSE ROAD BRIDGE OVER QUEEN CREEK



Location Map

Figure 1

BRIDGE 12: RITTENHOUSE ROAD OVER QUEEN CREEK (Structure #8038)

Assessment: Scour Critical

LOCATION: The Rittenhouse Road Bridge at Queen Creek is located in Section 25, T2S, R7E, Gila and Salt River Baseline and Meridian, on Rittenhouse Road near the Town of Queen Creek. The bridge is 140' downstream of the Southern Pacific Railroad (SPRR) bridge over Queen Creek. See Location Map, Figure 1.

STRUCTURE: The structure is a three-span, precast concrete I-girder bridge with a total length of 179.26' center-to-center of abutment bearings and a skew of 15 degrees to the right. (See Location Plan, Figure 2.) The flow rate used for design is unknown (not noted on the plans). The bridge was built in 1969 as Maricopa County Highway Department (MCHD) Project No. 812-30.

The abutments consist of a reinforced concrete cap beam supported on twelve 16" diameter fluted steel pipe piles (6 vertical piles and 6 battered piles). According to the plans, average tip elevations of the piles are Elevation 1414 at the north abutment and Elevation 1409 at the south abutment, approximately 25' and 30', respectively, below the bottom of the stream bed as shown on the plans. Wingwalls extend approximately 10' beyond the ends of the abutment wall.

The piers consist of a reinforced concrete cap beam supported by ten 16" diameter fluted steel pipe piles driven vertically to a distance below the stream bed of approximately 33' at Pier 1 and 31' at Pier 2.

EXISTING SCOUR PROTECTION: Scour protection at the bridge consists of sacked concrete along the stream banks extending from the downstream side of the abutments to the upstream face of the SPRR bridge. The sacked concrete is in fairly good condition with some undermining at the toe of the abutments. A review of bridge inspection reports showed that the undercutting of the sacked concrete scour protection was noted on consecutive reports dating from 1980. Some minor undermining of the sacked concrete at the toe of the abutments and of the grouted riprap bank protection was observed during the site inspection.

According to the plans, a 30" diameter corrugated metal pipe (CMP) culvert protruded through the left bank (looking downstream) of the sacked concrete bank protection. The CMP culvert conveyed water flowing to the northwest between the railroad embankment and the Rittenhouse Road embankment. Apparently the quantity of stormwater exceeded the capacity of the culvert because a section of the sacked concrete around the culvert has been washed out. The culvert itself no longer conveys water.

Scour protection on the right bank (looking downstream) of Queen Creek between the two bridges is a combination of sacked concrete, grouted riprap, and dumped riprap. Since the construction plans specified only sacked concrete, it may be assumed that the other materials have replaced sections of the original scour protection that have washed out.

Bank stabilization in the form of rubber tires stacked on posts have been constructed on the right bank (looking downstream) of Queen Creek upstream of the SPRR bridge. The tires and posts form a series of short walls that deflect flows at a bend in the stream. There is no information regarding when the deflectors were installed, but they appear to have prevented lateral migration of the bend.

No structural scour protection appears to have been placed around the piers. There was no evidence of scour around the piers observed during the site inspection.

STREAM FORM: Queen Creek has a transitional stream form between braided and meandering. (See Figure 3.) Upstream of the bridge, the stream exhibits braided characteristics while downstream its form is more that of a slow meander. Queen Creek forms low point, alternate and middle bars.

LAND USE: Land use in the vicinity of the bridge is primarily agricultural or low density residential. Urbanization will increase, but largely downstream of the bridge towards the Town of Queen Creek and along Rittenhouse Road to the northwest. Urbanization is not expected to have an impact on stream flows at the bridge.

Sand and gravel was extracted from the overbanks of Queen Creek upstream of the SPRR bridge in the past. These gravel extraction operations have apparently been closed for some time, judging by the size of vegetation and by general weathering of the mining sites.

SURFACE SOILS: Surface soils consist primarily of sand and gravel with occasional cobbles. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of Queen Creek in the vicinity of the Rittenhouse Road Bridge, estimated using U.S. Geological Survey (USGS) topographic maps, is 0.0034 ft/ft, or approximately 18' per mile.

VEGETATION: Vegetation includes trees such as mesquite, palo verde, ironwood and an occasional desert willow; desert broom is the dominant brush, with some creosote, ephedra and chuparosa. Dry grasses are also present. The larger vegetation occurs primarily on the south bank (left side looking downstream) and on sand bars, with a relatively dense stand of trees and shrubs located in mid-channel upstream of Pier 1. Downstream of the bridge the channel is relatively clear. Dry grasses and smaller varieties of shrubs are found on the stream banks and on the bars.

STREAM STABILITY: Overall lateral stability of the stream in the vicinity of the bridge is maintained by the sacked concrete bank protection and by the rubber tire deflector dikes upstream of the SPRR bridge, although the reliability of these latter structures may be somewhat doubtful. Downstream of the bridge, levees on the banks of Queen Creek provide lateral stability.

There are no grade control structures either upstream or downstream of the Rittenhouse Road Bridge. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade, although there is no measurable degradation observed at the site.

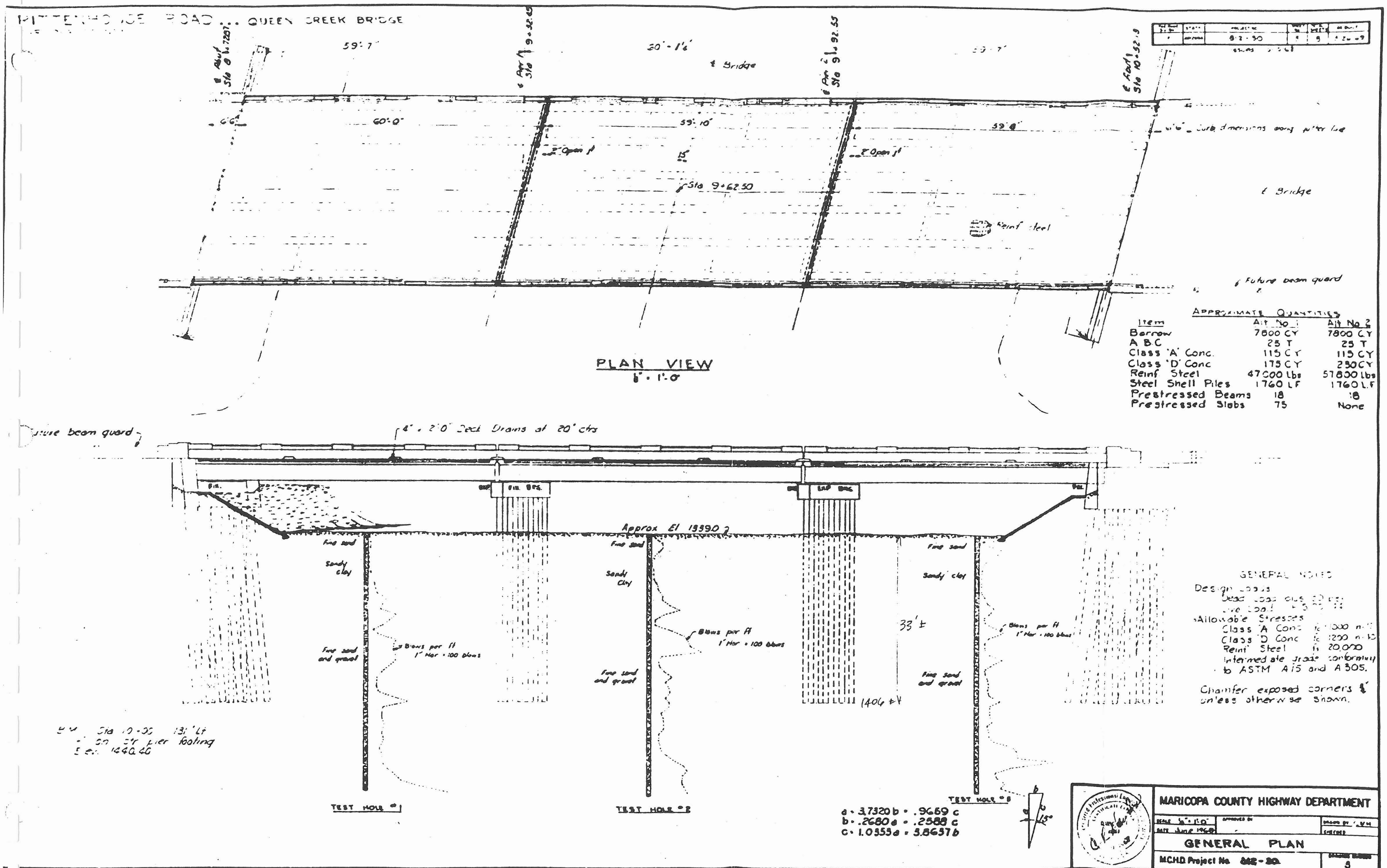

QUEEN CREEK BRIDGE
59'-7"
20'-1½"
℄ Bridge
60'-0"
59'-10"
59'-8"
2" Open jt
Sta 9+62.50
Reinf steel
Future beam guard
PLAN VIEW
APPROXIMATE QUANTITIES
Item | Alt No 1 | Alt No 2
Borrow | 7800 CY | 7800 CY
A B C | 25 T | 25 T
Class 'A' Conc. | 115 CY | 115 CY
Class 'D' Conc | 175 CY | 230 CY
Reinf Steel | 47000 lbs | 57800 lbs
Steel Shell Piles | 1760 LF | 1760 LF
Prestressed Beams | 18 | 18
Prestressed Slabs | 75 | None
Future beam guard
4" x 2'0" Deck Drains at 20' ctrs
Approx El 1339.0
Fine sand
Sandy clay
Fine sand and gravel
Blows per ft
1" Hor = 100 blows
33'±
1406±
GENERAL NOTES
Design Loads
Allowable Stresses
Class 'A' Conc
Class 'D' Conc
Reinf Steel
Intermediate grade conforming to ASTM A15 and A305.
Chamfer exposed corners ¾" unless otherwise shown.
TEST HOLE #1
TEST HOLE #2
TEST HOLE #3
a = 3.7320 b = .9659 c
b = .2680 a = .2588 c
c = 1.0355 a = 3.8637 b
MARICOPA COUNTY HIGHWAY DEPARTMENT
GENERAL PLAN
M.C.H.D. Project No.

Figure 2

RITTENHOUSE ROAD (SN 8038)

Water Course	Queen Creek
Stream Form	Transitional (US) Braided (DS) Slow Meander
Sinuosity	(DS) 1.06
General Channelization	Minor (see lateral controls)
Channel Slope	Steeper US
Estimated Channel Slope (ft/ft)	0.004674
Channel Contraction/Expansion	Main channel wider US
Primary Surface Sediment Type	sand/gravel
D50 Size	
Armoring Potential	Low
Channel Vegetation	
Type/Size	Trees include Mesquite, Palo Verde, Ironwood to 20 ft., occasional Desert Willow; Brush dominated by Desert Broom to 8 ft., also including Creosote, Ephedra, Chuparosa; dry grasses.
Density/Occurrence	Larger vegetation occurs primarily on bars with a relatively dense stand located mid-channel US of structure. Main channel is relatively clear. Dry grasses and smaller varieties occur on banks and occasionally on bars.
Relative Age	Mature
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Minor: Rubber tire deflector dikes on US north bank; minor bank protection.
Vertical	None
Sediment Deposits & Bars	US: Low point, alternate and middle bars forming in braided condition. DS: Low point and alternating bars forming in a slightly meandering condition.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	Undercutting of sacked concrete and trowelled concrete bank protection.
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agricultural or low density residential.
Sand & Gravel Extraction	Some evidence in overbanks US in past; no current extraction.
Freeway Construction	No
Dams	No
Drainage Channels	Rail road embankment drainage does provide inflows near structure.

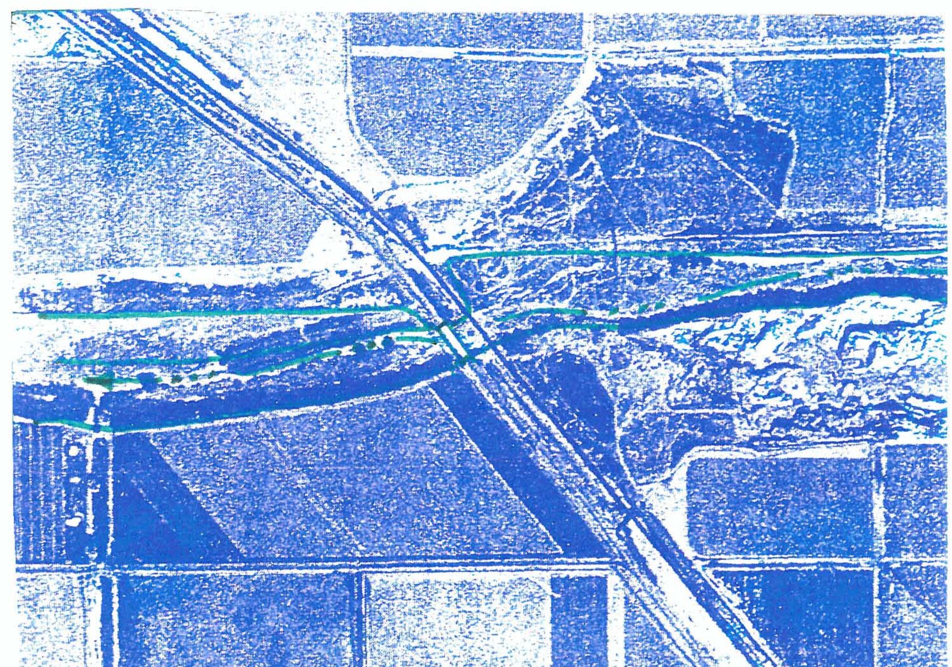


Figure 3

CURRENT HYDROLOGY AND FLOW ANALYSIS: Flow in Queen Creek comes from releases from Whitlow Ranch Reservoir, a flood control reservoir on Queen Creek located in Pinal County approximately 4 miles northeast of Florence Junction, Arizona, and from uncontrolled flows on the downstream watershed. According to the general design memorandum prepared by the U.S. Army Corps of Engineers for the Whitlow Ranch Reservoir project, the reservoir regulates the design flood from a peak inflow of 110,000 cfs to a maximum outflow of approximately 1,000 cfs.

Available plans, flow records, and hydrologic models provided the following information:

1. The design flow and design flood frequency are unknown.

2. USGS data show that the largest recorded flood between 1961 and the present was 42,900 cfs on August 19, 1954, as measured at Whitlow Reservoir Damsite.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 3,010 cfs.

4. The 500-year flood (superflood) is not reported on Federal Emergency Management Agency (FEMA) flood insurance study maps. USGS regression equations were used to estimate a 500-year flood of 5,150 cfs.

Generally, flows taken from published FEMA flood insurance studies (FIS) were given priority over other sources because of the substantial level of effort and review involved in their estimation. Although values for the more frequent recurrence intervals were included in the analysis for completeness, the critical discharge values were considered to be the 100-year flow and the lesser of the 500-year flow and the flow at the low chord elevation, based on HEC-18 criteria and MCDOT requirements.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1442.71	2,250	Q_{10}
1443.07	2,750	-
1443.24	3,010	Q_{100}
1444.48	5,150	Q_{500}
1449.00	16,900	Low Chord

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 5,150 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the critical flood (Q_{500}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Rittenhouse Road Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and a summary of pile embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{500} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

	Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1. Scour at Piers		
Contraction Scour, ft	0.0	0.0
Local Scour, ft	10.8	12.2
General Scour, ft	4.0	4.0
Total Scour, ft	**14.8**	**16.2**
2. Scour at Abutments		
Abutment Scour, ft	0.0	0.0
General Scour, ft	4.0	4.0
Total Scour, ft	**4.0**	**4.0**

Table 3. Summary of Pile Embedment

		Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1.	Embedment at Piers		
	Channel Elevation	1438.9	1438.9
	Total Scour, ft	14.8	16.2
	Bottom of Scour Hole Elev.	1424.1	1422.7
	Pile Tip Elev. (Pier 2)	1408.3	1408.3
	Embedment Remaining, ft	**15.8**	**14.4**
2.	Embedment at Abutments		
	Channel Elevation	1438.9	1438.9
	Total Scour, ft	4.0	4.0
	Bottom of Scour Hole Elev.	1434.9	1434.9
	Pile Tip Elev. (Abut. 1)	1414.0	1414.0
	Embedment Remaining, ft	**20.9**	**20.9**

5000

7000

STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{100} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads, live loads, and stream flow forces. The structural capacity of the piles, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was not warranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, the Rittenhouse Road Bridge at Queen Creek does not have sufficient capacity to resist the loads resulting from 100-year or 500-year flow events. The bridge is scour *critical*.

DEFICIENCIES AND COUNTERMEASURES:

Scour-related deficiencies include the following:

a. Insufficient embedment of the steel piles at the piers to support the vertical dead and live loads with scour produced by the 100-year and 500-year flows;

b. Minor undermining of sacked concrete riprap at the abutments and other scour protection on the bank upstream of the north abutment;

c. Failure of a 30-inch diameter culvert and surrounding sacked concrete riprap on the south bank upstream of the bridge.

Countermeasures to remedy scour-related deficiencies include the following:

a. Install scour monitoring devices and close the bridge to traffic if scour reaches a predetermined critical depth;

b. Construct a continuous concrete or grouted riprap sill across the width of the channel, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

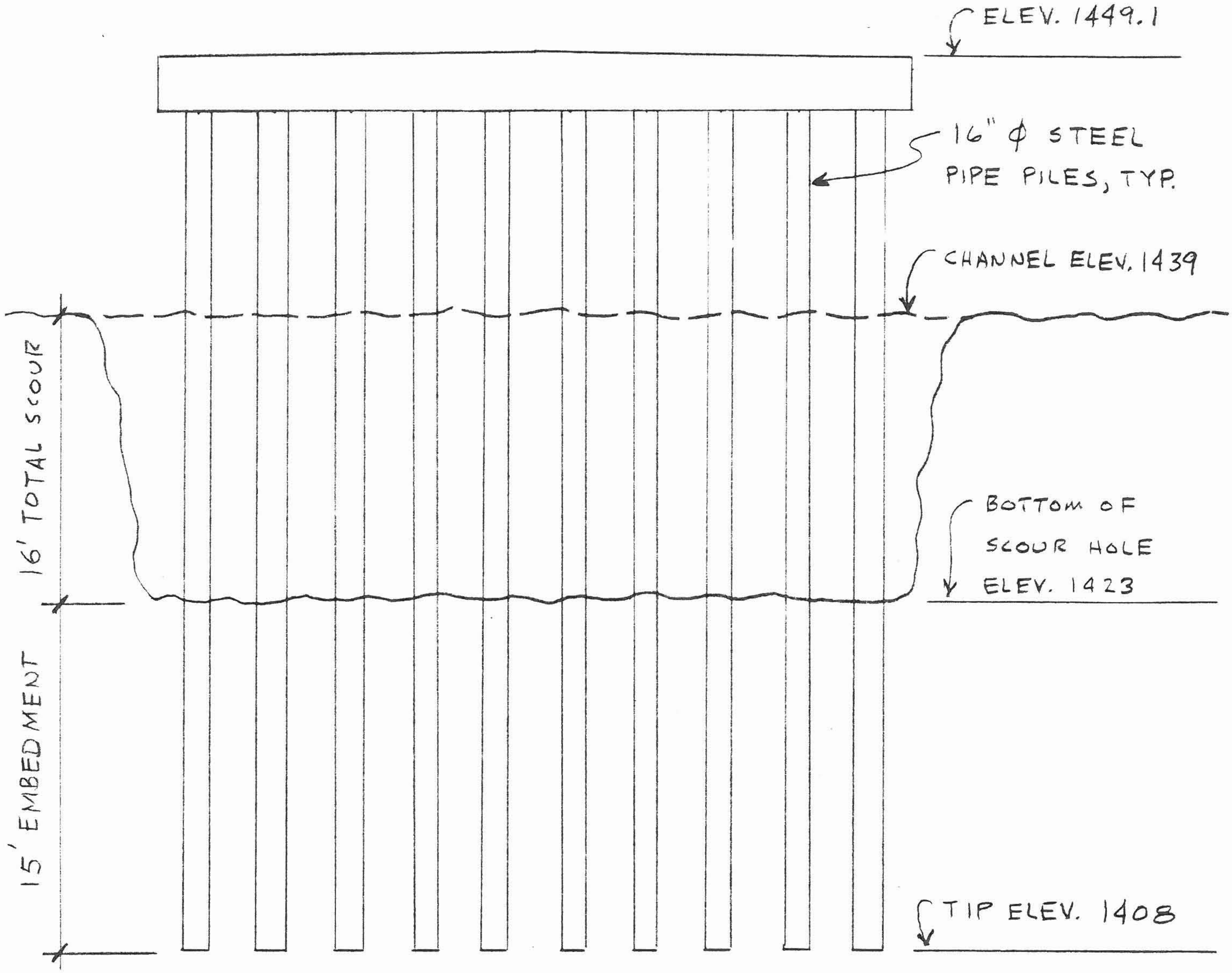
c. Encase the piers in a reinforced concrete beam supported on drilled shaft foundations (underpinning);

d. Remove the bridge and construct a new bridge on deeper foundations.

RITTENHOUSE ROAD BRIDGE OVER QUEEN CREEK

HYDRAULIC DATE (PER MM/CSSA):

Q_{500} = 5150 CFS
H.W. ELEV. = 1444.48
TOTAL SCOUR = 16'



PIER ELEVATION

Figure 4

Photo 1: View of upstream side of north abutment. Note sacked concrete rip-rap protecting north abutment outslope and trowelled concrete added later. Also note undercutting at base by impinging flow.

Photo 2: Same as previous. Note gravelly sand typical of primary flow channel.

Photo 3: View looking downstream from approximately mid-span of bridge deck. Note thalweg curving from lower right to upper left. Note low point bar to right and low alternate bar to left. Also note sandy, sparsely grassed downstream right overbank and mature trees providing stabilization to left bank downstream.

Photo 4: View looking along downstream rail toward south abutment. Note tree-lined banks and cultivated fields in background comprising downstream left overbank. Also note left overbank is not substantially elevated above channel.

Photo 5: View looking upstream from bridge deck toward SPRR bridge. Note upstream right overbank is grassed with occasional small trees and brush. Also note mature vegetation formed on middle bar between the two structures.

Photo 6: View looking approximately south across channel upstream of both bridges. Note tree-lined banks and grassy, vegetated upstream left overbank. Also note low vegetated bar being undercut adjacent to a primary channel consisting mainly of coarse sand.

Photo 7: View looking along upstream face of bridge toward north abutment. Note low area between embankments. Also note railroad box car used as bank protection just downstream of SPRR bridge. Further note low bars and established vegetation in channel between structures. Bar relief is typically 1-2 feet above low flow channels. Note locally braided condition of stream near structures.

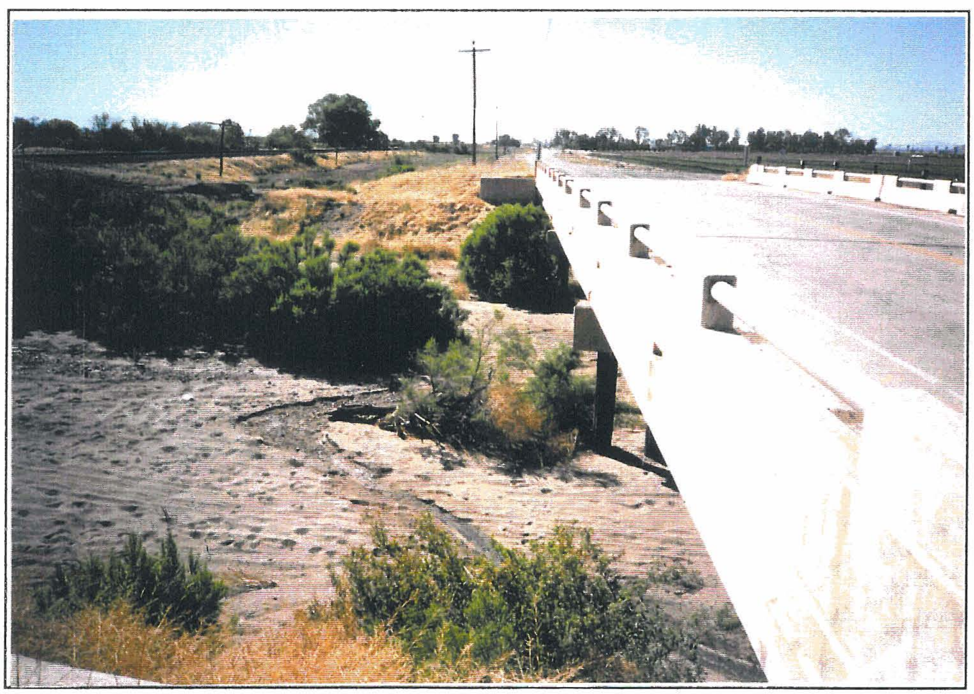


Photo 8: View along upstream face of structure looking toward south abutment. Note low area between embankments capturing runoff. Also note minor scour developed upstream of pier due to debris blocking flow.

Photo 9: View of I-beam and rubber tire dike used to deflect flow from north bank upstream of SPRR bridge. Note residual scour along base and deposition of sediment immediately upstream.

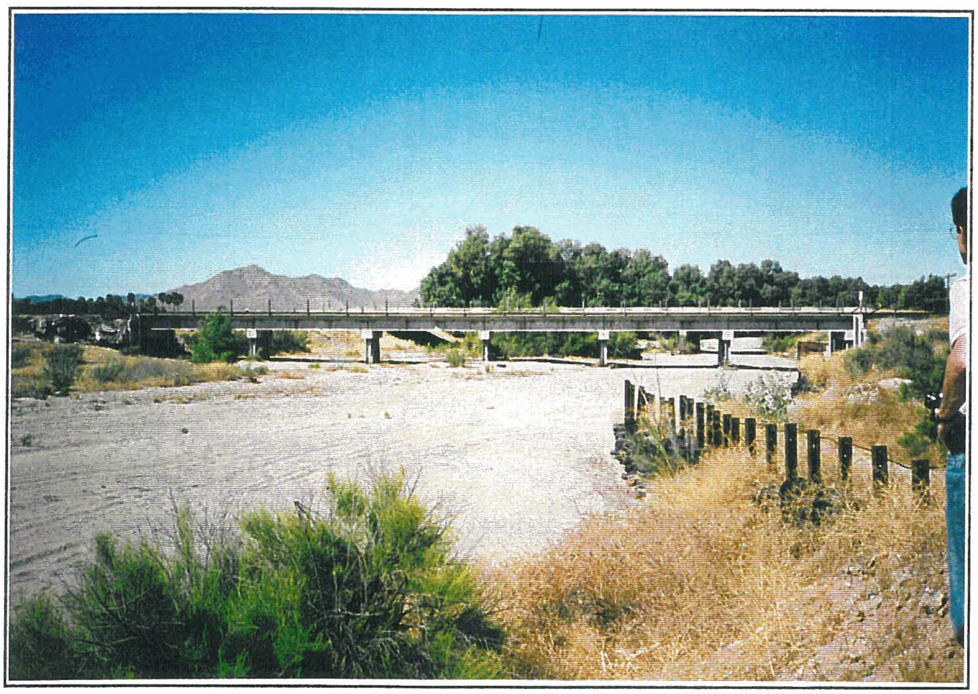
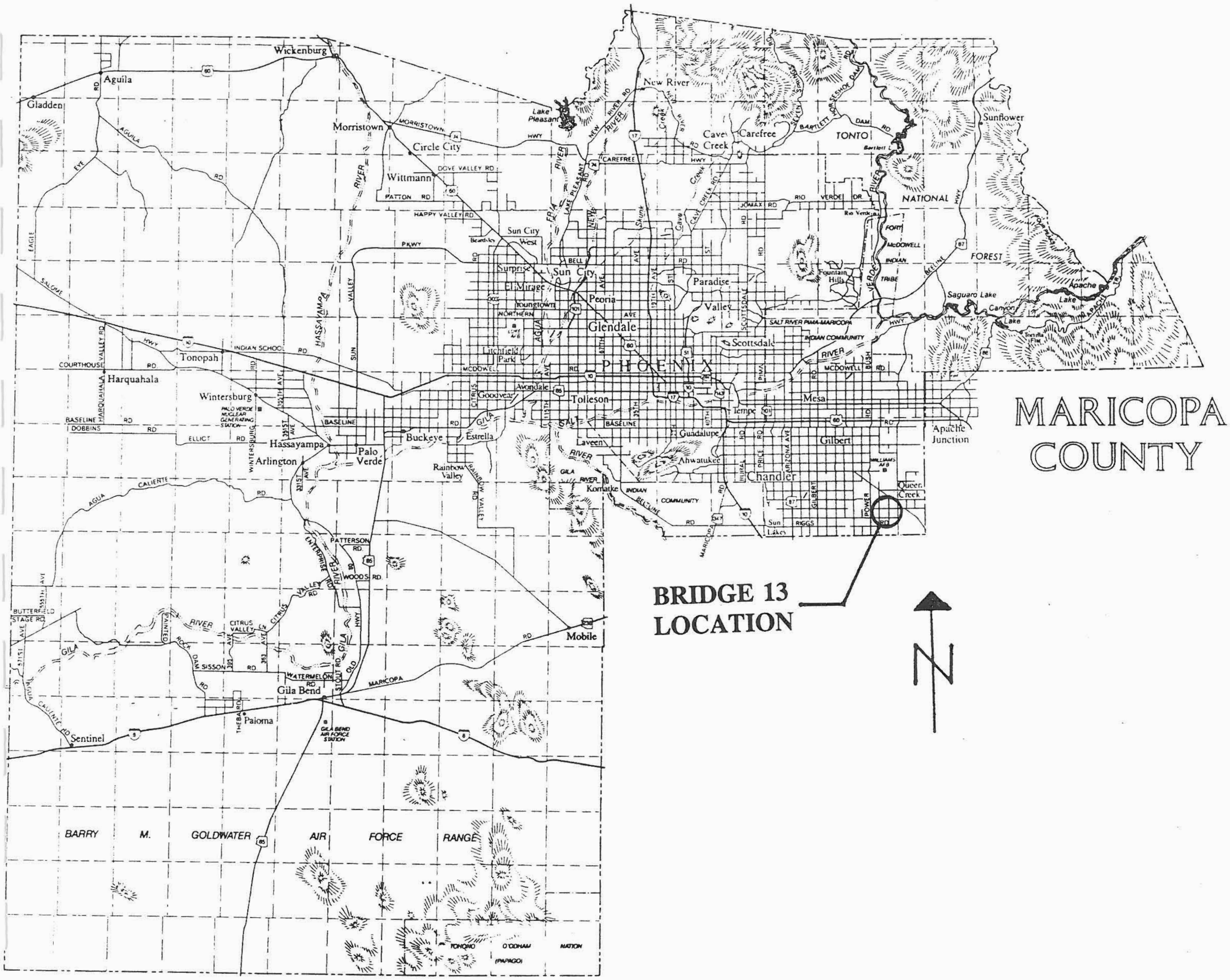


Photo 10: View looking downstream toward SPRR bridge from north bank. Note relative position of deflector dike along outside curve of impinging flow.

Photo 11: View of stream gage showing typical fluctuations of channel depth in vicinity of piles. Overall relief through piles is approximately 2 feet. Note staining of column at assumed original grade, however, substantial residual local scour at piers was not observed. Under high flows, close spacing of piles may assist debris capture and enhance local scour.

BRIDGE 13

HAWES ROAD BRIDGE OVER QUEEN CREEK



Location Map

Figure 1


Hawes Rd. Bridge #7818
over Queen Creek
Hawes Rd.
Queen Creek
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 13: HAWES ROAD OVER QUEEN CREEK (Structure #78818)

Assessment: Scour Stable

LOCATION: The Hawes Road Bridge at Queen Creek lies on the section line between Sections 16 and 17 of T2S, R7E, Gila and Salt River Baseline and Meridian, on Hawes Road near the Town of Queen Creek. The bridge is approximately 300' north of the intersection of Hawes Road and Ocotillo Road. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a three-span continuous concrete slab bridge with a total length of 130' center-to-center of abutment bearings and a skew of 23 degrees to the right. (See Location Plan, Figure 3.) The flow rate used for design is the 100-year flood of 3010 cfs. The bridge was designed by the Maricopa County Highway Department (MCHD) in 1986 and built in 1991 as MCHD Project No. 68268.

The abutments consist of a reinforced concrete cap beam supported on five 3' diameter drilled shafts. According to the plans, tip elevations of the drilled shafts are Elevation 1348, approximately 30' below the bottom of the stream bed. Short wing walls extend from the ends of the abutment wall parallel to the roadway centerline.

The piers also consist of a reinforced concrete cap beam supported by five 3' diameter drilled shafts. The tip elevation of the drilled shafts is Elevation 1340; embedment below the stream bed is approximately 38'.

EXISTING SCOUR PROTECTION: Scour protection at the abutments consists of an 18" thick layer of grouted riprap, sloped at 3:1 and keyed into the bottom of the channel to Elevation 1370 (approximately 8' below the stream invert). The riprap (dumped, not grouted) is carried around the ends of the abutments at a 1:1 slope, and keyed into the channel to Elevation 1370.

A 3' layer of dumped riprap was placed around the piers to a distance of 10' from the pier centerline. The top of the riprap layer is approximately 6' below the stream invert.

The channel upstream of the Hawes Road Bridge is unlined; downstream of the bridge the south bank has been lined with dumped riprap. Bridge inspection reports showed that ~~the~~ no scour was observed during inspections.

Residual scour holes between 1' and 2' deep around the drilled shaft columns were noted during a site inspection. This may indicate that deeper scour holes were formed during past flows and insufficient material was transported into the holes as the flow receded to completely fill them up. Also, dumped riprap placed along the upstream side of the south abutment is beginning to become destabilized. This may be due either to undermining of the toe of the riprap during flows in the creek or loss of support due to erosion caused by discharge of runoff from the bridge deck to the riprap.

STREAM FORM: The stream form of Queen Creek in the vicinity of the Hawes Road Bridge can be characterized as straight, with a nearly uniform trapezoidal section. (See Figure 4.) Sand bars are not well established, although a shallow point bar is forming upstream of the north abutment.

LAND USE: Land use in the vicinity of the bridge is primarily agricultural or low density residential. Urbanization will increase, but largely upstream of the bridge towards the Town of Queen Creek and along Hawes Road to the south. Urbanization is not expected to have an impact on stream flows at the bridge.

There is no evidence of sand and gravel extraction in the vicinity of the bridge.

SURFACE SOILS: Surface soils consist primarily of silt, sand and fine gravel with occasional cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 0.04 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of Queen Creek in the vicinity of the Hawes Road Bridge is 0.0034 ft/ft, or approximately 18.5' per mile, as estimated from U.S. Geological Survey (USGS) topographic maps.

VEGETATION: Vegetation includes trees such as palo verde and ironwood; desert broom is the dominant brush, with some creosote, ephedra and saltbush. Dry grasses are also present. The vegetation occurs primarily on the banks, especially on the banks downstream of the bridge, and only sparsely in the channel. Dry grasses and smaller varieties of shrubs are found on the stream banks and on the bars.

STREAM STABILITY: The Hawes Road Bridge is approximately 800' downstream from the Ocotillo Road Bridge at Queen Creek. The creek in the vicinity of these two bridges, both upstream and downstream has been effectively channelized by the construction of earthen levees on both banks. Lateral stability of the stream is maintained by the earthen, non-structural levees along the creek banks and by dumped riprap placed along the bank at the outside of bends. The presence of mature vegetation along the banks indicates some degree of lateral stability.

There are no grade control structures either upstream or downstream of the Hawes Road Bridge. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade, although there is no measurable degradation observed at the site.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Flow in Queen Creek comes from releases from Whitlow Ranch Reservoir, a flood control reservoir on Queen Creek located in Pinal County approximately 4 miles northeast of Florence Junction, Arizona, and from uncontrolled flows on the downstream watershed. According to the general design memorandum prepared by the U.S. Army Corps of Engineers for the Whitlow Ranch Reservoir project, the reservoir regulates the design flood from a peak inflow of 110,000 cfs to a maximum outflow of approximately 1,000 cfs.

Available plans, flow records, and hydrologic models provided the following information:

1. The design flow is 3,010 cfs and the design flood frequency is 100 years.

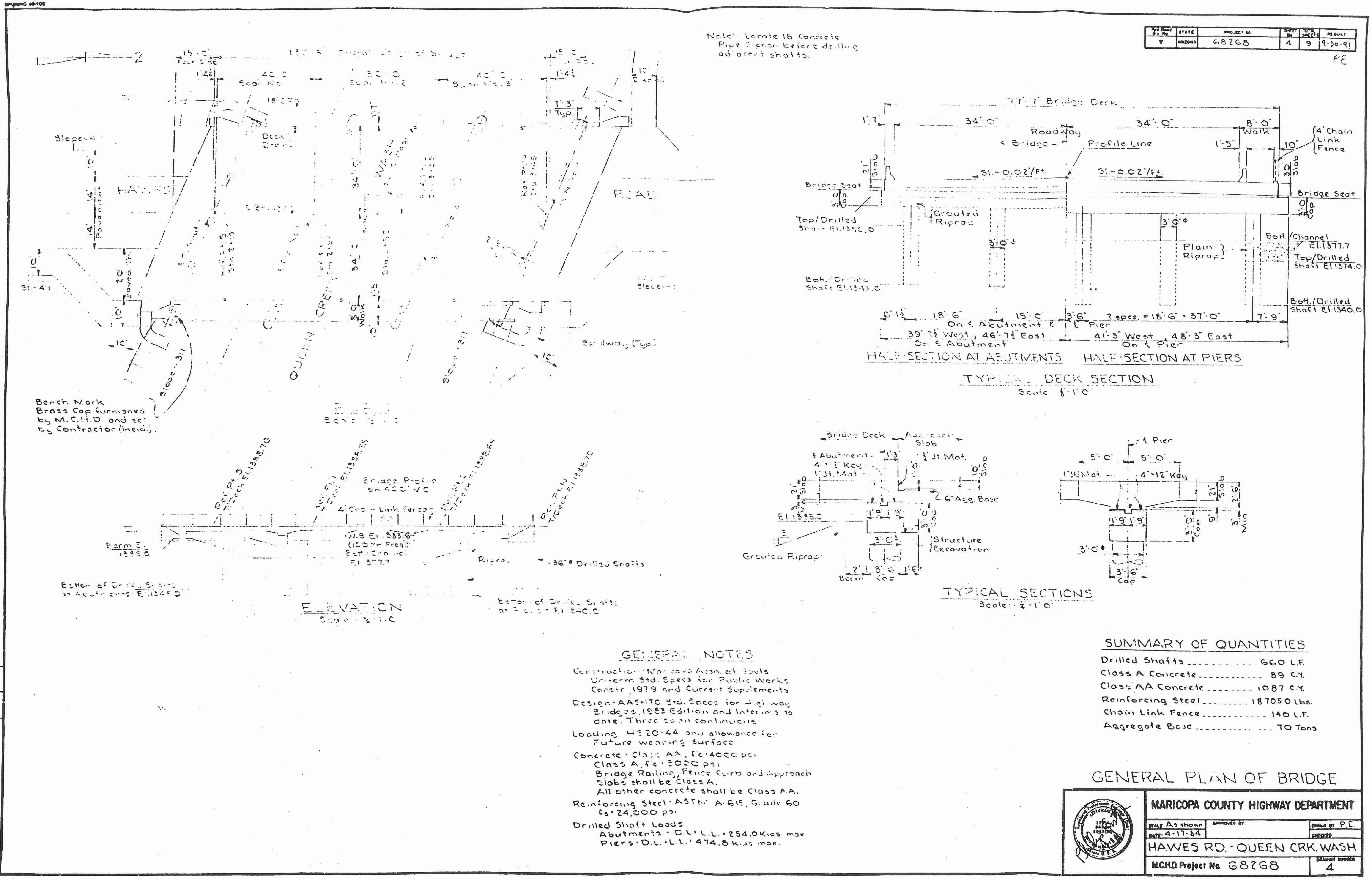
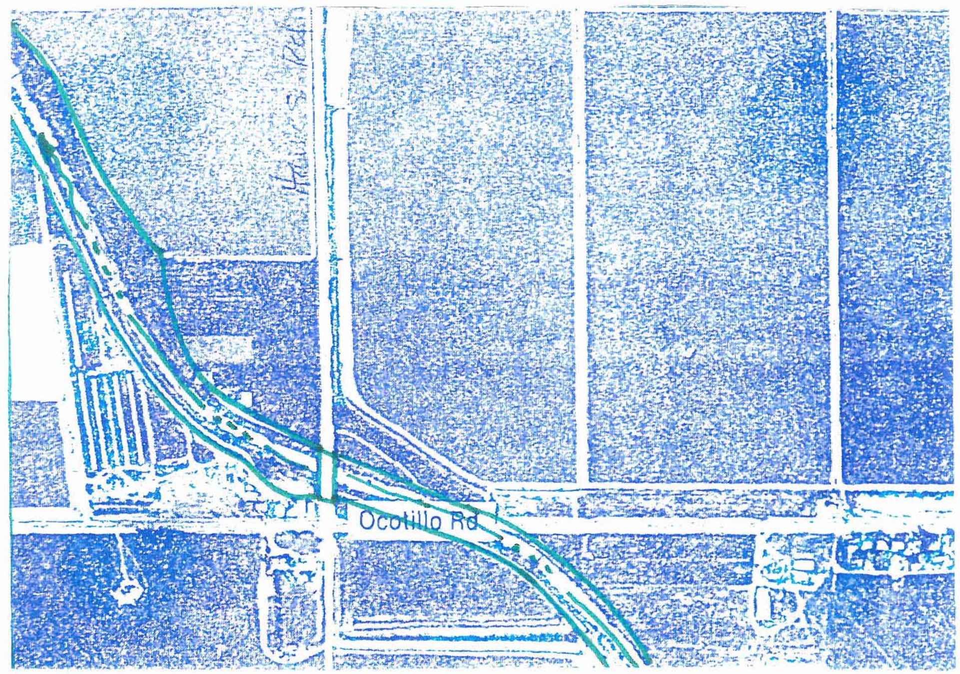

68268
4
9
9-30-91
Note: Locate 18" Concrete Pipe Siphon before drilling adjacent shafts.
77'-7" Bridge Deck
34'-0"
Roadway
34'-0"
8'-0" Walk
4' Chain Link Fence
℄ Bridge
Profile Line
Bridge Seat
Top/Drilled Shaft El.1390.0
Grouted Riprap
Plain Riprap
Bott./Channel El.1377.7
Top/Drilled Shaft El.1374.0
Bott./Drilled Shaft El.1340.0
HALF-SECTION AT ABUTMENTS
HALF-SECTION AT PIERS
TYPICAL DECK SECTION
TYPICAL SECTIONS
Structure Excavation
Grouted Riprap
6" Agg. Base
Bench Mark Brass Cap furnished by M.C.H.D. and set by Contractor (Incid.)
HAWES
QUEEN CREEK WASH
ROAD
Spillway (Typ.)
Bridge Profile
4' Chain Link Fence
36" Drilled Shafts
Riprap
ELEVATION
GENERAL NOTES
Construction: National Assn. of Govts Uniform Std. Specs for Public Works Constr. 1979 and Current Supplements
Design: AASHTO Std. Specs for Highway Bridges, 1983 Edition and Interims to date. Three span continuous
Loading HS20-44 and allowance for Future wearing surface
Concrete: Class AA, f'c = 4000 psi
Class A, f'c = 3000 psi
Bridge Railing, Fence Curb and Approach Slabs shall be Class A.
All other concrete shall be Class A.A.
Reinforcing Steel: ASTM A 615, Grade 60 fs = 24,000 psi
Drilled Shaft Loads
Abutments: D.L.+L.L. = 254.0 Kips max.
Piers: D.L.+L.L. = 474.8 kips max.
SUMMARY OF QUANTITIES
Drilled Shafts .... 660 L.F.
Class A Concrete .... 89 C.Y.
Class AA Concrete .... 1087 C.Y.
Reinforcing Steel .... 187050 Lbs.
Chain Link Fence .... 140 L.F.
Aggregate Base .... 70 Tons
GENERAL PLAN OF BRIDGE
MARICOPA COUNTY HIGHWAY DEPARTMENT
SCALE As shown
DATE 4-17-84
DRAWN BY P.C.
HAWES RD. - QUEEN CRK. WASH
M.C.H.D. Project No. 68268
4

Figure 3

<u>HAWES ROAD (SN 7818)</u>

Water Course	Queen Creek
Stream Form	Straight
Sinuosity	1.04
General Channelization	Non-structural (agricultural), trapezoidal, earthen levees
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.003968
Channel Contraction/Expansion	Main channel narrower US
Primary Surface Sediment Type	Silt/sand
D50 Size	0.040 MM
Armoring Potential	Low
Channel Vegetation	
Type/Size	Channel bottom rel. clear; banks sparse to moderate coverage including Palo Verde to 8 ft.; Desert Broom and Ironwood to 6 ft.; Occ. saltbush, creosote, ephedra; dry grasses dominate banks.
Density/Occurrence	Vegetation occurs on banks, sparse in channel; denser growth occurs DS of structure.
Relative Age	New to mature growth.
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Non-Structural, earthen levees
Vertical	None
Sediment Deposits & Bars	US: Bars not well established; shallow point bar forming US of north abutment. DS: Channel generally clear.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	North side shows 1-2 ft. residual scour.
Abutment	Rip-Rap placed along US side of south abutment is being destabilized by flow.
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agricultural
Sand & Gravel Extraction	No commercial extraction in vicinity
Freeway Construction	No
Dams	No
Drainage Channels	No



2. USGS data show that the largest recorded flood between 1961 and the present was 42,900 cfs on August 19, 1954, as measured at Whitlow Damsite, 4 miles northeast of Florence Junction.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 3,010 cfs.

4. The 500-year flood (superflood) is not reported on Federal Emergency Management Agency (FEMA) flood insurance study maps. USGS regression equations were used to estimate a 500-year flood of 5,150 cfs.

Generally, flows taken from published FEMA flood insurance studies (FIS) were given priority over other sources because of the substantial level of effort and review involved in their estimation. Although values for the more frequent recurrence intervals were included in the analysis for completeness, the critical discharge values were considered to be the 100-year flow and the lesser of the 500-year flow and the flow at the low chord elevation, based on HEC-18 criteria and MCDOT requirements.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage Discharge Curve

Stage	Discharge, cfs	Description
1382.11	2,250	Q_{10}
1382.57	2,750	Assumed
1382.79	3,010	Q_{100}
1384.36	5,150	Q_{500}
1385.67	7,400	Low Chord

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 5,150 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the critical flood (Q_{500}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18).

Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Hawes Road Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and a summary of drilled shaft embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{500} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1.	Scour at Piers		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	15.4	17.3
	General Scour, ft	4.0	4.0
	Total Scour, ft	**19.4**	**21.3**
2.	Scour at Abutments		
	Abutment Scour, ft	0.0	0.0
	General Scour, ft	4.0	4.0
	Total Scour, ft	**4.0**	**4.0**

Table 3. Summary of Drilled Shaft Embedment

		Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1.	Embedment at Piers		
	Channel Elevation	1377.9	1377.9
	Total Scour, ft	19.4	21.3
	Bottom of Scour Hole Elev.	1358.5	1356.6
	Drilled Shaft Tip Elev.	1340.0	1340.0
	Embedment Remaining, ft	**18.5**	**16.6**
2.	Embedment at Abutments		
	Channel Elevation	1377.9	1377.9
	Total Scour, ft	4.0	4.0
	Bottom of Scour Hole Elev.	1373.9	1373.9
	Drilled Shaft Tip Elev.	1348.0	1348.0
	Embedment Remaining, ft	**20.9**	**20.9**

STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{500} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads, live loads, and stream flow forces. The structural capacity of the concrete columns and drilled shafts, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was not warranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, the Hawes Road Bridge at Queen Creek has sufficient structural capacity to resist the loads resulting from flows up to and including 5150 cfs, i.e., the 500-year flow rate. The bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: There is some sloughing of riprap at the upstream side of the south abutment that should be repaired. ok

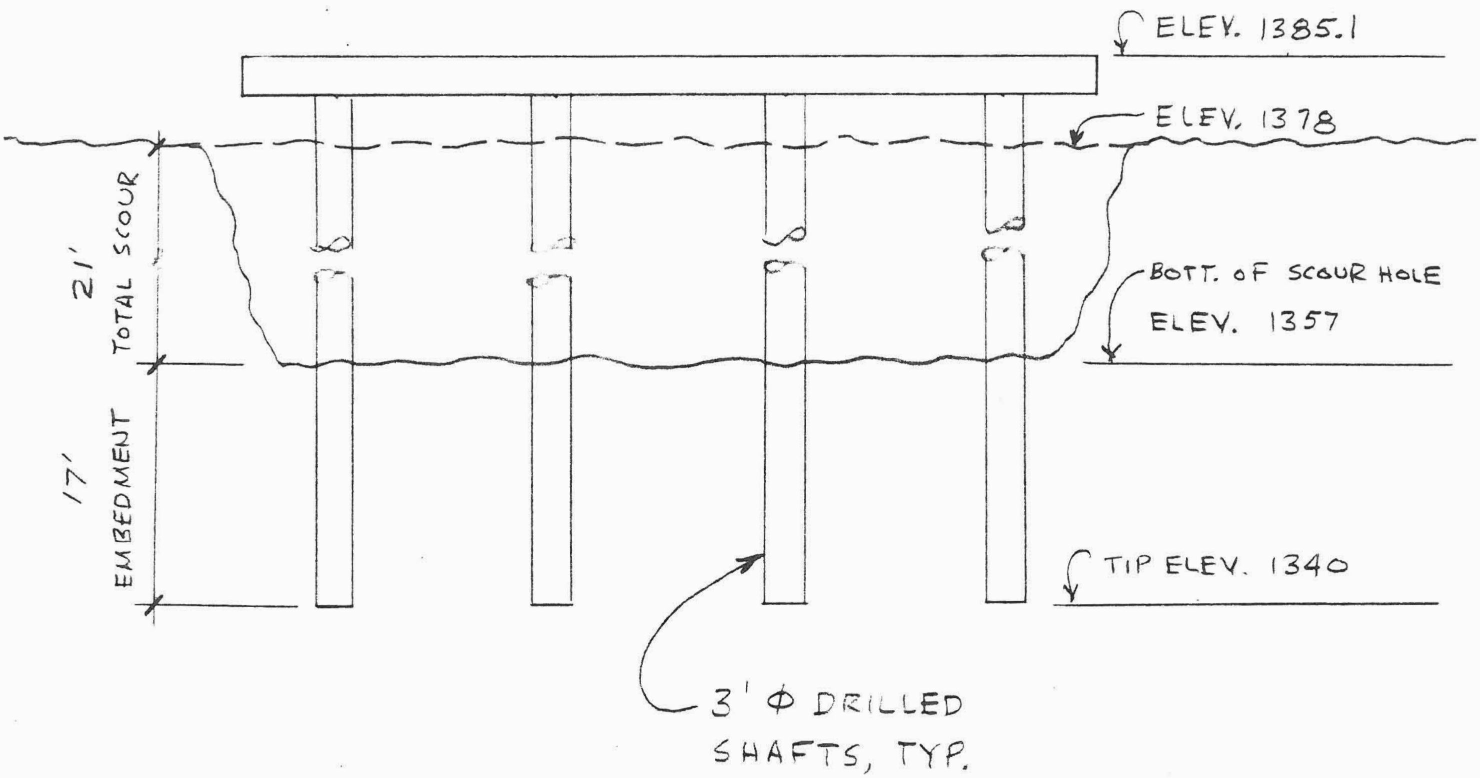
HAWES ROAD BRIDGE AT QUEEN CREEK

HYDRAULIC DATA (PER MM/CSSA):

Q_{500} = 5150 CFS

H.W. ELEV. = 1384.36

TOTAL SCOUR = 21'



PIER ELEVATION

Figure 5

Photo 1: Looking upstream from approximately mid-span of structure. Note generally uniform trapezoidal section formed by levee embankments. Note channel developing meandering form with thalweg shifting to impinge on south abutment while a low bar forms upstream of north abutment. Some mature vegetation is established along north embankment.

Photo 2: Looking downstream from approximately mid-span of structure. Note clear sand bed channel with somewhat greater density of mature vegetation on north side versus south. Note rip-rap placed along south bank.

Photo 3: Looking toward upstream face of structure. Note reduced clearance toward north abutment. Also note bar development upstream from north pile bent.

Photo 4: Looking toward south bank just downstream of south abutment. Note placement of rip-rap.

Photo 5: View of rip-rap placed along upstream side of south abutment. Note how scour by impinging flow has destabilized rip-rap at scupper outfall. Similar condition exists at downstream side of south abutment.

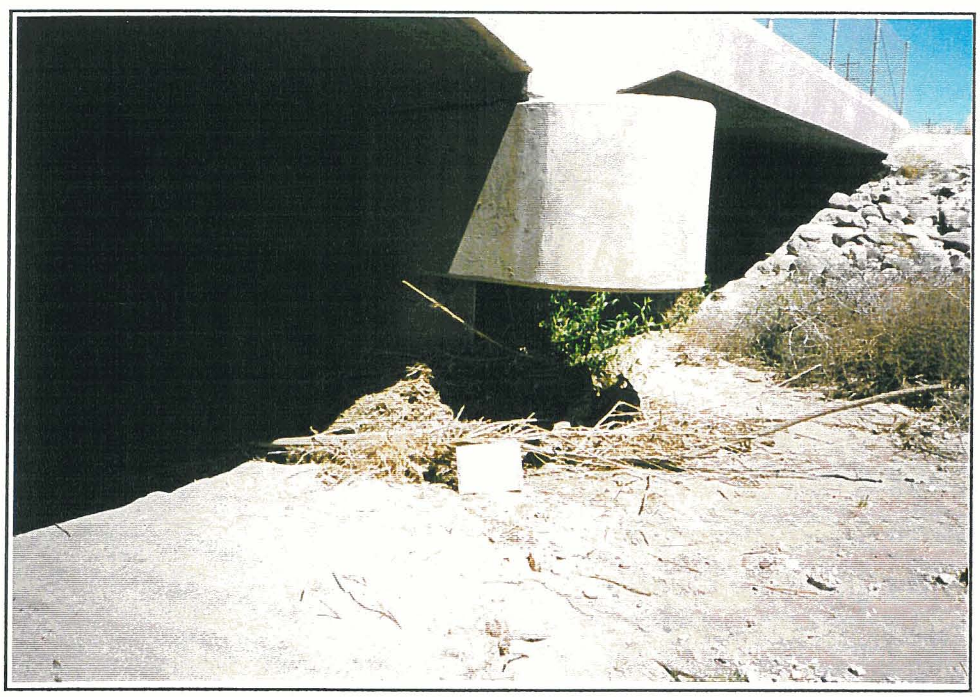


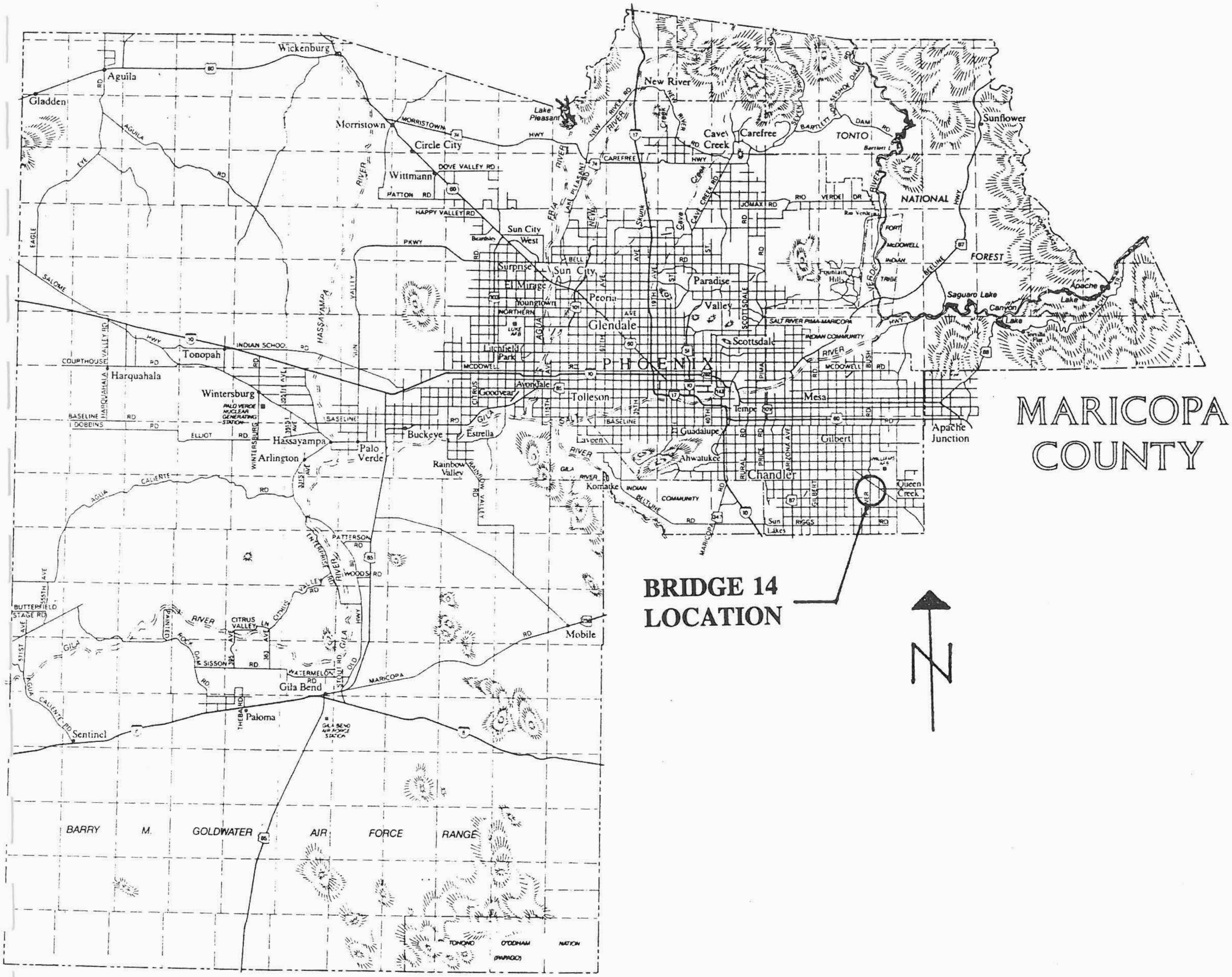
Photo 6: View of north side upstream pile. Note vegetation captured by pile. Similar debris capture was observed on in-stream vegetation.

Photo 7: View of south side levee upstream of structure. Note dry grass growing in sandy soil is typical of near stream overbanks. Levee crest is typically 3-5 feet minimum above surrounding overbanks. Cultivated fields generally surround stream beyond levees with roadway embankments and structures in overbanks locally.

Photo 8: View of typical north side pile. Note residual local scour of 1-2 feet.

BRIDGE 14

POWER ROAD BRIDGE OVER QUEEN CREEK



Location Map

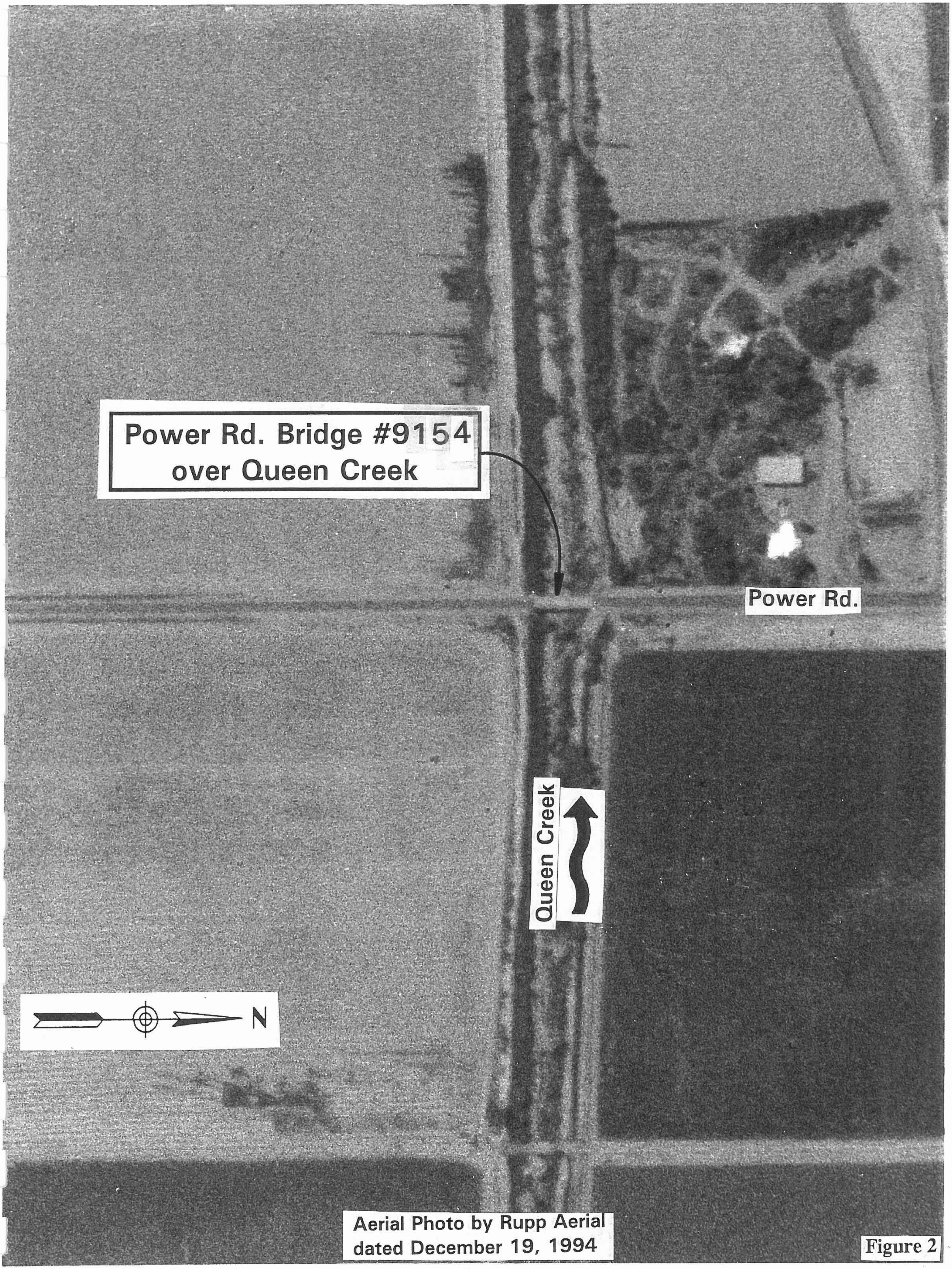

Power Rd. Bridge #9154
over Queen Creek
Power Rd.
Queen Creek
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 14: POWER ROAD BRIDGE OVER QUEEN CREEK (Structure #9154)

Assessment: Scour Critical

LOCATION: The Power Road Bridge at Queen Creek lies on the section line between Section 7 of T2S, R7E and Section 12 of T2S, R6E, Gila and Salt River Baseline and Meridian, on Power Road near the Town of Queen Creek. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a five-span concrete slab bridge with an total length of 135' center-to-center of abutment bearings and a zero degree skew. (See Location Plan, Figure 3). The flow rate used for design is not noted on as-built plans although a high water elevation of 1355' is indicated. The bridge was built in 1955 as Maricopa County Highway Department (MCHD) Project No. 56-C-8.

The abutments consist of four 10BP42 steel piles spaced at approximately 8' with a reinforced concrete curtain wall. According to as-built plans, the pile tips extend to Elevation 1305.

The piers consist of five 10BP42 steel piles spaced at approximately 6' with a reinforced concrete curtain wall. According to as-built plans, the pile tips extend to Elevation 1305 and the curtain wall terminates several feet below the bottom of the channel.

EXISTING SCOUR PROTECTION: The as-built plans do not show any special scour protection provided for the bridge or the channel at the time of construction. Although not shown on the plans, a concrete collar that extends around the pier near the base of the curtain wall has been constructed at Pier No. 2. The concrete collar may have been constructed to protect steel pilings exposed as a result of scour. It is not known if a similar collar has been constructed at all the piers and abutments. The collar was exposed approximately 2' at the time of the site visit; this condition appears in several consecutive reports, indicating that the channel does not appear to be degrading or eroding at a very fast rate.

STREAM FORM: The stream form of Queen Creek in the vicinity of the Power Road Bridge can be characterized as straight, with a nearly uniform trapezoidal section. (See Figure 4.) Regular sand bars are not well established, although occasional shallow bars form upstream and downstream of the bridge. Sediment deposition near the structure has reduced the open area of the bridge at the spans next to the abutments.

LAND USE: Land use in the vicinity of the bridge is primarily agricultural or low density residential. There is no evidence of sand and gravel extraction in the vicinity of the bridge. Urbanization will increase, although primarily along Power Road to the north, but not expected to have an impact on stream flows at the bridge.

SURFACE SOILS: Surface soils consist primarily of silt, sand and fine gravel with occasional cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 0.04 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of Queen Creek in the vicinity of the Power Road Bridge, estimated from U.S. Geological (USGS) topographic maps, is 0.0034 ft/ft, or approximately 18' per mile.

VEGETATION: Vegetation includes trees such as palo verde, mesquite and ironwood; brush includes desert broom, creosote and ephedra. Dry grasses are also present. The vegetation occurs as low to medium density on the banks, with greater density of vegetation upstream of the bridge than downstream. Dry grasses generally cover the banks, especially on the downstream side. There is sparse vegetation in the channel bottom, although there is a large clump of mature trees in the middle of the channel upstream of the bridge.

STREAM STABILITY: The creek, both upstream and downstream of the bridge, has been effectively channelized by the construction of earthen levees on both banks. The channel is slightly wider downstream of the bridge than upstream. Lateral stability of the stream is maintained by the earthen, non-structural levees along the creek banks. The presence of mature vegetation along the banks indicates some degree of lateral stability.

There are no grade control structures either upstream or downstream of the Power Road Bridge. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade, although there is no measurable degradation observed at the site.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Flow in Queen Creek comes from releases from Whitlow Ranch Reservoir, a flood control reservoir on Queen Creek located in Pinal County approximately 4 miles northeast of Florence Junction, Arizona, and from uncontrolled flows on the downstream watershed. According to the general design memorandum prepared by the U.S. Army Corps of Engineers for the Whitlow Ranch Reservoir project, the reservoir regulates the design flood from a peak inflow of 110,000 cfs to a maximum outflow of approximately 1,000 cfs.

Available plans, flow records, and hydrologic models provided the following information:

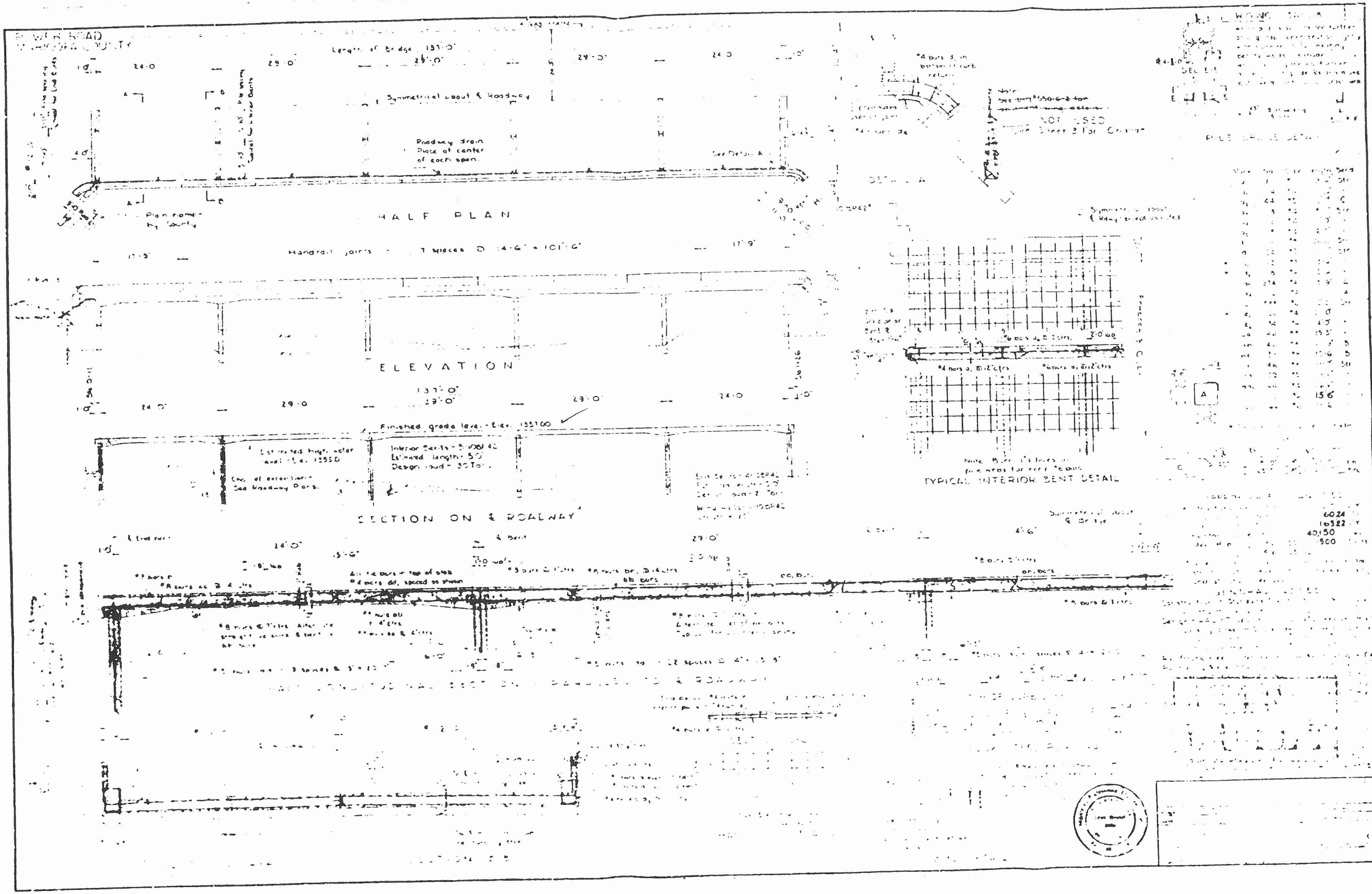
1. The design flow is 3,010 cfs and the design flood frequency is 100 years.

2. USGS data show that the largest recorded flood between 1961 and the present was 42,900 cfs on August 19, 1954, as measured at Whitlow Damsite, 4 miles northeast of Florence Junction.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 3,010 cfs.

4. The 500-year flood (superflood) is not reported on Federal Emergency Management Agency (FEMA) flood insurance study maps. USGS regression equations were used to estimate a 500-year flood of 5,150 cfs.

Generally, flows taken from published FEMA flood insurance studies (FIS) were given priority over other sources because of the substantial level of effort and review involved in their estimation. Although values for the more frequent recurrence intervals were included in the analysis for completeness, the critical discharge values were considered to be the 100-year flow



HALF PLAN
ELEVATION
SECTION ON ℄ ROADWAY
TYPICAL INTERIOR BENT DETAIL
DETAIL A
NOT USED
Symmetrical about ℄ Roadway
Roadway drain
Finished grade level

Figure 3

POWER ROAD (SN 9154)

Water Course	Queen Creek
Stream Form	Straight
Sinuosity	Not Applicable
General Channelization	Non-structural (agricultural), trapezoidal, earthen levees
Channel Slope	Steeper US
Estimated Channel Slope (ft/ft)	0.003941
Channel Contraction/Expansion	Main (clear) channel narrower US
Primary Surface Sediment Type	Silt/sand
D50 Size	0.040 MM
Armoring Potential	Low
Channel Vegetation	
Type/Size	Trees include Palo Verde, Mesquite, Ironwood to 15 ft.; brush includes Desert Broom, Creosote, Ephedra; dry grasses.
Density/Occurrence	Vegetation occurs as low to medium density on banks, sparse in channel; denser growth occurs US of structure. Dry grasses generally cover banks.
Relative Age	Mature
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Non-Structural, earthen levees
Vertical	None
Sediment Deposits & Bars	Regular bar formation is not established; occasional low bars form US and DS. Sediment deposition near structure has reduced clearance toward abutments.
Evidence of Degradation	Occasional cut banks visible US but little evidence of vertical incision.
Evidence of Aggradation	No
Evidence of Scour	
Pier	Exposed pile cap at pier 2.
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agricultural.
Sand & Gravel Extraction	No commercial extraction in vicinity.
Freeway Construction	No
Dams	No
Drainage Channels	Possible irrigation inflows.

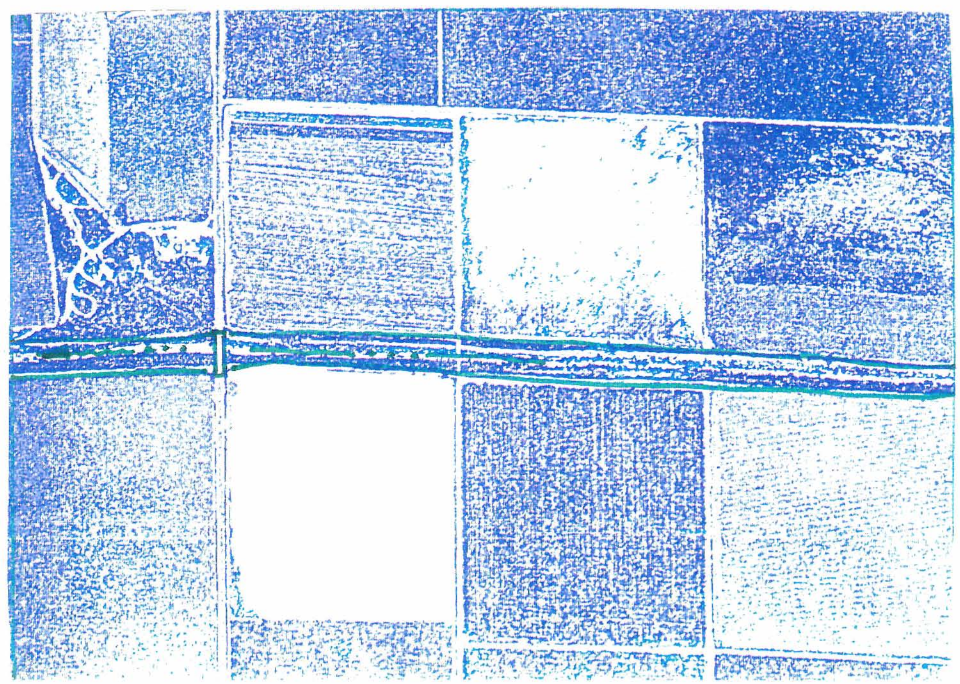


Figure 4

and the lesser of the 500-year flow and the flow at the low chord elevation, based on HEC-18 criteria and MCDOT requirements.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1349.27	2,250	Q_{10}
1349.82	2,750	-
1350.08	3,010	Q_{100}
1351.78	5,150	Q_{500}
1355.35	11,960	Low Chord

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 5,150 cfs.

A review of bridge inspection reports showed that the only scour problem of any significance is the exposure of approximately 2' of a concrete collar at Pier 2. This condition appears in several consecutive reports, indicating that the channel does not appear to be degrading or eroding at a very fast rate.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the critical flood (Q_{500}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Power Road Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and a summary of drilled shaft embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{500} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1.	Scour at Piers		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	10.5	11.4
	General Scour, ft	4.0	4.0
	Total Scour, ft	**14.5**	**15.4**
2.	Scour at Abutments		
	Abutment Scour, ft	0.0	0.0
	General Scour, ft	4.0	4.0
	Total Scour, ft	**4.0**	**4.0**

Table 3. Summary of Pile Embedment

		Q = 3,010 cfs (Q_{100})	Q = 5,150 cfs (Q_{500})
1.	Embedment at Piers		
	Channel Elevation	1342.7	1342.7
	Total Scour, ft	14.5	15.4
	Bottom of Scour Hole Elev.	1328.2	1327.3
	Pile Tip Elev.	1305.0	1305.0
	Embedment Remaining, ft	**23.2**	**22.3**
2.	Embedment at Abutments		
	Channel Elevation	1342.7	1342.7
	Total Scour, ft	4.0	4.0
	Bottom of Scour Hole Elev.	1338.7	1338.7
	Pile Tip Elev.	1305.0	1305.0
	Embedment Remaining, ft	**33.7**	**33.7**

STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{100} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads, live loads, and stream flow forces. The structural capacity of the piles, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was unwarranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, the Power Road Bridge at Queen Creek does not have sufficient structural capacity to resist the loads resulting from the 100-year and 500-year flow rates. The bridge is scour *critical*.

DEFICIENCIES AND COUNTERMEASURES:

Scour-related deficiencies include the following:

a. Insufficient strength of the steel piles to resist a combination of lateral and vertical forces.

Countermeasures to remedy scour-related deficiencies include the following:

a. Install scour monitoring devices and close the bridge to traffic if scour reaches a predetermined critical depth;

b. Construct a continuous concrete or grouted riprap sill across the width of the channel, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

c. Encase the piers in a reinforced concrete beam supported on drilled shaft foundations (underpinning);

d. Remove the bridge and construct a new bridge on deeper foundations.

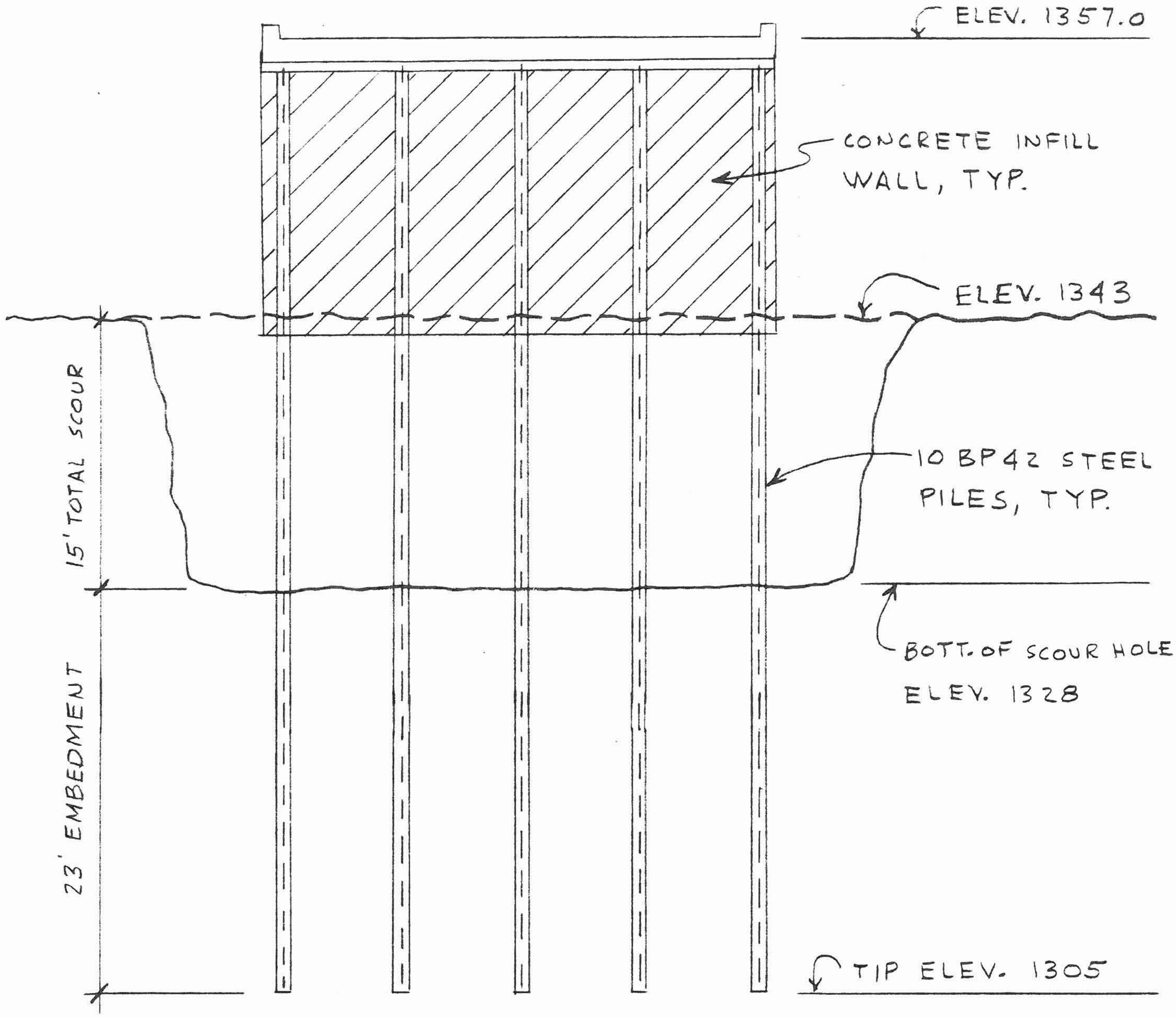
POWER ROAD BRIDGE OVER QUEEN CREEK

HYDRAULIC DATE (PER MM / CSSA):

Q 500 = 5150 CFS

H.W. ELEV. = 1351.78

TOTAL SCOUR = 15'



PIER ELEVATION

Figure 5

Photo 1: View looking upstream from approximately mid-span of structure. Note roughly trapezoidal cross-section and mature vegetation established at toe of levees. Also note relatively straight narrow sandy channel formed slightly south of centerline of levee crests. Upstream overbanks are generally cultivated fields.

Photo 2: View looking downstream from approximately mid-span. Note substantial expansion of clear trapezoidal channel immediately downstream of bridge. Also note downstream overbanks are generally cultivated fields with occasional structure on north side.

Photo 3: View from top of levee embankment looking south along upstream face of structure. Note substantially decreased cross-section through spans 1, 2, (nearest) and 5 resulting in constrained flow through spans 3 and 4.

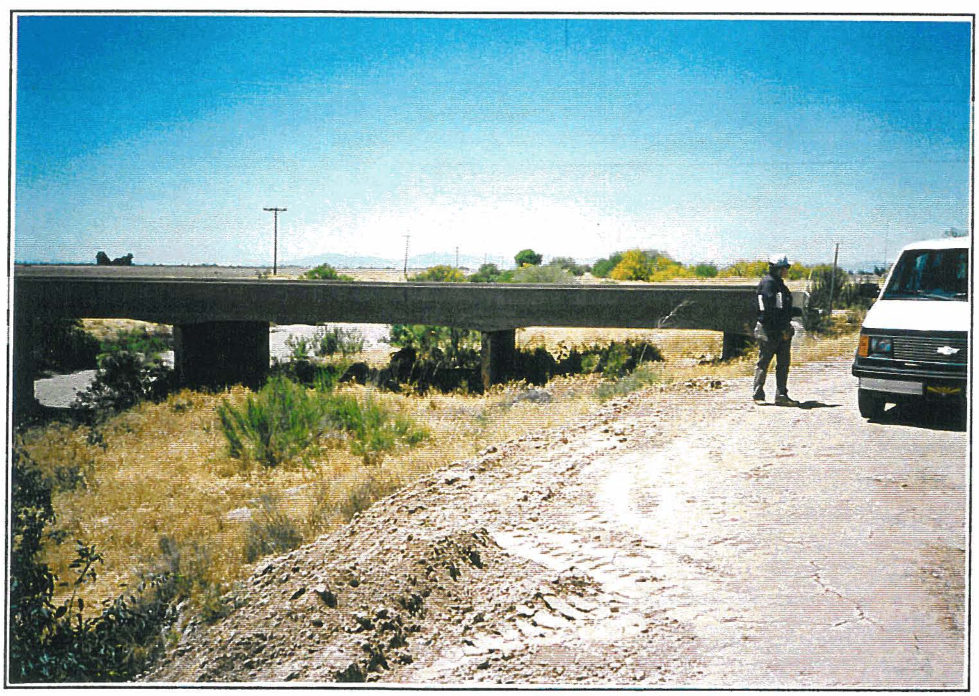


Photo 4: View of upstream face showing spans 1 and 2. Note significantly reduced cross-section.

Photo 5: View of spans 1 and 2 from upstream low flow channel. Note clean coarse sand typical of low flow channel. Also note low grasses established on bank.

Photo 6: View of upstream face of span 5. Note reduced cross-section and vegetation.

Photo 7: View of upstream end of pier 3. Note scour below base of pier cap.

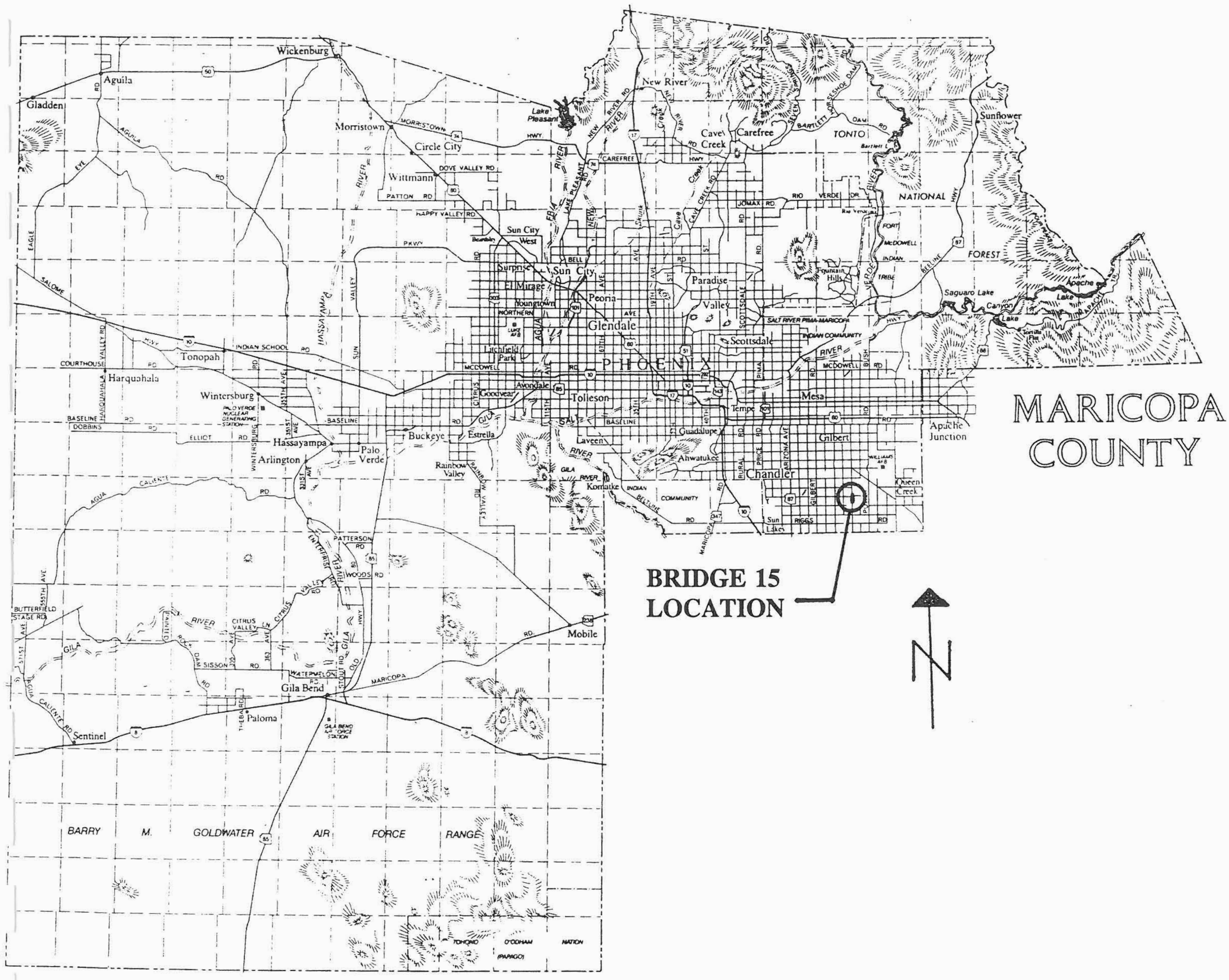
Photo 8: View of downstream end of pier 2. Note debris buildup typical of several locations during inspection. Note variability in channel cross-section.

Photo 9: View of north side bank approximately 800 feet upstream of structure. Note cut bank and destabilized vegetation.

Photo 10: View of south side bank approximately 800 feet upstream of structure. Note cut bank near toe of levee.

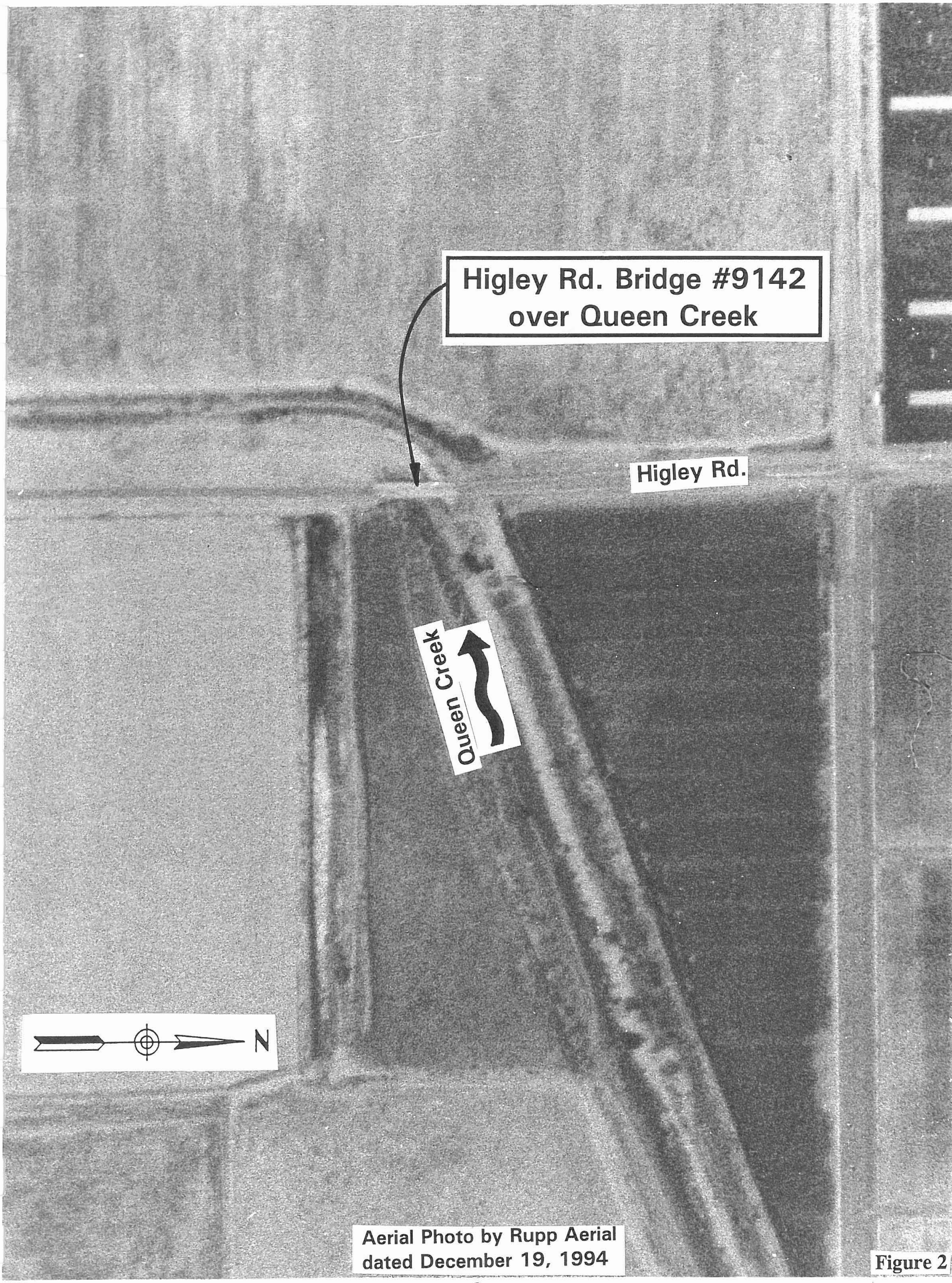
BRIDGE 15

HIGLEY ROAD BRIDGE OVER QUEEN CREEK



Location Map

Figure 1



Higley Rd. Bridge #9142
over Queen Creek
Higley Rd.
Queen Creek
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 15: HIGLEY ROAD BRIDGE OVER QUEEN CREEK (Structure #9142)

Assessment: Scour Stable

LOCATION: The Higley Road Bridge at Queen Creek is located in Section 14, T2S, R6E, Gila and Salt River Baseline and Meridian, on Higley Road approximately 3.5 miles south of the Town of Higley. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a three-span, continuous steel girder bridge with a total length of 152' center-to-center of abutment bearings and a 40 degree skew to the left. (See Location Plan, Figure 3.) The flow rate used for design is 3050 cubic feet per second (cfs), corresponding to a flood frequency of 100-year at the time of design. The bridge was built in 1964 as Maricopa County Highway Department (MCHD) Project No. S-296(3).

The abutments consist of a reinforced concrete abutment cap supported by seven 14" diameter concrete-filled steel pipe piles driven to Elevation 1258 according to the plans. Short reinforced concrete wingwalls extend from the abutment; the wing walls are supported by a total of three 14" diameter piles.

The piers consist of a reinforced concrete cap beam supported by nine 14" diameter concrete-filled steel piles driven to Elevation 1285, approximately 35' below the design stream invert of Elevation 1320.

EXISTING SCOUR PROTECTION: Scour protection consists of a grouted riprap lining (riprap and mortar, reinforced with welded wire fabric) that has been "staked" to the stream bank with steel fence posts. The lining starts at the upstream side of the north abutment and continues downstream on the right-hand (looking downstream) side of the channel to approximately 300' below the bridge. Queen Creek makes a sharp bend (deflection angle of approximately 65 degrees) to the right (looking downstream) at the bridge. Downstream of the bridge the channel slope is reduced and the channel width increases considerably.

No structural scour protection has been provided at the south abutment or at the piers. Bridge inspection reports showed that there were no significant scour problems at the Higley Road Bridge at Queen Creek. The only indication of scour observed during a site visit was some undercutting of the concrete bank protection at the north abutment. The overall condition of the bank protection is judged to be fair, although the lining appears to be eroding in places with subsequent exposure and corrosion of the welded wire fabric.

STREAM FORM: The stream form of Queen Creek in the vicinity of the Higley Road Bridge can be characterized as straight, with a slight meander of the clear (unvegetated) channel. (See Figure 4.) The cross-section is nearly uniform trapezoidal. Regular sand bars are not well established, although there is significant sediment deposition between the south abutment and Pier 2.

LAND USE: Land use in the vicinity of the bridge is primarily agricultural or low density residential. Urbanization will increase, although primarily along Higley Road to the north. Urbanization is not expected to have an impact on stream flows at the bridge.

There is no evidence of sand and gravel extraction in the vicinity of the bridge.

SURFACE SOILS: Surface soils consist primarily of silt, sand and fine gravel. The estimated median diameter (D_{50}) of the surface soil is approximately 0.04 mm. The armoring potential of the creek bed is estimated to be low.

SLOPE: The estimated slope of Queen Creek upstream of the Higley Road Bridge is 0.0024 ft/ft, or approximately 12.5' per mile. Downstream of the bridge the slope is 0.0009 ft/ft, slightly less than 5' per mile. Slopes were estimated using U.S. Geological Survey (USGS) topographic maps.

VEGETATION: Vegetation includes trees such as palo verde, cottonwood and desert willow; brush includes desert broom, creosote and brittle brush. Dry grasses are also present. The vegetation occurs on the channel banks. Larger trees and brush occur sparsely near the bottom of the channel; the banks are dominated by dry grasses.

STREAM STABILITY: The creek, both upstream and downstream of the bridge, has been effectively channelized by the construction of earthen levees on both banks. A part of the right bank has been armored, as described previously. Lateral stability of the stream is maintained by the earthen, non-structural levees along the creek banks, reinforced by concrete bank protection at the north abutment and at the outside of the bend in the channel. The presence of mature vegetation along the banks indicates some degree of lateral stability.

There are no grade control structures either upstream or downstream of the Higley Road Bridge. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade, although there is no measurable degradation observed at the site.

CURRENT HYDROLOGY: Flow in Queen Creek comes from releases from Whitlow Ranch Reservoir, a flood control reservoir on Queen Creek located in Pinal County approximately 4 miles northeast of Florence Junction, Arizona, and from uncontrolled flows on the downstream watershed. According to the general design memorandum prepared by the U.S. Army Corps of Engineers for the Whitlow Ranch Reservoir project, the reservoir regulates the design flood from a peak inflow of 110,000 cfs to a maximum outflow of approximately 1,000 cfs.

Available plans, flow records, and hydrologic models provided the following information:

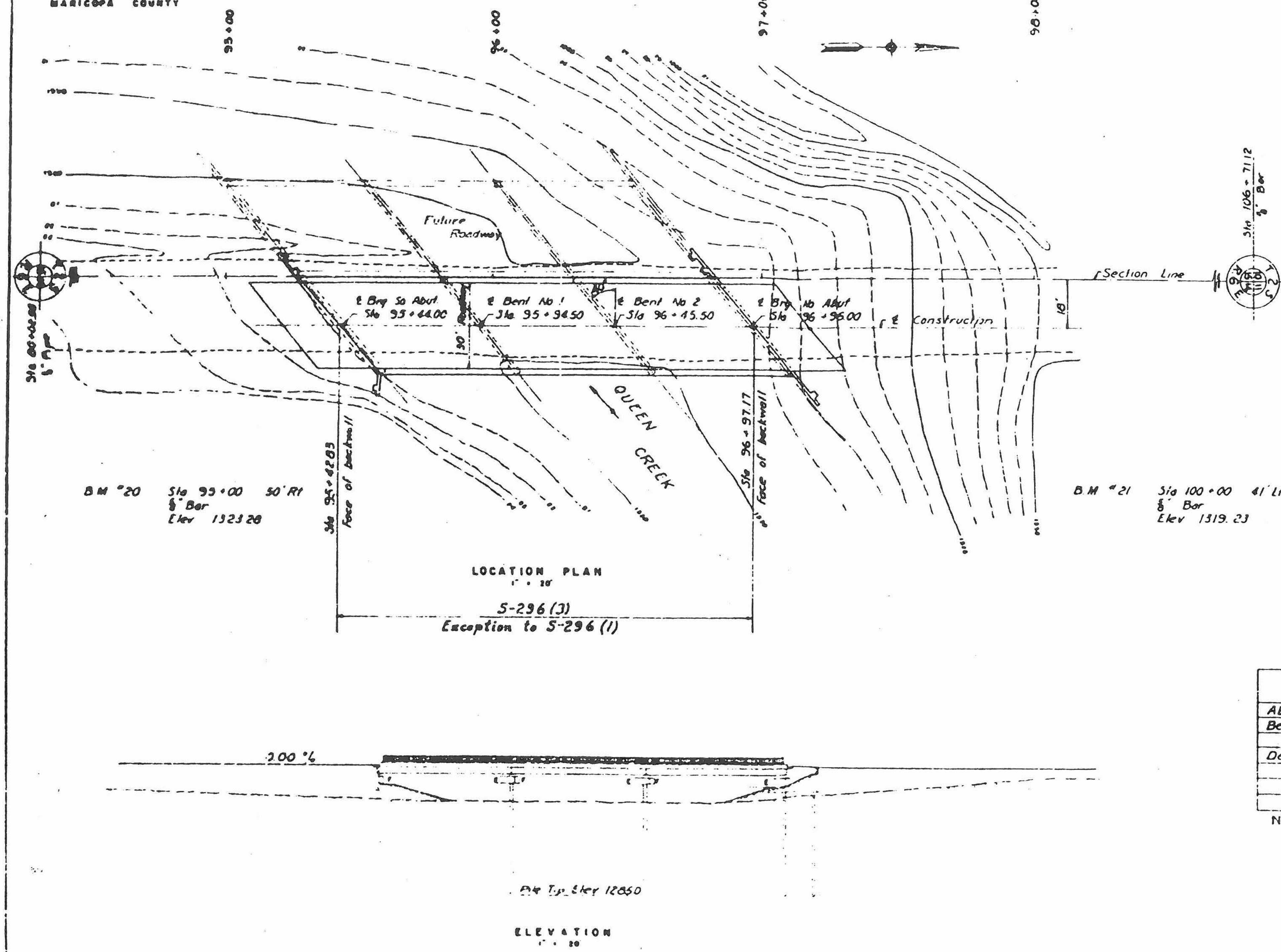
1. The design flow is 3,050 cfs and the design flood frequency is 100 years.

2. USGS data show that the largest recorded flood between 1961 and the present was 42,900 cfs on August 19, 1954, as measured at Whitlow Damsite, 4 miles northeast of Florence Junction.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 3,010 cfs.

HIGLEY ROAD BRIDGE ... QUEEN CREEK CHANNEL
MARICOPA COUNTY

STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
ARIZ.	S 296 (3)	2	7	5-8-64 -JRP



SPECIFICATIONS
Design: 1961 AASHO
Construction: AHD Standard Specifications 1960.

DESIGN LOADS
Dead Load: Wt of structure plus 30 psf for future wearing course.
Live Load: AASHO H20-S16-1944

ALLOWABLE STRESSES
Structural Steel: ASTM Spec A 36 $f_s = 20\,000$ psi
Reinforcing Steel: $f_s = 20{,}000$ psi for intermediate grade conforming to ASTM Spec A 15 & A 305.
Concrete: Substructure - Class 'A' $f'_c = 2500$ $f_c = 1000$ $n = 12$
Deck - Class 'D' $f'_c = 3000$ $f_c = 1200$ $n = 10$

PILES
Piles may be either cast in place concrete or precast concrete conforming to AHD Std Dwg CP-6 and shall be driven to the tip elevation shown or deeper if required to obtain a minimum capacity of 28 Tons. Pre-bore holes for pile shells.
Min. butt diameter of piles - 14"

HANDRAIL
Handrail to be furnished and installed by MCHD.

WELDING
All welding shall conform to American Welding Society Specifications for Welded Highway and Railway Bridges.

ITEM	SHEET NO.	CLASS D CONCRETE C.Y.	CLASS A CONCRETE C.Y.	STEEL - LBS REINF.	STEEL - LBS STRUCT.	CONCRETE PILES L.F.	
Abutments	5		[illegible]	[illegible]	115	[illegible]	841.85
Bents	4		[illegible]	[illegible]	115	[illegible]	753.31
		165.84	80.17	8276			
Deck	6	[illegible]		42 300	114 470		
				43452			
		165.84	95.6	51,072	114,965		
Total		[illegible]	[illegible]	[illegible]	114,350	[illegible]	1595

Non Pay Item - Embankment under bridge and abuts 240 C.Y. (Approx.)

Figure 3

HIGLEY ROAD (SN 9142)

Water Course	Queen Creek
Stream Form	Straight (slight meandering of clear channel.)
Sinuosity	Not Applicable
General Channelization	(US) Non-structural (agricultural), trapezoidal, earthen levees. (DS) Banks trapezoidal - East: earthen r/w embankment. West: wire-mesh reinforced grouted rip-rap.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.000282
Channel Contraction/Expansion	Channel expands DS
Primary Surface Sediment Type	Silt/sand
D50 Size	0.040 MM
Armoring Potential	Low
Channel Vegetation	
Type/Size	Cottonwood to 35 ft., Palo Verde to 10 ft., Desert Broom to 5 ft., Desert Willow, Creosote, Brittle Bush; dry grasses.
Density/Occurrence	Vegetation occurs on banks. Larger trees occur sparsely near main channel bottom; shrubs are sparse also. Banks dominated by dry grasses.
Relative Age	Mature
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Some bank protection along northwest bank; earthen levees elsewhere.
Vertical	None
Sediment Deposits & Bars	No significant bar development; significant sediment deposition between south abutment and nearest pier.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	Some scour and undercutting on north abutment slope protection.
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agricultural.
Sand & Gravel Extraction	No commercial extraction in vicinity.
Freeway Construction	No
Dams	No
Drainage Channels	Possible irrigation inflows.

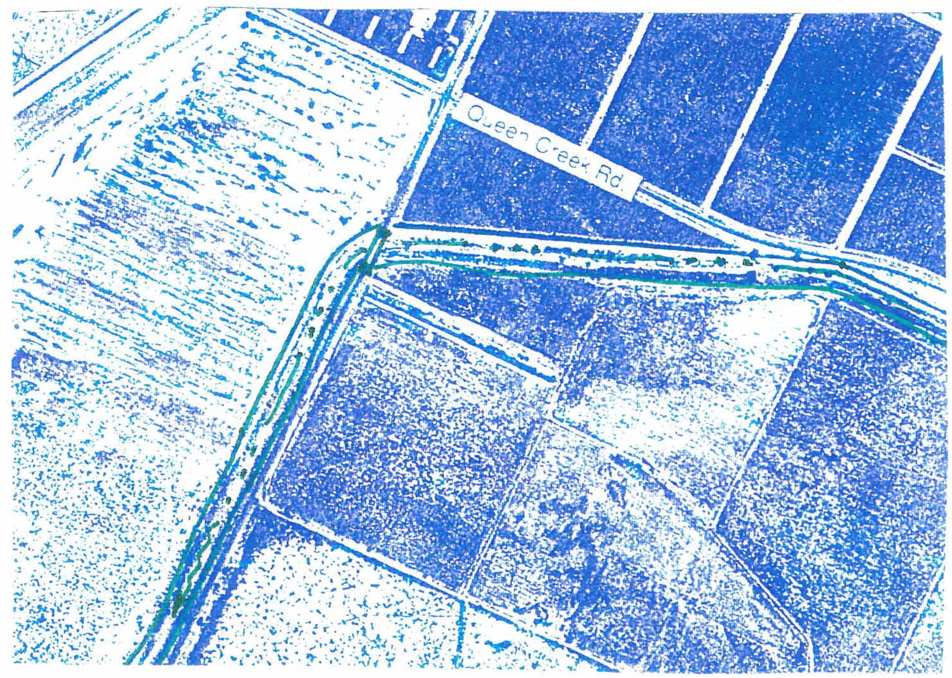


Figure 4

4. The 500-year flood (superflood) is not reported on Federal Emergency Management Agency (FEMA) flood insurance study maps. USGS regression equations were used to estimate a 500-year flood of 5,150 cfs.

Generally, flows taken from published FEMA flood insurance studies (FIS) were given priority over other sources because of the substantial level of effort and review involved in their estimation. Although values for the more frequent recurrence intervals are included in the analysis for completeness (see *Technical Appendix*), the critical design discharge values were considered to be the 100-year flow and the lesser of the 500-year flow and the flow at the low chord elevation, based on HEC-18 criteria and MCDOT requirements.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1327.08	2,250	Q_{10}
1327.94	2,750	-
1328.35	3,010	Q_{100}
1329.66	3,910	Low Chord
1331.25	5,150	Q_{500}

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 3,910 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the maximum flood (Q_{LC}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Higley Road Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and remaining pile embedment are shown in Tables 2 and 3, respectively.

A schematic representation of scour at the piers during Q_{LC} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 3,010 cfs (Q_{100})	Q = 3,910 cfs (Q_{LC})
1.	Scour at Piers		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	14.7	15.5
	General Scour, ft	4.0	4.0
	Total Scour, ft	**18.7**	**19.5**
2.	Scour at Abutments		
	Abutment Scour, ft	10.7	12.0
	General Scour, ft	4.0	4.0
	Total Scour, ft	**14.7**	**16.0**

Table 3. Summary of Pile Embedment

		Q = 3,010 cfs (Q_{100})	Q = 3,910 cfs (Q_{LC})
3.	Embedment at Piers		
	Channel Elevation	1317.6	1317.6
	Total Scour, ft	18.7	19.5
	Bottom of Scour Hole Elev.	1298.9	1298.1
	Pile Tip Elev.	1285.0	1285.0
	Embedment Remaining, ft	**13.9**	**13.1**
4.	Embedment at Abutments		
	Channel Elevation	1317.6	1317.6
	Total Scour, ft	14.7	16.0
	Bottom of Scour Hole Elev.	1312.9	1301.6
	Pile Tip Elev.	1285.0	1285.0
	Embedment Remaining, ft	**17.9**	**16.6**

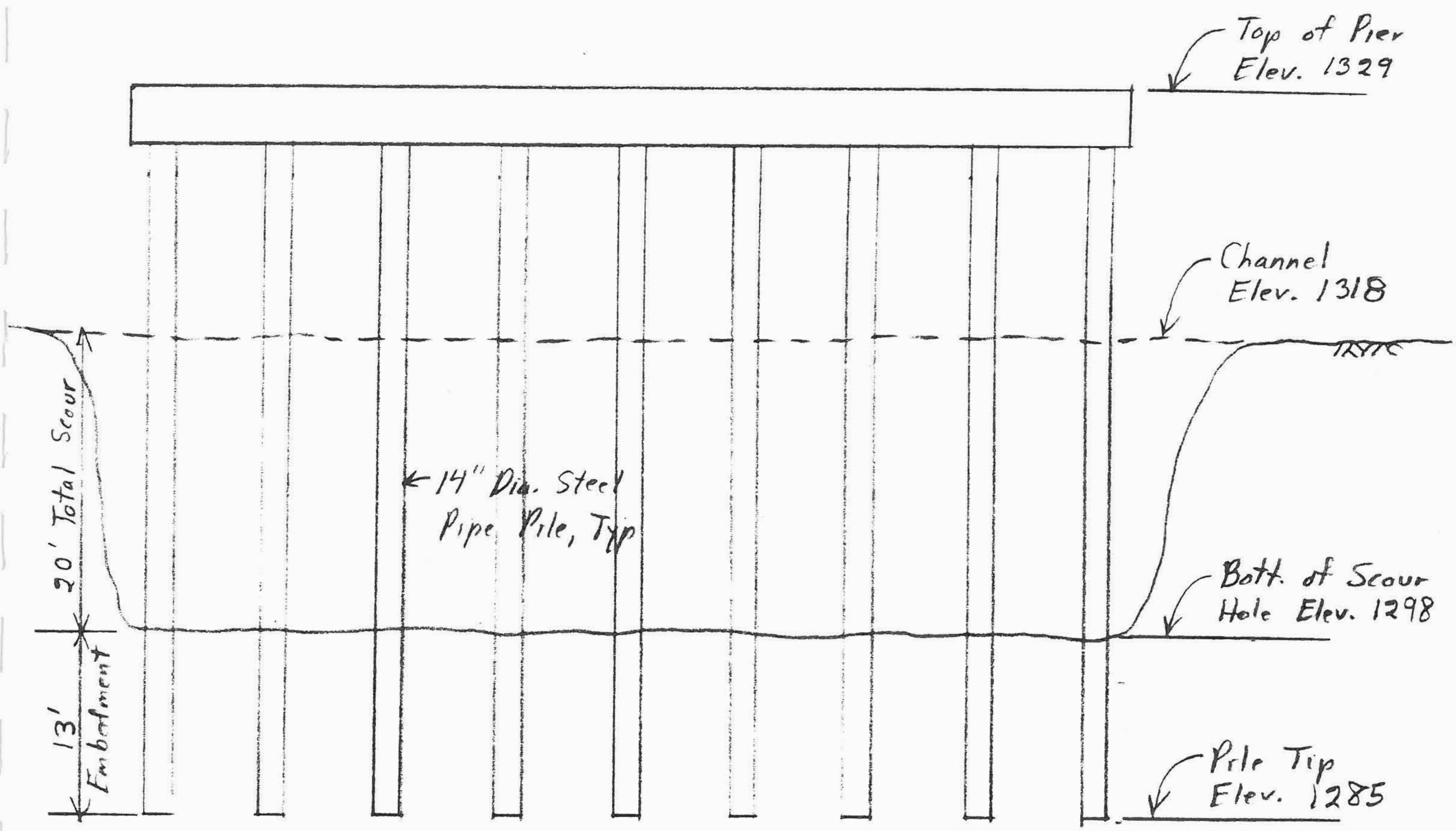
STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{LC} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads, live loads, and stream flow forces. The structural capacity of the concrete columns and drilled shafts, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was unwarranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS Based on the structural evaluation, the Higley Road Bridge at Queen Creek has sufficient structural capacity to resist the loads resulting from flows up to and including 3910 cfs , i.e., the low chord flow rate. The bridge is scour *stable*. ok

HIGLEY ROAD BRIDGE OVER QUEEN CREEK

HYDRAULIC DATA (Per MM/CSSA)

$Q_{LOW\ CHORD}$ = 3910 CFS
H.W. Elev. 1329.66
Total Scour = 20'



PIER ELEVATION

Figure 5

Photo 1: View looking upstream from approximately mid-span of structure. Note relatively stable trapezoidal section formed by levees upstream of bridge. Levees generally border cultivated fields with crests 3 - 8 feet above overbanks. Note relatively well established vegetation along levee banks. Channel bottom consists of coarse sand.

Photo 2: View looking downstream from approximately mid-span of structure. Note coarse sand channel and relatively wide trapezoidal section formed by levees. Also note wire mesh reinforced concrete bank protection extending from structure to completion of turn. Downstream right overbank is cultivated field while left bank is alongside roadway.

Photo 3: View of south abutment and pile bent. Note variability in sediment deposition on either side of bent. Also note rust on piles.

Photo 4: View looking upstream along south pile bent. Note depositional variation along bent. Maximum relief is approximately 6 feet. Note reduced cross-section.

Photo 5: View of south abutment. Note washout of concrete paving and staining pattern on abutment surface.

Photo 6: View of slope immediately upstream of north abutment. Note scour of concrete slope protection by impinging flow and exposure of wire mesh.

Photo 7: View of concrete pavement slope protection along west bank downstream of structure. Note scour of surface exposing wire mesh.

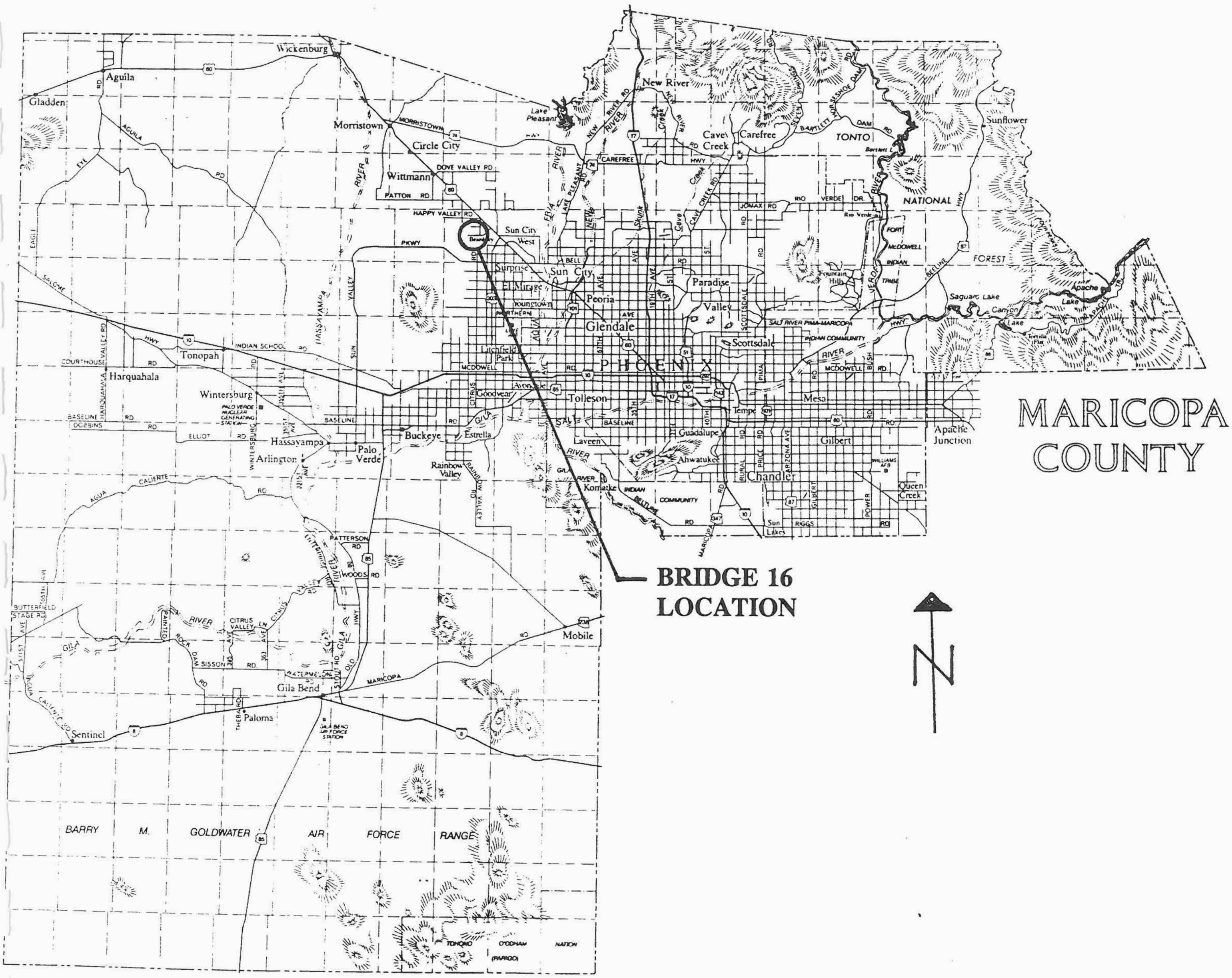
Photo 8: View of north bank upstream of structure. Note minor slumping of bank and destabilization of vegetation by impinging flow.

Photo 9: View of south bank immediately upstream of structure. Note erosion of levee embankment with vertical cut walls. Further instability may occur during severe flow conditions.

Photo 10: View of channel immediately after turning south. Note regular trapezoidal section running parallel with roadway alignment.

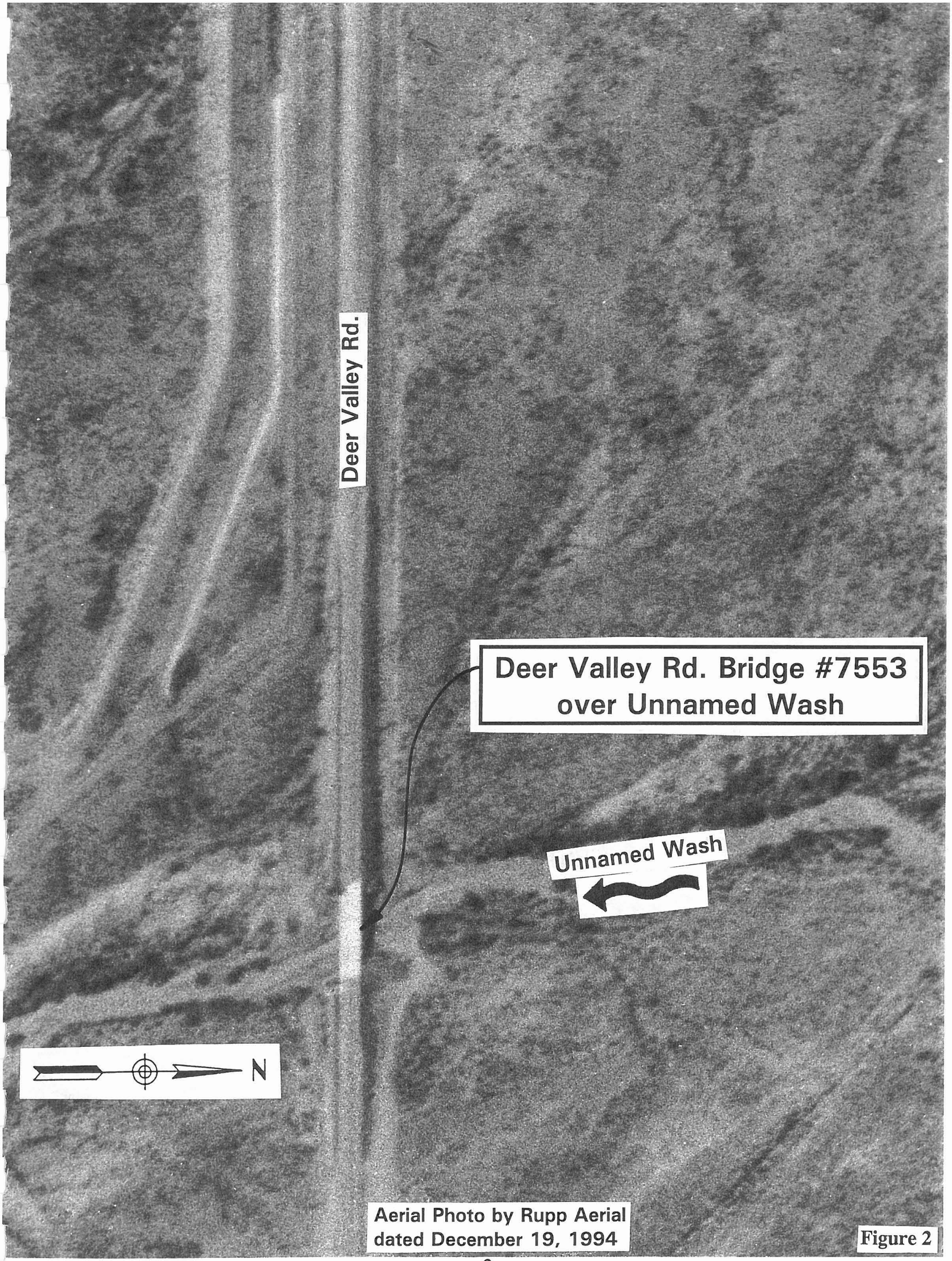
BRIDGE 16

DEER VALLEY ROAD BRIDGE OVER UNNAMED WASH



Location Map

Figure 1


Deer Valley Rd.
Deer Valley Rd. Bridge #7553
over Unnamed Wash
Unnamed Wash
N
Aerial Photo by Rupp Aerial
dated December 19, 1994
Figure 2

BRIDGE 16: DEER VALLEY ROAD BRIDGE OVER UN-NAMED WASH

(Structure #7553)
Assessment: Scour Critical

LOCATION: The Deer Valley Road Bridge at the unnamed wash near 189th Avenue (hereafter called "the wash") is located in Section 21 of T4N, R2W, Gila and Salt River Baseline and Meridian, on Deer Valley Road approximately three miles west of Grand Avenue (US Highway 60). See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The structure is a three-span concrete slab bridge with a total length of 162' center-to-center of abutment bearings and a skew of 20 degrees to the left. (See Location Plan, Figure 3.) The flow rate used for design is the 100-year flood of 3,925 cubic feet per second (cfs) as shown on the plans. The bridge was designed in 1986 by the Maricopa County Highway Department (MCHD) and built in 1988 as MCHD Project No. 68417.

The abutments consist of a reinforced concrete cap beam supported on three 3' diameter drilled shafts. According to the plans, tip elevations of the drilled shafts are at Elevation 1355, approximately 34' below the bottom of the stream bed. Short wingwalls extend from the ends of the abutment wall.

The piers also consist of a reinforced concrete cap beam supported by three 3' diameter drilled shafts. The tip elevation of the drilled shafts is Elevation 1345; embedment below the stream bed is approximately 44'.

EXISTING SCOUR PROTECTION: Scour protection at the abutments consists of an 24" thick layer of dumped riprap, sloped at 3:1 and keyed into the bottom of the channel approximately 5.5' below the stream invert. The riprap is carried around both ends of the east abutment to form spur dikes, with the larger of the two spur dikes on the upstream side. There is a riprap-lined spur dike on the downstream side of the west abutment; riprap on the upstream side wraps around the abutment to join a riprap blanket on the north side of the roadway embankment.

The north side of the roadway embankment serves as a side slope of a drainage channel along the road that intercepts runoff flowing to the southeast. The embankment is lined with a 24" thick layer of dumped riprap, keyed 3' below the invert of the drainage channel.

A 3' layer of dumped riprap was placed around the piers to a distance of 10' from the pier centerline. The top of the riprap layer was constructed flush with the stream invert.

The channel under the bridge was constructed as a flat surface between the riprap at the abutments, with an invert elevation at the bridge centerline of Elevation 1389.5. Flows in the wash have since cut a low flow channel approximately 2.5' below the post-construction grade. This low flow channel has partially undercut and destabilized the riprap along the west side of Pier 2.

Flows in the drainage channel along the embankment have caused headcutting and erosion of the channel. If this erosion is not stabilized, the riprap along the embankment could be undermined and destabilized.

STREAM FORM: The stream form of the wash in the vicinity of the Deer Valley Road Bridge can be characterized as braided. (See Figure 4.) The wash upstream of the bridge is shallow; flow is probably in the form of sheet flow. Downstream of the bridge the wash shows more definition, although still shallow.

LAND USE: Land use in the vicinity of the bridge is primarily undeveloped range. Although urbanization is proceeding northward along US 60, the close proximity of the Northwest Regional Landfill will probably inhibit commercial or residential development near the bridge.

There is no evidence of sand and gravel extraction in the vicinity of the bridge.

SURFACE SOILS: Surface soils consist primarily of sand and fine gravel with occasional cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 0.5 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The estimated slope of the wash in the vicinity of the Deer Valley Road Bridge is 0.0056 ft/ft, or approximately 29.5' per mile. The slope was estimated using U.S. Geological Survey (USGS) 7.5 minute topographic maps.

VEGETATION: Vegetation includes trees such as palo verde and mesquite; bushes include desert broom, creosote, brittle bush, ephedra and saltbush. Dry grasses are also present.

Trees occur in sparse to moderate density along natural washes. Desert broom grows primarily in and along riprap; other bushes are uniformly distributed with moderate density. Dry grasses occur outside the washes with moderate density.

STREAM STABILITY: Lateral stability of the stream at the bridge is maintained by the riprap-lined spur dikes and roadway embankment; there are no structural constraints on lateral migration of the natural wash other than occasional trees and vegetation along the banks.

There are no grade control structures either upstream or downstream of the Deer Valley Road Bridge. It is more likely that the long-term tendency of the stream is to degrade rather than aggrade.

CURRENT HYDROLOGY: Natural drainage in the area of the bridge flows from northwest to southeast in numerous washes that are generally shallow and poorly defined. The drainage channel on the north side of the road intercepts approximately one mile of drainage that formerly crossed the alignment of Deer Valley Road. The drainage channel created by the roadway embankment intercepts a significant quantity of water that would otherwise not reach the bridge site.

Available plans, flow records, and hydrologic models provided the following information:

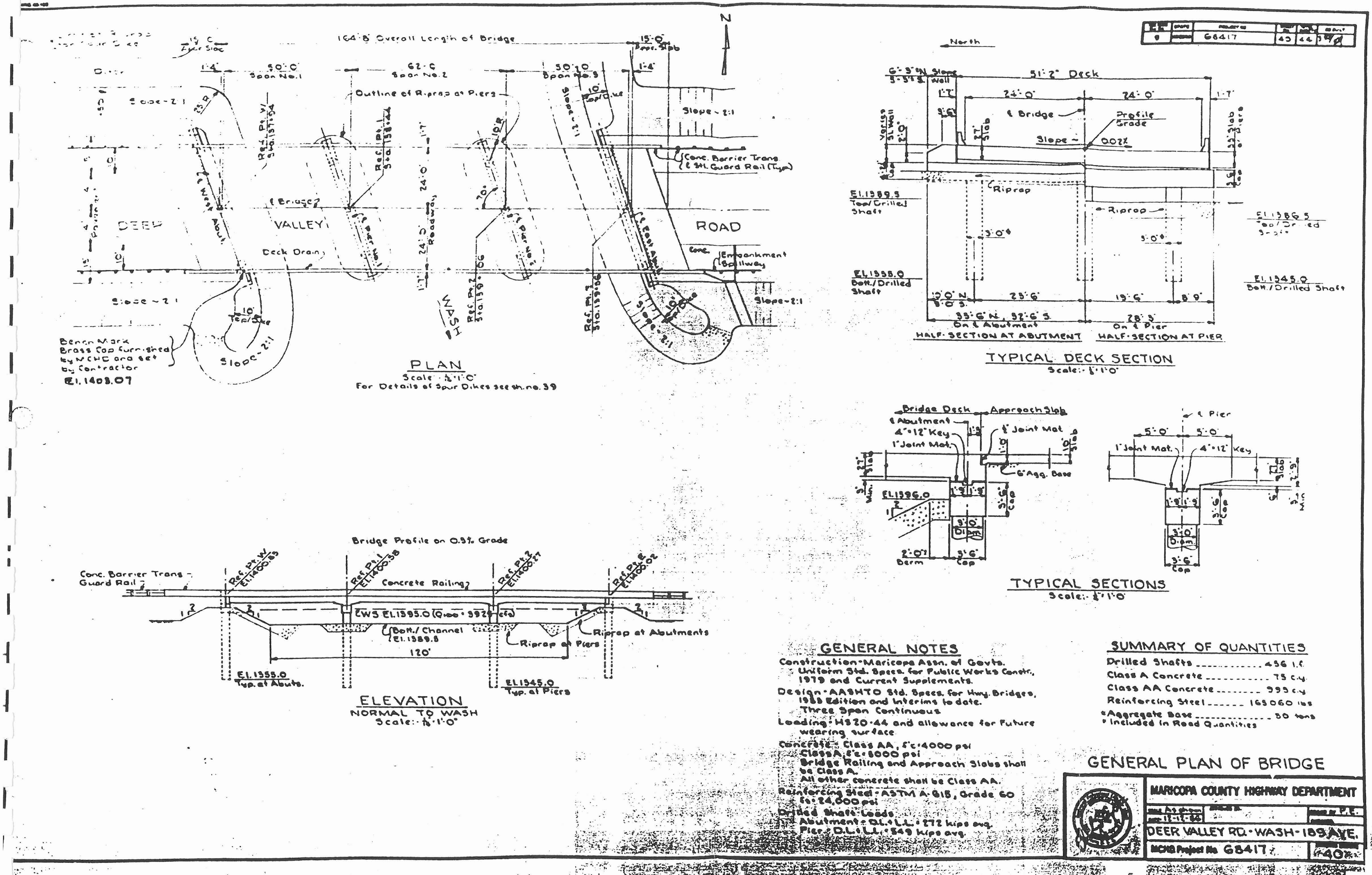

PLAN
Scale: 1/8"=1'-0"
For Details of Spur Dikes see sh. no. 39
164'-8" Overall Length of Bridge
50'-0" Span No. 1
62'-0" Span No. 2
50'-0" Span No. 3
DEER
VALLEY
ROAD
WASH
Outline of Riprap at Piers
Conc. Barrier Trans & Stl. Guard Rail (Typ)
Embankment Spillway
Deck Drain
Bench Mark Brass Cap furnished by MCHD and set by Contractor
El. 1403.07
TYPICAL DECK SECTION
Scale: 1/4"=1'-0"
51'-2" Deck
HALF-SECTION AT ABUTMENT
HALF-SECTION AT PIER
TYPICAL SECTIONS
Scale: 1/4"=1'-0"
Bridge Deck
Approach Slab
Bridge Profile on 0.5% Grade
Concrete Railing
Conc. Barrier Trans - Guard Rail
Riprap at Abutments
Riprap at Piers
120'
ELEVATION
NORMAL TO WASH
Scale: 1/16"=1'-0"
GENERAL NOTES
Construction - Maricopa Assn. of Govts. Uniform Std. Specs. for Public Works Constr., 1979 and Current Supplements.
Design - AASHTO Std. Specs. for Hwy. Bridges, 1983 Edition and Interims to date. Three Span Continuous
Loading - HS20-44 and allowance for Future wearing surface
Concrete: Class AA, f'c=4000 psi
Class A, f'c=3000 psi
Bridge Railing and Approach Slabs shall be Class A.
All other concrete shall be Class AA.
Reinforcing Steel - ASTM A-615, Grade 60 fs=24,000 psi
Drilled Shaft Loads
Abutment = D.L.+L.L. = 272 kips avg.
Pier = D.L.+L.L. = 549 kips avg.
SUMMARY OF QUANTITIES
Drilled Shafts .... 456 l.f.
Class A Concrete .... 75 c.y.
Class AA Concrete .... 999 c.y.
Reinforcing Steel ...... 165060 lbs
*Aggregate Base .... 80 tons
* Included in Road Quantities
GENERAL PLAN OF BRIDGE
MARICOPA COUNTY HIGHWAY DEPARTMENT
DEER VALLEY RD. - WASH - 189 AVE.
MCHD Project No. 68417
40

Figure 3

DEER VALLEY ROAD near 189th AVE. (SN 7553)

Water Course	Unnamed Wash near 189th Avenue
Stream Form	Braided (channel poorly defined US; secondary channel created by r/w embankment from west).
Sinuosity	Not Applicable
General Channelization	None; spur dike installed along US east abutment.
Channel Slope	(US) channel defined; (DS) uniform.
Estimated Channel Slope (ft/ft)	0.004804
Channel Contraction/Expansion	(DS) channel defined; (US) shallow channel flow to sheet wash.
Primary Surface Sediment Type	sand/gravel
D50 Size	5.0 MM
Armoring Potential	Low
Channel Vegetation	
Type/Size	Palo Verde to 10 ft., Mesquite to 8 ft., Desert Broom to 5 ft., Creosote to 5 ft., Brittle Bush, Salt Bush, Ephedra to 2 ft.; dry grasses.
Density/Occurrence	Trees sparse to moderate along channels; Desert Broom along rip-rap; others uniformly distributed with moderate density; dry grasses occur outside channels with moderate density.
Relative Age	Mature
Manning's Roughness Coef.	0.050
Controls on Stream Migration	
Lateral	None, spur dikes/roadway embankment guide flow near structure.
Vertical	None
Sediment Deposits & Bars	(US) No bar development (DS) Very low point and alternate bars occurring as of structure in shallow channel.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	Low flow channel has undercut and destabilized rip-rap along west side of pier 2.
Abutment	Channel paralleling r/w embankment has potential to destabilize rip-rap along north side of west abutment.
Land Use	
Urbanization of Upstream Watershed	Low rate; land generally undeveloped range.
Sand & Gravel Extraction	No
Freeway Construction	No
Dams	No
Drainage Channels	Channel created by r/w embankment.

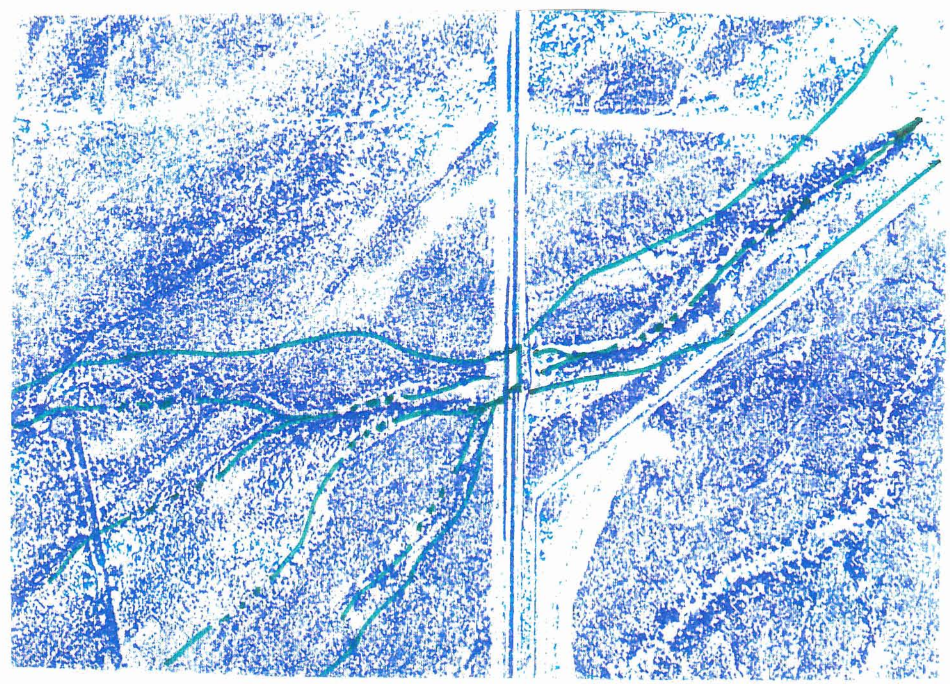


Figure 4

1. According to the plans, the design flow is 3,925 cfs and the design flood frequency is 100 years.

2. There is no USGS data available for this wash.

3. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge of 2,300 cfs. This study was completed prior to construction of Deer Valley Road and the bridge and does not reflect flow added by the drainage channel along the roadway embankment.

4. The 500-year flood (superflood) is not reported on Federal Emergency Management Agency (FEMA) flood insurance study maps. USGS regression equations were used to estimate a 500-year flood of 8,600 cfs.

Generally, flows taken from published FEMA flood insurance studies (FIS) were given priority over other sources because of the substantial level of effort and review involved in their estimation. Although values for the more frequent recurrence intervals were included in the analysis for completeness, the critical discharge values were considered to be the 100-year flow and the lesser of the 500-year flow and the flow at the low chord elevation, based on HEC-18 criteria and MCDOT requirements.

FLOW MODELING AND CALCULATION OF CRITICAL FLOW: In accordance with MCDOT requirements, the critical flow for use in scour calculations is the lesser of the 500-year flow and the flow that just reaches the low chord elevation of the bridge. In order to determine the controlling flow rate, a stage-discharge curve at a cross-section at the bridge was prepared using the Manning equation for uniform flow. The upstream approach channel was subdivided based on channel roughness and morphology, and a portion of the total flow was estimated for each subdivision. An iterative process was then used to balance water surface elevation and total discharge in the cross section. Flows were also classified as channel or overbank for the purpose of estimating flow contraction and abutment scour. Values for the energy slope and Manning's roughness coefficient used in the analysis were taken as averages of suitable upstream and downstream sections from HEC-2 modeling studies of the 100-year discharge case provided by FCDMC. The points on the stage-discharge curve generated by the modeling are summarized in Table 1.

Table 1. Stage-Discharge Curve

Stage	Discharge, cfs	Description
1390.87	785	Q_{10}
1393.25	3,000	-
1393.96	3,925	Q_{100}
1396.72	8,600	Q_{500}
1397.13	9,420	Low Chord

The lesser of Q_{500} and the low chord flow (Q_{LC}) is to be used to calculate scour during the critical event. The critical flow for scour calculations is therefore 8,600 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{100} and the maximum flood (Q_{500}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Because little numeric data is available regarding long-term channel grade changes, total scour depths include 4' of long-term degradation or general scour, for natural channel conditions without adjustment for downstream grade controls or potential for armoring, which in the case of the Deer Valley Road Bridge, is considered to be low. Scour calculations also adjust actual pier dimensions to allow for debris accumulation. The angle of attack was estimated as the difference between the approach angle of flow and the skew angle of the bridge. Results of the scour calculations and a summary of pile embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{500} is shown in Figure 5. Scour calculations are presented in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 3,925 cfs (Q_{100})	Q = 8,600 cfs (Q_{500})
1.	Scour at Piers		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	12.8	15.1
	General Scour, ft	4.0	4.0
	Total Scour, ft	**16.8**	**19.1**
2.	Scour at Abutments		
	Abutment Scour, ft	0.0	0.0
	General Scour, ft	4.0	4.0
	Total Scour, ft	**4.0**	**4.0**

Table 3. Summary of Drilled Shaft Embedment

		Q = 3,925 cfs (Q_{100})	Q = 8,600 cfs (Q_{500})
1.	Embedment at Piers		
	Channel Elevation	1387.0	1387.0
	Total Scour, ft	16.8	19.1
	Bottom of Scour Hole Elev.	1370.2	1367.9
	Drilled Shaft Tip Elev.	1345.0	1345.0
	Embedment Remaining, ft	**25.2**	**22.9**

2. Embedment at Abutments

Channel Elevation	1387.0	1387.0
Total Scour, ft	4.0	4.0
Bottom of Scour Hole Elev.	1383.0	1383.0
Drilled Shaft Tip Elev.	1355.0	1355.0
Embedment Remaining, ft	**28.0**	**28.0**

STRUCTURAL EVALUATION: A structural analysis of the bridge piers for the Q_{100} flood event was performed using a stiffness method of analysis that accounted for soil-structure interaction. The analysis included dead loads, live loads, and stream flow forces. The structural capacity of the concrete columns and drilled shafts, as well as the capacity of the soil, was evaluated. A separate analysis of the abutments was not warranted as they are similar in construction to the piers, yet their loadings are considerably less. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, the Deer Valley Road Bridge at Queen Creek does not have sufficient structural capacity to resist the loads resulting from 100-year or 500-year flow rates. The bridge is scour *critical*.

DEFICIENCIES AND COUNTERMEASURES:

Scour-related deficiencies include the following:

a. Insufficient embedment of the drilled shafts at the piers to support vertical dead and live loads with scour produced by the 100-year and 500-year floods;

b. Degradation of the channel along the north roadway embankment, with possible undermining of dumped riprap on the embankment;

c. Undermining and destabilization of riprap around the piers.

Countermeasures to remedy scour-related deficiencies include the following:

a. Install scour monitoring devices and close the bridge to traffic if scour reaches a predetermined critical depth;

b. Construct a continuous concrete or grouted riprap sill across the width of the channel, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

c. Encase the piers in a reinforced concrete beam supported on drilled shaft foundations (underpinning);

d. Remove the bridge and construct a new bridge on deeper foundations.

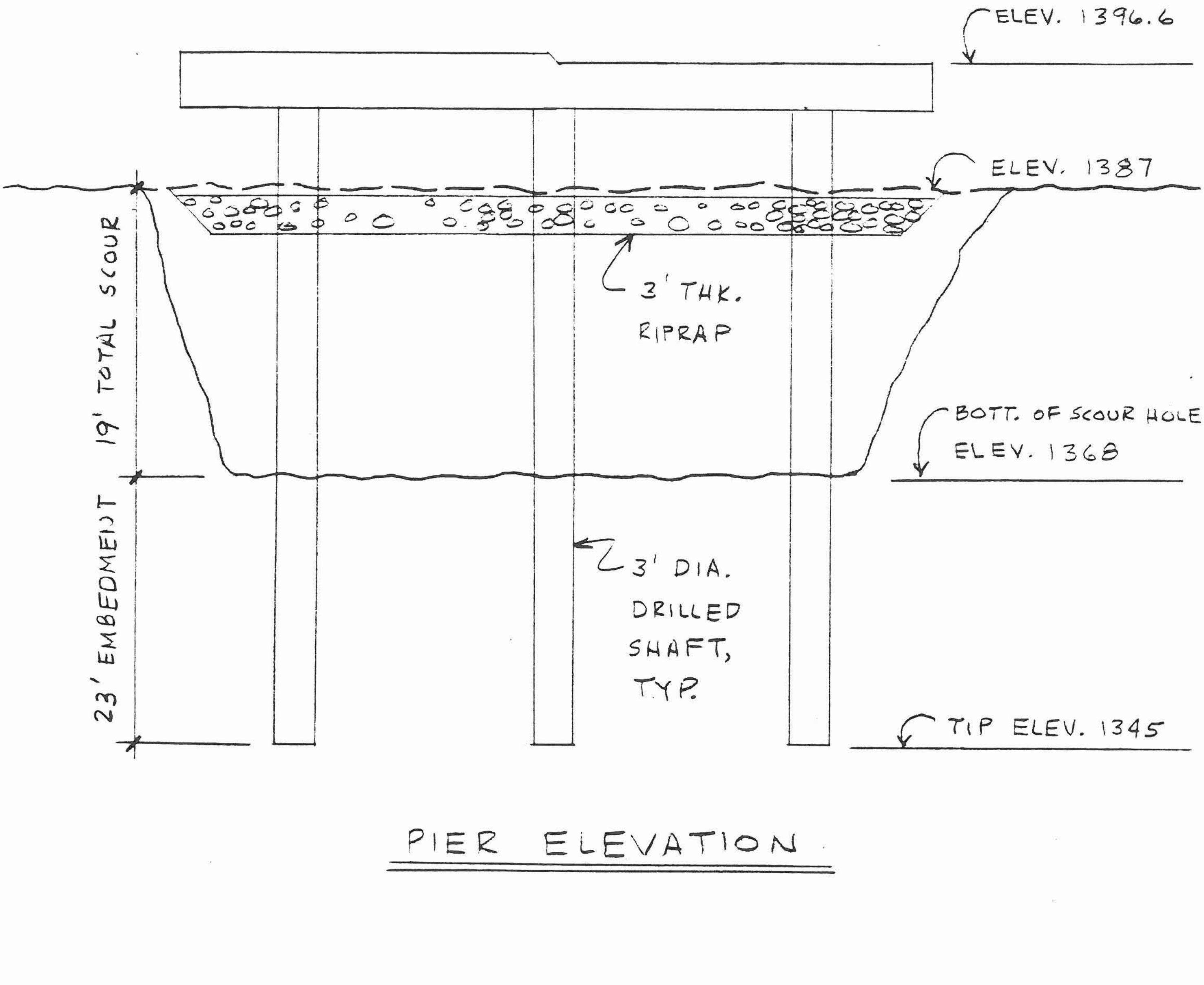
DEER VALLEY ROAD BRIDGE OVER UNNAMED WASH

HYDRAULIC DATA (PER MM/CSSA):

Q_{500} = 8600 CFS

H.W. ELEV. = 1396.72

TOTAL SCOUR = 19'



PIER ELEVATION

Figure 5

Photo 1: View looking approximately northwest in upstream direction from bridge deck. Note that wash is not well defined immediately upstream of bridge. Also note incision made by flow intercepted by west side roadway embankment. In background note upstream right overbank characterized by low grasses, creosote and salt bush, and occasional mesquite in a sandy soil.

Photo 2: View looking approximately northeast in upstream direction from bridge deck. Note spur dike and trash dumped in foreground. In background note upstream left overbank with levee formed by roadway embankment and vegetation similar to that observed in Photo 1.

Photo 3: View looking east along upstream roadway embankment west of structure. Note 2-3 foot incision made by concentrated flow and potential for destabilization of embankment under severe flow conditions.

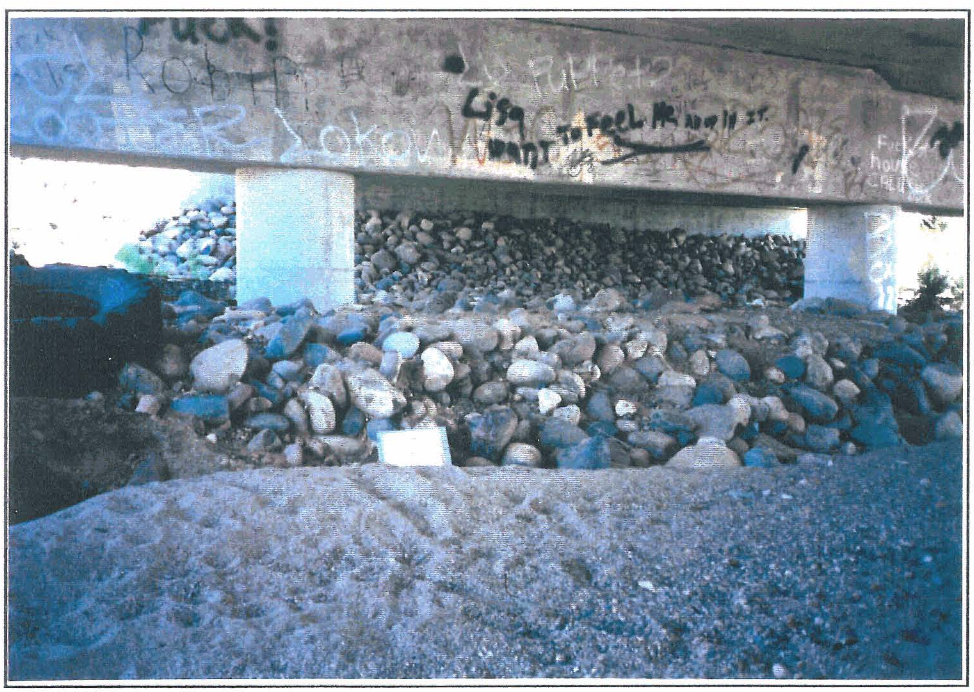


Photo 4: View of east side pier. Note undercutting and destabilization of rip-rap placed along shafts.

Photo 5: View looking south downstream of bridge. Note shallow cross-section with channel bottom composed of gravelly sand. Note low grasses, creosote and salt bush, palo verde and mesquite adjacent to channel.

Is this page missing some information?

- Ensure that roadway alignment geometry is firmly established, and initiate final superstructure widening studies (probably not more than a verification) and final superstructure design.

- Commence with the field geotechnical exploration. While the field investigation may be primarily a verification program, it is important to have the Bridge Foundation Report approved by MCDOT, and the Team-developed foundation systems and recommended foundation type accepted by MCDOT before completion of pier widening studies and final design of the pier and any associated scour countermeasure features.

Based on the foregoing concurrent Design Phase activities, the development of Project P.S.&E. documents will be orderly. The overall Project will be developed in accordance with the MCDOT Roadway Design Manual, November 3, 1993, revised to date; the Design Phase will be developed in accordance with Chapter 4, Design Procedure.

Throughout the Design Phase, Bridge Plans and any associated scour countermeasure plans and/or grade control structure alterations will be developed in general conformance with Section 4.8 Bridge Design of the Design Manual. Based on Project Scope of Work, assuming the Design Phase starts at 15% completion, and requiring submittals at 40%, 70%, 90% and 100% plan completions, the CA Team envisions the bridge plan submittals as follows:

- 40% - MCDOT Preliminary Plans submittal plus all bridge plan sheets included (but in varying stages of development).

- 70% - Development of all plan sheets well along, with approximately half the sheets complete or nearly complete. Completed/advance sheets will include Location Plan, General Bridge Plan(s) (except quantities), Foundation Plan, Boring Logs, all superstructure & approach slab details, and screed elevations.

- 90% - Approximates MCDOT PreFinal Plans, but is a 90% submittal: Detail plans will be 90% or better, most plan sheets will be complete, design calculations will be complete and checked, and preliminary draft special provisions will be in the submittal.

- 100% - Complies with MCDOT Final Plans.

The schedule included in this proposal leaves a short block of time (1 1/2 weeks + or -) to accommodate any revisions required by MCDOT final review comments.