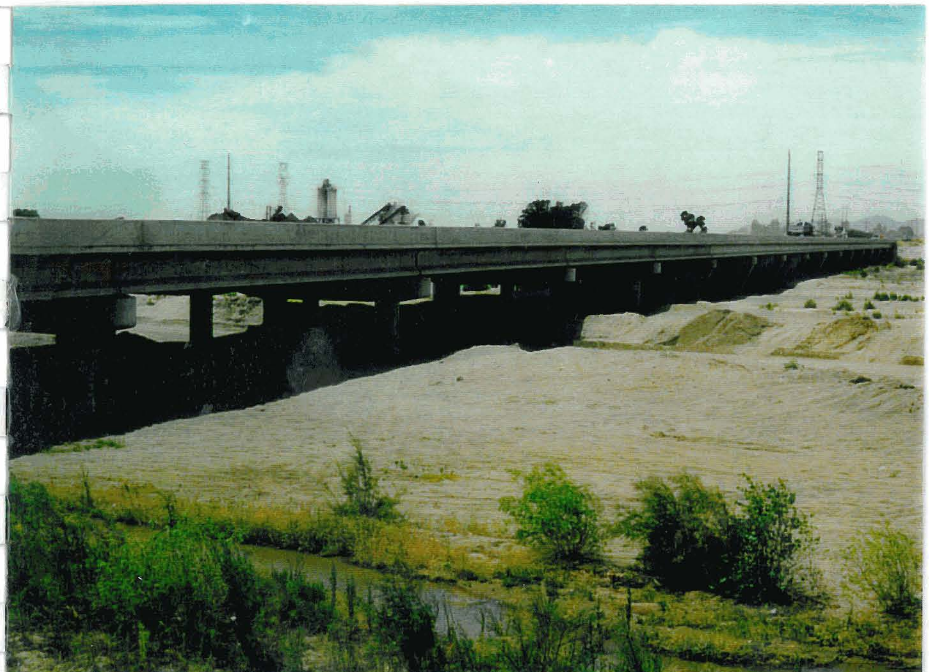

Final
BRIDGE SCOUR ASSESSMENT REPORTS
for 16 Maricopa County Bridges

VOLUME I
Structure Numbers
9691, 8981, 9301, 9859
9145, 7819, 8028, 8639

July, 1996

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

Registered Professional Engineer (Structural)
CERTIFICATE NO.
9161
JERRY A.
CANNON
Date Signed 7/9/96
ARIZONA, U.S.A.

Cannon & Associates, Inc.
Consulting Engineers

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CA 94046-1

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July, 1996

Prepared for
MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION (MCDOT)
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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

EXECUTIVE SUMMARY

INTRODUCTION

Bridge 1: Bell Road Bridge over Agua Fria River
Structure #9691

Bridge 2: Olive Avenue Bridge over Agua Fria River
Structure #8981

Bridge 3: Glendale Avenue Bridge over Agua Fria River
Structure #9301

Bridge 4: Camelback Road Bridge over Agua Fria River
Structure #9859

Bridge 5: Indian School Road Bridge over Agua Fria River
Structure #9145

Bridge 6: Maricopa County Highway 85 Bridge over Agua Fria River
Structure #7819

Bridge 7: New River Road Bridge over New River
Structure #8028

Bridge 8: I-17 Frontage Road Bridge over New River
Structure #8639

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 if the grouted riprap is 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 OK Please CHK
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:

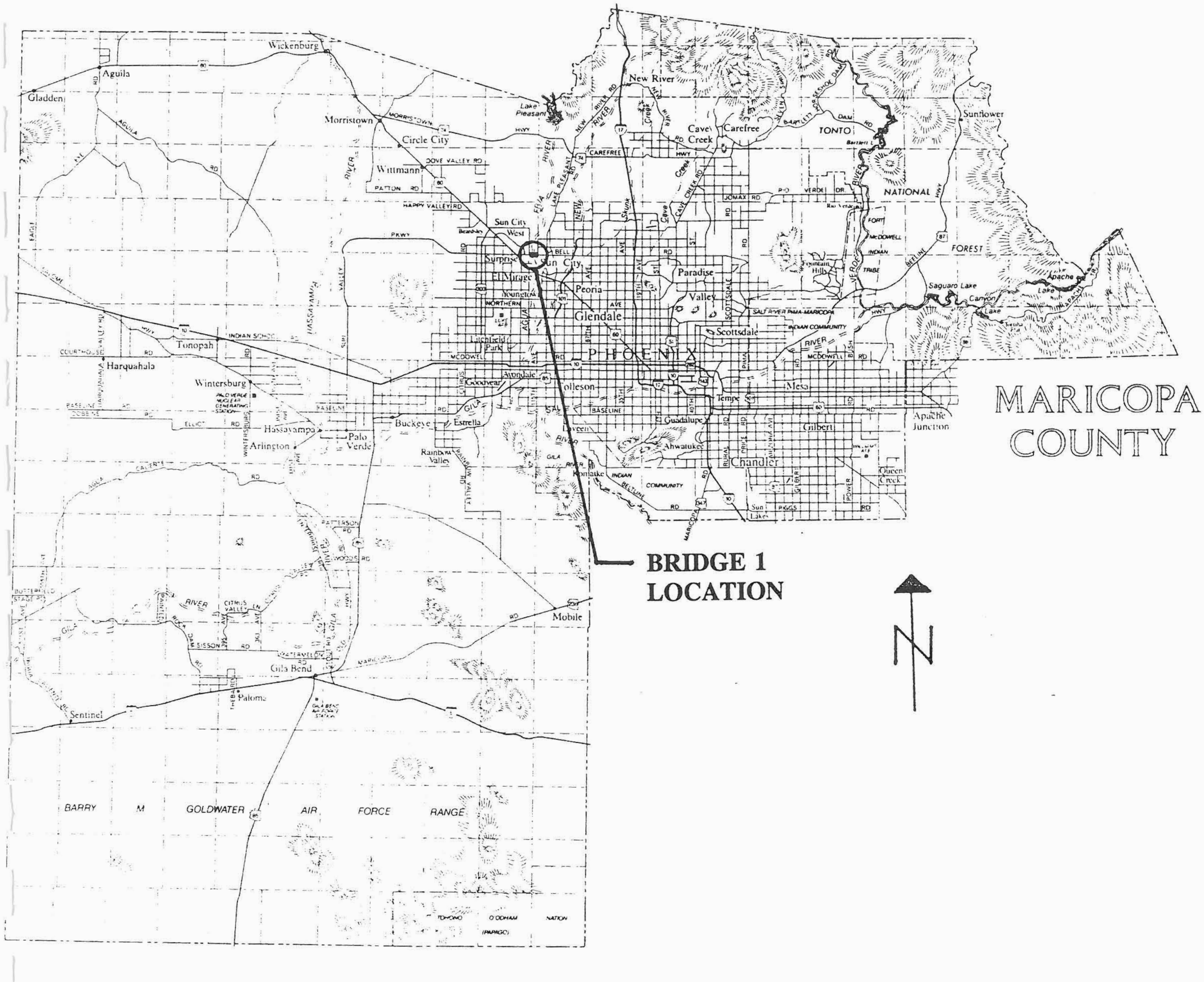
Scour Stable: Scour stable bridges are considered safe from catastrophic failure due to scour or erosion associated with a determinant discharge referred to as the evaluation discharge.

Scour Critical: 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 1

BELL ROAD BRIDGE OVER AGUA FRIA RIVER



Location Map

Figure 1

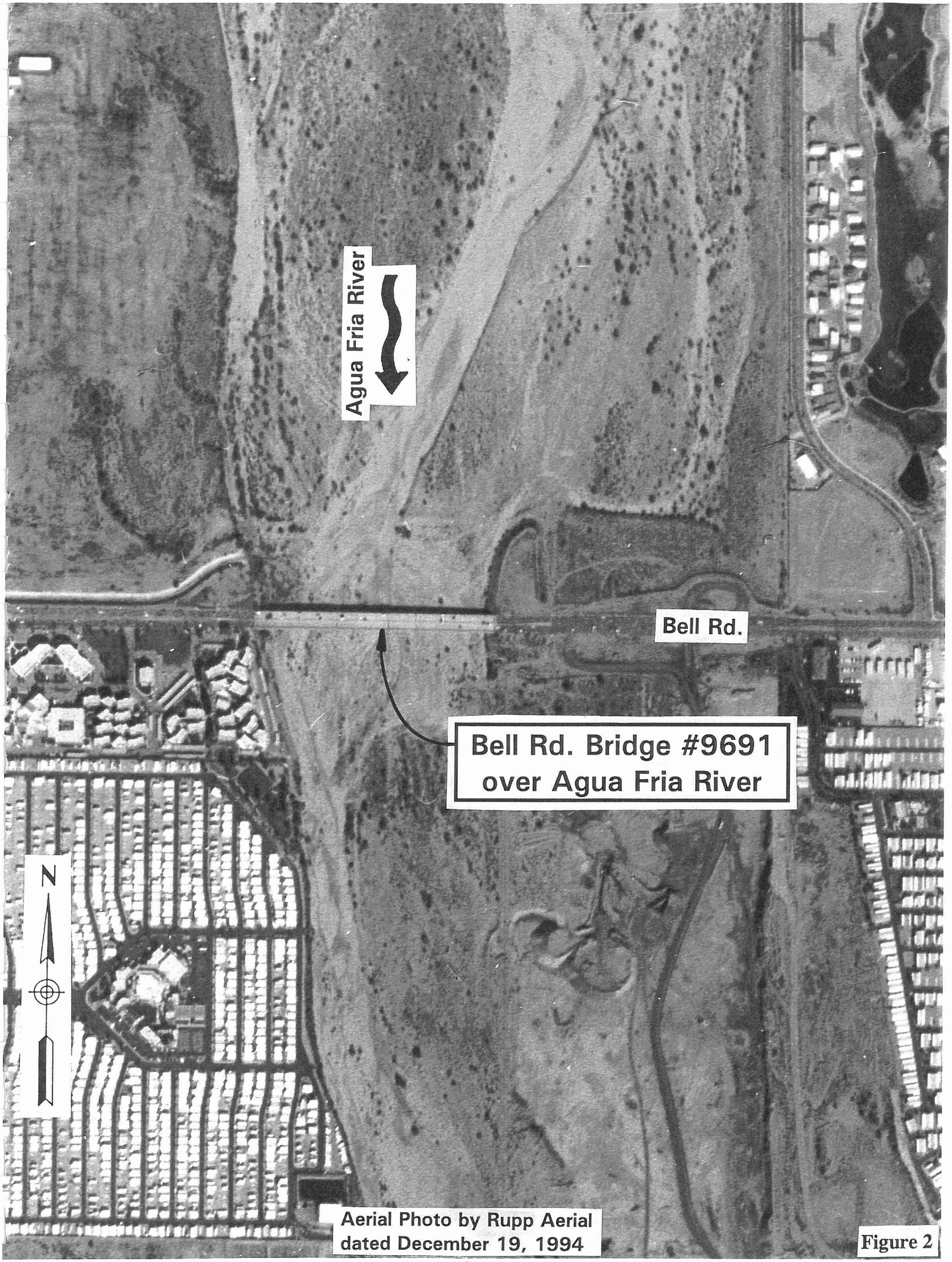

Agua Fria River
Bell Rd.
Bell Rd. Bridge #9691
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 1: BELL ROAD BRIDGE OVER AGUA FRIA RIVER (Structure #9691)

Assessment: Scour Stable

LOCATION: The Bell Road Bridge at the Agua Fria River lies on the section line between Section 1, T3N, R1W, and Section 36, T4N, R1W, Gila and Salt River Baseline and Meridian, coincident with the boundary line between Sun City West and the Town of Surprise, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The bridge has nine spans with a total length of 1,105' from center-to-center of abutment bearings. (See Location Plan, Figure 3.) It has two 36-foot-wide roadways separated by a concrete median barrier and a sidewalk on the downstream side. The bridge was designed for a stream flow rate of 83,000 cubic feet per second (cfs), corresponding to a flood frequency interval of 50 years. The bridge was designed by Benson & Gerdin Consulting Engineers in 1981; it was built in 1982 as MCDOT Project No. 68067.

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck. Expansion joints are provided at the abutments and at Pier Numbers 3 and 6.

The abutments consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on three 5'-0" diameter drilled shafts. The drilled shafts are founded approximately 68' below the exiting river flow line. Both abutments have zero skew with respect to the bridge centerline.

The piers consist of a concrete cap supported on three 5'-0" diameter formed columns. The formed columns extend approximately 11' below the existing river flow line and are supported on three 6'-0" diameter drilled shafts that are founded approximately 68' below the flow line. The piers have zero skew with respect to the bridge centerline.

EXISTING SCOUR PROTECTION: Scour protection at the east abutment consists of a gabion-lined elliptical guide bank extending approximately 470' upstream of the bridge centerline. The west abutment is protected by gabions along the west bank from a point approximately 300' upstream to a point approximately 600' downstream of the bridge centerline. The upstream end of the scour protection coincides with the outlet of a large drainage channel conveying stormwater from Sun City West. According to as-built plans, the gabions extend a minimum of 6' below the stream bed and approximately 33' out into the channel beyond the toe of the bank. Piers are also protected by gabions in the form of a 102' x 30' rectangle installed at a depth of 12' below grade. All gabions used for scour protection are 18" thick.

A site inspection showed that the gabions lining the guide bank and the west bank are in good condition, except at the outlet at the Sun City West drainage channel where undermining has caused failure of several gabions. Also, residual scour holes between 1' and 2' deep were noted around the drilled shaft columns. This may indicate that deeper scour holes were formed during past flows and that insufficient material was transported into the holes as the flow receded to completely fill them. Bridge inspection reports from between 1982 and 1994 indicate no significant scour problems.

STREAM FORM: The Agua Fria River at Bell Road is a braided stream with shallow to medium height point bars and middle bars distributed irregularly throughout the river bed. (See Figure 4.) There are two principal low flow channels--a large one flowing in a south-by-southwesterly direction near the center of the river bed and a smaller one near the west bank. The channels join immediately downstream of the bridge.

LAND USE: Land use in the surrounding area is primarily residential, with some commercial. Large gravel mining operations are located in or along the Agua Fria, approximately 2 to 4 miles upstream of the bridge and 2 miles downstream, near Grand Avenue. There is also some gravel mining in the east overbank of the river, downstream of the bridge.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest source of runoff will continue to be releases from the New Waddell Dam, approximately 15 miles upstream of the bridge.

SURFACE SOILS Surface soils consist primarily of sand and gravel, with some cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 3 mm. The armoring potential of the river bed is estimated to be low.

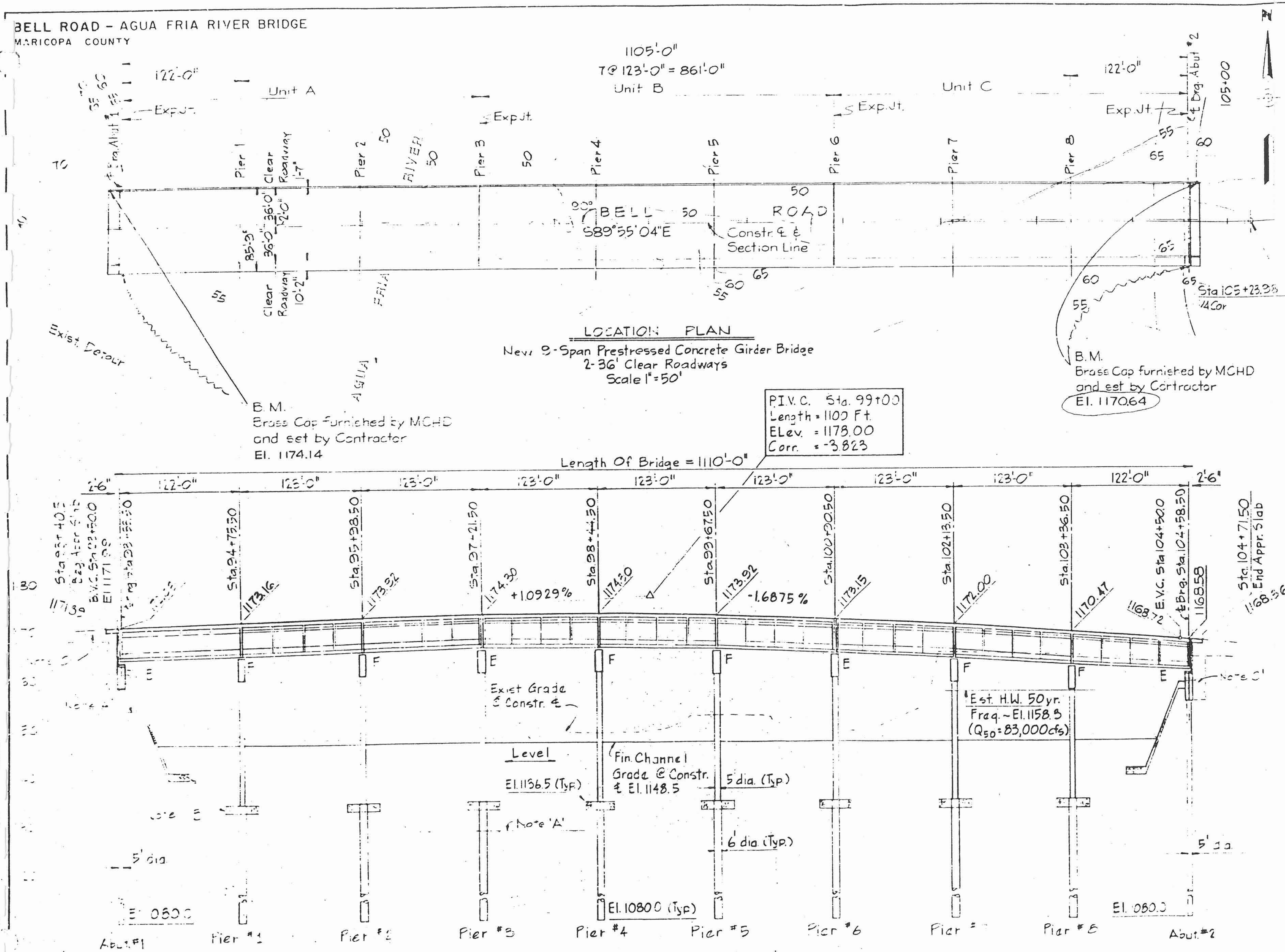
SLOPE: The slope of the river bed in the vicinity of the Bell Road Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0032 ft/ft or approximately 17' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River includes trees such as palo verde, mesquite, and ironwood; bushes and shrubs such as desert broom, creosote, ephedra, salt bush, and brittle bush; and low grasses. The larger trees occur with low to moderate density on established sand and gravel bars, and bushes and shrubs occur in stand of moderate density. The overall density of vegetation in the river is estimated to be low to moderate. Debris collecting on the piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Overall lateral stability of the river at the bridge is provided by wire-tied gabions on the banks for short distances upstream and downstream of the bridge. Berms constructed as part of residential developments along the river banks also serve to maintain the overall lateral stability of the river. Although aerial photographs showed that the low-flow channel configuration has not changed appreciably since 1982, it must be assumed that low flow channels may move laterally when water is flowing in the river.

There are no upstream or downstream grade control structures in the immediate area that would maintain vertical stability of the river bed at the bridge site. The gravel mining operations described previously are thought to have insignificant effect on channel elevations because they are either a significant distance away or are located in the overbanks. The long-term tendency of the stream bed is likely to degrade rather than aggrade, although no measurable degradation is observed at the bridge.

BELL ROAD - AGUA FRIA RIVER BRIDGE
MARICOPA COUNTY



SECTION ON CONSTRUCTION ℄
Scale: 1"=50' Horizontal, 1"=10' Vertical

NOTE C: Embankment beneath approach slabs shall be Select Material.
NOTE D: For Material Quantities see sht. 2 of 57

NOTE: See Sheet 6 of 24 for Minimum Lap Lengths For Reinf. Bars.

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ	6 8067	14	57	2-1-82

BENSON & GERDIN
CONSULTING ENGINEERS
PHOENIX, ARIZONA

DES MK | DR BB | CK MK | DATE 1-81

GENERAL NOTES

Construction:
Maricopa Association of Government's Uniform Standard Specifications for Public Works Construction, 1979 Edition, including all Supplements, and Standard Specifications, Arizona Department of Transportation, 1969 Edition, revised to date.

Design:
A.A.S.H.T.O. Standard Specifications for Highway Bridges, 1977 Edition, revised to date.

Loading Class: HS 20-44.

Allowable Stresses:

Class "B" Concrete -	f'c = 2500 psi ; n=10
Class "A" Concrete -	f'c = 3000 psi ; n=9
Class "AA" Concrete -	f'c = 4000 psi ; n=8
Class "S" Concrete -	f'c = 5700 psi ; n=7
Reinforcing Steel -	fs = 24000 psi
Structural Steel -	fs = 20000 psi

Reinforcing Steel shall conform to A.S.T.M. Spec. A615, Grade 60.

Structural Steel shall conform to A.S.T.M. Spec. A36.

Prestressing Steel shall be Type 270K, 7 wire, uncoated, stress-relieved strand. Min. ultimate strength for ½ ϕ strand ($A = 0.153\ in.^2$) = 41,300# conforming to A.S.T.M. Spec. A416.

All welding shall conform to the requirements of The American Welding Society Structural Welding Code D1.1-75, revised to date.

All dimensions for reinforcing steel shall be measured from the surface of the concrete to the center of the bars unless otherwise shown on the plans.

All reinforcing steel shall have 2" clearance unless otherwise noted.

All bend dimensions for reinforcing steel to be out-to-out unless otherwise noted on the plans.

Paint and Painting shall conform to Standard Specifications. Contact surfaces shall not be painted. Shop Paint - one coat #1A or #1B, #8 or #9 and #10. No field paint required.

Chamfer all exposed edges and corners ¾" unless otherwise noted on the plans.

Dimensions shall not be scaled from the drawings.

NOTE A: Existing Bridge Structure removed to Elev. [illegible], except exist. Abut. #1 Piles removed to Elev. 1150.0, all under separate contract.

NOTE B: For Typical Bank Protection At Piers & Abut. See Shts. 8 Thru 11 of 57.

PRESTRESSED CONCRETE ALTERNATE

MARICOPA COUNTY HIGHWAY DEPARTMENT
STA. 99+ BELL ROAD BRIDGE OVER AGUA FRIA RIVER

LOCATION PLAN | SHEET NO. 1 | TOTAL SHEETS 24

Figure 3

BELL ROAD (SN 9691)

Water Course	Agua Fria River
Stream Form	Braided
Sinuosity	Not applicable
General Channelization	East side has gabion spur dike US/DS to guide flow near structure. West side channel has gabion-lined banks for short distances US/DS.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.002544
Channel Contraction/Expansion	Wider US
Primary Surface Sediment Type	sand/gravel
D50 Size	3 MM
Armoring Potential	Low
Channel Vegetation	
Type/Size	Palo Verde to 10 ft., Mesquite, Ironwood; Desert Broom to 6 ft., Desert Willow, Creosote; Ephedra, Salt Bush, Brittle Bush; low grasses.
Density/Occurrence	Larger trees on established bars with low to moderate density; smaller shrubs low to moderate density in stands; smaller vegetation low density.
Relative Age	Mature
Manning's Roughness Coef.	0.030
Controls on Stream Migration	
Lateral	Limited; west side channel has gabion-lined banks for short distances US/DS.
Vertical	None
Sediment Deposits & Bars	Irregular distribution of shallow to moderate height point and middle bars.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Land use commercial, high density residential, industrial; general assumption is for increasing urbanization.
Sand & Gravel Extraction	Significant gravel mining operation in DS LOB
Freeway Construction	No, but general roadway improvements are likely in vicinity.
Dams	New waddel controls outflows from Lake Pleasant; McMicken Dam controls tail by wash watershed to Agua Fria through outfall channel.
Drainage Channels	Sun City West drainage channel outfall just US of west abutment.

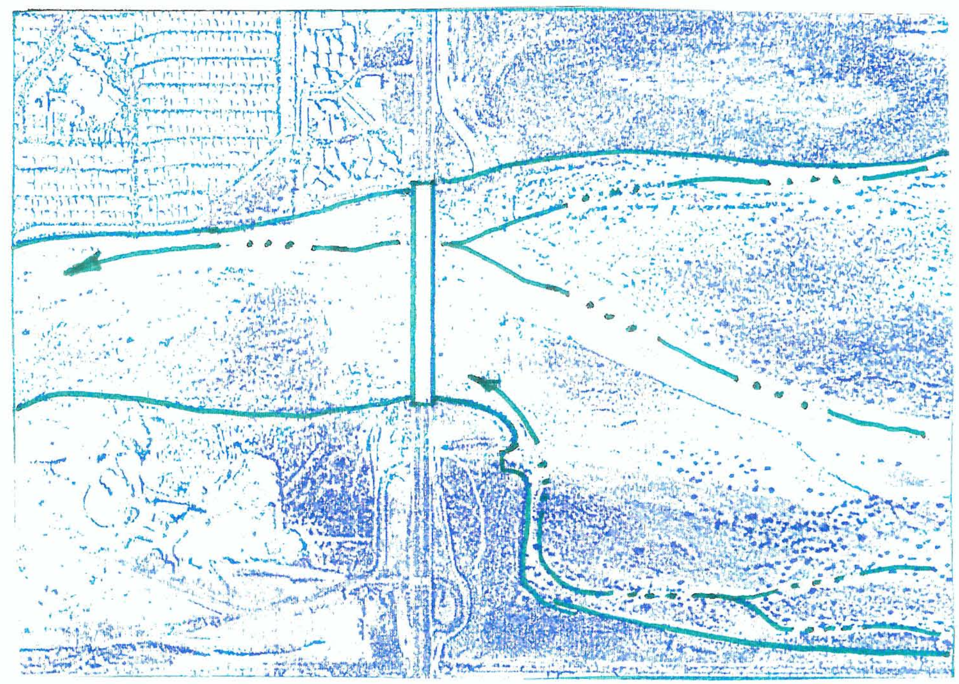


Figure 4

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Bell Road come from controlled releases from the New Waddell Dam at Lake Pleasant, approximately 15 miles upstream of the bridge. Detention basins along Trilby Wash at the base of the White Tank Mountains also discharge to the Agua Fria River approximately 5 miles upstream of the bridge via the McMicken Dam Outlet Channel. Smaller flows come from off-site developed and undeveloped land between Waddell Dam and the bridge.

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

a. The flow and flood frequency used in the design of the bridge are 83,000 cfs and 50 years, respectively.

b. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978 as measured at El Mirage, Arizona at Grand Avenue, approximately 2 miles downstream of the bridge.

c. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge at 115,000 cfs.

d. The 500-year flood (superflood) reported by the Federal Emergency Management Agency (FEMA) is 182,000 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 MAXIMUM FLOW: In accordance with MCDOT requirements, the critical flow for use in scour evaluations 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
1153.88	23,000	Q_{10}
1158.37	87,000	Q_{50}
1159.84	115,000	Q_{100}
1161.29	145,600	Low Chord
1162.85	182,000	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 145,600 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 Bell 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_{LC} is shown in Figure 5. Scour calculations are presented in full in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 115,000 cfs (Q_{100})	Q = 145,600 cfs (Q_{LC})
a.	Scour at Piers		
	Contraction Scour, ft	0.2	0.3
	Local Scour, ft	17.3	18.2
	General Scour, ft	4.0	4.0
	Total Scour, ft	**21.5**	**22.5**
b.	Scour at Abutments		
	Abutment Scour, ft	12.1	14.1
	General Scour, ft	4.0	4.0
	Total Scour, ft	**16.1**	**18.1**

Table 3. Summary of Drilled Shaft Embedment

		Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{LC})
a.	Embedment at Piers		
	Channel Elevation	1147.8	1147.8
	Total Scour, ft	21.5	22.5
	Elev. Bottom of Scour Hole	1126.3	1125.3
	Drilled Shaft Tip Ele.	1080.0	1080.0
	Embedment Remaining, ft	**46.3**	**45.3**
b.	Embedment at Abutments		
	Channel Elevation	1150.7	1150.7
	Total Scour, ft	16.1	18.1
	Elev. Bottom of Scour Hole	1134.6	1132.6
	Drilled Shaft Tip Elev.	1180.0	1080.0
	Embedment Remaining, ft	**54.6**	**52.6**

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.*

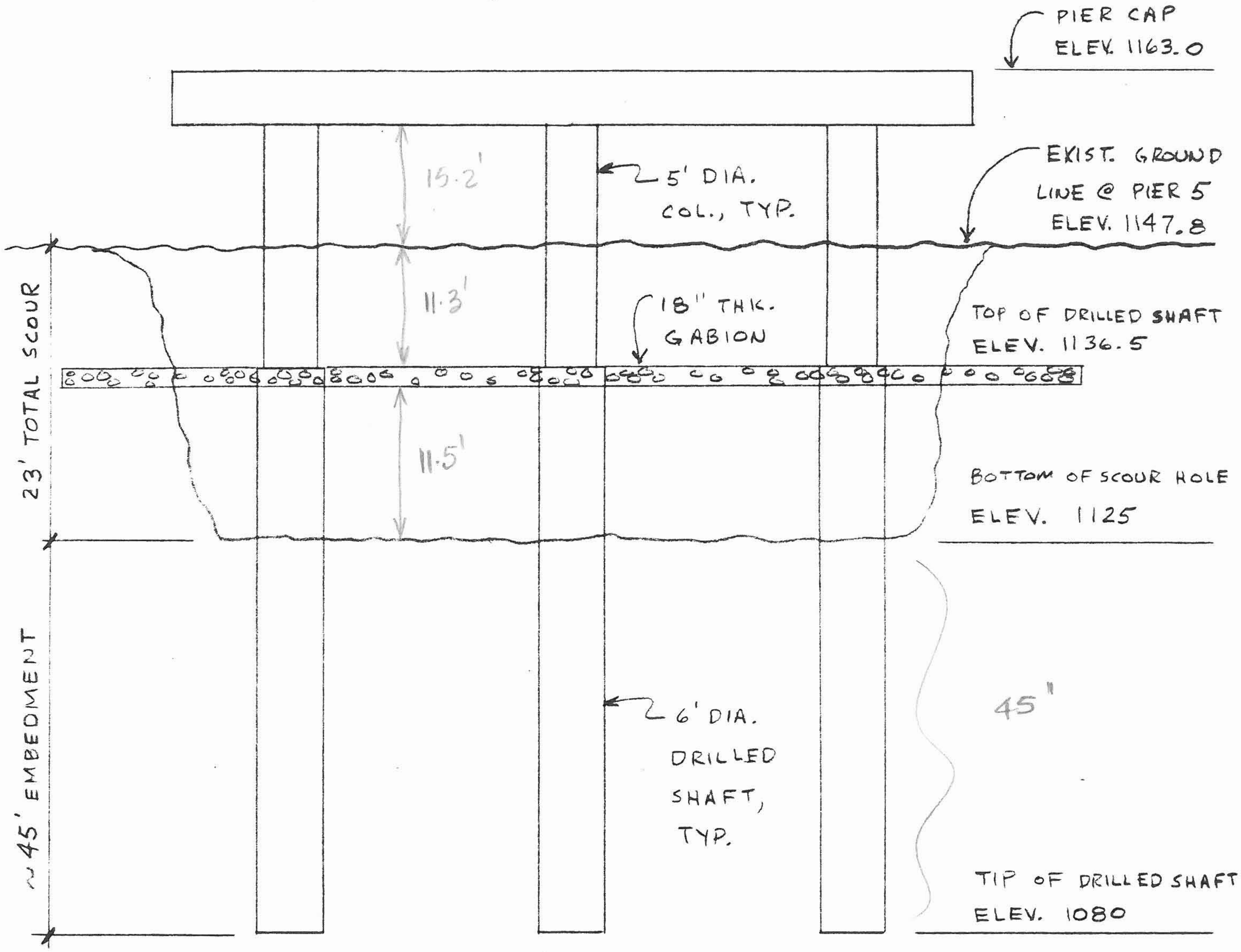
CONCLUSIONS: Based on the structural evaluation, the Bell Road Bridge at the Agua Fria River has sufficient structural capacity to resist the loads resulting from flows up to and including 145,600 cfs, *i.e.*, the low chord flow rate. The bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: Several gabions at the outlet of the Sun City West drainage channel have been damaged or destroyed by undermining and should be repaired or replaced.

BELL ROAD BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM/CSSA):

$Q_{LOW\ CHORD}$ = 145,600 CFS
H.W. ELEV. = 1161.29
TOTAL SCOUR = 23'



PIER ELEVATION

Figure 5

Photo 1: View looking toward upstream face of structure across primary low flow channel. Note channel is generally gravelly sand with finer and coarser gradations locally. Also note approximately 5 feet of relief between channel and adjacent bar is typical of upstream channel. Further note vegetation typical of upstream channel consisting of sparse grasses, scrub and occasional mesquite and palo verde trees.

Photo 2: View looking west along upstream face of structure. Note the generally sandy conditions existing immediately upstream of structure and the absence of vegetation.

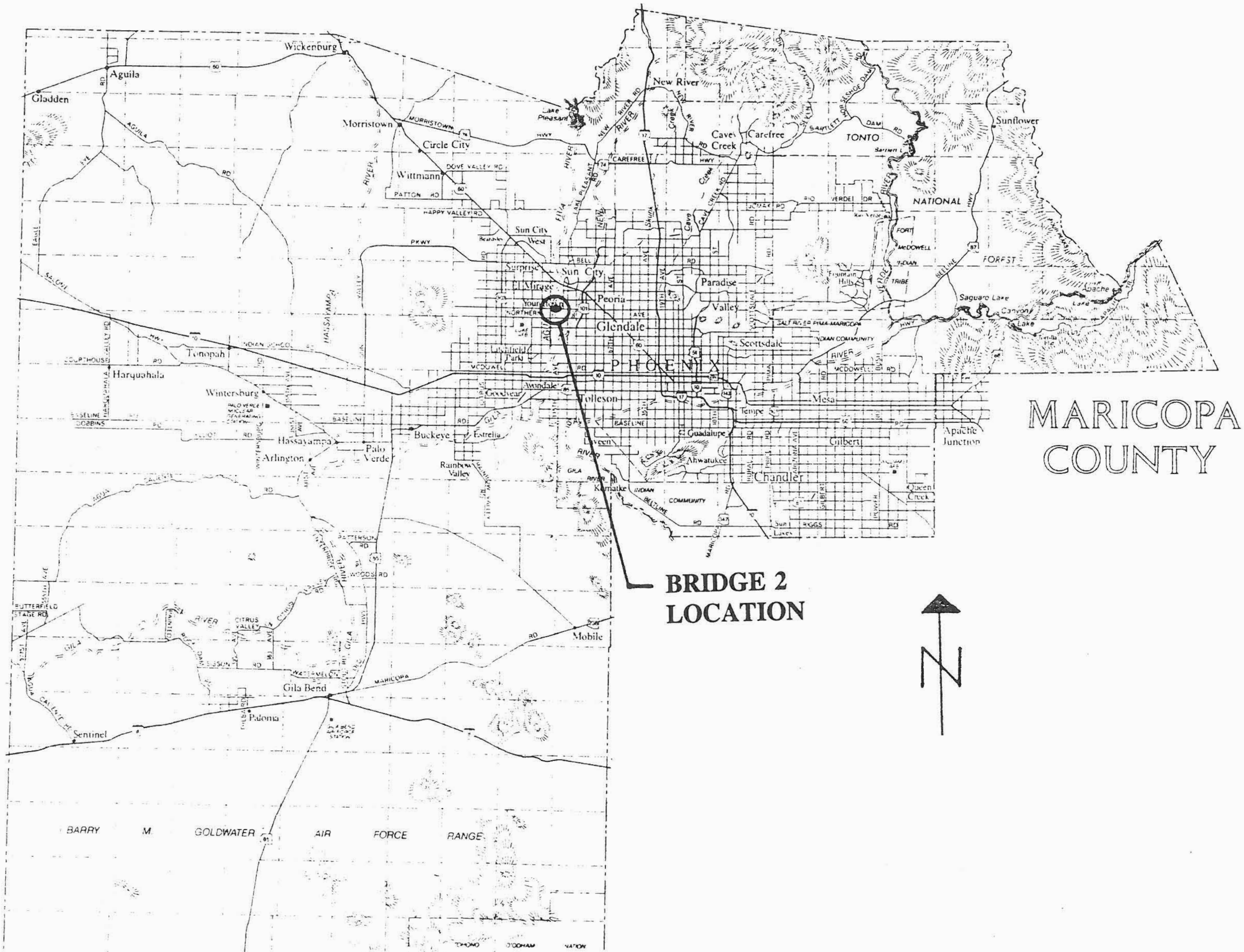
Photo 3: View looking downstream from bridge deck. Note primary low flow channel toward rip-rap along west bank and additional channel forms joining from the east in a generally braided form. Also note middle bar dividing the low flow channels.

Photo 4: View looking approximately west across channel downstream of structure. Note generally braided stream form in this location.

Photo 5: View looking east at upstream side of typical pile. Note variation in relief between bents of 2-3 feet was typical due primarily to variations in channel configuration. Also note that maximum depth of local scour at piers was 2-3 feet. Sediment type typically ranged from med. sand to cobbly gravel in vicinity of piers. Waterway was generally clear without significant variations in cross-section.

BRIDGE 2

OLIVE AVENUE BRIDGE OVER AGUA FRIA RIVER



Location Map

Figure 1


Agua Fria River
Olive Ave.
Olive Ave. Bridge #8981
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 2: OLIVE AVENUE BRIDGE OVER AGUA FRIA RIVER (Structure #8981)

Assessment: Scour Stable

LOCATION: The Olive Avenue Bridge the Agua Fria River lies on the section line between Sections 25 and 36 of T3N, R1W, Gila and Salt River Baseline and Meridian, near the boundary line between Sun City and the Town of Surprise, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The bridge has twelve spans with a total length of 1,498' from center-to-center of abutment bearings. (See Location Plan, Figure 3.) It has 68-foot-wide clear roadway and a sidewalk on the downstream side. The bridge was designed for a stream flow rate of 102,000 cubic feet per second (cfs), corresponding to a flood frequency interval of 100 years. The bridge was designed by Hoffman-Miller Engineers, Inc. in 1985; it was built in 1987 as MCDOT Project No. 68274.

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck. Expansion joints are provided at the abutments and at Pier Numbers 4 and 8.

The abutments consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on three 6'-0" diameter drilled shafts. The drilled shafts are founded approximately 56' below the exiting river flow line. Both abutments have zero skew with respect to the bridge centerline.

The piers consist of a concrete cap supported on three 5'-0" diameter formed columns. The formed columns extend approximately 2' below the existing river flow line and are supported on three 6'-0" diameter drilled shafts that are founded approximately 66' below the flow line. The piers have zero skew with respect to the bridge centerline.

EXISTING SCOUR PROTECTION: Scour protection at the east abutment consists of a riprap-lined elliptical guide bank extending approximately 290' upstream and downstream of the bridge centerline. The west abutment is protected by a riprap-lined dike approximately 100' long on each side of the bridge centerline. The riprap on the guide bank and dike is 2.5' thick, with a median diameter (D_{50}) of 15", and it is keyed 5' into the channel bed at the toe. Downstream of the west abutment, the bank is lined with dumped riprap approximately 3,400' to the south. The lower part of this lining, which is keyed into the soil, is grouted. According to the as-built plans, the piers are protected by a 6' thick layer of dumped riprap placed around each column several feet below grade.

A site inspection showed that all riprap linings are in good condition. The riprap around the piers was not visible.

STREAM FORM: The Agua Fria River at Olive Avenue is a braided stream with shallow to medium height point bars and middle bars distributed irregularly throughout the river bed. (See Figure 4.) There are two principal low flow channels--a large one flowing in a south-by-southwesterly direction near the center of the river bed and a smaller one near the west bank.

The channels join immediately downstream of the bridge.

LAND USE: Land use in the surrounding area is primarily agricultural and undeveloped desert, with some residential developments in the vicinity. Gravel mining operations are located in the west overbank downstream of the bridge; there was significant gravel mining in the channel downstream of the bridge. The remnant of this operation is still visible.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest source of runoff will continue to be releases from the New Waddell Dam, approximately 20 miles upstream of the bridge.

SURFACE SOILS: Surface soils consist primarily of sand and gravel, with some cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 3 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of the river bed in the vicinity of the Olive Avenue Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0025 ft/ft or approximately 13' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River includes trees such as palo verde, mesquite, and ironwood; bushes and shrubs such as desert broom, creosote, ephedra, salt bush, and brittle bush; and low grasses. The palo verde trees occur to moderate density upstream of the bridge and bushes and shrubs are sparsely distributed upstream but more densely downstream. The overall density of vegetation in the river is estimated to be low to moderate. Debris collecting on the piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Lateral stability of the river at the bridge is provided by dumped riprap on the banks for upstream and downstream of the bridge. Although aerial photographs showed that the low-flow channel configuration has not changed appreciably since 1982, it must be assumed that low-flow channels may move laterally when water is flowing in the river.

There are no upstream or downstream grade control structures in the immediate area that would maintain vertical stability of the river bed at the bridge site. The abandoned gravel mining operations described previously is thought to have a significant effect on channel elevation. Incising of the channel downstream of the bridge indicates that the long-term tendency of the stream bed is to degrade.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Olive Avenue come from the New Waddell Dam at Lake Pleasant, approximately 20 miles upstream of the bridge. Detention basins along Trilby Wash at the base of the White Tank Mountains also discharge to the Agua Fria River approximately 10 miles upstream of the bridge via the McMicken Dam Outlet Channel. Smaller flows come from off-site developed and undeveloped land between Waddell Dam and the bridge.

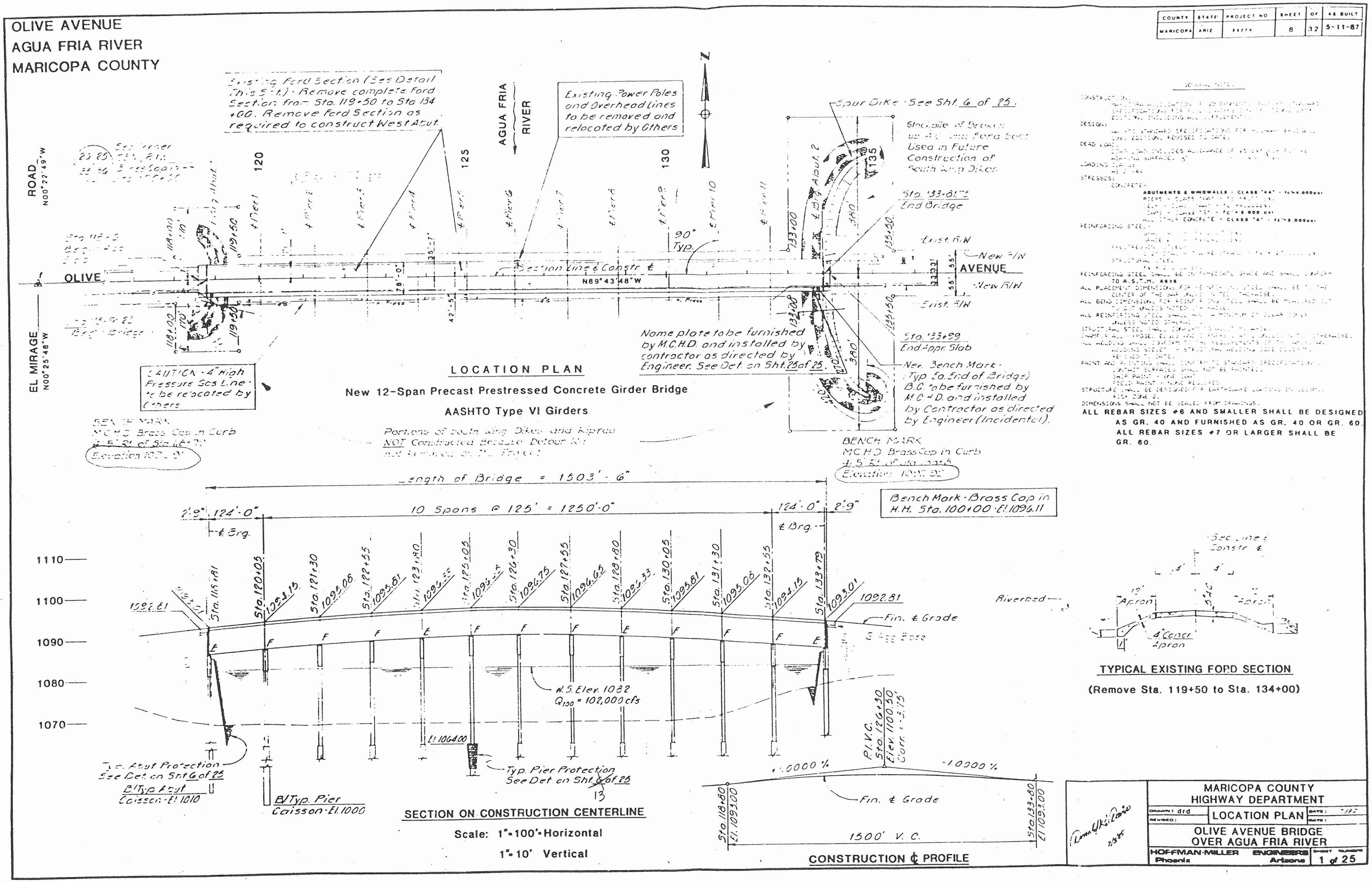

OLIVE AVENUE
AGUA FRIA RIVER
MARICOPA COUNTY
COUNTY | STATE | PROJECT NO. | SHEET | OF | AS BUILT
MARICOPA | ARIZ | | 8 | 32 | 5-11-87
Existing Power Poles and Overhead Lines to be removed and relocated by Others
AGUA FRIA RIVER
Spur Dike - See Sht. 6 of 25.
LOCATION PLAN
New 12-Span Precast Prestressed Concrete Girder Bridge
AASHTO Type VI Girders
OLIVE
AVENUE
EL MIRAGE
ROAD
N89°43'48"W
Name plate to be furnished by M.C.H.D. and installed by contractor as directed by Engineer. See Det. on Sht. 25 of 25.
Sta. 133+61.73 End Bridge
Sta. 133+99 End Appr. Slab
BENCH MARK
Bench Mark - Brass Cap in H.H. Sta. 100+00 - El. 1096.11
ALL REBAR SIZES #6 AND SMALLER SHALL BE DESIGNED AS GR. 40 AND FURNISHED AS GR. 40 OR GR. 60. ALL REBAR SIZES #7 OR LARGER SHALL BE GR. 60.
Length of Bridge = 1503' - 6"
10 Spans @ 125' = 1250'-0"
2'-9" 124'-0" 124'-0" 2'-9"
1110
1100
1090
1080
1070
W.S. Elev. 1082
Q_{100} = 102,000 cfs
Fin. ℄ Grade
Typ. Pier Protection See Det. on Sht. 13
B/Typ. Pier Caisson - El. 1000
SECTION ON CONSTRUCTION CENTERLINE
Scale: 1" = 100' - Horizontal
1" = 10' Vertical
1500' V.C.
CONSTRUCTION ℄ PROFILE
TYPICAL EXISTING FORD SECTION
(Remove Sta. 119+50 to Sta. 134+00)
MARICOPA COUNTY
HIGHWAY DEPARTMENT
LOCATION PLAN
OLIVE AVENUE BRIDGE
OVER AGUA FRIA RIVER
HOFFMAN-MILLER ENGINEERS
Phoenix Arizona
1 of 25

Figure 3

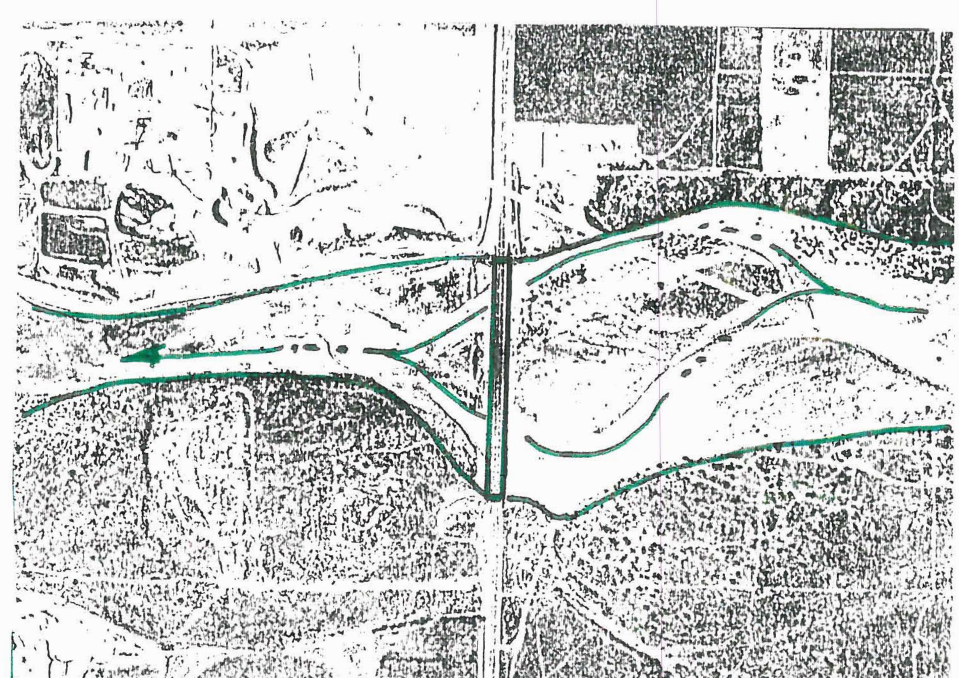
OLIVE AVENUE (SN 8981)

Water Course Agua Fria River

Stream Form Braided
Sinuosity Not applicable
General Channelization Channel DS of bridge along west bank is lined with dumped rip-rap for several hundred feet. Spur dikes US/DS east abutment guide flow near structure.
Channel Slope Significantly steeper DS.
Estimated Channel Slope (ft/ft) 0.002110
Channel Contraction/Expansion Wider US
Primary Surface Sediment Type sand/gravel
D50 Size 3 MM
Armoring Potential Low
Channel Vegetation
Type/Size Palo Verde to 8 ft., Desert Broom, Desert Willow, Creosote; Sage, Ephedra, Salt Bush; low dry grasses.
Density/Occurrence Palo Verde moderate density US; smaller brush sparsely distributed US, denser DS; grasses moderately dense US, sparse DS.
Relative Age Young to mature.
Manning's Roughness Coef. 0.030
Controls on Stream Migration
Lateral Channel DS of bridge along west bank lined with dumped rip-rap for several hundred feet
Vertical No
Sediment Deposits & Bars Irregular distribution of shallow to moderate height point and middle bars.
Evidence of Degradation Yes, incising of channel DS of structure.
Evidence of Aggradation No
Evidence of Scour
Pier Residual local scour observed to be a maximum of 2-3 feet.
Abutment No; potential for west abutment scour high due to main channel proximity.
Land Use
Urbanization of Upstream Watershed Low to moderate rate; land use primarily agricultural, high density residential.
Sand & Gravel Extraction Currently in overbanks; recently in channel immediately DS of structure.
Freeway Construction No

Dams No

Drainage Channels No



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

a. The flow and flood frequency used in the design of the bridge are 102,000 cfs and 100 years, respectively.

b. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978 as measured at El Mirage, Arizona at Grand Avenue, approximately 3 miles downstream of the bridge.

c. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge at 98,780 cfs.

d. The 500-year flood (superflood) reported by the Federal Emergency Management Agency (FEMA) is 179,000 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 MAXIMUM FLOW: In accordance with MCDOT requirements, the critical flow for use in scour evaluations 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
1072.00	10,970	Q_{10}
1076.86	69,700	-
1078.37	98,780	Q_{100}
1081.76	179,900	Q_{500}
1085.50	292,600	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 179,900 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 Olive Avenue 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 full in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 98,780 cfs (Q_{100})	Q = 179,900 cfs (Q_{500})
a.	Scour at Piers		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	23.9	27.1
	General Scour, ft	4.0	4.0
	Total Scour, ft	**27.9**	**31.1**
b.	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 = 98,780 cfs (Q_{100})	Q = 179,900 cfs (Q_{500})
a. Embedment at Piers		
Channel Elevation	1065.5	1065.5
Total Scour, ft	27.9	31.1
Elev. Bottom of Scour Hole	1037.6	1034.4
Drilled Shaft Tip Elev.	1000.0	1000.0
Embedment Remaining, ft	**37.6**	**34.4**

b. Embedment at Abutments

Channel Elevation	1065.5	1065.5
Total Scour, ft	4.0	4.0
Elev. Bottom of Scour Hole	1061.5	1061.5
Drilled Shaft Tip Elev.	1010.0	1010.0
Embedment Remaining, ft	**51.5**	**51.5**

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 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 structural evaluation, the Olive Avenue Bridge at the Agua Fria River has sufficient structural capacity to resist loads resulting from flows up to and including 179,000 cfs, *i.e.*, the 500-year flow rate. The bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: The existing riprap keyed into the channel banks should be grouted to avoid undermining of the riprap.

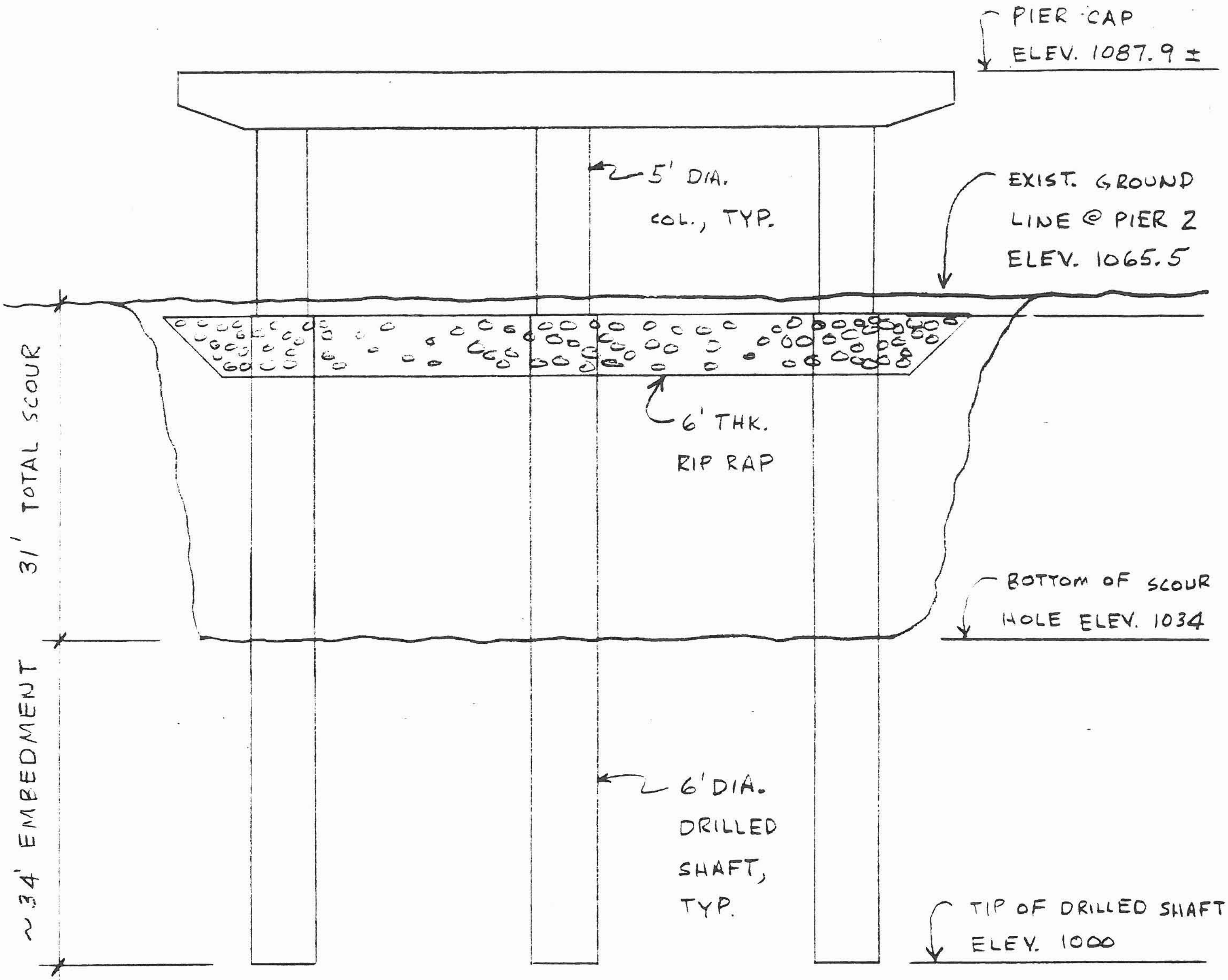
OLIVE AVENUE BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM/CSSA):

Q 500 = 179,900 CFS

H.W. ELEV. = 1081.76

TOTAL SCOUR = 31'



PIER ELEVATION

Figure 5

Photo 1: View looking upstream from west side of bridge deck. Note primary low flow channel composed of coarser material near bank with finer material toward bar. Also note west bank of main channel being undercut by flow and transition in bank relief from relatively shallow upstream to well defined near structure. Further note contrast in vegetation density between low flow channel and adjacent bar. Bar relief is approximately 5 feet above low flow channel.

Photo 2: View of west bank of main channel looking upstream from bridge deck. Note placement of rip-rap upstream of west abutment to control lateral migration of primary low flow channel.

Photo 3: View looking upstream from east side of bridge deck. Note secondary low flow channel impinging on east side of structure. Also note upstream left overbank is poorly defined with mild slope toward bench in background. Typical vegetation is sparse grasses, occasional low brush and moderately dense mesquite and palo verde trees.

Photo 4: View looking toward upstream mid-channel from east side of bridge deck. Note secondary low flow channel in upper right. Also note relatively coarse nature of surface sediments and absence of vegetation typical of east side of main channel upstream of bridge.

Photo 5: View looking downstream from west side of bridge deck. Note sharply cut banks of primary low flow channel as it trends toward middle of main channel downstream of structure. Also note effect of dip in channel (possible gravel pit) further downstream. Delta deposits forming from loss of stream energy and rapid deposition of sediment load.

Photo 6: View looking approximately southeast from west bank across channel downstream of structure. Note typical relief from upper delta surface to channel bottom is approximately 8 feet.

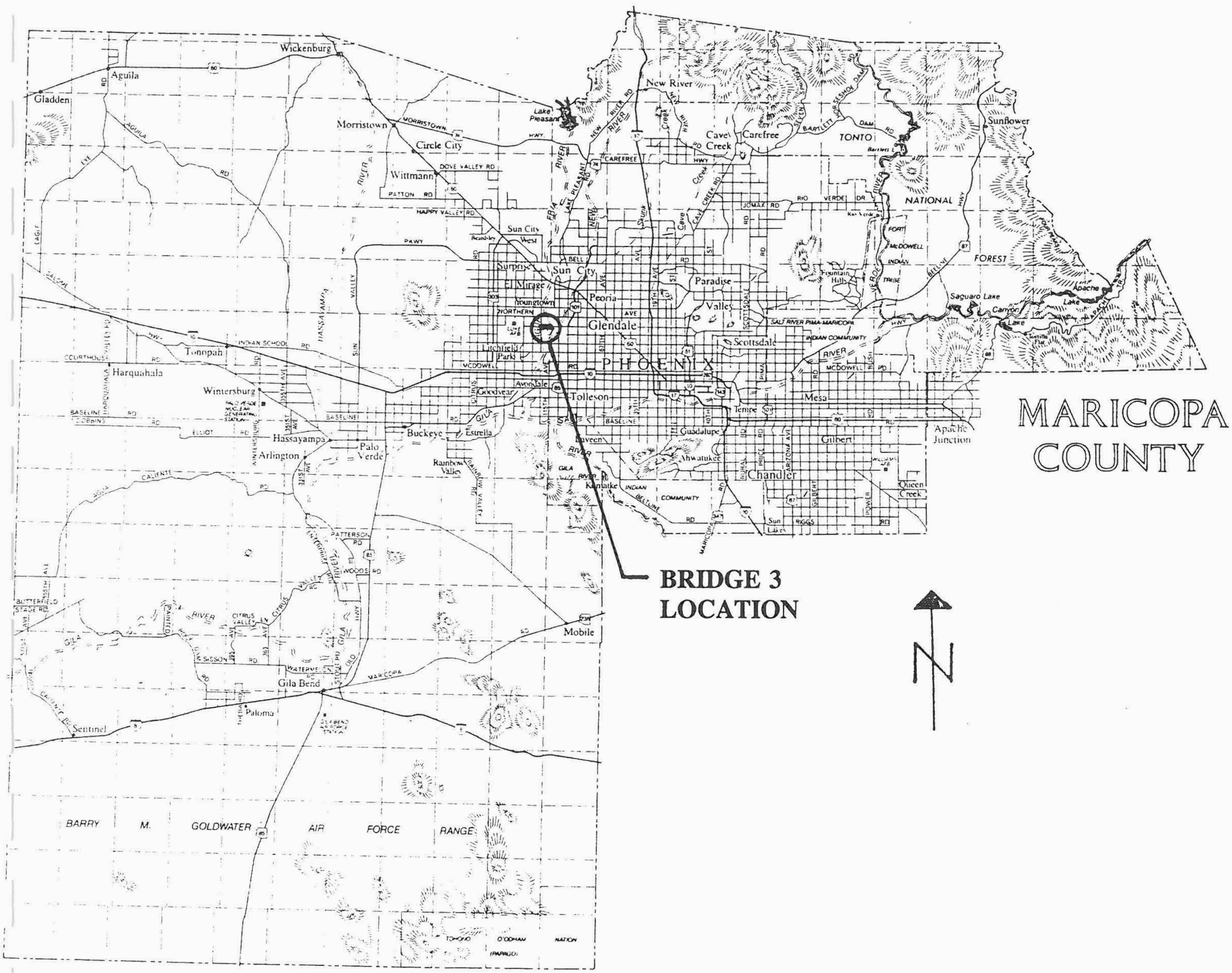
Photo 7: View looking upstream from channel toward west abutment. Note primary low flow channel trending toward center of main channel. Also note relatively steep channel slope.

Photo 8: View looking west beneath bridge. Note sandy depression near mid-span of structure may increase flow turbulence locally.

Photo 9: View of typical pile bent near west abutment. Residual local scour at piers was observed to be a maximum of approximately 2-3 feet.

BRIDGE 3

GLENDALE AVENUE BRIDGE OVER AGUA FRIA RIVER



<u>**Location Map**</u>

Figure 1


Agua Fria River
Glendale Ave.
Glendale Ave. Bridge #9301
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 3: GLENDALE AVENUE BRIDGE OVER AGUA FRIA RIVER

(Structure #9301)
Assessment: Scour Critical

at

LOCATION: The Glendale Avenue Bridge the Agua Fria River lies on the section line between Sections 1 and 2 of T2N, R1W, Gila and Salt River Baseline and Meridian, approximately 1 mile west of the Town of Glendale, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The bridge has six spans with a total length of 598' from center-to-center of abutment bearings and a 74' wide clear roadway. (See Location Plan, Figure 3.) The bridge was designed for a stream flow rate of 55,000 cubic feet per second (cfs), corresponding to a flood frequency interval of 50 years. The bridge was designed by Hoffman-Miller Engineers, Inc. in 1971; it was built in 1973 as MCDOT Project No. S-227(9).

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck. Expansion joints are provided at the abutments and at Pier Number 3.

The abutments consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on 17 driven steel piles. The piles are founded approximately 26' and 16' below the existing river flow line at Abutment Numbers 1 and 2, respectively. Both abutments are oriented with zero skew with respect to the bridge centerline.

The piers are the solid concrete wall type on spread footings. The upstream end of the wall is equipped with a 90 degree steel nosing. The footings are founded approximately 17' below the existing river flow line. The piers are oriented with zero degree skew with respect to the bridge centerline.

EXISTING SCOUR PROTECTION: According to as-built plans, scour protection is provided in the form of elliptical guide banks at each abutment, covered with a layer of 12" thick wire-tied riprap. The plans show the riprap keyed approximately 7' into the river bed. No special scour protection at the piers was shown on the plans; however dumped riprap has been placed around each pier.

A site inspection indicated that the wire-tied riprap lining was in generally good condition, although a break in the wire and minor unravelling of the riprap was noted at Abutment Number 2, the east abutment.

STREAM FORM: The Agua Fria River at Glendale Avenue is a braided stream with shallow to medium height point bars and middle bars distributed irregularly throughout the river bed. (See Figure 4.) Construction of a shallow earthen berm in the river bed intersecting the bridge at Pier Number 4 limits low flows to the four western spans of the six-span structure.

LAND USE: Land use in the surrounding area is primarily undeveloped desert or light industrial. Large gravel mining operations are located in or along the Agua Fria River approximately 1/2 mile upstream of the bridge and 2 miles downstream, near Camelback Road. There is also some gravel mining in the east overbank of the river approximately 1 mile downstream of the bridge.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest source of runoff will continue to be releases from the New Waddell Dam, is located upstream of the bridge.

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

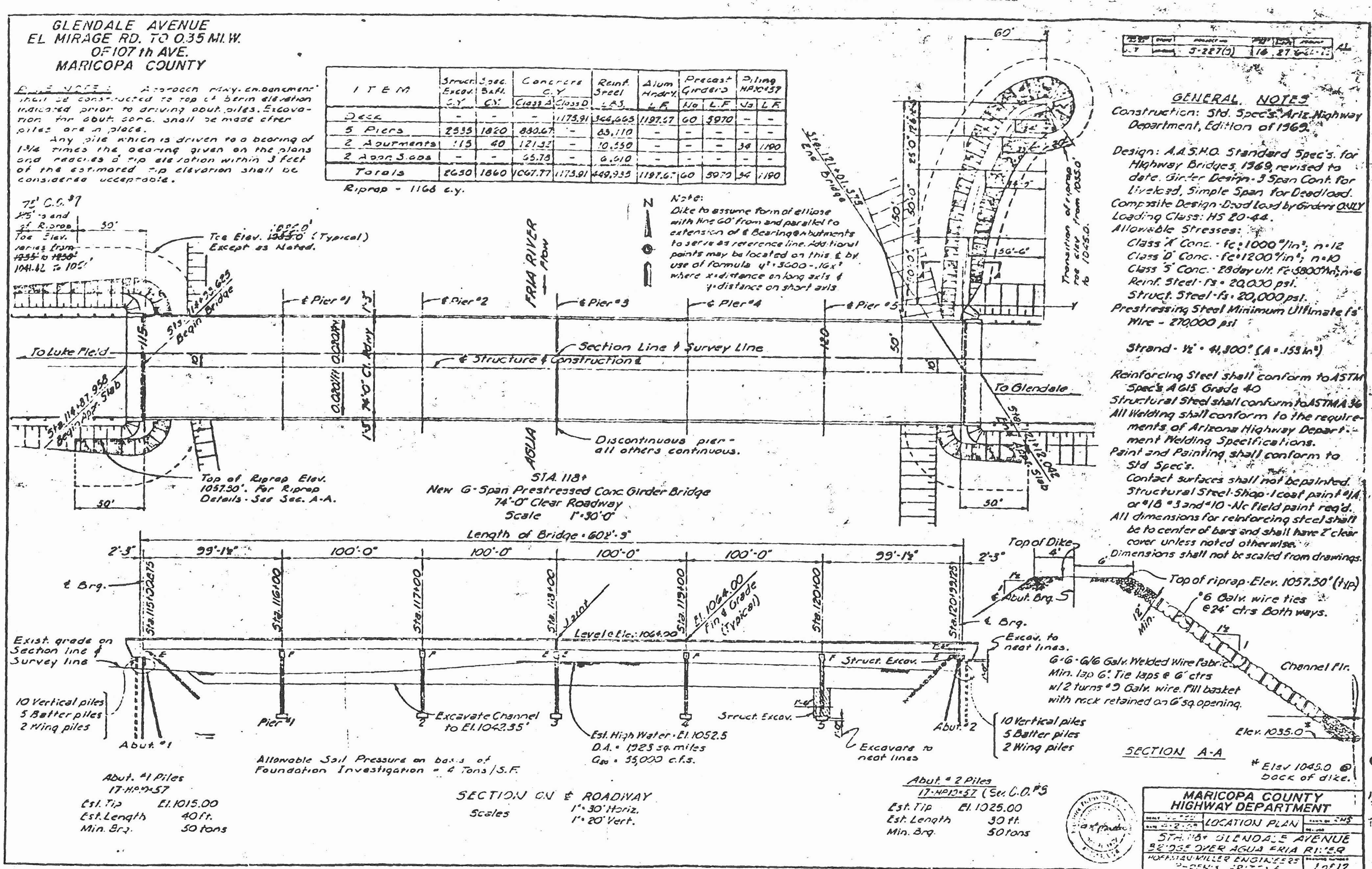
SLOPE: According to the as-built plans, the Agua Fria river in the vicinity of the bridge was graded at a slope of 0.002 ft/ft or approximately 10.5' per mile when the bridge was constructed. Slopes based on 7.5 minute USGS topographic maps range from 0.0023 ft/ft downstream of the bridge to 0.0028 ft/ft upstream of the bridge.

VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River includes trees such as palo verde, mesquite, and ironwood; bushes and shrubs such as desert broom, creosote, ephedra, salt bush, and brittle bush; and low grasses. Upstream of the bridge, the density of trees increases from low to moderate towards the west abutment; otherwise, vegetation is sparse, with low brush primarily towards the guide bank on the east side. Downstream of the bridge, there is a dense stand of trees and brush near a wastewater treatment plant outfall that discharges to the river downstream of the west abutment. Vegetation on the east side of the river downstream of the bridge is sparse to low density. Debris collecting on the piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Lateral stability of the river at the bridge is provided by the riprap-lined guide banks at both abutments. Upstream of the western guide bank, pieces of broken concrete have been placed intermittently along the west river bank for a distance of approximately 1,000'. Further upstream, a section of grouted riprap lines the west bank. Extensive gravel mining operations on the east overbank between Glendale Avenue and Northern Avenue have altered the stability of the east side of the river.

Although aerial photographs show that the location of the low-flow channel has not changed appreciably since 1984, it must be assumed that the thalweg may move laterally when water is flowing in the river. The earthen berm described previously provides some lateral stability for low flows but will not withstand erosive forces generated during extreme floods.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Glendale Avenue come from controlled releases from the New Waddell Dam at Lake Pleasant, approximately 22 miles upstream of the bridge. Detention basins along Trilby Wash at the base of the White Tank Mountains also discharge to the Agua Fria River approximately 12 miles upstream of the bridge via the McMicken Dam Outlet Channel. Smaller flows come from off-site developed and undeveloped land between Waddell Dam and the bridge.



ITEM	Struct. Excav. C.Y.	Spec. Bkfl. C.Y.	Concrete C.Y. Class A	Concrete C.Y. Class D	Reinf. Steel LBS.	Alum. Hndry. L.F.	Precast Girders No	Precast Girders L.F.	Piling HP10x57 No	Piling HP10x57 L.F.
Deck	-	-	-	1175.91	344,665	1197.67	60	5970	-	-
5 Piers	2535	1820	880.67	-	83,110	-	-	-	-	-
2 Abutments	115	40	121.32	-	10,550	-	-	-	34	1190
2 Appr. Slabs	-	-	65.78	-	6,610	-	-	-	-	-
Totals	2650	1860	1067.77	1175.91	449,935	1197.67	60	5970	34	1190

Figure 3

GLENDALE AVENUE (SN 9301)

Water Course	Agua Fria River
Stream Form	Braided
Sinuosity	Not applicable
General Channelization	Intermittent broken concrete rip-rap along west bank up to approximately 1000 ft. US, then approximately 1000 ft. of grouted rip-rap. Construction of shallow earthen berm intersecting bridge at pier 4 limits low flows to western 400 ft. of bridge.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.002047
Channel Contraction/Expansion	Constrained channel narrower US
Primary Surface Sediment Type	sand/gravel/cobbles
D50 Size	
Armoring Potential	Low
Channel Vegetation	
Type/Size	Cottonwood to 20 ft., Mesquite, occasional Palo Verde, Ironwood; Desert Broom to 7 ft., Desert Willow, Creosote; Sage, Ephedra, Salt Bush; dry grasses.
Density/Occurrence	(US) Density of trees increases to moderate toward west abutment, elsewhere sparse; low brush primarily toward east side spur dike. (DS) Dense strand of trees, shrubs DS of west abutment signifies ponding area; east side of channel sparse to low density.
Relative Age	Mature
Manning's Roughness Coef.	0.049
Controls on Stream Migration	
Lateral	Spur dikes US/DS east abutment guide flow locally; limited slope protection on west bank
Vertical	None
Sediment Deposits & Bars	Irregular distribution of shallow point and middle bars. Sedimentation occurring in east side spans.
Evidence of Degradation	No
Evidence of Aggradation	Graffiti obscured by sand/gravel accumulation indicates local aggradation.
Evidence of Scour	
Pier	No
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low rate; land use primarily agricultural or undeveloped, light industrial.
Sand & Gravel Extraction	Large gravel operation immediately US has significantly encroached on east side of channel and crossed east overbanks. Limited to operations in DS east overbank.
Freeway Construction	No
Dams	No
Drainage Channels	No

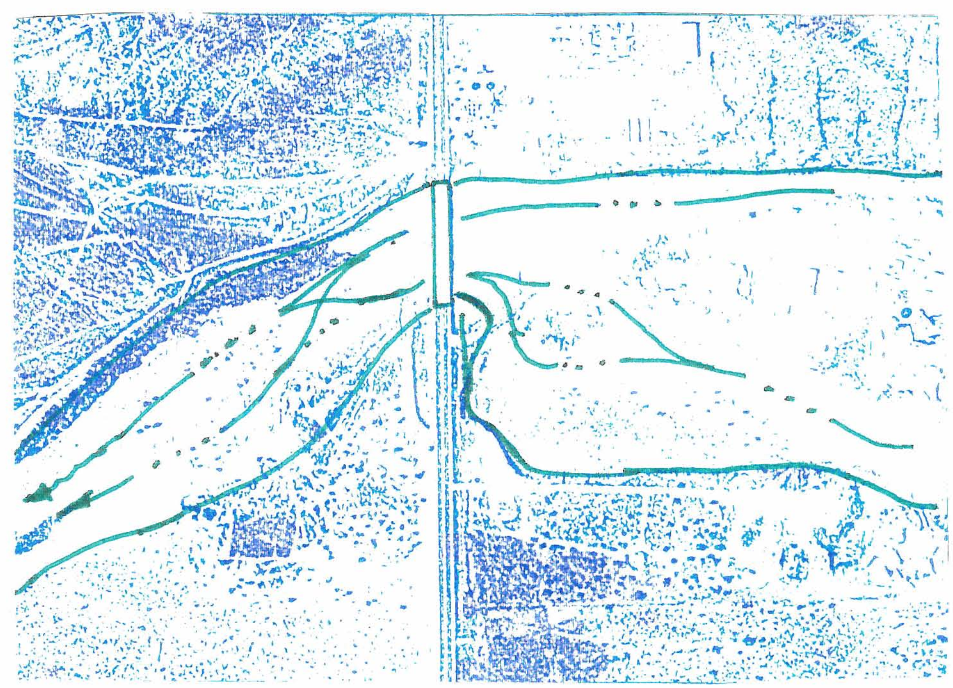


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Figure 4

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

a. The flow and flood frequency used in the design of the bridge are 55,000 cfs and 50 years, respectively.

b. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978 as measured at El Mirage, Arizona at Grand Avenue, approximately 5 miles downstream of the bridge.

c. The latest hydrologic model available from the Flood Control District of Maricopa County (FCDMC) estimates a 100-year flood at the bridge at 90,700 cfs.

d. The 500-year flood (superflood) reported by the Federal Emergency Management Agency (FEMA) is 177,000 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 evaluations 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
1050.95	18,000	Q_{10}
1053.38	30,000	Assumed
1055.10	40,000	Assumed
1058.14	60,800	Low Chord
1070.06	177,000	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 60,800 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for 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 Glendale Avenue 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 footing embedment are shown in Tables 2 and 3, respectively. A schematic representation of scour at the piers during Q_{LC} is shown is Figure 5. Scour calculations are presented in full in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 60,800 cfs (Q_{LC})
a.	Scour at Piers	
	Contraction Scour, ft	0.0
	Local Scour, ft	20.5
	General Scour, ft	4.0
	Total Scour, ft	**24.5**
b.	Scour at Abutments	
	Abutment Scour, ft	0.1
	General Scour, ft	4.0
	Total Scour, ft	**4.1**

Table 3. Summary of Footing Embedment

		Q = 60,800 cfs (Q_{LC})
a.	Embedment at Piers	
	Channel Elevation	1041.5
	Total Scour, ft	24.5
	Elev. Bottom of Scour Hole	1017.0
	Bottom of Footing Elev.	1025.0
	Embedment Remaining, ft	**-8.0**

CONCLUSIONS: Based on the scour calculations, the piers do not have sufficient embedment to resist flows of 60,800 cfs, i.e., the low chord flow rate and the footings do not bear on rock. The bridge is scour *critical.*

COUNTERMEASURES:

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 between and around the scour critical piers, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

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

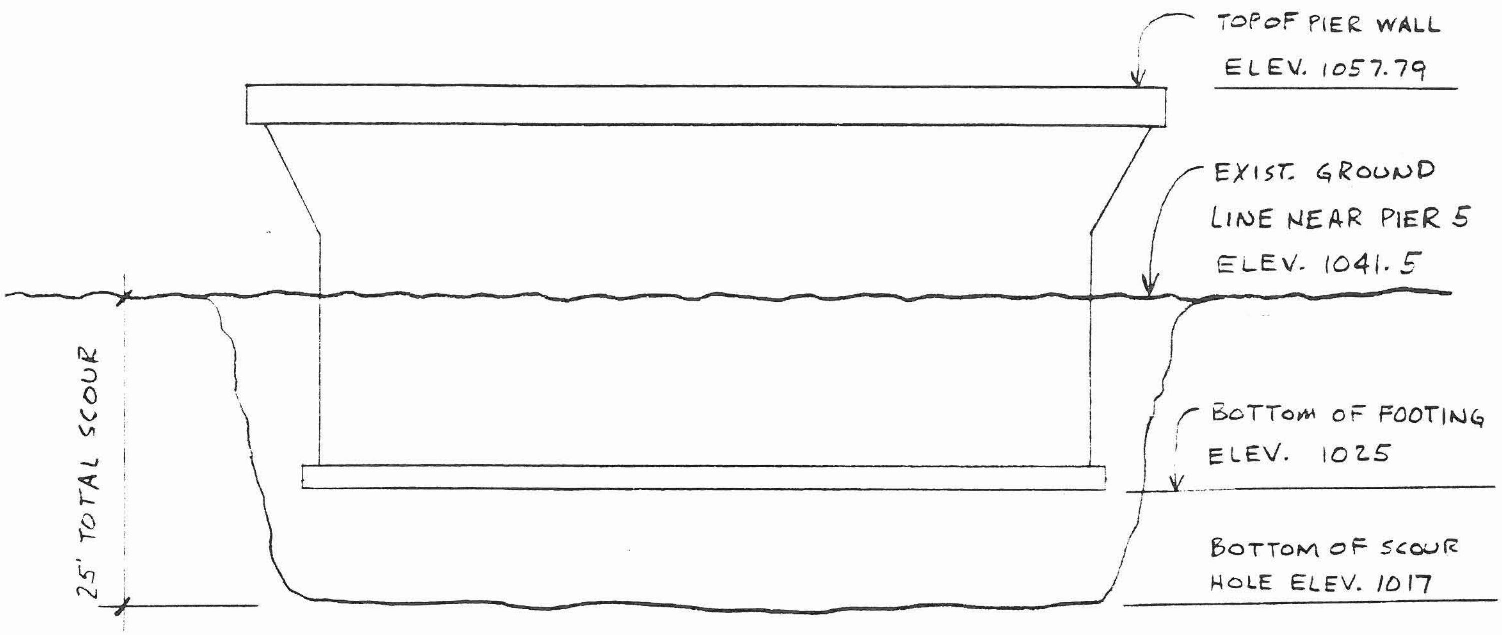
d. Replace the bridge with a new bridge supported on deep foundations.

GLENDALE AVENUE BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MW/CSSA):

H.W. ELEV. = LOW CHORD ELEV. = 1058.14

TOTAL SCOUR = 25'



PIER WALL ELEVATION

Figure 5

Photo 1: View looking upstream from approximately mid-span of structure. Note low dumped gravel levee forming east bank of a primary channel with haul road to east of levee. Large flows might potentially collapse poorly constructed levee and allow flow to expand east. Note short reach of white concrete grouted rip-rap along west bank in background. Upstream vegetation gives sparse to moderate coverage of channel and includes low brush and small trees. Channel surface sediments consist primarily of sandy gravel with occasional coarse sand bars.

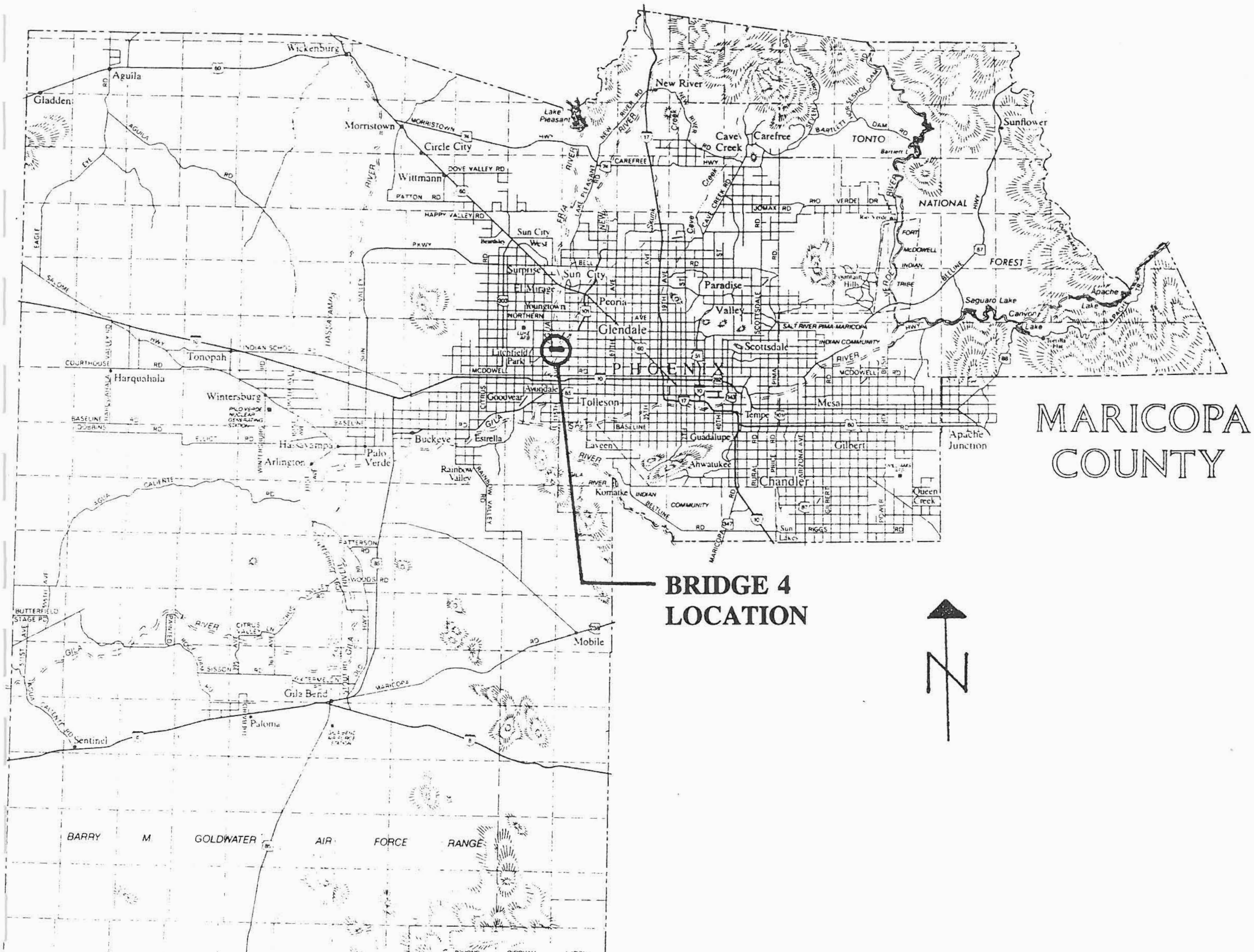
Photo 2: View of wire-tied rip-rap placed along upstream side of west abutment. Note overgrowth and potential for debris capture.

Photo 3: View looking downstream from approximately mid-span of bridge deck. Note extremely dense marshy vegetation immediately downstream of structure near west abutment.

Photo 4: View along upstream face of structure from approximately mid-span of deck. Note the presence of significant mature vegetation adjacent to west abutment along low flow channel. Note increasingly sandy nature of surficial sediments near structure.

BRIDGE 4

CAMELBACK ROAD BRIDGE OVER AGUA FRIA RIVER



Location Map

Figure 1

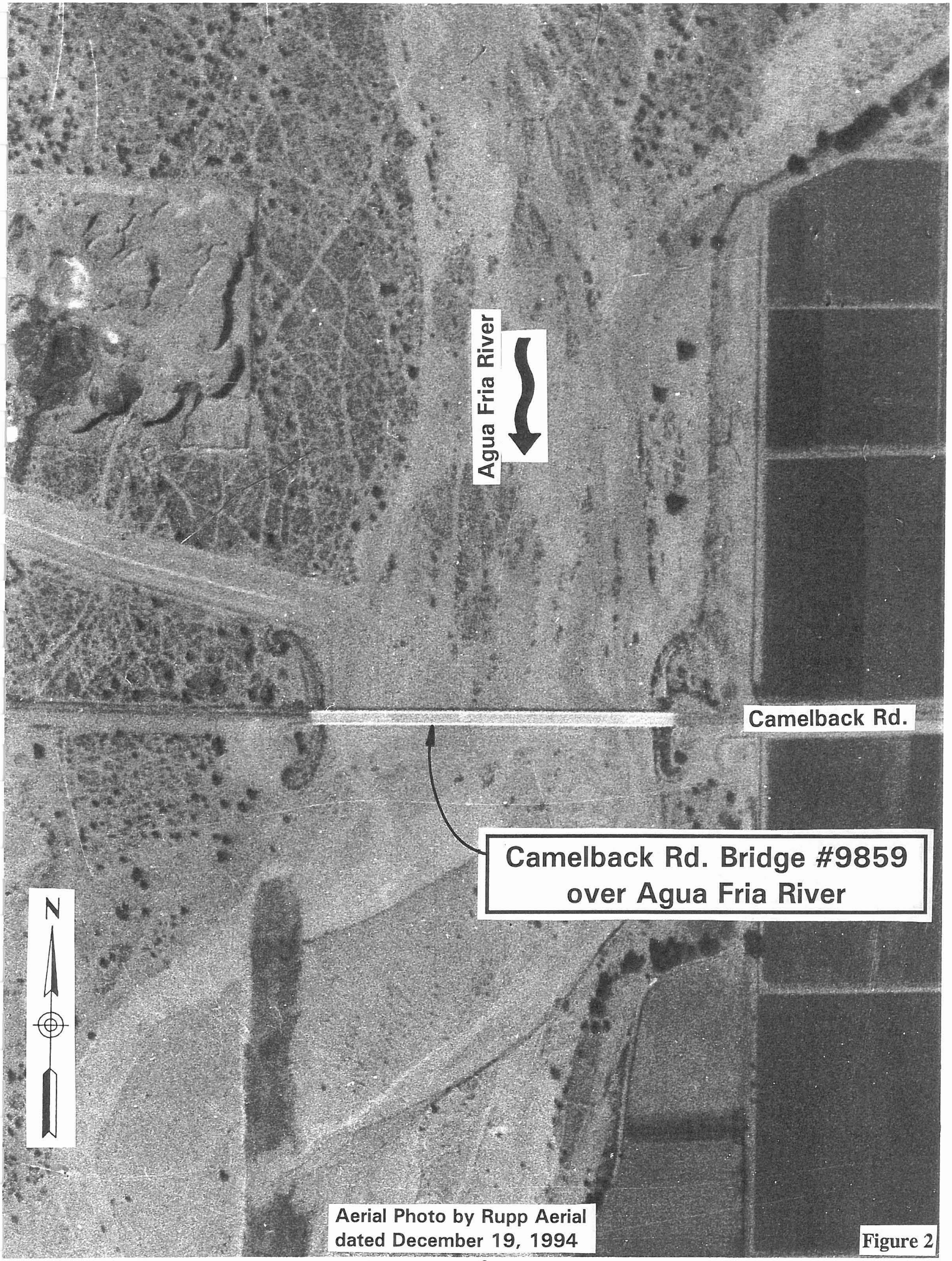

Agua Fria River
Camelback Rd.
Camelback Rd. Bridge #9859
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 4: CAMELBACK ROAD BRIDGE AT THE AGUA FRIA RIVER

(Structure #9859)
Assessment: Scour Stable

LOCATION: The Camelback Road Bridge at the Agua Fria River lies at the corner of T2N R1W Sections 13 and 24 and T2N R1E Sections 18 and 19, at the west side city limit of Phoenix, Arizona. See Location Map, Figure 1 and Aerial Photo Figure 2. Map.

STRUCTURE: The bridge has fifteen spans with a total length of 1,720' from center-to-center of abutment bearings. (See General Bridge Plans, Figure 3.) It has a 52-foot-wide clear roadway and a sidewalk on the upstream side. The bridge was designed for a stream flow rate of 95,000 cubic feet per second (cfs), corresponding to a flood frequency interval of 100 years. The bridge was designed by PRC Toups in 1982; it was built in 1984 as MCDOT Project No. 68104.

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck. Expansion joints are provided at the abutments and at Pier Nos. 3, 6, 9, and 12.

The abutments consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on three 4'-0" diameter drilled shafts. The drilled shafts are founded approximately 65' below the existing river flow line. Both abutments are oriented normal to the bridge roadway.

The piers consist of a concrete cap beam supported on three 4'-0" diameter formed columns. The formed columns are supported on three 4'-0" diameter drilled shafts that are founded approximately 65' below the existing groundline. The piers have zero skew with respect to the bridge centerline.

EXISTING SCOUR PROTECTION: According to as-built plans, scour protection at the west abutment consists of riprap-lined elliptical guide bank extending approximately 450' upstream and 300' downstream of the bridge centerline. The riprap covering the guide bank ranges between 2.5' and 3.5' thick, has a median stone diameter (D_{50}) of 16", and is keyed from 6' to 9' into the bank. (The larger dimensions occur at the upstream end of the guide bank.) The east abutment is protected by a guide bank, curved at the upstream end and straight at the downstream end. The upstream end is covered with a 1.5' thick layer of grouted riprap, the downstream end with a 2.5' thick layer of dumped riprap. The riprap is keyed approximately 4' to 6' into the stream bed. There is also a secondary dike branching from the back side of the east abutment guide bank that connects with the roadway embankment of the north side of the bridge. This secondary dike was designed to intercept runoff flowing west along the north side of the road and deflect it over the grouted riprap section of the guide bank, Piers are protected by a 5' thick layer of dumped riprap approximately 5' below grade.

A site inspection showed that for the most part the scour protection was in good condition. Some undermining and loss of riprap was noted on the downstream end of the east abutment guide bank, where a low flow channel had undercut the toe of the riprap.

STREAM FORM: The Agua Fria River at Camelback Road is a braided stream with shallow to medium height point bars, alternated and middle bars distributed irregularly throughout the river bed. (See Figure 4.) There is one principal low flow channel the near east bank.

LAND USE: Land use in the surrounding area is primarily agricultural or undeveloped desert, with some residential developments in the vicinity. Gravel mining operations are located in the west overbank downstream of the bridge; there was significant gravel mining in the channel downstream of the bridge, the remnant of which is still visible.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest sources of runoff will continue to be releases from New Waddell Dam on the Agua Fria River approximately 24 miles upstream of the bridge, and New River Dam on New River, approximately 15 miles upstream.

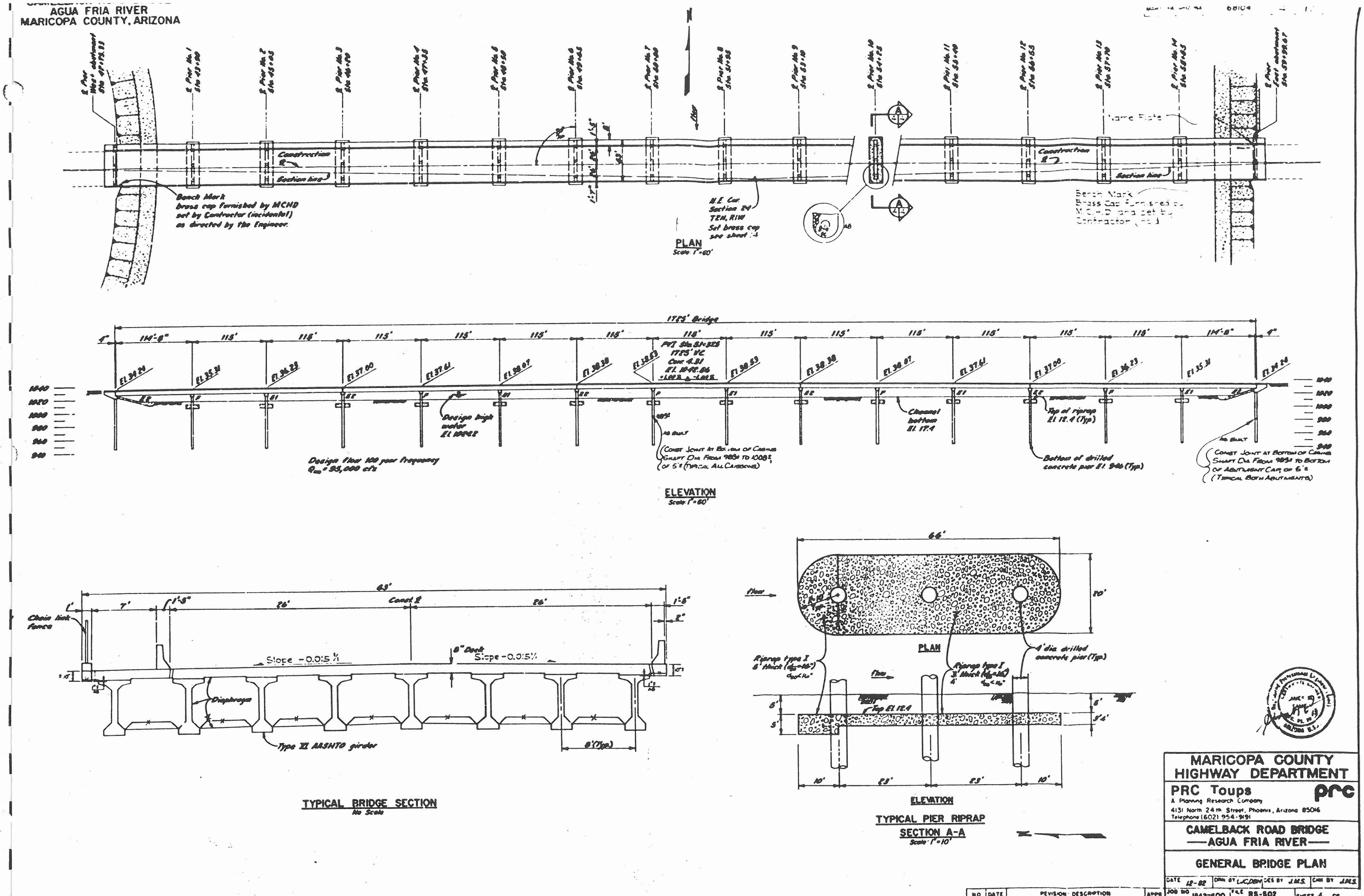
SURFACE SOILS: Surface soils consist primarily of sand and gravel, with some cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 1 mm. The armoring potential of the river bed is estimated to be low, although the low flow channel upstream of the bridge appeared to be well-armored during the site investigation.

SLOPE: The slope of the river bed in the vicinity of the Camelback Road Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0024 ft/ft or approximately 12.5' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River includes trees such as palo verde; bushes and shrubs such as desert broom, desert willow, creosote and ephedra; and low grasses. The overall density of vegetation in the river is estimated to be low to nearly barren, with the lower densities occurring downstream of the bridge, probably due to past gravel mining activities. Debris collecting on the piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Overall lateral stability of the river at the bridge is provided by the guide banks at the bridge abutments. A review of aerial photographs showed that between 1989 and 1995 the low-flow channel has varied between the east abutment and the eastern quarter of the bridge; therefore, it must be assumed that the low flow channel may move laterally to any of the piers when water is flowing in the river.

The location of the low flow channel (or channels) may depend on the flow rate in the three major channels upstream of the bridge: The Agua Fria River, New River, and the Colter Drainage Channel. New River discharges to the Agua Fria River approximately one-half mile above the bridge on the east side of the river; its angle of entry into the main river is such that a low flow channel would tend to form between the middle of the bridge and the east abutment if it were the only stream flowing. The Colter channel discharges to the Agua Fria River immediately upstream of the west abutment guide bank. Depending on the discharge, the Colter channel could cause low flow channels to form between the middle of the bridge and the west abutment.



AGUA FRIA RIVER
MARICOPA COUNTY, ARIZONA
PLAN
1725' Bridge
ELEVATION
TYPICAL BRIDGE SECTION
No Scale
Type IV AASHTO girder
Diaphragm
ELEVATION
TYPICAL PIER RIPRAP
SECTION A-A
MARICOPA COUNTY
HIGHWAY DEPARTMENT
PRC Toups
A Planning Research Company
4131 North 24th Street, Phoenix, Arizona 85016
Telephone (602) 954-9191
CAMELBACK ROAD BRIDGE
—AGUA FRIA RIVER—
GENERAL BRIDGE PLAN

Figure 3

CAMELBACK ROAD (SN 9859)

Water Course	Agua Fria River
Stream Form	Braided
Sinuosity	Not applicable
General Channelization	None
Channel Slope	Steeper DS
Estimated Channel Slope (ft/ft)	0.002343
Channel Contraction/Expansion	Wider US
Primary Surface Sediment Type	sand/gravel
D50 Size	1 MM (estimate)
Armoring Potential	Low, but low flow channel US is armored.
Channel Vegetation	
Type/Size	Palo Verde to 7 ft., Desert Broom to 5 ft., Desert Willow; Creosote, Ephedra; dry grasses.
Density/Occurrence	(US) Low density vegetation occurs on bars. (DS) Nearly barren except for a few isolated stands.
Relative Age	
Manning's Roughness Coef.	0.028
Controls on Stream Migration	
Lateral	Generally none; spur dikes US/DS of abutments guide flow locally.
Vertical	None
Sediment Deposits & Bars	(US) Low to moderate height point, alternate and middle bars.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	Some exposure due to position of low flow thalweg near east side piers.
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low to moderate rate; land use a combination of agricultural, high density residential, and industrial. General assumption is for increasing urbanization.
Sand & Gravel Extraction	Has occurred/currently underway US/DS of bridge. Note development of elongate pond in old pit approximately 1500 ft. DS of bridge.
Freeway Construction	No
Dams	No
Drainage Channels	Colter channel joins river approximately 400 ft. west abutment.

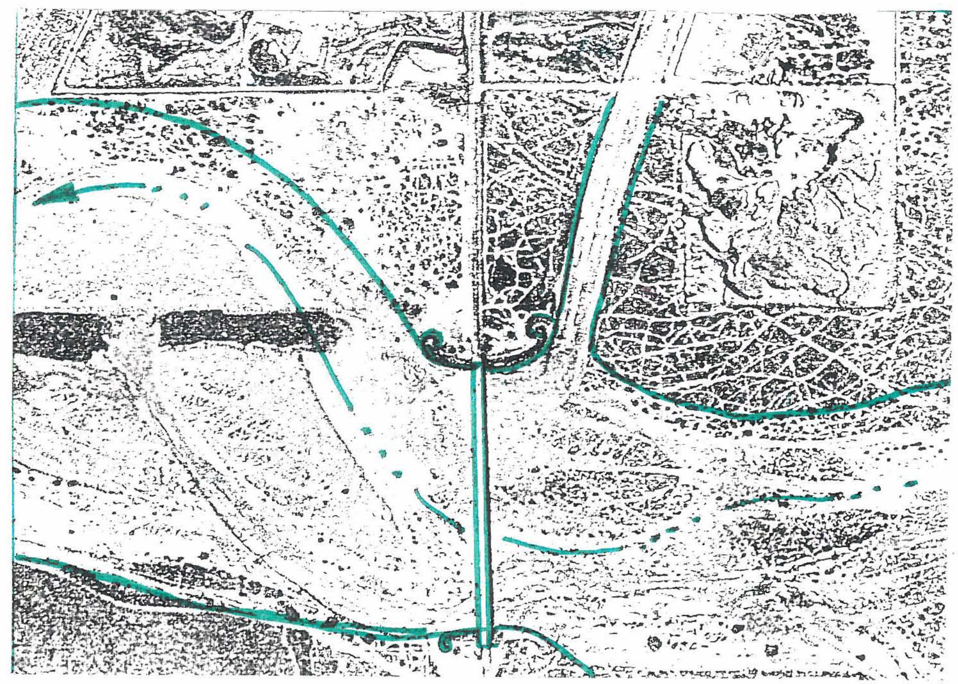


Figure 4

There are no upstream or downstream grade control structures in the immediate area that would maintain vertical stability of the river bed at the bridge site. The abandoned gravel mining operation in the channel described previously is thought to have a significant effect on channel elevation. Observation of incising of the channel downstream of the bridge indicates that the long-term tendency of the stream bed is to degrade.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Camelback Road come from controlled releases from the New Waddel Dam at Lake Pleasant, from New River Dam on New River and from the Colter Drainage Channel. Detention basins along Trilby Wash at the base of the White Tank Mountains also discharge to the Agua Fria River approximately 15 miles upstream of the bridge via the McMicken Dam Outlet Channel. 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 the design of the bridge are 95,000 cfs and 100 years, respectively.

2. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978, as measured at El Mirage, Arizona at Grand Avenue, approximately 3 miles upstream of the bridge.

3. The latest hydrologic model available from the Flood control District of Maricopa county (FDCMC) estimates a 100-year flood at the bridge of 95,000 cfs.

4. The 500-year flood (superflood) reported by FEMA is 184,000 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
1019.50	23,000	Q_{10}
1022.18	69,000	-
1023.32	95,000	Q_{100}
1026.45	184,000	Q_{500}
1027.16	207,300	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 184,000 cfs.

<u>**SCOUR CALCULATIONS:**</u> 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). Using approximate methods, it was calculated that long-term degradation or general scour, would be 10' at the bridge due to headcutting of a large gravel pit downstream of the bridge. (The armoring potential of the river against headcutting and degradation in general 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 present in full in the *Technical Appendix*.

Table 2. Summary of Scour Calculations

		Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{500})
a.	Scour at Piers		
	Contraction Scour, ft	0.1	0.2
	Local Scour, ft	25.4	29.2
	General Scour, ft	10.0	10.0
	Total Scour, ft	**35.5**	**39.4**
b.	Scour at Abutments		
	Abutment Scour, ft	6.7	11.4
	General Scour, ft	10.0	10.0
	Total Scour, ft	**16.7**	**21.4**

Table 3. Summary of Drilled Shaft Embedment

		Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{500})
a.	Embedment at Piers		
	Channel Elevation	1011.4	1011.4
	Total Scour, ft	35.4	39.4
	Elev. Bottom of Scour Hole	976.0	972.0
	Drilled Shaft Tip Elev.	946.0	946.0
	Embedment Remaining, ft	**30.0**	**26.4**
b.	Embedment at Abutments		
	Channel Elevation	1011.4	1011.4
	Total Scour, ft	16.7	21.4
	Elev. Bottom of Scour Hole	994.7	990.0
	Drilled Shaft Tip Elev.	946.0	946.0
	Embedment Remaining, ft	**48.7**	**44.0**

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 Camelback Road Bridge at the Agua Fria River has sufficient structural capacity to resist the loads resulting from flows up to and including 184,000 cfs, *i.e.*, the 500-year flow rate. The bridge is scour *stable*.

DEFICIENCIES AND COUNTERMEASURES: The existing riprap keyed into the channel banks should be grouted to avoid undermining of the riprap.

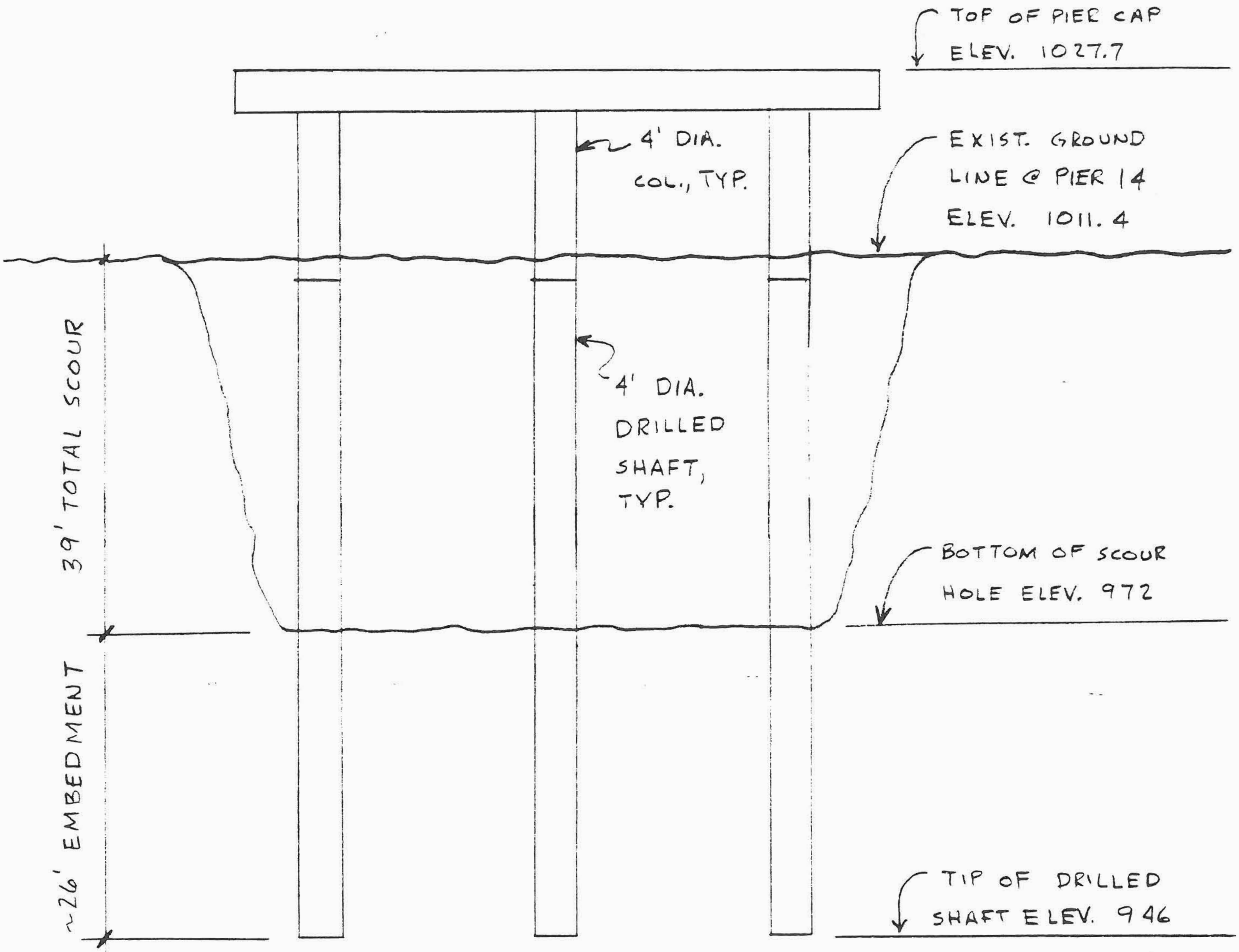
CAMELBACK ROAD BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM/CSSA):

Q500 = 184,000 CFS

H.W. ELEVATION = 1026.45

TOTAL SCOUR = 39'



PIER ELEVATION

Figure 5

Photo 1: View looking toward downstream face of structure from approximately mid-channel. Note that channel surface sediments consist primarily of gravelly sand with cobbles locally. Channel is not well vegetated near structure with occasional sparse grasses, low brush, and mesquite established on the low bars.

Photo 2: View looking northeast across thalweg of upstream low flow channel near east abutment. Note that channel banks are generally not sharply defined with overall vertical relief ranging from 3 to 8 feet.

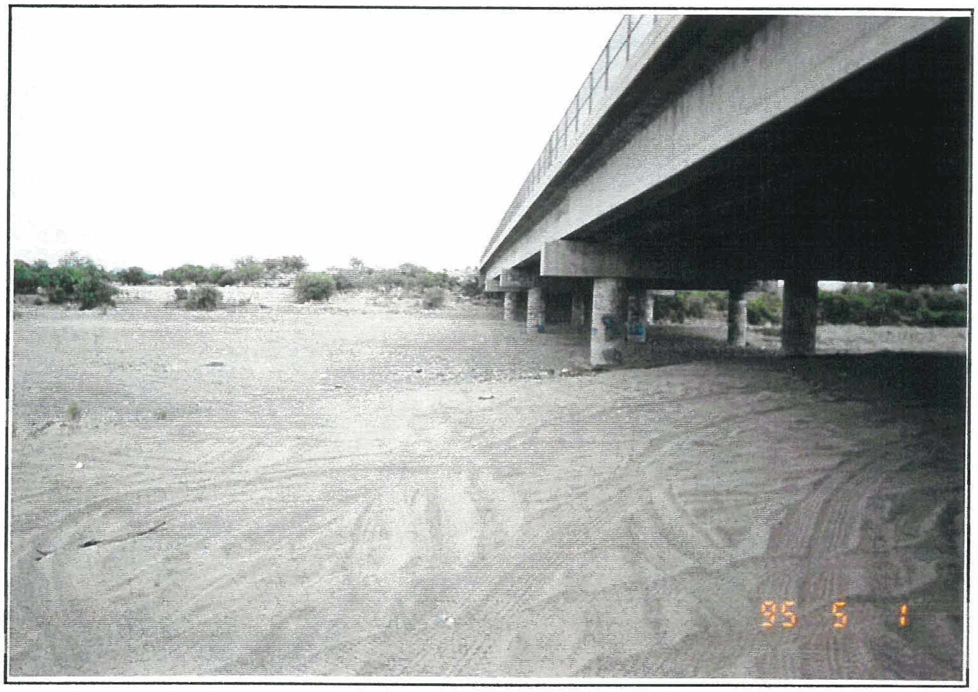


Photo 3: View looking toward east abutment along upstream face of bridge. Note position of low flow channel near abutment. Also note variation in surface sediments from coarse sand forming bar to gravel and cobbles visible in low flow channel.

Photo 4: View looking toward upstream end of third pile bent from east abutment. Note position of relatively coarse material armoring channel locally. Also note variation in pile exposure along bent is approximately 4 feet.

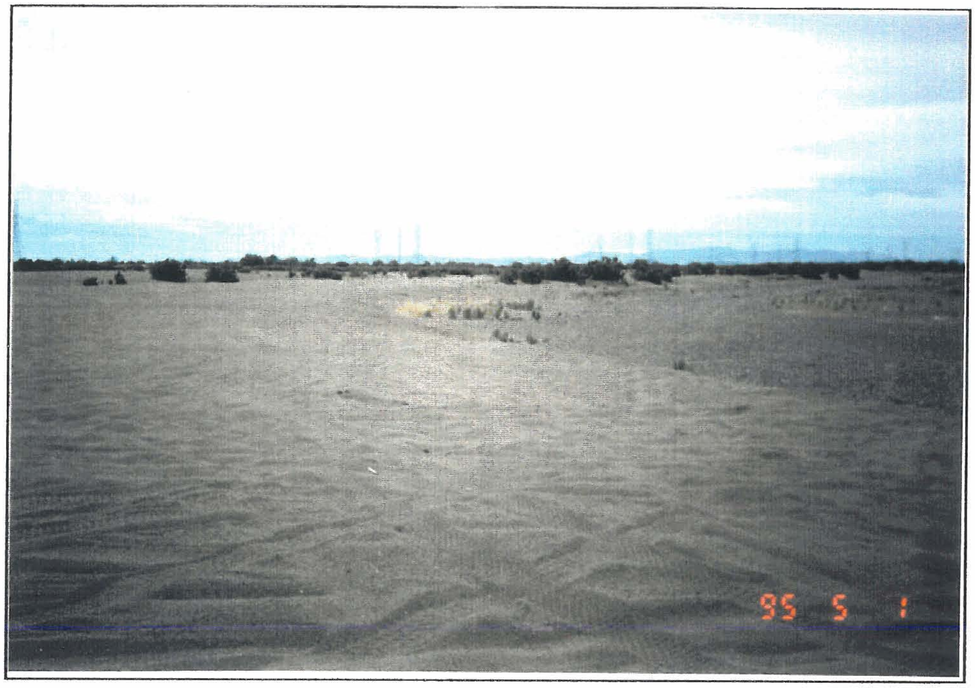


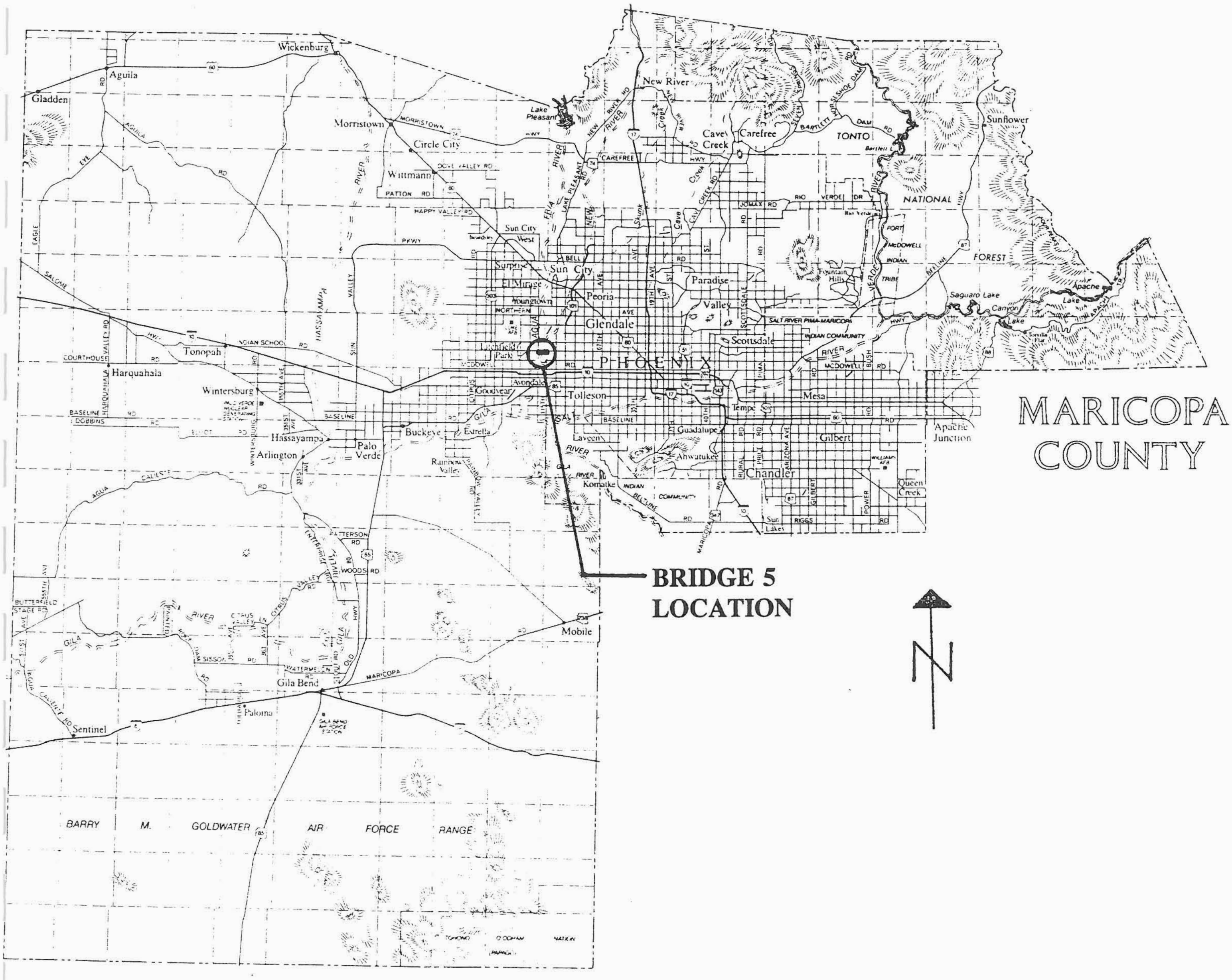
Photo 5: View looking approximately north upstream of structure. Note meandering form of low flow channel and relative coarseness of sediment in contrast to main channel.

Photo 6: View looking approximately northwest near right bank of main channel upstream of structure. Note trapezoidal channel joining main channel at right angle upstream of west abutment.

Photo 7: View looking downstream from a position approximately 1500 feet downstream of west abutment. Note formation of elongate pond possibly in location of former materials pit. Water surface is approximately 6 feet below main channel surface.

BRIDGE 5

INDIAN SCHOOL ROAD BRIDGE OVER AGUA FRIA RIVER



Location Map

Figure 1

BRIDGE 5: INDIAN SCHOOL ROAD BRIDGE OVER AGUA FRIA RIVER

(Structure #9145)
Assessment: Scour Critical

LOCATION: The Indian School Road Bridge at the Agua Fria River lies on the section line between Sections 24 and 25 of T2N R1W, coincident with the city limit of the Town of Avondale, Arizona. See Location Map, Figure 1.

STRUCTURE: The bridge has eighteen spans with a total length of 1,618' from center-to-center of abutment bearings. The original bridge was constructed in 1970 and widened in 1974 (MCDOT Project 60300) to provide two 36-foot-wide roadways separated by a concrete median barrier. The widening was designed for a stream flow rate of 73,800 cubic feet per second (cfs), corresponding to a flood frequency interval of 50 years. A plan and elevation of the bridge widening is shown in Figure 2. In 1983, the girders of Spans 13 through 18 and Pier Nos. 13 through 17 were replaced with new girders and piers (MCDOT Project No. 68074), replacing a section of the bridge that was destroyed by flooding in the river in 1978. The partial bridge replacement was designed for 94,000 cfs, corresponding to a flood frequency interval of 100 years. A plan and elevation of the replacement is provided in Figure 3. Plans for the original bridge were not found in MCDOT records.

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck. Expansion joints are provided at the abutments and at Pier Nos. 3, 6, 9, 12, and 15.

The abutments for the 1974 widening consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on eleven driven steel piles. The piles are founded approximately 17' below the existing river flow line. Both abutments are oriented at an 11 degree skew with respect to the bridge centerline.

The original bridge piers and the piers for the 1974 widening are the solid wall type on concrete spread footings. The upstream and downstream ends of the wall are constructed with a 90 degree nosing. The footings are founded approximately 14' below the existing river flow line. The bridge piers for the 1983 partial replacement consist of a concrete bent under each roadway. Each bent is comprised of a concrete cap beam supported by two 4' diameter drilled shafts that are founded approximately 65' below the existing groundline. All piers are oriented at an 11 degree skew with respect to the bridge centerline.

EXISTING SCOUR PROTECTION: Scour protection at the abutments is provided by banks stabilized with soil cement and grouted riprap. The soil-cement banks have been so constructed such the east abutment is approximately 50' behind the top of the bank; on the west end of the bridge the soil cement bank completely encases the west abutment and the first bridge pier (Pier 1).

Approximately 700' and 1,300' upstream of the bridge, two large soil cement-lined spur dikes have been constructed in the east overbank. The purpose of these dikes is to direct flows in the east overbank towards the bridge opening.

Scour protection of the piers founded on spread footings consists of dumped riprap around the pier above the footing. No scour protection was provided around the piers founded on drilled shafts.

A site inspection showed that the soil cement banks and grouted riprap at the abutments and the soil cement spur dikes were in good condition. Some residual scour was observed around the piers on spread footings; in some cases the riprap placed around the piers was exposed.

STREAM FORM: The Agua Fria River at Indian School Road is a braided stream, with shallow to medium height point and middle bars distributed irregularly throughout the river bed. (See Figure 4.) There is one principal low flow channel near east bank.

LAND USE: Land use in the surrounding area is primarily agricultural and industrial with some residential developments in the vicinity. Gravel mining operations are located in the west overbank downstream of the bridge; there was significant gravel mining in the channel upstream of the bridge, the remnant of which is still visible.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest sources of runoff will continue to be releases from New Waddell Dam on the Agua Fria River approximately 25 miles upstream of the bridge, and New River Dam on New River, approximately 16 miles upstream.

SURFACE SOILS: Surface soils consist primarily of sand and gravel, with some cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 1.5 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of the river bed in the vicinity of the Indian School Road Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0019 ft/ft or approximately 10' per mile.

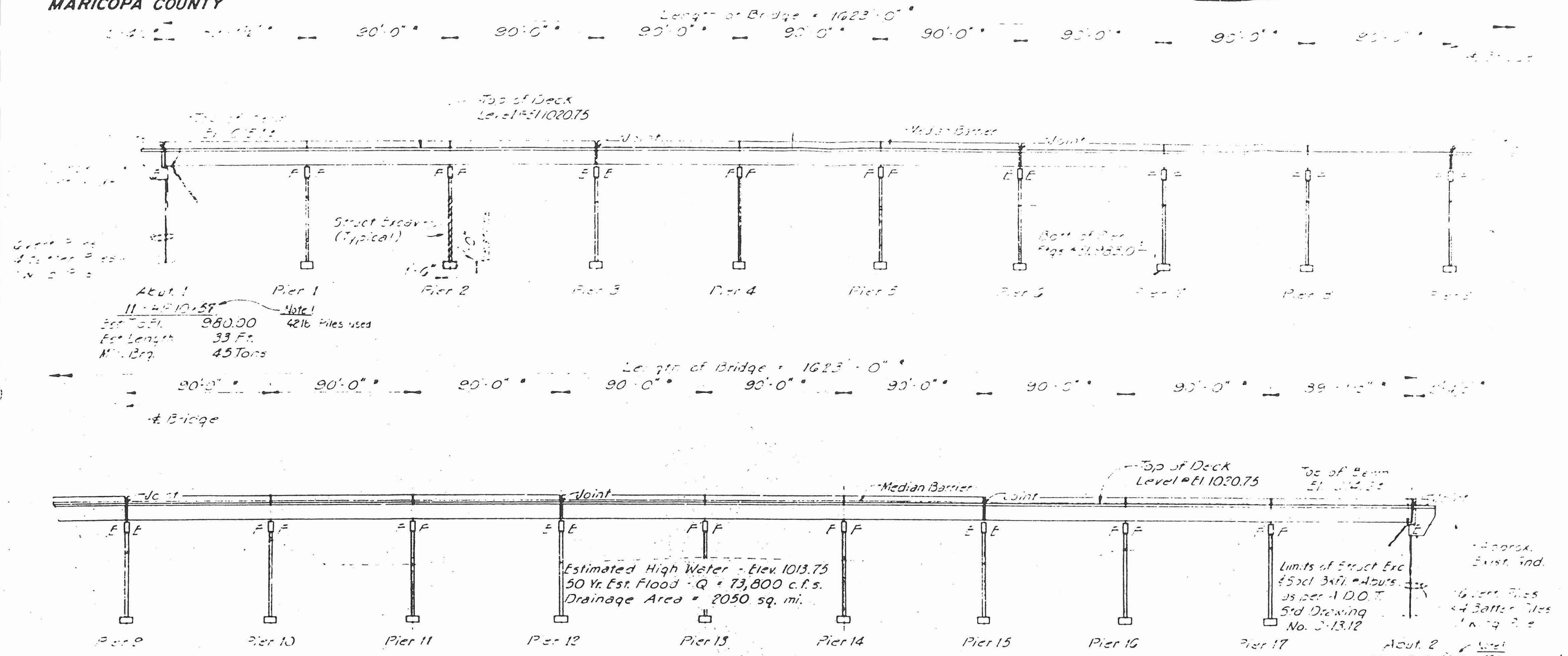
VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River includes bushes and shrubs such as desert willow, creosote, salt bush and ephedra; and low grasses. The overall density of vegetation in the river is estimated to be very low to nearly barren, probably due to past gravel mining and channelization of the river. Debris collecting on piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Local lateral stability of the river at the bridge is provided by the soil cement banks and riprap at the abutments. Aerial photographs showed that between 1982 and 1995 the low-flow channel has varied between the east abutment and the middle third of the bridge; therefore it must be assumed that the low flow channel may move laterally to any of the piers when water is flowing in the river.

Vertical stability of the channel is provided by a soil cement grade control structure approximately 150' downstream of the bridge. The grade control structure is assumed to prevent appreciable degradation (general scour) in the vicinity of the bridge.

INDIAN SCHOOL ROAD BRIDGE WIDENING AT AGUA FRIA RIVER
EL MIRAGE RD. to 115 th AVE.
MARICOPA COUNTY

		60300	10	32	4ET 78 RAK



E L E V A T I O N @ N E W C O N S T R U C T I O N ₵

Scales: 1" = 30' Horiz
1" = 15' Vert.

A P P R O X I M A T E   Q U A N T I T I E S

Item	Structural Excav.(CY)	Special Bckfl.(CY)	Concrete (CY) Class 'A'	Concrete (CY) Class 'B'	Reinforcing Steel (Lbs)	Exist. H-2-1 Hndr'l.[1] (LF)	Precast Girders (ea)	Piling No.	Piling Lin Ft.	#6 Dwls.[4] (Ea)
Deck[2]	—	—	1858.22	—	448,535	1620.14	90	—	—	—
2 Abuts.[2]	95	325	—	65.46	4820	—	—	22	726	26
17 Piers[2]	6880	—	—	1743.82	262,825	—	—	—	—	306
2 Appr. Slabs[3]	—	—	—	43.67	6,210	—	—	—	—	—
Med. Barr.[3]	—	—	165.85	—	27,040	—	—	—	—	—
Totals	6975	325	2024.07	1852.95	749,430	1620.14	90	22	726	332

[1] Remove and reset existing handrail. See Special Provisions.
[2] Quantities include respective closures.
[3] Does not include barrier transitions. See Roadway Sheets.
[4] Includes furnishing and installing #6 dowels. See Special Provisions.

* These dimensions are nominal. New abutment and pier extensions to be aligned in plan view with existing abutments and piers.

P I L E   N O T E

Any pile which is driven to a bearing of 1¾ times the bearing given on the plans and reaches a tip elevation within three feet of the estimated tip elevation shall be considered acceptable.

MARICOPA COUNTY HIGHWAY DEPARTMENT
Noted
Dec '76
ELEVATION
DRD
STA. 26+ INDIAN SCHOOL ROAD BRIDGE OVER AGUA FRIA RIVER
HOFFMAN-MILLER ENGINEERS INC. PHOENIX ARIZONA
2 of 20

FIGURE 2

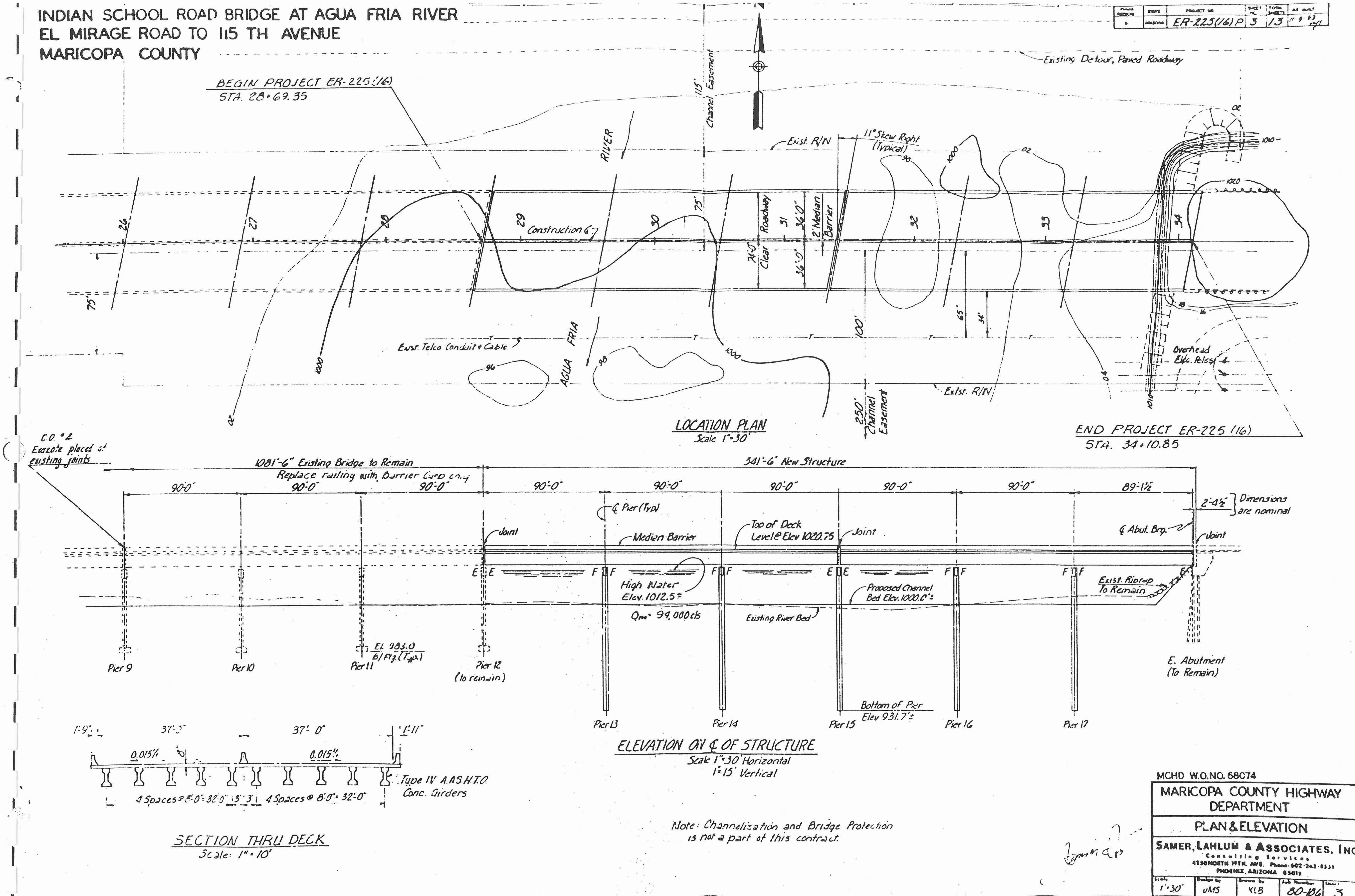

INDIAN SCHOOL ROAD BRIDGE AT AGUA FRIA RIVER
EL MIRAGE ROAD TO 115 TH AVENUE
MARICOPA COUNTY
ER-225(16)P 3 13
Existing Detour, Paved Roadway
BEGIN PROJECT ER-225(16)
STA. 28+69.35
115' Channel Easement
RIVER
Exist. R/W
11° Skew Right (Typical)
Construction ℄
Clear Roadway
2' Median Barrier
Exist. Telco Conduit & Cable
AGUA FRIA
250' Channel Easement
Exist. R/W
Overhead Elec. Poles
LOCATION PLAN
Scale 1"=30'
END PROJECT ER-225 (16)
STA. 34+10.85
C.O. #2
Evazote placed at existing joints
1081'-6" Existing Bridge to Remain
Replace railing with Barrier Curb only
541'-6" New Structure
90'-0"
89'-1½"
2'-4½"
Dimensions are nominal
℄ Pier (Typ)
Joint
Medien Barrier
Top of Deck Level @ Elev 1020.75
℄ Abut. Brg.
High Water Elev. 1012.5±
Q₁₀₀ = 94,000 cfs
Proposed Channel Bed Elev. 1000.0'±
Exist. Riprap To Remain
Existing River Bed
El. 983.0 B/Ftg. (Typ.)
Pier 9
Pier 10
Pier 11
Pier 12 (to remain)
Pier 13
Pier 14
Pier 15
Pier 16
Pier 17
Bottom of Pier Elev 931.7'±
E. Abutment (To Remain)
ELEVATION ON ℄ OF STRUCTURE
Scale 1"=30' Horizontal
1"=15' Vertical
0.015'/'
37'-0"
4 Spaces @ 8'-0" = 32'-0"
Type IV A.A.S.H.T.O. Conc. Girders
SECTION THRU DECK
Scale: 1"=10'
Note: Channelization and Bridge Protection is not a part of this contract.
MCHD W.O.NO. 68074
MARICOPA COUNTY HIGHWAY DEPARTMENT
PLAN & ELEVATION
SAMER, LAHLUM & ASSOCIATES, INC.
Consulting Services
4250 NORTH 19TH AVE. Phone: 602-263-8331
PHOENIX, ARIZONA 85015
1"=30'
UMS
KLB
80-106
3

Figure 3

INDIAN SCHOOL ROAD (SN 9145)

Water Course	Agua Fria River
Stream Form	Braided
Sinuosity	Not applicable
General Channelization	Soil cement-lined channel US/DS of bridge. Large soil-cement spur dikes on east side US of bridge.
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.001547
Channel Contraction/Expansion	Wider US
Primary Surface Sediment Type	sand/gravel
D50 Size	1.5 MM (estimate)
Armoring Potential	Low
Channel Vegetation	
Type/Size	Desert Broom to 4 ft., Desert Willow; Creosote, Salt Bush, Ephedra; low grasses.
Density/Occurrence	Generally very sparse US/DS.
Relative Age	Young
Manning's Roughness Coef.	0.025
Controls on Stream Migration	
Lateral	Soil cement-lined channel US/DS of bridge.
Vertical	Possible broken grade control structure approximately 150 ft. DS of bridge.
Sediment Deposits & Bars	Irregular distribution of point and middle bars.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	Maximum residual scour 3-4 ft. observed at some solid piers.
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low to moderate rate; land use a combination of agricultural and industrial. General assumption is for increasing urbanization.
Sand & Gravel Extraction	Past excavation of US/DS channel. Current excavation of US/DS overbanks with channel used as haul road.
Freeway Construction	No
Dams	No
Drainage Channels	No. Roosevelt Irrigation District (RID) canal DS of structure.

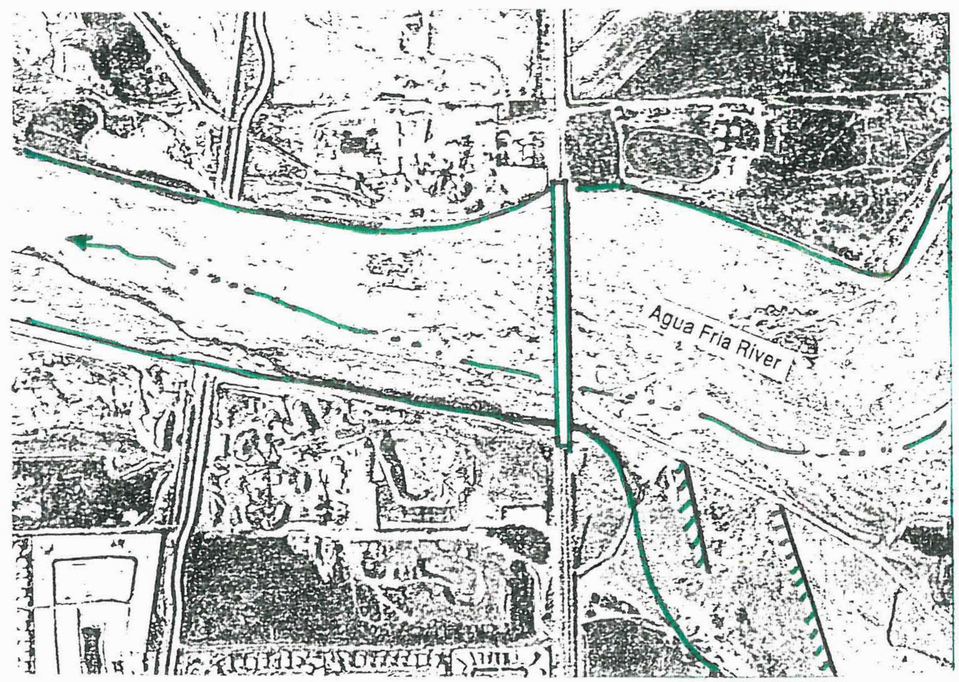


Figure 4

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Indian School Road come from controlled releases form the New Waddel Dam at Lake Pleasant, from New River Dam on New River, 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 the design of the bridge are 95,000 cfs and 100 years, respectively.

2. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978, as measured at El Mirage, Arizona at Grand Avenue, approximately 3 miles upstream of the bridge.

3. The latest hydrologic model available from the Flood control District of Maricopa County (FDCMC) estimates a 100-year flood at the bridge of 95,000 cfs.

4. The 500-year flood (superflood) reported by FEMA is 184,000 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 int the analysis for completeness, 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
1004.44	23,000	Q_{10}
1007.48	69,000	-
1008.79	95,000	Q_{100}
1012.43	184,000	Q_{500}
1015.03	261,600	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 184,000 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). Using approximate methods, it was calculated that long-term degradation or general scour, would be 10' at the bridge due to headcutting of a large gravel pit downstream of the bridge. (The armoring potential of the river against headcutting and degradation in general 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 foundation embedment are shown in Tables 2 and 3, respectively. Schematic representations of scour at the piers during Q_{500} are shown in Figures 5 and 6. Scour calculations are presented in the *Technical Appendix*.

Riprap at piers with spread footings was not considered too effective against scour because it is not continuous across the channel and therefore prone to undermining and displacement during high flows.

Table 2. Summary of Scour Calculations

		Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{500})
a.	Scour at Piers on Spread Footings		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	20.8	24.1
	General Scour, ft	0.0	0.0
	Total Scour, ft	**20.8**	**24.1**
b.	Scour at Piers on Drilled Shafts		
	Contraction Scour, ft	0.0	0.0
	Local Scour, ft	20.8	29.3
	General Scour, ft	0.0	0.0
	Total Scour, ft	**20.8**	**29.3**

Table 4. Summary of Foundation Embedment
(Piers 2 through 12)

	$Q = 95,000$ cfs (Q_{100})	$Q = 184,000$ cfs (Q_{500})
a. Embedment at Piers on Spread Footings		
Channel Elevation	999.6	999.6
Total Scour, ft	~~20.8~~ 30.8'	~~24.1~~ 34.1'
Bottom of Scour Hole Elev.	~~978.6~~ 968.8'	~~975.5~~ 965.5'
Bottom of Footing Elev.	983.0	983.0
Embedment Remaining, ft	-4.4 -14.2'	-7.5 -17.5'
b. Embedment at Piers on Drilled Shafts		
Channel Elevation	997.0	997.0
Total Scour, ft	~~20.8~~ 30.8'	~~29.3~~ 39.3'
Bottom of Scour Hole Elev.	~~976.2~~ 966.2'	~~967.7~~ 957.7'
Drilled Shaft Tip Elev.	932.0	932.0
Embedment Remaining, ft	~~44.2~~ 34.2	~~35.7~~ 25.7'

(Handwritten: a. — Not OK. Scour below the footing of bridge piers; b. — OK)

STRUCTURAL EVALUATION: A structural analysis of Piers 13 through 17 was performed for the Q_{500} flood event 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 because scour depths are negligible. Structural calculations are presented in the *Technical Appendix*.

CONCLUSIONS: Based on the structural evaluation, Piers 13 through 17 of the Indian School Road Bridge at the Agua Fria River have sufficient structural capacity to resist loads resulting flows up to and including 184,000 cfs, *i.e.*, the 500-year flow rate. The spread footings of Piers 2 through 12, however, do not bear on rock material and do not have sufficient embedment at Q_{100} or Q_{500}. The bridge is scour *critical*.

COUNTERMEASURES:

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 between and around the scour critical piers, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

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

d. Replace the bridge with a new bridge supported on deep foundations.

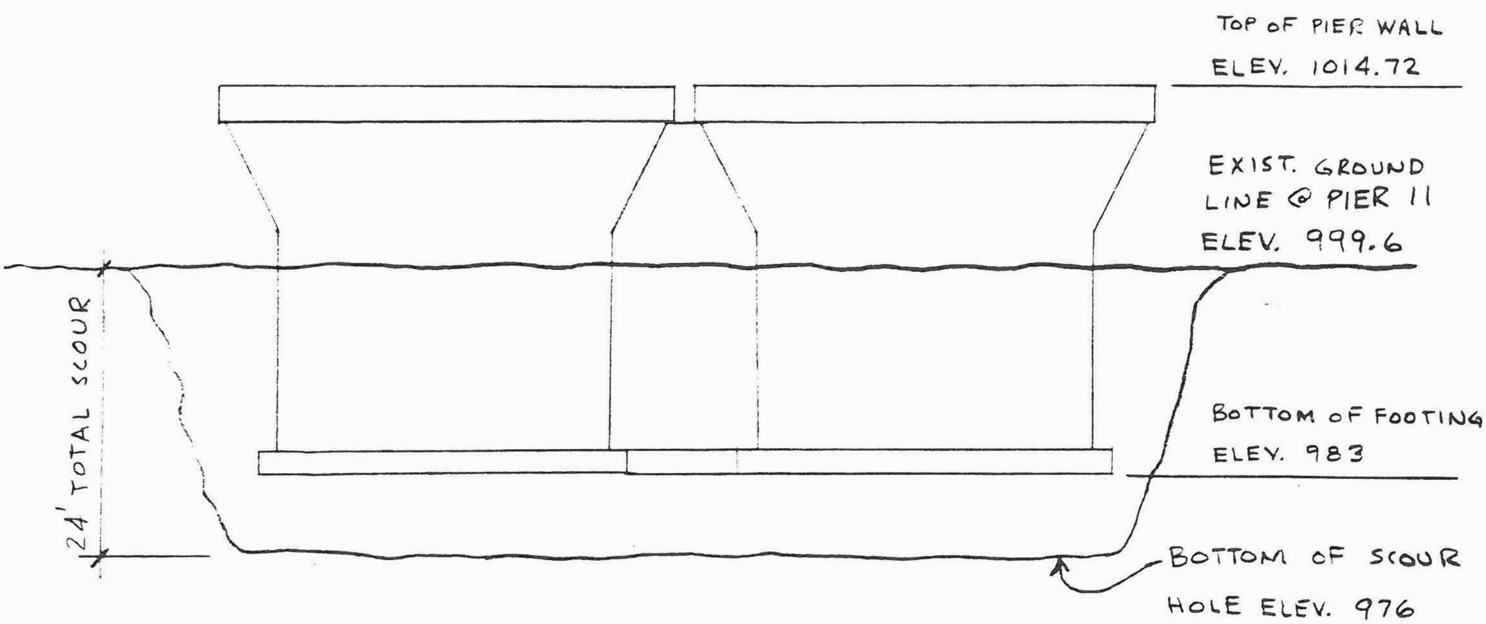
INDIAN SCHOOL ROAD BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM/CSSA):

Q500 = 184,000 CFS

H.W. ELEV. = 1012.43

TOTAL SCOUR = 24'



PIERS 1 THRU 12 ELEVATION

Figure 5

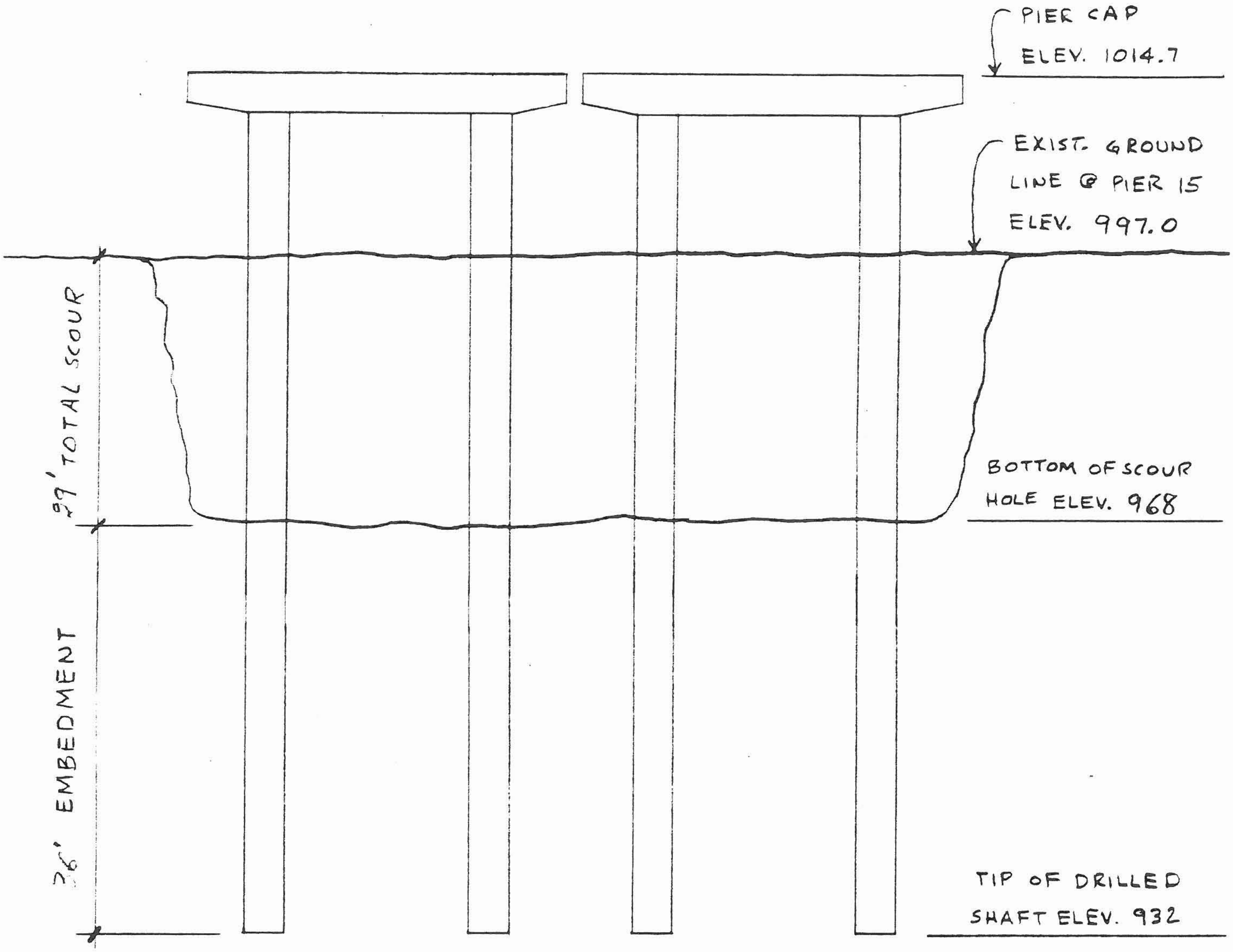
INDIAN SCHOOL ROAD BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM / CSSA):

Q500 = 184,000 CFS

H.W. ELEV. = 1012.43

TOTAL SCOUR = 29'



PIERS 13 THRU 17 ELEVATION

Figure 6

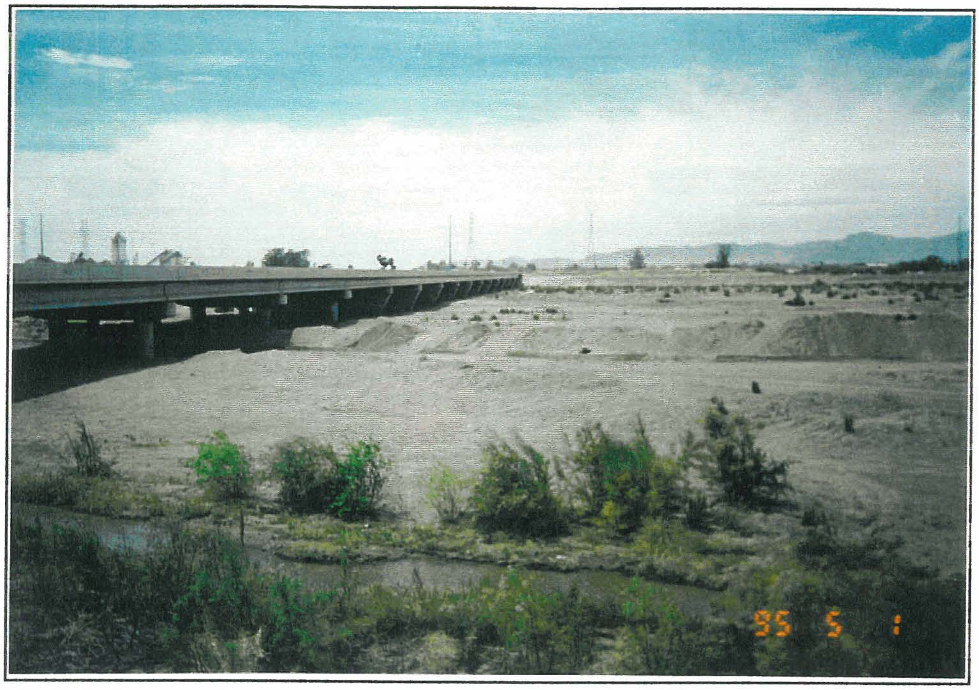


Photo 1: View looking west across upstream channel from top of spur dike. Note that upstream channel is composed primarily of gravelly sand with coarser material occurring locally. In general, vegetation is relatively sparse consisting primarily of creosote and salt bush on low bars. Also note materials pit haul road in foreground and irrigation return flow channel sustaining vegetation locally.

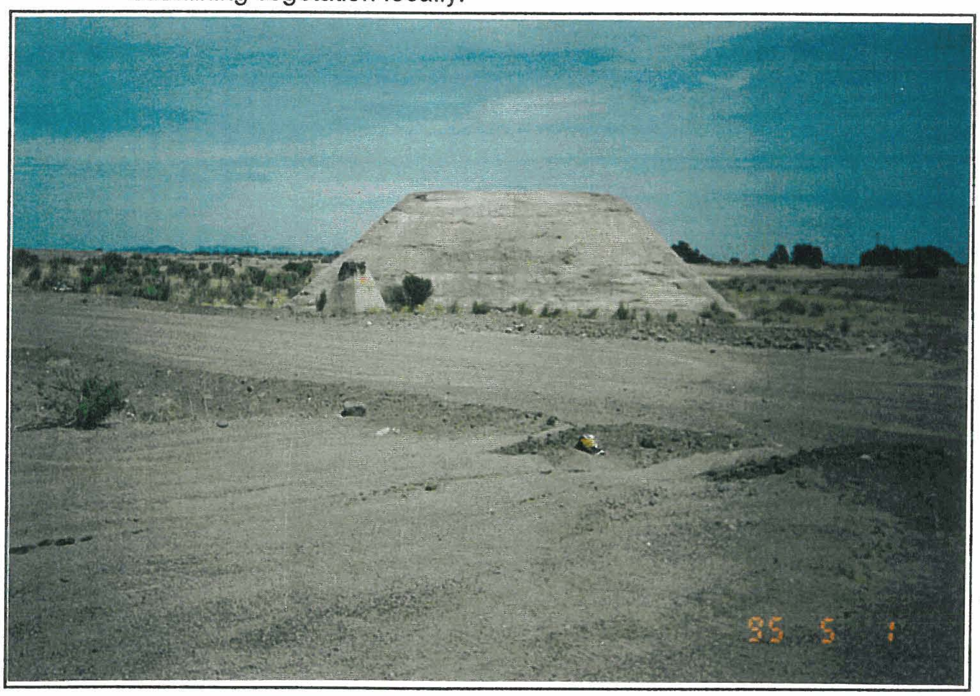


Photo 2: View looking northeast at large deflector dike located upstream of east abutment spur dike. Trapezoidal dike evidently directs flow away from east approach embankment inward toward bridge waterway.

Photo 3: View looking toward west abutment from east side of main channel downstream of structure. Note relatively coarse surficial sediments of low flow channel with finer sands forming low bars in background. Note relatively sparse distribution of small brush on bars. Also note channelized banks typical of upstream and downstream channel along this reach in background.

Photo 4: View looking downstream from approximately middle of downstream channel. Note relatively shallow low flow channel and sparse vegetation typical of this reach.

Photo 5: View looking at downstream face of structure. Note primary low flow channel crossing concrete slab.

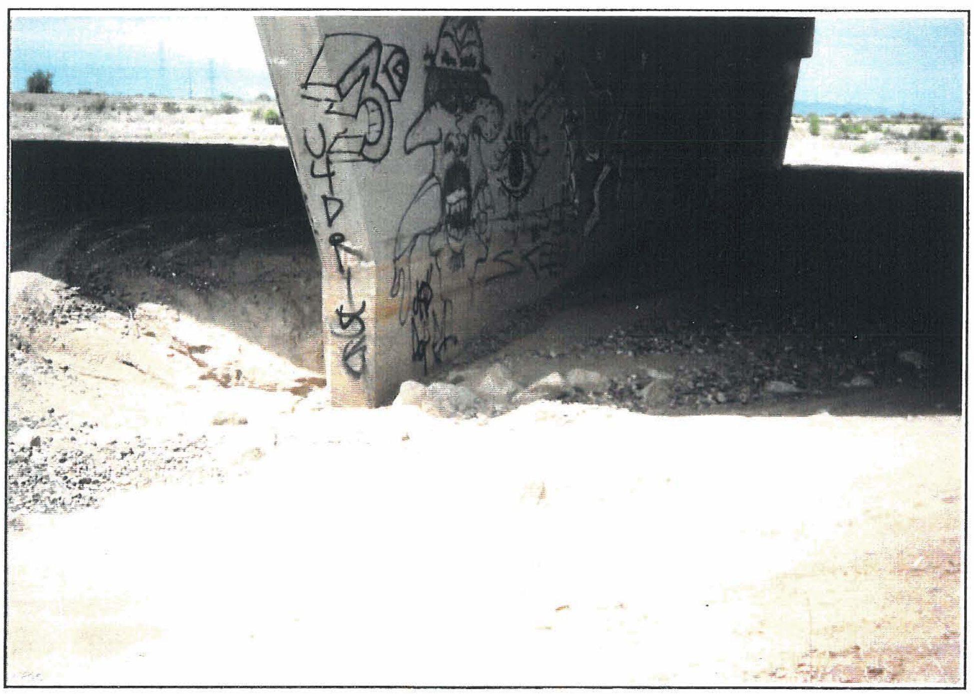
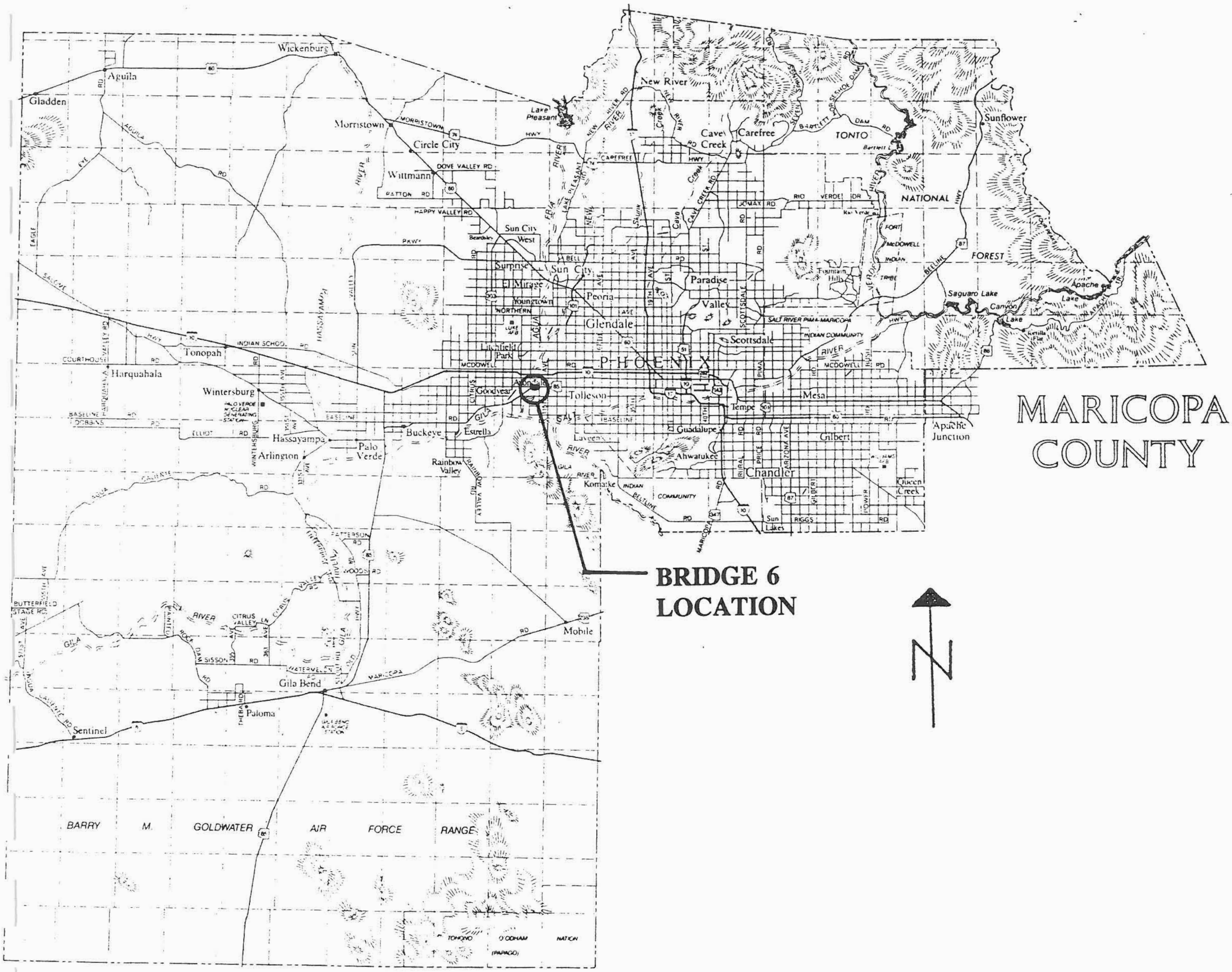


Photo 6: View looking at upstream face of typical solid pier. Note residual scour of 3-4 feet was most severe case observed. Also note staining of concrete and exposure of rip-rap.

Photo 7: View looking at upstream edge of first pier from west. Note the differential deposition on east side of pier and scour of west side was typical of several solid piers possibly indicating recent flow angle of attack may be slightly west of north.

BRIDGE 6

MARICOPA COUNTY HIGHWAY 85 BRIDGE OVER AGUA FRIA RIVER



Location Map

Figure 1

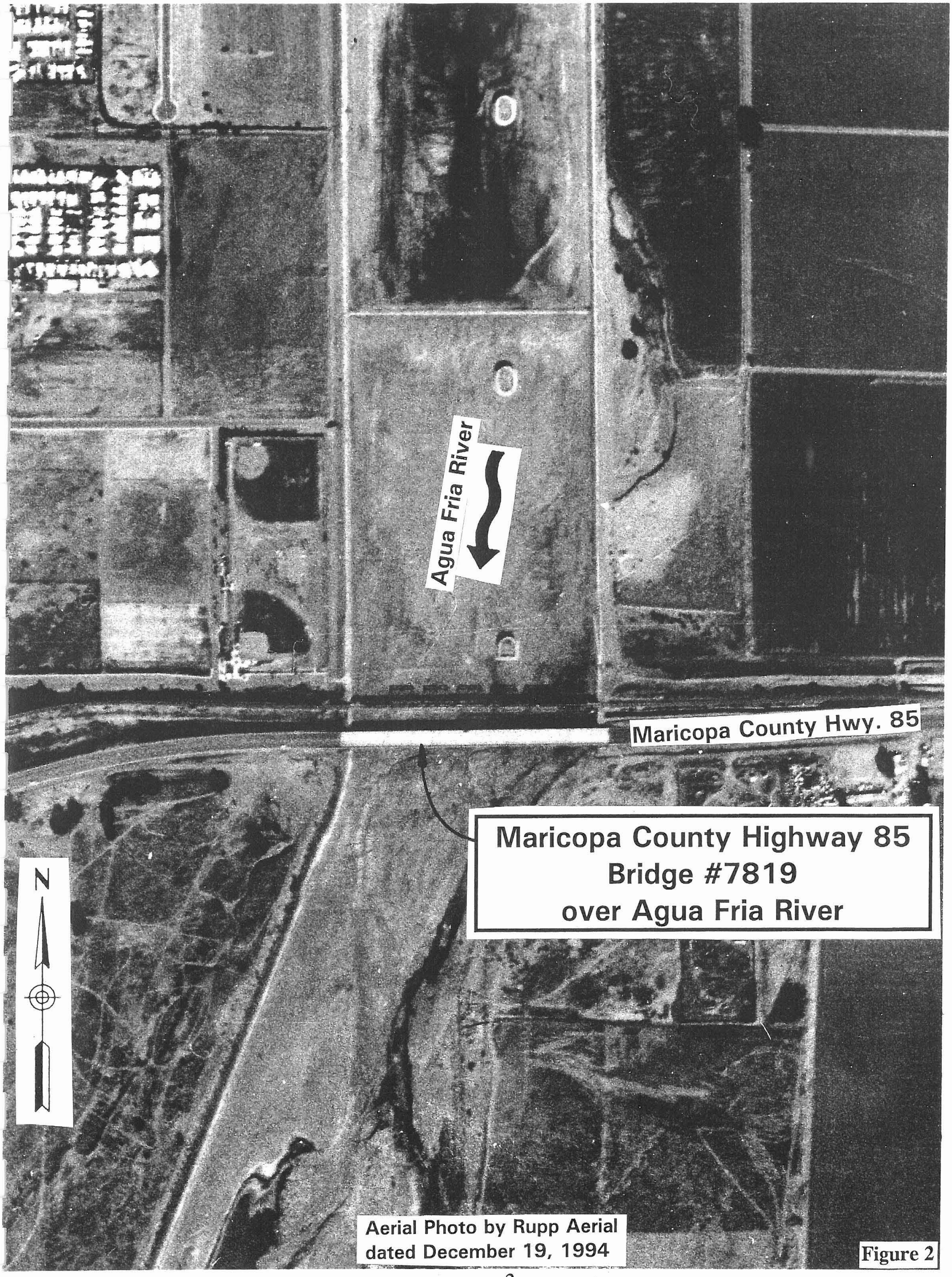

Agua Fria River
Maricopa County Hwy. 85
Maricopa County Highway 85
Bridge #7819
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

BRIDGE 6: MARICOPA COUNTY HIGHWAY 85 OVER AGUA FRIA RIVER

(Structure #7819)

Assessment: Scour Critical

LOCATION: The MC 85 Bridge at the Agua Fria River is located in Section 4, T1N, R1W, Gila and Salt River Baseline and Meridian, near the Town of Avondale, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The bridge has thirteen spans with a total length of 1,198.5' from center-to-center of abutment bearings. (See Location Plans, Figures 3 and 4.) It has a 68-foot-wide roadway and a sidewalk on the downstream side. The bridge was designed for a stream flow rate of 60,000 cubic feet per second (cfs), corresponding to a flood frequency interval of 50 years. The bridge was designed by the Bridge Division of the Arizona Highway Department (AHD) in 1969; it was built in 1974 as AHD Project No. BR-S-371(5).

The superstructure consists of prestressed concrete AASHTO I-girders with a cast-in-place reinforced concrete deck structure. Expansion joints are provided at the abutments and at Pier Nos. 3, 6, 9, and 12.

The abutments consist of a concrete cap beam, backwall, wingwalls, and an approach slab supported on sixteen driven steel pipe piles. Pile driving records indicate that the piles are founded between 12' and 17' below the existing river flow line. Both abutments have zero skew with respect to the bridge roadway.

The piers consist of a concrete cap beam supported on four 3'-6" diameter formed columns. The columns are supported on a pile foundation comprised of a concrete pile cap and twenty-six 16" diameter concrete-filled steel pipe piles. The bottom of the pile cap is approximately 3' below the existing river flow line. Pile driving records indicate that the piles are founded between 16' and 30' below the bottom of the existing pile cap. The piers are oriented normal to the roadway.

EXISTING SCOUR PROTECTION: No special scour protection was provided at the bridge when originally constructed. During a site investigation, it was found that the river has been lowered and channelized with soil cement banks. The bottom of the channel has been lowered approximately 4' from the 1974 elevation of Elevation 956 to approximately Elevation 952. The abutments are located approximately 35 feet behind the top of the soil cement banks, effectively encasing them in scour-resistant material. The soil cement lining under the bridge has been given a cap of sprayed-on gunite.

A 5' thick dumped riprap sill (estimated D_{50} = 20") has been constructed across the channel bottom. The sill measures 100' wide at the top, extending 25' on each side of the ends of the piers, and is keyed into the river bed at a 1.5:1 slope to a depth of approximately 15.5' on the downstream side of the bridge and 8.5' on the upstream side. During a site inspection it was noticed that some of the riprap has been transported downstream in places where the channel has suffered some localized erosion. Otherwise, the riprap appeared to be in good condition.

STREAM FORM: The Agua Fria River at MC 85 is a braided to straight stream with no significant bar formation upstream of the bridge and low to medium height middle bars distributed irregularly throughout the river bed downstream of the bridge. (See Figure 5.) There is no discernable low flow channel in the river bed upstream of the bridge. Downstream, a shallow channel has formed at approximately the fifth span from the west abutment. Irrigation return flows discharged to the river upstream of the east abutment have created a marshy strip along the east bank of the river downstream of the bridge.

LAND USE: Land use on the east side of the river is primarily agricultural with some commercial and industrial developments in the vicinity. Land use on the west side is the urbanized area of Avondale, Arizona. Former gravel mining operations are located in the river approximately one-half mile downstream of the bridge.

Urbanization is anticipated to increase, possibly increasing the magnitude and volume of runoff to the river from small thunderstorms of low-to-medium return frequency. In terms of significance to bridge scour, the largest sources of runoff will continue to be releases from New Waddell Dam on the Agua Fria River approximately 30 miles upstream of the bridge, and from New River Dam on New River, approximately 20 miles upstream.

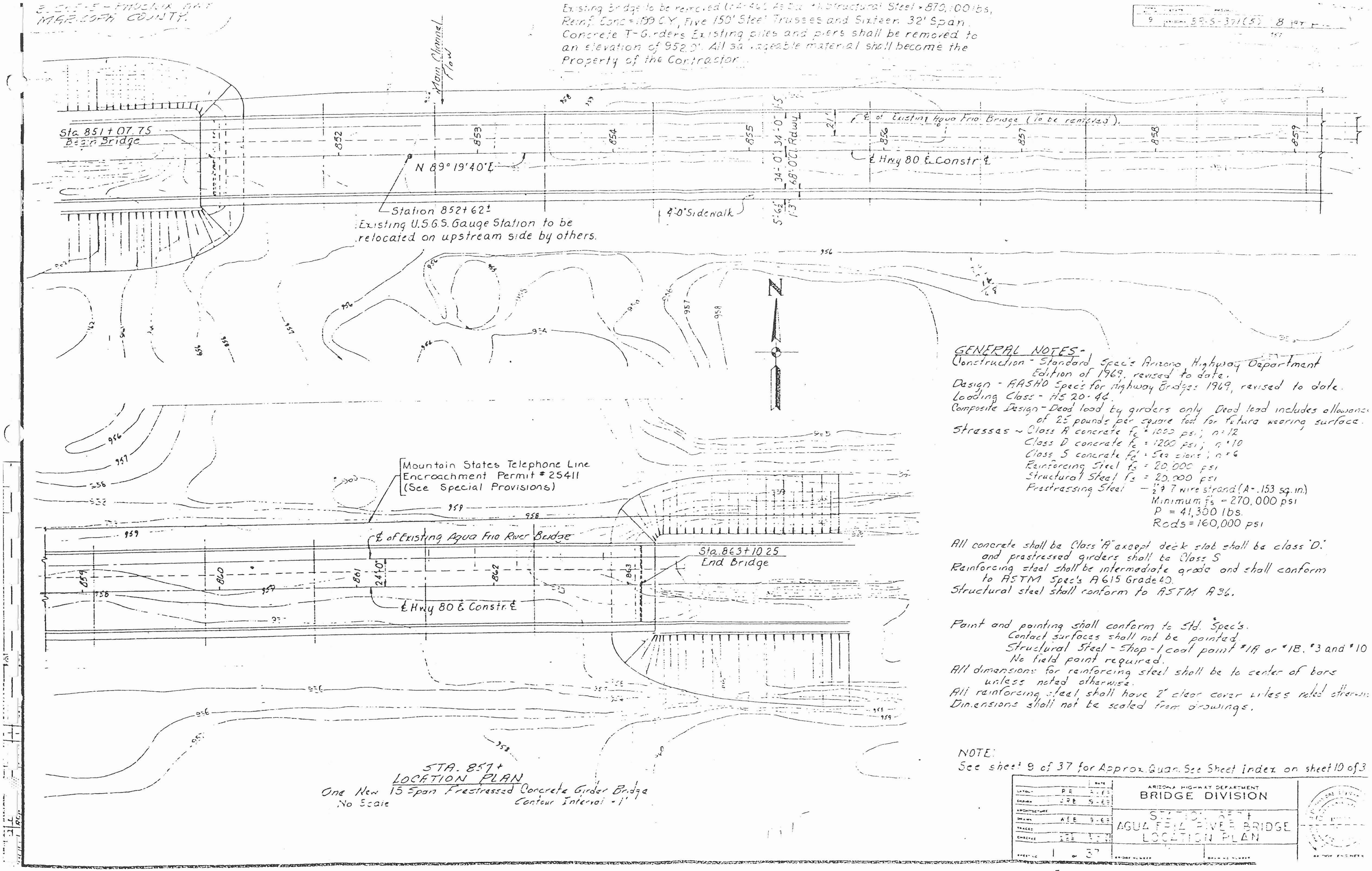
SURFACE SOILS: Surface soils consist primarily of sand and gravel, with some cobbles. The estimated median diameter (D_{50}) of the surface soil is approximately 1 mm. The armoring potential of the river bed is estimated to be low.

SLOPE: The slope of the river bed in the vicinity of the MC 85 Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0019 ft/ft or approximately 10' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the Agua Fria River include bushes and shrubs such as salt bush and ephedra; and low grasses. The overall density of vegetation in the river is estimated to be very low to nearly barren, probably due to past gravel mining and channelization of the river. Vegetation is taller and denser along the marshy strip described previously. Debris collecting on the piers is considered to be possible and is accounted for in the scour calculations.

STREAM STABILITY: Local lateral stability of the river at the bridge is provided by the soil cement banks and riprap at the abutments. Aerial photographs showed that between 1982 and 1995 the low-flow channel has varied between the east abutment and the middle third of the bridge; therefore, it must be assumed that the low-flow channel may move laterally to any of the piers when water is flowing in the river.

Vertical stability of the channel is provided by a soil cement grade control structure approximately 2000 feet upstream of the bridge. Vertical stability at the bridge is provided by large-diameter riprap placed across the bottom of the stream bed. This riprap is starting to lose stability in places, probably due to degradation of the channel downstream of the bridge.



Existing bridge to be removed. Structural Steel = 870,100 lbs, Reinf. Conc = 1159 C.Y., Five 150' Steel Trusses and Sixteen 32' Span Concrete T-Girders. Existing piles and piers shall be removed to an elevation of 952.0'. All salvageable material shall become the Property of the Contractor.
Main Channel Flow
Sta. 851+07.75 Begin Bridge
N 89° 19' 40" E
Station 852+62±
Existing U.S.G.S. Gauge Station to be relocated on upstream side by others.
4'-0" Sidewalk
Hwy 80 & Constr.
Existing Agua Frio Bridge (to be removed)
68'-0" Rdwy
Mountain States Telephone Line Encroachment Permit # 25411 (See Special Provisions)
of Existing Agua Frio River Bridge
Sta. 863+10.25 End Bridge
Hwy 80 & Constr.
GENERAL NOTES-
Construction - Standard Spec's Arizona Highway Department Edition of 1969, revised to date.
Design - AASHO Spec's for Highway Bridges 1969, revised to date.
Loading Class - HS 20-44.
Composite Design - Dead load by girders only. Dead load includes allowance of 25 pounds per square foot for future wearing surface.
Stresses ~ Class A concrete f'c = 1000 psi; n = 12
Class D concrete f'c = 1200 psi; n = 10
Class S concrete
Reinforcing Steel fs = 20,000 psi
Structural Steel fs = 20,000 psi
Prestressing Steel - ½" ø 7 wire strand (A = .153 sq. in.)
Minimum fs = 270,000 psi
P = 41,300 lbs.
Rods = 160,000 psi
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 Spec's A 615 Grade 40.
Structural steel shall conform to ASTM A 36.
Paint and painting shall conform to Std. Spec's.
Contact surfaces shall not be painted.
Structural Steel - Shop - 1 coat paint #1A or #1B, #3 and #10
No field paint required.
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:
See sheet 9 of 37 for Approx. Quan. See Sheet Index on sheet 10 of 37
STA. 857+
LOCATION PLAN
One New 15 Span Prestressed Concrete Girder Bridge
No Scale
Contour Interval = 1'
ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION
AGUA FRIA RIVER BRIDGE
LOCATION PLAN

Figure 3

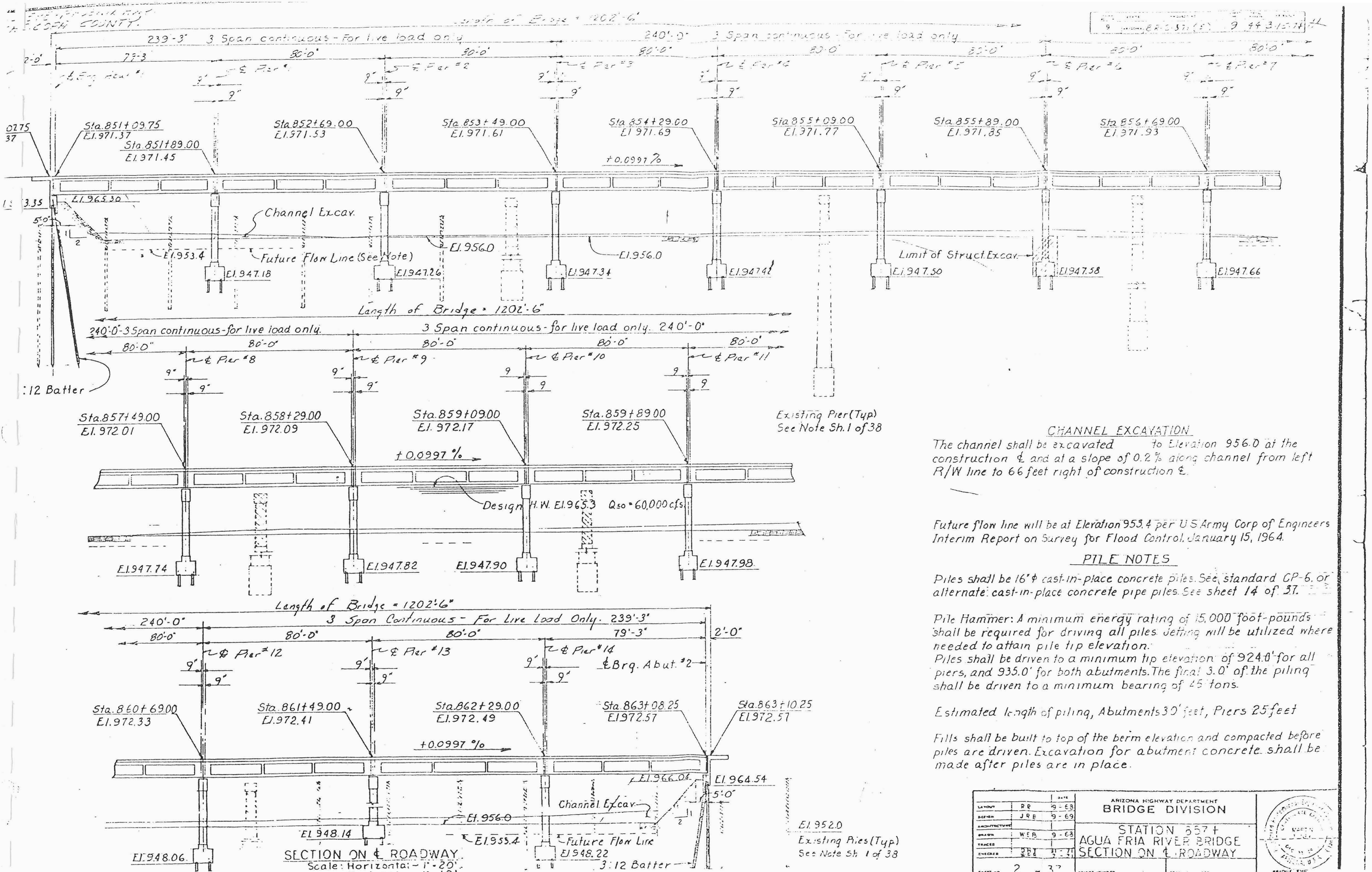

Length of Bridge = 1202'-6"
239'-3" 3 Span continuous - For live load only
240'-0" 3 Span continuous - For live load only
Sta. 851+09.75 El. 971.37
Sta. 851+89.00 El. 971.45
Sta. 852+69.00 El. 971.53
Sta. 853+49.00 El. 971.61
Sta. 854+29.00 El. 971.69
Sta. 855+09.00 El. 971.77
Sta. 855+89.00 El. 971.85
Sta. 856+69.00 El. 971.93
+0.0997 %
Channel Excav.
Future Flow Line (See Note)
El. 953.4
El. 956.0
Limit of Struct. Excav.
El. 947.18
El. 947.26
El. 947.34
El. 947.42
El. 947.50
El. 947.58
El. 947.66
3:12 Batter
240'-0" 3 Span continuous - for live load only.
3 Span continuous - for live load only. 240'-0"
Sta. 857+49.00 El. 972.01
Sta. 858+29.00 El. 972.09
Sta. 859+09.00 El. 972.17
Sta. 859+89.00 El. 972.25
Existing Pier (Typ) See Note Sh. 1 of 38
Design H.W. El. 965.3 Q50 = 60,000 cfs
El. 947.74
El. 947.82
El. 947.90
El. 947.98
3 Span Continuous - For Live Load Only. 239'-3"
Sta. 860+69.00 El. 972.33
Sta. 861+49.00 El. 972.41
Sta. 862+29.00 El. 972.49
Sta. 863+08.25 El. 972.57
Sta. 863+10.25 El. 972.57
℄ Brg. Abut. #2
El. 966.04
El. 964.54
El. 948.14
El. 948.06
El. 955.4
El. 948.22
Future Flow Line
El. 952.0
Existing Piers (Typ) See Note Sh. 1 of 38
SECTION ON ℄ ROADWAY
Scale: Horizontal - 1" = 20'
Vertical - 1" = 10'
CHANNEL EXCAVATION
The channel shall be excavated to Elevation 956.0 at the construction ℄ and at a slope of 0.2% along channel from left R/W line to 66 feet right of construction ℄.
Future flow line will be at Elevation 953.4 per U.S. Army Corp of Engineers Interim Report on Survey for Flood Control, January 15, 1964.
PILE NOTES
Piles shall be 16" ϕ cast-in-place concrete piles. See standard CP-6, or alternate cast-in-place concrete pipe piles. See sheet 14 of 37.
Pile Hammer: A minimum energy rating of 15,000 foot-pounds shall be required for driving all piles. Jetting will be utilized where needed to attain pile tip elevation.
Piles shall be driven to a minimum tip elevation of 924.0' for all piers, and 935.0' for both abutments. The final 3.0' of the piling shall be driven to a minimum bearing of 45 tons.
Estimated length of piling, Abutments 30 feet, Piers 25 feet
Fills shall be built to top of the berm elevation and compacted before piles are driven. Excavation for abutment concrete shall be made after piles are in place.
ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION
STATION 857+
AGUA FRIA RIVER BRIDGE
SECTION ON ℄ ROADWAY
2 of 37

Figure 4

MARICOPA COUNTY HIGHWAY 85 (SN 7819)

Water Course	Agua Fria River
Stream Form	Braided to straight ✓
Sinuosity	Not applicable
General Channelization	Soil cement-lined channel US of bridge and along west bank DS. Unlined on east.
Channel Slope	Steeper DS
Estimated Channel Slope (ft/ft)	0.002534
Channel Contraction/Expansion	Main channel narrower DS
Primary Surface Sediment Type	silt/sand/gravel
D50 Size	1 MM (estimate)
Armoring Potential	Low to moderate (locally)
Channel Vegetation	
Type/Size	Salt Bush, Ephedra, tall grasses, low dry grasses.
Density/Occurrence	(US) Very sparse low brush; dry grasses toward banks; moist grasses beneath bridge near east abutment. (DS) Dry grasses.
Relative Age	Mature
Manning's Roughness Coef.	0.028
Controls on Stream Migration	
Lateral	Soil cement-lined channel US and along west bank DS.
Vertical	Large diameter rip-rap placed around piers across channel bottom extending 25 ft. US/DS of pier; grade control structure approximately 2000 ft. US of structure.
Sediment Deposits & Bars	(US) No significant bar formation. (DS) Primarily low to moderate elongate middle bars.
Evidence of Degradation	Incised channel DS; headcutting of rip-rap DS.
Evidence of Aggradation	No
Evidence of Scour	
Pier	No
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Moderate rate; land use west of channel primarily high density residential and commercial; land use to east primarily agricultural; general assumption is for increasing urbanization to the east.
Sand & Gravel Extraction	Past operations in channel US/DS of bridge. Channel currently undergoing rapid adjustment to operations in channel immediately DS of bridge.
Freeway Construction	No
Dams	No
Drainage Channels	No

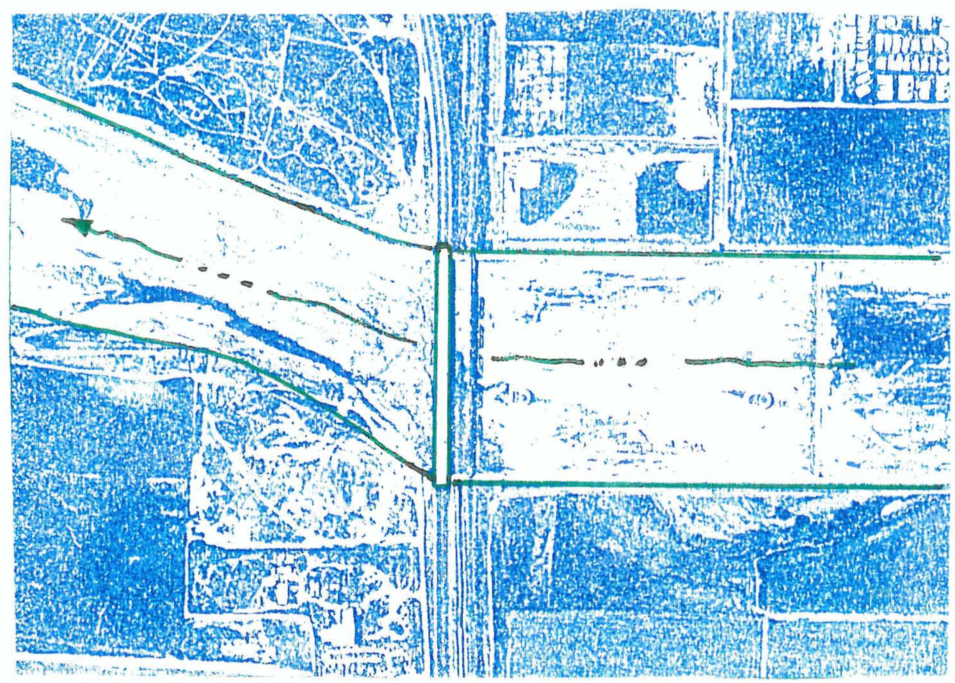


Figure 5

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in the Agua Fria River at Indian School Road come from controlled releases from the New Waddell Dam at Lake Pleasant, from New River Dam on New River, 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 the design of the replacement section of the bridge are 60,000 cfs and 50 years, respectively.

2. USGS data show that the largest recorded flood between 1961 and the present was 58,400 cfs on December 19, 1978, as measured at El Mirage, Arizona at Grand Avenue, approximately 12 miles upstream of the 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 95,000 cfs.

4. The 500-year flood (superflood) reported by FEMA is 184,000 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 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 instructions.

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
955.74	22,000	Q_{10}
959.04	69,000	-
960.44	95,000	Q_{100}
964.33	184,000	Q_{500}
966.25	236,200	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 184,000 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). Scour calculations 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 6. Scour calculations are presented in the *Technical Appendix*.

An incipient motion analysis was performed on the riprap on the channel bottom. Assuming a safety factor of three, the riprap is not large enough to resist motion by forces produced during the 500-year flood and cannot be considered to provide full scour protection at the piers during that event. Although the riprap does appear to be sufficiently large to protect against scour during the 100-year flood, the cover over the abutment cap is minimal. In some cases the top of the pile cap (Elevation 951) is visible below gaps in the riprap. The riprap sill, however, is considered sufficient to prevent long-term degradation (general scour) at the bridge piers.

Table 2. Summary of Scour Calculations

	Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{500})
1. Scour at Piers		
Contraction Scour, ft	0.0	0.0
Local Scour, ft	18.5*	21.0
General Scour, ft	0.0	0.0
Total Scour, ft	**18.5**	**21.0**

2. Scour at Abutments - Negligible (not tabulated)

*Assumes riprap sill not scour resistant

Table 3. Summary of Pile Embedment

	Q = 95,000 cfs (Q_{100})	Q = 184,000 cfs (Q_{500})
Channel Elevation	951.3	951.3
Total Scour, ft	18.5	21.0
Bottom of Scour Hole Elev.	932.8	930.3
Median Pile Tip Elev.	928.0	928.0
Embedment Remaining, ft	**4.8**	**2.3**

<u>**CONCLUSIONS:**</u> Based on the scour calculations, the piles supporting the piers of the Maricopa Highway 85 Bridge do not have sufficient embedment to resist flows of 184,000 cfs, i.e., the 500-year flow rate. The bridge is scour *critical.*

<u>**COUNTERMEASURES:**</u>

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 between and around the scour critical piers, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

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

d. Replace the bridge with a new bridge supported on deep foundations.

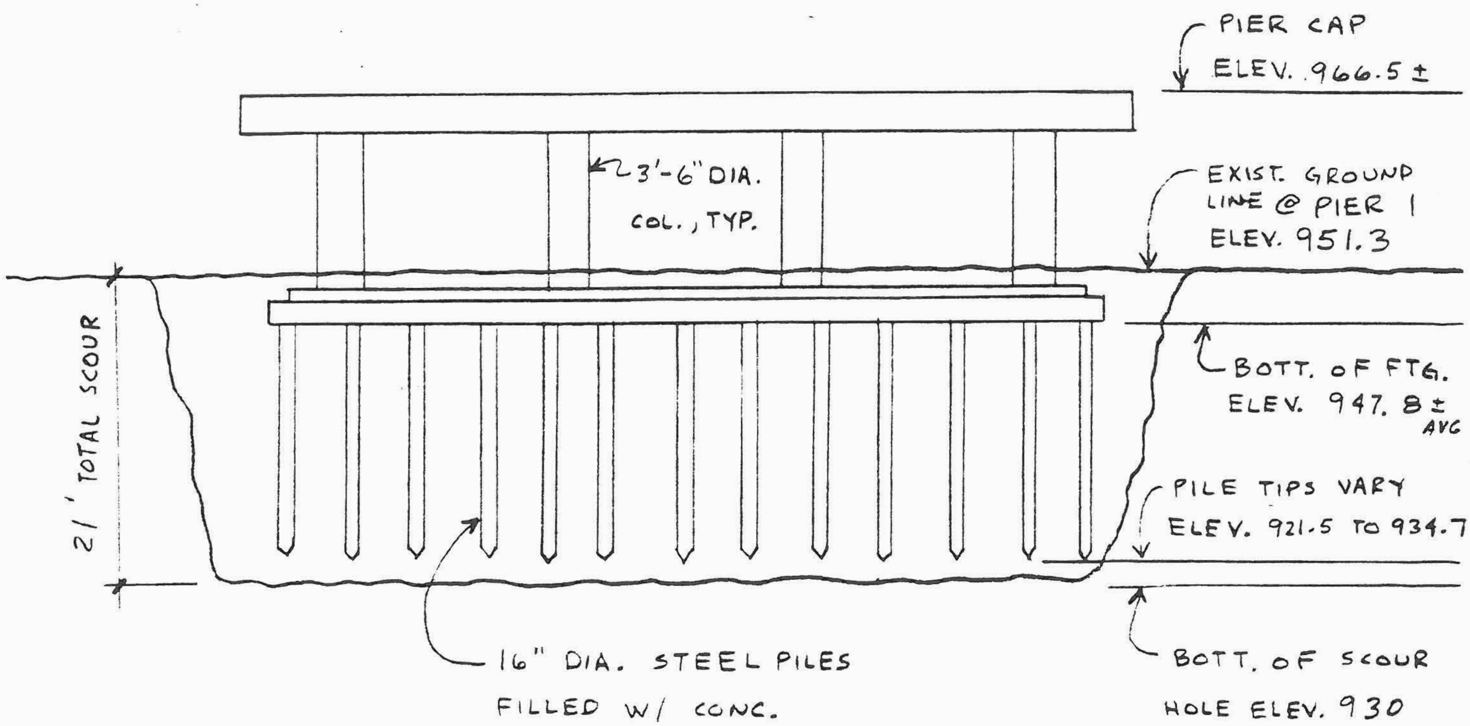
MARICOPA COUNTY HIGHWAY 85 BRIDGE AT AGUA FRIA RIVER

HYDRAULIC DATA (PER MM/CSSA):

Q500 = 184,000 CFS

H.W. ELEV. = 964.33

TOTAL SCOUR = 21'



PIER ELEVATION

NOTE: LARGE DIAMETER RIPRAP DUMPED ACROSS THE FULL WIDTH OF THE CHANNEL AND EXTENDING 25' UPSTREAM AND DOWNSTREAM OF THE COLUMNS.

Figure 6

Photo 1: View looking upstream from approximately mid-channel near SPRR bridge. Note slightly meandering, relatively clear, shallow low flow channel comprised of somewhat armored coarse surficial sediments. Also note low grasses typical of main channel on either side of shallow low flow channel. Further note power line islands positioned in east central portion of channel. Both upstream banks are formed of soil cement into relatively high trapezoidal levees.

Photo 2: View looking toward east bank and upstream face of structure from approximately mid-channel. Note significant density of moist grasses nourished by irrigation outfall located in east levee embankment slightly upstream of structure.

Photo 3: View looking downstream toward structure along west levee embankment. Note typical embankment configuration with channel access road.

Photo 4: View of downstream face of bridge from top of west levee embankment. Note placement of angular rip-rap around piers. Also note pattern of downcutting immediately downstream of rip-rap with several smaller channels forming into a larger low flow channel immediately downstream of structure. Note east bank is natural downstream of structure. Also note relatively coarse nature of downstream sediments exposed near structure. Some mining of channel sediments may have occurred immediately downstream of structure serving to direct low flow.

Photo 5: View looking toward west bank and upstream face of structure from approximately mid-channel. Note relatively coarse nature of exposed surficial sediments and minimal vegetation present. Also note close proximity of upstream SPRR bridge.

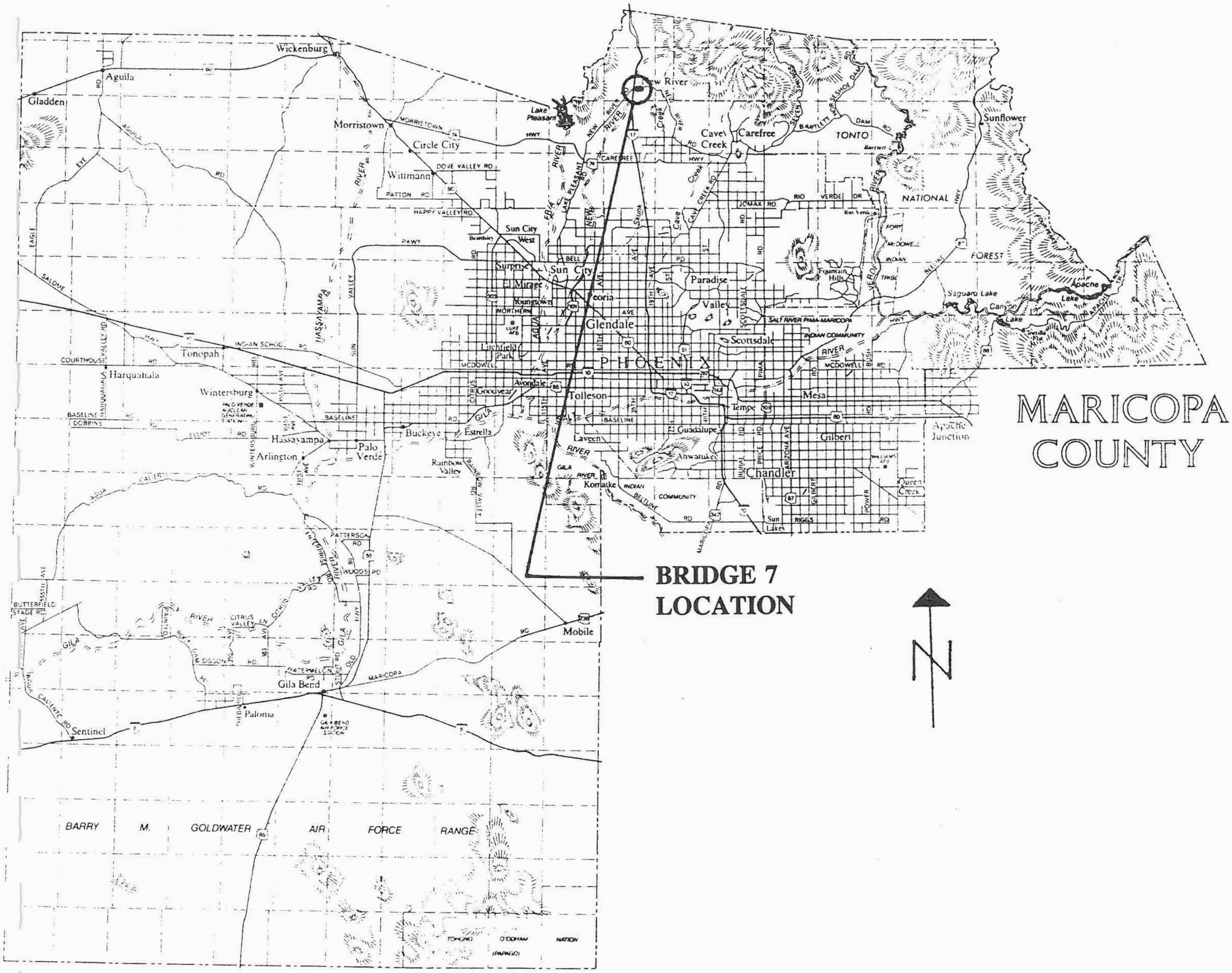
Photo 6: View of typical downstream edge of rip-rap placed around piers. Note approximately 2 feet of local erosion occurring in this position has destabilized downstream edge of rip-rap.

Photo 7: View of downstream side of narrow grade control structure located approximately 2000 feet upstream of structure. Note that channel grasses occur at significantly greater density upstream of this grade control (not shown).

Photo 8: View of downstream channel from bridge deck. Low flow channel drains toward ponding area for irrigation runoff. Confluence is identified by locally well developed vegetation visible in background.

BRIDGE 7

NEW RIVER ROAD BRIDGE OVER NEW RIVER



Location Map

Figure 1

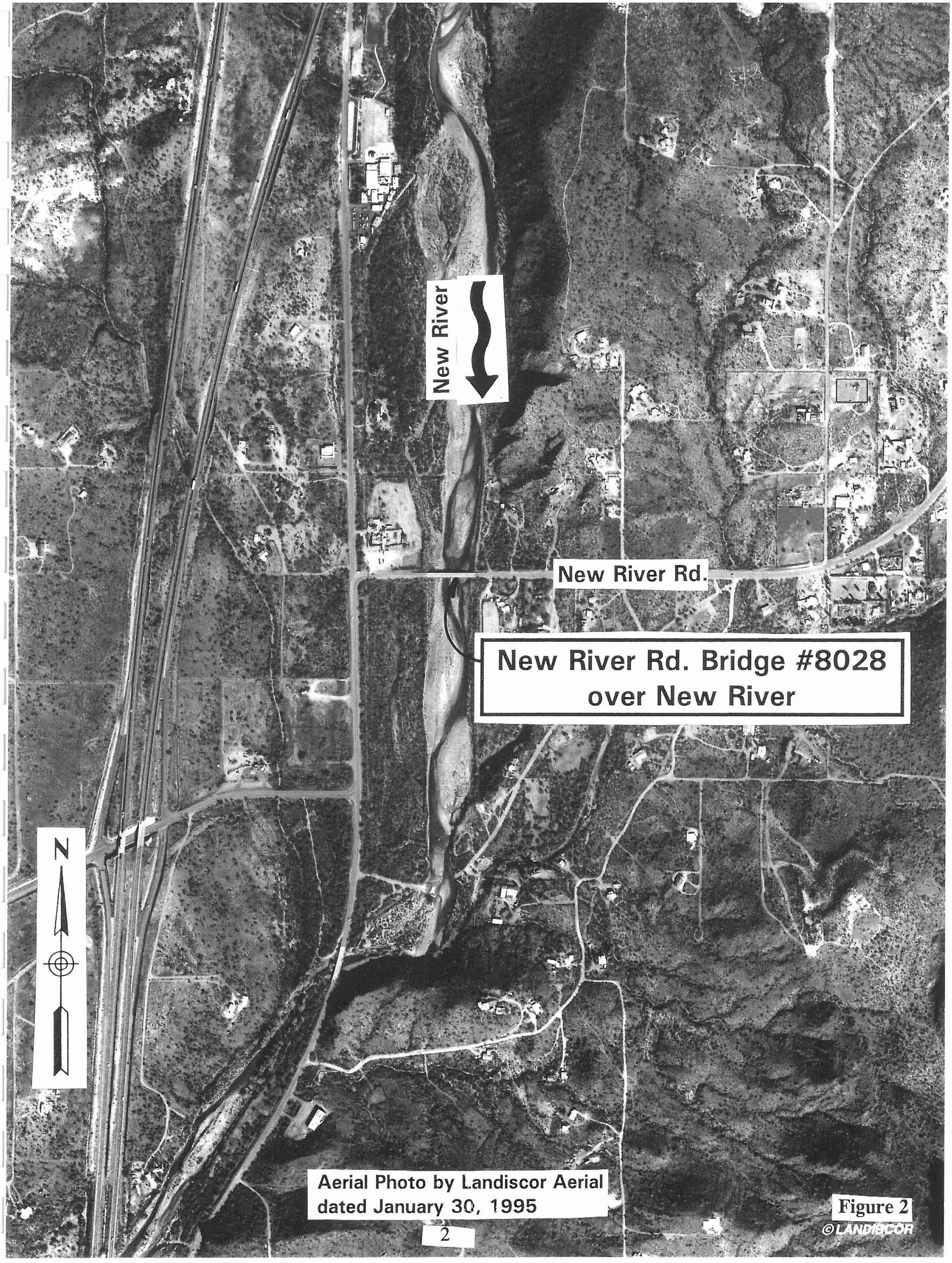

New River
New River Rd.
New River Rd. Bridge #8028
over New River
N
Aerial Photo by Landiscor Aerial
dated January 30, 1995
©LANDISCOR

Figure 2

<u>**BRIDGE 7:**</u> <u>**NEW RIVER ROAD OVER NEW RIVER (Structure #8028)**</u>

Assessment: Scour Critical

<u>**LOCATION:**</u> The New River Road Bridge at New River lies in Section 27 of T7N, R2E, Gila and Salt River Baseline and Meridian in the unincorporated area of New River, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

<u>**STRUCTURE:**</u> The bridge has six spans with an total length of 300' from center-to-center of abutment bearings and a 30-foot-wide roadway. (See Location Plan, Figure 3.) The design stream flow rate for the bridge is unknown. The bridge was designed by L.H. Bell and Associates in 1968; it was built in 1971 as MCDOT Project No. 72-262A.

The superstructure is a continuous concrete slab with a hinge located adjacent to Pier No. 3.

The abutments are the solid concrete wall type on spread footings oriented at zero skew with respect to the bridge roadway. According to as built plans, the footing at Abutment 1 is founded on shale and the footing at Abutment 2 is founded on hard red conglomerate. The footings are approximately 7' below the existing river flow line.

The bridge piers are the solid concrete wall type on spread footings oriented normal to the roadway. Both ends of the wall are constructed with a 116 degree nosing. According to as-built plans, the footings are founded on red granite hardpan, red conglomerate hardpan, and/or blue shale. The footings are approximately 7' below the existing ground line.

<u>**EXISTING SCOUR PROTECTION:**</u> No special scour protection measures were noted on the as-built plans, and none were observed during a site inspection. However, the large proportion of cobble and boulder sized material in the channel suggests a good potential for armoring.

<u>**STREAM FORM:**</u> New River at New River Road is a braided stream with low to medium height point, alternate and middle bars forming in a braided condition characteristic of streams with large sediment sizes and relatively steep slope. (See Figure 4.) At the time of the site inspection (May, 1995) there was water flowing in a low flow channel at Span 3 of the bridge (third span from the west abutment). A smaller dry channel was noted at Span 6 next to the east abutment. Aerial photographs suggest that the location of the main channel is fairly stable.

<u>**LAND USE:**</u> Land use in the surrounding area is primarily undeveloped desert with some light residential and commercial developments. No gravel mining, either former or active, was observed.

Urbanization is expected to increase, although at such low densities as to have little effect on runoff at the bridge.

<u>**SURFACE SOILS:**</u> Surface soils consist primarily of sand, gravel, cobbles and boulders. The armoring potential of the river bed is estimated to be high. Span 1 has reduced clearance due to sedimentation.

SLOPE: The slope of the river bed in the vicinity of the New River Road Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.011 ft/ft or approximately 58' per mile.

VEGETATION: Vegetation observed on the banks and bottom of the New River includes bushes and shrubs such as desert broom, creosote, desert willow ephedra, and others. Densities of vegetation in the river is estimated to be moderately dense on the banks and low to moderate in the channel. Debris collected on the piers was observed during the site inspection and is accounted for in the scour calculations.

STREAM STABILITY: Overall lateral stability of the river at the bridge is provided by earthen levees upstream and downstream of the bridge, except at the upstream east bank, which is a stable rock outcrop. The levees were constructed with the bridge and have occasional broken or missing sections. Although aerial photographs showed that the low-flow channel configuration has not changed appreciably, it must be assumed that the thalweg will shift laterally when water is flowing in the river.

There are no upstream or downstream grade control structures in the immediate area that would maintain vertical stability of the river bed at the bridge site. A rock outcrop downstream approximately one-half mile may act as a grade control for the upstream vertical stability of the river. It is likely that the long-term tendency of the stream bed over geologic time is to degrade rather than aggrade, although there is some sedimentation in Span 1 that has reduced the open area under the bridge.

CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in New River at New River Road come from uncontrolled runoff on the watershed. Available plans, flow records, and hydrologic models provided the following information:

1. The flow and flood frequency used in the design of the bridge were not shown on the plans.

2. USGS data show that the largest recorded flood between 1961 and the present was 18,600 cfs on September 5, 1970, as measured at Rock Springs, Arizona at Grand Avenue, located upstream of the 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 32,000 cfs.

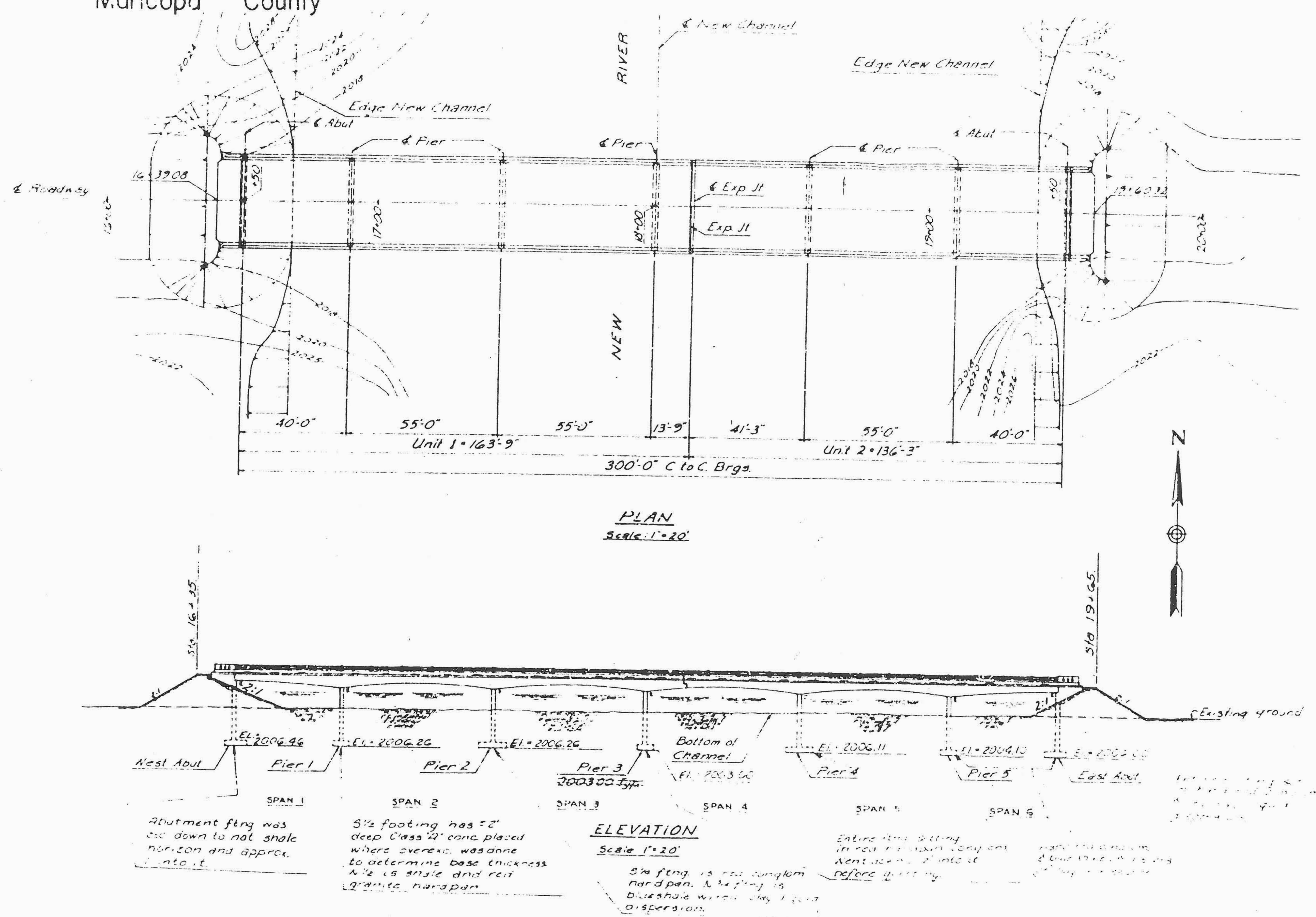
4. The 500-year flood (superflood) estimated by using the USGS regression equations for ungaged watersheds is 73,600 cfs.

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 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 instructions.

NEW RIVER ROAD–BRIDGE OVER NEW RIVER
Maricopa County

COUNTY	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
MARICOPA	ARIZ				

L.H. BELL & ASSOCIATES



General Notes

Construction – Standard Specifications [illegible] Highway Department – 1955 Edition

Design – A.A.S.H.O. Standard Specifications for Highway Bridges 19[illegible]

Loading – H 20-S16-44

Stresses – Class A Concrete fc = 1000 psi n = 12
Class D Concrete fc = 1200 psi n = 10
Reinforcing Steel fs = 20,000 psi
Structural Steel fs = 20,000 psi

All Substructure Concrete shall be Class "A". Superstructure Concrete shall be Class "D", with max. Aggregate 100% passing 1½".

Reinforcing Steel shall be intermediate grade and shall conform to A.S.T.M. Specs. A 15 and A 305.

Structural Steel shall conform to A.S.T.M. A 36 unless otherwise noted.

All Welding shall conform to American Welding Society Spec. for Welded Highway and Railroad Bridges, 1963 Edition.

All Dimensions for Reinforcing Steel shall be Center to Center of Bars.

2" Clear Cover shall be maintained for all Reinforcing Steel unless otherwise noted.

All Horizontal Measurements are at 70°F and Proper Adjustments must be made in Locating Anchor Bolts and Setting Bearing Plates and Expansion Angles for the actual temperatures at the time of Construction.

All Exposed Concrete Corners shall have ¾" Chamfer unless otherwise noted.

Exposed faces of outside girders, abutment wings and outside surfaces of the deck, curbs & parapets and all curb tops, inside faces of curbs, parapets shall be hand rubbed finish.

Structural Steel shall be included with concrete for payment.

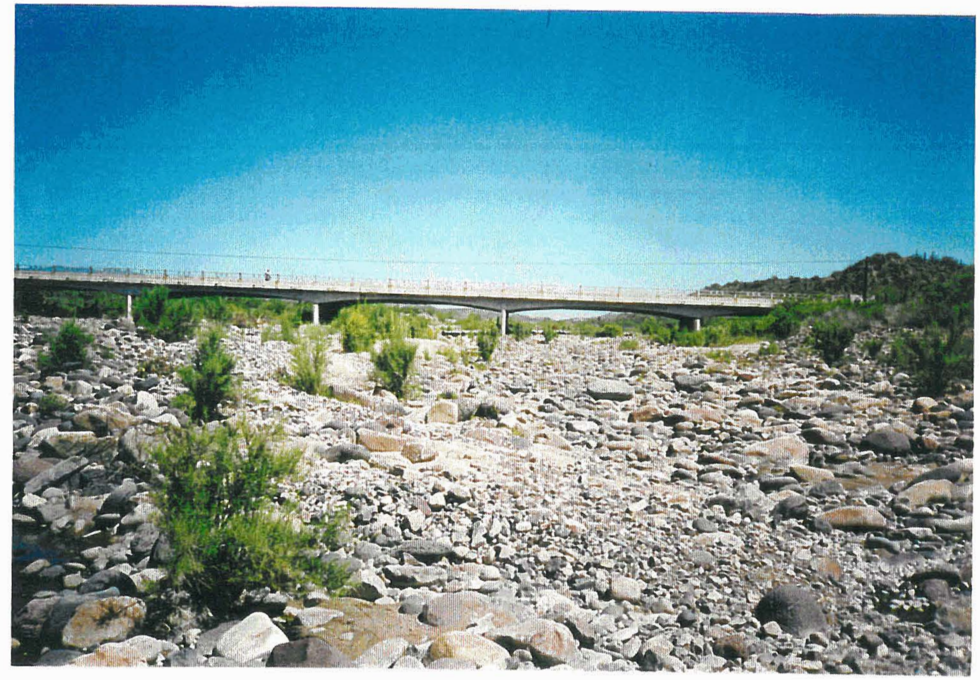
Estimated Quantities

Item	Units	Estimated Quantities
Borrow	CY	1200.
Concrete		
Class "A"	CY	420
Class "D"	CY	450.
Reinf. Steel	lbs	247,[illegible]
[illegible] Comp	CY	1655.
Handrail	Lin Ft	603.67
Struct. Excav	CY	[illegible]
[illegible] Base	Ton	30.

Figure 3

NEW RIVER ROAD (SN 8028)

Water Course	New River
Stream Form	Braided (first order low flow channel sinuous)
Sinuosity	Not applicable
General Channelization	Low earthen levees occasionally broken/missing) along channel banks.
Channel Slope	Steeper US
Estimated Channel Slope (ft/ft)	0.008817
Channel Contraction/Expansion	None
Primary Surface Sediment Type	sand/gravel/cobbles/boulders
D50 Size	
Armoring Potential	Moderate to high (locally)
Channel Vegetation	
Type/Size	Primarily Desert Broom to 5 ft., Desert Willow to 5 ft., Creosote to 4 ft., occasional Ephedra, others.
Density/Occurrence	Moderately dense on banks; sparse in main channel.
Relative Age	Young to mature.
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	Some control along US LB due to rock out crop; otherwise earthen levees relatively low/poor condition.
Vertical	None
Sediment Deposits & Bars	Low to moderate point, alternate and middle bars forming in braided condition. Characteristic of large sediment sizes and relatively steep slope. Reduced clearance in waterway toward west abutment due to sedimentation.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	Low flow channel has incised alongside mid-span piers.
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low to moderate rate; generally low density residential and light commercial.
Sand & Gravel Extraction	No
Freeway Construction	No
Dams	No
Drainage Channels	No



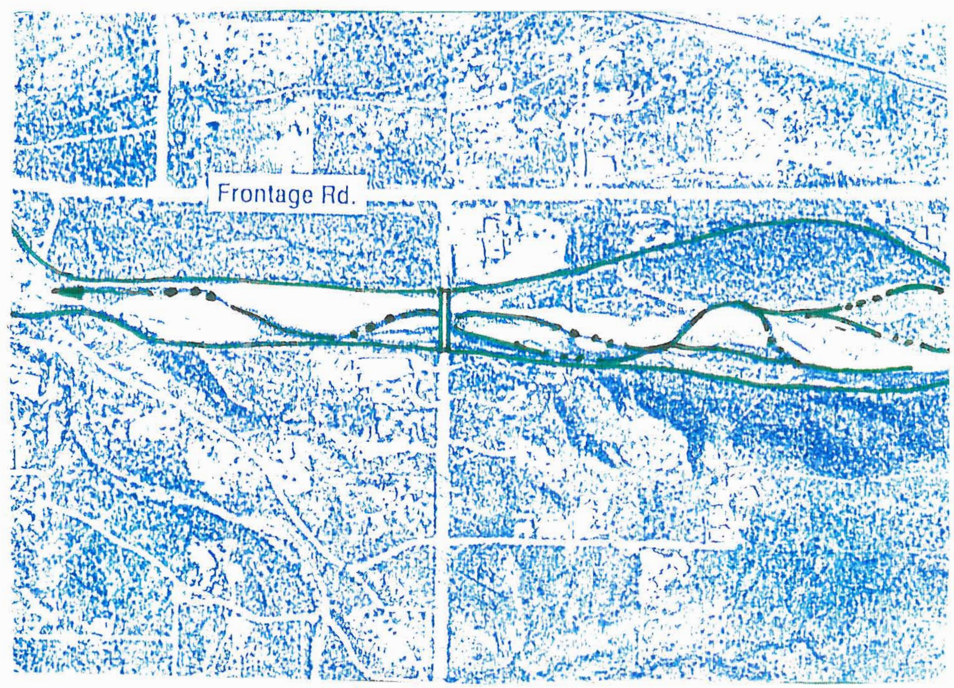


Figure 4

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
2023.00	21,900	Assumed
2024.00	28,400	Assumed
2024.51	32,000	Q_{100}
2024.60	32,700	Low Chord
2029.19	73,600	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 32,700 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). Scour calculations 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. Because of the high potential for armoring, long-term degradation or general scour, was assumed to be negligible. Results of the scour calculations and a summary of footing 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 = 32,000 cfs (Q_{100})	Q = 32,700 cfs (Q_{LC})
1. Scour at Piers		
Contraction Scour, ft	0.0	0.0
Local Scour, ft	16.4	16.5
General Scour, ft	0.0	0.0
Total Scour, ft	**16.4**	**16.5**

2. Scour at Abutments - Negligible (not tabulated)

Table 3. Summary of Footing Embedment

	Q = 32,000 cfs (Q_{100})	Q = 32,700 cfs (Q_{500})
Channel Elevation	2013.5	2013.5
Total Scour, ft	16.4	16.5
Elev. Bottom of Scour Hole	1997.1	1997.0
Spread Footing Bottom Elev.	2006.3	2006.3
Embedment Remaining, ft	**-9.2**	**-13.3**

GEOTECHNICAL EVALUATION: The refraction seismic survey data indicates that rock contact is somewhat variable at the New River Road bridge site. Higher velocity materials likely representative of rock (6,000 to 10,000 feet per second) vary in depth from about 2 to 13 feet below grade at the New River Road bridge. Since foundations are about 6 to 7 feet below grade, it appears that at least some of the footings will be susceptible to scour below the foundations.

CONCLUSIONS: Based on the scour calculations, the piers do not have sufficient embedment to resist flows of 32,700 cfs, i.e., the 500-year flow rate and the footings do not bear on rock. The bridge is scour *critical.*

COUNTERMEASURES:

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 between and around the scour critical piers, with the sill keyed deeply into the channel bed at the upstream and downstream ends;

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

d. Replace the bridge with a new bridge supported on deeper foundations.

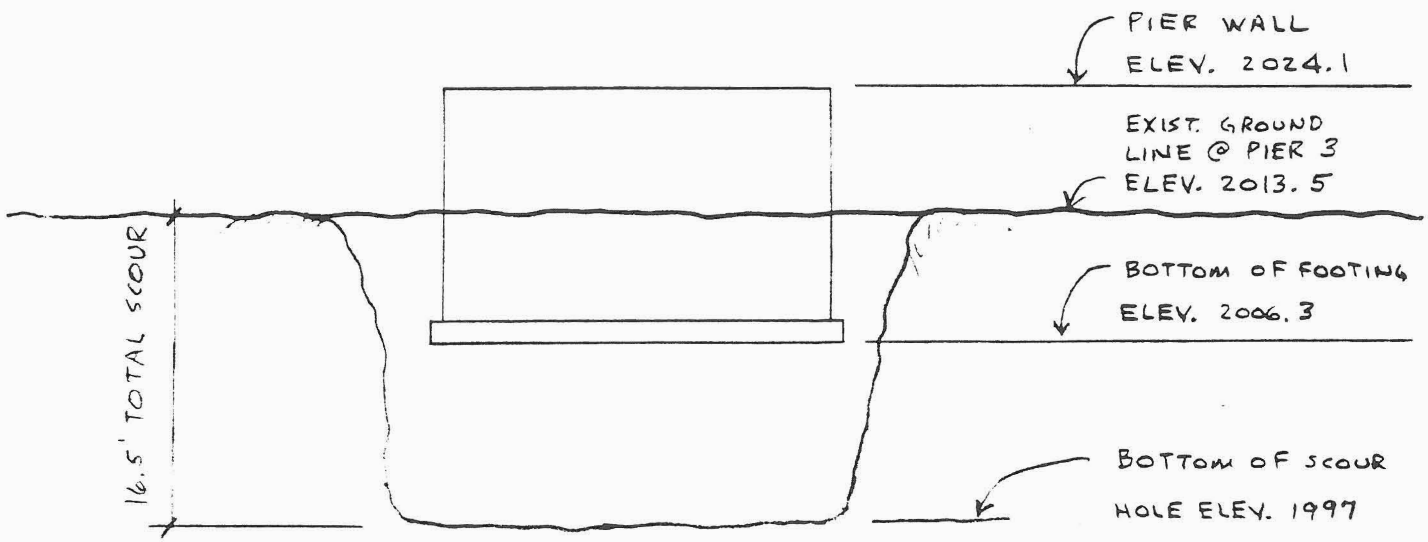
NEW RIVER ROAD BRIDGE OVER NEW RIVER

HYDRAULIC DATA (PER MM/CSSA):

$Q_{LOW\ CHORD}$ = 32,700 CFS

H.W. ELEV. = LOW CHORD = 2024.60

TOTAL SCOUR = 16.5'



PIER ELEVATION

2013.5
2006.3
0007.2

check modeling?

Figure 5

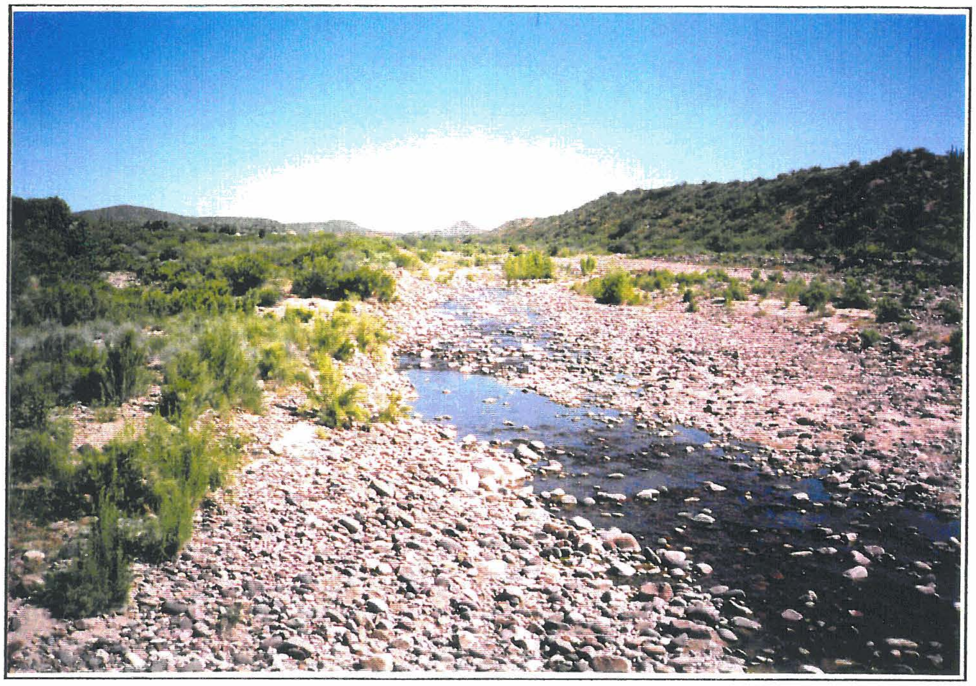


Photo 1: View of upstream channel from west side of bridge deck. Note relatively large proportion of cobble and boulder-sized material in primary channel suggesting good armor potential for significant portion of upstream channel. Also note vegetation established toward right side of main channel. Further note meandering pattern of low flow channel.

Photo 2: View of downstream channel from east side of bridge deck. Note large proportion of cobble and boulder-sized material in downstream channel. Downstream main channel is relatively straight and clear of obstructions. Also note fairly regular meandering pattern of low flow channel.

Photo 3: View of upstream channel looking west across low flow channel. Note unstable rocky slope in main channel being undercut by low flow channel. Vertical relief is approximately 8 feet.

Photo 4: View of upstream face of bridge. Note low flow channel between piers 3 and 4.

Photo 5: View of primary channel upstream of bridge. Note relatively high proportion of cobble and boulder-sized material may have good potential for armoring during high flows.

Photo 6: View of upstream edge of pier 3. Note significant debris accumulation. Accretion pattern on piers suggests recent flow angle of attack is slightly east of north.

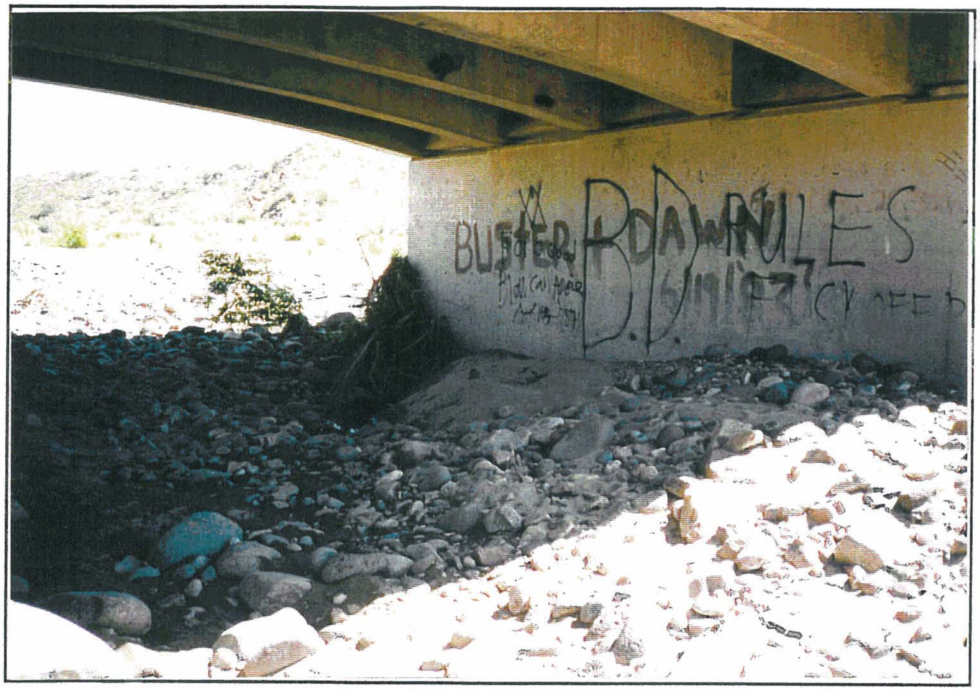
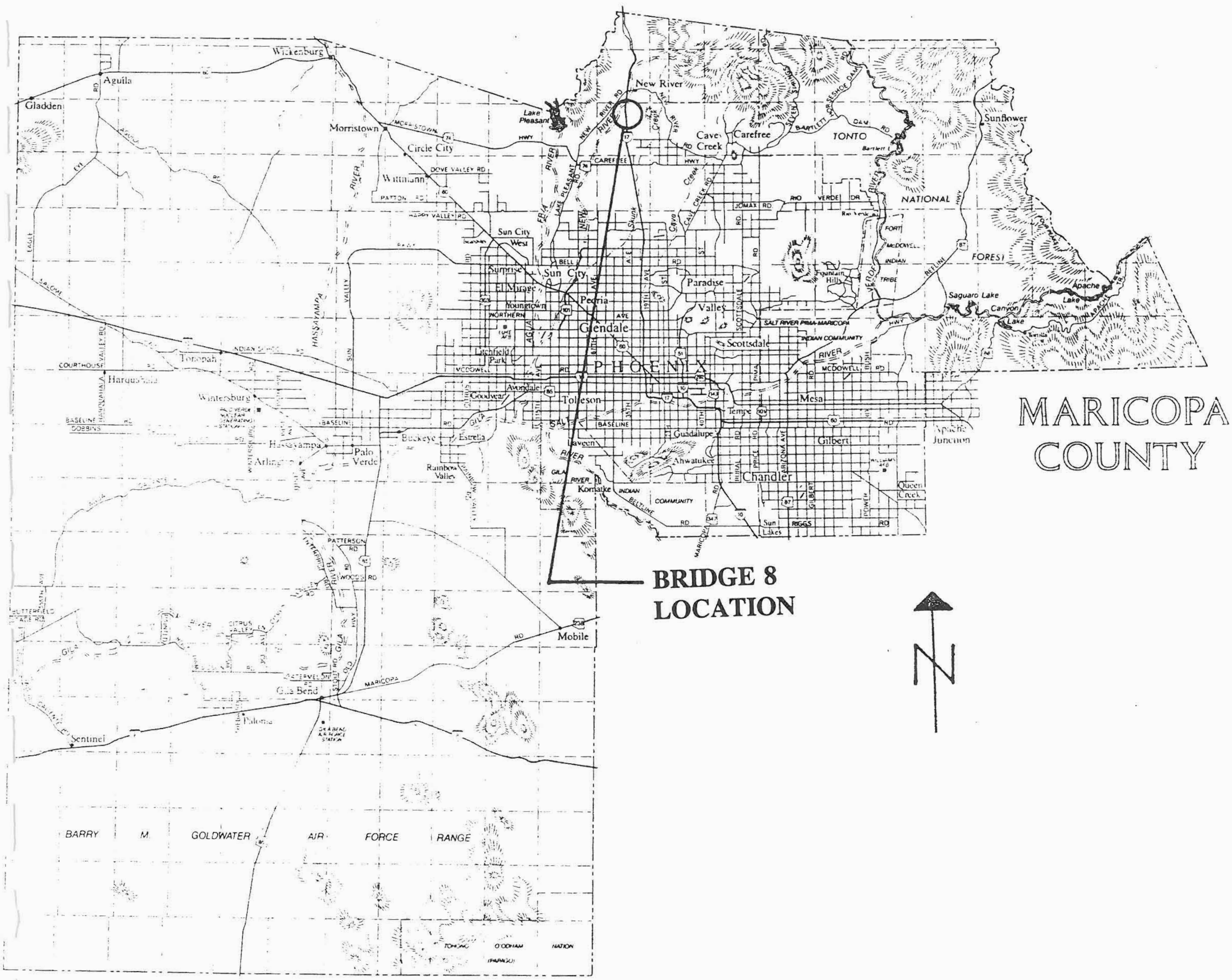


Photo 7: View of west side of pier 2. Note residual pattern of local scour at upstream face reflects debris accumulation on leading edge.

Photo 8: View of west abutment from downstream. Note this reduced clearance was also observed in waterway below adjacent span.

BRIDGE 8

I-17 FRONTAGE ROAD BRIDGE OVER NEW RIVER



Location Map

Figure 1

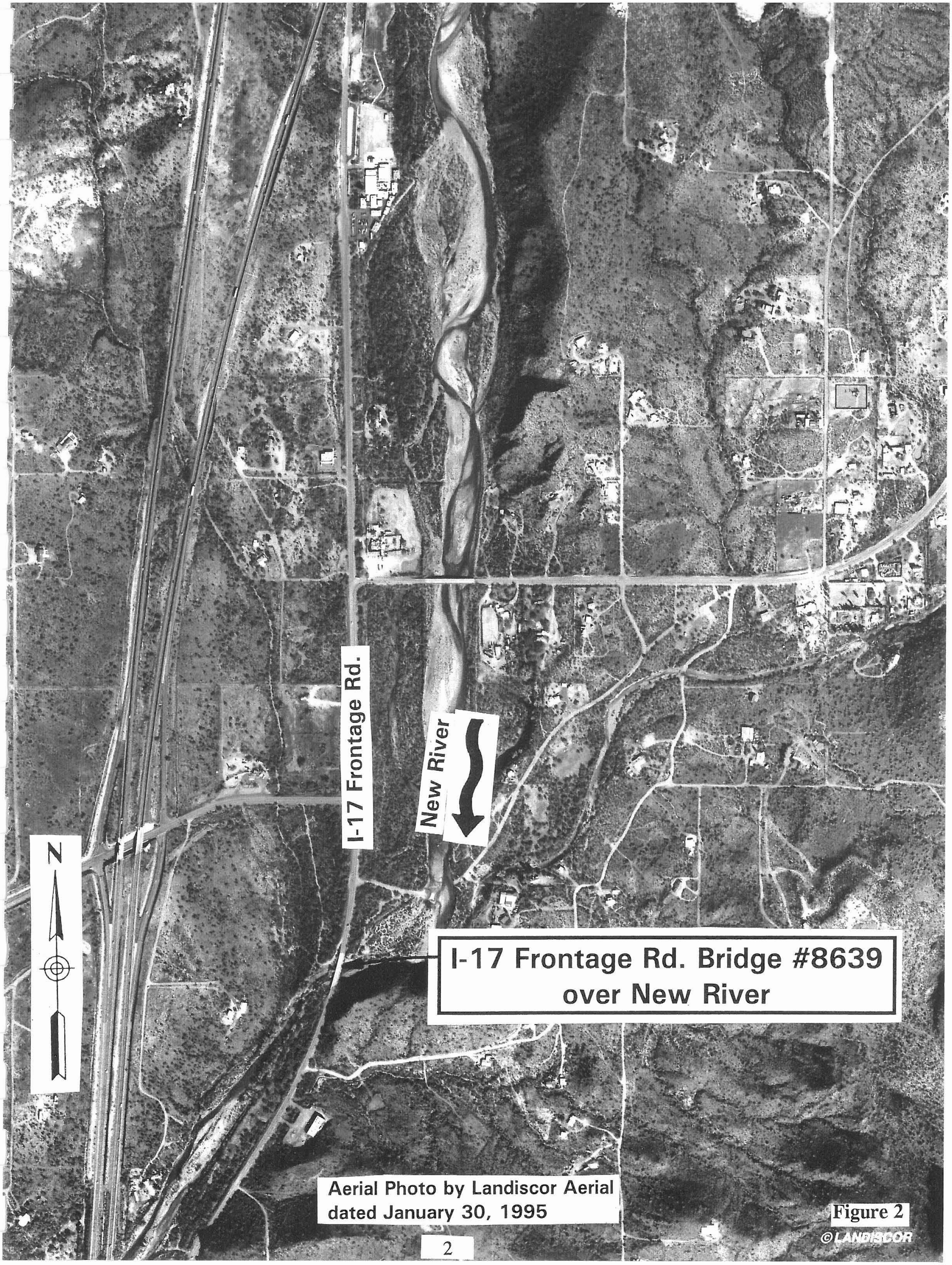

I-17 Frontage Rd.
New River
N
I-17 Frontage Rd. Bridge #8639
over New River
Aerial Photo by Landiscor Aerial
dated January 30, 1995
©LANDISCOR

Figure 2

BRIDGE 8: I-17 FRONTAGE ROAD BRIDGE OVER NEW RIVER

(Structure #8639)

Assessment: Scour Critical

LOCATION: The I-17 Frontage Road Bridge at the New River lies in Section, T7N, R2E, Gila and Salt River Baseline and Meridian, near the unincorporated area of New River, Arizona. See Location Map, Figure 1 and Aerial Photo, Figure 2.

STRUCTURE: The bridge has seven spans with an total length of 232' from center-to-center of abutment bearings and a 30-foot-wide roadway. (See Location Plan, Figure 3.) The bridge is on a horizontal curve of the roadway. The design stream flow rate for the bridge is unknown. The bridge was designed by the Bridge Division of the Arizona Highway Department (AHD) in 1947; it was built in 1948 as AHD Project No. NON FAS 39.

The superstructure is a continuous concrete slab with a hinge located adjacent to Pier No. 4.

The abutments are a closed wall type supported on concrete spread footings. Abutments 1 and 2 are oriented at a 31 degree and a 28 degree skew to the bridge roadway, respectively. According to as-built plans, the footing at Abutment 1 is founded approximately 8' below the existing river flow line and the footing at Abutment 2 is approximately 5' below the existing river flow line.

The bridge piers are the solid concrete wall type on spread footings. The piers are skewed approximately 30 degrees to the horizontal curve of the roadway. Both ends of the wall are constructed with a 90 degree nosing. According to as-built plans, the footings are founded approximately 8' below the existing river flow line.

EXISTING SCOUR PROTECTION: The as-built plans indicated that no special protection was provided at the abutments or piers, as confirmed during a site inspection. However, the large proportion of cobble and boulder sized material in the channel suggests a good armoring potential.

Why is concrete Grade Control structure provided for 2 ? What is the toe-down for it?

STREAM FORM: New River at the I-17 Frontage Road is a straight to braided stream with low bars forming in response to obstructions. (See Figure 4). Sedimentation at the north and south ends of the bridge has reduced the waterway in the spans adjacent to the abutments. Aerial photographs suggests that the location of the main channel is fairly stable.

LAND USE: Land use in the surrounding area is primarily undeveloped desert with some light residential and commercial developments. No gravel mining, either former or active, was observed.

Urbanization is anticipated to increase, although at such low densities as to have little effect on runoff at the bridge.

SURFACE SOILS: Surface soils consist primarily of sand, gravel, cobbles and boulders, with an estimated median size (D_{50}) of 50 mm. The armoring potential of the river bed is estimated to be moderate to high.

Slope: The slope of the river bed in the vicinity of the New River Road Bridge, estimated from U.S. Geological Survey (USGS) 7.5 minute topographic maps, is 0.0075 ft/ft or approximately 40' per mile.

VEGETATION: Vegetation observed on the banks and bottom of New River includes various species of trees up to 20' high; bushes and shrubs such as desert broom and ephedra; and low to medium height grasses. The upstream channel and banks are densely overgrown; the downstream channel is somewhat clearer with sparse growth in the channel becoming denser toward the banks. Debris piles more than 5' wide and 7' high were observed on Pier Nos. 3 and 4 during the site inspection. Debris was observed on nearly all the piers, and is therefore accounted for in the scour calculations.

STREAM STABILITY: Lateral stability of the river is provided by a rocky hillside upstream of the bridge which directs the southwardly-flowing river sharply to the west along the toe of the hill.

At the time of the site inspection (May, 1995) there was water flowing in a low flow channel at Spans 3 and 4 of the bridge (third and fourth spans from the south abutment). Although aerial photographs showed that the location of the low-flow channel has not changed appreciably over the years, the presence of debris on Pier 1 and a residual scour hole observed at Pier 2 require that it be assumed that the thalweg can shift laterally when water is flowing in the river.

A broken, abandoned concrete low-water crossing downstream of the bridge may act as a limited grade control structure, although only where intact. There is also a rock outcrop in the channel approximately 100' downstream that may act as a grade control for the upstream vertical stability of the river. It is likely that the long-term tendency of the stream bed over geologic time is to degrade rather than aggrade, although there is some sedimentation in Spans 1 and 7 that has reduced the open area under the bridge.

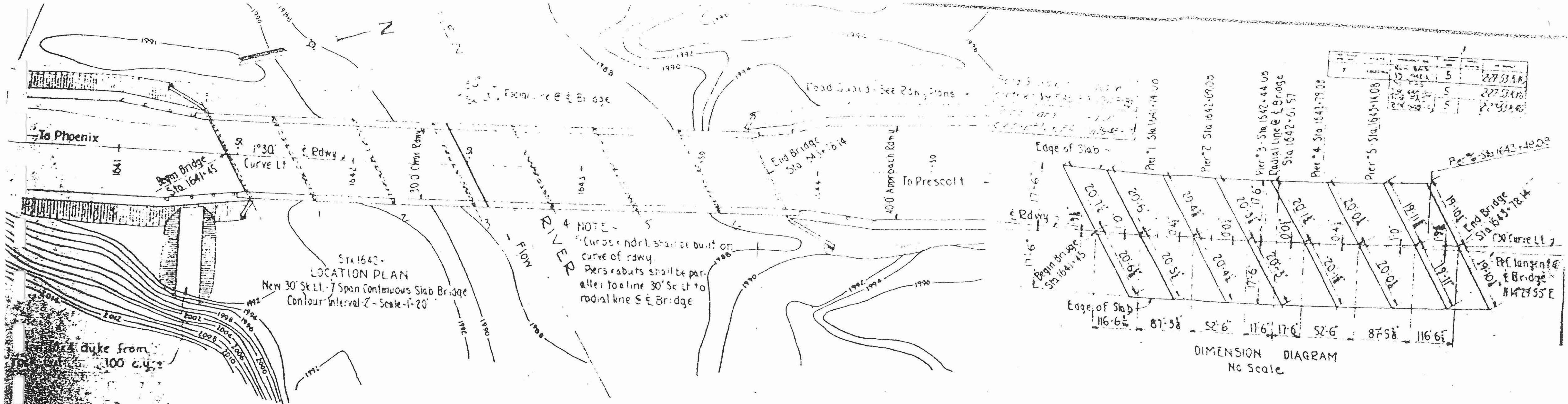
CURRENT HYDROLOGY AND FLOW ANALYSIS: Large flows in New River at the I-17 Frontage Road come from uncontrolled runoff on the watershed. Available plans, flow records, and hydrologic models provided the following information:

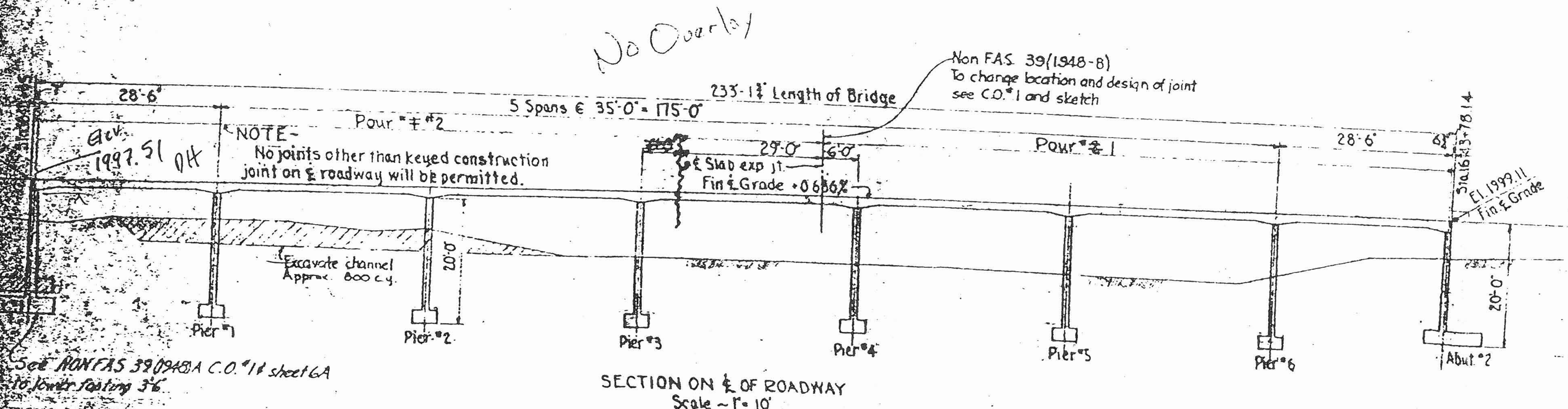
1. The flow and flood frequency used in the design of the bridge were not shown on the plans.

2. USGS data show that the largest recorded flood between 1961 and the present was 18,600 cfs on September 5, 1970, as measured at Rock Springs, Arizona at Grand Avenue, located upstream of the 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 33,400 cfs.

4. The 500-year flood (superflood) estimated by using the USGS regression equations for ungaged watersheds is 73,600 cfs.





GENERAL NOTES ~

Standard specifications Arizona Highway Department, edition of 1938 & supplements.

Design specs. AASHO 1944 revised to date.

Loading Clase - H20-S16-1944 (2 Lanes)

fs=18000 fc=1000 n=10 ~ superstructure.

fs=18000 fc=800 n=12 ~ substructure.

Structural steel shall conform to ASTM specifications A-7-42.

Welding shall conform to American Welding Society Specs. for Welded Highway & R.R. Bridges.

Paint as per spec. 4-7 - Contact surfaces not painted.

Shop - One coat paint #1, or U.S. Maritime Comm. Spec. 52 MC 25 quick drying red lead primer

Field - No paint

First contract - Non FAS 39(1948-A) FOUNDATIONS ONLY

Second contract - Non FAS 39(1948-B) SUPERSTRUCTURE

Third contract - Non FAS 39(1948-C) HANDRAIL AND APPROACH FILLS ONLY

APPROXIMATE QUANTITIES

Item	Std No	Sheet No	Struct. Excav. C.Y.	Concrete C.Y. A	Concrete C.Y. D	Steel Lbs Reinf.	Steel Lbs Struct.	Alum. Rail Lin. Ft	
Abut #1	Spec	6	390	100.17		7385			
Abut #2	–	7	550	127.74		9950			
6 Piers	–	8	900	287.11		13840	1220		
Deck	Spec & [illegible]	5, 10			461.57	92590	6160		
Handrail	HA-1	11						461.38	
(1948-A) ~ Totals			1840	515.02		34625	1220		
(1948-B) ~ Totals					427.57	37550	6160	461.38	

Handrail to be placed in Contract Non FAS 39(1948-C)

Handrail not included in contract Non F.A.S. 39(1948-B). Pockets for railing shall be provided as shown on sheet 7

Reinf. steel shall be intermediate grade.

Deck Concrete Class D - All Other Class A

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

STA 1642+
NEW RIVER BRIDGE
LOCATION DETAILS

Figure 3

INTERSTATE 17 FRONTAGE ROAD (SN 8639)

Water Course	New River
Stream Form	Braided to straight
Sinuosity	Not applicable
General Channelization	None
Channel Slope	Uniform
Estimated Channel Slope (ft/ft)	0.004399
Channel Contraction/Expansion	Clear channel wider DS.
Primary Surface Sediment Type	sand/gravel/cobbles/boulders
D50 Size	50 MM
Armoring Potential	Moderate to high (locally)
Channel Vegetation	
Type/Size	Various trees to 20 ft., Desert Willow, Desert Broom, Ephedra dominate smaller vegetation; low to medium height grasses.
Density/Occurrence	Upstream channel/banks densely overgrown; downstream somewhat clearer with sparse growth in channel becoming dense toward banks.
Relative Age	Mature to old.
Manning's Roughness Coef.	0.035
Controls on Stream Migration	
Lateral	South side - rocky hillside directs flow sharply westward.
Vertical	Broken concrete slab in channel may function as limited grade control; otherwise none.
Sediment Deposits & Bars	Low bars forming in response to obstructions mainly sedimentation reducing waterway toward north and south abutments.
Evidence of Degradation	No
Evidence of Aggradation	No
Evidence of Scour	
Pier	Mid-span piers show 2-3 ft. residual scour developed largely by flow angle and debris accumulated.
Abutment	No
Land Use	
Urbanization of Upstream Watershed	Low to moderate rate; generally low density residential and light commercial.
Sand & Gravel Extraction	No
Freeway Construction	No
Dams	No
Drainage Channels	Smaller channel joins upstream of structure; no rapid flow variations anticipated at present.

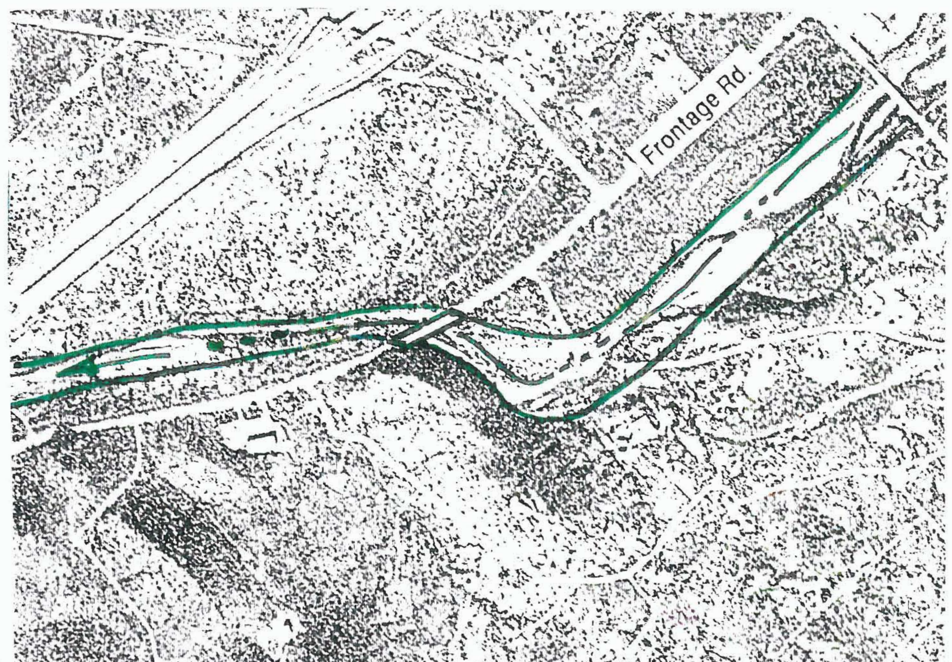


Figure 4

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 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
1993.07	10,000	Assumed
1994.00	12,750	Assumed
1994.93	15,900	Low Chord
1998.91	33,400	Q_{100}
2005.36	74,500	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. Because the low chord flow is less than the 100-year flow, the critical flow for scour calculations is taken as Q_{100} = 33,400 cfs.

SCOUR CALCULATIONS: Scour at the bridge was calculated for Q_{LC} and the critical flood (Q_{100}) using methods described in FHWA Hydraulic Engineering Circular No. 18 (HEC-18). Scour calculations 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. Because of the high potential for armoring, long-term degradation or general scour, was assumed to be negligible. Results of the scour calculations and a summary of footing 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 = 15,900 cfs (Q_{LC})	Q = 33,400 cfs (Q_{100})
1.	Scour at Piers		
	Contraction Scour, ft	0.0	0.9
	Local Scour, ft	17.1	31.7
	General Scour, ft	0.0	0.0
	Total Scour, ft	**17.1**	**32.6**
2.	Scour at Abutments		
	Abutment Scour, ft	0.0	5.1
	General Scour, ft	0.0	0.0
	Total Scour, ft	**0.0**	**5.1**

Table 3. Embedment Remaining at Piers

	Q = 15,900 cfs (Q_{LC})	Q = 33,400 cfs (Q_{100})
Channel Elevation	1984.3	1984.3
Total Scour, ft	17.1	36.2
Bottom of Scour Hole Elev.	1967.2	1953.1
Spread Footing Bottom Elev.	1977.2	1977.2
Embedment Remaining, ft	**-10.0**	**-24.9**

GEOTECHNICAL EVALUATION: The refraction seismic survey data indicates that rock contact is somewhat variable at the I-17 Frontage Road bridge site. Higher velocity materials likely representative of rock (6,200 to 8,000 feet per second) vary in depth from about 2 to 7 feet below grade at the I-17 Frontage Road bridge. Since foundations are about 8 feet below grade, it appears that these foundations are founded on rock.

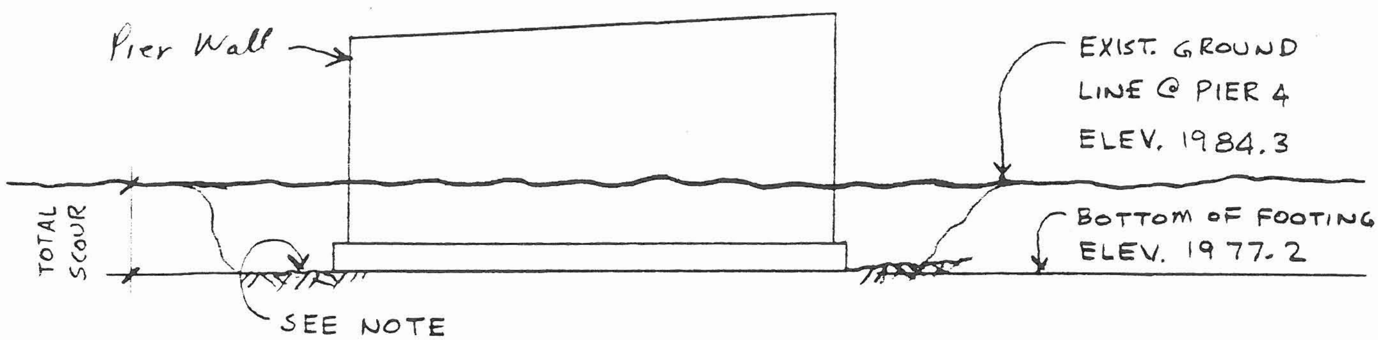
CONCLUSION: Although the I-17 Frontage Road Bridge at New River does not have sufficient hydraulic capacity to pass the 100-year flood, its bedrock foundations have provided adequate protection against scour for nearly 50 years during floods that probably overtopped the road. The bridge is scour *stable*.

I-17 FRONTAGE ROAD BRIDGE OVER NEW RIVER

HYDRAULIC DATA (PER MM/CSSA):

$Q_{LOW\ CHORD}$ = 15,900 CFS

H.W. ELEV. = LOW CHORD = 1994.93



PIER WALL ELEVATION

NOTE: Footings are founded on rock. See Bridge Scour Evaluation Report by AGRA Earth & Environmental, Inc. Job No. E95-36, dated April 15, 1996

Figure 5

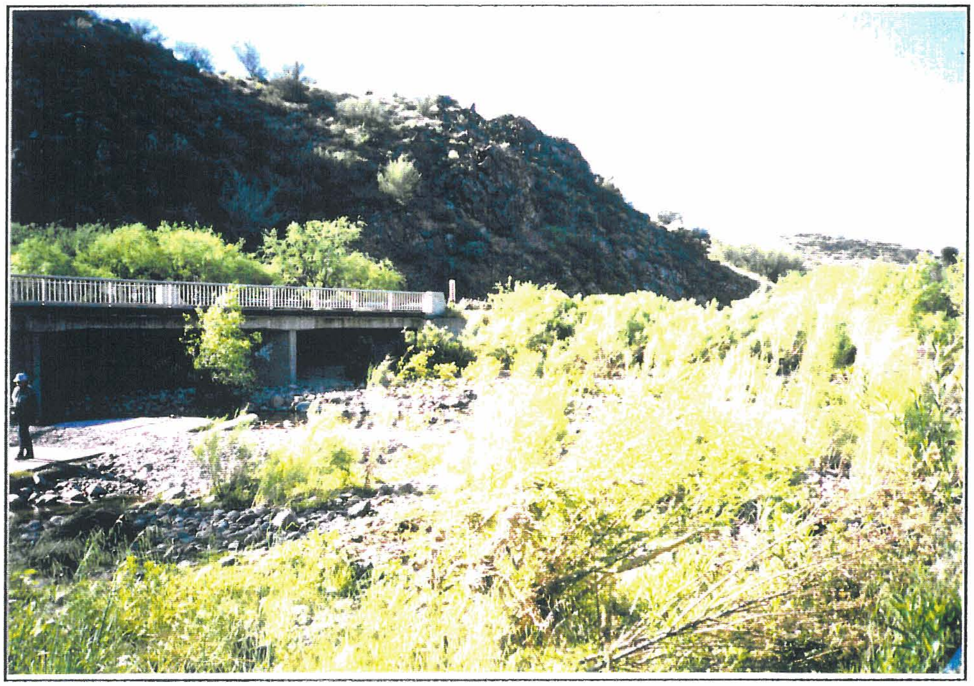


Photo 1: View looking toward downstream side of south abutment. Note cliffs in background indicating potential for near-surface bedrock to provide vertical and lateral stability. Also note mature vegetation in downstream overbanks and in primary channel.

Photo 2: View looking east along upstream left bank from bridge deck. Note boulders up to 2 feet on axis were commonly observed. Also note proximity of cliffs to right providing coarse material and lateral control of stream.

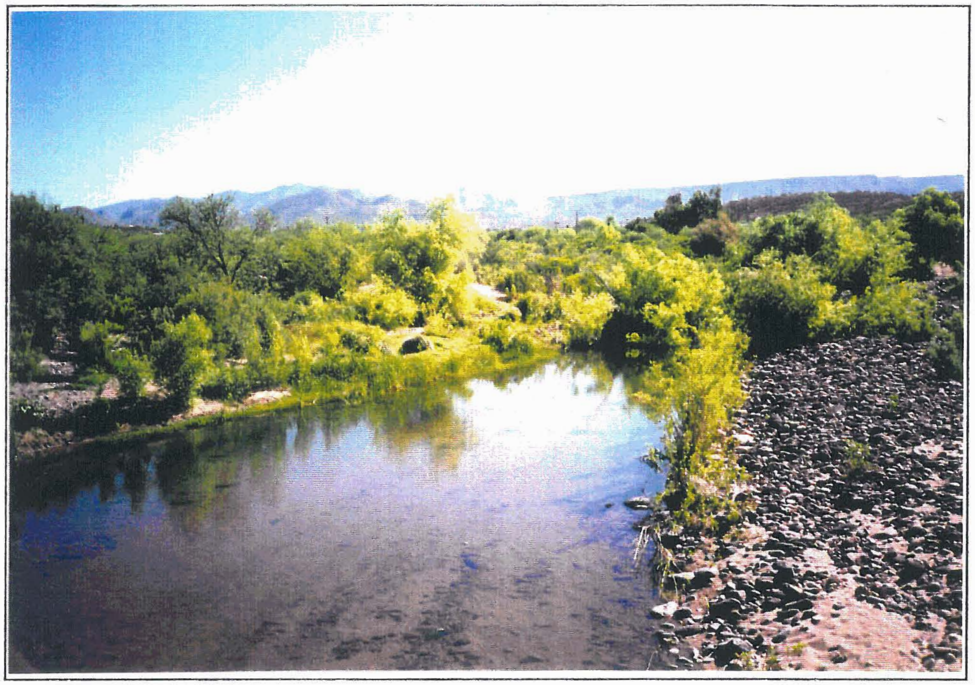


Photo 3: View looking upstream from bridge deck. Note relatively coarse material typical of upstream left bank near structure and well established vegetation along right bank of main channel. Also note rapid reduction in width of primary channel upstream of bridge.

Photo 4: View looking west toward upstream face of structure from right bank. Note mature vegetation on typical of right overbank. Also note large amount of debris captured.

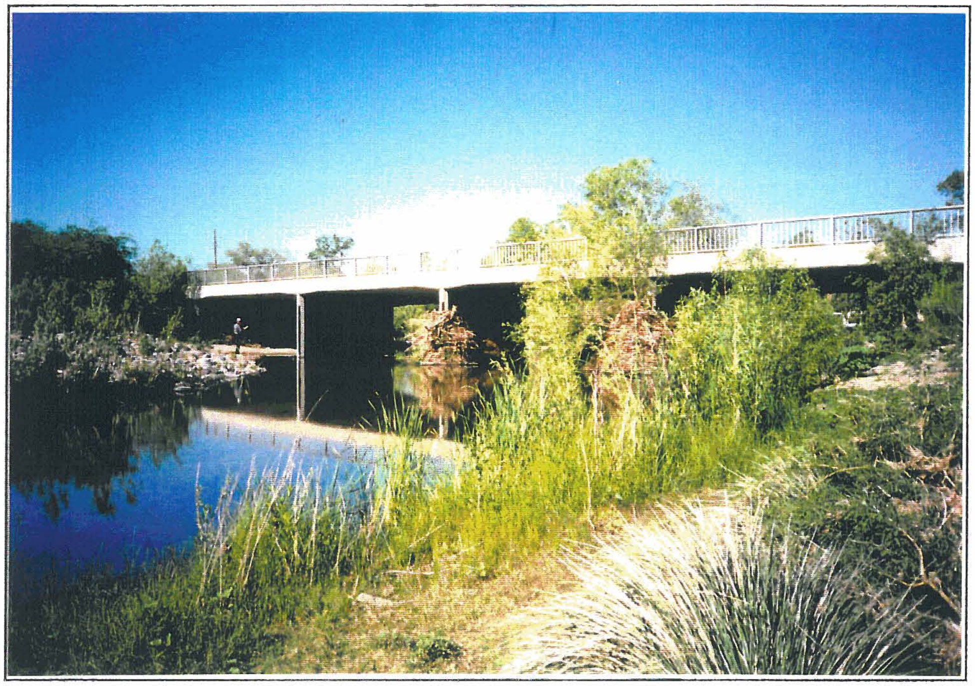


Photo 5: View of upstream face of bridge from right bank. Note expansion of relatively narrow low flow channel occurring at upstream face. Also note that main flow is through spans 2, 3, and 4. Further note substantial debris build-up on piers 2 and 3.

Photo 6: View of downstream face of bridge at piers 1 and 2. Note presence of concrete slab functioning as limited grade control and as flow diversion.

Photo 7: View of downstream face of bridge near pier 1. Note boulder-sized material up to 2 feet on axis may form armor surface in portions of channel and may possibly accelerate scour in less protected portions of channel.

Photo 8: View looking downstream near downstream face of bridge. Note coarse texture of primary channel material may form armor surface. Also note close proximity of mature vegetation to primary channel may provide significant flow resistance under high flow conditions. Note possible bedrock outcrop at mid-channel in background.

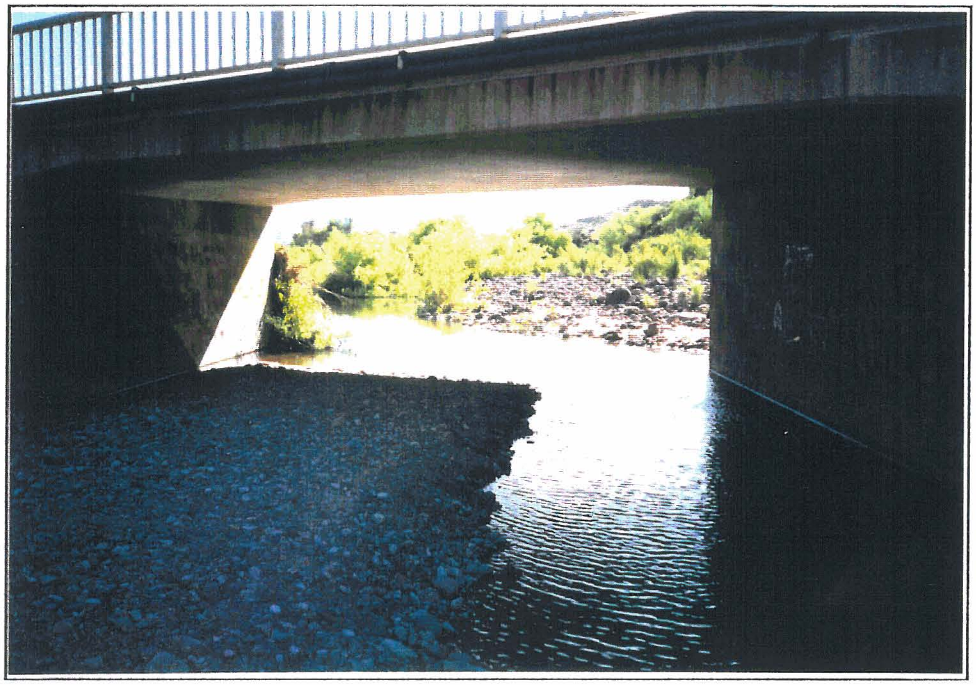


Photo 9: View of span 3 from downstream face. Note pattern of sediment deposition indicates that the angle of attack of recent flows may be north of pier skew.

Photo 10: View alongside low flow channel approximately 125 feet downstream. Note that captured debris was measured to heights up to 10 feet above channel in this vicinity.