Maricopa County
Department of
Transportation

Bridge Scour Investigation and Design of Corrective Measures

Contract No. CY 1997-26
Work Order No. 80407

Final Report

MC 85 Highway Bridge over Agua Fria River

Submitted August 28, 1997

Registered Professional Engineer (Structural)
CERTIFICATE NO.
30340
DENNIS J.
TREFREN
Date Signed 8/28/97
ARIZONA, U.S.A.

Prepared By:
INCA
INCA ENGINEERS, INC.
Wood/Patel & Associates
Maxim Technologies Inc.

TABLE OF CONTENTS

FUGURES

APPENDICES

Registered Professional Engineer (Structural)
CERTIFICATE NO.
30340
DENNIS J.
TREFREN
Date Signed 8/28/97
ARIZONA, U.S.A.

Bridge Scour Investigation and Design of Corrective Measures

FINAL REPORT

INTRODUCTION

The Maricopa County Department of Transportation retained two consultants in 1995 under Work Order Number 80407 to evaluate the scour potential during 100 and 500 year flood events for existing bridges in their jurisdiction over waterways. The results of that study classified some of the bridges as scour critical.

INCA Engineers, Inc. was retained by the County to review the previous reports for five bridges classified as scour critical, determine the extent of scour damage, recommend methods to prevent scour damage, and prepare contract documents for scour countermeasures.

The MC 85 Highway Bridge over Agua Fria River was evaluated as scour critical by Cannon and Associates, Inc. and documented in their report dated July 1996 (Revised November 1996).

Bridge Location and Description:

The Maricopa County Highway 85 Bridge crossing of the Agua Fria River is in Section 4, T1N, R1W, Gila and Salt River Baseline and Meridian near the town of Avondale, Arizona. The road parallels and is immediately downstream of the Union Pacific Railroad (UPRR) bridge crossing of the Agua Fria River.

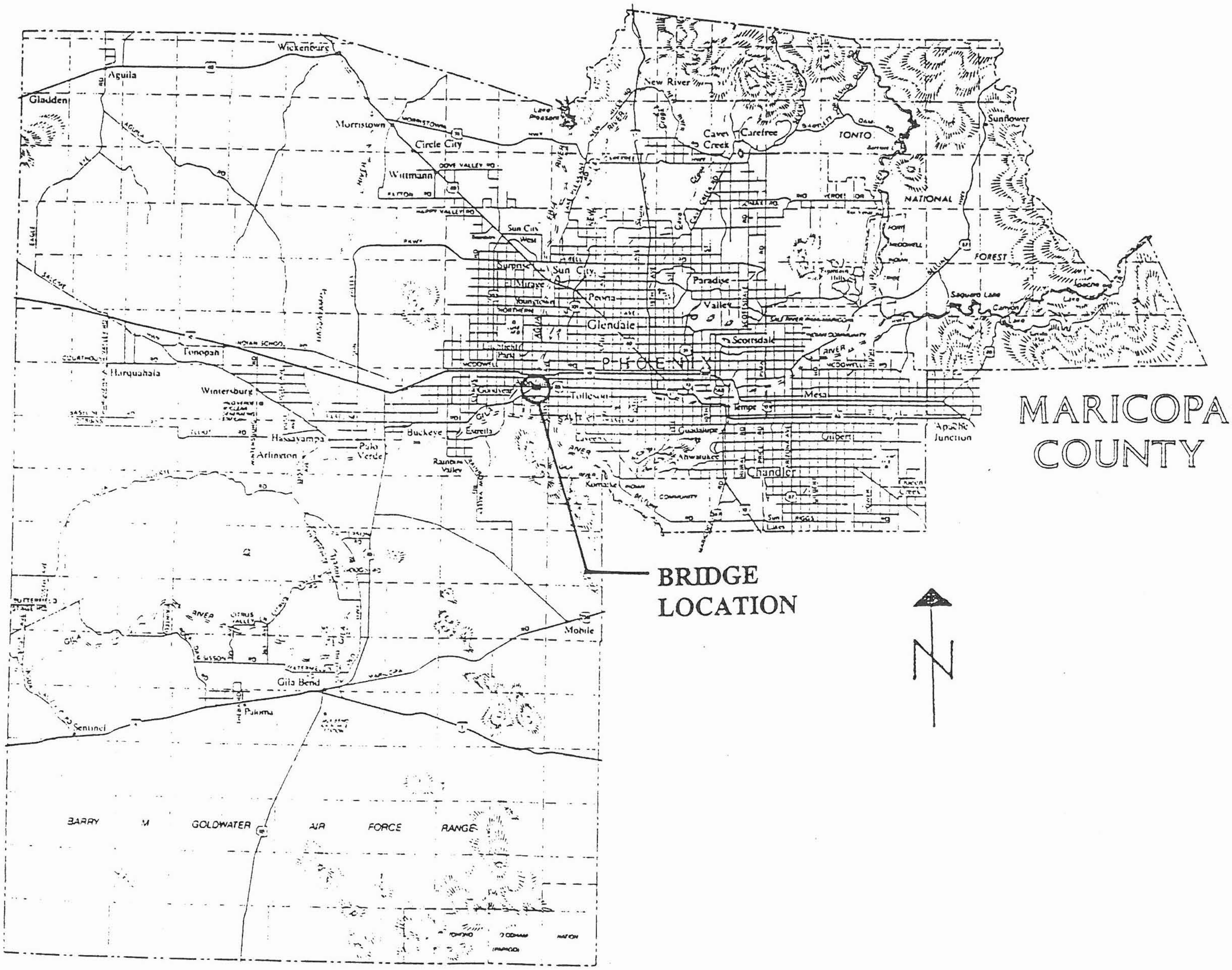
REVIEW OF PREVIOUS REPORT

Cannon & Associates, Inc. performed a scour investigation and structural stability analysis of this site and submitted a report dated July 1996 (Revised November 1996) documenting their findings. Wood/Patel has reviewed this report and offers the following comments:

- On page 1, paragraph 5, the report states that "Pile driving records indicate that the piles are founded between 12' and 17' below the existing river flow line." However, in the next paragraph on the same page the report states that "Pile driving records indicate that the piles are founded between 16' and 30' below the bottom of the existing pile cap." An examination of the bridge construction plans in relation to the current channel bed elevation indicates that the design depth of the piles at the piers is approximately 28' below the channel bed or 24.2' below the pile caps. The design depth of the piles at the abutments is approximately 17' below the channel bed or 29.5' below the abutment pile caps.

MC 85 Highway Bridge over Agua Fria River

Location Map



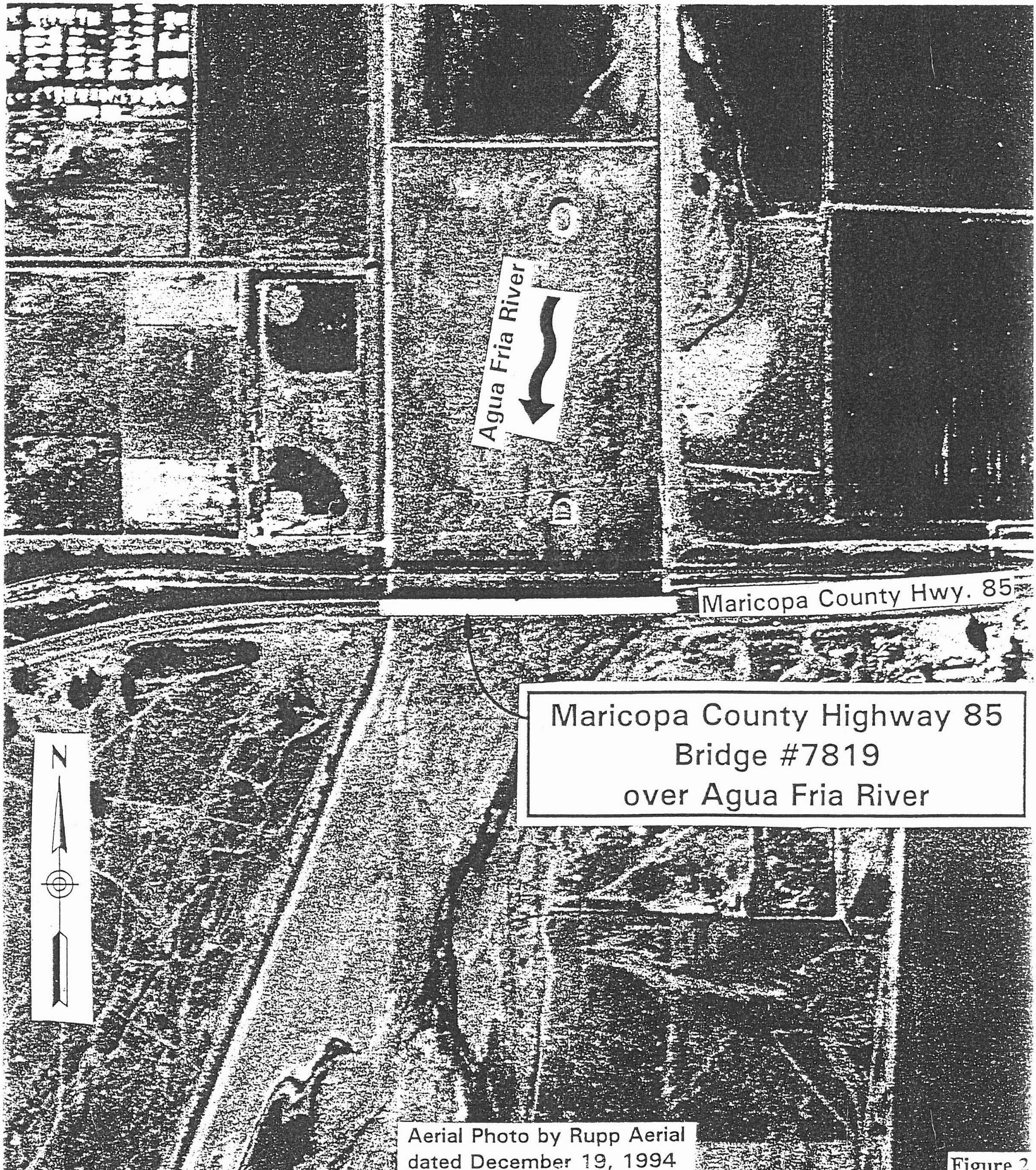

Agua Fria River
Maricopa County Hwy. 85
Maricopa County Highway 85
Bridge #7819
over Agua Fria River
N
Aerial Photo by Rupp Aerial
dated December 19, 1994

Figure 2

SITE INVESTIGATION

On June 17, 1997, a review of the site conditions was conducted by Dennis Trefren, P.E., and Richard Bruesch, P.E. of INCA, Jeff Holzmeister, P.E. and Rick Hiner, P.E. of Wood/Patel, Dave Thomas, P.E. of Maxim Technologies and Tom Sonnemann, P.E. of MCDOT. Observations were noted as the following:

1. Movement of individual stones from the dumped riprap is evident.

2. There is a three to five foot head cut near the downstream face of the dumped riprap and additional degradation noted further downstream. Based on this observation, it is recommended that a long-term scour estimate of five (5) feet be included in the total scour for the bridge site.

3. Approximately one quarter mile upstream of the bridge irrigation tailwater discharges into the river from three pipes. This runoff follows the east bank at the base of the soil-cement slope until it reaches the east end of the MC 85 bridge, at which point it flows parallel to and beneath the bridge deck, in the existing riprap blanket, for about 800 feet.

4. Irrigation tailwater also enters the river channel through the east hard bank between the highway and railroad bridges. This water has contributed to accelerated growth of grass and bushes in the low flow channel.

5. Both east and west river banks are protected by soil cement that is in good condition. Both abutments are adequately protected.

A discussion at the site concluded that a soil cement floor would appear to be the most feasible alternative. The in-situ material is well suited for soil cement. The dumped riprap should be placed at the downstream edge of the new floor cut off wall. Any material left over should be salvaged for use at other sites within the County.

HYDROLOGY RECOMMENDATIONS

Wood/Patel reviewed the hydrology from the Final Bridge Scour Assessment Report prepared by Cannon & Associates, Inc. The 100-year discharge of 95,000 cfs (FCDMD) and 500-year discharge of 184,000 cfs (FEMA) have been recently superceded by a Corps of Engineers study on the effects of the New Waddell Dam. The new discharges published by the Corps are approximately 51,000 cfs for the 100-year event and approximately 115,000 cfs for the 500-year event (values are interpolated based on the bridge location).

HYDRAULICS RECOMMENDATIONS

The hydraulics performed in the Final Bridge Scour Assessment Report prepared by Cannon & Associates, Inc. used a single section to determine the hydraulic characteristics of the bridge crossing. The analysis was performed in accordance with current methodology and appears to have been done correctly. Due to the high degree of uniformity in the channel cross section through this section of the river, a single section model should yield an acceptable estimation of the water surface elevation at the bridge structure. However, due to the change in discharge values, the analysis was reevaluated and new scour values were computed. The results are presented below and the calculations are contained in Appendix C.

SCOUR ANALYSIS

A review of the methodology previously used indicates that reasonable assumptions were made and the procedures utilized to compute scour are in accordance with HEC-18 methodology. The results of the revised analysis are presented below:

	100-year	**500-year**
Contraction Scour	0.00 feet	0.00 feet
Pier Scour	13.83 feet*	16.53 feet*
Long-Term Scour	5.00 feet	5.00 feet
Abutment Scour	0.00 feet	0.00 feet

*Assumes that dumped riprap sill is not scour resistant.

This yields a total scour at the piers of 18.83 feet for the 100-year event and 21.53 feet for the 500-year event (4.47 feet and 1.77 feet remaining pile embedment, respectively). The total scour at the abutments for both the 100-year and 500-year events is assumed to be 0.0 feet since the upstream channel is very uniform and there is no overbank flow.

Observations made during the field visit indicate that a head cut could be migrating upstream towards the bridge structure and it is recommended that five (5) feet of long-term scour be included for structural stability calculations.

LOCAL SCOUR AT DOWNSTREAM EDGE OF FLOOR:

Calculations were performed to estimate the local scour at the downstream edge of the proposed floor. These calculations were performed in accordance with the methodologies outlined in the Bureau of Reclamation document entitled Computing Degradation and Local Scour by Ernest Pemberton and Joseph Lara, 1984. The results are presented below and the calculations are included in the appendix.

100-year Local Scour at Floor	7.1 feet
500-year Local Scour at Floor	11.8 feet

If the long-term degradation estimate is added to these values, the recommended toe-down is 12 feet for the 100-year event and 17 feet for the 500-year event.

These estimates do not include the mitigating effect of placing the riprap on the downstream sill, which would tend to reduce the amount of scour significantly. The riprap should, therefore, provide an extra measure of safety but it is not integral to the performance of the structure.

ALTERNATIVE COUNTERMEASURES

The following is a discussion of the most feasible countermeasures.

Alternative 1:

This alternative consists of constructing a soil cement floor below the river bottom from the east bank to the west bank and ties into the existing soil cement bank protection. The soil cement floor will be keyed into the river bottom to a depth of 12 feet at a 1:1 slope on the upstream side and to a depth of 17 feet at a 2:1 slope on the downstream side. The riprap which is removed from beneath the bridge structure will be placed in the toe trench excavated for the soil cement floor on the downstream side.

Advantages of this alternative are:

- Provides scour resistant layer for all piers.
- Provides for grade control at the site.
- Uses a proven material and construction method, and is consistent with the existing banks.
- The least costly alternative.
- Utilizes salvaged existing dumped riprap at the toe of the soil cement floor on the downstream side.

Disadvantages of this alternative:

- Requires deep excavations for toe-down sections.
- Rigid system, one that could be damaged if undercut.
- May require temporary construction easements.
- Will require a temporary storage location for excavated material and processing soil cement.

The estimated cost for this alternative is $2,100,000.

Other Floor Systems:

Other floor systems such as wire tied riprap and reinforced concrete were not considered since their cost is at least twice that of soil cement.

Another alternative would be to grout over the existing riprap. This alternative wasn't considered since the existing riprap would have to be removed and cleaned before covering it with grout in order to develop a good bond between the grout and riprap. The cost of this alternative would be prohibitive.

PREFERRED ALTERNATIVE

We recommend Alternative 1 to be constructed since it offers the best scour protection at the lowest cost.

Appendix A
Photographs

Channel Degradation Downstream

Riprap Moving Downstream and Channel Degradation

West Bank Upstream

East Bank Upstream

West Bank Downstream

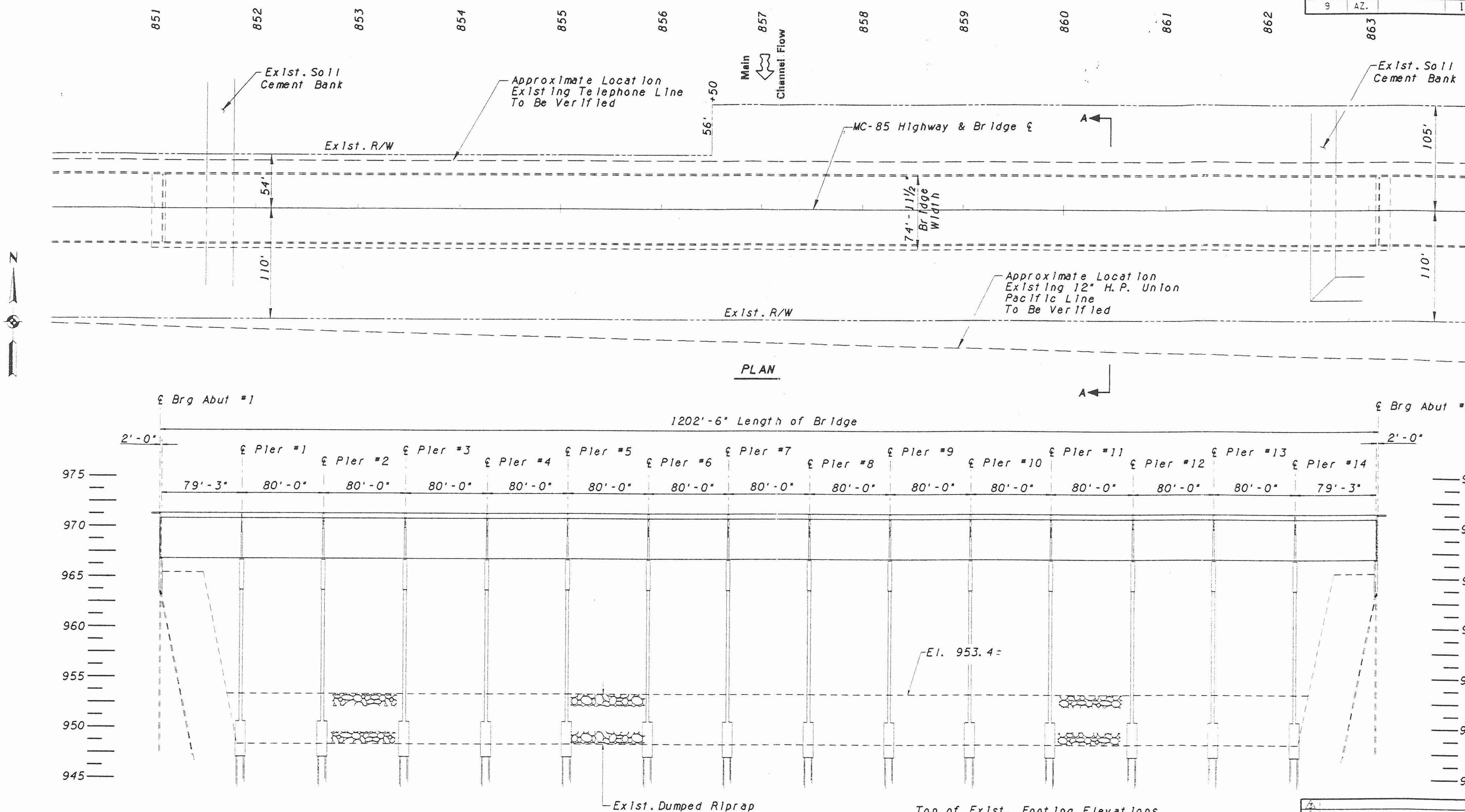
Southeast Corner of Bridge

Typical Size of Dumped Riprap

Channel Bottom Under Bridge Near East Side

Appendix B
Alternative 1 Details

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ.		1	2	



ELEVATION

Top of Exist. Footing Elevations

Pier 1 --- 950.43
Pier 2 --- 950.51
Pier 3 --- 950.59
Pier 4 --- 950.67
Pier 5 --- 950.75
Pier 6 --- 950.83
Pier 7 --- 950.91
Pier 8 --- 950.99
Pier 9 --- 951.07
Pier 10 -- 951.15
Pier 11 -- 951.23
Pier 12 -- 951.31
Pier 13 -- 951.39
Pier 14 -- 951.47

NO.	REVISION	BY	DATE

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ENGINEERING DIVISION

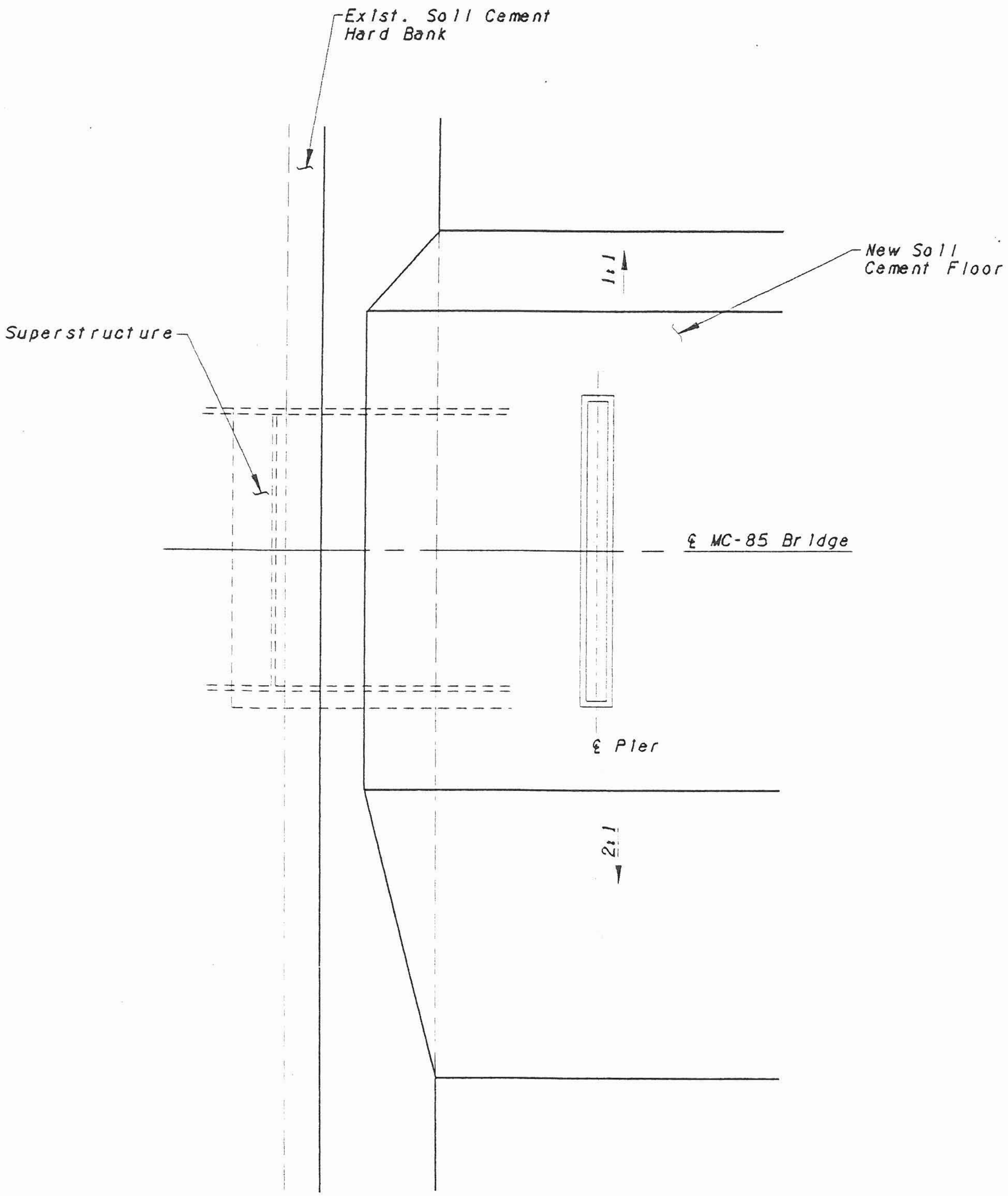
MC-85 HIGHWAY AT AGUA FRIA RIVER

		BY	DATE
PRELIMINARY NOT FOR CONSTRUCTION	DESIGNED		
	DRAWN	RON WIETZEMA	8/27/97
	CHECKED		

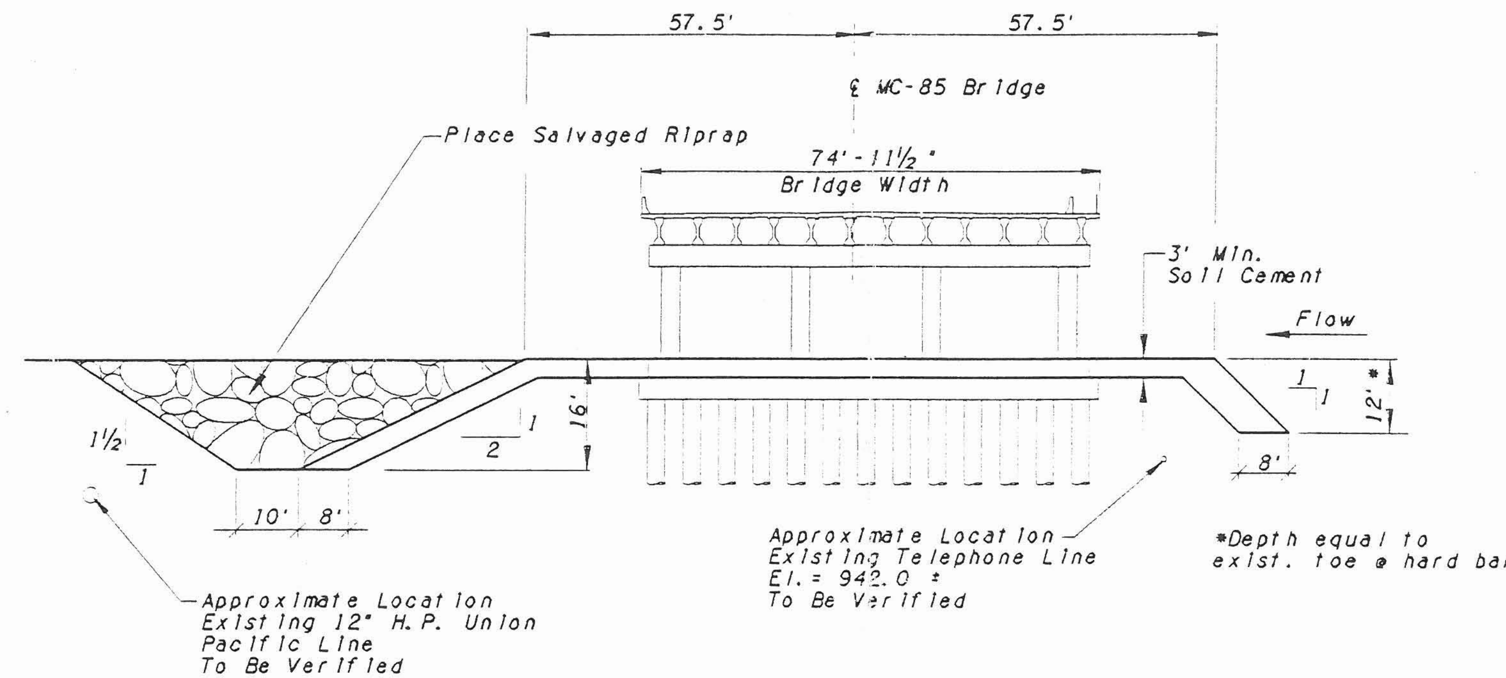
INCA ENGINEERS INC.

ALTERNATE NO. 1 — SHEET 1 OF 2

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ.		2	2	



PARTIAL PLAN - WEST ABUTMENT



SECTION A-A

NO.	REVISION	BY	DATE
3			
2			
1			

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ENGINEERING DIVISION

MC-85 HIGHWAY AT AGUA FRIA RIVER

	BY	DATE
DESIGNED		
DRAWN		
CHECKED		

INCA
INCA ENGINEERS INC.

ALTERNATE NO. 1

SHEET 2 OF 2

Appendix C
Hydraulic Calculations

PIER SCOUR CALCULATION SHEET

Consultant: Wood, Patel & Associates, Inc. Project # 95273 Sheet # 1 of 1

References: FHWA's HEC-18 (third edition) and Interum Procedure for Estimating Pier Scour with Debris

Project Name: MCDOT Bridge Scour Evaluation Date: 8-27-97

Engineer: R. HINER Checked By: ______ Structure #: MC85 @ AGUA FRIA

HYDROLOGIC/HYDRAULIC PARAMETERS

Return Interval	100 yrs.	Hydraulics Source	FLOWMASTER
Hydrology Source	FEMA	Flow Top Width (T)	1111.64 ft.
Discharge (Q)	51,000 cfs	Flow Area (A)	6683.78 ft^2
Water Surface Elev.	958.49 ft.	Channel Slope (S)	0.0019 ft/ft
Thalweg Elevation	951.3 ft.	Max. Velocity (Vm)	7.63 ft/sec
Max. Depth of Flow (Y1)	7.19 ft.	Froude Number (Fr)	0.55 (Vm^2*T/g*A)^.5

PIER SCOUR CALCULATIONS

Pier Type: Stemwall ______ Columns X

Foundation Type: Spread Ftng ______ Piles/Drilled Shaft X

K1 = 1.1 Correction factor for pier nose shape (assume square nose pier K1=1.1 Table 2, pg. 40, HEC-18. For multiple column piers and stemwalls skewed to the flow, K1=1.0)

Angle of Attack (theta) = 0° (15 degree min. for stemwall piers if there is potential channel meandering)

Pier Width (Wp) = 3.5 ft. Number of Columns/Piles per bent: ______

Dist. Between Columns = >5D ft. (Clear space must exceed 5 pier diameters for independent analysis)

Debris Blockout (Wd) = 4.0 ft. (Based on debris potential; low = 2 ft., medium = 3 ft., high = 4 ft.)

Length of Pier (L) = — ft. Coefficient for bed condition K3= 1.1 (Table 1, pg. 39)

Effective Pier Length (L') = — ft. L' = L or 12*Wp whichever is less.

Effective Pier Width (a) = 7.5 ft. (The greater of Wp*cos(theta)+Wd or (Wp*cos(theta)+Wd)/2+L'sin(theta))

K2 = 1.0 (For stemwall, multiple column, and single column piers K2=1.0)

Colorado State University Equation (HEC-18 pg. 52)

$$Y_s = Y_1 2.0 K_1 K_2 K_3 \left(\frac{a}{Y_1}\right)^{0.65} Fr^{0.43}$$

Depth of Pier Scour Hole = 13.83 ft. + 5' Long-Term Degradation = 18.83 ft

Elev. @ Btm of Scour Hole= 932.47 ft.

Elev. @ Bottom of Footing= 947.7 ft. 15.23' exposed piling

Elev. @ Min. Tip of Pile= 928.0 ft. 4.47' remaining embedment

FAILS – INSUFFICIENT EMBEDMENT

PIER SCOUR CALCULATION SHEET

Consultant: **Wood, Patel & Associates, Inc.** Project # **95273** **Sheet # 1 of 1**

References: **FHWA's HEC-18 (third edition) and Interum Procedure for Estimating Pier Scour with Debris**

Project Name: **MCDOT Bridge Scour Evaluation** Date: 8-27-97

Engineer: R. HINER Checked By: _____ Structure #: MC85 @ AGUA FRIA

HYDROLOGIC/HYDRAULIC PARAMETERS

Return Interval	500 yrs.	Hydraulics Source	FLOWMASTER
Hydrology Source	FEMA	Flow Top Width (T)	1120.1 ft.
Discharge (Q)	115,000 cfs	Flow Area (A)	10931.04 ft^2
Water Surface Elev.	962.29 ft.	Channel Slope (S)	0.0019 ft/ft
Thalweg Elevation	951.3 ft.	Max. Velocity (Vm)	10.52 ft/sec 30% over Avg
Max. Depth of Flow (Y1)	10.99 ft.	Froude Number (Fr)	0.59 (Vm^2*T/g*A)^.5

PIER SCOUR CALCULATIONS

Pier Type: Stemwall _____ Columns X

Foundation Type: Spread Ftng _____ Piles/~~Drilled Shaft~~ X

K1 = 1.1 Correction factor for pier nose shape (assume square nose pier K1=1.1 Table 2, pg. 40, HEC-18. For multiple column piers and stemwalls skewed to the flow, K1=1.0)

Angle of Attack (theta) = 0° (15 degree min. for stemwall piers if there is potential channel meandering)

Pier Width (Wp) = 3.5 ft. Number of Columns/Piles per bent: _____

Dist. Between Columns = > 50 ft. (Clear space must exceed 5 pier diameters for independent analysis)

Debris Blockout (Wd) = 4.0 ft. (Based on debris potential; low = 2 ft., medium = 3 ft., high = 4 ft.)

Length of Pier (L) = — ft. Coefficient for bed condition K3: 1.1 (Table 1, pg. 39)

Effective Pier Length (L') = — ft. L' = L or 12*Wp whichever is less.

Effective Pier Width (a) = 7.5 ft. (The greater of Wp*cos(theta)+Wd or (Wp*cos(theta)+Wd)/2+L'sin(theta))

K2 = 1.0 (For stemwall, multiple column, and single column piers K2=1.0)

Colorado State University Equation (HEC-18 pg. 52)

$$Y_s = Y_1 2.0 K_1 K_2 K_3 \left(\frac{a}{Y_1}\right)^{0.65} Fr^{0.43}$$

Depth of Pier Scour Hole = 16.53 ft. +5' Long-Term degradation = 21.53 ft

Elev. @ Btm of Scour Hole= 929.77 ft.

Elev. @ Bottom of Footing= 947.7 ft. avg. 17.93' exposed pilling

Elev. @ Min. Tip of Pile= 928.0 ft. avg. 1.77' remaining embedment

FAILS - INSUFFICIENT EMBEDMENT

MC85 Bridge over the Agua Fria River

100-year Scour Estimate Downstream of Floored Bridge Structure

Methodology from "Computing Degradation and Local Scour" by E. Pemberton and J. Lara, 1984,
Technical Guideline for Bureau of Reclamation, pages 40-45, equation type "D"

100-year Post-Waddell Dam Discharge =	51,000	cfs	5	ft Long-Term Degradation
Total Flow Area =	6683.78	ft^2		
Total Top Width =	1111.64	ft		
Mean Flow Depth =	6.01	ft		
Discharge per foot =	45.88	cfs/ft		

Schoklitsch (1932)

$$\boxed{d_s = \frac{K(H)^{0.2} q^{0.57}}{D_{90}^{\ 0.32}} - d_m}$$

ds = 7.7 ft

d_s = depth of scour (ft)

K =	3.15	3.15 inch-pound units
H =	5	difference between U/S and D/S WSEL
q =	45.88	discharge per unit width (cfs per ft)
D_{90} =	25	particle size for which 90% is finer (mm)
d_m =	6.01	D/S mean water depth

Veronese (1937)

$$\boxed{d_s = KH_T^{\ 0.225} q^{0.54} - d_m}$$

ds = 9.0 ft

d_s = depth of scour (ft)

K =	1.32	1.32 inch-pound units
H_T =	5	head from U/S to D/S
q =	45.88	discharge per unit width (cfs per ft)
d_m =	6.01	D/S mean water depth

Zimmerman & Maniak (1967)

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

ds = 4.7 ft

d_s = depth of scour (ft)

K =	1.95	1.95 inch-pound units
q =	45.88	discharge per unit width (cfs per ft)
D_{85} =	23.5	particle size for which 85% is finer (mm)
d_m =	6.01	D/S mean water depth

Average Scour Depth = 7.1 ft

Recommended Downstream Toe-Down = 13 ft (local scour + long-term)

Note: D85 and D90 estimated from field investigation, photographic data,

MC85 Bridge over the Agua Fria River

500-year Scour Estimate Downstream of Floored Bridge Structure

Methodology from "Computing Degradation and Local Scour" by E. Pemberton and J. Lara, 1984, Technical Guideline for Bureau of Reclamation, pages 40-45, equation type "D"

500-year Post-Waddell Dam Discharge =	115,000	cfs	5	ft Long-Term Degradation
Total Flow Area =	10931.04	ft^2		
Total Top Width =	1120.1	ft		
Mean Flow Depth =	9.76	ft		
Discharge per foot =	102.67	cfs/ft		

Schoklitsch (1932)

$$\boxed{d_s = \frac{K(H)^{0.2} q^{0.57}}{D_{90}^{\ 0.32}} - d_m}$$

ds = 12.0 ft

d_s =	depth of scour (ft)	
K =	3.15	3.15 inch-pound units
H =	5	difference between U/S and D/S WSEL
q =	102.67	discharge per unit width (cfs per ft)
D_{90} =	25	particle size for which 90% is finer (mm)
d_m =	9.76	D/S mean water depth

Veronese (1937)

$$\boxed{d_s = KH_T^{\ 0.225} q^{0.54} - d_m}$$

ds = 13.4 ft

d_s =	depth of scour (ft)	
K =	1.32	1.32 inch-pound units
H_T =	5	head from U/S to D/S
q =	102.6694	discharge per unit width (cfs per ft)
d_m =	9.76	D/S mean water depth

Zimmerman & Maniak (1967)

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

ds = 10.1 ft

d_s =	depth of scour (ft)	
K =	1.95	1.95 inch-pound units
q =	102.6694	discharge per unit width (cfs per ft)
D_{85} =	23.5	particle size for which 85% is finer (mm)
d_m =	9.76	D/S mean water depth

Average Scour Depth = 11.8 ft

Recommended Downstream Toe-Down = 17 ft (local scour + long-term)

Note: D_{85} and D_{90} estimated from field investigation, photographic data,

MC85 Bridge @ Agua Fria - 51,000 cfs
Worksheet for Irregular Channel

Project Description	
Project File	untitled
Worksheet	MC85 Bridge @ Agua Fria River
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Water Elevation

Input Data

Channel Slope 0.001900 ft/ft

Elevation range: 951.30 ft to 967.70 ft.

Station (ft)	Elevation (ft)	Start Station	End Station	Roughness
0.00	967.70	0.00	1144.00	0.028
6.00	966.40			
18.00	955.30			
50.00	954.00			
211.00	952.80			
451.00	952.10			
531.00	952.80			
611.00	952.30			
1091.00	951.70			
1118.00	951.30			
1135.00	966.20			
1144.00	966.80			

Discharge 51000.00 ft^3/s

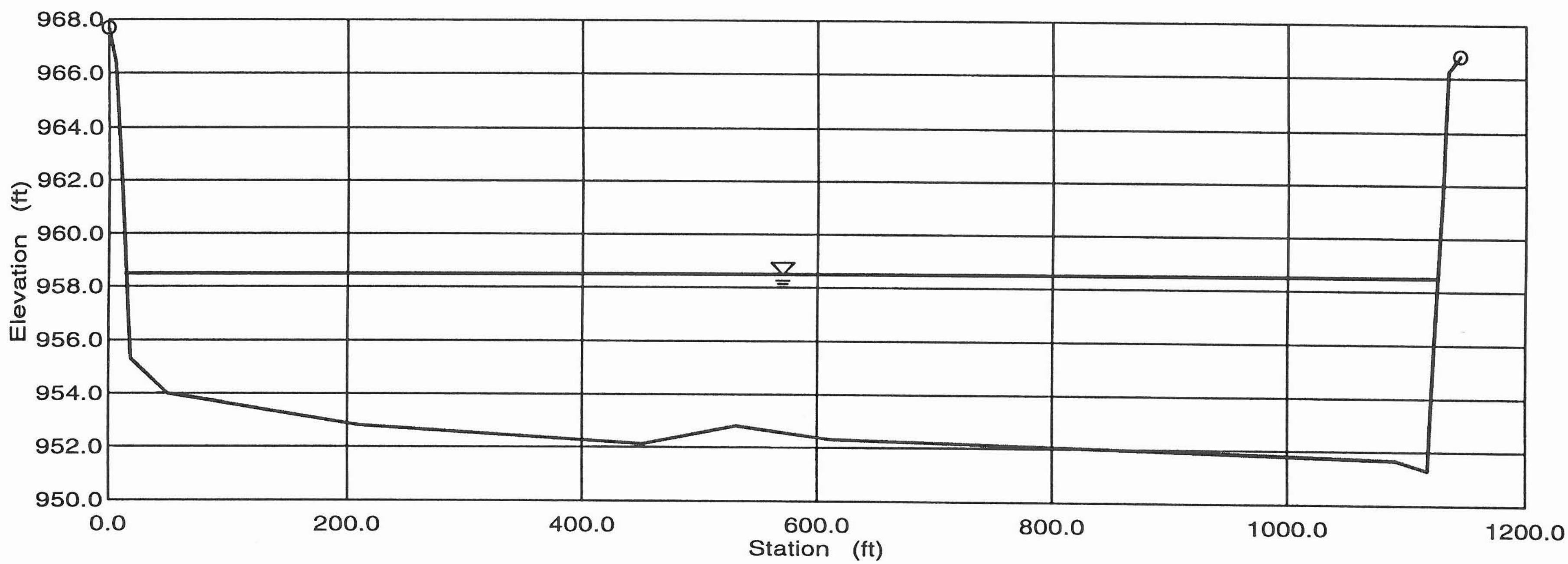
Results		
Wtd. Mannings Coefficient	0.028	
Water Surface Elevation	958.49	ft
Flow Area	6683.78	ft^2
Wetted Perimeter	1115.63	ft
Top Width	1111.64	ft
Depth	7.19	ft
Critical Water Elev.	956.49	ft
Critical Slope	0.007194	ft/ft
Velocity	7.63	ft/s
Velocity Head	0.90	ft
Specific Energy	959.39	ft
Froude Number	0.55	
Full Flow Capacity	236967.64	ft^3/s

Flow is subcritical.

MC85 Bridge @ Agua Fria - 51,000 cfs
Cross Section for Irregular Channel

Project Description	
Project File	untitled
Worksheet	MC85 Bridge @ Agua Fria River
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Water Elevation

Section Data		
Wtd. Mannings Coefficient	0.028	
Channel Slope	0.001900	ft/ft
Water Surface Elevation	958.49	ft
Discharge	51000.00	ft³/s



MC85 Bridge @ Agua Fria - 115,000 cfs
Worksheet for Irregular Channel

Project Description	
Project File	untitled
Worksheet	MC85 Bridge @ Agua Fria River
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Water Elevation

Input Data

Channel Slope 0.001900 ft/ft

Elevation range: 951.30 ft to 967.70 ft.

Station (ft)	Elevation (ft)	Start Station	End Station	Roughness
0.00	967.70	0.00	1144.00	0.028
6.00	966.40			
18.00	955.30			
50.00	954.00			
211.00	952.80			
451.00	952.10			
531.00	952.80			
611.00	952.30			
1091.00	951.70			
1118.00	951.30			
1135.00	966.20			
1144.00	966.80			

Discharge 115000.00 ft^3/s

Results		
Wtd. Mannings Coefficient	0.028	
Water Surface Elevation	962.29	ft
Flow Area	10931.04	ft^2
Wetted Perimeter	1127.01	ft
Top Width	1120.10	ft
Depth	10.99	ft
Critical Water Elev.	959.40	ft
Critical Slope	0.006029	ft/ft
Velocity	10.52	ft/s
Velocity Head	1.72	ft
Specific Energy	964.01	ft
Froude Number	0.59	
Full Flow Capacity	236967.64	ft^3/s
Flow is subcritical.		

MC85 Bridge @ Agua Fria - 115,000 cfs
Cross Section for Irregular Channel

Project Description	
Project File	untitled
Worksheet	MC85 Bridge @ Agua Fria River
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Water Elevation

Section Data		
Wtd. Mannings Coefficient	0.028	
Channel Slope	0.001900	ft/ft
Water Surface Elevation	962.29	ft
Discharge	115000.00	ft³/s