MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION

OLD US-80 GILA RIVER BRIDGE (GILLESPIE DAM BRIDGE)

Historic Bridge Rehabilitation

Structure No. 8021 Project No. SB 428 01C TRACS No. STP/BR –MMA -0(202) - A

Maricopa County Dept. of Transportation Contract No. 2005-056, Work Order No. TT188

Final Design Concept Report
Volume 1

July 26, 2007

Registered Professional Engineer (Civil)
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ARIZONA, U.S.A.

TranSystems

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VOLUME 1 – DESIGN CONCEPT REPORT
VOLUME 2 – TECHNICAL REPORTS
VOLUME 3 – MISCELLANEOUS INFORMATION

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Structural Grace – Structural Analysis
Southwest Traffic Engineering - Traffic
Primatech, LLC/Jacobs – Drainage/Scour
Olson Engineering, Inc. – Parallel Seismic
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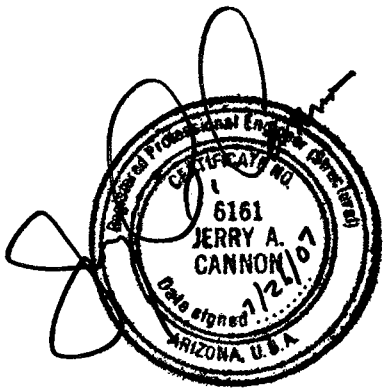

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VOLUME 2 – TECHNICAL REPORTS

Condition Evaluation Report Inspections March 13-16, 2006 dated November 17, 2006

Load Rating Report dated August 31, 2006

Final Traffic Inspection Analysis dated November 16, 2006 revised February 6, 2007

Drainage Report Dated June 2007

Final Geotechnical Engineering Report Dated January 26, 2007

Geotechnical Engineering Report Dated April 13, 2007

Nondestructive Testing Investigation Foundation Lengths and Support Conditions Dated January 4, 2007

Final Report Coating Assessment and Repainting Recommendations Dated July 9, 2006

Cultural Resources Survey and Historical Context of the Gillespie Dam Bridge Potential Impact Area Dated November 15, 2006

HAER Documentation of Gillespie Dam Bridge Dated July 2006

Biological Evaluation Dated October 2006

Section 404 Jurisdictional Delineation Dated August 2006

Preliminary Structural Calculations Dated August 2006

Structure Selection Report for New Bridge dated August 2006

VOLUME 3 – MISCELLANEOUS INFORMATION

Existing Bridge Plans (11" x 17" size) Dated November 1925

Scour Repair Plans Dated April 1994

Historic Property Inventory Form

Right-of-Way Narrative

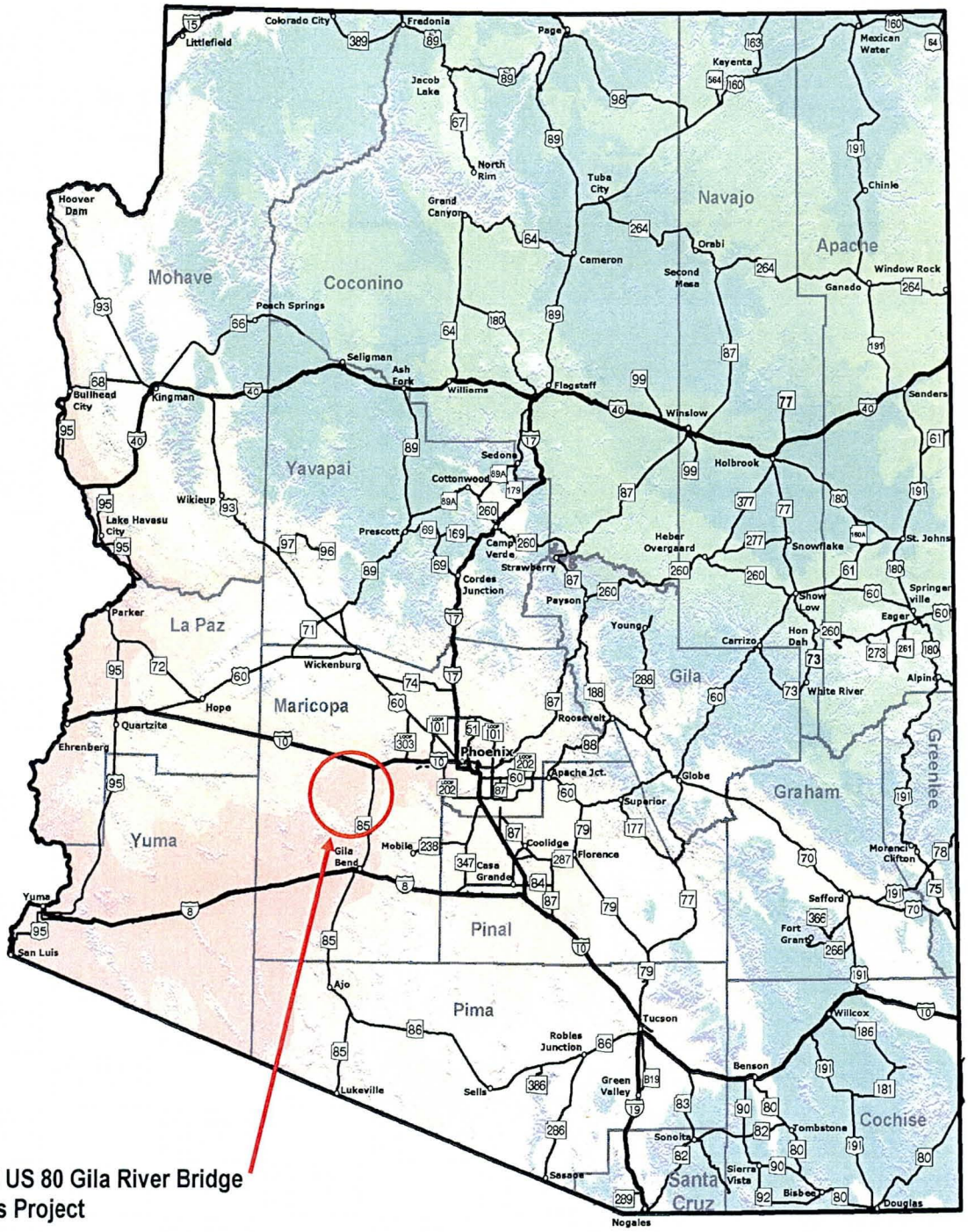


FIGURE 1 – VICINITY MAP

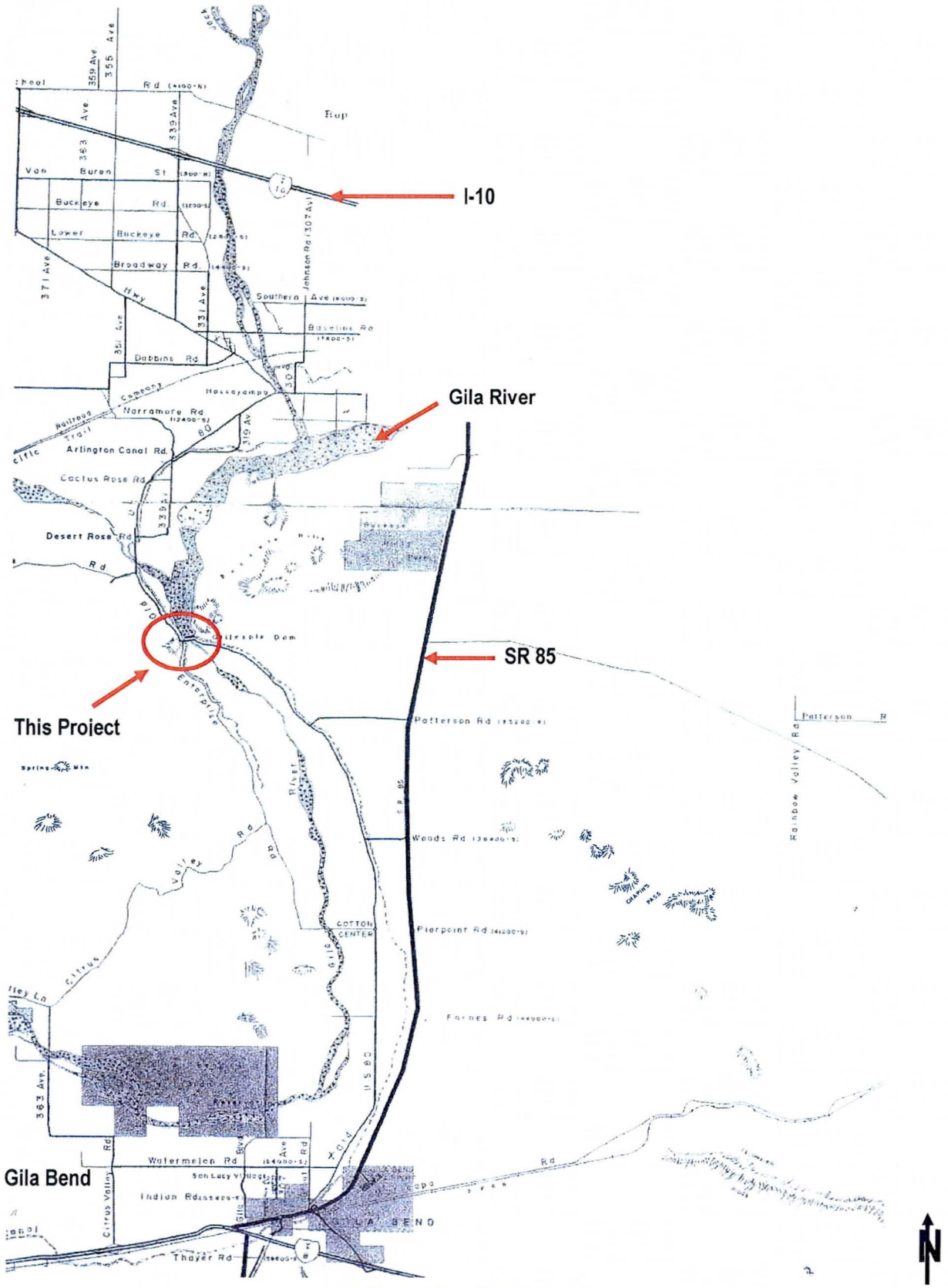


FIGURE 2 – SITE MAP

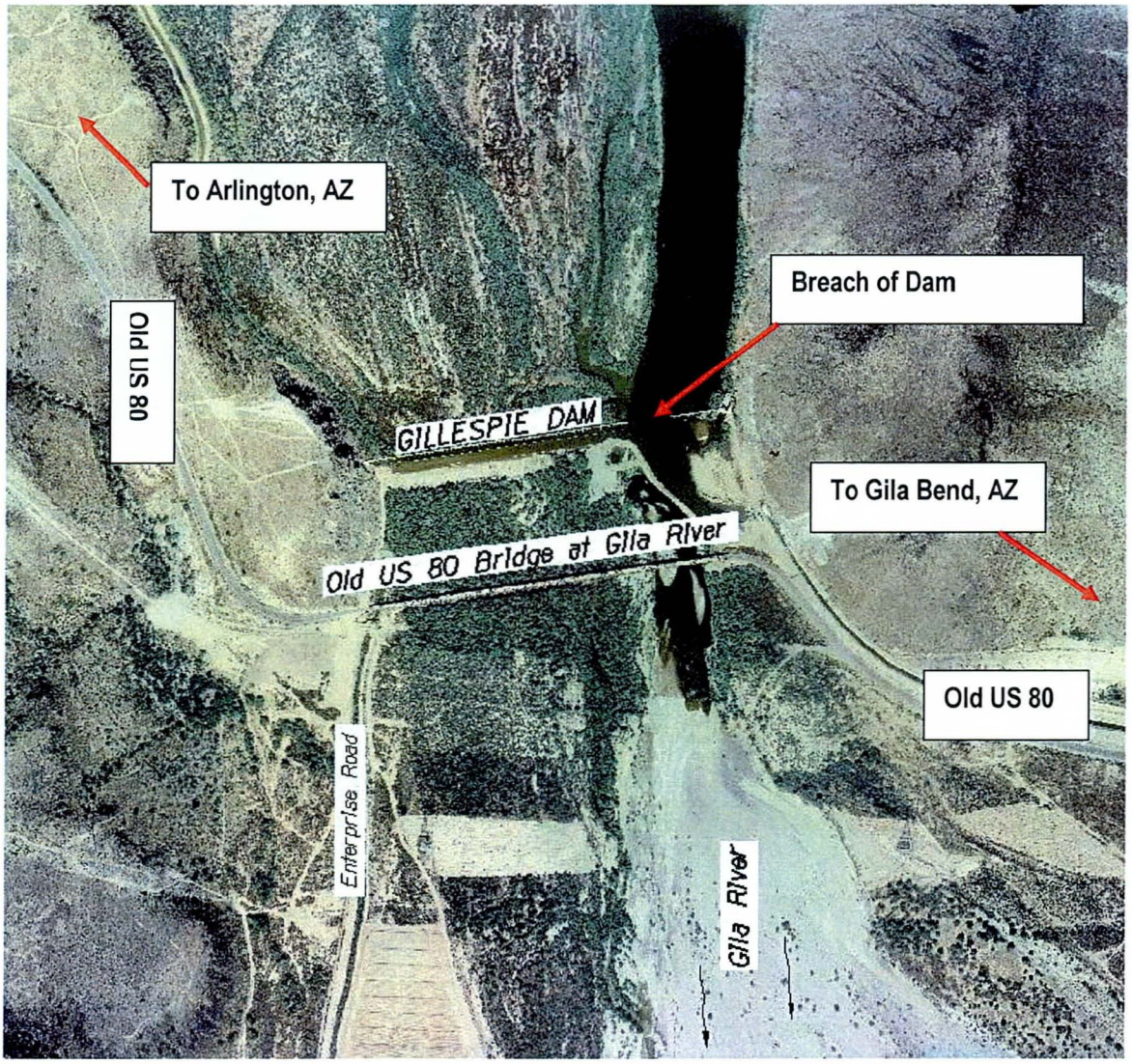


FIGURE 3 - AERIAL PHOTO OF BRIDGE SITE

This project consists of the preparation of an expanded Design Concept Report (DCR) for the historic steel camelback through truss, 9-span, 1665'-8" long bridge. The Historic Old US-80 Bridge at the Gila River is also known as the Gillespie Dam Bridge. It was built in 1927 in the general vicinity of Arlington in Maricopa County and was listed on the National Register of Historic Places in 1981. The bridge is one of the largest steel truss bridges in the state and was a vital link in the US-80 Highway (Ocean-to-Ocean Highway) that connected San Diego, California to Savannah, Georgia. The bridge is one of the most important examples of early bridge construction in Arizona.

The bridge is located on Old US-80 Highway between Arlington and Gila Bend in Arizona, about 6.7 miles south of Arlington. This expanded Design Concept Report is being conducted to determine the feasibility of rehabilitation of the existing bridge which includes an in-depth inspection of the bridge, determining the inventory and operating ratings, and the development of a program for rehabilitation of the bridge in coordination with the State Historic Preservation Office (SHPO). The DCR will be used to provide the Maricopa County Department of Transportation (MCDOT) staff and Transportation Advisory Board with information necessary for comparative evaluation of this project with other Transportation Improvement Program Projects (TIP), it will define the design concepts to be used in the final design, and it will serve as a supporting document for the federal aid request. This DCR is to provide information necessary to act as a coordination document to guide design and reconstruction of this historic bridge rehabilitation as well as the coordination of the future roadway improvements.

In addition to determining the feasibility for rehabilitation of the existing bridge, this study also evaluated the location of a new bridge to supplement the existing bridge when traffic demands or other factors require a new crossing of the Gila River.

This Design Concept Report is a comprehensive report that includes an in-depth steel inspection, structural analysis, traffic analysis, drainage/scour analysis, parallel seismic studies as a part of the geotechnical evaluation, coating assessment, public involvement, and environmental studies.

Based on these studies and evaluation of this information, five alternatives were considered as follows:

Alternative 1	Do Nothing (No Build/Status Quo) – Not Recommended
Alternative 2	Rehabilitate the Existing Bridge – Preferred Alternative
Alternative 3	New Bridge Parallel and Adjacent to Existing Bridge – Not Recommended
Alternative 4	New Bridge Downstream of Existing Bridge – Future Recommended Alternative
Alternative 5	New Bridge Between Existing Bridge and the Dam – Not Recommended

Alternative 2 – Rehabilitate Existing Bridge for Two-Way Traffic (with new pedestrian walkways) is the preferred alternative. The Historic Old US-80 Gila River Bridge (Gillespie Dam Bridge) will be rehabilitated so that it will be adequate to carry two-way traffic. Based on evaluation of engineering, environmental, and cost criteria, the alternative was determined to best meet the objectives and requirements of the project. It preserves the historic bridge and it has the fewest environmental impacts. This alternative will provide an all-weather crossing of the Gila River for the least cost and it meets current traffic demands of the area.

The bridge rehabilitation will require the following work:

- Bridge concrete deck replacement
- Replace exterior stringers
- Concrete wing wall modifications
- Repaint bridge
- Install approach guard rail
- Add pedestrian walkway on each side of the bridge
- Deck joint repair
- Temporary detour during construction using SR 85
- Repair truss bearings
- New bridge barrier rails
- Heat straightening of bent steel members
- Scour protection of six piers
- Provide protective beam at each entrance to bridge
- Remove posting for bridge live loads
- Historic marker
- Miscellaneous other repairs

The estimated project cost for Alternative 2 is $15.9 million.

Alternative 4 – New Bridge Downstream of Existing Bridge is recommended when future development of the area requires a new crossing of the Gila River.

1.1 CHARACTERISTICS OF THE PROJECT

This project consists of the preparation of an expanded Design Concept Report (DCR) for the historic steel camelback through truss, 9-span, 1665'-8" long bridge. The Historic Old US-80 Bridge at the Gila River is also known as the Gillespie Dam Bridge. It was built in 1927 in the general vicinity of Arlington in Maricopa County and was listed on the National Register of Historic Places in 1981. The bridge is one of the largest steel truss bridges in the state and was a vital link in the US-80 Highway (Ocean-to-Ocean Highway) that connected San Diego, California to Savannah, Georgia. The bridge is one of the most important examples of early bridge construction in Arizona.

The bridge is located on Old US-80 Highway between Arlington and Gila Bend in Arizona, about 6.7 miles south of Arlington. See Figures 1, 2 and 3 on pages iv, v, and vi. This expanded Design Concept Report is being conducted to determine the feasibility of rehabilitation of the existing bridge which includes an in-depth inspection of the bridge, determining the inventory and operating ratings, and the development of a program for rehabilitation of the bridge in coordination with the State Historic Preservation Office (SHPO). The DCR will be used to provide the Maricopa County Department of Transportation (MCDOT) staff and Transportation Advisory Board with information necessary for comparative evaluation of this project with other Transportation Improvement Program Projects (TIP), it will define the design concepts to be used in the final design, and it will serve as a supporting document for the federal aid request. This DCR is to provide information necessary to act as a coordination document to guide design and reconstruction of this historic bridge rehabilitation as well as the coordination of the future roadway improvements.

In addition to determining the feasibility for rehabilitation of the existing bridge, this study also evaluated the location of a new bridge to supplement the existing bridge when traffic demands or other factors require a new crossing of the Gila River. This Design Concept Report is a comprehensive report that includes in-depth steel bridge inspection, structural analysis, traffic analysis, drainage/scour analysis, parallel seismic studies as a part of the geotechnical evaluation, coating assessment, public involvement, and environmental studies.

1.2 DESCRIPTION OF THE PROJECT

The main focus of this project is the rehabilitation of the Gillespie Dam Bridge. The existing bridge is a nine-span steel thru-truss bridge with spans between the piers of 162'-0", 162'-6", 202'-6", 202'-6", 202'-6", 202'-6", 202'-6", 162'-6", and 162'-0" the total back to back length between abutments is 1665'-8".

The existing bridge has a 19-foot wide clear roadway width which provides for two 9'-6" wide travel lanes for two-way traffic. While the width is adequate for passenger vehicles, it is marginally adequate for trucks, recreational, and farm vehicles which sometimes require that other vehicles stop and wait until the wider and larger vehicle passes over the bridge. There are no pedestrian walkways on the bridge and pedestrians that use the bridge now share the roadway with vehicles on the bridge. When two vehicles are passing each other on the bridge, there is no space left for pedestrians. This presents an unsafe condition for users of the bridge. See Figures 4 and 5 on pages 4-5.

The bridge has a vertical clearance of 14 feet and some of the top chord bracing members have been damaged by vehicles with a height greater than 14 feet. The bridge is posted for 13'-6" maximum vertical clearance. The bridge is also posted for a maximum live load of 20 tons.

The bridge has the following deficiencies that need to be repaired:

1. Floods that occurred in 1993 caused the failure of the Gillespie Dam located upstream of the bridge. The failure of the Gillespie Dam directed major flows between Pier No. 1 (east abutment) and Pier No. 5 causing scour damage to Pier Nos. 2, 3, and 4.

2. The bridge bearings consist of nested roller bearings to provide for thermal expansion. The roller bearings are frozen, skewed, or misaligned and are heavily corroded which has reduced the movement capability.

3. The bridge coating system is in poor condition, paint deterioration and surface corrosion are prevalent and the coating system has failed.

4. The bridge overhead sway bracing struts, diagonal bracing, some of the main truss members and the bridge railing have been impacted by oversized vehicles and need to be repaired.

5. There are no traffic barriers on the bridge to resist vehicular impact. Existing pedestrian railing is substandard for pedestrians.

6. The concrete deck has deteriorated with numerous cracks and it is not connected to the floor beams.

The Structure and Inventory Appraisal for the bridge that was completed in 2004 gave the bridge a sufficiency rating of 47.20%.The bridge is also functionally obsolete. The bridge was evaluated in detail including a traffic study, environmental studies, in-depth inspection, inventory and operating ratings, geotechnical and parallel seismic testing, drainage and scour study, and structural analysis (see Figure 6 on page 6). The results of this evaluation will be incorporated into the DCR as well as concept studies for the future new bridge.

1.3 TRAFFIC & ACCIDENT DATA

1.3.1 Future Traffic Projections

The area along Old US-80 is primarily agricultural. With over 6,700 acres of developable land, there will be a large increase in traffic volumes in the future. There are currently several master planned communities proposed for the northeast quadrant of the Patterson Road intersection with Old US-80. With almost 9,000 proposed homes, these developments will have a significant impact on Old US-80 when completely built out.

Due to the uncertainty of the home construction it is difficult to predict exactly how many homes will be constructed along the corridor by 2025. The daily volumes along Old US-80 Highway near the Gila River could be as much as 8,500 daily vehicles with some development south of the Gila River and minimal development north of the Gila River. Much of the new development traffic will go to SR 85 and will not travel over the US-80 Gila River Bridge.

The existing roadway alignment and bridge over the Gila River can handle up to 8,000 vehicles per day, but probably cannot accommodate the projected traffic volumes in 2025 depending on where development occurs. A four-lane roadway cross section may need to be constructed, which will also require a new bridge crossing over the Gila River. Ultimately, a six-lane roadway cross section will be required to accommodate the build out volumes that are predicted to be over 35,000 vehicles daily.

Current ADT (2005)	200 vehicles per day
Projected ADT (2025)	Depends on development in area

1.3.2 Accident History

Accident history for Old US-80 was obtained from the Maricopa County Department of Transportation (MCDOT) for the five year timeframe of January 1997 to December 2001. A total of two accidents were reported, neither of which were within the study boundary. The accident history is shown in the table below for reference.

Reported Accidents – Year 1997 to 2001

Year	Accident Type				Remarks
	Angle	Rear-End	Other	Injuries	
1997	-	-	1	-	Collision w/ Guardrail
2001	-	-	1	2	Collision w/ Traffic Sign
5-Year Total	0	0	2	2	

1.4 PROJECT FUNDING

The Rehabilitation of the Historic Old US-80 Bridge at the Gila River (Recommended Alternative No. 2) will be funded by Maricopa County using local funds and Federal Highway Bridge Replacement & Rehabilitation funds.

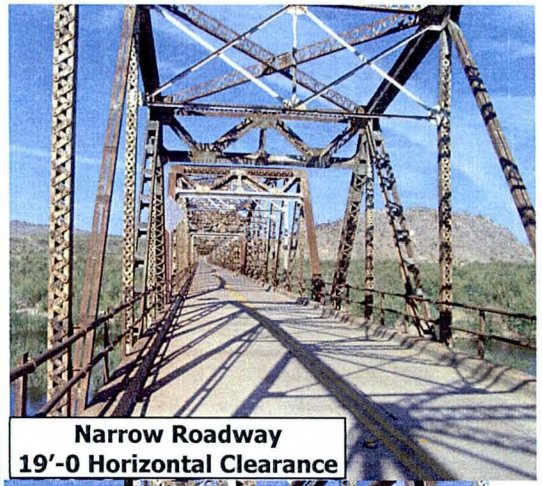
Federal Highway Bridge Replacement & Rehabilitation Funds	$ 1,000,000
Maricopa County Funds	$14,941,434
Total Project Cost	$15,941,434

FIGURE 4

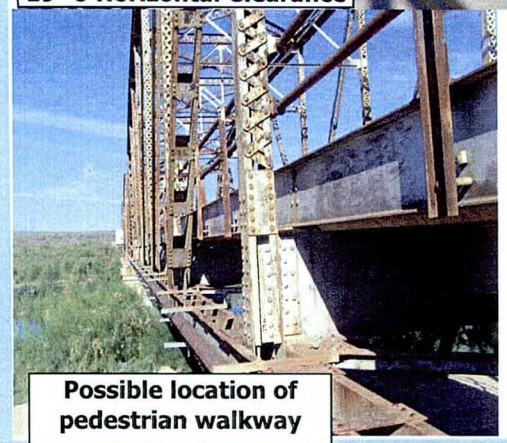
MCDOT RightRoads PROGRAM
The Right System The Right Time The Right Cost

Historic Old US 80 Bridge at Gila River
(Gillespie Dam Bridge)
Design Concept Report (DCR)

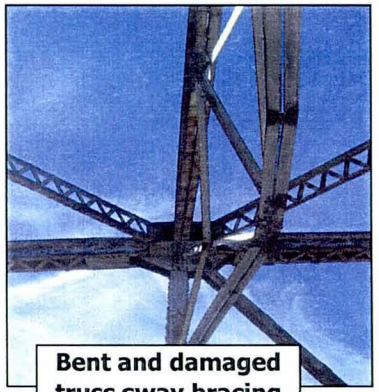
Bridge Rehabilitation Issues & Challenges



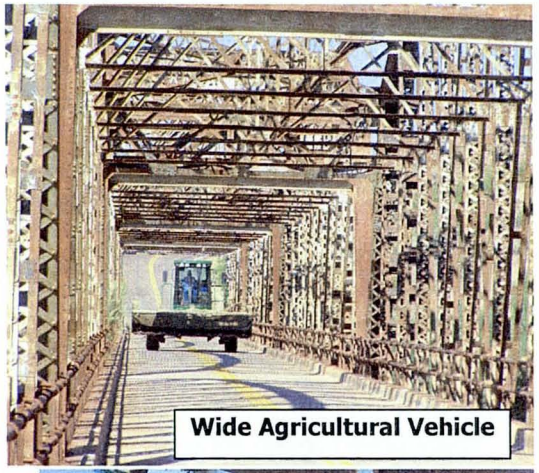
Narrow Roadway
19'-0 Horizontal Clearance



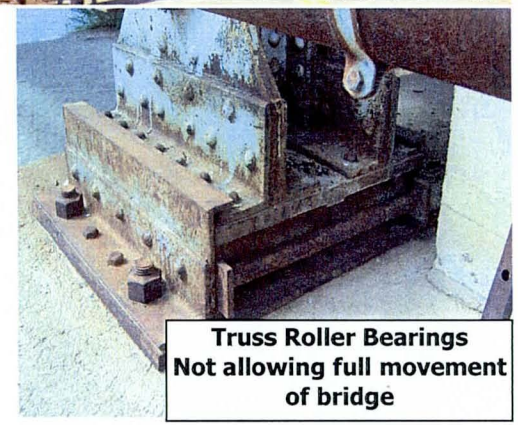
Possible location of pedestrian walkway



Bent and damaged truss sway bracing



Wide Agricultural Vehicle



Truss Roller Bearings
Not allowing full movement of bridge

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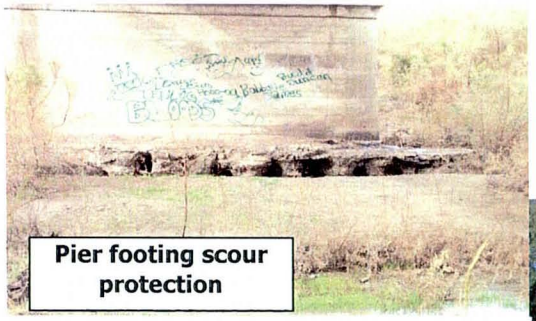
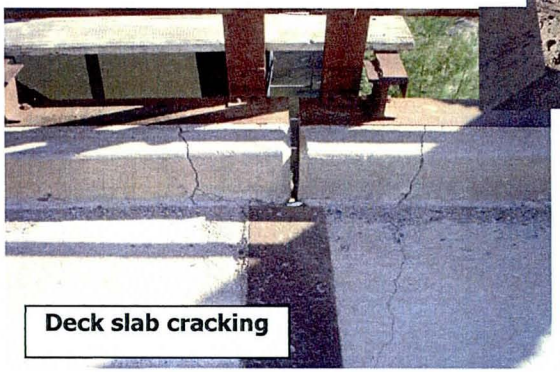
FIGURE 5

MCDOT RightRoads PROGRAM
The Right System The Right Time The Right Cost

Historic Old US 80 Bridge at Gila River
(Gillespie Dam Bridge)
Design Concept Report (DCR)

Bridge Rehabilitation Issues & Challenges





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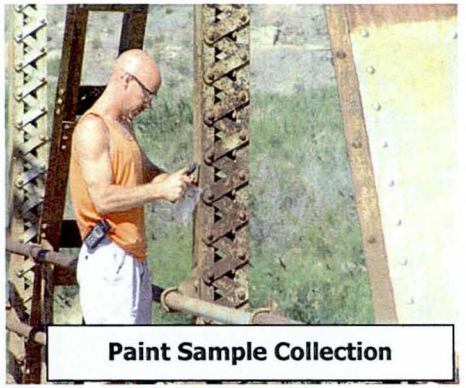
MCDOT RightRoads PROGRAM
The Right System The Right Time The Right Cost

Historic Old US 80 Bridge at Gila River
(Gillespie Dam Bridge)
Design Concept Report (DCR)

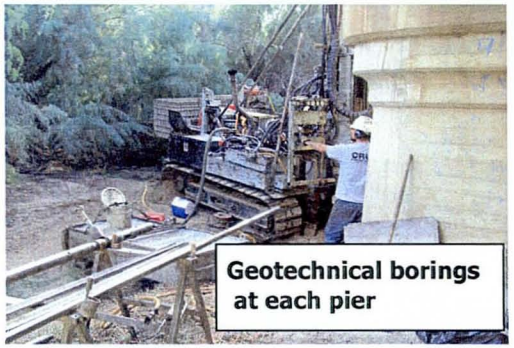
Bridge Rehabilitation Data Collection

On-site Traffic Survey

Bridge Site Inspection


Paint Sample Collection

Parallel seismic studies to verify foundation depths


Geotechnical borings at each pier

Geotechnical log boring samples

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FIGURE 6

2.1 BRIDGE INSPECTION & EVALUATION

A detailed inspection of all of the bridge members was performed by Burgess & Niple using adapted climbing and industrial rope access techniques to safely access all truss members. Piers 2 and 3 were accessed by Zodiak Kayak to probe for scour. Each stringer – floor beam connection was accessed using a 25-foot ladder from the ground or applying industrial rope techniques with a beam-roller from the floor beam bottom flange.

Bridge members were investigated for material deterioration, weathering and aging effects, load induced distress and indication of overloading, excessive deflection, inadequate bearing and collision damage. Steel members were inspected for signs of corrosion, loss of rivets or bolts, cracks at locations subject to fatigue and bent or damaged members.

The inspection is in accordance with the following documents revised to date:

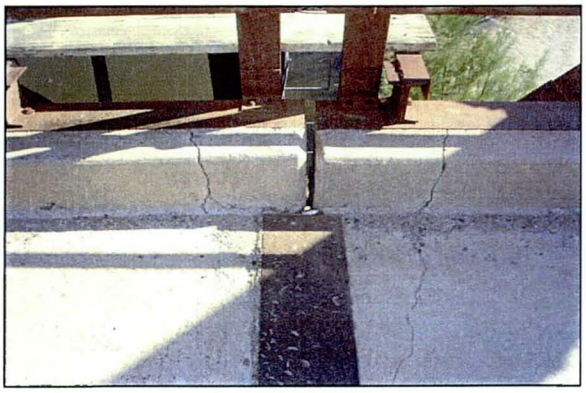
- *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFD) of Highway Bridges*, American Association of State Highway and Transportation Officials (AASHTO), October 2003
- *Manual for Condition Evaluation of Bridges*, American Association of State Highway and Transportation Officials (AASHTO), 2001
- *Bridge Inspector's Reference Manual (BIRM),* Federal Highway Administration, October 2002.
- *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges*, Federal Highway Administration, 1995
- *National Bridge Inspection Standards*, U.S. Department of Transportation, 1998

2.1.1 Deck

- Transverse cracking, up to 1/8-inch, occurs over every floorbeam throughout the bridge deck as well as other minor transverse cracking of the deck (See Photo 1). Cracking over the floorbeams is due to the negative moment developed in the deck over the floorbeams and the lack of reinforcing steel to resist this negative moment.
- Light scaling is present on the deck wearing surface (See Photo 2).
- Pack rust is developing between the floorbeam and the deck due to deck drainage passing through the cracks in the floor over the floorbeams, causing the deck to lift off the roadway stringer as much as 3/8-inch. This is causing diagonal cracks in the traffic curb which extend into the bridge deck wearing surface at each expansion joint (See Photo 3). The deck is lifting off the roadway stringer but is being held down by the curb stringer initiating cracking in the deck at the joints and into the curb.

Photograph No. 1 – Typical cracking in deck over floorbeams.

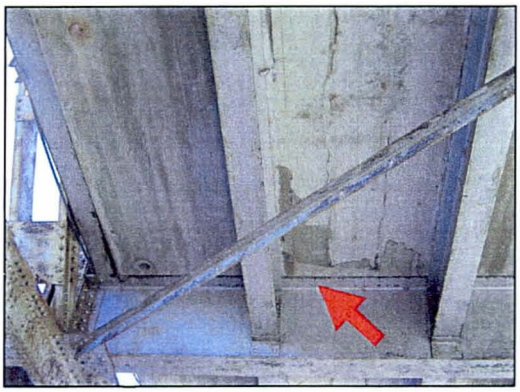
Photograph No. 2 – Typical scaling on deck wearing surface.



Photograph No. 3 – Typical cracking of deck and curb due to the lifting of the deck off the curb stringer.

The 'Deck Underside' of the bridge is in satisfactory condition. Spalls typically exist under the joint armor near the curb line (See Photo 4); otherwise, there are no signs of spalling, distress, delamination, or cracking throughout the rest of the deck underside. A typical barrier wrap used during fabrication between the forms and the concrete still adheres to most of the deck underside. This does not affect the load capacity of the deck (See Photo 5).

Photograph No. 4 – Looking up at the west face of Floorbeam 8 in Span 1 under the north curb. Typical spalling exists beneath the joint armor.



Photograph No. 5 – Looking up at the east face of Floorbeam 7 in Span 3 at the north curb. Note the typical barrier wrap adhering to the underside.

2.1.2 Bridge Barrier Railing

The condition of the bridge is fair. However, the 'Bridge Railings' do not meet current AASHTO code. This railing consists of dual steel pipes supported by the verticals and intermittent steel posts. The steel posts are riveted onto the steel curb stringer. The bridge railing has surface corrosion over 95 percent of its surface. There are many

locations of broken rails due to collision damage. This occurs over 10 percent of the entire rail length. There is debris adjacent to the curve that covers the deck drains and the barrier rail doesn't conform to current AASHTO code (See Photo 6).

2.1.3 Rail Transitions

The 'Rail Transitions' are in fair condition but do not meet current AASHTO code.

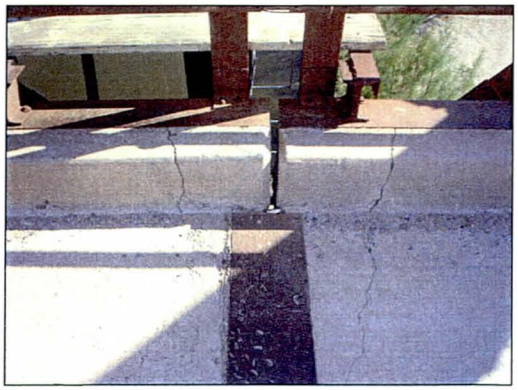
2.1.4 Deck Joint

The 'Deck Joints' on the bridge are in fair condition. The joints along with the bearings allow the structure to expand and contract due to temperamental changes. Both provide valuable information about each other. The sliding plate joints measurements were taken between the interior faces of the vertical joint armor plates (See Photo 7) at the upstream and downstream curbs.

Typical transverse cracks in the deck and diagonal cracks adjacent to the curbs at the joints reveal that the bearings are not allowing proper expansion and contraction (See Photos 7 and 8). Debris in the joint, surface corrosion on the armor and minor spalling along the joint are typical (See Photo 9). The joint openings at each abutment were covered with asphalt and measurements were not taken.

Photograph No. 6 Debris on the edge of the deck in Spans 8 and 9 covering drain holes. Barrier Railing does not meet current AASHTO code.

Photograph No. 7 – The top of the sliding plate joint above Pier 4 on the downstream side – the concrete deck and curb are cracked.



Photograph No. 8 – Looking down at the joint above Pier 3 on the downstream side. Note the cracks in the curb and deck side has the typical debris and the cracking of concrete.

Photograph No. 9 – Looking up at the same joint at the downstream side above Pier 3. Note the spalling of the concrete adjacent to the joint armor.

TABLE NO. 1 – HISTORICAL JOINT MOVEMENT

Location	Temp (°F)	UPSTREAM Opening (North)	Temp (°F)	UPSTREAM Opening (North)	Temp (°F)	UPSTREAM Opening (North)
Date Measurement Taken	*1999*	*1999*	*1999*	*1999*	*2006*	*2006*
Pier No. 2	62	2-⅜"	75	2-⅛"	70	2-¼"
Pier No. 3	62	1-⅝"	75	1-½"	70	1-¼"
Pier No. 4	62	2-½"	75	2-⅝"	70	2-¾'
Pier No. 5	62	1-¾"	75	1-½"	70	1-½'
Pier No. 6	62	2"	75	2"	70	2-⅛"
Pier No. 7	62	1-¾"	75	1-⅝"	70	1-¾"
Pier No. 8	62	2-⅛"	75	2-½"	70	2-¼"
Pier No. 9	62	2"	75	2-⅛"	70	2-¼"

Location	Temp (°F)	DOWNSTREAM Opening (South)	Temp (°F)	DOWNSTREAM Opening (South)	Temp (°F)	DOWNSTREAM Opening (South)
Date Measurement Taken	*1999*	*1999*	*1999*	*1999*	*2006*	*2006*
Pier No. 2	62	2-⅝"	75	2-¾"	70	2-⅜"
Pier No. 3	62	1-⅛"	75	1"	70	1-⅛"
Pier No. 4	62	2-½"	75	2-⅝"	70	2-¾"
Pier No. 5	62	1-½"	75	1-⅝"	70	1-⅞"
Pier No. 6	62	2"	75	2"	70	2-⅜"
Pier No. 7	62	1-¾"	75	2"	70	1-¼"
Pier No. 8	62	2"	75	2"	70	2"
Pier No. 9	62	2"	75	2"	70	2"

The table above shows historical joint openings measured in inches at three different temperatures taken at the upstream and downstream sides of the joints: the 1999 values are from the previous inspection report performed by Michael Baker, Inc.

The openings, under ideal conditions, should have the same measurements at the upstream and downstream curb for each joint. This condition only occurred at Pier 4. In addition, the openings would be greatest for the 62 degrees Fahrenheit (F) measurements, then a smaller opening for the 70 degrees F and finally a smaller opening for the 75 degrees F measurement. This condition only occurred at the upstream joint at Pier 2. Although evidence reveals that the joint openings are changing under different temperatures, they're not following typical calculations. This, for the most part, can be attributed to the roller bearings not functioning as designed and will be discussed later in the 'bearing devices' section (See Table 1 above).

Note the downstream joint openings above Piers 8 and 9 have not changed since 1999. These joints appear frozen and are not allowing proper expansion.

2.1.5 Drainage System

The 'Drainage System' on the bridge is in satisfactory condition. Heavy accumulation of debris has accumulated over time along the traffic curb in Spans 8 and 9 and is covering 90 percent of the drain holes (See Photo No. 9). The drain holes in Spans 1 through 7 are functioning as designed and only a few are covered or blocked.

2.1.6 Superstructure

The superstructure is in fair condition.

Main Members: The 'Main Members' are in fair condition. The inspection comments are divided into members: Verticals, Diagonals, Lower Chord, Upper Chord and Floorbeams and Stringers.

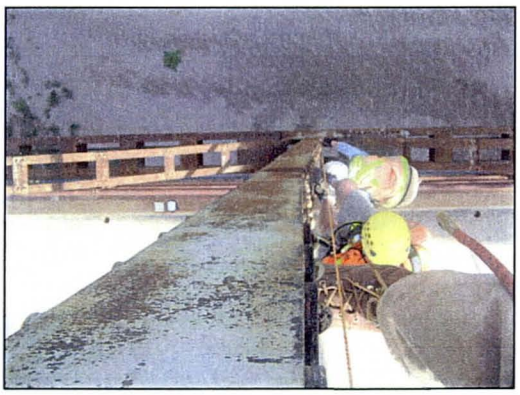
Verticals: The verticals are in generally good condition and consist of two channels with lacing bars. Surface corrosion and peeling paint are present throughout the structure. No appreciable section loss has occurred to the members. The vertical-to-floorbeam connections are in good condition with no significant deficiencies noted. Many of the overhead sway bracing struts have been impacted by oversize vehicles which have caused 12 verticals to be out of alignment (Table No. 2, and Photograph Nos. 10, and 11). Aside from the end verticals, (Panel Points 1 and 7 on the 160-foot spans, and Panel Points 1 and 9 on the 200-foot spans) these are compression members. The out of plane verticals will cause the load to be eccentric, and may cause bowing of the members (See Table No. 2 below). Photos 12 and 13 show the vertical truss member connection details to better illustrate the existing bridge design details for the bridge.

TABLE NO. 2 – VERTICALS AFFECTED BY IMPACT DAMAGE TO THE SWAY BRACING

Span	Truss	Member	Magnitude of Inches Bending	Direction
1	Downstream	L2U2	1"	North
1	Upstream	L2U2	1-1/4"	South
1	Upstream	L2U2	15/16"	West
1	Upstream	L3U3	3/8"	South
1	Upstream	L3U3	1/2"	West
1	Upstream	L4U4	7/16"	South
1	Upstream	L4U4	1/8"	West
1	Downstream	L3U3	3/8"	South
1	Downstream	L3U3	1/4"	West
1	Upstream	L5U5	1-1/16"	South
1	Upstream	L5U5	1/2"	West
1	Upstream	L6U6	1-9/16"	South
1	Upstream	L6U6	1-15/16"	West
3	Upstream	L3U3	3/4"	South
3	Upstream	L3U3	3/4"	West
3	Upstream	L4U4	3"	South
3	Upstream	L4U4	3-11/16"	West

TABLE NO. 2 – VERTICALS AFFECTED BY IMPACT DAMAGE TO THE SWAY BRACING CONT'D

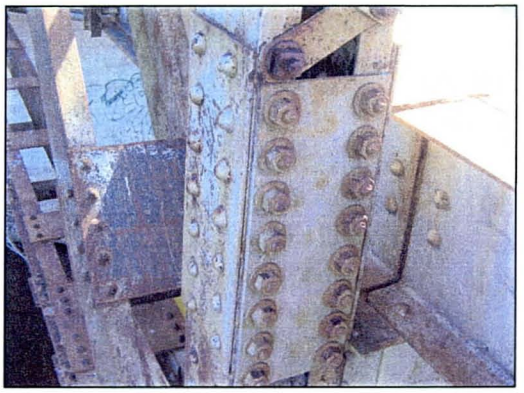
Span	Truss	Member	Magnitude of Inches Bending	Direction
3	Upstream	L5U5	1-1/8"	South
3	Upstream	L5U5	3/8"	West
3	Upstream	L6U6	1"	South
3	Upstream	L6U6	5/8"	West
6	Upstream	L7U7	3/8"	North
6	Upstream	L7U7	1/2"	West



Photograph No. 10 – Span 1, upstream member L6U6, note vertical member is bent out of plane. Inspectors use plumbing device to acquire out of plane measurements.

Photograph No. 11 –Span 1, upstream truss, member L2U2. Note: out of plane bending to vertical member due to collision damage to sway bracing.

Photograph No. 12 – Span 4, upstream truss, note the lower chord is too wide at lower chord connection causing outside gusset plate to bow outward.



Photograph No. 13 – Span 2, upstream truss, Panel Point 2, note plates bolted / riveted to bottom of vertical.

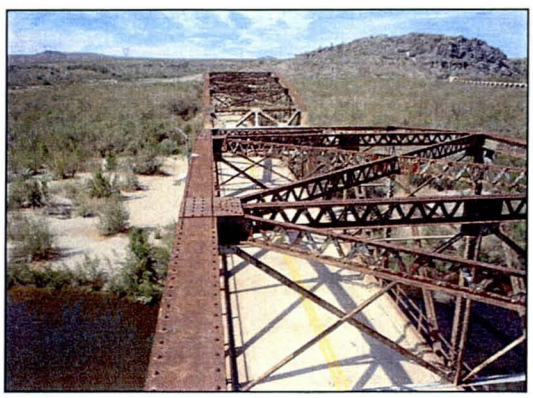
Diagonals: The diagonals are in generally good condition (See Photo No. 14) and consist of either built-up sections with two channels or two angles connected with batten plates. All diagonals on the structure carry tension. Surface corrosion and peeling paint are present throughout the structure. No appreciable section loss has occurred to the members. All diagonal member connections are in good condition with no significant deficiencies.

Photograph No. 14 – Typical diagonal member, note surface corrosion and paint failure. No other significant deficiencies noted.

Lower Chord: The lower chord is generally in good condition and consists of two channels back to back, connected by batten plates. Surface corrosion and peeling paint is present throughout the structure. No appreciable section loss has occurred to the members (See Photo No. 15). All lower chord connections are good with no significant deficiencies noted. As previously noted, the lower chord is wider than the vertical in three locations which likely occurred during construction (See Photo 12).

Photograph No. 15 – Typical lower chord member, note surface corrosion and paint failure. No other significant deficiencies noted.

Upper Chord: The upper chord is generally in good condition and consists of two channels with a solid plate on the top and batten plates on the bottom. Surface corrosion and peeling paint is present throughout the structure. No appreciable section loss has occurred to the members (See Photo No. 16). All upper chord connections are in good condition with no significant deficiencies noted.

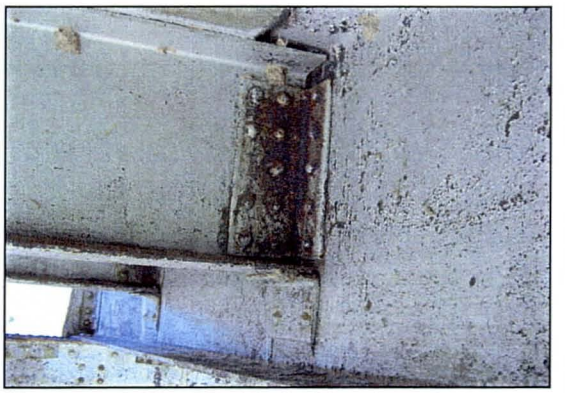


Photograph No. 16 – Typical upper chord, note surface corrosion and failure of paint. No other significant deficiencies noted.

Floorbeams and Stringers: The steel floorbeams consist of W28X106 wide flanged rolled shapes fabricated by the Bethlehem Steel Company. Cover plates and stiffeners were not utilized.

There are four roadway stringers and two exterior curb stringers in each span. The roadway stringers frame into each side of the floorbeam web with riveted clip angles and stringer seats. The curb stringers are riveted to the top face of the top flange of each floorbeam and are partially encased with concrete along the deck fascia. Curb stringers are attached to the next adjacent stringer by a 1 inch diameter tie rod through the deck. The two exterior stringers are W15X38.5 shapes and the four interior stringers are W18X49 and were all fabricated by the Bethlehem Steel Company (See Photo Nos. 17 and 18). The floor system is in good condition. Surface corrosion and paint deterioration are typical throughout with no appreciable section loss.

Photograph No. 17 – Looking up in Span 9 at the typical conditions. The exterior stringers are enclosed in concrete and the interior stringers are not enclosed in concrete.



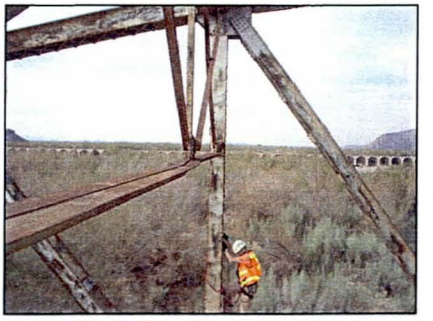
Photograph No. 18 – Typical interior stringer-floorbeam connection in Span 3 on the west face of floorbeam.

2.1.7 Secondary Members

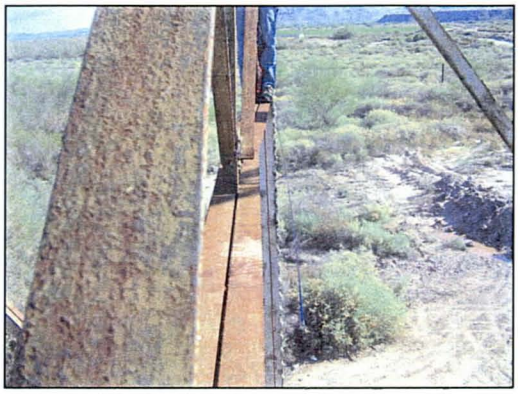
The 'Secondary Members' are in fair condition. The inspection comments are divided into members: Sway and Portal Bracing, and Upper and Lower Lateral Bracing.

Sway and Portal Bracing: Low vertical clearance and oversized vehicles have caused several of the overhead sway bracings to be bent out of plane due to collision damage (Photograph Nos. 19 through 23). New sway bracing members were installed and many of them have since been damaged due to collision. In many locations in Spans 1, 3 and 6, the collision to the sway bracing has caused the verticals to become out of plane (see verticals). The portal bracing has been modified at the connection to the end posts on every span to allow for a higher vertical clearance. The vertical clearance was raised 3 feet 4 inches in the 160-foot spans and 4 feet in the 200-foot spans (see Table 3 on page 15.)

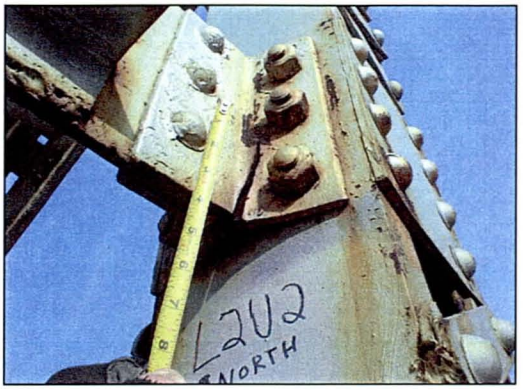
Photograph No. 19 - Span 1, Panel Point 2, note collision damage to sway bracing causing the lower strut and diagonal sway bracing members to be out of plane.



Photograph No. 20 – Span 4, Panel Point 8. Note: collision damage to sway bracing lower strut.



Photograph No. 21 – Span 9, Panel Point 5. Note: collision damage to sway bracing lower strut and inspectors using plumb device to acquire out of plane measurements.



Photograph No. 22 – Span 1, upstream truss, member L2U2. Note: crack and tear in sway bracing connection angle due to collision damage.

Photograph No. 23 – Span 1, upstream truss, member L5U5. Note: crack in sway bracing connection angle due to collision damage.

TABLE NO. 3 – SWAY BRACING DEFICIENCIES AND LOCATIONS

Span	Member	Location	Damage
1	Lower Portal Strut	East Portal Bracing	Lower flange bent 3" westward for 1' in length in two places (above eastbound lane and at center).
1	Upper Lateral Strut	Bracing for L2U2	Twisted at center with bottom flange bent 2" westward.
1	Lower Strut Connection to Vertical	L2U2 Upstream	Clip angle is cracked - 3 1/2" long, 3/16" wide at bottom.
1	Lower Lateral Strut	Bracing for L2U2	Heavy collision damage - bent 3' westward.
1	Lower Lateral Strut	Bracing for L3U3	New sway brace, no damage.
1	Top Chord Horizontal Bracing	U3 US to U2 DS	Heavy collision damage - bent 1' upward and 1'-6" westward with bottom angle broken free from center connection.
1	Lower Lateral Strut	Bracing for L4U4, L5U5, L6U6	New sway bracing bent West.

TABLE NO. 3 – SWAY BRACING DEFICIENCIES AND LOCATIONS CONT'D

Span	Member	Location	Damage
1	Lower Strut Connection to Vertical	L5U5 Upstream	Clip angle is cracked - 2 1/4" in length.
1	Lower Lateral Strut	West Portal Bracing	Impact damage to lower flange above eastbound lane 2' in length.
2	Lower Lateral Strut	Bracing for L2U2 and L3U3	Bent 1" westward, strut angles and cross-bracing members have been replaced.
2	Lower Lateral Strut	Bracing for L4U5 and L5U5	Bent 1" westward, strut angles have been replaced, other lateral brace members also bent up.
2	Lower Lateral Strut	Bracing for L6U6	Bottom strut has been replaced. No damage
2	Gusset Plate Connection	U7 Downstream	1/4" Pack rust between top strut and gusset plate.
2	Lower Lateral Strut	East and West Portal Bracing	Impact damage to lower flange, 1' in length.
3	Lower Lateral Strut	Bracing for L4U4	Bent 4" westward, other lateral brace members also bent up 1' westward.
3	Lower Lateral Strut	Bracing for L5U5	Strut angles and cross-bracing members have been replaced, no damage noted.
3	Lower Lateral Strut	Bracing for: L2U2, L3U3, L6U6, L7U7	Bent 1" westward, strut angles have been replaced, other lateral brace members also bent.
3	Lower Lateral Strut	Bracing for L8U8	Bent 2" westward, strut angles have been replaced, other lateral brace members also bent.
4	Lower Lateral Strut	Bracing for L3U3	Bent 2" westward.
4	Lower Lateral Strut	Bracing for L4U4	Bent 2-1/2" westward.
4	Lower Lateral Strut	Bracing for L2U2 and L5U5	Bent 3" westward.
4	Lower Lateral Strut	Bracing for L6U6	Bent 4" westward.
4	Lower Lateral Strut	Bracing for L7U7	Bent 4-1/2" westward.
4	Lower Lateral Strut	Bracing for L8U8	Bent 1" to 2" eastward above eastbound lane and 1-1/2" westward above westbound lane.

TABLE NO. 3 – SWAY BRACING DEFICIENCIES AND LOCATIONS CONT'D

Span	Member	Location	Damage
5	Lower Lateral Strut	Bracing for L5U5	Bent 2" westward and twisted.
5	Lower Lateral Strut	Bracing for: L4U4, L6U6, L7U7	Bent 3" westward and twisted.
5	Lower Lateral Strut	Bracing for L8U8	Bent 1" westward.
6	Lower Lateral Strut	Bracing for L2U2 and L3U3	Bent 2" westward
6	Lower Lateral Strut	Bracing for L5U5	Bent 2" eastward above eastbound lane and 3" westward above westbound lane.
6	Lower Lateral Strut	Bracing for: L4U4, L7U7	Bent 3" westward.
6	Lower Lateral Strut	Bracing for L6U6	Bent 2-1/2" eastward.
7	Lower Lateral Strut	Bracing for L6U6 and L8U8	Bent 1" eastward at center and kinked.
8	Lower Lateral Strut	Bracing for L3U3	Bent 1/2" westward at center and kinked
8	Lower Lateral Strut	Bracing for L5U5	Bent 1" eastward at center.
9	Lower Lateral Strut	Bracing for L5U5	Bent 10" eastward at center, other lateral brace members also bent, end connection angles to truss pried open, but not cracked.
9	Lower Lateral Strut	Bracing for: L3U3, L4U4, L6U6	Bent 4" eastward with ding at center.
9	Lower Lateral Strut	West Portal Bracing	Lower flange bent 2" westward for 1' in length above westbound lane.

Upper and Lower Lateral Bracing: The upper lateral bracing is in generally good condition with surface corrosion and peeling paint throughout. On Span 1, between Panel Points 2 and 3, the upper lateral bracing is damaged resulting in the bottom angle being bent 1 foot out of plane and fractured at the cross-buck connection (Photograph No. 24). The damage appears to be the result of over-height vehicular damage. The other upper lateral bracing is in good condition with no significant deficiencies noted.

The lower lateral bracing is in generally good condition with surface corrosion and peeling paint throughout. No significant deficiencies were noted.

Photograph No. 24 – Span 1, between Panel Points 1 and 2. Note: kink in upper lateral bracing due to previous collision damage.

2.1.8 Bearing Devices

The 'Bearing Devices' of the bridge are in poor condition.

Each span is independently supported with fixed, pinned-plate bearings on one end and steel roller bearings on the other. The east side of each simply-supported span is supported by the roller bearings (Photograph No. 25) and the west side of each span is supported by the fixed, pinned-plate bearings (Photograph No. 26). The roller bearings house a single line of four, five-inch diameter rollers that transfer vertical load between the sole plate into the masonry plates and allow longitudinal expansion and contraction (Figure 6).

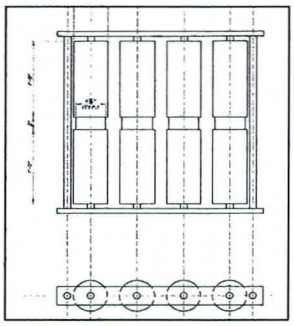
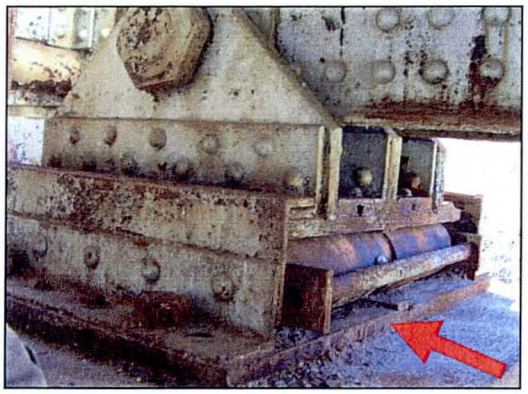
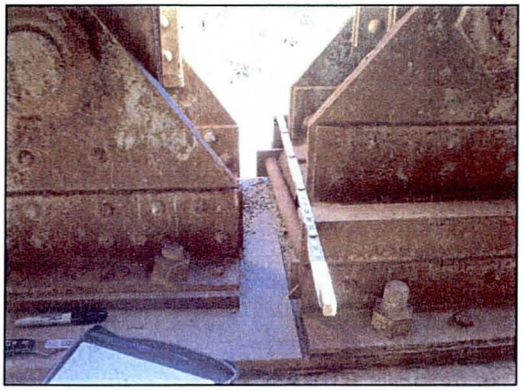


Figure 7 – Nested Roller Plan and Profile view

The centerline of the four rollers housed inside the bearing follows the centerline truss lines by tracking on an elevated guide-key along the centerline of the masonry plate (Photograph No. 27). A number of rollers have rotated in some manner inside the bearing housing unit off of the guide-key (Photograph No. 34); two have shifted laterally off of the guide-key (Photograph No. 28) and two appear 'frozen' (Table No. 4 on page 20). Surface corrosion and debris throughout the bearings are typical.



Photograph No. 25 – The downstream roller bearing at Pier 5 has rotated inside the steel housing.



Photograph No. 26 – The downstream bearing at Pier 8 is rotated.

Photograph No. 27 –The exterior roller has rotated inside the upstream bearing at Abutment 1.

Bearing notes compared with the joint notes are provided in the following table. The 1999 bearing values were taken at 85 degrees F by Michael Baker Inc., and the 2006 values were recorded at 70 degrees F. Measurements are recorded in Table No. 4 below.

The 2006 inspection found thirteen of eighteen bearings in expansion at 70 degrees F (Table No. 4). The bearings noted 'frozen' in the 1999 report have all contracted in 2006 and are showing signs of movement. However, the joint openings above the downstream side of Piers 8 and 9 have no signs of movement since 1999.

Seven of the eighteen roller bearings have rotated in some manner. There are a number of possible reasons of why these roller bearings would rotate, and although the evidence does not present a definitive answer, it is recommended that these rollers along with the two shifted rollers and the two frozen bearings be reset in order to function as designed. If left unchecked, there are bearings in danger of 'walking off' the masonry plate. Note the joint openings at the downstream Piers 8 and 9 have not moved, however the bearings have moved due to expansion and contraction.

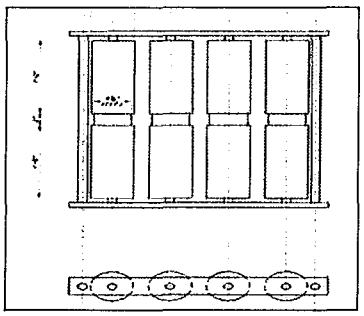
Two rollers have bent the vertical masonry plates due to the rotation: the upstream bearing at Abutment 1 (Photograph No. 28) and the upstream bearing at Pier 8.

TABLE NO. 4 – BRIDGE JOINT SUMMARY

Joint Location	Upstream 1999 Bearing Notes				Upstream 2006 Bearing Notes			
	(in)	Exp/Con	Temp (°F)	Comment	(in)	Exp/Con	Temp (°F)	Comment
Abut. No. 1	2-1/4	Expansion	85	Rotated	1-1/4	Expansion	70	Rotated
Pier No. 2	2-1/8	Expansion	85		1	Expansion	70	Shifted Laterally North
Pier No. 3	2-1/2	Expansion	85		1-1/2	Expansion	70	
Pier No. 4	1	Expansion	85		-	Contraction	70	
Pier No. 5	2-1/2	Expansion	85	Frozen	1-1/16	Expansion	70	Rotated
Pier No. 6	1-1/2	Expansion	85		1/4	Expansion	70	
Pier No. 7	2-1/2	Expansion	85		1-1/4	Expansion	70	Rotated
Pier No. 8	1-1/2	Expansion	85		1/16	Expansion	70	Rotated
Pier No. 9	0	Contraction	85	Frozen	1	Contraction	70	

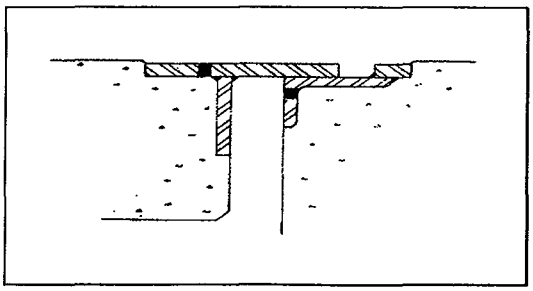
Joint Location	Downstream 1999 Bearing Notes				Downstream 2006 Bearing Notes			
	(in)	Exp/Con		Comment	(in)	Exp/Con		Comment
Abut. No. 1	2-1/8	Expansion			1-3/16	Expansion		
Pier No. 2	2	Expansion			3/8	Expansion		
Pier No. 3	3-1/2	Expansion			1-9/16	Expansion		Shifted Laterally North 3/16"
Pier No. 4	1-1/8	Expansion			1/8	Contraction		
Pier No. 5	1-5/8	Expansion		Rotated	1/4	Expansion		Rotated
Pier No. 6	1-1/8	Expansion			-	Contraction		
Pier No. 7	2-1/2	Expansion			9/16	Expansion		Rotated
Pier No. 8	7/8	Expansion		Rotated, Frozen	3/8	Expansion		Rotated
Pier No. 9	0	Contraction		Frozen	1-1/2	Contraction		

Roller Bearing:

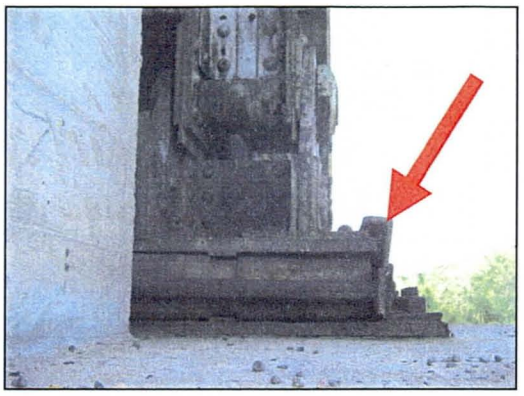


Roller Bearing plan view and profile view. Expansion and contraction measurements were taken from the centerline of the roller assembly to the centerline of the vertical masonry plate.

Sliding Plate Joint:



Sliding Plate Joint profile view. Measurements were taken between the interior faces of the vertical joint armor plates.



Photograph No. 28 – The upstream roller bearing at Pier 1 (abutment). Note: the north vertical masonry guide-plate bent outward due to the roller rotation.

Possible explanations for the roller rotation include:

1. The sun heats the truss lines at different temperatures when the sun is in a different location. For example, the south truss line (downstream) was hotter to the touch than the north truss line (upstream) during the March 2006 inspection. The northern truss was partially shaded by the southern truss as the sun was in the southern hemisphere. This difference in temperature will allow one side to expand at a faster rate and thus rotating the span and the bearings slightly. Note from Table No. 5 below the longitudinal difference in joint openings between the upstream and downstream ends of each joint. The highlighted rows are the joints above the bearings that have rotated (the seventh rotated bearing is at Pier 1 where we are unable to measure the joint opening due to the asphalt overlay). This theory, however, would only hold true if the downstream measurements were consistently smaller.

TABLE NO. 5 – JOINT MEASUREMENT DIFFERENTIALS

	2006 Joint Measurements		
	Upstream	**Downstream**	**Difference**
Pier No. 2	2 ¼"	2 3/8"	- 1/8"
Pier No. 3	1 ¼"	1 1/8'	1/8"
Pier No. 4	2 ¾	2 ¾"	0"
Pier No. 5	**1 ½"**	**1 7/8"**	**- 3/8"**
Pier No. 6	2 1/8"	2 3/8"	- ¼"
Pier No. 7	**1 ¾"**	**1 ¼"**	**½"**
Pier No. 8	**2 ¼"**	**2 "**	**¼"**
Pier No. 9	2 ¼"	2 "	¼"

2. Following the flood from the Gillespie Dam breach in 1993, the high waters and debris impacted the bridge and the force may have caused the bearings to rotate. In addition, the buoyancy force from the water may have lifted the bearings and as the water level dropped the bearings reset in the current rotated state. Although the bridge was overtopped, this buoyancy theory does not explain why the other bearings are properly aligned and function as designed.

3. The roller bearings could have been set wrong during fabrication and the rollers were misguided from the beginning.

4. Incompressible debris rested within the roller-housing and caused the rollers to move away from the centerline of the truss-line.

5. Extreme temperatures may have closed the joint opening creating a continuous span over the pier which would lift the sole plate from the bearings. When temperatures cooled the steel contracted and rested onto rotated rollers. This theory does not explain how the rollers rotated when the structure lifted from the bearings. Furthermore, the highest ambient temperature in Gila Bend, Arizona was 125 degrees F since the bridge was constructed. The lowest temperatures needed to close the joints over Piers 5, 7 and 8 are 158 degrees F, 150 degrees F and 222 degrees F, respectively. Although the temperature of the steel gets hotter than the ambient temperature, these temperatures have not been reached and seem unreasonable.

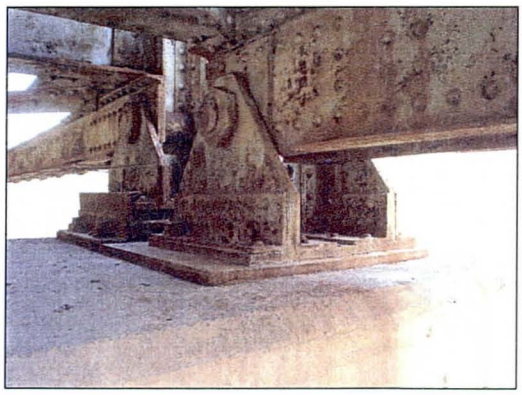
6. Moisture retained within the roller housing corroded the guide-key on the masonry plate. This allowed enough section loss to the key so that the rollers rolled over the guide key laterally instead of along it, longitudinally. This theory could not be verified because in order to inspect the guide-key the bearing assembly must be dismantled. If the bearings are reset the masonry guide will have to be inspected and possibly remedied if section loss exists.

In conclusion, the rotated, shifted and frozen bearings are not functioning as designed. Any one or a combination of the above theories would explain why the roller-bearings have rotated. Again, despite the reasons of how they were altered, it is recommended the upstream bearings at Piers 1, 2, 5, 7 and 8 and the downstream bearings at Piers 3, 5, 7, 8 and 9 be reset to allow proper distribution of loads from the superstructure to the substructure.

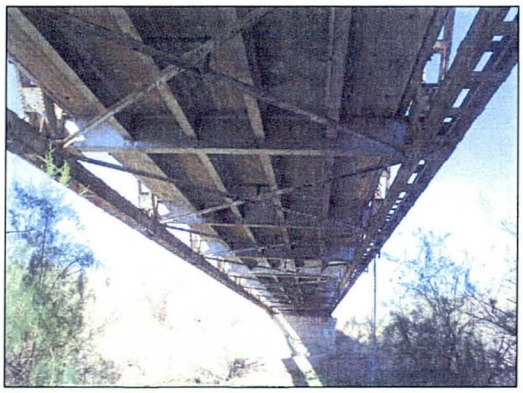
2.1.9 Paint System

The 'Paint System' of the bridge is in poor condition. Paint deterioration and surface corrosion is prevalent throughout the entire floor system steel with no apparent loss of section. The bearings have varying degrees of surface corrosion with little, if any, section loss. Pockets of laminating corrosion were found on the masonry plates of the fixed bearings at Pier 10 (abutment). Surface corrosion and peeling paint also occur above the deck on all of the steel truss and bracing members. No section loss is noted anywhere on the truss. The paint condition on the new sway bracing members is in good condition. In general, the paint system has failed and this is not surprising considering the age of the structure.

Paint deterioration and surface corrosion on the floor system covers roughly 20 percent of the steel with no loss of section (previous Photograph Nos. 25-27, and the following Photograph Nos. 29, 30). Insect and bird nests exist above more than half of the stringer clip angles in Spans 1, 2 and 3. The stringers and floorbeams have no visible signs of excessive deflection or misalignment.



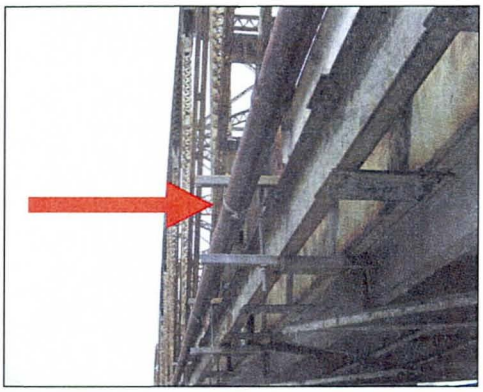
Photograph No. 29 – Typical surface corrosion on the bearings. This bearing is supporting Spans 7 and 8 at the downstream end of Pier 8.



Photograph No. 30 – General under view taken under Span 4.

2.1.10 Utilities

The 'Utilities' carried by the bridge are in satisfactory condition. There is a steel pipe sheath along the downstream lower chord supported by brackets spaced every nine feet (Photograph No. 31). A small number of brackets have separated from the pipe and the lower chord. In one location the pipe has separated, allowing the conduits inside to be exposed (Photograph No. 32). This utility has recently been repaired and appears to be in service.



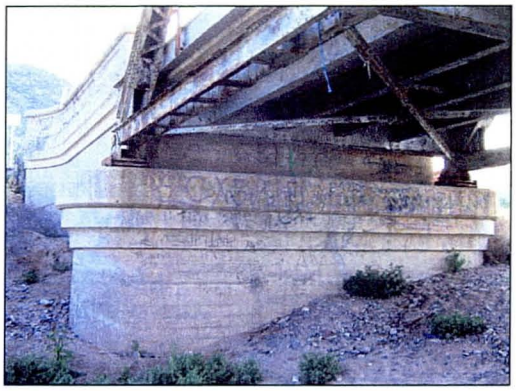
Photograph No. 31 – Utility sheath and support detail on the lower chord of Span 1.

Photograph No. 32 – Broken utility sheath on the downstream side of Abutment 1.

2.1.11 Substructure

The substructure is in satisfactory condition. The condition ratings are divided and rated into the following categories:

Abutment: The two 'Abutments' are in satisfactory condition and are functioning as designed. These substructure units are reinforced concrete wall-type abutments which are supported by spread footings that bear on bedrock. The walls are tapered at the top to form the pier cap (Photograph Nos. 33 and 34). Scour exists at the east Abutment most likely from the flood in 1993.



Photograph No. 33 – East abutment looking east, Pier 1 (east abutment). Note the scour hole on the upstream side.

Photograph No. 34 – West abutment looking west, Pier 10 (west abutment). Irrigation channel runs along the east face.

Piers: The 'Piers' are in satisfactory condition and are functioning as designed. Similar to the abutments, these substructure units are reinforced concrete wall-type piers which are supported by spread footings bearing on bedrock. The pier walls are tapered at the top to form the pier cap (Photograph No. 35). Scour holes exist on the upstream faces most likely from the flood in 1993.
Specific pier comments include:

- One vertical crack exists down the west face of the pier walls at the bridge centerline. The widest of these is on the west face of Pier 2 and it has been epoxy injected (Photograph No. 36). The epoxy is still bonding and the crack has not widened.

- Another, slightly smaller vertical crack on Pier 6 is 1/16-inch wide on the west face. These cracks are not significant, however their condition should be monitored in future inspections.

Photograph No. 35 – Looking south at the upstream face of Pier 8. This pier is in good condition, which is typical of all the piers.

Photograph No. 36 – Looking at the west face of Pier 3. Note: the vertical crack down the centerline that has been epoxy injected.

Wingwalls, Dados, etc.: The 'Wingwalls and Dados' are in satisfactory condition. One three-foot diameter spall was found on the upstream face of the north wingwall at Abutment 1 (Photograph No. 37). Small temperature and shrinkage surface cracks exist on the wingwalls and dados and do not affect their condition.

Photograph No. 37 – Three-foot diameter spall on Abutment 1 above the upstream bearing.

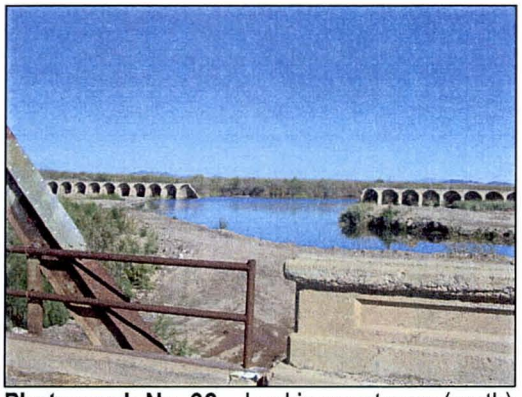
2.1.12 Waterway

The Waterway is in satisfactory condition.

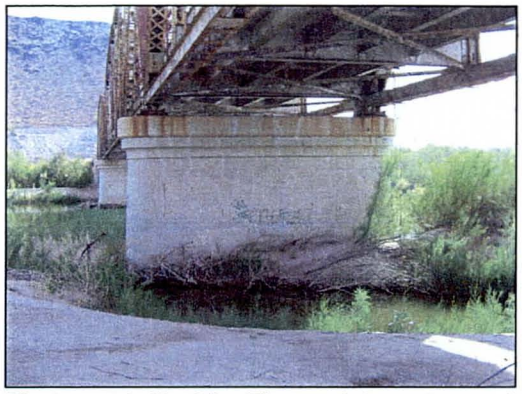
2.1.13 Channel

A flood in 1993 breached the Gillespie Dam upstream of the bridge (Photograph No. 38). The resulting flood altered the riverbed configuration under the bridge. However, subsequent floods, as well as dredging by the Canal Company have further altered the riverbed in the vicinity of the bridge since the 1993 Flood. Evidence of flooding exists in all spans, with deep scour holes on the upstream face of each pier and in isolated areas under Spans 3, 4, 5 and 6 (Photograph No. 39, 40, 41 and 42).

There are three bodies of water underneath the Gillespie Bridge. Spans 1, 2, 3, 4 and 9 have a waterway or a portion of a waterway underneath the structure. A pond exists under Span 2 and a portion of Spans 1 and 3, the Gila River runs under Span 4, and an irrigation channel runs under Span 9. Scour holes exist in the pond around Piers 2 and 3, and the irrigation channel is flowing against Abutment 10. The Gila River, however, is not meandering or affecting the substructure in Span 4 in any manner.



Photograph No. 38 – Looking upstream (north) from the Bridge at the breach in the Gillespie Dam.

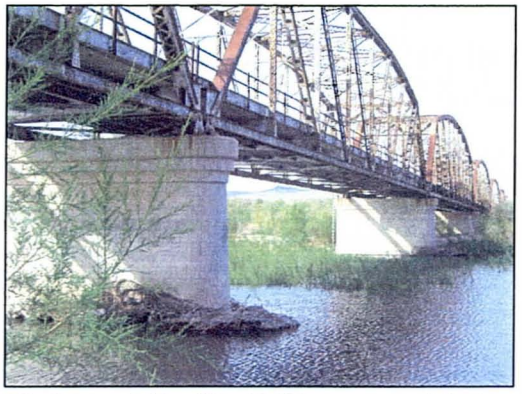


Photograph No. 39 – The pond downstream of the breached dam has created scour and deposited debris that surrounds the upstream portion of Pier 3.

Photograph No. 40 – Underneath Span 4, flooding and drainage from the deck has cut channels into the soil.

Photograph No. 41 – Looking east underneath Span 5, flooding and drainage from the deck has cut channels into the soil.

Photograph No. 42 - Looking east underneath Span 6, a six foot deep scour hole from high water.

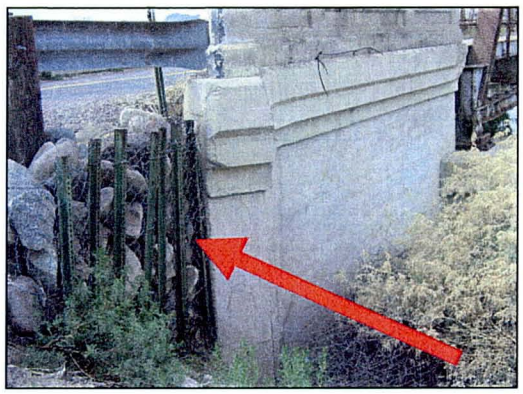


Photograph No. 43 – Drift has accumulated above the waterline on Piers 2 and 3 high water.

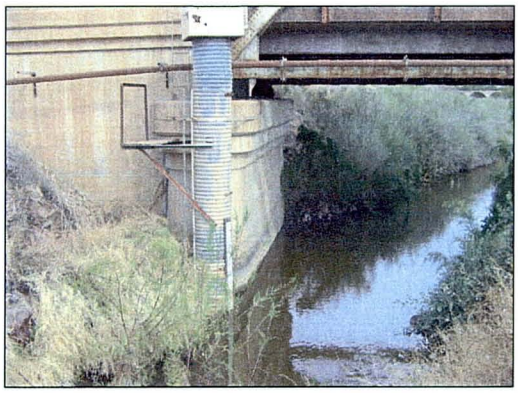
Photograph No. 44 –Elevation change between the upstream and downstream corners of Pier 5. The arrow is pointing to the upstream face.

2.1.14 Bank Protection

The slope at Abutment 1 is protected by a fence along the north curb of the approach. This is holding up well and functioning as designed (Photograph No. 45). The irrigation channel is undercutting the embankment on either side of Abutment 10 and will continue to do so unless arrested (Photograph No. 46). Pier 3 does not have adequate slope protection as well. The pond has circumnavigated the upstream nose of the pier, depositing debris and creating a large scour hole up to six feet deep. Since there is no flow along the pier, the scour hole should not increase and slope protection is not needed. However, monitor the scour depth in future inspections.



Photograph No. 45 – Fence gabion holding approach slope and guardrail post on the north curb of the east abutment.



Photograph No. 46 – Looking north along Abutment 10 at the irrigation channel flowing under Span 9.

2.1.15 Approaches

The 'Approaches' are in satisfactory condition. Debris is collecting and plants are growing along the curb-lines of the west approach. The asphalt overlay is elevated two-inches above the deck wearing surface on both approaches (Photograph Nos. 47 and 48). This height difference causes a rough transition for vehicular traffic.

Photograph No. 47– The west approach asphalt is two inches above the bridge deck.

Photograph No. 48 – The east approach asphalt is two inches above the bridge deck.

2.1.16 Approach Guard Rail

The 'Approach Guard Rail' does not meet current AASHTO code.

2.1.17 Guard Rail Ends

The 'Guard Rail Ends' do not meet current AASHTO code.

2.2 BRIDGE LOAD RATING & EVALUATION

As part of the bridge inspection and testing program, a load rating analysis was performed. The report presents the results of the structural analysis and load rating of the Old US-80 Bridge as a part of this Design Concept Report. This report is based on an in-depth inspection performed in March 2006. The rating results and recommendations reflect the existing structure condition, including bent members, as well as proposed modifications to the structure to enhance safety.

The load rating was based on the AASHTO Manual for Conditional Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, 2003, with 2005 Interim Revisions and the AASHTO LRFD Bridge Design Specifications, Third Edition, with 2005 and 2006 Interim Revisions. The approach for the load and resistance factor rating of bridges is comprised of three distinct procedures: 1) design load rating, 2) legal load rating, and 3) permit load rating.

The design load rating evaluates the performance of existing bridges utilizing the LRFD design loading and design standards. Bridges with a design load operating rating factor greater than one are considered to have adequate capacity for all AASHTO legal loads. Bridges with a design load operating rating factor less than one need to be rated for legal loads to determine the necessity of posting or strengthening the bridge. Permit load rating verifies the adequacy of the bridge in the review of permit applications for loads above the legal weights and was not addressed in this report.

The load rating analysis was based on the following conditions:

1. The as-built structure based on geometry and member sizes shown on existing drawings.

2. The existence of bending moments in the bent vertical members due to axial compression loading.

3. The compression flanges of some floor stringers were considered to be unbraced due to gaps existing between the top flange of the stringer and the underside of the concrete deck, which may have some relationship to the load carried by those stringers.

4. In addition to the existing structure, the bridge was analyzed considering added dead loads which would result from the proposed addition of new pedestrian walkways and traffic barriers.

The possibility of increased member stresses due to poorly functioning roller bearings was also proposed. Field measurements were taken to determine the movement of sole plates and the roller bearings of each truss for a temperature differential of about 35°F. Calculations showed that the trusses are not significantly restrained by the bearings and as a result, temperature effects are not included in the load rating of the bridge. However, field measurements also showed that at least five of the twenty roller bearings are severely distressed and that further deterioration could lead to more serious problems. Thus, it was recommended that the roller bearings be repaired or replaced.

Upon completion of the analysis, three types of truss members were found to have a design load operating rating factor less than one: bent verticals in the 160-foot truss, diagonals in the 160-foot truss which exhibited stress reversal from tension to compression under some configurations of live loading, and deck stringers which were regarded as having unbraced compression flanges due to the existence of gaps between the top flanges of the stringers and the underside of the deck. However, all but a single truss member were found to have legal load rating factors greater than one. The only truss member with a legal load rating factor less than one is the bent vertical L6U6 on the upstream side of span 1. If this truss member remains in its current condition, the loading rating analysis indicates that the bridge should remain posted for 40,000 pounds. However, if this member were to be straightened, there would not be a need to post the bridge as all primary load carrying members of the bridge would have adequate capacity for all three AASHTO legal truck loads.

2.2.1 Bent Vertical Members

Of the twelve bent vertical members noted in the Condition Evaluation Report, member L6U6 on the upstream side of Span 1 is the lowest rated of all members. In its bent condition, it does not have sufficient capacity to resist the compression loads caused by AASHTO legal trucks. If not repaired, this member will control the rating of the entire bridge and require posting. Therefore, as a minimum to avoid posting, we recommend that this member be straightened.

All other bent verticals have sufficient capacity to resist the compression loads caused by the legal trucks. Therefore, any recommendations to straighten these verticals are left to the judgment of the bridge Owner.

2.2.2 Other Truss Members

Except for the specific bent vertical member noted above, all primary load-carrying members of the bridge have adequate capacity for all three AASHTO legal trucks and do not need repairs or strengthening

2.2.3 Roller Bearings

Field measurements show that at least five of the twenty roller bearings are severely distressed. It is thought that sliding behavior in combination with rolling behavior may be occurring within many bearings rather than pure rolling behavior. Even though the bearings may not be performing as designed, measured movements suggest that the trusses are not significantly restrained by the bearings at this time, but further deterioration of bearing performance could lead to more serious problems. Therefore, we recommend that the roller bearings be repaired or replaced as part of the bridge rehabilitation.

2.2.3 Posting

TABLE NO. 6 – RATING FACTORS FOR THE EXISTING BRIDGE

Member Class		HL-93		AASHTO Legal Loads		
		Inventory	Operating	Type 3	Type 3S2	Type 3-3
160' Trusses	Top Chord	0.931	1.207	2.813	2.162	2.045
	Bottom Chord	0.937	1.215	2.822	2.152	2.040
	Straight Verticals	1.141	1.479	2.539	2.074	1.996
	Bent Verticals	0.452	0.585	1.040	0.821	0.790
	Diagonals	0.654	0.848	1.508	1.330	1.372
200' Trusses	Top Chord	0.997	1.293	3.310	2.487	2.325
	Bottom Chord	0.831	1.077	2.750	2.050	1.935
	Straight Verticals	1.210	1.569	2.516	2.219	2.107
	Bent Verticals	1.001	1.298	1.873	1.479	1.422
	Diagonals	1.003	1.300	2.021	1.659	1.634
Floor Members	Braced Stringers	0.937	1.218	1.546	1.694	1.873
	Unbraced Stringers	0.601	0.780	1.245	1.365	1.512
	Floor Beams	0.781	1.012	1.435	1.532	1.748

While several truss members have rating factors below 1.00 for the HL-93 design load, only bent vertical member L6U6 on the upstream side of Span 1 has a rating factor below 1.00 for AASHTO legal loads. Tables 7 and 8 below give a rating summary for this critical bent member. The enhanced bridge includes replacing the concrete deck and adding new traffic barriers and pedestrian walkways.

TABLE NO. 7 – RATING SUMMARY WITHOUT ENHANCEMENTS

Condition	Rating Level		Rating Factor	Safe Load Capacity (Tons)	Posting Load (Tons)
Existing	AASHTO Legal Loads	Type 3	1.040	26.0	Not needed
		Type 3S2	0.821	29.6	26.8
		Type 3-3	0.790	31.6	28.0
After Minimum Recommended Repairs	AASHTO Legal Loads	Type 3	1.245	31.1	Not needed
		Type 3S2	1.160	41.8	Not needed
		Type 3-3	1.116	44.6	Not needed

TABLE NO. 8 – RATING SUMMARY WITH ENHANCEMENTS

Condition	Rating Level		Rating Factor	Safe Load Capacity (Tons)	Posting Load (Tons)
Enhanced but Not Repaired	AASHTO Legal Loads	Type 3	0.907	22.8	21.7
		Type 3S2	0.717	25.8	21.4
		Type 3-3	0.690	27.6	22.3
Enhanced and After Minimum Recommended Repairs	AASHTO Legal Loads	Type 3	1.336	33.4	Not needed
		Type 3S2	1.055	38.0	Not needed
		Type 3-3	1.016	40.6	Not needed

2.3 TRAFFIC ANALYSIS & EVALUATION

The Old US-80 Bridge over the Gila River is scheduled for repair/rehabilitation. As part of the bridge evaluation a traffic analysis was performed to determine the approximate number of vehicles on Old US-80 near the bridge during the design year 2025. The purpose of the trip generation was to determine the appropriate future cross section of Old US-80 to be evaluated as part of the alternative selection.

2.3.1 Existing Traffic Data

Old US-80 is currently carrying approximately 200 daily vehicles evenly split between northbound and southbound. Approximately 20% of the vehicles are larger than passenger cars/pick up trucks. The majority of the traffic (75%) is using Old US-80 between 6 am and 6 pm. The segments of Old US-80 on either side of the bridge are predicted to operate at level of service A.

A driver survey was performed on Thursday, April 13, 2006. The following questions were asked:

- What is your basic trip destination area?
- Where did your trip basically begin?
- What is the purpose of your trip (business/work, school, appointment, recreation)?
- How many times a week/month do you drive this route?
- Are you familiar with alternative routes to reach your destination?
- If Old US-80 was temporarily closed to traffic, what alternative route would you use? How much extra travel would it be for you?

The study was able to determine the following information:

- 56% of the drivers use the route for work related trips.
- 90% of the drivers would use SR 85 as an alternative route (with approximately 15 extra miles of travel associated with using SR 85).
- 60% of the drivers drive the route every week.

Discussions with the Arlington Elementary School and Buckeye Union High School Districts indicated that approximately 15 students are picked up on the east side of the bridge and transported across the bridge. In addition, several other students are picked up at the bus stop at the intersection of Old US-80 and Enterprise Road (immediately west of the bridge). If the bridge was closed, neither of the school districts have the money nor the buses to provide a separate route to pick up students on both sides of the bridge.

A review of the 5-year accident history from January 1997 to December 2001 revealed that there were only two recorded accidents on Old US-80 within the study limits. Neither of the accidents was fatal and both involved vehicles striking fixed objects off the paved surface (guard rail and sign post).

2.3.2 Future Traffic Projections

The area along Old US-80 is primarily agricultural. With over 6,700 acres of developable land, there is the potential for a large increase in traffic volumes in the future. There are master planned communities proposed for the northeast quadrant of the Patterson Road intersection with Old US-80. With almost 9,000 proposed homes, these developments will have a significant traffic impact on Old US-80 when completely built out.

Due to the uncertainty of the home construction industry it is difficult to predict exactly how many homes will be constructed along the corridor by 2025. The daily volumes along Old US-80 near the Gila River could be as much as 8,500 daily vehicles with some development south of the Gila River and minimal development north of the Gila River.

The existing roadway alignment and bridge over the Gila River can not accommodate the projected traffic volumes in 2025 or 2015. A 4-lane roadway is the minimum roadway cross section that should be constructed, which will also require a new bridge crossing over the Gila River. Ultimately, a 6-lane roadway cross section will be required to accommodate the build-out traffic volumes that are predicted to be over 35,000 vehicles daily.

2.3.3 Impacts of Construction to Traffic

Any reductions to the number of lanes across the bridge will have an impact to the vehicles that use Old US-80. A temporary/portable traffic signal could be used to control traffic during non-work periods and supplemented with flaggers during construction periods. If any necessary bridge closures are required they should be coordinated with school districts to occur when school is not in session. Any restrictions will require advance signing on Old US-80. In addition, advance notice of any lane restrictions across the bridge should be provided to the area farmers.

2.3.4 One-Way Traffic Signal

The installation of a permanent traffic signal to control traffic if the bridge is converted to a one-lane bridge is possible. There is certain design criteria that will be required by the County's Traffic Engineering Department in order to implement the traffic signal.

2.3.5 Recommendations

The projected traffic volumes on Old US-80 associated with the proposed development south of the Gila River and the available developable land in the area will exceed the capacity of the existing roadway and bridge. Eventually Old US-80 should be constructed to a 6-lane facility to accommodate the projected future traffic volumes (which are over 35,000 daily vehicles).

The existing roadway is two lanes, which could typically accommodate up to 10,000 daily vehicles adequately. However, due to the narrow bridge crossing (approximately 19'-0" wide) and the sharp horizontal curves on either side of the bridge, the capacity of the bridge crossing is significantly reduced. In fact, if the existing bridge is not widened and the horizontal curves are not improved, the bridge will become a bottle neck causing significant delays to through traffic as the daily traffic volume on Old US-80 exceeds 8,000 daily vehicles.

A four-lane divided roadway can accommodate up to 35,000 daily vehicles before the capacity of the roadway is exceeded and significant traffic delays begin to occur. The improvements to Old US-80 should be phased to not only minimize initial construction costs, but to also allow for additional improvements to be constructed later with minimal impact to the traveling public. A recommended phasing schedule is provided as follows:

Initial Phase (see Figure 8 on page 33)-

- Rehabilitate the existing bridge.
- Maintain existing roadway geometry.
- Maintain traffic on existing Old US-80.
- Capacity of bridge crossing is approximately 8,000 daily vehicles.
- Begin design of new bridge crossing when daily traffic volume approaches/exceeds 5,000 daily vehicles.

Second Phase (See Figure 9 on page 34)-

- Construct new bridge over Gila River – bridge should be able to accommodate a future three-lane section.
- Construct two-lane roadway to connect to new bridge. Should connect to Old US-80 north and south of existing bridge crossing.
- Shift southbound Old US-80 traffic to new roadway and new bridge crossing.
- Convert existing Old US-80 Bridge crossing to northbound direction only.
- Capacity of bridge crossings over Gila River (new and existing) is approximately 25,000 vehicles.
- Begin design of new bridge crossing when daily traffic volume approaches/exceeds 20,000 daily vehicles.

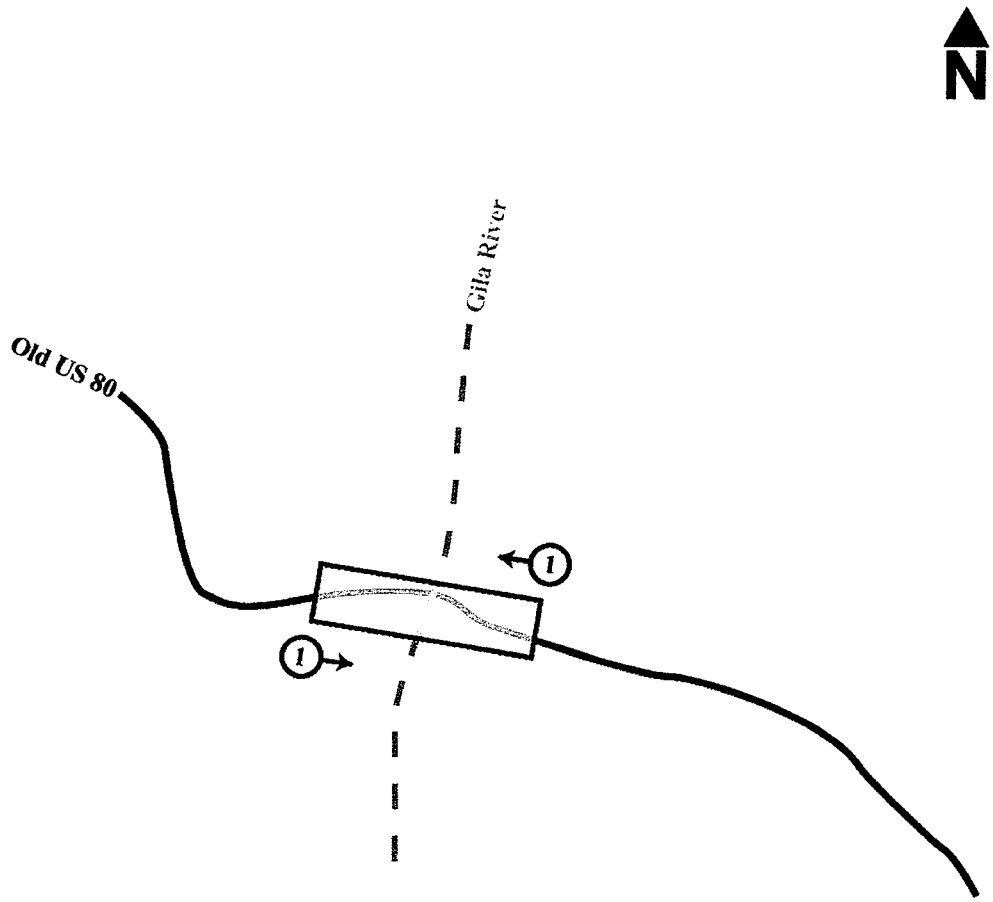
It should be noted, that in order for the daily volume on Old US-80 to approach 25,000 vehicles, the roadway will need to be improved to a four-lane section beyond the area of the Gila River Crossing (south towards Patterson Road and north towards the communities of Palo Verde, Arlington and Hassayampa).

Third Phase (See Figure 10 on page 35)-

- Construct new northbound bridge over Gila River and associated roadway.
- Shift northbound Old US-80 traffic to new roadway and bridge crossing.
- Convert existing Old US-80 Bridge to tourist destination (no vehicles).
- Capacity of new bridge crossings (four-lane divided roadway) is approximately 35,000 daily vehicles.
- Begin design of additional through lanes on Old US-80 when daily traffic volume approaches/exceeds 30,000 daily vehicles.

Final Phase (See Figure 11 on page 36)-

- Add additional through lanes to Old US-80 (three-through lanes in each direction).
- Modify striping across bridges to provide three-through lanes.
- Capacity of new six-lane roadway is approximately 60,000 daily vehicles.



LEGEND

(X) → **# of Travel Lanes**

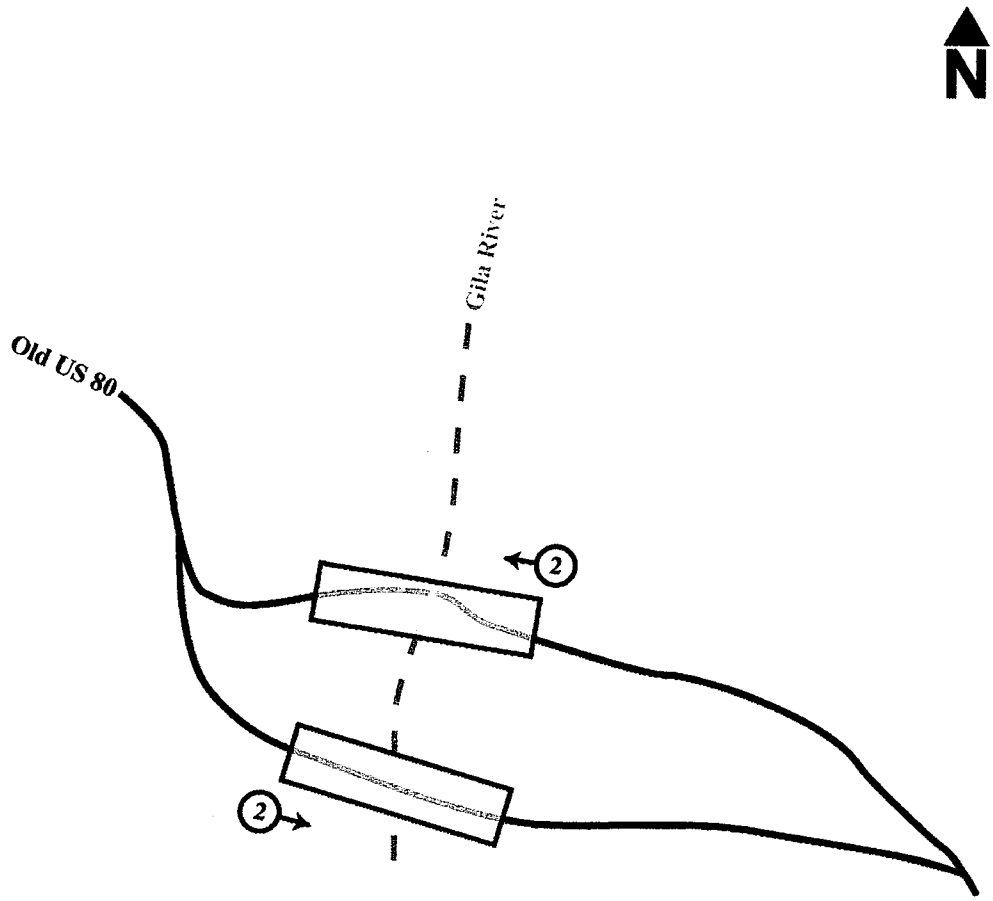
Initial Phase

* Rehab the existing bridge.
* Maintain existing roadway geometry.
* Maintain traffic on existing Old US 80.

CAPACITY: Up to 8,000 daily vehicles.

TRIGGER: 5,000 daily vehicles. Begin design of new bridge crossing.

FIGURE 8 – INITIAL PHASE IMPROVEMENTS



LEGEND

(X)→ **# of Travel Lanes**

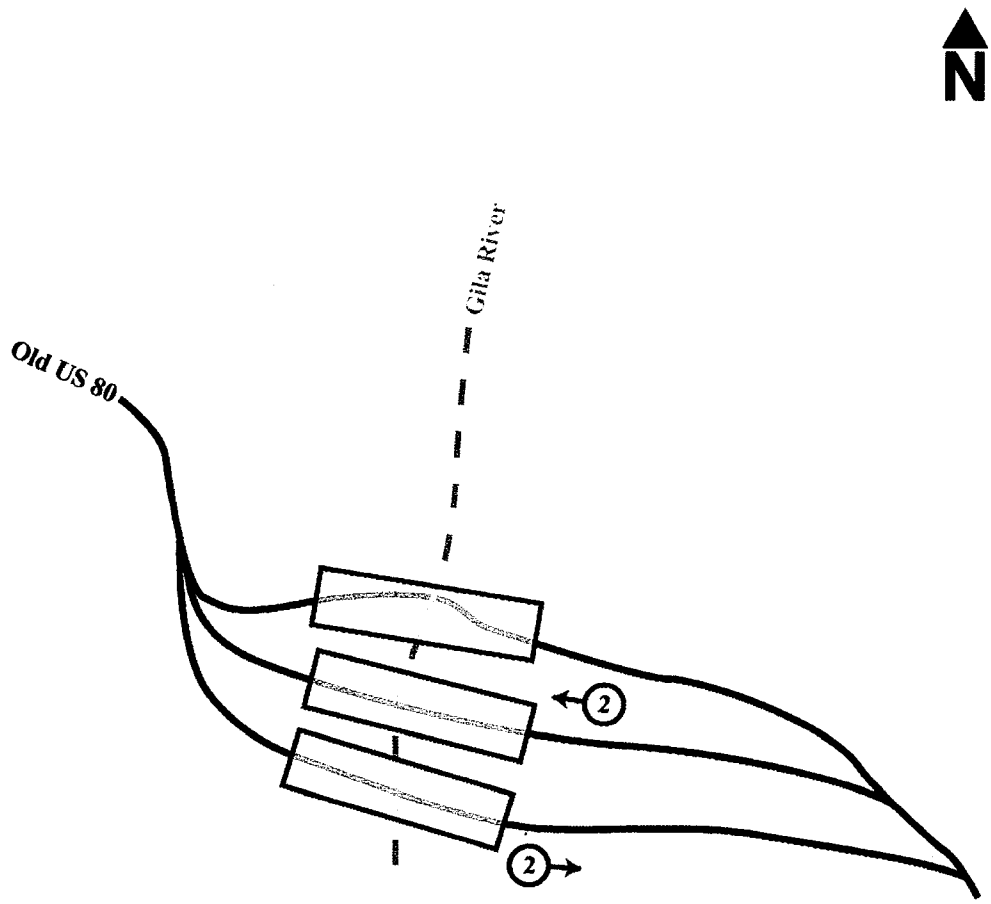
Second Phase

* Construct new southbound bridge over Gila River.
* Construct new southbound two-lane roadway to connect to new bridge.
* Shift Old US 80 traffic to new bridge crossing.
* Convert existing Old US 80 bridge crossing to northbound only.

CAPACITY: Up to 25,000 daily vehicles.

TRIGGER: 20,000 daily vehicles. Begin design of additional bridge crossing.

FIGURE 9 – SECOND PHASE IMPROVEMENTS



LEGEND

(X)→ **# of Travel Lanes**

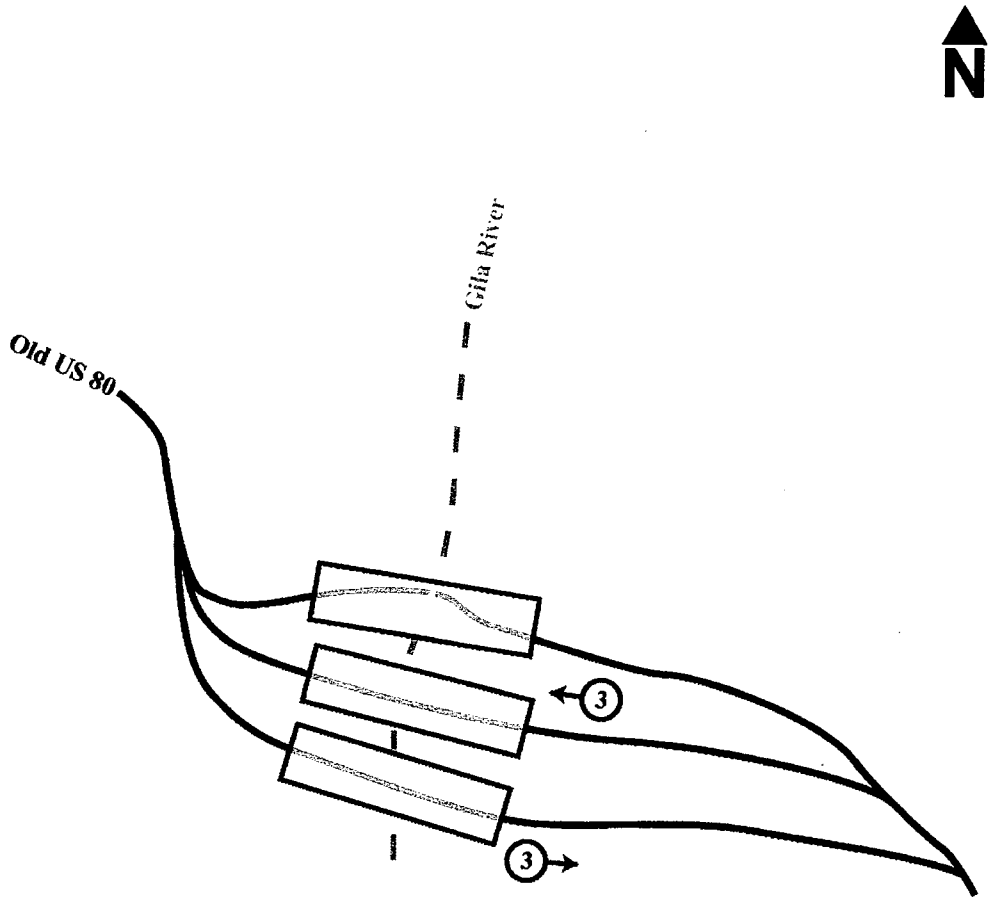
Third Phase

* Construct new northbound bridge over Gila River and associated roadway.
* Shift northbound Old US 80 traffic to new roadway and bridge crossing.
* Convert existing Old US 80 bridge to tourist attraction.

CAPACITY: Up to 35,000 daily vehicles.

TRIGGER: 30,000 daily vehicles. Begin design of additional third lane in each direction.

FIGURE 10 – THIRD PHASE IMPROVEMENTS



LEGEND

(X)→ **# of Travel Lanes**

Final Phase

* Construct additional through lane on Old US 80 in each direction.

CAPACITY: Up to 60,000 daily vehicles.

FIGURE 11 – FINAL PHASE IMPROVEMENTS

2.4 BRIDGE SUBSTRUCTURE EVALUATION

2.4.1 General

The existing Old US-80 Gila River Bridge is a nine-span steel truss bridge that spans the Gila River. The bridge is located about 500 feet downstream of the Gillespie Dam in Maricopa County. The foundations for the bridge are spread footings supported on igneous bed rock from about 15 feet to over 40 feet below the river bed. The spread footings are about 10 feet wide and 33 feet in length. The available plans for the bridge do not adequately show the depth of the foundations, and in 1993 during a major flood event some of the foundations were damaged due to flows in the Gila River which resulted in the failure of the Gillespie Dam. Repairs were made to Piers 2, 3, and 4 as a result of this flood event, but concern remains to the stability of the bridge during a major flood event. This work includes a detailed evaluation of the existing bridge foundation and the scourability of the spread footings.

2.4.2 Existing FEMA Study

In 1990, Cella Barr Associates performed the Gila River Flood Insurance Study for the Flood Control District of Maricopa County under contract FCD No 88-02. The report is available at the District library, A650.015.010. Included in the report are the hydrology and water surface elevations. In the study the following discharges are given at CP 1218 – Gillespie Dam:

HYDROLOGY

Return Period	Discharge, cfs
5-yr	37,000
10-yr	78,000
20-yr	124,000
50-yr	186,000
100-yr	235,000
200-yr	277,000
500-yr	335,000

2.4.3 FEMA Hydraulics

The study determined both floodplain and floodway elevations for the reach of the Gila River from Painted Rock Dam at stream mile 126.0 up to Gillespie Dam at stream mile 166.58. Cross-sections of interest are HB, HC, HD, HE, and HF. Cross-section HB and HC are near the alignment of the future new bridge alignment downstream crossing of the Gila River (Alternative 4). Cross-sections HE and HF are at the existing bridge. Cross-section HF is at the upstream face of the existing bridge and cross-section HE is at the downstream face of the existing bridge.

TABLE NO. 9 – GILA RIVER CHANNEL CROSS-SECTION FLOODWAY DATA

Cross-Section	Location	Distance from Bridge	Floodway Width	Velocity	Water Surface Elevation	
					w/o Floodway	Floodway
HA	166.06	1900	2427	8.7	742.3	742.6
HB	166.15	1426	2225	8.4	744.0	744.2
HC	166.25	898	2060	8.4	745.9	746.2
HD	166.34	422	1880	8.4	747.6	747.9
HE	166.41	52	1753	8.3	749.0	749.4
HF	166.42	0	1608	8.0	750.1	750.5

NOTE: Elevations listed here are on NVGD 29.

2.4.4 Bridge Hydraulics – Existing Bridge

The existing Old US80 highway bridge is a 9-span truss bridge that carries two lanes of vehicular traffic across the Gila River. The bridge is situated approximate 700 feet downstream of Gillespie Dam in Maricopa County. The bridge was designed in 1925 by R.V. Leeson, Consulting Engineer. It was constructed in 1926-1927. It is approximately 1661.5 feet long, with the substructure consists of ten units; eight piers and two abutments. The substructures are supported on spread footings supported on rock or "caliche" from 25 feet to 42 feet below the riverbed. The footings are about 10 feet to 14 feet wide and 32 feet to 36 feet in length. In the bridge plans and in this report the all the foundation elements are called piers with the numbering beginning at the east abutment, Pier 1 through the west abutment (Pier 10).

In January 1993, a major flood occurred that resulted in the over topping and subsequent failure of the Gillespie Dam. The collapse of a portion of the dam concentrated flows on the easternmost portion of the bridge. This in turn caused some scouring of the east bridge abutment (Pier No. 1) and Piers Nos. 2 through 6. A repair project was undertaken for piers 2, 3, and 4 (east end of the bridge) in late 1995. The repair project was designed by DMJM Consulting Engineers. The repair consisted of jet grouting a curtain wall on the upstream side of the footings. That repair addressed cracks in some piers and provided a curtain wall around the upstream and sides of several piers.

SH&B, now AMEC, performed borings in 1993. The borings do indicate the presence of a basalt and agglomerate rock bed. The geotechnical report states that the bottom of footing elevations for piers 2, 3, and 4 are as follows:

Pier No.	Bottom Footing Elevation
2	711.75
3	712.30
4	708.50

Below are excerpts from the report, "The existing channel grade varies from elevation 731 to 735." "The soil encountered consisted of sand, silty sand, and sand gravel mixture." "Underlying the looser granular deposits is basalt beneath Abutment No.1, Pier No. 2, and the south side of Pier No. 3. agglomerate beneath the north (upstream) side of Pier No. 3 and moderately to strongly cemented clayey sand and sandy clay beneath Pier No. 4." "The rock (basalt and agglomerate) is hard to moderately hard, and the cemented soils are very firm to hard. These materials do not appear to have been scoured."

2.4.5 Parsons Brinckerhoff Bridge Scour Evaluation

In 1995 through 1997, Parsons Brinckerhoff performed a Bridge Scour Evaluation of the bridge. The study followed FHWA and generally accepted procedures for evaluating bridge foundation regarding scour. The study concluded that the bridge capacity flow is 210,000 cfs at a distance of 3 feet below the top of the roadway. This flow is less than the 100-year flood discharge of 235,000 cfs as determined by the Flood Control District of Maricopa County. In 2007, the revised 100-year discharge is 195,000 cfs. Due to the repairs at piers 2, 3, and 4, it was concluded that the bridge is not "scour critical" and therefore no recommendations for scour countermeasures were provided.

2.4.6 Primatech/Jacobs Bridge Scour Evaluation

It was determined that the bridge is susceptible to scour and should be considered scour critical until repairs are completed.

In March 1996, the US Army Corps of Engineers issued the Section 7 Study for Modified Roosevelt Dam, Arizona that lists revised values for discharges at multiple locations and frequencies. In that document the following discharges are listed for the with project condition:

Frequency	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr
Discharge, cfs	12,000	38,000	65,000	145,000	195,000	215,000	270,000

Long-term Degradation: The existing bridge has been in place about 80 years. Downstream at the pipeline crossing measurements of degradation have been made over the years. Those measurements show some degradation followed by some refilling, but more important, there has been a stable bed for a long time. Therefore we believe that long-term degradation has occurred and no estimate is included in the prediction of the future scour at the existing bridge.

General or Contraction Scour: The bed lowering during an event may be estimated by either General Scour or Contraction Scour. At the existing bridge, the bridge opening is about the same size as the dam opening upstream resulting in no contraction Scour. The general scour was evaluated using methodology by Abbott or Lacey. The general scour including a 1.3 safety factor is 9.2 feet.

Pier Scour: The pier scour for the existing bridge was evaluated using a minimum skew angle and rectangular piers with consideration of debris or round nose with no debris. The no debris case results in the deeper scour. This is 20.9 feet.

Summary: The results of this scour study of the existing bridge determined that the predicted scour elevation for the piers is Elevation 695.9.

2.4.7 Scour for flows of lesser events

The scour from flows of events less than the 100-year can cause damage to the bridge foundations. For the piers at the east end of the bridge, not including Pier 1 (east abutment), Pier 4 is at elevation 708.5 which is the lowest bottom of pier footing and Pier 2 is at elevation 711.0 which is the highest bottom of pier footing. This is 5.3 feet to 9.8 feet higher than the predicted 100-year scour elevation of 695.9. Tabled below are the scour elevations for the 20- and 50-year storm events. As can be seen from the table even for a 20-year flow of 65,000 cfs, the predicted scour elevation is 706.9, at least 1.6 feet lower than the footing of Pier 4.

Frequency, Yr	Discharge, cfs	Elevation, ft
20	65,000	706.9
50	145,000	705.2

2.4.8 Impact of existing breach at Gillespie Dam

If the flow is concentrated through the breach, a very rough approximation can be made by assuming that it is flow through a weir. For a width of 255 feet and a depth of 10 feet, about half the height of the dam, with a weir coefficient of 2.63 the discharge through the weir would be about 21,200 cfs. As the bridge is about 700 feet downstream of the dam, the flow would spread out and decrease in velocity. The flow would spread out at about a 1:3 ratio, therefore the flow width downstream is about 720 feet. Assuming a slope of 0.00283 and a Manning's n of 0.024, the resultant depth is 4.23 feet with a velocity of 8.53 fps. Then Manning's n of 0.024 is very low, but gives us the highest velocity. The resulting scour is 14.4 feet for a pier with debris. Assuming 3.0 feet of bed loss, this would result in a bottom of scour elevation of 710.6. For flows much greater than the breach weir flow listed above the existing breach at Gillespie Dam will have a minor effect on the predicted pier scour. The 100-year discharge will overtop the dam. At the bridge, the water surface will hit the superstructure, thereby forcing a redistribution of flow throughout the length of the bridge opening.

2.4.9 2006 Geotechnical Investigation

In November 2006, 10 borings were taken at the bridge, one at each pier. Eight of the borings were taken adjacent to the pier wall and penetrated to and through the footings. This allowed for determination of the elevation of the footings and the material below the footings.

The results of the geotechnical investigation revealed that the pier footings bear on about 2'-0" above igneous bed rock consisting of basalt or agglomerate. Pier No. 1 footing bears on sand and gravel. It is apparent that during the construction evaluation of the founding material was made, and if it was not up to the desired quality, the foundation was deepened. The geotechnical investigation supports why the bridge has not failed, it also does not alleviate concern for the piers that were affected in the 1993 flow event. We would still consider the bridge to be "scour critical" until repairs are made to fully protect the footings and ensure that the flow would not scour out the foundation material.

2.4.10 Foundation Analysis

During the field exploration, test borings were located as close as practical to the side of the existing bridge piers in order to install the PVC casing for use in the parallel seismic testing that would be used to determine the depth of the spread footings. As a result of the test borings and subsequent coring operations we were able to encounter the depth of the concrete foundation for eight of the ten bridge piers.

The test borings were extended below the bottom of the footings to verify the adequacy of the soil material to support vertical loads as well as to determine the scourability of the foundation material. The existing foundations are bearing on a soil that is capable of supporting allowable bearing pressure of 20,000 pounds per square foot. The analysis of the bearing capacity of the existing foundations was determined in accordance with the 2002 Edition of the AASHTO Specifications for Highway Bridges. Using a maximum pier load of 2000 kips, the foundation pressure is about 6500 pounds per square foot less than the allowable soil bearing pressure.

2.4.11 Elevation Bottom of Spread Footings

The elevation of the bottom of the spread footings was obtained by three methods – (1) using the 1993 repairs plans prepared by DMJM Consultants and 1997 PBQD Study, (2) log borings performed by Terracon, and (3) by parallel seismic testing performed by Olson Engineering. Terracon drilled one log boring at each of the ten piers and was able to core drill a hole through eight of the ten pier footings. This was supplemented and checked using parallel seismic testing with an accuracy of plus or minus two feet. Table No. 10 on the following page summarizes the bottom of the footing elevations for the bridge. Figure 12 on page 42 provides a graphic description of the bottom of the footing and scour data.

TABLE NO. 10 – BOTTOM OF FOOTING ELEVATIONS

Location	As-built Plans/Repair Elevations	Based of Terracon Test Borings Depth of Footing	Based on Olson Parallel Seismic Results Depth of Footing	Bottom of Footing Elevations Used in Evaluation of Foundations
Pier 1 (east abutment)	716.75 (DMJM Plans)	Unable to Verify	714.45	716.75
Pier 2	711.75 (DMJM Plans)	711.0	712.2	711.0
Pier 3	712.30 (DMJM Plans)	713.0	715.35	713.0
Pier 4	±708.5(DMJM Plans)	Unable to Verify	704.30	708.5
Pier 5	±712	699.0	697.72	699.0
Pier 6	±712	694.0	None	694.0
Pier 7	±712	687.0	None	687.0
Pier 8	±712	699.0	None	699.0
Pier 9	±712	713.0	None	713
Pier 10 (west abutment)	±720	718.5	None	719.5

Datum used based on bench mark Q13 need elevation (1929 survey datum) located on the north side of Pier No. 1 (east abutment) and benchmark P13 need (1929 survey datum) located on the north side of Pier No. 10 (west abutment).

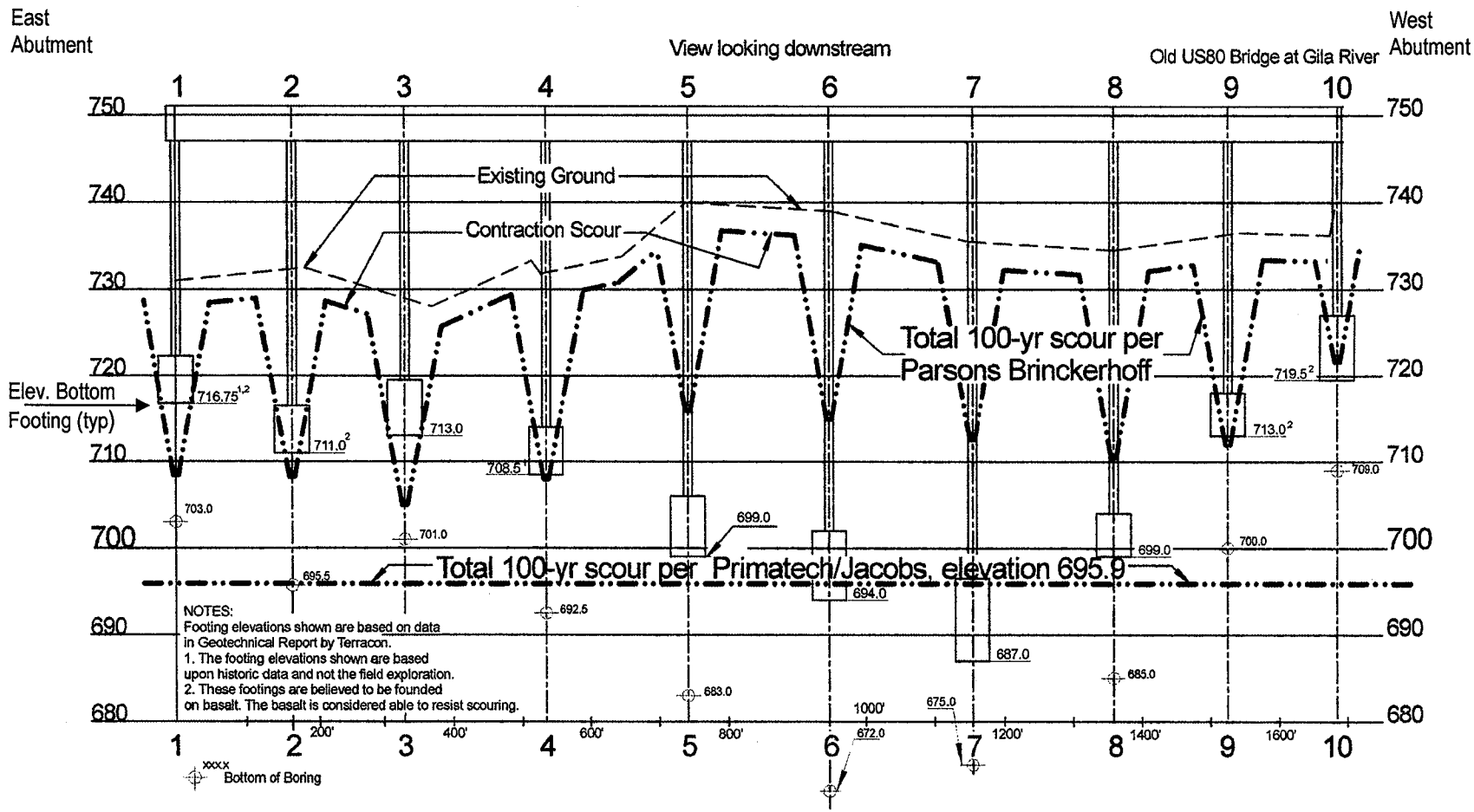
2.4.12 Scourability of Spread Footings

The predicted 100-year scour elevation is 695.9 feet. Piers 6 and 7 bottom of footing elevations are all deeper than the predicted scour elevation and are not susceptible to scour. Piers 1, 2, 3, 4, 5, 8, 9, and 10 bottom of footing elevations are above the predicted scour elevation of 695.9 feet and for these piers the stability of the bridge is dependent on the scour resistance of the material underneath the spread footings.

The Federal Highway Administration Publication "Evaluating Scour at Bridges" (FHWA Publication NH1 01-001) dated 2001 provides well documented equations and methods that are employed to evaluate the scour potential of either cohesive or cohensionless soils. That publication further references the FHWA 1991 Memorandum "Scourability of Rock Formations" that provides direct and empirical methods to determine if rock are resistant to scour. The empirical methods of scour resistance evaluation for rock include:

- Evaluation of geological formation and rock discontinuities
- Evaluation of the Rock Quality Designation (RQD)
- Unconfined compressive strength of the rock
- The Slake Durability Index
- Soundness of rock in accordance with AASHTO T104
- Abrasion of rock in accordance with AASHTO T96

FIGURE 12 – SECTION AT BRIDGE CONSTRUCTION CENTER LINE
(Summary of bottom footing elevations and scour depth)



Of these empirical indicators the first five have been used as initial indicators of scour resistance of bed rock at the site. Compression test results of intact pieces of rock core ranged from 950 psi to 8560 psi. Based on the FHWA Memorandum, rock with an unconfined compressive strength less than 250 psi should be considered to behave as soil. Based on compressive strength, the bed rock would be considered scour prone. However, the Memorandum also indicates that rock with an RQD less than 50, a Slake Durability Index of less than 90, and a sodium soundness loss of greater than 12% should be considered as soil-like with respect to scour potential. The measurements of RQD included on the boring logs and the results of Slake Durability and sodium sulfate soundness laboratory test results indicates potential scourability of portions of the rock formations beneath foundations at Piers 1, 2, 3, 8, 9, and 10. The foundation material under Piers 4 and 5 appears to be adequate to resist scour but needs further study. These were core runs indicating RQD above and below the critical value of 50% at various depths beneath the existing foundations. Similarly, the laboratory test results indicate intervals of foundation materials to have Slake Durability indices of less than 90 and sodium sulfate soundness loss of greater than the threshold value of 12%. All of the factors used to evaluate the scour potential of the bed rock on the site have been based on empirical indicators as outlined in the FHWA Memorandum.

Piers 2, 3, and 4 were repaired in 1993. Photographs and a review of the repair plans show that the grout repair did not enclose the material under the footings. The grout repair was done to only the upstream side of the footing and two-thirds of the footing length on each side of the footing, but not on the remainder of the footing thus leaving the remainder of the footing exposed to scour. In addition, photos taken at the footing repair show that the grouting is intermediate and doesn't appear to provide a continuous concrete protection around the footing.

2.4.13 Conclusions & Recommendations

Based on our analysis of the spread footing on the existing bridge, it is our professional opinion that the foundation materials under Piers 1, 2, 3, 8, 9 and 10 should be stabilized to prevent possible scouring of that material under the footing. The proposed method to be used to stabilize the soil materials under the pier footings is pressure grouting of the soil materials directly under these pier footings. This method needs further study which will need to be done during the final design.

2.5 BRIDGE SUPERSTRUCTURE EVALUATION

2.5.1 Portal Bracing

Some portals on both the 160' and 200' spans have impact damage to the bottom chord of the bracing. Vertical clearance from original drawings is 14'-0" to the centerline of the bottom chord member. Previous repair replaced the bottom chord full-width with a rolled I-beam installed perpendicular to the inclined portal. Bottom flange protrudes down below the 14'-0" clearance line.

Repair Options	No. of Locations	Cost Each Location	Totals	Discussion	Recommendation
A - Repair existing portal members by use of heat-straightening, welded or bolted splices at member tears, and replacing severed gusset plates in kind.	5	$2,000	$10,000	Minimal repair to secondary members is adequate for structural requirements and restores "original" condition. Requires adequate posting both at bridge and approach roadways. "Headache" bars could also be added.	Recommended
B- Replace existing portal bottom chords with rolled channel section.	18	$12,000	$216,000	Restores original vertical clearance, but still substandard. Minimal visual impact, but significant cost.	Not Recommended
C- Reconfigure portal bracing to provide required vertical clearance. Cut portal truss diagonals as required and use welded connections for frame, as opposed to truss configuration of portal.	18	$16,000	$288,000	Provides adequate vertical clearance. Fairly strong visual impact due to reconfiguration. Must be checked for structural capacity due to change in portal bracing height. It also encourages higher loads on the bridge.	Not Recommended

2.5.2 Intermediate (Sway) Bracing

Original intermediate sway bracing had 14'-0" vertical clearance at centerline. Several members have been replaced, but at the same height. Impact damage to these relatively light-weight members is severe in several locations, and includes deflections of the truss verticals. This bracing is required for overall lateral stability and reducing the unbraced length of the truss verticals.

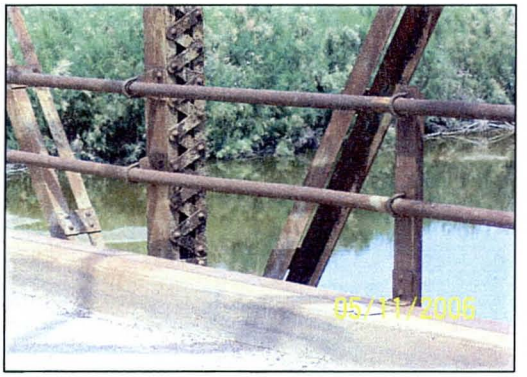
Repair Options	No. of Locations	Cost Each Location	Totals	Discussion	Recommendation
A - Replace bottom chord angle and gusset plates at damaged locations only. Heat-straighten other bracing members as required.	28	$10,000	$280,000	Restores original vertical clearance, but still substandard. Minimal visual impact, but significant cost.	Recommended
B - Replace all the intermediate sway bracing in the same configuration but with adequate vertical clearance. Use welded connections.	83	$16,500	$1,369,500	Extremely high cost. May affect the structural capacity of the verticals due to change in bracing. Minimal visual impact. Encourages higher loads on the bridge.	Not Recommended

2.5.3 Vertical Truss Members

The bent vertical members need to be straightened in order to increase the load capacity of the bridge. This will require that they be heat straightened. The cost of this has been included in the cost estimate. There are no other feasible options.

2.5.4 Bridge Railings

The existing bridge railing (both sides) consists of a continuous, 2-rail system of two inch diameter steel pipes mounted 15" on center above the 9" curb. The railing is supported either by 5 inch x 6.7 channel section posts bolted to the curb stringer or by U-bolt clips on the truss members. The channel section posts are spaced at about 6'-8" on center. This system has survived nearly 80 years of use, but is heavily rusted throughout with some areas of collision damage. The railing does not meet current AASHTO design requirements for strength, geometry, or crash-worthiness. The railing also transfers vehicle impacts directly to truss members.



Repair Options	No. of Locations	Cost Each Location	Totals	Discussion	Recommendation
A - Repair existing historic railing members where damaged. Remove paint and apply a new coating system. Provide silicon lubricant to each slip joint, after painting.	3331 LF	$20	$66,620	Restores original condition. Addresses none of the concerns as to vehicular safety. Trusses remain at risk from impact transferred through connections.	Recommended
B - Construct a new bridge barrier railing with a modern crash worthy two rail steel Wyoming Barrier Rail. The post for this system would be anchored into the new concrete curb. The historic railing will remain and be repaired.	3331 LF	$150	$499,650	Provide a modern crash tested barrier rail adjacent to the existing pipe railing.	Recommended
C - Install "safety shape" reinforced concrete barrier full length as part of redecking.	3331 LF	$80	$266,480	Cheaper alternative, but blocks view and creates "tunnel effect" on a long, narrow bridge. Visual impact is significant and structure must be checked for capacity.	Not Recommended

2.5.5 Nested Roller Bearings

Both the 160' and 200' spans use a system of four nested rollers under the rotational bearing to provide for thermal expansion. The rollers were kept in alignment by two outer "keeper" plates and a centerline groove with a corresponding guide bar on the underside of the sole plate. Several of these bearings are frozen, skewed or misaligned. At least one location shows the sole plate keeper bar out of the roller groove. All rollers are heavily corroded with doubtful movement capacity.

Repair Options	No. of Locations	Cost Each Location	Totals	Discussion	Recommendation
A - Jack up the expansion end of each truss. Remove roller nests and polish the surface of each roller. Replace keeper bars. In field, grind the under side of the sole plate and top of masonry plate to a smooth finish. Position roller nest for ambient temperature, and lower truss down on to the roller nest.	18	$45,000	$810,000	Restores original thermal movement capacity. Time required to polish and reinstall bearings may keep the bridge out of service for many months, depending upon other work planned, this may or may not be a concern since thermal movement will continue with trusses on temporary supports.	Not Recommended

Repair Options	No. of Locations	Cost Each Location	Totals	Discussion	Recommendation
B - Provide new roller nests to avoid extensive period of temporary support.	18	$42,000	$756,000	Restores original condition and movement capacity. Significant reduction in risk due to unknown condition of rollers and time each truss must be temporarily supported. Cost of new rollers may be less than refurbishing the existing and can be ordered before construction begins. Lead time for rollers will be considerable.	Recommended
C - Similar to other options except replace the roller nests with Teflon on stainless steel sliding plates, with appropriate shims to meet existing height. Provide heavier gauge keeper plates to ensure alignment.	18	$50,000	$900,000	Provides full thermal movement capacity with a long-term, no maintenance solution. Similar to Option B in that material can be ordered ahead so that temporary support of trusses is minimized.	Not Recommended

2.5.6 Jacking of Trusses at Expansion Joints

To complete repairs to the truss bearings, the trusses must be raised from their bearings. Currently, the trusses sit on concrete piers with limited width for machinery in the longitudinal direction. Two options are available for this type of work. One is to jack the truss from underneath through the floor beam. The other is to use a crane or some other type of equipment to lift the truss from an adjacent span. Of the two options, the jacking option is the simpler of the two and is the preferred alternative. Below is a discussion of the requirements and/or repairs needed before jacking of the trusses is permitted.

An analysis of the existing floor beam indicates that there is enough capacity in the beam to jack the truss. The limiting factor for the jacking of the trusses is the moment capacity of the floor beam. In order to prevent the capacity from being exceeded, the location of the jacks along the floor beam will have to be located within 5'-0" of the floor beam to truss connection for the 200-foot spans and within 6'-6" of the floor beam to truss connection for the 160-foot spans.

While the floor beam has enough capacity to take the loads induced by jacking, the connection between the floor beam and truss does not. The existing connection consists of two 6x4x1/2 angles connected to the floor beam with 11-3/4" diameter rivets and to the truss with 16-3/4" diameter rivets. These rivets do not have the capacity to carry the weight of either the 160-foot or 200-foot truss. Two options were considered to accommodate this issue. One is to remove the concrete deck and curb prior to jacking. By removing the concrete deck, the dead load of the trusses reduces enough to allow jacking without any other modification made to the existing connection. This solution, however, is limited to situations where the deck is already planned on being replaced since the cost of removing and replacing the deck makes this option prohibitively costly if done for no other reason than repairing the truss bearings. The other option is to replace the rivets in the existing connection with high strength A325 bolts. This option requires the existing rivets to be removed and new ¾" bolts placed through existing holes. Support of the floor beam would also be required during operations to prevent movement while the connection is being worked on.

Another option is to provide a spreader beam under the truss with a jack on each side of the truss bearing. The gusset plate on the truss is very deep and appears to be adequate to support the truss jacking loads. It also may be possible to provide a spreader beam on the end of the truss that could be attached to the end of the truss. This option needs further evaluation, but may allow jacking of the trusses without removal of the concrete deck.

2.5.7 Strengthening of Stringers and Floor Beams

The existing bridge was built using a non-composite system with the deck acting independently from the existing steel members. The original construction plans indicate the bridge was designed in 1925 and show that the design truck was two 15-ton trucks. The current design vehicle recommended by "LRFD Bridge Design Specifications" Third Edition, 2005 by the American Association of State Highway and Transportation Officials (AASHTO) is the HL-93 Truck, which is heavier than the truck load used in the original design. Initial analyses showed that using the HL-93 design truck produces loads that exceed the strength capacity of the interior stringers and floor beams. The floor beam strength capacity was exceeded based on the size of the beam with the section being smaller than what is required. The interior stringers were exceeded due to the possibility of lateral torsional buckling. Currently, the deck is not connected to the stringer and does not provide any lateral stability. There are sections of the bridge where there is a visible gap between the stringer and deck. This allows the stringer to rotate thereby reducing its strength capacity. The section itself is strong enough to resist the design loads; however, the stringer would need to be sufficiently braced in order to develop its full strength capacity. The design truck did not produce loads that exceeded the strength capacity of the exterior stringers.

Several alternatives were considered for strengthening the existing members to accommodate the HL-93 Truck load. It was determined that the most efficient way to increase the capacity of the system was to take advantage of a composite system. A composite system is advantageous in that it allows the steel member and concrete deck to act in tandem as one section. This system allows the concrete in the deck to take a large portion of the compressive load. This in turn allows the tension stresses to be applied to a larger portion of the existing stringers and beams, which in turn increases overall load capacity of each. Additionally, providing a composite section fully braces the member preventing lateral torsional buckling and allowing the members to develop their full capacity.

The design of the composite system requires shear studs on the interior stringers and floor beams (the exterior stringers are currently aligned on top of the floor beam and so cannot be made composite with the deck). Preliminary analysis shows that for the interior stringers, 2-7/8" diameter welded studs at a 6" pitch for a total of 80 studs. For the floor beams, 2-7/8" diameter welded studs at a 7" pitch are required for a total of 72 studs per beam. A new 8½" deck will need to be placed on top of the stringers as well, thereby embedding the shear studs. With the deck and members acting as one unit, the load capacities are increased and are sufficient to carry the HL-93 Truck loading.

Currently, the exterior stringers are braced at 1/3 points along each bay by 1" diameter rods. This aids in developing the strength capacity required to accommodate the design loads. When the deck is replaced, these rods will need to remain in place to ensure lateral stability of the exterior stringers. In addition, it would be beneficial to place additional shear studs on the inside of the stringer to create a connection between the stringer and curb. This would ensure the strength capacity required is achieved. A system using 2-3/4" diameter welded studs at 4'-0" would be sufficient.

Below is a summary of the requirements for strengthening the stringers and beams. The deck is not included and is discussed further in the following section.

Repair Requirement	Unit	Total	Cost	Total
Place 2-7/8" Diameter Stud @ 6" pitch at each interior stringer	EA	26,240	$10.00	$262,400
Place 2-7/8" Diameter Stud @ 7" pitch at each floor beam	EA	6,552	$10.00	$65,520
Place 2-3/4" Diameter Stud @ 4'-0" between exterior stringer and curb	EA	984	$10.00	$9,840
			Total	**$337,760**

2.5.8 Concrete Deck Slab Replacement

The deck has nearly 80 years of use and it shows signs of distress and deterioration. There is transverse cracking at every floor beam and minor transverse cracking throughout. Water has begun to seep through the cracks and cause rusting at the concrete/floor beam interface causing uplift. Spalling has begun to occur along the underside of the deck at the joints. Scaling is present on the deck wearing surface. The existing concrete deck has a maximum thickness of 8-½ inches tapering to 7-½ inches at the curbs.

From a strength perspective, the deck does not meet current code requirements. Transverse reinforcement consists of ½" diameter rods spaced at 8". Longitudinal reinforcement consists of ½" diameter rods spaced at roughly 2'-0". The transverse reinforcement meets current code requirements, but the longitudinal reinforcement does not. The lack of longitudinal reinforcement prevents the loads from distributing to the stringers properly and can lead to excessive cracking at the bottom of the slab between stringers. Additionally, the deck is currently designed as a continuous member. No indication is given in the design plans that negative reinforcement was provided over the joints. This has lead to the heavy cracking in the deck over the floor beams. The top longitudinal reinforcement in the deck will have to be significantly increased to accommodate the negative moment that occurs at these areas.

Due to the combination of wear and failure to meet current code requirements, combined with the need to replace the deck to place shear studs to allow composite action with the interior stringers and beams, it is recommended that the deck be replaced. To meet AASHTO code requirements, an 8-½ inch thick deck slab with #5 reinforcing bars is to be used. The use of normal weight concrete is preferred over the use of light weight concrete due to the increase durability and lower cost. The light weight concrete would reduce the dead load, but the existing framing is strong enough to carry the normal weight concrete.

The deck joints are also showing signs of deterioration. Expansion joints are currently provided at each pier and abutment. The joints consist of an open joint covered by a sliding plate system. Transverse cracks have developed at the joints and extended into the curbs. Two joints, at Piers 8 and 9, appear to be frozen and show no movement during thermal changes. This is most probably caused by the frozen bearings. It is proposed that joints will be modified when the deck is replaced.

Below is a summary of the requirements for replacing the deck.

Repair Requirement	Unit	Total	Cost	Total
Remove Existing Concrete Deck	LS	1	$200,000.00	$200,000.00
Place new concrete (f'c=4000psi @ 28 days) (MAG Class AA)	CY	1015	$900.00	$913,500.00
Provide new reinforcing steel (fy=60,000 psi)	LB	297,980	$0.90	$268,180.00
Modification of Deck Joints	EA	10	$5,000.00	$50,000.00
			Total	**$1,431,680.00**

2.6 BRIDGE PEDESTRIAN WALKWAYS EVALUATION

Pedestrian safety is an issue with the existing structure. Pedestrians currently are not protected from vehicular traffic. This is compounded by the limited 19'-0" clear roadway width. When more than one vehicle is on the bridge, there is nowhere for the pedestrians to go to avoid traffic. To accommodate this, it has been proposed to add a pedestrian walkway to each side of the existing structure. This would remove the pedestrians from the roadway creating a safe place to cross. See Appendix A for examples of historic bridges with pedestrian walkways.

The pedestrian walkway is intended to be 5'-0" wide and connected to the bridge through outriggers at third points of each bay. A decorative steel railing would be provided at the exterior of the walkway for pedestrian protection. This walkway will cantilever from the bridge with connections at the exterior stringer and the outside interior stringer.

This design increases the design load going to the exterior stringer. This increase causes the exterior stringer load to exceed its capacity in service and strength. To accommodate a pedestrian walkway, the exterior stringer will have to be replaced. The recommended solution is to replace the exterior stringer with a new S15x42.9, 50 ksi stringer. The shape of the stringer plus the configuration of the pedestrian walkway framing eliminates the torsion from the pedestrian walkway. The S15x42.9 provides sufficient capacity for both the service and strength conditions while maintaining the member height and overall look by using an "S" member which is similar to the existing standard beam member currently used.

Below is a summary of the requirements for placing a new pedestrian walkway on both sides of the bridge.

Repair Requirement	Unit	Total	Cost	Total
Replace exterior stringer w/ S15x42.9	LB	144,000	$4.00	$576,000.00
Place new outriggers (W12x30) @ third points	LB	107,000	$4.00	$428,000.00
Decking for sidewalk	SF	22,150	$30.00	$664,500.00
Pedestrian Railing (posts included)	LB	55,000	$4.00	$220,000.00
Channel (C8x12)	LB	80,000	$4.00	$320,000.00
			Total	**$2,208,500.00**

2.7 BRIDGE COATING ASSESSMENT & EVALUATION

KTA Tator completed a coating assessment and repainting recommendation for the US-80 Bridge. The Bridge is predominantly covered in thin tightly bonded surface rust. In crevice areas there are some heavier flaky films, but no appreciable section loss was observed. The coating that is left is poorly bonded and severely degraded. The coatings contain high levels of lead and low levels of chromium.

Complete coating removal and replacement is required. Two methods of removal are viable:

1. Complete abrasive blast cleaning within a fully sealed negative pressure containment which removes all coating, rust and mill scale.

2. Water jetting which removes all coating.

Water jetting would be supplemented with localized power tool cleaning to remove corrosion, or the water stream enhanced with abrasive injection to remove the rust and to better clean around the rivets, bolt heads and crevices. All applicable environmental and industrial hygiene regulations associated with lead abatement will need to be followed throughout the surface preparation, clean-up and waste disposal operations.

Coating system selection will be dependent upon the method of preparation selected. If the surface is abrasive blast cleaned, a zinc/epoxy/urethane system should be applied. If water jetting is selected, a rust inhibitive alkyd/silicone alkyd system should be applied.

Any new steel that is to be added should be abrasive blasted in the shop to meet the requirements of SSPC SP-10, "Near White Blast Cleaning." The surfaces should be shop primed with the respective primer from the system selected for painting the existing bridge and two field coats applied after installation. In the case of the epoxy/urethane system, a shop primer with ample recoat time should be used.
The following are the results of the investigation:

1. The steel surfaces of the bridge have lost much of the existing coating. The surfaces were predominantly covered in rust. A description and a breakdown of the degree of rusting are included in Table No. 11 below.

TABLE NO. 11 – SURFACE RUSTING OF STEEL ON OLD US-80 BRIDGE

Location	Percent of Surface Covered by Rust
Floor Beams	Approximately 75% of the surfaces displayed pinpoint rust through of the coating. The existing coating is flaking and delaminating.
Stringers	Approximately 85% of the surfaces displayed pinpoint rust through of the coating. The existing coating is flaking and delaminating.
Laterals	Approximately 75% of the surfaces displayed pinpoint rust through of the coating. The existing coating is flaking and delaminating.
Roller Bearings	Approximately 65% of the surfaces displayed pinpoint rust through of the coating. The existing coating is flaking and delaminating.
Bearing Plates	100% covered in tightly bonded surface rust.
Top of Lower Flanges (Floor Beams and Stringers)	100% covered in tightly bonded surface rust.
Joiner and Gusset Plates	Approximately 40% of the surfaces displayed pinpoint rust through of the coating. The existing coating is flaking and delaminating.
Upper Chords - Undersides	100% covered in tightly bonded surface rust.
Hand Rails	100% covered in tightly bonded surface rust.

2. In most areas, the rust is superficial and tightly bonded to the substrate.

3. In other areas (crevices and undersides of structural components) the rust is flaky and loose. However, no appreciable section loss was observed.

4. Adhesion of the existing coating is poor (ASTM D3359: 0A and ASTM D6677: 0) in all areas tested (ten to fifteen tests).

5. Dry film thickness measurements, taken in random areas, ranged from 4 to 9 mils with most readings falling into the 6 to 9 mil range.

6. When the coating film was viewed with a Tooke Gage (destructive device used for examining paint film cross-sections) some evidence of a red primer was observed. In addition, a single silver layer could be discerned. There was no evidence of any additional coats.

7. The substrate beneath the paint is covered with mill scale. No evidence of original abrasive blast cleaning was found.

2.7.1 Discussion and Recommendations

The Old US-80 Bridge is significantly rusted with only around 15% of the surface area still covered in coating. In most cases the rust is superficial and tightly bonded. In some areas there are accumulations of flaky rust. However, removal of the rust shows little evidence of section loss.

The coatings that remain are heavily worn, poorly bonded, and have reached the end of their useful service life. The underlying substrate is covered with mill scale.

Dry film thickness measurements of paint indicate that when still intact there are layers of coating that typically range from 6 to 9 mils. This coating is old, deteriorated, delaminating and worn, and must be completely removed and replaced.

Chemical analysis indicated that the paint has an extremely high lead content and also low concentration of chromium. As such, project controls need to be selected that control exposure of the workers and the environment to lead and chromium.

Two options are available for removal of the coatings. The first, which is the recommended scheme, is to abrasive blast cleaning which is the more expensive option but will provide a 35-year system with almost no maintenance. A total cost of approximately $5,000,000 is estimated to perform this recoating option. The second option, high-pressure water jetting is significantly less expensive but will require some maintenance after about a ten-year period. A total cost of $3,000,000 is estimated to perform the less rigorous recoating option.

Dry abrasive blast cleaning to SSPC-SP10, "Near White Blast Cleaning" will efficiently remove the coating, rust and mill scale and impart a surface profile on the steel. While this method is very efficient for cleaning, it will require the installation of a fully sealed containment system with negative pressure ventilation and dust collection. If complete abrasive blast cleaning is specified, a three-coat epoxy zinc/epoxy/urethane system should be applied.

As an alternative, water jetting can be used to remove all of the existing coating, while allowing mill scale to remain over less than 5% of the surface area. It is often desirable to remove the layer of mill scale because of the possibility of future delaminations when exposed to moisture. Due to the dry desert conditions, it is not necessary in this case.

Water jetting, if selected, should be performed in accordance with SSPC-SP 12, "Surface Preparation and Cleaning of Steel and Other Hard Materials by High- and Ultrahigh-Pressure Water Jetting Prior to Recoating." A WJ-2 degree of cleaning should be specified to remove the coating. Water jetting will remove the paint, but will keep the lead-containing dust suppressed and in the wash water. The wash water with the entrained debris is collected and disposed of appropriately.

Coating removal efficiencies can be increased with the addition of an abrasive to the water. This would be ideal for the bridge since there are heavy layers of coating in a number of areas. Likewise, while the overall surface area is not heavily rusted, there are areas of local rusting that would be better cleaned using the water with abrasive injection.

Following the water jetting, power tool cleaning should be performed in accordance with SSPC-SP15, "Commercial Grade Power Tool Cleaning" to remove all rust and to impart a profile to the substrate. The power tool cleaned areas can then be touched-up with a rust inhibitive alkyd primer. The entire surface should then be coated with a full coat of the rust inhibitive primer and a full coat of silicone alkyd finish, formulated with a minimum of 30% silicone.

Regardless which coating system is applied, special care will need to be taken to stripe coat all rivets, bolt heads and joints with all coats of paint, in addition to working the material into the crevices.

Any new steel that is to be installed on the bridge should be abrasive blasted in the shop to meet the requirements of SSPC SP-10, "Near White Blast Cleaning." The surfaces should be shop primed with an inorganic zinc primer or rust inhibitive alkyd primer, corresponding with the coating system selected above. The same two field coats used for the existing steel should be applied after installation of the steel.

It should be noted that because the existing steel is coated with a lead and chromium containing paint system, all modifications (cutting, grinding, etc.) would have to be performed compliant with the applicable lead abatement regulations. Consideration should be given to removal of the paint prior to performing the modifications. After clean up, the modifications can be performed in a normal environment without all of the requirements (personnel protection, air monitoring, special disposal requirements) associated with lead abatement.

3.0 ALTERNATIVES

3.1 GENERAL

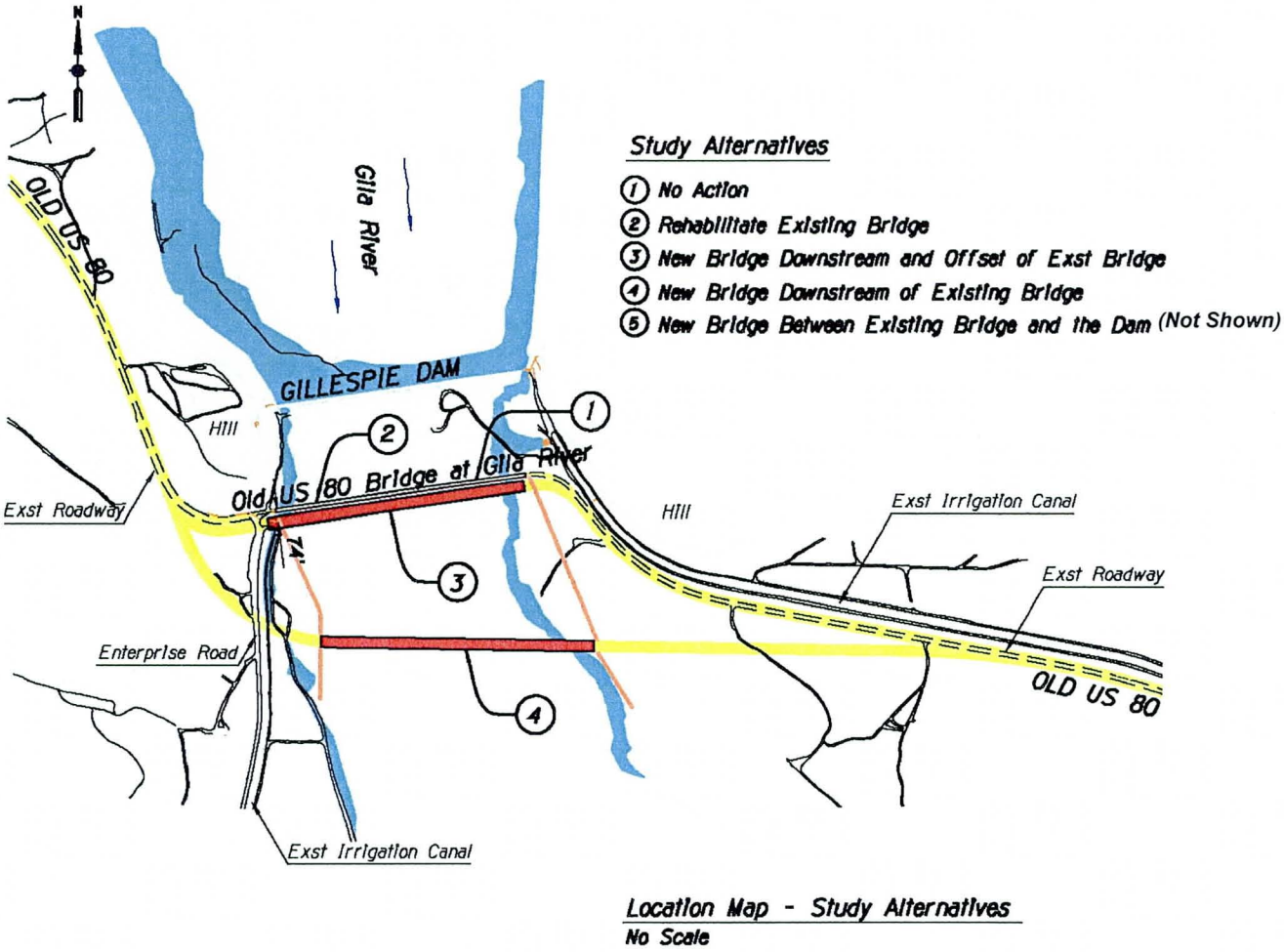
The location of the existing bridge crossing was probably influenced by the location of the existing river crossing along the spillway of the Gillespie Dam (built in 1921), the narrow river section, and the presence of relatively shallow bed rock. An overview of potential alignment corridors upstream and downstream of the existing crossing was performed. Evaluation of potential corridors upstream of the existing crossing did show that they appear to be less desirable than the crossing at the downstream corridor. Areas to cross the Gila River upstream of the existing bridge were approximately a half a mile to a mile upstream due to the terrain obstacles along the river, in particular the mountains along the east side and the location of the dam which is about 500 feet upstream from the existing bridge. The realignment of a new route through this area would substantially increase the length of new roadway required to reconnect to the existing alignment. In addition, the corridor did not provide for a potential reduction in new structure length and the connection to the existing roadway alignment along the east side of the river could potentially be more costly due to the rolling/mountainous terrain. Further evaluation of alignments upstream of the structure was not continued based on this overview.

The existing Old US-80 Bridge and approach roadways were constructed around 1927 on and along an existing roadway alignment. The roadway provided an 18-foot wide paved surface allowing a 9-foot wide lane in each direction and 3-foot tapered shoulders. The current roadway has been improved with the construction of a wider paved roadway surface of 28 feet providing a 12-foot wide lane in each direction along with a 2-foot paved shoulder. While improvements to the pavement width and the addition of safety devices such as guardrail have been made to the roadway, the original 1927 alignment and profile do not appear to have been modified. The existing steel truss bridge includes a 19-foot wide clear roadway from face of curb to face of curb. Currently, the bridge is posted as 18 feet wide and is striped for two travel lanes, one in each direction. The vertical clearance on the existing bridge is posted at 13'-6" and a maximum weight limit of 40,000 lbs which is less than the legal load limit. Due to the substandard deck width there are operational problems due to wide farm equipment and large trucks that use the structure and cannot maintain two-way traffic when they cross.

The study area limits were established to include approximately one half mile east and west of the existing bridge structure. The terrain on the east approach is level while the terrain on the west approach is considered rolling.

3.2 ALTERNATIVES STUDIED & EVALUATION CRITERIA

Further investigation of alignment alternatives downstream of the existing crossing was continued. One alignment that was investigated and discontinued followed the midsection line across the Gila River approximately 200 to 300 feet downstream of the existing bridge. The alignment was located immediately upstream of the El Paso Gas right-of-way and pipeline. Due to limited improvement of the roadway alignment, the presence of the pipeline, and the potential for additional scour to the pipeline further consideration of the alignment was discontinued.



OLD US-80 GILA RIVER BRIDGE (GILLESPIE DAM BRIDGE)
MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION
CONTRACT 2005-056, WORK ORDER NO. TT188
STUDY ALTERNATIVES

FIGURE 13 – LOCATION MAP STUDY ALTERNATIVES

The alternatives presented in the scope of work were evaluated to determine the feasibility relative to the project requirements, budget, and schedule. See Figure 13 on page 54. We have evaluated these five alternatives:

Alternative 1	Do Nothing (No Build/Status Quo)
Alternative 2	Rehabilitate the Existing Bridge
Alternative 3	New Bridge Parallel and Adjacent to Existing Bridge
Alternative 4	New Bridge Downstream of Existing Bridge
Alternative 5	New Bridge Between Existing Bridge and the Dam

The alternatives studied were developed and evaluated by the following criteria:

1. Does the alternative provide a bridge structure which meets current AASHTO standards?
2. Does the alternative provide pedestrian walkways?
3. Does the alternative provide AASHTO HS20-44 live load capacity?
4. Is the alternative eligible for FHWA Bridge Replacement and Rehabilitation Funding?
5. How difficult will the alternative be to construct? Will closure of the bridge be necessary for construction?
6. Does the alternative impact residents and businesses in the area?
7. Does the alternative retain the existing Old US-80 Gila River Bridge and is it acceptable to SHPO?
8. What is the estimated project cost of the alternative?

A brief description of each alternative including advantages and disadvantages of each are as follows:

Alternative 1 Do Nothing or Maintain Status Quo (No action)

Roadway Design: No modification to the existing roadway.

Structure Design: No modification to the existing structure.

Impact on Environmental Resources: No additional impact on environmental resources.

Impact on Historic Resources: No modification to historic structure.

Public Involvement: At the public meeting for the bridge, the community supported the idea of rehabilitation of the existing bridge. The public supported the need for pedestrian walkways on the bridge. Area farmers suggested the need for a wider bridge with more vertical clearance.

Advantages:

1. There will be no construction cost for this alternative.
2. There will be no additional environmental impacts with this alternative.
3. There will be no effect on the historic integrity of the bridge.

Disadvantages:

1. This alternative will retain all the substandard roadway features including deficient shoulders and approach roadway geometry.
2. This alternative would not provide for safe pedestrian/bicycle use of the existing bridge requiring pedestrians and bicycles to share the roadway with vehicle use of the bridge.
3. This alternative would retain all the substandard bridge features including, but not limited to weakened or bent truss members, the roller bearings will continue to deteriorate and not provide for expansion/contraction of the steel trusses which could lead to failure of the bridge and reduction in live load capacity. A failure of the bridge would render it unusable since the structure will have little if any redundancy. This alternative would also not provide protection for trusses with a barrier rail or would not allow for repainting of the steel trusses.

4. This alternative will retain a scour critical rating which could require closure which could result in settlement of the bridge and its possible closure in the event of a major flood.
5. This alternative will retain structural deficiencies and the continued deterioration of the structure would eventually require that the load capacity be reduced which would prevent school buses and heavier vehicles from using the bridge.

Alternative 2 Rehabilitation of the Existing Bridge (Preferred Alternative)

This alternative would rehabilitate the existing Old US-80 Gila River Bridge for two-way traffic with pedestrian walkways on each side of the bridge. The bridge width would remain at 19'-0" clear roadway width and vertical clearance would remain at 13'-6". The live load would remain 20 tons and the bridge would be posted even though the bridge can support the legal load limit. See Appendix B.

Consideration was given to using the bridge for alternating one-way traffic, but this concept was felt not to be practical and the bridge will remain a two-way bridge with two 9'-6" wide travel lanes.

The rehabilitation would include:

- Replace concrete deck slab
- Repair truss roller bearings
- Strengthen floor beams and stringers
- Replace exterior raised stringers with stronger members to support pedestrian walkway
- Provide new crash tested bridge barrier rail
- Rehabilitate the concrete wing walls
- Heat straighten bent and damaged truss and bracing members and the bridge railing
- Repaint bridge
- Scour protect piers
- Install approach guard rail
- Provide protective beams at each entrance to the bridge
- Add pedestrian walkways to each side of the bridge
- Repave and regrade roadway approaches to bridge

Roadway Design: Safety improvements would be made to the existing roadway alignment including guard rail modifications and improved stopping sight distance on the west approach by laying back the cut slope on the north side of the roadway.

Structure Design: The existing bridge would be rehabilitated including replacement of the concrete deck slab, repair of truss roller bearings, strengthening floor beams and stringers, new crash tested bridge barrier, rehabilitation of concrete wing walls, heat straighten bent and damaged truss members, repaint bridge, add pedestrian walkways, and scour protect the piers.

Impact on Environmental Resources: Rehabilitation of the existing bridge has fewer environmental impacts that any other build alternative. Most of the work could be done from the existing bridge except scour protection of piers which will require accessing the Gila River to allow access for equipment. The replacement of the concrete deck will probably require access to the Gila River, but no removed concrete or debris will be allowed in the Gila River. Repainting of the bridge can be done in such a manner as to have minimal environmental impacts.

Impact on Historic Resources: The rehabilitation of the existing bridge will be done in such a manner as to not cause an adverse impact to this important historical resource. Repairs will be made to replace deteriorated portions of the bridge

that are compatible with the existing construction. The bridge will be repainted, the pedestrian walkway will be added to the bridge and the metal barrier rail added which will need concurrence with SHPO.

Public Involvement: At the public meeting for the bridge, the community supported the idea of rehabilitating the Old US-80 Bridge. The public supported the need for a pedestrian walkway on the bridge. Area farmers suggested the need for a wider bridge with more vertical clearance which cannot be obtained with the rehabilitation of the existing bridge. During construction, it was felt to be important to provide a detour route if the existing bridge was to be closed during construction.

Construction Detour: It will be necessary to close the bridge while repairs are being made to the bridge. We estimate that it will be necessary to close the bridge for approximately 10 months during which time a construction detour will be required. See Section 6.6 on page 75.

Estimated Project Cost: The estimated project costs for this alternative including engineering and construction costs are $16.7 million.

Advantages:

1. The existing construction cost associated with the rehabilitation of the bridge would be less than the construction of a new bridge and will serve the community for a number of years until a new bridge is needed.
2. The existing bridge life would be extended making use of an available County resource.
3. The environmental impacts to rehabilitate the existing bridge would be less compared to building a new bridge.
4. The existing bridge currently has a traffic volume of 200 vehicles per day and has a maximum capacity of 8,000 vehicles per day. Due to the narrowness of the bridge, drivers would tend to slow down when transversing the bridge with oncoming traffic. Large trucks and wide farm equipment would probably cause people to wait at the other end. This would considerably reduce the traffic capacity across the bridge from a standard two lane road capacity of approximately 8000 vehicles per day.
5. The pedestrian walkway can be added to each side of the bridge which will make the bridge more useful and reduce the potential hazards for pedestrians using the bridge.
6. The rehabilitation of the existing bridge has public support and by rehabilitating the bridge we make use of a Maricopa County historic resource.
7. The existing bridge is eligible for Federal Bridge Replacement and Rehabilitation Funds.
8. The bridge would no longer need to be posted.

Disadvantages:

1. The narrow bridge width of 19'-0" clear roadway and the 9'-6" wide travel lanes would remain.
2. The vertical clearance of 13'-6" would remain and over-height vehicles could not use the existing bridge.
3. The addition of the pedestrian walkways on each side of the bridge will have a visual impact to the bridge and will require concurrence with SHPO.
4. A detour would be needed to rehabilitate the existing bridge that will cause delays to current users of the bridge.

Alternative 3 New Bridge Parallel and Adjacent to the Existing Bridge (35 MPH) (Not Recommended)

This alternative would include building a new bridge 74 feet downstream from the existing bridge. The existing bridge could be used to maintain traffic during construction while the new bridge is built. This alternative would not include any improvements to the existing bridge. The advantage to this alternative is that the structure would accommodate standard traffic lane widths associated with the reconstructed approach roadways. The disadvantages include increased project cost, environmental and aesthetic disturbances, continued use of low design speeds to connect into the existing roadways, reconstruction to adjacent irrigation facilities and turnouts and the use of superelevation transitions on the new bridge due to the adjacent curves. The new bridge cost would also be higher since the pier/span configuration of 200' and 160' span of the existing structure would need to match the existing structure to

minimize the scour and maximize the flow capacity of the structure. This alternative was eliminated from further consideration due to its high cost and impacts the historic bridge.

Alternative 4 New Bridge Downstream from Existing Bridge (55 MPH) (Future Recommended Alternative)

This alternative would include building a new bridge about 1,000 feet downstream of the existing bridge. The new bridge would be about 1,800 feet long to span the floodway of the Gila River. This alternative would not include any improvements to the existing bridge. The approach roadways would be aligned to accommodate 55 mph design speed and the bridge width increased to accommodate future traffic demands. The existing bridge would be used to maintain traffic during construction while the new bridge is built. The width of the bridge was assumed to be 87 feet, but could be sized for needed future traffic demands.

Roadway Design: The approach roadway would be designed to meet current MCDOT and AASHTO requirements.

Structural Design: The new bridge would be 14-span, 1,800-foot long, AASHTO Type 6 Prestressed Concrete I-girders with cast-in-place concrete deck slab supported on drilled shaft foundations. The bridge width would be based on traffic demands.

Impact of Environmental Resources: Construction of a new bridge will require additional environmental studies that will need to be done prior to the construction of the bridge.

Impact on Historic Resources: The construction of a new bridge about 1,000 feet from the historic bridge will have minimal visual impact on the historic resource, but will need SHPO concurrence. The existing bridge could remain in service for one-way traffic or it could be made into a pedestrian bridge.

Public Involvement: Additional public involvement will be needed while the new bridge is constructed.

Estimated Project Cost: The estimated project costs for this alternative including engineering and construction costs is $45.3 million.

Advantages:

1. This alternative provides a new bridge that can carry future and current traffic without any height, width, or load restrictions.
2. Allows for future growth in the area without restriction.
3. The new bridge can be built using the existing bridge as a detour.

Disadvantages:

1. This alternative is the most costly alternative and provides for traffic demands that currently do not existing at this time.
2. This alternative would appear to be unneeded at this time.
3. Environmental impacts will be greater than Alternative 2 Rehabilitate the Existing Bridge.
4. This bridge may not be eligible for Federal Bridge Replacement and Rehabilitation Funds.

Alternative 5 New Bridge Between Existing Bridge and the Gillespie Dam (Not Recommended)

This alternative would include building a new bridge upstream of the existing bridge between the existing bridge and the Gillespie Dam. This alternative was eliminated from further evaluation due to possible impacts to the Gillespie Dam and the existing bridge and the roadway alignment would require cutting through adjacent mountains to provide for the roadway alignment.

Alternative 2 – Rehabilitate Existing Bridge for Two-Way Traffic (with new pedestrian walkways) is the preferred alternative. The Historic Old US-80 Gila River Bridge (Gillespie Dam Bridge) will be rehabilitated so that it will be adequate to carry two-way traffic. Based on evaluation of engineering, environmental, and cost criteria, the alternative was determined to best meet the objectives and requirements of the project. It preserves the historic bridge and it has the fewest environmental impacts. This alternative will provide an all-weather crossing of the Gila River for the least cost and it meets current traffic demands of the area. See Figures 14 thru 18 on pages 62-70 and Appendix B. The bridge rehabilitation will require the following work:

a. Bridge Concrete Deck Replacement – The existing concrete deck has extensive cracking and deterioration and it is not properly connected to the floor stringers. We propose to replace the concrete deck with a new concrete deck that will be reinforced to reduce cracking of the concrete and meet current AASHTO code requirements. It also facilitates the modifications of the bridge bearing by reducing the load on the trusses and making it easier to jack up the trusses and it allows for the addition of welded studs to the floor stringers so that the concrete slab and stringers become a composite member that is stronger than the stringer and slab acting separately. The steel studs will also provide a positive attachment to the steel framing to prevent the separation between the concrete and the steel stringers.

b. Repair Truss Bearings – The existing truss roller bearings are to be replaced to allow for temperature movement of the trusses. We propose to jack up the expansion end of each truss and replace the roller nests with new rollers.

c. Replace the Exterior Raised Steel Stringers on both sides of the bridge with stronger steel framing to support the pedestrian walkways. The pedestrian walkway will be part of this alternative unless SHPO objects. The depth of the new steel stringers will remain the same and the new member strength will be increased by using heavier stringers and high strength steel.

d. We propose to provide New Bridge Barrier Rails – The existing bridge barrier rail consists of 5-inch channel post with 2-inch diameter pipe railing. This railing is also attached to the steel trusses so that when a vehicle impacts the railing it could damage the steel trusses. The new bridge barrier rails will be a crash tested barrier rail and will be placed in front of the existing barrier rail and on top of the curb and not connected to the trusses.

e. Concrete Wing Wall – The concrete wall at each wing wall is lightly reinforced and would be strengthened with a thicker, stronger wall with the same appearance of the existing wall. The concrete wall also needs to protect the blunt end of the new steel barrier rail in order to meet AASHTO code requirements. By making the wall thicker it accommodates this requirement. Also a new attachment will be provided to the concrete attachment wall to support the pedestrian walkway transition between the new pedestrian walkway and the embankment in back of the new guard rail.

f. Heat Straightening Bent and Damaged Steel Members – Use the heat straightening method to repair bent or damaged sway bent vertical truss members and portal bracing, existing pedestrian railing, and other related framing.

g. Repaint Bridge – The repainting of the bridge is necessary. This will first include an abrasive blast cleaning to remove the existing paint and then a recoating of the steel. This system will provide a minimum of 75 years of protection for the bridge.

h. Scour Protection of Piers – We propose to pressure grout the existing foundation material to piers 1, 2, 3, 8, 9, and 10 so that the bridge is no longer scour critical. This will stabilize the soil materials under the footing to prevent the material under the pier footings from scour in the event of a major flood event.

i. Install Approach Roadway Guardrail – New approach guardrail to be attached to the concrete wing walls to protect the walls and ends of the bridge trusses.

j. Provide Protective Beam at Each Entrance to the Bridge – Rather than raise the portal or sway bracing on the bridge, a protective beam will be installed at each end of the bridge to prevent over-height vehicles from traveling on the bridge and hitting the existing portal and sway bracing.

k. Add Pedestrian Walkway to each side of Bridge – In order to provide a bridge that pedestrians can safely use, the addition of a pedestrian walkway is proposed for both sides of the bridge. The design of the walkway will need to be approved by the State Historic Preservation Office (SHPO) and needs to have no adverse effect on the bridge. This decision will need to be made after consultation with SHPO.

l. Posting Bridge – The existing historic bridge will no longer need to be posted for live loads and the height limited to a maximum of 13'-6" and the width limited to 19'-6". The design speed to remain 25 mph on the bridge.

m. Repave/Regrade Roadway Approach to Bridge – The existing roadway paving needs to be replaced with new paving at the approaches to the bridge and additional embankment material will be needed to provide for a walkway in back of the guardrails on the approaches. Consideration should be given to extending the concrete approach slabs so as to provide a smoother transition from the paving to the bridge.

n. Deck Joint Repair – The existing deck joints will be repaired where needed and reused. Cover plates at the curbs will be provided and consideration will be given to the adding of an upturn to prevent water from going directly into the bridge roller bearings.

o. Two-Way Traffic – The bridge will remain a two-lane bridge with 9'-6" wide lanes, with one lane in each direction. Signage will be provided at each end of the bridge to warn users of the bridge that if a wider vehicle is on the bridge, to stop and wait for it to cross the bridge.

p. Installation of Historic Markers – National Register Markers and a Marker that tells the history of the bridge and the crossing of the Gila River will be provided at each end of the bridge.

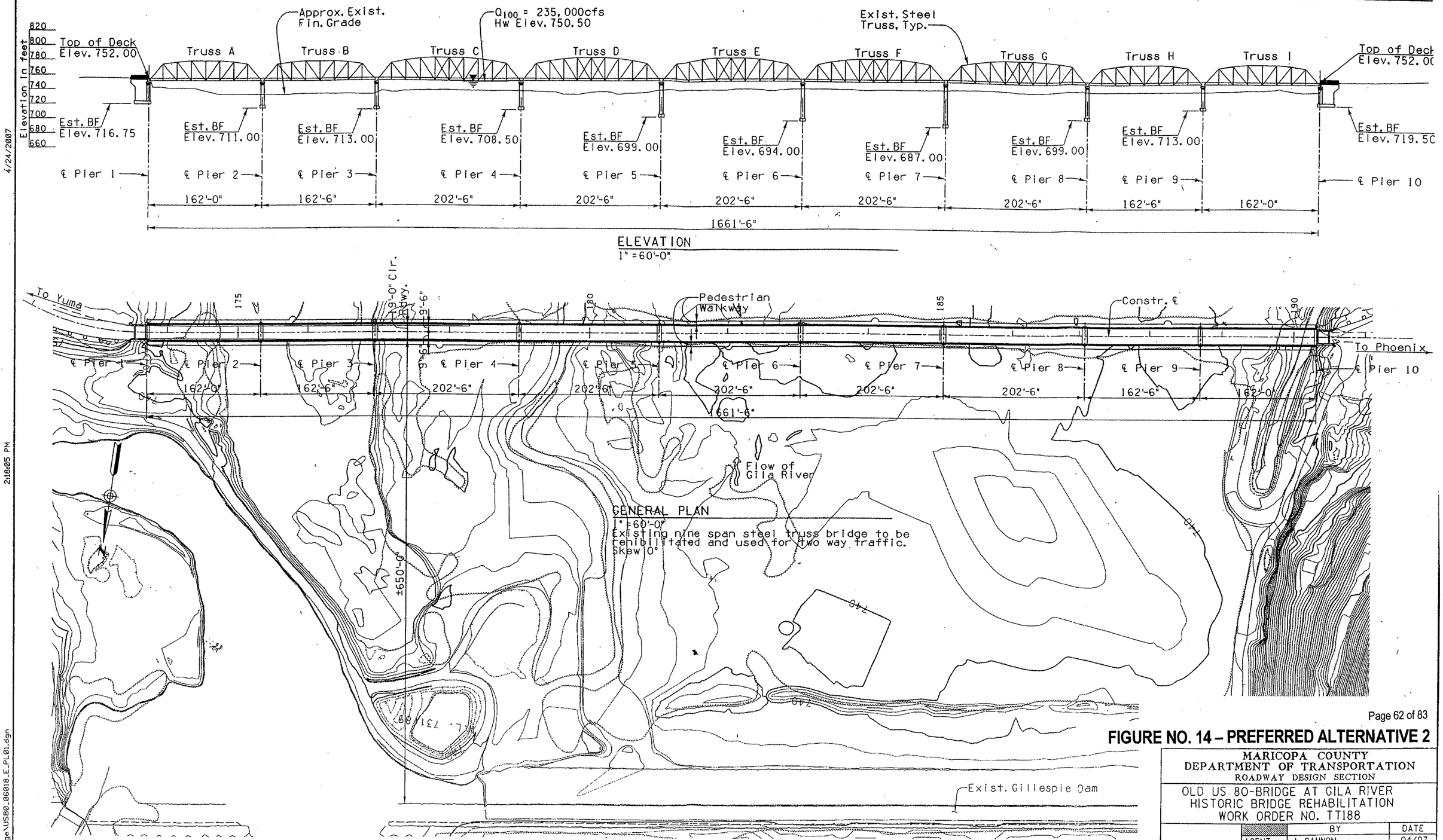
q. Temporary Detour During Construction – The bridge will need to be closed during construction. We propose using SR 85 as the detour route rather than an at-grade crossing of the Gila River. This detour will require extended travel for vehicles using Old US-80 Highway. Since the school buses need to use the bridge during construction, MCDOT should assist the local school district with their travel costs.

4.1 COST ESTIMATE

Preferred Alternative 2 – Rehabilitate Existing Bridge for Two-Way Traffic with New Pedestrian Walkways

Item No.	Item Description	Estimated Construction Cost
a	Concrete Bridge Deck Replacement	$1,431,680.00
b	Repair Truss Bearings	$756,000.00
c	Add Welded Studs to Stringers and Floor Beams	$337,760.00
d	Provide New Bridge Barrier Rail	$566,270.00
e	Concrete Wing Wall Modification	$85,000.00
f	Heat Straighten Bent/Damaged Members	$400,000.00
g	Repaint Bridge (remove all existing paint)	$5,000,000.00
h	Scour Protect Piers 1, 2, 3, 8, 9, & 10)	$1,500,000.00
i	Install Guard Rail (4 corners of bridge)	$35,000.00
j	Protective Beam (each entrance)	$50,000.00
k	Pedestrian Walkway on Each Side of Bridge (includes replacing exterior stringers)	$2,208,500.00
l	Post Bridge & Other Signage	$2,500.00
m	Repave/Regrade Roadway Approaches	$30,000.00
n	Deck Joint Repair	$60,000.00
p	Install Historic Markers	$10,000.00
	Subtotal	$12,472,710.00
	Engineering Design	$500,000.00
	Environmental Mitigation (1%)	$124,727.00
	Miscellaneous Work (5%)	$623,636.00
	Maintenance of Traffic (2%)	$249,454.00
	Mobilization (7%)	$873,090.00
	Survey	$100,000.00
	Contractor Quality Control (2%)	$249,454.00
	Construction Administration during Construction (6%)	$748,363.00
	TOTAL PROJECT COST FOR ALTERNATIVE 2	**$15,941,434.00**

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



NOTE: Elevations listed are survey control NVGD 29 (1929)

FIGURE NO. 14 – PREFERRED ALTERNATIVE 2

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

		BY	DATE
PRELIMINARY NOT FOR CONSTRUCTION	AGENT	J. CANNON	04/07
	DRAWN	DY	04/07
	CHECKED		04/07

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

GENERAL PLAN AND ELEVATION

SHEET OF

4/24/2007

2:18:05 PM

g:\TC06\0018\Bridge\US80_06018_E_PL01.dgn

P60506018

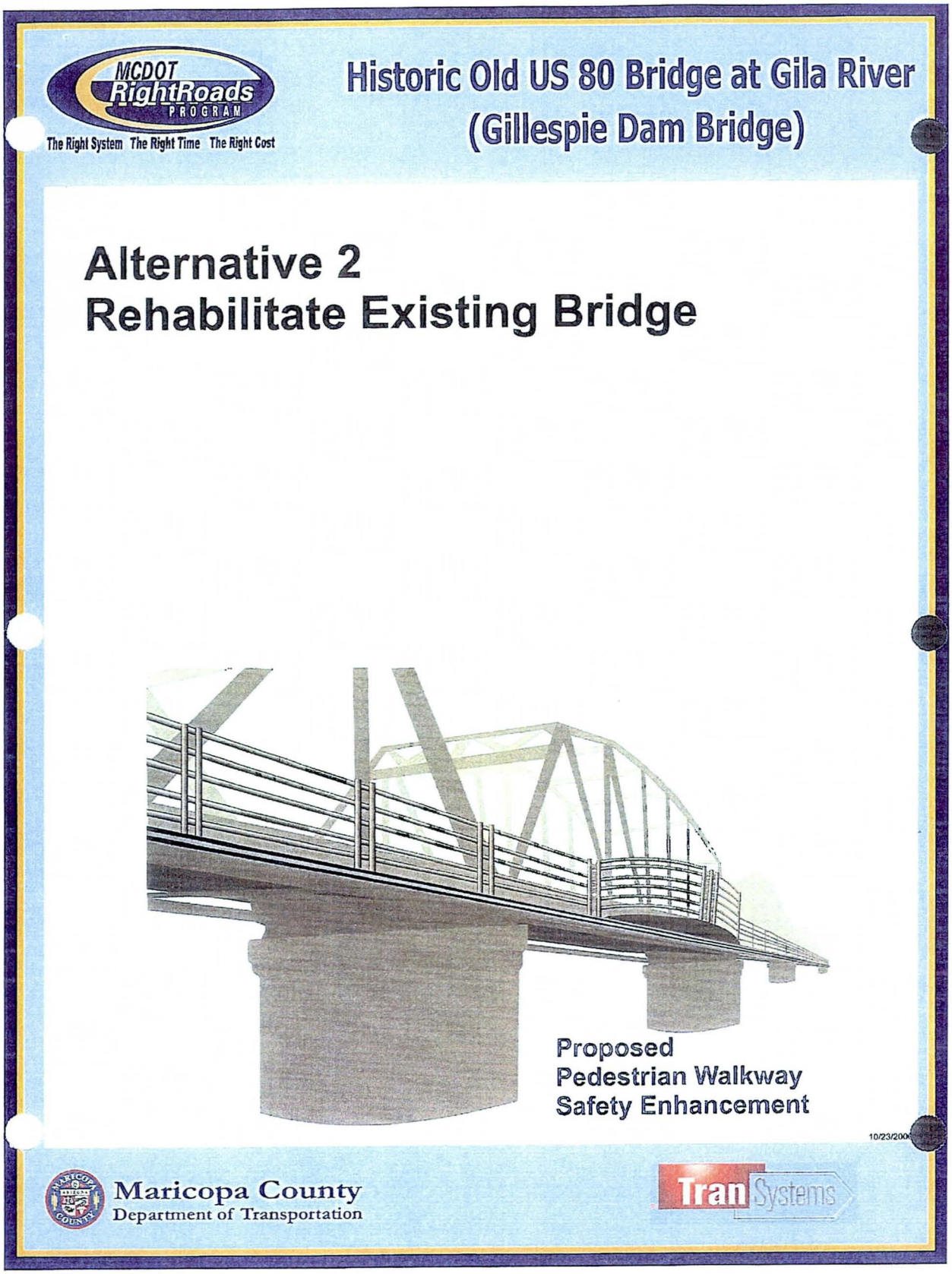


FIGURE NO. 15 – PREFERRED ALTERNATIVE 2

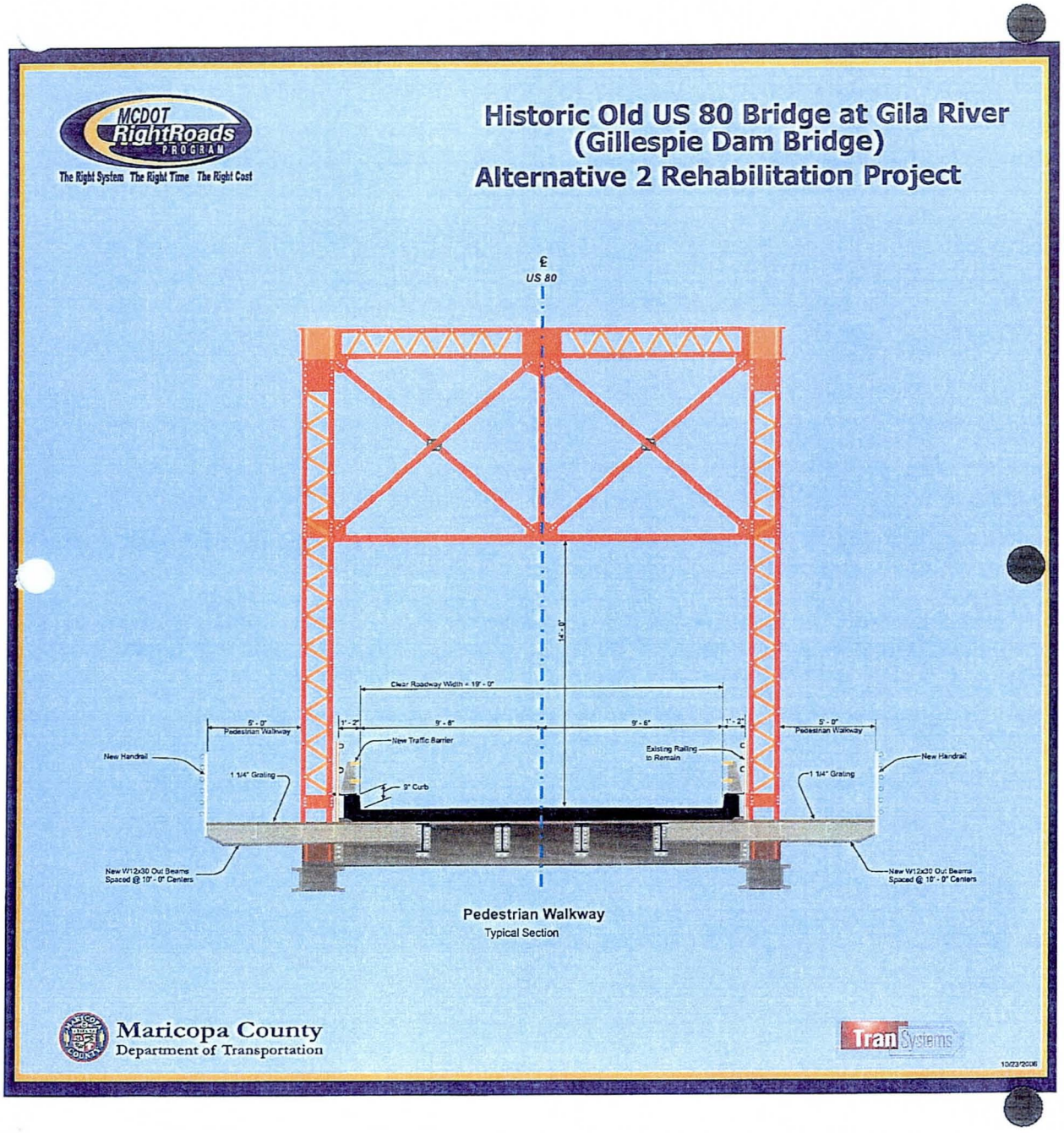


FIGURE NO. 16 – PREFERRED ALTERNATIVE 2

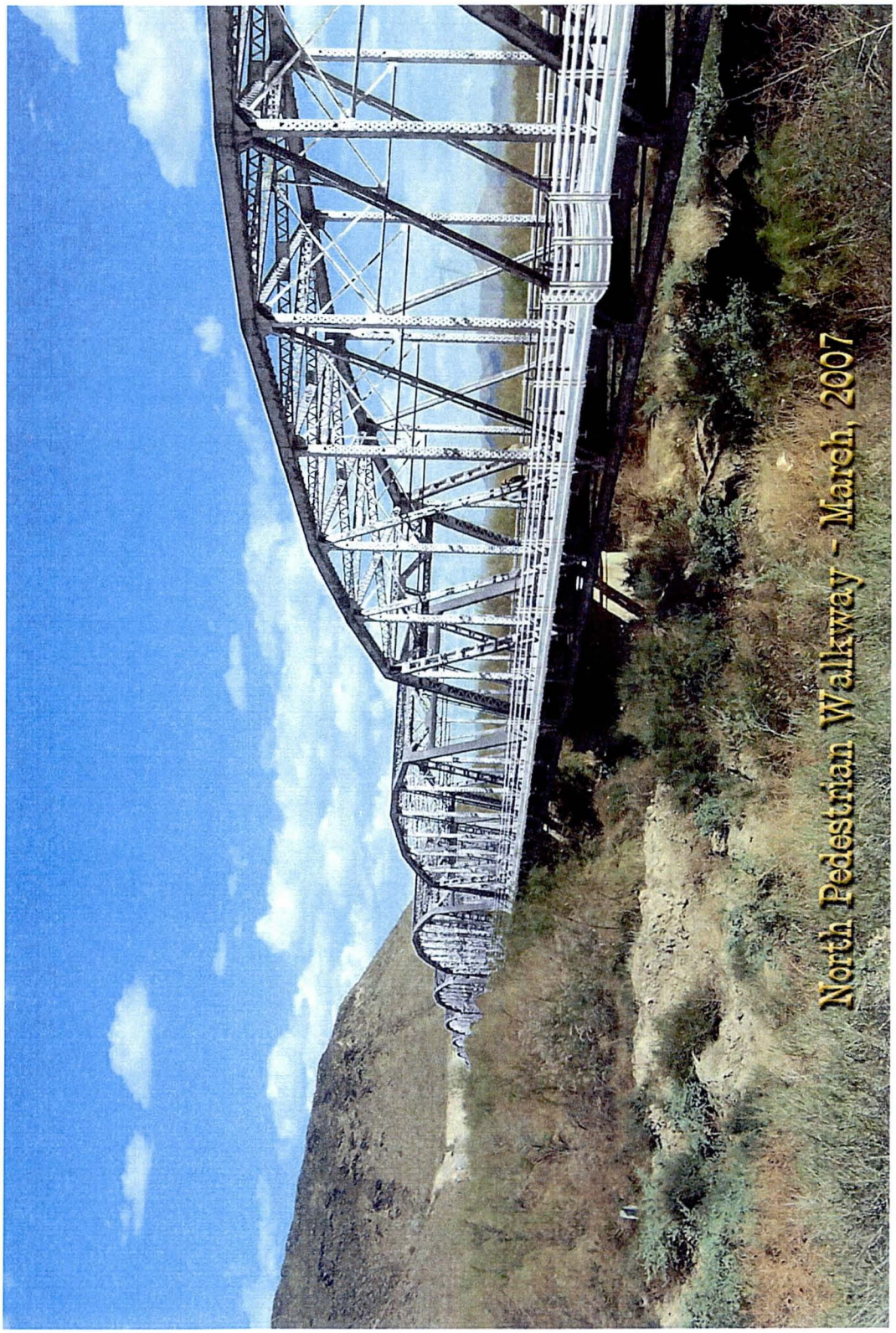


FIGURE NO. 17 – ILLUSTRATION OF ALTERNATIVE 2 WITH NEW PEDESTRIAN WALKWAY ADDED TO THE SIDES OF THE TRUSSES

4.2 PHASED CONSTRUCTION COST ESTIMATE FOR ALTERNATIVE 2

Depending on available funds, this project may need to be built in different phases. The following is a cost breakdown for four construction phases.

PHASE 1

Item No.	Item Description	Estimated Construction Cost
a	Concrete Bridge Deck Replacement	$1,431,680.00
b	Repair Truss Bearings	$756,000.00
c	Add Welded Studs to Stringers & Floor Beams	$337,760.00
d	Provide New Bridge Traffic Barrier Rail	$566,270.00
e	Concrete Wing Wall Modification	$85,000.00
f	Heat Straighten Bent/Damaged Members	$400,000.00
g	Install Guard Rail (4 corners of bridge)	$35,000.00
h	Protective Beam (each entrance)	$50,000.00
i	Post Bridge & Other Signage	$2,500.00
j	Repave/Regrade Roadway Approaches	$30,000.00
k	Deck Joint Repair	$60,000.00
l	Install Historic Markers	$10,000.00
	Subtotal	$3,764,210.00
	Miscellaneous Work, Maintenance of Traffic, Mobilization, Survey & Contractor Quality Control (17%)	$639,916.00
	Design Engineering	$350,000.00
	Construction Engineering (10%)	$376,421.00
	TOTAL PROJECT COST FOR ALTERNATIVE 2 - PHASE 1	**$5,130,547.00**
		USE $5.1 MILLION

PHASE 2

Item No.	Item Description	Estimated Construction Cost
h	Repaint Bridge (remove all existing paint)	$5,000,000.00
	Subtotal	$5,000,000.00
	Miscellaneous Work, Maintenance of Traffic, Mobilization, Survey & Contractor Quality Control (17%)	$850,000.00
	Design Engineering	$100,000.00
	Construction Engineering	$200,000.00
	TOTAL PROJECT COST FOR ALTERNATIVE 2- PHASE 2	**$6,150,000.00**
		USE $6.15 MILLION

PHASE 3

Item No.	Item Description	Estimated Construction Cost
	Pedestrian Walkway Each Side of Bridge (includes replacing exterior stringers)	$2,208,500.00
	Subtotal	$2,208,500.00
	Miscellaneous Work, Maintenance of Traffic, Mobilization, Survey & Contractor Quality Control (17%)	$375,445.00
	Design Engineering (5%)	$110,425.00
	Construction Engineering (10%)	$220,850.00
	TOTAL PROJECT COST FOR ALTERNATIVE 2 - PHASE 3	**$2,915,220.00**
		USE $3.0 MILLION

PHASE 4

Item No.	Item Description	Estimated Construction Cost
	Foundation Stabilization (Piers 1, 2, 3, 8, 9, & 10)	$1,500,000.00
	Subtotal	$1,500,000.00
	Miscellaneous Work, Maintenance of Traffic, Mobilization, Survey & Contractor Quality Control (17%)	$255,000.00
	Design Engineering	$50,000.00
	Construction Engineering	$80,000.00
	TOTAL PROJECT COST FOR ALTERNATIVE 2 - PHASE 4	**$1,885,000.00**
		USE $1.9 MILLION

5.0 ALTERNATIVE 4 – FUTURE RECOMMENDED ALTERNATIVE

Alternative 4 New Bridge Downstream from the Existing Bridge is the preferred future alternative and will be needed when future development of the area requires a new bridge to increase the capacity of the crossing of the Gila River. See Figure 18 on page 70.

This alternative would include building a new bridge approximately 1,000 feet downstream of the existing bridge and would be 14 spans and 1,800 feet long to span the floodway of the Gila River. For additional information and discussion of this alternative see pages 58-59 and Appendix B for plan sheets.

The width of the bridge is shown as a five-lane bridge, but two separate phased structures of smaller widths could be substituted for the single five-lane bridge as described in the traffic analysis section of this report. See pages 31 through 37 for additional information on phasing for the new bridge. The ultimate roadway and bridge sections are shown on Figure 18 on page 70. For a detailed cost estimate of the single, five-lane bridge see Section 5.1.

5.1 ALTERNATIVE 4 – FUTURE RECOMMENDED ALTERNATIVE COST ESTIMATE

	ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
1	REMOVAL OF ASHPALTIC CONCRETE	SQ.YD	19,094	$5.00	$95,470.00
2	REMOVAL OF EXISTING GUARDRAIL	L.FT	270	$3.00	$810.00
3	SAWCUTTING	L.FT	451	$3.00	$1,353.00
4	ROADWAY EXCAVATION	CU.YD	256,541	$25.00	$6,413,525.00
5	DYKE (BANK PROTECTION EMBANKMENT)	CU.YD	32,100	$7.00	$224,700.00
6	BORROW IN PLACE	CU.YD	53,913	$20.00	$1,078,260.00
7	AGGREGATE BASE, CLASS 2	CU.YD	12,933	$35.00	$452,655.00
8	ASPHALTIC CONCRETE (MISCELLANEOUS STRUCTURAL)	TON	26,189	$50.00	$1,309,450.00
9	MISCELLANEOUS WORK (SIGNS)	L.SUM	1	$5,000.00	$5,000.00
10	FURNISH AND INSTALL TEMPORARY TRAFFIC CONTROL DEVICES	L.SUM	1	$5,000.00	$5,000.00
11	PAVEMENT MARKING	L.FT	80,360	$0.50	$40,180.00
12	SEEDING (CLASS II)	ACRE	24	$750.00	$18,000.00
13	SOIL CEMENT BANK PROTECTION	CU.YD	11,724	$120.00	$1,406,880.00
14	BOX CULVERT AND WING WALL	L.SUM	1	$125,000.00	$125,000.00
15	RECONSTRUCT IRRIGATION DITCH	L.SUM	1	$20,000.00	$20,000.00
16	RECONSTRUCTION EXISTING CROSS DRAINAGE STRUCTURES	L.SUM	1	$10,000.00	$10,000.00
				TOTAL COST OF ROADWAY ITEMS ABOVE	**$11,206,283.00**

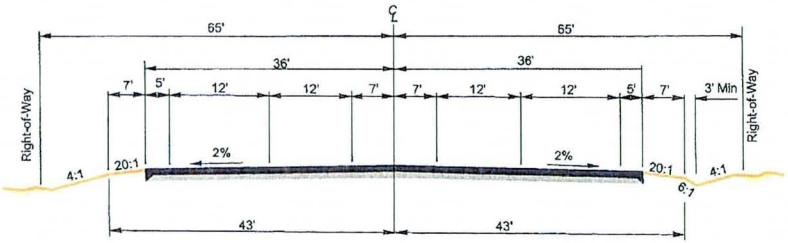
17	STRUCTURAL EXCAVATION	CU.YD	475	$10.00	$4,750.00
18	STRUCTURE BACKFILL	CU.YD	4,700	$20.00	$94,000.00
19	STRUCTURAL CONCRETE (MAG CLASS A) (F'C=3,000 PSI)	CU.YD	0	$220.00	$0.00
20	STRUCTURAL CONCRETE (MAG CLASS AA) (F'C=4,000 PSI)	CU.YD	248	$450.00	$111,600.00
21	STRUCTURAL CONCRETE (MAG CLASS AA) (F'C=4,000 PSI)	CU.YD	8,161	$500.00	$4,080,500.00
22	F-SHAPE BRIDGE CONCRETE BARRIER (32 INCH)	L.FT	3,724	$70.00	$260,680.00
23	CONCRETE BARRIER AND TRANSITION (32 INCH)	EACH	4	$4,000.00	$16,000.00
24	BARRIER RAILING	L.FT	3,600	$100.00	$360,000.00
25	DECK JOINT ASSEMBLY (STRIP SEAL)	L.FT	348	$200.00	$69,600.00
26	APPROACH SLAB (SD 2.01)	SQ. FT.	1,172	$12.00	$14,064.00
27	PRECAST, P/S MEMBER (AASHTO TYPE VI MOD. GR.)	L.FT	19,520	$175.00	$3,416,000.00
28	RESTRAINERS (FIXED)	L.FT	22	$100.00	$2,200.00

	ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
29	RESTRAINERS (EXPANSION)	EACH	88	$200.00	$17,600.00
30	BEARING	EACH	88	$1,400.00	$123,200.00
31	REINFORCING STEEL	LB.	3,460,915	$0.85	$2,941,778.00
32	DRILLED SHAFT (60")	L.FT	480	$900.00	$432,000.00
33	DRILLED SHAFT (120")	L.FT	2,160	$2,000.00	$4,320,000.00
34	**LUMP SUM STRUCTURE (TOTAL OF PRECEEDING STRUCTURE ITEMS)**	**L.SUM**			**$16,263,972.00**

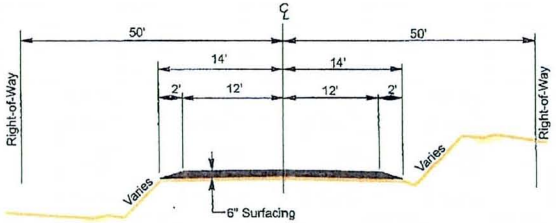
(15% contingency)	$2,439,596.00
TOTAL COST OF BRIDGE ITEMS ABOVE	**$18,703,568.00**
SUBTOTAL BRIDGE AND ROADWAY COST	**$29,909,851.00**
ENVIRONMENTAL PROTECTION DURING CONSTRUCTION @ 1%	$299,099.00
MISCELLANEOUS WORK @ 5%	$1,495,493.00
MISCELLANEOUS WORK AND PROTECTION OF TRAFFIC @ 2%	$598,197.00
EROSION CONTROL @ 1%	$299,099.00
MOBILIZATION @ 7%	$2,093,690.00
CONTRACTOR QUALITY CONTROL @ 2%	$598,197.00
CONSTRUCTION SURVEYING & LAYOUT @ 2%	$598,197.00
Subtotal Construction Cost	$35,891,823.00
Design Engineering at 10%	$3,589,182.30
Construction Engineering 15%	$5,383,773.45
Right-of-Way	$439,100.00
Utility Relocations	$10,000.00
TOTAL PROJECT COST	**$45,313,878.75**

MCDOT RightRoads PROGRAM
The Right System The Right Time The Right Cost

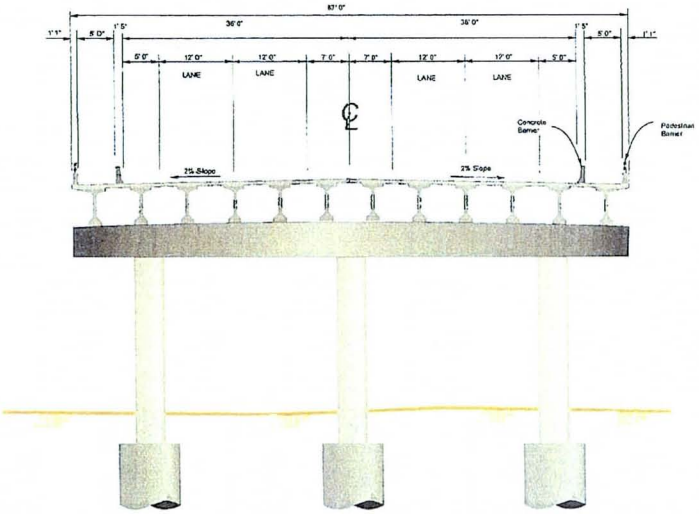
Historic Old US 80 Bridge at Gila River (Gillespie Dam Bridge) Design Concept Report (DCR)



Ultimate (Year 2026) Alternative 3&4 Roadway Section



Existing Roadway Section



Ultimate (Year 2026) Alternative 3&4 Bridge Crossing

Maricopa County
Department of Transportation

TranSystems

12/06/2005

FIGURE NO. 18

6.1 DESIGN STANDARDS

Design standards and safety requirements for the roadway and structures will comply with the following:

MAG Uniform Standard Specifications for Public Works Construction and Maricopa County Department of Transportation Supplement to MAG Standard Specifications

MAG Standard Details for Public Works Construction

Maricopa County Department of Transportation Roadway Design Manual (November 1993; Revised April 2004)

Drainage Design Manual for Maricopa County, Volumes I-III by Flood Control District (FCD) of Maricopa County (Revised January 1996)

Maricopa County Department of Transportation Pavement Marking Manual (2005)

Maricopa County Department of Transportation Sign Manual

Geometric Design of Highways and Streets, AASHTO (2004)

AASHTO LRFD Bridge Design Specifications (3rd Edition, 2004 with Interim 2005)

AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFD) of Highway Bridges (October 2003)

AASHTO Manual for Subsurface Investigation (for geotechnical investigation and foundations)

6.2 ROADWAY

The following roadway requirements and dimensions will be used:

ITEM	CRITERIA
Functional Classification	Old US-80 is a Rural Minor Arterial
Level of Service	Desired LOS of C
Design Year	2030
Design Vehicle	WB-67
Design Speed	No Action: Posted Speed 35 mph
	Alternative 2: Posted Speed 35 mph
	Alternative 3: Posted Speed 35 mph
	Alternative 4: 55 mph (MCDOT Roadway Design Manual, Rural Minor Arterial, rolling terrain pg 5-16)
Posted Speed	35 mph (500 ft from bridge), 25 mph (250 ft from bridge)
Maximum Super-Elevation	8% (MCDOT Roadway Design Manual pg 5-19)
Minimum Radii	No Action: N/A
	Alternative 2: R_{min} = 350 ft (AASHTO exhibit 3-23, V=35mph, e_{max} = 8%)
	Alternative 3: R_{min} = 350 ft (AASHTO exhibit 3-23, V=35mph, e_{max} = 8%)
	Alternative 4: R_{min} = 965 ft (AASHTO exhibit 3-23, V=55mph, e_{max} = 8%)
Lane Widths	12 ft through lanes (MCDOT Roadway Design Manual pg 5-1, 5-4)

ITEM	CRITERIA
Shoulder Width	5 ft paved shoulder, 7 ft unpaved shoulder (MCDOT Roadway Design Manual pg 5-1, 5-4)
Median	N/A
Roadway Cross-Slope	2% (MCDOT Roadway Design Manual pg 5-4)
Shoulder Cross-Slope	2% paved shoulder, 20:1 unpaved shoulder (MCDOT Design Manual pg 5-4)
Embankment Cut/Fill Slopes	4:1 maximum (MCDOT Roadway Design Manual pg 5-4)
Clear Zone	Based on 35 and 55 mph and an ADT over 6000
Minimum Stopping Sight Distance	No Action: N/A
	Alternative 2: SSD_{min} = 250 ft (MCDOT Roadway Design Manual, V=35 mph pg 5-39)
	Alternative 3: SSD_{min} = 250 ft (MCDOT Roadway Design Manual, V=35 mph pg 5-39)
	Alternative 4: SSD_{min} = 495 ft (MCDOT Roadway Design Manual, V=55 mph pg 5-39)
Minimum Passing Sight Distance	No Action:
	Alternative 2: PSD_{min} = 1,280 ft (MCDOT Roadway Design Manual, V=35 mph pg 5-39)
	Alternative 3: PSD_{min} = 1,280 ft (MCDOT Roadway Design Manual, V=35 mph pg 5-39)
	Alternative 4: PSD_{min} = 1,985 ft (MCDOT Roadway Design Manual, V=55 mph pg 5-39)
Sidewalks	MAG Standard 5 foot or greater in width, with minimum clear width of 4 feet, and ADA compliant. (MCDOT Roadway Design Manual, pg 5-50)
Maximum Longitudinal Grade	No Action: N/A
	Alternative 2: Match existing maximum longitudinal grade
	Alternative 3: Match existing maximum longitudinal grade
	Alternative 4: Maximum longitudinal grade = 5% (MCDOT Roadway Design Manual, V=55mph, rolling terrain pg 5-32)
Minimum Longitudinal Grade	+/- 0.25% desirable, +/- 0.15% in special cases (MCDOT Roadway Design Manual pg 5-31)
Minimum K-Factors for Sag and Crest Vertical Curves	No Action: N/A
	Alternative 2: K_{min} = 29 (MCDOT Roadway Design Manual, V=35 mph pg 5-34)
	Alternative 3: K_{min} = 29 (MCDOT Roadway Design Manual, V=35 mph pg 5-34)
	Alternative 4: K_{min} = 114 (MCDOT Roadway Design Manual, V=55 mph pg 5-34)
Curb and Gutter Types	Where required, curb and gutter shall be constructed in accordance with MAG Standard Detail 220, with 'H' normally 6 inches. Curb shall only be used where posted speed is 45 mph or less.
Curb Return Radii	Shall be in accordance with Table 6.1 of the MCDOT Roadway Design Manual, pg 6-1.

ITEM	CRITERIA
Pavement Tapers	When a proposed roadway is to connect to an existing roadway of smaller width the following guidelines should be followed. (a) $L=WS^2/60$ (design speed less than or equal to 40 mph) (b) L=WS (design speed greater than 40 mph), where: W = Offset from drivable through lane in feet S = Design Speed L = Taper Length (c) Use 8:1 Taper from narrow section to wide section (d) (MCDOT Roadway Design Manual, pg 5-41)
Roadside Barrier Flare Rates	Flare rates for roadside barrier for each alternative are as follows: Alternative 2: 16:1 inside shy line and 10:1 outside of shy line Alternative 3: 16:1 inside shy line and 10:1 outside of shy line Alternative 4: 24:1 inside shy line and 16:1 outside of shy line (AASHTO Roadside Design Guide, Table 5-7, pg 5-32)
Guardrails	Guardrail and Concrete Barrier to be constructed per MCDOT Standard Details as shown in the Maricopa County Supplement to the MAG Uniform Standard Details. Length and placement of guardrail shall be determined using AASHTO's "Roadway Design Guide." (MCDOT Roadway Design Manual, pg 5-47)

6.3 STRUCTURE GEOMETRICS (EXISTING BRIDGE – ALTERNATIVE 2)

The existing structure has the following geometric dimensions:

1. The length of the existing bridge is 1665'-8" – back to back of abutments.
2. The bridge clear roadway width is 19'-0" for two 9'-6" travel lanes.
3. The bridge deck elevation is 752.0 (as shown on the as-built bridge plans using NGVD 29).
4. The 100-year high water elevation is 750.50.
5. The existing bridge will be posted for maximum vertical clearance of 13'-6".
6. The scour elevation is 695.9 feet.

6.4 RIGHT-OF-WAY

Existing Right-of-Way:

According to the Maricopa County Assessors Office, the entire area is zoned Rural-190 (1 (one) residential unit per 190,000 SF (4.36 AC) minimum) and the Zoning Code is R-190.The existing right-of-way width at the existing bridge is 50 feet on each side of the bridge for a total width of 100'-0".

New Right-of-Way:

Alternative 3 – New Bridge Parallel and Adjacent to the Existing Bridge (35 MPH)

Property Ownership	Area of Property	
	(SF)	(Acres)
El Paso Natural Gas	740	0.02
Gila Ventures LLC	28,780	0.66
Gillespie Dam Investments LLC	12,188	0.28
Paloma Investment Limited Partnership	285,565	6.56
Total	327,273	7.51
Total Cost (Estimated Value: $10,000 per Acre)		$75,100

El Paso Natural Gas: Parcel 401-61-930
Gila Ventures LLC: Parcels 401-62-010-C, 401-62-010-D
Gillespie Dam Investments LLC: Parcel 401-61-01B
Paloma Investment Limited Partnership: Parcels 401-61-019, 401-61-018-C, 401-61-017-L, 401-61-017-H

Alternative 4 – New Bridge Downstream from Existing Bridge (55 MPH)

Property Ownership	Area of Property	
	(SF)	(Acres)
El Paso Natural Gas	478,959	11.00
Gila Ventures LLC	282,545	6.49
Gillespie Dam Investments LLC	38,160	0.88
Jagow Emile	397,183	9.12
Long Melissa / Rania	223,729	5.14
Paloma Investment Limited Partnership	457,332	10.50
Sutter Ricky L / Karen K	34,676	0.80
Total	1,912,584	43.91
Total Cost (Estimated Value: $10,000 per Acre)		$439,100

El Paso Natural Gas: Parcels 401-61-931, 401-61-014-B, 401-013-B, 401-61-930
Gila Ventures LLC: Parcels 401-62-010-D, 401-62-010-C
Gillespie Dam Investments LLC: Parcel 401-61-01B
Jagow Emile: Parcel 401-61-011
Long Mellissa / Rania: Parcel 401-61-012-B
Paloma Investment Limited Partnership: Parcels 401-61-019, 401-61-018-C
Sutter Ricky L / Karen K: Parcel 401-62-007

6.5 UTILITIES

Utility companies having facilities in the study area of the project have been notified of the study and have been requested to provide information regarding their facilities. Known utilities in the area include Qwest, El Paso Natural Gas, SRP Power Transmission lines and irrigation. A list of the utility companies and their contacts are listed below.

Name	Organization	Mailing Address	Phone
Matt Phillips	Qwest	6350 S. Maple Ave. Room #125 Tempe, AZ. 85283	480-234-2032
Jeff Meaf	SWG (Natural)	9th S. 43rd Ave. Phoenix, AZ. 85009	602-763-4000
Dennis Segars	El Paso Natural Gas	7815 S. 48 St. Phoenix, AZ. 85044	602-438-4224
Greg Wilson	SRP	PO BOX 52025 Main Station XCT315 Phoenix, AZ. 85072-2025	602-390-7538

Qwest facilities located on the south side of the bridge may need to be relocated on the bridge to accommodate the addition of the pedestrian walkway.

6.6 TEMPORARY DETOUR DURING CONSTRUCTION

The Old US-80 Bridge will need to be closed for about ten months during the rehabilitation of the existing bridge. Three detour alternatives have been evaluated by length of detour, environmental impact, and impacts to school and emergency vehicles.

6.6.1 Length of Detour (See Figure 18)

Enterprise Road Detour – The detour is 13 miles long compared to 12 miles using Old US-80. While this detour is similar in length to the existing route, the condition of the roadway (alignment and dirt pavement material) is substantially different. The time to drive the detour due to this is approximately 26 minutes compared to 13 minutes using US80.

Gila River Crossing Detour – The detour would be located slightly downstream and parallel to the existing bridge. This detour would reestablish an old crossing that has been obliterated through the river but can still be located on the river banks. This detour is similar in length to the existing route.

SR 85 Detour – The detour is 31 miles long compared to 4 miles using Old US-80. The time to drive the detour due to its length is approximately 35 minutes compared to 4 minutes using US-80. While this detour appears to be longer it would not be as long for most of the drivers since their origin or destination point is not in the immediate vicinity of the bridge.

6.6.2 Cost of Detour

Enterprise Road Detour – The detour is located on private property. In addition to the easement cost it is anticipated that the roadway surface would have to be upgraded. Utilizing the pavement structure for a typical PM 10 roadway the cost to upgrade the roadway will be over $400,000 per mile.

Gila River Crossing Detour – Reestablishing the existing crossing will require the placement of fill, placement of storm drainage pipe to handle the low flow conditions and construction of a paved surface. It is anticipated that these improvements would have to be removed at the end of the project. This detour probably could not be constructed totally within the existing 50-foot half right-of-way, so an additional easement would be needed for this alternative. There is also a buried gas line in the proximity of the proposed detour road.

SR 85 Detour – While no roadway improvements are anticipated for this detour alignment there would be substantial cost for the detour signing for the duration of the closure over the 31 mile detour.

Old US-80 Gila River Bridge (Gillespie Dam Bridge)
Detour Alternatives

1. *Enterprise Road Detour (13 miles)*
2. *Gila River Crossing Detour (0.25 miles)*
3. *SR 85 Detour (31 miles)*

North
NTS

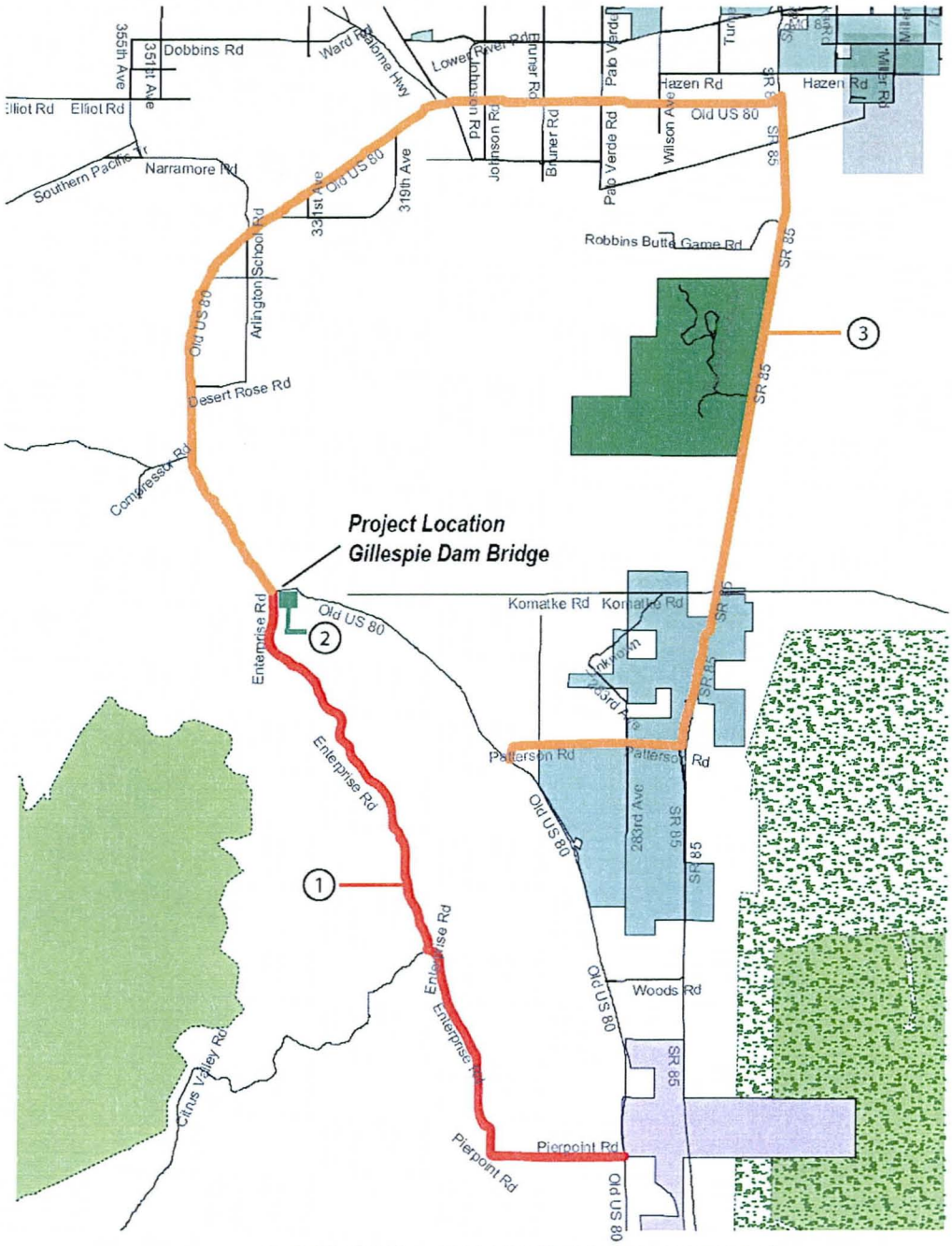


FIGURE 19 – DETOUR ALTERNATIVES

6.6.3 Environmental Impacts

Enterprise Road Detour – The detour would utilize an existing private road which had been used recently to transport generators to the Palo Verde Nuclear Power Plant. While it is not anticipated that the detour would provide biological or historic impacts, it does require an at-grade crossing of the Gila River which will have environmental impacts. Much like the Gila River Crossing Detour, a permit might be needed for this alternative at the Pierpoint Road crossing of the Gila River. Pierpoint Road as is does not seem to be adequate for the anticipated traffic volume and type. Work (grading, asphalt, culverts) may be needed to provide a suitable roadway across the River. Regardless, any 404 work needed for a detour route can be added to the anticipated permit for the overall US-80 Bridge construction.

Gila River Crossing Detour – Similar to the Enterprise Road Detour, this alternative requires reestablishing the existing crossing will require the removal of existing vegetation, primarily the invasive Tamarisk. There are potential biological impacts due to habitat loss for the Southwest Willow Flycatcher. The construction of the crossing will require consultation with the U.S. Fish and Wildlife Service and an individual 404 permit. In addition, this reach of the Gila River is an impaired water which would have to be addressed in a 402 permit.

SR 85 Detour – It is not anticipated that the detour would have any significant impacts.

6.6.4 Impact to School and Emergency Vehicles

Discussions with the Arlington Elementary School and Buckeye Union High School Districts indicated that approximately 15 students are picked up on the east side of the bridge and transported across the bridge. In addition, several other students are picked up at the bus stop at the intersection of Old US-80 and Enterprise Road (immediately west of the bridge). They indicated that they operate one bus and that a major detour would make it difficult to impossible to service those students during the closure of the bridge. In addition, emergency services from Gila Bend and local emergency services such as Buckeye Valley Fire District could be impacted by the closure.

Enterprise Road Detour – The detour would add approximately one hour to the travel time to pick up the students located just south of the bridge. The detour would also have a similar impact to emergency services.

Gila River Crossing Detour – The detour would have minimal to no impact on the school or emergency services.

SR 85 Detour – Due to the detour length and location this detour would have impacts similar to the Enterprise Road Detour.

Based on this review the following table was developed to compare the alternatives.

TABLE NO. 12 – OLD US-80 DETOUR ALTERNATIVE COMPARISON

Detour Number	Detour Name	Length of Detour	Cost of Detour	Environmental Impact	Impact to School and Emergency Vehicles
1	Enterprise Road Detour	◒	○	○	◒
2	Gila River Crossing Detour	●	○	○	◒
3	SR 85 Detour	○	●	●	○

● **Best** ◒ **Average** ○ **Worst**

6.6.5 Conclusion

In order to make the necessary repairs to rehabilitate the existing bridge, it will be necessary to close the bridge for about ten months. Both the Enterprise Road Detour and the Gila River Crossing Detour will be very costly and have extensive environmental impacts and in the event of flood event will need to be closed to traffic. The SR 85 Detour would be inconvenient, but based on the amount of traffic using the existing bridge; this alternative would have the lowest cost and minimal environmental impacts (See Table No. 12).

6.6.6 Recommendation

It is recommended that the SR 85 Detour be used during construction of this project and that MCDOT provide financial assistance to the school district and notify users of the bridge through public meetings, other notifications, and provide adequate signage for the detour. This detour would be the least costly, have the least environmental impact, and be the most practical for this project. The major disadvantage is that the detour would sever the Old US-80 Route for ten months and would cause some delays and inconveniences to local users. During the final design, the additional route should be evaluated.

7.0 SOCIAL, ECONOMIC, & ENVIRONMENTAL CONSIDERATIONS

7.1 SOCIAL & ECONOMIC CONSIDERATIONS

The project area falls within the jurisdictional boundaries of Maricopa County. No residential or commercial properties are located within or immediately adjacent to the project limits. However, the bridge is an important transportation link to the surrounding area. The impacts of project construction and planned bridge improvements to existing access for the surrounding community will be analyzed during the environmental analysis. No existing low income or minority populations are anticipated to be disproportionately impacted by the proposed improvements. Access restrictions, displacement, pedestrian/bicycle/vehicular traffic impacts, as well as possible effects on minority, low income, elderly, or female head of household populations will be identified during the environmental analysis and will be documented in the Categorical Exclusion.

7.2 HISTORICAL SIGNIFICANCE OF BRIDGE

Prior to 1927, traffic on the Ocean-to-Ocean Highway at this point was often halted by flooding on the Gila River. The Old US-80 Gila River Bridge was thus strategically important to Arizona transportation in that it finally allowed all-weather travel over this vital transcontinental route. Technologically, the bridge is noteworthy as one of the longest vehicular structures in the state. Arizona's longest bridges have historically been built over the Gila. In fact, more effort and money was spent building – and maintaining – bridges over the Gila than any other river in the state.

The Arizona Highway Department (AHD) began planning for a concrete girder bridge over the Gila River at this point even before Frank Gillespie completed his dam west of Phoenix in 1921. In the interim, a novel crossing was devised in which automobiles were pulled by horse team across an apron poured at the dam's downstream toe. Mindful of the problems encountered at other large-scale concrete bridges over the Gila, AHD in April 1925 contracted for soundings and borings and then hired a consulting engineer to help locate and design the structure. For its superstructure, the engineers dropped the concrete bridge design and instead delineated a series of rigid-connected through trusses weighing a total of 2.3 million pounds. The trusses featured Camelback web configurations, with built-up box beams for the upper and low chords with spans up to 202'-6". These were supported by solid concrete walls and spread footings placed on bedrock-basalt agglomerate, with the deepest pier extending over 40 feet below the riverbed. The 19-foot wide concrete deck is supported on steel stringers and was bounded on the sides by steel pipe railings. See Photos 49-52 on pages 81 and 82.

In January 1926, eleven contractors submitted competitive bids for the construction. The highway department let the contract to the lowest bidder, Lee Moor Construction Company of El Paso, Texas. Moor's crew began work on the piers immediately and completed the immense structure in July 1926. Total cost: $320,000. The Gillespie Dam Bridge carried mainline traffic on the Ocean-to-Ocean Highway (US-80) until a route realignment in 1956 moved the road. At that time the bridge reverted to county road status, under which it now functions today in unaltered condition.

The bridge is listed on the National Register of Historic Places for its technological and historic significance. It is an outstanding multiple span truss bridge and a major crossing on a important highway.

7.3 ENVIRONMENTAL CONSIDERATIONS

The following information is a summary of the environmental considerations associated with the proposed project. Federal funds may be used for the project. The Categorical Exclusion to be prepared for the project will disclose the anticipated project impacts and applicable mitigation measures.

7.3.1 Biological Resources

A Biological Evaluation will be completed for this project. The project area does not occur within any critical habitat, designated or proposed, under the Endangered Species Act. However, the bridge is located within cactus ferruginous pygmy-owl (CFPO) Zone 3, which is an area of historic range of the pygmy-owl but has a low potential for occupancy[1]. The area also contains suitable habitat for the CFPO, Yuma clapper rail, and southwestern willow flycatcher (WIFL). According to the Arizona Game and Fish Department (AGFD), surveys for the CFPO were conducted within the project area from March 2000 to April 2002 and no CFPOs were detected. AGFD detected Yuma clapper rails above Gillespie Dam during surveys in 2006 and previous years. Yuma clapper rail surveys south of the dam were completed in 2006 and none were detected. AGFD surveys in 1994, 1996, and 1999 detected no WIFLs. However, in 2006 a transient male was identified during the field season but was not detected again in subsequent surveys. A Biological Evaluation will be prepared to analyze the potential effects to the CFPO, Yuma clapper rail and WIFL and will also investigate the need for avoidance or mitigation measures.

7.3.2 Floodplain Encroachment

A review of the Federal Emergency Management Agency Flood Insurance Rate Map indicates that the project is within a 100-year floodplain. Impacts on floodplains typically occur when the topography of the project area is substantially modified by either placement or removal of materials within the floodplain. The potential for the project to affect the 100-year floodplain will be determined during hydrological analysis associated with the design process and will be incorporated into the Categorical Exclusion.

[1] CFPO was de-listed on May 15, 2006, but the de-listing is the subject of a current lawsuit and could be re-listed in the future.

7.3.3 Clean Water Act Section 404 Permit Requirements

A Jurisdictional Delineation is currently being prepared to identify any Waters of the United States (WOUS) under the jurisdiction of the US Army Corps of Engineers (COE). The Gila River and tributaries to the river are recommended to be WOUS under the jurisdiction of the COE. Additionally, a potential jurisdictional wetland has also been identified in the project vicinity. If the COE agrees that there are WOUS/wetlands in the project area that will be affected by construction, a Clean Water Act Section 404 permit will be required. Mitigation requirements and/or in lieu fees to compensate for the loss of WOUS/wetland areas will be determined during the Section 404 permit application process.

7.3.4 Wetland/Riparian Areas

Wetlands and riparian areas are present in the project area. The project's impact to these resources, as well as any required mitigation, will be determined during project specific environmental activities.

7.3.5 Cultural Resources

A Class III survey of a large parcel defined as the area of potential effect (APE) centered at the Gillespie Dam Bridge, which included MCDOT right-of-way along Old US-80, was conducted for a proposed bridge repair or construction project. Because of several historical structures in the area, including the US-80 Bridge – which is listed on the National Register of Historic Places (NRHP), a historical context overview was prepared for the project area. Sites identified in the project area include the bridge, the historic US-80 highway, Gillespie Dam and associated features/artifacts, and prehistoric sites. A determination of impacts to cultural resources, and any applicable mitigation requirements, will be completed during the Section 106 process associated with the environmental investigations for the project.

7.3.6 Section 4(f) Resources

The NRHP-listed bridge, as well as Old US-80, the Gillespie Dam, and other historic features in the project area are, or may be, considered Section 4(f) properties. Impacts to Section 4(f) properties, alternatives for avoidance of these resources, measures to minimize harm, and the applicability of programmatic evaluations will be evaluated during project environmental activities. Results of coordination/consultation with the State Historic Preservation Officer as well as land managing agencies with jurisdiction over any present Section 4(f) resources will be included in the Section 4(f) analysis in the Categorical Exclusion.

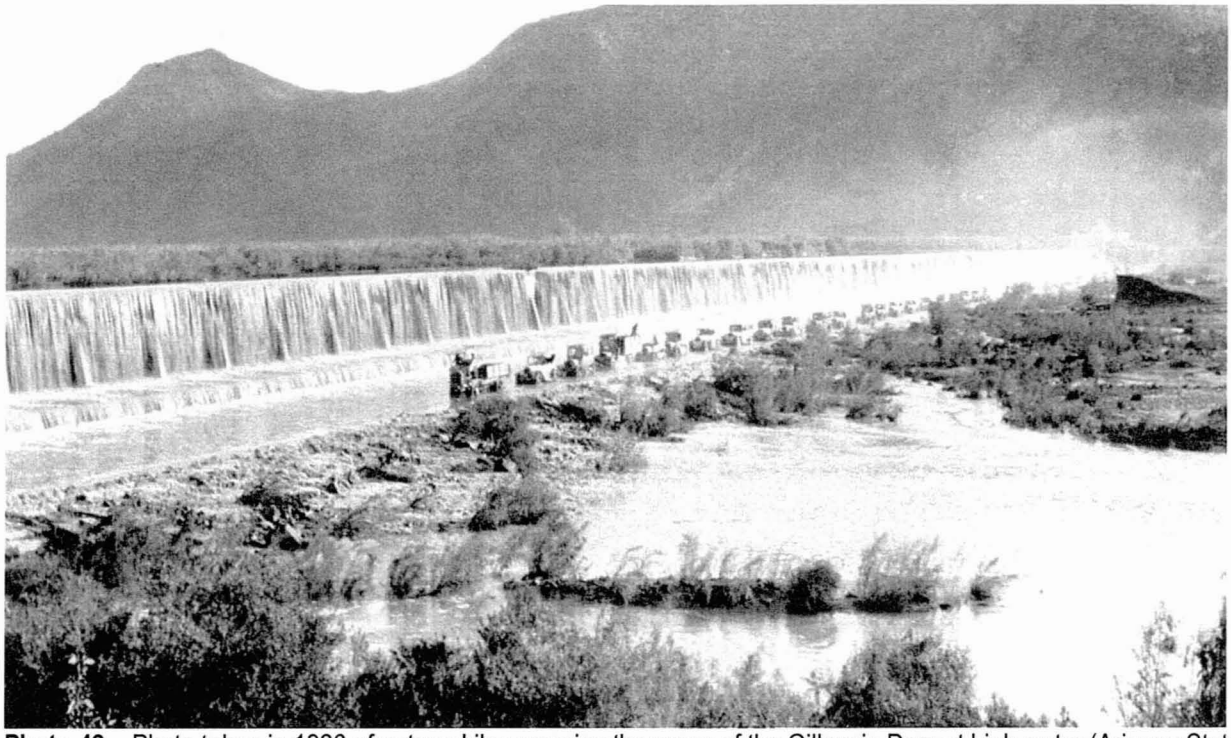


Photo 49 – Photo taken in 1926 of automobiles crossing the apron of the Gillespie Dam at high water (Arizona State Library, Archives and Public Records: no. 98-2339 and 98-2645).

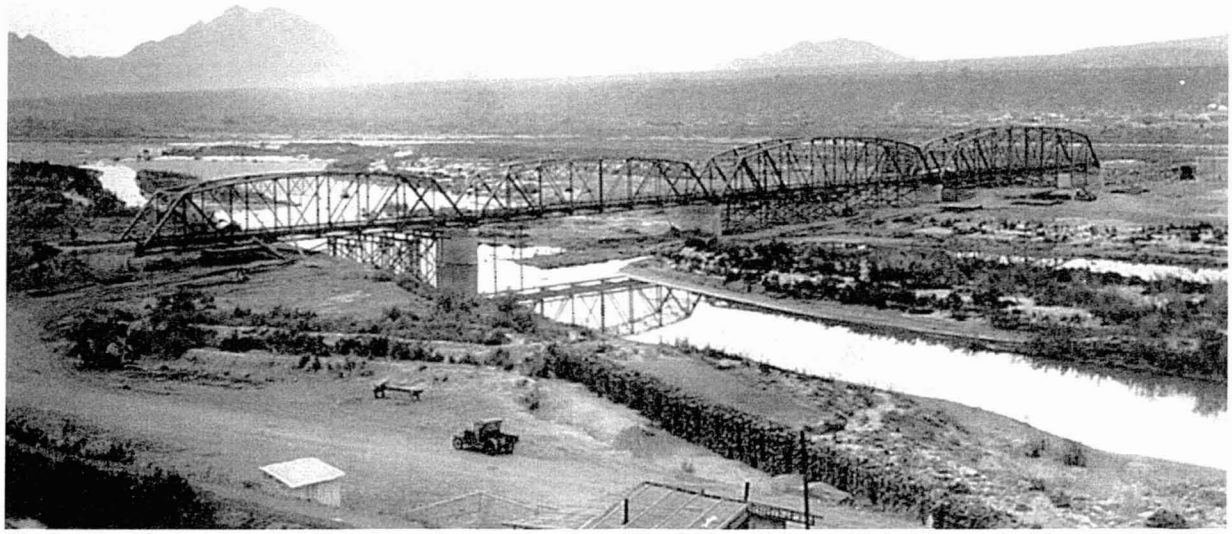


Photo 50 – Photo taken in 1926 of ongoing construction of the bridge (Arizona State Library, Archives and Public Records: no. 98-2360).

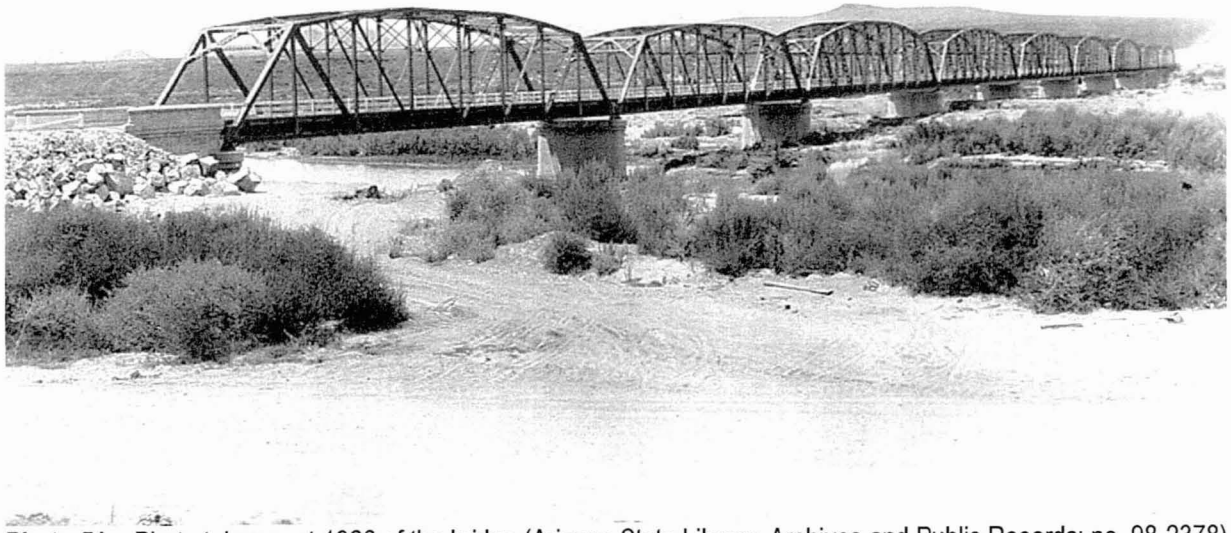


Photo 51 – Photo taken post-1926 of the bridge (Arizona State Library, Archives and Public Records: no. 98-2378).

Photo 52 – Photo taken post-1926 of the bridge (Arizona State Library, Archives and Public Records: no. 98-2376).

8.0 PUBLIC INVOLVEMENT

A public meeting was held on October 23, 2006 at the Gila Bend Elementary School in Gila Bend, Arizona. See Appendix C for public meeting handouts.

8.1 Public Meeting Purpose

Public Involvement --- the Maricopa County Department of Transportation (MCDOT) RightRoads program conducted this informational community open house meeting held on October 23, 2006 at the Gila Bend Elementary School in Gila Bend, Arizona to discuss and gather public comment on proposed plans for bridge rehabilitation.

8.2 Public Comments

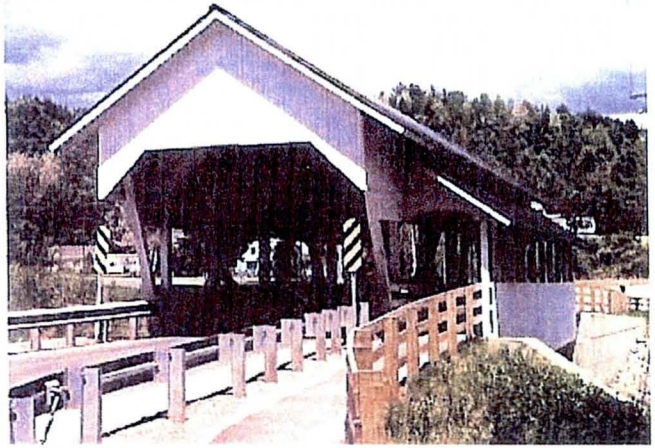
Approximately 20 people attended the public open house meeting to discuss and review plans for Gillespie Dam Bridge rehabilitation and conceptual design alternatives for future roadway and river crossing improvements at Gila Bend Elementary School in Gila Bend. Project information and comment sheets were distributed to all those in attendance. See Appendix C for the public meeting handouts. The following are comments received during discussions that project team members had with the attendees during the meeting:

- Farmers need wider bridge with more vertical clearance or dry crossing near the bridge.
- During construction and bridge closure, it is imperative a working bypass be provided, otherwise, there will be no public crossing available within many miles south and north.
- All proposals are acceptable ...
- Community attendees support idea of rehabilitation of the Gillespie Dam Bridge - it is an important bridge to them.
- The idea of having a pedestrian walkway on the bridge was felt to be a good idea and needed. If only one walkway was to be added to the bridge, most liked the pedestrian walkway on the upstream side of the bridge.
- Concern was expressed that care be taken to ensure the addition of a walkway to the bridge be esthetically pleasing/complimentary.
- Community support was expressed for the repainting of the bridge.
- Response was positive to the preliminary image of the pedestrian walkway and to the idea of viewing/rest areas.
- Comments seem to confirm need for pedestrian walkway as safety feature. (No objections voiced)
- "Important" that old bridge and dam be preserved and not be demolished following future improvements.
- Area farmers move their equipment back and forth on SR 85. (US-80 bridge is not wide enough to accommodate large equipment). New bridge should be designed to accommodate wide farm equipment/vehicles.

added

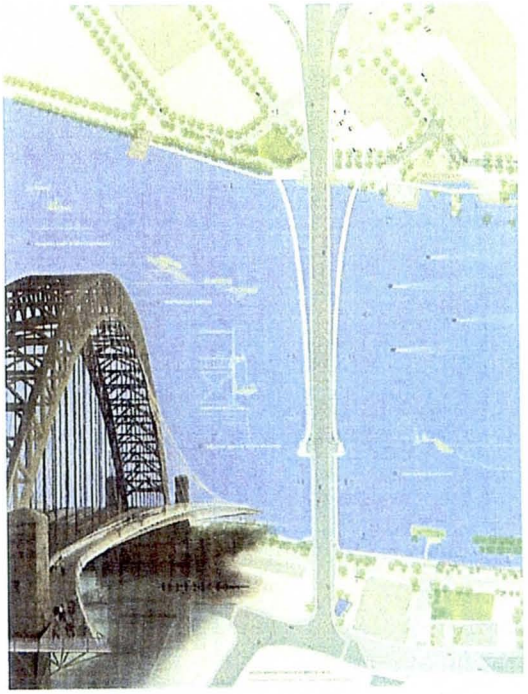
APPENDIX A – Examples of Historic Bridges with Pedestrian Walkways

Pedestrian Walkway Additions to Historic Bridges



Miller's Run Covered Bridge
Built in 1878 and restored in 1995
One-span, 56-feet
Pedestrian Walkway Addition - 1995

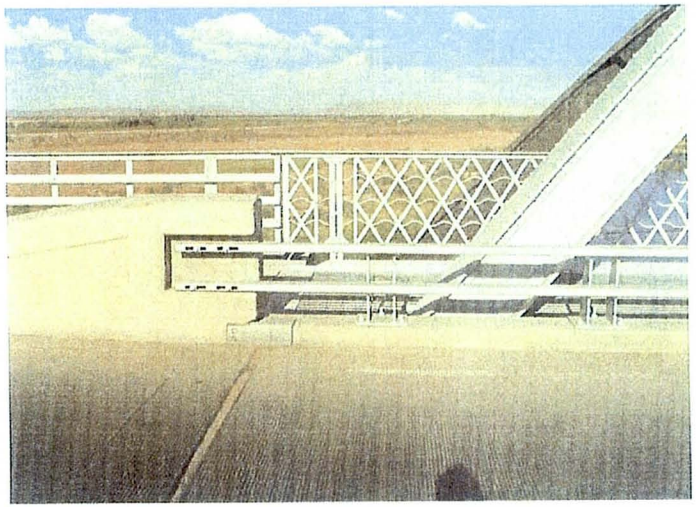
Harpersfield Covered Bridge (Ohio)
Built in 1868
Two-span, 230-feet
Pedestrian Walkway and Canopy Addition



West End Bridge (Pittsburgh)
Built in 1932
Conceptual Plans for Pedestrian Walkway Addition

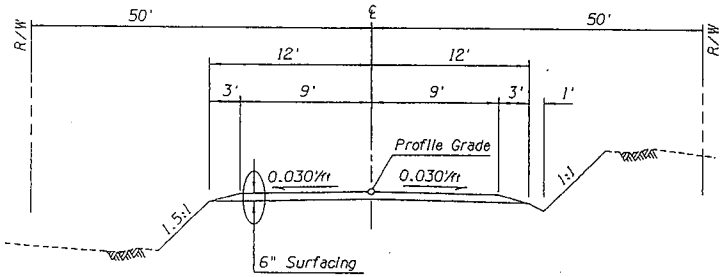
Ocean-to-Ocean Bridge Rehabilitation
Interstate 8
Yuma County, Arizona

Ocean-to-Ocean Highway Bridge Pedestrian walkway with metal grating and new barrier - this is what we propose for the Old US-80 Gila River Bridge.

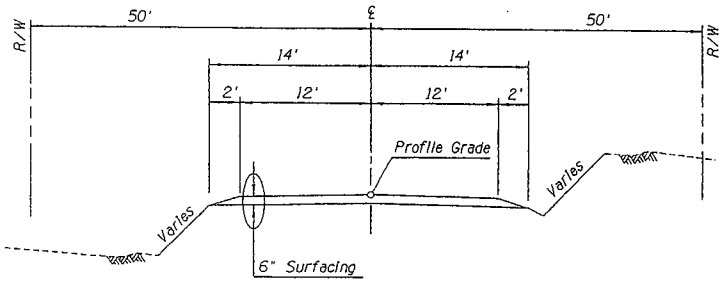


Ocean-to-Ocean Highway Bridge Barrier transitions to concrete wall

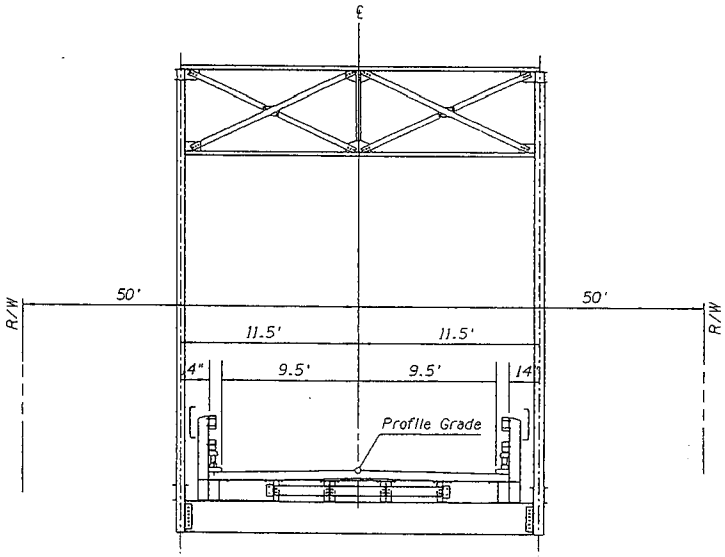
APPENDIX B – Preliminary Concept Plan Sheets for Alternatives 1, 2, and 4



EXISTING AS-BUILT ROADWAY TYPICAL SECTION (1926)



EXISTING ROADWAY TYPICAL SECTION (Field Observation)



EXISTING BRIDGE TYPICAL SECTION
ALTERNATIVE 1 (DO NOTHING)

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

PRELIMINARY
NOT FOR
CONSTRUCTION

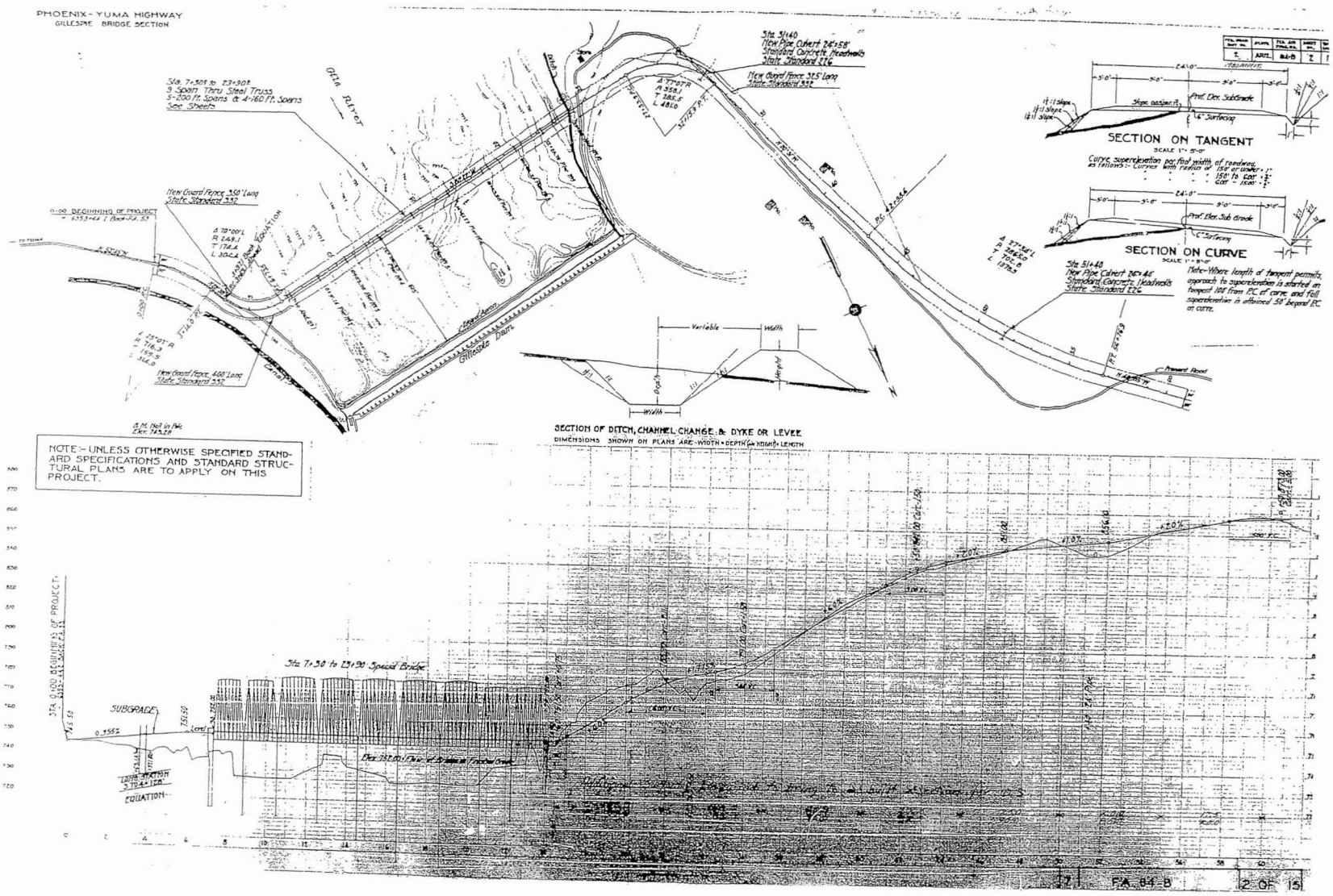
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3688

ROADWAY TYPICAL SECTION

SHEET
OF

Alternate No. 1 - Do Nothing (Not Recommend)

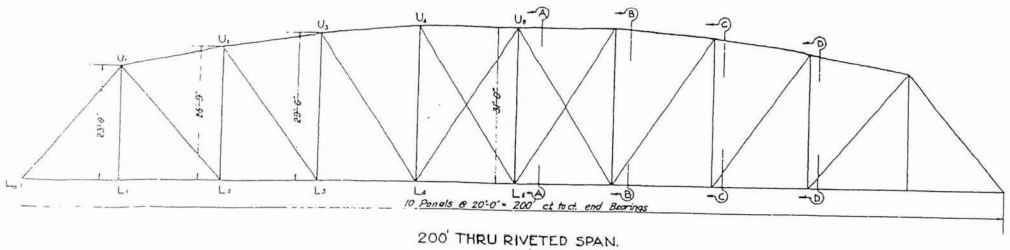


As built Roadway Plan

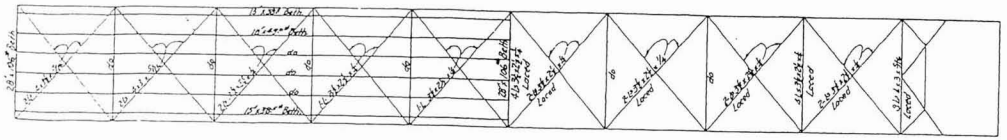
GILLESPIE DAM HIGHWAY BRIDGE
ACROSS THE GILA RIVER
PHOENIX-YUMA HIGHWAY

FED. ROAD DIST. NO.	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS
2	ARIZ.	64-B	5	13

Sheet 2 of 5 of Bridge
As Built - JRF



200' THRU RIVETED SPAN.
Scale: 1" = 10'-0"



HALF PLAN LOWER LATERAL SYSTEM.

HALF PLAN UPPER LATERAL SYSTEM.

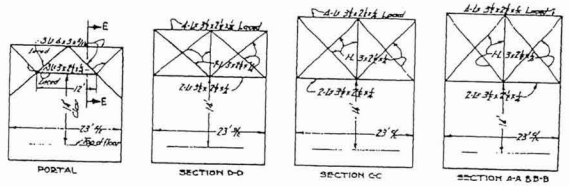
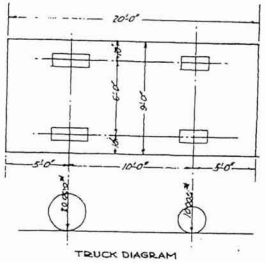


TABLE OF STRESSES - 200' FT. SPAN.

MEMBER	STRESSES D.L.	L.L.	IMPACT	TOTAL	SECTIONS	AREA IN. SQ. INCHES
L$_1$U$_1$	+207200	+72800	+21800	+297,600	2 ∠'s 15"×33.9# - 2-PL 15"×35"	29.21
U$_1$U$_2$	+207000	+74400	+22,300	303,700	do 2 ∠'s 15"×33.9#	29.21
U$_2$U$_3$	+244,000	+87,200	+26,300	358,000	do 2 ∠'s 15"×35#	29.21
U$_3$U$_4$	+253,300	+94,600	+28,400	376,300	do 2 ∠'s 15"×40#	32.15
U$_4$U$_5$	+274,200	+98,600	+29,300	402,100	do 2 ∠'s 15"×42#	32.15
L$_1$L$_2$	-46,900	+24,500	+ 7,300	- 78,600	2 ∠'s 10"×15.3# (Lacing)	8.94
L$_2$L$_3$	+18,000	+18,000	+ 5,400	+ 41,400	2 ∠'s 6"×11.5#	6.72
L$_3$L$_4$	- 2,700	+15,000	+ 4,500	- 18,900	do	6.72
L$_4$L$_5$	0	0	0	0	do	6.72
L$_1$L$_2$	-133,000	-47,700	-14,300	-195,000	2 ∠'s 15"×33.9#	17.60
L$_2$L$_3$	-133,000	-47,700	-14,300	-195,000	do	17.60
L$_3$L$_4$	-203,400	-72,000	-21,700	-298,300	2 ∠'s 15"×40#	21.20
L$_4$L$_5$	-240,000	-87,000	-28,800	-355,100	2 ∠'s 15"×45#	24.14
L$_5$L$_6$	-253,000	-94,600	-28,300	-383,700	2 ∠'s 15"×50#	26.88
L$_1$U$_1$	-34,000	-25,000	- 7,500	-66,500	2 ∠'s 8"×11.5# (Lacing)	5.60
U$_1$L$_2$	-107,100	-41,800	-12,800	-161,400	2 ∠'s 12"×20.7#	10.50
U$_2$L$_3$	-84,400	-31,200	- 9,400	-105,000	2 ∠'s 10"×15.3#	7.42
U$_3$L$_4$	-37,500	-25,000	- 7,500	-70,300	2 ∠'s 8"×3/8" × 8#	4.78
U$_4$L$_5$	-20,200	-22,000	- 6,600	-48,800	2 ∠'s 5"×3"×3/8"	3.70
U$_5$L$_6$	0			-4,900	2 ∠'s 8"×2½"×½"	2.18

— Tension + — Compression

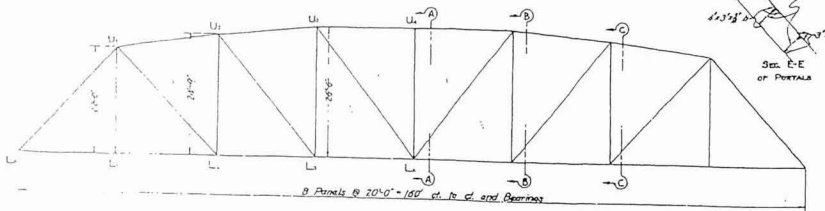


TRUCK DIAGRAM

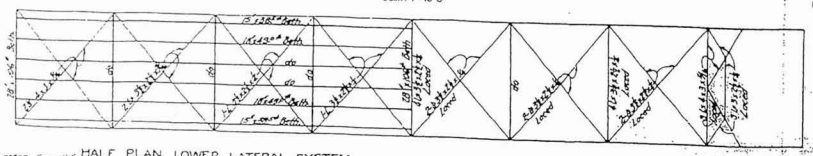
LOADING - 200' Span, -
Live Load - 2-15 Ton trucks as per diagram or 64# per sq. ft. of roadway.
Impact - 30% of Live Load Stresses.
Dead Load - 3600# per ft. of span (estimated)

LOADING - 160' Span, -
Live Load - 2-15 Ton trucks as per diagram or 68# per sq. ft. of roadway.
Impact - 30% of Live Load Stresses.
Dead Load - 3060# per ft. of span (estimated).

GENERAL NOTES, -
All rivets ¾"ø
State Standard Specifications to apply except as noted in "Special Provisions."
Lacing bars for 15" ∠'s & 12" ∠'s 2½"×⅜".
All other lacing bars 2"×⅜".
Gusset plates at L₀ ⅝", all others ½".
Paint - 1 shop coat, 2 field coats, see Special Prov.
Cast Steel Pedestals shall be used for bed plate.
Roller Nests shall be entirely enclosed with angles secured to shoe by steel bolts.
Gusset plates in Portals - ⅜".
Shop drawings to be made on standard sheets outside dimensions 22"×36"

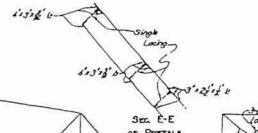


160' THRU RIVETED SPAN.
Scale: 1" = 10'-0"



HALF PLAN LOWER LATERAL SYSTEM.

HALF PLAN UPPER LATERAL SYSTEM.



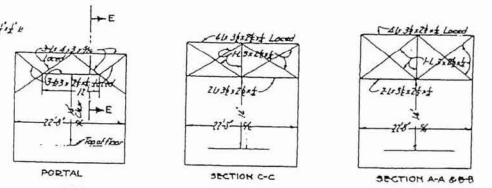


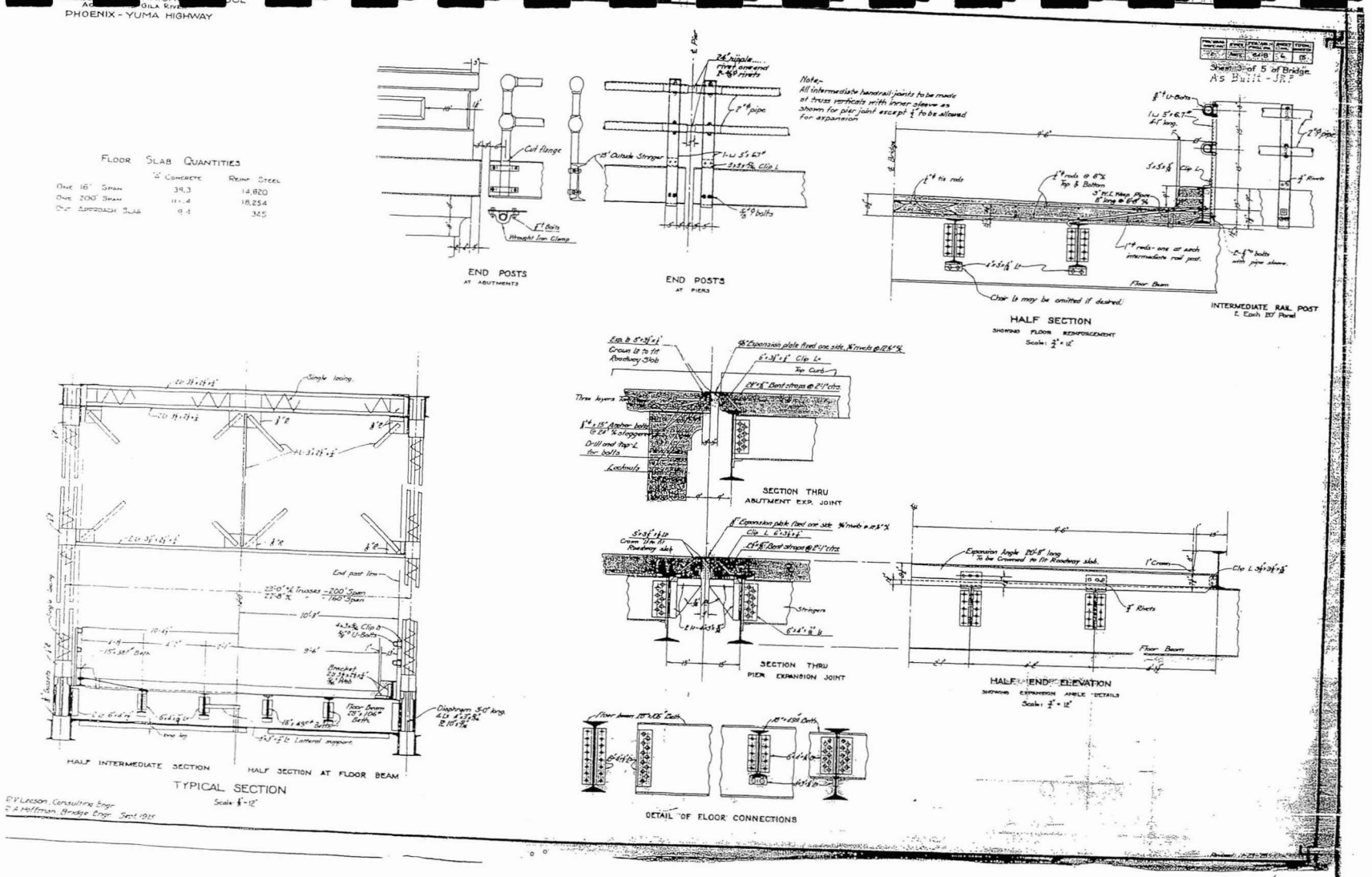
TABLE OF STRESSES - 160' FT. SPAN.

MEMBER	STRESSES D.L.	L.L.	IMPACT	TOTAL	SECTIONS	AREA IN. SQ. INCHES
L$_1$L$_2$	+144,000	+61,050	+18,300	+224,000	2 ∠'s 15"×33.9# - 2-PL 15"×35"	21.64
L$_2$L$_3$	+149,000	+63,300	+18,900	+231,300	do 2-∠'s 15"×33.9#	21.64
L$_3$L$_4$	+173,300	+75,600	+22,800	+271,600	do 2-∠'s 15"×40#	24.58
L$_4$L$_5$	+189,800	+77,900	+23,400	+291,100	do 2-∠'s 15"×50#	29.38
L$_1$U$_1$	+7,600	+18,700	+5,600	+44,900	2-∠'s 6"×11.5# (Lacing)	6.72
U$_1$L$_2$	-31,300	-12,900	-3,900	-48,100	do	6.72
L$_2$U$_2$	0	0	0	0	do	6.72
L$_2$L$_3$	-97,400	-41,050	-12,300	-150,750	2-∠'s 12"×20.7# *	10.50
L$_3$L$_4$	-97,400	-41,050	-12,300	-150,750	do	10.50
L$_3$L$_4$	-148,800	-62,500	-18,750	-229,600	2-∠'s 15"×50# *	15.82
L$_4$L$_5$	-172,200	-72,800	-21,800	-267,000	2-∠'s 15"×50# *	18.32
L$_5$U$_5$	-50,800	-25,200	-7,800	-83,100	2-∠'s 8"×3"×½" (Lacing)	8.40
U$_4$U$_5$	-270,900	-98,300	-29,700	-398,900	2-∠'s 15"×42# *	17.20
U$_4$U$_5$	-244,300	-88,000	-26,700	-359,000	2-∠'s 15"×40#	15.70
U$_4$U$_5$	-174,700	-63,300	-19,100	-257,100	2-∠'s 15"×33.9#	15.70

Revised 11-23-25

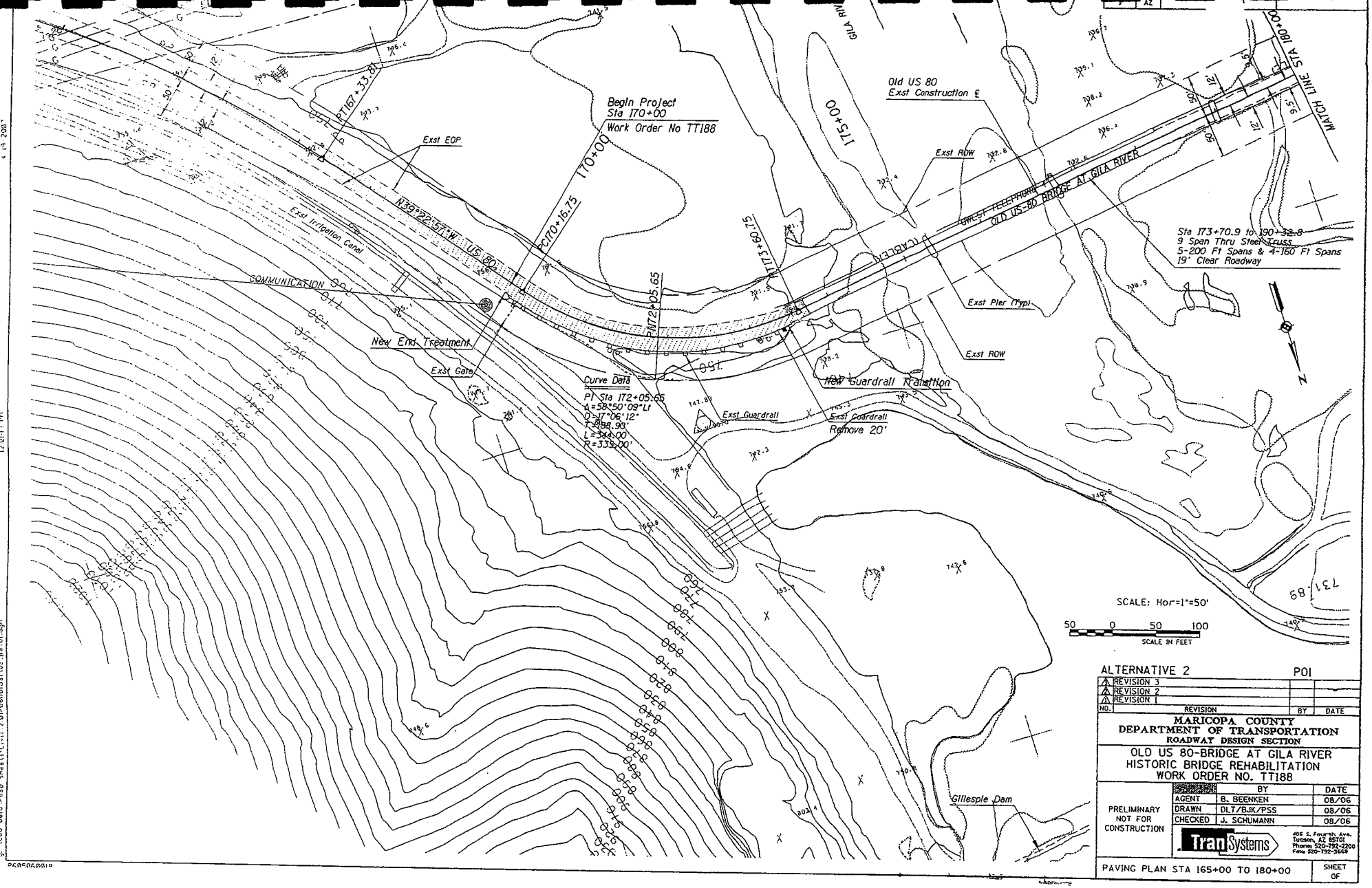
P.V. Lawson, Consult. Engr.
R.A. Hoffman, Bridge Engr. Sept. 1925

As built Truss Plan

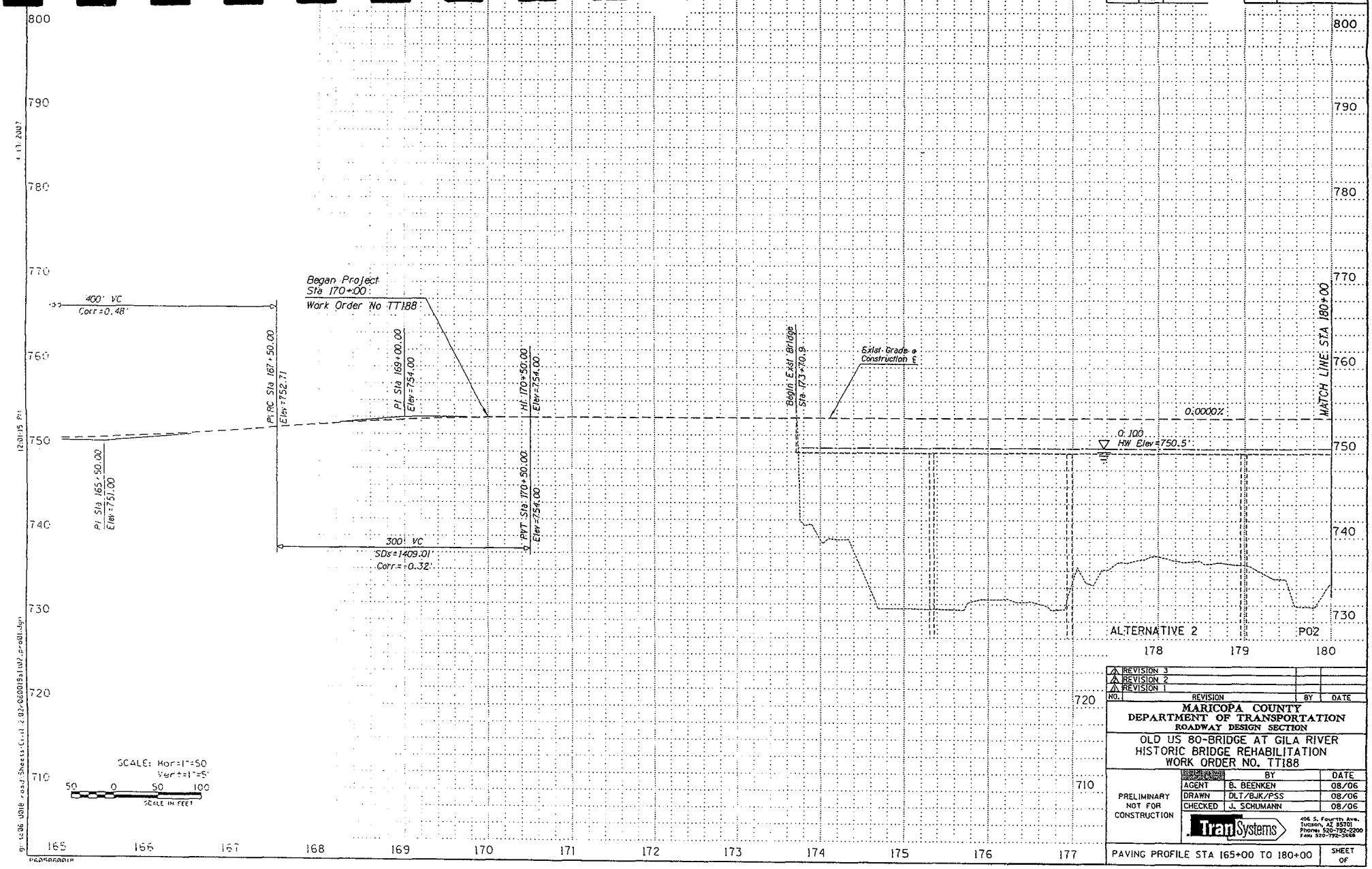


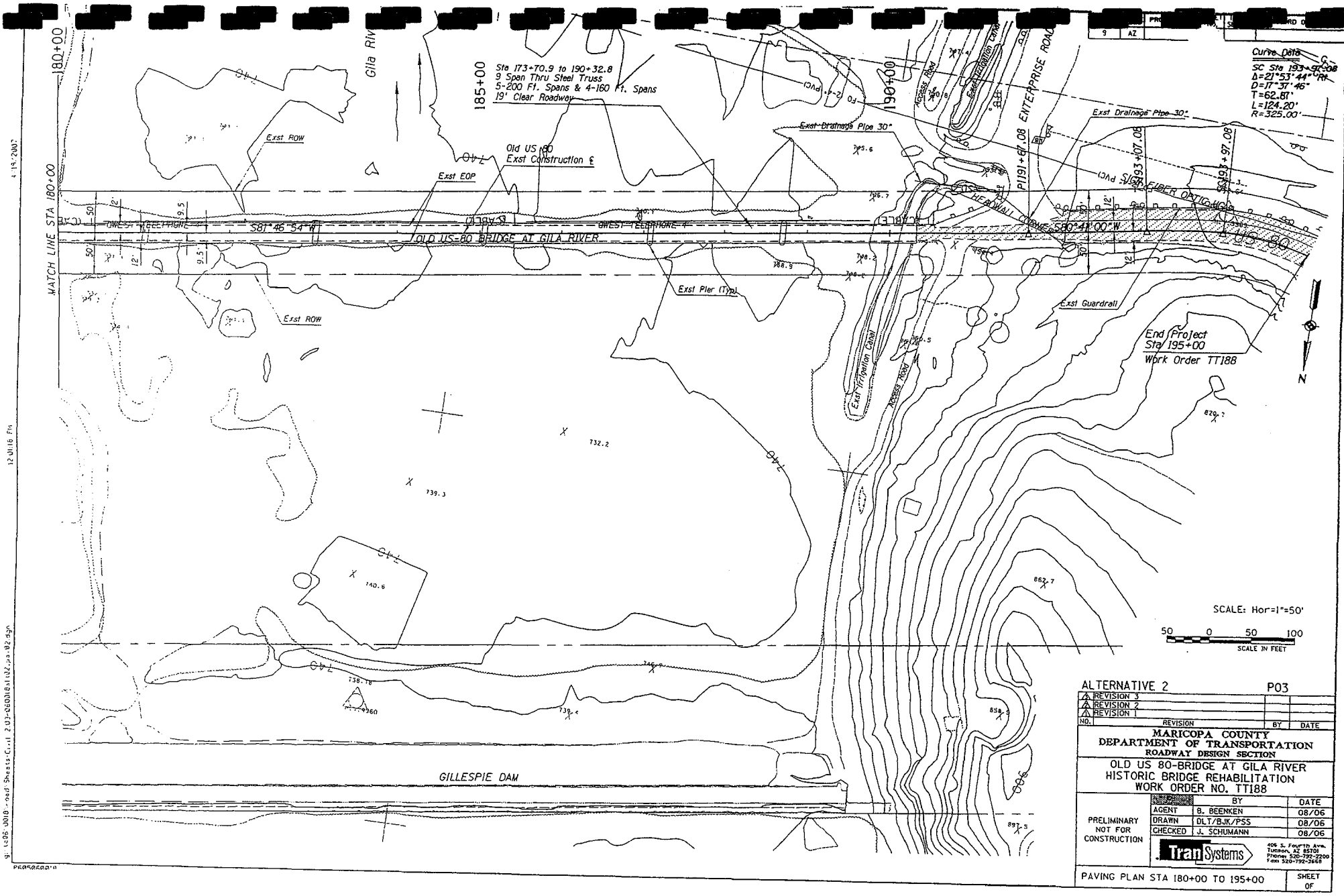
409

As built Bridge Plan



Alternate No. 2 – Preferred Alternate


Began Project
Sta 170+00
Work Order No TT188
400' VC
Corr=0.48'
PI Sta 165+50.00
Elev=751.00
PI RC Sta 167+50.00
Elev=752.71
PI Sta 169+00.00
Elev=754.00
HI 170+50.00
Elev=754.00
PVT Sta 170+50.00
Elev=754.00
300' VC
SDs=1409.01'
Corr.=0.32'
Begin Exist Bridge
Sta 173+70.9
Exist Grade @
Construction ℄
0.0000%
0.100
HW Elev=750.5'
MATCH LINE STA 180+00
ALTERNATIVE 2
P02
SCALE: Hor=1"=50
Ver=1"=5'
SCALE IN FEET
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
BY DATE
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3088
PAVING PROFILE STA 165+00 TO 180+00
SHEET
OF



ALTERNATIVE 2 P03

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
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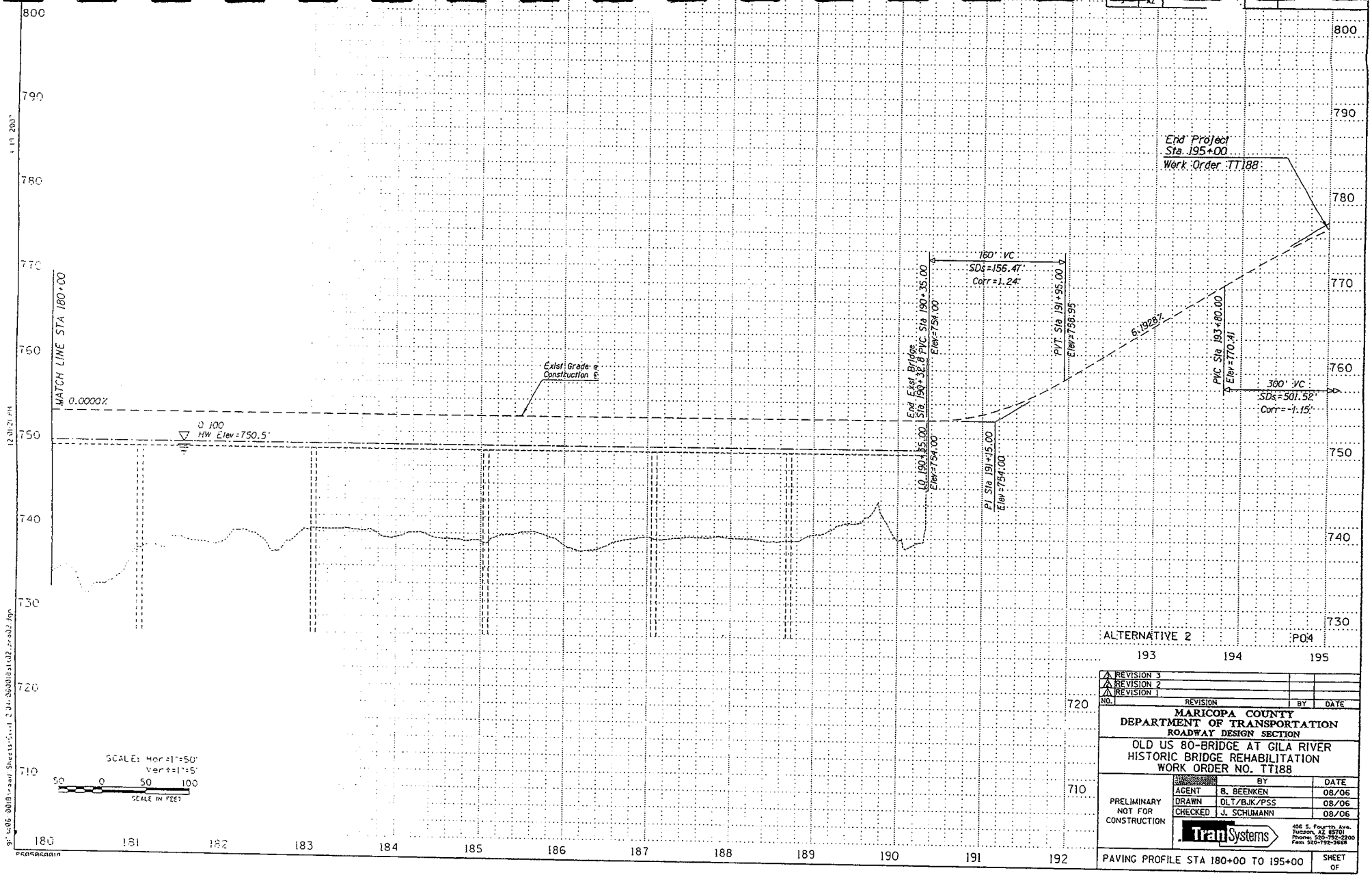
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-2668

PAVING PLAN STA 180+00 TO 195+00

SHEET OF



NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

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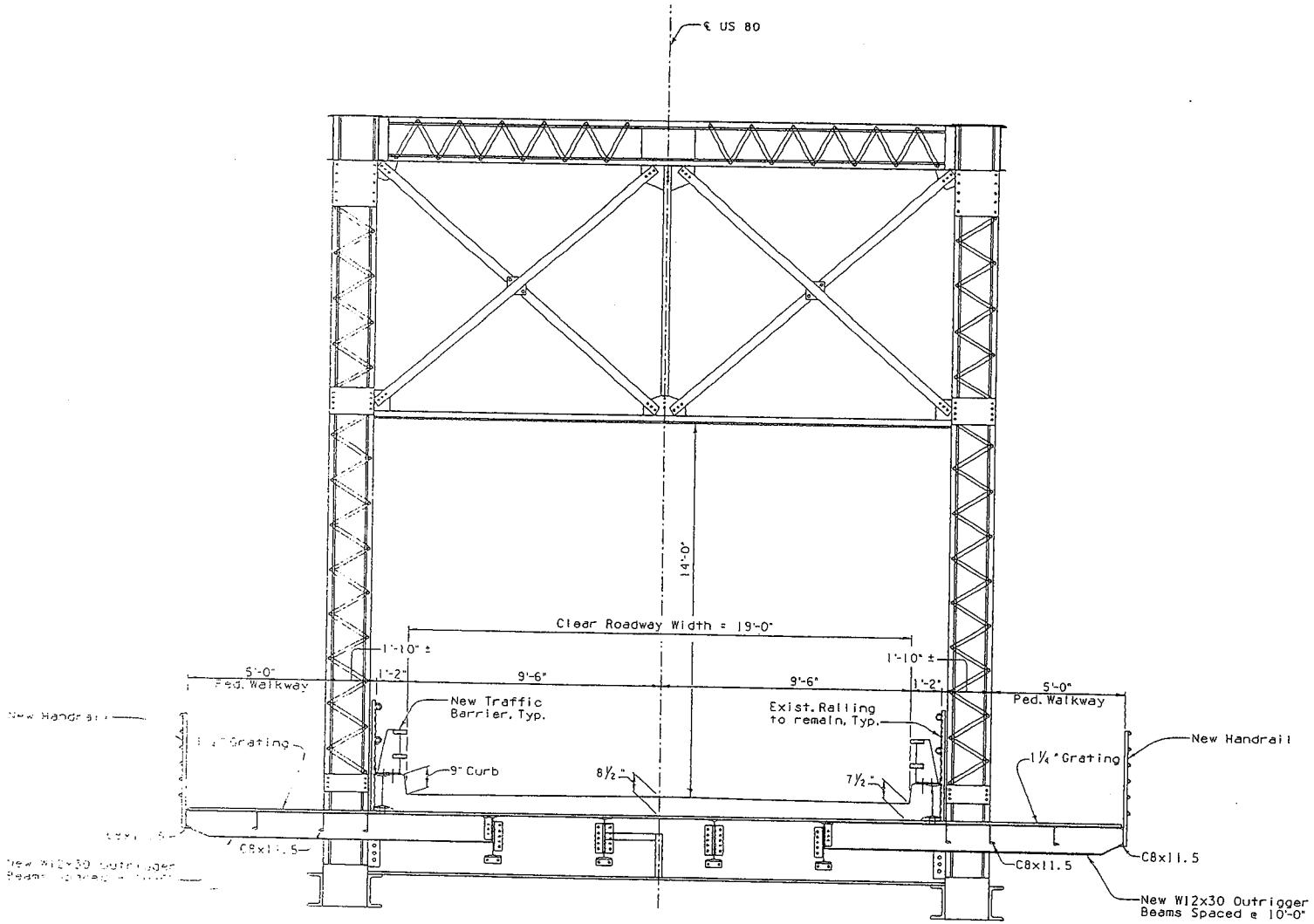
PAVING PROFILE STA 180+00 TO 195+00

SHEET
OF



TYPICAL SECTION
½" = 1'-0"

PEDESTRIAN WALKWAY
ALTERNATIVE 2
(SHEET 1 OF 3)

ALTERNATIVE 2

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

PRELIMINARY
NOT FOR
CONSTRUCTION

	BY	DATE
AGENT	J. CANNON	08/06
DRAWN	DY/AP	08/06
CHECKED		08/06

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405 S. Fourth Ave.
Tucson, AZ 85701
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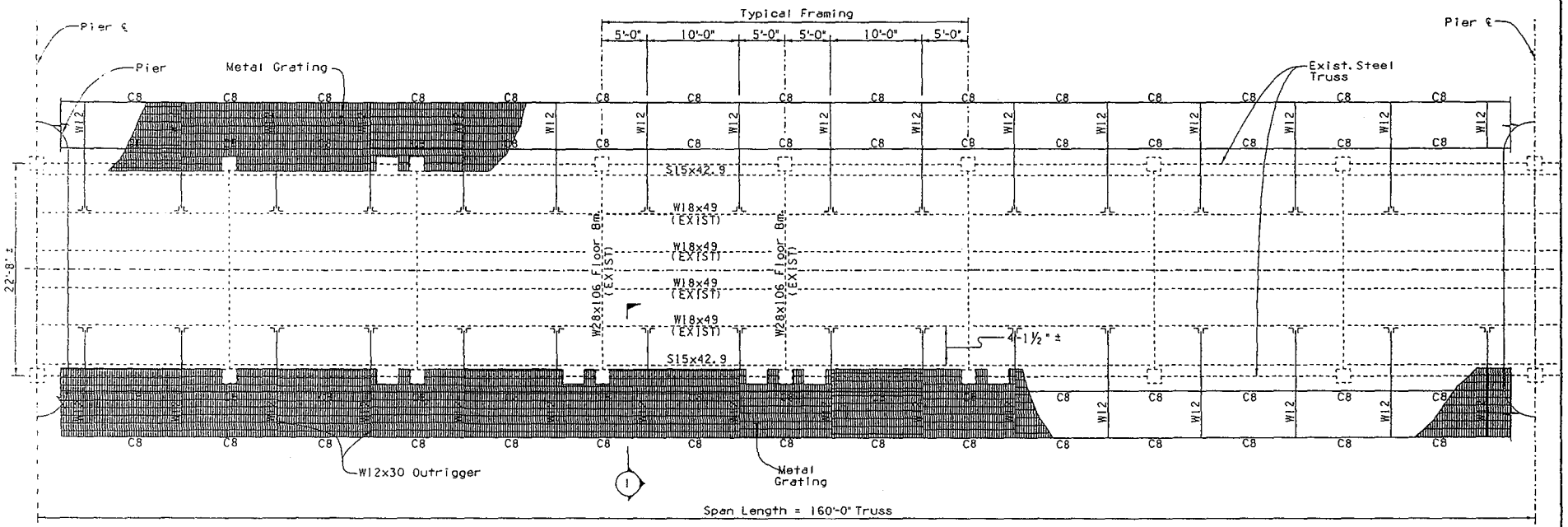
Fax: 520-792-3668

SHEET
OF

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



FRAMING PLAN FOR 160'-0" TRUSS

3/16" = 1'-0"

PEDESTRIAN WALKWAY
ALTERNATIVE 2
(SHEET 2 OF 3)

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

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	DRAWN	D. YRIGOYEN	08/06
	CHECKED		08/06

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Tucson, AZ 85701
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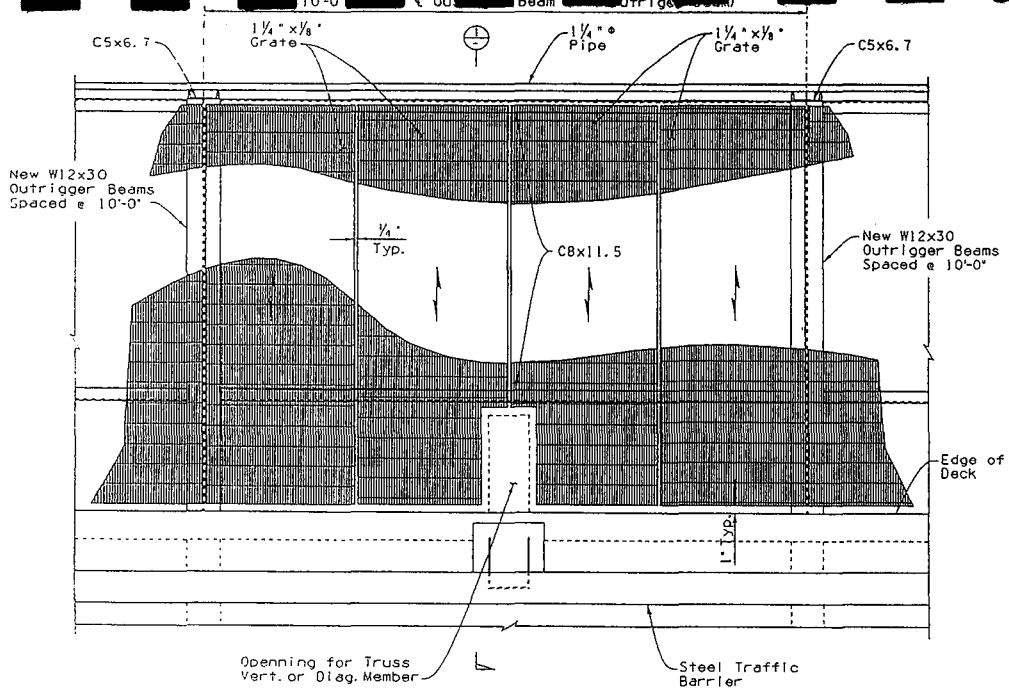
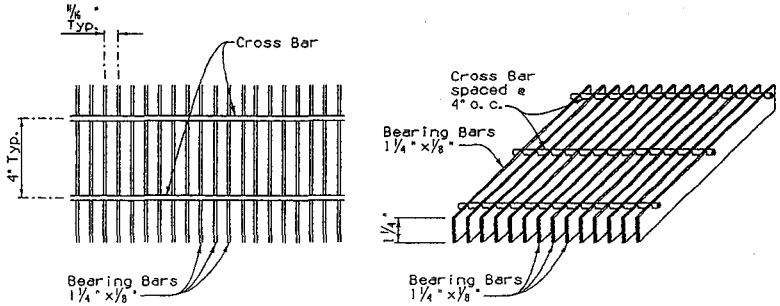
SHEET
OF

P605060018

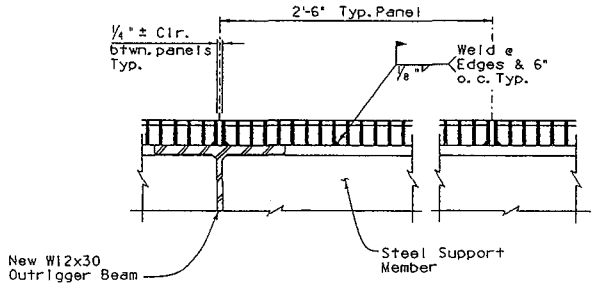
F.H.W.A. REGION	STATE	PROJECT NO.	NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ				



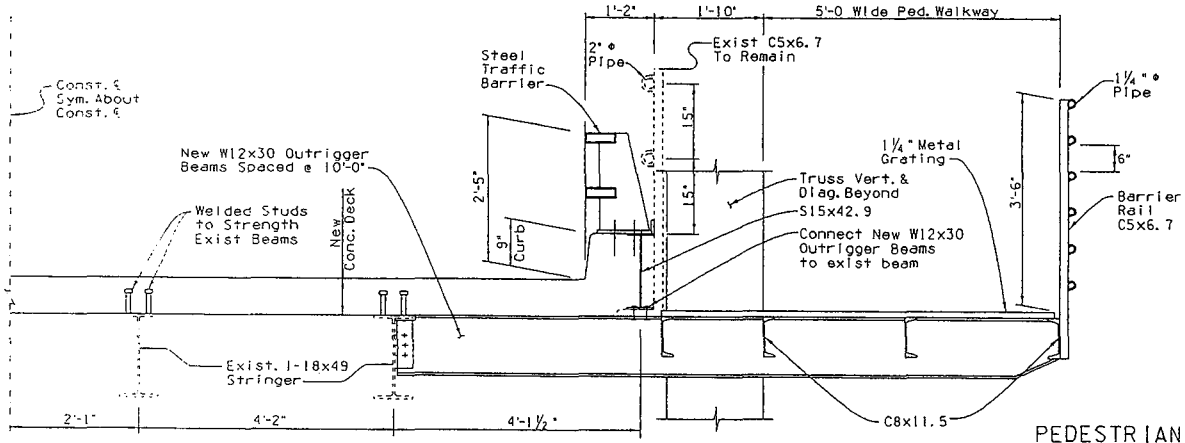
TYPICAL GRATE LAYOUT
1" = 1'-0"



TYPICAL CLOSE MESH PLAIN STEEL GRATING (2)
No Scale



TYPICAL GRATING-WELDING DETAIL (3)
No Scale



TYPICAL PED. SECTION (1)
1" = 1'-0"

NOTES:

1. Steel grating to be GCM-4 (McNichols) or similar
2. Attach steel grating to supports w/ ⅛" x 1" fillet welds spaced as follow:
   a- Parallel to supports-space @ 6" o. c.
3. Steel grating to have primer coat plus two finish coats to match color of the bridge.
4. All Steel to be ASTM A36.

PEDESTRIAN WALKWAY
ALTERNATIVE 2
(SHEET 3 OF 3)

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

		BY	DATE
PRELIMINARY NOT FOR CONSTRUCTION	AGENT	J. CANNON	08/06
	DRAWN	D. YRIGOYEN	08/06
	CHECKED		08/06

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3468

SHEET OF

4/30/2007
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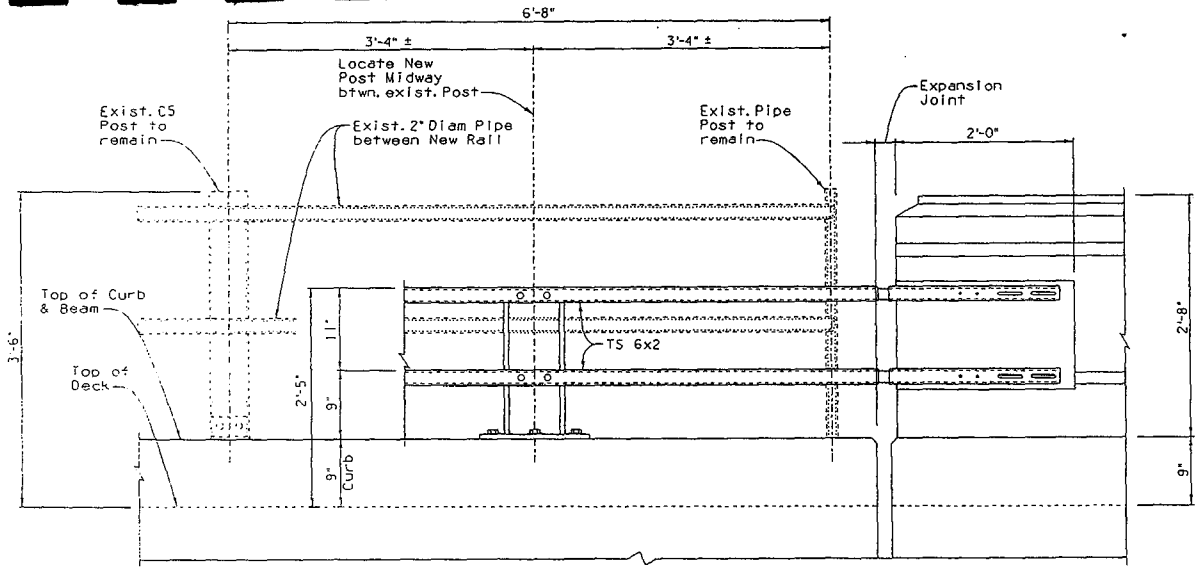
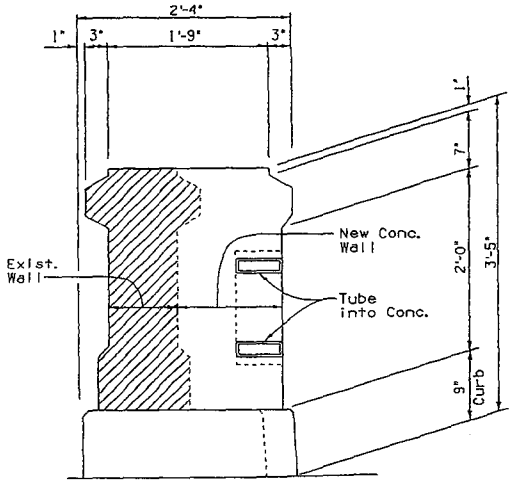
PR050680818

FOR REGIONAL		PROJECT	NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ				



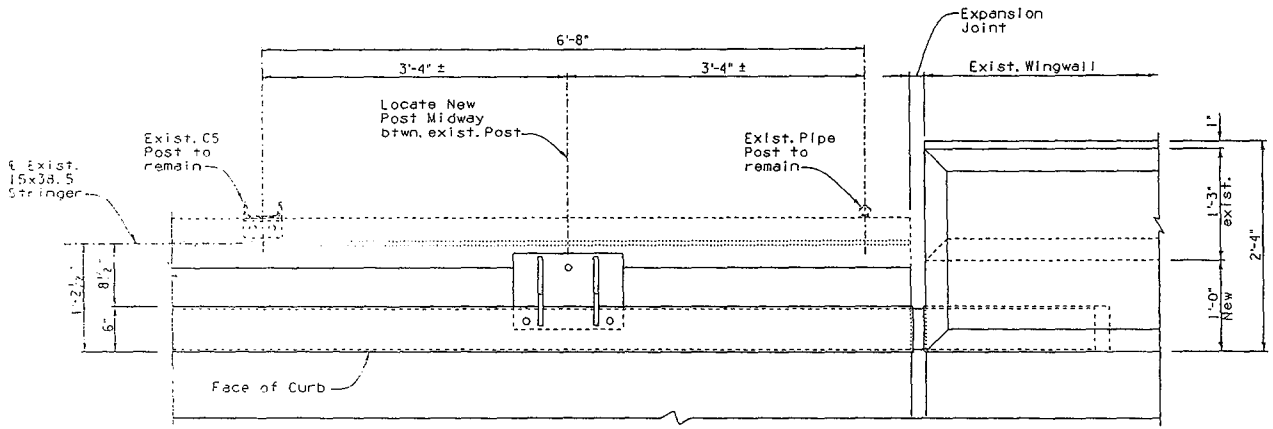
ELEVATION BARRIER RAIL TRANSITION & POST SPACING

1½" = 1'-0"



MODIFY CONC. WINGWALL FOR BARRIER RAIL TRANSITION

1½" = 1'-0"



PLAN - BARRIER RAIL TRANSITION

1½" = 1'-0"

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
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	DRAWN	D. YRIGOYEN	08/06
	CHECKED		08/06

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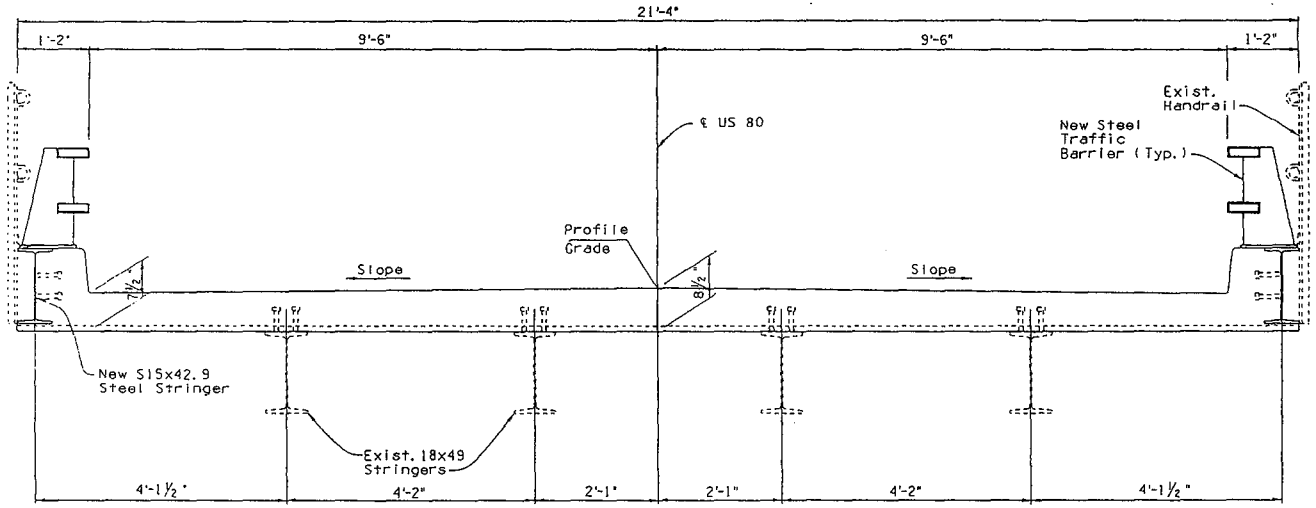
SHEET
OF

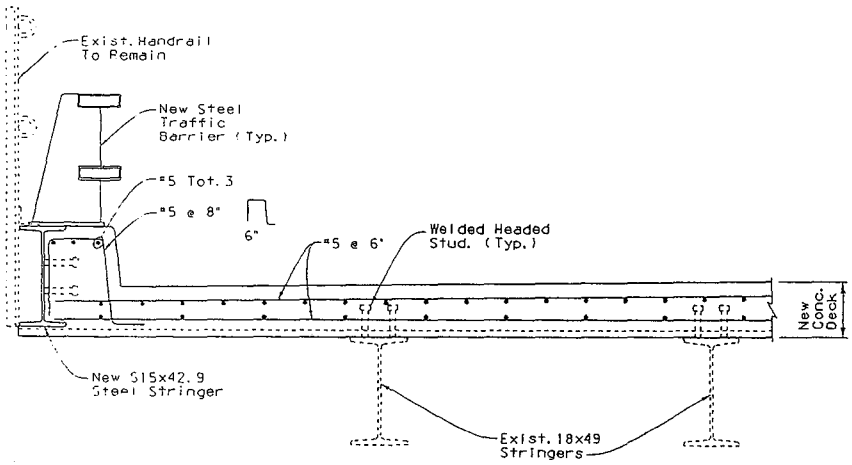
P605060918

FED. RD. REGION	STATE	PROJECT NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ			



TYPICAL BRIDGE DECK SECTION

1" = 1'-0"



PARTIAL TYPICAL DECK SECTION

No Scale

NOTES:

1- Longitudiani deck reinforcement shall be placed parallel to ℄ girders.

2- Transverse deck reinforcement shall be placed parallel to ℄ Brg. at piers with spacing measured along const. ℄

3- Bars shall not be spliced with in the required lap length of the adjacent bars.

4- Top transverse straight bars in the deck slab shall be spliced at center of girder span.

5- Bottom transverse straight bars in the deck slab shall be spliced at centerline of girder.

6- Contractor to submit shop drawings showing lap splice locations to the Engineer for approval prior to fabrication.

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
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ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
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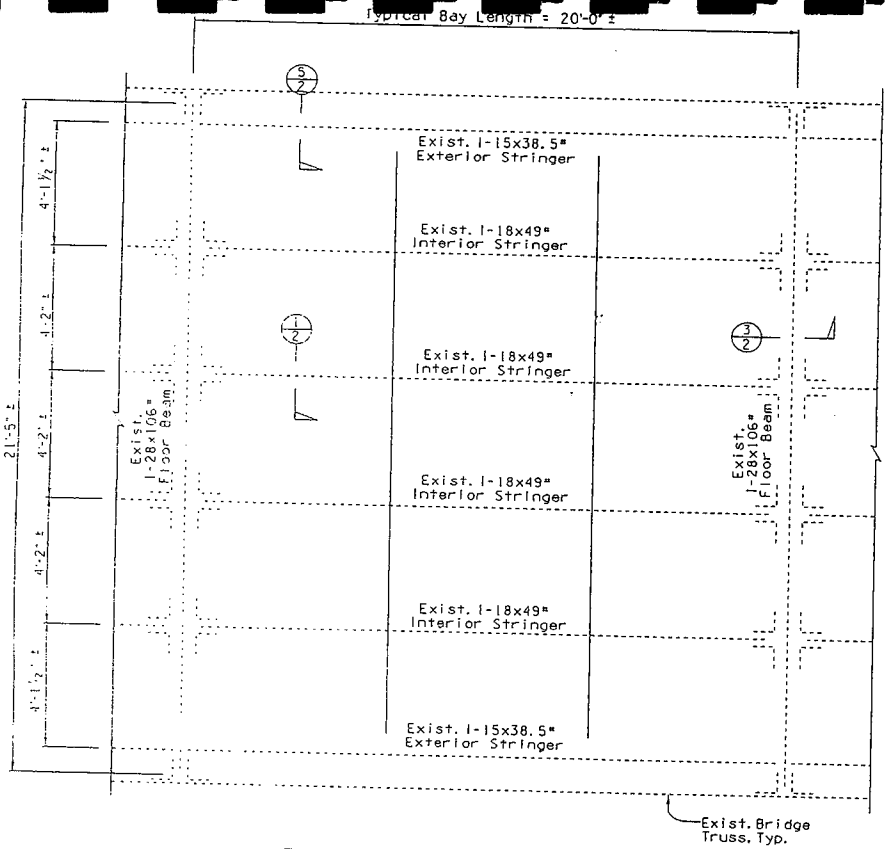
CONCRETE DECK REPLACEMENT

SHEET OF

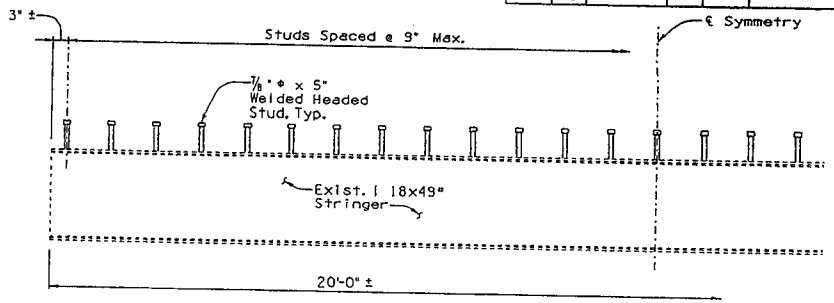
P605060018

REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ				



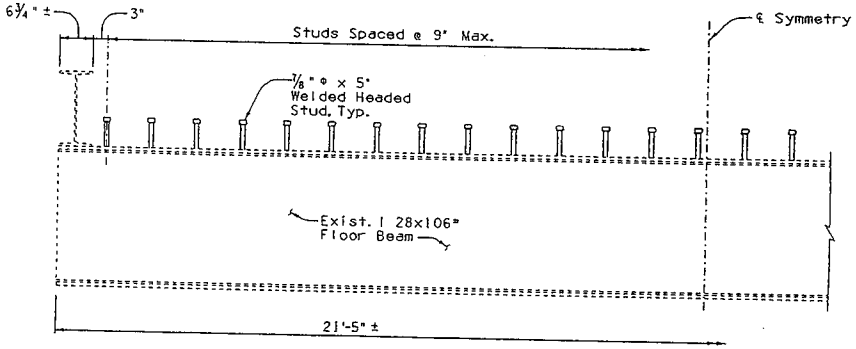
FRAMING PLAN - TYPICAL BAY

½" = 1'-0"



STEEL INTERIOR STRINGER ELEVATION (1)

1" = 1'-0"



STEEL FLOOR BEAM ELEVATION (2)

1" = 1'-0"

Notes:

1. Design of steel members is composite for live load and superimposed dead load only. Other dead load is carried by the steel members only.
2. Shear connectors shall be fatigue stress Category A through C for Case I stress cycles.
3. Steel stud connectors shall conform to ASTM A108, grades C-1015, C-1018, or C-1020 cold rolled steel. Solid or granular flux-filled studs shall be used. The studs shall be automatically end welded to the steel girders with complete fusion.
4. All welding shall conform to the ANSI/AASHTO/AWS Bridge Welding Code, D1.5: Current.

COMPOSITE MEMBER DESIGN ALTERNATIVE
(SHEET 1 OF 2)

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
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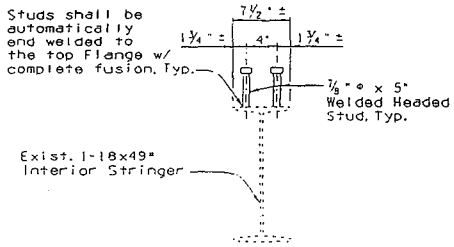
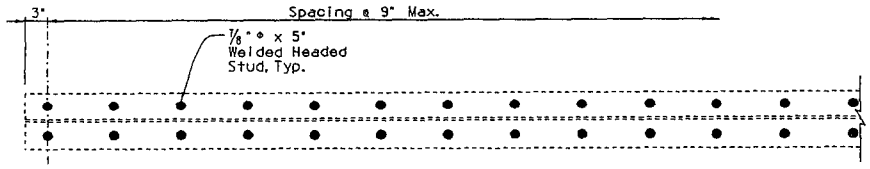
SHEET
OF

P695060018

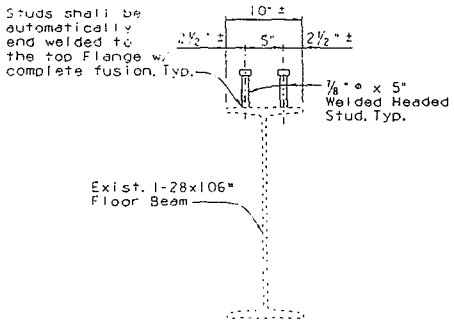
FHWA REGION	STATE	PROJECT NO.	NO.	TOTAL SHEETS	RECORD DRAWING
9	AZ				



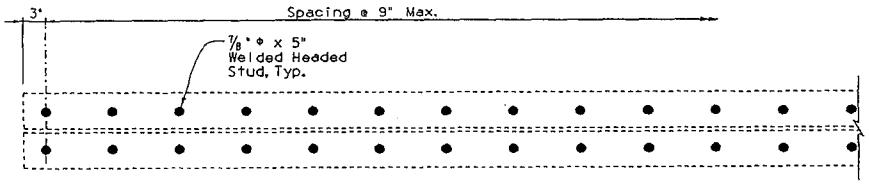
SECTION - STEEL INTERIOR STRINGER (1)
1½" = 1'-0"



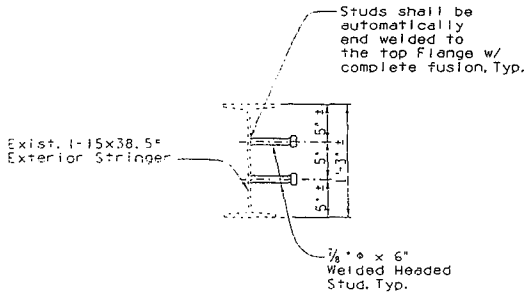
PARTIAL PLAN - STEEL INTERIOR STRINGER (2)
1½" = 1'-0"



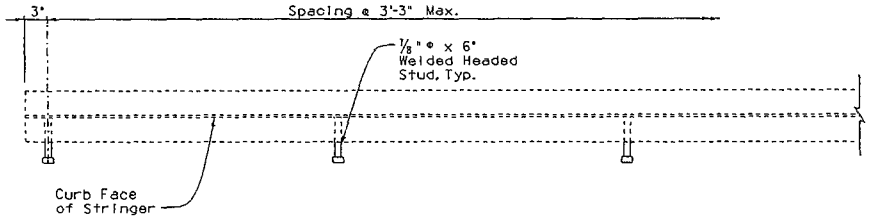
SECTION - STEEL FLOOR BEAM (3)
1½" = 1'-0"



PARTIAL PLAN - STEEL FLOOR BEAM (4)
1½" = 1'-0"



SECTION - STEEL EXTERIOR STRINGER (5)
1½" = 1'-0"



PARTIAL PLAN - STEEL EXTERIOR STRINGER (6)
1½" = 1'-0"

COMPOSITE MEMBER DESIGN ALTERNATIVE
(SHEET 2 OF 2)

NO.	REVISION	BY	DATE
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2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

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PRELIMINARY NOT FOR CONSTRUCTION	AGENT	J. CANNON	08/06
	DRAWN	D. YRIGOYEN	08/06
	CHECKED		08/06

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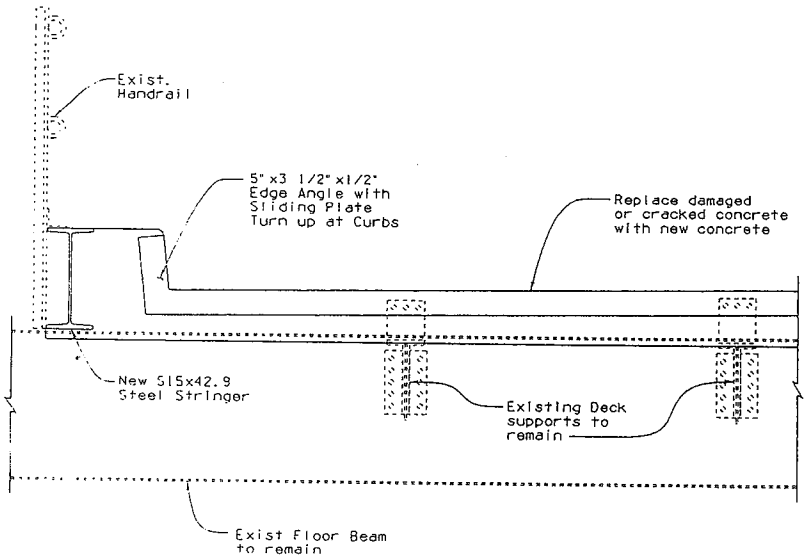
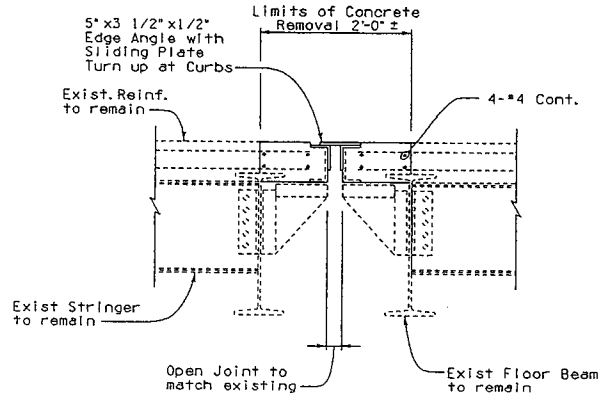
SHEET
OF



PARTIAL SECTION AT PIER EXPANSION JOINTS (1)
1" = 1'-0"



PARTIAL SECTION THRU PIER EXPANSION JOINTS (2)
1" = 1'-0"

Notes:

1. Remove damaged or cracked concrete at joints. When removing concrete, take care not to damage exist. longitudinal reinforcement.
2. Clean and/or replace damaged or deteriorated edge angles and plates.
3. Place new edge angle as shown and turn up at curb. Pour new concrete to match existing slab dimensions and grades.
4. Weld new sliding plate to one side of joint allowing other end to move freely.

DECK JOINT REPAIR ALTERNATIVE

NO.	REVISION	BY	DATE
3	REVISION 3		
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MARICOPA COUNTY
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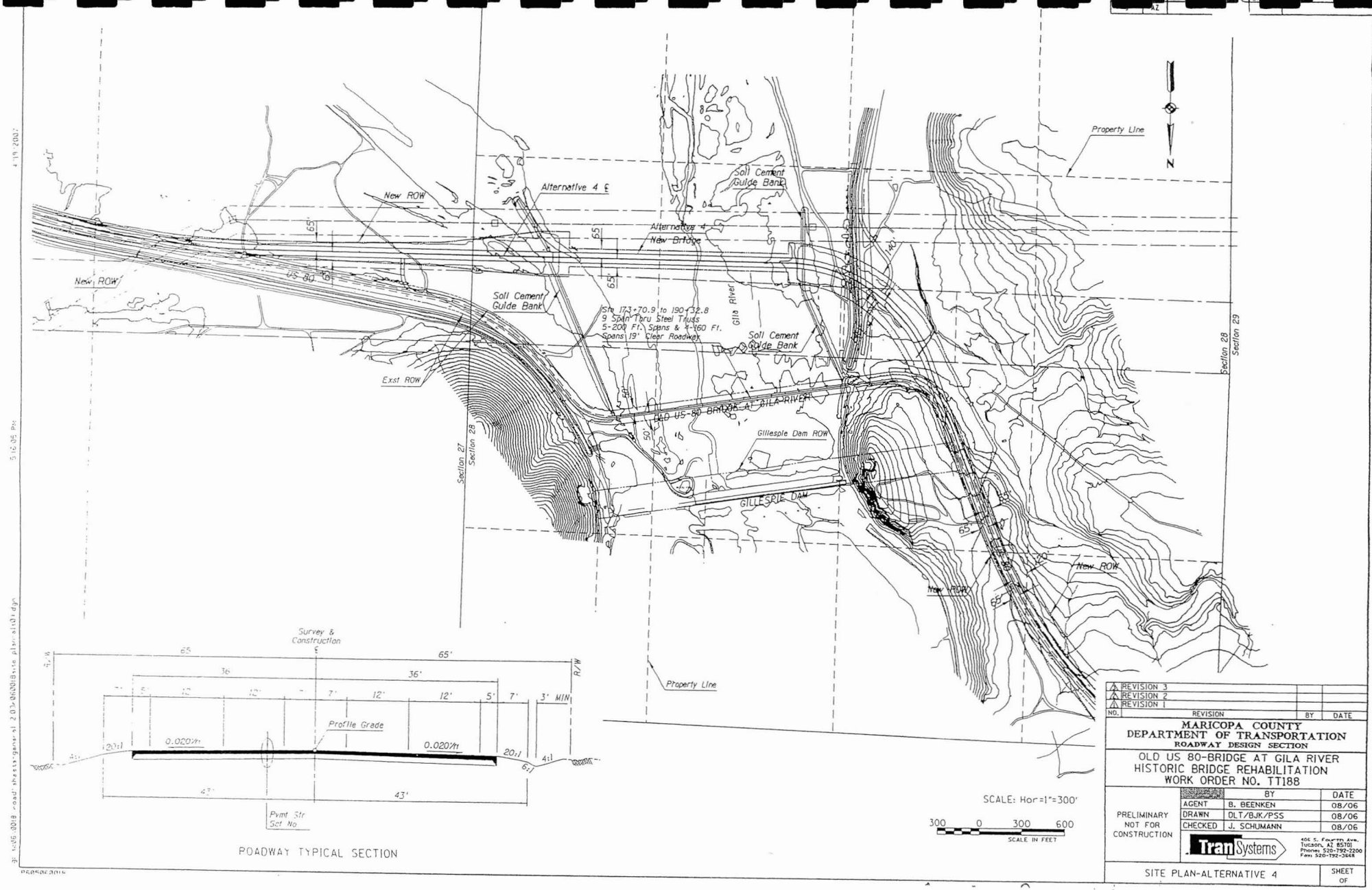
SHEET
OF

P605060018



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CHECKED	J. SCHUMANN	08/06

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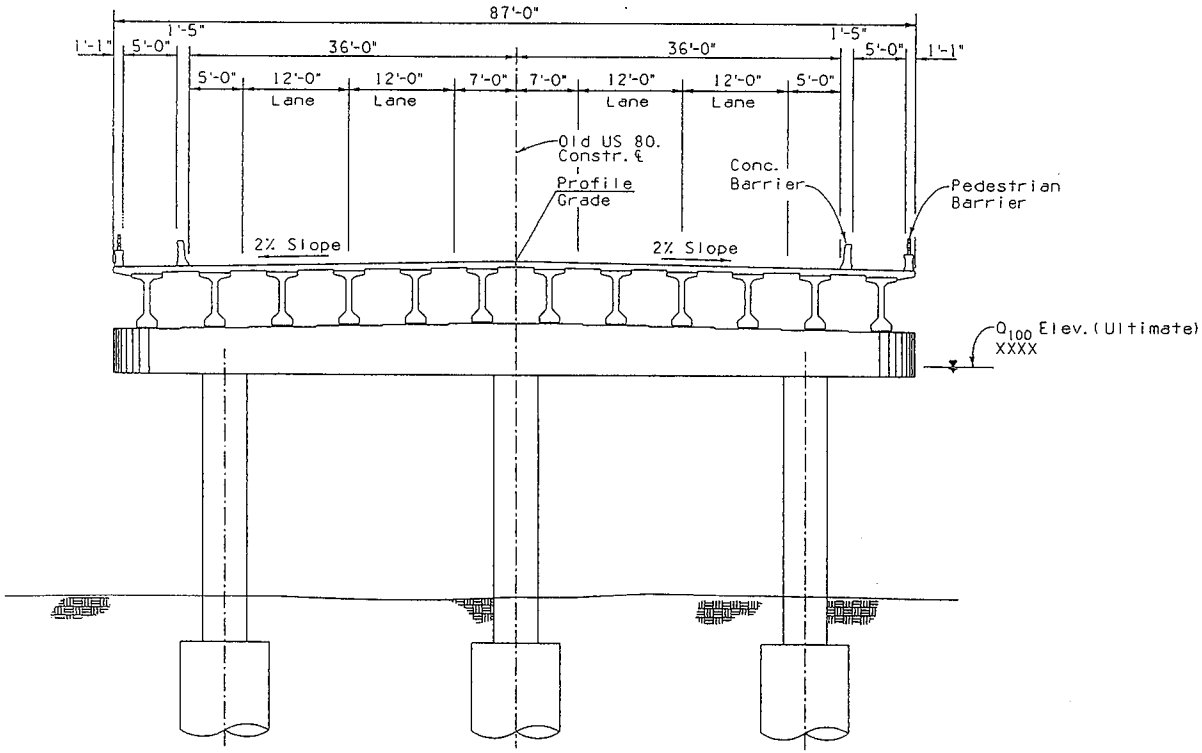
SITE PLAN-ALTERNATIVE 4

SHEET OF

Alternate Not. 4 – Future Recommended Alternate



TYPICAL SECTION ON STRUCTURE

(STRUCTURE SHOWN FOR GRAPHICAL REPRESENTATION ONLY.
DOES NOT INDICATE PREFERED ALTERNATIVE.)

ALTERNATIVE 4 (RECOMMENDED FOR FUTURE WHEN TRAFFIC REQUIRES NEW BRIDGE)

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
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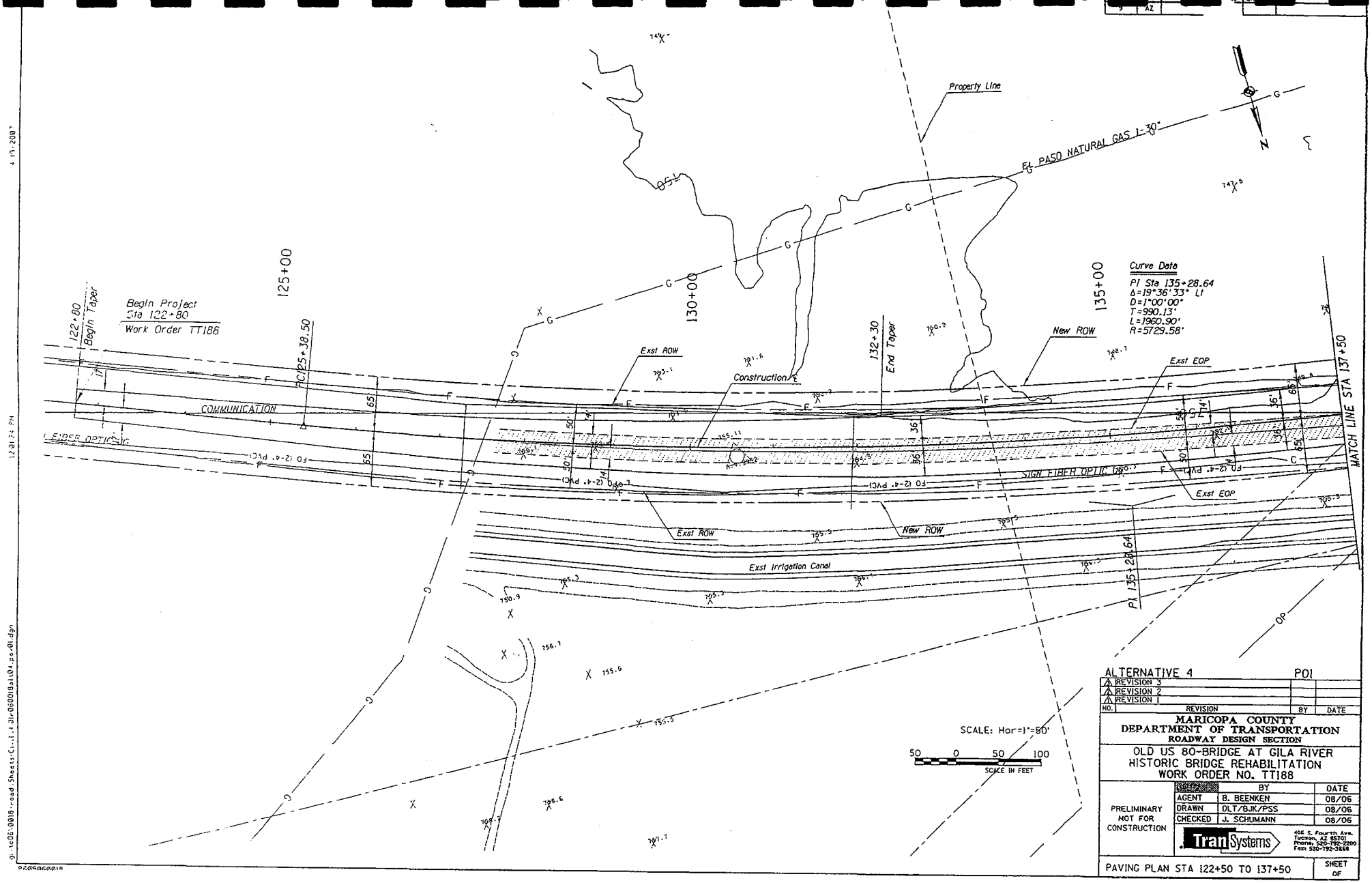
CONSTRUCTION

	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
405 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

BRIDGE TYPICAL SECTION

SHEET
OF



ALTERNATIVE 4 P01

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

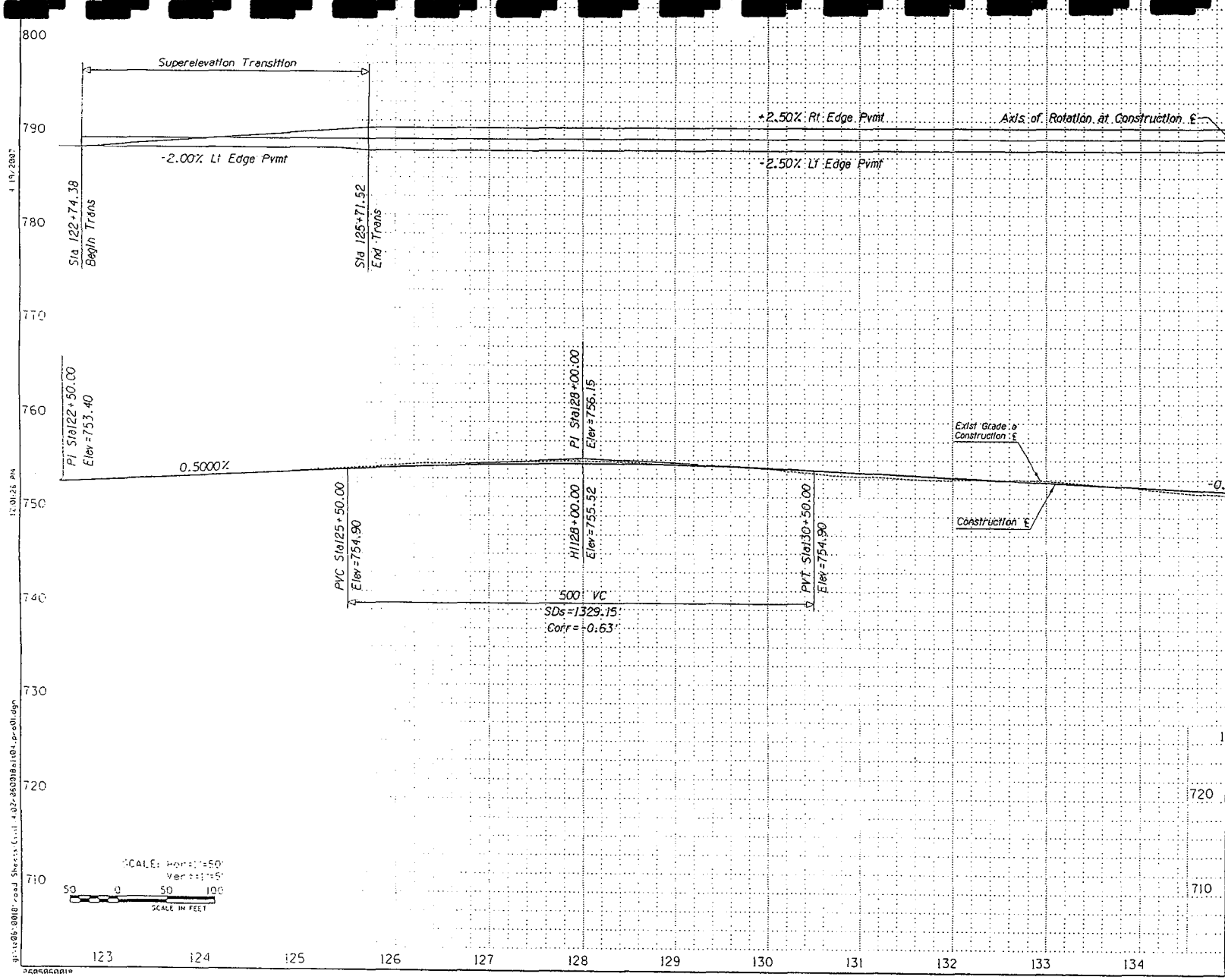
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

		BY	DATE
PRELIMINARY NOT FOR CONSTRUCTION	AGENT	B. BEENKEN	08/06
	DRAWN	DLT/BJK/PSS	08/06
	CHECKED	J. SCHUMANN	08/06

TranSystems
40E S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3666

PAVING PLAN STA 122+50 TO 137+50

SHEET OF



ALTERNATIVE 4 P02

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

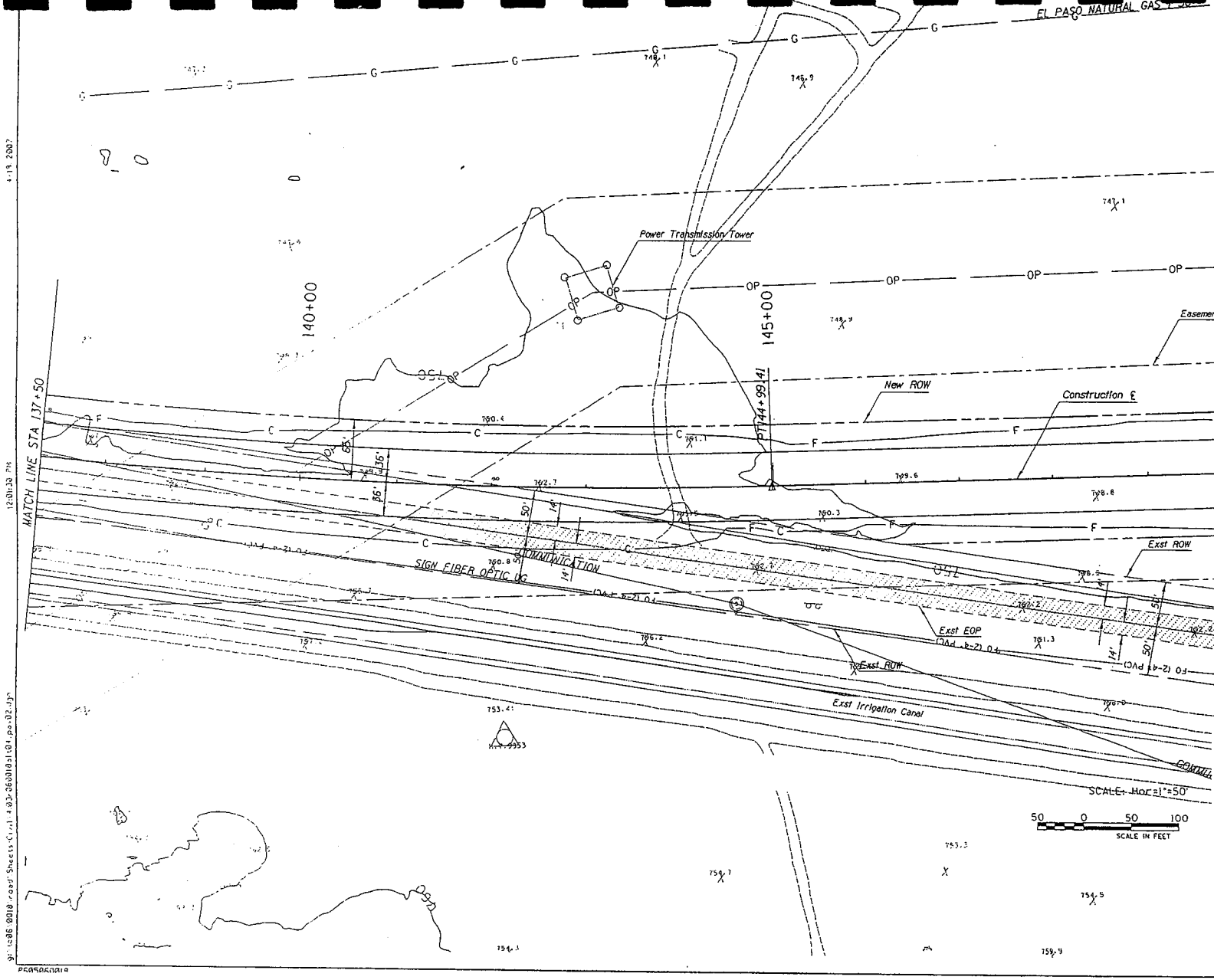
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

PRELIMINARY NOT FOR CONSTRUCTION

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-2668

PAVING PROFILE STA 122+50 TO 137+50

SHEET OF



ALTERNATIVE 4 P03

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

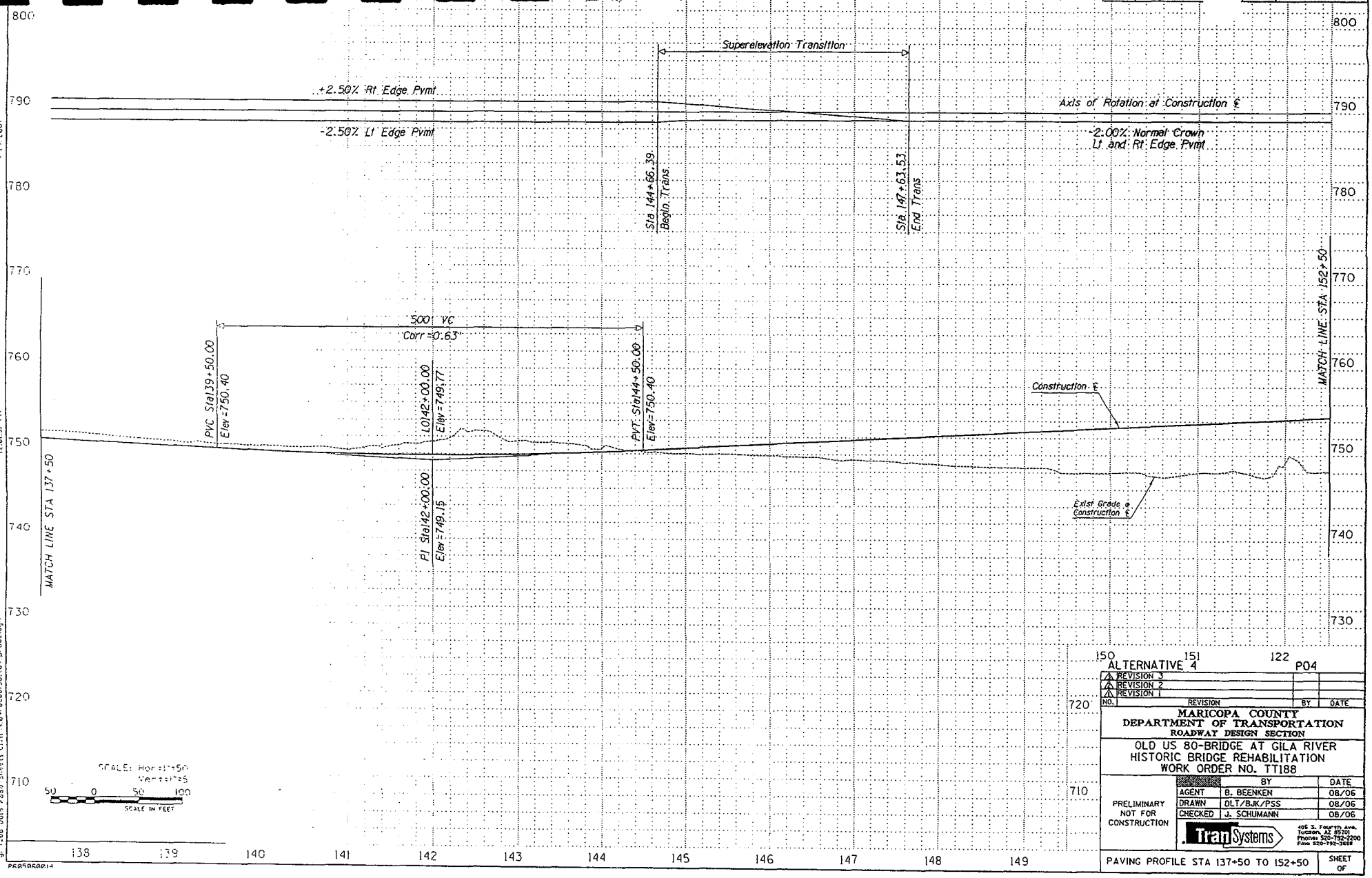
PRELIMINARY NOT FOR CONSTRUCTION

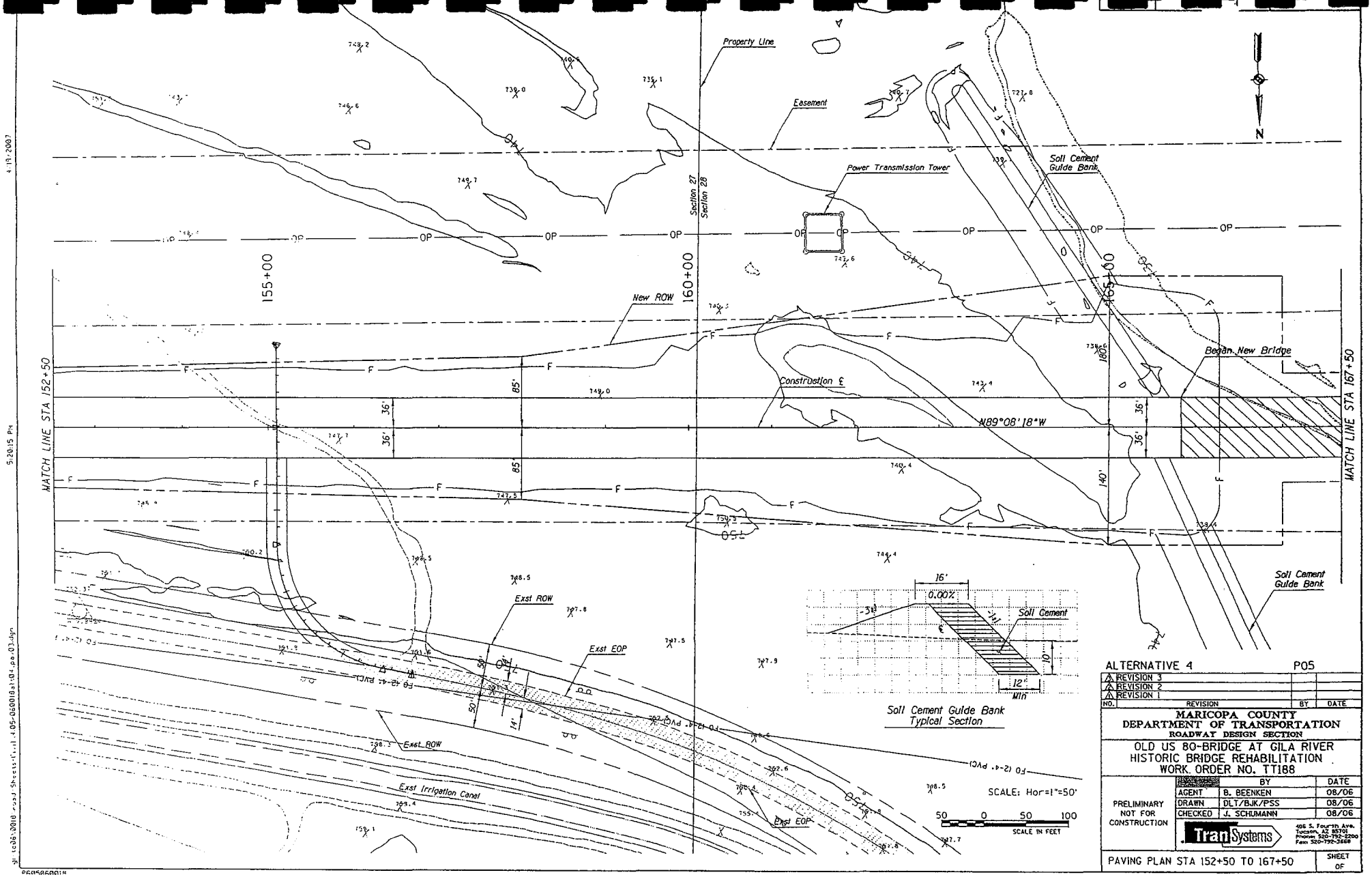
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

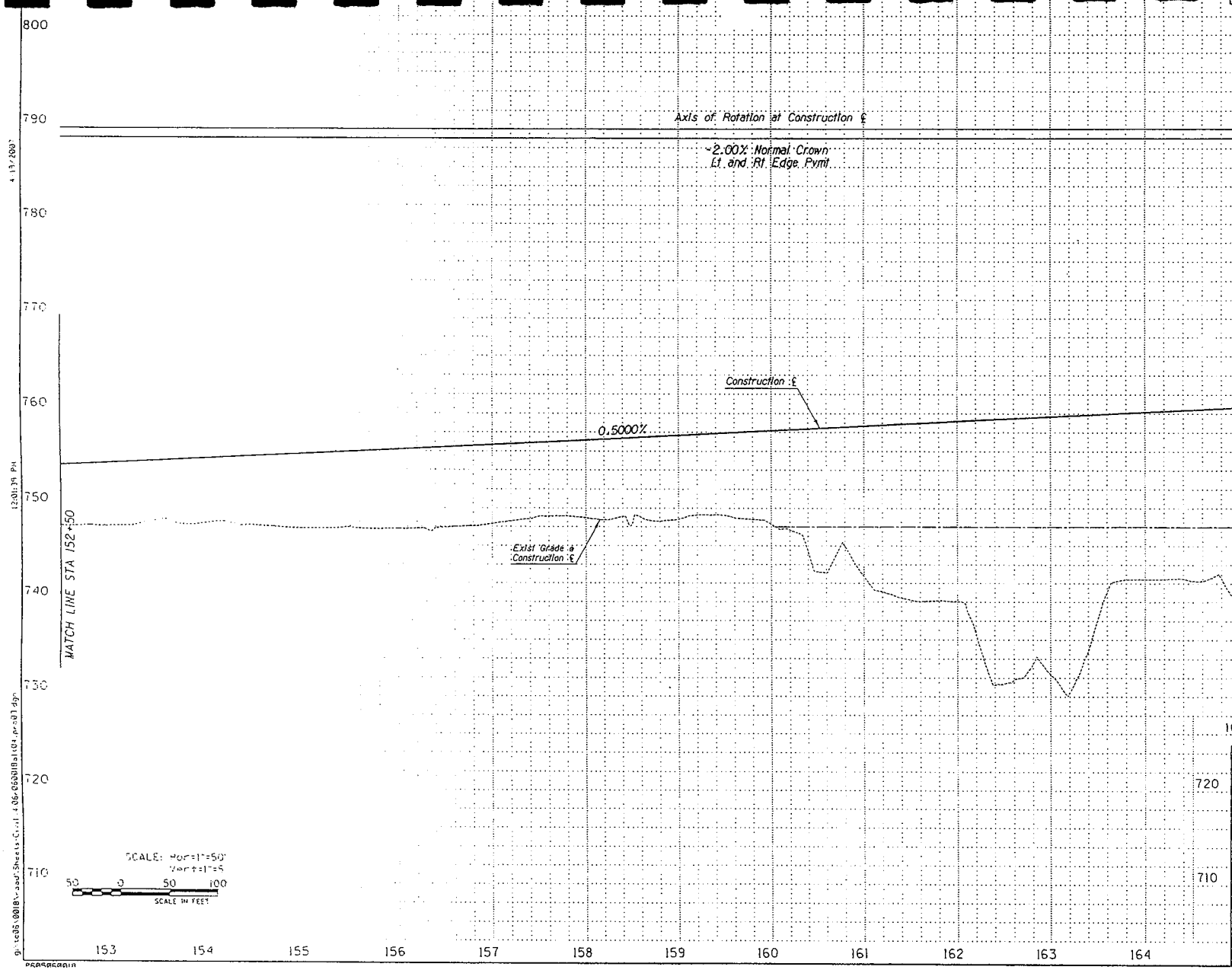
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

PAVING PLAN STA 137+50 TO 152+50

SHEET OF


Superelevation Transition
+2.50% Rt Edge Pvmt
-2.50% Lt Edge Pvmt
Axis of Rotation at Construction ℄
-2.00% Normal Crown
Lt and Rt Edge Pvmt
Sta 144+66.39
Begin Trans.
Sta 147+63.53
End Trans
500' VC
Corr = 0.63'
PVC Sta139+50.00
Elev = 750.40
L0142+00.00
Elev = 749.77
PI Sta142+00.00
Elev = 749.15
PVT Sta144+50.00
Elev = 750.40
Construction ℄
Exist Grade @ Construction ℄
MATCH LINE STA 137+50
MATCH LINE STA 152+50
SCALE: Horz 1"=50'
Vert 1"=5'
SCALE IN FEET
ALTERNATIVE 4
P04
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3468
PAVING PROFILE STA 137+50 TO 152+50
SHEET
OF


Property Line
Easement
Power Transmission Tower
Soil Cement Guide Bank
Section 27
Section 28
155+00
160+00
165+00
New ROW
Construction ℄
N89°08'18"W
Begin New Bridge
MATCH LINE STA 152+50
MATCH LINE STA 167+50
Exst ROW
Exst EOP
Exst Irrigation Canal
Soil Cement
Soil Cement Guide Bank Typical Section
SCALE: Hor=1"=50'
SCALE IN FEET
ALTERNATIVE 4
P05
REVISION 3
REVISION 2
REVISION 1
NO.
REVISION
BY
DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
PAVING PLAN STA 152+50 TO 167+50
SHEET
OF



ALTERNATIVE 4 P06

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

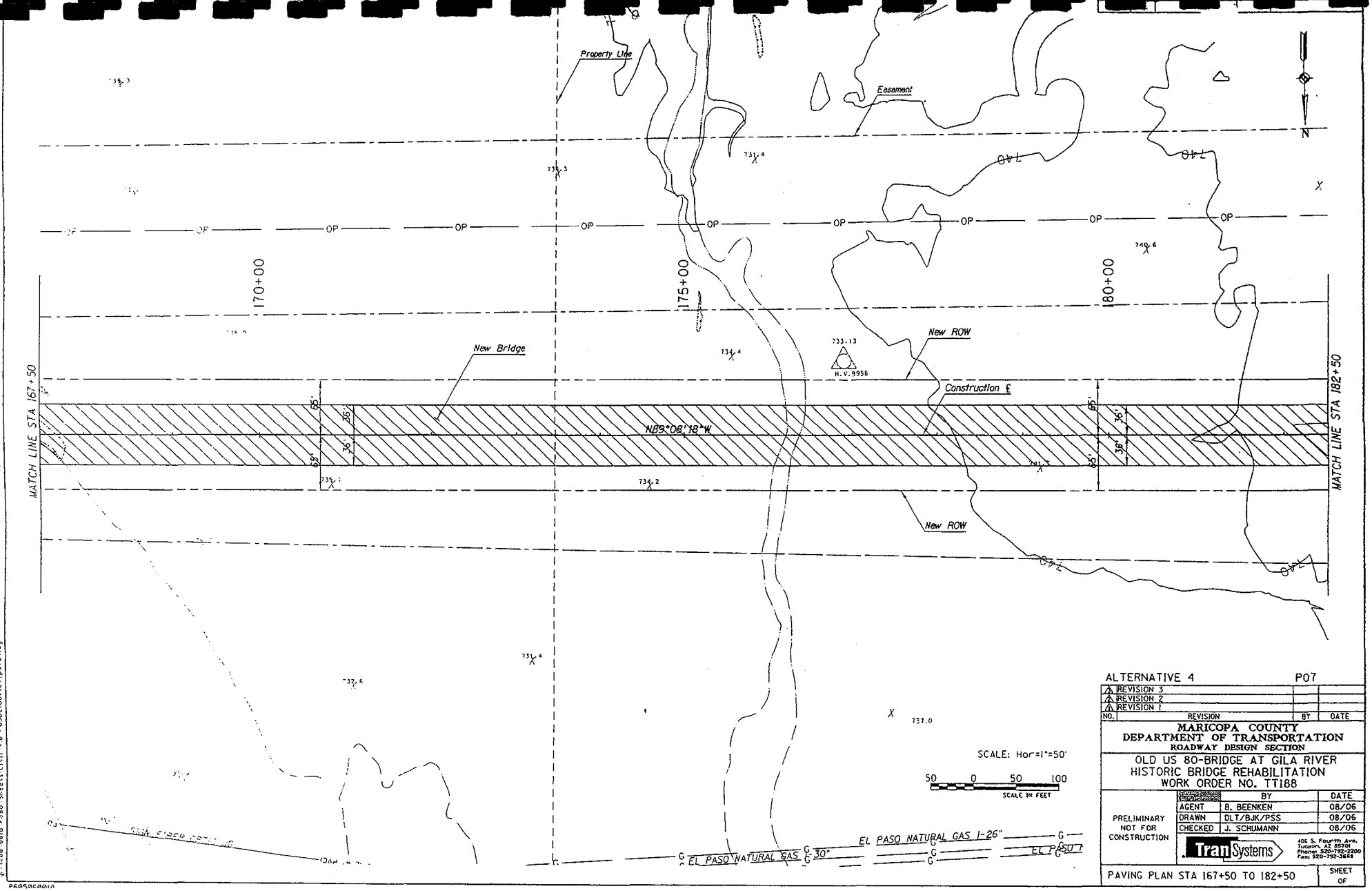
PRELIMINARY NOT FOR CONSTRUCTION

	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

PAVING PROFILE STA 152+50 TO 167+50

SHEET OF



ALTERNATIVE 4 P07

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

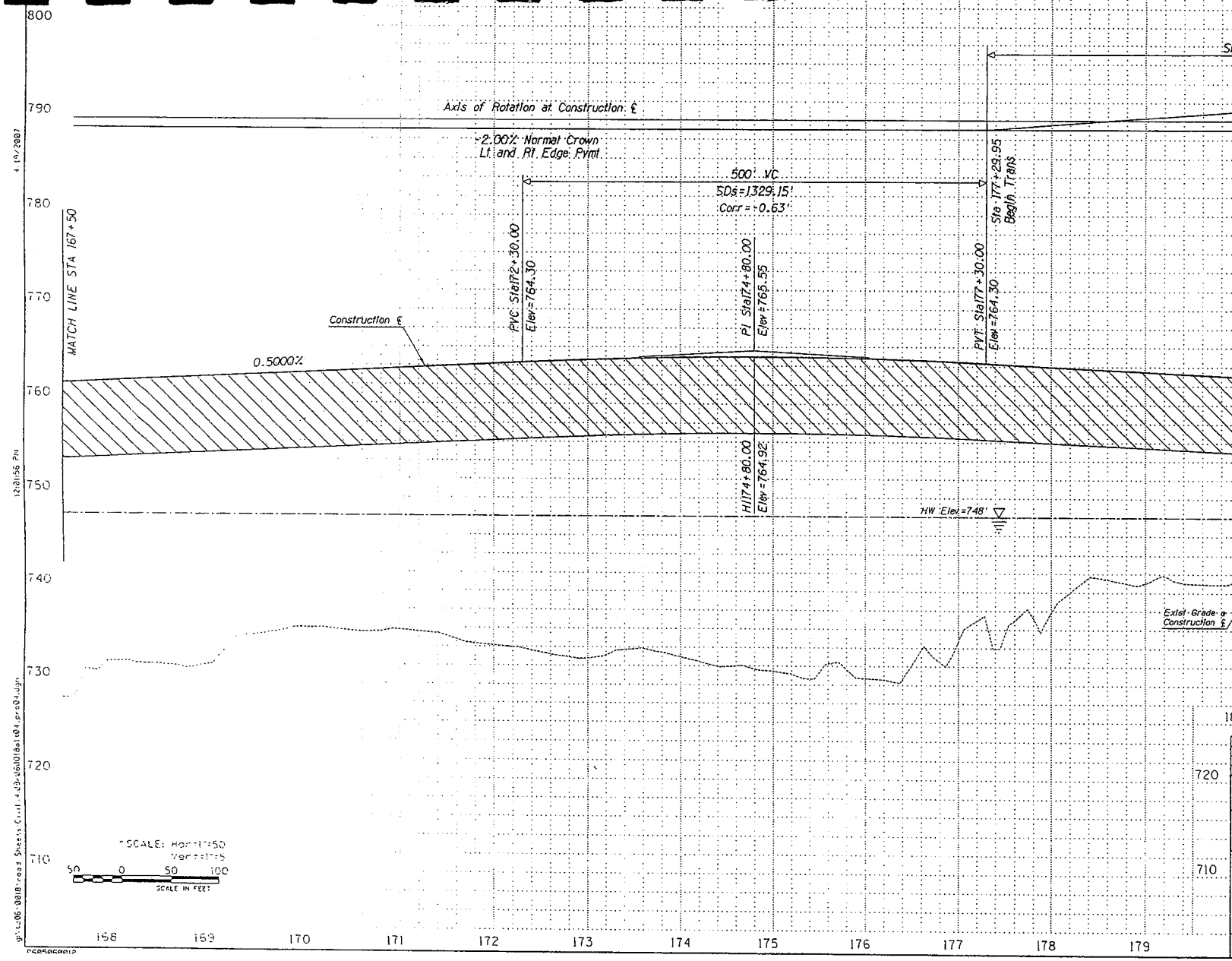
PRELIMINARY
NOT FOR
CONSTRUCTION

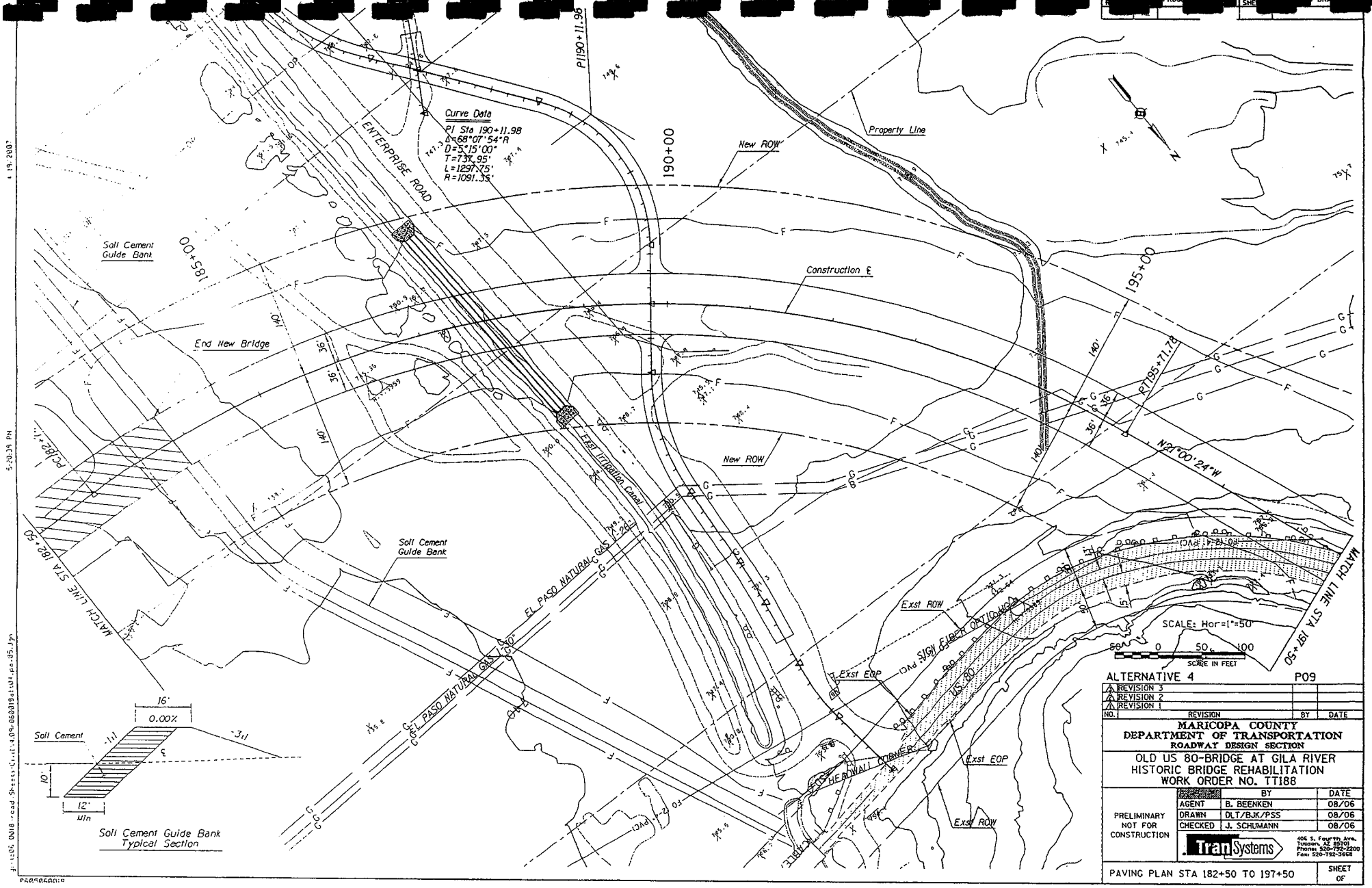
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

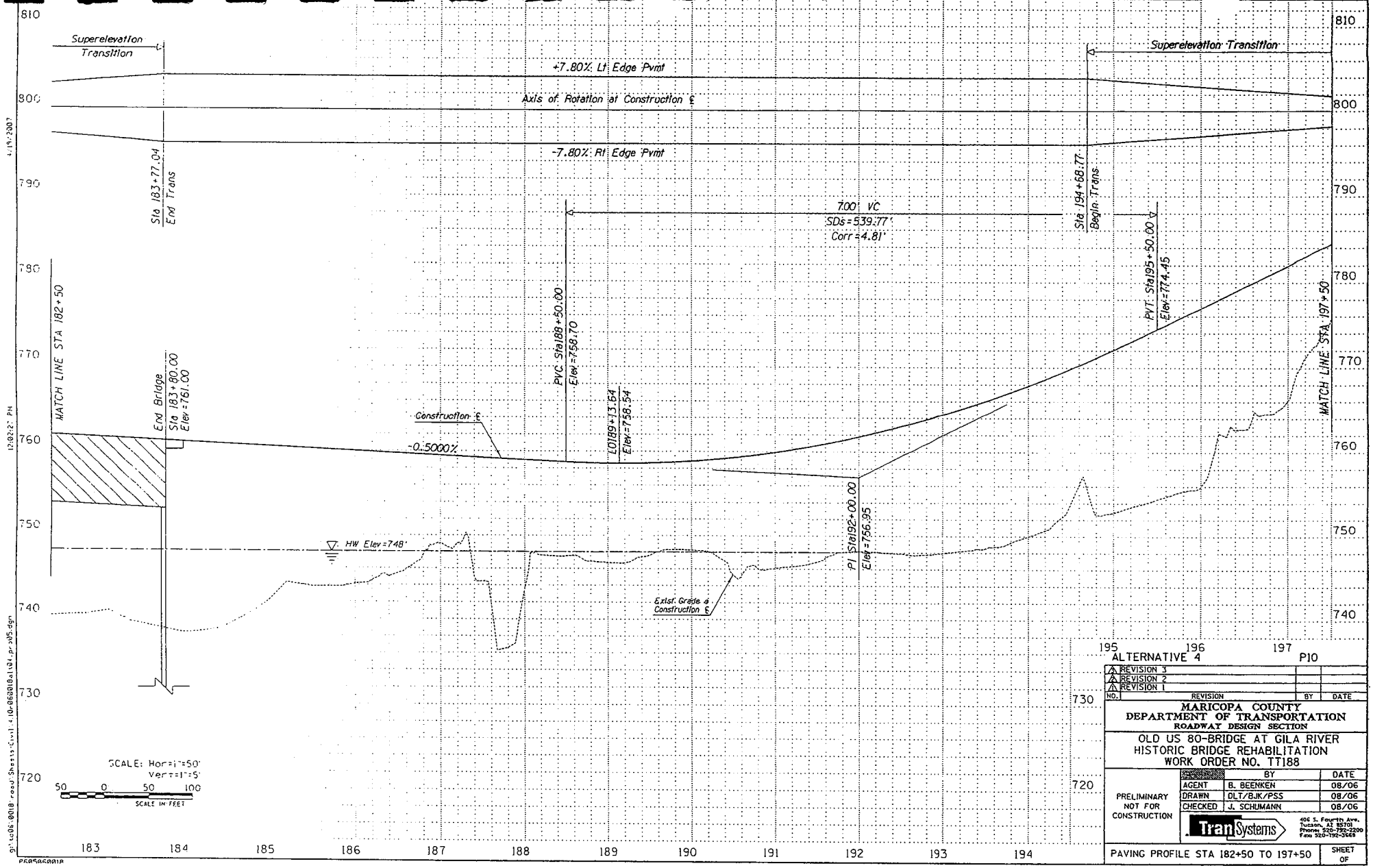
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3688

PAVING PLAN STA 167+50 TO 182+50

SHEET
OF


Superelevation Transition
Axis of Rotation at Construction ℄
-2.00% Normal Crown
Lt and Rt Edge Pvmt
500' VC
SDs=1329.15'
Corr=-0.63'
Sta 177+29.95
Begin Trans
MATCH LINE STA 167+50
MATCH LINE STA 182+50
PVC Sta172+30.00
Elev=764.30
PI Sta174+80.00
Elev=765.55
PVT Sta177+30.00
Elev=764.30
HI174+80.00
Elev=764.92
Construction ℄
0.5000%
-0.5000%
HW Elev=748'
Exist Grade @ Construction ℄
SCALE: Horz 1"=50
Vert 1"=5
SCALE IN FEET
ALTERNATIVE 4
P08
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY NOT FOR CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN OLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668
PAVING PROFILE STA 167+50 TO 182+50
SHEET OF


Curve Data
PI Sta 190+11.98
Δ=68°07'54"R
D=5°15'00"
T=737.95'
L=1297.75'
R=1091.35'
PI190+11.96
190+00
185+00
195+00
ENTERPRISE ROAD
Soil Cement Guide Bank
End New Bridge
New ROW
Property Line
Construction ℄
PT195+71.78
N27°00'24"W
Exst Irrigation Canal
EL PASO NATURAL GAS
Exst ROW
Exst EOP
SIGN FIBER OPTIC
US 80
HEADWALL CORNER
MATCH LINE STA 182+50
MATCH LINE STA 197+50
SCALE: Hor=1"=50'
SCALE IN FEET
Soil Cement
Soil Cement Guide Bank Typical Section
ALTERNATIVE 4
P09
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY NOT FOR CONSTRUCTION
BY DATE
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
405 S. Fourth Ave. Tucson, AZ 85701 Phone: 520-792-2200 Fax: 520-792-3668
PAVING PLAN STA 182+50 TO 197+50
SHEET OF


Superelevation Transition
+7.80% Lt Edge Pvmt
Axis of Rotation at Construction ℄
-7.80% Rt Edge Pvmt
Sta 183+77.04
End Trans
MATCH LINE STA 182+50
End Bridge
Sta 183+80.00
Elev=761.00
Construction ℄
-0.5000%
PVC Sta188+50.00
Elev=758.70
LOI189+13.64
Elev=758.54
HW Elev=748'
PI Sta192+00.00
Elev=756.95
Exist. Grade @ Construction ℄
7.00' VC
SDs=539.77'
Corr=4.81'
Sta 194+68.77
Begin Trans
PVT Sta195+50.00
Elev=774.45
Superelevation Transition
MATCH LINE STA 197+50
SCALE: Horz=1"=50'
Vert=1"=5'
SCALE IN FEET
ALTERNATIVE 4
P10
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY NOT FOR CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3468
PAVING PROFILE STA 182+50 TO 197+50
SHEET OF

ALTERNATIVE 4 P11

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

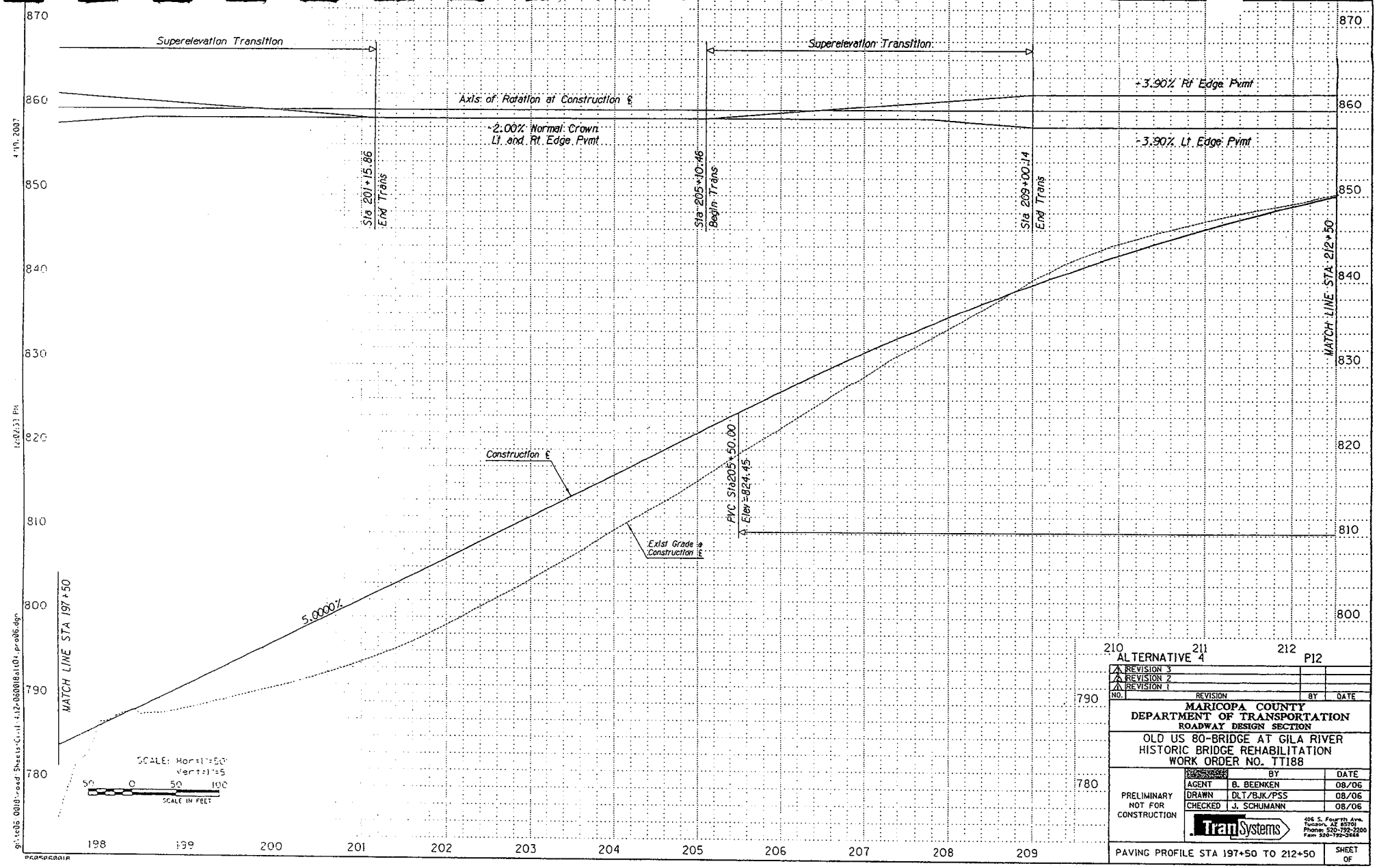
PRELIMINARY
NOT FOR
CONSTRUCTION

	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
405 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

PAVING PLAN STA 197+50 TO 212+50

SHEET
OF



ALTERNATIVE 4

P12

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

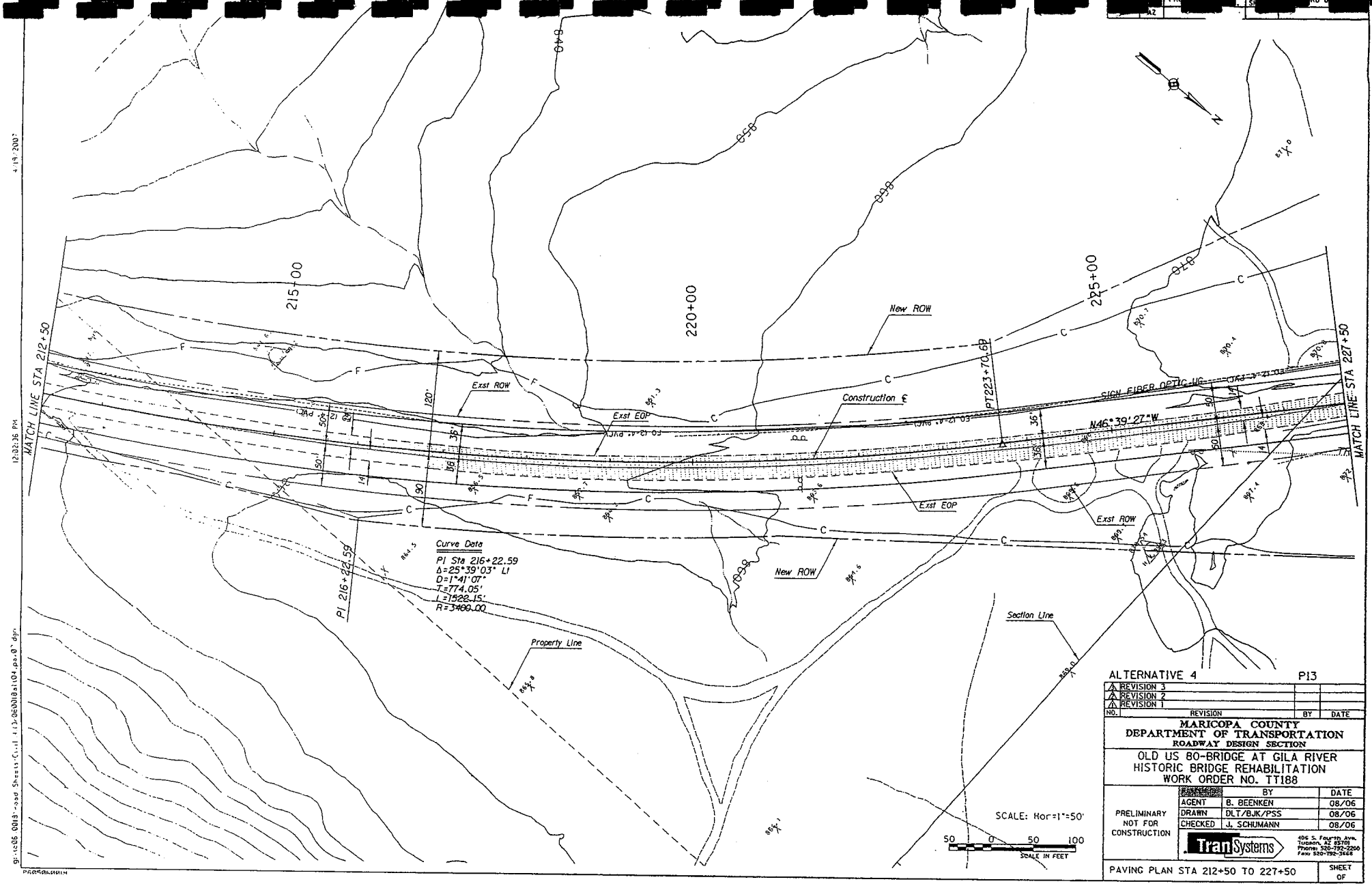
PRELIMINARY
NOT FOR
CONSTRUCTION

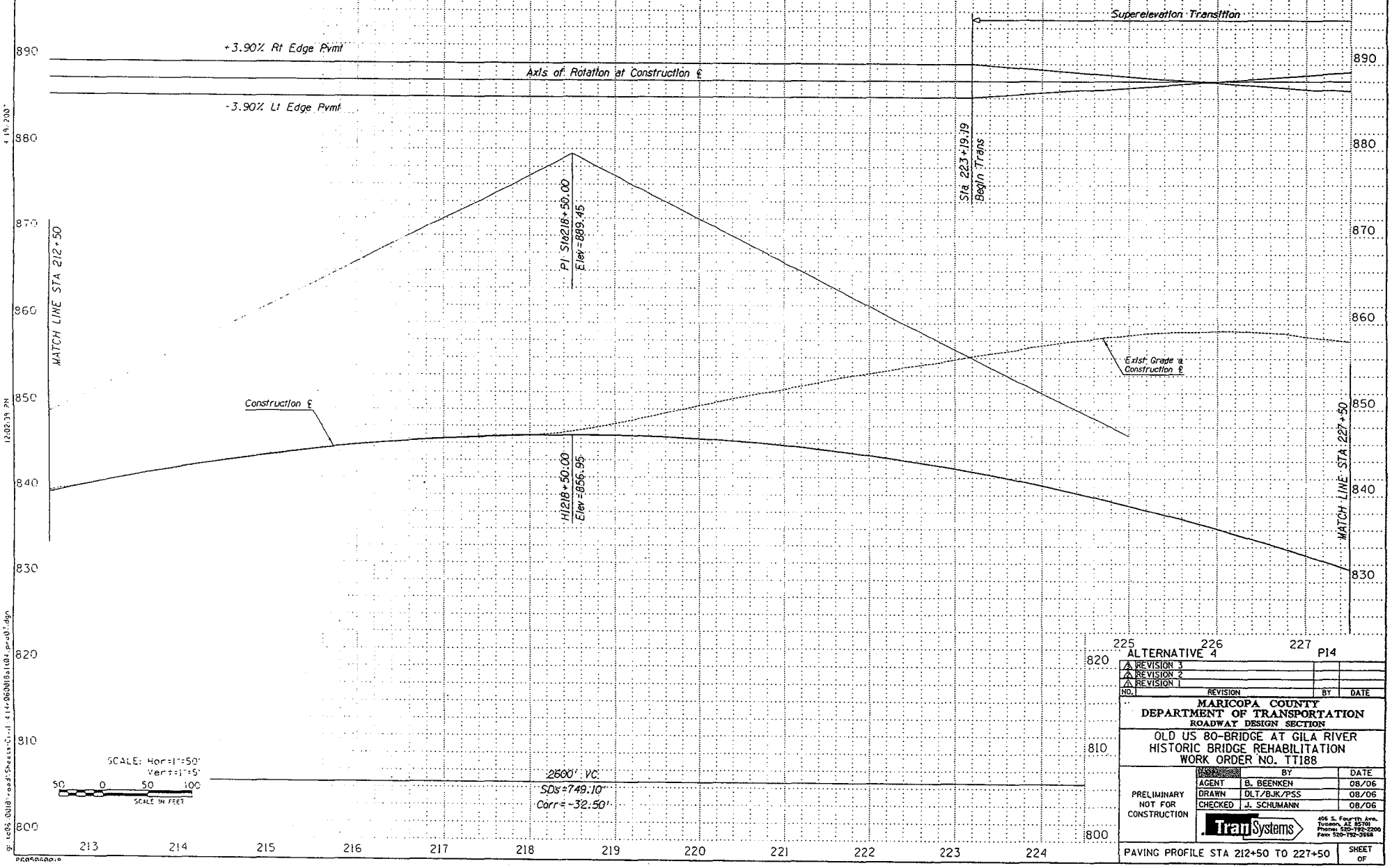
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

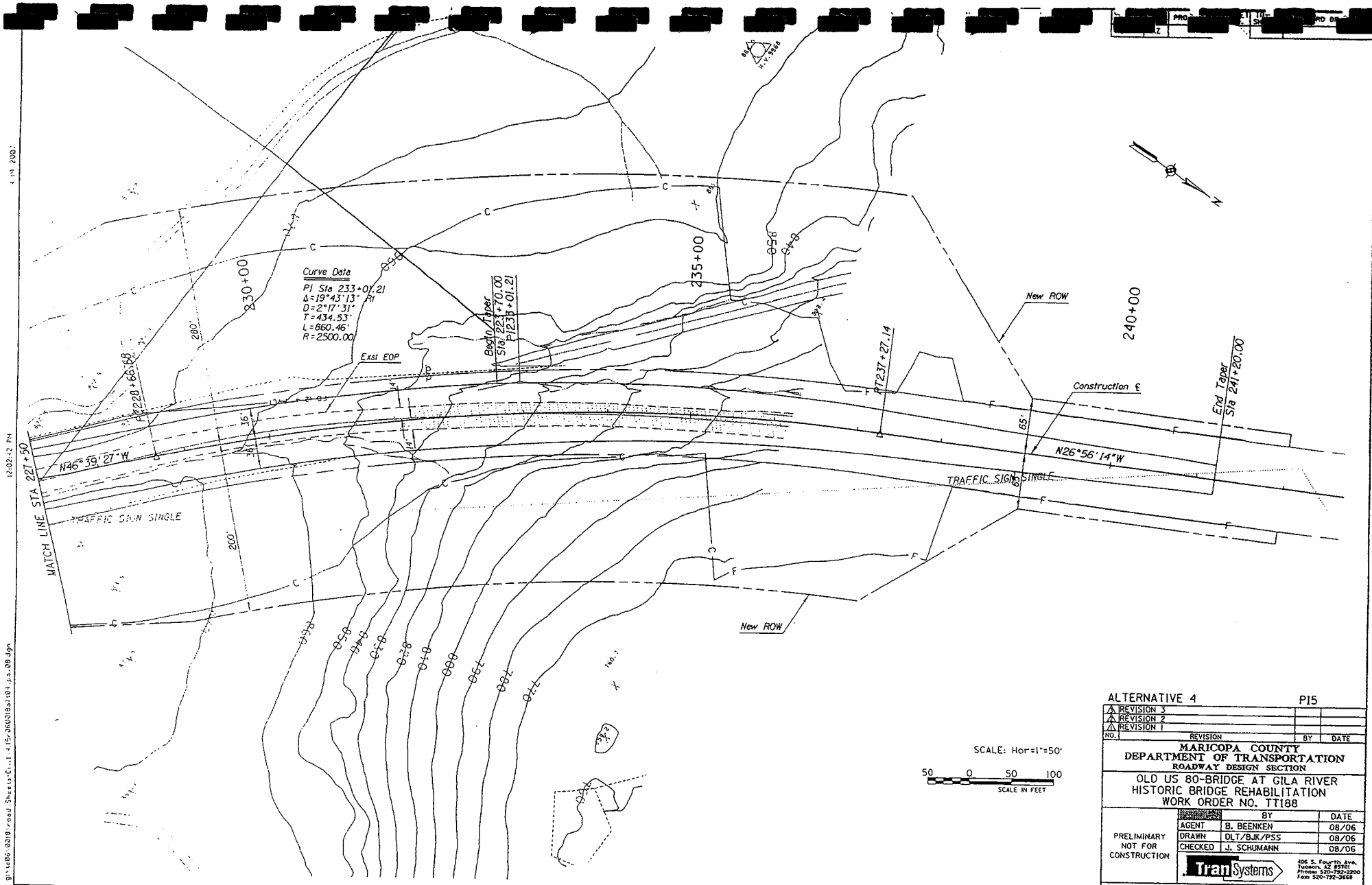
TranSystems
405 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-2668

PAVING PROFILE STA 197+50 TO 212+50

SHEET
OF


MATCH LINE STA 212+50
MATCH LINE STA 227+50
215+00
220+00
225+00
New ROW
Exst ROW
Exst EOP
Construction ℄
PT223+70.68
SIGN FIBER OPTIC UG
N46°39'27"W
Curve Data
PI Sta 216+22.59
Δ=25°39'03" LT
D=1°41'07"
T=774.05'
L=1522.15'
R=3400.00
PI 216+22.59
Property Line
Section Line
SCALE: Hor=1"=50'
SCALE IN FEET
ALTERNATIVE 4
P13
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668
PAVING PLAN STA 212+50 TO 227+50
SHEET
OF


Superelevation Transition
+3.90% Rt Edge Pvmt
Axis of Rotation at Construction ℄
-3.90% Lt Edge Pvmt
MATCH LINE STA 212+50
PI Sta 218+50.00
Elev=889.45
Sta 223+19.19
Begin Trans
Exist Grade @ Construction ℄
Construction ℄
HI 218+50.00
Elev=856.95
MATCH LINE STA 227+50
890
880
870
860
850
840
830
820
810
800
213
214
215
216
217
218
219
220
221
222
223
224
225
226
227
SCALE: Hor=1"=50'
Ver=1"=5'
50 0 50 100
SCALE IN FEET
2600' VC
SDs=749.10'
Corr=-32.50'
ALTERNATIVE 4
P14
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
BY DATE
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
405 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668
PAVING PROFILE STA 212+50 TO 227+50
SHEET
OF



ALTERNATIVE 4 PI5

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

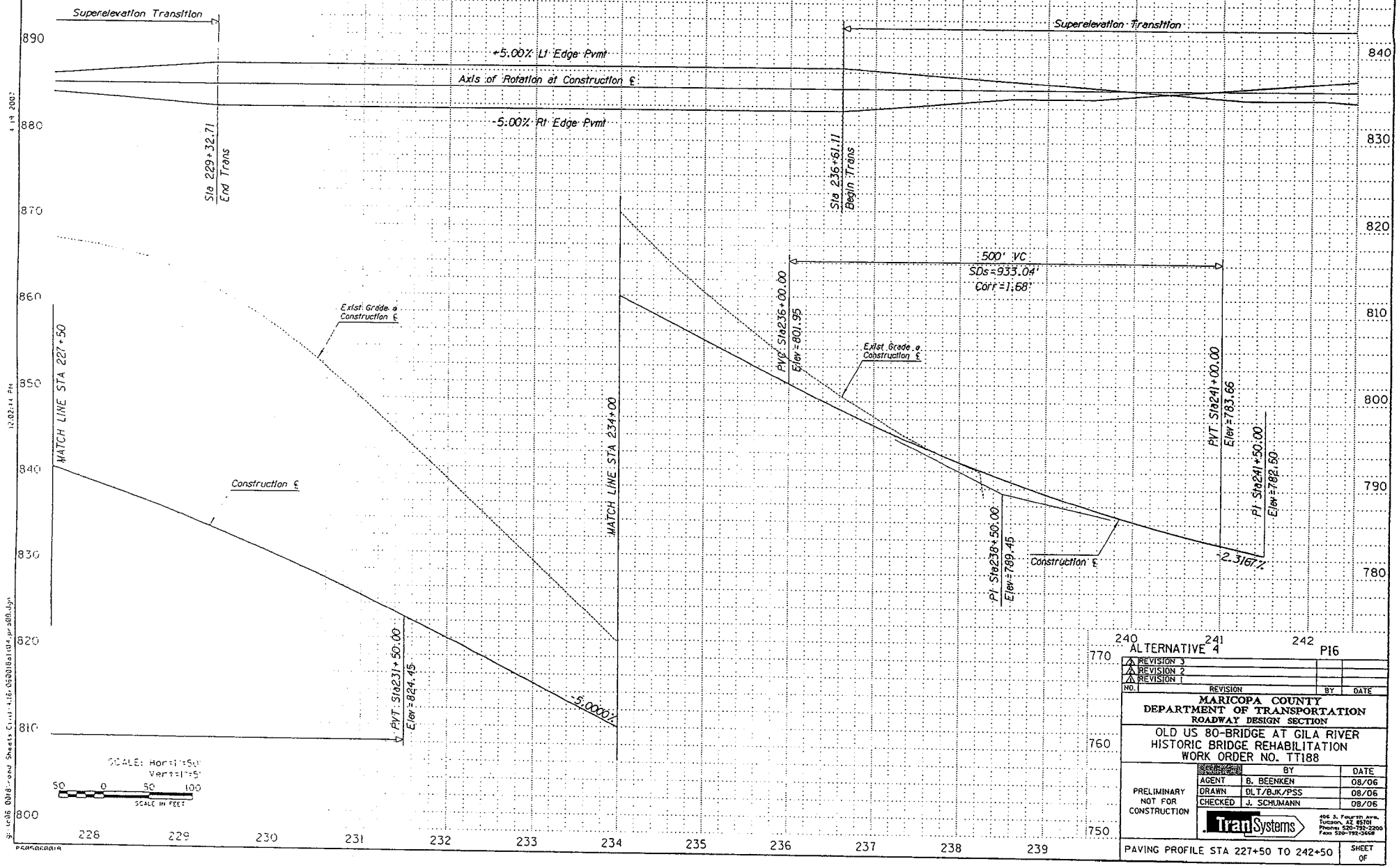
PRELIMINARY NOT FOR CONSTRUCTION

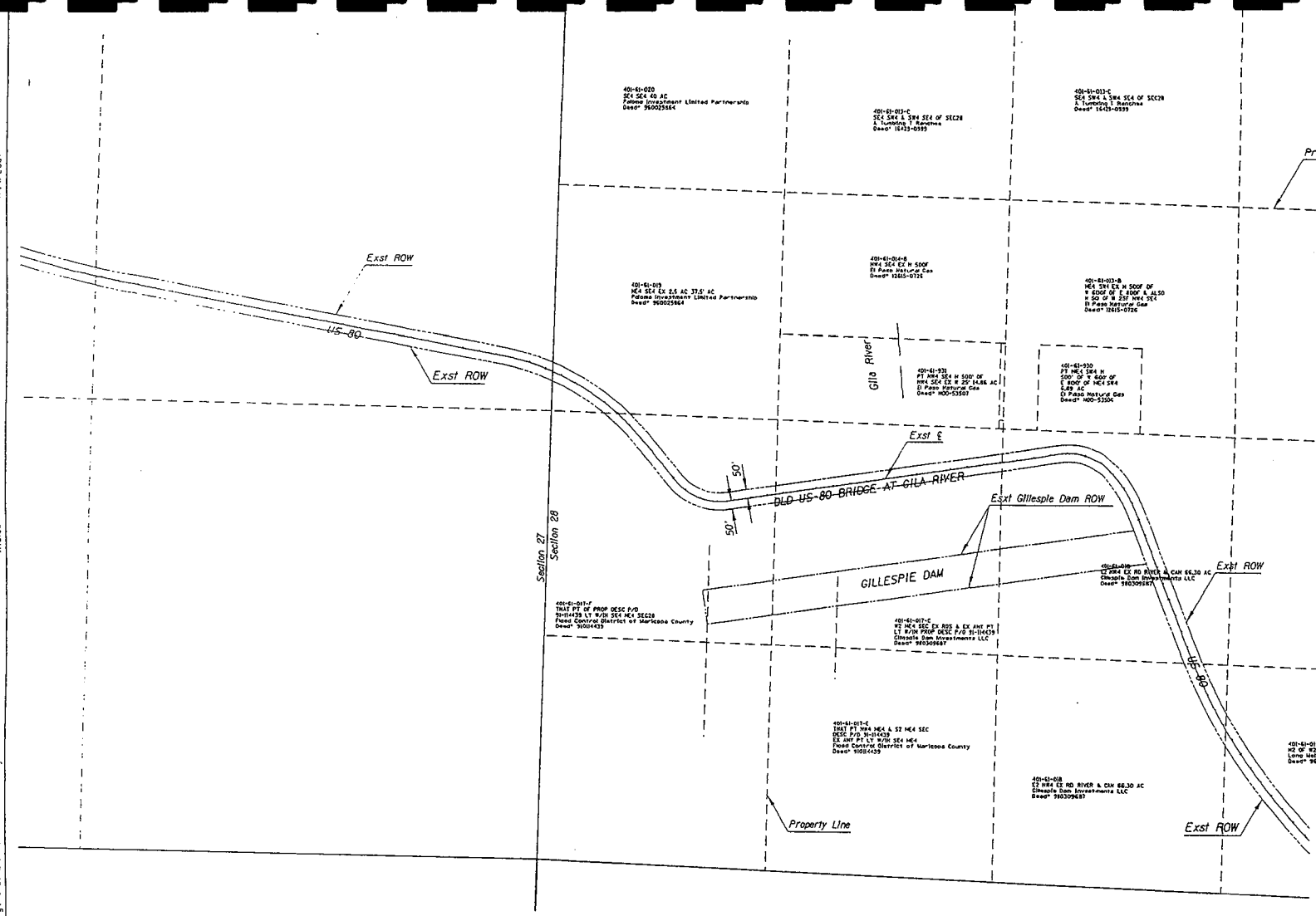
	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

PAVING PLAN STA 227+50 TO 242+50

SHEET OF


Superelevation Transition
+5.00% Lt Edge Pvmt
Axis of Rotation at Construction ℄
-5.00% Rt Edge Pvmt
Sta 229+32.71
End Trans
Sta 236+61.11
Begin Trans
Superelevation Transition
500' VC
SDs=933.04'
Corr=1.68'
PVC Sta236+00.00
Elev=801.95
PVT Sta241+00.00
Elev=783.66
PI Sta241+50.00
Elev=782.50
PI Sta238+50.00
Elev=789.45
-2.3167%
MATCH LINE STA 227+50
MATCH LINE STA 234+00
Exist Grade @ Construction ℄
Construction ℄
PVT Sta231+50.00
Elev=824.45
-5.0000%
SCALE: Hor=1"=50'
Vert=1"=5'
SCALE IN FEET
ALTERNATIVE 4
PI6
REVISION 3
REVISION 2
REVISION 1
NO. REVISION BY DATE
MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION
OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188
PRELIMINARY
NOT FOR
CONSTRUCTION
AGENT B. BEENKEN 08/06
DRAWN DLT/BJK/PSS 08/06
CHECKED J. SCHUMANN 08/06
TranSystems
PAVING PROFILE STA 227+50 TO 242+50
SHEET
OF



SCALE: Hor=1"=300'

300 0 300 600

SCALE IN FEET

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

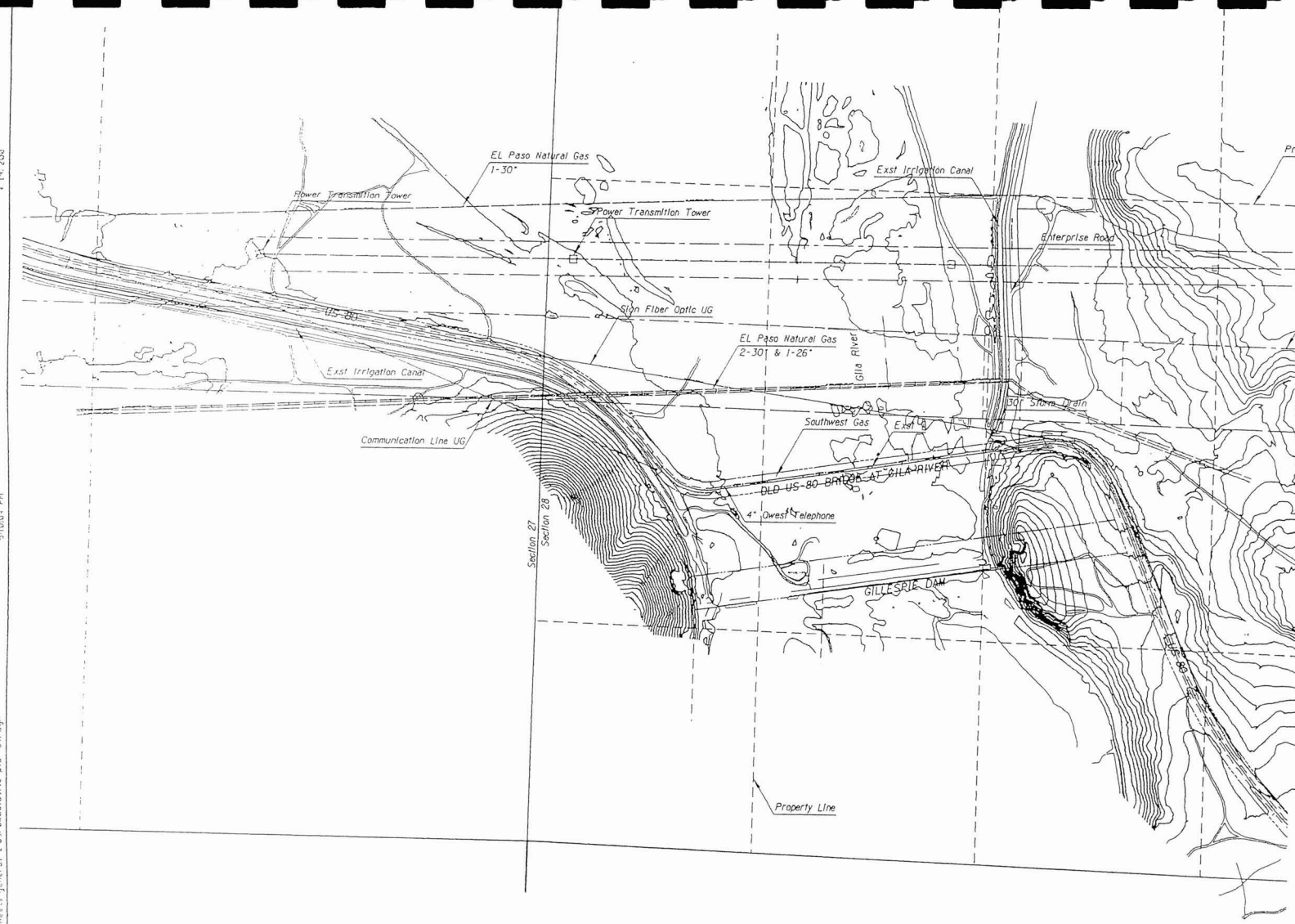
PRELIMINARY
NOT FOR
CONSTRUCTION

	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

TranSystems
406 S. Fourth Ave.
Tucson, AZ 85701
Phone: 520-792-2200
Fax: 520-792-3668

EXISTING RIGHT-OF-WAY PLAN

SHEET
OF



SCALE: Hor=1"=300'

300 0 300 600
SCALE IN FEET

NO.	REVISION	BY	DATE
3	REVISION 3		
2	REVISION 2		
1	REVISION 1		

MARICOPA COUNTY
DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN SECTION

OLD US 80-BRIDGE AT GILA RIVER
HISTORIC BRIDGE REHABILITATION
WORK ORDER NO. TT188

PRELIMINARY
NOT FOR
CONSTRUCTION

	BY	DATE
AGENT	B. BEENKEN	08/06
DRAWN	DLT/BJK/PSS	08/06
CHECKED	J. SCHUMANN	08/06

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EXISTING UTILITY PLAN

SHEET
OF

APPENDIX C – Public Meeting Handouts

Historic Old US 80 Bridge at Gila River (Gillespie Dam Bridge)

Bridge Rehabilitation & Design Concept Report

MCDOT RightRoads PROGRAM

The Right System The Right Time The Right Cost

Maricopa County Department of Transportation **October 23, 2006**

Project Description

The Maricopa County Department of Transportation (MCDOT) is conducting a Design Concept Report (DCR) of the historic Gillespie Dam Bridge on US 80 at the Gila River to evaluate existing and future traffic needs. Over time, crossing vehicles have damaged the bridge. Some bridge components, due to age and natural deterioration, are in need of rehabilitation or replacement. Damage and pier scour caused by the Gillespie Dam failure in January 1993 have raised additional concerns about the structural integrity of the bridge. This study will examine rehabilitating the existing historic bridge and develop a scope of work and associated cost estimate.

In addition to evaluating bridge rehabilitation, the study is evaluating long-term transportation needs of the area. A future location for a proposed new bridge is also being evaluated to address forecasted area growth, development and resulting traffic demand.

Project Background

The Gillespie Dam Bridge on Old US 80 was constructed in 1927 and is listed on the National Register of Historic Places. The bridge is located between Gila Bend and the community of Arlington. At a length of 1665 feet, the bridge is one of the longest in the state and was a vital link in the US 80 (Ocean-to-Ocean Highway) that connected San Diego, California to Savannah, Georgia.

The DCR is a comprehensive report that uses more complex projects as a prelude to actual design. Its objective is to develop a project scope; estimated construction cost and identify a proposed schedule. This information is then used to program the project (reserve funds) into MCDOT's Transportation Improvement Program.

The DCR includes the collection of relevant data and the development of several reports and studies including:

- Geotechnical Investigation Report
- In-depth Bridge Inspection Report
- Bridge Load Rating Report
- Existing and Future Traffic Analysis Report
- Hydrologic, Hydraulic and Scour Analysis
- Environmental Reports
- Archeological Report

The study incorporates all report findings and information, along with public input, to evaluate and to develop different rehabilitation and future conceptual design alternatives. Based on this information, a "preferred" alternative is selected and a scope of work, estimated construction cost and an implementation plan is formulated.

Goals/Objectives

- Provide for current vehicle usership and pedestrian uses on existing bridge
- Determine necessary repairs for the existing bridge
- Address future traffic demand
- Develop alternative alignment for new bridge and roadway
- Develop cost estimates for current and future needs

Issues/Challenges

- Incorporate projected future growth and development
- Identify and address future land use plans
- Forecast future traffic needs
- Coordinate with impacted stakeholders and National Register of Historic Places
- Identification of project funding sources
- Identification of new right-of-way requirements
- Utility coordination and accommodation

For more information, contact Andrzej Wojakiewicz at (602) 506-8625 or write to him at:
MCDOT, 2901 W. Durango Street, Phoenix, AZ 85009, or e-mail at: AndrzejWojakiewicz@mail.maricopa.gov.

Existing and Future Traffic Analysis

With the improvements to SR 85 (additional through lanes), the traffic volumes on Old US 80 have declined. This present reduction in traffic, however, is not anticipated to last due to present and forecasted area development. In preparation of the Traffic Analysis Report, a traffic count and driver survey has been conducted. Traffic counts found the average daily traffic to be slightly more than 200 vehicles per day with 20-percent of those vehicles being larger than passenger car/pick up truck.

Based on the zoning of the surrounding area and available or undeveloped land, future traffic projections were formulated in order to evaluate the existing roadway's capabilities and to identify the roadway type and size that will be needed in the future to accommodate increased travel demand.

Over 6,700 acres of land are available for potential development in the traffic influence area. The zoning in the region currently allows three homes per acre. Depending on the development forces, the traffic across the Gillespie Dam Bridge could potentially increase to almost 18,000 vehicles per day using a future development density projection of one home per acre and to over 40,000 vehicles a day traveling over Gillespie Dam Bridge using a future development density projection of three homes per acre.

This information will be used in the DCR to help the project team determine the future bridge and roadway type (number of lanes/configuration/access control) and in the development of an implementation plan for the rehabilitation of the existing historic bridge and the construction of a new bridge.

Advanced Design Concept Report Alternatives

Alternative 2 Rehabilitate Existing Bridge

This alternative will determine the feasibility of rehabilitating the existing structure and provide an estimated construction cost. The historic Gillespie Dam Bridge will be evaluated using state-of-the-art design software in accordance to current design and safety standards. An extensive investigation will be performed to evaluate the bridge piers and scour susceptibility. As a project enhancement, MCDOT is also investigating the addition (attachment) of a pedestrian walkway onto the existing narrow bridge due to its travel way width of only 19-feet.

Alternative 3 Construct New Bridge Parallel to Existing Bridge

This alternative will determine the feasibility of constructing a new bridge immediately south of the existing bridge and provide an estimated construction cost. This alternative minimizes estimated construction cost and the impact to the connecting Old US 80 roadway.

The sharp roadway curves preceding the approaches to the bridge (design speed of 35 MPH) limit the vehicle speed. As a result, under future increased traffic conditions, this alignment will limit the operational capacity of the roadway and bridge crossing.

The elevation of the new bridge will need to be approximately ten feet higher than the existing bridge to place the new bridge above the Gila River 100-Year Event for storm water elevation, the width and cross slope of the roadway and the potential height of the new bridge structure (beams, deck, etc.).

Raising the roadway would also make it difficult to maintain current roadway access points at each end of the bridge. The new bridge cost would also be increased since the pier/span configuration of both the old and new bridge need to match, due to such close proximity, to minimize scour and maximize the flow of the Gila River.

Concern also exists under this alternative for the additional scour potential the new bridge could have to the existing natural gas pipelines located immediately down stream.

Alternative 4 Construct New Bridge Downstream of Existing Bridge

This alternative will determine the feasibility of constructing a new bridge approximately 1000-feet downstream of the existing bridge and provide an estimated construction cost. This corridor alignment was selected because it is located between the El Paso Natural Gas property to the north and the APS power transmission line easement to the south. With the new bridge alignment downstream of the El Paso Natural Gas pipelines, there is no concern of impacting the pipelines due to additional scour potential.

The widening of the Gila River at this location would require a longer bridge (1800-feet) than Alternative 3. This alternative would also require

approximately 9,000 feet of new roadway to connect the new crossing and the Old US 80 roadway. This alternative would, however, allow maintenance of the existing access connections and maintain visual and physical access to the exiting historic Gillespie Dam Bridge for use as a pedestrian crossing.

With this alternative, the new roadway approaches (design speed of 55 MPH) to the new bridge would not limit the operational capacity of the new roadway and bridge crossing evidenced in Alternative 3.

At this location, the bridge pier/span configuration does not need to match the configuration of the existing old bridge upstream and potentially more economical bridge structure types could be utilized. This alternative would increase project cost due to additional roadway, bridge length and right-of-way needs.

Design Concept Phase Project Schedule

Project Start March 2006

Data Collection

- Existing and Future Traffic information
- Area Drainage Features/Characteristics
- Existing and Future Land Use
- Right-of-Way Identification/Requirements
- Environmental Assessment
- In-depth existing bridge inspection
- Utility information
- Historic Bridge Rehabilitation Report
- Geotechnical information
- Field Survey & Mapping

Engineering

- Facility type and roadway alignment conceptual alternatives
- New Bridge Type Selection Report
- Preferred alternative selection
- Cost estimates

Draft Report Submittal September 27, 2006

Public Input Meeting October 23, 2006

Completed DCR Environmental Study December 2006

This project is currently not funded for either interim ·habilitation constriction or new bridge/roadway construction.

Gillespie Dam Bridge Rehabilitation

This project will prepare an expanded Design Concept Report (DCR) for Alternative 2, the rehabilitation of the historic Old US 80 Bridge over the Gila River (Gillespie Dam Bridge). This nine-span, 1665-foot long, through-truss steel bridge was built in 1927 in the general vicinity of Arlington in Maricopa County and is listed in the National Historic Sites Register. The project limits include the bridge and an approximately half-mile-long segment of Old US 80 roadway alignment extending from each bridge abutment, for a total project length of approximately 1.32 miles.

Rehabilitation Goals/Objectives

- Rehabilitate the existing bridge for vehicular use
- Provide space for pedestrians to use the bridge
- Restore a valuable historic resource

Existing Deficiencies/ Issues/Challenges

- Narrow two-lane bridge
- Posted load capacity is 20 tons
- Posted vertical clearance is 13'-6"
- Pedestrians must share bridge with vehicles
- Bridge roller bearings need repair
- Bridge concrete deck is cracking
- Bridge barrier rail not strong enough to prevent damage to trusses
- Bent truss members need to be straightened
- Bridge needs to be repainted

Proposed Bridge Rehabilitation Project Schedule

Project Start March 2006

Draft DCR Submittal September 27, 2006

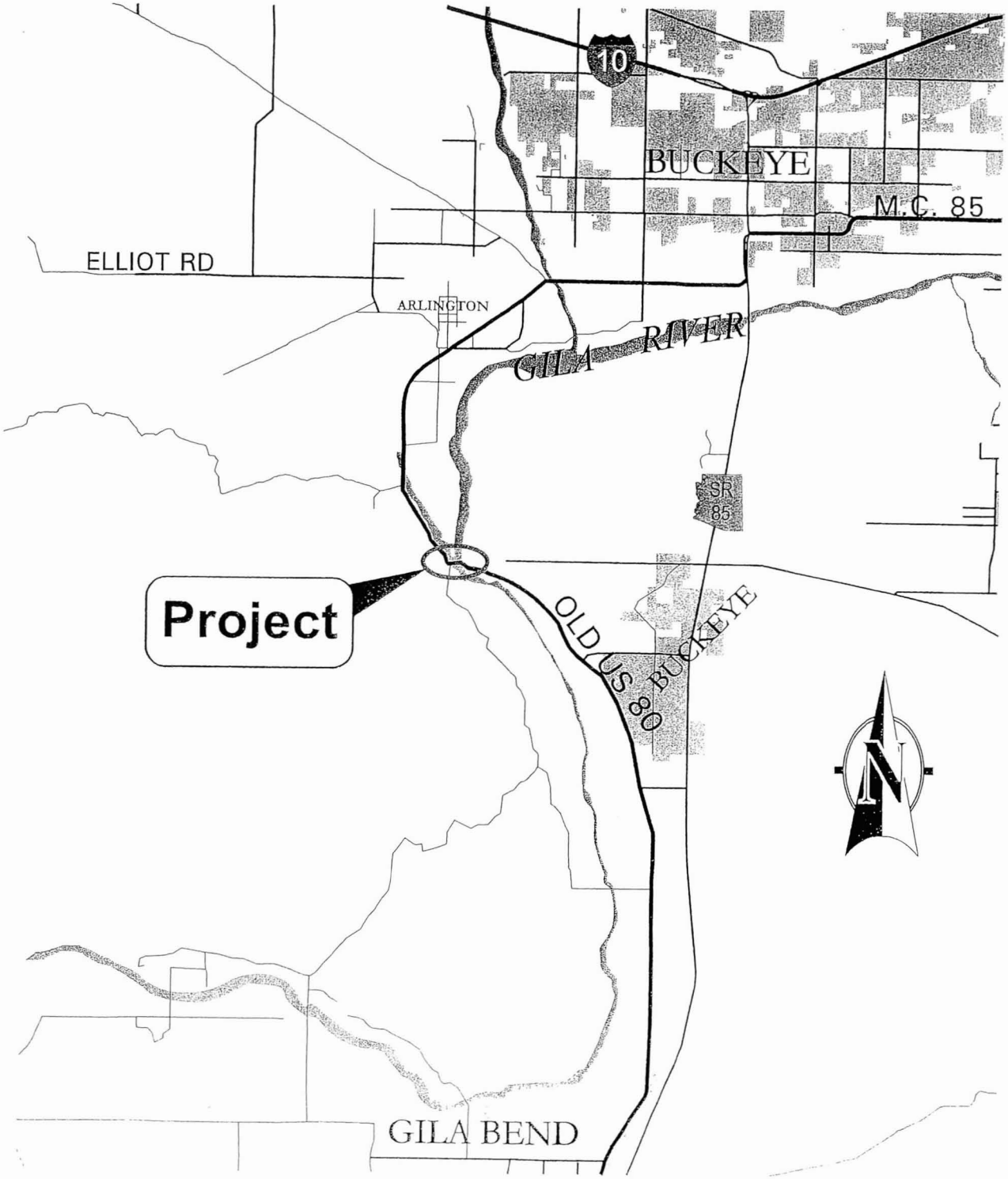
Public Input Meeting October 23, 2006

DCR Environmental Study December 2006

Start Final Design July 2007

Construction Start 2008/2009 FY

Construction Duration 240 Days



Maricopa County
Department of Transportation

www.mcdot.maricopa.gov

October 23, 2006

Maricopa County Department of Transportation

Old US 80 at the Gila River Bridge Rehabilitation Design concept Phase

Comment Sheet

Please complete and submit this form to a staff member before leaving or mail to: **Maricopa County Department of Transportation, Attn:** Andrzej Wojakiewicz, 602-506-8625, at 2901 **W. Durango St., Phoenix, AZ 85009.** Include your name and mailing address.

Please Print.

Name: _____ **Phone number:** _____

Address: _____

_____

E-Mail : _____

Meeting Survey

How would you rate the knowledge and helpfulness of staff members who assisted you?

Knowledge	Helpfulness
❒ Very knowledgeable	❒ Very helpful
❒ Somewhat knowledgeable	❒ Somewhat helpful
❒ Not very knowledgeable	❒ Not very helpful

Was all the project information presented in an understandable manner? Yes _____ No _____

Did staff answer your questions? Yes _____ No _____. If not, what didn't they answer?

_____

_____

_____

Do you want more information about MCDOT projects? Yes _____ No _____ . If yes, please make sure your name and address are filled in so we can add you to our mailing list.

How did you hear about the meeting?

Newspaper _____ Radio _____ Flyers _____ Trail Signs _____

Friends/Neighbors _____ Other:

_____

Additional Comments: _____

_____

_____

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