DICK PERREAULT

Bridge Evaluation Study
Ash Avenue Bridge
(Salt River Crossing)

City of Tempe
Project 876191B
Maricopa County, Arizona

Donohue ENGINEERS
ARCHITECTS
SCIENTISTS

ASH AVENUE BRIDGE
(SALT RIVER CROSSING)

BRIDGE EVALUATION STUDY

City of Tempe
Tempe Project No. 876191B
Maricopa County, Arizona

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Donohue Project No. 17685

May 4, 1990

Registered Professional Engineer (Civil)
CERTIFICATE NO.
23184
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Date Signed 5/4/90
ARIZONA, U.S.A.

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EXECUTIVE SUMMARY

The City of Tempe has proposed to channelize the reach of the Salt River from the Southern Pacific Railroad structure upstream through the Ash Avenue and Mill Avenue bridges to the McClintock bridge. The Flood Control District of Maricopa County will maintain the river channel after construction is completed.

OBJECTIVES

The objective of this study is to determine the adequacy of the Ash Avenue bridge to survive a flood with the current non-channelized and future channelized conditions and also the capacity of the bridge to be converted to a functional pedestrian bridge to provide access across the Salt River. With these objectives, four alternatives were identified and analyzed. These alternatives are "Do Nothing", "Structure Rehabilitation/Repair", "Structure Modification", and "Structure Removal". The "Do Nothing" alternative basically allows the structure to remain in its existing closed condition without repair or rehabilitation. The "Structure Rehabilitation/Repair" alternative involves rehabilitation and repair to return the structure to the original design configuration and condition. The "Structure Modification" alternative involves a combination of rehabilitation and repair with modifications to correct structural deficiencies. The "Structure Removal" alternative involves complete removal of the structure.

HISTORY

The Ash Avenue bridge once served as a major structure crossing the Salt River and linking the City of Tempe to western and northern portions of the state, including Phoenix. However, since abandonment in 1934 and the completion of the adjacent Mill Avenue bridge this structure has been closed to traffic by ADOT due to its deteriorated physical condition and serious structural inadequacies.

Constructed in 1911-1913 by inmates from the Territorial Prison at Florence, the Ash Avenue bridge is an eleven span open spandrel, reinforced concrete, three-hinged arch with two arch and column abutment spans. The overall structure length is 1507.75 feet providing an 18 foot clear roadway width. The design loading is based on a liveload of 100 pounds per square foot plus a 15 ton traction force.

During the structure's relatively short and eventful service life of approximately 20 years, it experienced considerable distress seriously affecting the condition of the structure and its ability to carry imposed loads. This structural distress is caused by a combination of factors, including unanticipated traffic growth, impact loading, river scour, substructure settlement, thermal forces, and original design and construction detail deficiencies. The result of these distress factors is a significantly reduced load carrying capacity, significant structural deterioration, failure of all primary bridge members and connections, and a general feeling that the structure is in need of extensive repairs and modifications or removal.

The "Fourth Biennial Report of the State Engineer to the Governor" for the period of July 1, 1918 to December 31, 1920, stated, "an analysis of stresses calls the sufficiency of the floor system and arch rings seriously in question".

The Arizona Highways, "Days of Tempe Bridge Are Numbered", dated May 1925, by Ralph Hoffman, State Bridge Engineer, states, "we are quite certain that its (Tempe bridge) days are numbered"; and, "The deflection of the slab takes a form of a smooth reverse curve, such as would be expected in a series of continuous girders with one span loaded, but on an exaggerated scale. Thus we might picture a series of see-saws end to end, and each linked to the other. Strike one joint of this series and the shock would be transmitted in a wave motion throughout the entire length of the series. Some such action undoubtedly takes place in the transmission of the impacts on the bridge as it is quite apparent that there is a periodic wave which is transferred through the crown hinges".

CURRENT CONDITION

The overall condition of the structure varies from poor to failed. The concrete deck is in poor condition with transverse cracking near the piers. The curb and traffic barrier members are in poor condition with several segments completely failed. The spandrel posts are in poor condition with numerous members completely failed and virtually all connections at the arch rib cracked. The arch ribs are generally in poor condition with longitudinal cracking along the arch rib near the main reinforcing steel. The pier units are in poor condition; and, due to the scour of the river bed, several of the caissons are exposed.

The original design assumptions and methods of analysis no longer apply to the Ash Avenue bridge because of the numerous structure modifications and the extent of failed members and connections. The Standard Specifications for Highway Bridges, Fourteenth Edition, 1989, as adopted by the American Association of State Highway and Transportation Officials must be used as a basis for the evaluation of the structure.

Based upon the overall condition and the current structural analysis criteria, the structure has "failed". This failure has not yet caused a physical collapse. The potential does exist for the structure to collapse under its own weight as it exists today. Although it cannot be said with any degree of accuracy as to when there will be a sudden catastrophic collapse, heavy construction in the area does cause vibrations that could contribute to that collapse.

REVIEW OF ALTERNATIVES

Both the "Do Nothing" and "Structure Rehabilitation/Repair" alternatives do not correct the structural and physically deficient condition of the structure. The "Do Nothing" alternative may lead to the complete collapse of the structure. In the existing channelized condition, the structure has the potential to overturn and be a hazard to the Southern Pacific Railroad bridge and all facilities downstream by causing accumulation of debris and a hydraulic "jump". The "Structure Rehabilitation/Repair" alternative, which returns the structure to the original design configuration and condition, was structurally inadequate originally and would remain so now.

The "Structure Modification" alternative requires significant and costly rehabilitation, repair, and modifications to the main arch ribs and other primary members to correct design deficiencies, repair physical deterioration, provide an adequate load carrying capacity, and a factor of safety against overturning. This alternative can be made to solve the structural deficiencies but the newly configured structure will not retain its original identity. The cost of this alternative will exceed the cost estimate recently completed by the City of Tempe for a new structure near this location which was $8 million. This alternative is not economically viable.

The fourth alternative of "Structure Removal" requires complete removal of the structure and eliminates future liability for the owning agency. This is the only alternative that provides absolute assurance that the proposed channelization of the Salt River can be safely accomplished without threatening other existing bridges.

The Ash Avenue bridge is on the National Register of Historic Places and any proposed construction operations must be coordinated with the State Historic Preservation Officer. For additional information, refer to the report "Section 404 Permit and Historic Permit Investigation", Ash Avenue, City of Tempe Project Number 876191B.

ASH AVENUE BRIDGE
(SALT RIVER CROSSING)
BRIDGE EVALUATION STUDY
CITY OF TEMPE
PROJECT 876191B
MARICOPA COUNTY, ARIZONA

INTRODUCTION

This Bridge Evaluation Study provides the City of Tempe with engineering documentation regarding the structural adequacy and capacity of the Ash Avenue bridge to survive a flood in the current non-channelized and future channelized condition and to be converted to a functional pedestrian bridge to provide access across the Salt River.

The information contained in this study is based upon a limited visual review of the structure, review and evaluation of record drawings, review of maintenance records and reports, review of historic documentation and records, evaluation of destructive testing (DT) results, and structural computations. General loading combinations under consideration are in accordance with the Standard Specifications for Highway Bridges (Specification adopted by the American Association of State Highway and Transportation Officials (AASHTO). These loading combinations include deadload (the weight of the structure), pedestrian liveload, hydraulic loading, and windloading for both the non-channelized and channelized river conditions.

Since its construction during 1911-1913, the Ash Avenue bridge has undergone considerable distress due to changes in loading conditions which have seriously affected the condition of the structure and its ability to withstand imposed loads. Furthermore, flooding conditions of the Salt River have caused significant scour of the river bed with undermining and settlement of substructure units resulting in additional distress to the structure.

The City of Tempe has proposed to channelize the Salt River as part of the Rio Salado improvement plan. The proposed channelization extends from the downstream Southern Pacific Railroad structure, through the Ash Avenue and Mill Avenue bridges, to the McClintock Drive bridge upstream. This improvement to the Salt River will modify the hydraulic flow characteristics imposed on the structure thereby further changing the loading conditions applied to the structure. This project will be a joint effort of the City of Tempe, the Arizona Department of Transportation, and the Flood Control District of Maricopa County.

DESCRIPTION OF THE STRUCTURE

The structure is an open-spandrel, reinforced concrete, 3-hinged arch with an overall length of approximately 1507.75 feet and overall width of approximately 20 feet. The clear roadway width is approximately 18 feet. The structure consists of 11 main spans and 2 abutment spans. Individual main span lengths are approximately 131 feet between the centerline of the pier units. The south and north abutment span lengths are approximately 40.33 feet and 32.42 feet, respectively.

The superstructure of the main spans consists of a reinforced concrete slab supported by transverse floorbeams at 10.83-foot spacing. The transverse floorbeams are supported by vertical spandrel posts rising from the main arch ribs. The two variable depth main arch ribs have a center-to-center spacing of approximately 12.66 feet. The concrete slab overhang and handrail are supported by a longitudinal beam at the outer edge of the slab. The longitudinal beams are supported, respectively, by the spandrel posts and overhanging corbel beams which are a continuation of the transverse floorbeams.

The superstructure of the abutment spans is similar to the main spans except for the vertical spandrel columns. The columns are supported on a large footing at the south abutment with caissons extending to bedrock and on bedrock at the north abutment.

All of the pier units are massive reinforced concrete shafts supported by various types of foundation combinations. Piers Nos. 1, 3, 5, 6, and 8 are supported on two (2) 6-foot diameter excavated caissons spaced at 13-foot centers. Piers Nos. 2 and 10 are supported directly on bedrock. Pier No. 4 is supported on six (6) excavated caissons with a transverse spacing of 13 feet and a longitudinal spacing of 20 feet. Pier No. 9 is founded on six (6) 5-foot diameter excavated caissons with 13-foot transverse and longitudinal spacing.

The parapet railing, curb, and post have a combined height of three (3) feet. The handrail is located on 4-inch diameter balusters spaced at 9-inch centers with an 8-inch by 12-inch post located at each spandrel post. This arrangement is repeated between each spandrel post.

The Ash Avenue bridge has been closed to traffic since 1933 following construction of the adjacent Mill Avenue bridge. Closure was a result of structural distress and the inability of the bridge to carry ever increasing traffic volume and weight.

No utilities are attached to the structure; however, an overhead high voltage transmission line crosses the Salt River within the immediate vicinity of the bridge. This facility may present safety and hazard concerns during future reconstruction and/or removal operations due to its close proximity.

HISTORIC STRUCTURE INFORMATION

Historic data was found through the review of articles from Engineering News Record and Arizona Highways. Reports from the State Engineer to the Governor were used to substantiate information reviewed.

Final construction plans for the structure were completed during early 1911, and under arrangements with the Territorial Prison at Florence, Arizona, prisoners were employed to construct the bridge. Construction supervision consisted of 2 engineers, 5 foreman, 2 carpenters, and several prison guards. The structure was opened to traffic on September 20, 1913.

Original plans and specifications detailed a 9 span solid arch ring bridge approximately 1225 feet long with a 16-foot roadway. During early construction operations, the plans and specifications were revised to an 11-span arch rib with open spandrel walls and a roadway width of 18 feet. Reasons for this change in structure type have not been documented, and may, in fact, be a cause for some of the structural deterioration experienced during the early life of the bridge.

The design loading for the structure was based upon a liveload of 100 pounds per square foot plus a 15-ton traction engine. Early documentation indicates that the completed structure was statically indeterminate with the sufficiency of the floor system and arch rings in serious doubt.

During the period from Thanksgiving, 1919, through March 3, 1920, flooding of the Salt River caused severe scour resulting in excessive settlement of Pier No. 9. Emergency repairs were made; however, there were concerns whether the structural adequacy of Pier No. 9 and 5 other piers could sustain vertical loadings.

Traffic volumes and weights, particularly trucks, increased dramatically over design projections, resulting in distress to the members caused by increased impact and vibrations. At numerous times during the early life of the bridge, limitations on truck traffic and speed were considered to minimize impact loading.

Serious cracking of structural members, including the arch ribs, floor, spandrel posts, handrail, and expansion joints required repairs. Within 10 years of its construction, it was necessary to inform the State Legislature of the poor conditions of the structure, and the possibility of restricting traffic or closing the bridge.

During the early to mid 1920's, the condition of the bridge had deteriorated so much that plans for a new structure crossing the Salt River were implemented. Following the completion of the adjacent Mill Avenue bridge in 1931, the Ash Avenue structure was abandoned on May 22, 1933.

ANALYTICAL EVALUATION

The procedures used to analyze the structure consist of a multi-step approach:

Step No. 1 - Initial review and evaluation of the available structure plans and documentation. This review identified structural members with potential problems and allowed for the chronological review of the structural capacity of the structure.

Step No. 2 - Visual field review of the structural members and joints noting deficiencies, including: spalls; scaling; delaminations; and, the general condition of the appropriate elements. Field review of the deck surface, handrail, arch ribs, spandrel post, spandrel columns, floorbeams, piers, and abutment units was completed using a walk-by procedure to substantiate past inspection reports. The inspection team noted the condition of the various structural members. After the inspection of two spans, it became apparent that the majority of the deficiencies were consistent throughout each span, as well as similar members. Measurements of the structure, including the slab, transverse floorbeams, spandrels, corbels, spandrel arches, longitudinal stringers, and main arch ribs were completed with the assistance of a hydraulic man-lift machine.

Step No. 3 - Inspection procedure involved preliminary evaluation of the condition of the various members with the intent to identify the location where to take concrete core samples to determine the member strengths, depth of delaminations, and extent of deterioration.

Step No. 4 - Testing procedures were implemented to substantiate the condition of the concrete and reinforcing steel to verify design strengths.

Step No. 5 - Structural analysis of the structure to determine the overall condition and to compute the structural capabilities of the members. Capacities were computed for the stability of the structure to withstand a design flood event in the current non-channelized and future channelized river configuration with and without pedestrian loading. Capacities were computed for the deck slab, transverse floorbeams, longitudinal center support beam, curb beams, spandrels, and main arch ribs.

FIELD REVIEW, EVALUATION, AND RATING SPECIFICATIONS

The following specifications must be used as a basis for the walk-by field review and evaluation of the structure:

Standard Specifications for Highway Bridges, 14th Edition, 1989, as adopted by the American Association of State Highway and Transportation Officials.

Manual of Maintenance Inspection of Bridges, 1983 as adopted by the American Association of State Highway and Transportation Officials and amended by periodic revisions through 1989.

SUMMARY OF CONDITION

The structure has undergone considerable distress as evidenced by the numerous areas of cracking, spalling, and general deterioration of the load carrying members. A majority of this distress occurred during the early life of the structure and prior to its abandonment in 1933. In general, the deterioration has been caused by settlement, vibration or impact loading from trucks, increased traffic volumes, and thermal forces.

Deck Surface

The asphalt wearing surface is in a poor to failed condition. The asphalt is severely cracked, weathered, and spalled at several locations.

Concrete Deck

The section of the concrete deck between the arch ribs is in fair condition. The section of the concrete deck supported between the arch rib and the curb beam is in poor condition. Since the concrete deck is covered with an asphalt wearing surface, only the bottom side of the concrete deck could be visually reviewed. The underside of the deck has transverse cracks throughout the bridge exhibiting leaching and efflorescence. Between the pier and the first spandrel on each side of Pier Nos. 1, 2, 3, and 4, transverse cracks extend the full width of the deck, and in several instances extend down the spandrel arches. There are several locations where the reinforcing steel is exposed. The underside of the deck between the arch rib and the curb beam is severely cracked, spalled, and delaminated and exhibiting efflorescence with exposed reinforcing steel. The underside of the deck in Span Nos. 4, 7, 8, and 9 is darkened and could have been exposed to fire.

Expansion Joints - Over Piers

The expansion joints over the piers are failed and the surrounding concrete is spalled and delaminated.

Deck Expansion Joints - Over Crown Hinge

The expansion joints over the crown hinges have failed and are leaking. The concrete under them is spalled, delaminated, and exhibiting efflorescence.

Parapet Railing, Posts, and Curbs

The parapet railings, posts, and curbs are in a poor to failed condition with extensive spalling and delamination. The parapet railing and post are completely missing on some portions of the bridge. The exterior edge of the curb section is badly spalled with the longitudinal reinforcing steel exposed throughout the bridge.

Spandrel Posts

The spandrel posts are in a poor to failed condition. Many of the spandrel posts are severely spalled, delaminated, and cracked horizontally and vertically. At some of the spandrel posts, only the reinforcing steel is left in place versus concrete encasing them. Many of the spandrel posts were repaired with Gunite which is cracked, spalled, and delaminated.

Spandrel Columns

The spandrel columns are in fair condition. The spandrel columns exhibited some minor spalling. Many of the spandrel columns have horizontal cracks at the pier to column connection. Several of the spandrel columns have vertical cracks. The spandrel columns where expansion joints were installed are cracked at the expansion joint near the top of the column.

Main Arch Ribs

The main arch ribs are in poor condition. Most of the arch ribs exhibit severe cracking and some spalling at the crown hinges. Many of the arch ribs have longitudinal cracks either near the top or bottom on the side of the arch paralleling the main reinforcing steel. These cracks may be full width of the arch rib since some appear on the interior and exterior surfaces. In some instances, these cracks run near the quarter and three-quarter points of the spans.

At the west arch rib in Span No. 11, there is a horizontal crack extending through the arch rib.

At the east arch rib in Span No. 10, the bottom main reinforcing steel is exposed and buckled. The concrete surrounding this location is severely cracked with the cracks extending longitudinally each direction and extending to the top side of the arch.

At the east arch rib in Span No. 9, there are vertical cracks perpendicular to the arch rib which show on all four surfaces of the arch rib.

In summary, the Ash Avenue Bridge is in poor to failed condition with major structural concerns. No maintenance has been performed on the bridge during the last 57 years of its life.

The field review identified several structural design deficiencies which are affecting the structural capacity, serviceability, and functional aspects. These deficiencies have resulted in an overstressed, or failure condition in the structure. These deficiencies must be corrected if the structure is to provide an acceptable and adequate level of service.

The slenderness requirements (depth, width, and effective length) and lateral bracing spacing of the main arch members are not satisfactory, thereby seriously decreasing the member capacity.

Bar development and lap lengths are not adequate to transfer the imposed loading at the connection between the spandrel posts to the main arch rib. Therefore, many of these joints are overstressed with resultant failure.

Liveload deflections during the early years of the structure's life were documented as being excessive. Due to the closed condition of the structure, no field verification of liveload deflections were possible; however, based upon the slim nature of the load carrying members and from calculations, it can be assumed that deflections were in fact of a magnitude to raise concern.

FIELD SAMPLING AND TESTING

During the walk-by field review and initial evaluation of the structure, additional testing procedures were implemented to substantiate the condition of the concrete and reinforcing steel and verify design strengths to be used in the analysis phase.

Fifteen (15) concrete core samples were taken at various locations in the deck slab, main arch ribs, spandrel arch diaphragms, pier footings, and caissons. The spandrel posts were not used to obtain core samples due to the extent of deterioration. Core samples consisted of both 2-inch and 6-inch diameter specimens and were tested in accordance with the American Society of Testing Materials (ASTM) Specification C-42. The core samples exhibiting large aggregate to size of sample ratio were not tested because the test results would reflect aggregate strength, and not concrete strength. A summary of the location, core size, and compressive strength of the specimens are as follows:

Location	Core Size	Compressive Strength
Deck Slab, Span 6	2-inch	5230 psi
Deck Slab, Span 8	2-inch	3657 psi
Spandrel Arch, Span 5	2-inch	3590 psi
Spandrel Arch, Span 7	2-inch	3020 psi
Main Arch Rib, Span 6	2-inch	3540 psi
Main Arch Rib, Span 2	6-inch	2115 psi
Pier Shaft, Pier 6	2-inch	3060 psi
Footing, Pier 9	2-inch	6960 psi

Three (3) reinforcing steel specimens were taken from various locations in the deck and sampled in accordance with ASTM procedures to determine strength characteristics. A summary of the location, yield strength, and ultimate strength of the specimens are as follows:

Location	Yield Strength	Ultimate Strength
Span 2	50,000 psi	68,000 psi
Span 3	58,500 psi	79,000 psi
Span 4	72,500 psi	97,500 psi

The results of the field material sampling generally agree with the strength of the materials used during the era of the bridge construction. However, due to the wide variety of the compressive and yield strengths of the concrete and reinforcing steel, respectively, the following values will be used as a basis to determine the structural capacity of the bridge:

$f'c$ = 2000 psi (Concrete Strength at 28 days)
fc = 800 psi (Allowable Concrete Stress, Inventory)
fc = 1200 psi (Allowable Concrete Stress, Operational)
Ec = 2,400,000 psi (Elastic Modulus of Concrete)
n = 15 (Modular Ratio)
fy = 33,000 psi (Yield Strength of Steel)
fs = 18,000 psi (Allowable Steel Stress, Inventory)
fs = 25,000 psi (Allowable Steel Stress, Operational)

LOADING COMBINATIONS AND ANALYSIS APPROACH

The following loading assumptions and combinations were used for the purpose of determining the structural capacity of the bridge:

1. The general loading conditions, as outlined in the AASHTO Specifications, were used as a basis for applying various loading combinations. Loading combinations were limited to deadload, buoyancy, pedestrian liveload, streamflow, and windloading.

2. Plan dimensions and "as new construction" conditions of the bridge were used as a basis for all structural computations. No reduction factors for loss of section properties or strength were taken into consideration.

3. Deadload is the structural weight of the bridge members applied in accordance with the design specifications assuming a unit weight of 150 pounds per cubic foot.

4. Buoyancy is the effective weight of bridge members when submerged to a depth of the design flood elevation. The design flood elevation for both the current non-channelized and future channelized river configuration were used for the purpose of determining the maximum buoyancy force.

5. Streamflow is the horizontal force due to the effect of flowing water on the structural members below the design flood elevation. For the purpose of determining the maximum overturning moments and forces on the structure, streamflow was applied to an effective area equal to the structure member size plus an additional 2.0 feet on each side of the member to account for potential debris blockage. Due to the open spandrel nature of the arch members, the streamflow force was applied to 150% of the upstream effective area to account for application of the force to the upstream face of the downstream members.

6. Windloading is the applied horizontal force on the exposed area of structural members based on a wind velocity of 100 miles per hour. For the purpose of determining the maximum overturning moments and forces on the structure, windload was applied at the centroid of an effective area of 150% of the exposed frontal area.

7. Pedestrian liveload is the application of a vertical load with an intensity of 85 pounds per square foot assuming the structure will be solely for pedestrian usage. The full roadway width is assumed to be loaded with pedestrian liveload with no reduction for multiple span loading. Partial width pedestrian loading was not considered.

8. The design flood is assumed to be an event with a respective design discharge of 250,000 cubic feet per second. The resultant design flood water surface elevation and velocity are 1148.0 feet and 11.2 feet/second, respectively, for the non-channelized river condition. Under the future channelized river condition, the water surface elevation and velocity are 1155.9 and 11.2, respectively, assuming 4 feet of debris build-up on submerged structural elements.

9. Evaluation of the structural capacity is based upon gravity loads (deadload and pedestrian liveload) to determine overall bridge capacity plus a

factor of safety against overturning based upon deadload, streamflow, buoyancy, and windloading.

STRUCTURE ALTERNATIVES

Four alternatives were identified for detailed investigation and evaluation under this study. Each of these alternatives have a distinct impact on the function and serviceability of the structure and the ability to carry loads, longevity, cost, and liability to the owning agency. The alternatives under consideration included the following:

Do Nothing

This alternative basically allows the structure to remain in its existing closed condition without repair or rehabilitation, and will not provide for a functioning structure for pedestrian loading.

Structure Rehabilitation/Repair

This alternative involves rehabilitation and repair to return the structure to the original design configuration and condition. Various construction methods would be used to repair the cracking, spalling, and overall deterioration; however, structural deficiencies with respect to the Standard Specifications would not be corrected.

Structure Modification

This alternative involves a combination of rehabilitation and repair with modification to correct structural deficiencies and improve liveload capacity. Major modifications to the structure would be necessary to correct deficiencies. Modifications to the substructure units and foundation support system would be necessary to improve the factor of safety against overturning.

Structure Removal

This alternative involves either partial or complete removal of the structure to eliminate future liability concerns in regard to the bridge being capable of supporting the require liveload, stream flow and wind overturning forces. Partial removal would also be possible to allow the end spans to remain in place, as these would be located behind the proposed dikes of the future channelized river condition.

ANALYTICAL RESULTS

The major components of the structure were analyzed to determine their structural capacity. The method of analysis and approach is as follows:

Modeling Condition One

The structure was first analyzed assuming that the concrete deck was continuous over the piers and that the structure as a whole is resisting the imposed loads. This method of modeling will usually be the most beneficial to the major components of the structure. This model overstressed the concrete deck at several locations. These overstressed locations were verified in the field by the transverse cracks noted in the deck.

Modeling Condition Two

The structure was then analyzed assuming that the concrete deck was discontinuous over the piers, as is actually the case, at Pier Nos. 2, 3, and 4. The following results are based on this model:

1. The structure was analyzed for stability about the main arch ribs, socket to pier, for both the non-channelized and future channelized conditions.

   Minimum Allowable Factor of Safety = 1.0
   Desirable Factor of Safety = 1.5
   Actual Factor of Safety = 1.98 (non-channelized condition)
   Actual Factor of Safety = 1.76 (channelized condition)

   The stability of the main arch ribs meet AASHTO Specifications.

2. The structure was analyzed for stability for the critical location about the substructure units for both the non-channelized and future channelized condition.

   Minimum Allowable Factor of Safety = 1.0
   Desirable Factor of Safety = 1.5
   Actual Factor of Safety = 0.99 (non-channelized condition)
   Actual Factor of Safety = 0.88 (channelized condition)

   The stability of the structure in the non-channelized condition meets the minimum allowable factor of safety but not the desirable factor of safety. The stability of the structure in the channelized condition is less than the minimum allowable factor of safety. Additional caissons may be added to the substructure units to meet Specifications.

3. The main arch ribs were analyzed for deadloads and for pedestrian loadings.

   The main arch ribs do not meet AASHTO specifications for buckling for deadload alone. The structure cannot be used unless the size of the main arch ribs are increased.

4. The main arch ribs were analyzed for deflection, for pedestrian loading, for full-width structure loading.

Allowable deflection = 1.57"
Actual deflection = 0.75" full-width loading

The structure meets AASHTO Specifications for pedestrian loading deflection criteria.

5. The spandrel posts were analyzed for the forces due to deadload, pedestrian load, and windload.

   The reinforcing steel connection between the spandrel post and the arch rib does not meet AASHTO Specifications. The structure is not adequate unless the connection between the spandrel post and arch rib is improved.

The structure may be in a failure mode; however, because of the elastic action of the structure, which allows non-catastrophic failure, the structure is still standing. Through the review of the analysis, condition, and history of the structure, many components have been overstressed and have failed. Overstressing of a member may deteriorate the bond between the concrete and reinforcing steel.

STRUCTURE ALTERNATIVE EVALUATION RESULTS

Evaluation of the four structure alternatives, for the identified loading combinations that determined the structural capacity and factor of safety against overturning, minimizes the viable options available to the owning agency. The following comments can be associated with each of the alternatives:

Do Nothing

Structural deficiencies of the existing bridge dictate that the do nothing alternative is totally unacceptable. The structure no longer is functioning, as originally designed, due to failure of the numerous spandrel posts, localized failure of the main arch ribs, transverse cracking to the slab, failure of the spandrel post to main arch rib connection, and settlement of the pier supports.

The structural capacity of the bridge should be considered as zero because numerous members are totally deteriorated and joints have failed. The overall structural integrity is jeopardized and failure could occur at any time.

This alternative places liability on the owning agency to either provide rehabilitation repairs or to modify the structural members to provide an acceptable level of service or to remove the structure. Consideration should be given to other alternatives.

Structure Rehabilitation/Repair

Rehabilitation and repair of the structure to regain the original structure configuration will not eliminate the structural deficiencies. Inadequate lap and bar development lengths and inadequate size of main arch ribs will still exist. Without improvements and modifications to the structural details, future localized failures will continue to occur.

The factor of safety against overturning during the application of streamflow, and windload is not adequate for the existing structure configuration. Future structure failure could occur which would jeopardize both the Ash Avenue bridge as well as other downstream structures. Considerations should be given to other alternatives.

Structure Modification

To correct inadequate design details, provide structural capacity for pedestrian liveload, and develop an acceptable factor of safety against overturning requires significant modifications to various structural members.

The major structure components requiring modification are the main arch ribs. Removal and reconstruction to a larger size will be necessary to improve the slenderness ratio to provide adequate compressive capacity to withstand buckling. At the same time, modifications to the spandrel posts will be required to supply the necessary lap and development length for the main reinforcing steel, and provide for a fixed joint condition.

Placement of additional expansion joints will be necessary in the slab at those piers not already modified and placement of new expansion joints at those piers already modified.

Modification to the foundation and caisson support system will be required to increase the factor of safety against overturning due to stream flow and windloading.

In order to retain the Ash Avenue structure, the structure modification alternative, complete reconstruction of all members above the piers, is required. The cost of this alternative would exceed the cost of a complete new bridge, which was estimated at $8 million.

Structure Removal

The alternative to remove the structure which is required of alternatives "Do Nothing" and "Structure Rehabilitation/Repair" requires complete demolition.

The probable estimate of cost to remove the structure is in the range of $90,000 to $135,000, depending upon the contractor's method of removal and regulatory agency requirements. Deposition of material was assumed to be placed behind the dikes.

SOURCES OF INFORMATION

The following information was used in the review, inspection, and evaluation of the Ash Avenue Bridge:

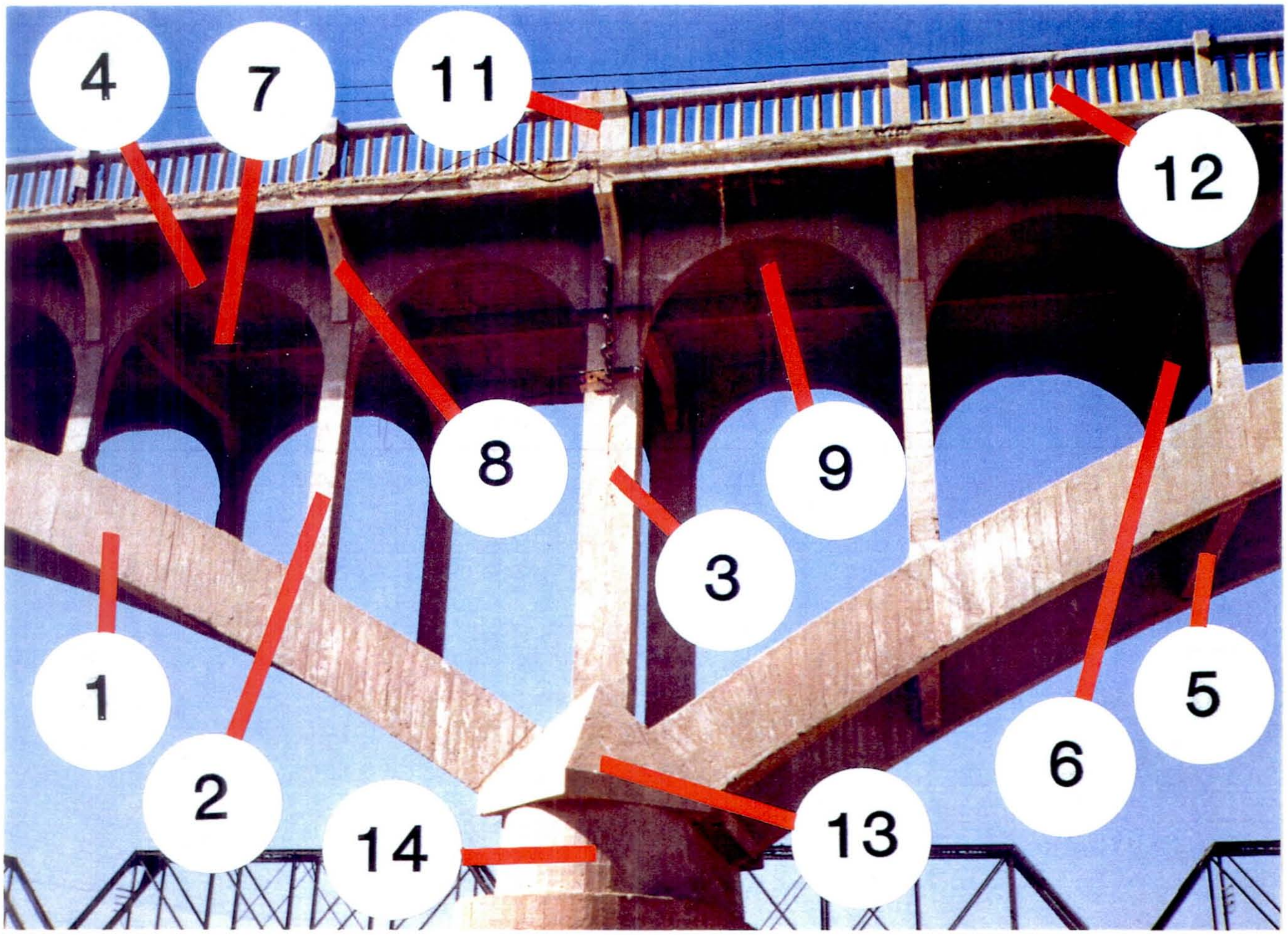
- o Bridge plans dated October 31, 1911, prepared by the Territory of Arizona Engineering Department.
- o Repair bridge plans dated July 3, 1921, prepared by the Arizona Highway Department of Transportation.
- o Engineering News Record article dated March 28, 1912, page 578, "Description of the Ash Avenue Bridge".
- o Engineering New Record article dated April 21, 1921, "Repair of Tempe Concrete Arch Bridge Damaged by Settlement and Floor Expansion".
- o Tempe Daily News dated December 1, 1980, "Inmates Gave Bridge Crew Exciting Time".
- o Arizona Highways, May, 1925, "Days of Tempe Bridge are Numbered".
- o Arizona Highways, June 1931, "Tempe Bridge Soon to Be Ready for Traffic".
- o Ash Avenue Bridge Analysis, by Reed, Jones, Christofferson, Inc.
- o Final Report Channel Improvement Study, Mill Avenue Reach Project No. 876191, By CRSS, Inc.
- o Report of the State Engineer of the State of Arizona, July 1, 1909 to June 30, 1914.
- o Second Report of the State Engineer to the State of Arizona, July 1, 1914 to June 30, 1915, and July 1, 1915 to July 30, 1916.
- o Fourth Biennial Report of the State Engineer to the Governor of the State of Arizona, July 1, 1918 to December 31, 1920.

  Fifth Biennial Report of the State Engineer to the Governor of the State of Arizona, July 1, 1920 to June 30, 1922.
- o Sixth Biennial Report of the State Engineer to the Governor of the State of Arizona. July 1, 1922 to June 30, 1924.
- o Seventh Biennial Report of the State Engineer to the Governor of the State of Arizona, July 1, 1924 to June 30, 1926.

REFERENCES

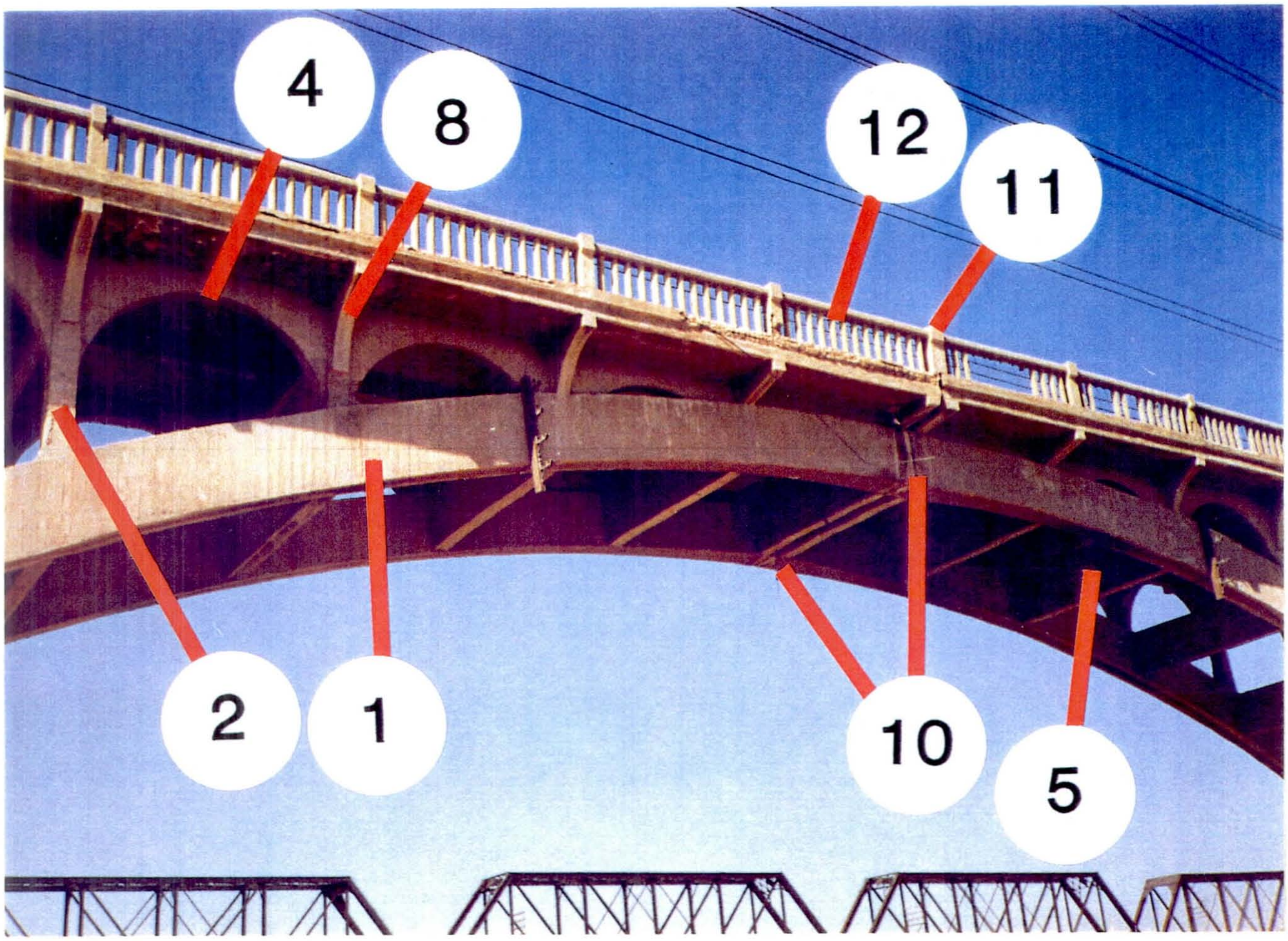
- Reinforced Concrete and Masonry Structures, by Hool & Kinne, Edition 1924, "Section 8-Arches", page 433-529.
- The Theory of Continuous Structures and Arches, by Charles M. Spofford, S.B., Edition 1937, Chapter III, Page 158-183.

51M/MI/AL6



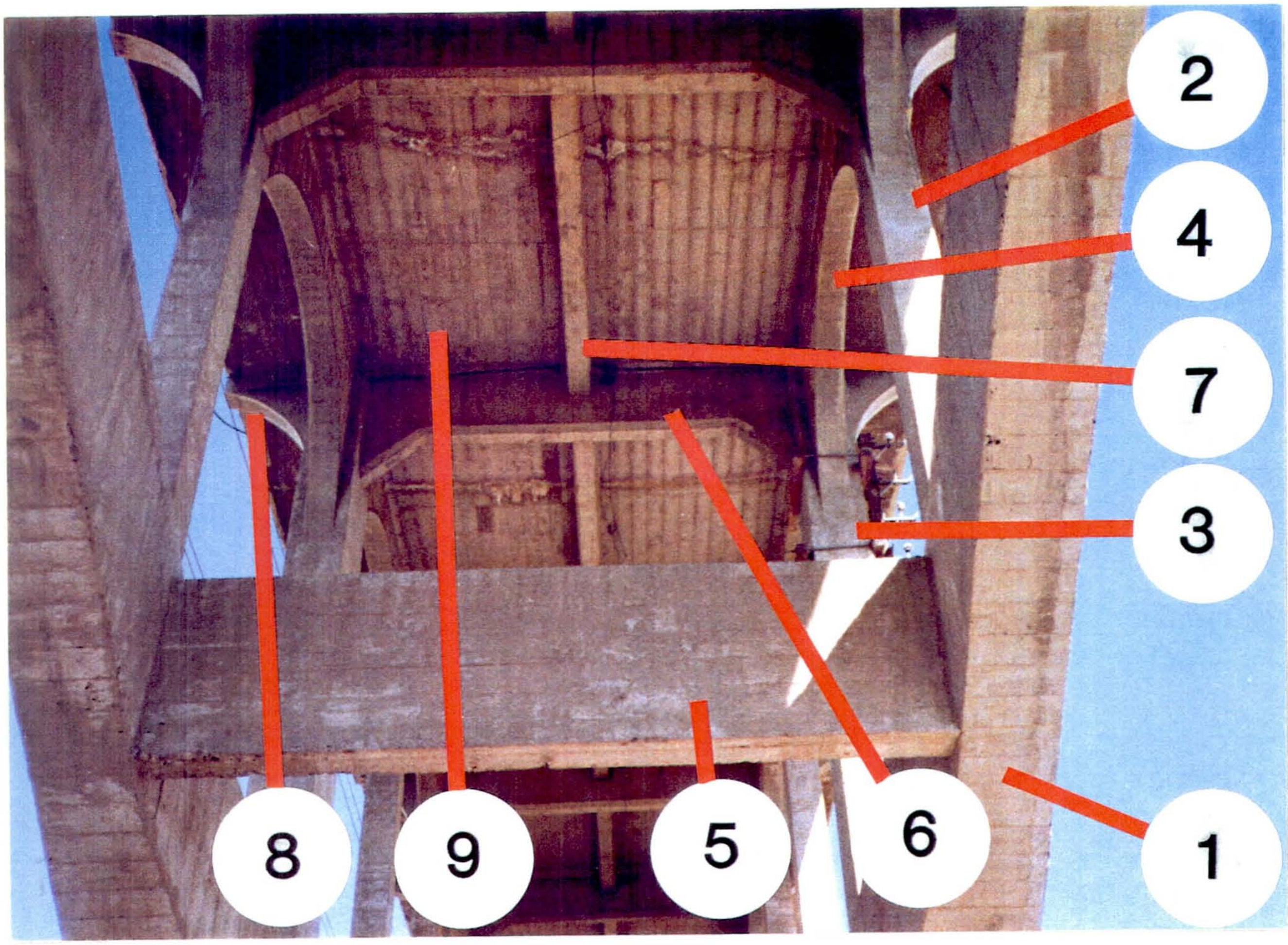
LEGEND

1. Arch Rib
2. Spandrel Post
3. Spandrel Column
4. Spandrel Arch
5. Sway Bracing
6. Transverse Deck Beam
7. Longitudinal Deck Beam
8. Bracket
9. Deck
11. Parapet Post
12. Parapet Railing
13. Pier Cap
14. Pier



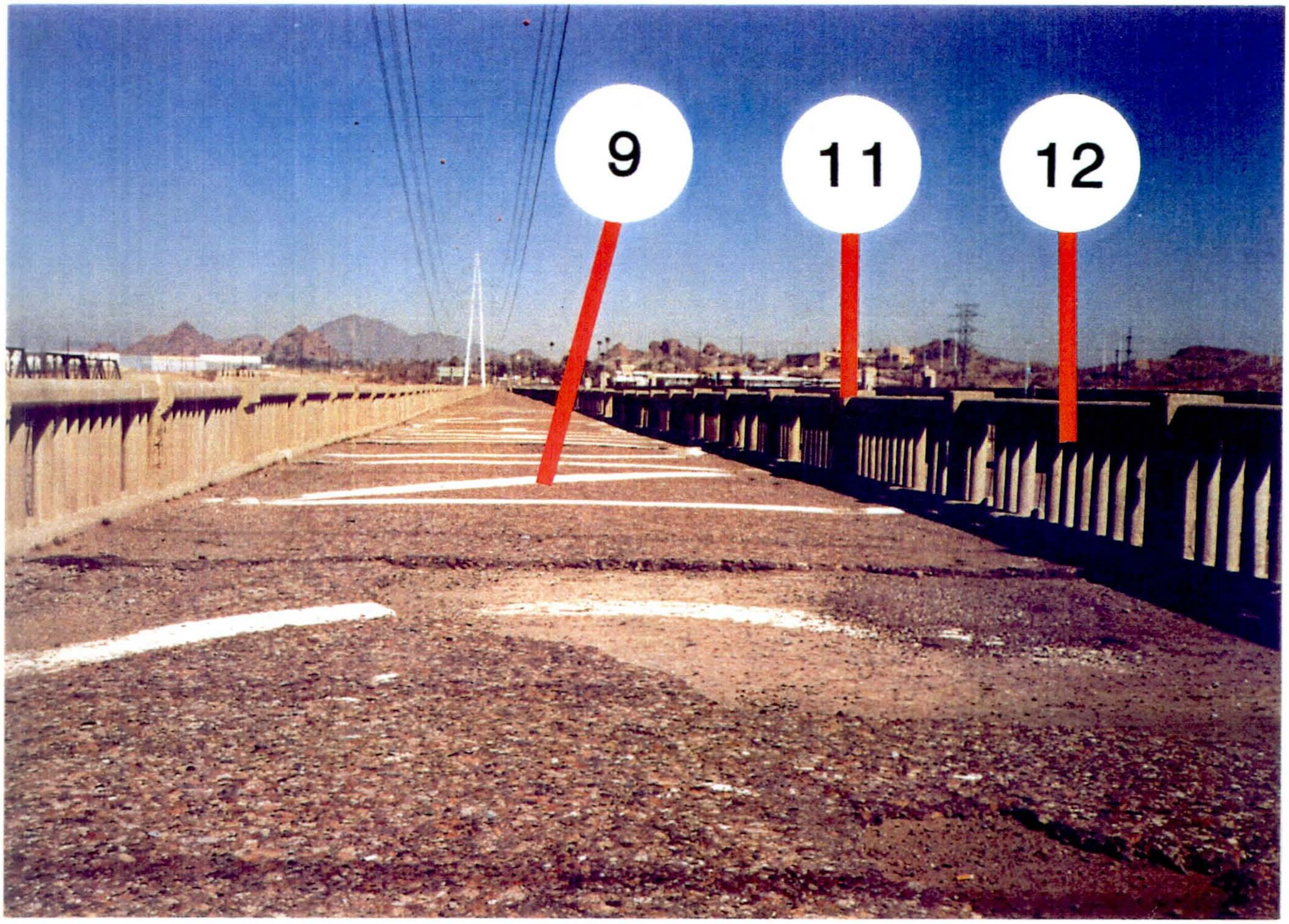
LEGEND

1. Arch Rib
2. Spandrel Post
4. Spandrel Arch
5. Sway Bracing
8. Bracket
10. Crown Hinge
11. Parapet Post
12. Parapet Railing



LEGEND

1. Arch Rib
2. Spandrel Post
3. Spandrel Column
4. Spandrel Arch
5. Sway Bracing
6. Transverse Deck Beam
7. Longitudinal Deck Beam
8. Bracket
9. Deck

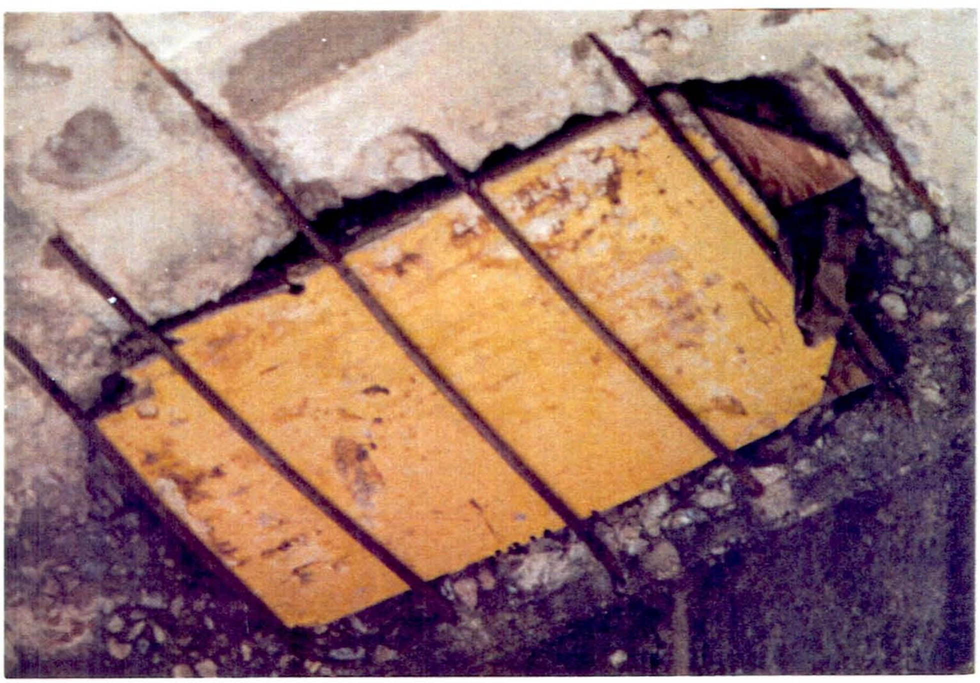


LEGEND

9. Deck
11. Parapet Post
12. Parapet Railing

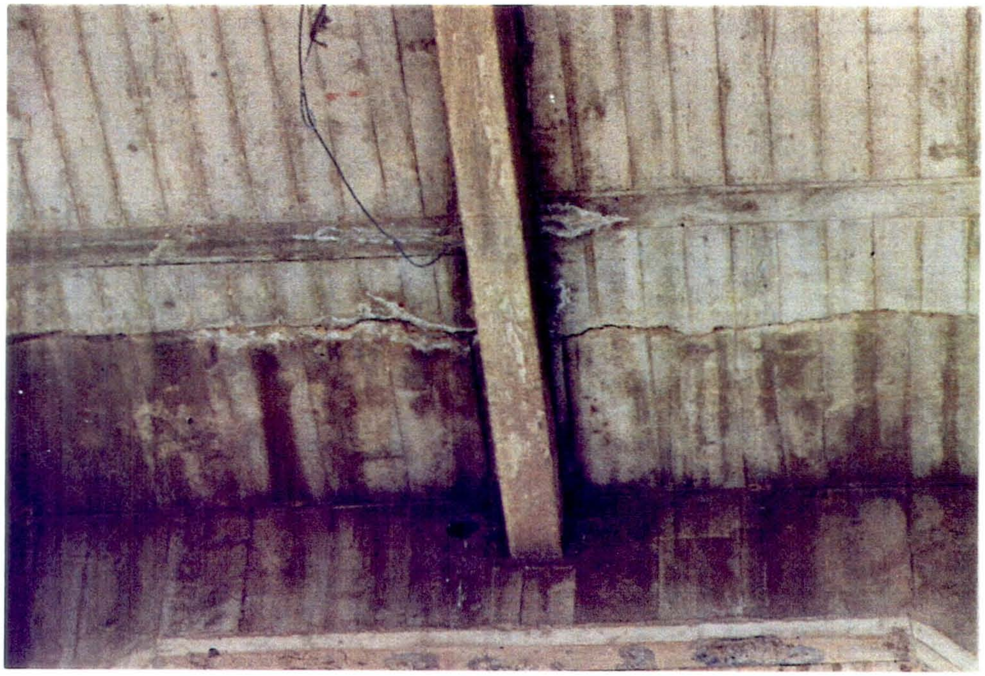
BRIDGE DECK, LOOKING NORTH

NOTE: Missing parapet railing
NOTE: Missing pavement
NOTE: "Sagging" of parapet railing



SPAN No. 1, SPANDREL BAY No. 3

NOTE: Exposed deck reinforcing



SPAN No. 4, SPANDREL BAY No. 1

NOTE: Transverse crack in deck

SPAN No. 4, SOUTH OF PIER No. 4

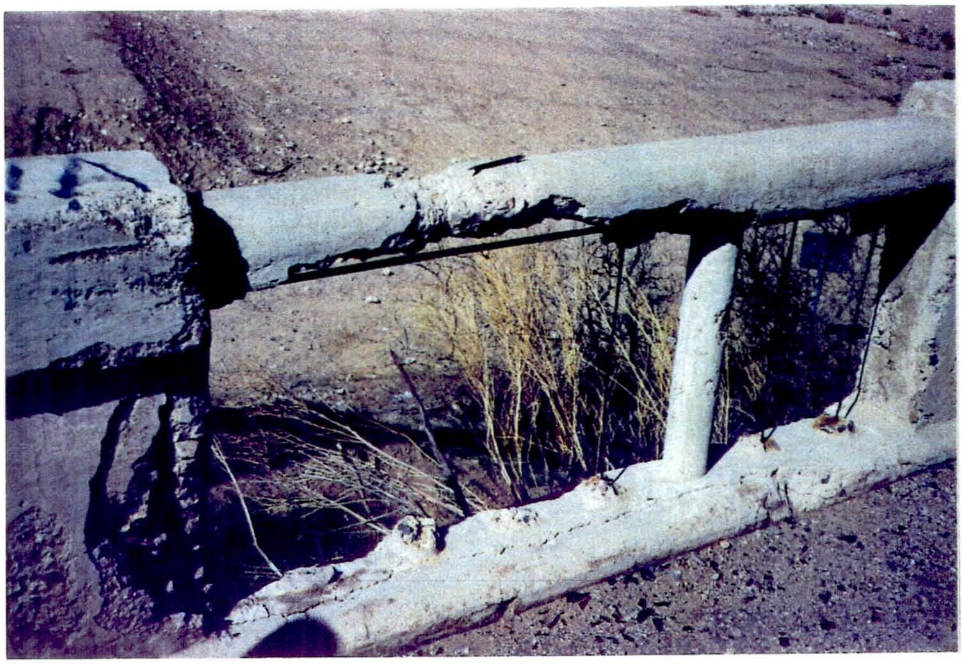
NOTE: Crack across deck continuing through spandrel arches

SPAN No. 3, SPANDREL BAY No. 4
WEST SIDE

NOTE: Exposed reinforcing steel
NOTE: Delamination and spalling concrete

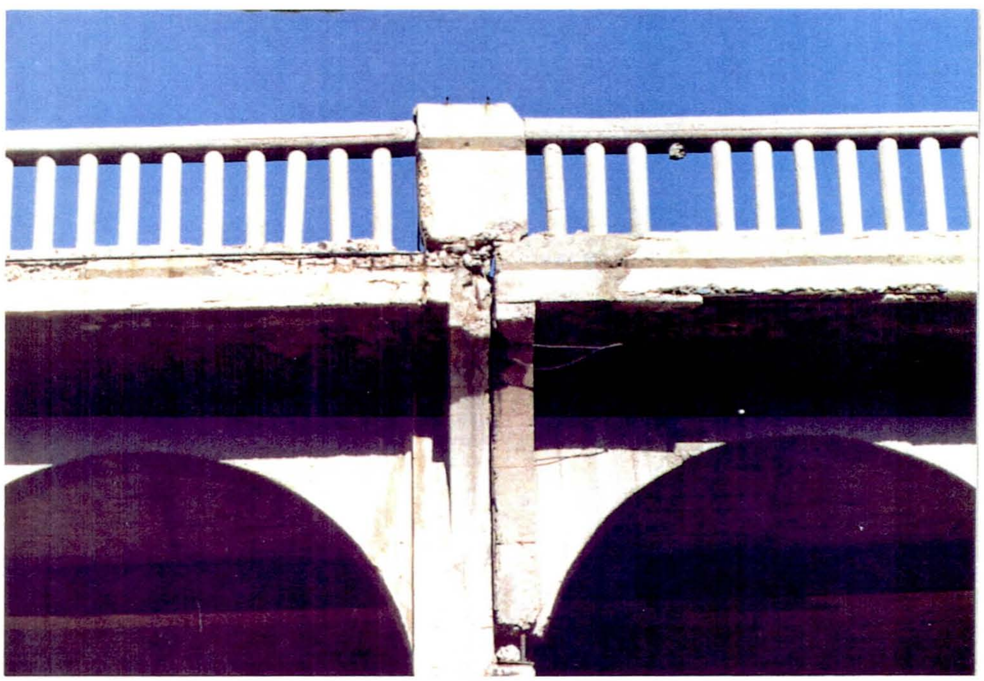
SPAN No. 4, SPANDREL BAY No. 7

NOTE: Delamination and spalling concrete
NOTE: Efflorescence
NOTE: Darkened area (may have been exposed to fire)



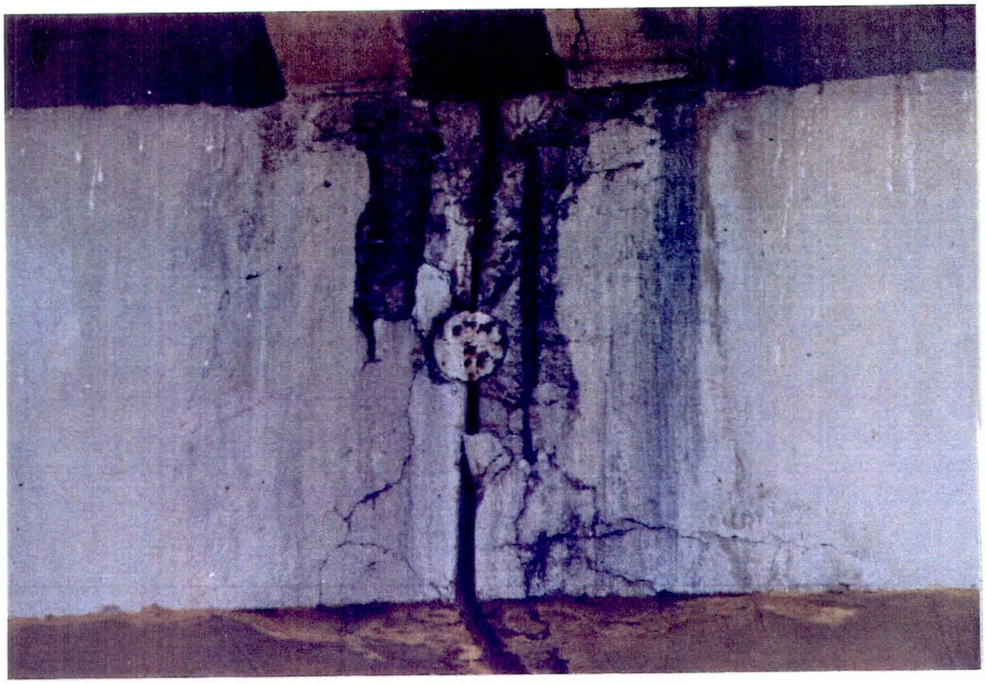
PARAPET POST AND RAILING AT SOUTH END

NOTE: Missing parapet rails
NOTE: Delamination and spalling of parapet post



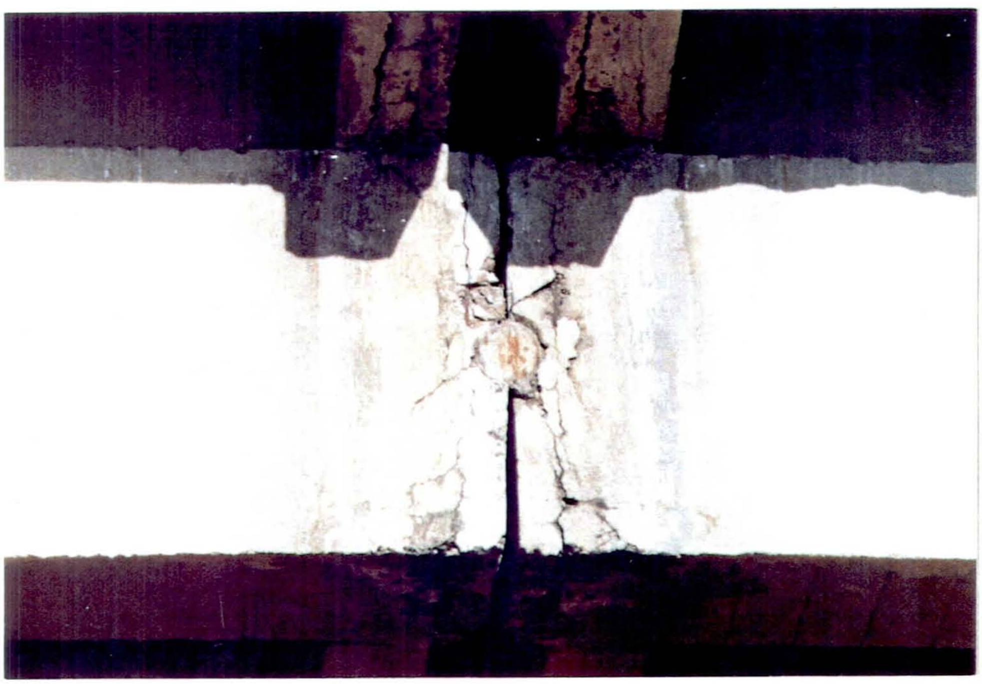
BRACKETS, PARAPET, AND EXPANSION JOINT - PIER No. 4

NOTE: Exposed reinforcing steel at base of railing and parapet post
NOTE: Exposed reinforcing steel at base of bracket and spandrel arch at spandrel column



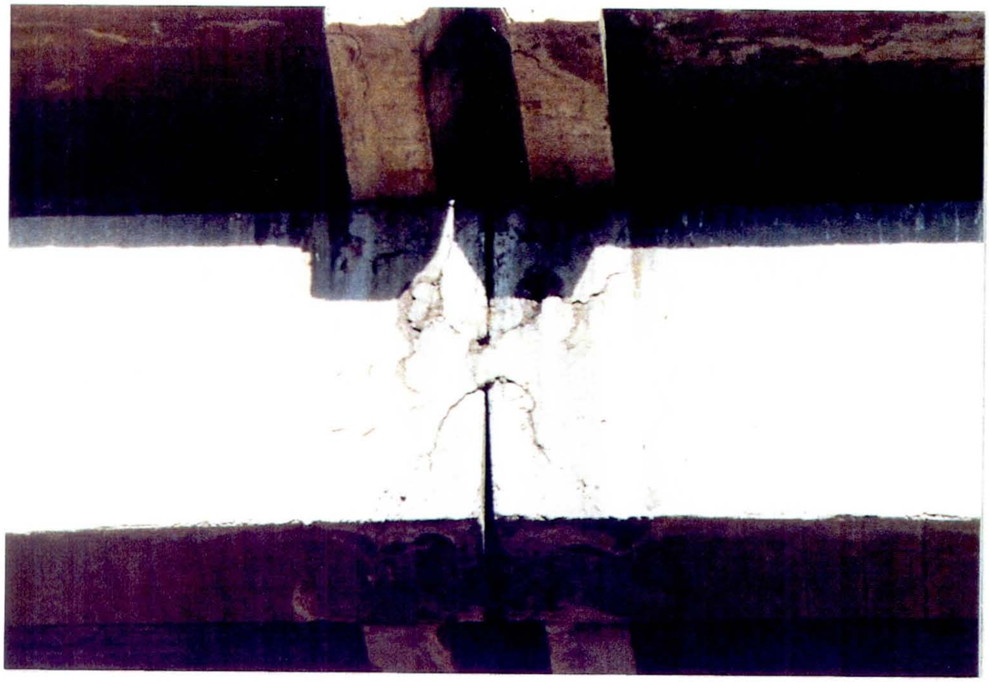
SPAN No. 1, EAST CROWN HINGE

NOTE: Exposed crown hinge pin
NOTE: Exposed reinforcing steel
NOTE: Fractures in arch rib near hinge pin



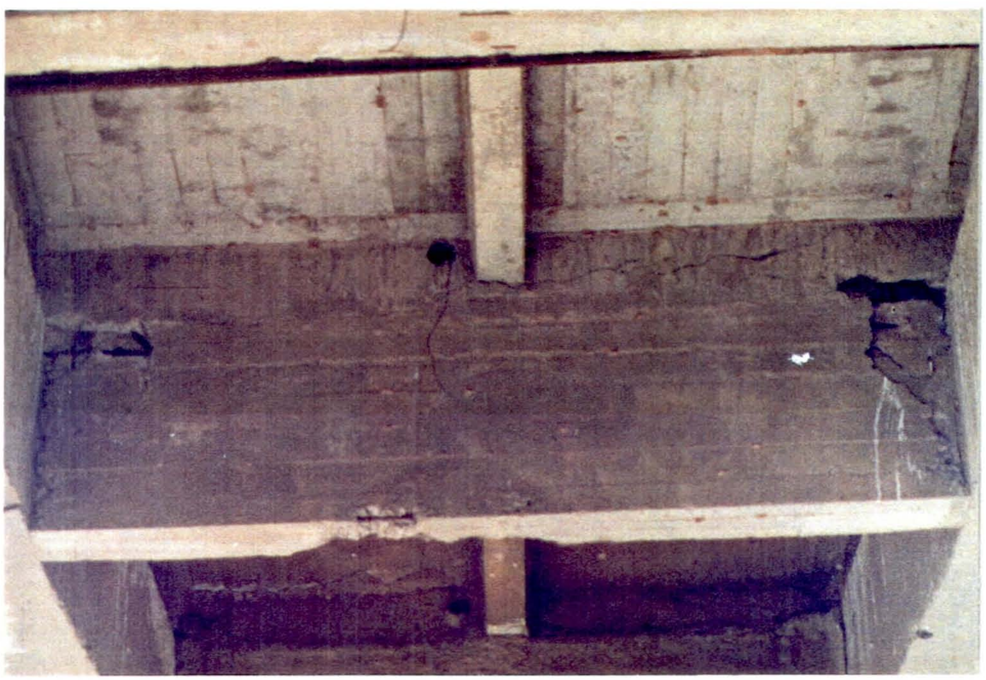
SPAN No. 1, WEST CROWN HINGE

NOTE: Cracks around hinge



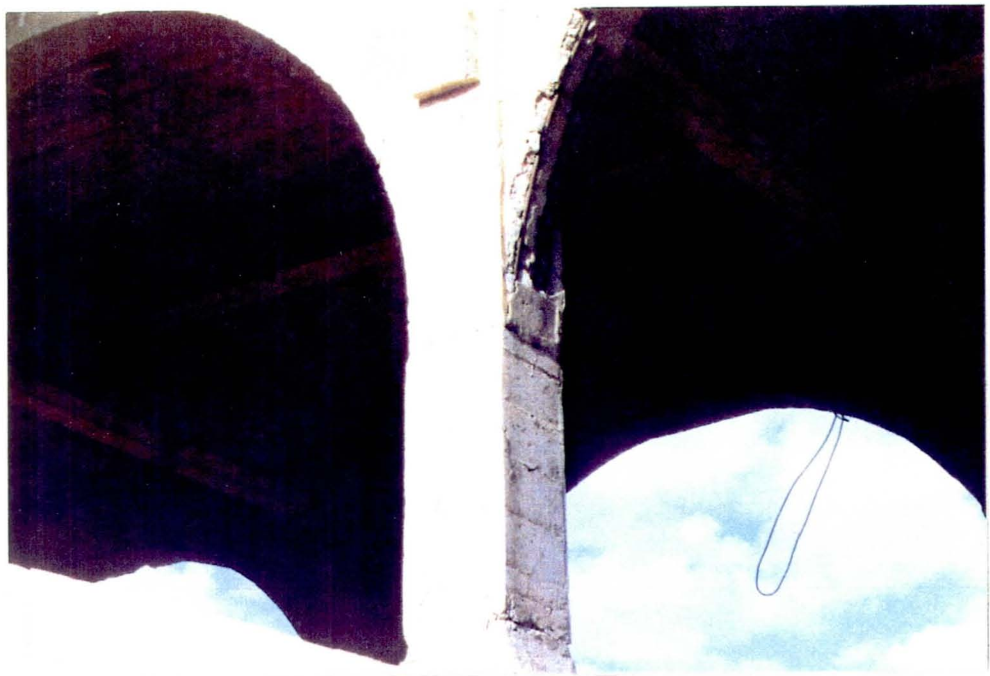
SPAN No. 5, WEST CROWN HINGE

NOTE: Cracks around hinge



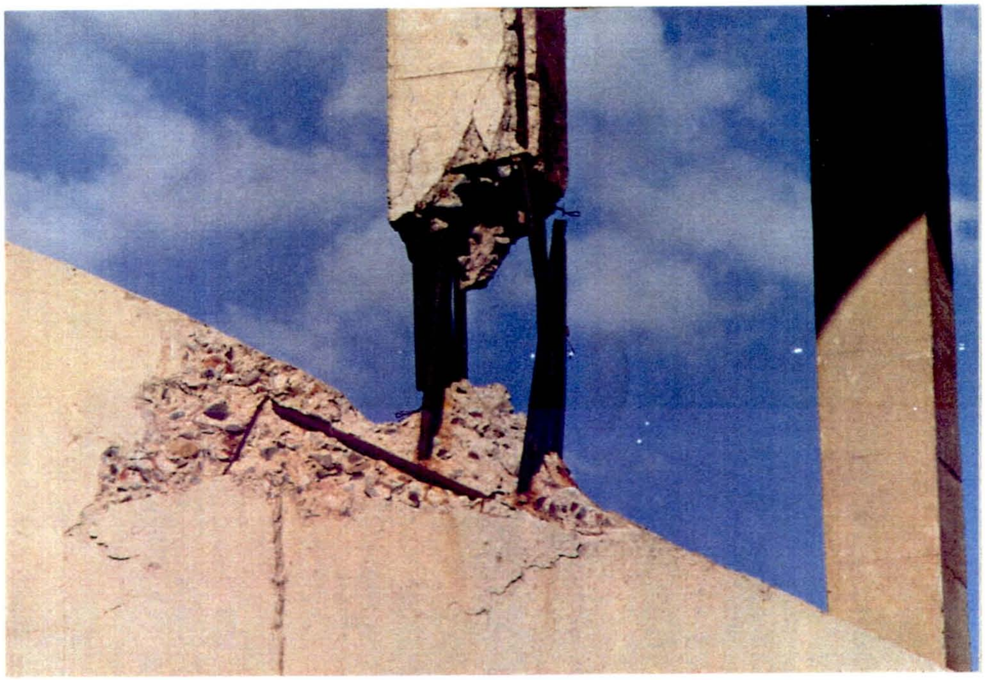
SPAN No. 10, TRANSVERSE FLOOR BEAM No. 9

NOTE: Cracks at the ends of the floorbeam
NOTE: Horizontal cracks in tranverse floorbeam
NOTE: Exposed reinforcing steel at the bottom of the floorbeam



SPAN No. 10, SPANDREL BAY No. 2

NOTE: Exposed reinforcing steel in spandrel arch
NOTE: Horizontal cracks at spandrel arches

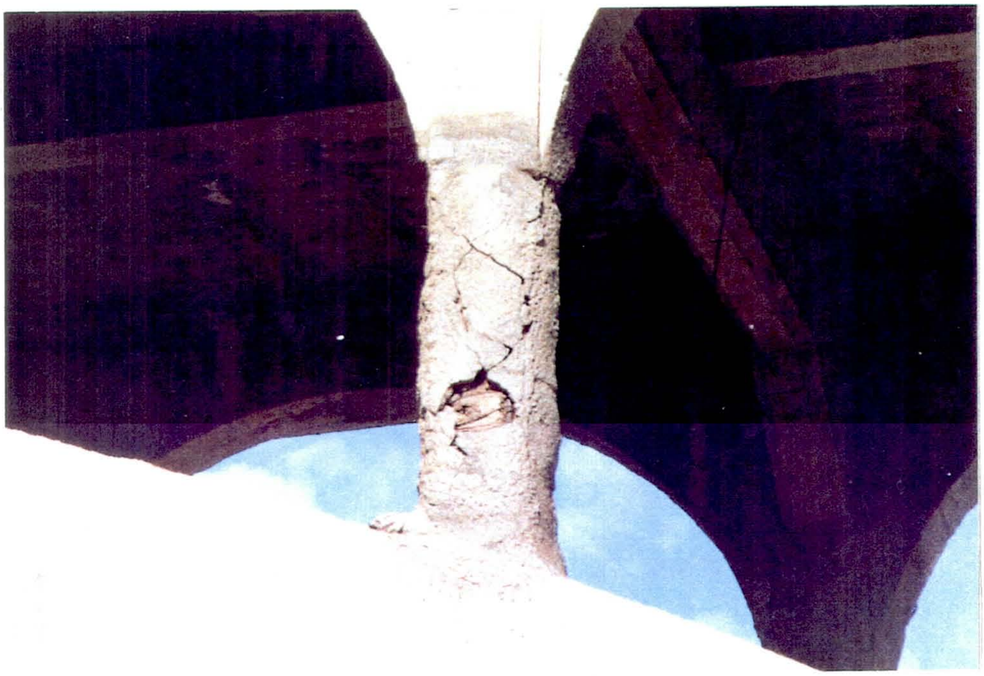


SPAN No. 4, SPANDREL POST No. 4, WEST SIDE

NOTE: Exposed reinforcing steel in arch rib
NOTE: Spandrel post severed

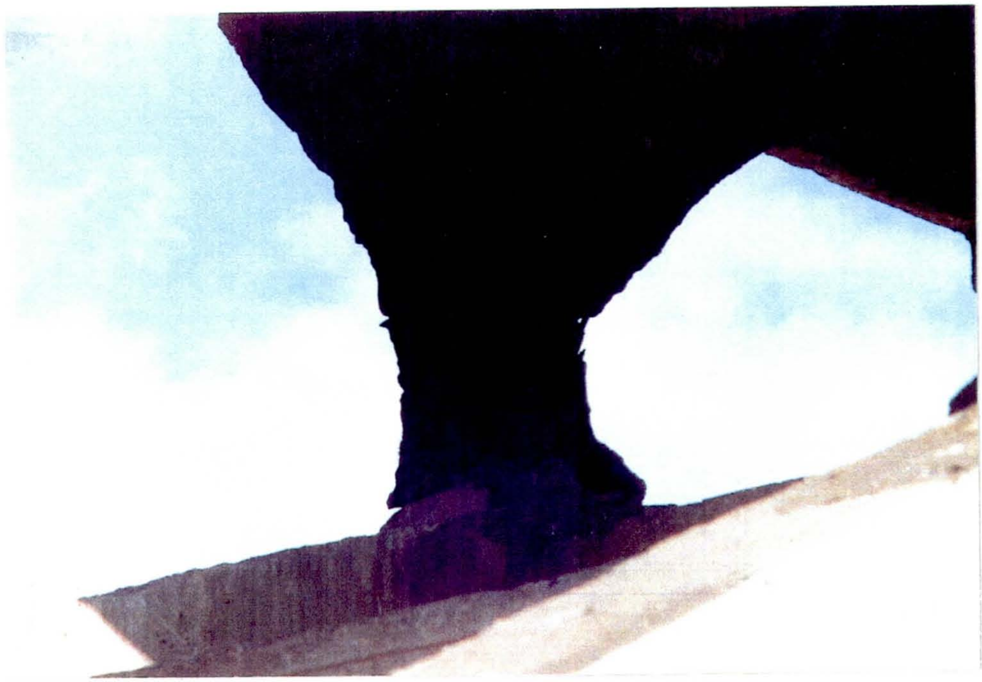
SPAN No. 10, SPANDREL POST No. 2

NOTE: Exposed reinforcing steel



SPAN No. 9, SPANDREL POST No. 2, WEST SIDE

NOTE: Delamination and spalling of concrete



SPAN No. 11, SPANDREL POST No. 10, EAST SIDE

NOTE: Exposed reinforcing steel in spandrel post

SPAN No. 1, SPANDREL POST No. 2, EAST SIDE

NOTE: Fractures in spandrel post
NOTE: Fractures in arch rib

SPAN No. 10, EAST ARCH RIB

NOTE: Exposed reinforcing steel
NOTE: Buckling of exposed reinforcing steel

SPAN No. 10, EAST ARCH RIB

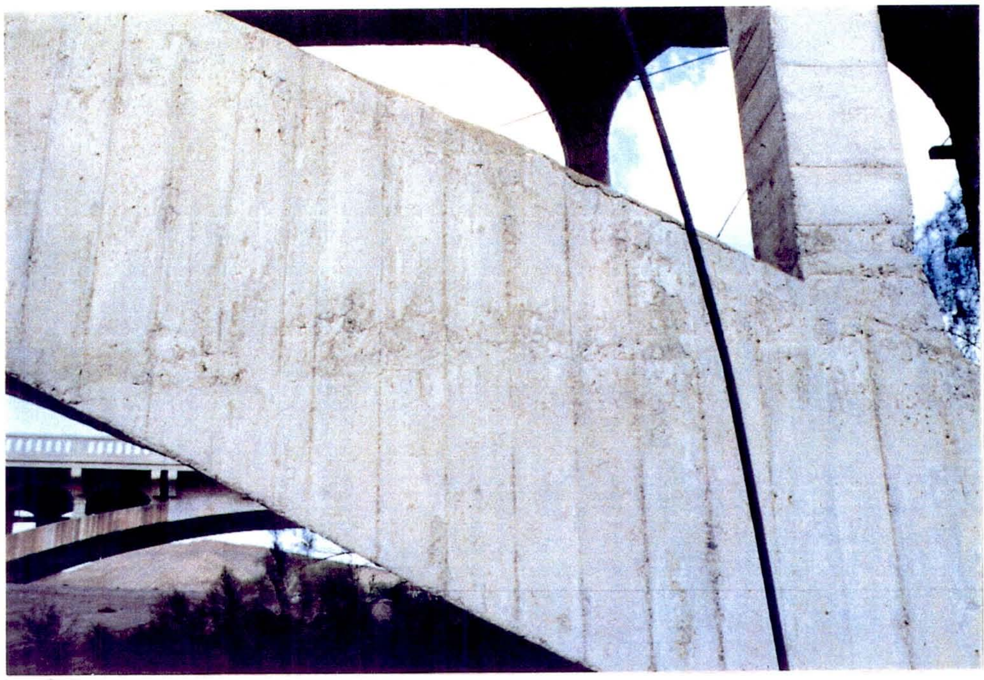
NOTE: Exposed reinforcing steel
NOTE: Buckling of exposed reinforcing steel
NOTE: Cracks parallel and perpendicular to arch rib

SPAN No. 1, EAST ARCH RIB

NOTE: Delamination and spalled concrete
NOTE: Exposed reinforcing steel

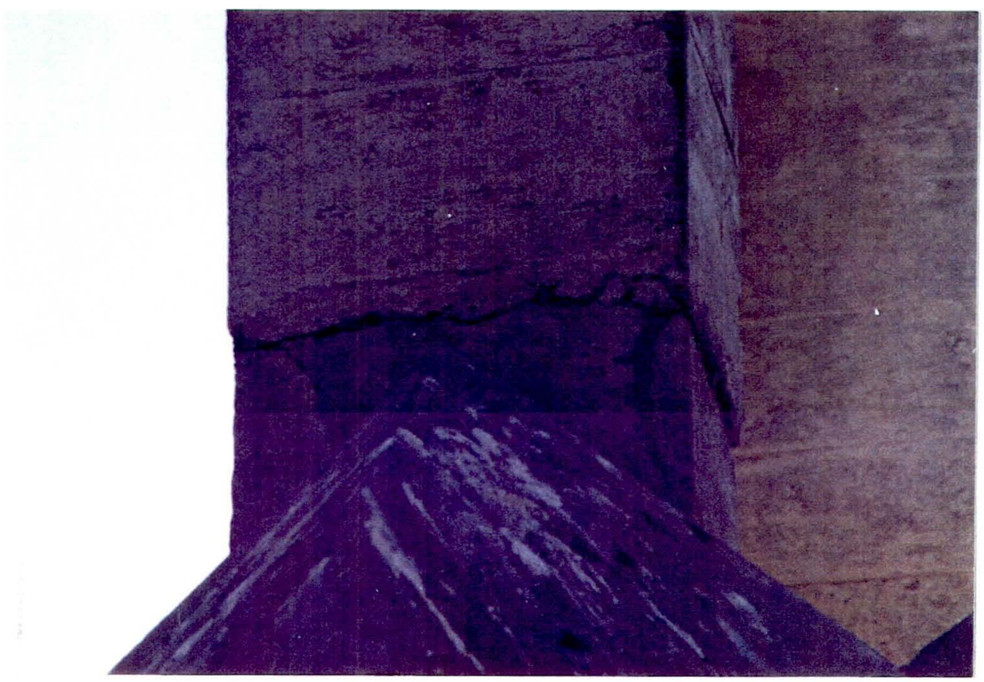
SPAN No. 10, WEST ARCH RIB BETWEEN POST No. 2 AND No. 3

NOTE: Crack perpendicular to arch rib



SPAN No. 11, WEST ARCH RIB BETWEEN
SPANDREL POST No. 1 AND No. 2

NOTE: Horizontal crack in arch rib



PIER No. 8, EAST SPANDREL COLUMN

NOTE: Fracture horizontal through spandrel column

PIER No. 7, EAST SIDE

NOTE: Fractures in pier cap and pier

PIER No. 4

NOTE: Scour at pier

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March 26, 1990

Donohue & Associates, Inc.
3055 West Indian School Road
Phoenix, Arizona 85017

SHB Job No. FT90-3234
Report No. 1

Attention: Mr. Dale Schaub

Re: Ash Avenue Bridge
Ash Avenue & Salt River
Tempe, Arizona

Gentlemen,

Transmitted herewith are results of testing performed on concrete cores and reinforcing steel sampled from the Ash Avenue Bridge. A sketch of each core failure made is enclosed. The maximum aggregate size was approximately 1 1/2 inches to 2 inches in all cores except the footing. That core had aggregate as large as 4 inches.

Reinforcing steel was sampled from the curb area on the deck. That steel is round, deformed bar. Results indicate that the steel probably was manufactured to the requirements of ASTM A7 (an obsolete specification).

Core sample 15 had steel included that was 1 1/8 inches in diameter, round and nondeformed. No testing was performed on that steel because of the small amount of the sample.

Should any questions arise concerning this report, please do not hesitate to contact us.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By Albert C. Ruckman
Albert C. Ruckman, P. E.

Copies: Addressee (3)

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RESULTS OF COMPRESSION TESTS

Test DATE 3-6-90

PROJECT Ash Avenue Bridge JOB NO. FT90-3234

LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road
Phoenix, AZ 85017

SOURCE OF SAMPLE Pier Cap - Pier #6

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, $in.^2$ 2.447 UNIT WEIGHT, PCF 140.0

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

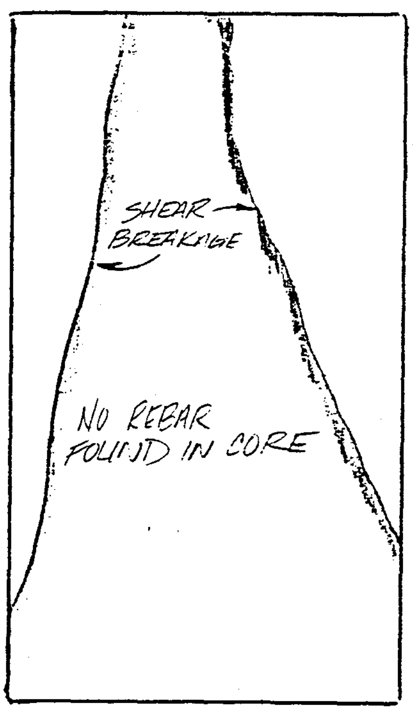
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec ± 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	PSI
-	Core #2	Shear	3.485	-	7,614	2.0	-	3060



RESPECTFULLY SUBMITTED,
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Albert C. Ruckman, P.E.

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PROJECT Ash Avenue Bridge JOB NO. FT90-3234

LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road Phoenix, AZ 85017

SOURCE OF SAMPLE Arch Support Footing - Pier #9

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.400 UNIT WEIGHT, PCF 144.3

TIME IN MIXER WATER ADDED ON JOB, GAL

DATE CONCRETE PLACED - AGE -

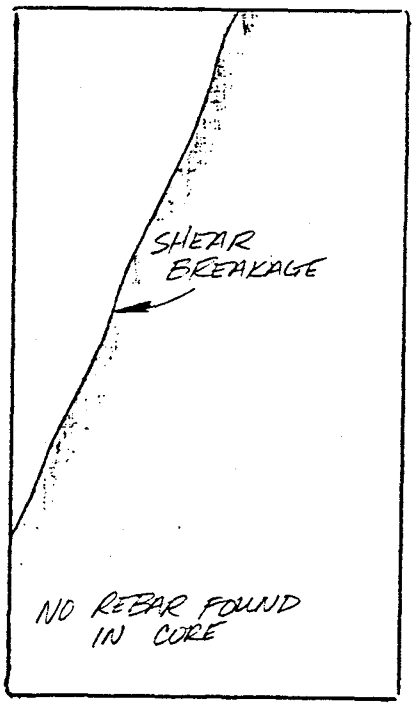
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec ± 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	PSI
-	Core #5	Shear	3.600	-	16,769	2.0	-	6960



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Albert C. Ruckman, P.E.

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Test DATE 3-6-90

PROJECT Ash Avenue Bridge JOB NO. FT90-3234

LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road Phoenix, AZ 85017

SOURCE OF SAMPLE Deck between Piers 7 & 8

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.433 UNIT WEIGHT, PCF 137.0

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

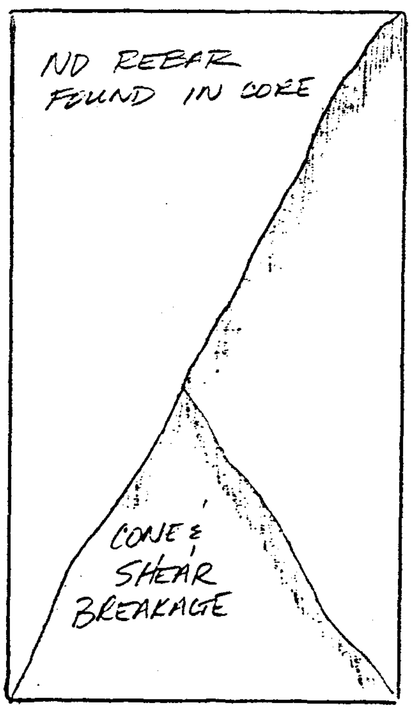
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec $\pm$ 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	Corrected PSI
-	Core #6	Cone/Shear	2.450	-	9,769	1.4	0.96	3657



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Test DATE 3-6-90

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LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road
Phoenix, AZ 85017

SOURCE OF SAMPLE Deck between Piers 5 & 6

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.378 UNIT WEIGHT, PCF 130.0

TIME IN MIXER WATER ADDED ON JOB, GAL.

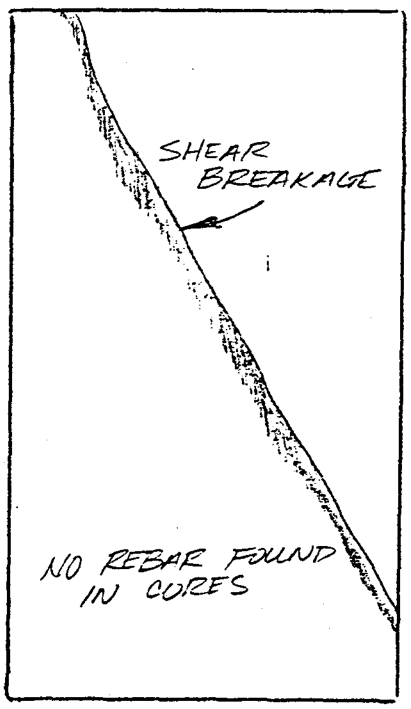
DATE CONCRETE PLACED - AGE -

SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked
Loading Rate: 20-50 psi/sec ± 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	PSI
-	Core #7	Shear	3.540	-	12,438	2.0	-	5230



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LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road Phoenix, AZ 85017

SOURCE OF SAMPLE Main Arch 10' S of Center

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.461 UNIT WEIGHT, PCF 144.3

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

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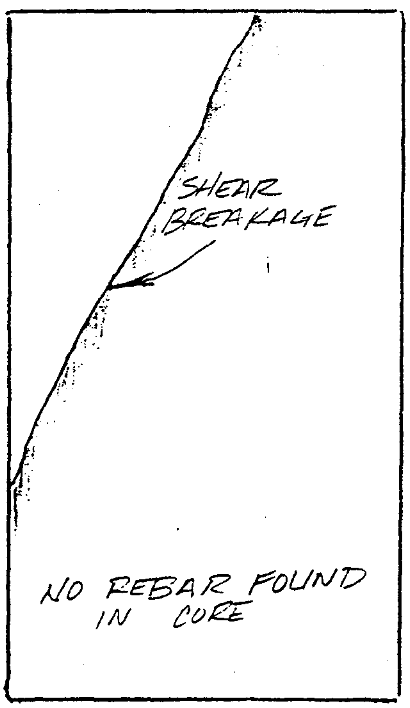
DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec $\pm$ 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L* / D	Factor	PSI
-	Core #10	Shear	3.560	-	8,724	2.0	-	3540



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CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road Phoenix, AZ 85017

SOURCE OF SAMPLE Deck Arch N of Pier #6

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.433 UNIT WEIGHT, PCF 135.2

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

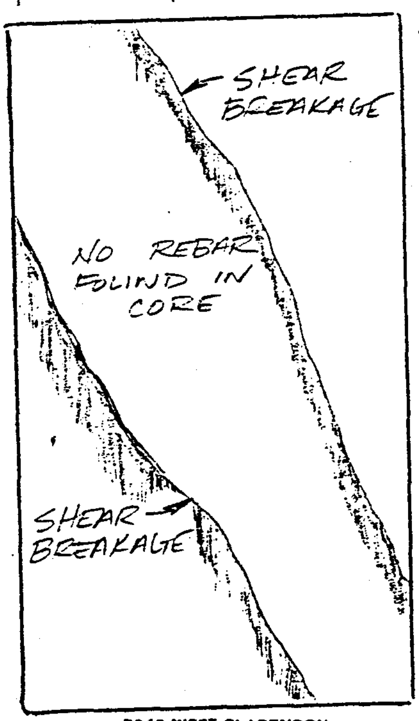
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec ± 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	PSI
-	Core #11	Shear	3.490	-	7,379	2.0	-	3020



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SOURCE OF SAMPLE 4th Deck Arch N of Pier #4

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 2.419 UNIT WEIGHT, PCF 138.4

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

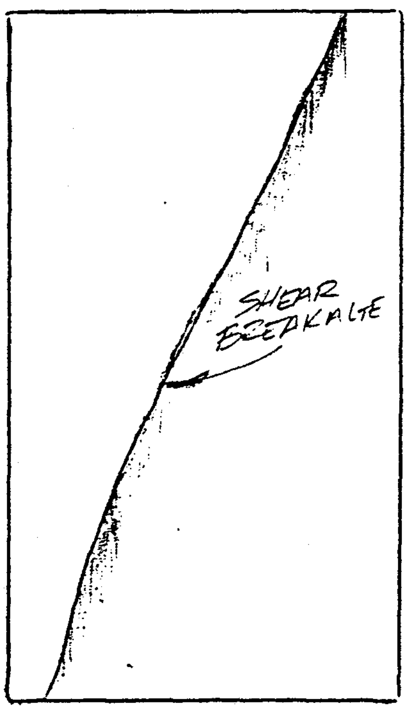
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec $\pm$ 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L*/D	Factor	Corrected PSI
- -	Core #13	Shear	2.700	-	9,057	1.5	0.96	3590



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LOCATION Washington & Mill Avenue

CLIENT Donohue & Associates, Inc. ADDRESS 3055 West Indian School Road Phoenix, AZ 85017

SOURCE OF SAMPLE Main Arch between Piers 3 & 4 - S 1/4 Span

SOURCE OF MATERIAL - DESIGN STRENGTH, PSI -

SPECIMEN AREA, in.^{2} 27.57 UNIT WEIGHT, PCF 138.8

TIME IN MIXER WATER ADDED ON JOB, GAL.

DATE CONCRETE PLACED - AGE -

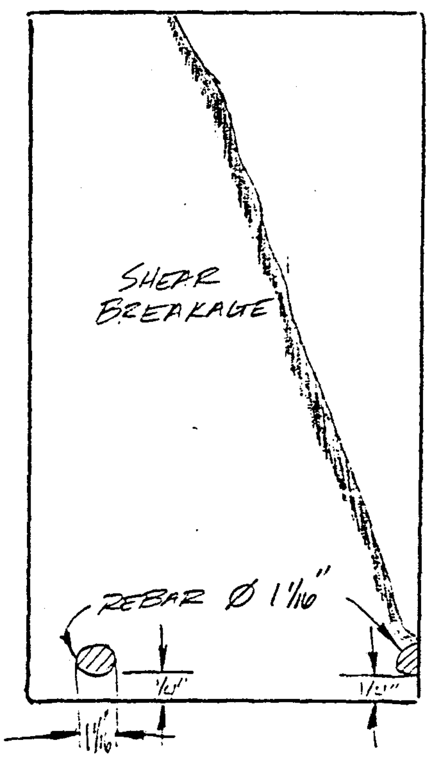
SUBMITTED BY SHB/DRL CORED BY SHB/DRL

DATE CORED 2-21-90 DATE RECEIVED 2-21-90

REMARKS: Core Specimens/ASTM C-42 Break: Dry Soaked

Loading Rate: 20-50 psi/sec ± 500 lbs/sec

LAB NO.	IDENTIFICATION NO.	Failure Type	Ht. (B)* Inches	Age Days	Load Pounds	L* / D	Factor	Corrected PSI
-	Core #15	Shear	7.460	-	66,432	1.3	0.94	2115



RESPECTFULLY SUBMITTED,
SERGENT, HAUSKINS & BECKWITH ENGINEERS

By Albert C. Ruckman

Albert C. Ruckman, P.E.

ALBUQUERQUE, NEW MEXICO
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2011 SOUTH CERRILLOS ROAD
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METL

METALS ENGINEERING & TESTING LABORATORIES
3701 W. Thomas Rd. • Phoenix, AZ 85019 • (602) 272-4571

DATE March 5, 1990

YOUR P.O. NO. F2828

TO: SERGENT, HAUSKINS & BECKWITH

MATERIAL Rebar A-7

ATTN:

SPECIFICATION

LAB No. 30-79

P/N

HEAT

LOT

S/N

SIZE

Job No. FT90-3234

DENTIFICATION	SIZE	AREA	YIELD STRENGTH		TENSILE STRENGTH		ELONGATION		REDUCTION OF AREA		CODE
			LOAD	POUNDS PER SQ. IN.	LOAD	POUNDS PER SQ. IN.	IN. 2 IN.	PERCENT	DIM.	PER CENT	
	.625	.3100	15,500	50,000	21,100	68,000	.300	15.0			
	.625	.3100	18,200	58,500	24,500	79,000	.320	16.0			
	.625	.3100	22,500	72,500	30,250	97,500	.542	27.0			
	MINIMUM REQUIREMENTS										
	MAXIMUM REQUIREMENTS										

YIELD STRENGTH DETERMINED BY: ☒ 0.2% OFFSET ☐

(F) Indicates flaw.

) Indicated fracture outside gauge mark.

) Indicates fracture through guage mark or within specimen width of gauge marks.

☐ MEETS SPECIFICATION REQUIREMENTS

☐ DOES NOT MEET SPECIFICATION REQUIREMENTS

RESPECTFULLY SUBMITTED

Carlos Chapa
Quality Coordinator

This is a summary of a limited field review of the Ash Avenue bridge noting the general condition of the appropriate elements of the bridge. The field review was completed with a walk-by procedure.

Span No. 1 East Side of Bridge

There is a longitudinal crack near the top of the arch rib extending from the base of arch to approximately the third spandrel post. Near Spandrel Post No. 1, this crack appears on both the exterior and interior surface of the arch rib. On the northern portion of this arch rib, there are many longitudinal cracks in the bottom section near Spandrel Post No. 11. Spandrel Post Nos. 1, 2, 11, and 12 have been repaired with gunnite. Spandrel Post No. 2 is severely cracked vertically. Spandrel Post No. 3 is severely cracked near the base to arch rib connection. The hinge at the crown is severely cracked, spalled, and delaminated. The curb section on the underside of the deck is severely spalled, delaminated, and exhibits efflorescence. Reinforcing steel is exposed on both the curb section and the parapet section for the full length of the east side of Span No. 1.

Span No. 1 West Side of Bridge

Near Spandrel Post No. 1, the main arch rib has a longitudinal crack located near the top running from about Spandrel Post No. 1 to Spandrel Post No. 2. This crack appears on both the interior and exterior surfaces of the arch rib. There is a longitudinal crack near the bottom side, along the side, extending from Pier No. 1 south to approximately the three-quarter point. The crown hinge is severely cracked, spalled, and delaminated. The underside of the curb section exhibits efflorescence and the parapet curb is delaminated; much of the reinforcing steel is missing for approximately one-quarter of the length. Pier No. 1 exhibits much cracking in the triangular section of the pier with horizontal cracks and vertical cracks around the shaft near where the arch rib connects into the shaft.

Span No. 2 East Side of Bridge

There is a longitudinal crack near the top side of the arch on the interior surface near Spandrel Post No. 1. Spandrel Post Nos. 10 and 11 have been repaired by gunnite. Spandrel Post Nos. 1, 2, and 3 exhibit vertical cracks near the base at the connection between the spandrel post and the arch rib. The underside of the parapet curb exhibits efflorescence, spalling, and has exposed reinforcing steel in much of the area. The parapet curb is spalled and delaminated most of the length. The crown hinge exhibits vertical cracks near the pin.

Span No. 2 West Side of Bridge

There is a longitudinal crack near the bottom side in the main arch rib near the quarter point and the three-quarter point. Spandrel Post Nos. 2, 10, and 11 have been repaired by gunnite. The curb section underside exhibits much efflorescence and reinforcing steel is exposed in some areas. Pier No. 2 exhibits many vertical and horizontal cracks near where the arch ribs connect into the shaft. Most of the parapet curb is intact.

Span No. 3 East Side of Bridge

There is a longitudinal crack near the top of the arch, running between Spandrel Post Nos. 1 and 2; it appears on both the interior and exterior surfaces of the arch rib. There is a longitudinal crack near the top of the arch rib between Spandrel Post Nos. 11 and 12, appearing on both the interior and exterior surfaces of the arch rib. Spandrel Post Nos. 3, 10, and 11 have many vertical cracks. The underside of the deck is spalled and delaminated and exhibits efflorescence, with much of the reinforcing steel exposed in both the curb and parapet section for approximately three-quarters of the length. The crown hinge is severely cracked, spalled, and delaminated.

Span No. 3 West Side of Bridge

The underside of the overhang is spalled and delaminated with the reinforcing steel exposed in some of the area. Reinforcing steel is exposed at the parapet curb at about one-quarter of the length.

Span No. 4 East Side of Bridge

The arch rib is cracked longitudinally near the top from Spandrel Post Nos. 1 and to halfway between Spandrel Post Nos. 2 and 3. This crack appears on both the interior and exterior surfaces of the arch rib. Between Spandrel Post Nos. 11 and 12, the arch rib is cracked near the top longitudinally on the exterior surface. The crown hinge is severely cracked. More than half of the curb section is spalled and delaminated with the reinforcing steel exposed and also hanging. The underside of Span No. 4 is darkened and appears it has been exposed to a fire. Spandrel Post Nos. 1 and 2 have been repaired by gunnite. Spandrel Post Nos. 2, 3, and 11 are severely cracked.

Span No. 4 West Side of Bridge

There is a crack running parallel near the top of the arch rib near Spandrel Post No. 1. The crack appears on the interior and exterior surfaces of the arch rib; there is a crack near the top side of the arch between Spandrel Post Nos. 11 and 12 located on the exterior surface. The crown hinge is severely cracked. The underside of the overhang has reinforcing bars exposed for approximately 2,550 feet of the length of the span. The parapet curb is spalled, delaminated, with much of the reinforcing steel hanging for approximately one-quarter of the span. At Pier No. 4, there is scour with much of the footing exposed. The top of the pier shaft is cracked horizontally around each end of the pier shaft under the arch ribs.

Span No. 5 East Side of Bridge

The arch rib is cracked longitudinally near the top side and between Spandrel Post Nos. 1 and 2 and shows on both the interior and exterior surfaces of the arch rib. The top side of the arch rib is cracked longitudinally on the interior surface near Spandrel Post No. 12. The crown hinge has much cracking. Spandrel Post No. 2 has failed horizontally. Spandrel Post No. 3 connection to the arch is spalled with the reinforcing steel exposed. Spandrel Post No. 10 is cracked. Spandrel Post No. 11 has the reinforcing steel exposed near the face to arch connection. The underside of the curb section is cracked longitudinally much of the length. The parapet curb section is spalled, delaminated, with reinforcing steel exposed much of the length.

Span No. 5 West Side of Bridge

There is a crack near the top side of the arch rib that goes from the base to Spandrel Post No. 2 on the exterior surface. There is a longitudinal crack near the side of the arch rib near Spandrel Post No. 11. It shows on the exterior surface of the arch rib. The crown hinge is severely crack and has signs of efflorescence. The underside of the overhang shows signs of efflorescence, with some longitudinal cracking. The parapet curb is delaminated most of the length with the reinforcing steel showing. The reinforcing steel is exposed in the spandrel column over Pier No. 4 at the expansion device. Spandrel Post Nos. 2 and 3 have vertical cracks with the reinforcing steel showing. Spandrel Post No. 10 has been repaired by gunnite and is cracked diagonally. Spandrel Post No. 11 is cracked diagonally.

Span No. 6 East Side of Bridge

There is a crack in the top of the arch rib extending from Spandrel Post Nos. 1 through 3 on the exterior surface of the arch rib. There is a longitudinal crack which extends from Spandrel Post No. 10 to the end of the arch rib near the top on the exterior surface. The crown hinge is cracked with some of the reinforcing steel showing. The underside of the overhang has transverse cracks and exhibits efflorescence. The parapet curb is delaminated and spalled most of the length with some of the reinforcing steel exposed. Spandrel Post Nos. 1 and 2 are cracked horizontally. Spandrel Post No. 3 has been repaired by gunnite but is also cracked. Spandrel Post No. 4 appears to be cracked at the base to arch rib connection. Spandrel Post Nos. 9 and 10 are cracked near the base to arch rib connection.

Span No. 6 West Side of Bridge

Spandrel Post Nos. 3 and 11 have been repaired by gunnite. Spandrel Post No. 2 is cracked horizontally. Spandrel Post No. 3 has the reinforcing steel exposed . Spandrel Post No. 4 is cracked horizontally at the arch rib connection. Spandrel Post No. 9 is cracked at the base to arch rib connection. Spandrel Post No. 10 has map cracking throughout. The underside of the overhang exhibits efflorescence with transverse cracks throughout. The parapet curb is delaminated and spalled with reinforcing steel exposed in much of the area. Pier No. 6 is cracked around the shaft underneath the arch rib support.

Span No. 7 East Side of Bridge

On the east side of the structure on the main arch rib between Spandrel Post Nos. 1 and 2, there is a longitudinal crack at the top of the arch rib; it shows for part of the length on the interior surface of the arch. It shows for the full length between Spandrel Posts Nos. 1 and 2 on the exterior surface. There is a longitudinal crack near the top of the arch between Spandrel Post Nos. 10 and 11 on the exterior surface of the arch. It partially shows on the interior surface also. Spandrel Post Nos. 3 and 10 have been repaired by gunnite. Spandrel Post Nos. 1 and 2 are cracked horizontally near the base to arch rib connection. Spandrel Post No. 2 is also cracked horizontally approximately halfway up. Spandrel Post No. 3 has vertical and horizontal cracks throughout. Spandrel Post Nos. 10 and 11 are cracked and delaminated near the arch rib connection.

Span 7 West Side of Bridge

There is a longitudinal crack near the top side of the arch rib on the exterior surface, running from Pier No. 6 to Spandrel Post No. 2. There is longitudinal crack between Spandrel Post Nos. 11 and 12 near the top on the interior surface, running parallel to the top of the arch rib. Spandrel Post Nos. 2, 3, and 12 have been repaired by gunnite. Spandrel Post No. 1 has the reinforcing steel exposed near the base to arch rib connection. Post No. 3, which was repaired, is cracked at a diagonal through the post. Spandrel Post No. 3 is cracked horizontally at the base to arch connection with the reinforcing steel exposed. Spandrel Post No. 4 is cracked horizontally at the arch rib connection. Spandrel Post No. 10 is severely cracked horizontally at the base to arch rib connection, with some vertical cracks. Spandrel Post No. 11

is cracked at the base horizontally, with a diagonal crack extending up the post. Spandrel Post No. 11 is cracked horizontally at the base with vertical cracks. Pier No. 7 is cracked horizontally around the full width of the shaft, approximately one foot down where the arch ribs are supported, with cracking located throughout.

Span No. 8 East Side of Bridge

There is a longitudinal crack which parallels the top of the arch rib extending from Spandrel Post Nos. 1 through 3 on the exterior surface of the arch rib. There is a longitudinal crack on the arch rib, paralleling the top of the arch between Spandrel Post Nos. 11 and 12 on the exterior surface. Spandrel Post No. 2 has been repaired by gunnite. Spandrel Post No. 1 is cracked horizontally at the spandrel post base to arch rib connection. Spandrel Post No. 2 exhibits map cracking horizontally and vertically. Spandrel Post No. 3 has a horizontal crack at the base to arch rib connection. Spandrel Post No. 10 has vertical and horizontal cracks near its base. Spandrel Post No. 11 is spalled and delaminated with the vertical cracks running diagonally. The underside of the overhang exhibits efflorescence with transverse cracks. Parapet curb is cracked longitudinally with the reinforcing steel exposed.

Span No. 8 West Side of Bridge

There is a longitudinal crack extending from Spandrel Post Nos. 1 and 2 in the bottom side of the arch rib. There is a longitudinal crack near the bottom of the arch rib on the exterior surface running from Spandrel Post No. 10 to the end of the arch rib. Spandrel Post Nos. 2, 3, and 11 have been repaired by gunnite. Spandrel Post No. 10 is cracked horizontally near the base with some map cracking up through the spandrel arch. Spandrel Post No. 11 is delaminated and spalled. Spandrel Post No. 12 has the vertical reinforcing bars exposed. The underside of the overhang shows signs of efflorescence with the parapet curb spalled and delaminated for approximately one-quarter of the length and longitudinal reinforcing steel exposed. The crown hinge shows map cracking and exhibits efflorescence.

Span No. 9 East Side of Bridge

There is a longitudinal crack near the top of the arch rib running parallel to the arch rib between Spandrel Post Nos. 1, 2, and 3 on the exterior surface. Between Spandrel Post Nos. 11 and 12 there is a longitudinal crack near the top of the arch on the exterior surface; it also partially shows on the interior surface. Spandrel Post Nos. 2, 10, 11, and 12 have been repaired by gunnite. Spandrel Post No. 1 has exposed reinforcing steel at the base to arch rib connection. Spandrel Post No. 2 shows cracked gunnite throughout. Spandrel Post No. 3 is cracked and shattered near the base. Spandrel Post No. 4 is cracked horizontally at the spandrel post to arch rib connection. Spandrel Post Nos. 5, 6, 7, and 9 appear to be cracked at the arch rib connection. Spandrel Post No. 10 is spalled and delaminated with the reinforcing steel exposed. At Spandrel Post No. 11, the gunnite is cracked and shattered throughout. Spandrel Post No. 12 is cracked horizontally near the top with the reinforcing steel exposed. The crown hinge shows map cracking. The underside of the deck exhibits efflorescence. The railing is missing for about three-quarters of the length of the span.

Span No. 9 West Side of Bridge

There is a longitudinal crack near the top side of the arch rib at Spandrel Post No. 2 on the exterior surface of the arch. There is a longitudinal crack running parallel to the arch rib near the top at Spandrel Post No. 11. Spandrel Post Nos. 2, 3, 4, 10, 11, 12, and 13 have been repaired by gunnite. Spandrel Post No. 1, the vertical reinforcing steel is exposed. Spandrel Post No. 2, the gunnite repairs are cracked and spalled. Spandrel Post No. 3, the vertical reinforcing steel is exposed with much of the gunnite delaminated and spalled. Spandrel Post No. 4 is cracked horizontally at the base to arch rib connection. Spandrel Post Nos. 6 and 7 are also cracked horizontally at the arch rib connection. Spandrel Post No. 9 and spandrel arches are cracked, spalled, and delaminated. At Spandrel Post No. 10, much of the gunnite is missing with only the vertical reinforcing steel exposed. At Spandrel Post No. 11, the gunnite is spalled, delaminated, with the horizontal wiring and some of the vertical reinforcing steel exposed. Spandrel Post No. 12 is spalled with some of the vertical reinforcing steel exposed. The underside of the overhang shows signs of efflorescence and longitudinal cracks. In the parapet curb, the longitudinal reinforcing steel is exposed for approximately ten percent of the span length. Approximately one-half of the rail is missing for the span.

Span No. 10 East Side of Bridge

There is a longitudinal crack near the top of the arch rib extending between Spandrel Post Nos. 1 and 2 on the exterior surface and partially showing on the interior surface. The bottom side of the arch rib, between Spandrel Post Nos. 10 and 11, is spalled with the main reinforcing steel exposed. The main reinforcing steel is buckled. At this point, the arch rib is cracked diagonally and horizontally throughout. It appears that these cracks run the full width of the arch rib. Between Spandrel Post Nos. 9 and 10, there are vertical cracks in the arch rib. Spandrel Post Nos. 2, 8, 10, 11, and 12 have been repaired by gunnite. Spandrel Post No. 1 is spalled with the reinforcing steel exposed near the arch rib to connection. At Spandrel Post No. 2, the gunnite is cracked vertically. Spandrel Post No. 3 is cracked horizontally at the arch rib connection. Spandrel Post No. 4 and the spandrel arch are cracked diagonally. This crack also extends through the bracket for the overhang of the slab. Spandrel Post Nos. 6 and 7 are cracked horizontally. Spandrel Post No. 9 is cracked horizontally through the spandrel post and spandrel arch. Spandrel Post No. 10 is spalled and delaminated with much of the reinforcing steel exposed. At Spandrel Post No. 11, the gunnite is spalled, with the wires for the gunnite and vertical reinforcing steel exposed. Spandrel Post No. 12 is cracked horizontally near the top of the spandrel arch to the spandrel post connection and at the base; main reinforcing steel is also exposed. The crown hinge is cracked with much map cracking. The underside of the deck is spalled with reinforcing steel exposed. It also exhibits efflorescence and transverse cracks through approximately one-half of the span. The railing is missing for approximately half of the span.

Span No. 10 West Side of Bridge

There is a vertical crack in the main arch rib between Spandrel Post Nos. 1 and 2 which shows on the exterior surface of the arch rib and on the interior surface. There is a longitudinal crack near the top of the arch rib extending between Spandrel Post No. 1 and 3 on the exterior surface of the arch rib. This crack partially shows on the interior surface also. There is a vertical crack in the main arch rib located between Spandrel Post Nos. 9 and 10, extending from the bottom to the top. There is also a vertical crack in the arch rib extending from the top to about halfway through the arch just north of Spandrel Post No. 10. The vertical crack halfway between Spandrel Post Nos. 9 and 10 can also be noted on the interior surface of the arch rib. Spandrel Post Nos. 2, 3, 4, 9, 10, 11, and 12 have been repaired by gunnite. Spandrel Post No. 1 is cracked horizontally near the base to arch rib connection. Spandrel Post No. 2 is cracked and spalled throughout. Spandrel Post No. 3 is spalled and delaminated, with the main vertical reinforcing steel showing. Spandrel Post No. 4 and the spandrel arch are crushed and cracked. Spandrel Post Nos. 6 and 7 are cracked horizontally. Spandrel Post No. 9 is cracked horizontally with the main vertical reinforcing bars bent slightly. Spandrel Post No. 10 is spalled and delaminated with much of the gunnite missing. Spandrel Post No. 11 is spalled and delaminated full height. The underside of the deck for half of the span shows signs of efflorescence with longitudinal cracks and curb reinforcing steel exposed. The railing is missing for approximately one-half length of the span. The crown hinge has much map cracking .

Span No. 11 East Side of Bridge

There is a longitudinal crack running parallel to the top of the arch rib near Spandrel Post No. 10 down to the support shown on the exterior surface only. Spandrel Post Nos. 2, 9, 10, and 11 have been repaired with gunnite. The reinforcing bars are exposed in Spandrel Post No. 2. Spandrel Post No. 3 is cracked horizontally near the base to arch rib connection. It appears that Spandrel Post No. 4 may also be cracked near the base to arch rib connection. The crown hinge exhibits map cracking with efflorescence. The underside of the deck exhibits some signs of efflorescence with some transverse cracks in the overhang. The parapet section reinforcing bars are exposed for about 10% of the span.

Span No. 11 West Side of Bridge.

There is a diagonal crack running horizontally from Spandrel Post No. 1 in the arch rib. Spandrel Post Nos. 3, 6, 7, 9, 10, and 11 have been repaired with gunnite. The spandrel columns at Pier No. 10 are cracked horizontally near the base and near the top on both the east and west side. Spandrel Post No. 1 is cracked horizontally near the top at the spandrel arch connection. Spandrel Post No. 2 exhibits map cracking with vertical cracks throughout the length with a horizontal crack in the spandrel arch. Spandrel Post No. 3 has the main reinforcing steel exposed. The crown hinge exhibits much map cracking throughout.

Span No. 1, Deck

There is a transverse crack between the abutment spandrel column and Spandrel Post No. 1 in the deck exhibiting signs of efflorescence. There is a transverse crack extending full width in Bay No. 3 also through the spandrel arches also showing signs of efflorescence. There is a transverse crack approximately three feet long in Bay No. 4. There are several transverse cracks approximately three feet long showing signs of efflorescence in Bay No. 7. There are several transverse cracks exhibiting efflorescence in Bay No. 8. The deck is severely cracked in Bay No. 10, full width. There is a patch with a wooden piece of plywood placed in it approximately two by three feet square. There is a transverse crack the full width of the deck in Bay No. 11, extending down the spandrel arches.

Span No. 2, Deck

There is a transverse crack the full width of the deck in the first bay approximately three feet from the pier. There is a transverse crack in Bay No. 2 running the full width of the deck. There is a transverse crack in Bay No. 3 running the full width of the deck and running down the spandrel arches. There is a transverse crack in Bay No. 12 running the full width of the deck and down the spandrel arches.

Span No. 3, Deck

There is a transverse crack in Bay No. 1 running the full width of the deck and down the spandrel arches.

Span No. 4, Deck

There is a transverse crack running the full width of the deck and down the spandrel arches. The underside of Span No. 4 is darkened and appears to have been exposed to fire at some time. There is approximately a three-foot transverse crack in Bay No. 9. There are two transverse cracks in Bay No. 10 running the full width of the deck and down the spandrel arches. There is a transverse crack in Bay No. 12 running the full width of the deck and down the spandrel arches.

Span No. 5, Deck

There are several small transverse cracks in Bay No. 1 which shown signs of efflorescence. There are several small transverse cracks in Bay No. 9. There is a transverse crack in Bay No. 11, running between the spandrel arches. There is a transverse crack in Bay No. 12 extending from spandrel arch to spandrel arch.

Span No. 6, Deck

There is a full width crack transverse across the deck and through the spandrel arches. Near the mid-span, the bottom reinforcing steel is exposed. There are several small transverse cracks in Bay No. 9. There is a transverse crack running full width of the deck in Bay No. 10 exhibiting efflorescence. In Bay No. 11, there are two transverse cracks; one extending from spandrel arch to spandrel arch and the other from spandrel arch to the longitudinal center girder. In Bay No. 12, there are three transverse cracks; one transverse crack extends full width of the deck, and the other two transverse cracks extend from spandrel arch to center longitudinal girder.

Span No. 7, Deck

There are two transverse cracks in Bay No. 1; one transverse crack runs the full width of the deck, and the other crack runs from spandrel arch to center longitudinal girder. In Bay No. 2, there are several small cracks extending from spandrel arch to center longitudinal girder. At Bay No. 4, there are two transverse cracks; one crack from spandrel arch to spandrel arch, the other for approximately half the length. Span No. 7 is darkened and appears to have been exposed to fire at some time.

Bay No. 8 has a transverse crack full width. Bay No. 9 has a transverse crack full width between spandrel arches. There is a transverse crack in Bay No. 10 from spandrel arch to spandrel arch. There are several small transverse cracks at Bay No. 11, with one extending from spandrel arch to spandrel arch.

Span No. 8, Deck

There is a transverse crack in Bay No. 1 extending approximately one-half the width. In Bay No. 2, there are three transverse cracks, approximately one-half the width of the deck. In Bay No. 3, there are two transverse cracks; one extending from spandrel arch to spandrel arch, and the other half the width of the deck. In Bay No. 4, there are four transverse cracks extending about one-half the width of the deck. In Bay No. 5, there are two transverse cracks extending one-half the width of the deck. Span No. 8 is darkened and appears to have been exposed to fire at some time. Bay No. 8 has two transverse cracks about half the width of the deck. Bay No. 9 has six transverse cracks, approximately one-half the width of the deck. Bay No. 9 has six transverse cracks approximately one-half the width of the deck exhibiting efflorescence. Bay No. 10 has two transverse cracks; one approximately extending from spandrel arch to spandrel arch, and the other about one-half the width of the deck. Bay No. 11 has approximately nine transverse cracks, all discontinuous. Bay No. 12 has a transverse crack approximately two feet from the pier extending from spandrel arch to spandrel arch.

Span No. 9, Deck

The bottom side of the deck is darkened and appears to have been exposed to fire at some time. Bay No. 2 has several transverse cracks (approximately seven) about one-half the width of the deck. Bay No. 2 has one transverse crack from spandrel arch to spandrel arch and two approximately one-half the width of the deck. Bay No. 4 has five transverse cracks about one-quarter width of the deck. Transverse Floorbeam No. 5, 6, 7, 8 have been repaired with gunnite. Bay No. 10 has two transverse cracks exhibiting efflorescence. Bay No. 11 has one transverse crack exhibiting efflorescence.

Span No. 10, Deck

Bay No. 1 has one transverse crack about one-quarter the width of the deck. Bay No. 2 has a transverse crack extending from spandrel arch to spandrel arch. Bay No. 3 has one transverse crack extending from spandrel arch to spandrel arch. Bay No. 6 has exposed reinforcing steel for an area of approximately 1-1/2'x1-1/2'. Transverse Floorbeam No. 6, 7, and 8 have been repaired by gunnite. Bay No. 8 has two transverse cracks about one-quarter the width of the deck. Bay No. 9 has one transverse crack extending from small spandrel arch to spandrel arch. Bay No. 10 has several cracks exhibiting efflorescence. Bay No. 11 has several cracks exhibiting efflorescence. Bay No. 12 has several cracks exhibiting efflorescence.

Span No. 11, Deck

Bay No. 1 longitudinal center girder is cracked diagonally. Both spandrel arches are cracked diagonally near the center. Bay No. 2 has three transverse cracks exhibiting efflorescence. Bay No. 3 has two transverse cracks extending from spandrel arch to spandrel arch. Bay No. 4 has one transverse crack extending from spandrel arch to spandrel arch and one which is about one-quarter of the width of the deck.

51M/MI/AL5

No allowance shall be made for wire ties, spacers, etc.

In estimating reinforcement the bars shall be measured by the lineal foot as laid. All laps shall be allowed for.

The bars of each different size shall be measured and described separately, as also straight bars, bent bars, stumps, and hooping.

Pipe sleeves, turnbuckles, clamps, threaded ends, nuts, and other forms of mechanical bond shall be measured separately by number and size and allowed for in addition.

Wire cloth, expanded metal, folded fabric and other steel fabrics sold in sheets or rods, shall be measured and described by the square foot. The size of mesh steel in tension and weight per square foot shall be stated. No allowance shall be made for waste, cutting, etc.

Deformed bars shall be measured separately from plain.

4. SURFACE FINISH

The unit of measure for finish of concrete surfaces shall be the square foot. Finish shall always be measured and described separately.

No measurement or allowance shall be made for going over concrete work after removing of forms and patching up voids and stone pockets, removing fins, etc.

Granolithic finish shall be measured by the square foot and shall include all labor and materials for the specified thickness.

Finish laid integral with the slab shall be measured separately from finish laid after slab has set.

No allowance shall be made for protection of finish with sawdust, sand or testing.

Grooved surfaces, gutters, curbs, etc., shall be measured separately from plain granolithic and shall be measured by the square foot to lineal foot as the case may require.

The following shall be measured by the square foot:

Cement wash (state how many coats).
Rubbing with carborundum.
Scrubbing with wire brushes.
Tooling.
Picking.
Plastering, etc.

The Reinforced-Concrete Bridge at Tempe, Ariz.

The recently admitted State of Arizona had under its territorial government an engineering department which comprised a well organized highway division. As a part of the work of this division, in the spring of 1911 it was ordered by the board of control of the territory to build a bridge across the Salt River at Tempe, Maricopa County, about nine miles east of Phoenix, to carry the main line of a north and south territorial highway then under construction. The then territorial engineer, J. B. Girand, proceeded to carry out the instructions and started the design of the bridge a year ago, and since then has begun construction. The bridge, which is described below, from information forwarded us by Mr. Girand, is somewhat out of the ordinary in design.

Salt River at the location of the bridge is normally a small stream only about 40 ft. in width, running through the typical sandy country of southern Arizona. It lies in a wide valley some 1600 ft. from height to height of land, a large portion of which is filled at the time of occasional floods. It was, therefore, necessary to carry the highway on some sort of a structure to a distance of approximately 1600 ft., and after a study of the situation it was decided to build a series of reinforced-concrete arches, each 125 ft. in span, making a total length of 1507 ft. 9 in. The decision to use concrete was reached largely because of the fact that convict labor could be used in the construction, thus reducing the cost of the bridge to superintendence and material.

On account of the failure of two piers of a large railroad bridge about 500 ft. upstream from the proposed bridge site, it was decided that all piers and abutments of the new bridge should be built on bed rock. Test holes at intervals of 100 ft. along the center line showed that bed rock would be found at an average of 30 ft. below the surface, except in the main channel at the north side of the river bed where the rock had a considerable sag, the greatest depth being 44 ft. below the surface. The general design finally decided upon and the profile of the river bed and the rock bottom is shown in Fig. 1. As shown there, it was decided to make every third pier an abutment pier beginning with Pier 10 and extending to Pier 4. The remaining spans were equally divided by making Pier 2 an abutment pier. The intermediate piers did not require a great width of base for bearing, and as the height of stream line above rock would require an amount of concrete which would have been excessive in cost if carried down to bed rock, it was decided to use two steel cylinders of 6 ft. diameter, driven to rock under each intermediate pier. On account of the depth to rock, this same scheme was used for the south abutment foundation and for Pier 4, except that the number of cylinders was increased in these two.

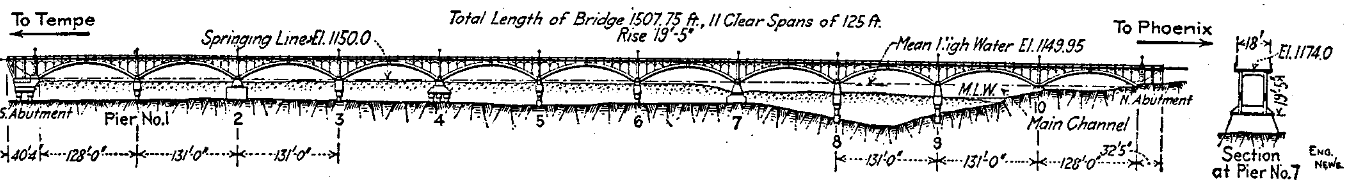
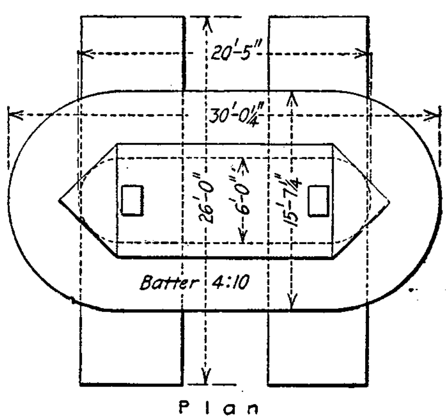


Fig. 1. Outline Elevation of the Tempe Reinforced-concrete Bridge across the Salt River, Near Phoenix, Ariz.



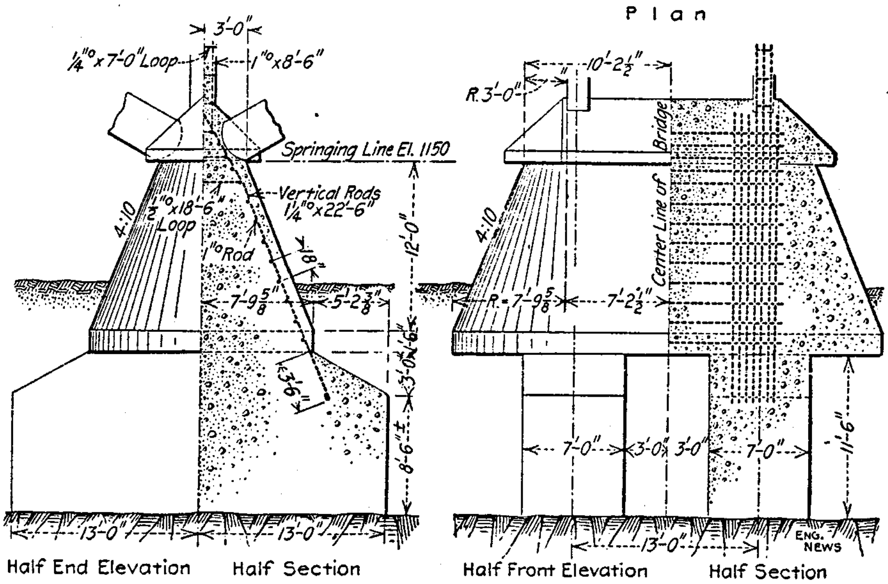


Fig. 2. Details of Pier 7, Typical of Solid-bottom Piers

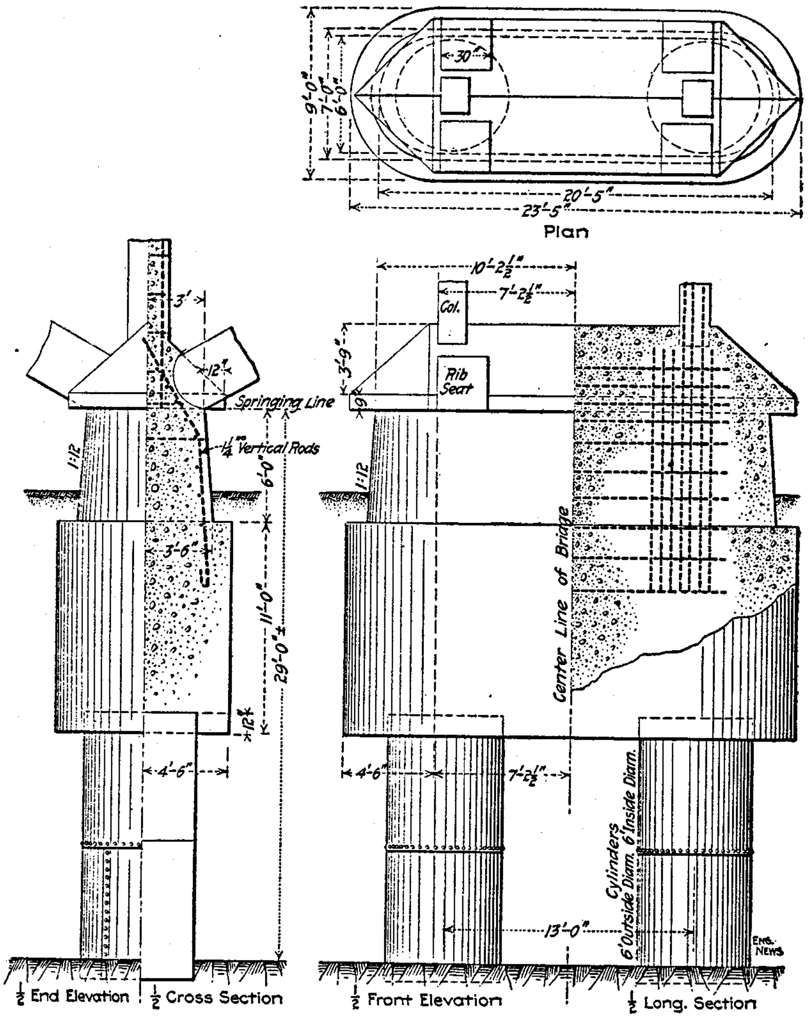


FIG. 3. DETAILS OF PIER 1, TYPICAL OF CAISSON-FOUNDED PIERS

Figs. 2 and 3 show the construction of two typical piers; Fig. 2 representing the abutment pier, No. 7, brought down to bed rock by solid construction, and Fig. 3 showing one of the intermediate piers, No. 1, founded on the steel cylinders. The abutment pier (No. 7) consists of two plinths of concrete 7 ft. wide, transverse to the bridge line, placed 6 ft. apart, and carrying on their top a solid section which is battered up to above the ground level where the skewbacks to the bridge enter. This pier is reinforced, as shown, only against possible contraction or expansion and not to carry any designed load. It will be noted that it is of peculiar design in that the main bearing portion, which is superposed on the two lower plinths, is overhung to a considerable extent.

Pier 1 (Fig. 3) is typical of the intermediate piers founded on the steel cylinders. These piers were constructed by excavating an open cut to the elevation shown at the top of the cylinders and then by sinking the steel cylinders to bed rock by the tubbing method. This tubbing method, as is well known, consists in placing the cylinders on the ground and excavating from them in open, allowing them at the same time to sink to the bed-rock level. When these cylinders reached the bed rock, they were filled with concrete and upon them was then built the solid block which forms the footing to the pier and on this block was placed the battered block shown, which varied in height according to the pier. The same peculiar skewback design was placed on these piers as is shown on the solid abutment piers. In calculating the stability of the piers, the surrounding earth was not taken into account.

The arch proper comprises two three-hinged segmental arch ribs placed 13 ft. center to center and carrying the reinforced-concrete slab roadway on spandrel columns. The design of the ribs is shown in Fig. 4. They are, as shown, there, segmental and have a depth of 36 in. at each end and 40 in. at the middle. Their minimum thickness is 17 in. Near the ends they are widened out, having a thickness of 30 in. at the lower end and 24 in. at the crown. These ribs are reinforced with 1⅛-in. round steel rods,

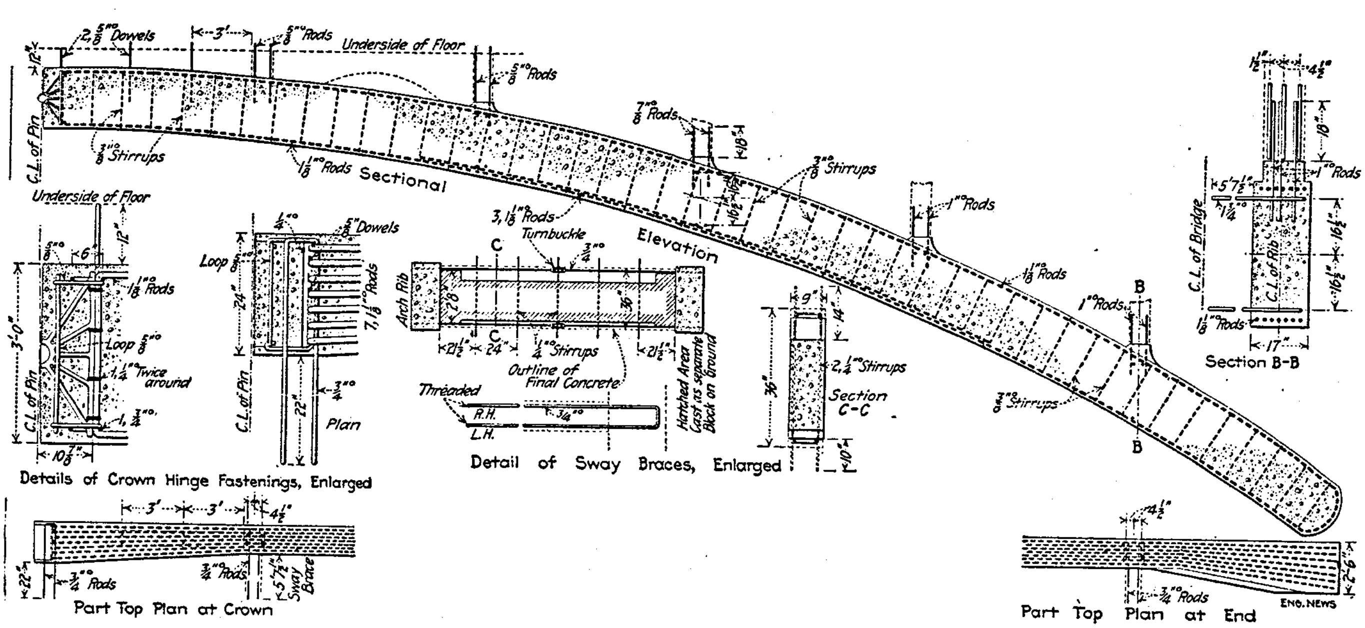


FIG. 4. DETAILS OF ARCH-RIB CONSTRUCTION OF THE TEMPE BRIDGE

longitudinally connected by ⅜-in. round steel stirrups. The hinges at the pier consist merely of a rounded end fitting into a similarly rounded depression in the pier. Galvanized sheet metal is placed in the sockets to separate the concrete of the pier from that of the ribs. At the crown the cast-iron hinge bearing on a 4-in. pin is introduced.

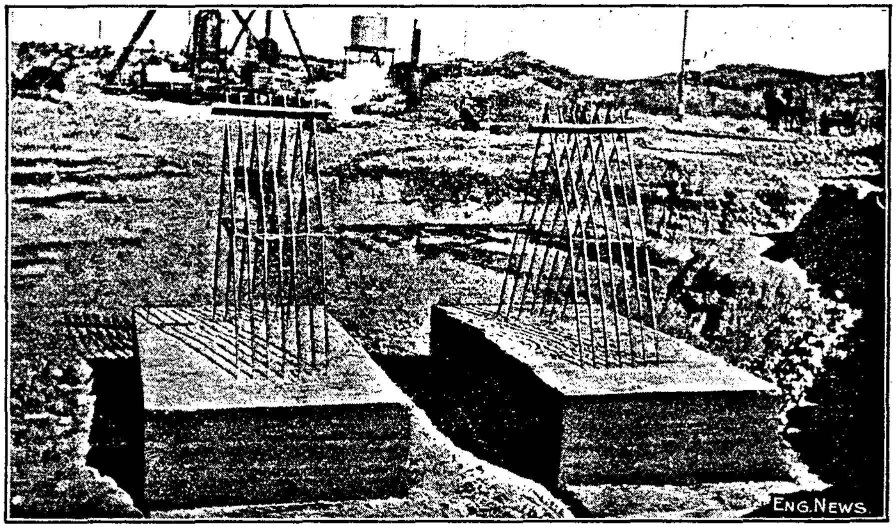


FIG. 5. VIEW OF ONE OF THE SOLID-BOTTOM PIERS

The cross-bracing for the arch rib is somewhat novel, as is shown in the right-hand part of Fig. 4. Here it will be noted that the bracing consists of a solid concrete strut cast in forms on the ground and set in its proper place on the arch ribs. At the locations of these cross-braces steel rods bearing a turnbuckle at the middle are introduced between the two arch ribs during their construction. When the cross-bracing is to be placed, it is lifted into place from the ground and the face of the ribs having been chipped and rich mortar applied it is fitted into place. Then the turnbuckles are tightened until the rods connecting the ribs are as tight as possible and concrete is then placed around the cross-braces, covering both the concrete strut and the turnbuckle rods, leaving the final section a 9x36-in. rectangle.

The floor is carried by 12x12-in. spandrel columns placed about 11 ft. center to center and connected at the top by semicircular spandrel arches longitudinally with the bridge and transversely by girders, which latter carry a beam on the center line of the bridge. Semi-arch brackets cantilevered out from the spandrel columns carry the curb which is designed as a beam and which carries the floor balustrade between the brackets. The balusters are round posts 4 in. in diameter and are cast on the ground.

The floor slab has a thickness of 7 in. at the center line of the bridge and 5 in. at the curb. Expansion joints are provided for this floor at the crown of each arch. The hinges at the crown are masked by a concrete cover-plate. The bridge was designed to carry a 15-ton traction engine plus a live load of 100 lb. per sq.ft. The maximum stresses allowed in the concrete are 700 lb. per sq.in. compression and in the steel 16,000 lb. per sq.in. in tension. As noted above, the entire construction is being carried out under the direction of Mr. Girand, State Engineer; and is being done by convict labor, under supervision of J. C. Ryan, Bridge Engineer. The work of construction was started in the summer of 1911 and is progressing at a fair rate.

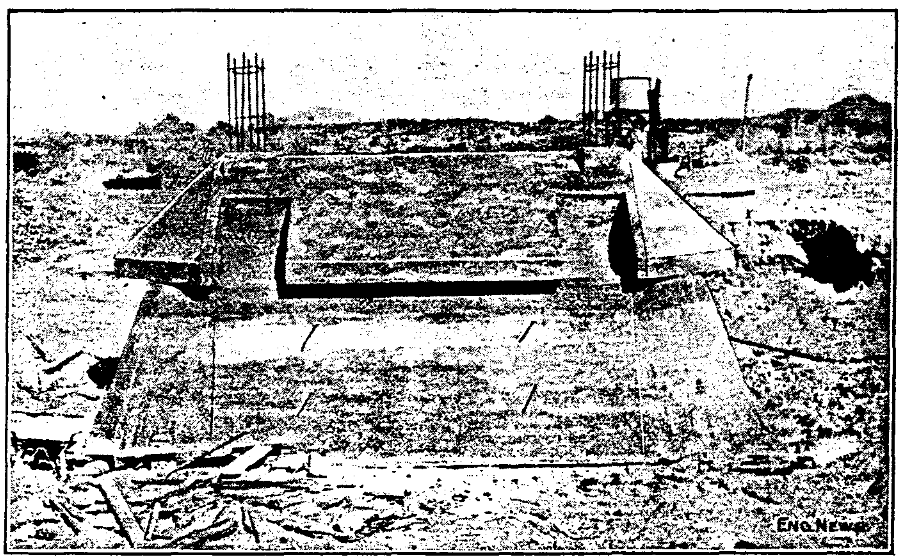


FIG. 6. VIEW OF PIER 7 UNDER CONSTRUCTION

Electric Trunk Railway Operation is actively being planned for in many of the German states. Prussia, Bavaria and Baden have led in this planning, and Saxony is about to consider the subject. The Bavarian and Baden state railways, it is now stated, have adopted as standard the single-phase system, with 15,000 volts trolley-line potential and 16⅔ cycles frequency. This is the same standard as adopted for the Prussian development.

Field Compression Tests of Concrete

BY G. H. BAYLES*

In the spring of 1910 it became the duty of the writer to design and superintend the construction of a reinforced-concrete warehouse in the Borough of Brooklyn, City of New York, for the New York Dock Co. At that time the building regulations of the borough fixed the limit of compressive stress of concrete in flexure at 500 lb. per sq.in. in the outer fiber, with other similarly low unit stresses. From previous experience these stresses seemed to the writer too low, but the borough regulations could not be transgressed, so it was necessary to design the buildings under these specifications. With a view, however, to establishing some authentic strength value of field-made concrete for use in determining the safe allowable unit stress and also to give some criterion of strength by which the time for form removal might be determined, the author decided to carry on during the construction of this and adjacent similar reinforced-concrete buildings a complete series of field tests on concrete cubes made from the concrete as it was placed in the forms for the buildings. Through 15 months, tests were regularly made and proved very useful to the contractor and to the superintending engineer in determining the time for form removal.

On the first contract the tests were put in charge of the two inspectors, both of whom were trained experts in the making and placing of concrete. One superintended the preparation of the concrete and the other placing it in the forms in

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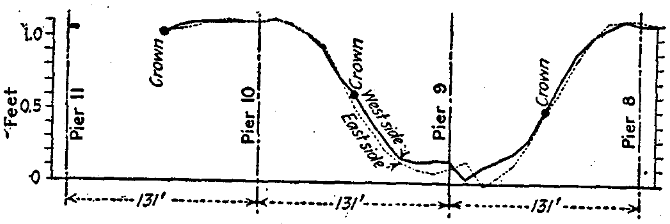
Repair of Tempe Concrete Arch Bridge Damaged by Settlement and Floor Expansion

Bridge Over Salt River Endangered by Failure of Cylinder Pier Supposed to Rest on Rock—Load Transferred to New Cylinders Sunk Alongside Pier—Cracking of Floor and Spandrels—Expansion Joints

BY MERRILL BUTLER
Bridge Engineer, Arizona Highway Department, Phoenix

SETTLEMENT of one pier of the concrete arch bridge across the Salt River at Tempe, Ariz., placed a serious problem before the State Highway Department in the latter part of 1919 and the early part of 1920. The bridge, built in 1911-1913, is a link in the main highway route leading from Phœnix and the Salt River Valley to the eastern and southern portions of the state, and to New Mexico and the East, and is crossed by about 2,500 vehicles per day. It consists of eleven two-rib three-hinged arches of 125 ft. clear span, with open spandrels. The piers were intended to be founded on rock, as stated in a description of the bridge in *Engineering News* of March 28, 1912, p. 578, "on account of the failure of two piers of a large railroad bridge about 500 ft. upstream from the proposed bridge site." Some of the piers were carried to rock in open excavation, but others rest on concrete-filled steel cylinders sunk to rock. It was one of the latter that settled.



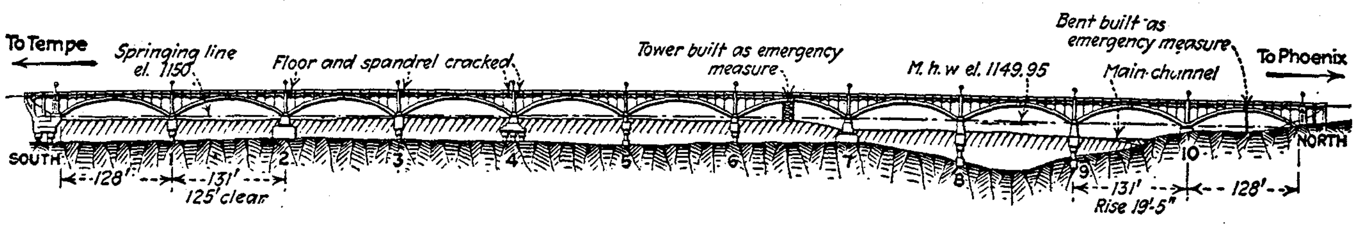


FIG. 1. GENERAL ELEVATION OF TEMPE BRIDGE, AND PROFILE OF ROADWAY IN SPANS ADJACENT TO THE SETTLED PIER NO. 9

Shortly after the floods of Thanksgiving, 1919, the second pier from the north end of the bridge (Pier 9) settled about 4½ in. Traffic was maintained, except during high water, until Feb. 13, 1920, when a further settlement occurred, about ½ in. A two-ton limit was then placed on the loads permitted to cross the bridge. On March 2 an additional settlement of 1⅓ in. occurred, and the bridge was closed to traffic. The following day there was a sudden drop of nearly 5 in. At this time also it was noticed that the pier had shifted out of line about 0.1 ft., downstream.

A profile of the bridge roadway between piers 8 and 10 is shown by a small sketch in Fig. 1, drawn to an exaggerated vertical scale. The sag was strikingly noticeable in looking along the pavement, as in the view, Fig. 2. Evidently the structure adapted itself to the 1-ft. settlement of pier 9 in fairly flexible manner. No evidence of any crack in the floor was found in the region of settlement, although the hand rail was badly cracked; the longitudinal steel in the floor slab probably prevented any serious break in the concrete.

Emergency measures were taken at once to insure the stability of the structure, and the bridge was thrown open to pedestrian traffic on March 4. Material and equipment for sinking cylinders to underpin the defective pier had been in process of being assembled for some time, but repair work could not be started till late in the month as the timber that had been ordered was delayed in shipment. Once begun the work progressed so rapidly that vehicular traffic could be allowed to cross the bridge again on May 11.

CONDITIONS AT THE SETTLED PIER

Prior to the flood of Thanksgiving, 1919, pier 9 was entirely surrounded by sand and gravel, which, being in its undisturbed state, served to carry a considerable portion of the load by way of the base of the pier block (note the pier construction, Fig. 4). The flood swept away all this material and left the pier supported on the two cylinders, which proved inadequate to carry the load. Soundings taken in March, 1920, indicated that except for some thin layers of gravel overlying the bedrock everything had been scoured out. In the light of the difficulties subsequently experienced in sinking the new cylinders it is very probable that the concrete in the bottom of the original cylinders was of inferior grade, or that a foot or so of sand had filtered in after the rock had been cleaned off. The natural consequence would be a crumpling of the steel shells of the cylinders, and this is what actually happened, it is believed. Unbalanced live-load thrust would tend to accelerate such failure.

Other defective conditions in the bridge had also developed, and the plan for the repair work included them. A great number of the spandrel columns were found broken in horizontal shear near the extrados and several spandrel walls near the crown had pulled loose from the arch rings. In the vicinity of piers 2, 3 and 4 the roadway slab and spandrel arches had cracked completely through; in the spans adjacent to these piers none of the spandrel columns were cracked.

There was also trouble at the floor expansion joint. The type of joint used had proved unsatisfactory and large chuck holes had formed alongside each joint, causing serious impact whenever a heavy vehicle passed

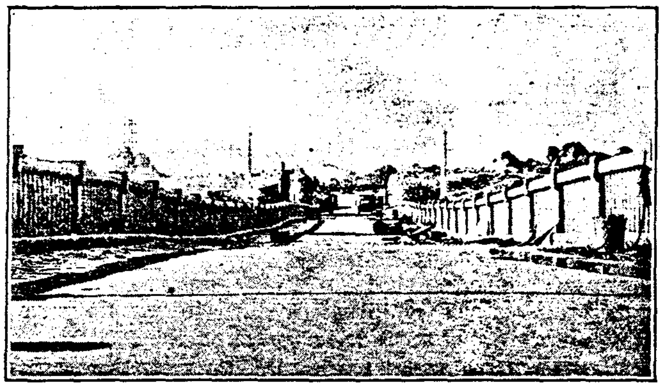


FIG. 2. SAG OF ROADWAY OVER SETTLED PIER

over the bridge; in some cases the concrete supporting the wooden strips which bridged the joints was found to be cracked and broken from traffic action. The reconstruction of these joints as well as the repair of the other bridge defects will be mentioned farther on.

Underpinning the Pier

In April the channel of the river, scoured out by the November flood, began filling up again, and by the middle of May a considerable deposit of silt and fine sand had accumulated around pier 9. This condition and the desirability of maintaining traffic on the bridge were the main factors of the problem when the state highway department entered upon the repair work in the spring of 1920.

Falsework under the bridge was considered necessary as a safeguard if traffic was to be carried during reconstruction. The department had no steel sheetpiling available, and market conditions were such that none could be obtained for immediate delivery. It was decided to place new cylinders around the old pier, which would allow the underpinning to be done without disturbing the existing structure; it was feared that there was a chance that the pier might tip over if the old cylinders were left without lateral support. The total dead load at the base of the cylinders of pier 9, not allowing for buoyancy of the water, was about 1,650,000 lb. Salt River is subject to sudden freshets, and all work had to be planned to withstand a sudden rise of the river at any time. Fortunately no rise greater than a foot occurred during the whole undertaking.

As already mentioned, emergency provision for holding the structure against further movement was made early in March, when, after a sudden new drop of pier 9, the total settlement had reached a foot. A repair gang was hurried to the bridge and put in a 36-hour shift. About 500 sand bags were thrown around pier 9, in the hope that the bearing would be increased and further scour prevented. Towers were built at the crown of spans 7-8 and 10-11, with the object of saving the remaining portion of the bridge if the two spans between piers 8 and 10 went out; the depth of water in the river prevented the construction of supports any nearer to pier 9. With respect to the arrangement of the temporary supports it should be remarked that the deck of the bridge is continuous over the piers, and has expansion joints at the crowns of the arches only (except as subsequently reconstructed). Since this emergency work no further settlement of the pier has been observed.

Later in the month, in preparation for the underpinning work on pier 9, falsework piles were driven in spans 8-9 and 9-10. These had to be placed outside the side lines of the bridge because the driver leads reached above the deck. Framed bents were erected on these piles, and their caps wedged against the intrados of the arch ribs by oak wedges (Fig. 3); a man inspected these wedges every second day, to make sure that none worked loose under the action of traffic.

Work on a cofferdam around pier 9 was started immediately upon completion of the falsework. It was made up of Wakefield piles consisting of three 2 x 6's, 2 x 8's or 2 x 10's, 20 ft. long, driven by a small steam hammer hung from a pair of short leads mounted on skids. A jet was used to facilitate the sinking, but despite the jet there was considerable difficulty in getting the piles down, owing to the compact nature of the sand.

The general scheme of underpinning is clearly indicated by the drawing in Fig. 4. The work of sinking the steel cylinders began early in July. A small stiff-leg derrick was rigged to handle a 2½-ft. orangepeel bucket, operated from an engine near pier 10. A 40-hp. gasoline engine and a belt-driven 8-in. centrifugal pump were installed on a barge near the downstream side of the cofferdam; this outfit at all times kept the water level below the bottom of the new concrete block.

All of the six new cylinders went down easily with-

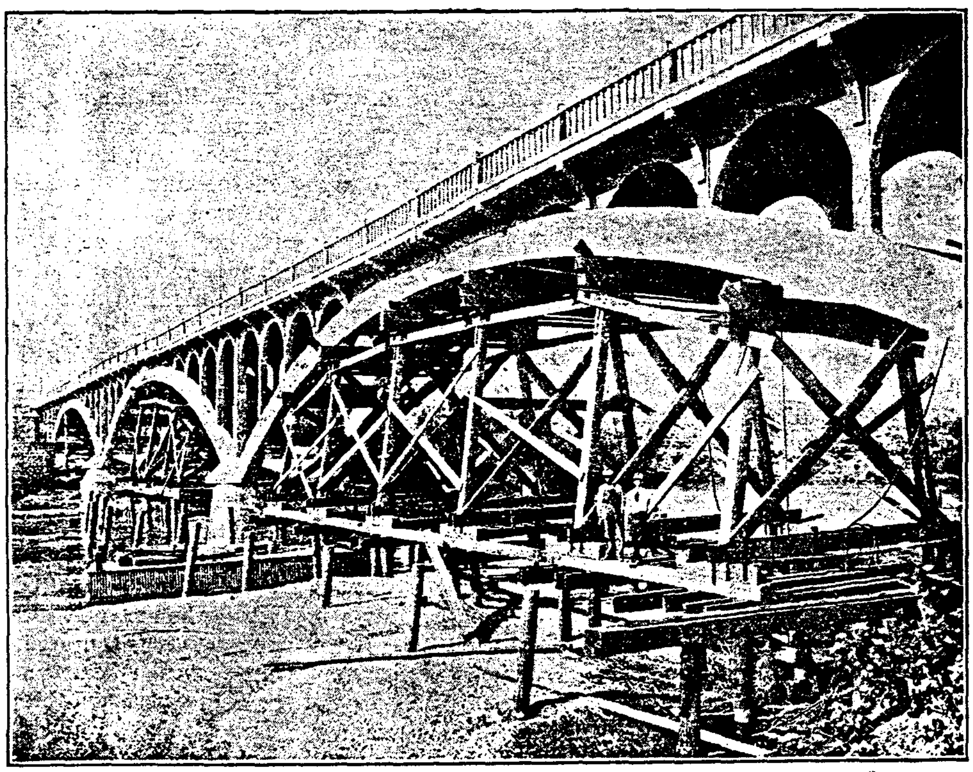


FIG. 3. FALSEWORK ON BOTH SIDES OF PIER 9

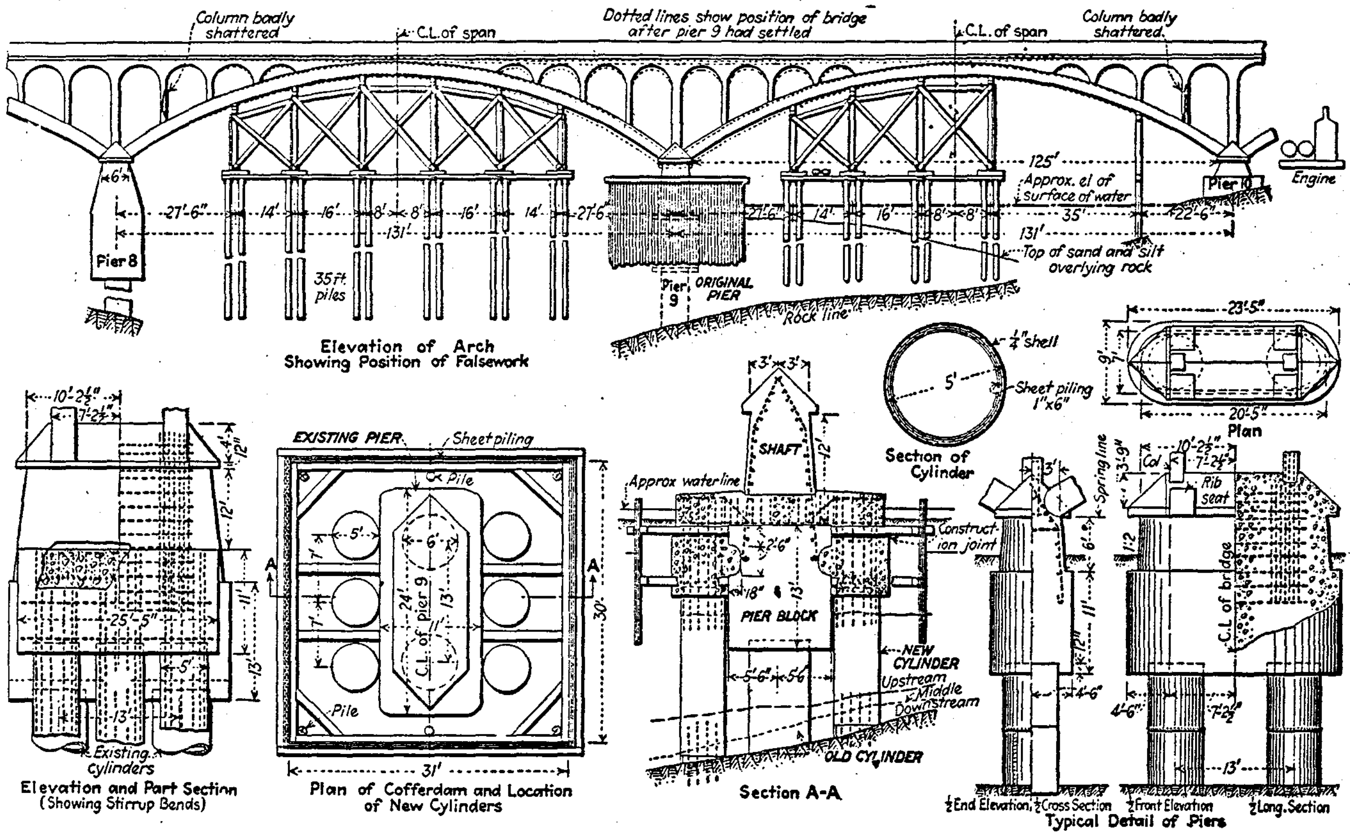


FIG. 4. PIER 9 UNDERPINNED BY NEW CYLINDERS SUNK ALONGSIDE, WITH REINFORCED-CONCRETE BEAM CAP

out loading, until they lodged on logs or branches of trees that had wrapped themselves around the pier during floods. These obstructions caused considerable delay, especially as the fine sand flowed almost with the consistency of water, and it was impossible to pump these cylinders down. Dynamiting was tried, to break up the logs, with little success, and in two cases with rather damaging results, as events proved. Cutting tools of 1⅛-in. drill steel were then made up and these proved very satisfactory for getting the timber obstructions out of the way. Cottonwood logs 8 in. in diameter were removed by this means. After this clearing, no difficulty was experienced in getting the cylinders down to rock. Even then, however, the flow of sand was such that it was impossible to reach the rock for cleaning it off and anchoring the cylinders.

To solidify the sand around each cylinder, at least sufficiently to allow the cylinders to be pumped out, grout apparatus was made up and placed on the job. Steam pressure was found to bake the cement until it clogged the 6-in casing; water pressure worked quite satisfactorily. The general scheme of operation was as follows: Grout was poured into the storage chamber after removing the cap, and when the cap had been replaced the water pressure from the pump was turned on. The valve in the 2-in. pipe was kept closed until pressure was on the storage chamber, in order to prevent sand from working into the bottom of the jet pipe. At first considerable inconvenience was experienced from the entry of sand, but by installing the valve and keeping it closed until full pressure from the pump was on the storage chamber the trouble was largely overcome. The bleeder at the bottom of the storage chamber was used to determine when the grout had been forced out.

In some cases the grout set up, but when excavation was continued, a week after grouting, much material of pasty consistency was removed. Evidently where the sand carried a considerable amount of silt the cement had failed to set. The grouting proved to be only partially successful, in no case entirely stopping the flow of sand and water.

Wakefield sheetpiles of three 1 x 6 boards were then driven inside the cylinders by hand, after the sand had been excavated as low as possible with the orange-peel bucket. Once the piles were down to rock, it was possible to pump the cylinders. The sand was then removed, the rock cleaned off, four anchor holes drilled for 1¼-in. square rods set in pairs, and the anchor rods grouted in. The sheetpiles showed little tendency to kick in at the bottom.

When examined, cylinders 1 and 5 proved to be so badly distorted by the blasting tried for removing logs that it was necessary to pull them and sink new cylinders. The seams were broken and the lower sections forced out of round. Blasting had been done with quarter-stick charges of dynamite.

The bottoms of the cylinders were sealed with concrete, and twelve 1-in. square rods were set in place with sufficient lap below the tops of the anchor rods to provide bond. The cylinders were then concreted up to the construction joints just below the cap concrete. The steel shells were cut off at the proper elevation with an oxyacetylene torch. Then the concrete caps were poured, up to the top of the old pier block. After they had set, the existing shaft was cut out in sections and the reinforced beam which transfers the pier load to the new cylinders was poured.

No effort was made to raise the pier or the floor back to the original elevation. The hand rail, however,

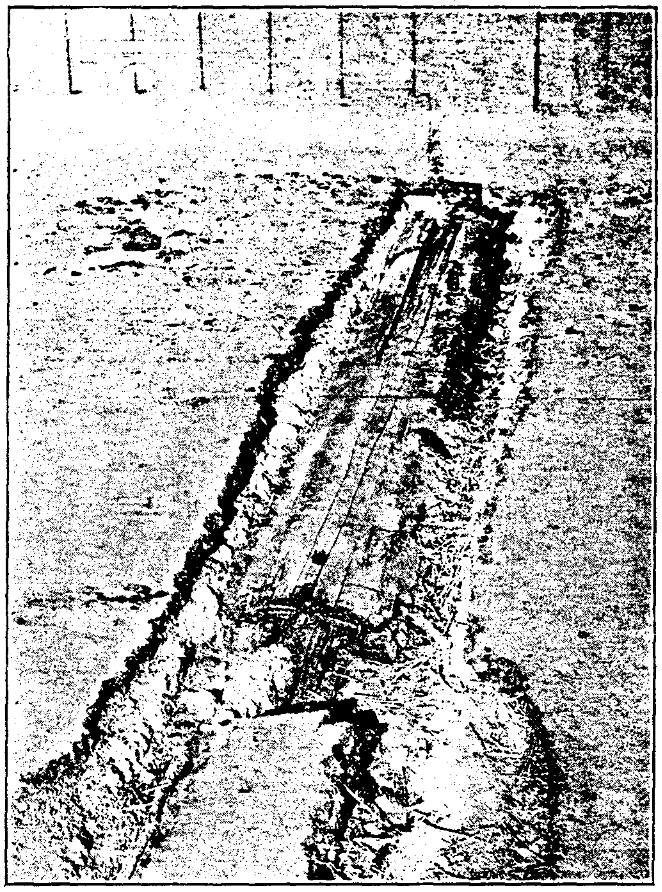


FIG. 5. OLD ROADWAY EXPANSION JOINT

has been rebuilt, to remove the appearance of sag, and it is propcsed to take out part of the dip in the floor by filling in with a wood block pavement between crown expansion joints on either side of pier 9.

At piers 2, 3 and 4 the original plans for the floor did not contemplate the use of the longitudinal steel subsequently decided upon and used in the other panels; instead the slab was thickened and longitudinal steel rails were added here. The reinforcing in the center longitudinal beam, however, remained unchanged throughout the construction. These beams were found to be cracked or broken in diagonal tension in four cases, at the three piers in question, and the roadway slab and spandrel arches were cracked substantially as shown on section *MM*, Fig. 4. In two instances these cracks were so wide that a man's little finger could be inserted.

To repair the damage, the old concrete was cut back to the middle of the panel, and a new girder built alongside the old cross-girder, with an expansion joint as shown. Additional stirrups were placed in the longitudinal beams to take care of horizontal shear. During this work all vehicle traffic was excluded from the bridge.

Renewal of Crown Expansion Joints

As originally built, the floor expansion joints over the crowns of the arches consisted of a beveled strip of redwood spanning the 4-in. gap in the concrete. seated in a properly beveled recess in the floor. This strip was stiffened by another strip attached below as a rib, and was covered with asphalt continuous with that over the concrete. These expansion joints had never been repaired, except for additions to the wearing surface, during the seven-year interval since the bridge was built. Large chuck holes had formed at each joint, Fig. 5, and in some cases the concrete supporting what was left of the redwood filler was cracked and broken.

Experience had shown the impossibility of keeping a satisfactory asphalt surface over an expansion joint supported on both sides of the crown opening. There-

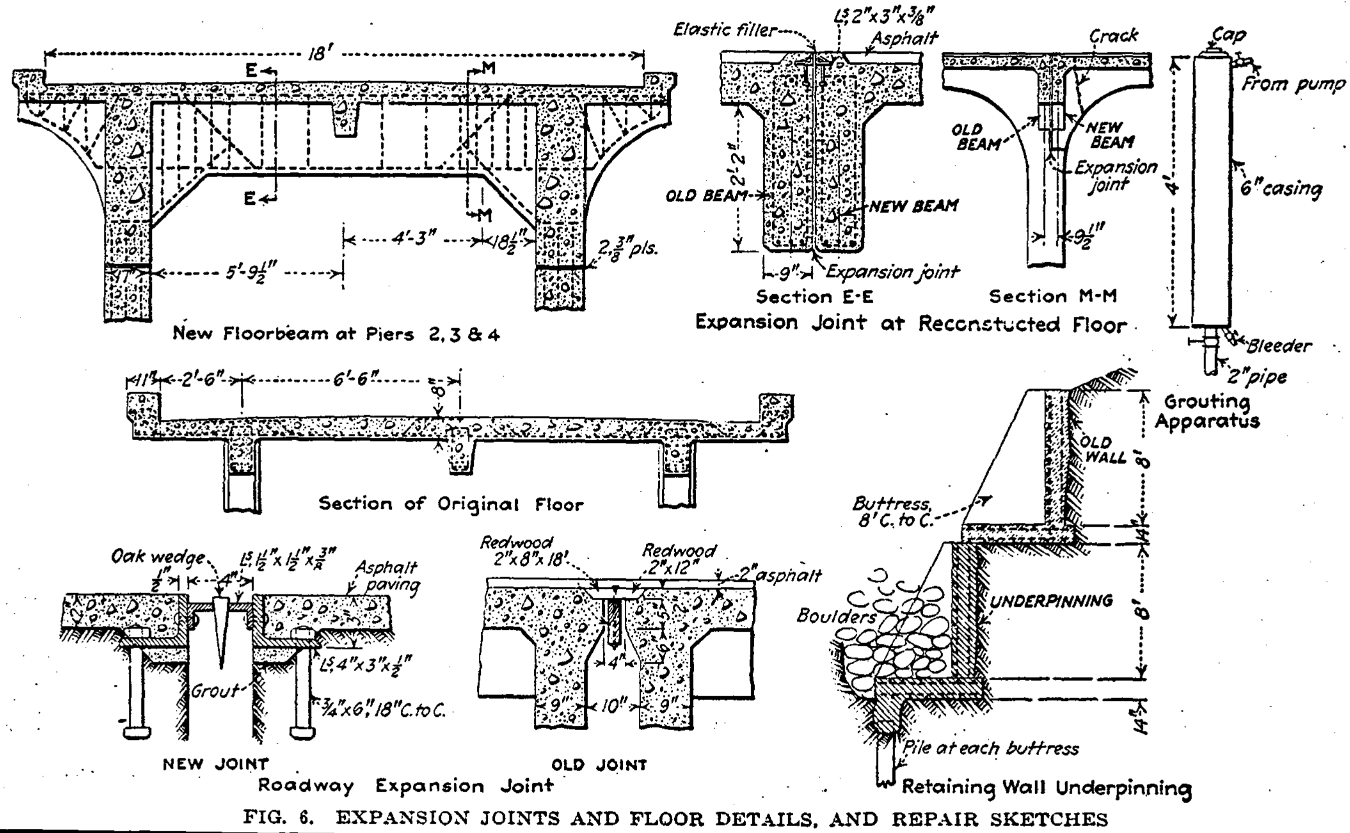


FIG. 6. EXPANSION JOINTS AND FLOOR DETAILS, AND REPAIR SKETCHES

fore the design shown in Fig. 6 was decided upon for the reconstruction. The $1\frac{1}{2}$ x $1\frac{1}{2}$ x $\frac{3}{8}$-in. angles are for the purpose of reducing the width of the opening, and are dropped below the general surface of the pavement to prevent their taking much weight from traffic.

One of these new joints has broken out from the effects of stresses to which it was subjected, but has been replaced. Oak wedges have been driven as shown, and these seem to have been of material help, as no other trouble has been experienced at the joints, which have been in service now a little over six months.

Underpinning Retaining Wall

The retaining wall adjoining the east abutment was a reinforced-concrete buttress wall founded on gravel. Floods had scoured out the gravel adjacent to this wall until the edge of the footing was exposed and slightly undercut on a length of about 30 ft. It was considered necessary to underpin the wall for a 50-ft. length.

This work was carried out in sections, the gravel bank being utilized as back form. Under the toe of each buttress there is a pile, for added security. The nature of the construction is clearly shown in Fig. 6.

No attempt has been made to repair the broken spandrel columns, as their condition is not considered to involve any direct danger.

The entire work described was carried on under the direction of Thomas Maddock, state engineer, F. N. Holmquist, assistant state engineer, the writer as bridge engineer, J. H. Zeitler as construction engineer during the foundation work, and J. M. Brown in the same capacity during falsework construction and floor repair. All work was done by state forces, and most of the equipment used was state owned.

Good Scholarship and Engineering Eminence

A close correspondence between good scholarship in college and eminence in engineering is shown in an investigation made under the auspices of the American Association of Collegiate Registrars by Prof. Raymond Walters of Lehigh University, who presents a report in the current issue of *School and Society*. It was found that of 392 distinguished engineers graduated at 75 technical schools, colleges and universities 182, or 46.4 per cent, stood in the highest fifth of their classes scholastically upon graduation, 109, or 27.8 per cent, stood in the second highest fifth, 72, or 18.3 per cent, in the middle fifth, 14, or 3.6 per cent, in the next to lowest fifth, and 15, or 3.8 per cent, in the lowest fifth. Figures for a group of 189 alumni of five Eastern engineering schools were somewhat different in the upper classes, the second highest scholastic fifth having the largest percentage. In all groupings of the eminent engineers there were less than 4 per cent in each of the two lowest scholastic fifths. Of 730 names on the Registrars' Association list of distinguished engineers practically 80 per cent were found to be collegiate graduates, 16 per cent men of secondary school education and practical training, and less than 5 per cent men who started in college but did not finish. The arbitrary basis of eminence in this study of a professional group was taken to be the holding of office, membership in important committees and service as representatives of the four founder engineering societies, civil, mechanical, electrical, and mining and metallurgy, for five years, 1915-1919.

Allocation of Nile Waters Between Egypt and the Sudan

Minority Report by H. T. Cory, American Member of the Nile Project Commission, Gives Principles of Water and Cost Division

INDORSEMENT of the immense project for the full utilization and a larger measure of flood control of the Nile advocated by the Egyptian Ministry of Public Works, but with divided opinion as to the allocation of water and cost between Egypt and the Sudan are the outstanding features of the report of the Nile Project Commission (see review, p. 689 of this issue), created early in 1920, whose report was dated in August of that year but not made public until 1921.

Unusual interest for American engineers attaches to this report because one of its three members was H. T. Cory, of San Francisco, Cal. This interest is heightened by the fact that Mr. Cory felt compelled to make a minority report as regards allocation of water and cost between Egypt and the Sudan, in which report he embodied his conception of the fundamental principles that should govern the allocation of water between two countries having a common interest in the same stream, as well as the basic principle for joint action in developing the water supply and distributing the cost of the work between the two countries.

Briefly, the two majority members of the commission—F. St. John Gebbie, inspector general of irrigation in India and head of all the irrigation service in that country, and Dr. G. C. Simpson, since last September head of the Weather Service of Great Britain and before that chief of the Weather Bureau of India—attempted no definite and comprehensive answer as to the allocation of water and cost, but merely proposed that Egypt and the Sudan should each go on with water storage developments already under way—Egypt with the Gebel Aulia reservoir on the White Nile to be formed by a dam just above the junction of the White with the Blue Nile, and the Sudan with the Sennar or Makwar reservoir, being formed by a dam at Sennar on the Blue Nile some distance above Khartum—each country to pay the entire cost of its reservoir and to have the use of all the water stored by it, with relatively minor exceptions and adjustments based upon local conditions.

As to Mr. Cory's proposal for allocation of cost and water the majority members of the commission say:

"With regard to Mr. Cory's report, we feel that while his findings may be theoretically correct it is impossible, on financial and other points, to apply them in present circumstances."

It should be noted that the complete project contemplates a large amount of additional storage and also the construction of a long channel through the Sudd in order that water proposed to be stored in Lake Albert may not be lost in its passage to the Nile proper through what is now a vast overflowed area. It should also be noted that data for the design of these additional works are still lacking. The majority of the commission apparently bases its failure to attempt to allocate water and cost from the ultimate development to a lack of data and to other difficulties. This lack of engineering data and all the other unenumerated difficulties Mr. Cory appears to think do not stand in the way of an allocation of water and cost on broad general principles and to a certain extent upon experience. Mr. Cory also sets forth

Days of Tempe Bridge are Numbered

By RALPH HOFFMAN, Bridge Engineer

A QUESTION of great importance to many persons of the Salt River valley is the ultimate life of the Tempe bridge. We are quite certain that its days are numbered. The life of the structure has been variously estimated and almost from the time of its inception the design has been of sufficient importance to call forth articles by some of the most noted consulting bridge engineers, as evidenced by articles appearing in Engineering News and Engineering and Contracting about the time of the construction, 1911 to 1913.

It was thought that predictions were realized when Pier No. 9 settled 4½ inches during the floods of November, 1919. Although considerable alarm was felt at the time, the bridge was not closed to traffic except during the peak of the floods. The excitement had almost subsided when in February and March of the next year further settlement occurred at the same pier, making a total sag of about one foot in the roadway.

The bridge was closed to traffic and repairs made as described in the Engineering News of April 12, 1921. These consisted of reinforcing the weak pier by sinking additional cylinders on each side and casting a reinforced cap in sections extending through the old pier The expansion joints were also repaired at this time but the type of joints placed were not satisfactory and their failure resulted in excessive impact from trucks due to the depressions.

CONSIDERABLE STRAIN SEEN

The settlement of the pier mentioned subjected the superstructure to considerable strain and the deck took remarkable deflections without showing fractures, but these have been gradually developing under the impact vibrations set up by the passage of heavy traffic. New developments could be seen at each inspection and these were made at frequent intervals. It was thought that the immediate danger lay in a gradual destruction from vibrations, resulting from the impact at the faulty expansion joints and the recent repairs to these have sustained that belief.

These vibrations were transmitted the full length of the bridge so that the effect of one truck passing over each of the thirteen joints was a succession of violent shocks. The traffic count for this highway was in the neighborhood of 3500 to 4000 per day, and hence some idea may be had of the destructive action of such forces.

There is some doubt as to the cause for this transmission of shock from end to end of the structure. It is quite possible that the effect might be felt in the spans immediately adjacent to the loaded span, on account of the absence of footings that would ordinarily be expected to compensate an unbalanced thrust at the pier. Some degree of flexibility must necessarily be present in the pier but it could hardly be conceived that the span with a hinge at the pier would act as an elastic unit. It is thought however, that due to the placing of heavy longitudianal reinforcement in the deck slab—a last minute revision in construction—a cantilever action is obtained at each pier. This additional steel was placed continuously over the pier, extending from crown to crown and was sufficient to hold the slab under excessive deflection without apparent fracture.

The deflection of the slab takes a form of a smooth reverse curve, such as would be expected in a series of continuous girders with one span loaded, but on an exaggerated scale. Thus we might picture a series of see-saws end to end, and each linked to the other. Strike one joint of this series and the shock would be transmitted in a wave motion throughout the entire length of the series. Some such action undoubtedly takes place in the transmission of the impacts on the bridge as it is quite apparent that there is a periodic wave which is transferred through the crown hinges.

HOLIDAYS POSTPONE WORK

Plans were prepared for the replacement of the joints before the holiday season of 1924, but owing to the Christmas shopping period, the actual work was postponed. The greatest problem that confronted the bridge engineer was to place a joint that could be securely anchored to the thin concrete slab, without removing much of the old concrete to make a smooth connection. A joint composed of two heavy angles and a plate one-half inch in thickness and eight inches wide was selected. The plate was securely riveted to one angle and the angles provided with anchor bolts at four foot centers on both legs.

The problem of backing these angles up with a thin section of concrete that would stay, was still with us until it was determined that the State had many uses for a cement gun other than making repairs to the columns and beams of the Tempe bridge, and that valuable piece of equipment was purchased.

The cement gun was used for placing the joints as well as for the column repairs. A strip of concrete and asphalt surfacing twelve inches wide and six inches in depth was removed on either side of the joint by means of jack-hammers, the angles were then blocked in position and the anchors sulphured into holes previously drilled with the air hammers. The air compressor for operating the cement gun was also used for operating the jack-hammers. The angles were backed up with a slab of gunite, which gave a maximum density of concrete and bond to the old surface, and being placed under pressure, was securely packed under the horizontal leg of the angles.

QUICK-SETTING CEMENT USED

One outstanding feature was the use of Lumnite Cement for a majority of the concrete work. This was probably the first practical use of this quick-setting cement in the state. It was estimated that the use of the bridge was worth approximately $1,000 a day to the public and the use of the Lumnite Cement, giving twenty-eight day strength in twenty-four hours was a considerable advantage, shortening the period of closing by at least two weeks. All operations were carried out with the idea of shortening the closing period. The old joints were removed under traffic, only the heavy trucks and busses being detoured by way of the South Center street bridge and the Lower Tempe road.

All of the thirteen crown joints were replaced with the new type. Several spandrel columns were entirely rebuilt with wire mesh and gunite and slight repairs made on others. Seven new steel cross-beams were placed at the crown sections of the two spans adjacent to Pier No. 9. These were also encased with gunite. The work was carried out with a crew of State forces under the direction of District Engineer, George B. Shaffer, and was completed on March 1, 1925, with only about two weeks interruption to traffic.

The new joints have reduced the vibration to a minimum and many engin-

(Continued on page 18)

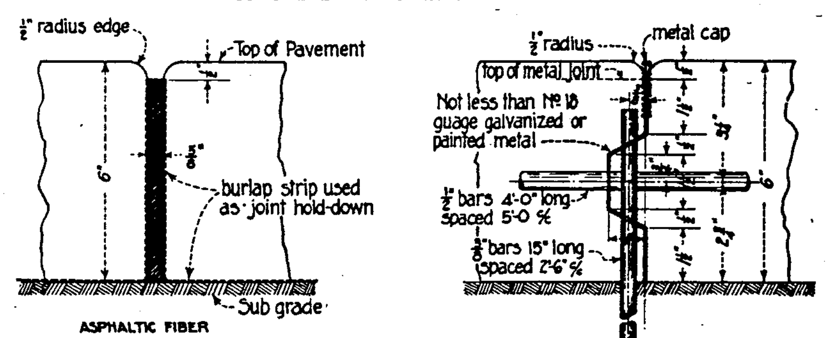
tightly against the sides. This avoids any slack forming in the burlap when the concrete is deposited about the joint. The finishing machine is allowed to pass over the joint one or more times before the header board is removed. The header board should be removed very slowly with one end slightly above the other and care being taken that the space under the header board is replaced by spading thoroughly while the removal is taking place.

CONCRETE ADDED

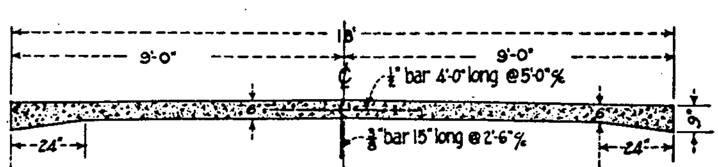
Concrete is now added to the extent of a surplus and the finishing machine is passed over as many times as are necessary to cut the surface to crown and grade. The entire surface has now been given the same tamping, rodding and belting process and is ready to receive whatever hand finishing that is necessary to even the surface. The submerged joint allows a free and continuous passage of the longitudinal float. The use of the longitudinal float is apparently the best assurance of an easy riding surface and its merits are most emphasized by its free and uninterrupted passage properly lapped over the entire surface of the run. The half inch of mortar which remains above the top of the joint is now removed and the slabs are edged on both sides of the joint.

This type of joint seems satisfactory. No defective joints have appeared where placed in this manner and for additional information cores were taken to show cross section of joint. These cross-sections showed 100 per cent slab and 100 per cent joint in its proper position. The cores showed that the burlap did not allow concrete to pass under nor between the ends of the sections of the joint material, a very harmful condition which so often exists at expansion joints.

ARIZONA HIGHWAY DEPARTMENT
STANDARD EXPANSION JOINTS
FOR
CONCRETE PAVEMENT



TRANSVERSE JOINT LONGITUDINAL JOINT



CROSS SECTION OF
CONCRETE PAVEMENT
THE (MARICOPA) ARIZONA SECTION

DAYS OF TEMPE BRIDGE ARE NUMBERED

(Continued from page 16)

eers declared that the bridge is in better condition in this respect than ever before. But we have only checked the onslaught of destruction and disintegration. There is a tremendous increase of traffic on that highway each year. It is a common point to both the main East and West highway and the North and South highway through the state, and the bridge, even though it could be made as good as new, would be greatly overloaded by the present traffic. It is my belief that it will be only a matter of a short time until the destruction, temporarily allayed, will continue.

No idea of the value of this link can be realized until it is closed to the public. At the time of closing for repairs in 1920, the traffic census was 2500 per day, while the count now runs close to the 4000 mark, and at the present rate of increase in the motor vehicle registration, this mark will soon be passed. The life of the structure could undoubtedly be prolonged if the public would take a lesson from the previous closings and observe slow and careful driving. This should be especially emphasized to those who operate heavy trucks and busses. It is not particularly the load but the impact of that load moving at high speed which causes the destructive forces.

CONTROL LOADS AND SPEED

Care should be taken by authorities to prohibit loads in excess of ten tons, total load including the truck, and above all to hold down the speed of all traffic. These rules might work a temporary hardship on some, but it would be nothing in comparison to the complete loss of the structure, which might occur in the very near future.

It is believed that the State should be prepared to construct a new bridge at or near this location within the next four or five years. Even though the present bridge withstands the onslaughts for that long or longer, its roadway will not be sufficient to care for the traffic. It is scarcely wide enough at present, and the next bridge should carry a roadway of not less than 24 feet. It may not be necessary to junk the present bridge, but merely limit it to light traffic which would relieve congestion at other points.

UNBEARABLE.

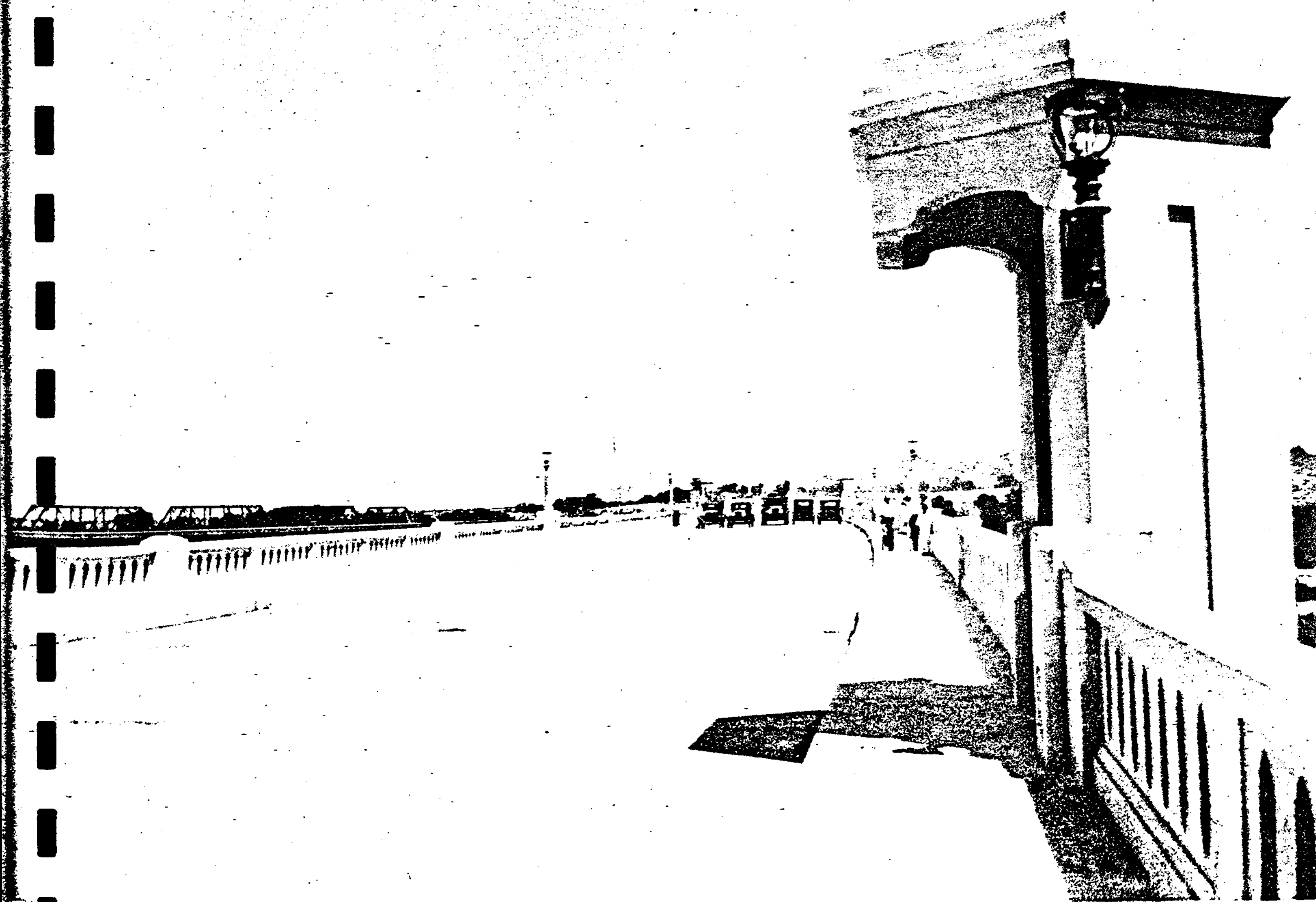
The applicant for cook was untidy and insolent in appearance.

"Don't hire her," whispered Jones to his wife. "I don't like her looks."

"But," remonstrated his wife, "just consider the reputation for cooking she bears."

"That doesn't matter," said Jones testily. "We don't want any she bears cooked. We don't like them."—Vancouver Province.

Arizona Highways



Five Cars Passing Abreast on New Tempe Bridge

Volume 7
Number 6

June

Copy Ten Cents
Yearly One Dollar

ARIZONA HIGHWAYS
CIVILIZATION FOLLOWS THE IMPROVED HIGHWAY

Volume VII. June, 1931. Number 6

Tempe Bridge Soon To Be Ready For Traffic

By RALPH HOFFMAN, Bridge Engineer.

The completion of the new Tempe Bridge, Arizona's largest and most magnificent causeway, adds another triumph of engineering skill and closes another chapter in the history of Arizona highway construction.

Many readers will recall the dedication of that spectacular structure, the Grand Canyon Bridge, and now just two years later plans are being carried forward for the dedication on July 4th, 1931, of the Tempe Bridge.

This new structure, although not so spectacular as the former, is the largest bridge ever built in the state of Arizona, both in length and width of roadway. The total length is 1577 feet; the width of the roadway is thirty-six feet between curbs and provides room for four lanes of traffic. In addition a five-foot sidewalk is provided on each side, maknig a total width, inside the concrete handrails, of forty-six feet.

Comparing the above dimensions with those of the old bridge,—an 18-foot roadway and no sidewalks,—those who have driven over it in periods of heavy traffic will realize the easy comfort of driving on the new structure.

The old bridge, designed for the traffic of 20 years ago, has been replaced with a modern structure in which the engineers have attempted to visualize the future needs of this highway.

Within City Limits

The bridge is located at the south end of Mill Avenue within the city limits of Tempe, and carries the traffic of three main U. S. Highways, namely: U. S. Route 89, the only north and south highway through Arizona; U. S. Route 80, a transcontinental highway, and U. S. Route 60, the new transcontinental route recently established through Arizona. Thus it will be seen that, with the completion of Route 60, a large percentage of the tourist traffic must pass over this bridge in addition to the ever increasing local traffic.

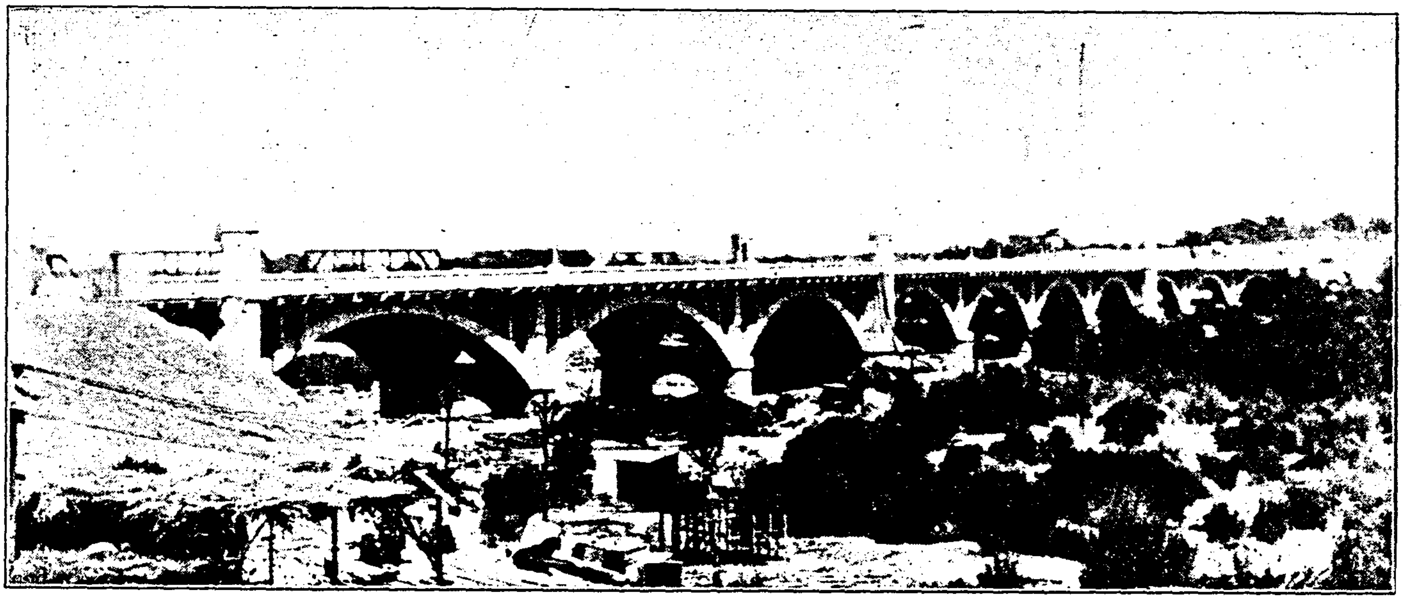
The recent traffic counts show a total of about 8,000 vehicles each 24 hours traversing this section of the highway; and this total has been increasing rapidly. If the old bridge carried this traffic it is safe to say that the new one will handle three or more times this total on account of the width of roadway and the increased speed made possiby by that width.

The extension of Mill Avenue was the only logical location, as it maintains the present line of travel through the main part of town and eliminates two right angle turns in the town of Tempe.

A survey was made first extending the center line of Mill Avenue straight across the river and re-entering the present highway with a long curve on the north side of the river, and a contract was let for foundation borings. The results of these borings were at first very discouraging. The data obtained showed shallow rock foundation for less than half the length of the bridge and the remaining portion a soft caliche. The rock apparently dipped abruptly and was not encountered at depths up to 75 feet beyond the center of the channel.

Side Found Unfavorable

The experience with the railroad bridge only 300 feet up stream, on which two steel spans were lost by the failure of a pier, was sufficient evidence that the caliche material was not adequate for foundations except at a depth which would preclude all possibilities of scour under the footings. This depth was considered to be 40 to 45 feet below low



View of new Tempe Bridge looking down stream, showing arch construction. Each arch has a span of 140 feet.

water elevat'on, which meant only one tyre of d-si-n—long steel spans.

In addition to the deep foundations this site required extensive bank protection and a long, high fill at the north end of the bridge and the loss of consi'erable length of the existing paved highway on that side.

The profile plotted from the test borings d'd. howeve-, show a high point in the rock formation toward the center of the channel. In studying this profile on the ground it was discovered that the high point lined with an outcrop of rock on the north bank under the old bridge and a ridge extending out from the Tempe Butte.

This discovery indicated the possibility of a rock ridge or dyke extending across the river diagonally across our line. The indication of the existence of such a formation was so strong that our own drilling force was moved on the job to prove our theory.

Located Diagonal Ridge

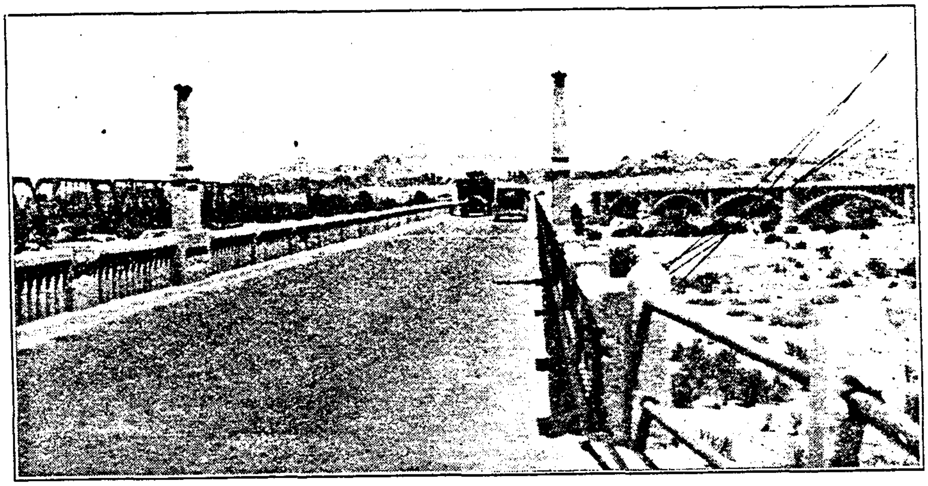
An extensive drilling program was laid out and the ridge located as expected. Contours of this formation under the bed of the river were plotted and a paper location for the new center line laid out.

This location, by spanning a small underground channel in the rock near the north bank, made possible a fairly shallow rock foundation for the entire length of the bridge and also made possible the adoption of the concrete arch design. The line extends from a point on Mill Avenue on the south bank diagonally across the river to an intersection with the present highway at the north end of the old bridge. An easy curve (one degree) extending onto the bridge from each end was not difficult to take care of in the design.

The estimated saving of this line over the original was more than $100,000 on foundations and roadway.

For economy the design was practically limited to the deck type structure on account of the width of roadway to be provided. It was also desirable to keep the roadway on as low a level as possible, which limited the span length on account of available head room. For these reasons only two types were considered: the concrete arch and the steel plate girders.

The limit of the span length for the concrete arches was about 140 feet and for the steel girders about 100 feet; the problem resolved itself into the comparison of relative merits of two designs on this basis.



The old Tempe Bridge, which the new structure, to be seen at the right, will soon supplant. It was a close squeeze for a truck and auto to pass on this old bridge. On the new bridge 5 cars can pass, as shown on cover scene.

Prevailing steel prices at that time and the additional piers required for the steel design resulted in a slightly lower cost for the concrete arch type. The concrete structure was to be preferred on account of the inherent architectural effects to be secured without additional cost and probably would have been the accepted design even at a slightly higher cost.

Ten Spans in Bridge

Final plans were worked up for the arch bridge consisting of ten spans, 140 feet each. The spans were of the two r b open spandrel type, with the concrete roadway supported on beam and webbed columns above the two ribs.

Each rib is two feet nine inches in thickness by nine feet wide at the crown, seven feet thick in the vertical plane at the piers.

The reinforcement consists of 1¼ inch square bars at 12 inch centers in the top and bottom of the rib throughout its length, except that this steel is doubled in the extractors (top) at each end for a distance of about 30 feet out from the pier.

The ribs were designed as hingeless arches fixed at the piers and the stresses analyzed by the elastic theory involving long, tedious calculations and a mass of figures which have no place in this article.

Two types of piers were used in the des'gn. It was considered advisable to provide at least two abutment piers for convenience and safety in construction. With this in view the spans were divided into three groups of three, four and three spans each and the groups separated by abutment piers. The piers are of the same general design below the top of the arch except in size, the abutment pier being 15 feet in girth at the spring line of the arch while the intermediate piers are only 7½ feet. These are constructed with two separate shafts on separate footings and the shafts are tied together with an arched tie strut., built intergral with the pier cap at the junction with the arch rings.

Above the arch the intermediate piers carry a typical column construction, while the abutment piers are surmounted by a sand box extending the entire length of the piers, to give additional weight. The ends of these boxes are carried up above the roadway in a hexagonal tower effect, terminating in a canopy over a retreat bay in the pier end. These piers are capable of resisting the full deal load thrust of the arches from one side only.

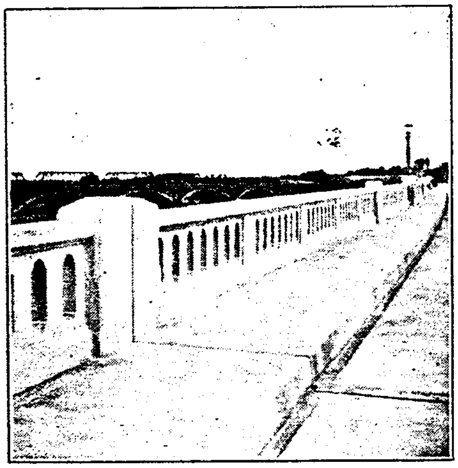
On account of the height of 32 feet from the spring line of the arch to foundation some degree of flexibility was anticipated in the intermediate piers. The movement of the pier top under live load on the bridge was calculated and its effect on the stresses in the arch ring analyzed. The result required an increase in the size of reinforcement in the rib.

Open Type Abutments

The abutments are of the open type with the earth fill spilling around the end columns. The same effect as at the abutment piers has been maintained with a tower on each side of the roadway with the addition of a hexagonal pylon at the ends of the railing.

The roadway slab is reinforced as a continuous slab between expansion joints with bent steel providing for negative movement over the supporting beams. Four expansion joints are provided to each span at about the third points of the span and at each pier. A feature of the design is the elimination of all sliding joints by supporting all ends on separate columns.

The handrail details were worked out after a careful study of those built in other cities and a design arrived at



The hand rail and sidewalk on the new bridge adds beauty to the bridge, and safety for pedestrians.

which is sturdy as well as distinctive and in keeping with the rest of the structure.

The lighting fixtures and poles were selected to harmonize with the rest of the handrail structure. Mounted upon handrail block over each intermediate pier is a spun concrete pole surmounted by a standard street lighting unit. At the towers these units are supported on heavy bronze brackets mounted on each side of the tower.

In all there are thirty-four of these units on the bridge. The bracket lights on the towers are specified to be arranged in a circuit to burn all night and the rest to be controlled by an automatic time clock, so that they will burn only during the early hours of the night. In this manner ample lighting will be assured at all times.

The lighting will not be maintained by the state but is placed there for the use of the city of Tempe, as the bridge is within the city limits.

The sections of the members throughout the bridge were designed to a minimum required for the stress and practically no concrete was added for mass effect or architectural treatment except in the work above the deck,—handrails and towers.

The contract was awarded January 22, 1930, to the Lynch-Cannon Engineering Co. of Los Anglees, the low bidder on the job. This company began work under their contract in March, 1930.

Anchored In Solid Rock

The first work was that of excavating for the piers and abutments. Cofferdams of heavy steel sheet piling were driven to rock and the sand and gravel taken out with a crane. The design required the concrete footings to be anchored three feet into the solid rock, which required blasting the rock out to the footing lines.

While the excavation of the first hole was in progress a central mixing plant was erected adjacent to a commercial gravel plant ,which was to furnish the sand and gravel for the entire job. Belt conveyors transported the material from the plant to large storage bins above the mixer. From these bins the sand and gravel were weighed in a batcher and dumped into the mixer. The cement was supplied from an adjacent storage shed by means of a skip which dumped directly into the hopper.

From the mixer the concrete was hauled on an industrial railroad to the job. Here the batch boxes were lifted from the cars and dumped into the forms. For the footings and piers the gasoline operated crawler crane was used in depositing the concrete.

In concreting the arch rings and deck it was necessary to have a machine which could reach the entire height and width of the structure so that concrete could be deposited at any desired point with a minimum of moving. For this purpose a traveling gantry crane was constructed, consisting of a four heavy post frame mounted on four flanged wheels. Supported on top of this frame at deck level was a boom derrick, operated by a gasoline hoisting engine. Two steel rails laid parallel to the bridge permitted this crane to be shifted to any span in a short space of time.

Each rib was poured in five main sections and four keys, each five feet six inches long, which were omitted until the other sections of concrete had taken shrinkage. Two of these keys were near the crown and two at the haunches and were placed at the lap in the reinforcing steel. The purpose of the keys was to eliminate as much of the initial stress in the ring as possible.

Fooled by Big Boulder

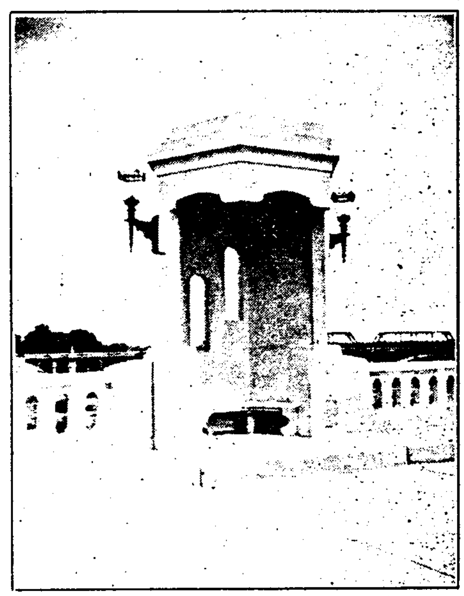
The sections of rib were poured symmetrically about the center of the span to balance the load and the timber false work and prevent distortion.

The two abutments were completed first, as specified, in order to allow a contract to be let for approach fills. Then the piers were completed in order, beginning at the Tempe end.

When pier number 9 was reached in the process of construction, rock foundation was found at plans elevation on the upstream side and extending over about one-half the area of the pier. Original tests for rock made at this pier showed rock at the same elevation at both ends, but developments showed that the drill had struck a large boulder at one end and the crew, confident in the discovery of bed rock, had moved on without the usual check.

Steel rails were at once driven on the perimiter of the pier and a profile of the solid rock plotted to determine its actual location and slope. On the low side the rock was found at an elevation about 30 feet below the high side.

The construction here called for special treatment. The work on this pier was the most difficult encountered on the entire job and required very careful preparations. In order not to delay the rest of the construction the work on this hole was carried on in three eight-



One of the eight rest stations on the new bridge.

hour shifts until the pier was finished. More than 3,000 cubic yards of material were excavated from the hole, of which about 25 per cent was solid rock.

The last concrete was poured in the deck during May, 1931, and the last concrete in the bridge, a small dado in the handrail, was poured on Wednesday, June 3, 1931—just fifteen months after starting the work.

Throughout the entire job is reflected

(Continued on page 23)

construction in Oak Creek canyon. Sedonia to Flagstaff slow in wet weather.

STATE ROUTE 74, WICKENBURG TO EHRENBERG—74 miles. Surface, low type improved. Condition good, Wickenburg to Aguilla and Quartzsite to Ehrenberg, balance fair.

STATE ROUTE 81, DOUGLAS TO SAFFORD—128 miles. Gravel surfaced. Condition good.

STATE ROUTE 187, SACATON DAM TO CASA GRANDE—13 miles. Gravel surfaced. Condition good.

STATE ROUTE 83, VAIL TO SONOITA—28 miles. Gravel surfaced. Good.

STATE ROUTE 82, NOGALES TO TOMBSTONE JCT. 70 miles. Gravel surfaced. Good.

STATE ROUTE 84, TUCSON TO GILA BEND—124 miles. Gravel surfaced. Condition good excepting Tucson to Rillito being oil surfaced, 1 mile detour near Rillito; ten mile detour between Rillitio and Red Rock. Detour fair. Observe caution in driving.

STATE ROUTE 87, MESA TO PICACHO—60 miles. Paved oiled or gravel surfaced. Condition good.

NEW TEMPE BRIDGE SOON TO BE READY FOR TRAFFIC

(Continued from page 7)

a careful and excellent workmanship which is a credit to the engnieers and contractors. A full measure of credit should be given to the engineering force under the direction of A. F. Rath and to the general foreman, E. C. Moore and his men for the pride they have taken in a piece of work well done.

The Lynch-Cannon firm was represented by F. L. Holser, general manager of the company, who by frequent contacts with the work made it his duty to promote harmony and give the state a finished product in which there could be no fault.

ARIZONA IS NATION'S LARGEST VACATIONLAND

(Continued from page 8)

and vine shaded streams. And after a blissful day, home to dinner, where even the most inexpert, with never a trout to his credit, shared bountifully in the day's catch.

These are but a few of the vacation spots in Arizona. Flagstaff has many points of interest and wonders. No one should fail to see Sunset Peak, weird and unique. A cone-shaped mountain of gray volcanic cinder until within a short distance of the tip of the cone, where the red cinder begins, giving the peak the appearance of being heated red-hot. Meteor Mountain, caused by the impact of some wanderer of the skies, who collided with Mother Earth by some miscalculation in his schedule.

And then there are the many beautiful lakes, Mary's, Stoneman's Lake, Mormon Lake, all offering entertainment, boating and fishing and both camping and lodging facilities. Not to speak of such sights as the Petrified Forest and the Painted Desert, to be seen only in our own state, time and space forbidding a fuller description but which any Arizonan should feel shame at having failed to see.

Inmates gave bridge crew exciting time

FA-3[illegible] 14-3

Tempe Daily News Dec. 1, 1980

By Bob Kelly
Daily News writer

The fate of the old Tempe Bridge — a product of early 20th Century prison labor — is in limbo. But its history, like its skeleton, remains cast in concrete.

The Tempe City Council last week rejected a proposed $25,000 feasibility study on the structure, also called the Ash Avenue Bridge, which crosses the Salt River bed just west of the Mill Avenue Bridge.

The study would have determined, among other things, the practicality of repairing the span to accommodate bicycle and pedestrian traffic.

After examining Arizona Department of Transportation reports, some council members felt the project would be too expensive.

So the old Tempe Bridge — now a shell of its original form — probably will remain as it is until a private developer offers to renovate it.

But no matter what end is in store for the bridge, its beginning makes for colorful conversation.

Construction began in March 1911. Crews consisted almost entirely of state prison inmates who were housed in a stockade near the site.

The transportation department reports say the labor force averaged about 57 men and at times ranged upward to 70.

Although he didn't initiate the idea of using convict labor, Arizona's first governor, George Hunt, supported the concept.

Tom Davis pointed out in an Arizona State University term paper that Hunt, who was elected chief executive in 1912 when Arizona became a state, was a firm believer in prison reform.

Davis said Hunt once defended the honor system at a Chamber of Commerce meeting in Prescott.

"He said he was so confident in the men that he would resign if one should escape," Davis wrote. "This was a promise he was not to keep.".

Perhaps one could defend Hunt's decision. Not one prisoner escaped.

Fifteen did.

These runs for freedom were far from mundane. Davis reported one prisoner who made a break on June 21, 1912 was soon caught, but not before he had accumulated a little prosperity.

"A short time after he was noticed absent, he was found by fellow convicts on a search party walking along the Salt," Davis wrote.

"In place of his prison clothes he was wearing a fine tailored suit. He had a gold locket with a diamond stud hanging from his breast pocket and a silver dollar in his trouser pocket.

"As with all captured escapees, he was returned to Florence to serve out his complete term under less-desirable circumstances."

Please turn to page A5, Bridge

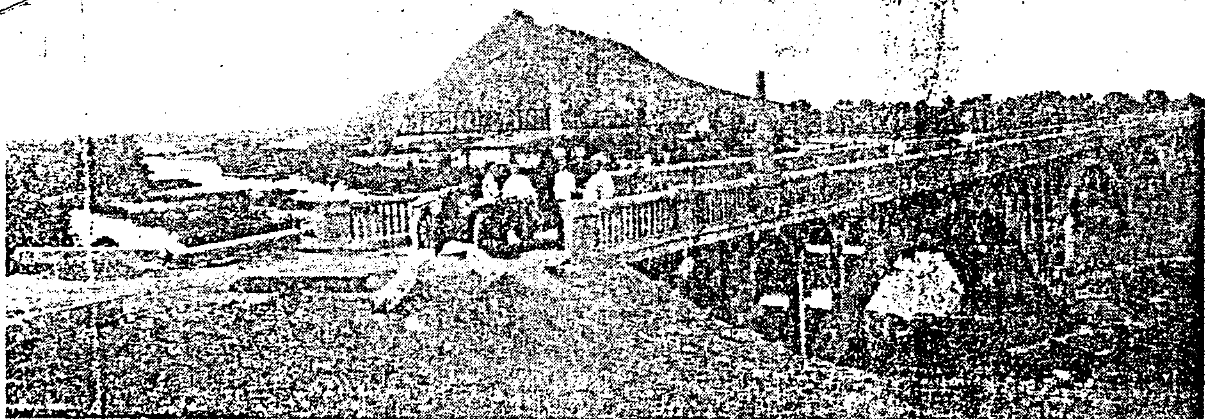


Photo courtesy of the Arizona Collection at Arizona State University.

Daily News Photo by Kay Avery.

The Ash Avenue Bridge has undergone a transformation in the past 67 years. The bridge, shown above in August 1913 about a month before its opening, was abandoned by the Arizona Department of Transportation in 1933. The deteriorating structure, shown below as it now looks, now is closed to all traffic and has been criticized as a hazard and eyesore.

FA-III 14-3

Tempe Daily News Dec. 1, 1980 P. A5.

Daily News photo by Ray Avery

A commercial jetliner flies over Hayden Butte on its way out of Phoenix Sky Harbor International Airport. Residents of North Tempe have become increasingly concerned by aircraft they claim have deviated from established take-off and landing patterns.

FA-34.

Bridge

Sunday baseball games were adopted to keep the labor force from becoming victims of the old ennui.

Davis reported Tempe residents, apparently impressed with the prisoners' athletic prowess, attended the games and even passed the hat to buy the team better equipment.

But this also got out of hand.

"On one Sunday early in January 1912, more than the hat was passed," Davis wrote. "The local citizenry apparently was quite chummy with the bridge squad and one local barber was found passing a bottle of whiskey to one of the black convicts.

"The barber was fined 15 bucks and the convict was returned to Florence."

Despite the mayhem, the project survived and the Ash Avenue Bridge was finished in September 1913. But it was doomed to a relatively brief life span.

The structure eventually fell victim to time and wear. Its road was two narrow to handle increasing numbers of cars and its foundation wasn't strong enough to support heavy vehicles.

One engineer said in the May 1925 *Arizona Highways* that weighty vehicles caused the entire span to vibrate "so that the effect of one truck passing over each of the 13 joints was a succession of violent shocks."

The structure was abandoned in 1933, shortly after the Mill Avenue Bridge was opened.

"The cost of continued repair was getting to be excessive," said Tempe Councilman William Ream.

But he added the bridge still could be put to practical use.

"At the time they abandoned it it was limited to about 10 or 15 tons," Ream said. "You still could put bicycle and pedestrian traffic on it without too big a sweat."

However, the question of who owns the old bridge remains unanswered. When state officials abandoned it, they did so without transferring the title.

Now, no one knows where the title is.

"I think the city of Tempe is probably the owner," Ream said. "In any case, it appears the abutments on both sides belong to the city. So whether we want it or not, it belongs to the city of Tempe.

"Sometimes it's hard to turn down a gift you don't want. This is one of those cases."

Collins

encourage increased resources to combat prostitution.

"That is a real problem," he said. the prosecutors are running a probation system."

He said he would rather spend

the past several weeks have meeting

Heisman

he apparently was to spend the night.

Rogers carried four of the six regions into which the country is divided — the Mid-Atlantic, South, Southwest and Far West. Green captured the Northeast, while Purdue quarterback Mark Herrmann, who finished fourth overall with 405 points, won the Midwest.

Rounding out the top 10 Heisman finishers were: Jim McMahon, Brigham Young quarterback; Art Schlichter, Ohio State quarterback; Neil Lomax, a quarterback from small college Portland State University and college football's all-time passing lea Jarvis Redw Nebraska run back; Kenny Eas UCLA safety; and three-way tie for among Anthony Car Michigan receiver; M Singletary, Ba linebacker, and D Wilson, Illinois quar back.

Rogers became eighth consecutive r ning back to win Heisman and the 31s the 46 years of award. Green was the third lineman finish as high as sec without doubling as pass-catching end. others were Alex K ras of Iowa in 1957 John Hicks of Ohio St in 1973.

REPORT

OF THE

STATE ENGINEER

OF THE

STATE OF ARIZONA

JULY 1, 1909 to JUNE 30, 1914

Published by Board of Control, by authority of Chapter 53, Session Laws of the Second Special Session of the First State Legislature

The Arizona State Press

#1

Clifton-San Carlos Highway:

Preliminary survey between Clifton and San Carlos, a distance of 114 miles, completed.

Engineering .... $ 818.43

MARICOPA COUNTY

Prescott-Phoenix Highway, Grand Avenue Division:

This section consists of that part of Grand Avenue extending from the city limits of Phoenix to Glendale. Portions of this road were surfaced with caliche, gravel and disintegrated granite at a cost of $2.130.57.

There was expended on this section of highway $4,612.32 for maintenance work to June 30, 1912, two teams and three men being employed for the joint maintenance of this section and the Phoenix-Tempe Highway.

Phoenix-Tempe Highway:

There was expended to June 30, 1912, on this section of highway $6,920.64. This expenditure covering surfacing with caliche in some places and general maintenance work.

Phoenix-Yuma Highway:

Reconnaissance and some preliminary survey work made on the highway extending from the City of Phoenix to the Town of Yuma in Yuma County, a distance of 202 miles.

Proportion Engineering costs.................................$ 650.26

Tempe Bridge:

Plans and specifications prepared for the construction of a bridge over the Salt River at Tempe and construction under way with prison labor June, 1911.

#1

Expended to June 30, 1912..................................$56,023.83

For detail, see Prison Labor section of this report.

A survey of the Fairbanks bridge site with soundings for foundations was made.

The highway was located 9 miles out of Bisbee.

Plans and estimates for this work are now in course of preparation in this office.

A caretaker with team was employed to keep the Douglas-Bisbee highway in repair.

Maricopa County.

County portion State Road Fund	$36,749.91
State portion State Road Fund	12,249.97
	$48,999.88

#1

The law provided that Maricopa County should have all of the State Road Fund contributed by said county for the purpose of completing the Tempe bridge. As this fund was not available until after January 1, 1913, arrangements were made to borrow sufficient to meet the payrolls for labor on the bridge and the merchants and others agreed to wait for settlement until the above date.

To continue the employment of prisoners on this work, arrangement was made by the Board of Supervisors and the State Engineer with the Board of Control whereby the prison would pay into the Road Fund for each prisoner so employed the net per capita cost of maintenance of prisoners at the prison. Similar arrangement was made for the employment of prisoners in other counties.

When the grading outfit of prison labor engaged on the Florence-Mesa highway reached the Maricopa line it was decided to have them grade from that point to Higley, a distance of seven miles, which was done.

Minor repairs and some dragging was done on the Glendale road.

The County Road Superintendent was appointed as superintendent of state highways.

Clifton-Duncan Highway:

Survey made of seven miles of highway from the end of Clifton-Solomonville Highway, Sec. 1, toward Duncan.

Engineering:

Salaries	$ 440.92	
Travel and subsistence	343.92	
Team hire	66.00	
		$ 850.84

MARICOPA COUNTY

Board of Supervisors:
W. A. Moeur, Chairman.
Frank Luke, Member.
Lin B. Orme, Member.
Jas. Miller, Jr., Clerk.

Tempe Bridge:

#1

Finishing and paving work completed in September, 1913, and bridge opened for traffic September 20, 1913.

Expenditures 1913-1914	$ 7,574.82	
*Refunds	7,542.74	
		$ 32.0

(For detail see Prison Labor Section of this report.)

Phoenix-Tempe Highway:

Construction work on this highway with prison labor continued to completion, December, 1913. (For detail see Prison Labor section of this report.)

Expenditures 1913-1914	$43,543.18	
*Refunds	9,781.48	
		$33,761.7

#1

Reimbursement made to General Fund for loan of $30,000.0 in fiscal year 1912-1913. Expenditures during that year $24,440.0. Amount expended current year from this fund, $5,559.26, shown as regular expenditures against proper projects.

Lumber	5,506.98	
Reinforcing steel	7,940.87	
Hardware and miscellaneous	2,049.49	
		19,952.29
Fuel and Oil		524.62
Tools and Equipment—		
Charged to job	6,496.18	
Less recovered (est.)	4,210.34	
		2,286.84
Miscellaneous Expense—		
Camp supplies—Eng'r mess	1,012.87	
Camp equipment, etc.	48.66	
Office expense	69.62	
Miscellaneous	309.57	
		1,440.72
Total Cost		$ 45,417.36

#1

TEMPE BRIDGE

Including South Approach with San Francisco Canal Culvert

J. C. Ryan, Division Engineer.

Under instructions from Board of Control, the Territorial Engineer submitted plans for construction of bridge over Salt River at Tempe, February 24, 1911, and under date of May 31, 1911, the Superintendent of the Prison was instructed to send twenty-five prisoners and six guards to the camp established at the bridge site. The following resolution was adopted on this date by the Board of Control relative to use of prison labor on this work:

> "Whereas, certain prisoners now confined in the Territorial Prison at Florence can be advantageously employed by the Territory in the construction of the Territorial bridge across the Salt River near Tempe; and
>
> Whereas, it is deemed advisable for the Board of Control that men so employed be rewarded for faithful and efficient service on the construction of said bridge, and it is

the belief of the members of the Board of Control that by so rewarding prisoners so employed a greater standard of efficiency may be maintained and better results can be achieved on the work by giving the prisoners an incentive for good behavior:

Whereas, be it resolved that for every day of faithful and conscientious labor performed by a territorial prisoner in the construction of the said bridge, a credit of two days shall be allowed to the said prisoner to be deducted from his sentence in addition to the regular good time allowance."

On July 13, 1911, the Superintendent of the Prison was instructed to send twenty-five additional men to Tempe for employment on this work, bringing the prison force up to fifty men and shortly thereafter increased to bring average up to fifty-seven men for the period of twenty-seven months for completion of the bridge. Of the average of fifty-seven prisoners at the Tempe bridge site, forty-eight were employed on the bridge proper and nine on camp work (one cook and waiter for engineer's and foremen's mess, one cook and waiter for prison mess, one barber, one laundryman and one corralman). The paid force consisted of one engineer, one assistant engineer, five foremen, two carpenters, seven guards and one bookkeeper.

The original plans and specifications called for a nine span solid arch ring bridge 1225 feet in length for 16-foot roadway and estimate on this basis was made of $78,397.92. Later, these plans and specifications were revised to call for an eleven span arch rib type bridge for 18-foot roadway with open spandrel walls and various other changes were made, necessitating additional paid skilled labor in the way of carpenters, etc., together with increase in reinforcing materials. Detail of construction costs for project, which includes south approach with San Francisco Canal Culvert; (North approach included in Phoenix-Tempe Highway.)

Material:

Cement	$ 8,343.74
Steel	17,496.76

Lumber	7,493.27	
Hardware, etc.	6,593.62	
		$ 39,927.39

Freight on Materials:

Cement	$ 4.792.00	
Steel	813.91	
Lumber	623.63	
Hardware, etc.	876.79	
		$ 7,106.33

Miscellaneous Supplies:

Oil and Coal	$ 3,821.43	
Freight	905.67	
		$ 4,727.10

Tools and Equipment:

Construction	$ 6,875.51	
Engineering	619.77	
Freight	937.51	
	$ 8,432.79	
Equipment credits	286.30	
		$ 8,146.49

Teams:

Rentals	$ 5,785.22	
Feed	4,024.18	
		$ 9,809.40

Labor:

Engineer and Assistants	$ 8,702.81	
Foremen	8,591.42	
Carpenters	10,726.88	
Office Draftsman	1,483.36	
Time and Bookkeeper	1,024.14	
Office Engineer and Force	1,222.71	
Other piad labor	703.37	
		$ 32,454.69

Supplies, Engineer's Mess:	$ 6.473.72

Miscellaneous Expense:

Miscellaneous expense of Engineers and Engineering Parties, including railroad fare, rent of quarters, telephone and telegraph charges, office supplies, etc.	$ 2,510.03
Oiling approach to bridge	130.00
Rent of land for storage of material	256.50

Right of Way:		624.70
Paving:		2.866.53
Bridge Plate:		126.55

Prisoners:

Camp Supplies, food, clothing, etc.	$ 16,913.63	
Salary of Guards	17,722.89	
Transportation	718.45	
Medical attention	365.75	
Escapes	342.65	
Expense of Guards	84.44	
Stockade expense, building, wiring, telephone, lights, etc.	1,382.91	
	$ 37,530.72	
Maintenance refunds from Prison	25,770.74	
		$ 11,760.01
		$126,919.44
Credit due account equipment recovered as State Equipment		1,439.47
		$125.479.97

Included in cost of bridge are the following:

South approach	$2,600.86	
San Francisco Canal Culvert	3,960.00	
		$ 6,560.86
Net Cost Tempe Bridge		$118,919.11

Credit has not been allowed this construction for the stockade and buildings now standing, also lumber left over, probable value, $1.000.00.

UNIT COSTS OF LABOR

Total men days 46,859
Total working men days 33,726

Average number of men on job	57
Average number of men on bridge work	48
Average number of men on camp work	9
Average cost per man per day, exclusive of maintenance refund	.80
Average cost per man per day, less refund	.25
Average cost per working man per working day, exclusive of refund	1.11

Av. cost per working man per working day, less refund....	.348
Average cost per man per day for guards..............................	.378*
Average cost per working man per working day for guards	.53**

*Included in Item 4.
**Included in Item 6.

Escapes	11
Recaptured	2
Engineers and Assistants per working day........	$16.22
Foremen per working day	19.28
Carpenters per working day	16.43
Office per working day	3.51
Other per working day	2.95

Efficiency report of Division Engineer in charge of this construction shows the following:

"Paid labor force necessary to do the same amount of work per day as 48 prisoners:

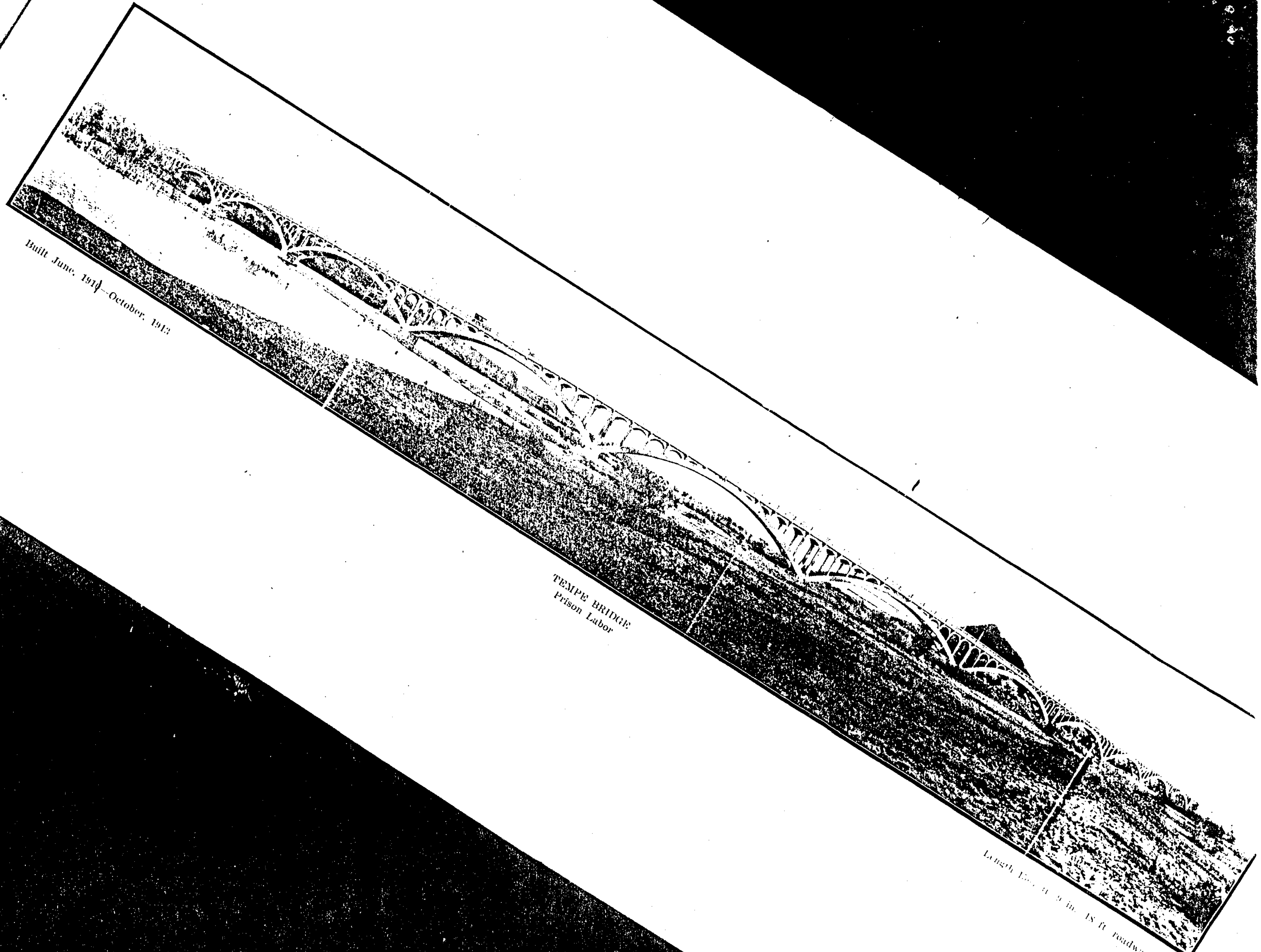
1 Blacksmith	$ 4.00	
3 Derrick Engineers @ $3.50	10.50	
14 White laborers on foundation work, etc. @ $2.50..	35.00	
8 Laborers on concrete work @ $2.00........................	16.00	
4 White teamsters @ $2.50	10.00	
6 Laborers on rock crusher @ $2.00..........................	12.00	
1 Cook for Engineer's Mess	2.50	
		$ 90.00
48 Prisoners @ $1.11..		53.28
Difference in favor of Prison Labor per day....		$ 36.72"

Cooks and waiters for general mess not included under paid labor as boarding house should be self-sustaining.

GLOBE-RAY HIGHWAY

Section Two.

In July, 1912, it was decided to start work on what is designated Globe-Ray Highway, Section Two, extending from the City of Globe toward Ray to the Gila County boundary, a distance of 21.0 miles. After investigation of honor system as then being used in Colorado and other states, it was deemed expedient to adopt that



Built June, 1911—October, 1913

TEMPE BRIDGE
Prison Labor

Length [illegible] ft. 9 in. 18 ft. roadway

SECOND REPORT

OF THE

STATE ENGINEER

TO THE

State Highway Commission

(State Board of Control)

GEO. W. P. HUNT	Governor
J. C. CALLAGHAN	State Auditor
CHAS. R. OSBURN	Citizen Member
LAMAR COBB	State Engineer

AND TO THE

Boards of Supervisors of the Several Counties

FOR THE PERIODS

July 1, 1914 to June 30, 1915
AND
July 1, 1915 to June 30, 1916

Excavation—

Supt.			7.30	
Labor			88.40	

Building Flumes—

Labor			102.00	
Supt.			8.76	
Mdse.: Labor	99.10			
Nails	2.11		101.21	
Team hire			18.00	325.67
125.5 lin. ft. cost per lin. ft.		2.638		625.63
M. & G. acct. on 426.90				64.04
				$716.67

Phoenix-Tempe Highway—

This section maintained by caretaker J. B. Blakley and assistants as required.

Labor	$1,320.28	
Material	196.32	
Team hire, feed, etc.	765.79	$2,282.39

Tempe Bridge—

Caretaker on Phoenix-Tempe Highway charged with keeping clean and lights in order.

Labor	6.40	
Material, etc.	66.54	72.94

Florence-Mesa Highway Sec. 1—

This section extending from Higley 7.1 miles to Pinal County line dragged after rains and minor repairs made.

Labor	57.38	
Team hire and miscellaneous	112.55	169.83

Division Engineer F. W. Twitchell

Mesa-Roosevelt Highway

Upon completion of improvement work with Honor Prison

#2

Division Engineer:

Salaries	34.20	
Expense	25.35	59.55

Office:

Salaries		70.36
		$129.91

Prescott-Phoenix Reconnaissance—

Reconnaissance of proposed highway from Phoenix to Prescott commenced.

Salaries	$ 72.58	
Travel expense	75.10	
Miscellaneous	39.25	186.93

GENERAL

Phoenix-Tempe Highway Prison—

Delayed charges to this work completed in previous year amounting to $65.18 paid and credit of $428.10 for returned cement sacks.

Tempe Bridge:

Disputed charges from 1912 adjusted.

#2

Arizona State Prison	$235.45
South Side Gas Electric Co.	38.08
	$273.53
Loading signs erected at each end of bridge	14.00

General Expense—

Team hire for steel	$ 20.00
Powder on hand	35.51
Miscellaneous labor	11.07
	$66.58

#2

Tempe Bridge:

Description circulars of escape ....$ 6.80

Phoenix-Yuma Highway, Arlington-Agua Caliente Division:

This project under direction Division Engineer F. G. Twitchell, whose report follows:

Up to this year, no improvements had been made on the Phoenix-Yuma Highway in Maricopa County, and the route to be followed west of Arlington had not been determined. It having been decided to spend the available funds between Arlington and Agua Caliente, on account of this section being the worst part of the road, it became necessary to make a choice between a route by way of Wolsey Peak and Point of Rocks, which had been the one generally used; and another route which ran further to the north by way of Fourth of July Butte and Yellow Medicine Wash. A careful investigation of both of these routes was made, and the northern route was decided upon as being the better.

It was found that both routes presented many difficulties and disadvantages. The southern route would have required a great deal of heavy rock work to get through Wolsey Wash and past the Point of Rocks. It also ran for many miles through the silt bottom land of Cottonwood Wash and the Gila River—the poorest kind of material for road purposes—and the outlook for obtaining anything better for surfacing was very discouraging, as there was nothing suitable that would give a shorter average haul than about ten miles. There were also many large and unconfined washes to cross.

The northern route ran through a somewhat rougher country, and was a few miles longer, but the material was, in the main, of a suitable character for surfacing, and there was a great deal less drainage to be looked after. For these reasons it was decided upon as being the one that would prove most economic eventually.

The expenditure of the money available was concentrated on the worst part of the road, which was between the Yellow Medicine and Lowdermilk Washes, a distance of three miles. The grading

#2

Tempe Bridge—

During floods early in 1916, the south approach to this structure was endangered and it was necessary to have some protection work done.

Labor	$ 125.47	
Teams	24.50	
Teams' feed	6.00	
Lumber	15.78	
Cement	5.40	
Globes	26.03	
Repairs to equipment	13.20	
Miscellaneous expense	34.15	
Expense Assistant Engineer	1.30	$ 251.83

Equipment—

Purchased 1915-6	$ 291.99
Refund of proportion of cost on State Highway Department Automobile	494.00
Net Credit	$ 202.01

Equipment on Hand—

Two Axes	$ 3.00	
One Bar, Pinch	.75	
Nine Brooms, Street	4.84	
Two Checkers, Traffic	4.60	
One Drag, No. 2 Prairie, 7-ft	18.95	
One Drag, Steel, 10-ft	33.00	
Two Drags, Wooden	23.00	
2 Fresnos, 5-ft.	50.75	
One Hammer, 8 lb.	.70	
Two Harrows	19.00	
One Injector	15.00	
Two Picks	2.25	
Two Screens, Gravel	8.80	
Seventeen Shovels	8.50	
One Sprinkler, 600-gal.	480.00	
Forty-two ft. Steel Drills	5.50	
One Tamper, Cement	1.50	
Eighteen Teeth, Scarifier	45.63	
One Wagon Tank, 12 bbls.	184.90	
One Wagon, Freight, 3¼ in.	75.00	$ 1,045.67

THIRD BIENNIAL REPORT

OF THE

STATE ENGINEER

TO THE

GOVERNOR AND THE COMMISSION OF STATE INSTITUTIONS

GEO. W. P. HUNT .... Governor
LEROY A. LADD .... Chairman
LOUIS B. WHITNEY .... Member
CHAS. R. OSBURN .... Member

AND TO THE

Boards of Supervisors of the Several Counties

FOR THE PERIOD

July 1, 1916, to June 30, 1918

FOURTH BIENNIAL REPORT

OF THE

STATE ENGINEER

TO THE

GOVERNOR

OF THE

STATE OF ARIZONA

For the Period
July 1, 1918, to December 31, 1920

4

spans. Total length of bridge, 195'-4". Two westerly spans washed out during the fall of 1916, and were replaced by a steel truss. The Thanksgiving flood of 1919 wrecked this structure entirely. Except at the westerly abutment, the foundations were all placed on sand and had inadequate depth to withstand the scour during the high water. This stream carries considerable large drift, and, if the reports of eye witnesses are correct, the bridge was constructed with insufficient headroom. The largest flood experienced during twenty years occurred in August, 1920, and served to demonstrate the utter inadequacy of this bridge. This Department has begun the construction of a 3-span, 303-foot over all bridge with steel trusses and concrete floor. The length will be greater by more than fifty per cent and the headroom will be nearly double that of the former structure.

Previous efforts to construct and maintain this bridge have cost the taxpayers of Maricopa County something over $20,000, together with a part-time loss of use. This Department contemplates an additional expenditure of about $70,000, making a total of over $90,000. A properly designed bridge in the first place would have cost about $30,000 or, in other words, would have saved $60,000 and much inconvenience.

Tempe Bridge: Repairs

#4

The second pier from the north end of this bridge settled about 4½ inches shortly after the Thanksgiving, 1919, floods. Traffic was maintained, except during high water, until the second drop, which occurred February 13, 1920, and amounted to about ½". Two tons was then named as the maximum load permitted to cross. On March 2, 1⅛" additional deflection necessitated closing the bridge and on the following day a sudden drop of nearly 5" was recorded. Emergency measures to insure the stability of the structure were completed and the bridge opened to pedestrian traffic March 4. Material and equipment for sinking caissons and underpinning the defective pier had been in process of assembling for some time and the belated arrival of timber permitted permanent repair work to start late in the month; the Governor having declared that an emergency existed and set aside $45,000.00 for the repairs on March 25.

Serious cracks in the superstructure made necessary extensive repairs over three piers at the south end. This work was completed, the false work to support the sunken arches erected, and the bridge opened to vehicular traffic on May 11, 1920. Since that time the retaining wall at the south approach has been underpinned, the crown expansion joints renewed, sheetpile cofferdam driven and at this time the sunken pier is underpinned and the false work removed. Considerable repairs are yet needed on the handrail, floor and spandrel columns.

It may be pertinent to call attention at this time to the existence of other very serious cracks in the arch rings and floor system. The bridge was originally designed for a live load of 100 pounds per square foot, plus a 15-ton traction engine. The present State requirements are 150 pounds per square foot of roadway surface, or two 15-ton trucks. The bridge as built is a more or less indeterminate structure, but an analysis of stresses calls the sufficiency of the floor system and arch rings seriously in question.

Besides the one undergoing repair, five additional piers must be considered as of doubtful capacity to sustain the loads to which they are subjected.

In view of the extreme economic value of the Tempe Bridge—about 2500 vehicles per day use this structure at an estimated saving of $1.00 each over the longer route—it is the recommendation of this Department that the Legislature be informed of the unsatisfactory condition of this bridge, the liability of serious accident which may require restricting traffic or closing the bridge altogether and the possibility of requests for maintenance funds during the next two years.

The general scheme of repair would be to follow the procedure at the sunken pier for underpinning operations; the placing of additional arch rings of reinforced concrete or steel; and necessary strengthening alterations in the floor system to provide for temperature variation and to transfer the loads to the new arch rings. Work would have to be carried on without stopping traffic; a condition which would increase the cost to the State by many thousands of dollars.

The original cost of the Tempe Bridge was $151,250.71 and it will probably require somewhere in the neighborhood of $450,000.00 at present prices to build a new bridge adequate to carry present day traffic. The expenditure of about one-half of this latter amount should serve to increase the life of the bridge for several years over that now estimated, which is about five at best.

Miscellaneous Duties

Plans have recently been prepared for County bridges of an aggregate value of nearly $70,000.00. This work is done for the Counties of the State for the actual cost of preparing the plans and specifications.

FIFTH BIENNIAL REPORT

OF THE

STATE ENGINEER

TO THE

GOVERNOR

OF THE

STATE OF ARIZONA

For the Period

July 1, 1920, to June 30, 1922

5

building adds to rather than detracts from the looks of the surroundings.

The costs of the various improvements are as follows:

Office Building	$ 52,045.24
Warehouse	14,499.76
Sheds	9,469.08
Machine Shop	7,542.94
Paint Shop	7,311.04
Grounds and Fence (inc. cost of land)	18,545.49
Spur track	1,575.63
Paving	2,003.56
	$112,992.74

6. RECOMMENDATIONS

#5

TEMPE BRIDGE: This bridge is showing slowly progressing evidences of failure in the superstructure. Numerous cracks have appeared in the floor slabs and beams and in the spandrel arches and columns. The increasing number of these cracks indicates the possibility of an ultimate failure which may be serious in its consequences. Frequent inspections should therefore be made to determine the condition of this bridge. The floor system as originally constructed had no expansion joints. It was continuous from the crown hinge of one arch to the crown hinge of the next. From what appears to be the consequence of this form of construction, the floor slab, floor beams and spandrel arches cracked transversely to the center-line of the bridge at three piers. In various other spans the spandrel columns cracked. This Department cut out portions of the floor and spandrels at the above mentioned three piers and built in expansion joints. This work has proved to be entirely satisfactory and it is recommended that a similar procedure be followed at several other piers. The provision of these roadway expansion joints should have the effect of eliminating thermal stresses in the floor and should therefore lengthen the life of the bridge.

FLORENCE BRIDGE: The Florence Bridge consists of 29 girder spans, each 50 ft. in length. For the most part these girders are continuous over two piers. A recent inspection discloses that numerous cracks are appearing in the beams adjacent to the fixed piers. None of these footings, it should be noted, are founded on unyielding material, but on the contrary are supported by piling driven into the silt of the river bed.

SIXTH BIENNIAL REPORT

OF THE

STATE ENGINEER

TO THE

GOVERNOR

OF THE

STATE OF ARIZONA

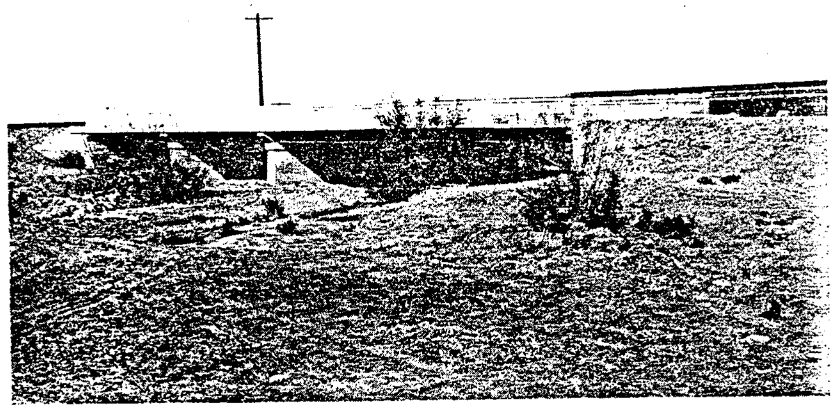
For the Period

July 1, 1922, to June 30, 1924

MANUFACTURING STATIONERS INC., PHOENIX, ARIZONA

forced concrete box culverts, a steel rail cattle guard, and a wire guard fence. A set of 4- girder reinforced concrete decks, ranging in span from 20 feet to 40 feet, have been worked up and are being used in the place of the old 3-girder standard which has become obsolete. These new spans, although designed for heavier loads than the old, are more economical in materials and have been used exclusively in the past two years where such spans were required.

TYPICAL 4-GIRDER BRIDGE



FOUR 30-FOOT SPANS NEAR MARINETTE

Proposed Work

The accomplishments of the Department, in the past two years, have been very gratifying to the members and the future looks bright with new and interesting work of greater dimension. A bridge over the Gila River near the Gillespie Dam is practically assured. This bridge will be a Federal Aid Project costing approximately $300,000. The moving of the San Carlos Bridge on the Gila River to a point above the new dam and the proposed bridges on the Colorado River are other interesting projects

Repairs are contemplated on the Tempe Bridge, chiefly in replacing the expansion joints. A paint crew is being sent out, with an air brush outfit, to paint all of the old steel bridges which are badly in need of such work. The first of these will be the San Carlos Bridges.

#6

SEVENTH BIENNIAL REPORT

OF THE

STATE ENGINEER

TO THE

GOVERNOR

OF THE

STATE OF ARIZONA

FOR THE PERIOD
JULY 1, 1924, TO JUNY 30, 1926

of our older concrete girder bridges, namely, New River near Marinette and Granite Creek near Whipple Barracks, Prescott.

In these bridges steel expansion plates were used which had become frozen together from corrosion. A roller nest consisting of four rollers—three rollers for Granite Creek—and a pin joint was designed and fabricated. These were placed as units under one end of each girder, the concrete of the pier or abutment being chipped out to make room for the new shoe. The spans were raised by means of false-work timbers and oak wedges and lowered back on the new shoes. These spans are now moving in a satisfactory manner, and safe from further destruction by temperature stresses.

In many cases the cracks which had developed in the girders and piers closed up after installation of the shoes. These shoes were placed at a cost of about $250 per shoe, whereas the same shoe placed during the construction of the bridge would have cost less than $70, but the expenditure at this time saved many thousands of dollars had the spans been left as they were built. Twelve of these shoes were placed at New River and nine at Granite Creek. The cost at New River being $500 per span on a span worth $4,000 to replace.

On the Tempe bridge considerable work was necessary to make it safe and prevent further disintegration. The bridge is designed for light traffic only and that feature combined with the stresses produced by the failure and settlement of one of the piers has been the cause of the partial failure of many members, especially the spans adjacent to the pier on which the settlement occurred.

The work of repairing this bridge was started in January, 1925, and consisted of placing new expansion plates in the roadway slab and rebuilding several columns and beams by means of gunite concrete.

Steel expansion plates were not provided in the original construction and angle irons place at these joints during the repairs in 1920 were a complete failure, leaving large holes in the floor and causing enormous stresses in the structure due to the impact of heavy loads. It was seen that in order to save the bridge from complete destruction by these forces that these joints would need immediate replacement. The old joint and part of the concrete slab was cut out and a joint

consisting of angles and a heavy plate was securely anchored in place and backfilled with concrete placed with a cement gun. This method of placing of concrete was used on account of its great strength and the bond which could be secured between the new and the old concrete. Plans were ready for this work in December, 1924, but owing to the necessity of closing the bridge for the major portion of the work, operations were delayed until after the holiday season and the work was done in January, 1925.

The bridge now has a smooth riding surface and is in a better condition structurally than after the repairs of 1920, but is still too light for present day traffic conditions. The ultimate solution is a new bridge to which heavy traffic can be diverted, thus leaving the old bridge for light cars and local traffic.

Standard Plans

As stated before, the present standard plans are virtually all out of date on account of recent changes in loading specifications and manufacturers' specification for reinforcing bars. It would have been useless to change these plans until these specifications were satisfactorily completed.

The revision of these standard plans will take many weeks of tedious work and the department is making plans for immediate revision of those standards most frequently used. Those which will come first in the list will be: box culverts, abutments and decks for slab and girder bridge.

As has been stated before, the value of standard plans has been considerably reduced by advanced type of location, but nevertheless their use greatly reduces the labor and time required for the preparation of highway plans and are also valuable and necessary to the locating engineer in selecting the proper type and size of structure for a particular location.

Along with the revision of old standards will be several new ones such as double boxes, four-girder spans from 22 feet to 44 feet, three-girder spans from 44 feet to 60 feet, and a group of standard U-abutments, many of which have been used on special structures in the past two or three years.

PUSH AVENUE

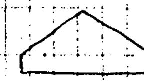
BRIDGE

PENTAX

Field Book

PB-1

3/2/0 ASH AVE BRIDGE ELEV'L FRANKLEY COOL CLOUDY SET

SHOT #	VERT ∢	D	EL. DIFF	ELEV	DESCRIPTION
				~~(1138.93)~~	
BMK #1 -	88°09'02"	151.40	+4.89	1134.04	SE Cor of E. Pier Ftg @ Pier #4 - ASH AVE BRIDGE
1	88°41'34"	252.52	5.69/13.69	1147.73	Pier #6 @ Bottom of Cap  Elev here
2	89°05'28"	356.10	5.64/13.64	1147.68	Pier #7 @ " " "
3	89°19'34"	496.74	5.84/13.84	1147.88	Pier #8 @ " " "
4	89°32'58"	623.96	4.90/12.90	1146.94	Pier #9 @ " " "
5	89°32'40"	750.23	+5.96/13.96	1148.00	Pier #10 @ " " "
6	87°06'00"	754.05	+38.20	1172.24	Top of Deck elevation @ Pier #10
7	86°36'46"	625.58	+37.03	1171.07	" " " " @ " #9
8	85°38'29"	499.38	+38.06	1172.10	" " " " @ " #8
9	84°12'45"	375.14	+38.03	1172.07	" " " " @ " #7
10	81°39'39"	258.99	+37.96	1172.00	" " " " @ " #6
11	77°01'30"	165.94	+38.21	1172.25	" " " " @ " #5

SHOT #	VERT ∡	D	EL. DIFF	ELEV	DESCRIP.
12	75° 39' 25"	149.75	+38^{31}	1172.35	Top of Deck @ Pier #4
13	80° 23' 39"	226.55	+38^{34}	1172.38	" " " @ Pier #3
14	83° 34' 35"	339 89	+38^{25}	1172.29	" " " @ Pier #2
15	85° 14' 56"	461.33	+38^{34}	1172.38	" " " @ Pier #1
16	89° 15' 26"	459^{82}	5.96/13.96	1148^{09}	Bottom of Pier Cap @ Pier #1
17	89° 10' 11"	459.82	6.66/14.66	1148.70	" " Arc @ Pier #1
18	88° 59' 48"	334.59	5.91/13.91	1147.95	Bottom of Pier Cap @ Pier #2
19	88° 51' 30"	334.59	6.67/14.67	1148.71	Bottom of Arc @ Pier #2
20	89° 29' 32"	222.65	5.86/13.86	1147.90	Bottom of Cap @ Pier #3
21	88° 18' 05"	222.65	6.61/14.61	1148.65	Bottom of Arc @ Pier #3
22	88° 30' 49"	252.52	6.55/14.55	1148.59	Bottom of Arc @ Pier #6
23	88° 58' 50"	356.10	6.34/14.34	1148.38	" " " " " #7

SHOT #	VERT ∡	D	EL DIFF	ELEV	DESCRIP.
23	89°13'51"	496.74	6.67/14.67	1148.71	Bottom of Arc - Pier #8
24	89°28'40"	623.96	5.69/13.69	1147.73	" " " - " #9
25	89°28'52"	750.23	6.79/14.79	1148.83	" " " - " #10
26	89°13'13"	235.65	+3.21/11.21	1145.25	WATER MARK - WEST SIDE of MILL AVE BRIDGE (5th Pier from North)
27	89°25'24"	314.95	+3.17/11.17	1145.21	WATER MARK @ Mill Ave - 4th Pier from North
28	89°32'24"	419.71	+3.37/11.37	1145.41	" " " " " - 3RD Pier from North
29	87°09'24"	839.64	+41.71	1175.75	Top of ~~Deck~~ CURB - Mill Ave (@ North Abut)
30	86°33'19"	700.77	+42.18	1176.22	" " " - Mill Ave (1st Pier SOUTH)
31	85°38'25"	560.55	+42.73	1176.77	" " " - " " (2nd Pier SOUTH)
32	84°11'23"	424.00	+43.14	1177.18	" " " - " " (3RD PIER SOUTH)
33	84°19'00"	425.44	+42.35	1176.39	Top of Deck - Mill Ave (3RD Pier SOUTH)
34	82°05'30"	312.10	+43.32	1177.36	" " CURB - " " (4th Pier SOUTH

HI = 5.44 (89°25'45") 6.16

SHOT #	VERT ∡	D	EL DIFF	ELEV	DESCRIP	
35	79°47'00"	240.05	+43.36	1177.40	TOP of CURB - MILL AVE	(5th PIER SOUTH)
36	80°06'13"	248.25	+43.28	1177.32	" " " - " "	(6th " ")
37	80°25'03"	251.79	+42.56	1176.60	" " DECK - " "	(" " ")
38	82°30'20"	328.05	+43.14	1177.18	" " CURB - " "	(7th PIER SOUTH)
39	84°33'30"	448.15	+42.68	1176.72	" " " - " "	(8th PIER SOUTH)
40	85°48'50"	575.21	+42.10	1176.14	" " " - " "	(9th " ")
41	86°37'00"	705.38	+41.71	1175.75	" " " - " "	(10th " ")
42	86°41'08"	708.22	+41.00	1175.04	" " DECK - " "	(" " ")
43	88°10'52"	93.0	+2.9	1136.9	STREAM BED	West Ash Avenue
44	89°39'25"	132.9	+0.8	1134.8	" "	
45	89°43'50"	139.0	+0.6	1134.6	" "	
46	90°10'38"	180.8	-0.5	1133.5	" "	

SHOT #	VERT ∢	D	ELEV DIF	ELEV	DESCRIP
47	90°12'28"	231^{1}	-0^{8}	1133^{2}	STREAM BED
48	90°09'15"	256^{0}	-0^{6}	1133^{4}	" "
49	89°51'49"	308^{0}	$+0^{7}$	1134^{7}	" "
50	89°59'06"	359^{3}	$+0^{1}$	1134^{1}	" "
51	89°56'35"	482^{2}	$+0^{4}$	1134^{4}	" "
52	89°24'00"	593^{2}	$+6^{2}$	1140^{2}	" "
53	89°57'16"	279^{4}	$+0^{2}$	1134^{2}	" "