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Final Bridge Selection Report

East Papago Salt River Viaduct
Segmental Concrete Bridge Alternate
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For
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I. GENERAL

Purpose

This report summarizes the studies made for the segmental concrete alternate for Segment 6 of the East Papago Freeway. The East Papago Freeway connects I-10 (the Papago Freeway) on the west to the Outer Loop (Pima Freeway) and Red Mountain Traffic Interchange on the east.

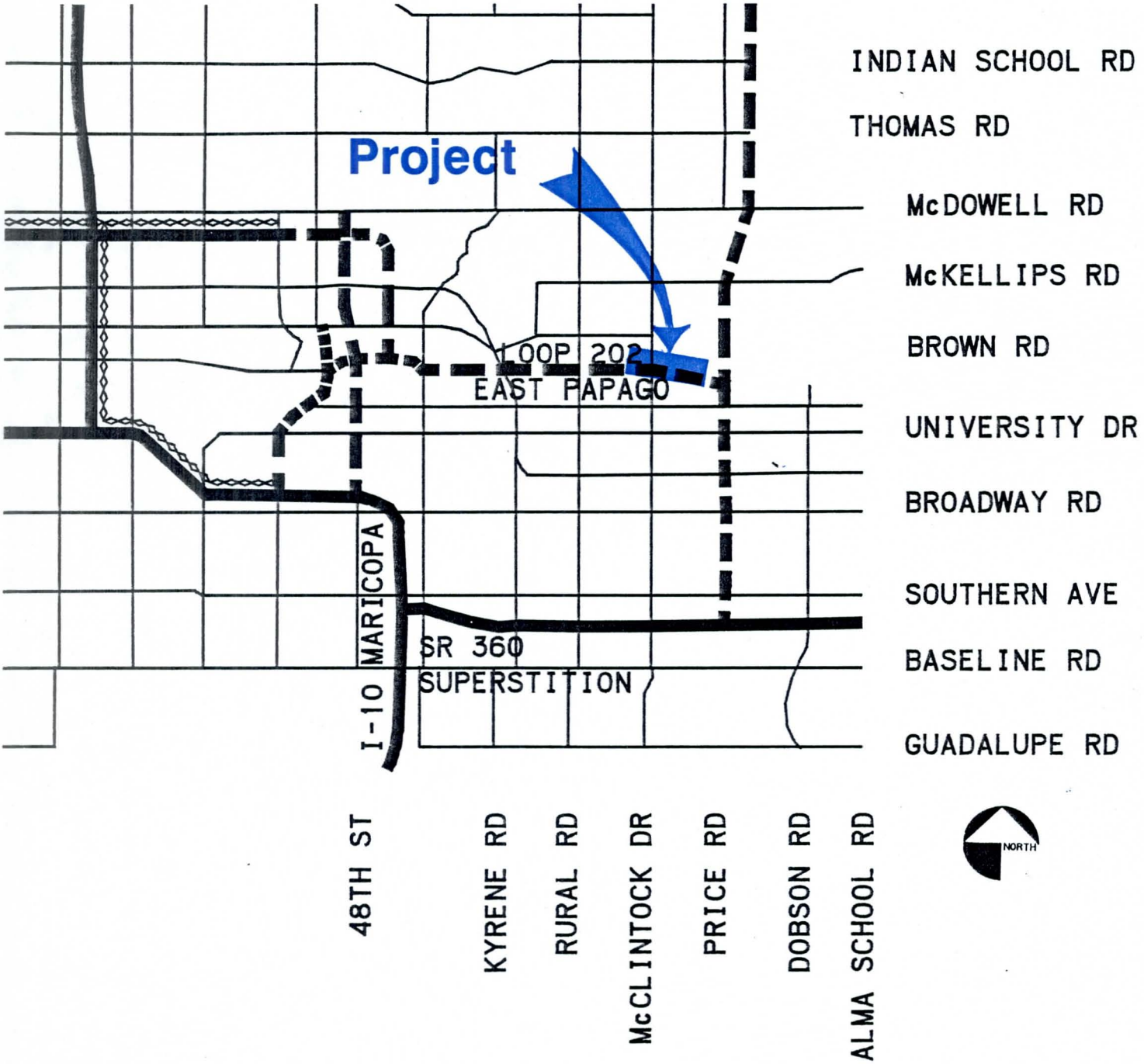
Segment 6 of the East Papago Freeway begins at the west side of Indian Bend Wash and terminates at the east with the Red Mountain Interchange. After the freeway crosses Indian Bend Wash, it travels eastward to the north bank of the Salt River, just west of McClintock Drive. The Freeway passes over the north end of the McClintock Drive bridge and continues eastward about 2600 feet, traveling along the Salt River bed. The alignment then curves southward to connect with the Red Mountain Interchange on the south bank of the river. A plan of the proposed freeway is shown in Figure 1.

The purpose of the proposed viaduct is to carry the East Papago Freeway over the McClintock Drive Bridge and over the Salt River.

Summary of Study Results

The recommendations reached in this report indicate the desired structure is a precast segmental box girder bridge with a superstructure consisting of two single-cell box girders for each roadway with 150 foot spans prestressed using external tendons. The substructure consists of 9 foot round columns flared at the top under each box girder with 10 foot diameter drilled shafts founded in the SGC layer and underlying clay.

FIGURE 1



LOCATION PLAN

II. BRIDGE GEOMETRICS

Roadway Cross-Sections

To the east of the viaduct the East Papago Freeway consists of four 12-foot traffic lanes plus a 12-foot High Occupancy Vehicle (HOV) lane in each direction. Each of the roadways has a 10-foot shoulder on both the right and left. The two 80-foot wide roadways continue across the Salt River to the Red Mountain Interchange. Each roadway carries two traffic lanes and an HOV lane through the Interchange. In addition, exit lanes on the eastbound roadway and entrance lanes on the westbound roadway merge with the mainline to form flared roadways at the eastern terminus of the viaduct.

The four-lane roadway with HOV lanes and ten-foot shoulders are the ultimate width of the freeway. No provision for future widening is provided. Roadway cross sections are shown in Figure 2.

Bridge Cross Section

The viaduct cross sections match the east and west approach roadway sections described above. The two roadways on structure are separated by a 27-foot wide median, edge of HOV lane to edge of HOV lane. The viaduct consists of two separate structures with 3-foot 10-inches between interior fasciae of the bridges.

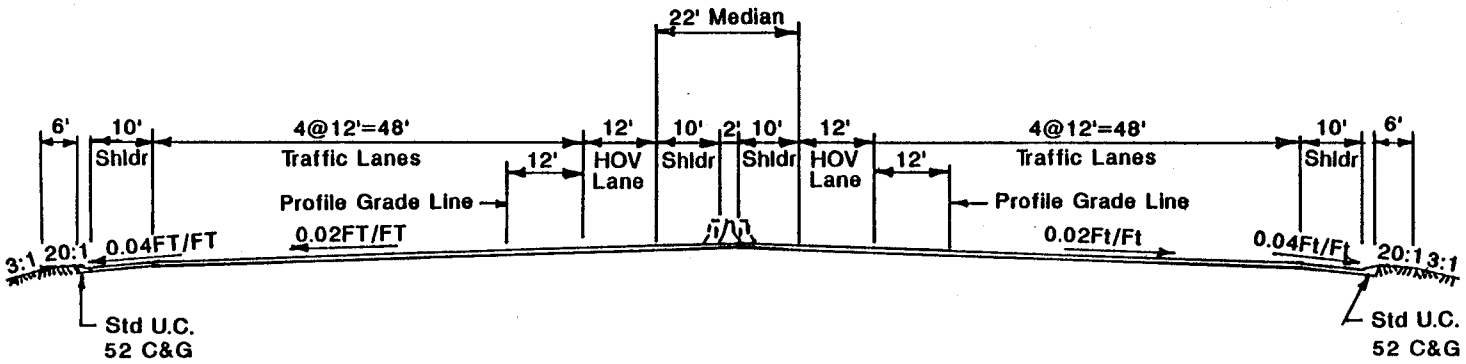
Cross sections on the viaduct will vary from the normal cross slope of 0.02 feet per foot to a maximum super elevation of 0.04 feet per foot. A typical bridge cross section is shown in Figure 2.

Right-of-Way

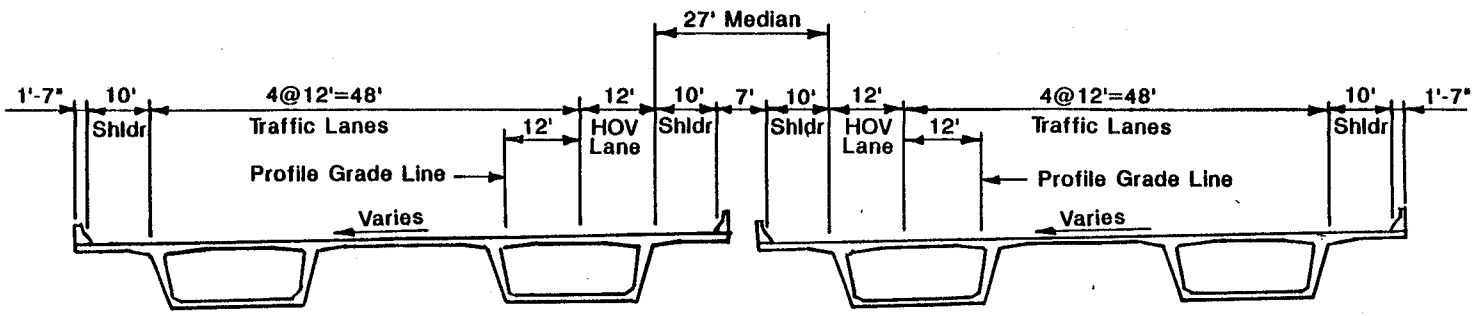
The proposed viaduct will be built within the right-of-way limits to be provided for the project.

Roadway Alignment

The horizontal alignment of the Salt River Viaduct consists of a 3770-foot long tangent section at the west end. A one degree thirty minute curve, 1042 feet long, carries the roadways to the south bank of the Salt River, where a 585-foot long tangent section terminates the viaduct.



ROADWAY SECTION



SEGMENTAL BOX GIRDER ALTERNATE

TYPICAL SECTIONS

FIGURE 2

The profile rises at the west approach on a two percent grade to provide clearance over McClintock Drive. Once over McClintock Drive, the grade rolls gently downward to meet with the Red Mountain Interchange alignment. The profile is controlled by clearance over Ramp "B", McClintock Drive and freeboard over the Salt River high water level. Also, the grades adjacent to the viaduct structure must not be influenced.

The east and west abutments were set to match the abutment locations previously established for the prestressed concrete I-girder alternate. Other common roadway approach items will also be set to match the I-girder alternate. These items include retaining walls, approach slabs and anchor slabs. Abutments and piers are generally set normal or radial to the centerline of the project, except for the piers adjacent to the off-ramp at McClintock Drive. An eastbound pier over Ramp B is skewed $29^{\circ}45'$. This pier is skewed to optimize the span lengths from the west abutment to McClintock Drive.

Constraints for pier locations occur at Ramp B, at the McClintock Drive Bridge and at utilities adjacent to McClintock Drive. Following the Management Consultant's recommendation, the pier to the east of McClintock Drive was placed at the same location as the prestressed I-girder alternate. Using two 160-foot spans west from this pier allows clear spanning of McClintock Drive and keeps pier placement away from the 230-KV overhead lines. These lines cannot be de-energized for construction of the drilled shafts.

Several span arrangements for the eastbound roadway over Ramp B were considered. Clear spanning of the ramp requires a span of about 200 feet. This span length is beyond the capabilities of an eight-foot deep box section utilizing external tendons. Because other more cost effective spans are available, the clear spanning of Ramp B by the eastbound roadway is not recommended. Shorter spans could be achieved using a straddle bent normal to the roadway with columns outside clear limits to Ramp B. The bent would have to be framed into the superstructure to provide vertical clearance over Ramp B. Framing the superstructure into the cap beam complicates segment placement, post-tensioning and other

construction procedures. The recommended skewed pier placement simplifies the construction. All of the dead load, except barrier curbs, would be placed on normal bents. During placement of the longitudinal closure pours, transverse diaphragms would be cast between the parallel box segments, normal to each column, to provide for more equal distribution of live loads to the skewed pier.

Pier locations at the eastern end of the viaduct are placed to optimize the placement of box girders in the flared roadway portions of the viaduct. Piers are required where the number of boxes changes and where the direction of the box girders changes. Recommended pier locations are shown on the General Plan and Elevation drawings of the preliminary plans.

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III. HYDRAULICS

Channel Flow

The hydraulic and scour parameters for the project were provided by Simons, Li and Associates, Inc. (SLA). Information required for the preliminary and final design of the viaduct was taken from Addendum No. 4 to the "Preliminary Hydraulic Analysis of the Salt River for the East Papago Expressway and Red Mountain Interchange" and from memos prepared by SLA.

The SLA addendum indicates the hydraulic analyses were run for a baseline condition and for a concept condition. Two baseline conditions were analyzed: 1) Existing conditions by using 1986 topographic mapping; 2) Modified baseline conditions by removing the effect of gravel pits in the study area. The modified baseline was used to analyze project impacts. This structures concept study has used only the data related to the concept condition for the preparation of this report.

The SLA concept conditions for Alternative 4 are briefly summarized as follows:

1. The northern alignment of the East Papago Freeway is entirely on structure through the study area.

2. A levee on the south bank of the Salt River prevents overtopping of the bank. ELEVATION?

3. The north bank is protected to Elevation 1170.0.

4. The effect of existing mining in the stream bed is included in the study.

5. The preliminary configuration of the Red Mountain Interchange was used.

6. The structure consists of 135-foot spans with 9-foot diameter columns, two columns per roadway.

7. Bridge crossing computer routines were not used, instead bridge piers were modeled as obstructions in the flow field.

8. Each column is exposed to flow of the river.

9. The effective length of mainline crossing Salt River is approximately 1200 feet.

All of the above conditions are valid for the segmental concrete alternate except for span length. The segmental concrete alternate will use slightly longer spans, 150 feet. The longer spans should make the use of the SLA studies slightly conservative and, therefore, valid for the preliminary phase of this project.

Table 1 presents water surface profiles taken from the SLA data for the 100-year flood; Q = 215,000 cfs, and for the superflood, Q = 250,000 cfs. The effect of two feet of debris build-up on each side of the columns is included in the tables. A minimum freeboard of two feet, distance from 100-year water profile to low structure, is recommended by SLA. Table 2 presents computed water surface elevations, average velocities, and top widths for the 100-year flood. The data presented in these two tables will be used, where appropriate, in the design of the viaducts.

Scour

The minimum predicted scour invert elevations computed by SLA, for both the 100-year frequency, Q = 215,000 cfs and the superflood conditions, Q = 250,000 cfs, are reproduced in Table 3. The total assumed scour, for the design of the structure, is a combination of general scour, bed-form scour, long-term degradation of the stream bed, plus 30% of these conditions as a factor of safety, plus scour caused by pier influence.

TABLE 3 PREDICTED SCOUR INVERT ELEVATION
EAST PAPAGO SALT RIVER VIADUCT

PIER ZONE	STATIONING	PREDICTED SCOUR INVERT ELEV (ft)	
		Q=215,000 cf	Q=250,000 cfs
1	286+00 - 295+	N/A	1106
2	297+13.71 - 3	1105	1102
3	302+00 - 310+	1101	1098
4	310+00 - 319+	1093	1092
5	319+00 - 334+	1102	1100
6	334+00 - 337+	1093	1091
7	337+00 -	1093	1091

TABLE 1 CHANNEL INVERT ELEVATIONS

EAST PAPAGO STATION	Q = 215,000 cfs		Q = 250,000 cfs		PHYSICAL FEATURE
	CHANNEL INVERT ELEVATION (feet)	WATER SURFACE ELEVATION (feet)	CHANNEL INVERT ELEVATION (feet)	WATER SURFACE ELEVATION (feet)	
294+90	1148.2	1171.7	1148.2	1173.2	McClintock Drive Bridge
295+75	1148.3	1172.6	1148.3	1174.0	
299+20	1148.5	1173.0	1148.5	1174.6	
303+50	1148.9	1173.1	1148.9	1174.7	
307+50	1149.2	1173.2	1149.2	1174.8	
311+50	1149.5	1173.7	1149.5	1175.4	Old Scottsdale Landfill
316+20	1149.8	1174.5	1149.8	1176.1	
320+20	1150.1	1175.5	1150.1	1176.9	
324+20	1150.5	1176.5	1150.5	1177.8	
328+20	1150.8	1177.4	1150.8	1179.0	
332+45	1151.0	1178.3	1151.0	1180.0	Old Tempe Landfill
336+50	1151.4	1178.9	1151.4	1180.8	
342+25	1152.2	1179.9	1151.8	1181.6	
347+00	1151.8	1180.1	1152.2	1181.9	
	1152.6	1180.1	1152.6	1182.0	Outer Loop Highway

TABLE 2 HYDRAULIC INFORMATION -- Concept Condition for Alternative 4, 100-Year With Debris Buildup

EAST PAPAGO STATION	CALCULATED WATER SURFACE ELEV (ft)	HYDRAULIC DEPTH (ft)	CHANNEL VELOCITY (fps)	TOPWIDTH (ft)	PHYSICAL FEATURE
294+90	1171.7	22.4	8.4	1537.0	McClintock Drive Bridge
295+75	1172.6	23.2	8.1	1737.0	
299+20	1173.0	22.7	7.3	2181.0	
303+50	1173.1	22.0	9.0	2098.0	
307+50	1173.2	21.3	11.6	2038.0	
311+50	1173.7	21.6	12.6	1793.0	Old Scottsdale Landfill
316+20	1174.5	21.4	12.9	1741.0	
320+20	1175.5	22.0	12.5	1482.0	
324+20	1176.5	22.7	11.8	1620.0	
328+20	1177.4	23.9	11.1	1535.0	
332+45	1178.3	23.8	9.9	1645.0	Old Tempe Landfill
336+50	1178.9	24.5	9.3	1322.0	
342+25	1179.9	26.3	7.4	1564.0	
347+00	1180.1	26.5	7.5	1554.0	
	1180.3	26.4	8.8	946.0	Outer Loop Highway

The SLA data has grouped the piers into zones based on the numbering of the piers in the prestressed I-girder alternate and the location of these piers with respect to the various levels of the existing Salt River bed. For this study the pier zones have been related to stationing along the freeway. This stationing was established by relating the I-girder pier numbers to freeway stationing and adjusting the stationing for variations in the stream bed.

The structure will be designed for stability under existing river conditions, for scour due to the 100-year flood, and for scour conditions due to the superflood. Pier foundations will consist of large diameter drilled shafts. Foundation design will ignore side friction above the assumed super flood scour elevations. The weight of the pier and drilled shaft above the assumed super flood scour elevation will be included in the foundation reaction. The weight of the drilled shaft below the assumed scour elevation will not be included in the foundation calculations.

Deck Drainage

The bridge deck will be drained by allowing rain runoff to free fall through scuppers located along the low gutterline. Discharge onto Ramp B and McClintock Drive will be prevented by placing scuppers outside of these roadways. Additional drainage criteria is presented in the SLA addendum.

IV. BRIDGE SUBSTRUCTURE

Piers

Several factors influence the pier type selection for the Salt River Viaduct. The piers are to be founded on large diameter drilled shafts, 80 to 136 feet below stream bed. The piers can be subjected to scour depths ranging from 28 to 58 feet. The piers must be stable for both the unscoured and scoured river bed condition. In addition, the pier type must be compatible with the method of construction of the superstructure.

Three pier sections and two conditions of fixity were investigated. Pier sections included round columns, obround columns and octagonal columns. Each section was investigated for both solid and hollow shapes. Two round columns were investigated, an eight-foot and a nine-foot diameter section. In addition, the pier column tops were investigated for fully fixed, total moment transfer between column and superstructure, and pinned, no moment transfer between column and substructure.

The study indicated that the optimum pier section consists of two columns supporting each roadway or a total of four columns at each pier. Each pier column was assumed to be supported by a single large diameter drilled shaft. The maximum column dimension was assumed to be 1 foot narrower than the diameter of the shaft. This provides a drilled shaft construction tolerance of plus or minus six inches.

The results of the pier study indicated that four or five span units could be supported on a 9-foot round pier supported on a 10-foot drilled shaft. These columns could be pinned at the top and still be stable for all conditions. Design of the column in the fully scoured condition will require a non-linear analysis to account for KL/r values in excess of 100.

The pier study was developed simultaneously with the constructability analysis. The 9-foot round column with a 10-foot diameter drilled shaft was found to require less steel than the 8-foot round column. In addition the balanced cantilever construction method required more reinforcing steel than did the span-by-span method of construction.

Hollow and segmental precast columns were also studied but the additional complications and expense and impact on schedules were felt to outweigh any savings in substructure dead loads.

For the column fixity analysis, only the 9-foot round column was investigated. This study indicated about 1.7% reinforcing steel was required for the fully fixed column. With the critical loading condition being dead load, plus creep, shrinkage and temperature on the non-scoured condition. The critical loading condition for the top pinned column was the fully scoured condition with moments magnified for slenderness effects. Moment magnification for the linear analyses was based on a theoretical p-delta analysis which considered geometric non-linearity using AASHTO's empirical relation for EI. This condition required 1.6% reinforcing steel in the column.

The piers are relatively slender and elastomeric bearing pads of two to three inch thickness are feasible for all but the expansion joint piers. Because of the flexibility of the column/shaft, superstructure movement demands can be provided as a combination of pier deflection and bearing pad shear movement. Preliminary analysis indicated the force required to shear (translate) an elastomeric bearing pad one inch was equal to the force required to deflect the pier top one inch. The validity of the preliminary design results will be verified in the final design when actual bearing movement requirements due to creep, elastic shortening and temperature are more accurately determined.

Abutments

The abutment locations are set to match the locations for the prestressed I-girder alternate. These locations are set to minimize the length of structure required, with embankment placed as near to riverbanks and ramps as possible. The west abutment is, therefore, a semi-deep type with retaining walls used to keep embankment from spilling into the Salt River and onto Ramp B. The east abutments are more shallow, requiring short wingwalls to retain the embankment.

During the superflood, the assumption is made that the embankment scours; however, the viaduct is to be otherwise unaffected by the flood. The abutment will, therefore, be designed as a free-standing column for this condition. For normal operating conditions, provisions will be required to resist horizontal earth loads produced by the approach roadway embankment. Consideration will be given to the use of horizontal earth tie-backs on the abutment and wing wall sections to reduce lateral bending of the columns.

The faces of abutments, wingwalls and retaining walls will be provided with a vertical, random pattern rustication which matches the vertical wall treatment of the rest of the East Papago Freeway.

Foundations

Extensive geotechnical data has been obtained for the design of the viaducts. This work was conducted by Sergent, Hauskins & Beckwith Engineers (SH&B) and Thomas-Hartig & Associates, Inc. and is summarized in the report, "Final Foundation Design Report - Salt River Bridge - East Papago Freeway Section 6" prepared by Thomas-Hartig. Test data includes drilling, sampling, laboratory testing, load testing and production drilling.

The Thomas-Hartig report characterizes the geological conditions as consisting of two generalized soil strata. The upper strata consists of stratified sandy gravels and cobbles. These soils are generally dense and contain scattered sand lenses. The lower strata is primarily stiff to hard, stratified silty clays and clayey silts, generally with low to medium plasticity. Scattered lenses of sandy silt, silty sand, clayey sand, and sand exist. This layer is generally calcareous, with intermittent light to heavy cementation. Sandy lenses and zones are more numerous toward the east end of the alignment.

The Thomas-Hartig report recommends the use of deep large diameter drilled shafts for the foundations of the Salt River Viaduct. The report provides allowable design curves for several different diameters of drilled shafts based on the axial load to the shaft. These curves allow

selection of different axial loads for the various zones along the viaduct. Zones correspond to those developed by SLA for the scour study, and are referenced in Table 3 of this report.

The allowable axial load curves from the Thomas-Hartig report were used to complete the studies required for this report. Tip elevations for nine, ten and eleven-foot diameter drilled shafts were calculated from the curves and were used to evaluate the substructure and superstructure elements of the viaduct.

Final design will be based on the ten-foot diameter drilled shafts. Founding tip elevations will be determined from the Thomas-Hartig curves using design loads. Additional studies, considering the use of 10-foot drilled shafts in the upper strata and 8-foot drilled shafts in the lower strata, will be developed, provided this information is made available by the geotechnical consultants. These studies will be performed in the second stage of this project and the results incorporated in the final design, where appropriate.

V. BRIDGE SUPERSTRUCTURE

Constructability Analysis

The design of a concrete segmental bridge is greatly influenced by the method of construction used to build the structure. Construction loads quite frequently are some of the largest the structure will have to carry. Construction loads also can be "locked into the structure" and become service loads when they cannot be removed. Age of concrete at the time of loading is also an important factor in determining stresses.

A constructability analysis was made to determine a feasible method of construction. This analysis considered time allowed for construction, cost of construction, and foundation conditions. Also considered in the analysis was the existence of conditions peculiar to the site which may affect the construction method. Vertical clearance over McClintock Drive and under the power lines was considered. Height above the river bed was a factor as was the possibility of high flows in the river during construction.

Cast-in-place construction was eliminated early in the analysis. Conventional cast-in-place box girder construction was not considered as the use of false work in the river was not to be permitted. Cast-in-place segmental construction, either balanced cantilever or span-by-span, was also not considered because it would be very difficult for this type of construction to meet the very limited construction timetable of 20 months.

Launching of the viaduct was also eliminated as a viable construction method. This type of construction requires the use of constant alignment, both horizontally and vertically. The use of intermediate launch points to accommodate the changes in vertical and horizontal alignment was not considered feasible because of the height of the viaduct, approximately 60 feet above the river. In addition, embankment and/or falsework for casting the segments and resisting the launching force, would have presented a similar obstruction to river flow as the falsework for cast-in-place construction.

The analysis, therefore, concentrated on the use of precast concrete. Two methods of construction were considered; balanced cantilever and span-by-span construction.

Cross Section

Concurrent with the constructability analysis was a study to determine the box girder configuration. Three box girder cross sections were investigated.

1. Each roadway supported by a two-cell box girder. Cross sections including web and top and bottom thicknesses were developed using minimum dimensions to conserve material and weight. Since the minimum section was found to be less economical than the two single-cell box girders, further study was not made to verify the assumed section.

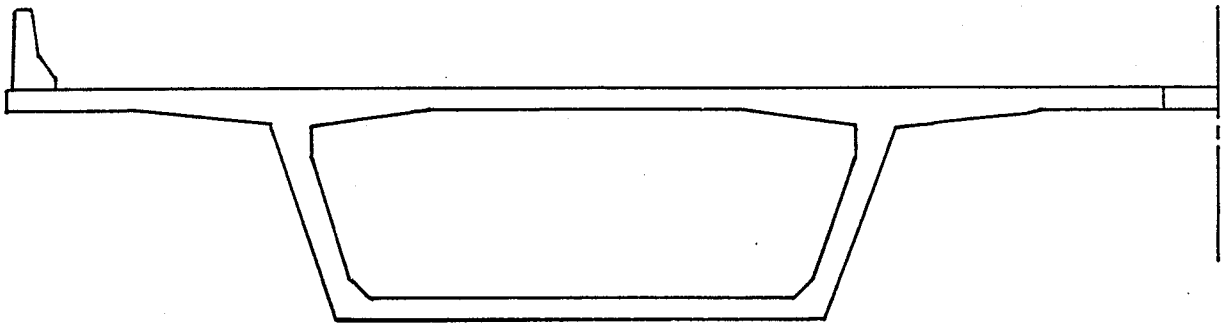
2. Each roadway supported by two single-cell box girders connected by a longitudinal closure pour. Longitudinal post-tensioning was assumed to be internal, completely encased in the concrete.

3. Each roadway supported by two single-cell box girders connected by a longitudinal closure pour. Longitudinal post-tensioning was assumed to be external, contained within the cell of the box and anchored at the pier diaphragms.

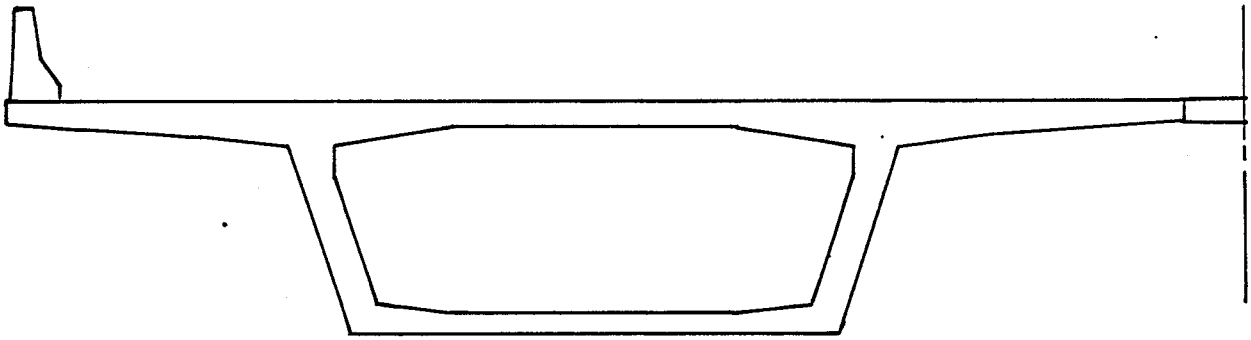
Sections of the box girders studied are shown in Figure 3. See also drawing number 10 for dimensions of the recommended alternate box section.

The study included a four span preliminary superstructure design for the first two box girder sections described above. In addition a preliminary substructure design, including foundations, was made for each study, to obtain total structure costs. A table of the design alternates studied is included in Table 4. Cost versus span curves developed from the studies are included in Appendix A. These cost curves are only to be used to evaluate the relative cost of each of the alternates studied.

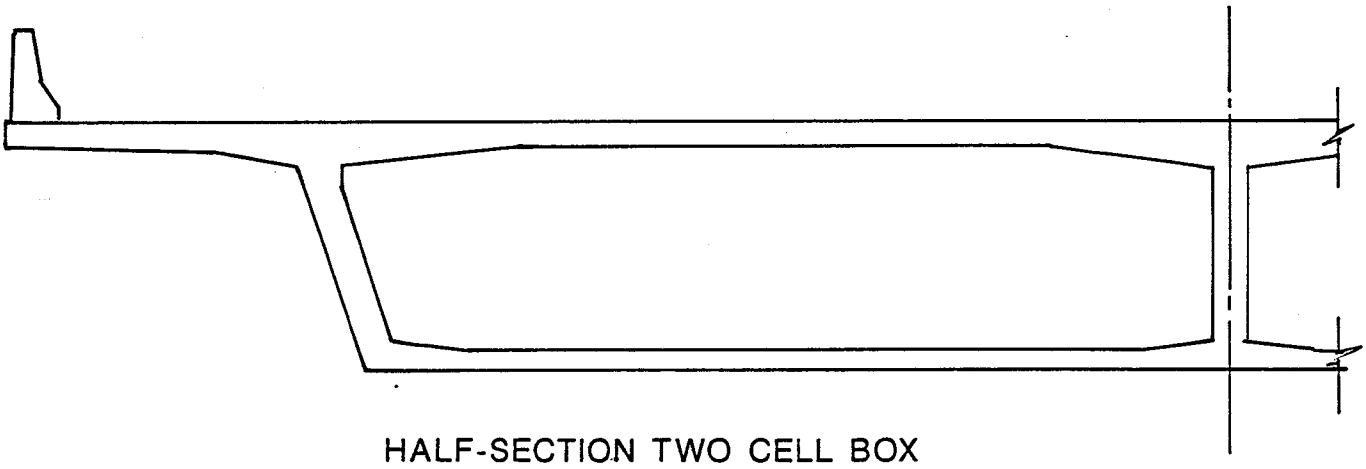
FIGURE 3



SINGLE CELL BOXES
EXTERNAL TENDONS



SINGLE CELL BOXES
INTERNAL TENDONS



HALF-SECTION TWO CELL BOX

ALTERNATE BOX GIRDER SECTIONS

The curves are based on the items that vary for each of the alternates. These curves do not include items that would be constant for each alternate, such as abutments, retaining walls, wearing surface, mobilization, etc. The curves were based on unit prices developed during the early phase of the project. The curves have not been revised to reflect unit prices used in the final cost estimates.

TABLE 4 - CONSTRUCTABILITY STUDY

	Span-by-Span		Balanced Cantilever
Spans	Two-Cell	Single Cell	Two-Cell
145	X	X	
170	X	X	X
190	X	X	X
220	X	X	X
250			X

For each of the above schemes, cost studies were prepared for both 10-foot diameter and 11-foot diameter drilled shafts. The 10-foot diameter drilled shafts were found to be more economical, as is shown in the cost curves in Appendix A.

VI. RECOMMENDATIONS

Design Construction Method

The recommended method of construction for the design of the Salt River Viaduct is to use 150-foot spans erected by the span-by-span method of construction. This method of construction is recommended for the following reasons:

1. Span-by-span is the most economical method of construction.

2. The 150-foot span is within the economical span range as shown in the curves presented in Appendix A.

3. The use of 150-foot spans with an eight-foot depth of structure should eliminate or minimize the need for thickening of the bottom flange.

4. The use of 150-foot spans minimizes the number of tendons required.

5. The optimum structure depth can be utilized without having to alter grades to the west, where the freeway has already been designed, and to the east where the complicated geometry of the Red Mountain Interchange exists.

6. The 150-foot span provides for potential use of existing trusses for erection of the bridge; however, the estimates have been based on the use of new trusses.

7. The use of 150-foot spans provides greater opportunity for alternative supporting systems.

8. The use of 150-foot spans is not new; therefore, contractors need not necessarily develop new techniques and construction schedule time can be more readily met.

Cross Section

The use of two single-cell box girders to support each roadway is recommended for economic reasons and for achieving the construction schedule. The cost curves presented in Appendix A show the two single-cell boxes to be more economical than the two-cell box. In addition, the ease of handling the lighter single-cell box girder in the fabrication yard and in erection will contribute to the Contractor's ability to meet the very short time schedule allocated for this project.

One of the more important aspects of the cross section is the riding surface. A thin latex modified concrete overlay, one and one half inches thick, is one possible wearing surface. This wearing surface is fairly expensive, $40 to $45 per square yard, and would add about $4,000,000 to the cost of the project. A major disadvantage of the overlay is that during construction it is very sensitive to atmospheric conditions, wind, heat and humidity, which would make placement of the overlay difficult in the Phoenix environment.

Construction in Florida has proven that smooth riding surfaces can be achieved in precast segmental construction. Texture of the precast segments is not always uniform, resulting in variations in tire noise from segment to segment. It is therefore recommended that the top reinforcing steel cover be increased to 2 3/4 inches within the nine inch top slab thickness. This will allow minor grinding and total transverse machine grooving to achieve a uniform wearing surface and texture.

The design of the bridge will include an allowance of twenty-five pounds per square foot future wearing surface.

Longitudinal Post-Tensioning

The use of external post-tensioning is becoming more and more common. It's use in France, where most major bridges are built with external tendons, is widespread. In the United States it has been used for ten years, and is being used in Belgium, Venezuela, Switzerland and Germany. Obviously a construction technique which has grown so popular in use and which has been tested and proven satisfactory in the laboratory must be given consideration.

Although the use of external post-tensioning goes back to at least 1936, its use has not met with overwhelming enthusiasm and acceptance. This reluctance to accept the external post-tensioning concept has been based on the justified conservative nature of bridge engineers. The bridge engineer wants to be sure that he is designing a structure that reacts linearly to service loads, has adequate strength, provides for serviceability, and preferably has redundant load paths.

Research conducted worldwide on bridges with unbonded tendons has done much to allay the fears of the bridge designer. Work conducted at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin, Research Report 365-3f, is of special interest. This research indicates the structure does behave linearly for service level loads and even for loads beyond the factored ultimate load. The research also indicated that the use of epoxied joints provided a safety factor of two against joint opening when the zero stress criteria was applied at service behavior.

For serviceability the Salt River Bridge is in a near-ideal environment for the use of external tendons. The bridge is located in an area of very low seismic probability. The mild winter climate of Phoenix eliminates the problems related to freeze-thaw and roadway de-icing by salt application. Phoenix has very good aggregates and a proven history of concrete construction, indicating sound durable concrete can be provided.

Providing adequate load paths in case of loss of a critical structural element is desirable. The integrity of the post-tensioning system must be maintained in a prestressed concrete box girder bridge. It is anticipated that a minimum of six tendons will be provided. Loss of all six tendons through a calamitous event, such as an earthquake, is highly unlikely. Loss of a single tendon, due to fatigue, anchorage failure, etc., also has a low degree of probability. However, the loss of a single tendon would be repairable as provisions for contingency tendon anchorages will be furnished. Tendon loss due to anchorage failure also has a built-in factor of safety. The external tendons are to be anchored

in diaphragms at the piers. These diaphragms, which are about eight feet thick, will provide bonding capacity to the tendon after the tendons are grouted.

One of the big advantages in the use of external post-tensioning is in the reduction in weight through the use of thinner webs. Weight savings represents savings in substructure costs which are about one half the total viaduct cost.

The use of external tendons also simplifies the placement of concrete and simplifies cable layout. This represents time savings to the Contractor indicating that the use of external tendons would be beneficial for meeting the construction schedule.

Based on the above advantages which represent savings in both dollars and time, the use of external post-tensioning is recommended for the Salt River Viaduct.

Joints

Three types of jointing procedures are proposed for the recommended structure.

1. Cast-in-place transverse closure joints with longitudinal post-tensioning which make make the pre-cast segments continuous spans.

2. Cast-in-place longitudinal closure joints with transverse post-tensioning which make the roadway continuous from gutterline to gutterline.

3. Joints between individual precast segments. All joints will be Type A joints.

The recommended cross section for the normal width roadway is two single-cell box girders. The recommended length for the precast segments is 10 feet. This length was selected considering time to cast segments, size of segment for transportation and weight of segment for placement.

A narrow cast-in-place closure pour between the pier segments and the span segments is proposed. This requires two transverse closure pours, of about 6 inches in length, to be placed in each span. These closure pours are placed to allow minor geometric adjustments of the precast elements. The closure pour also takes the pier segment out of the critical path as these segments are not match cast with the span segments.

The use of two single-cell box girders requires a longitudinal cast-in-place closure pour to make the roadway continuous. This closure pour will have mild reinforcing steel placed both transverse and longitudinal to the roadway. The transverse reinforcing steel will be lapped spliced with transverse reinforcing steel extending out of the segment cantilevers. Transverse post-tensioning through the closure pour is required to complete the continuity tie between the two boxes. This transverse post-tensioning will be integrated with transverse post-tensioning supplied in the casting yard for handling and erection stresses. The closure pour width will generally be four feet. This width allows for lap splicing of transverse reinforcing steel, tolerance for placement of transverse ducts between box sections, and connection of vertical misalignment between the two box girder sections. A variable width section is required between box sections in the roadway width transition sections at the east end of the viaduct.

The joint between precast segments shall have shear keys detailed in accordance with Article 25 of the Guide Specifications. One segment surface of the match cast joint shall be coated with epoxy to provide a complete and uniform bonding of all match cast surfaces.

Construction Schedule

A schedule for construction of the bridge in twenty months is included in Appendix B. This is a very tight schedule for construction of a bridge of the magnitude of the Salt River Viaduct. The schedule is dependent on the Contractor having access to the site immediately upon notice to proceed. The schedule assumes all right-of-way and clearances have been obtained before the contract is awarded.

The schedule anticipates an accelerated schedule by the Contractor, both in procuring equipment and in constructing the bridge. Equipment procurement, especially forms for casting segments, is a critical scheduling item. If an average of 8 segments per day is assumed and one ten-foot segment per form per day and a 24 work day month are assumed, then all forms must be at the site producing segments five months after notice to proceed to complete segment casting within 16 months of notice to proceed. If segment casting completion can be delayed until 18 months after notice to proceed, and four segments per day are cast for 13 months, production by the other four forms can be delayed up to five months, or ten months after notice to proceed.

Segment erection is also on the critical path. To achieve placement of all segments within 18 months, two sets of erection trusses are required. The first pair of trusses should be placing segments 6 months from notice to proceed. The second pair of trusses should be placing segments two weeks later.

Piers

Nine-foot round piers on ten-foot round drilled shafts are recommended. The use of elastomeric bearing pads is also recommended. The piers are transversely flared at the top to match the box girder and to allow maximum spread of the bearings.

Typical pier details are shown in the preliminary plans.

Construction Documents

To assure the Department that the Contractor has the means and equipment to meet the schedule additional items may be required in the Contractor's bid.

1. A statement of how the work will be executed.

2. A bar graph of the work items the bid is based on indicating start and finish dates for major items.

3. An equipment schedule.

4. An organizational chart including experience resumes of key personnel.

Cooperation between Owner, Contractor and Designer is a must for the schedule to be met. For a smoother handling of problems, a Dispute Review Board could be established to quickly and effectively handle major disputes. This may require the bid documents to be placed in escrow so that they are readily available for the Review Board to effectively resolve pay and work disputes.

The schedule for constructing the bridge is only 20 months long. Although this is a very tight timeframe for building the bridge, it is attainable provided construction progresses smoothly without interruptions. To provide an incentive to the Contractor to meet the equipment schedule, a pay item should be included to pay for equipment when it arrives at the site and is placed in production of either casting or erecting segments.

Approval of Contractor-produced drawings should also be expedited. One method of expediting shop drawing review is to provide a performance specification whereby the erection and equipment shop drawings are the Contractor's responsibility and acceptance of equipment is based on the equipment meeting certain performance specifications. Shop drawings for bridge structure elements would continue to be reviewed by the Designer.

Construction Sequence

It is recommended that the final design assume the following erection sequence:

Construction to start at the west abutment of the eastbound roadway and proceed eastward. This allows clearance of the hazardous waste sites to the west to occur simultaneously with the bridge construction. The first box girder segments to be placed would be the inside box or the box adjacent to the construction centerline. The second box girder placement should follow two or more spans behind.

If possible, the eastbound construction should be completed before starting the westbound. Upon completion of the exterior box girder segment, one pair of trusses should be removed for beginning placement of the inside segments of the westbound roadway. The other pair of trusses should place the center segments in the flared roadway portion.

Should the hazardous waste sites not be cleared in time for segment placement, the eastbound roadway should be constructed as far as possible. Construction should then start on the westbound superstructure and proceed to the east abutment before completing the eastbound road.

Cost Estimates

Two estimates of the probable cost of construction have been prepared. Table 5 gives the estimated cost of construction, assuming the viaduct is constructed using internal tendons. Table 6 gives the estimated cost of construction for the recommended viaduct, assuming the viaduct is constructed using external tendons. The tables are based on the best estimate of unit prices for the various bid items using present cost conditions. No provision has been made for increasing the unit prices to account for inflation between September, 1990 and when the project will be bid. Also included are two sheets showing quantities factors used to determine conventional and post-tensioning steel quantities.

The cubic yard price of $290.00 for box segment concrete has been retained since it was arrived at by using labor, material and erection costs. The total cost per cubic yard for the superstructure including concrete, mild steel reinforcing, transverse and longitudinal post-tensioning and erection is equivalent to $423 per cubic yard. It should also be noted that there is a large body of local pricing experience which substantiates the reinforcing prices used in this report.

GCW/lbm/DS
14244.5

QUANTITIES FACTORS

INTERNAL TENDONS:

Box Segment :	Ax=65.60 sf (Mainline sec)
	Ax=57.10 sf (ramp sec)
	Reinforcement=9.52 lb/sf
	Longitudinal P.T.= 3.34 lb/sf
	Transverse P.T.=0.98 lb/sf
Pier Diaphragm:	Concrete=10.0 cy
	Post-Tensioning=262 lb
	Reinforcement=2000 lb
Deviation Block:	Concrete=0.72 cy/each
	Reinforcement=236 lb/cy
Closure Pour:	Reinforcement=160 lb/cy
	Post-Tensioning=0.98 lb/sf

TABLE 5 Estimate of Probable Construction Cost
Segments with Internal Tendons
Span-by-Span Construction
September, 1990

ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
SUPERSTRUCTURE				
Box Segment Concrete	CY	53,850	$290.00	$15,616,500
Diaphragm Concrete	CY	1,410	$290.00	$408,900
Deviation Block Concrete	CY	215	$290.00	$62,350
Closure Concrete	CY	1,420	$200.00	$284,000
Reinforcing Steel	LB	8,973,000	$0.40	$3,589,200
Post-tensioning Steel	LB	3,857,900	$1.00	$3,857,900
Wearing Surface	SY	96,754	$4.80	$464,419
Barrier Concrete	CY	2,040	$215.00	$438,600
Expansion Joints	LF	1,870	$600.00	$1,122,000
Bearings	EACH	326	$500.00	$163,000
Vertical Restrainers	EACH	978	$75.00	$73,350
Total Superstructure				$26,080,219
SUBSTRUCTURE				
Structural Excavation	CY	1,360	$10.00	$13,600
Structure Backfill	CY	9,530	$15.00	$142,950
Column Concrete	CY	21,360	$200.00	$4,272,000
Abutment Concrete	CY	775	$190.00	$147,250
Reinforcing Steel	LB	5,009,000	$0.35	$1,753,150
Drilled Shafts (10' dia)	LF	18,520	$700.00	$12,964,000
Drilled Shafts (6' dia)	LF	3,015	$350.00	$1,055,250
Wall Concrete	CY	625	$190.00	$118,750
Approach Slab Concrete	CY	180	$180.00	$32,400
Anchor Slab Concrete	CY	810	$180.00	$145,800
Total Substructure				$20,645,150
Subtotal				$46,725,369
Mobilization (5%)				$2,336,270
Contingencies (15%)				$7,359,250
TOTAL ESTIMATED COST				$56,420,889

Area, square feet	914810
Superstructure cost	$34.42
Substructure Cost	$27.25
Total cost	$61.67

Solid Column

QUANTITIES FACTORS

EXTERNAL TENDONS:

Box Segment :	Ax=57.70 sf (Mainline sec)
	Ax=49.20 sf (ramp sec)
	Reinforcement=8.64 lb/sf
	Longitudinal P.T.= 2.85 lb/sf
	Transverse P.T.=1.18 lb/sf
Pier Diaphragm:	Concrete=22.2 cy
	Post-Tensioning=525 lb
	Reinforcement=4440 lb
Deviation Block:	Concrete=1.58 cy/each
	Reinforcement=200 lb/cy
Closure Pour:	Reinforcement=160 lb/cy
	Post-Tensioning=1.18 lb/sf

TABLE 6 Estimate of Probable Construction Cost
Segments with External Tendons
Span-by-Span Construction
September, 1990

ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
SUPERSTRUCTURE				
Box Segment Concrete	CY	47,330	$290.00	$13,725,700
Diaphragm Concrete	CY	3,140	$290.00	$910,600
Deviation Block Concrete	CY	470	$290.00	$136,300
Closure Concrete	CY	1,396	$200.00	$279,200
Reinforcing Steel	LB	8,597,000	$0.40	$3,438,800
Post-tensioning Steel	LB	3,631,700	$1.00	$3,631,700
Wearing Surface	SY	96,754	$4.80	$464,419
Barrier Concrete	CY	2,040	$215.00	$438,600
Expansion Joints	LF	1,870	$600.00	$1,122,000
Bearings	EACH	326	$500.00	$163,000
Vertical Restrainers	EACH	978	$75.00	$73,350
Total Superstructure				$24,383,669
SUBSTRUCTURE				
Structural Excavation	CY	1,360	$10.00	$13,600
Structure Backfill	CY	9,530	$15.00	$142,950
Column Concrete	CY	21,360	$200.00	$4,272,000
Abutment Concrete	CY	775	$190.00	$147,250
Reinforcing Steel	LB	5,009,000	$0.35	$1,753,150
Drilled Shafts (10' dia)	LF	18,000	$700.00	$12,600,000
Drilled Shafts (6' dia)	LF	3,015	$360.00	$1,085,400
Wall Concrete	CY	625	$190.00	$118,750
Approach Slab Concrete	CY	180	$180.00	$32,400
Anchor Slab Concrete	CY	810	$180.00	$145,800
Total Substructure				$20,311,300
Subtotal				$44,694,969
Mobilization (5%)				$2,234,750
Contingencies (15%)				$7,039,460
TOTAL ESTIMATED COST				$53,969,179

Area, square feet	914810
Superstructure cost,per square foot	$32.19
Substructure Cost, per square foot	$26.81
Total Cost, per square foot	$58.99

Solid Column

TABLE 6 Estimate of Probable Construction Cost
Segments with External Tendons
Span-by-Span Construction
September, 1990

ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
SUPERSTRUCTURE				
Box Segment Concrete	CY	47,330	$290.00	$13,725,700
Diaphragm Concrete	CY	3,140	$290.00	$910,600
Deviation Block Concrete	CY	470	$290.00	$136,300
Closure Concrete	CY	1,396	$200.00	$279,200
Reinforcing Steel	LB	8,597,000	$0.40	$3,438,800
Post-tensioning Steel	LB	3,631,700	$1.00	$3,631,700
Wearing Surface	SY	96,754	$4.80	$464,419
Barrier Concrete	CY	2,040	$215.00	$438,600
Expansion Joints	LF	1,870	$600.00	$1,122,000
Bearings	EACH	326	$500.00	$163,000
Vertical Restrainers	EACH	978	$75.00	$73,350
Total Superstructure				$24,383,669
SUBSTRUCTURE				
Structural Excavation	CY	1,360	$10.00	$13,600
Structure Backfill	CY	9,530	$15.00	$142,950
Column Concrete	CY	17,375	$200.00	$3,475,000
Abutment Concrete	CY	775	$190.00	$147,250
Reinforcing Steel	LB	5,009,000	$0.35	$1,753,150
Drilled Shafts (10' dia)	LF	17,845	$700.00	$12,491,500
Drilled Shafts (6' dia)	LF	3,015	$360.00	$1,085,400
Wall Concrete	CY	625	$190.00	$118,750
Approach Slab Concrete	CY	180	$180.00	$32,400
Anchor Slab Concrete	CY	810	$180.00	$145,800
Total Substructure				$19,405,800
Subtotal				$43,789,469
Mobilization (5%)				$2,189,470
Contingencies (15%)				$6,896,840
TOTAL ESTIMATED COST				$52,875,779

Area, square feet	914810
Superstructure cost,per square foot	$32.19
Substructure Cost, per square foot	$25.61
Total Cost, per square foot	$57.80

5' Hollow Column

TABLE 6 Estimate of Probable Construction Cost
Segments with External Tendons
Span-by-Span Construction
September, 1990

ITEM	UNIT	QUANTITY	UNIT PRICE	AMOUNT
SUPERSTRUCTURE				
Box Segment Concrete	CY	47,330	$290.00	$13,725,700
Diaphragm Concrete	CY	3,140	$290.00	$910,600
Deviation Block Concrete	CY	470	$290.00	$136,300
Closure Concrete	CY	1,396	$200.00	$279,200
Reinforcing Steel	LB	8,597,000	$0.40	$3,438,800
Post-tensioning Steel	LB	3,631,700	$1.00	$3,631,700
Wearing Surface	SY	96,754	$4.80	$464,419
Barrier Concrete	CY	2,040	$215.00	$438,600
Expansion Joints	LF	1,870	$600.00	$1,122,000
Bearings	EACH	326	$500.00	$163,000
Vertical Restrainers	EACH	978	$75.00	$73,350
Total Superstructure				$24,383,669
SUBSTRUCTURE				
Structural Excavation	CY	1,360	$10.00	$13,600
Structure Backfill	CY	9,530	$15.00	$142,950
Column Concrete	CY	17,375	$260.00	$4,517,500
Abutment Concrete	CY	775	$190.00	$147,250
Reinforcing Steel	LB	5,009,000	$0.35	$1,753,150
Drilled Shafts (10' dia)	LF	17,845	$700.00	$12,491,500
Drilled Shafts (6' dia)	LF	3,015	$360.00	$1,085,400
Wall Concrete	CY	625	$190.00	$118,750
Approach Slab Concrete	CY	180	$180.00	$32,400
Anchor Slab Concrete	CY	810	$180.00	$145,800
Total Substructure				$20,448,300
Subtotal				$44,831,969
Mobilization (5%)				$2,241,600
Contingencies (15%)				$7,061,040
TOTAL ESTIMATED COST				$54,134,609

Area, square feet	914810
Superstructure cost,per square foot	$32.19
Substructure Cost, per square foot	$26.99
Total Cost, per square foot	$59.18

5' Hollow Column

APPENDIX A

ASSUMED UNIT PRICES
(For Span Optimization Curves)

Superstructure Concrete: (Includes Erection Equipment)

	Span-By-Span	Balanced Cantilever
145'	$290/cy	
170'	$300/cy	$320/cy
195'	$310/cy	$325/cy
220'	$330/cy	$330/cy
250'		$335/cy

Substructure Concrete: $200/cy

Post-Tensioning: $1.30/lb

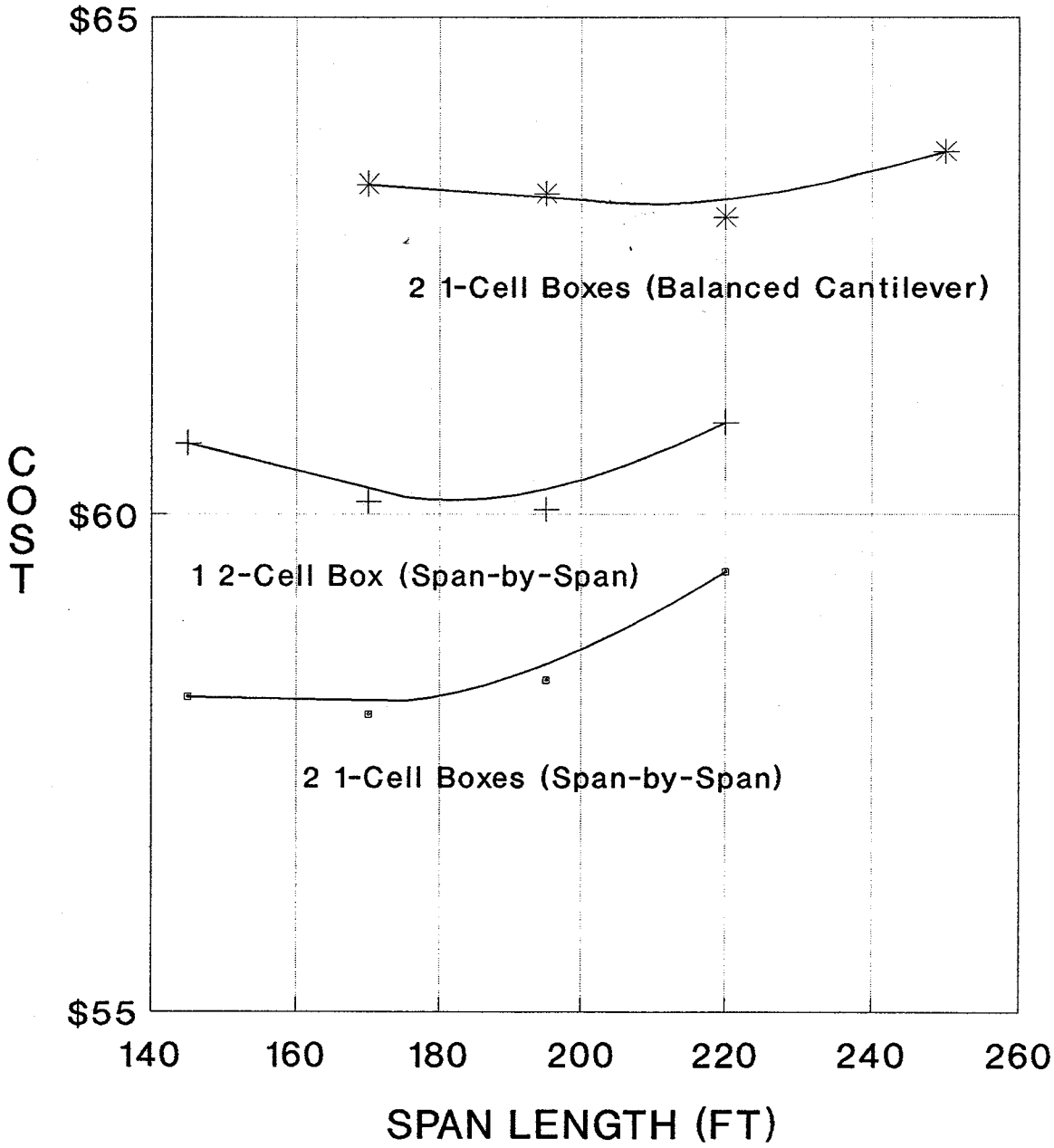
Reinforcement: $0.40/lb

10' Dia. Drilled Shafts: $985.0

Barrier Curbs: $30.00/lf

TOTAL STRUCTURE COST

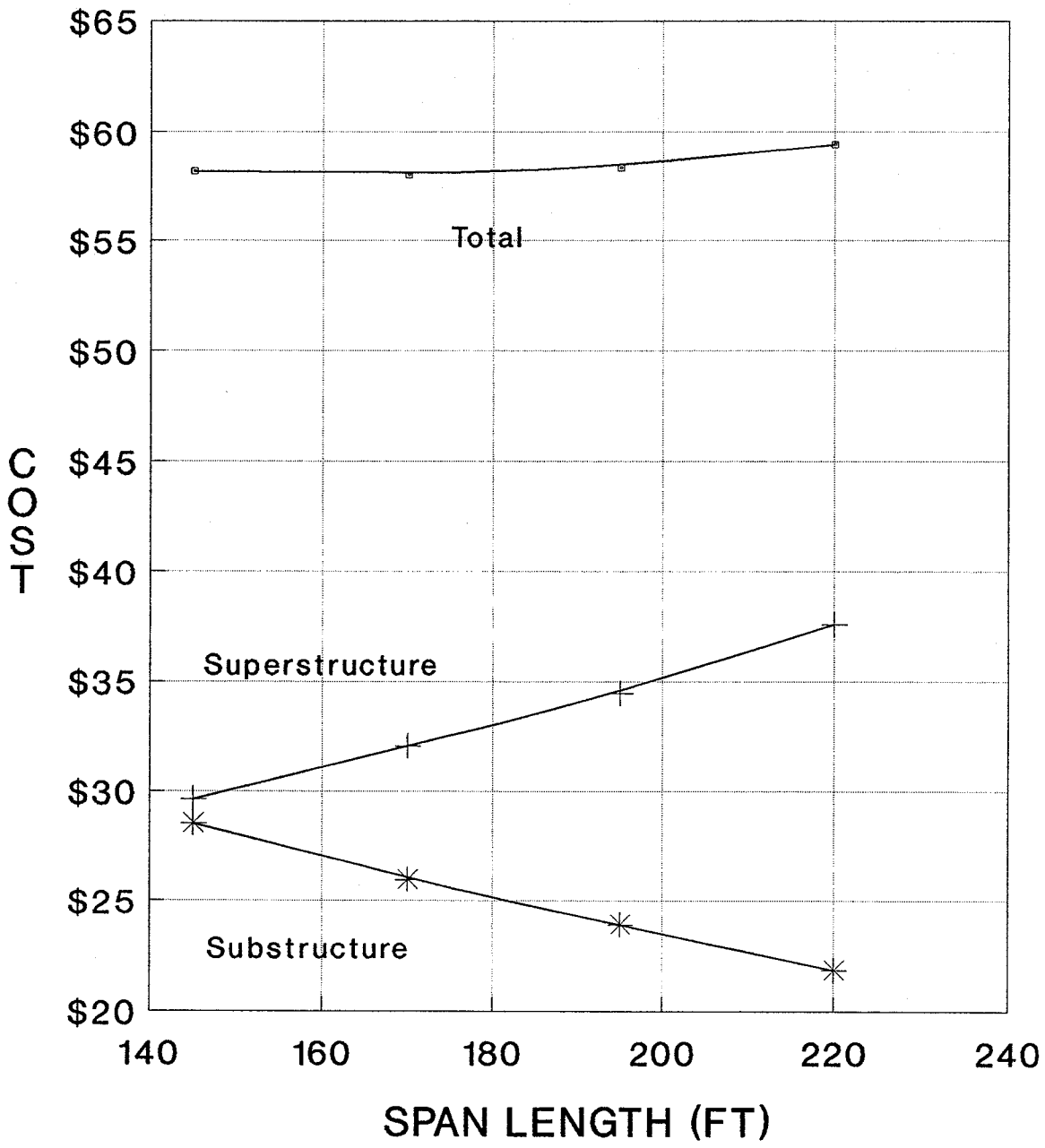
Segmental Concrete Alternates



10 FT DIAMETER DRILLED SHAFTS

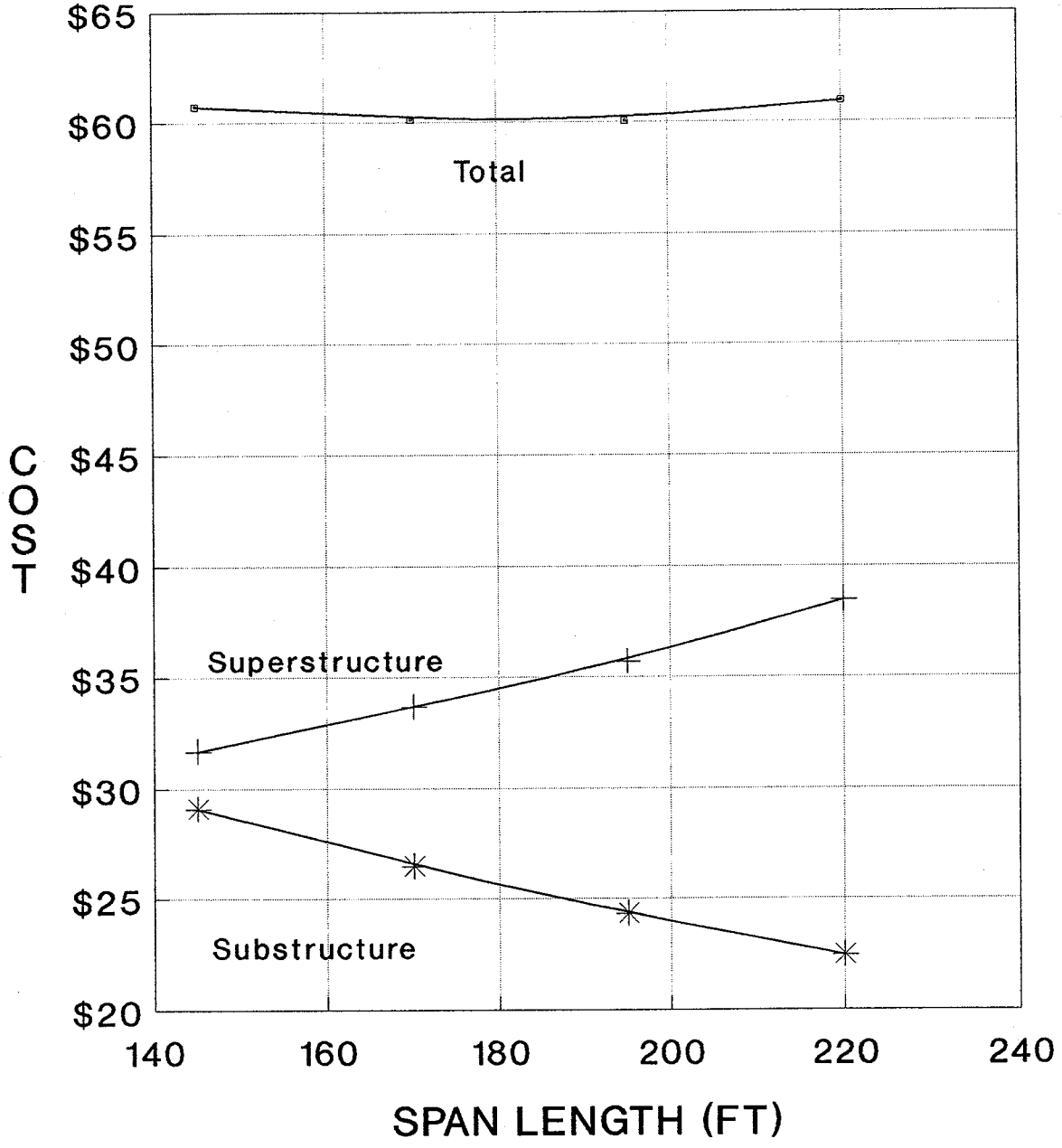
2 1-Cell Boxes (Span-By-Span)

Structure Cost Summary



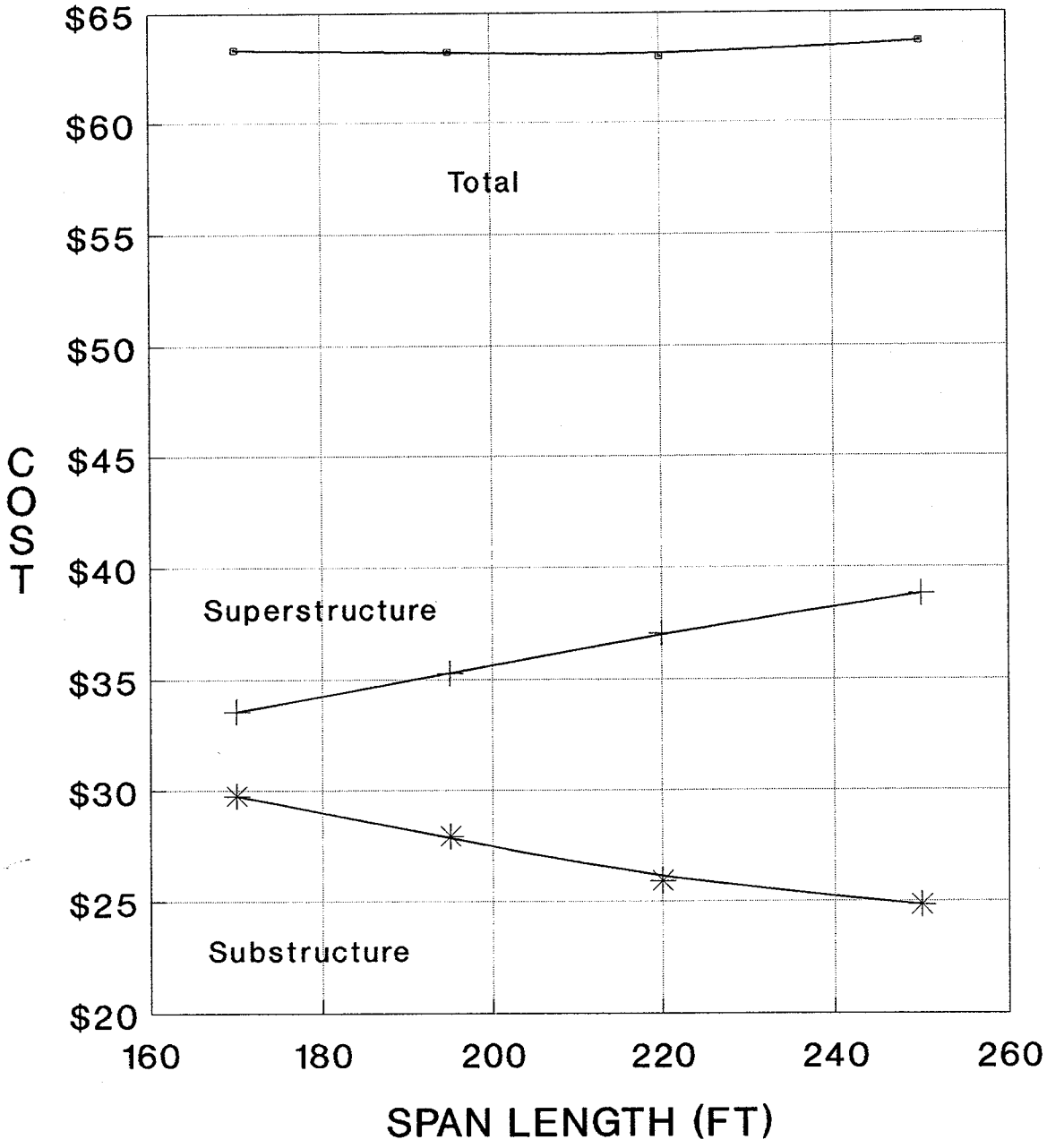
10 FT DIAMETER DRILLED SHAFTS

1 2-Cell Box (Span-By-Span)
Structure Cost Summary



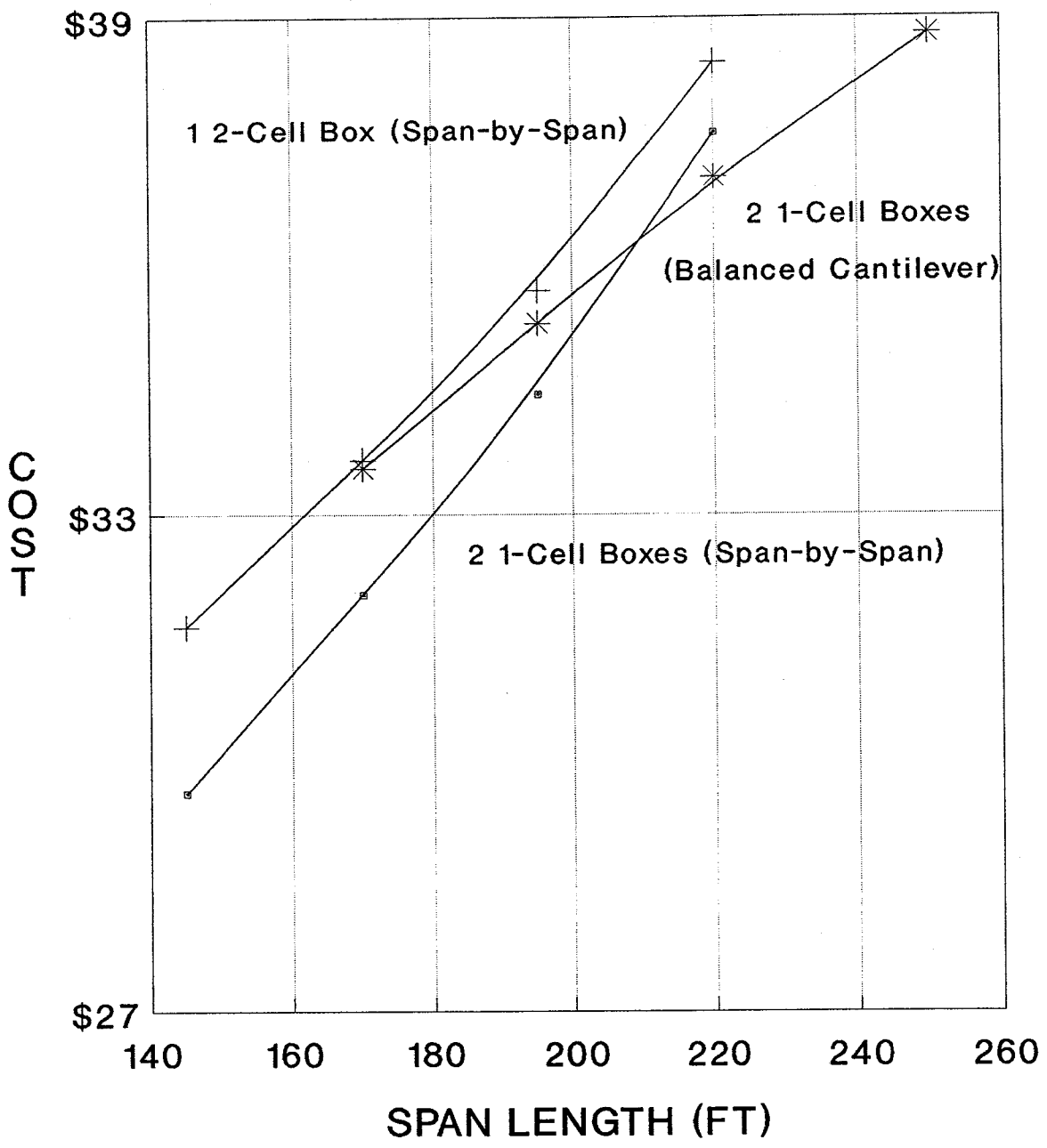
10 FT DIAMETER DRILLED SHAFTS

2 1-Cell Boxes (Balanced Cantilever)
Structure Cost Summary

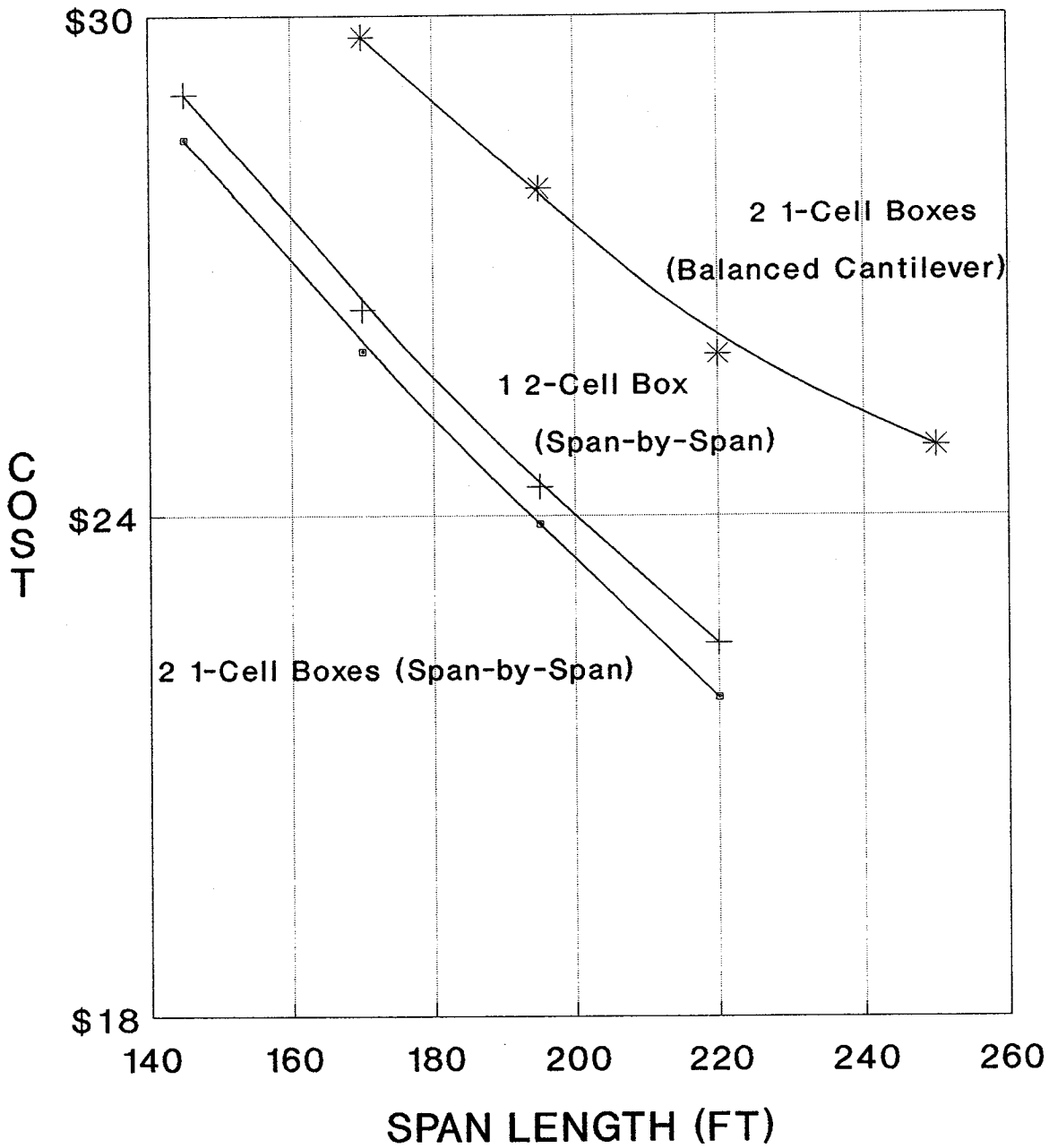


10 FT DIAMETER DRILLED SHAFTS

SUPERSTRUCTURE COST
Segmental Concrete Alternates



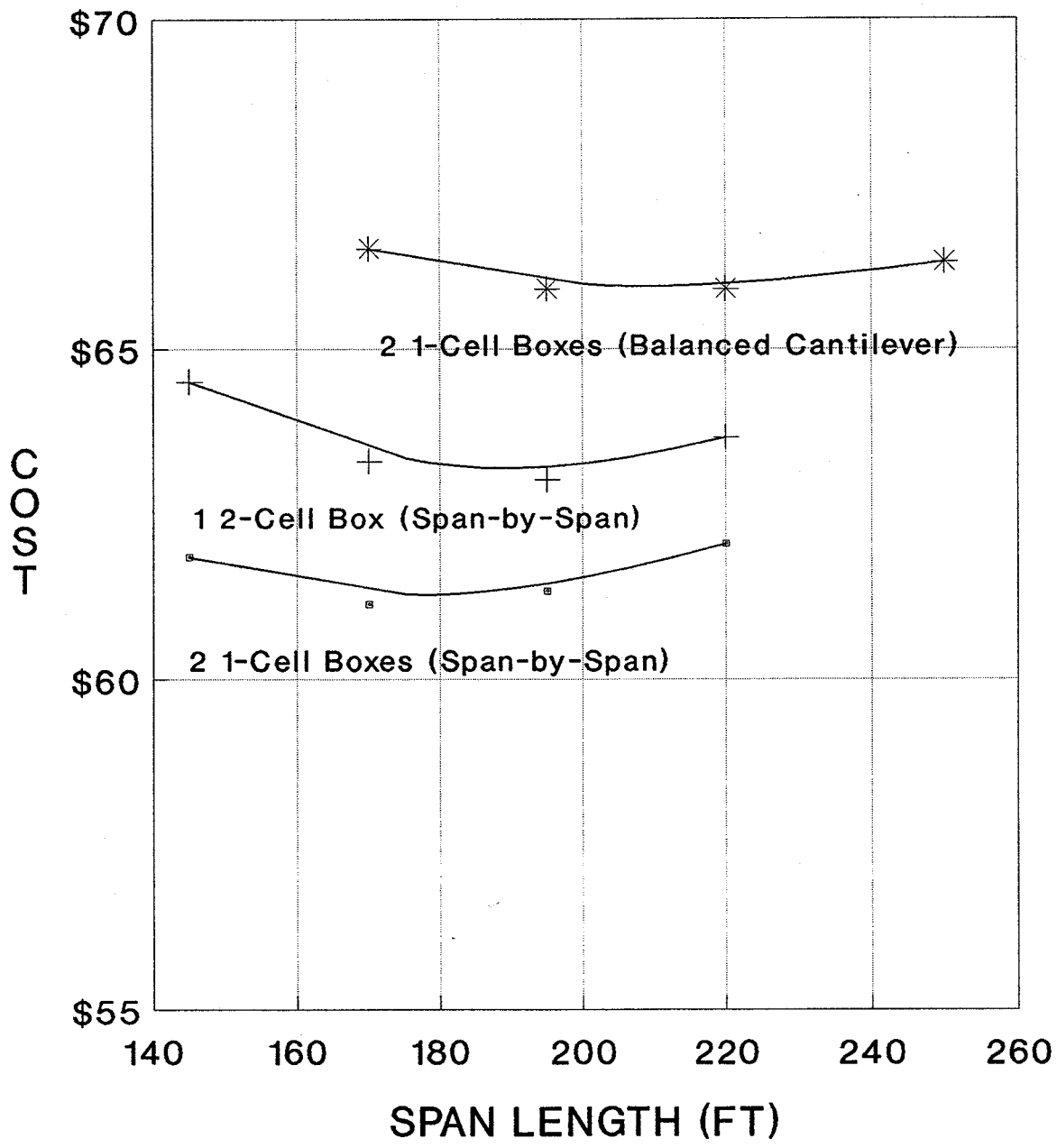
SUBSTRUCTURE COST
Segmental Concrete Alternates



10 FT DIAMETER DRILLED SHAFTS

TOTAL STRUCTURE COST

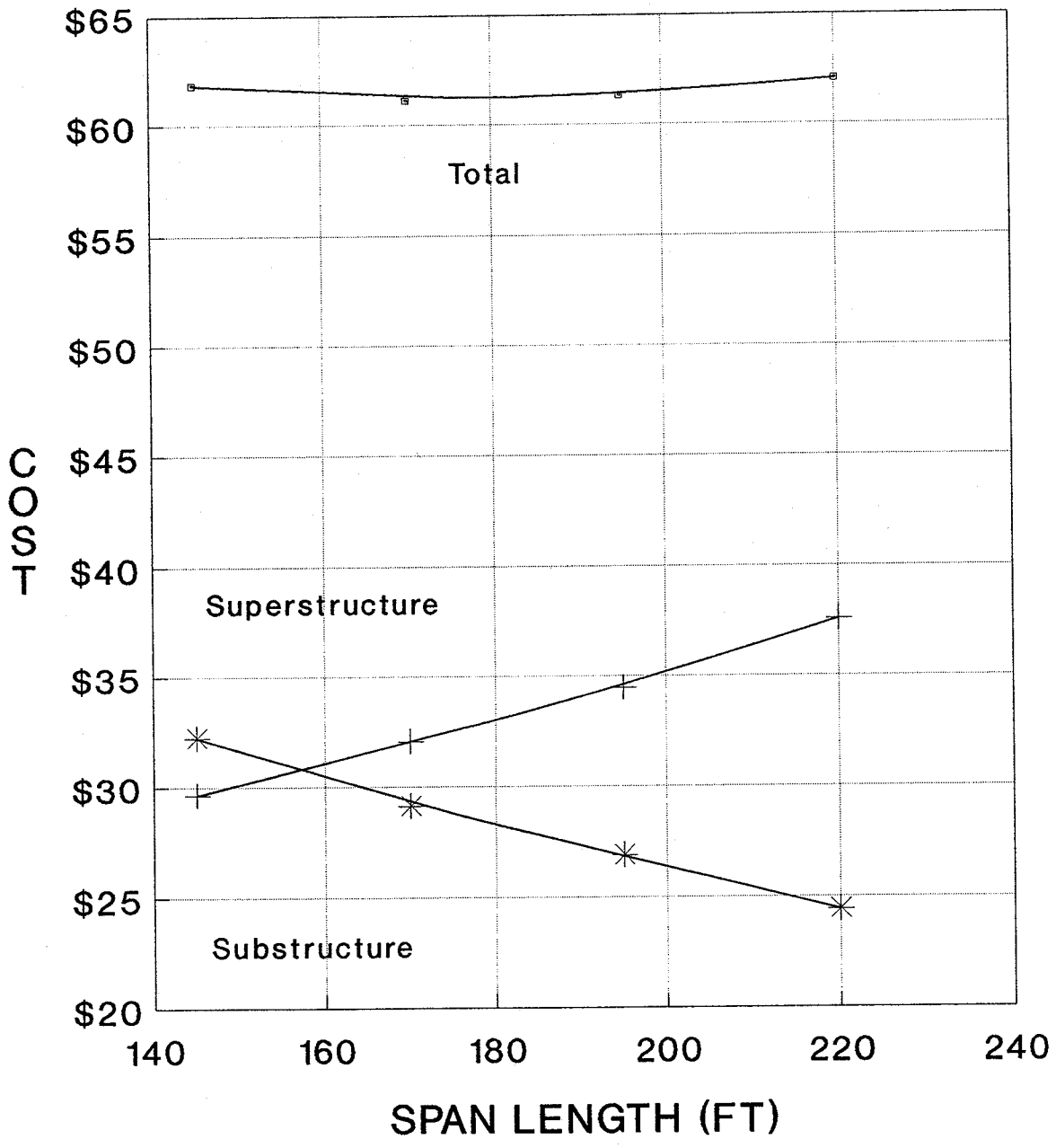
Segmental Concrete Alternates



11 FT DIAMETER DRILLED SHAFTS

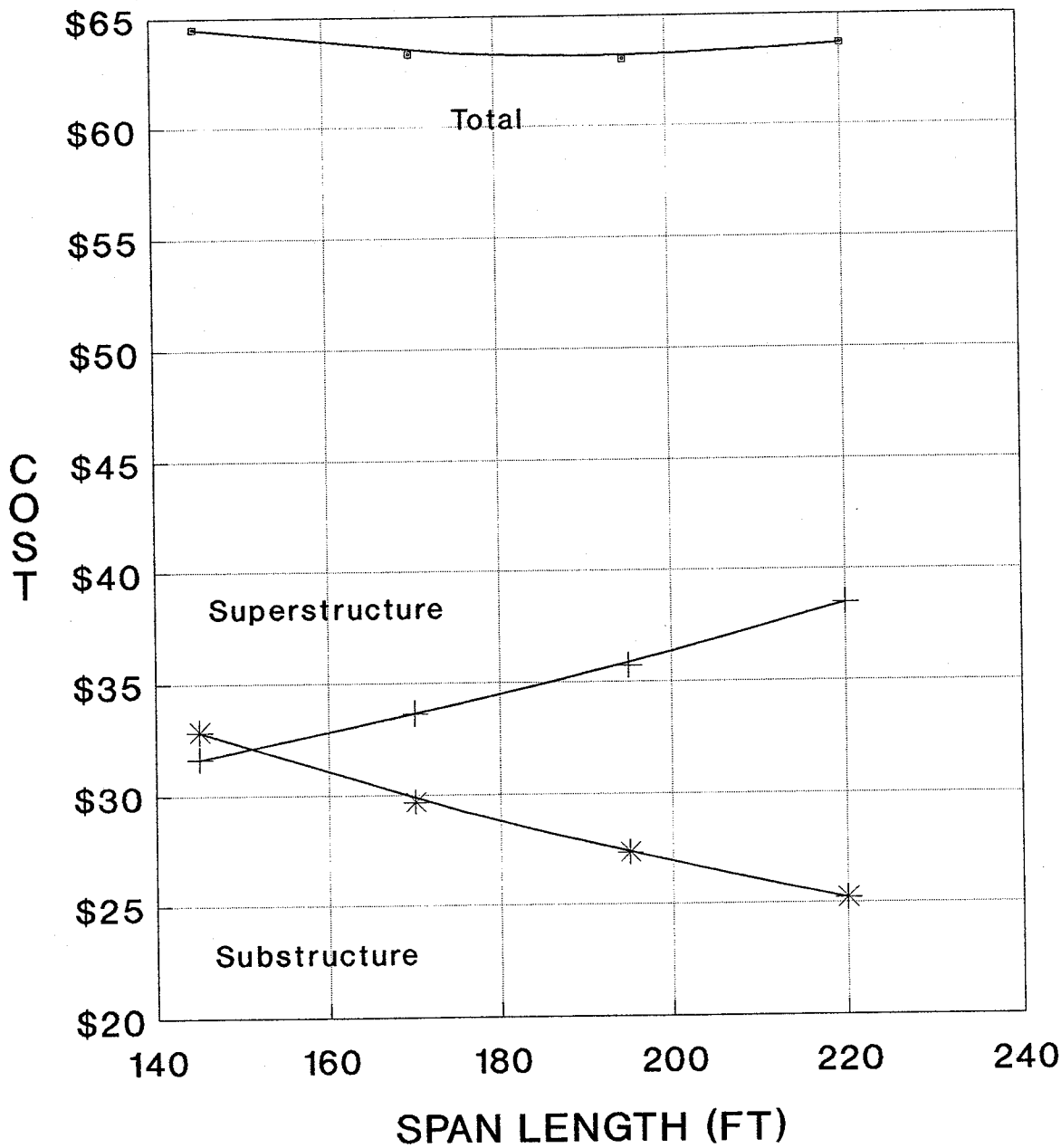
2 1-Cell Boxes (Span-By-Span)

Structure Cost Summary



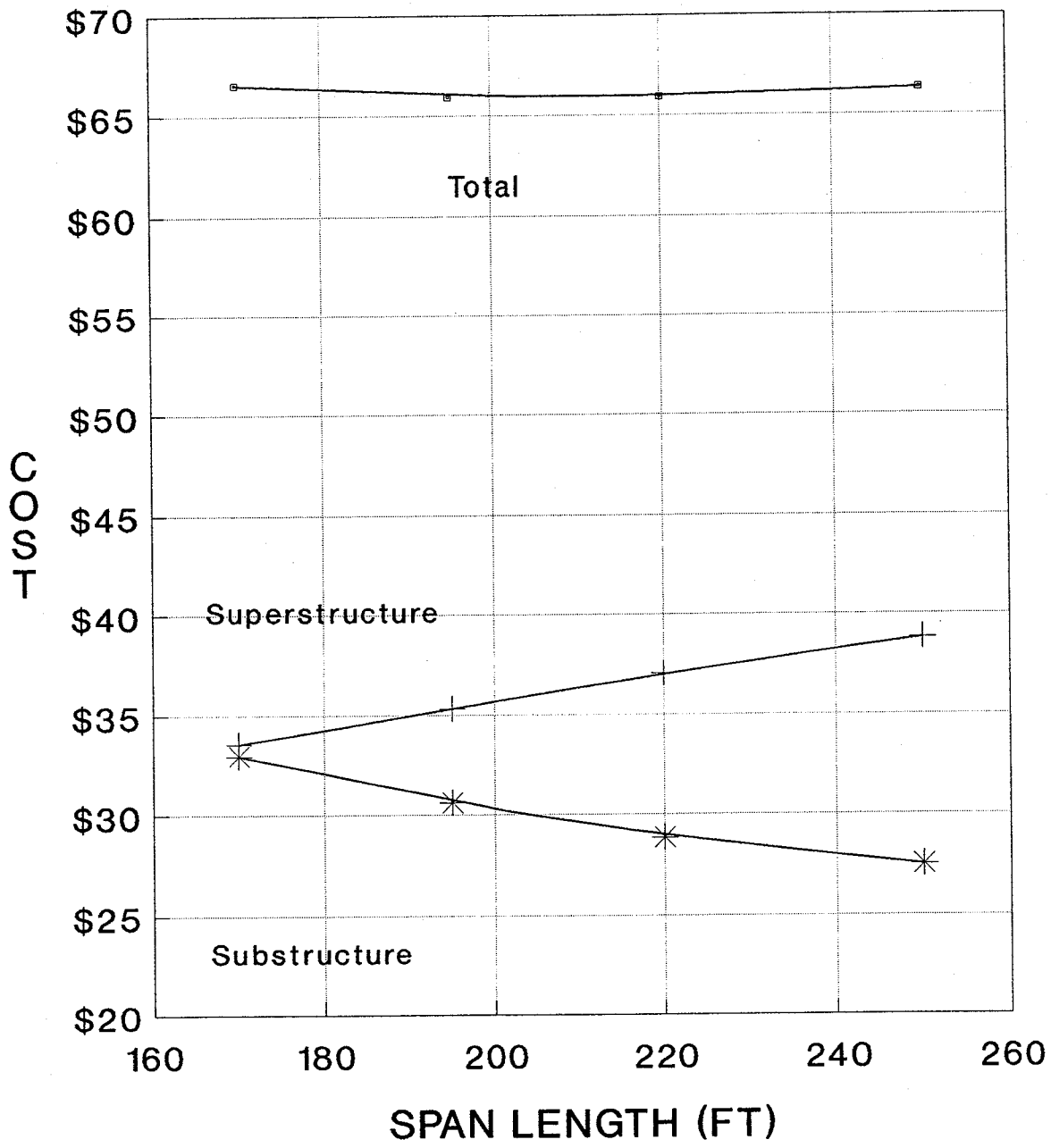
11 FT DIAMETER DRILLED SHAFTS

1 2-Cell Box (Span-By-Span)
Structure Cost Summary



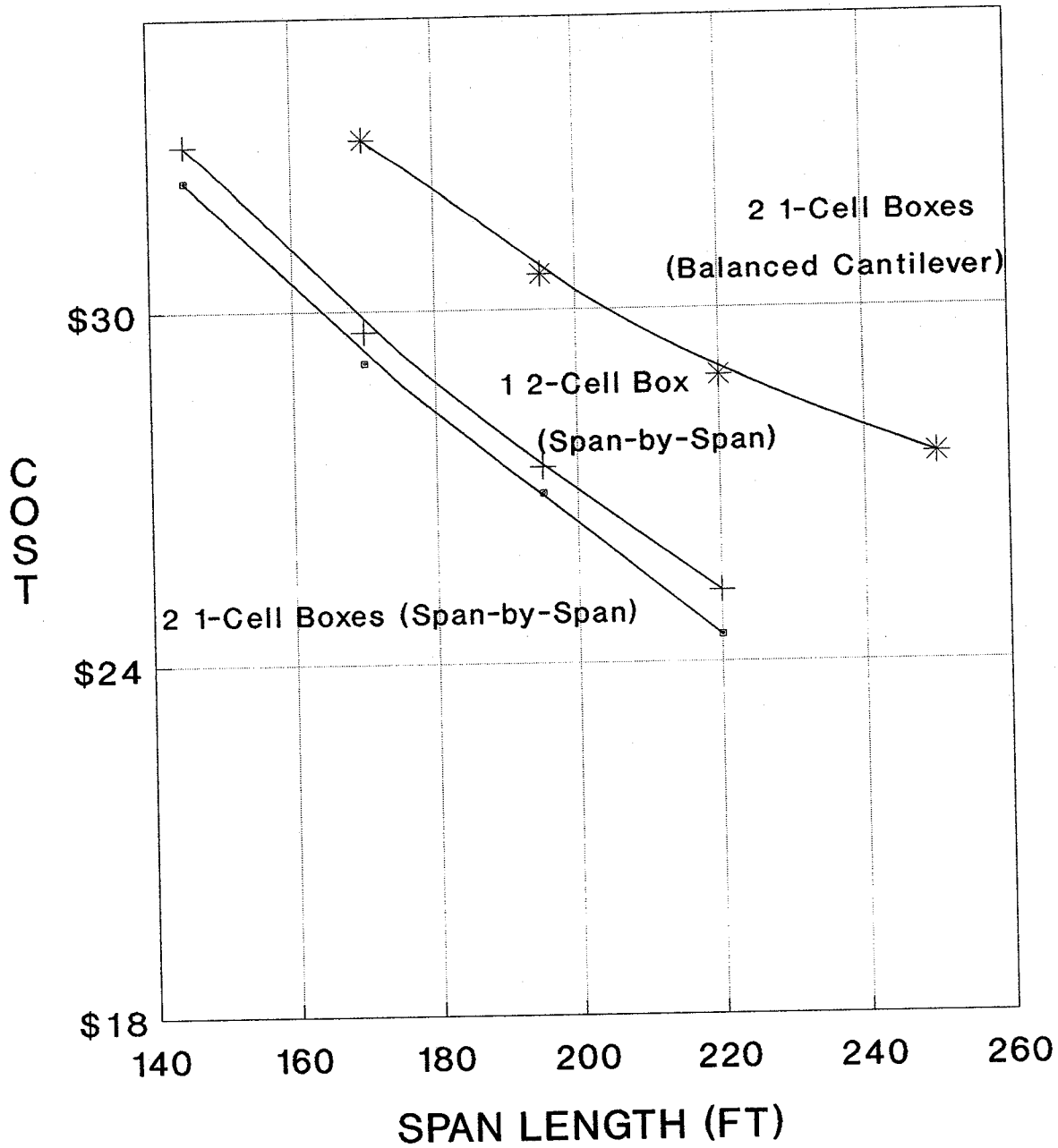
11 FT DIAMETER DRILLED SHAFTS

2 1-Cell Boxes (Balanced Cantilever)
Structure Cost Summary



11 FT DIAMETER DRILLED SHAFTS

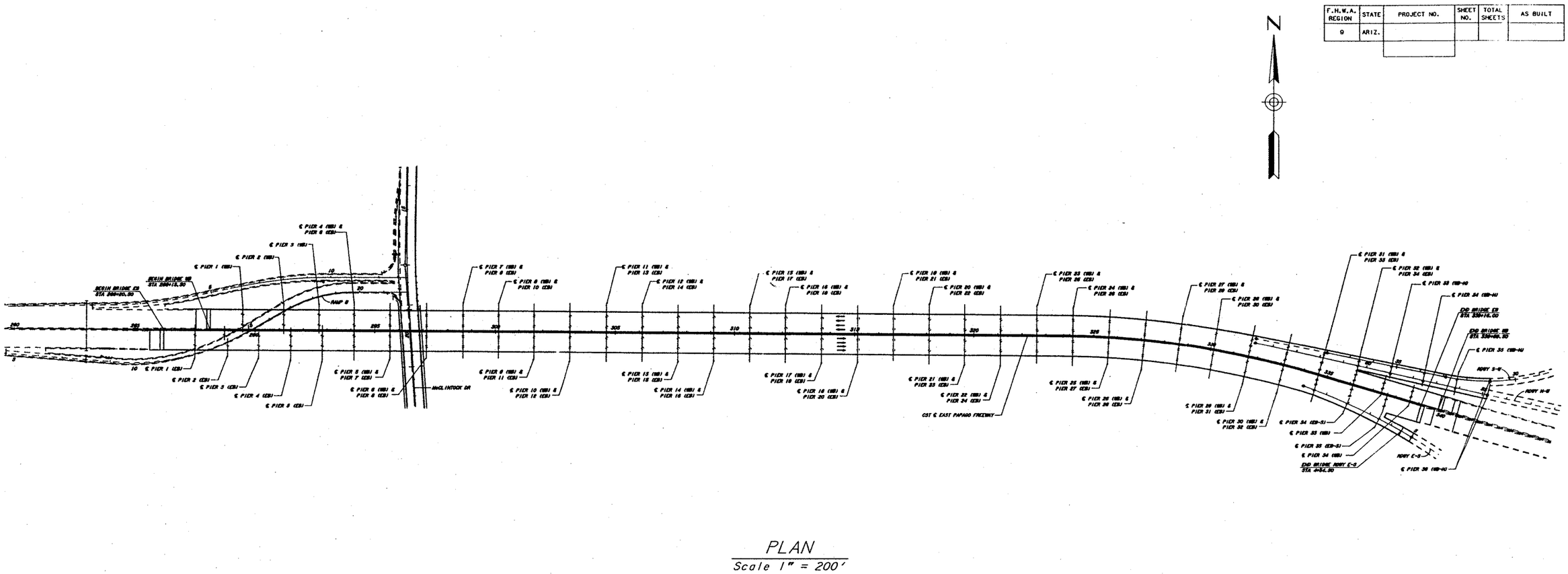
SUBSTRUCTURE COST
Segmental Concrete Alternates



11 FT DIAMETER DRILLED SHAFTS

APPENDIX B

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN

Scale 1" = 200'

36 SPAN EB, 35 SPAN WB PRESTRESSED SEGMENTAL BOX GIRDER BRIDGE

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

GENERAL PLAN

NOT FOR CONSTRUCTION

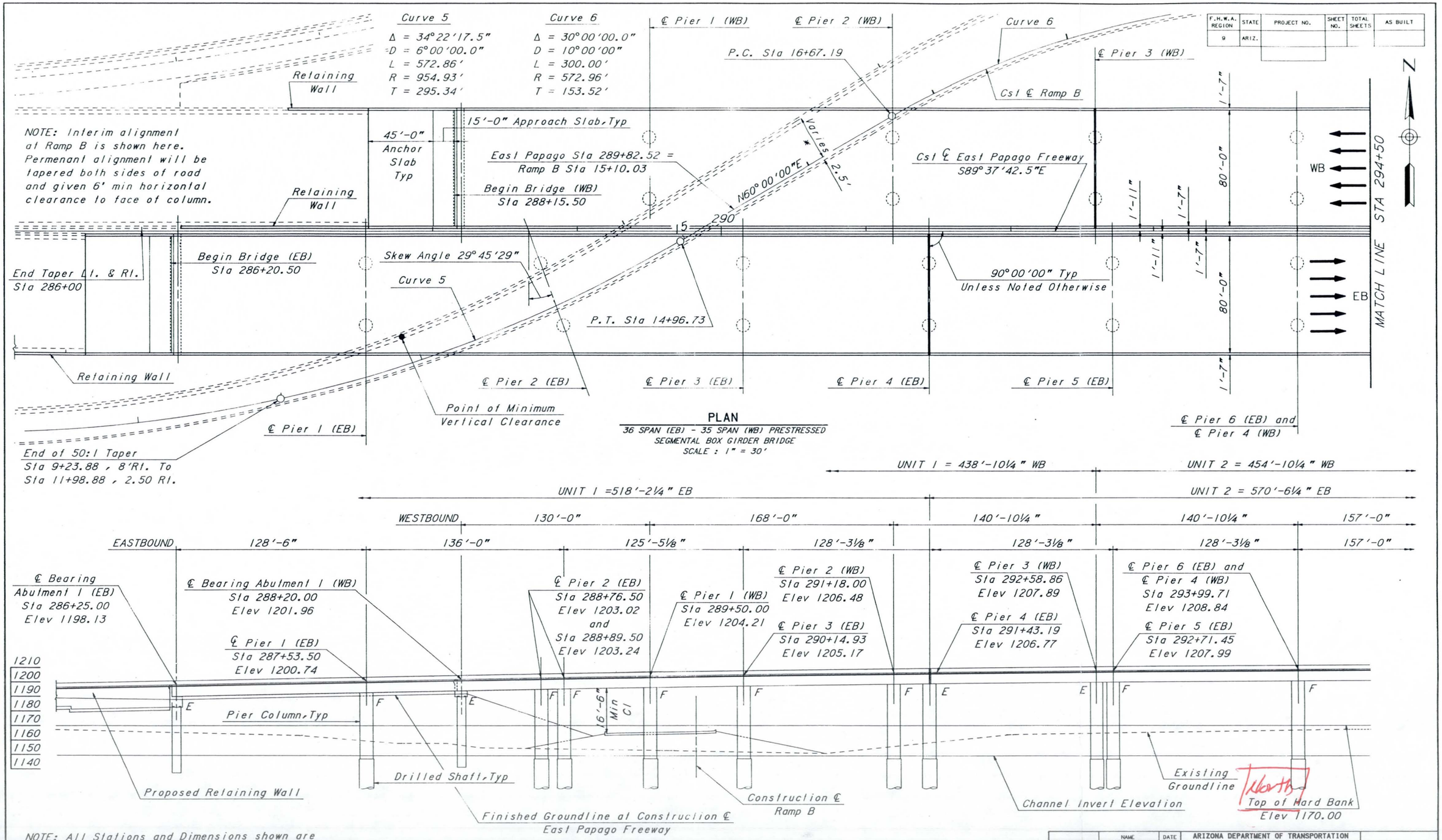
LOOP 202 | ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

BRIDGE ENGINEER

DWG. NO. 1

___ OF ___

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
36 SPAN (EB) - 35 SPAN (WB) PRESTRESSED SEGMENTAL BOX GIRDER BRIDGE
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

ELEVATION
SCALE : 1" = 30'

	NAME	DATE
DESIGN	MM	9-90
DRAWN	EMG	9-90
CHECKED	TK	9-90

HNTB

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

NOT FOR CONSTRUCTION

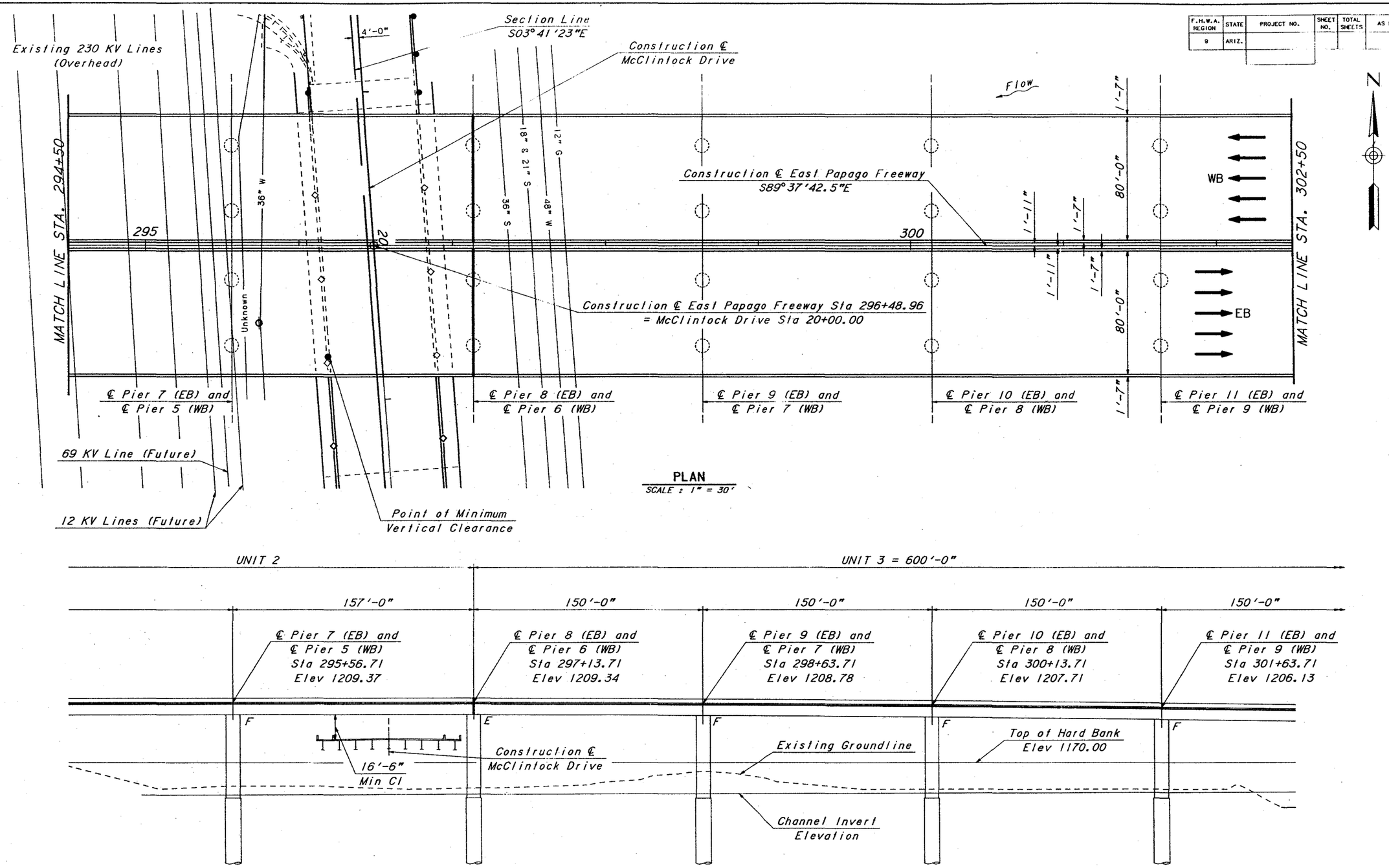
BRIDGE ENGINEER

DWG. NO. 2

___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'

ELEVATION
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

NOT FOR CONSTRUCTION

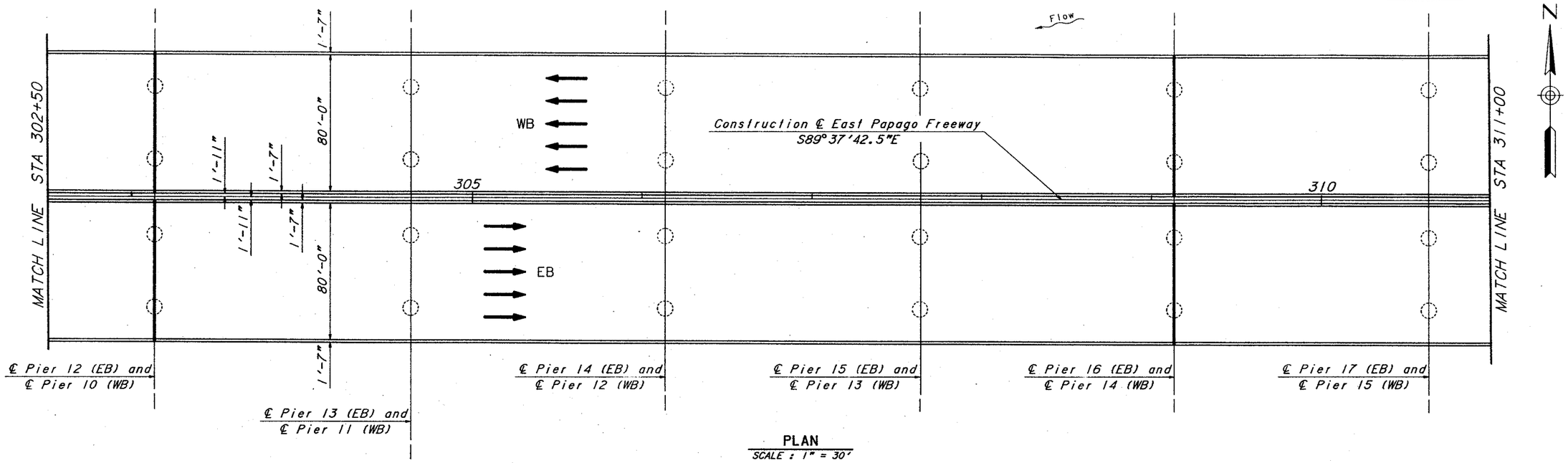
BRIDGE ENGINEER

DWG. NO. 3

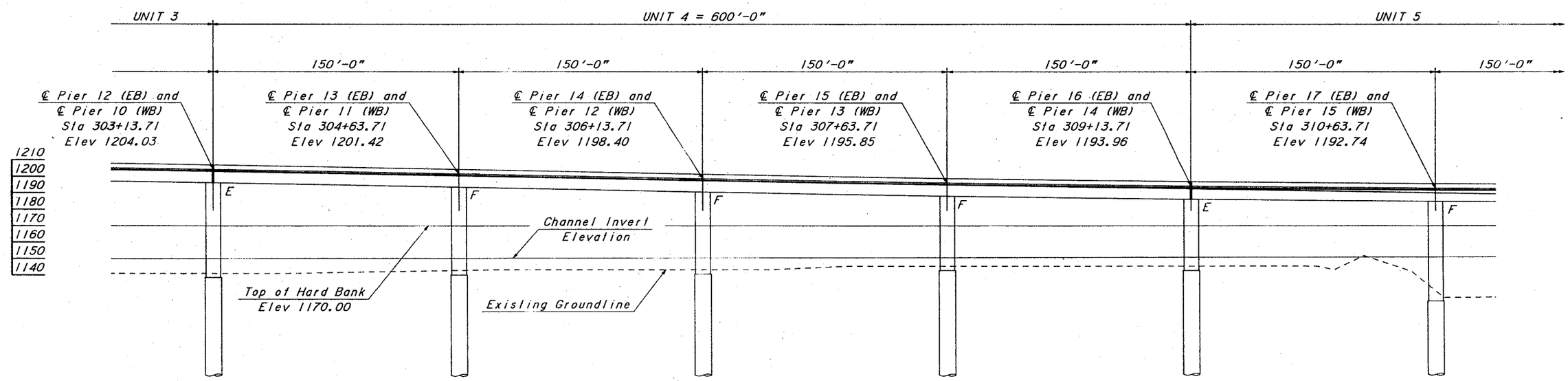
___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'



ELEVATION
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

NOT FOR CONSTRUCTION

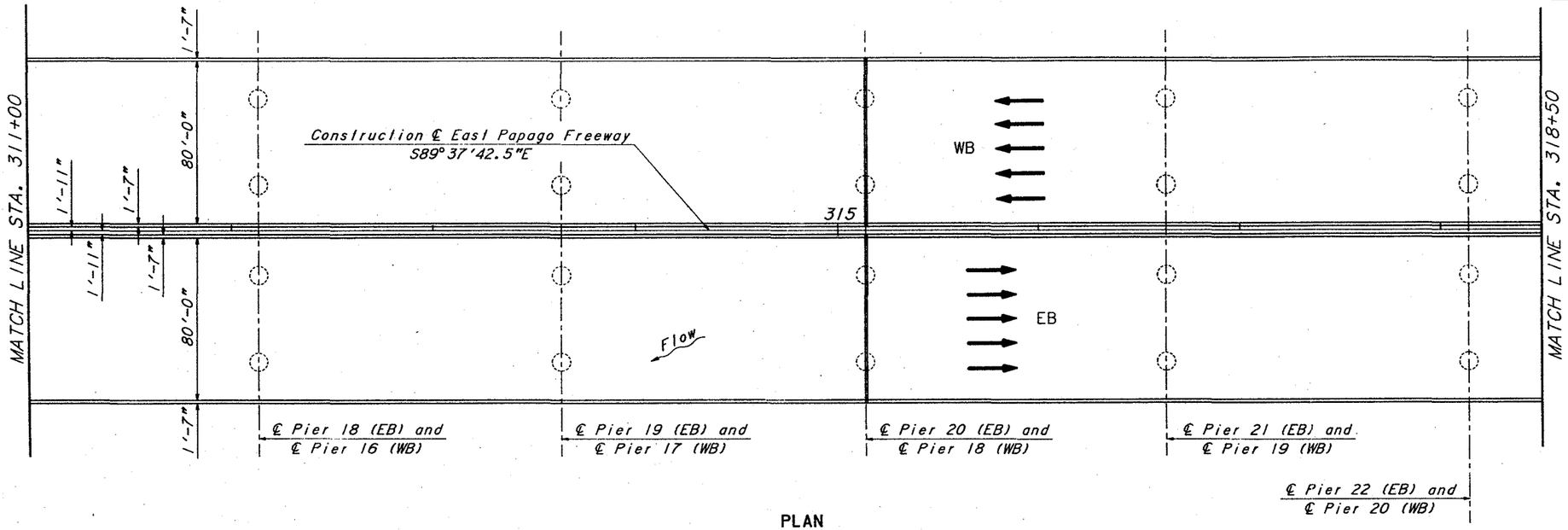
LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

BRIDGE ENGINEER

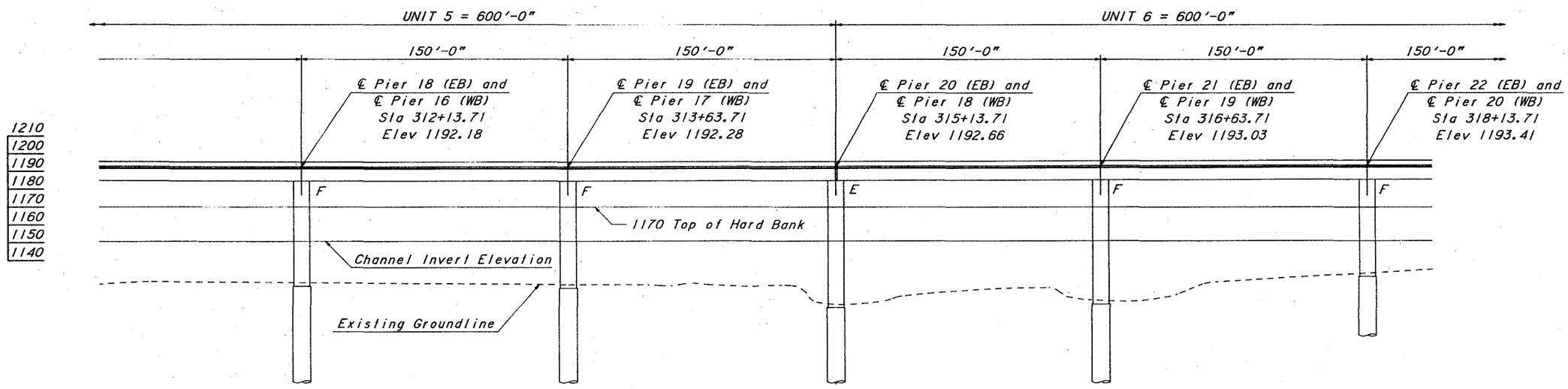
DWG. NO. 4

___ OF ___

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'



ELEVATION
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

LOCATION

NOT FOR CONSTRUCTION

BRIDGE ENGINEER

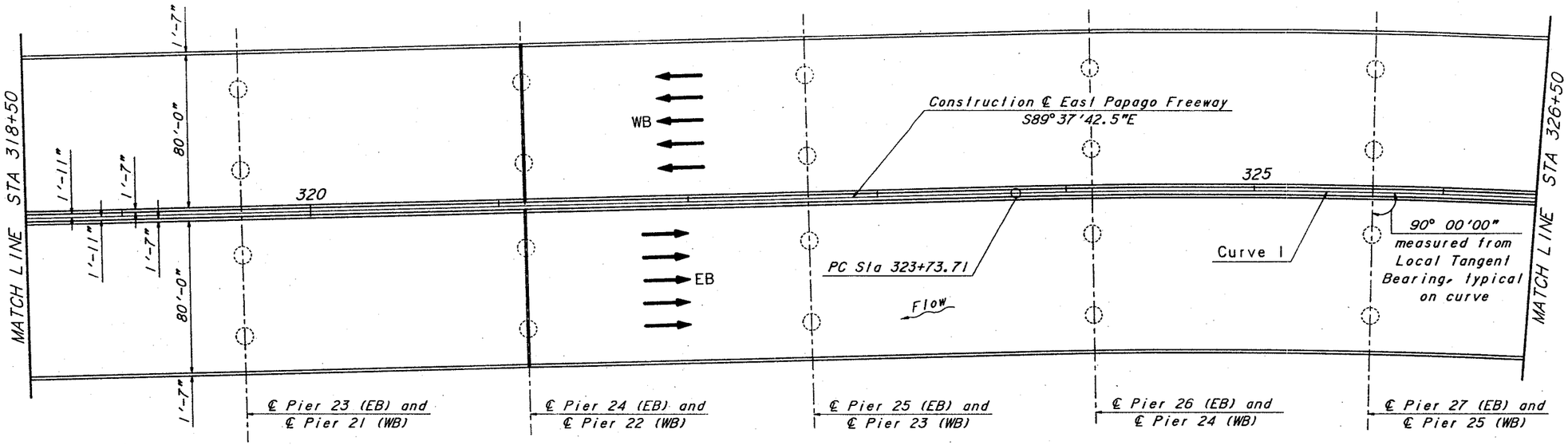
DWG. NO. 5

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO.

___ OF ___

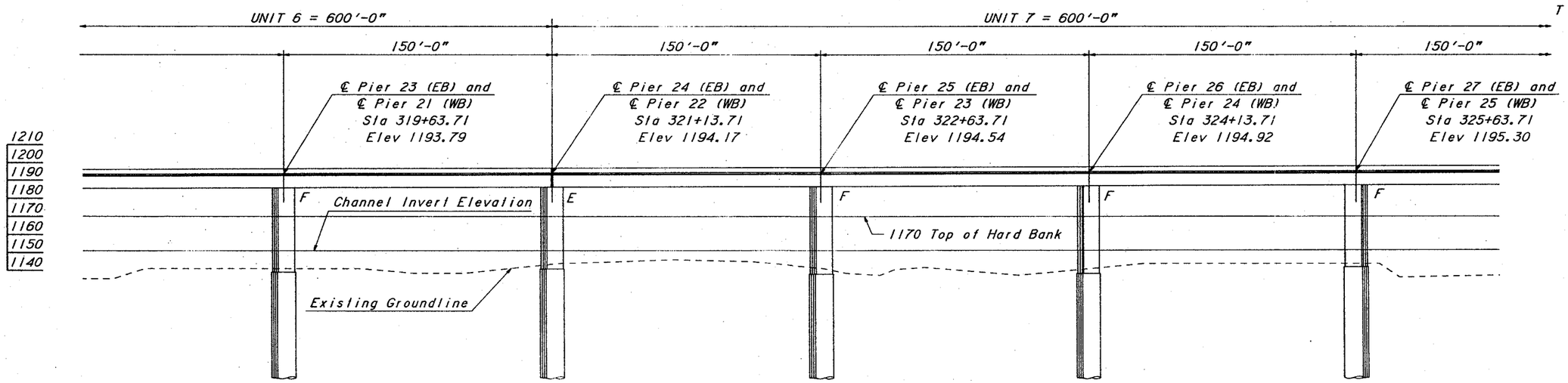
SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'

CURVE I
Δ = 15°37'42.5"
D = 1°30'00"
L = 1041.90'
R = 3819.72'
T = 524.20'



ELEVATION
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

NOT FOR CONSTRUCTION

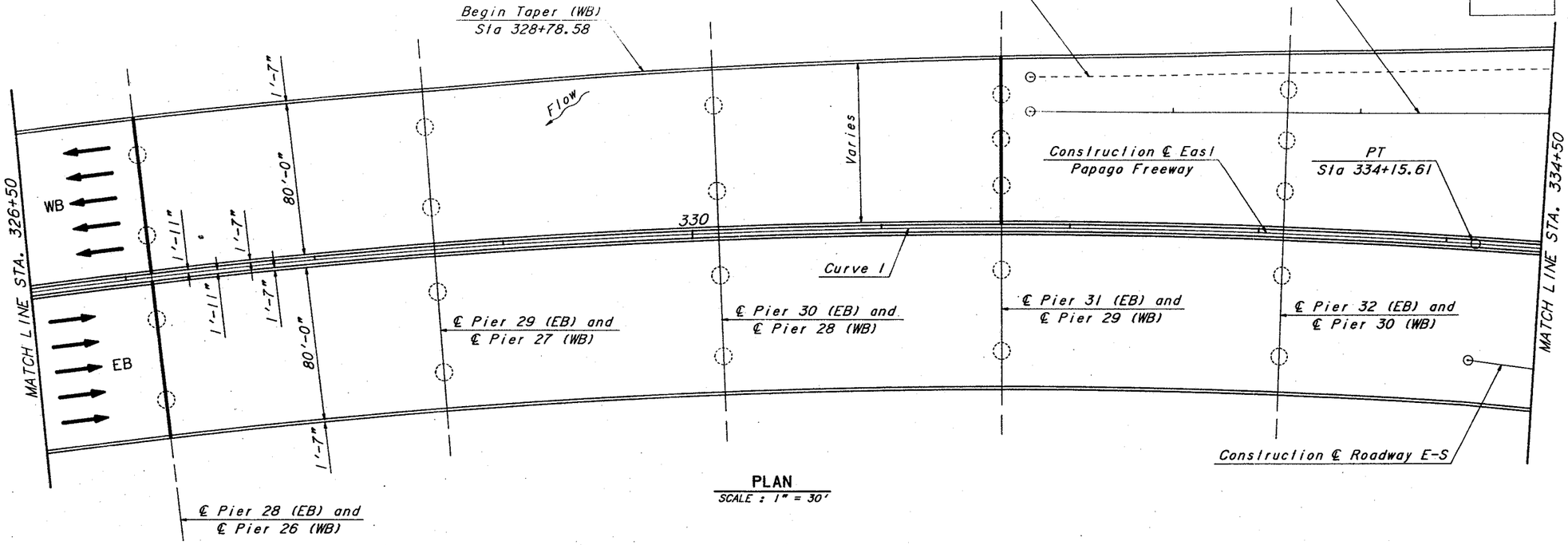
BRIDGE ENGINEER

LOOP 202 ROUTE	MILEPOST	STRUCTURE NO.	LOCATION

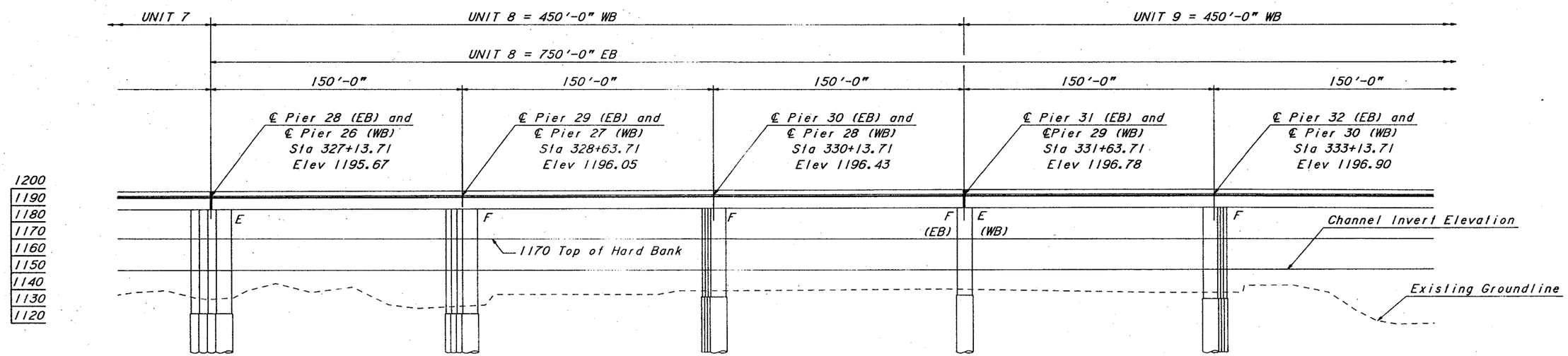
DWG. NO. 6

___ OF ___

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'



ELEVATION
SCALE : 1" = 30'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

	NAME	DATE
DESIGN	MM	9-90
DRAWN	EMG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

NOT FOR CONSTRUCTION

LOOP 202 ROUTE	MILEPOST	STRUCTURE NO.	LOCATION

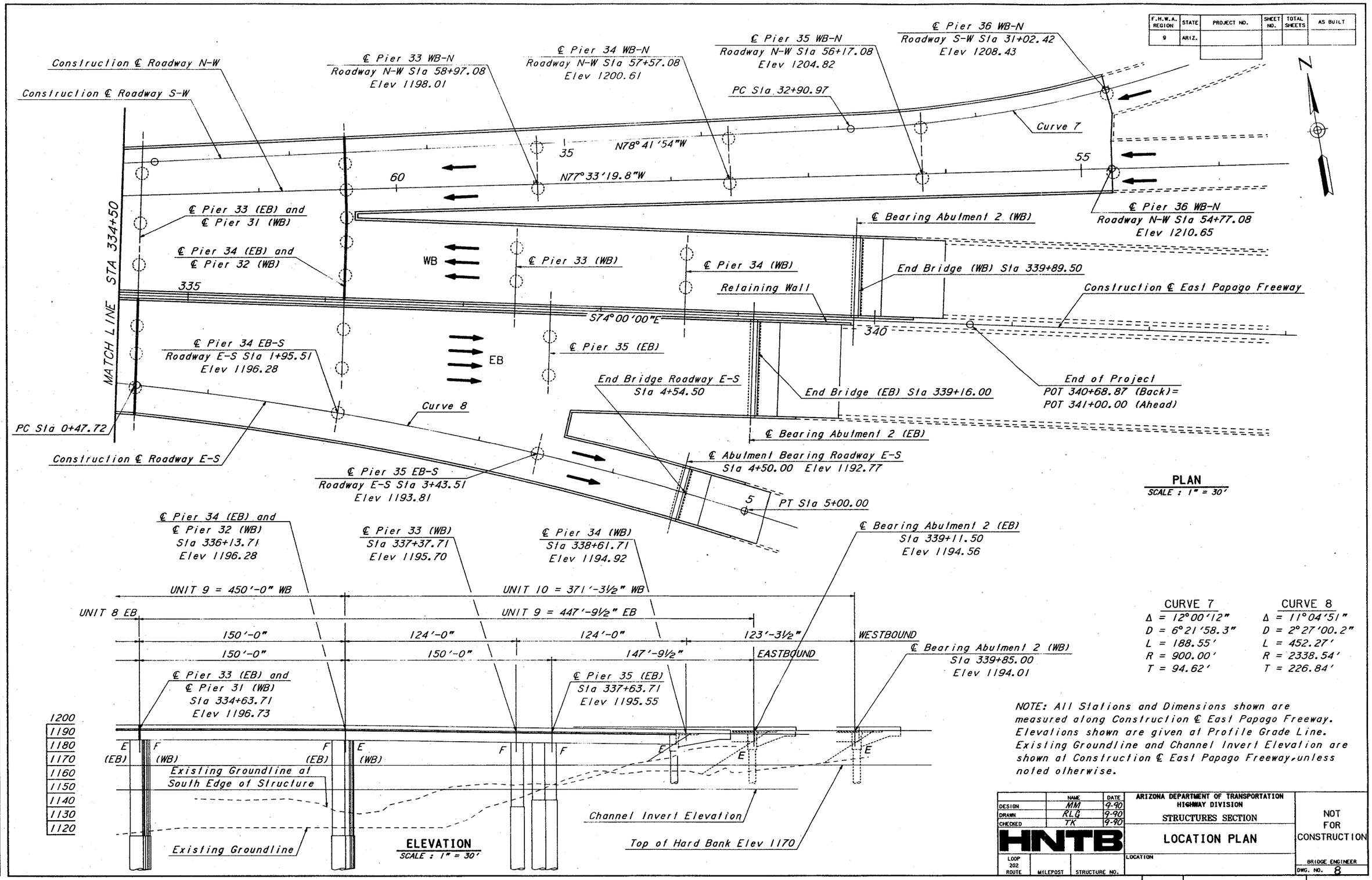
BRIDGE ENGINEER

DWG. NO. 7

___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



PLAN
SCALE : 1" = 30'

ELEVATION
SCALE : 1" = 30'

CURVE 7	CURVE 8
Δ = 12°00'12"	Δ = 11°04'51"
D = 6°21'58.3"	D = 2°27'00.2"
L = 188.55'	L = 452.27'
R = 900.00'	R = 2338.54'
T = 94.62'	T = 226.84'

NOTE: All Stations and Dimensions shown are measured along Construction ℄ East Papago Freeway. Elevations shown are given at Profile Grade Line. Existing Groundline and Channel Invert Elevation are shown at Construction ℄ East Papago Freeway, unless noted otherwise.

	NAME	DATE
DESIGN	MM	9-90
DRAWN	RLG	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

LOCATION PLAN

NOT FOR CONSTRUCTION

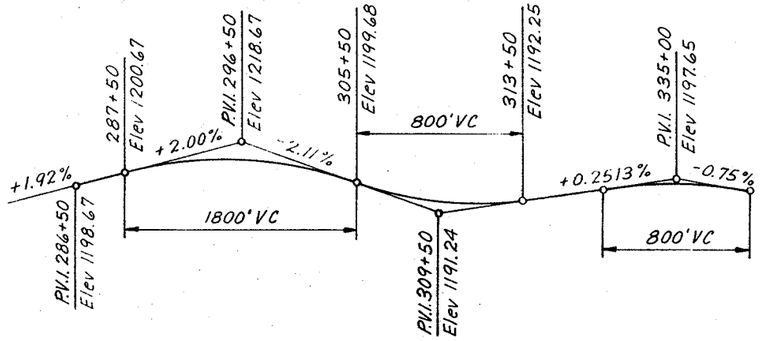
LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

BRIDGE ENGINEER

DWG. NO. 8

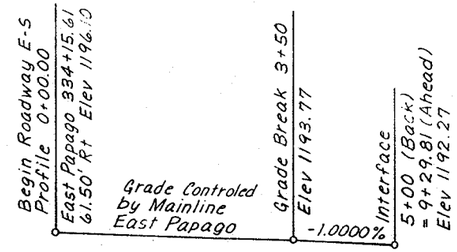
___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE



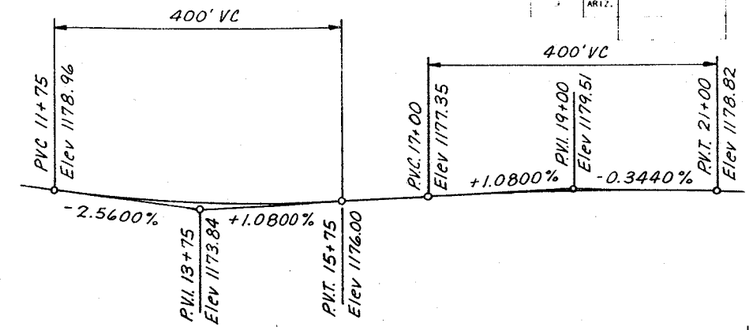
PROFILE GRADE EAST PAPAGO FREEWAY

NO SCALE



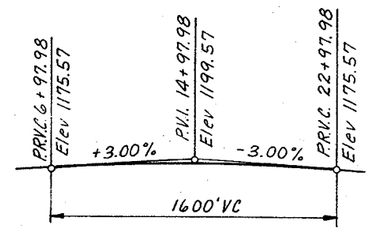
PROFILE GRADE ROADWAY E-S

NO SCALE



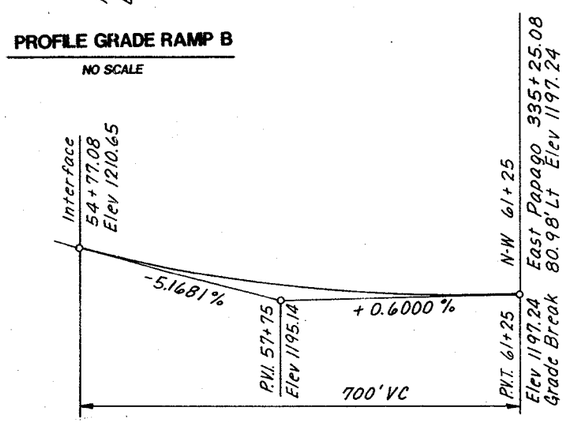
PROFILE GRADE RAMP B

NO SCALE



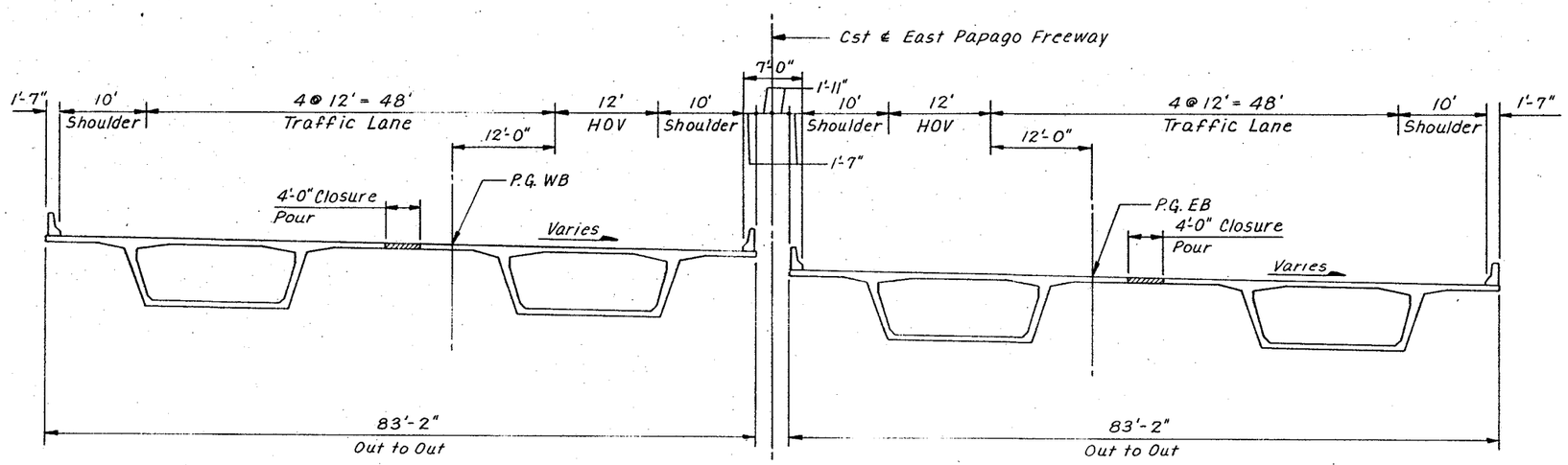
PROFILE GRADE McCLINTOCK DRIVE

NO SCALE



PROFILE GRADE ROADWAY N-W

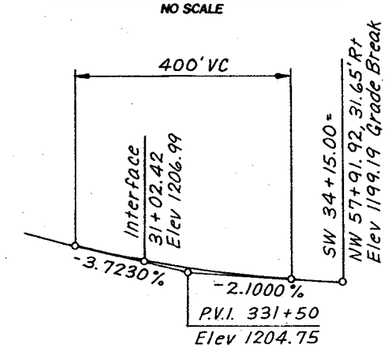
NO SCALE



TYPICAL SECTION

(SHOWN AT MAINLINE)

SCALE $\frac{1}{8}$ = 1'-0"



PROFILE GRADE ROADWAY S-W

NO SCALE

	NAME	DATE
DESIGN	TK	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO.

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

PROFILE GRADES AND TYPICAL SECTION

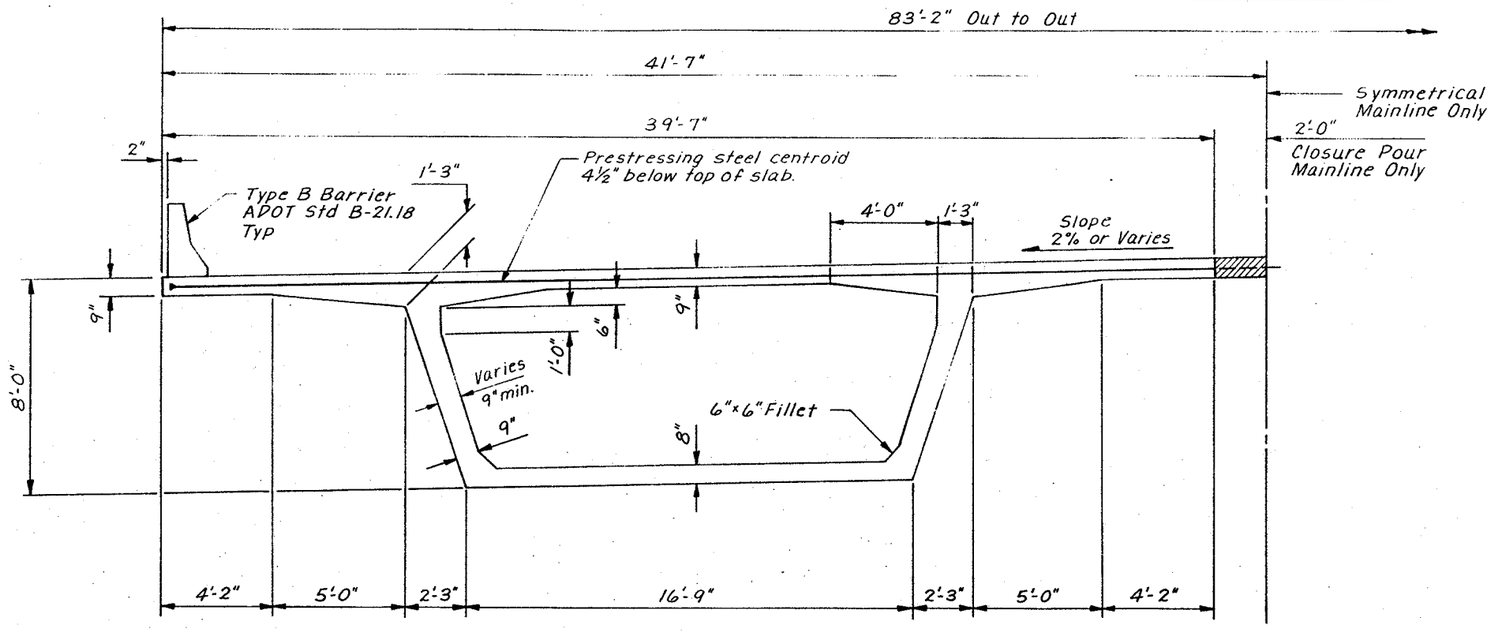
LOCATION

NOT FOR CONSTRUCTION

BRIDGE ENGINEER

SHT. NO. 9 OF

TRACS NO.



TYPICAL SECTION AT MAINLINE AND RAMP E-S - DIMENSIONS

SCALE $\frac{3}{8}$" = 1'-0"

NOTE:

Transverse prestressing consists of 4-0.6" strands in 1"X3" flat duct at 2'-0" spacing. Use 5 ducts @ 2' centers per 10 foot segment. Two ducts to be stressed in yard, three ducts to have strands continuous across roadways and strssed after closure pour is made. Mild steel reimforcing will provide top slab strength for movement of segment between casting bed and curing yard.

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

TYPICAL SECTION

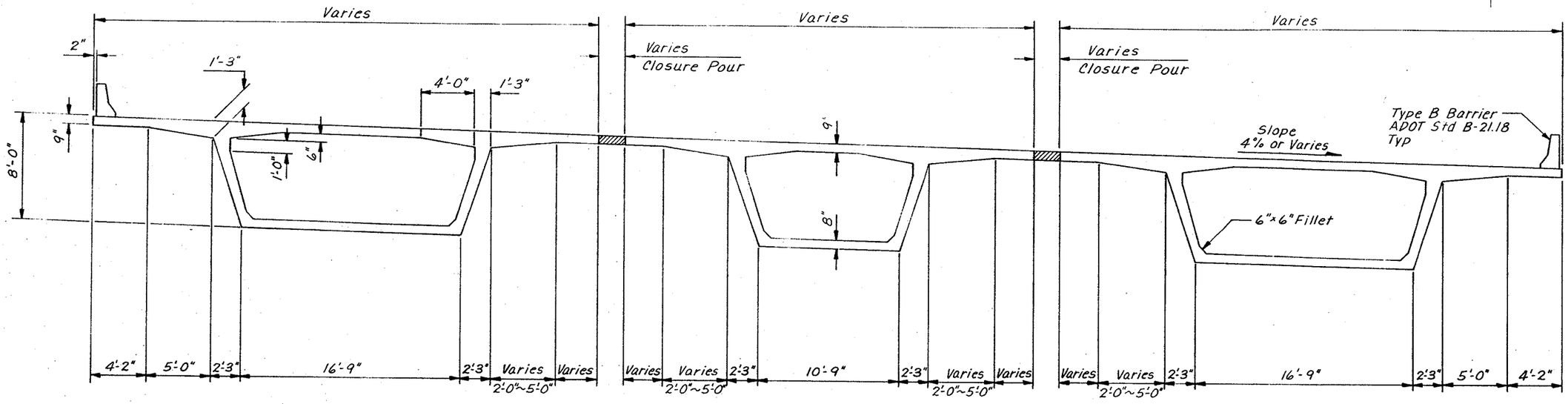
NOT FOR CONSTRUCTION

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

SHEET NO. 10 OF

TRACS NO.

SURVEY NO. | FINISH: PLANS | REF. SIDE: | SCALE: 0:



TYPICAL SECTION AT FLARE - DIMENSIONS

SCALE $\frac{1}{4}$" = 1'-0"

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

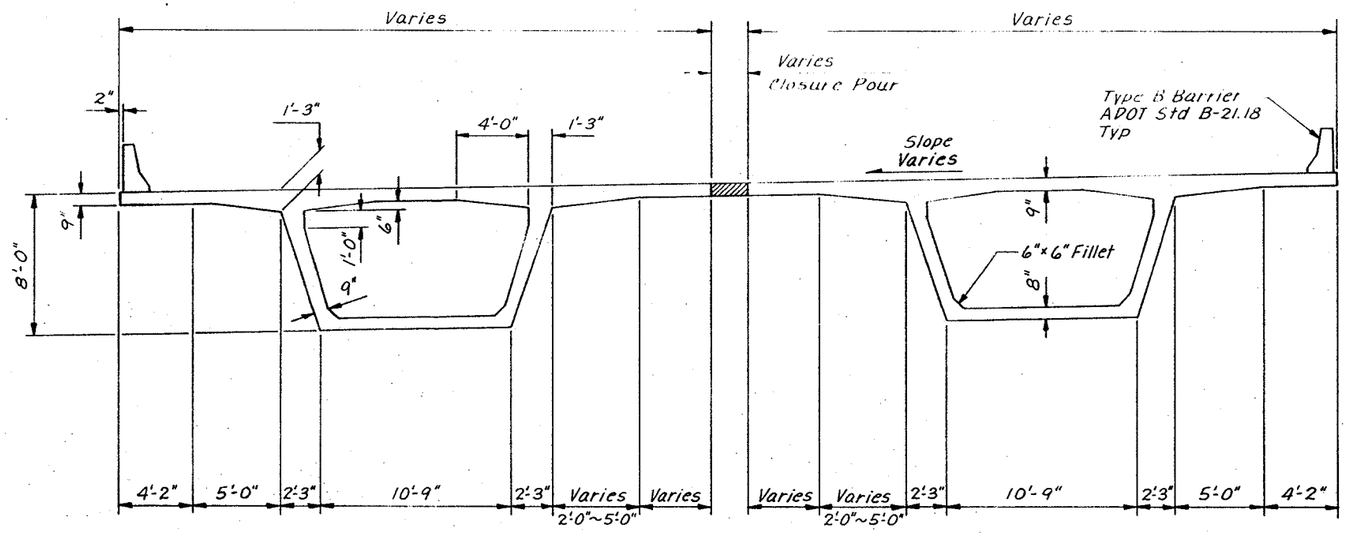
TYPICAL SECTION

NOT FOR CONSTRUCTION

LOOP 202 ROUTE	MILEPOST	STRUCTURE NO.	LOCATION

TRACS NO.

SHEET NO. 11 OF



TYPICAL SECTION* AT RAMP S-W AND N-W - DIMENSIONS

SCALE $\frac{1}{4}$ = 1'-0"

* Section applies EB Sta 334+63.71 / WB Sta 336+13.71 } to end of Bridge also

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION
TYPICAL SECTION

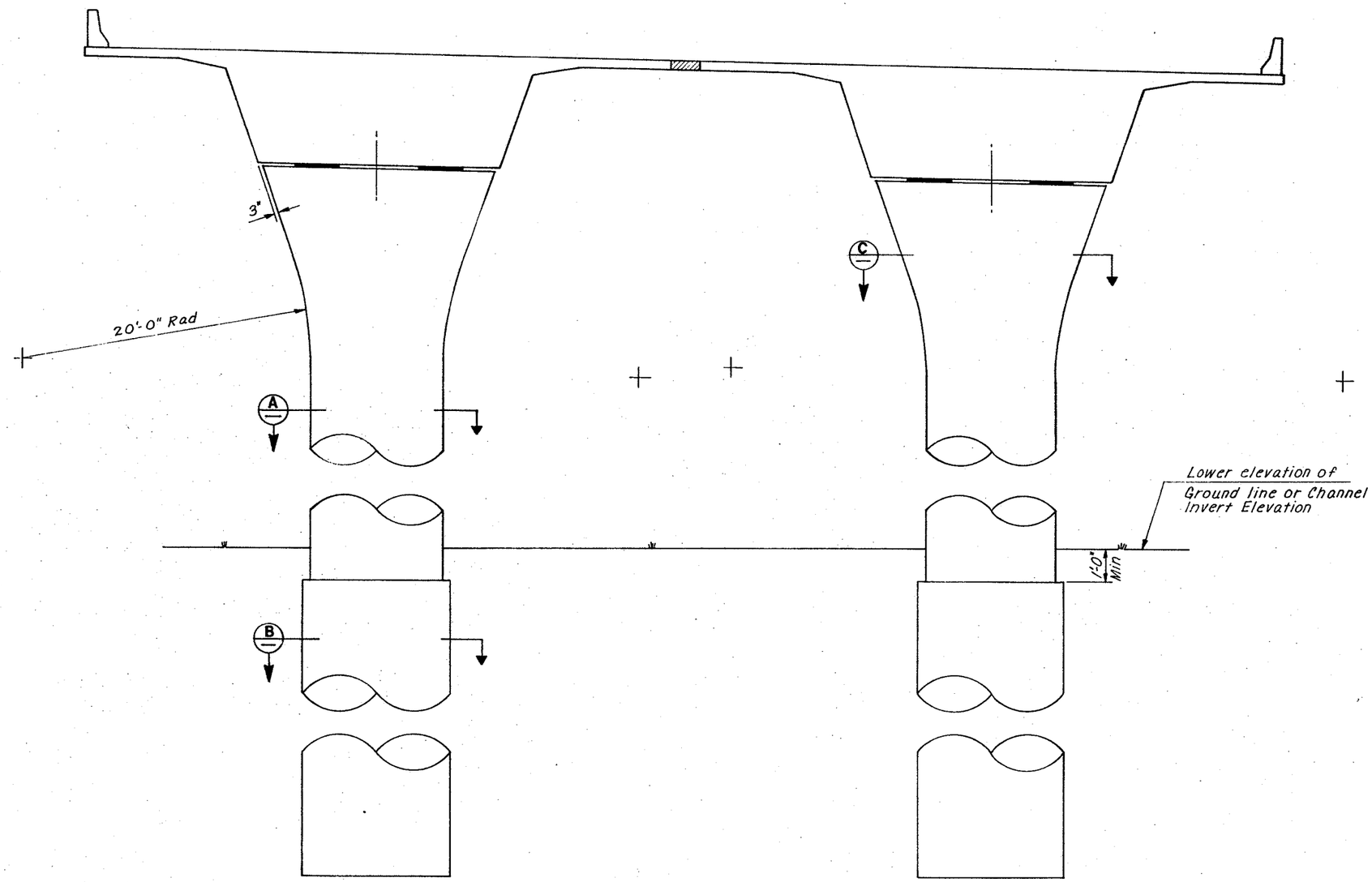
NOT FOR CONSTRUCTION

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION

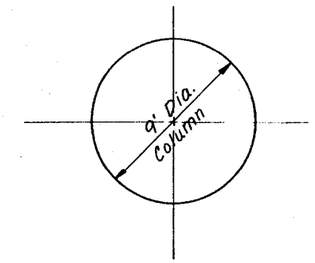
TRACS NO.

SHEET NO. 12

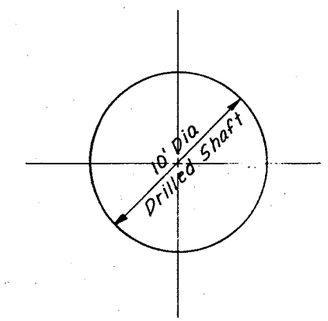
F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



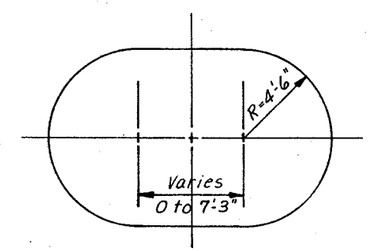
ELEVATION
SCALE $\frac{1}{4}$ = 1'-0"



A SECTION
SCALE $\frac{1}{4}$ = 1'-0"



B SECTION
SCALE $\frac{1}{4}$ = 1'-0"



C SECTION
SCALE $\frac{1}{4}$ = 1'-0"

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

PIER DETAILS

LOCATION

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO.

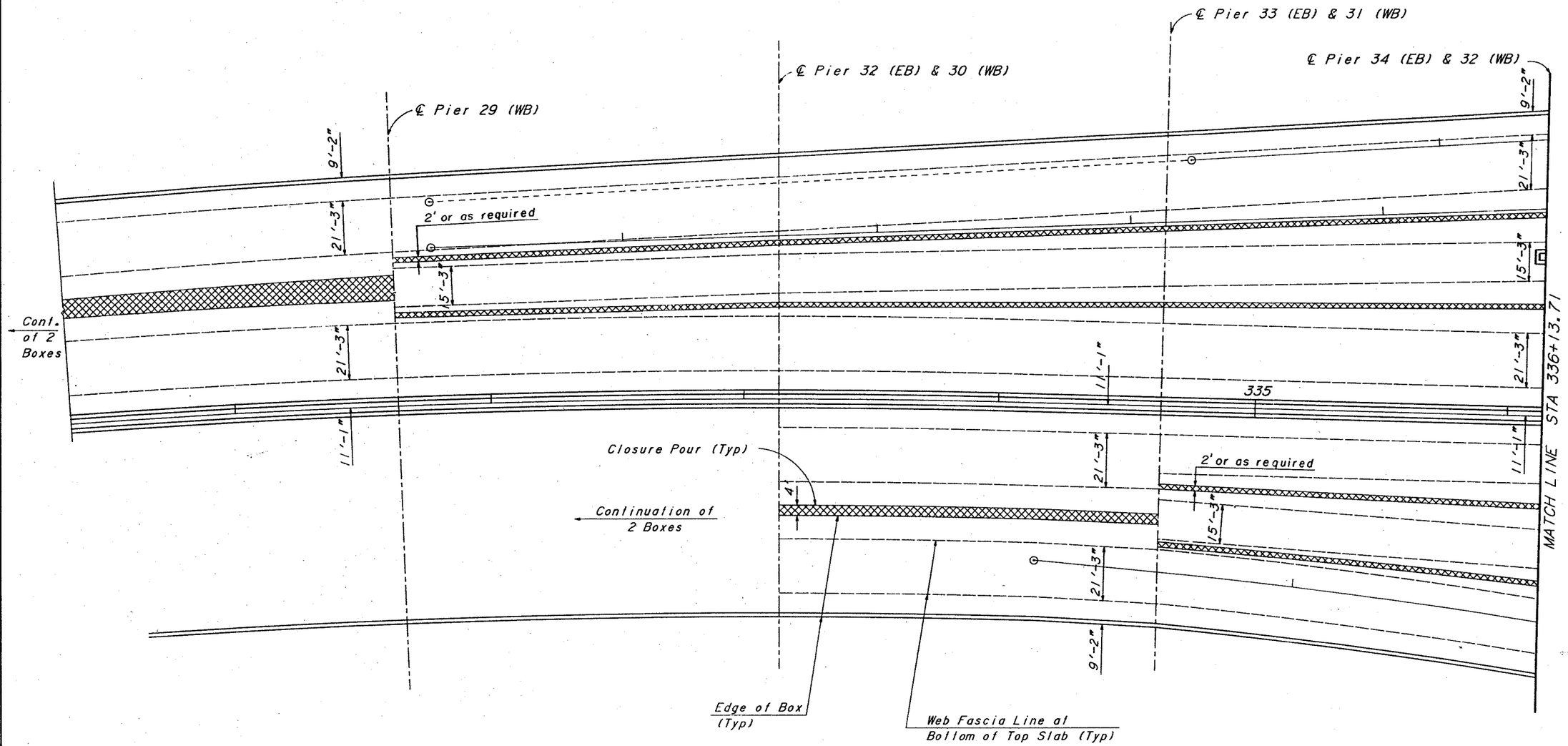
NOT FOR CONSTRUCTION

BRIDGE DRAWING NO. 13 OF

TRACS NO.

DATE	CHK	REVISION	CHK	SHEET NO.

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



FRAMING PLAN
Scale 1" = 20'

	NAME	DATE
DESIGN	MM	9-90
DRAWN	TOU	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

TAPER & GORE DETAILS

NOT FOR CONSTRUCTION

LOOP 202 ROUTE	MILEPOST	STRUCTURE NO.	LOCATION

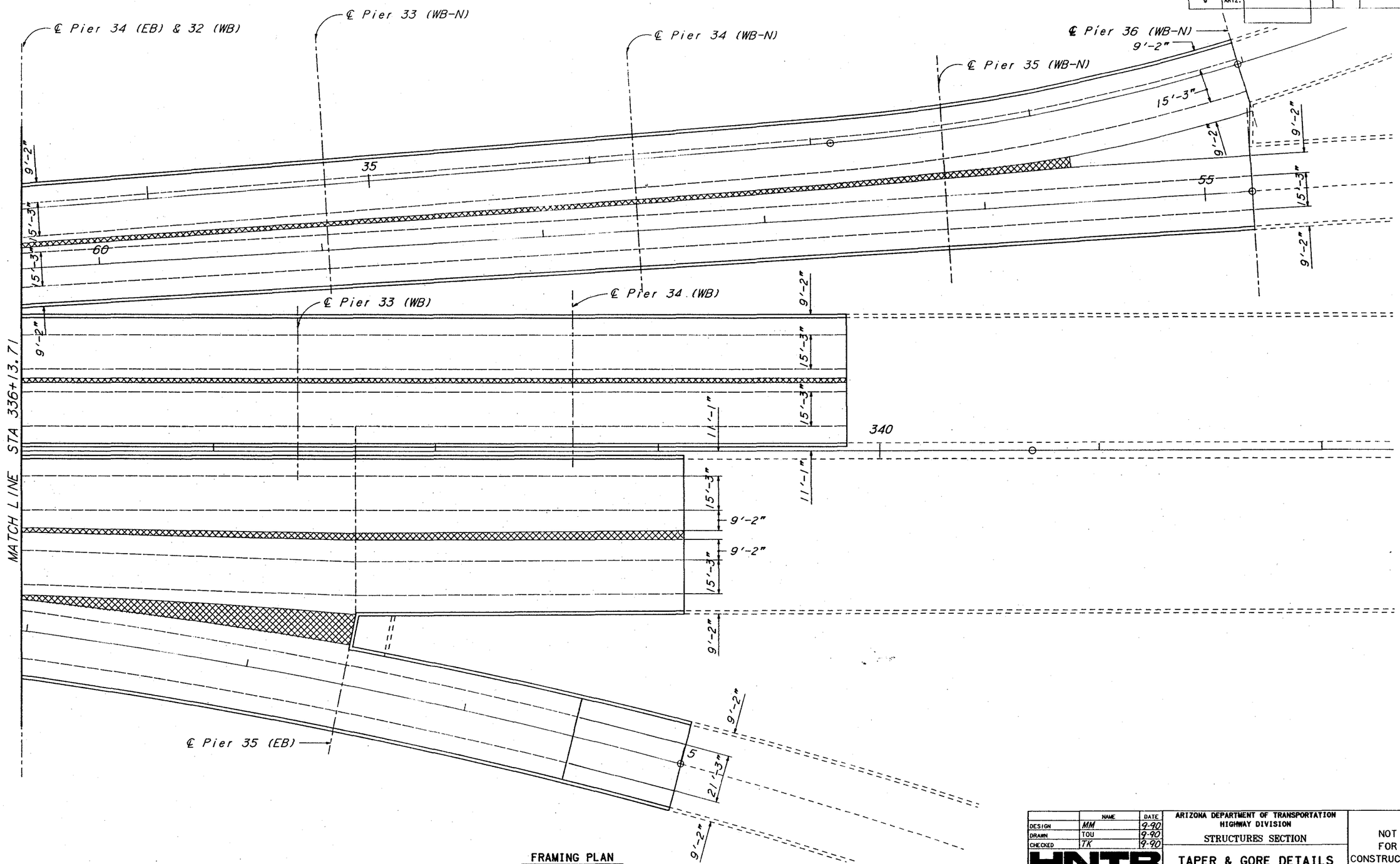
BRIDGE ENGINEER

DWG. NO. 14

___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



FRAMING PLAN
Scale 1" = 20'

	NAME	DATE
DESIGN	MM	9-90
DRAWN	TOU	9-90
CHECKED	TK	9-90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

TAPER & GORE DETAILS

LOCATION

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO.

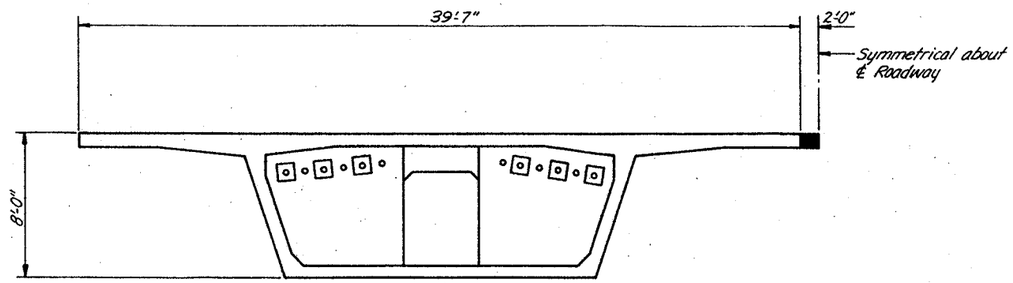
NOT FOR CONSTRUCTION

BRIDGE ENGINEER

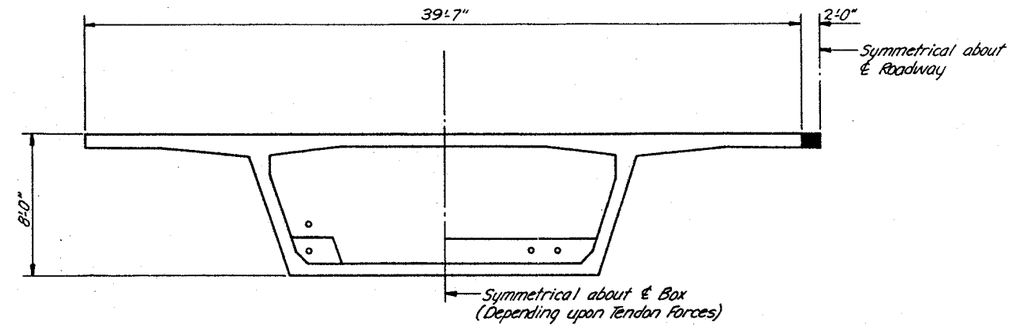
DWG. NO. 15

___ OF ___

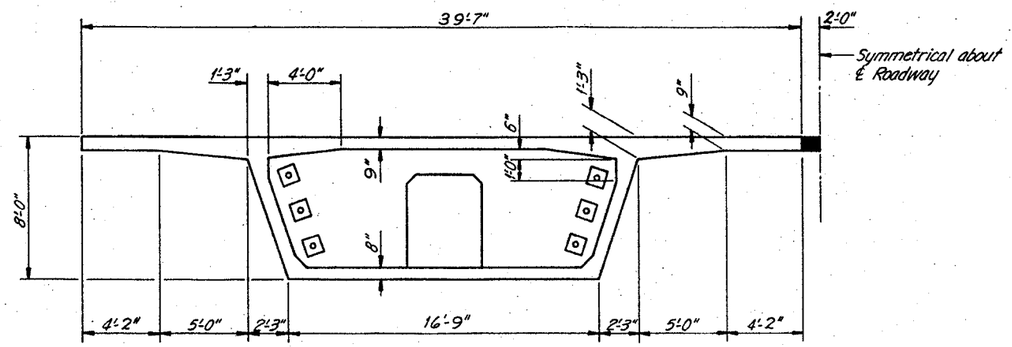
SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE



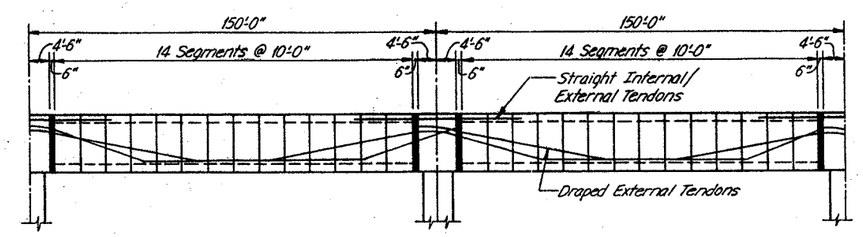
PIER DIAPHRAGM
NO SCALE



DEVIATION BLOCKS
NO SCALE



EXPANSION JOINT DIAPHRAGM
NO SCALE



SCHEMATIC TENDON PROFILE
NO SCALE

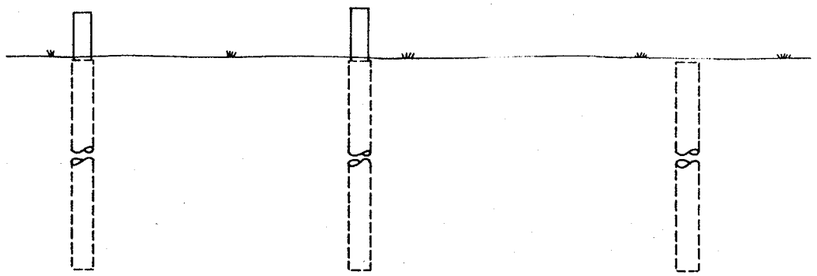
NOTE:
Diaphragm thickness to be determined at final design.

	NAME	DATE	
DESIGN	MM	9-90	ARIZONA DEPARTMENT OF TRANSPORTATION HIGHWAY DIVISION STRUCTURES SECTION **PIER DIAPHRAGM AND DEVIATION BLOCK DETAILS** LOCATION
DRAWN	JMC	9-90	
CHECKED	TK	9-90	
HNTB			
LOOP 202 ROUTE	MILEPOST	STRUCTURE NO.	

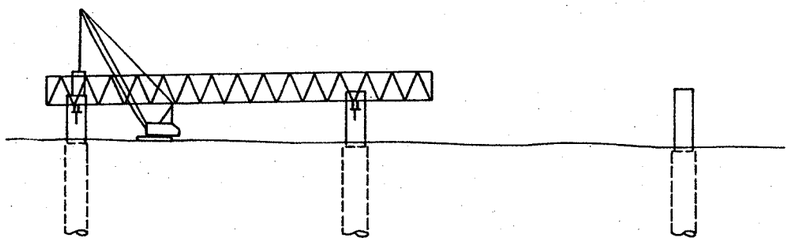
NOT FOR CONSTRUCTION

BRIDGE ENGINEER
DWG. NO. 16

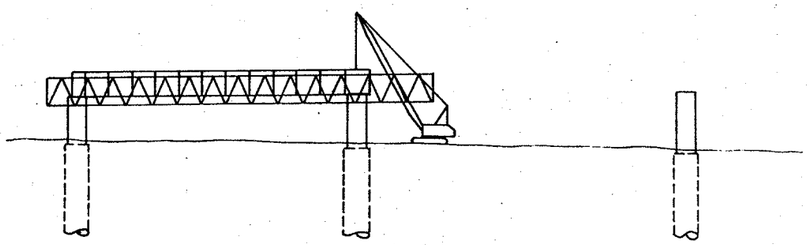
TRACS NO.



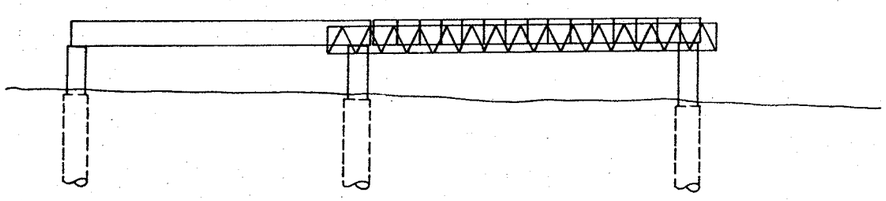
STAGE 1



STAGE 2



STAGE 3



STAGE 4

ERECTION SEQUENCE

STAGE 1

1. Construct drilled shafts.
2. Construct piers.

STAGE 2

1. Erect inside trusses on piers.
2. Place expansion joint segment.

STAGE 3

1. Place pier segment and adjust to grade.
2. Place span segments, adjust to grade and epoxy joints.
3. Cast closure pours at pier segments.
4. Stress longitudinal post-tensioning.
5. Release truss.

STAGE 4

1. Advance inside trusses to next span.
2. Place inside pier segment and adjust to grade.
3. Place inside span segments, adjust to grade and epoxy joints.
4. Cast inside transverse closure pours at pier segments.
5. Stress inside box girder longitudinal post-tensioning.
6. Release trusses for placing inside box girders.

Note: Outside refers to segment boxes on outside of structure.
Inside refers to segment boxes adjacent to roadway median.
Eastbound roadway to be constructed first.

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	ROUTE
9	ARIZ.				

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

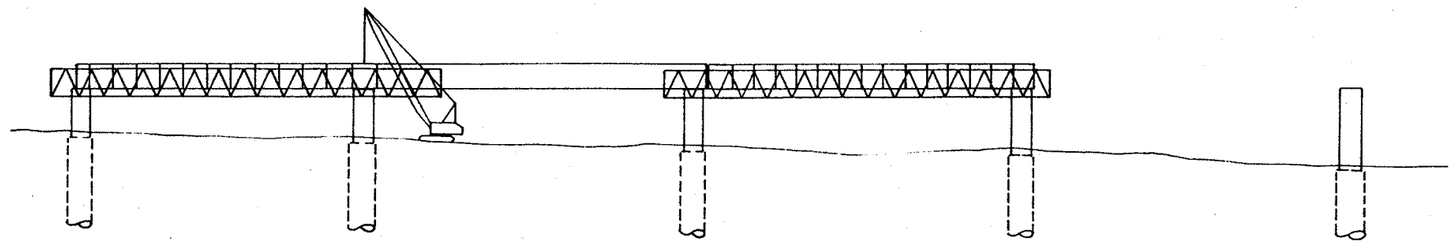
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION
ERECTION SEQUENCE

LOCATION

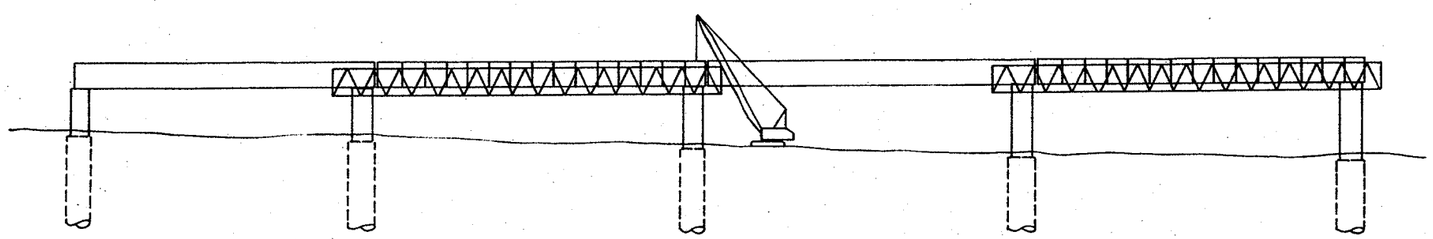
NOT FOR CONSTRUCTION

DRAWING NO. 17 OF

TRACS NO.



STAGE 5



STAGE 6

ERECTION SEQUENCE

STAGE 5

1. Advance inside box girder trusses to next span.
2. Place outside box girder trusses on end span.
3. Place expansion joint segment on outside box girder truss.
4. Place span segments, adjust to grade and epoxy joints.
5. Place pier segment and adjust to grade for both spans supported by trusses.
6. Cast transverse closure pours both trusses.
7. Stress longitudinal post-tensioning for both spans supported by trusses.

STAGE 6

1. Repeat steps 1 through 7 of Stage 5.
2. Pour longitudinal closure pour between end span segments.
3. Stress transverse tendons through closure pour.

STAGE 7

Continue span-by-span erection with inside box girder construction leading outside box girder by at least two spans.

	NAME	DATE
DESIGN	MM	9-90
DRAWN	JMC	9-90
CHECKED	TK	9-90

HNTB

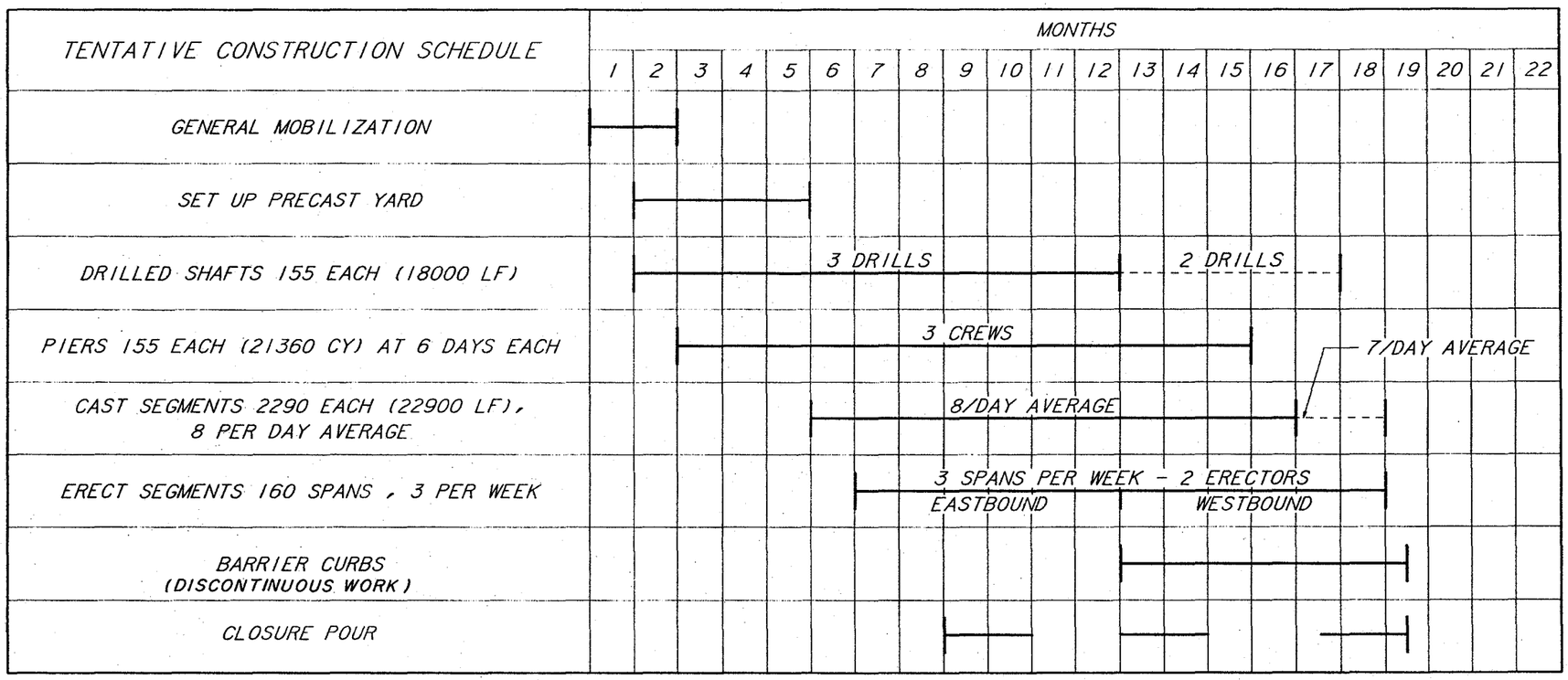
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION
ERECTION SEQUENCE

NOT FOR CONSTRUCTION

BRIDGE STRUCTURE NO. 18

TRACS NO.

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
9	ARIZ.				



NOTE:

Idealized schedule assumed.

Schedule is based on
2 shifts per day, 6 days per week.

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN	MM	8/90
DRAWN	EMG	8/90
CHECKED	TK	9/90

HNTB

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
STRUCTURES SECTION

CONSTRUCTION SCHEDULE

NOT FOR CONSTRUCTION

LOOP 202 ROUTE | MILEPOST | STRUCTURE NO. | LOCATION | BRIDGE ENGINEER

DWG. NO. 19

___ OF ___