I-10 Mobile River Bridge and Bayway Widening

Preliminary Bridge Type Study – High Level Approach Spans

Technical Memorandum

Mobile, Alabama

May 30, 2017

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1.0 Introduction

The I-10 Mobile River Bridge and Bayway Widening project proposes to increase the capacity of I-10 by constructing a new bridge with 215 feet of vertical clearance and 600 feet horizontal minimum clearance across the Mobile River and widen the existing I-10 bridges across Mobile Bay from four to eight lanes.

2.0 Purpose of Preliminary Bridge Type Study

The Preliminary Bridge Type Study for the high level approach spans to the Mobile River crossing (henceforth referred to as bridge study) will focus on the feasibility, type, and location of the bridge in support of the Final EIS decision making process. It will identify potential options for the location and type of a bridge that may cross the Mobile River as part of the I-10 Mobile River Bridge and Bayway Widening project. The study will provide the basis for comparison of effectiveness of bridge alternatives .

The new I-10 Mobile River Bridge can be divided into two general regions:

1. High level approach spans to the east and west of the main span unit. These spans are approximately 8,000 feet in total length and will cross several existing and future roads including railroad tracks. The pier heights will vary from approximately 25 feet near the abutment to 185 feet at the main span transition pier. The east end of the high level approach spans will tie-in with the new Bayway Bridges.

2. Cable-stayed main span unit that crosses the Mobile River. (Refer to the Main Span Preliminary Bridge Type Study technical memorandum submitted under separate cover.)

For both the main span unit and high level approach spans, various span arrangements, substructure and superstructure types and foundations were studied. The preliminary bridge concepts were evaluated through a screening process and recommendations are presented that can be used for the future planning and design development of the Mobile River Bridge project.

The scope of this memorandum is to document the development process for the high level approach spans, the alternatives developed and considered, to evaluate the functional and technical merits of those alternatives, and to prepare recommendations for further development during Preliminary and Final Design of the project.

Other project components, such as highway geometry, maintenance of traffic plans, toll plazas and interchanges, drainage, roadway lighting, signing, Bayway widening, etc., are evaluated concurrently by ALDOT and others and are not included in this memorandum.

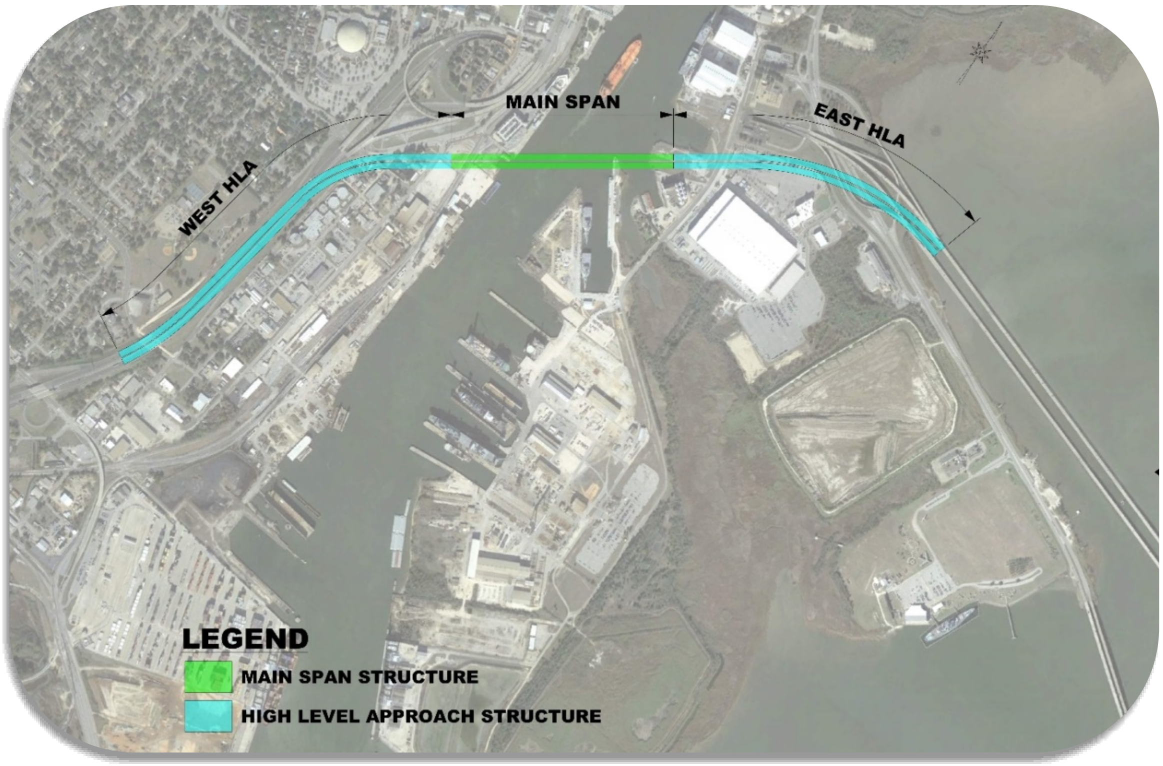


Figure 1: Bridge Layout

3.0 Bridge Type Evaluation Process Methodology

As part of the I-10 Mobile River Bridge and Bayway Widening project, a bridge type selection process was initiated in 2016 for the high level approach spans. The evaluation process consists of three steps which involve developing and analyzing numerous bridge alternatives. These steps will culminate in the recommendation of a new bridge type(s) that will be designed and built across the Mobile River. As described below, the evaluation process is collaborative in nature and is intended to capture project specific requirements.

- Step 1 – Develop Preliminary Bridge Concepts.

Working in close coordination with ALDOT, key design criteria and guidelines were developed to evaluate bridge concepts.

Key design criteria included items such as:

- Alignment (tangent or curved)
- Construction Cost
- Maintenance and Durability
- Constructability
- Right-of-Way and Access
- Structural Limitations
- Environmental Commitments

Evaluation guidelines were also developed as part of the overall project. Some of the guidelines reflected navigational, structural and highway limitations, and physical restrictions that exist at the bridge site. Other guidelines represented environmental commitments and financial constraints necessary to meet budgetary goals.

Once a consensus was reached on the design criteria and evaluation guidelines, several concepts were developed and presented to ALDOT. Three concepts were then selected and carried forward to Step 2.

- Step 2 - Develop Three Bridge Type Alternatives

Step 2 consisted of the following general tasks:

- Perform conceptual engineering on three concepts approved in Step 1.
- Further refine for conformance to the design parameters and best meeting the design guidelines.
- Prepare comparative construction cost estimates.
- Prepare and present updated evaluation matrix.

Step 2 culminated with recommendations on the bridge type that would be further developed in Step 3.

- Step 3 – Develop Preferred Bridge Alternative for TS&L (Future Phase)

Step 3 consists of the following general tasks:

- Refine final bridge alternative.
- Prepare cost estimate.
- Prepare construction time.
- Complete type, size and location report.

The project is currently at the end of Step 2. During Step 1, nine preliminary bridge concepts were developed and evaluated, and on September 14, 2016 a meeting was held with ALDOT to present the preliminary concepts. Three alternatives reflecting feasible bridge types were identified as best meeting the objectives of Step 1 and selected to be carried forward into Step 2. A copy of the PowerPoint presentation and meeting minutes for the September 14, 2016 meeting can be found in Appendix A of this technical memorandum.

Prior to completion of Step 2, ALDOT decided to evaluate the Public-Private Partnership (P3) project delivery method for the project and directed the design team to complete Step 2 and cease any further work which in essence included the tasks outlined under Step 3. Assuming this delivery method is used, the future P3 developer, design engineer and construction contractor will make the final bridge type selection based on the contract requirements.

4.0 Step 1 - Preliminary Bridge Concepts

In coordination with ALDOT, a preliminary bridge design criteria document was prepared for the various bridge concepts and to provide the necessary technical framework for the study. The design criteria is included in Appendix B.

In addition to design criteria, other project specific evaluation guidelines were developed as part of the overall project requirements. Some of the guidelines reflected navigational, structural and highway limitations, and physical restrictions that exist at the bridge site. Other guidelines represented environmental commitments and financial constraints necessary to meet budgetary goals. These guidelines and criteria were summarized into a bridge type evaluation matrix to assist in the review of the various bridge types.

4.1 Structure Types Considered

4.1.1 Superstructure - Precast “I” and “U” Beams

To recommend structure types for the Mobile River Bridge high level approach spans, initial assumptions must be made as to span arrangements and structure types to allow for initial screening of concepts and to prepare comparative costing of the various options.

During the main span meeting on July 28, 2016 the design team was directed by ALDOT to eliminate structural steel alternatives from further consideration due to concerns with life-cycle costs associated with structural steel in a coastal environment. In addition to the main span unit, the bridge types studied for the high level approaches exclusively focused on concrete sections either prestressed, post-tensioned or a combination of both.

As noted in the ALDOT Structural Design Manual, the prestressed concrete girder design policy requires the use of standard shape AASHTO-PCI type girders designed as simple spans. These bridges are considered to be standard design because their regular geometry and short spans allow the use of readily available bridge components and conventional construction techniques.

Even though ALDOT’s criteria for standard designs is reasonable and clear, due to the large quantity of approach span construction combined with project specific constraints such as bridge height, construction in an urban environment, curved alignment, etc. it was considered advantageous for the project to investigate other structure types that could be beneficial in addressing those challenges. The next step in developing additional feasible concrete bridge types included an expanded search of structure types that may not ordinarily be in ALDOT’s inventory.

In recent years some Departments of Transportation (DOT) throughout the country have adopted the use of precast, spliced girders. The primary advantages of spliced girders include: longer spans than can be achieved with pretensioned-only precast girders; cost effectiveness versus steel superstructures; flexibility to span at grade obstacles and reduce the number of substructures; and improved bridge appearance.

Spliced precast girders are not commonly used by ALDOT so the design team researched and reviewed details and practices from other Owners including their project experience. This became the basis for a white paper prepared for ALDOT’s review and concurrence.

The paper, included in Appendix C, presented background information on the two most common structural configurations: spliced I-beams and spliced U-beams, and is organized into basic information for each type; brief discussion of potential application on the Mobile River Bridge high level approaches; summary of example projects from across the U.S.; and additional resources.

Spliced girder bridges are typically used for continuous structures in order to facilitate longer spans. In the past, spliced girders have been limited to straight concrete I-beam sections but new spliced U-beam technologies now allow for curved U-shaped bridge options. Several have already been successfully constructed in Colorado.

Construction of these bridges generally includes:

- Precasting either straight or curved u-shaped sections.
- Supporting the pier and "drop in" sections, and splicing the sections together on temporary supports using post-tensioning.
- A lid slab is cast before the final post-tensioning is applied to increase the torsional resistance of the section.
- After the section is closed and stressed, conventional forms are placed between the boxes and a full depth deck is cast.

Some of the advantages to this type of construction compared to conventional construction include:

- Lower fabrication times.
- Faster construction.
- Ability to span longer distances.
- Increased aesthetics by providing a unified appearance.

Challenges of this type of construction include:

- Requires extensive shoring.
- Typically entails heavy beam sections and larger cranes to place.

As this type of construction gains wider acceptance amongst Owners, the industry is moving towards standardizing their use by providing design and construction guidelines and sharing their experience with these beams. Some of these industry resources include:

- The Precast/Prestressed Concrete Institute (PCI) developed a set of concept drawings for curved, spliced U-beams.
- Colorado Department of Transportation has been building spliced beam bridges since the early 1990s, including a site-precast curved U-beam bridge in 1995.
- The Florida Department of Transportation has adopted the PCI Zone 6 sections as the preferred spliced girder.

Table 1 below summarizes additional pros and cons of spliced precast I-beams and U-beams.

Type	Use	Pros	Cons
Spliced I-Girders	• Simple or continuous spans • Straight (possible curved application) • Section depth 7', up to 12' if haunched • 170' to 325' spans	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Can be used in curved alignment by providing a kink at splice • Regional precasters with experience and formwork	• Some standards are available; depending on beam type, may require web and end block modification • Less torsionally stiff than U-beams, therefore not curved • Requires shoring or strongbacks for erection until splicing is complete • Less aesthetic option than U-beams
Spliced U-beams	• Simple or continuous spans • Straight or curved • Section depth 6'-8', up to 11' if haunched • Radius down to 700' • Spans up to 300' • Minimum 2 beams per cross-section	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Fewer beams required in cross-section than with I-girders • Considered more aesthetic than I-girders • Standards are available (PCI Zone 6 Concept drawings) • Regional precasters with experience and formwork	• Heavy, therefore larger demand on substructure • For transport, field sections limited to approx. 100' • Super-heavy permits likely required for transport (approx. 115-125 tons per section) • Larger erection equipment necessary • Requires shoring or strongbacks until splicing is complete

Table 1 - Comparison of Spliced Precast I-Beams vs. U-Beams

As a result of this paper and after a series of discussions with ALDOT, the following approach was agreed upon for precast splice beams.

- Eliminate from further consideration spliced precast I-beams. The use of spliced precast I-beams in curved alignments is theoretically possible but was considered too risky due to the lack of track record of projects with similar scope as the Mobile River Bridge.
- U-beams have a higher aesthetic value. An important DEIS environmental commitment.
- It is likely that a combination of simple and continuous spans will be needed to accommodate the project geometry. U-beams will maintain a uniform bridge appearance.

In addition to continuous beam types, the possibility of using longer simple spans was also investigated. ALDOT typically uses the BT-72 section for simple spans in the 120-140 feet so it was desired to investigate the feasibility of using a section that could span longer distances and still meet ALDOT's standard practice of simple span I-beams. After researching current practice throughout the Gulf states and the availability of precasters to produce the precast sections, the Louisiana Department of Transportation and Development's (LaDOTD) Louisiana Girder Shape LG-78 with a maximum span length of 183 feet was selected as a feasible section.

4.1.2 Superstructure - Precast Segmental Box Girders

Precast concrete segmental construction is ideally suited for long elevated structures; very few structure types offer the production speed, cost effectiveness, aesthetics, and quality afforded by a precast concrete system. Off-site casting and on-site post-tensioning results in the production of a high quality and durable structure.

Segmental construction expands the range of feasible span lengths from 150 feet which is the optimum span length for span-by-span construction to over 300 feet for cantilever construction. Precast concrete segmental construction techniques are flexible enough to accommodate placing precast concrete segments in compact work zones over major utilities, railroad tracks and service roads thus minimizing the impact to existing heavy vehicular flow in and around the construction site. The trapezoidal box girder shape also has superior aesthetic value due to its smooth, continuous flat planes.

Precast segmental construction offers the following advantages:

- Build-from-Above construction that minimizes impacts to existing vehicular and railroad traffic.
- Production speed and quality afforded by a precast concrete system. Precast concrete segmental construction is ideally suited for long elevated structures.
- Off-site casting and on-site post-tensioning resulting in a high quality structure.
- Minimum disruption to vehicular and rail traffic.
- High aesthetic value.
- Geometric Flexibility – easily accommodates curved and tangent alignments.
- Use of local labor and materials.

4.1.3 Superstructure – Summary of Structure Types

On the basis of ALDOT’s current practice for precast girders, the adoption of precast splice beams by various DOT’s and the benefits that precast segmental concrete offers, the following bridge categories were selected to be further developed:

Considered = ● Not Considered = ○

	Structure Types			
Bridge Alternative	Simple Span	Spliced/ Continuous	Straight	Curved
Precast I Girders	●	○	●	○
Precast U Girder	●	●	●	●
*Steel Plate Girder	○	○	○	○
Precast Segmental	●	●	●	●

* Eliminated from High Level Approach Spans Scope

Table 2: Summary of Structure Types Categories

To develop superstructure types for each of the categories listed in Table 2, the following screening criteria was used:

- Baseline solution uses ALDOT's preferred girders and structural design criteria.
- Consider alternatives that have a higher aesthetic value.
- Consider alternatives that minimize maintenance costs and increase durability.
- Consider alternatives that accommodate superstructure continuity to minimize the number of deck joints and bearings.
- Eliminate steel alternatives as requested by ALDOT.
- Consider span ranges that capture the best solution for the bridge height.
- Consider alternatives with span lengths that have historically been competitive with steel.
- Consider alternatives that accommodate bridge curvature and gore areas.
- Due to the limited tangent portions of the alignment eliminate spliced precast I-beams.

Similar to the main span unit studies, high level approach spans bridge types were studied to determine favorable span arrangements and structure depth which meet project requirements, expectations and provide economy. Determination of the appropriate structure type requires a complete analysis of site geometry, condition, and bridge layout. In most situations, the placement of piers and abutments is constrained to particular locations due to conflicts with the features crossed by the bridge. Some portions of the structure, such as the area in the vicinity of the Austal property, have flexibility in placement of substructure units and selection of span lengths.

Table 3 and Figure 2 below summarize the superstructure types selected for further development.

Bridge Alternative	Max. Span Length (ft.)	Alignment	Span Continuity
1. Precast Concrete Segmental (Cantilever)	300-350	TAN/CURVED	Continuous
2. Precast Concrete Segmental (Cantilever)	280	TAN/CURVED	Continuous
3. Precast Concrete Segmental (Span-by-Span)	160	TAN/CURVED	Continuous
4. PCI Spliced U-Beam (U72)	220	TAN/CURVED	Continuous
5. PCI Spliced U-Beams (U96)	280	TAN/CURVED	Continuous
6. Florida U-Beam Simple Span (FUB-72) and PCI Spliced U-Beam (U72)	Simple -185 Continuous – 220	TAN/CURVED	Simple/Continuous
7. LaDOTD Bulb T (LG-78)	183	TAN/CURVED	Simple
8. Florida U-Beam (FUB-72)	160	TAN/CURVED	Simple
9. Steel Plate Girders	Not Considered. Steel Eliminated from Scope.		
10. AASHTO-PCI Bulb T (BT-72)	160	TAN/CURVED	Simple

Table 3: High Level Approach Spans Bridge Alternatives – Step 1

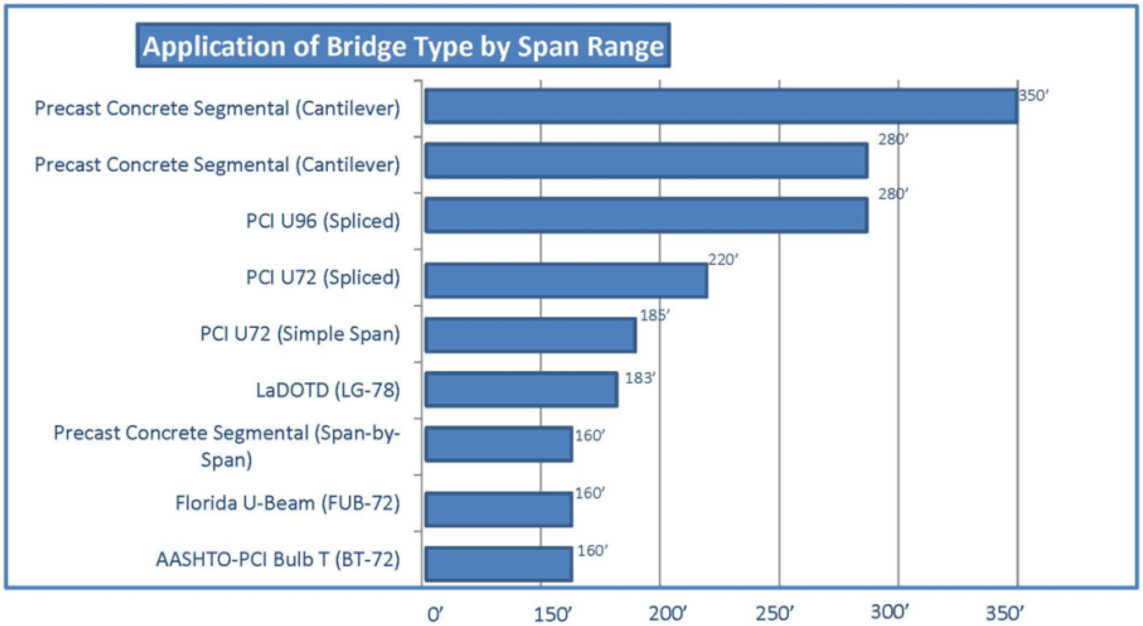


Figure 2: Application of Bridge Type by Span Range

Cross-section drawings for all nine concepts can be found in Appendix D of this technical memorandum.

4.1.4 Foundations

Concurrently with the high level approach spans bridge study, ALDOT commissioned the design team to assess the potential vibration impacts to historic properties close to the project. Per the DEIS, *"ALDOT will conduct a study to evaluate potential vibration impacts for pile driving and to help identify construction methodologies that would avoid vibration impacts to historic properties in proximity to the project".*

In addition to the goal stated above, this initial study also had the additional benefit of providing early indications of feasible foundation types including their:

- Initial construction costs.
- Constructability benefits and challenges.
- Construction schedule.

Based on the overall evaluation of the subsurface data obtained during this investigation and consideration of the project background information, it is anticipated that deep foundation systems will be required to support the bridge elements. Such systems may consist of large diameter drilled shafts, steel and concrete driven piles, or concrete filled driven pipe piles.

The foundation alternatives considered in the study included the following:

- 4-ft diameter drilled shafts
- 6-ft diameter drilled shafts
- 8-ft diameter drilled shafts
- 6-ft diameter steel pipe piles

- 30” PSC piles
- 36” PSC piles
- 54” diameter precast cylinder piles
- 60” diameter precast cylinder piles
- HP12x53 steel piles
- HP14x117 steel piles

Since several superstructure types were still under consideration during Step 1, it was not practical to develop foundation concepts for a multitude of structure types that may or may not be selected at the culmination of the study phase. To capture the feasible range of foundation sizes, the heaviest and lightest superstructure types for both the high level approach spans and main span superstructure were selected and the corresponding preliminary foundation loads calculated.

The high level approach spans study used these superstructures:

- Precast Bulb-T girders (lightest alternative)
- Segmental box girder (heaviest alternative)

Due to the early nature of the bridge study, preliminary foundation demands were estimated by combining results from previous similar projects and limited analytical modeling.

The preliminary foundation loads accounted for:

- Superstructure type
- Span length
- Bridge deck width
- Wind area
- Design wind speed

The completed I-10 Mobile River Bridge and Bayway Widening – West Side Foundation Alternatives Evaluation report is included in Appendix E.

4.1.5 Substructure - Pier Locations

Pier locations for each superstructure type were determined based on the following considerations:

- In general, piers are located to avoid existing and proposed roadways, and minimize impacts on the ground.
- Consistency - where possible, piers are located to maximize the number of equal length spans.
- Pier spacing that is conducive to optimizing each superstructure type will be considered.
- Straddle bents are required at pier locations where the crossing alignments are relatively coincident and the span capabilities of the superstructure alternative are maximized.
- Assume same Bayway tie-in location for all alternatives.

4.1.6 Substructure – Pier Types

Pier configurations were determined based on pier location, clearance requirements and anticipated gravity and lateral loads. Pier heights vary in height from 25 feet near the abutment to approximately 185 feet near the main span. In addition to height, there are gore areas where the bridge deck width is in excess of 110 feet.

To accommodate these project requirements the following pier types were considered:

- Multi-column piers – used for lower level and gore area piers supporting line-beams.
- Single column piers – used for higher level piers supporting line-beams and all segmental spans. Hollow and solid cross-section columns were considered.
- In several locations, straddle bents are used to limit span length or provide space for facilities crossing under the structure. Integral and non-integral caps were considered depending on the vertical clearance requirements and beam type of the supported spans. Integral pier caps are also used where required vertical underclearance precludes the use of a standard drop cap.

4.1.7 Substructure – Pier Caps

Pier caps have been preliminarily sized to have the cross sectional area and shape of a corresponding efficiently designed column. In general, the pier caps have been sized to maintain sufficient bearing area, adequate cross section for gravity loads, and depth of section for column reinforcing embedment

With the exception of straddle bents for segmental alternatives, all pier caps are non-integral. Integral or non-integral straddle bent pier caps are expected to require post-tensioned cast-in-place concrete construction in order to maintain an efficient cross section and reduce reinforcing congestion.

4.2 Bridge Evaluation

4.2.1 Bridge Evaluation Criteria

Table 4 below identifies the criteria that were used to evaluate the bridge concepts. Each feasible concept was evaluated in each of the six selection criteria. However, since Step 1 is a structure evaluation and not a type selection, a preferred structure type was not selected.

Qualitative and quantitative methods were used to evaluate the various structure concepts. With the exception of cost, a qualitative scoring scale was used to assign a score based on quantifiable numbers and engineering judgment to establish a relationship between the concepts. As illustrated in Table 5, a scale of good, fair and poor was used to rank each alternative.

Bridge Evaluation Criteria and Description

Criteria	Description
Constructability	This criterion was primarily used to rate the bridge types and their impact to the construction schedule, active rail lines and traffic lanes. The superstructure types that had the shorter construction duration, least impact to existing traffic, and required the least amount of construction falsework were rated higher. This criterion also rated the bridge superstructure on proposed material type and availability.
Maintenance and Durability	This criterion assesses the long term durability of bridge materials and the need and ease of maintenance.
Environmental Commitments	This criterion assesses compliance of alternatives with DEIS commitments such as clearance to Union Hall, Aesthetics, minimize construction vibration. The structure types that best comply with this criteria were rated higher. For aesthetics, generally box girders were rated higher than line-girder superstructures.
Design Considerations	This criterion assesses compliance of alternatives with project specific criteria such as clearance to county jail, tie-in to Bayway. The structure types that best comply with this criteria were rated higher.
Construction Schedule	This criterion was used to assess the construction duration of bridge types. The superstructure types that had the shorter construction duration were rated higher.
Comparative Construction Costs	This criterion was used to economically differentiate between the practical superstructure alternatives. Relative costs were based on adjusted historical unit cost information.

Table 4 - Bridge Evaluation Criteria and Description

Criteria	●	◒	○
Constructability	Good	Fair	Poor
Maintenance and Durability	Good	Fair	Poor
Environmental Commitments	Good	Acceptable	Not Acceptable
Design Considerations	Good	Fair	Poor

Table 5 - Sample Scoring Criteria

Multiple evaluation matrices were prepared and presented to ALDOT for the various concepts. These are discussed in the next sections.

4.2.2 Constructability

This criterion was primarily used to rate the bridge concepts based on their impact to the construction schedule, active rail lines and existing and proposed traffic lanes. The bridge types that had the shorter construction duration, least impact to traffic, and required the least amount of construction falsework/shoring were rated higher. This criterion also rated the bridge superstructure on proposed material type and availability.

Constructability

Bridge Alternative	Construction Access	Disruption to Existing Traffic	Established Construction Methods	Impacts to Existing Utilities	Availability of Precasters	Bridge Height	Overall
1. Precast Concrete Segmental (Cantilever)	◒	◒	●	●	◒	●	◒
2. Precast Concrete Segmental (Cantilever)	●	●	●	●	◒	●	●
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	◒	●	●
4. PCI Spliced U-Beam (U72)	◒	◒	◒	●	◒	◒	◒
5. PCI Spliced U-Beams (U96)	◒	◒	◒	●	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒	◒	◒	●	◒	◒	◒
7. LaDOTD Bulb T (LG-78)	◒	◒	●	◒	●	●	◒
8. Florida U-Beam (FUB-72)	◒	◒	●	◒	◒	●	◒
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	●	◒	●	●	◒

Table 6 – Constructability Evaluation of Alternatives

4.2.3 Maintenance and Durability

Long-term maintenance typically includes routine maintenance and replacement of wearing surfaces, bearings, and expansion joints. Expansion joints are located on the bridge deck and are used to provide articulation to the bridge as it expands and contracts through the temperature rise and fall cycles. Bearings are structural elements used to support the bridge superstructure and are located on top of the piers to transfer the loads from the superstructure to the piers.

Because joints and bearings typically contain movable parts, they have on-going maintenance costs and increased maintenance requirements as structures age. Minimizing the number of expansion joints and bearings decreases long-term maintenance costs. The number of joints and bearings required for each concept were evaluated.

Ease of access to inspect and maintain the bridge is also essential for the long-term health of the structure. To establish the minimum separation between structures for adequate snooper truck access, a series of clearance checks were done to ensure that the A-62 can reach the under deck portion of the bridge. Appendix F includes these study drawings.

In addition to bearings, joints and access, each concept was also evaluated on the feasibility of future deck replacement and widening.

Maintenance and Durability

Bridge Alternative	Access	Bearings.	Deck Joints	Deck Replacement	Future Widening	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	●	◒	◒	●
2. Precast Concrete Segmental (Cantilever)	●	●	●	◒	◒	●
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	◒	●
4. PCI Spliced U-Beam (U72)	◒	◒	●	◒	◒	◒
5. PCI Spliced U-Beams (U96)	◒	◒	●	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒	◒	●	◒	◒	◒
7. LaDOTD Bulb T (LG-78)	◒	◒	◒	●	●	◒
8. Florida U-Beam (FUB-72)	◒	◒	◒	●	●	◒
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	◒	●	●	◒

Table 7 – Maintenance and Durability Evaluation of Alternatives

4.2.4 Environmental Commitments

The commitments included in the DEIS will be legally binding and necessary for completion of the Mobile River Bridge and Bayway Widening project. This criterion assesses compliance of concepts with DEIS commitments such as avoidance of impacts to the Union Hall, aesthetics, and avoidance of vibration impacts to historic properties. The bridge types that best comply with this criteria were rated higher. For aesthetics, generally box girders were rated higher than line-beam superstructures.

4.2.5 Design Considerations

This criterion assesses compliance of alternatives with project specific criteria such as clearances to the county jail, future sidewalk, tie-in to Bayway, gore areas, curved alignment, and main span compatibility. The bridge types that best comply with this criteria were rated higher

Environmental Commitments

Bridge Alternative	Union Hall	Aesthetics	Permitting	ROW	Construction Vibration	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	●	●	●	●
2. Precast Concrete Segmental (Cantilever)	●	●	●	●	●	●
3. Precast Concrete Segmental (Span-by-Span)	◒	●	●	●	●	●
4. PCI Spliced U-Beam (U72)	●	●	●	●	●	●
5. PCI Spliced U-Beams (U96)	●	●	●	●	●	●
6. PCI Simple Span and Spliced U-Beam (U72)	●	●	●	●	●	●
7. LaDOTD Bulb T (LG-78)	◒	◒	●	●	●	◒
8. Florida U-Beam (FUB-72)	◒	●	●	●	●	●
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	●	●	●	◒

Table 8 – Environmental Commitments Evaluation of Alternatives

Design Considerations

Bridge Alternative	Clearance to Jail	Vert. Clearance	Foundation Size	Potential For Future Sidewalk	Curved Alignment	Gore Area	Bayway Tie-In	Main Span Compatibility	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	◒	◒	●	○	○	●	◒
2. Precast Concrete Segmental (Cantilever)	●	●	◒	◒	●	○	○	●	◒
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	●	◒	◒	●	●
4. PCI Spliced U-Beam (U72)	●	●	●	●	●	○	◒	◒	◒
5. PCI Spliced U-Beams (U96)	●	●	●	●	●	○	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	●	●	●	●	●	●	◒	◒	●
7. LaDOTD Bulb T (LG-78)	●	●	●	●	◒	●	◒	◒	●
8. Florida U-Beam (FUB-72)	●	●	●	●	◒	●	◒	◒	●
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	●	●	●	●	◒	●	◒	◒	●

Table 9 – Design Considerations Evaluation of Alternatives

4.2.6 Comparative Construction Cost Estimates

This criterion was used to economically differentiate between the practical concepts. Relative costs were based on adjusted historical unit cost information. An initial comparative cost estimate was prepared to aid in screening of all bridge configurations developed in Step 1 and identify the three recommended alternatives to be carried forward to Step 2 of the process. Since limited structural design was performed during Step 1, the initial cost estimate consisted of limited potential bid items, recent bid tabs for similar projects, comparative regional costs and other historic data. This cost estimate was based on the major construction items required for each bridge type.

For each of the nine concepts developed in Step 1, a preliminary summary of quantities was developed to include structural only items expected to contribute significantly to the cost. These material quantities and an assumed construction method were the basis for the estimated comparative construction cost. The comparative construction cost was developed based on significant amount of historical data augmented with conceptual level design work with appropriate adjustments for inflation to current year and for geographic differences.

The comparative cost estimate approach consisted of the following general steps:

- Establish a combination of quantity based and historical unit costs.
- Prepare preliminary bridge layouts for each alternative to aid in the computation of quantities.
- Perform sensitivity study to establish impact of pier height on bridge costs.

Using the unit costs provided in Appendix G, the comparative cost estimate for each of the nine concepts under consideration was developed. Table 9 summarizes the comparative construction cost for the nine alternatives in 2017 dollars. These cost estimates include a 15% contingency; however, due to the preliminary nature of the concepts life-cycle costs are not included.

Bridge Alternative	Cost ($/sf)
1. Precast Concrete Segmental (Cantilever)	328
2. Precast Concrete Segmental (Cantilever)	295
3. Precast Concrete Segmental (Span-by-Span)	209
4. PCI Spliced U-Beam (U72)	207
5. PCI Spliced U-Beams (U96)	215
6. PCI Simple Span and Spliced U-Beam (U72)	215
7. LaDOTD Bulb T (LG-78)	178
8. Florida U-Beam (FUB-72)	167
9. Steel Plate Girders	N/A
10. AASHTO-PCI Bulb T (BT-72)	178

Table 10 - Comparative Construction Cost Estimates (2017 $/sf)

4.2.7 Construction Schedule

This criterion was used to assess the construction duration of bridge types. A qualitative assessment of construction durations was made using historical production rates of similar projects. The bridge types with shorter construction duration were rated higher.

Bridge Alternative	Ranking
1. Precast Concrete Segmental (Cantilever)	○
2. Precast Concrete Segmental (Cantilever)	◒
3. Precast Concrete Segmental (Span-by-Span)	●
4. PCI Spliced U-Beam (U72)	◒
5. PCI Spliced U-Beams (U96)	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒
7. LaDOTD Bulb T (LG-78)	◒
8. Florida U-Beam (FUB-72)	◒
9. Steel Plate Girders	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒

Table 11 – Construction Schedule Evaluation of Alternatives

5.0 Step 1 - Recommendations

During Step 1 of the bridge type evaluation process, nine preliminary bridge concepts for the high level approach spans were developed, evaluated and presented to ALDOT on September 14, 2016. As a result of this meeting, three preliminary bridge alternatives were recommended to be carried forward for further study. The three preliminary bridge alternatives represented all feasible bridge types and engineering solutions that addressed the ALDOT's criteria and best met the Step 1 objectives.

The recommend alternatives for further development in Step 2 of the bridge type evaluation process are:

- Precast Concrete Segmental (Span-by-Span Construction)
- U-beams (simple spans)
- U-beams (continuous spans)

Criteria	1. Precast Concrete Segmental (Cantilever)	2. Precast Concrete Segmental (Cantilever)	3. Precast Concrete Segmental (Span-by-Span)	4. PCI Spliced U-Beam (U72)	5. PCI Spliced U-Beams (U96)	6. PCI Simple Span and Spliced U-Beam (U72)	7. LaDOTD Bulb T (LG-78)	8. Florida U-Beam (FUB-72)	9. Steel Plate Girders	10. AASHTO-PCI Bulb T (BT-72)
	High Level Approach Spans Bridge Alternatives									
Initial Construction Cost	○	○	◒	◒	◒	◒	●	●	N/A	●
Construction Schedule	○	◒	●	◒	◒	◒	◒	◒	N/A	◒
Constructability	◒	●	●	◒	◒	◒	◒	◒	N/A	◒
Maintenance and Durability	●	●	●	◒	◒	◒	◒	◒	N/A	◒
Environmental Commitments	●	●	●	●	●	●	◒	●	N/A	◒
Design Considerations	◒	◒	●	◒	◒	●	●	●	N/A	◒
Overall	○	◒	●	●	◒	●	◒	◒	N/A	◒

Table 12 – Recommended Alternatives for Step 2

6.0 Step 2 – Develop Three Bridge Alternatives

During Step 2 of the bridge type evaluation process, the bridge types that were recommended for further consideration in Step 1 were refined for conformance with the design parameters and project guidelines. During this process, each of the three bridge alternatives was evaluated for construction cost, constructability/construction time, maintenance and durability, environmental commitments and design considerations.

The general tasks of Step 2 consisted of the following:

- Perform conceptual engineering on alternatives approved in Step 1.
- Refine for conformance to the design parameters and project guidelines.
- Prepare updated comparative construction cost estimates.
- Prepare and present updated evaluation matrix.

6.1 Conceptual Engineering

6.1.1 Number of Lanes

For consistency between all bridge alternatives, during Step 1 it was assumed each roadway was composed of four 12-foot-wide lanes and two 12-foot-wide shoulders. Per direction from ALDOT, this assumption was revised for Step 2. For conceptual engineering, three 12-foot-wide lanes and two 12-foot-wide shoulders were used for the continued development of the bridge alternatives. Refer to Appendix H for project cross-sections.

6.1.2 Foundations

For the high level approach spans, the West Side Foundation Alternatives Evaluation report (Appendix E) concluded that steel HP 14x117 piles are considered the most likely foundation type for these elements. This foundation type provides the lowest construction risks; lowest construction costs and lowest driving vibrations for the high level approach spans.

However, as noted in Section 4.1.4, during Step 1 preliminary foundations were prepared for only the lightest and heaviest superstructure types:

- Precast Bulb-T girders (lightest alternative)
- Segmental box girder (heaviest alternative)

At the time there were numerous superstructure and pile types under consideration and it was not practical to develop foundation concepts for all the possible combinations. Additionally, four lanes of traffic were assumed in Step 1. Therefore, it was necessary to develop HP14x117 foundation concepts for the remaining bridge alternatives in Step 2 using the revised guidelines.

The process of developing feasible deep foundation concepts involved the following general steps:

- Calculate preliminary foundation demands for the bridge types under consideration.
- Using the foundation demands and pile/shaft capacities provided, size the foundations elements for each alternative.

6.1.3 Substructure

Pier locations used during Step 1 were reviewed and revised as necessary using the latest horizontal alignment for the west side and the east tunnel interchange, the revised bridge width and latest maintenance of traffic phasing.

The typical pier shapes consist of a single column with a hammerhead cap for U-beams and a smaller cap for the trapezoidal segmental box girder. Caps were sized to maintain sufficient bearing area, adequate cross section for gravity loads, and depth of section for column reinforcing embedment With the exception of straddle bents for segmental alternatives all pier caps are non-integral. Refer to Appendix I for typical pier shapes.

6.1.4 Span Arrangement

Precast Concrete Segmental

For precast concrete segmental construction using the span-by-span construction technique the optimum span length historically has been approximately 150 feet. This construction method utilizes an assembly truss spanning between permanent piers to support precast segments prior to installation and stressing of post-tensioning tendons. Segments are delivered to the assembly truss from the completed spans or from the ground below. After all segments comprising a span are assembled, the post-tensioning tendons are installed and stressed.

Even though the majority of the high level approaches can be built using span-by-span construction, there are some locations where straddle bents are not feasible and the pier spacing required to avoid existing and future roads, utilities and historic properties requires a span length that would exceed the maximum span possible for span-by-span construction. These areas include spans at or near the county jail, Union Hall property and Austal parking lot.

Where longer spans are required, modified-balanced cantilever construction can be used. Unlike typical cantilever construction which consists of placing segments in a balanced fashion on each side of the pier and proceeding in two directions, modified-cantilever erects cantilevers in only one direction. The benefits of using the modified-balanced cantilever technique include:

- Same assembly truss can be used to build span-by-span and modified-cantilever spans.
- Span lengths can be selected to allow the use of same exterior cross-sectional dimensions thus maintaining a uniform appearance of the structure.
- Maintaining the same cross-section promotes the efficient use of the casting-yard.

U-Beams

Similar to the precast concrete segmental alternative the typical simple U-beam span is in the range of 140 feet. A large portion of the high level approaches can be accommodated in this span range.

In locations where this span length is not sufficient to avoid existing at-grade facilities, continuous units with spliced U-beams can be used to achieve spans in the 200 feet to 240 feet range. Haunched pier sections can further be used to extend span range to approximately 320 feet. These areas include spans at or near the county jail, Union Hall property, and east tunnel interchange.

Another benefit of span continuity is that curved U-beams can be used in curved sections of the alignment thus maintaining a smooth fascia for the entire length of the high level approach spans.

Simple span pretensioned U-beams and spliced precast U-beams are available in the same depths, thereby creating a seamless superstructure appearance while optimizing the use of each beam in appropriate areas of the bridge.

6.2 Constructability

The precast segmental and U-beam alternatives were evaluated with respect to their erection schemes and potential impacts to the project study area. Impacts to the environment, existing roads, proposed roads, railroads and utilities are generally created by the construction and location of new bridge foundations, any temporary falsework/shoring towers, large construction equipment such as cranes, and beam haulers.

The structure type alternatives were also evaluated based on their ability to maintain traffic under and adjacent to the bridge during construction. Delivery of construction materials and bridge components, crane placement, and arrangement of the bridge deck erection equipment were taken into consideration. Structure type alternatives that utilized top down construction methods that minimized disruptions to traffic were preferred.

Precast Concrete Segmental

In evaluating the constructability of the segmental alternative, consideration was given to the delivery of the precast segments and temporary vertical clearances required for any overhead construction equipment

One of the advantages of precast segmental construction over other structure types is the ability to develop construction schemes that minimizes at-grade impact. Span-by-Span construction, unlike construction of other structure types, can be achieved with the use of ground based cranes or overhead launching gantries. Ground based cranes for this project are not considered desirable therefore our studies focused on other means of constructing the segmental alternative.

One technique that has been successfully used worldwide, including several major projects in the United States, is the use of overhead launching gantries.

With an overhead gantry:

- Bridge segments are delivered along completed deck.
- Most work proceeds above the terrain as well; therefore little to no disruption is caused to existing traffic, railroads, etc.
- All temporary loads are introduced directly into piers, there is no need for temporary shoring towers.
- Overhead gantries can build span-by-span and cantilever spans.
- Overhead gantries can accommodate curved alignments and gore areas.
- Can achieve faster segment erection and provide schedule savings.

A similar construction sequence is being used for the construction of the precast segmental approaches on the Bayonne Bridge in New York designed by HDR. As seen in Figures 3 and 4, the overhead gantry is supported by the permanent piers and the precast segments are delivered with a segment hauler on the previously completed spans.

Figures 5 and 6 below illustrate the key steps for span-by-span construction with launching gantry.

The use of an overhead gantry for construction of the precast segmental spans will mitigate impacts to existing vehicular and railroad traffic. The precast segments can be delivered to the gantry utilizing the previously completed spans thus eliminating the need for large cranes and segment haulers competing for space with vehicular and railroad traffic below.

Appendix J includes the construction sequence and clearance studies prepared by Armeni Consulting Services, LLC for the segmental alternative.

Figure 3 - Bayonne Bridge Overhead Gantry

Figure 4 - Bayonne Bridge Segment Delivery

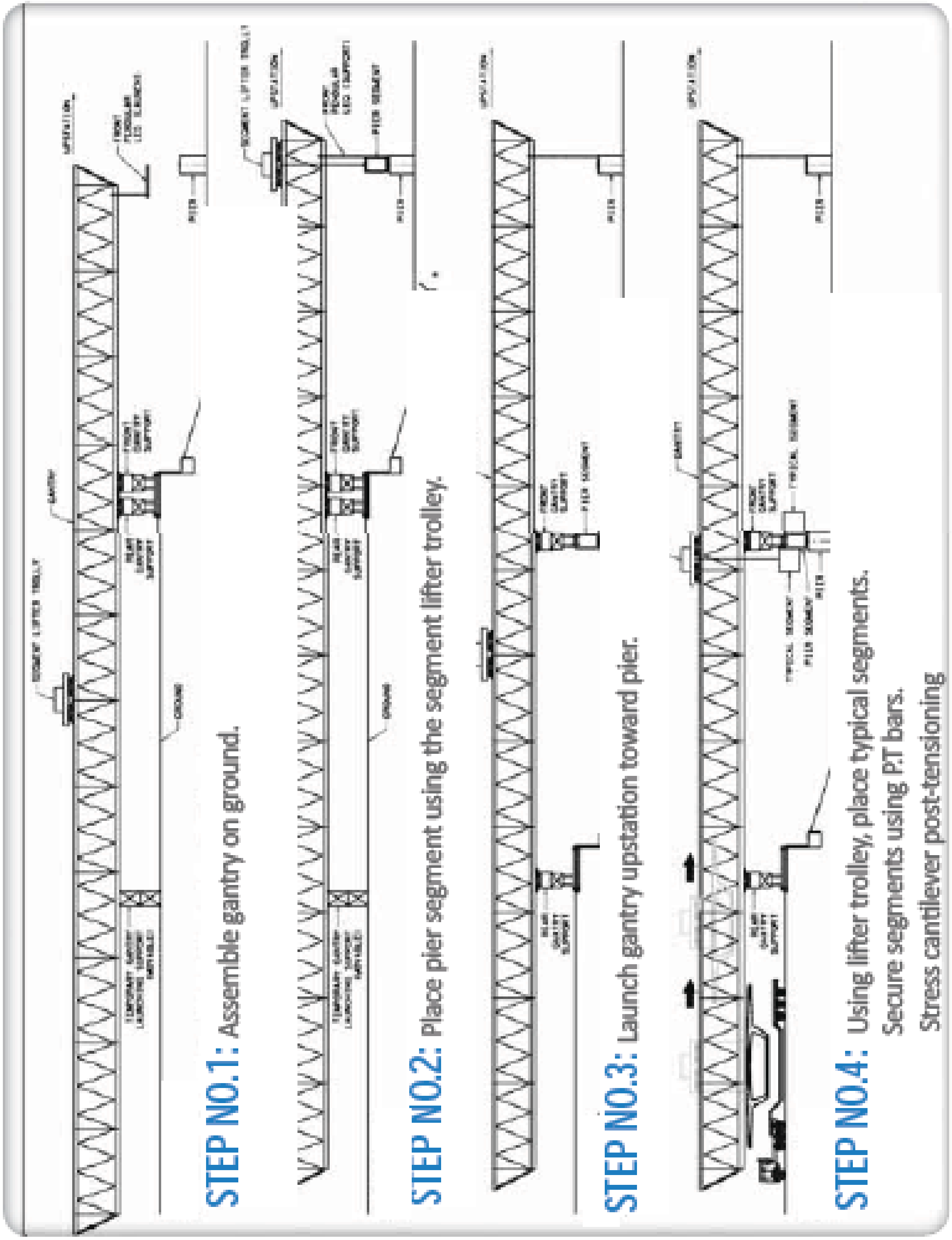


Figure 5 - Steps 1-4 Span-by-Span Construction with Overhead Gantry

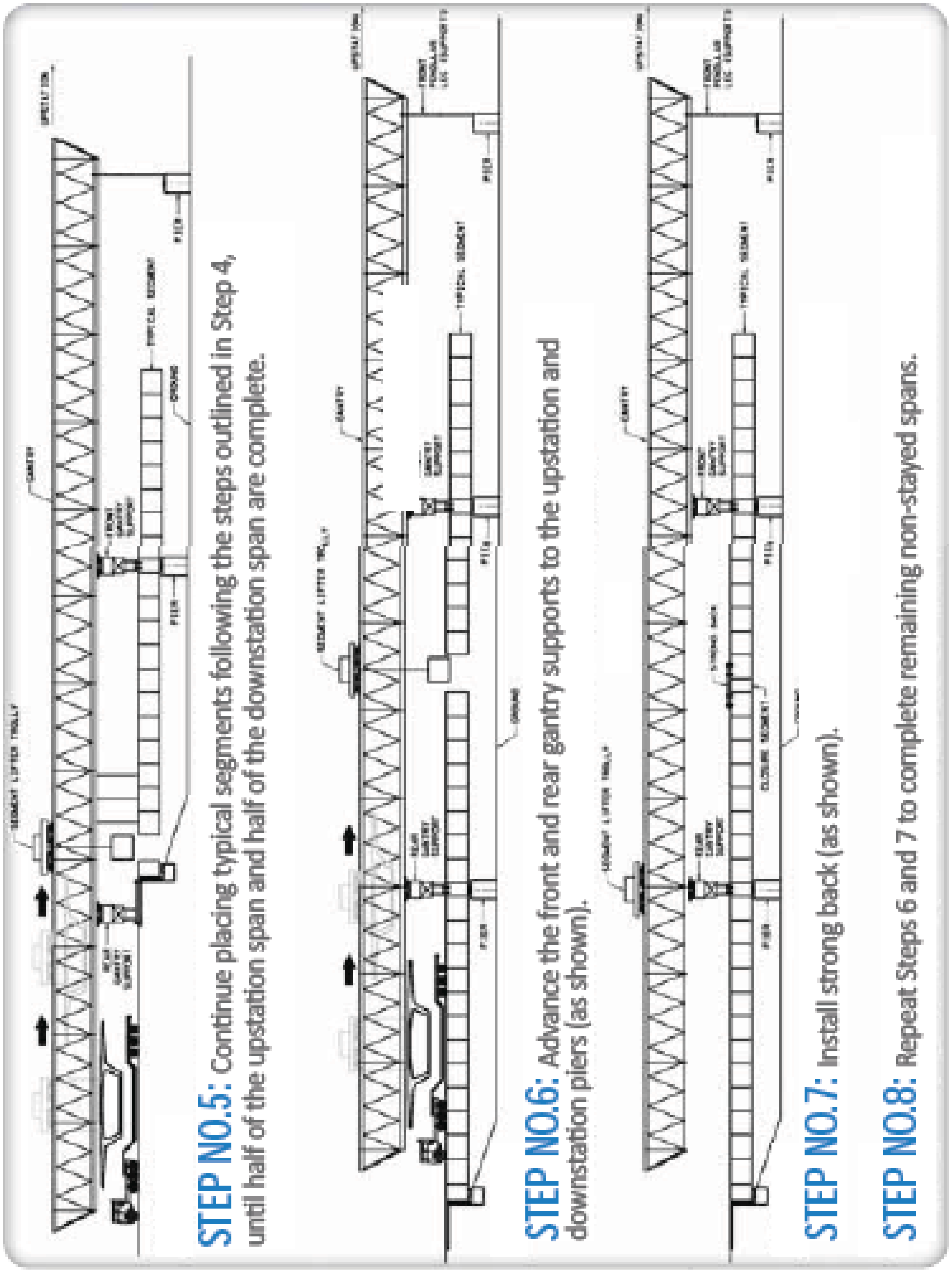


Figure 6 - Steps 5-8 Span-by-Span Construction with Overhead Gantry

U-Beams

Construction of precast U-beam bridges with simple spans in the span range and height required for the high level approach spans typically consists of two crawler cranes lifting the beam from the beam hauler and placing it on the bent cap. It is necessary to position the cranes relative to the structure and provide the haul roads or temporary MOT access necessary to deliver the beams and for the cranes to maneuver. This process is efficient and quick when there is enough space to maneuver large cranes and trucks.

In Appendix K, Armeni Consulting Services, LLC developed a series of figures to estimate crane size and placement, haul road width, and illustrate the major steps in pier construction sequence and superstructure erection sequence utilizing ground based cranes.

Where spliced beams are used to achieve longer spans, construction is typically more expensive than traditional girder construction. This construction method requires temporary shoring to support the pier section in place prior to setting the middle or end sections using strongback beams. Once the closure joint is cast and the longitudinal post-tensioning is complete the shoring towers can be removed.

In Appendix L, Armeni Consulting Services, LLC developed a series of figures to illustrate feasible ways to place construction equipment and temporary supports to build continuous spans at critical project locations such as the east tunnel interchange and the county jail and Union Hall property on the west side of the Mobile River.

The typical construction sequence of spliced beams consists of the following steps:

- The over-the-pier precast girders are erected on piers and temporary falsework – tall temporary towers will be required for the high level units. In situations where at-grade space is limited, these towers will also include temporary steel beams to straddle existing roadways and other obstacles.
- Mid-span girder sections are lifted and temporarily secured by falsework or strongbacks.
- The precast girders are made continuous by casting a splice between the pier and mid-span girder sections.
- Post-tensioning is performed in one or two stages. Single stage post-tensioning is completed prior to deck placement. In two stage post-tensioning, Stage 1 is carried out prior to casting of the deck slab and stage 2 is performed after casting of the deck slab.
- Traffic barriers are cast and future wearing surface is added.

Since large ground construction equipment is not desirable, alternative construction schemes were investigated for construction of the U-beams. One alternative that has been successfully used in other projects where it is not possible or desirable to have construction equipment at-grade is the use of a deck mounted launcher. Deck mounted launchers function in a manner similar to segmental construction. The launcher is supported by the permanent piers and the beams are trucked to the site and delivered to the launcher using the previously completed structure, eliminating the need for large cranes and segment haulers on the ground. This system is typically slower than crane based construction and can be very costly.

Deck mounted beam launchers can be effective for the simple U-beam spans in the high level approach spans. One disadvantage of the beam launchers is that they are typically limited to a span length of approximately 140 feet, consequently, for construction of long span continuous units with spliced U-beams temporary shoring will still be required.

Construction of this structure type requires good at-grade access for crane placement, delivery of precast sections and for installation of the temporary shoring towers required to set the beams.

6.3 Maintenance and Durability

Precast Concrete Segmental

An important aspect of the precast segmental span-by-span alternative is that provisions are made to provide additional structural capacity by adding another post-tensioning tendon in the future if it is needed. This capability to increase load capacity is not provided by beam bridges.

Deck replacement is not typically required during the bridge's life span for the precast segmental box girder structure type. This structure type is designed to minimize cracking and prevent chlorides from penetrating the concrete and reaching the reinforcing steel through a compressed deck design. The combination of the external longitudinal tendons and the internal transverse tendons provided in the bridge deck creates a bi-directionally post-tensioned structure that is much more durable than cast-in-place concrete decks. Cast-in-pace decks used in beam bridges need to be completely replaced about every 50 years, whereas the concrete box girder deck should last the full life of the bridge. Long continuous units for this alternative minimize the number of expansion joint locations. Precast segmental box girder structures also require fewer bearings than beam bridges.

Precast segmental box girders offer ease of inspection, the inside of the box girder can be inspected with no special equipment. An access hatch with ladder entry is provided at the bottom slab near abutments. This access hatch is secured with a metal door and lock. The exterior can be inspected using a snooper truck.

U-Beams

Simple U-beam spans have a conventional cast-in-place deck, and the associated durability and life-span concerns. This deck, though, is relatively easy to replace with standard procedures and can be readily widened.

Spliced continuous U-beam spans have two options for deck construction, with differing impacts on durability. The first option is to place the deck after all post-tensioning of the U-beams is complete. Like simple spans, this deck is a superimposed dead load with similar durability and life-span concerns, but can be removed and replaced with only slightly modified procedures.

The second option is to place the deck after the first stage of U-beam post-tensioning, and then complete the second stage of post-tensioning when the deck has attained strength. The deck is longitudinally post-tensioned and becomes an integral load-carrying part of the superstructure. Similar to precast concrete segmental, the deck is more durable, resistant to cracking and chloride penetration. Deck replacement and future widening are significantly more complicated because of the post-tensioning.

Spliced U-beams have inspection access similar to precast segmental. An access hatch is located at the end piers of each continuous unit. This access hatch is secured with a metal door and lock. The exterior can be inspected using a snooper truck.

6.4 Comparative Construction Cost Estimates

Once the preferred alternatives were identified in Step 1, a second cost estimate was prepared for just those alternatives. Major construction item quantities were computed or revised and a 15% contingency cost was included to account for the preliminary level of design

Similar to the cost estimate in Step 1, the updated cost estimate consists of potential bid items, using recent bid tabs for similar projects, comparative regional costs and other historic data.

Using the unit costs provided in Appendix M, the comparative cost estimate for each of the alternatives under consideration was developed. Table12 summarizes the comparative construction cost for the alternatives in 2017 dollars. These cost estimates include a 15% contingency; however, due to the preliminary nature of the alternatives life-cycle costs are not included.

Bridge Alternative	Cost ($/sf)
1. Precast Concrete Segmental	270
2. Precast U-Beam	257

Table 13 - Comparative Construction Cost Estimates (2017 $/sf)

7.0 Step 2 - Recommendations

Design and construction of large infrastructure projects in urban areas can be a very complex task, where issues such as constructability, aesthetics, and environmental constraints are addressed concurrently. The purpose of this study is to provide structural recommendations to be used for the future planning and design development of the Mobile River Bridge project.

The evaluation presented herein is preliminary and is subject to verification based upon preliminary and final design. Various methods for constructing the high level approach spans were considered and although a number of different construction methods are considered feasible, certain methods of construction appear to offer advantages in terms of least risk and lowest probable cost. The recommended bridge type and construction methods are for the purpose of this feasibility study and for feasibility-level cost estimation. It is expected that upon preliminary and final design, these suggested methods will be refined and modified to reflect the final design requirements.

Precast 72 in. deep U-beams on hammerhead style bents supported by HP 14x117 steel piles is considered the most likely structure type for the high level approach spans. The majority of the span consists of simply supported beams; however, there are locations where curvature in the alignment or the need for a longer span required spliced precast curved U-beams.

This structure type provides:

- Lower construction costs vs. precast segmental span-by-span construction.
- Precasting facilities readily available in the Gulf Coast area.
- Method familiarity for local contractors.
- Flexibility in construction sequencing vs. precast segmental span-by-span which tends to be very linear.
- High aesthetic value to meet the DEIS commitments.
- Easily accommodates geometric constraints such as curved alignments, gore areas and longer spans where required.

Some bridge pier locations require the use of straddle bents. Straddle bents are required at pier locations where the crossing alignments are relatively coincident and the span capabilities of the superstructure alternative are maximized. Straddle bents are post-tensioned in order to minimize the span-to-depth ratio of the bent cap. At some locations, realignment of the bridge should be considered in order to eliminate the use of a straddle bent(s). The two bridge locations where straddle bents are required are the new westbound I-10 over the new westbound I-10 Business and the new westbound and eastbound I-10 at the east tunnel interchange.

The evaluations and preliminary recommendations presented in this memorandum have been formulated on the basis of preliminary concepts for the bridge and foundations. As such, all of the preliminary conclusions presented herein are considered appropriate for concept-level evaluations of the design and for concept-level cost estimating.

As this project moves towards the preliminary and final design phase, many factors may affect the proposed construction sequencing and the bridge selection criteria. The method of design and construction would have a large effect on the construction sequencing and packaging. The factors with the most effect on bridge selection are the foundation costs, the development of tangent/curved precast concrete box girder sections, and the development of precast concrete elements. In addition to this list, horizontal or vertical realignment of some portions of the alignment may further simplify some of the construction issues by reducing span lengths and eliminating some straddle bents. As with many complicated design and construction projects, the necessity for further study and design development is required in order to move the Mobile River Bridge project into the final design phase.

High Level Approach Spans Bridge Alternatives							
Bridge Alternative	Initial Construction Cost	Construction Schedule	Constructability	Maintenance and Durability	Environmental Commitments	Design Considerations	Overall
1. Precast Concrete Segmental	◒	●	●	●	●	●	●
2. Precast U-Beam	●	●	◒	◒	●	●	●

Table 14 – Recommended Alternative for Step 3

8.0 Step 3 – Develop Preferred Bridge Alternative for TS&L (Future Phase)

This step was removed from the current scope of work

Appendix A:

September 14, 2016 Meeting

- PowerPoint Presentation
- Meeting Attendance
- Meeting Minutes

Mobile River Bridge

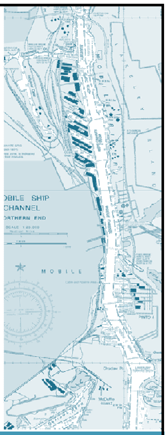
High Level Approach Spans

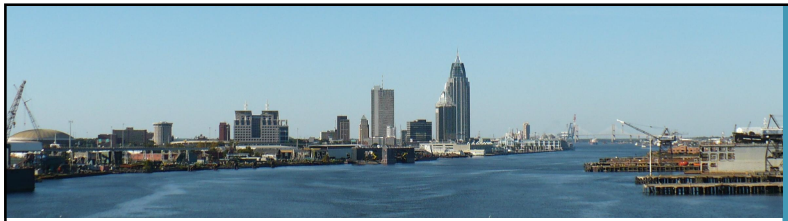
Bridge Type Selection Process

September 14, 2016

General Presentation Outline – High Level Approach Spans Bridge Type Selection Process

1. Scope of Work
2. Bridge Type Selection Process
3. Design Guidelines/Constraints
4. Superstructure Types Considered
5. Design Approach
6. High Level Approach Spans Bridge Alternatives
7. High Level Approach Spans Alternatives Evaluation
8. High Level Approach Spans Recommendations

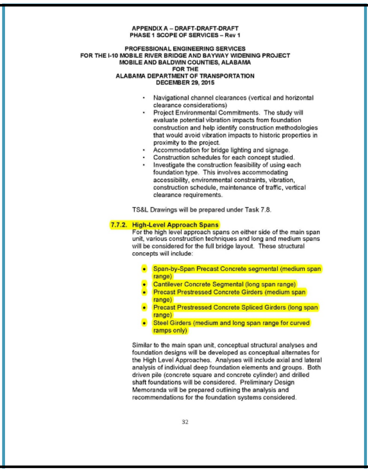




01 Scope of Work

1. Scope of Work

- Span-by-Span Precast Concrete segmental (medium span range)
- Cantilever Concrete Segmental (long span range)
- Precast Prestressed Concrete Girders (medium span range)
- Precast Prestressed Concrete Spliced Girders (long span range)
- Steel Girders (medium and long span range for curved ramps only)



APPENDIX A – DRAFT-DRAFT-DRAFT
PHASE 1 SCOPE OF SERVICES – Rev 1

PROFESSIONAL ENGINEERING SERVICES
FOR THE I-10 MOBILE RIVER BRIDGE AND BAYWAY WIDENING PROJECT
MOBILE AND BALDWIN COUNTIES, ALABAMA
FOR THE
ALABAMA DEPARTMENT OF TRANSPORTATION
DECEMBER 29, 2015

- Navigational channel clearances (vertical and horizontal clearance considerations)
- Project Environmental Commitments. The study will evaluate potential vibration impacts from foundation construction and help identify construction methodologies that would avoid vibration impacts to historic properties in proximity to the project.
- Accommodation for bridge lighting and signage.
- Construction schedules for each concept studied.
- Investigate the construction feasibility of using each foundation type. This involves accommodating accessibility, environmental constraints, vibration, construction schedule, maintenance of traffic, vertical clearance requirements.

TS&L Drawings will be prepared under Task 7.8.

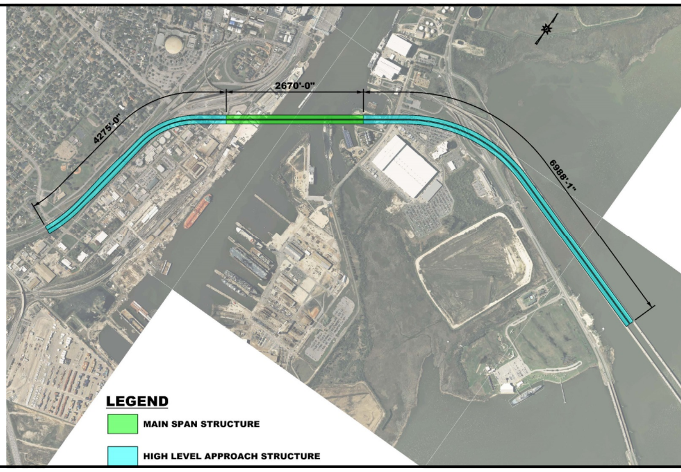
7.7.2. High-Level Approach Spans

For the high level approach spans on either side of the main span unit, various construction techniques and long and medium spans will be considered for the full bridge layout. These structural concepts will include:

- Span-by-Span Precast Concrete segmental (medium span range)
- Cantilever Concrete Segmental (long span range)
- Precast Prestressed Concrete Girders (medium span range)
- Precast Prestressed Concrete Spliced Girders (long span range)
- Steel Girders (medium and long span range for curved ramps only)

Similar to the main span unit, conceptual structural analyses and foundation designs will be developed as conceptual alternates for the High Level Approaches. Analyses will include axial and lateral analysis of individual deep foundation elements and groups. Both driven pile (concrete square and concrete cylinder) and drilled shaft foundations will be considered. Preliminary Design Memoranda will be prepared outlining the analysis and recommendations for the foundation systems considered.

32

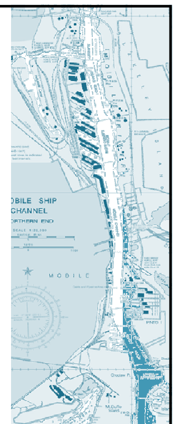


02 Bridge Type Selection Process

2. Bridge Type Selection Process

- Three Step Process which Involves Developing and Analyzing Numerous Bridge Concepts
- **Step 1 – Develop Preliminary Bridge Concepts**
- Step 2 - Develop 3 Bridge Type Alternatives
- Step 3 – Develop Preferred Bridge Alternative for TS&L

We are Here

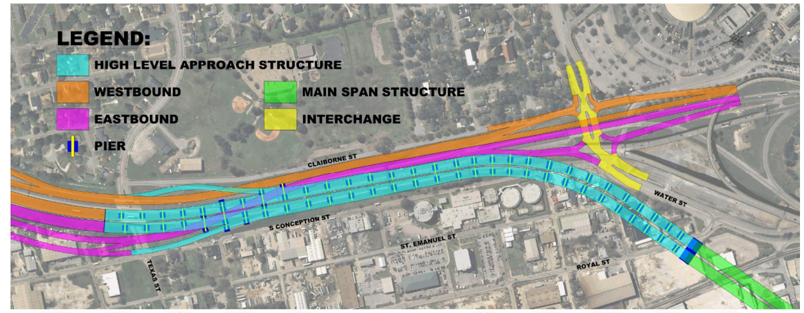


03 Design Guidelines and Constraints

3. Design Guidelines/Constraints

- Bridge Structural Configuration
  - Each Roadway is Composed of four 12-foot-wide Lanes and two 12-foot-wide shoulders (assumed for this initial ranking of Alternatives). Once Final determination is made revisions to layouts, cross-sections, costs estimates, etc. will be incorporated into the Preferred Alternative
- Union Hall
- Local Streets - Construction Phasing
- County Jail
- Inspection Access
- Tie-in To Bayway
- Bridge Height
- DEIS Environmental Commitments

3. Design Guidelines/Constraints



PLAN

Mobile River Bridge
Bridge Types – Evaluation Matrix

High Level Approach Span Criteria

Construction Cost	Constructability	Maintenance and Durability	Construction Schedule	Environmental Commitments	Design Considerations
Initial Construction Cost (for Ranking of Alternatives)	Construction Access Disruption to Existing Traffic / Maintenance of Traffic (Construction Phasing on the West Side) Established Construction Methods Impacts to Existing Utilities Shoring Tower Heights Availability of Local Precasters Erecting Precast Girders at 150ft (+) Heights	Accessibility Bearings Deck Joints Deck Replacement Flexibility for Future Widening	Construction Duration	Union Hall Contextual Design and Aesthetics Permitting ROW Construction Vibrations	Clearance to Jail Vertical Clearance Foundation Sizes Accommodates Future Sidewalk Chording of Spans Around the Curve Gore Area Local Streets Tie-in to Bayway

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04 Structure Types Considered

4. Structure Types Considered

- We Started with the Structural Design Manual

Structural Design Manual — *Section 5: Concrete Structures*

Bridge Component	Location	Clearance (Inches)
Bridge deck slab	Top	2
	Bottom	1
Columns	All faces	3
Caps	Top and bottom	2
Pile footing	From top of pile	4
Spread footing	Bottom	4
Abutments	Bottom	3
	Top of cap	2
	Faces of backwall	2
Drilled shafts	Sides	6

Table 5-2
Design Clearances (Concrete Cover) for Reinforced Concrete Structures

The following reinforcing steel spacing requirements shall be satisfied:

1. The minimum and maximum spacing of reinforcement shall be in accordance with *AASHTO LRFD* Article 5.10.3.

ALDOT exception to AASHTO:

The maximum spacing of flexural reinforcement shall not exceed 9 inches.

2. Where flexural reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above the bottom layer with layers not less than 4 inches on center.

Prestressed Concrete Girder Design Policy

The following policies shall be used for the design of prestressed concrete girders:

1. The following standard shape AASHTO-PCI type girders shall be used: Type I, Type II, Type III, BT-54, BT-63, and BT-72, as well as solid and voided slab beams. Modifications of these girders may be used under special circumstances (such as

February 2015 5-2

4. Structure Types Considered

- BT-72 using Simple Spans is the Baseline option for this Initial Comparison of Alternatives

Structural Design Manual — *Section 5: Concrete Structures*

clearance problems or freeboard limitations) when approved by the State Bridge Engineer.

2. All AASHTO-PCI type girders shall be designed as simple spans for all dead and live loads. In no case shall they be made continuous.
3. The transformed area of bonded reinforcement shall not be included in the calculations of section properties for prestressed concrete girders.
4. Pretensioning strands shall be 0.5 or 0.6 inches in diameter, low relaxation, 270 ksi Ultimate Tensile Strength. Use strand areas as follows:
   a. For 0.5" diameter low relaxation strand, use A = 0.153 square inches
   b. For special 0.5" diameter low relaxation strand, use A = 0.167 square inches
   c. For 0.6" diameter low relaxation strand, use A = 0.217 square inches
5. Prestressed concrete members should be designed so that no tension occurs after losses under the Service III load combination limit state. In no case shall tension in Service III conditions exceed 0.0948 $\sqrt{f'_c}$ (ksi). Exceptions to this policy shall require prior approval of the State Bridge Engineer.
6. Girder shear reinforcing steel shall not be spaced greater than 18 inches on center. Shear reinforcing steel in the girder ends shall be spaced 4 inches on center for a distance equal to the girder depth. Shear reinforcing steel shall be spaced so that no additional reinforcing steel is necessary to address horizontal shear forces at the top of the girder. Confining steel shall be #3 bars spaced at 4" and running a distance equal to the beam depth. Vertical shear steel shall be no smaller than #5 bars.
7. Calculation of camber due to prestress prior to pouring the bridge deck shall be based on a 60-day interval between release of the strand and erection of the girder.
8. Debonding of prestressed concrete girders shall be as given in AASHTO LRFD Section 5.11.4.3.
9. For calculating losses, use the *AASHTO LRFD Approximate Method*, neglecting gains. The following values shall be used for calculating losses:
   Time at release: 0.75 days
   Age of deck placement: 60 days
   Final age: 27500 days
   Relative Humidity: 75%

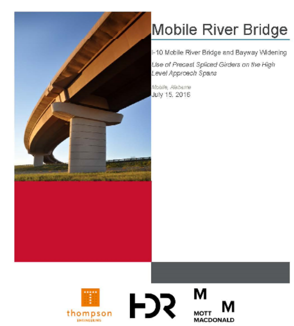
February 2015 5-3

4. Structure Types Considered - ALDOT

ESTIMATED BRIDGE COSTS - 2011					
SPAN LENGTH	PIER HEIGHT	ALIGNMENT	GIRDER TYPE	COST / SF	GDR. DEPTH
< 45'	< 30'	TAN - 10 DEG. CURVE	AASHTO T - I	$ 51.00	2.3
< 45'	< 30'	> 10 DEG. CURVE	STEEL W-BEAM	$ 58.00	3.0'
45' - 60'	< 30'	TAN - 10 DEG. CURVE	AASHTO T - II	$ 58.00	3.0'
45' - 60'	< 30'	> 10 DEG. CURVE	STEEL W-BEAM	$ 65.00	3.0'
60' - 80'	< 30'	TAN - 10 DEG. CURVE	AASHTO T - III	$ 65.00	3.8'
60' - 80'	< 30'	> 10 DEG. CURVE	STEEL W-BEAM	$ 73.00	3.0'
80' - 100'	< 30'	TAN - 5 DEG. CURVE	BT - 54	$ 73.00	4.5'
80' - 100'	< 30'	> 5 DEG. CURVE	STEEL PL GDR.	$ 87.00	4.0'
100 - 120	< 40'	TAN - 3 DEG. CURVE	BT - 63	$ 80.00	5.3'
100' - 120'	< 40'	> 3 DEG. CURVE	STEEL PL GDR.	$ 94.00	5.0'
100' - 120'	< 40'	> 3 DEG. CURVE	CONCRETE BOX	$ 102.00	6.0'
100' - 120'	40' - 60'	TAN - 3 DEG. CURVE	BT - 63	$ 87.00	5.3'
100' - 120'	40' - 60'	> 3 DEG. CURVE	STEEL PL GDR.	$ 102.00	5.0'
100' - 120'	40' - 60'	> 3 DEG. CURVE	CONCRETE BOX	$ 109.00	6.0'
120' - 140'	40' - 60'	TAN - 3 DEG. CURVE	BT - 72	$ 94.00	6.0'
120' - 140'	40' - 60'	> 3 DEG. CURVE	STEEL PL GDR.	$ 109.00	6.0'
120' - 140'	40' - 60'	> 3 DEG. CURVE	CONCRETE BOX	$ 113.00	6.0'
140' - 180'	60' - 80'	TAN	CONC. SPLICE GDR.	$ 109.00	6.5'*
140' - 180'	60' - 80'	ALL	STEEL PL GDR.	$ 116.00	6.5'
140' - 180'	60' - 80'	ALL	CONCRETE BOX	$ 116.00	6.5'
180' - 200'	80' - 100'	TAN	CONC. SPLICE GDR.	$ 116.00	6.5'*
180' - 200'	80' - 100'	ALL	STEEL PL GDR.	$ 122.00	7.0'
180' - 200'	80' - 100'	ALL	CONCRETE BOX	$ 122.00	8.0'
200' - 250'	80' - 100'	TAN	CONC. SPLICE GDR.	$ 122.00	6.5'*
200' - 250'	80' - 100'	ALL	STEEL PL GDR.	$ 130.00	9.0'
200' - 250'	80' - 100'	ALL	CONCRETE BOX	$ 130.00	9.0'
250' - 300'	80' - 100'	ALL	STEEL PL GDR.	$ 144.00	12.0'
250' - 300'	80' - 100'	ALL	CONCRETE BOX	$ 144.00	8.0'*
300' - 350'	80' - 120'	ALL	STEEL PL GDR.	$ 166.00	16.0'
300' - 350'	80' - 120'	ALL	CONCRETE BOX	$ 166.00	10.0'*
350' - 400'	80' - 120'	ALL	STEEL PL GDR.	$ 180.00	12.0'*
350' - 400'	80' - 120'	ALL	CONCRETE BOX	$ 180.00	10.0'*
			* MID-SPAN DEPTH OF HAUNCED GIRDER		

4. Structure Types Considered

- We Expanded our Search of Possibilities and made recommendations to ALDOT on other Feasible Structure Types including precast, spliced girders.
- The Paper presented the two most common structural configurations: spliced I-girders and spliced U-beams.



4. Structure Types Considered

- Spliced girder bridges are typically used for continuous structures in order to facilitate longer spans.
- In the past, spliced girders have been limited to straight concrete I-girder sections.
- New spliced U-girder technologies now allow for curved U-shaped bridge options. Several have already been successfully constructed in Colorado.

2.6 Summary

Type	Use	Pros	Cons
Spliced I-Girders	• Simple or continuous spans • Straight (possible curved application) • Section depth 7', up to 12' if haunched • 170' to 325' spans	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Can be used in curved alignment by providing a kink at splice • Regional precasters with experience and formwork	• Some standards are available; depending on beam type, may require web and end block modification • Less torsionally stiff than U-beams, therefore not curved • Requires shoring or strongbacks for erection until splicing is complete • Less aesthetic option than U-beams
Spliced U-beams	• Simple or continuous spans • Straight or curved • Section depth 6'-8', up to 11' if haunched • Radius down to 700' • Spans up to 300' • Minimum 2 beams per cross-section	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Fewer beams required in cross-section than with I-girders • Considered more aesthetic than I-girders • Standards are available (PCI Zone 6 Concept drawings) • Regional precasters with experience and formwork	• Heavy, therefore larger demand on substructure • For transport, field sections limited to approx. 100' • Super-heavy permits likely required for transport (approx. 115-125 tons per section) • Larger erection equipment necessary • Requires shoring or strongbacks until splicing is complete

4. Structure Types Considered

- Construction of these bridges includes:
  - Pre-casting either straight or curved u-shaped sections
  - Supporting the pier and drop in sections, and splicing the sections together on temporary supports using post-tensioning.
  - A lid slab is cast before the post-tensioning is applied to increase the torsional resistance of the section.
  - After the section is closed and stressed, conventional forms are placed between the boxes and a full depth deck is cast.

4. Structure Types Considered

- Advantages to this type of construction compared to conventional construction include:
  - Lower fabrication times
  - Faster construction
  - Ability to span longer distances
  - Increased aesthetics by providing a unified appearance

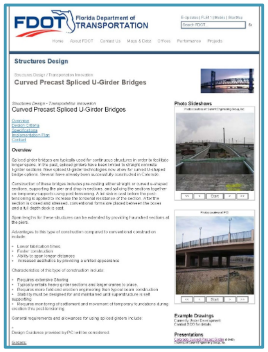
4. Structure Types Considered

- Characteristics of this type of construction include:
  - Requires extensive Shoring
  - Typically entails heavy girder sections and larger cranes to place.

4. Structure Types Considered

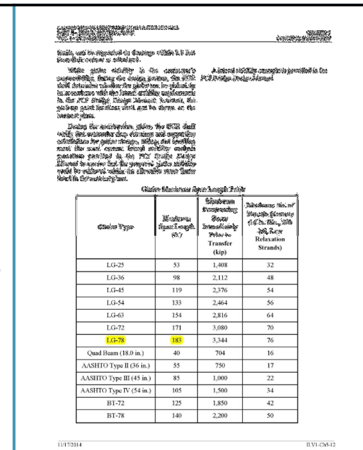
- Additional References provided in the Paper:
  - The Precast/Prestressed Concrete Institute (PCI) developed a set of concept drawings for curved, spliced U-Girders.
  - CDOT has been building spliced girder bridges since the early 1990s, including a site-precast curved U-beam bridge in 1995.
  - FDOT has adopted the PCI Zone 6 sections as the preferred spliced girder.

PCI
Precast/Prestressed Concrete Institute



4. Structure Types Considered

- AASHTO Type II, III, IV, BT-72, BT-78, are the standard shapes of prestressed concrete girders.
- Louisiana Girder Shapes LG-25, LG-36, LG-45, LG-54, LG-63, LG-72, and LG-78 are allowed with the approval of the Bridge Design Engineer Administrator.

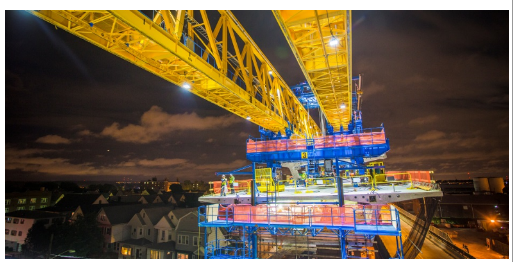


[illegible]	[illegible]	[illegible] Transfer (kip)	[illegible] Relaxation Strands)
LG-25	53	1,408	32
LG-36	98	2,112	48
LG-45	119	2,376	54
LG-54	133	2,464	56
LG-63	154	2,816	64
LG-72	171	3,080	70
LG-78	183	3,344	76
Quad Beam (18.0 in.)	40	704	16
AASHTO Type II (36 in.)	55	750	17
AASHTO Type III (45 in.)	85	1,000	22
AASHTO Type IV (54 in.)	105	1,500	34
BT-72	125	1,850	42
BT-78	140	2,200	50

11/17/2014

4. Structure Types Considered – Precast Segmental

- Construction Methods that Maintain Traffic
- Cost Competitive
- Geometric Flexibility – Curved and Straight Alignments
- High Aesthetic Value
- Local Labor and Materials
- "Factory Like" Quality
- Minimum Maintenance



4. Structure Types Considered – Summary

Considered = ✓ Not Considered = ○

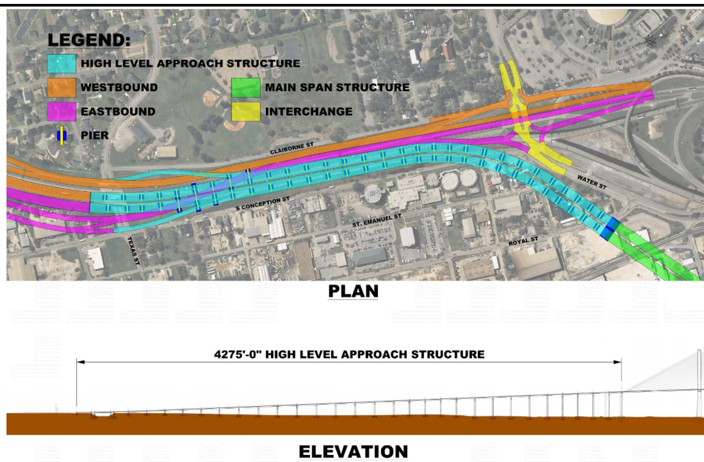
Bridge Alternative	Structure Types: Simple Span	Spliced/ Continuous	Straight	Curved
Precast I Girders	✓	○	✓	○
Precast U Girder	✓	✓	✓	✓
*Steel Plate Girder	○	○	○	○
Precast Segmental	✓	✓	✓	✓

* Eliminated from scope; applies to curved ramps only

05 Design Approach

5. Design Approach

- Identify Girder Types to be Studied
- For Girders not commonly used by ALDOT research and review details and practices from other Owners including their project Experience.
- Establish Preliminary Span Layouts Maximizing each Structure Type Strengths
- Key Assumptions:
  - Project Alignment
  - Number of Lanes on Bridge
  - Tie-in Geometry (terminus) with Bayway
  - Construction Phasing
  - No Bike/Ped Facility

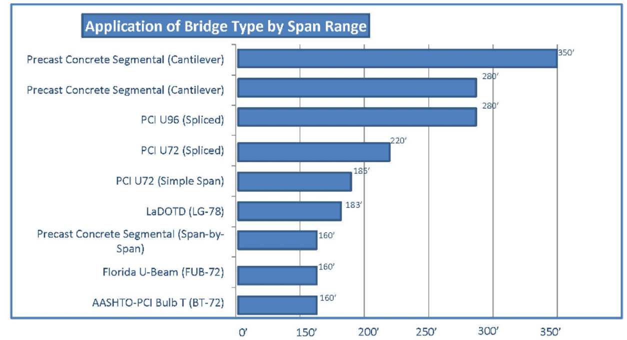


06 High Level Approach Spans Alternatives

6. High Level Approach Spans Alternatives

- After Reviewing ALDOT Structural Design Manual and the references previously Mentioned, the following Guidelines were developed:
  - o Baseline Solution Uses ALDOT's preferred Girders and Structural Design Criteria
  - o Alternatives that have a higher aesthetic value
  - o Alternatives that minimize maintenance costs and increase durability
    - Superstructure continuity
    - Eliminate steel alternatives
  - o Span ranges that capture the best solution for our bridge height
  - o Alternatives with span Lengths that have Historically been competitive with Steel
  - o Alternatives that Accommodate Bridge Curvature
    - Eliminate spliced I-Girders

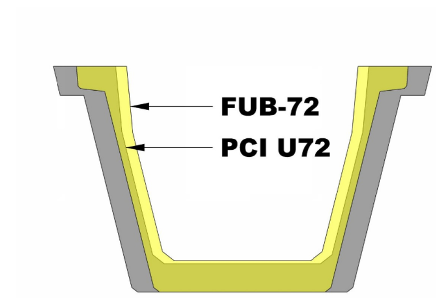
6. High Level Approach Spans Alternatives – Span Range

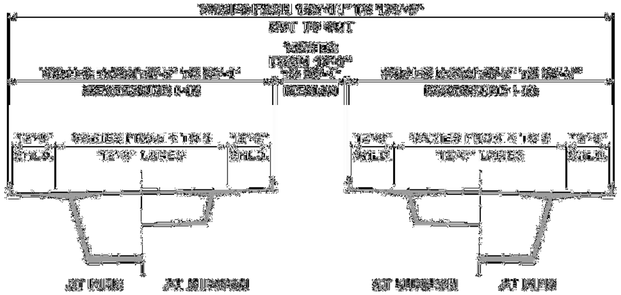


6. High Level Approach Spans Alternatives - Summary

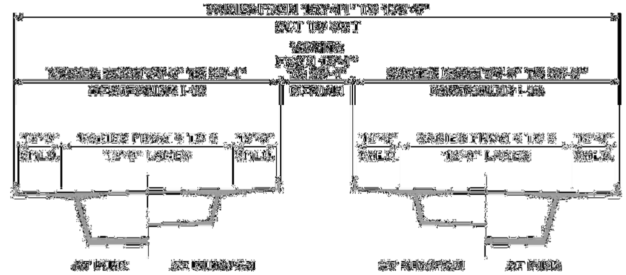
Bridge Alternative	Max. Span Length (ft.)	Alignment	Span Continuity
1. Precast Concrete Segmental (Cantilever)	300-350	TAN/CURVED	Continuous
2. Precast Concrete Segmental (Cantilever)	280	TAN/CURVED	Continuous
3. Precast Concrete Segmental (Span-by-Span)	160	TAN/CURVED	Continuous
4. PCI Spliced U-Beam (U72)	220	TAN/CURVED	Continuous
5. PCI Spliced U-Beams (U96)	280	TAN/CURVED	Continuous
6. Florida U-Beam Simple Span (FUB-72) and PCI Spliced U-Beam (U72)	Simple -185 Continuous – 220	TAN/CURVED	Simple/Continuous
7. LaDOTD Bulb T (LG-78)	183	TAN/CURVED	Simple
8. Florida U-Beam (FUB-72)	160	TAN/CURVED	Simple
9. Steel Plate Girders	Not Considered. Steel Eliminated from Scope.		
10. AASHTO-PCI Bulb T (BT-72)	160	TAN/CURVED	Simple

6. High Level Approach Spans Alternatives

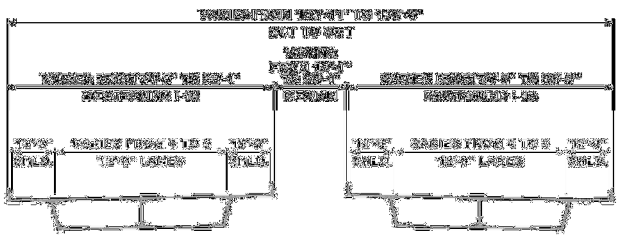




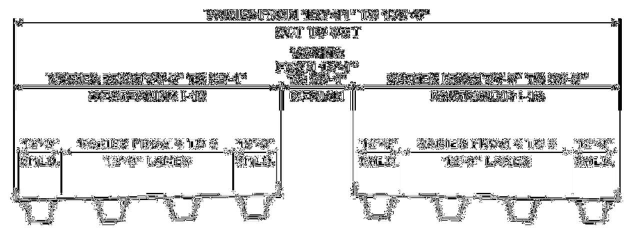
OPTION 1 - BALANCED CANTILEVER



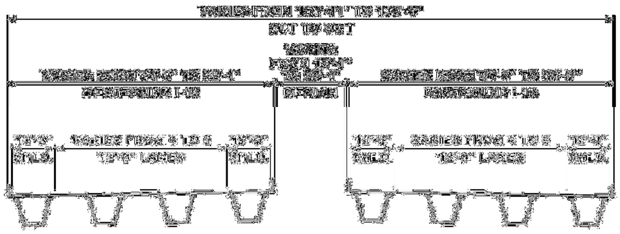
OPTION 2 - BALANCED CANTILEVER (GANTRY)



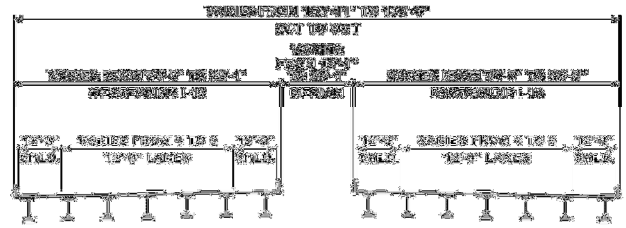
OPTION 3 - SPAN BY SPAN



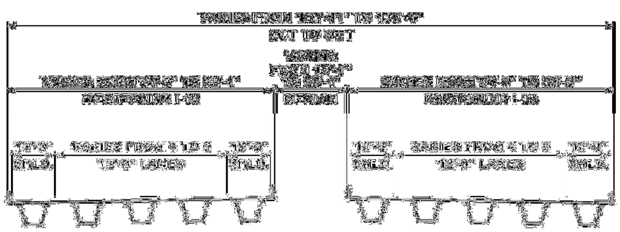
OPTION 4 - SPLICED U72 BEAMS



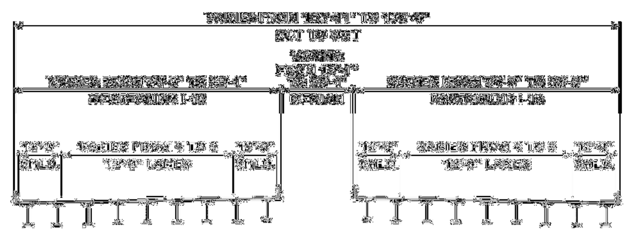
OPTION 5 - SPLICED U86 BEAMS



OPTION 7 - SIMPLE SPAN LG-70 BEAMS



OPTION 8 - SIMPLE SPAN FUB-72 BEAMS



OPTION 10 - SIMPLE SPAN BT-72 BEAMS

07 High Level Approach Spans Concept Evaluation

7. High Level Approach Span Concepts Evaluation

- Qualitative and quantitative methods were used to evaluate the various structure alternatives
- With the exception of cost, each criterion was assigned a "bubble" based on quantifiable numbers and engineering judgment to establish a relationship between the alternatives.

Criteria	●	◒	○
Constructability	Good	Fair	Poor
Maintenance and Durability	Good	Fair	Poor
Environmental Commitments	Good	Acceptable	Not Acceptable
Design Considerations	Good	Fair	Poor

7. High Level Approach Span Concepts Evaluation

- All of the High Level Approach Spans Concepts we ranked using the Evaluation Matrix

HDR
Mobile River Bridge
Bridge Types – Evaluation Matrix
High Level Approach Span Criteria

Construction Cost	Constructability	Maintenance and Durability	Construction Schedule	Environmental Commitments	Design Considerations
Initial Construction Cost (for Ranking of Alternatives)	Construction Access Disruption to Existing Traffic / Maintenance of Traffic (Construction Phasing on the West Side) Established Construction Methods Impacts to Existing Utilities Shoring Tower Heights Availability of Local Precasters Erecting Precast Girders at 150ft (+) Heights	Accessibility Bearings Deck Joints Deck Replacement Flexibility for Future Widening	Construction Duration	Union Hall Contextual Design and Aesthetics Permitting ROW Construction Vibrations	Clearance to Jail Vertical Clearance Foundation Sizes Accommodates Future Sidewalk Chording of Spans Around the Curve Gore Area Local Streets Tie-in to Bayway

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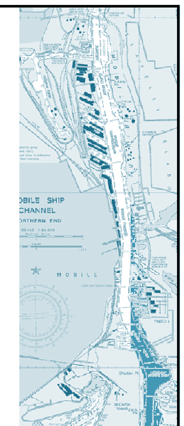
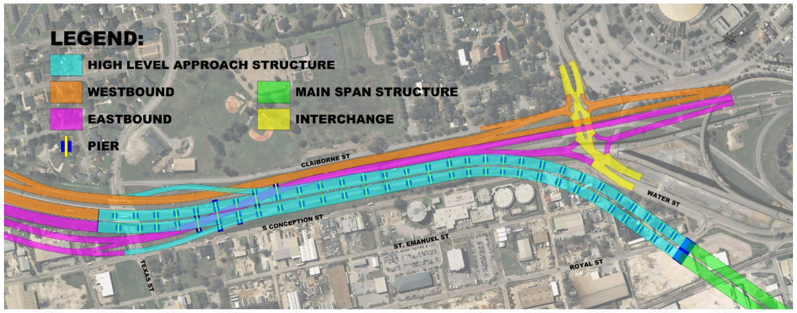
7. High Level Approach Span Concepts Evaluation – Constructibility

- Construction Access
  - How is Equipment and Material Delivered?
- Disruption to Existing Traffic/Maintenance of Traffic
  - Impacts to Existing Traffic During Construction – Construction Phasing
- Established Construction Methods
- Impacts to Existing Utilities
- Shoring Tower Heights
- Availability of Precasters (for I and U Girders)
- Bridge Height (Crane Placement)

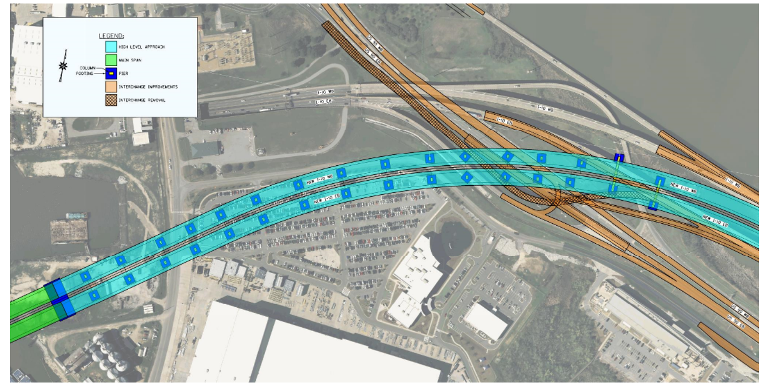


7. Constructibility – West Side



PLAN

7. Constructibility – East Side



7. Constructibility – Precast Segmental (Span-by-Span)

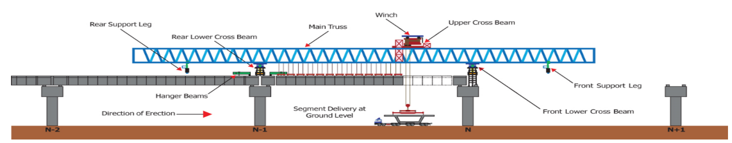
Segments Can Be Erected Using:

Under Slung Truss

Overhead Truss

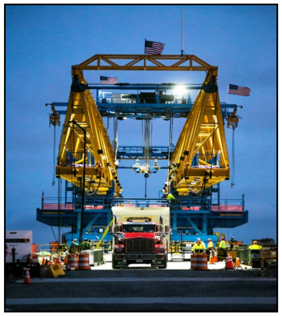
(Segment Delivery Can Be From Above or Below)

7. Constructibility – Precast Segmental (Span-by-Span)



Span-by-Span Construction with Overhead Truss
(Segment Delivery Can Be From Above or Below)

7. Constructibility – Precast Segmental (Span-by-Span)



Segment Delivery Can Be From Above

7. Constructibility – Precast Segmental (Span-by-Span)

Span-by-Span Construction with Underslung Truss
(Segment Delivery From Below)

7. Constructibility – Precast Segmental (Span-by-Span)

Span-by-Span Construction with Underslung Truss
(Segment Delivery From Above)

7. Constructibility – Precast Segmental (Span-by-Span)

Span-by-Span Construction with Underslung Truss
(Segment Delivery From Below)

7. Constructibility – Precast Segmental (Cantilever)

Segments Can Be Erected Using:

Beam and Winch

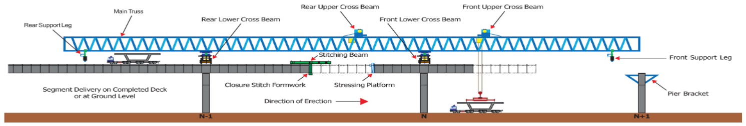
Overhead Truss

7. Constructibility – Precast Segmental (Cantilever)

Segments Can Be Erected Using:

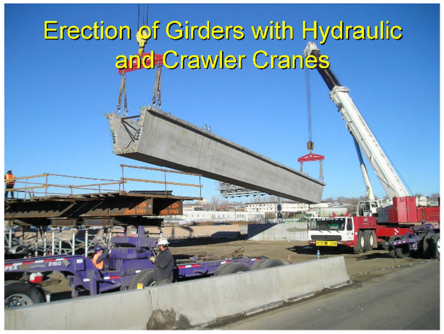
Cranes

7. Constructibility – Precast Segmental (Cantilever)



Cantilever Construction with Overhead Truss
(Segment Delivery Can Be From Above or Below)

7. Constructibility – Spliced U Girders



7. Constructibility – Spliced U Girders

7. Constructibility – Spliced U Girders

7. Constructibility – Spliced U Girders

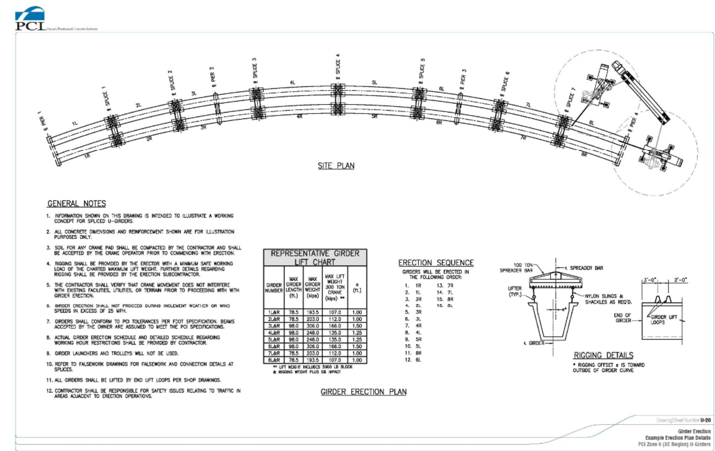
7. Constructibility – Spliced U Girders

7. Constructibility – Simple Span I and U Girders

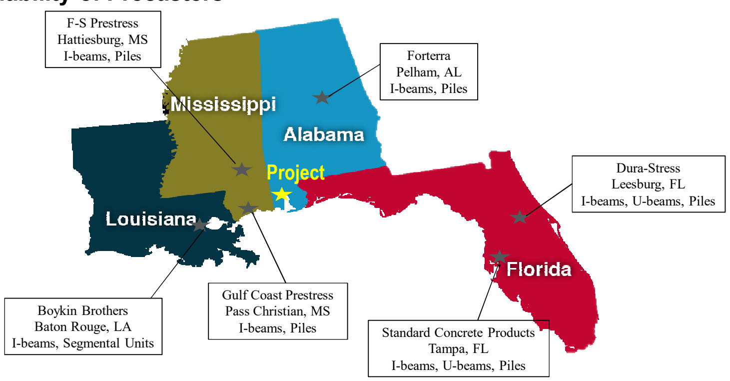
Girder Launchers:

From Below

From Above



7. High Level Approach Span Concepts Evaluation – Availability of Precasters



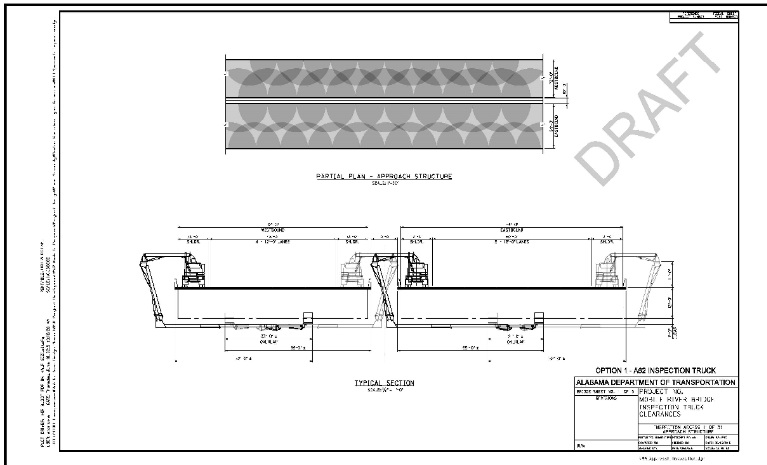
Constructability

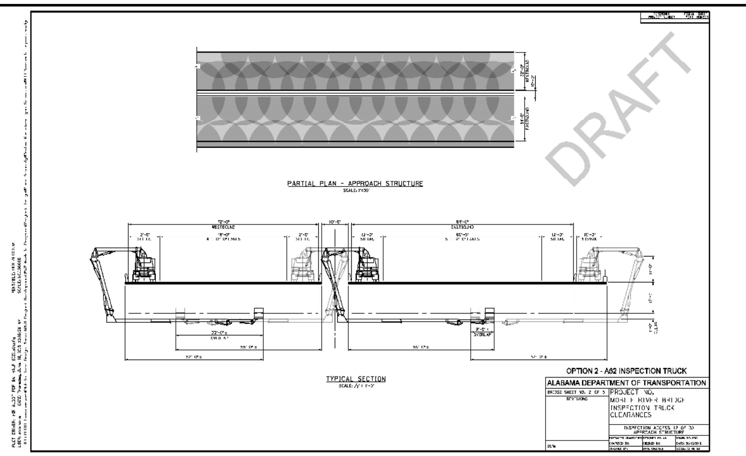
Good = ● Acceptable = ◒ Not Acceptable= ○

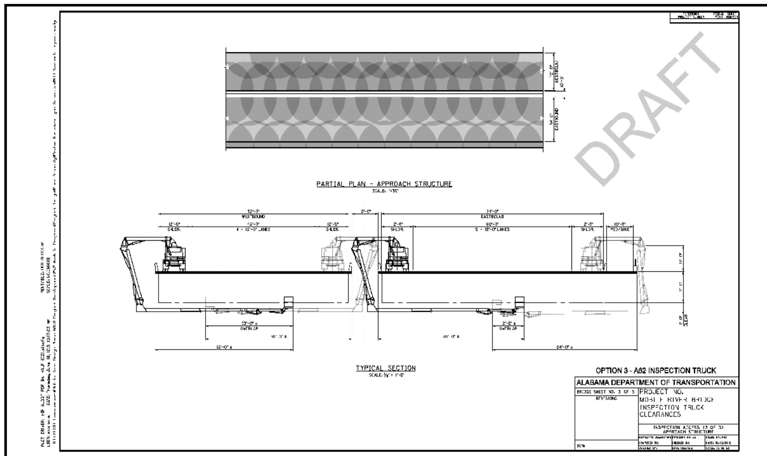
Bridge Alternative	Construction Access	Disruption to Existing Traffic	Established Construction Methods	Impacts to Existing Utilities	Availability of Precasters	Bridge Height	Overall
1. Precast Concrete Segmental (Cantilever)	◒	◒	●	●	◒	●	◒
2. Precast Concrete Segmental (Cantilever)	●	●	●	●	◒	●	●
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	◒	●	●
4. PCI Spliced U-Beam (U72)	◒	◒	◒	●	◒	◒	◒
5. PCI Spliced U-Beams (U96)	◒	◒	◒	●	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒	◒	◒	●	◒	◒	◒
7. LaDOTD Bulb T (LG-78)	◒	◒	●	◒	●	●	◒
8. Florida U-Beam (FUB-72)	◒	◒	●	◒	◒	●	◒
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	●	◒	●	●	◒

7. High Level Approach Span Concepts Evaluation – Maintenance and Durability

- Maintenance and Inspection Access
- Bearings
- Deck Joints
- Deck Replacement
- Future Widening







Maintenance and Durability

Good = ● Acceptable = ◒ Not Acceptable= ○

Bridge Alternative	Access	Bearings.	Deck Joints	Deck Replacement	Future Widening	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	●	◒	◒	●
2. Precast Concrete Segmental (Cantilever)	●	●	●	◒	◒	●
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	◒	●
4. PCI Spliced U-Beam (U72)	◒	◒	●	◒	◒	◒
5. PCI Spliced U-Beams (U96)	◒	◒	●	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒	◒	●	◒	◒	◒
7. LaDOTD Bulb T (LG-78)	◒	◒	◒	●	●	◒
8. Florida U-Beam (FUB-72)	◒	◒	◒	●	●	◒
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	◒	●	●	◒

7. High Level Approach Span Concepts Evaluation – Environmental Commitments

- Union Hall
- Contextual Design and Aesthetics
  - Superstructure Shape and Pier Spacing
- Permitting
- ROW
- Construction Vibrations

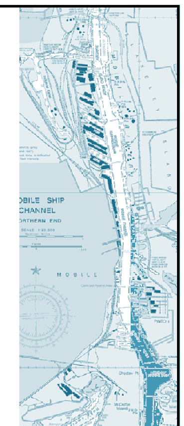
Environmental Commitments

Good = ● Acceptable = ◒ Not Acceptable= ○

Bridge Alternative	Union Hall	Aesthetics	Permitting	ROW	Construction Vibration	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	●	●	●	●
2. Precast Concrete Segmental (Cantilever)	●	●	●	●	●	●
3. Precast Concrete Segmental (Span-by-Span)	◒	●	●	●	●	●
4. PCI Spliced U-Beam (U72)	●	●	●	●	●	●
5. PCI Spliced U-Beams (U96)	●	●	●	●	●	●
6. PCI Simple Span and Spliced U-Beam (U72)	●	●	●	●	●	●
7. LaDOTD Bulb T (LG-78)	◒	◒	●	●	●	◒
8. Florida U-Beam (FUB-72)	◒	●	●	●	●	●
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒	◒	●	●	●	◒

7. High Level Approach Span Concepts Evaluation – Design Considerations

- Clearance to Jail
- Vertical Clearance
- Foundation Types and Sizes
- Accommodates Future Sidewalk
- Accommodates Curved Alignment
  - Chording of Spans Around Curve or curved Superstructure
- Gore Areas
- Tie-in to Bayway
- Compatibility with Main Span



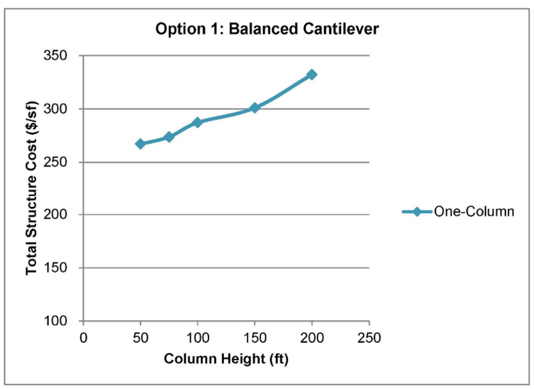
Design Considerations

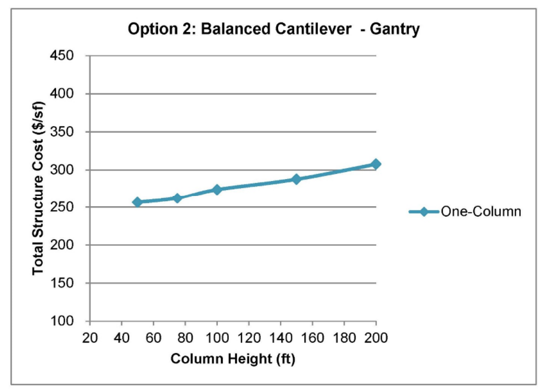
Good = ● Acceptable = ◒ Not Acceptable= ○

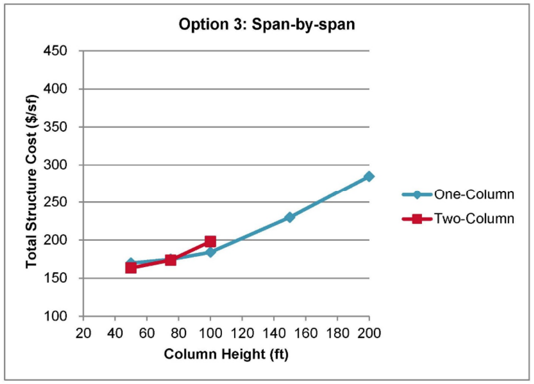
Bridge Alternative	Clearance to Jail	Vert. Clearance	Foundation Size	Future Sidewalk	Curved Alignment	Gore Area	Bayway Tie-In	M.S. Compatibility	Overall
1. Precast Concrete Segmental (Cantilever)	●	●	◒	◒	●	○	○	●	◒
2. Precast Concrete Segmental (Cantilever)	●	●	◒	◒	●	○	○	●	◒
3. Precast Concrete Segmental (Span-by-Span)	●	●	●	◒	●	◒	◒	●	●
4. PCI Spliced U-Beam (U72)	●	●	●	●	●	○	◒	◒	◒
5. PCI Spliced U-Beams (U96)	●	●	●	●	●	○	◒	◒	◒
6. PCI Simple Span and Spliced U-Beam (U72)	●	●	●	●	●	●	◒	◒	●
7. LaDOTD Bulb T (LG-78)	●	●	●	●	◒	●	◒	◒	●
8. Florida U-Beam (FUB-72)	●	●	●	●	◒	●	◒	◒	●
9. Steel Plate Girders	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10. AASHTO-PCI Bulb T (BT-72)	●	●	●	●	◒	●	◒	◒	●

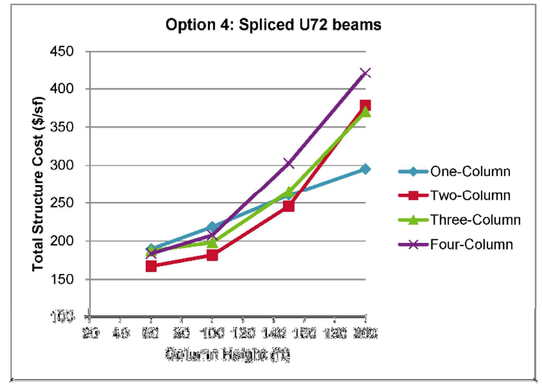
7. High Level Approach Span Concepts Evaluation – Initial Construction Costs

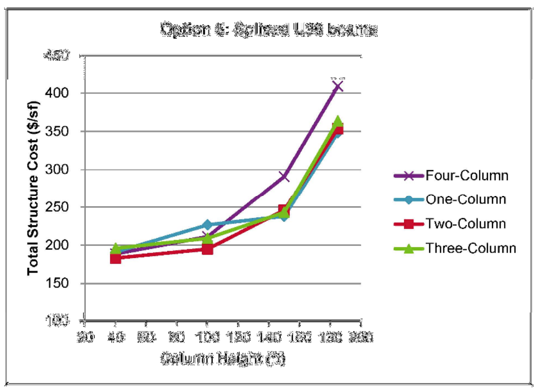
- Approach
  - Combination of Quantity Based and Historical Unit Costs
  - Proofed vs. Historical Costs of Similar Structures
  - Performed Sensitivity Study to Establish Impact of Pier Height on Bridge Costs

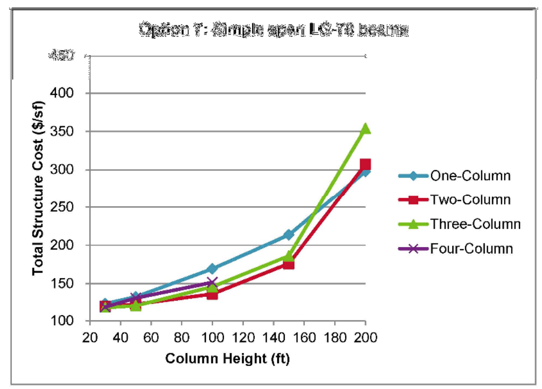


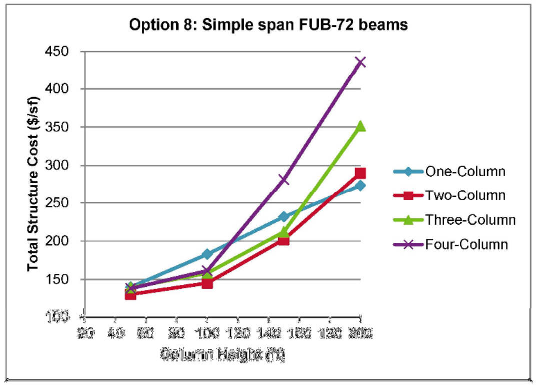


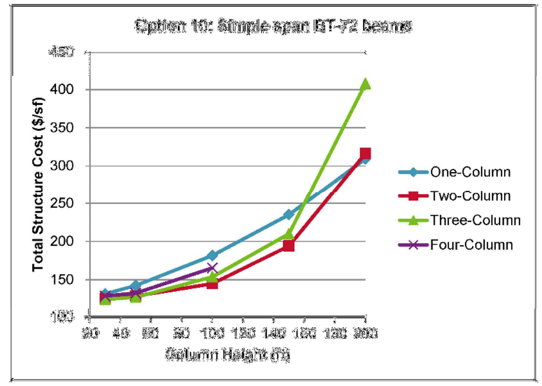

Option 3: Span-by-span
Total Structure Cost ($/sf)
450
400
350
300
250
200
150
100
20 40 60 80 100 120 140 160 180 200
Column Height (ft)
One-Column
Two-Column

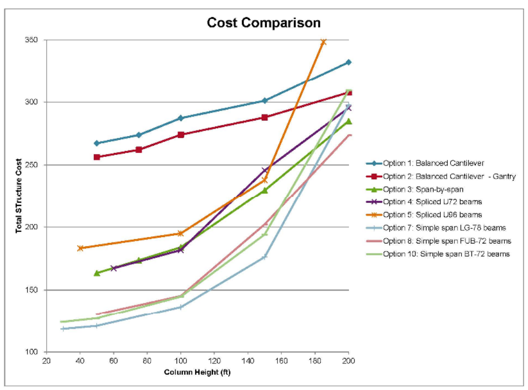

Option 4: Spliced U72 beams
Total Structure Cost ($/sf)
450
400
350
300
250
200
150
One-Column
Two-Column
Three-Column
Four-Column

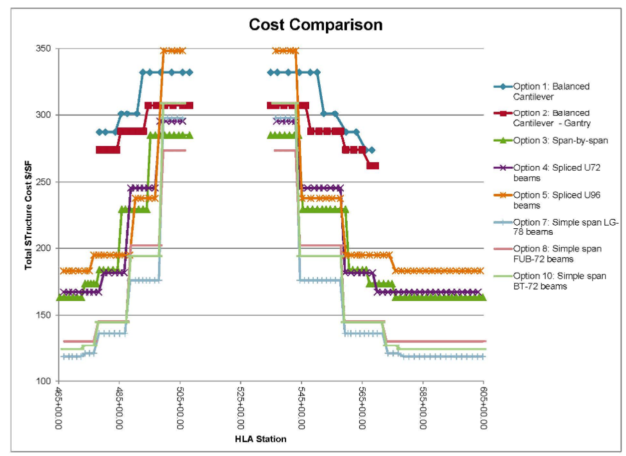

Total Structure Cost ($/sf)
400
350
300
250
200
150
Four-Column
One-Column
Two-Column
Three-Column


Total Structure Cost ($/sf)
400
350
300
250
200
150
100
20 40 60 80 100 120 140 160 180 200
Column Height (ft)
One-Column
Two-Column
Three-Column
Four-Column


Option 8: Simple span FUB-72 beams
450
400
350
300
250
200
150
Total Structure Cost ($/sf)
One-Column
Two-Column
Three-Column
Four-Column


400
350
300
250
200
150
Total Structure Cost ($/sf)
One-Column
Two-Column
Three-Column
Four-Column


Cost Comparison
350
300
250
200
150
100
Total STructure Cost
20
40
60
80
100
120
140
160
180
200
Column Height (ft)
Option 1: Balanced Cantilever
Option 2: Balanced Cantilever - Gantry
Option 3: Span-by-span
Option 4: Spliced U72 beams
Option 5: Spliced U96 beams
Option 7: Simple span LG-78 beams
Option 8: Simple span FUB-72 beams
Option 10: Simple span BT-72 beams


Cost Comparison
350
300
250
200
150
100
Total STructure Cost $/SF
465+00.00
485+00.00
505+00.00
525+00.00
545+00.00
565+00.00
585+00.00
605+00.00
HLA Station
Option 1: Balanced Cantilever
Option 2: Balanced Cantilever - Gantry
Option 3: Span-by-span
Option 4: Spliced U72 beams
Option 5: Spliced U96 beams
Option 7: Simple span LG-78 beams
Option 8: Simple span FUB-72 beams
Option 10: Simple span BT-72 beams

7. High Level Approach Span Concepts Evaluation – Initial Construction Cost

- 2017 Construction Costs

Bridge Alternative	Cost ($/sf)
1. Precast Concrete Segmental (Cantilever)	328
2. Precast Concrete Segmental (Cantilever)	295
3. Precast Concrete Segmental (Span-by-Span)	209
4. PCI Spliced U-Beam (U72)	207
5. PCI Spliced U-Beams (U96)	215
6. PCI Simple Span and Spliced U-Beam (U72)	215
7. LaDOTD Bulb T (LG-78)	178
8. Florida U-Beam (FUB-72)	167
9. Steel Plate Girders	N/A
10. AASHTO-PCI Bulb T (BT-72)	178

7. High Level Approach Span Concepts Evaluation – Initial Construction Cost

- 2017 Construction Costs
- Include 15% Contingency

* Cantilever Construction was only Considered for areas of the Project where the bridge height might Justify longer Spans

Bridge Alternative	Cost ($)M
1. Precast Concrete Segmental (Cantilever)	*
2. Precast Concrete Segmental (Cantilever)	*
3. Precast Concrete Segmental (Span-by-Span)	410
4. PCI Spliced U-Beam (U72)	406
5. PCI Spliced U-Beams (U96)	422
6. PCI Simple Span and Spliced U-Beam (U72)	422
7. LaDOTD Bulb T (LG-78)	349
8. Florida U-Beam (FUB-72)	328
9. Steel Plate Girders	N/A
10. AASHTO-PCI Bulb T (BT-72)	349

7. High Level Approach Span Concepts Evaluation – Construction Schedule

Good = ● Fair = ◒ Poor = ○

Bridge Alternative	Ranking
1. Precast Concrete Segmental (Cantilever)	○
2. Precast Concrete Segmental (Cantilever)	◒
3. Precast Concrete Segmental (Span-by-Span)	●
4. PCI Spliced U-Beam (U72)	◒
5. PCI Spliced U-Beams (U96)	◒
6. PCI Simple Span and Spliced U-Beam (U72)	◒
7. LaDOTD Bulb T (LG-78)	◒
8. Florida U-Beam (FUB-72)	◒
9. Steel Plate Girders	N/A
10. AASHTO-PCI Bulb T (BT-72)	◒

08 High Level Approach Spans Recommendations

8. High Level Approach Spans – Recommendations

Good = ● Fair = ◒ Poor = ○

High Level Approach Spans Bridge Alternatives

Criteria	1. Precast Concrete Segmental (Cantilever)	2. Precast Concrete Segmental (Cantilever)	3. Precast Concrete Segmental (Span-by-Span)	4. PCI Spliced U-Beam (U72)	5. PCI Spliced U-Beams (U96)	6. PCI Simple Span and Spliced U-Beam (U72)	7. LaDOTD Bulb T (LG-78)	8. Florida U-Beam (FUB-72)	9. Steel Plate Girders	10. AASHTO-PCI Bulb T (BT-72)
Initial Construction Cost	○	○	◒	◒	◒	◒	●	●	N/A	●
Construction Schedule	○	◒	●	◒	◒	◒	◒	◒	N/A	◒
Constructability	◒	●	●	◒	◒	◒	◒	◒	N/A	◒
Maintenance and Durability	●	●	●	◒	◒	◒	◒	◒	N/A	◒
Environmental Commitments	●	●	●	●	●	●	◒	●	N/A	◒
Design Considerations	◒	◒	●	◒	◒	●	●	●	N/A	◒
Overall	○	◒	●	●	◒	●	◒	◒	N/A	◒

Next Steps

- Finalize Bridge Configuration and Bridge Laneage
- Bayway Configuration
- East Tunnel Interchange Alternative
- Final Main Span Configuration
- Update Layouts, Costs and Schedule for Remaining 3 High Level Approach Spans Alternatives
- Optimize Foundation Design

Thank you for your time

QUESTIONS?

A

Cost Estimate Backup

ALDOT Bridge Costs – Escalated to 2017

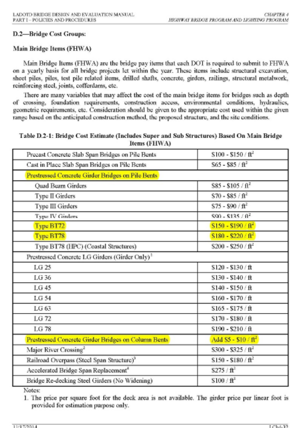
Girder Type	Year	2011 Bridge Cost / sf	Years of 4% Inflation to 2017	INFLATION Adjusted Cost / sf	Span Length (ft.)	Pier Height (ft.)
BT-72	2011	$94	6	$119	120-140	40-60
Concrete Spliced Girder	2011	$109	6	$138	140-180	60-80
Concrete Spliced Girder	2011	$116	6	$147	180-200	80-100
Concrete Spliced Girder	2011	$122	6	$154	200-250	80-100

Birmingham

- AASHTO BT-63 prestressed concrete girders.

LaDOTD

- AASHTO Type II, III, IV, BT-72, BT-78, are the standard shapes of prestressed concrete girders.
- Louisiana Girder Shapes LG-25, LG-36, LG-45, LG-54, LG-63, LG-72, and LG-78 are allowed with the approval of the Bridge Design Engineer Administrator.



LADOTD BRIDGE DESIGN AND EVALUATION MANUAL
PART I – POLICIES AND PROCEDURES

CHAPTER 4
HIGHWAY BRIDGE PROGRAM AND LIGHTING PROGRAM

D.2—Bridge Cost Groups:

Main Bridge Items (FHWA)

Main Bridge Items (FHWA) are the bridge pay items that each DOT is required to submit to FHWA on a yearly basis for all bridge projects let within the year. These items include structural excavation, sheet piles, piles, test pile related items, drilled shafts, concrete, girders, railings, structural metalwork, reinforcing steel, joints, cofferdams, etc.

There are many variables that may affect the cost of the main bridge items for bridges such as depth of crossing, foundation requirements, construction access, environmental conditions, hydraulics, geometric requirements, etc. Consideration should be given to the appropriate cost used within the given range based on the anticipated construction method, the proposed structure, and the site conditions.

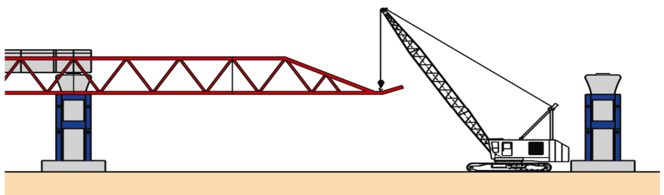
Table D.2-1: Bridge Cost Estimate (Includes Super and Sub Structures) Based On Main Bridge Items (FHWA)

Precast Concrete Slab Span Bridges on Pile Bents	\$100 - \$150 / ft^2
Cast in Place Slab Span Bridges on Pile Bents	\$65 - \$85 / ft^2
Prestressed Concrete Girder Bridges on Pile Bents	
Quad Beam Girders	\$85 - \$105 / ft^2
Type II Girders	\$70 - \$85 / ft^2
Type III Girders	\$75 - \$90 / ft^2
Type IV Girders	\$90 - \$135 / ft^2
Type BT72	\$150 - \$190 / ft^2
Type BT78	\$180 - \$220 / ft^2
Type BT78 (HPC) (Coastal Structures)	\$200 - \$250 / ft^2
Prestressed Concrete LG Girders (Girder Only)[1]	
LG 25	\$120 - \$130 / ft
LG 36	\$130 - \$140 / ft
LG 45	\$140 - \$150 / ft
LG 54	\$160 - \$170 / ft
LG 63	\$165 - \$175 / ft
LG 72	\$170 - \$180 / ft
LG 78	\$190 - \$210 / ft
Prestressed Concrete Girder Bridges on Column Bents	Add \$5 - \$10 / ft^2
Major River Crossing[2]	\$300 - \$325 / ft^2
Railroad Overpass (Steel Span Structure)[3]	\$150 - \$180 / ft^2
Accelerated Bridge Span Replacement[4]	\$275 / ft^2
Bridge Re-decking Steel Girders (No Widening)	\$100 / ft^2

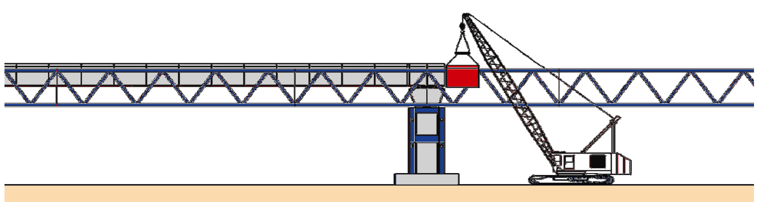
Notes:
1. The price per square foot for the deck area is not available. The girder price per linear foot is provided for estimation purpose only.

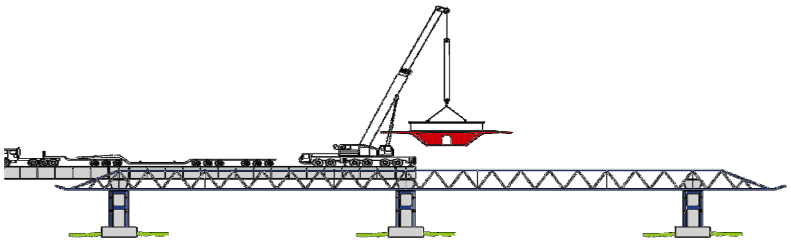
11/17/2014 I.Ch4-32

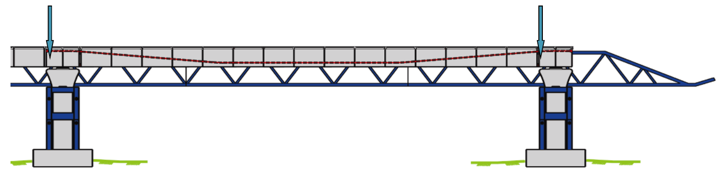
Typical Span-by-Span Construction Sequence

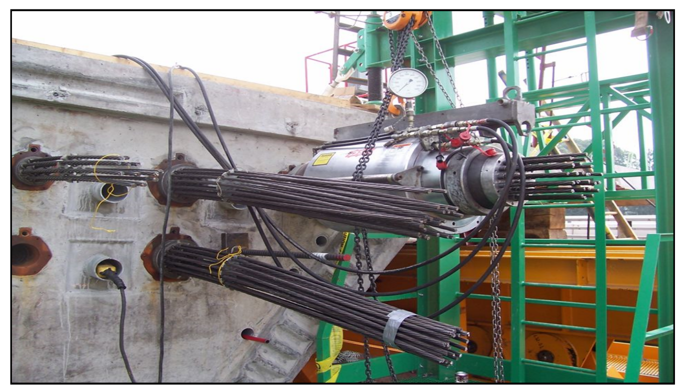


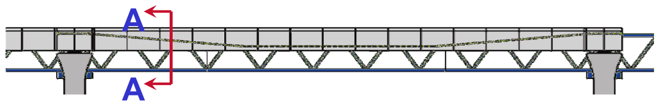

MALL

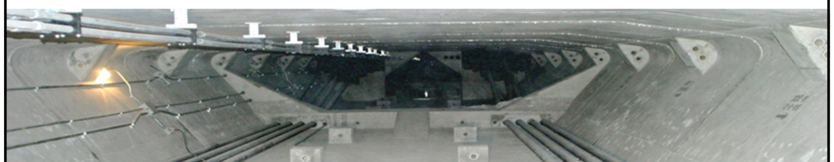












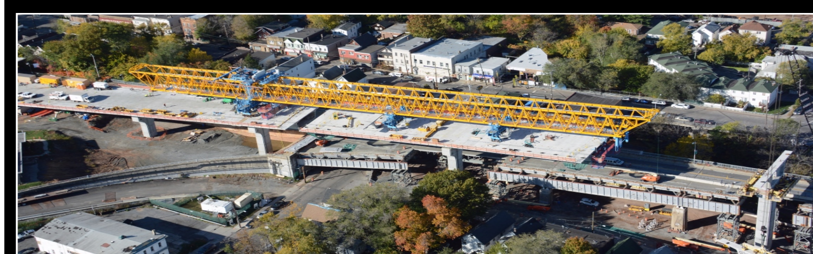
Precast Balanced Cantilever Construction

Precast Segmental – Cantilever

Erection with Launching Gantry

- Span up to 350 ft±
- Speed of Erection (up to 6 pairs of segments)
- Delivery from behind or below
- Overhead Construction
- Self Launching – min. crane support
- Parallel Structures Simultaneously
- Temp loads directly into Piers

Precast Balanced Cantilever Construction – Overhead Gantry



Bayonne Bridge, NY

Overhead Gantry – Advantages and Disadvantages

Advantages:
- Reduced impact on ground below
- Eliminates need for Adjustments to Vertical Profile

Disadvantages:
- Cost of Specialized Equipment

Ground Based Cranes – Advantages and Disadvantages

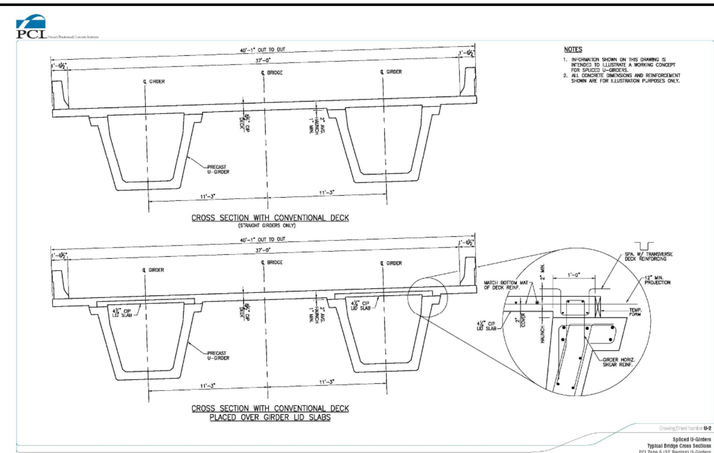
Advantages:
- No Cost for Specialized Equipment
- Eliminates need for Adjustments to Vertical Profile

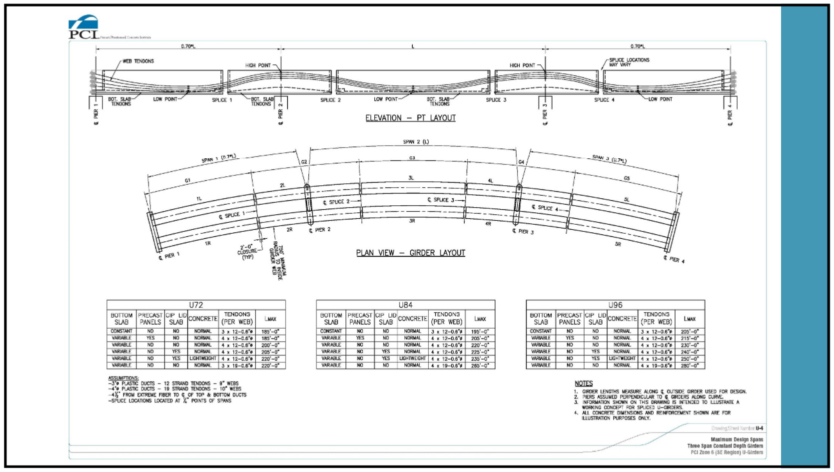
Disadvantages:
- Higher Impact on Ground Below

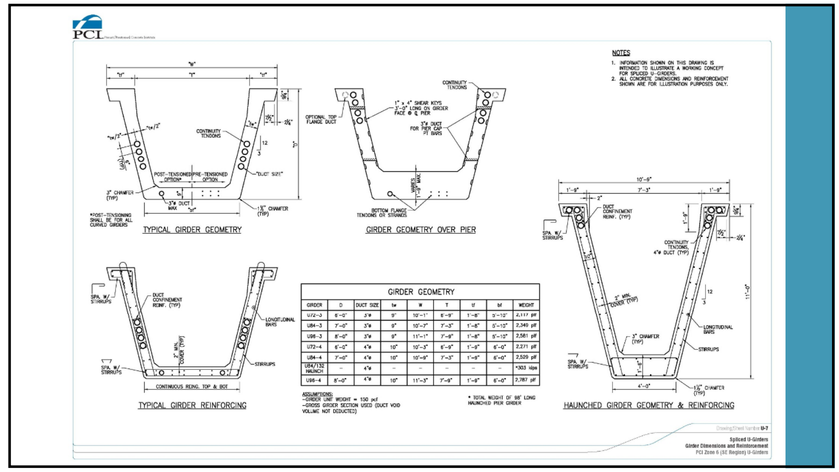
5. Design Approach – References (PCI Zone 6 U-Girder Drawings)

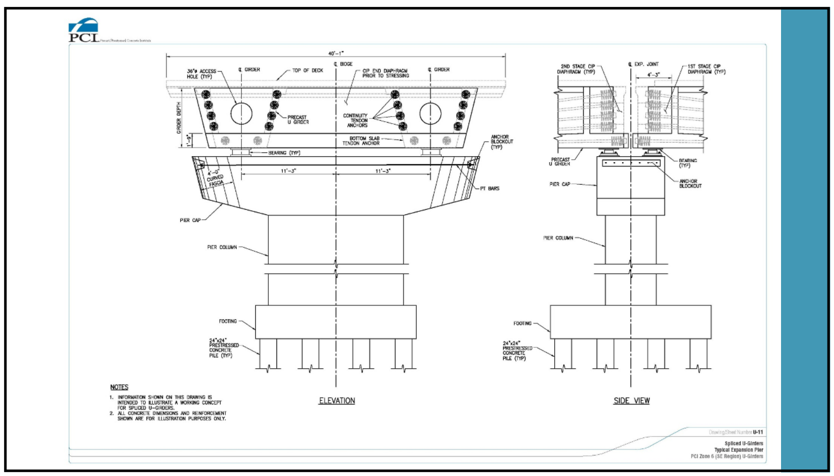
- The Precast/Prestressed Concrete Institute (PCI) developed a set of concept drawings for curved, spliced U-Girders.
- The drawings include:
  - Cross-sections
  - Prestressing layouts
  - Recommended span lengths for simple span, continuous spans, and continuous spans with haunched pier sections
  - Erection sequences

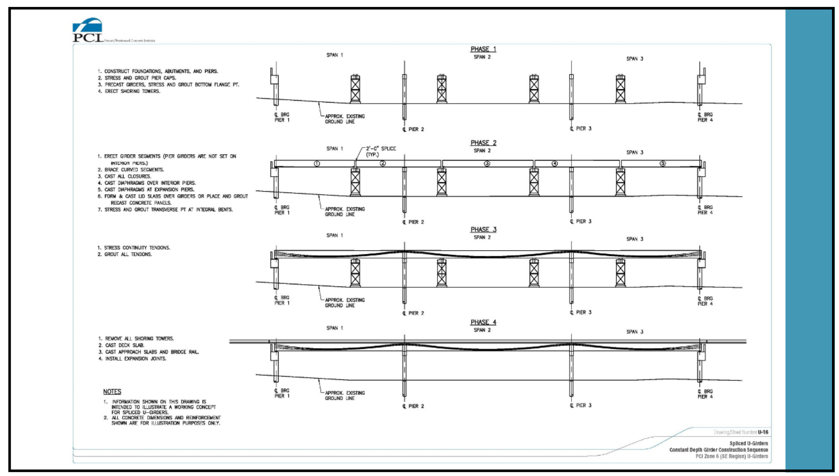
PCI
Precast/Prestressed Concrete Institute




ELEVATION – PT LAYOUT
PLAN VIEW – GIRDER LAYOUT
U72
U84
U96
NOTES
1. GIRDER LENGTHS MEASURE ALONG ℄ OUTSIDE GIRDER USED FOR DESIGN.
2. PIERS ASSUMED PERPENDICULAR TO ℄ GIRDERS ALONG CURVE.
3. INFORMATION SHOWN ON THIS DRAWING IS INTENDED TO ILLUSTRATE A WORKING CONCEPT FOR SPLICED U-GIRDERS.
4. ALL CONCRETE DIMENSIONS AND REINFORCEMENT SHOWN ARE FOR ILLUSTRATION PURPOSES ONLY.
Drawing/Sheet Number U-4
Maximum Design Spans
Three Span Constant Depth Girders
PCI Zone 6 (SE Region) U-Girders


TYPICAL GIRDER GEOMETRY
GIRDER GEOMETRY OVER PIER
TYPICAL GIRDER REINFORCING
GIRDER GEOMETRY
HAUNCHED GIRDER GEOMETRY & REINFORCING
Drawing/Sheet Number U-7
Spliced U-Girders
Girder Dimensions and Reinforcement
PCI Zone 6 (SE Region) U-Girders


ELEVATION
SIDE VIEW
Drawing/Sheet Number U-11
Spliced U-Girders
Typical Expansion Pier
PCI Zone 6 (SE Region) U-Girders


PHASE 1
PHASE 2
PHASE 3
PHASE 4
Drawing/Sheet Number U-16
Spliced U-Girders
Constant Depth Girder Construction Sequence
PCI Zone 6 (SE Region) U-Girders

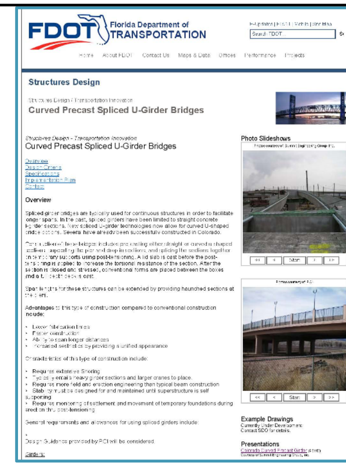
5. Design Approach – References (CDOT)

- CDOT has been building spliced girder bridges since the early 1990s, including a site-precast curved U-beam bridge in 1995.
- By early 2000, CDOT had standardized the U-beam section, and began using this section to construct curved, spliced U-beams using plant-manufactured beams.

Bridge	Spans	Horizontal radius
IH25/SH270 Ramp K	200'	800'
IH70/E470 Ramp H	100'-195'	1,400'
IH76/SH270 Ramp Y	100'-230'	760'
IH70/SH58 Ramp A	150'-235'	820'
Austin Bluffs Overpass	110'-210'	700'
IH25 Viaduct, Trinidad	100-256'	1,200'

5. Design Approach – References (FDOT)

- As part of their Invitation to Innovation program, the Florida Department of Transportation (FDOT) has a webpage dedicated to Curved Precast Spliced U-Girder Bridges.
- The page provides an overview and general requirements for use of curved spliced girders.
- FDOT has adopted the PCI Zone 6 sections as the preferred spliced girder.



5. Design Approach – References (LaDOTD)

- AASHTO Type II, III, IV, BT-72, BT-78, are the standard shapes of prestressed concrete girders.
- Louisiana Girder Shapes LG-25, LG-36, LG-45, LG-54, LG-63, LG-72, and LG-78 are allowed with the approval of the Bridge Design Engineer Administrator.

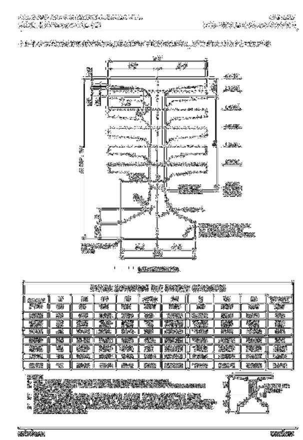
Girder Maximum Span Length Table

Girder Type	Maximum Span Length (ft.)	Maximum Prestressing Force Immediately Prior to Transfer (kip)	Maximum No. of Strands (0.6 in. Dia., 270 ksi, Low Relaxation Strands)
LG-25	53	1,408	32
LG-36	98	2,112	48
LG-45	119	2,376	54
LG-54	133	2,464	56
LG-63	154	2,816	64
LG-72	171	3,080	70
LG-78	183	3,344	76
Quad Beam (18.0 in.)	40	704	16
AASHTO Type II (36 in.)	55	750	17
AASHTO Type III (45 in.)	85	1,000	22
AASHTO Type IV (54 in.)	105	1,500	34
BT-72	125	1,850	42
BT-78	140	2,200	50

11/17/2014 II.V1-Ch5-12

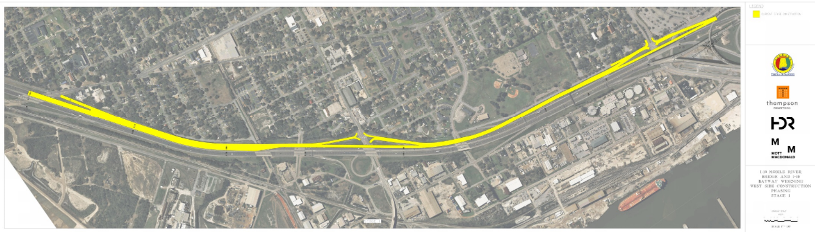
5. Design Approach – References (LaDOTD)

- LG Girder Dimensions, Section Properties and Strand Parameters



7. Constructibility – West Side Construction Phasing

Phase 1

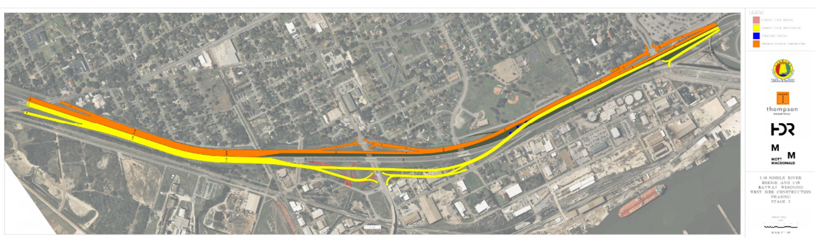


7. Constructibility – West Side Construction Phasing

Phase 2

7. Constructibility – West Side Construction Phasing

Phase 3



7. Constructibility – West Side Construction Phasing

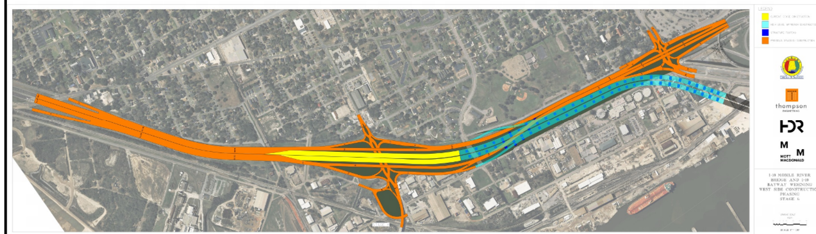
Phase 4

7. Constructibility – West Side Construction Phasing

Phase 5

7. Constructibility – West Side Construction Phasing

Phase 6



7. Constructibility – West Side Construction Phasing

Phase 7

Attendance List

thompson | HDR | MOTT MACDONALD

Meeting: Mobile River Bridge- West Side Foundation & Main Span Unit Studies Progress Meeting	Time: 1:30 p.m.	Date: July 28, 2016

Name	Firm	Phone	Email
Edwin Perry III	ALDOT SW	251-470-8243	perrye@dot.state.al.us
Andrew Wood	ALDOT SW	251-470-8320	wooda@dot.state.al.us
Manuel Carballo	HDR	469.559.5663	manuel.carballo@HDRInc.com
Cory Shipman	HDR	972.732.2014	cory.shipman@HDRInc.com
Kendall Kilpatrick	Mott MacDonald	251.287.9400	kendall.kilpatrick@mottmac.com
Stan Biddick	ALDOT-Design	334-242-6833	biddicks@dot.state.al.us
William Adams	ALDOT Design	334-242-6488	adamswi@dot.state.al.us
Matt Ericksen	ALDOT-SW Region	251-470-8364	ericksen@dot.state.al.us
Steve Flukinger	Thompson	251-525-3038	sflukinger@thompsonengineering.com
Sam Sternberg	Thompson	251-509-7786	ssternberg@thompsonengineering.com
Tim Colquett	ALDOT-Bridge	334-242-6007	colquettt@dot.state.al.us
Randall Mullins	" "	6004	mullinsr@ " "
Patt Hickox	HDR	251-295-6874	patrick.hickox@hdrinc.com
on the phone: Justin Doornik, Bradley Touchstone,			
Jake Perkins, Mike Lamont,			
Bart Hendricks (MM)			
Don Arkle			

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Agenda

Project:	I-10 Mobile River Bridge & Bayway Widening
Subject:	West Side Foundation & Main Span Unit Studies Progress Meeting
Date:	Thursday, July 28, 2016
Location:	Alabama Department of Transportation -SWR

1. **Main Span Bridge Type Selection Process**
   - **a. Scope of Work**
   - **b. Bridge Type Selection Process**
   - **c. Design Guidelines/Constraints**
   - **d. Superstructure Types Considered**
   - **e. Design Approach**
   - **f. Main Span Bridge Alternatives**
   - **g. Main Span Alternatives Evaluation**
   - **h. Main Span Recommendations**

2. **West Side Foundation Study**
   - **a. Purpose of Study**
   - **b. Scope of Work**
   - **c. Foundation Types Studied**
   - **d. Design Assumptions**
   - **e. Design Approach**
   - **f. Design Concepts**
   - **g. Constructability**
   - **h. Construction Costs**
   - **i. Vibration Due to Pile Driving**

Meeting Minutes

Project:	I-10 Mobile River Bridge & Bayway Widening
Subject:	West Side Foundation and Main Span Unit Study
Date:	Thursday, July 28, 2016, 1:30 PM
Location:	Alabama Department of Transportation - SWR

Attendees:	Patrick Hickox (HDR)	Tim Colquett (ALDOT)
	Manuel Carballo (HDR)	Randall Mullins (ALDOT)
	Cory Shipman (HDR)	Edwin Perry (ALDOT)
	Kendal Kirkpatrick (MM)	Andrew Wood (ALDOT)
	Steve Flukinger (TE)	Matt Ericksen (ALDOT)
	Sam Sternberg (TE)	William Adams (ALDOT)
	Don Arkle (ALDOT)	Stan Biddick (ALDOT)
Conference Call:	*Justin Doornink (MM)*	*Jake Perkins (HDR)*
	Bradley Touchstone (TA)	*Mike Lamont (HDR)*
	Bart Hendricks (MM)	

Manuel Carballo presented information for the Mobile River Bridge project covering the below topics (see attached pdf of the presentation).

1. **Main Span Alternatives**
   a. Explained the 7 Main Span Alternatives in Detail
   b. Covered the Pros and Cons for Each Alternative
   c. Scored the Alternatives and Recommended 4 Alternatives for Further Study
      i. Precast Concrete Segmental Alternative #1
      ii. Steel Edge Girder Alternative #2
      iii. CIP Concrete Edge Girder Alternative #3
      iv. CIP Concrete Segmental Split Deck Alternative #7
   d. Tim Colquett expressed concern with maintaining the steel edge girder options and recommended that these be eliminated from consideration. Pat Hickox reiterated the potential upfront savings for the Steel Concrete Edge Girder Alternative #2. ALDOT selected the following options for continued study:
      i. Precast Concrete Segmental Alternative #1
      ii. CIP Concrete Edge Girder Alternative #3
      iii. CIP Concrete Segmental Split Deck Alternative #7
   e. Miscellaneous
      i. Alternative #1 may be eliminated if a sidewalk is required due to excessive bridge width causing transverse bending issues with the superstructure cross section.

2. **West Side Foundations**
   a. Discussed 10 foundation options
      i. 8 ft. drilled shafts
      ii. 6 ft. drilled shafts
      iii. 4 ft. drilled shafts
      iv. 30” prestressed concrete piles
      v. 36” prestressed concrete piles
      vi. 4.5 ft. concrete cylinder piles
      vii. 5 ft. concrete cylinder piles
      viii. 12” HP piles
      ix. 14” HP piles
      x. 6 ft open ended steel pipe piles
   b. ALDOT expressed concern over constructability issues associated with drilled shaft and eliminated all drilled shaft options for future considerations.
   c. Tim Colquett will consider steel piling pending the results from studying the corrosiveness of the soil. Sam Sternberg will take samples and send the laboratory for testing.
   d. ALDOT is reconsidering allowable axial pile capacities, since adjoining states all use different capacities and will provide any updates.
   e. ALDOT stated the need to conduct pre-construction property surveys to local properties.

Appendix B:

Structures Design Criteria

thompson
ENGINEERING

LETTER OF TRANSMITTAL

Document No. TTT-0046

To: **Alabama Department of Transportation**

Date: 8/12/16 **Job No.** 15-1101-0300
Attention: Edwin Perry, III, PE
Re: MRB- Structures Design Criteria

WE ARE SENDING YOU: ☒ Attached ☐ Under separate cover via _____ the following items:
☐ Change Order ☐ Prints ☐ RFI ☐ Drawings ☐ Specifications ☐ Copy of letter ☐ Contract
☐ Work Authorization Order

Copies	Date	No.	Description
1	8/12/16	TTA-0046	Mobile River Bridge Structures Design Criteria

THESE ARE TRANSMITTED as checked below:
☒ For approval ☐ Approved as submitted ☐ Resubmit _____ copies for approval ☐ Approved as noted
☐ For your use ☐ Returned for corrections ☐ Submit _____ copies for distribution ☐ For review and comment
☐ As requested ☐ For bids due ☐ Return _____ corrected prints ☐ Prints returned after loan to us

Remarks: Please find attached the proposed structures design criteria for the project.

Let us know if you have any questions or comments.

FROM: Patrick Hickox, PE

Patrick Hickox

COPY TO: File…/contracts/HDR
Greg Kochersperger, PE
Steve Flukinger, PE
Manuel Carballo, PE

If enclosures are not as noted above, please notify sender immediately

2970 Cottage Hill Road, Suite 190
Mobile, AL 36606
251-666-2443 ph. / 251-666-6422 fax
www.thompsonengineering.com

A THOMPSON HOLDINGS, INC. COMPANY

STRUCTURES DESIGN CRITERIA

For

I-10 Mobile River Bridge and Bayway Widening Project
Mobile, Alabama

August 2016

Version DRAFT B

DISCLAIMER

This document is intended to provide clarification of ALDOT Policy as it applies to the I-10 Mobile River Bridge and Bayway Widening Project and to provide direction on elements not specifically addressed in other documents. This document does not relieve the designer's responsibility to possess a thorough knowledge of the ALDOT Structures Design Manual and AASHTO LRFD Bridge Design Specifications.

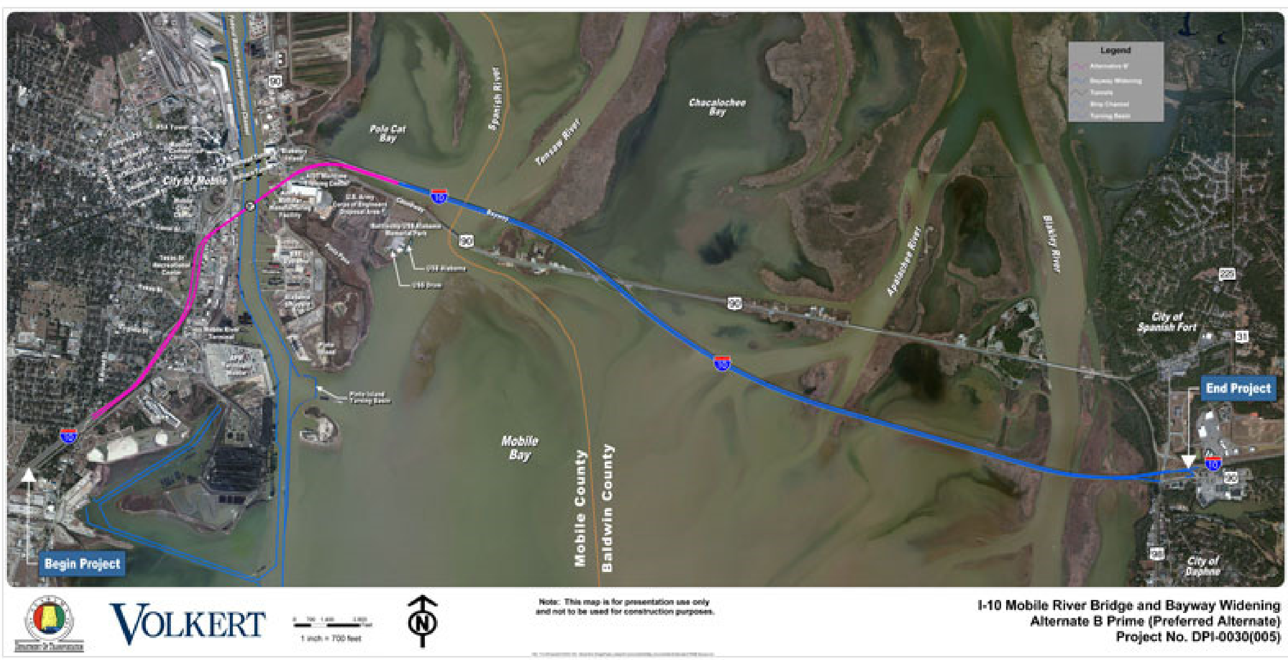
REVISIONS

Date	Version	Description
7/15/16	DRAFT A	Initial Draft document circulated for internal comment
8/3/2016	DRAFT B	Submitted for ALDOT Review

Table of Contents

1.0 Project Description

This project includes the I-10 Mobile River Bridge and Bayway Widening from Exit 24 (Broad Street) to East of SR-16 (US-90) at Spanish Fort, Mobile and Baldwin Counties. As detailed in the draft EIS, the recommended alignment of the project is Alternative B'. The project proposes to increase the capacity of I-10 by constructing a new eight-lane bridge with 215 feet of vertical clearance and 800' horizontal clearance across the Mobile River and widen the existing I-10 bridges across Mobile Bay from four to eight lanes. Other project improvements include interchanges at I-10/Virginia Street, I-10/Water Street, I-10/East Tunnel, I-10/Mid-bay and I-10/US 90/US 98.



This document establishes the structural design criteria to be used for all bridges, retaining walls, and miscellaneous highway structures, except for toll facility structures.

	Reference	Revision Date
2.0 Specifications, Codes and Design Directives		
American Association of State Highway and Transportation Officials. *LRFD Bridge Design Specifications*, 7th Edition, with 2016 Interim Revisions. 2016.	LRFD	
American Association of State Highway and Transportation Officials. *Manual for Bridge Evaluation*, 2nd Edition, with 2016 Interim Revisions. 2010.	MBE	
American Association of State Highway and Transportation Officials. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, 6th Edition, with 2015 Interim Revisions . 2013.	LTS-6	
American Association of State Highway and Transportation Officials. *LRFD Guide Specifications for the Design of Pedestrian Bridges*, 2nd Edition, with 2015 Interim Revisions.	Gd Spec Ped	
American Association of State Highway and Transportation Officials. *Guide Specification for LRFD Seismic Bridge Design,* 2nd Edition, with 2015 Interim Revisions.	Gd Spec Seismic	
American Association of State Highway and Transportation Officials. *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges,* 2nd Edition, with 2010 Interim Revisions.	Gd Spec Vessel	
American Association of State Highway and Transportation Officials. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 2nd Edition, with 2003 Interim Revisions.	Gd Spec Segm	
Alabama Department of Transportation. *Structural Design Manual*, February 2015.	ALDOT SDM	
Alabama Department of Transportation. *QC/QA Guidelines.*	ALDOT QC/QA	
Alabama Department of Transportation. *Guide to Developing Construction Plans.*	ALDOT GDCP	

	Reference	Revision Date
Alabama Department of Transportation. Bridge Special Project Drawings.	ALDOT Br Sp Dwg	
Alabama Department of Transportation. *Standard Specifications for Highway Construction*, 2012; and Special Provisions	ALDOT Std Spec Spec Provision	
Alabama Department of Transportation. Special and Standard Highway Drawings, 2015	ALDOT St Dwg	
Alabama Department of Transportation. *Bridge Plans Detailing Manual*, 2014	ALDOT Detailing Manual	
Alabama Department of Transportation. *Structural Design Manual,* 2015	ALDOT SDM	
Alabama Department of Transportation. *Guidelines for Operation*, March, 2016	ALDOT Gd Op	
Alabama Department of Transportation. *Roadway Plan Preparation Manual*, December, 2008	ALDOT PPM	
AASHTO/AWS D1.5M/D1.5-2015, *Bridge Welding Code*, 2015.	BWC	
AWS D1.1, *Structural Welding Code Steel*, 2010.	SWC	
American Institute of Steel Construction. *Steel Construction Manual*, 14th Edition, 2010.	AISC	
Project-related Geotechnical Reports, Inspection Reports and Hydraulic Reports		
Mobile River Bridge Draft Environmental Impact Statement	DEIS	
Mobile River Bridge Scope of Services	MRB Scope	
Mobile River Bridge Coordination Call Issues Log	MRB Issues Log	

	Reference	Revision Date
Post Tensioning Institute. DC45.1-12: *Recommendations for Stay Cable Design, Testing, and Installation*. 2012.	PTI Stay Cable	
CEB-FIP Model Code 1990: Design Code	CEB-FIP ‘90	
EN 1993-1-5:2006 Eurocode 3	Eurocode 3	

3.0 Geometric Constraints

3.1 Design Speed

3.1.1 See roadway plans for corresponding design speed for each bridge.

3.2 Span Lengths:

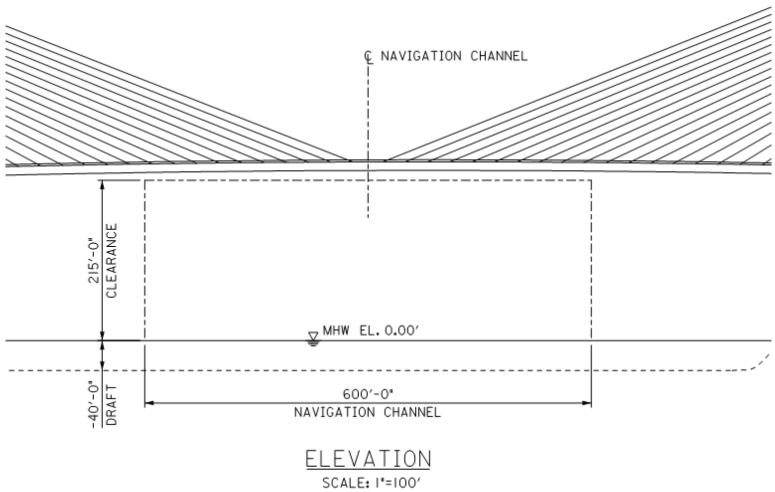
3.2.1 See Approved Type, Size and Location (TS&L) Drawings [TS&L Development is pending]

3.3 The typical overall structure widths:

3.3.1 See Approved Type, Size and Location (TS&L) Drawings [TS&L Development is pending]

3.4 Horizontal Clearance

	Reference	Revision Date
3.4.1 Required horizontal clearance for the main span navigation channel is shown in Figure 1.	MRB Scope	



<u>Figure 1</u>

	Reference	Revision Date
3.5 Vertical Clearance		
3.5.1 Final vertical clearance over roadways shall be 17’-0” minimum preferred at edge of travel lane; in no case less than 16’-3” at edge of paved shoulder for interstate system and state routes, and 14’-3” for city streets and county roads. Final minimum vertical clearance over roadways for existing bridges to be widened shall not be less than existing minimum vertical clearance.	ALDOT Gd Op Section 3-25	
3.5.2 Proposed vertical clearance over Mobile Harbor Federal Navigation Channel shall be 215’-0”, as shown in Figure 1.	DEIS	
3.5.1 Final vertical clearance over railroads shall be 24'-3" minimum.		
3.6 Clearance Calculations		
3.6.1 All vertical and horizontal clearance calculations		

		Reference	Revision Date
	done by roadway need to be verified by the bridge designer (this includes checking clearance of pier caps projecting over adjacent roadways as well as clearance checks for signals, if any, hung from the bottom of the girders). Factors such as cross-slopes of deck, bearing assembly height and pedestal height shall be accounted for in the vertical clearance calculations.		
3.6.2	Actual minimum clearances are to be shown in plans instead of the required minimums. The location where these minimums occur shall be shown in the Plan & Elevation sheets		
3.7	Maintenance and Inspection Access		
3.7.1	For Cable Stayed Main Span provide access to underside of all areas of bridge utilizing an Aspen A-62 under-bridge inspection vehicle, cat-walks or a maintenance traveler.	MRB Issues Log	
3.7.2	For High Level Approach Spans and Bayway, provide access to underside of all areas of bridge utilizing an Aspen A-62 under-bridge inspection vehicle. Provide minimum of 10'-0" between backs of railing to allow access for inspection.	MRB Issues Log	

	Reference	Revision Date
4.0 Loadings		
4.1 Dead Loads (DC)		
4.1.1 Structural dead loads shall be based on unit weights of materials and the computed volumes of the structural elements.		
4.1.1.1 Non-Reinforced concrete – 145 pound per cubic foot	LRFD Table 3.5.1-1	
4.1.1.2 Structural concrete - 150 pound per cubic foot (includes weight of reinforcing steel)	LRFD Table 3.5.1-1	
4.1.1.3 Prestressed concrete with f'c greater than 5.0 ksi – 155 pounds per cubic foot	LRFD Table 3.5.1-1	
4.1.1.4 Steel - 490 pound per cubic foot	LRFD Table 3.5.1-1	
4.1.2 Additional Dead Loads (DC1)		
4.1.2.1 Stay-in-Place Metal Forms - 15 pounds per square foot dead load (this includes the dead weight of concrete in the forms).	ALDOT SDM Section 3	
4.1.3 Additional Dead Loads (DC2)		
4.1.3.1 The barrier rail dead load shall be considered equally distributed across all girders. However, the dead load for girder design shall not be less than 25% of a single barrier rail weight.	ALDOT SDM Section 3	
4.1.3.2 Bridge Barrier Rail – 377 pound per linear foot of railing for standard barrier.	ALDOT Sp Dwg BBR-1	

	Reference	Revision Date
4.1.3.3 See bridge plans for weights of TL-5 barriers for the cable-stayed bridge.		
4.2 Wearing surfaces and utilities (DW)		
4.2.1 Future Wearing Surface		
4.2.1.1 No future wearing surface allowance required		
4.2.1.2 Integral wearing surface on precast deck panels for cable stay bridge shall be included as DC loading according to design details.		
4.2.2 Utilities – For weight of structure mounted utilities contact the respective discipline leads (Utilities, drainage, ITS, etc.) and calculate weight based on information received in Final Design Phase		
4.3 Vehicular Live Loads (LL+IM)		
4.3.1 Highway Loading – AASHTO HL93	LRFD 3.6.1.1	
4.3.2 Permit Vehicle		
4.3.2.1 Alabama Permit Loads – See Section 8.3.1 for loads to review as part of the Load Rating tabulations.	ALDOT SDM Section 16	
4.3.1 Dynamic Load Allowance (IM) – Per AASHTO	LRFD 3.6.2	
4.4 Pedestrian / Bicycle Loadings (PL)		
4.4.1 Bridges with Sidewalks	LRFD 3.6.1.6	
4.4.1.1 A pedestrian load of 0.075 ksf shall be	LRFD 3.6.1.6	

			Reference	Revision Date
		applied to all sidewalks wider than 2 ft. and considered simultaneously with the vehicular design live load in the vehicle lane.		
	4.4.2	Railing Loads	LRFD 13.8 and 13.9	
		4.4.2.1 For design loads for pedestrian and bicycle railings, see AASHTO LRFD		
4.5	Centrifugal Forces (CE)		LRFD 3.6.3	
4.6	Braking Force (BR)		LRFD 3.6.4	
4.7	Vehicular Collision Force (CT)		LRFD 3.6.5	
	4.7.1	All piers located within 30 ft to the edge of travel way shall be investigated for vehicle collision.	LRFD 3.6.5.1	
	4.7.2	Provide mitigation or design for forces per AASHTO LRFD		
4.8	Water Loads (WA)		LRFD 3.7	
	4.8.1	Stream Pressure	LRFD 3.7.3	
		4.8.1.1 The longitudinal drag force shall be taken as the product of longitudinal stream pressure and the projected surface exposed thereto.	LRFD Table 3.7.3.1-1	
		4.8.1.2 The lateral drag force shall be taken as the product of the lateral stream pressure and the surface exposed thereto.	LRFD Table 3.7.3.2-1	
	4.8.2	Storm Surge and Wave Loading		
		4.8.2.1 See Bayway Storm Surge Analysis for loadings on Bayway Structure	MRB Scope	
4.9	Wind Loading on Structures (WS)		LRFD 3.8	

	Reference	Revision Date
4.9.1 Cable-Stayed Bridge Structure		
4.9.1.1 Preliminary wind studies and preliminary analysis are needed to establish and validate the superstructure and tower cross sections of the cable stayed bridge.	MRB Scope	
4.9.1.2 Final design activities such as a full aeroelastic model will be accomplished in Final Design Phase		
4.9.1.3 Wind loadings on cable-stayed bridges to be based on site specific wind climate study, sectional model tests, and buffeting analysis results.		
4.9.1.4 Final wind load demands shall be based on dynamic buffeting analyses. Wind forces shall be applied in accordance with AASHTO and the wind engineering study. Wind load during construction in accordance with AASHTO LRFD 5.14.2.3 and the wind engineering study.		
4.9.1.5 Wind speeds for Strength and Aerodynamic Stability Design shall be based on the following return period and averaging time.		
4.9.1.5.1 Strength Design:		
i. Final Design – 100 year return period with a 1-hour mean.		
ii. Construction Stage – 20 year return period with a 1-hour mean.		

	Reference	Revision Date
4.9.1.5.2 Aerodynamic Stability Design: i. Final Design – 10,000 year return period with a 10-minute mean. ii. Construction Stage – 1000 year return period with a 10-minute mean.		
4.9.1.6 Design Wind Speeds for at deck level for Strength and Stability design are provided in the site specific wind climate study.		
4.9.1 Other Bridge Structures	LRFD 3.8	
4.9.2 Mast Arms Signal Structures To be established in Final Design Phase		
4.9.3 Sign Structures To be established in Final Design Phase		
4.10 Wind loadings on Vehicles (WL)	LRFD 3.8.1.3	
4.11 Earthquake (EQ)		
4.11.1 Seismic Design will be in accordance with AASHTO Guide Specification for Seismic Design and LRFD 3.10.	ALDOT SDM Section 15	
4.12 Earth Pressure (EH) (on substructure and retaining walls) To Be established in Final Design Phase		
4.13 Thermal Loadings	LRFD 3.12.2	

	Reference	Revision Date
4.13.1 Uniform Temperature (TU)		
4.13.1.1 Erection temperature shall be assumed to be 70° F	ALDOT SDM Section 3	
4.13.1.2 Temperature Ranges for superstructure materials are as shown below:	LRFD 3.12.2.2	
4.13.1.2.1 For concrete structures:		
i. High: 105 ° F		
ii. Low: 35 ° F		
4.13.1.2.2 For concrete deck on steel girder:		
i. High: 110 ° F		
ii. Low: 30 ° F		
4.13.1.3 Note the maximum and minimum design temperatures on drawings for girders, expansion joints, and bearings.		
4.13.2 Differential thermal gradients (TG)	LRFD 3.12.3	
4.13.2.1 Consideration of differential thermal gradient is only required for cable-stayed and segmental concrete bridges.		
4.13.2.2 Uniform temperature differential between the stay cables and the bridge (deck, towers, anchor piers) is +/- 25 ° F.		
4.14 Friction (FR)	LRFD 3.13	

	Reference	Revision Date
4.14.1 For the longitudinal force due to friction of non-fixed bearings (other than elastomeric pads, i.e. pot bearings) use 7 percent of dead load.		
4.15 Vessel Collision (CV)		
4.15.1 Main Span Towers shall be designed for vessel impact in accordance with AASHTO.	LRFD 3.14	
4.15.2 Operational classification for the main span bridge shall be “critical or essential”.	LRFD 3.14.3	
4.15.3 Vessel impact forces shall be computed in accordance with Method II per AASHTO LRFD Section C3.14.1. A site specific vessel traffic study shall be performed together with a risk assessment and calculation of impact loads.	MRB Scope	
4.15.4 For the evaluation vessel impact, the following two vessel collision events combined with scour conditions shall be considered:		
4.15.4.1 A drifting empty barge breaking loose from its moorings and striking the bridge. The vessel impact loads should be combined with one-half of the predicted long-term scour plus one-half of the predicted short term scour. The flow rate, water level, and short-term scour depth are those associated with the design flood for bridge scour (100-year flood event).		
4.15.4.2 A ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions. The vessel impact loads should be combined with the effects of one-half of the long-term scour and no short-term scour. The flow rate and water level should be taken as the yearly mean conditions.		

	Reference	Revision Date
4.16 Construction Loading		
4.16.1 Beam/Girder Bridges To be established in Final Design Phase		
4.16.2 Cable-stayed and Segmental Bridges		
4.16.2.1 Construction loading for segmentally erected bridges shall be in accordance with LRFD 5.14.2.	LRFD 5.14.2	
4.16.2.2 Wind loads during construction shall be based on project-specific wind study recommendations.		
4.17 Cable Loss and Cable Replacement		
4.17.1 Cable Replacement		
4.17.1.1 The cable-stayed bridge shall be designed for the following Strength Limit State for the Cable Replacement Case: 1.2DC + 1.4DW + 1.5(LL+IM) + 1.0 Cable Exchange Forces		
4.17.1.2 At least one lane of live load shall be shifted away from the cable under exchange.		
4.17.1.3 The above limit state shall be evaluated for axial only stay cable demand as per the PTI Strength A limit state defined in Clause 5.3.2.1 of PTI. The definitions of the symbols are given in Section 3.3.2 of AASHTO LRFD. A resistance factor of $\varphi = 0.78$ shall be used for the Strength Limit State of the cables for the Replacement case. Resistance factors for other components shall be in accordance with AASHTO LRFD	LRFD 3.3.2 LRFD 5.5.4 LRFD 6.5.4	

	Reference	Revision Date
Sections 5.5.4 and 6.5.4.		
4.17.2 Cable Loss Operating Condition		
4.17.2.1 The cable stayed bridge shall be designed for the following Extreme Limit State for the Cable Loss Case	PTI Stay Cable 5.5	
Loss of One Cable:	PTI Stay Cable 5.3.3	
1.1DC+1.35DW+0.75(LL+IM) +1.1CLDF		
Where,		
CLDF = Cable Loss Dynamic Forces		
4.17.2.2 The bridge shall be designed for the loss of any single cable at one time. This condition is applicable for the bridge in the final operating condition with traffic running in the striped lanes.		
4.17.2.3 A resistance factor of ϕ = 0.95 shall be used for the Extreme Limit State for the cables. Resistance factors for other components shall be in accordance with LRFD Sections 5.5.5 and 6.5.5.		
4.17.2.4 The dynamic force resulting from the sudden fracture of a cable shall be in accordance with Article 5.5 of the PTI Stay Cable recommendations.	PTI Stay Cable 5.5	

	Reference	Revision Date
5.0 Loading Combinations		
Loading combinations are in accordance with AASHTO LRFD	LRFD Table 3.4.1-1	
5.1 Load Modifiers		
5.1.1 The value of the load modifier η_i and its factors , η_D, η_R, and η_I, shall all be set equal to 1.00, unless otherwise directed by the State Bridge Engineer	ALDOT SDM Section 1	
5.2 Jacking of Superstructure – Bearing Replacement		
5.2.1 Jacking loads should consider that traffic is to be maintained during jacking operations. Load factors for both service and strength conditions should be based on AASHTO. The design of the substructure and foundation components for this condition should include live load impact when live load impact is to be considered.	LRFD Table 3.4.1-1	

	Reference	Revision Date

6.0 Material Properties

6.1 Concrete

6.1.1 28-Day Compressive Design Strength

Concrete Description	Concrete Class	Min f_c' (psi)
Retaining walls	Class A	3,000
Bridge substructure	Class A	3,000*
Box culverts, bridge superstructure (including barrier rails and deck) and substructure	Class B	4,000
Machine laid curbs, gutters, slope paving,	Class C	3,000
Underwater concrete	Class D	3,000
Prestressed Concrete Girders	XX	5,000-8,000
Prestressed Concrete Piles	XX	5,000-6,000
Main Span Bridge Towers	XX	8,000
Main Span Bridge Superstructure – Cast-in-Place Concrete	XX	6,000
Main Span Bridge Superstructure – Precast Concrete	XX	8,000

* Higher strengths may be specified in the plans. Per the Quality Control Manual for Bridge Plan Detailing "flag and add note indicating the required 28 day strength if not per the Standard Specification 3000psi (i.e., "4000 PSI").

Reference: ALDOT Std Spec Section 501 Spec Provision 12-0676(2)

ALDOT SDM Section 5

6.1.2 Coefficient of thermal expansion is taken as 0.0000060 per degree F.

Reference: LRFD 5.4.2.2

6.1.3 Coefficient of shrinkage is taken as 0.0002.

Reference: LRFD 5.4.2.3

6.1.4 Modulus of Elasticity shall be 1,820*√f'c (ksi)

Reference: LRFD 5.4.2.4

	Reference	Revision Date
6.1.5 Poisson's ratio shall be taken as 0.2.	LRFD 5.4.2.5	
6.1.6 Modulus of Rupture shall be 0.24*√f'c (ksi) for all normal weight concrete except when used to calculate the cracking moment of a member in LRFD 5.8.3.4.3, then use 0.20*√f'c (ksi).	LRFD 5.4.2.6	
6.1.7 Corrosion control		
6.1.7.1 For the purpose of crack control calculations, assume the following exposure factors:	LRFD 5.7.3.4	
6.1.7.1.1 Interchange Bridges Superstructure: γ_e = 0.75 Substructure: γ_e = 1.00		
6.1.7.1.2 Bayway Superstructure: γ_e = 0.75 Substructure: γ_e = 0.75		
6.1.7.1.3 High Level Approaches Superstructure: γ_e = 0.75 Substructure: γ_e = 1.00		
6.1.7.1.4 Cable Stay Span – See 7.9.3.2		

	Reference	Revision Date
6.1.7.2 Design clearances (concrete cover)	ALDOT SDM Section 5	

Design Clearances (Concrete Cover) for Reinforced Concrete Structures		
Bridge Component	**Location**	**Clearance (inches)**
Bridge Deck Slab	Top	2
	Bottom	1
Columns	All faces	3
Caps	Top and bottom	2
Pile Footing	From top of pile	4
Spread Footing	Bottom	4
Abutments	Bottom	3
	Top of cap	2
	Faces of backwall	2
Drilled Shafts:	Sides	6

	Reference	Revision Date
6.2 Structural Steel		
6.2.1 Main members (e.g., girders, rolled beams, lateral bracing, diaphragms, stiffeners, and vertical connection plates) shall be AASHTO M 270 Grade 36, Grade, 50 or Grade 70	ALDOT SDM Section 6	
6.2.2 Miscellaneous members (such as armor plates, ladders, catwalks, and clip angles) shall be AASHTO M 270 Grade 36 or Grade 50	ALDOT SDM Section 6	
6.2.3 The use of weathering steel is prohibited	ALDOT SDM Section 6	
6.2.4 Anchor bolts shall conform to AASHTO M 314	ALDOT SDM Section 6	
6.2.1 Steel Sheet Piling		
6.2.1.1 Permanent and temporary steel sheet piling shall conform to the requirements of AASHTO M 202 {AASHTO M 202M}	ALDOT Std Spec Section 834	

	Reference	Revision Date
6.2.2 Modulus of elasticity is taken as 29,000 ksi.	LRFD 6.4.1	
6.2.3 Coefficient of thermal expansion is taken as 0.0000065 per degree F.	LRFD 6.4.1	
6.2.4 When applicable, the Charpy V-Notch impact requirements for structural steel shall be Temperature Zone 1.		
6.3 Reinforcing Steel		
6.3.1 Reinforcing steel shall meet the requirements of AASHTO M31, ASTM A615 Grade 60 unless otherwise noted.	ALDOT Std Spec	
6.4 Pretensioning Strand		
6.4.1 Strands for precast, prestressed AASHTO girders or bulb tee girders shall be Grade 270, low-relaxation, straight prestressing strands	ALDOT SDM Section 5	
6.4.1.1 Use either 0.5” or 0.6” diameter strands with yield strength at 90% of ultimate: fpu = 270 ksi fy = 243 ksi	LRFD Table 5.4.4.1-1	
6.4.1.2 Modulus of elasticity = 28,500 ksi	LRFD 5.4.4.2	
6.5 Post-tensioning Strand and Bars		

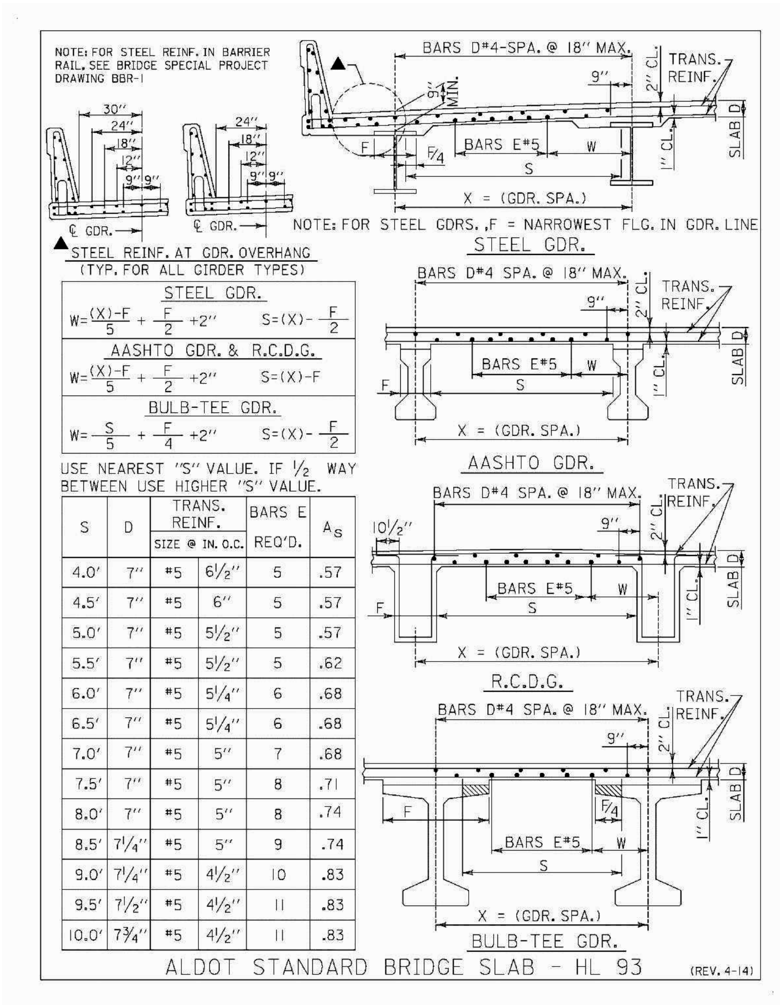
	Strands	Bars	Comments	Reference	Revision Date
Specification	A416 Grade 270 Low-Relaxation	A722 Grade 150 Type II	ASTM		
Size	0.6” strand	1 3/8” bar			
Ultimate Tensile Strength (f_{pu})	270 ksi	150 ksi		LRFD 5.4.4.1	
Yield Strength	90% f_{pu} = 243 ksi	80% f_{pu} =120 ksi		LRFD 5.4.4.1	
Modulus of Elasticity	28,500 ksi	30,000 ksi		LRFD 5.4.4.2	
Anchor Set	3/8 in	1/16 in		Gd Spec Segm 10.3	
Maximum Jacking Stress	216 ksi	108 ksi	Strand – 80% of the Ult. Tensile Strength Bar – 90% of Yield Strength	LRFD Table 5.9.3-1	
Maximum Anchoring Stress	189 ksi	105 ksi	70% of the Ultimate Tensile Strength	LRFD Table 5.9.3-1	
Maximum Stress @ Int. Location	200 ksi	-	74% of the Ultimate Tensile Strength	LRFD Table 5.9.3-1	
Friction (1)	0.23/rad	0.30/rad	Based on the use of Polyethylene Ducts	LRFD Table 5.9.5.2.2b-1	
Friction (2)	0.25/rad	-	Based on the use of Rigid Steel Pipe	LRFD Table 5.9.5.2.2b-1	
Wobble	0.0002/ft	0.0002/ft	Based on the use of either Polyethylene Ducts or Rigid Steel Pipe	LRFD Table 5.9.5.2.2b-1	

	Reference	Revision Date
6.5.1 1 ¾" bars may be utilized as temporary bars		
6.6 Stay Cable Strand		
6.6.1 Stay cables shall consist of 0.60 inch or 0.62 inch diameter seven-wire weldless low-relaxation strands conforming to ASTM A416, Grade 270. The strands shall be individually sheathed and coated with a corrosion inhibiting material.		
6.6.2 Strand modulus of elasticity = 28,500 ksi		
6.6.3 Epoxy-coated stay cable strand is not allowed.		
6.7 Fasteners		
6.7.1 Field Connections shall be bolted with 7/8 inch diameter minimum high strength AASHTO M 164 (ASTM A325) bolts in 15/16 inch diameter holes.	ALDOT SDM Section 6	
6.7.2 All nuts, washers, and bolts shall be mechanically galvanized.	ALDOT SDM Section 6	
6.7.3 High Strength Bolts: ASTM A325, type III; class B faying surface unless noted otherwise.		
6.7.4 Anchor Bolts: AASHTO M314.	ALDOT SDM Section 6	
6.8 Shear Stud Connectors		
6.8.1 In addition to LRFD requirements for shear connectors, studs shall be a minimum of 5 inches in length and shall conform to AASHTO M 169	ALDOT SDM Section 6	
6.9 Steel Cable Stayed Option – Precast Concrete Deck Panels		
6.9.1 Precast concrete deck panels shall be cast and stored a minimum of 180 days prior to installation.		

	Reference	Revision Date
7.0 Design Methodology		
7.1 General		
7.1.3 All design to be done in accordance with the LRFD specifications.	ALDOT SDM Section 1	
7.2 Construction Vibrations		
7.2.1 Vibration		
7.2.1.1 The FEIS will document commitments made regarding the development of a construction vibration monitoring system during the final design phase. The FEIS will include the University of South Alabama's vibration study and development of a monitoring plan as required.	MRB Scope	
7.2.1.1 The West Side Alternative Foundation Analysis study will also evaluate construction phasing and constructability of foundation alternates. This includes evaluation of potential vibration impacts on adjacent structures as well as environmental risks of driven versus drilled foundations for each structure.	MRB Scope	
7.3 Superstructure		
7.3.1 Deflection Control		
7.3.1.1 Live Load deflection shall be limited to L/800 without pedestrian and L/1000	LRFD 2.5.2.6.2	

		Reference	Revision Date
	with pedestrian.		
7.3.1.2	Live load deflection shall be checked for both loading options as given in LRFD 3.6.1.3.2.	ALDOT SDM Section 2	
7.3.1.3	For steel I-shaped girders, the provisions of LRFD 6.10.4.2 regarding the control of permanent deflections through flange stress controls, shall apply.		
7.3.2	Deck slab		
7.3.2.1	Required slab thickness and reinforcement based on girder type and girder spacing are provided in the table below. Designs shall be prepared based on this information. Any exceptions will require prior approval of the State Bridge Engineer.	ALDOT SDM Section 9	

	Reference	Revision Date



STEEL GDR.

$W = \frac{(X)-F}{5} + \frac{F}{2} + 2''$ $\quad S = (X) - \frac{F}{2}$

AASHTO GDR. & R.C.D.G.

$W = \frac{(X)-F}{5} + \frac{F}{2} + 2''$ $\quad S = (X) - F$

BULB-TEE GDR.

$W = \frac{S}{5} + \frac{F}{4} + 2''$ $\quad S = (X) - \frac{F}{2}$

USE NEAREST "S" VALUE. IF 1/2 WAY BETWEEN USE HIGHER "S" VALUE.

S	D	TRANS. REINF. SIZE	TRANS. REINF. @ IN. O.C.	BARS E REQ'D.	A_S
4.0'	7"	#5	6 1/2"	5	.57
4.5'	7"	#5	6"	5	.57
5.0'	7"	#5	5 1/2"	5	.57
5.5'	7"	#5	5 1/2"	5	.62
6.0'	7"	#5	5 1/4"	6	.68
6.5'	7"	#5	5 1/4"	6	.68
7.0'	7"	#5	5"	7	.68
7.5'	7"	#5	5"	8	.71
8.0'	7"	#5	5"	8	.74
8.5'	7 1/4"	#5	5"	9	.74
9.0'	7 1/4"	#5	4 1/2"	10	.83
9.5'	7 1/2"	#5	4 1/2"	11	.83
10.0'	7 3/4"	#5	4 1/2"	11	.83

7.3.2.2 Finished Grade Deck Elevations for Steel Girder Spans

Check point elevations for steel girder spans shall be as shown

Reference: ALDOT SDM Section 9

Span Length (feet)	Points per Span
≤ 100	10
100 ≤ 200	20
200 ≤ 300	30
300 ≤ 400	40
400+	50

	Reference	Revision Date
7.3.2.3 Finished Grade Deck Elevations for Concrete Girder Spans	ALDOT SDM Section 9	
7.3.2.3.1 Tenth point elevations at gutter lines and centerlines of girders shall be provided for all spans located in a vertical curve or superelevation transition		
7.3.2.3.2 Twentieth point elevations shall be provided for spans greater than 100 feet in length		
7.3.2.4 See ALDOT SDM Section 2 for deck drainage requirements additional to LRFD 2.6.6.	ALDOT SDM Section 2	
7.3.2.5 Deck reinforcement requirements for negative flexure areas for continuous steel girders shall be met.	LRFD 6.10.1.7	
7.3.2.6 Continuous ½” drip grooves shall be provided along the underside of concrete decks.		
7.3.2.7 For steel girders, haunch thickness shall be decided based on factors such as SIP form depth (function of girder spacing and slab depth), girder top flange thickness, and width and cross slope of deck. Minimum haunch thickness over edge of top flange shall not be less than ½”		
7.4 Reinforced Concrete Structures		
7.4.1 Reinforcing Steel Spacing		
7.4.1.1 Minimum and Maximum spacing of	LRFD 5.10.3	

		Reference	Revision Date
	reinforcement shall be in accordance with LRFD 5.10.3		
7.4.1.2	The maximum spacing of flexural reinforcement shall not exceed 9 in.	ALDOT SDM Section 5	
7.4.1.3	Where flexural reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above the bottom layer with layers not less than 4 inches on center.	ALDOT SDM Section 5	
7.5	Prestressed Concrete Structures		
7.5.1	Allowable AASHTO-PCI type girders to be used:	ALDOT SDM Section 5	
7.5.1.1	Type I, Type II and Type III		
7.5.1.2	BT-54, BT-63 and BT-72		
7.5.1.3	Solid and voided slab beams		
7.5.2	A Type Selection Study will be performed during the Preliminary Design Phase and will consider additional prestressed girder shapes for the High Level Approaches	MRB Scope	
7.5.2.1	BT-78 (Based on either Florida or Louisiana Standard Shapes)		
7.5.2.2	U-beams sections based on PCI SE Region recommendations for curved spliced girders.		

	Reference	Revision Date
7.5.3 All AASHTO-PCI type girders shall be designed as simple spans for all dead and live loads. In no case shall they be made continuous.	ALDOT SDM Section 5	
7.5.4 The transformed area of bonded reinforcement shall not be included in the calculations of section properties for prestressed concrete girders.	ALDOT SDM Section 5	
7.5.5 Concrete Stress Limits – Before Losses – per AASHTO LRFD	LRFD 5.9.4.1	
7.5.6 Concrete Stress Limits – After Losses	LRFD 5.9.4.2	
7.5.6.1 Maximum allowable compressive stress shall be per AASHTO LRFD	LRFD Table 5.9.4.2.1-1	
7.5.6.2 Maximum allowable tensile stress shall be limited to the following:		
7.5.6.2.1 For service load conditions that involve traffic loading, the stress limits shall be investigated using the load combination Service III.	LRFD 5.9.4.2.2	
7.5.6.2.2 Prestressed concrete members should be designed so that no tension occurs after losses under the Service III load combination limit state.	ALDOT SDM Section 5	
7.5.6.2.3 In no case shall tension in Service III conditions exceed $0.0948\sqrt{f'_c}$ (ksi).	ALDOT SDM Section 5	

	Reference	Revision Date
7.5.7 Prestress losses relative to the stress immediately before transfer may be taken as the sum of all losses or gains due to elastic shortening and losses due to long term shrinkage and creep of concrete and relaxation of the steel.	LRFD 5.9.5.1	
7.5.8 The transformed area of bonded reinforcement shall not be included in the calculations of section properties for prestressed concrete girders.	ALDOT SDM Section 5	
7.5.9 Debonded strands – per AASHTO LRFD	LRFD 5.11.4.3	
7.5.10 Beam camber due to prestress prior to pouring of the beam deck shall be based on 60 day interval between release of stand and erection of girder	ALDOT SDM Section 5	
7.5.11 Time Dependent Losses – Precast Girders		
7.5.11.1 Use the approximate method for time dependent losses. Neglect gains.	LRFD 5.9.5.3 ALDOT SDM Section 5	
7.5.11.2 Time at release is 0.75 days		
7.5.11.3 Time at deck placement is 60 days		
7.5.11.4 Final age is 27500 days		
7.5.11.5 Relative humidity is assumed to be 75%.		
7.5.11.6 The estimate of relaxation loss for low-relaxation strands is 2.4ksi.	LRFD 5.9.5.3	
7.5.12 Shear Reinforcement	ALDOT SDM Section 5	
7.5.12.1 Vertical shear steel shall be no smaller than #5 bars		
7.5.12.2 Shear reinforcing steel in the girder ends shall be spaced 4 inches on center		

	Reference	Revision Date
for a distance equal to the girder depth		
7.5.12.3 Confining steel shall be #3 bars spaced at 4" and running a distance equal to the beam depth		
7.5.12.4 Shear reinforcing steel shall be spaced so that no additional reinforcing steel is necessary to address horizontal shear forces at the top of the girder		
7.5.13 Build-up over girders	ALDOT SDM Section 5	
7.5.13.1 A minimum 1 inch haunch shall be provided at girder midspan, calculated at the critical edge of the girder flange		
7.5.13.2 Neglect haunch width and depth for section properties.		
7.5.14 Diaphragms	ALDOT SDM Section 5	
7.5.14.1 Edge beams shall be provided		
7.5.14.2 Intermediate diaphragms shall be used only as required by calculation		
7.6 Steel Structures		

		Reference	Revision Date
7.6.1	Girders shall be designed as a composite section in the region where the concrete slab is in compression under dead load. For continuous girders, the regions where the slab is in tension shall be designed as non-composite.	ALDOT SDM Section 6	
7.6.2	Use of hybrid sections (flange and web of different materials) in bridge sections (such as over bents) shall require prior approval of the State Bridge Engineer.	ALDOT SDM Section 6	
7.6.3	Shop flange splices required by a change in plate thickness or width shall not be used unless a minimum of 1,500 pounds of structural steel can be saved by the addition of the shop splice	ALDOT SDM Section 6	
7.6.4	Minimum dimensions		
7.6.4.1	Flange plates shall be a minimum of 1 inch thick and 12 inches wide.	ALDOT SDM Section 6	
7.6.5	Bolted connections		
7.6.5.1	Field connections shall be bolted with 7/8 inch diameter high strength AASHTO M 164 bolts in 15/16 inch diameter holes. All nuts, washers, and bolts shall be mechanically galvanized.	ALDOT SDM Section 6	
7.6.6	Minimum Size of Fillet Welds		
7.6.6.1	Using the table below, the minimum weld size shall be determined by the thicker of the two parts joined. The minimum fillet weld size shall be used unless a larger size is required by design based on the calculated stress. The weld size need not exceed the thickness of the thinner part joined. Fabrication of ancillary members, as defined in the currently used AWS Bridge Welding Code, is exempted	ALDOT SDM Section 6	

	Reference	Revision Date
from these requirements.		

Material Thickness of Thicker Part Joined	Minimum Size of Fillet Weld (inches)
To ¾” inclusive	1/4
Over ¾” to 1½”	5/16
Over 1½” to 2¼”	3/8
Over 2¼” to 6”	1/2

7.6.7 Camber Ordinates for Steel Girders — ALDOT SDM Section 6

7.6.7.1 The following two sets of camber ordinates shall be calculated and shown: — ALDOT SDM Section 6

7.6.7.1.1 Camber due to dead load of steel only

7.6.7.1.2 Total non-composite dead load camber (dead load of both steel and concrete)

7.6.7.2 Camber ordinates shall be calculated and shown at the same points required for finished grade elevations, as follows: — ALDOT SDM Section 6

7.6.7.2.1 Camber ordinates at tenth points shall be provided on the plans for all simple and continuous spans

7.6.7.2.2 Camber ordinates at twentieth points shall be provided for spans greater than 100 feet in length

7.6.7.2.3 Camber ordinates at fiftieth points shall be

	Reference	Revision Date
provided for spans greater than 200 feet in length		
7.6.7.2.4 Camber ordinates shall be provided at all girder splice point locations		
7.6.8 Lateral Bracing		
7.6.8.1 The connection of lateral bracing gusset plates to girders and cross frames, and members to gusset plates, shall be made using 7/8-inch diameter (minimum) ASTM A325 High Strength Bolts in standard sized holes.	ALDOT SDM Section 6	
7.6.8.2 Holes in gusset plates for lateral bracing member connections may be oversized. Hole spacing and edge distances must also be increased if oversized holes are used, according to the ALDOT exception table below.	ALDOT SDM Section 6	

Bolt Diameter (inches)	Sheared Edges (inches)	Rolled Edges of Plates or Shapes, or Gas Cut Edges (inches)
7/8	1-3/4	1-1/4
1	2	1-1/2
1-1/8	2-1/4	1-5/8
1-1/4	2-3/8	1-3/4

	Reference	Revision Date
7.6.9 Intermediate Stiffeners		
7.6.9.1 For I-girder bridges, spacing of cross-frames is recommended not to exceed 25 ft (with an absolute maximum of 30 feet). This provision will be relaxed for special cases such as extremely skewed bridges to alleviate 'nuisance stiffness' effects. In such cases a refined analysis	LRFD 6.7.4	

	Reference	Revision Date
will be required to ensure that lateral flange and web buckling issues are addressed and it's effect on allowable stresses are evaluated.		
7.6.9.2 For I-girder bridges, transverse stiffeners shall be placed on one (1) side of web plates except at cross-frames, and shall be on the inside face of exterior girders.		
7.6.10 Bearing Stiffeners		
7.6.10.1 For plate girder bridges with grade less than or equal to 4%, place bearing stiffeners normal to the bottom flange. The effect of the grade shall be considered in design of the stiffener. For grades greater than 4%, orient bearing stiffeners to be vertical under full dead load.		
7.6.10.2 For box girder bridges, place bearing stiffeners normal to the bottom flange.		
7.6.10.3 For all bearing stiffeners, jacking stiffeners and auxiliary stiffeners, specify a "Finish-to-bear" finish on the bottom flange and specify fillet welded connections to both top and bottom flanges.		
7.6.11 Longitudinal Stiffeners		
7.6.11.1 Longitudinal stiffeners may be provided for bottom flanges over piers of steel box girders to improve allowable compression stress in the bottom flanges. Bottom flange longitudinal stiffeners are cumbersome to design and require extensive detailing and fabrication. For boxes with wide		

	Reference	Revision Date
bottom flanges, longitudinal stiffeners may not be avoidable. Explore thickening of bottom flanges between shop splices straddling the pier. The additional cost of these stiffeners (including the details) should be weighed against the cost of thickening the bottom flange.		
7.6.12 Fatigue		
7.6.12.1 The details of fasteners and members including splices, stiffeners, shear connectors and bracings subjected to repeated variations or reversals of stress shall be designed for categories A through C details shown in LRFD 6.6.1.2.3 Detail Categories.		
7.6.12.2 See roadway plans for truck percentage and annual average daily traffic on each bridge to be used for fatigue design	LRFD 3.6.1.4	
7.6.13 Fracture critical members (FCM)		
7.6.13.1 The contract plans shall clearly delineate the components that are FCMs as well as those components which are not fracture critical members but require Charpy V-Notch testing.		
7.6.13.2 Two I-girder systems on non-movable structures are not allowed without prior written approval.		

	Reference	Revision Date
7.6.14 Field welding shall be permitted only for bearings and shear studs	ALDOT SDM Section 6	
7.6.15 Cantilevered deck slabs shall have their bottom surface aligned with the bottom of the girder flange.		
7.6.16 Girder Transportation		
7.6.16.1 The length of shipped pieces (girder flanges and web) joined by bolted field splices shall not exceed 140 feet.	ALDOT SDM Section 6	
7.6.16.2 The gross weight of shipped pieces shall not exceed 50 tons.	ALDOT SDM Section 6	
7.7 Segmentally Erected Concrete Box Girder		
7.7.1 The design is based on the bridge being constructed using the balanced cantilever erection technique or span by span erection.		
7.7.2 Minimum concrete strengths		
7.7.2.1 Prior to lifting segment – 2500 psi		
7.7.2.2 Prior to transverse post-tensioning – 4000 psi		
7.7.2.3 Prior to erection – 6000 psi		
7.7.3 Allowable transverse stresses during construction	LRFD Table 5.14.2.3.3-1	
7.7.4 Allowable longitudinal stresses after construction		
7.7.4.1 Allowable concrete compressive stresses	LRFD Table 5.9.4.2.1-1	
7.7.4.2 Allowable concrete tensile stresses		

	Reference	Revision Date
7.7.4.2.1 Precast - Zero tension under all service limits state loading combinations	LRFD Table 5.9.4.2.2-1	
7.7.4.2.2 CIP - 0.095 SQRT (f'c) tension under all service limits state loading combinations	LRFD Table 5.9.4.2.2-1	
7.7.4.3 Only Service Load Combination III applies for checks of longitudinal superstructure tension	LRFD 5.9.4.2.2	
7.7.5 Allowable longitudinal stresses during construction		
7.7.5.1 Allowable concrete compressive stresses, 0.50*f'ci	LRFD 5.14.2.3.3	
7.7.5.2 Allowable concrete tensile stresses	LRFD Table 5.14.2.3.3-1	
7.7.6 Allowable transverse stresses in deck after construction		
7.7.6.1 Allowable concrete compressive stress	LRFD Table 5.9.4.2.1-1	
7.7.6.2 Allowable concrete tensile stresses		
7.7.6.2.1 0.095 SQRT (f'c) tension in transverse direction in precompressed tensile zone	LRFD Table 5.9.4.2.2-1	
7.7.7 The ϕ (phi) factors to use are as follows		
7.7.7.1 Flexure factor (transverse) is 1.0.	LRFD 5.5.4.2.1	
7.7.7.2 Precast Segmental		
7.7.7.2.1 Flexure factor (long.) is 1.0. Values will be adjusted in accordance with criteria for compression controlled section requirements.	LRFD Figure C5.5.4.2.1-1	

	Reference	Revision Date
7.7.7.2.2 Shear factor is 0.90.		
7.7.7.3 Cast-in-Place Segmental		
7.7.7.3.1 Flexure factor (long.) is 1.0. Values will be adjusted in accordance with criteria for compression controlled section requirements.	LRFD Figure C5.5.4.2.1-1	
7.7.7.3.2 Shear factor is 0.90.		
7.7.8 Creep and shrinkage strains and effects shall be calculated in accordance with the CEB-FIP Model Code for Concrete Structures (1990).	CEB-FIP '90	
7.7.8.1 Relative humidity shall be 75%.	LRFD Figure 5.4.2.3.3-1	
7.7.8.2 For notional thickness calculations, the perimeter in contact with the atmosphere shall be the sum of the internal and external perimeters.		
7.7.9 Web Design		
7.7.9.1 Mild steel web reinforcement will be in accordance with shear / torsion requirements of AASHTO LRFD.	LRFD 5.8.6	
7.7.9.2 Design web reinforcement shall be the greater of:		
7.7.9.2.1 Maximum transverse moment demand plus 50% of the maximum longitudinal shear demand.		
7.7.9.2.2 Maximum longitudinal shear demand plus 50% of the maximum transverse moment demand.		

	Reference	Revision Date
7.7.10 Bottom Slab Design		
7.7.10.1 Mild steel reinforcement will be designed considering resistance to box shear demands, torsion demands, and out-of-plane flexural demands.		
7.7.10.2 Design bottom slab reinforcement to be the greater of:		
7.7.10.2.1 Maximum transverse moment demand plus 50% of the maximum shear/torsion demand.		
7.7.10.2.2 Maximum shear/torsion demand plus 50% of the maximum transverse moment demand.		
7.8 Spliced Girders		
To be developed as part of Type Selection Study	MRB Scope	
7.8.1 Pre-stressed precast girders will follow applicable sections of this design criteria		
7.8.2 Post-tensioning will follow AAHSTO LRFD	LRFD 5.9	
7.9 Cable-Stayed Main Span Design		
7.9.1 Reference Condition		
7.9.1.1 The structure reference condition corresponds to a temperature of 70º F and to time of 100 years after the first concrete element is cast. Unless noted on the Plans, the structure geometry, forces, and displacements shall be shown at the reference condition.		

	Reference	Revision Date
7.9.2 Analysis		
7.9.2.1 The dead load analysis shall include the time dependent effects of creep and shrinkage for the stage-by-stage cantilever construction and the completed bridge.		
7.9.2.2 Three dimensional computer analyses for strength, service, and extreme load combinations may be based on models utilizing section properties derived from moment-curvature analysis as appropriate. The effective section properties shall be consistent with the load levels at the sections.		
7.9.2.3 Second-order analysis shall be performed on the structure to determine the p-delta effect on the tower and superstructure under combined factored axial load and bending for the critical strength limit states. Both geometric and material nonlinearity shall be considered in the analysis.		
7.9.3 Reinforced and Prestressed Concrete Design		
7.9.3.1 All reinforced and prestressed concrete members of the bridge shall be designed in accordance with AASHTO LRFD. Concrete members may be designed as fully or partially prestressed.		
7.9.3.2 Control of Cracking		
Concrete reinforcement shall be distributed so as to meet the requirements of AASHTO LRFD at the Service limit state considering the crack		

Reference **Revision Date**

control exposure factors provided in the table below:

Component	**Exposure Factor**, γ_e	**Exposure Zone** (Most severe exposure controls exposure factor)
Pile Cap – Top Surface	0.75	Splash
Pile Cap – Sides	0.75	Splash
Pile Cap – Underside	1.0	N/A (underside of structural pile cap is separated from exposure to the river water by 18" of cast-in-place soffit forms)
Tower Leg – Below El. 187.5'	0.75	Splash / Atmospheric
Tower Leg – Above El. 187.5'	1.0	Atmospheric
Tower Cross Beam	0.75	Atmospheric (but with post-tensioning, so the more severe exposure factor is considered)
Anchor Pier Column	0.75	Splash / Atmospheric
Anchor Pier Cap Beam	0.75	Atmospheric (but with post-tensioning, so the more severe exposure factor is considered)

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Deck – Top Surface	0.75	Atmospheric (but with post-tensioning, so the more severe exposure factor is considered)
Deck – Underside	0.75	Atmospheric

7.9.3.1 Alternatively, Service Limit States may be verified by calculating theoretical crack widths directly. In this case, theoretical crack widths shall be calculated in accordance with the crack control model by R. J. Frosch, "Flexural Crack Control in Reinforced Concrete," Design and Construction Practices to Mitigate Cracking, SP-204. American Concrete Institute, Farmington Hills, MI, pp. 135–154 (reference from AASHTO LRFD) and verified to be less than the crack widths corresponding to the exposure factors given in the table above.

Exposure Factor, γ_e = 1.0: Limiting crack width = 0.017"

Exposure Factor, γ_e = 0.75: Limiting crack width = 0.012"

7.9.4 Reinforced and Prestressed Concrete Design during Construction

7.9.4.1 Reinforcing steel stresses at the construction Service limit state shall be limited to 30 ksi to control flexural cracking of concrete.

	Reference	Revision Date
7.9.5 Cable-Stayed Concrete Superstructure Design		
7.9.5.1 Concrete segmental box girder and edge girder superstructures shall be designed per section 7.7 Segmentally Erected Concrete Box Girder		
7.9.6 Structural Steel Design - General		
7.9.6.1 Fatigue design shall be in accordance with AASHTO LRFD.		
7.9.7 Structural Steel Design - Edge Girders		
7.9.7.1 The design of the edge girder, flanges, webs, and transverse stiffeners shall be based on Eurocode 3 - Design of steel structures - Plated structural elements. For all other miscellaneous details including weld capacities, shear stud capacities, fatigue categories and fatigue stress limit, splice design and bolt capacities, the provisions of AASHTO LRFD shall be used.	Eurocode 3	
7.9.7.2 To ensure that the minimum requirements of AASTHO LRFD are satisfied with respect to plate buckling, a supplementary design check will be performed based on the AASHTO LRFD requirements. The presence of the axial load only acts to shift the neutral axis such that more of the web is in compression. Therefore, the stability of the web under the combined axial and bending stresses can be appropriately addressed with the web load shedding factor, R_b, specified in AASHTO, which is a function of the depth of the web in compression.	LRFD 6.10.1.10.2	
7.9.7.3 The following summarizes the checks		

	Reference	Revision Date
that shall be performed to ensure the minimum requirements of AASHTO LRFD are satisfied:		
7.9.7.3.1 For the completed bridge, at the Strength and Service limit states, the girder longitudinal steel stresses (i.e. parallel to the bridge axis) shall be calculated based on an elastic stress distribution accounting for stress history. The calculated stresses shall be less than the following limits:		
i. Girder bottom flange stress limit in compression: $\Phi_s F_{nc} R_b$		
ii. Girder bottom flange stress limit in tension: $\Phi_s F_y$		
iii. Girder top flange stress limit in tension or compression: $\Phi_s F_y$		
Where:		
Φ_s = the material resistance factor based on the values specified in AASHTO. For combined axial compression and bending a combination of the two resistance factors shall be used as appropriate, based on the ratio of applied axial compression stress to bending stress.	LRFD 6.5.4.2	
R_b = the web load shedding factor as	LRFD 6.10.1.10.2	

		Reference	Revision Date
	specified in AASHTO. The depth of the web in compression shall consider both flexure and axial load on the section.		
	F_{nc} = the compression capacity of the discretely braced bottom flange. The compression capacity is based on flange local buckling and flange lateral torsional buckling as specified in AASHTO LRFD.	LRFD 6.10.8.2.2 LRFD 6.10.8.2.3	
	F_y = the specified minimum yield stress of the relevant top or bottom flange.		
7.9.7.4	In addition to the longitudinal stress checks, the girder shall satisfy the following AASHTO LRFD requirements:		
	7.9.7.4.1 As the steel girders are designed elastically for the Strength and Service limit states, the ultimate axial/bending capacity interaction checks of AASHTO LRFD Section 6.9.2.2 are not necessarily appropriate; however, they will be performed.	LRFD 6.9.2.2	
	7.9.7.4.2 The girder web shear capacity and transverse stiffener sizes shall be checked based on Section 6.10.9.3.1;	LRFD 6.10.9.3.1	
	7.9.7.4.3 The edge girder shall satisfy the cross section proportion limits specified in Section 6.10.2; and	LRFD 6.10.2	
	7.9.7.4.4 During construction, the non-composite girder shall		

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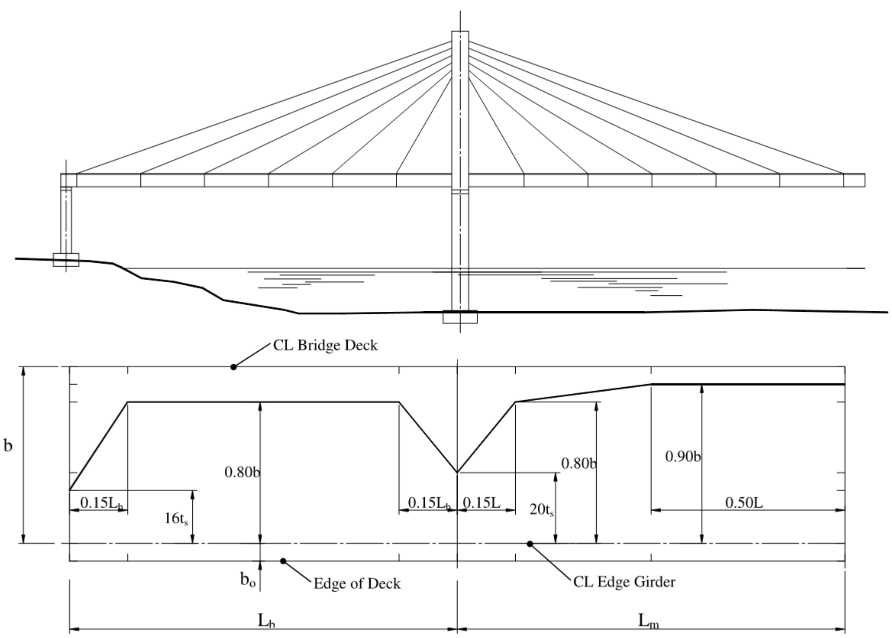
satisfy all relevant clauses of AASHTO LRFD.

7.9.7.5 Fabrication tolerances shall be to the requirements of the AASHTO LRFD Bridge Construction Specifications and applicable EN 1993-1-5:2006 Eurocode 3 requirements.

7.9.8 Composite Edge Girder Effective Width

7.9.8.1 Effective flange width for composite steel edge girders with concrete decks shall be in accordance with IBC-99-21, “Modified Effective Slab Width for Composite Cable-Stayed Bridges”, by D.D. Byers and S.L. McCabe. See figure below:



7.9.9 Stay Cable Design

7.9.9.1 Stay cables shall be designed for the service, strength, and extreme event limit states in accordance to the

	Reference	Revision Date
requirements of PTI Recommendations for Stay Cable Design, Testing and Installation. The static design of cables shall account for axial loads and the bending stresses near the anchorages that result from angle changes caused by cable sag changes, geometry changes from joint displacement and change of angle due to rotation of girder and tower.		
7.9.9.2 As defined in the PTI recommendations, the design shall allow for the replacement of any individual cable with a reduction of the live load in the area of the cable under exchange. Similarly, the design shall also be capable of withstanding the loss of any one stay cable without the occurrence of structural instability.		
7.9.9.3 The design shall allow all individual stay cables to be removed by de-tensioning at the live end anchorage.		
7.9.9.4 The stay cable anchorages shall allow for future force adjustments (increase or decrease) of at least 2.5% of the guaranteed ultimate strength of each stay cable. The Operations & Maintenance Manual shall include a procedure with details and procedures for removing/de-tensioning strands and re-installing strands.		
7.9.9.5 Stay cable anchorage assemblies and all their components shall be designed for the following minimum lateral loads:		
7.9.9.5.1 Deck Anchorages:		

	Reference	Revision Date
i. Strength Limit State: The maximum of the transverse factored load acting on the cable, or 2.5% of the maximum static cable force, applied at the centerline of cable support at the exit point of the guide pipe. The maximum of 2.5% of the maximum static cable force, or maximum of the transverse factored load acting on the cable**,** plus lateral load from 0.5° misalignment (angle) plus 3/16 in. misalignment (position), applied at the cable bearing plate. ii. Fatigue Limit State: 4% of the cable fatigue load or 1.5% of the maximum live load force, whichever is greater, applied at the centerline of cable support at the exit point of the		

	Reference	Revision Date
guide pipe.		
7.9.9.5.2 Tower Anchorages:		
i. Strength Limit State: The maximum of 2.5% of the maximum static cable force, or maximum of the transverse factored load acting on the cable**,** plus lateral load from 0.5° misalignment (angle) plus 3/16 in. misalignment (position), applied at the cable bearing plate.		
ii. Fatigue Limit State: 4% of the cable fatigue load or 1.5% of the maximum live load force, whichever is greater, applied at the cable bearing plate.		
7.9.10 Tie-Down Cable Design		
7.9.10.1 The tie-down cables shall be designed for the strength and fatigue limit states in accordance to the requirements of PTI Recommendations for Stay Cable Design, Testing and Installation. The		

	Reference	Revision Date
design of the tie-down cables shall account for axial loads and the bending stresses that result from relative movements between the cable-stayed deck and anchor piers.		
7.9.10.2 The tie-down cables shall be designed such that they can be replaced using the strand-by-strand cable replacement method.		
7.9.10.3 The tie-down cables shall be post-tensioned such that the anchor pier bearings will not be subjected to uplift at the service or strength limit states.		
7.9.10.4 At the strength limit state, the tie-down cable anchorage shall be designed for the maximum of 2.5% of the maximum static cable force, or maximum of the transverse factored load acting on the cable, plus lateral load from 0.5° misalignment (angle) plus 3/16 in. misalignment (position), applied at the bearing plate.		
7.9.11 Bearing Design		
7.9.11.1 Bearings shall be designed and detailed to be replaceable by jacking the superstructure to remove the load. The longitudinal and transverse analysis of superstructure shall consider the redistribution of reactions and forces when jacks are engaged to replace the bearings. The plans shall indicate the intended position of the jacks.		
7.9.11.2 A 1/8th inch thick elastomeric bearing pad shall be used under the masonry plate of all bearings.		

	Reference	Revision Date
7.9.12 Lock-Up Devices		
7.9.12.1 Lock-up devices (LUDs) are hydraulic cylinders whose behavior depends on the rate of load application. Quickly applied loads (all dynamic loads) are transmitted between the components connected by the LUDs (LUDs locked), whereas slowly applied loads (thermal effects and time dependent effects in concrete) result in relative displacements between the connected components, but relatively little force transfer (LUDs free).		
7.9.12.2 The LUD restraint diminishes as the force on the LUD is applied over prolonged time period. Therefore, the influence of the LUDs on live load and wind load force effects is bounded by running the live load and wind load analyses with the LUDs locked and free. For the live load case, the dynamic impact allowance is not considered for the case when the LUDs are free, as this case corresponds to the traffic loading being fixed for a prolonged period (i.e., essentially not moving). For the wind loads, the following two scenarios are considered: (1) The full wind load (dynamic and static) is applied to a model where the LUDs are locked, corresponding to a condition where the winds act relatively suddenly on the bridge. (2) The dynamic wind is applied to a model where the LUDs are locked and the static wind is applied to a model where the LUDs are free, corresponding to a case where the prolonged static forces in the LUDs have bled away.		

	Reference	Revision Date
7.9.12.3 The LUDs are also assumed locked for the following load cases: i. Vessel collision; ii. Earthquake; iii. Longitudinal braking forces.		
7.9.12.4 The LUD performance specification will define all of the required movement characteristics; however, it is currently assumed that thermal movements are accommodated by a permitted movement speed of 0.0002 in/sec and that the LUDs lock at a movement speed of 0.3 in/sec.		
7.9.13 Expansion Joint Design		
7.9.13.1 Expansion joint design shall be in accordance with AASHTO LRFD.		
7.9.14 Foundation Geotechnical Design – Cable Stay Bridge		
7.9.14.1 The Geotechnical Engineer will provide unfactored geotechnical soil design parameters and nominal geotechnical resistances of the piles. For geotechnical design of the piles, resistance factors for all load combinations shall be in accordance with AASHTO LRFD.:	MRB Scope	
7.9.14.2 Accordingly, geotechnical design resistance factors for Extreme limit states shall be as follows		
7.9.14.2.1 Axial compression resistance: ϕ = 0.		
7.9.14.2.2 Axial uplift resistance: ϕ = 0.5 (Dynamic Load Testing)		

	Reference	Revision Date
/ 0.6 (Static Load Testing)		
7.9.14.3 Geotechnical resistance factors for Strength limit states shall be as follows:		
7.9.14.3.1 Axial compression resistance: ϕ = 0.8		
7.9.14.3.2 Axial uplift resistance: ϕ = 0.5 (Dynamic Load Testing) / 0.6 (Static Load Testing)		
7.9.14.4 The factored pile axial and lateral load demands will be determined for a pile group using FB Multi-Pier.		
7.9.14.5 Force and bending moment demands acting at the underside of pile cap will be obtained from the global analysis model. The global analysis model will be furnished with soil springs obtained from the FB Multi-Pier analysis to capture the foundation stiffness.		
7.9.14.6 The factored pile resistances shall be determined by the Geotechnical Engineer by applying the appropriate resistance factors to the nominal pile axial capacities. The pile lengths shall be determined such that the factored geotechnical axial resistances of the piles exceed the factored pile axial demands.		
7.9.15 Foundation Structural Design - Piles		
7.9.15.1 Loads from the global bridge analysis will be factored and combined in accordance with this Design Criteria. Pile loads will be determined from FB Multi-Pier analysis. The results from the analyses will include moment,		

	Reference	Revision Date
shear and axial load demands in individual piles at each foundation location. The structural capacity of the piles will be verified for the resulting factored pile demands for various load combinations as per the design requirements.		
7.9.16 Foundation Structural Design - Pile Cap		
7.9.16.1 Pile cap demands will be determined from the FB Multi-Pier analysis in which the pile caps and piles are modeled explicitly.		
7.9.16.2 The pile cap model will provide axial, shear and bending moment demands suitable for pile cap sectional design. Detailing of the joints will be performed using the strut and tie method		
7.10 Bearings		
7.10.1 For pretensioned concrete beam units, elastomeric bearings Type 2, Type 4, or Type 5 shall be used in accordance with the Standard Specifications.	ALDOT SDM Section 14	
7.10.1.1 Design bearings in accordance with LRFD Method A.		
7.10.1.2 Use durometer hardness of 50 for laminated bearings.		
7.10.1.3 The minimum distance from edge of the elastomeric bearing (or the corner of the pad when pads are skewed to the pedestal) to the face of the cap shall be 4 inches.		

	Reference	Revision Date

	Reference	Revision Date
7.10.2 For steel bridge bearings, refer to AASHTO / NSBA Steel Collaboration Standard G9.1-2004 Guidelines for Steel Bridge Bearing Design and Detailing. Also, use guidance from bearing manufacturers regarding bearing 'assembly' height (includes masonry plate and beveled sole plates). Pier cap elevations and top of pedestal elevations shall be determined based on an assumed 'maximum bearing assembly height'.		
7.10.3 For post-tensioned segmental and cable-stayed bridges, high-load rotational bearings such as disc or spherical bearings may be used. Live-load rotations (particularly at anchor piers for cable-stayed bridges) shall be considered when selecting bearing types.		
7.10.4 Uplift restrains are undesirable and should only be used with written approval.		
7.10.5 Design and provide provisions for the removal and replacement of bearings.		
7.10.5.1 For steel I-girder bridges, design so that the jacks are placed directly under girder lines. Additional jacking stiffeners will be provided for this purpose. The eccentricity of the jacking load with respect to the pile shall be accounted for when designing the end bent. Alternately, the jacks may be placed between the beams below the plate diaphragms.		
7.10.5.2 For steel box girder bridges, design so that the jacks are placed directly under the jacking stiffeners attached to the interior plate diaphragms. Locate jacking stiffeners to avoid conflict of jacks with bearing assembly.		

	Reference	Revision Date
7.10.6 Determine if lateral restraint of the superstructure of a bridge is required and make necessary provisions to assure that the bridge will function as intended.		
7.10.7 Multi-rotational bearings may be guided transversely or longitudinally. For curved girder bridges, transversely guided bearings shall be guided radially. Longitudinally guided bearings shall be chorded along the direction of thermal movement (straight line between the subject guided bearing and the adjacent fixed bearing).		
7.10.8 Design masonry plates based on applied loading and bearing resistance of concrete.	LRFD 5.7.5	
7.11 Traffic Railings		
7.11.1 Utilize ALDOT Standard BBR-1 rail for typical bridges except as modified herein.		
7.11.2 Provide 42-inch TL-5 bridge traffic railings on cable-stayed bridges and high-level approaches		
7.11.3 Provide 42-inch TL-5 bridge traffic railings on lower level bridges adjacent to pier columns of upper level bridges (e.g. bridges on multi-level interchanges) if the gutter line of the lower level bridge traffic railing is within 5 feet of the upper level bridge pier column.	LRFD 3.6.5.1	
7.12 Expansion Joints		

	Reference	Revision Date
7.12.1 Generally use open joints as shown on Bridge Special Project Drawing SBD-1.	ALDOT SDM Section 14	
7.12.2 Joints for bridges using PSPC concrete girders shall be set for a 1½“ opening (perpendicular to joint) at 70°F unless otherwise required and subsequently indicated on the plans.	ALDOT SDM Section 14	
7.12.3 For steel bridges, any span arrangement (simple, continuous, or any combination) in which the fixed point to either girder end or opposing fixed point (with a joint between), exceeds 275 feet shall use a structural steel expansion dam. The joint opening at 70°F shall be calculated based on a joint opening of ½” at 120°F.	ALDOT SDM Section 14	
7.12.4 For cable-stayed and segmental bridges, modular or finger joints may be required.		
7.13 Approach Slabs		
To be established in Final Design Phase		
7.14 Substructure Design		
7.14.1 General		
7.14.1.1 Design shall be performed per LRFD		
7.14.1.2 For spans of 50 feet or less, pile bents with reinforced concrete caps may be considered if each of the following conditions are met:	ALDOT SDM Section 10	
7.14.1.2.1 Calculated scour does not prohibit the use of this structure type		
7.14.1.2.2 Subsurface material is such that piles can be driven to obtain 10 feet of penetration into natural		

	Reference	Revision Date
ground		
7.14.1.3 Reinforced concrete framed bents (or hammerhead piers when approved by the State Bridge Engineer) should generally be used when span lengths exceed 50 feet.	ALDOT SDM Section 10	
7.14.1.4 ALDOT discourages the use of hammerhead piers on a single drilled shaft. Any design utilizing hammerhead piers on a single drilled shaft shall have prior approval of the State Bridge Engineer.	ALDOT SDM Section 10	
7.14.1.5 For designs utilizing drilled shafts, the substructure cap shall be a minimum of 6 inches wider than the column (or the drilled shaft, if the shaft extends to the bottom of the cap).	ALDOT SDM Section 10	
7.14.2 End Bent and Intermediate Bent		
7.14.2.1 Live load impact will be included for exposed pier piling. Live load impact will not be included for fully buried pier piling. Live load impact will be included for end bent caps, intermediate pile bent caps, intermediate pile bent piling and exposed pier footers. Live load impact will not be included for buried pier footers and end bent piling.		
7.14.2.2 Backwalls		
7.14.2.2.1 In general, use 1’-0” backwalls on BT54, BT-63, and BT-72. Use 9” backwalls on AASHTO Type I, II and III girders.	ALDOT Detailing Manual	

	Reference	Revision Date
7.14.2.2.2 Use No. 4 bars at 8 inches on center for horizontal reinforcement and No. 6 bars at 9inches on center for vertical reinforcement unless design specifies otherwise.		
7.14.2.3 Cap		
7.14.2.3.1 When bearing elevation difference is greater than 12” between the exterior girders, sloping of the cap is required. Less than 12” difference in bearing elevations can be handled by varying the height of the pedestals or stepping the cap.	ALDOT Detailing Manual	
7.14.2.3.2 Space stirrups at 1’-0” on center (max) when pedestals are used. If cap is stepped, space stirrups at 6” on center under girders and at 1’-0” between girders	ALDOT Detailing Manual	
7.14.2.3.3 Detail pedestals as level.	ALDOT Detailing Manual	01/11/2015
7.14.3 Columns		
7.14.3.1 Columns shall be designed utilizing effective length and slenderness effects or a P-Delta analysis.		
7.14.3.2 Columns shall be assumed to be fully fixed to the footing and bents designed as rigid frames above the footing.		

	Reference	Revision Date
7.14.3.3 Long-term creep effects on the p-delta analysis will be considered.		
7.14.3.4 Column reinforcement should meet the minimum reinforcing ratio requirements of AASHTO LRFD Equation 5.7.4.2-3. A reduced effective area may be used when the cross-section is larger than that required to resist the applied loading. The minimum reinforcing ratio using the reduced area is to be the greater of one percent of the gross section or the value obtained from Eqn 5.7.4.2-3. In lieu of the reduced effective area a reduced effective concrete strength (f'c) can be used. For the reduced concrete strength the minimum reinforcing ratio obtained using Equation 5.7.4.2-3 is to be one percent of the gross section.	LRFD 5.7.4.2	
7.14.4 Pile Footings		
7.14.4.1 Footings are to be designed in accordance with the LRFD Method with applicable service criteria checks.		
7.14.4.2 Sour Considerations		
7.14.4.2.1 The top of footing for pile footing foundations shall be located below the streambed a depth equal to the estimated long-term degradation and contraction scour depth.	ALDOT SDM Section 10 FHWA Hydraulic Engineering Circular No. 18	
7.14.4.2.2 Influences such as corrosion due to exposed piling, debris collection on piling, and unbraced pile		

	Reference	Revision Date
length (exposed pile below footing plus depth to fixity) shall be considered in the design of pile footing foundation whenever the top of footing is to be constructed above the estimated long-term degradation and contraction scour depth.		
7.14.5 Driven Piling		
7.14.5.1 Pile section and length shall be determined by LRFD method		
7.14.5.2 Minimum pile sizes: Prestressed Concrete Piles (PCP): 24” square Steel H Piles: HP 14 Steel Pipe Piles: 20” diameter		
7.14.5.3 Maximum Factored Design Load per Pile (unless otherwise approved by ALDOT)		
7.14.5.3.1 Loads below are for foundation (footing) pile only. Max load will be less for pile bents depending on the height of the bent and the condition of the subsurface.	ALDOT SDM Section 10	
7.14.5.3.2 Steel “H” Piling	ALDOT SDM Section 10	

Pile Designation	Maximum Factored Design Load Allowed
HP 10 x 42	80 tons

		Reference	Revision Date

HP 12 x 53	100 tons
HP 14 x 73	140 tons
HP 14 x 89	170 tons
HP 14 x 102	195 tons
HP 14 x 117	224 tons
HP 10 x 42	410 tons

7.14.5.3.3 Concrete Prestressed Piling — ALDOT SDM Section 10

Size of Pile	Maximum Factored Design Load Allowed
14-inch Square	90 tons
16-inch Square	120 tons
18-inch Square	150 tons
20-inch Square	180 tons
24-inch Square	220 tons
30-inch Square	310 tons
36-inch Square	410 tons

7.14.5.4 Piles without lateral support shall be designed as columns. — ALDOT SDM Section 10

7.14.5.5 The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 inches. — LRFD 10.7.1.2

7.14.5.6 Minimum pile spacing center-to-center is not less than 30 inches or 2.5 pile diameters. — LRFD 10.7.1.2

7.14.5.7

7.14.5.8 Minimum pile embedment is 1' for a pinned connection.

7.14.5.9 Minimum pile embedment is 3' for a fixed connection.

	Reference	Revision Date
7.14.5.10 Test Piles		
7.14.5.10.1 A test pile shall be driven in the designated location and loaded to verify the minimum bearing capacity by static testing methods.	ALDOT Std Spec 505.03.f.2	
7.14.5.10.2 Test piles shall be driven at such locations as will permit their use in the finished structure.	ALDOT Std Spec 505.03.f.2	
7.14.5.10.3 Test piles shall be at least 10’ longer than the estimated length of the production piles.	ALDOT Std Spec 505.03.f.2	
7.14.6 Design Software for Substructure Design: LEAP Bridge RC Pier: Version 13.00.00.74 FB-MultiPier: Version 4.18.1		
7.15 Concrete Box Culverts		
To be established in Final Design Phase		
7.16 Miscellaneous Bridge		
To be established in Final Design Phase		
7.17 Retaining Walls		
To be established in Final Design Phase		
7.18 Miscellaneous Structures		
7.18.1 Lighting		
To be established in Final Design Phase		

	Reference	Revision Date
7.18.2 Sign Structures		
To be established in Final Design Phase 7.18.2.1		
7.18.3 Signal Structures		
To be established in Final Design Phase 7.18.3.1		

		Reference	Revision Date
8.0	**Load Rating**		
8.1	General		
8.1.1	Bridges shall be analytically load rated in accordance with the requirements of the current AASHTO Manual For Bridge Evaluation and the FHWA October 30, 2006, policy memorandum on Bridge Load Ratings for the National Bridge Inventory.	ALDOT SDM Section 16	
8.1.2	Rating models for pre-stressed girder bridges, non-curved steel girder bridges, and steel reinforced concrete bridges will be built on the most current version of the AASHTO Bridgeware bridge rating program or a program which can produce a bridge model that can be imported into the Department's version of Bridgeware.	ALDOT SDM Section 16	
8.1.3	Curved steel girder bridges shall be rated using the program it was designed with or another method deemed acceptable by the State Bridge Engineer.	ALDOT SDM Section 16	
8.1.4	When a bridge rating model is completed, the Maintenance Bureau and the Federal Highway Administration shall be notified with the results in writing.	ALDOT SDM Section 16	
8.1.5	The rating results in electronic format and a compatible bridge model shall be provided to the Bridge Bureau when or before the final design (Mylar drawings) is delivered.	ALDOT SDM Section 16	
8.2	Component Specific Evaluation.		

	Reference	Revision Date

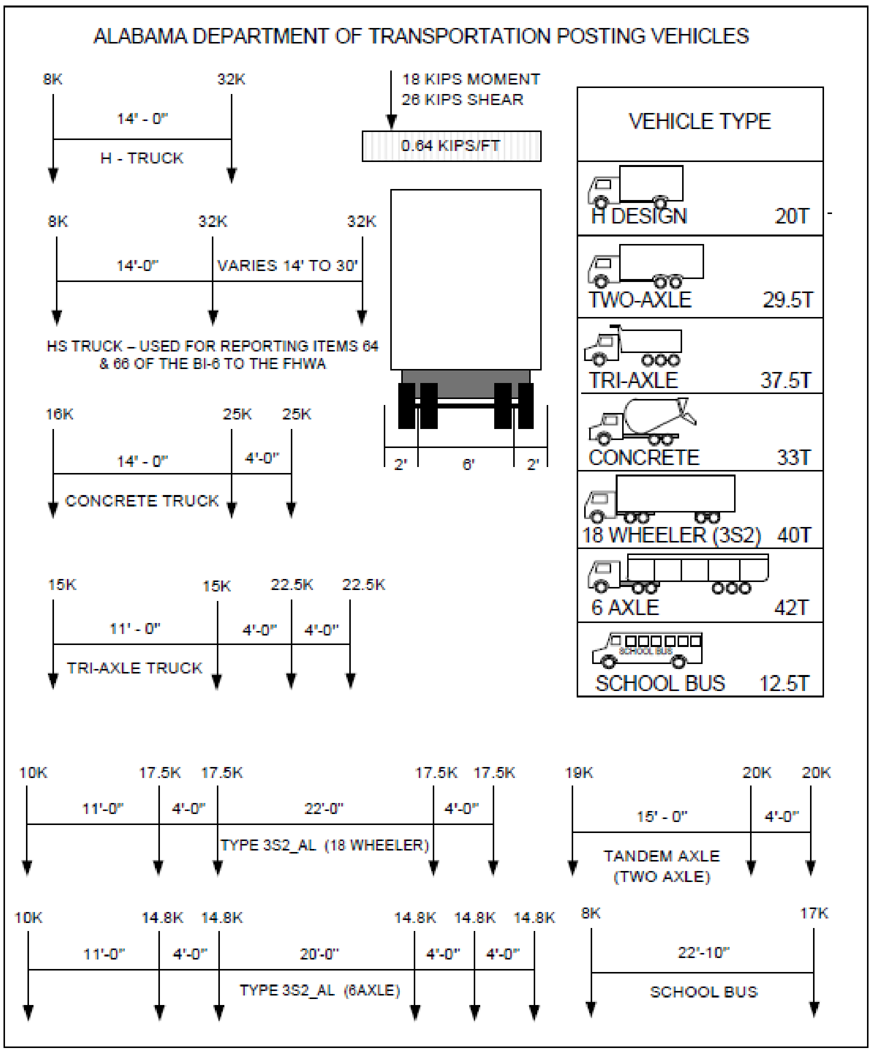
8.2.1 Transverse diaphragms do not require load rating.

8.2.2 Support of Expansion Joint Devices do not need to be load rated.

8.2.3 Anchorage for PT tendons will not be load rated.

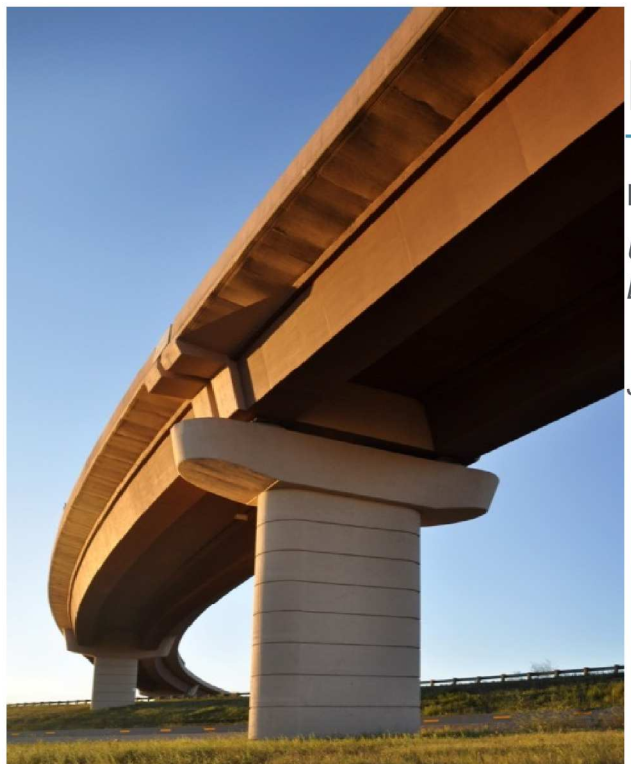
8.3 Load Rating Procedure

8.3.1 Bridge will be rated using the current Posting Trucks shown below:



Appendix C:

Use of Precast Spliced Girders on the High Level Approach Spans – White Paper



Mobile River Bridge

I-10 Mobile River Bridge and Bayway Widening

Use of Precast Spliced Girders on the High Level Approach Spans

July 15, 2016

thompson ENGINEERING

HDR

MOTT MACDONALD

1 Introduction

In recent years some Departments of Transportation throughout the country have adopted the use of precast, splice girders. The primary advantages of spliced girders include: longer spans than can be achieved with pretensioned-only precast girders; cost effectiveness versus steel superstructures; flexibility to span at grade obstacles and reduce the number of substructures; and improved bridge appearance. This report presents background information on the two most common structural configurations: spliced I-girders and spliced U-beams, and is broken into basic information for each type; brief discussion of potential application on the Mobile River Bridge high level approaches; summary of example projects from across the U.S.; and further resources

2 Basic Information on Spliced Girders

2.1 Geometry

2.1.1 Spliced I- Girders

Spliced I-girders are typically utilized for straight bridges. While research has been conducted to fabricate a curved girder, constructed bridges have used chorded segments which were angled at splice points.

Figure 1 Bellaire Causeway Bridge in Pinellas County, Florida

2.1.2 Spliced U-Beams

Because of their higher torsional rigidity, U-beams are the beam type of choice for alignments where a truly curved superstructure is desired or required. Curved U-beams have been built to radii as low as 700 feet. Beams are spaced at 20- to 22 feet, which can reduce the number of beams lines for a roadway width.

Figure 2 IH25/SH270 Ramp K, CDOT (photo [Summit Engineering Group](#))

2.2 Beam Sizes and Span Length

2.2.1 Spliced I-Girders

Many different I-beam shapes have been used for spliced girders, including AASHTO shapes, PCI shapes, and shapes developed by individual agencies. Modifications of web thickness and end blocks may be necessary to accommodate post-tensioning ducts and anchorages. Commonly used depths are 72 in., 84 in., 96 in., and 108 in. When haunched pier sections are used to achieve longer spans, these can be up to 16 foot deep over the pier.

Spliced I-girders can be used as simple spans or continuous span units. Simple spans can be in the 170 foot to 230 foot range. In continuous units, it is possible to reach a maximum span length of approximate 320 feet. At this upper length, haunched pier segments are required.

Typical concrete strengths are 8,000-10,000 psi.

Figure 3 Haunched I-girder pier segment with shoring tower

2.2.2 Spliced U-Beams

Typical spliced U-beams sections are 72 in., 84 in., and 96 in. deep.

Spliced U-beams can be used as simple spans or continuous span units, with the following approximate span limits.

	Continuous Spans			Simple Span

Figure 4 Placing interior diaphragm for curved U-beams at bent (photo [Summit Engineering Group](#))

2.3 Constructability

Spliced I-girders and U-beams have similar constructability considerations. Hauling is a significant consideration for these large beams. Because the U-beams are very heavy, field segments are limited to approximately 100 feet and may require weight permits. I-girders are lighter, therefore will generally be governed by the length that can be transported to the site, with an upper limit of approximately 170 feet. Lifting weight of field sections is also a major consideration in urban areas where placement of large cranes may prove difficult.

Either shoring towers or strongbacks are needed to support field pieces until the splice is completed and the beams are self-supported.

Figure 5 Splicing an I-girder using a strongback to support segments

An additional step for curved U-beams is to close the section after erection and prior to placing any construction or composite loads, further increasing the torsional stiffness. This is done by erecting a lid slab, either precast or cast-in-place, to connect the webs.

2.4 Durability

Both spliced I-girders and spliced U-beams are plant-manufactured precast elements, which have the inherent durability benefit of optimal casting conditions and good quality control. An informal survey of Gulf Coast precasters shows that there are multiple precast concrete fabricators with experience and formwork to produce these sections and economically deliver them to the bridge site.

Precast concrete units have better durability and lower maintenance costs than steel units in a corrosive coastal environment.

Continuous, spliced U-beam units can be designed to have integral pier diaphragms, thus eliminating bearings on interior bents. This reduces the maintenance cost of the structure over its lifespan.

2.5 Cost

The span lengths achieved with spliced girders are competitive with spans that have been traditionally reserved for steel unit construction. By introducing a competitive alternative, construction costs for these types of bridges are reduced.

As reported for the Colorado Department of Transportation Curved U-beam Bridges (refer to Section 4.3) the cost for spliced U-beams was $1,063 per foot, versus $1,393 per foot for a steel design.

2.6 Summary

Spliced I-Girders	• Simple or continuous spans • Straight (possible curved application) • Section depth 7', up to 12' if haunched • 170' to 325' spans	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Can be used in curved alignment by providing a kink at splice • Regional precasters with experience and formwork	• Some standards are available; depending on beam type, may require web and end block modification • Less torsionally stiff than U-beams, therefore not curved • Requires shoring or strongbacks for erection until splicing is complete • Less aesthetic option than U-beams
Spliced U-beams	• Simple or continuous spans • Straight or curved • Section depth 6'-8', up to 11' if haunched • Radius down to 700' • Spans up to 300' • Minimum 2 beams per cross-section	• Plant-manufactured precast element • Longer spans and fewer substructures than with pretensioned girders • Fewer beams required in cross-section than with I-girders • Considered more aesthetic than I-girders • Standards are available (PCI Zone 6 Concept drawings) • Regional precasters with	• Heavy, therefore larger demand on substructure • For transport, field sections limited to approx. 100' • Super-heavy permits likely required for transport (approx. 115-125 tons per section) • Larger erection equipment necessary • Requires shoring or strongbacks until splicing is complete

3 Potential Application for High Level Approaches

3.1 Locations of Ground Obstacles

There are several locations along the alignment of the high level approaches where spans greater than 200 feet would be useful for avoiding ground obstacles, and/or minimizing the number of substructure units over 100 feet in height.

West Side Approach Spans:

- the existing structure over Texas Street (curved alignment)
- eastbound ramp crossing under approach (curved and straight alignment)
- Palmetto Street and county jail (curved alignment)

East Side Approach Spans:

- Old Spanish Trail (curved alignment)
- Existing ramps at IH-10 and Old Spanish Trail (curved alignment)

3.2 Extension of Main Span Aesthetics

Large spliced girders can compliment the superstructure type selected for the cable-stayed main spans. If a box structure is selected, the spliced U-beams provide a similar smooth, sloped side and flat soffit.

Large splice girders also provide a smaller, and therefore less noticeable, depth transition from the cable-stayed superstructure than could be achieved with traditional pretensioned precast beams.

4 Example Projects

4.1 US27 Bridge at Moore Haven, Florida

The US27 Bridge at Moore Haven, Florida, uses a continuous, spliced bulb-T unit to span the Caloosahatchee River. The main span over the river currently holds the longest span record at 320 feet. It was constructed in 1999 at a cost of $60 per square foot. The girders are 6.75 or 8 feet deep, with 15 foot deep haunched pier sections. No falsework was used in the river; instead strongbacks secured the drop-in segment and shoring towers in the back spans were designed for uplift.

Figure 6 US 27 Bridge at Moore Haven, FL (photo FDOT)

4.2 Shelby Creek Bridge, Kentucky

The Shelby Creek Bridge is a high level bridge utilizing a five-span, continuous, spliced girder unit. The bridge was constructed without any falsework, and used precast, prestressed deck subpanels. Completed in 1991, the spliced girder bridge was $88 per square foot, which beat out the steel plate girder alternate by $417,000.

Figure 7 Shelby Creek Bridge (photo PCI Journal)

Spliced Segmental Prestressed Concrete I-Beams for Shelby Creek Bridge

4.3 Colorado Department of Transportation Curved U-beam Bridges

The following article and presentation demonstrate the Colorado Department of Transportation's (CDOT) experience in implementing curved U-beams. CDOT has been building spliced girder bridges since the early 1990s, including a site-precast curved U-beam bridge in 1995. By early 2000, CDOT had standardized the U-beam section, and began using this section to construct curved, spliced U-beams using plant-manufactured beams. Below is a summary of these projects.

Bridge	Spans	Horizontal radius

Long-Span Precast U-Girders in Colorado

Innovative Applications of Spliced Precast Concrete U Girders on Complex Longer Span Bridge Projects

5 Further Resources

5.1 NCHRP Report 517: Extending Span Ranges of Precast Prestressed Concrete Girders

This report documents the findings of NCHRP Project 12-57, which had the objective "to develop recommended load and resistance factor design (LRFD) procedures, standard details, and design examples for achieving longer spans using precast, prestressed concrete bridge girders." The report discusses design and construction issues, and provides example designs.

NCHRP Report 517: Extending Span Ranges of Precast Prestressed Concrete Girders

5.2 PCI Zone 6 U-Girder Drawings

The Precast/Prestressed Concrete Institute (PCI) developed a set of concept drawings for curved, spliced U-Girders. The drawings include cross-sections; prestressing layouts; recommended span lengths for simple span, continuous spans, and continuous spans with haunched pier sections; and erection sequences. In the preface of the drawings, the notes state that the details were derived from experience with many projects, and that the use of standard sections supports widespread industry utilization.

PCI Zone 6 (SE Region) U-Girders

5.3 *Aspire* "Sharing New Technology through PCI Bridge Technoquests"

This article, published by PCI in its *Aspire* magazine, documents the development of curved, spliced U-beams.

Sharing New Technology through PCI Bridge Technoquests

5.4 FDOT Curved Precast Spliced U-Girder Bridges

As part of their Invitation to Innovation program, the Florida Department of Transportation (FDOT) has a webpage dedicated to Curved Precast Spliced U-Girder Bridges. The page provides an overview and general requirements for use of curved spliced girders. FDOT has adopted the PCI Zone 6 sections as the preferred spliced girder.

FDOT Curved Precast Spliced U-Girder Bridges

5.5 TxDOT Extended Span Precast Girders

The Texas Department of Transportation (TxDOT) has information posted on extended span Precast I-Girders and Precast U-Girders. The U-girders are identical in cross-section to the PCI Zone 6 drawings, while the I-girders are an expansion of TxDOT's family of standard pretensioned I-girders.

TxDOT Extended Span Precast Girders

Appendix D:

Step 1:

Bridge Cross Sections

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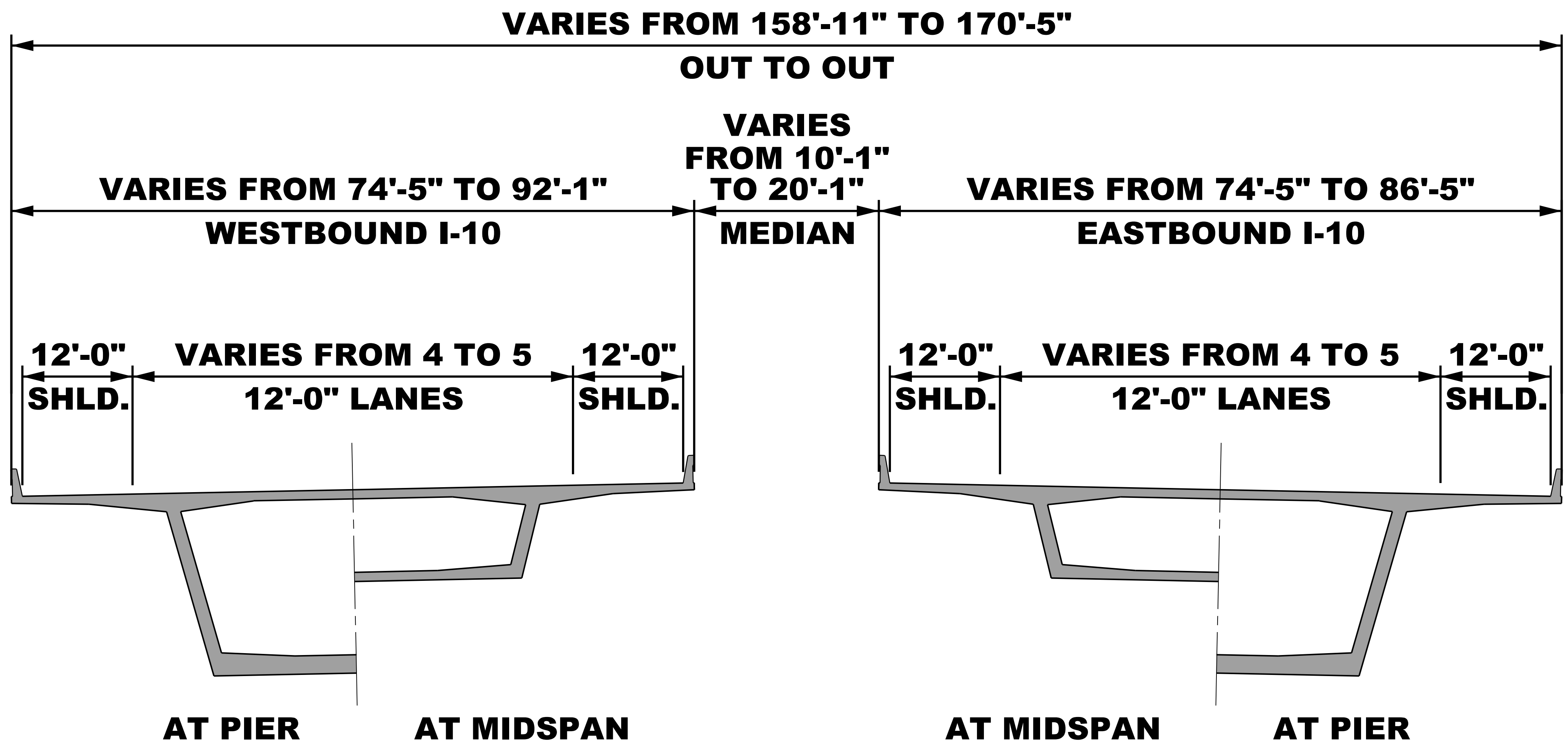

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE I
BALANCED CANTILEVER

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
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VERIFIED BY:	DATE CHECKED:	SCALE:

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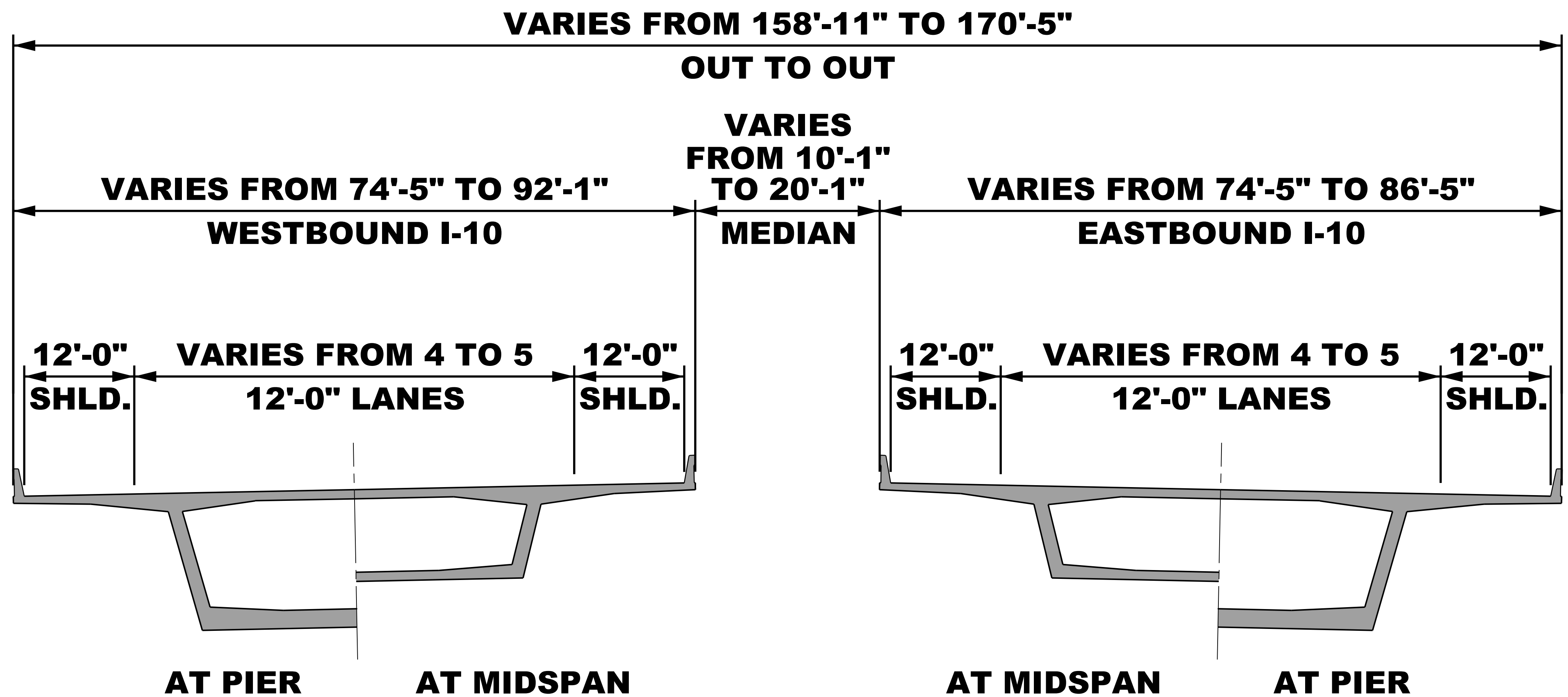

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 2
BALANCED CANTILEVER
(GANTRY)

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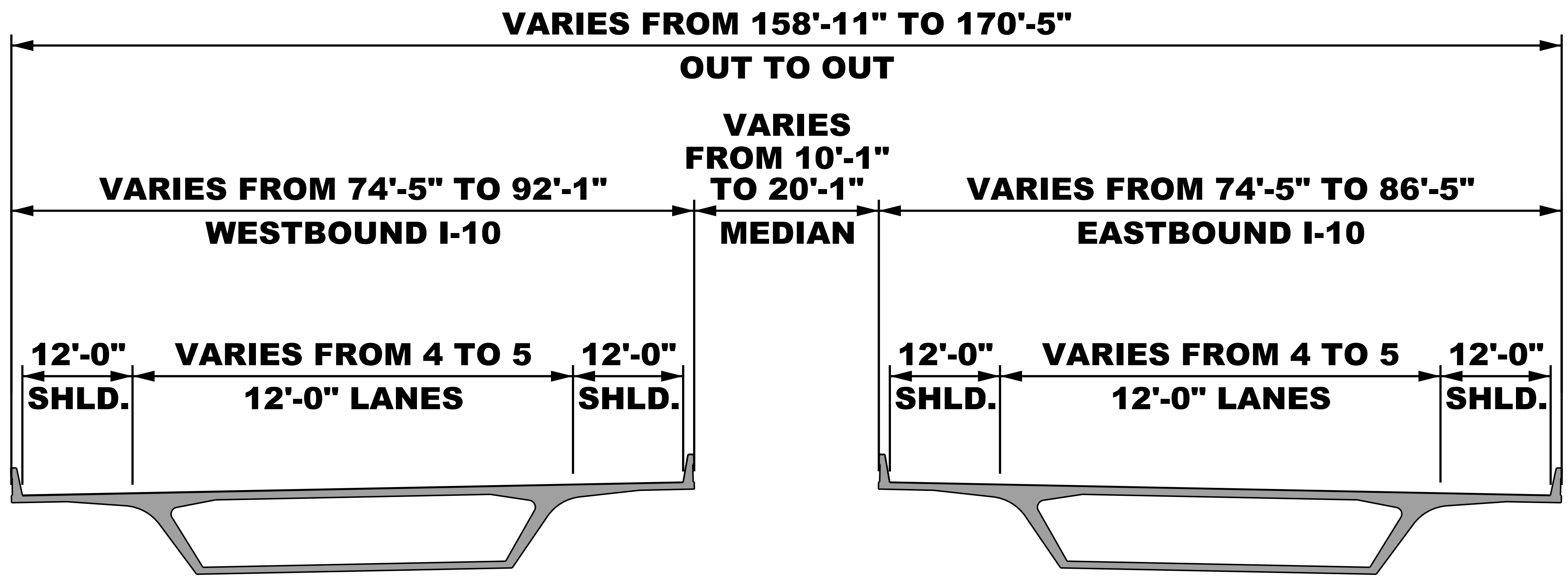

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 3
SPAN BY SPAN

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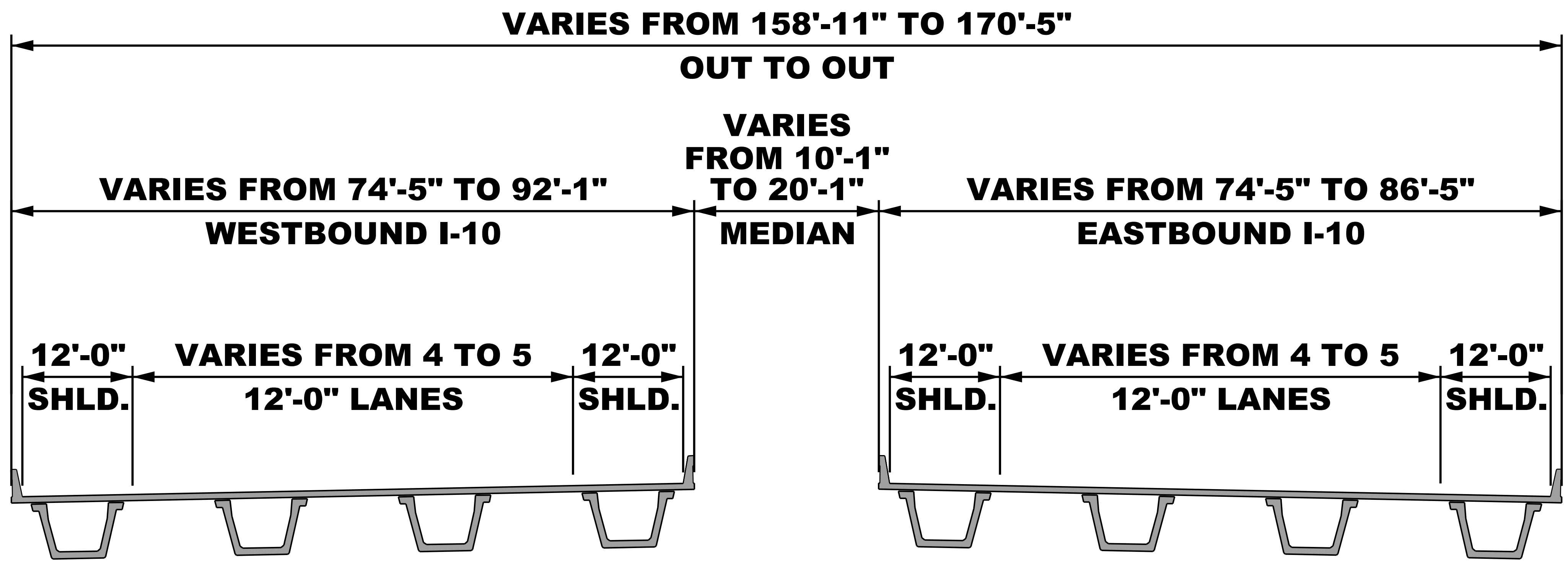

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 4
SPLICED U72 BEAMS

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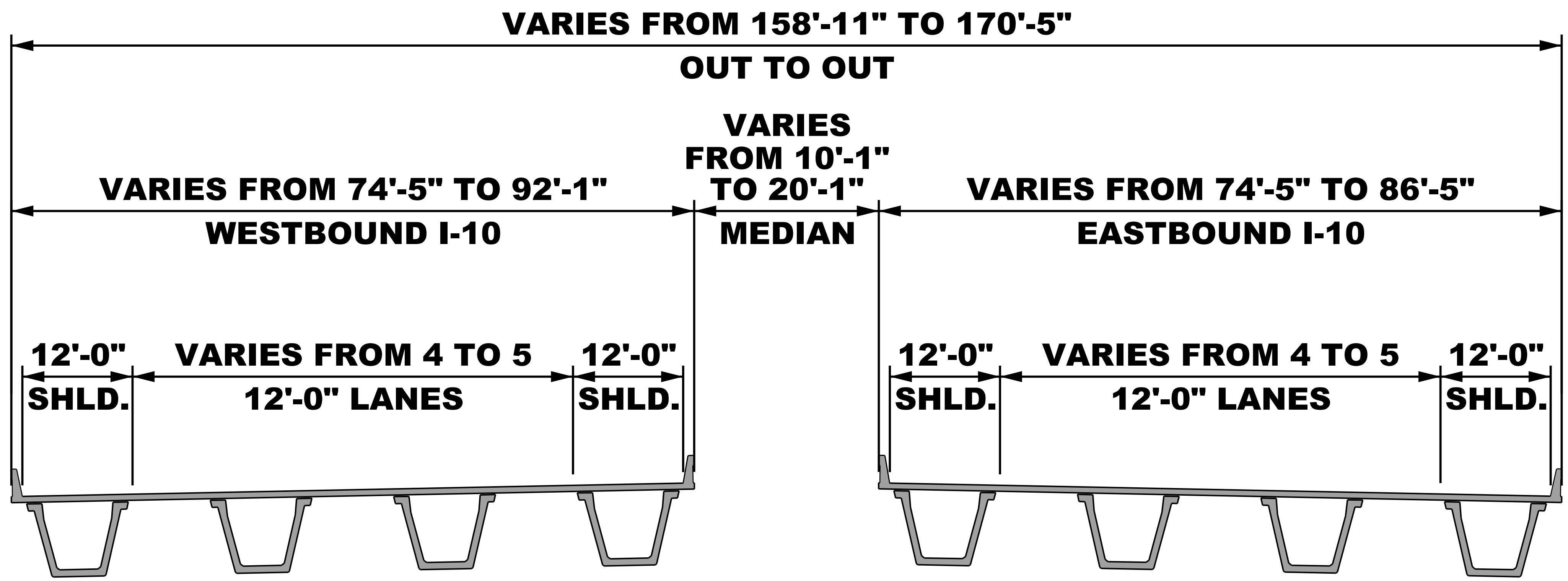

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ALABAMA DEPARTMENT OF TRANSPORTATION

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REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 5
SPLICED U96 BEAMS

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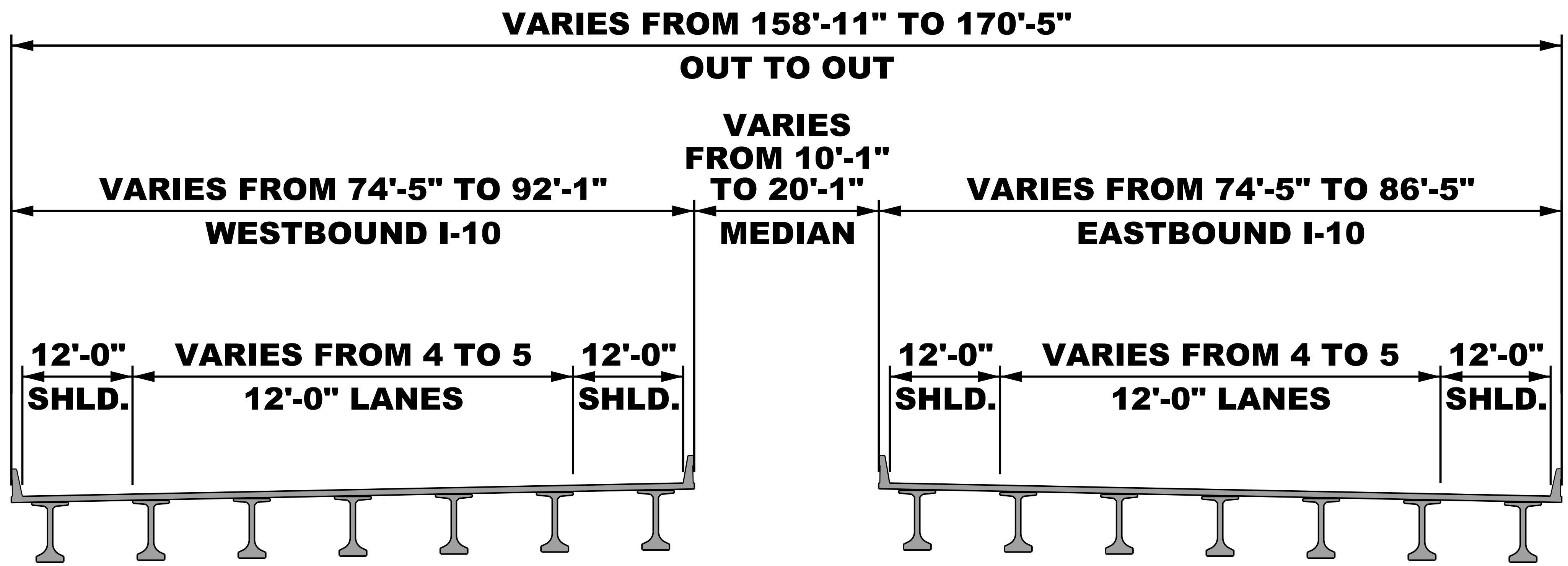

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 7
SIMPLE SPAN LG-78 BEAMS

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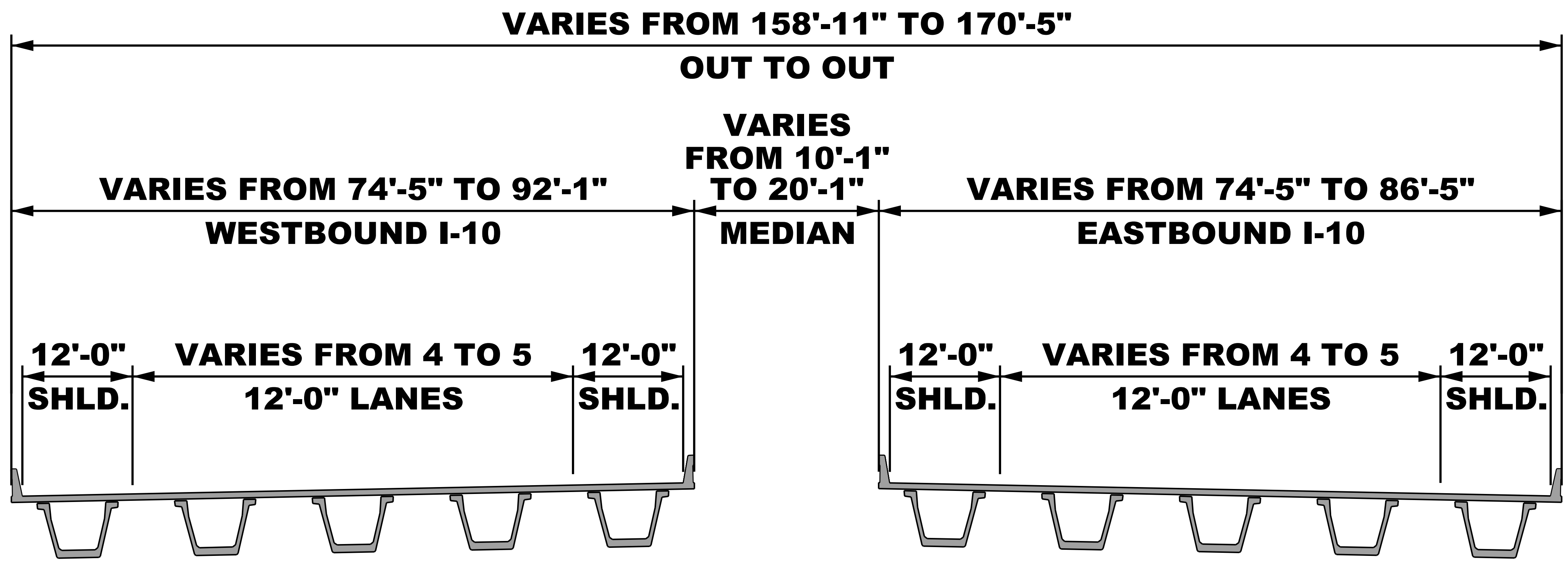

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
ALTERNATIVE 8
SIMPLE SPAN FUB-72 BEAMS

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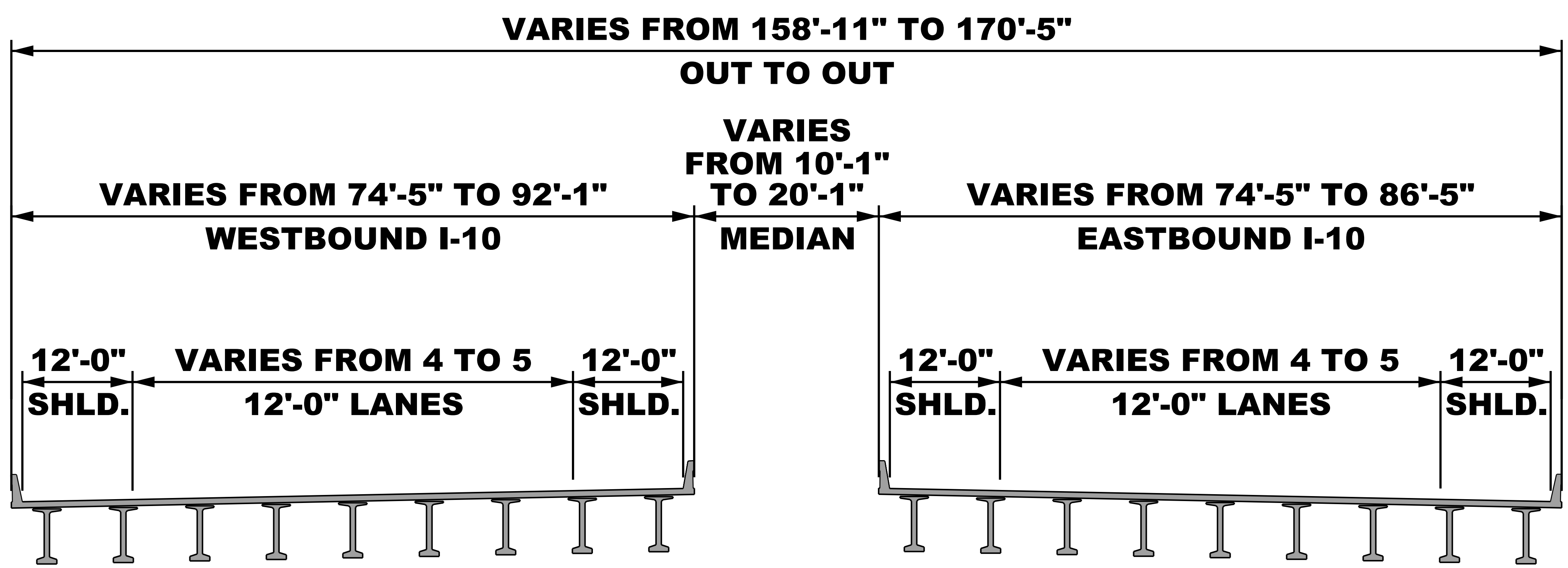
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH ALTERNATIVE 10 SIMPLE SPAN BT-72 BEAMS		
REVISIONS			
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Appendix E:

West Side Foundation Alternatives Evaluation

I-10 Mobile River Bridge and Bayway Widening

West Side Foundation Alternatives Evaluation

Mobile, Alabama

March 03, 2017

WEST SIDE FOUNDATION ALTERNATIVES EVALUATION

1 Introduction

In support of the environmental document, this study was commissioned by the Alabama Department of Transportation (ALDOT) to assess the potential vibration impacts to historic properties close to the project. Per the DEIS, *"ALDOT will conduct a study to evaluate potential vibration impacts for pile driving and to help identify construction methodologies that would avoid vibration impacts to historic properties in proximity to the project".*

In addition to the goal stated above, this initial study will also have the additional benefit of providing early indications of feasible foundation types including their:

- Initial construction costs
- Constructability benefits and challenges
- Construction schedule

Within the context of this report, the west side foundations include the west high level approach spans to the main span and the west tower and transition pier of the main span unit. The deep foundation elements considered in the preliminary foundation concepts include large diameter drilled shafts and a variety of precast and steel driven piles.

2 Approach

The process of developing feasible deep foundation concepts for the west side involved sizing foundations for each pile type and performing a relative comparison of cost and constructability among the alternatives. The steps in this process were as follows:

- Develop preliminary foundation demands for the bridge superstructure types under consideration.
- Determine preliminary foundation capacities for each foundation alternative being considered.
- Using the foundation demands and pile/shaft capacities, size the foundations elements for each alternative.
- Perform constructability review and develop construction costs for each foundation alternative.
- Perform a comparison among the foundation alternatives to assess relative cost, construction and environmental impacts.

The relative comparison used among the foundation alternatives in this study was generally based on the following considerations:

- Geometric compliance with the existing navigational channels and historic district.
- Ability to function in the expected bridge configuration and ability to resist expected structural demands.
- Constructability and construction cost.
- Vibration impacts to the historic district during construction.

Due to the preliminary nature of the loads and capacities, the comparison between the various foundation alternatives should be considered relative rather than absolute and is intended to screen foundation options and select those foundation types that will be studied further in the TS&L and final design phases of the project.

3 Preliminary Foundation Demands

For the main span unit and high level approach spans, there are several structure types currently under consideration so it was not practical to develop foundation concepts for a multitude of structure types that may or may not be selected at the culmination of the TS&L phase. To capture or bracket the feasible range of foundation sizes, the heaviest and lightest superstructure types for both the high level approach spans and main span superstructure were selected and the corresponding preliminary foundation loads calculated. Due to the early nature of the study, preliminary foundation demands were estimated combining results from previous similar projects and limited analytical modeling. Preliminary factored strength demands for the main span tower, anchor pier and high level approach spans foundations were determined using this approach. Our preliminary foundation loads accounted for:

- Superstructure type (Steel edge girder, concrete edge girder, box section)
- Span length
- Bridge deck width
- Wind area
- Design wind speed

Preliminary demands were calculated at the center of gravity at each foundation, and the factored self-weight of the foundation was separately added to the demands. Note that only factored strength demands were determined and used in the preliminary design process, however, during final design the chosen pile type will also include consideration of its ability to resist extreme events, for example, the main span tower foundations that are susceptible to vessel collision demands.

4 Preliminary Foundation Resistances

Based on the overall evaluation of the subsurface data obtained during this investigation and consideration of the project background information, it is anticipated that deep foundation systems will be required to support the bridge elements.

Such systems may consist of large diameter drilled shafts, steel and concrete driven piles, or concrete-filled driven pipe piles. Therefore, the foundation alternatives considered in this study include the following:

- 4-ft diameter drilled shafts
- 6-ft diameter drilled shafts
- 8-ft diameter drilled shafts
- 6-ft diameter steel pipe piles
- 30” PSC piles
- 36” PSC piles
- 54” diameter precast cylinder piles
- 60” diameter precast cylinder piles
- HP12x53 steel piles
- HP14x117 steel piles

Preliminary axial geotechnical resistances and estimated tip elevations for the above foundation alternatives were provided by Thompson Engineering and Dan Brown and Associates and are documented in technical memorandum dated June 27, 2016 (see Appendix A). The provided resistances assume that drilled shaft and pile layouts utilize center-to-center spacings of three diameters or larger.

No uplift or axial geotechnical resistances higher than those allowed in the ALDOT’s Structural Design Manual were used in the development of these concepts.

5 Preliminary Foundation Design Process

At the time of this study, multiple superstructure types are being considered for the high level approach spans and main span unit so it is not practical to develop foundation alternatives for all of the possible

combinations of superstructure and foundation types. Instead, preliminary designs were determined for each foundation alternative for two superstructure types – heaviest and lightest.
Main Span Unit:

- Steel edge girder option (lightest option)
- Segmental box section option (heaviest option)

High Level Approach Spans

- Precast Bulb-T Girders (lightest option)
- Segmental box girder (heaviest option)

Generating two sets of foundation designs enabled inclusion of the superstructure type relative comparison of this study. Refer to Appendices B and C for conceptual foundation design drawings.

The overall approach in the preliminary design for each foundation alternative included application of the previously described foundation demands to the center of gravity of the footing, and pile/shaft axial demands were determined using a rigid footing analysis. An iterative approach was used to add or remove shafts from the analysis until the controlling compression axial Demand/Capacity (D/C) ratio was within the range of 0.90–1.00 without uplift in any pile or drilled shaft. A target range of $0.90 \leq D/C \leq 1.00$ was implemented in order to develop a suite of foundation designs with approximately the same performance and efficiency that could be compared in a relative manner. In a few instances, adding or removing piles/shafts from the foundation layout resulted in D/C ratios that were either above or below the target range, but not within the target range. In those instances, the design that produced a D/C below the target range was used. In addition, while the pile/shaft axial demands were determined assuming a rigid footing in this study, the locations of the piles/shafts within the footings were based on best practices from previous experience. We recognize that during final design a flexible footing analysis will be performed on the selected foundation type for improved efficiency.

The resulting foundation sizes and layouts for each alternative are based only on geotechnical capacities and are comparable to those of other similar existing bridges. Our experience with structures of similar scope indicates that achieving the required structural capacity for each foundation alternative is possible. For the main span unit, the average thickness of the tower footing was held constant at 20.0-ft for each alternative. This thickness was determined from a parametric analysis involving an assumed tower vertical load on a flexible footing supported by PSC piles. The footing thickness was modified in multiple iterations of the analysis, and distribution of the assumed tower vertical load to the PSC piles was compared among the iterations. It was determined that an average thickness of 20.0-ft resulted in adequate distribution of the tower load to the piles without inducing excessively high dead load demands into the piles. The average thickness of the anchor pier footing was held constant at 14.0-ft for each alternative based on previous experience.

The overall approach to preliminary foundation design was previously described. However, the following list contains the specific steps within that approach that resulted in the foundation layouts for the main span tower foundations:

1. A single-mast pylon was assumed for the geometry of the tower pier. Because the superimposed loads were subjected to the footing at a single location, the center-center spacings between piles/shafts were minimized in order to generate a foundation with the smallest geometric footprint.
2. Rows and columns of piles/shafts were added to or removed from the foundation in the iterative design process as required until the controlling compression D/C was within or as close as possible to the target range.
3. The design was confirmed to not produce uplift in any pile or drilled shaft.

The specific steps of the anchor pier design approach were similar to that of the tower piers and are as follows:

1. A three-column configuration was assumed for the geometry of the anchor pier. The columns were assumed to be spaced at 60-ft centers. Piles/shafts were positioned within the estimated footprint at minimum center-to-center spacings to develop a layout for the first iteration of the design process.
2. Piles and shafts were added to or removed from the foundation in the iterative design process as required until the controlling compression D/C was within or as close as possible to the target range. The center-to-center spacings among the piles/shafts were modified as necessary to appropriately occupy the foundation footprint.
3. The design was checked for uplift. If uplift existed in any of the piles/shafts, the center-to-center spacing of the piles/shafts was increased to create a larger foundation footprint, and the foundation analysis was repeated with this approach until uplift no long existed in the foundation.

For the high level approach spans an approach similar to the anchor pier design approach was used.

6 Comparative Construction Cost Estimates

Foundation concepts for all ten (10) shaft/pile options were developed for the heaviest and lightest superstructure types of the high level approach spans and main span unit. These foundation concepts became the basis for a summary of quantities that was prepared for each foundation option. These costs therefore represent a rolled-up summary of a large number of cost items related to a particular element. To establish initial construction cost estimates, Armeni Consulting Services provided recommended unit prices that were applied to the summary of quantities. Unit price values were derived from a combination of historical unit price information from other similar projects. The construction estimates are in 2017 U.S. dollars and do not include right-of-way, utility relocation, engineering inspection and ALDOT overhead. At this stage of development the cost estimates should only be used for alternative comparison purposes.

Tables 1 through 3 below summarize the cost estimates for the main span towers, main span anchor pier and for the high level approach spans.

Foundation Type	Steel Superstructure	Concrete Superstructure
8' Dia. Drilled Shafts	$ 22.7M	$ 32.9M
6' Dia. Drilled Shafts	$ 20.3M	$ 30.9M
4' Dia. Drilled Shafts	$ 18.9M	$ 29,4M
6' Dia. Steel Pipe Piles	$ 11.0M	$ 17.5M
30" PSC	$ 12.2M	$ 19.0M
36" PSC	$ 13.5M	$ 20.7M
54" Concrete Cylinder Piles	$ 10.3M	$ 15.6M
60" Concrete Cylinder Piles	$ 9.1M	$ 14.7M
HP 12x53 Piles	$ 10.9M	$ 16.5M
HP 14x117 Piles	$ 8.5M	$ 11.8M

Table 1 - Main Span Towers Foundation Costs (Each Foundation)

Foundation Type	Steel Superstructure	Concrete Superstructure
8' Dia. Drilled Shafts	$ 5.5M	$ 6.9M
6' Dia. Drilled Shafts	$ 6.3M	$ 7.6M
4' Dia. Drilled Shafts	$ 6.1M	$ 7.4M
6' Dia. Steel Pipe Piles	$ 5.3M	$ 5.2M
30" PSC	$ 3.7M	$ 4.1M
36" PSC	$ 3.6M	$ 3.9M
54" Concrete Cylinder Piles	$ 3.3M	$ 3.4M
60" Concrete Cylinder Piles	$ 3.1M	$ 3.3M
HP 12x53 Piles	$ 5.4M	$ 5.2M
HP 14x117 Piles	$ 4.1M	$ 4.3M

Table 2 - Main Span Anchor Piers Foundation Costs (Each Foundation)

Foundation Type	Bulb-Ts Superstructure	Segmental Superstructure
8' Dia. Drilled Shafts	$ 1.8M	$ 3.1M
6' Dia. Drilled Shafts	$ 2.1M	$ 2.7M
4' Dia. Drilled Shafts	$ 2.2M	$ 3.3M
6' Dia. Steel Pipe Piles	$ 1.2M	$ 3.9M
30" PSC	$ 1.3M	$ 1.5M
36" PSC	$ 1.2M	$ 1.6M
54" Concrete Cylinder Piles	$ 1.0M	$ 1.9M
60" Concrete Cylinder Piles	$ 1.0M	$ 1.8M
HP 12x53 Piles	$ 1.6M	$ 1.7M
HP 14x117 Piles	$ 1.1M	$ 1.5M

Table 3 - High Level Approach Spans Foundation Costs (Average for Pier Heights 50 ft. – 190 ft., Each Foundation)

7 Constructibility Review

Based on the information gathered during this investigation, various methods for constructing the several different foundation elements were reviewed to establish the relative merits between all options. Availability of material, local expertise with the foundation type, vibration impacts, past experience, foundation footprint size, spoil removal, durability, long term maintenance and corrosion were considered to be the major differentiators between options and are included in the evaluation of the data presented in Tables 4 through 8 below.

Foundation Type	Pros	Cons	Recommendation
Drilled Shafts	-Reduced Vibration Impacts -Smaller footing footprint has lower potential to impact existing utilities and disturbed area -Noise-level is less than pile driving, potential to be installed at night -8 ft. and 6 ft. - Large pool of experienced drilling contractors, though not necessarily local -4 ft. - Large pool of experienced drilling contractors, including local	-Limits number of potential local subcontractors -Re-handling of spoil removals either by truckloads of material through the city or by muck barges -Potential disturbance and removal of hazardous waste within the spoils -Potential for anomalies/voids within the drilled shaft concrete during installation	High Construction Risk

Table 4 - Drilled Shafts

Foundation Type	Pros	Cons	Recommendation
6' Steel Pipe Piles	-Provides one of the smallest disturbed area footprints, in turn there is less potential impact on existing utilities -Non-displacement pile, densification and heave should be limited, resulting in less variation of pile lengths (will be driven to set tip) -Large Axial and Lateral Capacities - Beneficial to Main Span Unit -Ability to add Sections of Pipe to Continue Driving to Minimum tip and Capacity - Less Excavation of Spoil Material than Drilled Shatfs	-Risk of Vibration Impact to Adjacent Properties -Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends -Specialty equipment required to install these large diameter piles is not typically owned by local contractors -Large lead time required for procurement of steel pipe -Limited number of material suppliers for this diameter - Excavation and Hauling of Spoil Material within the Piles -Susceptible to Corrosion	Lower Construction Risk-Re-Assess once the Corrosion Studies are Complete

Table 5 - 6 ft. Diameter Pipe Piles

Foundation Type	Pros	Cons	Recommendation
Precast Square Piles	-Material is readily available, minimal lead time -Does not limit number of potential local subcontractors -Installed using conventional equipment owned by local contractors -Not Susceptible to Corrosion	-Risk of Vibration Impact to Adjacent Properties -Additional effort required to re-drive piles within a confined cofferdam due to pile heave or densification of soil within footing due to displacement from driving -Large footprint of footing has higher potential to impact existing utilities and disturbed area -Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends -Risk of piles breaking or damaged during shipping and handling -Risk of pile damage due to improper driving or poor hammer performance	Lower Construction Risk

Table 6 - Precast Square Piles

Foundation Type	Pros	Cons	Recommendation
Steel H Piles	-Material is readily available, minimal lead time -Installed using conventional equipment owned by local contractors (used on Cochrane Bridge) -Non-Displacement Pile type, no spoils to remove -Does not limit number of potential local subcontractors	-Risk of Vibration Impact to Adjacent Properties -Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends -Susceptible to Corrosion	Lower Construction Risk - Re-Assess once the Corrosion Studies are Complete

Table 7 - Steel H Piles

Foundation Type	Pros	Cons	Recommendation
Concrete Cylinder Piles	-Material is locally available, minimal lead time -Large Axial and Lateral Capacities - Beneficial to Main Span Unit -Not Susceptible to Corrosion	-Risk of Vibration Impact to Adjacent Properties -Limited number of suppliers, pricing controlled by the precaster not the contractor -Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends -Risk of piles breaking or damaged during shipping and handling -Risk of pile damage due to improper driving or poor hammer performance -Difficult to add Additional Sections to the Pile for Tip Depth -May Impact size of Footing due to Shallower tip Depths	High Construction Risk

Table 8 - Concrete Cylinder Piles

8 Vibration due to Pile Driving

On June 12, 2015, the University of South Alabama released a report commissioned by ALDOT to monitor pile driving vibration for the future Mobile River Bridge (See Appendix D). The study was conducted in response to concerns raised by ALDOT related to possible damage of nearby structures from ground-borne vibrations. The primary objective of this project was to determine the distance that pile driving operations can be conducted with minimal risk to nearby structures. An investigation and vibration monitoring program was developed for four pile sizes that are often used by ALDOT. The piles included thirty-six inch square and twenty-four inch square concrete piles, as well as two steel H-Piles. The piles were driven using typical installation techniques and the vibration levels at various distances from the piles were monitored.

The investigation concluded that the largest vibrations were observed while driving the thirty-six inch concrete pile. The maximum vibrations observed had a magnitude of 0.82 inches per second at fifty feet from the pile. The vibrations at 150 feet from the pile had dissipated to 0.15 inches per second. The results of the monitoring program and a literature review determined that an allowable vibration level of 0.5 inches per second for modern structures and 0.1 inches per second for potentially sensitive structures should be established for construction activity at or near the location of the project site. Additionally, a survey distance of 150 feet for modern structures and 250 feet for potentially sensitive structures was recommended.

As illustrated in Figure 1 below, the main span west tower, which is the mainline foundation element closest to the historic properties, is approximately 1,600 ft. away. This distance exceeds the minimum distance of 250 ft. recommended by the report for potentially sensitive structures.

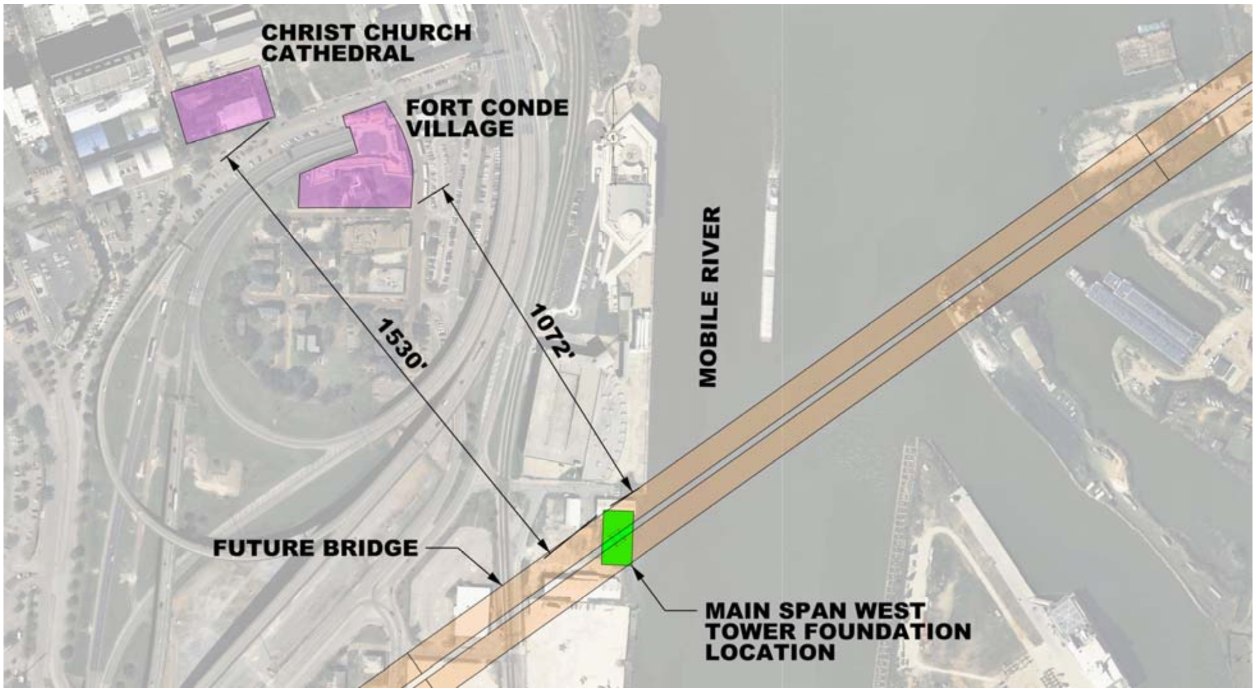

Figure 1– Distance to Historic Properties

Since the high level approach spans foundations would be further away, it is reasonably to expect those elements will not have vibration impacts to the historic properties in the vicinity of the project either.

9 Steel Pile Durability

The durability of a pile foundation can be defined by how long it performs satisfactorily, without unforeseen high maintenance costs relative to the foundations expected service life. Hence, structure foundation durability over the design life is an important consideration. In general, driven piles have proved very durable in most environments. However, environments exist for driven piles as well as any other deep foundation type where durability is an important design consideration to satisfy design life requirements.

During the West Side Foundation meeting held on July 28, 2016 in Mobile, Alabama, the design team presented ALDOT the preliminary results of the foundation studies (Refer to Appendix E for the PowerPoint presentation). The preliminary results indicated that steel piles of different shapes and sizes were cost competitive and offered several constructability benefits. To fully vet the use of steel piles on the project, ALDOT requested that the project site's corrosiveness be evaluated before further advancing steel piles to the TS&L phase.

Site specific corrosion investigations are needed to determine the corrosivity of a site and to provide appropriate corrosion mitigation measures to obtain the desired design lives. Factors that contribute to corrosion include the presence of soluble salts, soil and water resistivity, soil and water pH, and the presence of oxygen. Therefore, a limited field and laboratory testing program was implemented on land borings west of the river to establish an initial indication of the corrosiveness of the soils.

Characterizing the corrosivity of an environment is complicated due to the interaction of variables described above. Some agencies and organizations characterize the corrosivity of soil or water using a broad range of descriptors such as “severely”, “highly”, “moderately”, or “slightly”. Although the classification lists vary somewhat in the descriptions, most lists use resistivity as a leading indicator of the potential for soil and/or water corrosion.

Thompson Engineering drilled two (2) borings at MB-1 and WHLA-3 to a depth of 30 feet below existing ground surface. Using mud rotary drilling techniques, a Diedrich D50 geotechnical drilling rig equipped with an automatic standard penetration test (SPT) hammer was utilized to advance the borings. All borings were grouted to grade upon field work completion.

Samples were obtained using the split barrel sampling technique in accordance with AASHTO T-206. Recovered samples were initially examined and visually classified in the field by Thompson Engineers and were then placed on ice. Samples were transported to Test America and Thompson laboratories for electrochemical testing.

Electrochemical series soil tests (total chlorides, total sulfates, resistivity and pH) were performed on the samples. The samples were then classified following soil limit conditions for corrosive environments for steel piles provided by ALDOT and FHWA:

ALDOT:

- Resistivity less than 3,000 ohm-cm
- pH less than 5 or greater than 10
- Chlorides greater than 50 ppm
- Sulfates greater than 100 ppm

FHWA (Publication NHI-05-042):

- Resistivity less than 2,000 ohm-cm
- pH less than 4.5
- Resistivity is between 2,000 and 5,000 ohms-cm and chlorides are greater than 100 ppm or sulfate greater than 200 ppm
- Sulfates greater than 100 ppm

On the basis of the soil limits provided by ALDOT and FHWA the majority of the samples are classified as non-aggressive. It is therefore recommended that steel piles be considered feasible until additional water and soil testing is done for the main span foundation located in the Mobile River. The electrochemical test results are summarized in Table 1 in Appendix F.

Once the Mobile River samples are tested and classified, project wide steel pile corrosion mitigation recommendations will be prepared and presented to ALDOT. These mitigation strategies may include the use of sacrificial thicknesses for the steel piles. To account for a reduction in steel cross section due to corrosion, sacrificial steel thickness may be added to all permanent steel substructures. Depending on the soil classification and pile location, corrosion rates (in/yr) can be assigned to the foundation element and used to determine the necessary sacrificial thickness required to achieve the desire service life of the element. Appendix G includes two projects under construction where this corrosion strategy is being implemented.

10 Recommendations

The evaluation presented herein is preliminary and is subject to verification based on performing additional soil borings at the final locations of the bridge foundation units. Once the final foundation locations are selected and loading information, settlement tolerance, and construction methods are known, this evaluation can be finalized to design the required foundation units.

Various methods for constructing the several different foundation elements were considered and although a number of different construction methods are considered feasible for the various foundation elements, certain methods of construction appear to offer advantages in terms of least risk and probable lowest cost. The following sections provide recommended foundation types and construction methods for the individual foundation elements. It should be noted that these recommended methods are for the purpose of this feasibility report and for feasibility-level cost estimation. It is expected that upon final design, these suggested methods will be refined and modified to reflect the final design requirements.

10.1 Main Span Unit

6 ft. diameter steel pipes filled with reinforced concrete are considered the most likely foundation type for these elements. This foundation type provides the smallest footprint; lower construction risks, construction costs and driving vibrations for the main span unit (see Table 9). The pipe piles are filled with cast-in-place reinforced concrete for the upper 30 to 50 ft. This foundation type provides excellent structural resistance against horizontal loads and is a good option under the following conditions: 1) where poor soil conditions exist, such as soft mud deposits or loose sands; 2) if scour potential exist that will cause long unsupported pile lengths; or 3) if large lateral loads are anticipated. Final pile tip elevation and depth of reinforced concrete section should be determined based on the applied loading and additional soil borings taken at their final locations of the foundation unit.

10.2 High Level Approach Spans

Steel HP 14x117 piles are considered the most likely foundation type for these elements. This foundation type provides the lowest construction risks; construction costs and driving vibrations for the high level approach spans (see Table 9).
.
The evaluations and preliminary recommendations presented in this report have been formulated on the basis of generalized data in the vicinity of the proposed bridge crossings, together with current preliminary concepts for the bridge and foundations. As such, all of the preliminary conclusions presented herein are considered appropriate for concept-level evaluations of the design and for concept-level cost estimating. Experience indicates that the actual sub-soil conditions at the actual final locations of all the foundation elements may vary from those explored and presented in this report. Therefore, a comprehensive final design-specific geotechnical investigation should be performed to provide geotechnical exploration and analysis at the locations of each foundation element.

11 Considerations for Subsequent Development

As the structures are carried forward into Preliminary Design additional studies should be carried out including the following:

11.1 Pile Load Test Program

Pile load testing provides an opportunity for continuous improvement in foundation design and construction practices, while at the same time fulfilling its traditional role of design validation and routine quality control of the piling works. Load testing plays an important part in the geotechnical and structural optimization of foundation solutions and should be recognized as a potential source of project savings. Ideally, the program should be done early enough in the design phase to allow sufficient time for an objective appraisal of the test results and subsequent design revisions to incorporate the program results.
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11.2 Alternate Pile Designs

If cost data prepared after a comprehensive final design-specific geotechnical investigation doesn't clearly point to one pile material having an advantage over the other, consider providing alternate designs with both steel and concrete to promote competition within the construction industry.

Foundation Type Recommendations for High Level Approach Spans and Main Span Unit											
Bridge Type	**Item Evaluated**	**8' Dia. Drilled Shafts**	**6' Dia. Drilled Shafts**	**4' Dia. Drilled Shafts**	**6' Dia. Steel Pipe Piles**	**30" PSC**	**36" PSC**	**54" Concrete Cylinder Piles**	**60" Concrete Cylinder Piles**	**HP 12x53 Piles**	**HP 14x117 Piles**
Main Span Unit	**Construction Cost per Foundation ($M)**	27.8	25.6	24.2	14.3	15.6	17.1	13	12	13.7	10.2
	Constructibility	High Construction Risk			Lower Construction Risk - Provides Large Lateral Capacity Needed for Vessel Collision	Lower Construction Risk - Low Lateral and Vertical Capacities require large Footprint		High Construction Risk - Provides Large Lateral Capcity Needed for Vessel Collsion		Lower Construction Risk - Low Lateral and Vertical Capacities require large Footprint	
	Recommendation	Not Recommended - Highest Construction Cost and Construction Risk			Recommended - Re-Assess once Corrosion Studies are Complete	Not Recommended - Large Footprint required with Highest Cost of Driven Pile Types		Not Recommended - High Construction Risk		Not Recommended - Larger footprint required not Recommended for Main Span Unit	
High Level Approches	**Construction Cost per Foundation ($M)**	2.5	2.4	2.8	2.6	1.4	1.4	1.5	1.4	1.7	1.3
	Constructibility	High Construction Risk			Lower Construction Risk	Lower Construction Risk		High Construction Risk		Lower Construction Risk	
	Recommendation	Not Recommended - Highest Construction Cost and Construction Risk			Not Recommended - High Construction Cost	Re-Assess once Corrosion Studies are Complete		Not Recommended - High Construction Risk		Recommended - Re-Assess once Corrosion Studies are Complete	

Table 9 – Foundation Type Recommendations Summary

12 Appendices

Appendix A – Preliminary Foundation Analysis

Appendix B – Main Span Conceptual Foundation Design Drawings

Appendix C - High Level Approach Spans Conceptual Foundation Design Drawings

Appendix D - Final Report on Vibration Due to Pile Driving at the Mobile River Bridge

Appendix E - July 28, 2016 Presentation

Appendix F – West High Level Approach Corrosion Test Results

Appendix G – Sacrificial Thickness Sample Project

Appendix A – Preliminary Foundation Analysis

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TECHNICAL MEMO

Preliminary Foundation Analysis
West Side Main Pier and High Level Approach
Mobile River Bridge Project - Mobile, Alabama
ALDOT Project No.: DPI-0030(005)
Thompson Project No.: 15-1101-0300
DBA Project No.: 15-049D-02-7.3
Scope of Work Task: 7.3 West Side Alternative Foundation Analysis

To:	Mr. Manuel Carballo, P.E. - HDR, Inc.
From:	Aaron B. Hudson, P.E. W. Robert Thompson, III, P.E., D.GE
CC:	Sam Sternberg III, P.E. – Thompson Engineering, Inc.
Date:	June 27, 2016

Introduction

This Technical Memorandum (TM) presents our preliminary analysis of deep foundation systems for the West Side Main Pier and High Level Approaches. This work is part of Task 7.3 West Side Alternative Foundation Analysis being performed by the design team in support of completion of the environmental documents for the project.

The span configuration and alignment of the bridge have not been finalized, so HDR is utilizing assumed bridge loads based on experience with similar projects for their evaluations. The main span foundations are assumed to be on land or otherwise protected from vessel impact loads. The goal of this study is to provide order of magnitude estimate information to ALDOT on the general relative quantities and costs of the different foundation systems that will be considered. Selection and optimization of foundations systems will be done during future design phases once the environmental documents are completed and approved.

During a conference call on June 7, 2016, HDR requested that the following nine foundation type and size combinations be evaluated based on discussions with ALDOT:

- 8ft diameter drilled shafts
- 6ft diameter drilled shafts
- 4ft diameter drilled shafts
- 30in square prestressed concrete (PSC) piles (voided)
- 36in square prestressed concrete (PSC) piles (voided)
- 54in diameter precast concrete cylinder piles
- 60in diameter precast concrete cylinder piles
- 72in diameter steel open ended pipe (OEP) piles (assume 1in wall thickness)
- Steel HP piles (12x53 and 14x117)

The preliminary recommendations for these foundation options are to include considerations for casing for drilled shafts as well as the use of both ALDOT and Florida DOT (FDOT) maximum design values for concrete pile nominal resistance. HDR will discuss with ALDOT if they want to consider steel piling, either OEP or HP, in light of the fact that the upstream Cochrane-Africatown Bridge is supported on steel HP piles.

Since the ALDOT design guidance has relatively low allowed resistance values for PSC piles relative to other states, HDR has requested that DBA and Thompson Engineering (TE) prepare a white paper on the topic of driven concrete pile resistance for bridge structures. The intent of the white paper will be for the design team to provide ALDOT with case histories and documented studies on concrete pile resistances in order for ALDOT to make revisions to their design guidance with respect to the upper limits allowed for these piles. This white paper will be issued at a later date.

The information provided in this memo is suitable for concept development in order to begin evaluating possible deep foundation configurations. As design progresses, we will provide more detailed recommendations for foundation design based upon anticipated axial and lateral loads developed by HDR for the actual bridge configuration, including Service Limit State analyses and considerations of construction techniques and processes relative to other foundation types.

Background Information for Analysis

Initial recommendations were provided in TMs dated April 27, 2016 (drilled shafts) and May 2, 2016 (driven piles). The recommendations in those two TMs were based on:

- Available historic borings, all 100 ft. or less in depth, from nearby ALDOT projects and other structures including:
  - I-10 Tunnel Interchange
  - Alabama Cruise Terminal
  - National Maritime Museum
- A single 300-ft boring drilled near the planned East Side Main Pier location in late 2015 (not yet reported).
- General knowledge of conditions in downtown Mobile from deeper borings for commercial building projects such as the RSA tower.
- The report from a recent ALDOT driven pile research project by Steward and Cleary[1] conducted in the general location of the West Side Main Pier. This report included a soil boring to about 115ft in depth, static and dynamic load tests on a 24in PSC pile and dynamic load tests on a 36in PSC pile.
- Conservative estimates relative to the nature and strength of the soil strata below approximate elevation -100ft, based on the single 300-ft deep boring drilled on the East side.

[1] E. Steward and J. Cleary, "INVESTIGATION OF PILE SETUP (FREEZE) IN ALABAMA – Development of a Setup Prediction Method and Implementation into LRFD Driven Pile Design", Univ. of South AL, Mobile, AL, Research Project 930-839R, June 2015.

Subsurface Conditions

Three preliminary exploratory borings have been completed for the West Side Main Pier and High Level Approach structures: WHLA-1, WHLA-3 and MB-1. The boring locations were selected by TE based on the pier location information provided by HDR to be drilled in the general locations of the piers. MB-1 and WHLA-3 were both drilled to a depth of 300ft while WHLA-1 was drilled to a depth of 280ft.

WHLA-1 consisted of silty sand (SP-SM) to an approximate elevation of -90ft, followed by a layer of high-plasticity clay (CH) to approximate elevation -105ft. Underlying the high plasticity clay is another silty sand (SP-SM) to approximate elevation -185ft followed by a low-plasticity clay (CL) to approximate elevation -230ft. Silty sand (SM) was then encountered to the boring termination depth.

MB-1 was similar to WHLA-1 with the strata breaks occurring at slightly different elevations. The uppermost silty sand (SP-SM) layer extended to an approximate elevation of -85ft, followed by a layer of high-plasticity clay (CH) to approximate elevation -100ft. Underlying the high plasticity clay is the same silty sand (SP-SM) layer to approximate elevation -185ft followed by the low-plasticity clay (CL) to approximate elevation -215ft. Silty sand (SM) was then encountered to the boring termination depth.

WHLA-3 varied from WHLA-1 and MB-1 in that the second silty sand layer ended at a higher elevation. WHLA-3 began with a silty sand (SP-SM) to an approximate elevation of -85ft, followed by a layer of high-plasticity clay (CH) to approximate elevation -105ft. Underlying the high plasticity clay is the second silty sand (SP-SM) layer to approximate elevation -130ft followed by a low-plasticity clay (CL) to approximate elevation -225ft. Silty sand (SM) was then encountered to the boring termination depth.

The three borings were used to develop a general soil profile for these preliminary analyses. A basic soil profile is attached. The results of several UU triaxial shear tests were used in addition the SPT data and the experience of TE and DBA to estimate soil properties for analysis. For this preliminary study, the borings indicate that with the exception of PSC piles, most foundation systems will bear in medium dense to dense sands below elevation -100ft. Boring WHLA-3 illustrates that some piers may have more clay soils in this elevation range, but for the purposes of this study the strength limit state pile and shaft resistance has been estimated accounting for the presence of some clay in this zone. The detailed analysis for strength and service limit state conditions in future design phases will address the specific soil profiles at each pier using the actual bridge loads determined by HDR.

Preliminary Drilled Shaft Recommendations

The three borings drilled to this point on the West Side indicate good bearing conditions for drilled shafts at elevations between elevations -140ft and -180ft. A stiff to hard clay strata is present starting at approximately -180ft. While this is a suitable strata, higher values of unit base resistance are available for shafts bearing in the dense sands above, particularly if base grouting is considered during later stages of design.

Due to the broad assumptions being made at this point in the design, we are providing a recommended factored resistance for the various diameter drilled shafts bearing at elevations -140ft and -180ft, corresponding to the most likely range for suitability based on ground conditions and construction considerations. The factored resistance for depths in between may be estimated by interpolation. The use of base grouting to increase base resistance and/or deeper shafts can be evaluated to reduce the number of shafts per foundation unit as the design process continues.

Factored resistance is estimated using a **resistance factor of 0.7 for compression and 0.5 on side resistance for uplift** on the basis that field load tests will eventually be performed to confirm the design. The listed uplift resistance does NOT include shaft or cap weight. **Table 1** below lists the recommended **factored** resistance for the **Strength Limit State** based on Boring MB-1. Shafts for the interior approach piers could be 10 to 20 feet shorter for the same resistance.

Table 1: Preliminary Estimated Factored Resistance for Strength Limit State – Drilled Shafts

Drilled Shaft Diameter (ft)	Tip Elevation (ft)	Factored Resistance - (kips)	
		Compression	Uplift
4	-140	2,300	1,350
	-180	3,000	1,900
6	-140	3,700	2,000
	-180	4,800	2,800
8	-140	5,400	2,700
	-180	6,900	3,800

For estimating purposes, the use of a temporary casing to a tip elevation of -30ft (approximately 40 ft) should be included. Permanent casing will likely not be required for shafts constructed on land, but will be needed for any shafts constructed in the river.

Preliminary Driven Pile Recommendations

The boring data, along with the included load test information results, indicate that suitable bearing of PSC piles is available at elevations of -60 feet and below in medium dense sandy soils. Concrete cylinder, OEP, and HP piles will be driven to deeper elevations to bear in either stiff to hard clays and medium dense to dense sands.

30in and 36in PSC

The 2015 ALDOT Structural Design Manual lists a "maximum factored design load allowed" for 30in and 36in PSC piles of 620 kips and 820 kips, respectively. The "maximum factored design load" would thus be the maximum factored resistance allowed for design.

Based on the broad assumptions being made at this point in the design, the ALDOT maximum factored resistance for the PSC piles can be achieved at tip elevations of -65ft and deeper as listed in Table 2. The tip elevations are estimated at the **Strength Limit State** using a **resistance factor of 0.8 for compression and 0.6 on side resistance for uplift** on the basis that static and dynamic field load tests will be performed to confirm the design. The listed uplift resistance does NOT include pile or cap weight.

It is likely that higher resistance values can be achieved based on the experience of Florida DOT (FDOT) and Mississippi DOT (MDOT) along the Gulf Coast, as well as the ALDOT load test at this site. FDOT, which routinely uses PSC piles for bridge foundations, allows nominal resistance of up to 1,200 kips for 30in piles, corresponding to factored resistance of 960 kips for 30in piles using a resistance factor of 0.8. This indicates that these piles can routinely be installed to higher factored loads than currently allowed by ALDOT. FDOT does not list a maximum for 36in PSC piles. We have assumed 1,500 kips for the nominal resistance for 36in piles.

Table 2: Preliminary Estimated Factored Resistance for Strength Limit State – PSC Piles

	ALDOT Standards			FDOT Standards		
Pile (in)	Tip Elev. (ft)	Compression (kips)	Uplift (Kips)	Tip Elev. (ft)	Compression (kips)	Uplift (Kips)
30	-65	620	300	-90	960	640
36	-65	820	400	-90	1200	750

54in and 60in Concrete Cylinder Piles

ALDOT does not currently have published maximum factored load values for concrete cylinder piles. FDOT allows nominal resistance of up to 3,100 kips for 54in piles and 4,000 kips for 60in piles, corresponding to factored resistances of 2,480 kips for 54in piles and 3,200 kips for 60in piles using a resistance factor of 0.8. Table 3 below lists the estimated tip elevation to achieve **factored** resistances for 54in and 60in concrete cylinder piles for the **Strength Limit State** based on the broad assumptions being made at this point in the design. The factored resistance below is estimated for the FDOT maximum loads using **a resistance factor of 0.8 for compression and 0.6 on side resistance for uplift** on the basis that static and dynamic field load tests will be performed to confirm the design. The listed uplift resistance does NOT include pile or cap weight.

Table 3: Preliminary Estimated Factored Resistance for Strength Limit State – Cylinder Piles

Driven Pile Size (in)	Pile Type	Factored Resistance - (kips)		Tip Elevation (ft)
		Compression	Uplift	
54	cylinder	2,480	1,280	-110
60		3,200	1,600	-120

72in Open End Pipe Piles

ALDOT does not dot have any experience with steel OEPs, however they have suggested to HDR that they may be receptive to considering them if the subsurface conditions are favorable. HDR has requested our evaluation of 72in diameter OEPs based on their experience on similar projects. Our experience indicates that to be effective, they would need to be driven very deep to achieve a high resistance. Based on the soil conditions, we would target driving OEPs to a tip elevation of around -270 feet for a **factored** resistance of 9,000 kips in compression and 6,000 kips in uplift. These would be for the **Strength Limit State** using **a resistance factor of 0.8 for compression and 0.6 on side resistance for uplift** on the basis that static and dynamic field load tests will be performed to confirm the design. The listed uplift resistance does NOT include pile or cap weight.

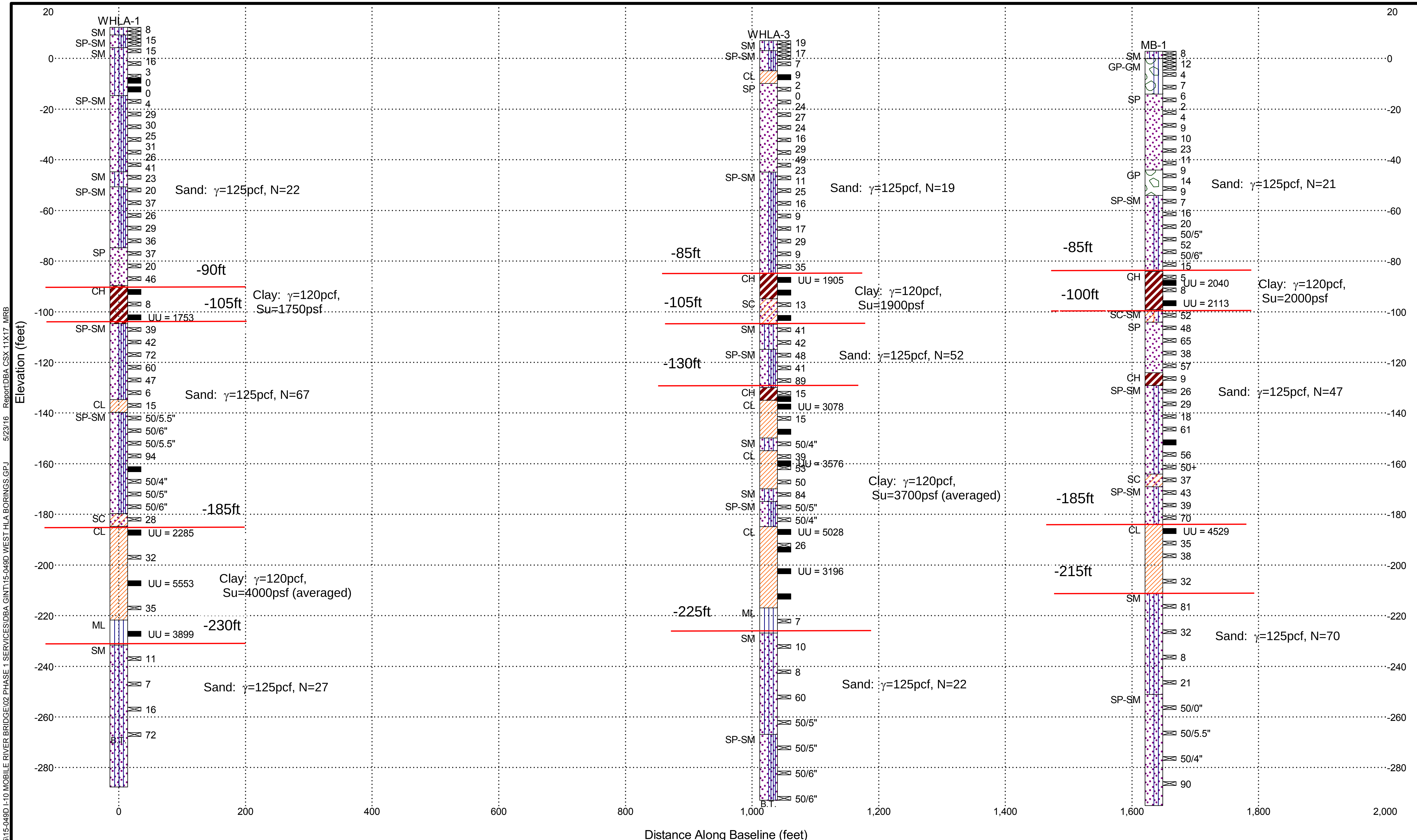
HP Piles

The 2015 ALDOT Structural Design Manual lists a "maximum factored design load allowed" for 12x53 and 14x117 HP piles of 200 kips and 448 kips, respectively, The "maximum factored design load" would thus be the maximum factored resistance allowed for design.

Based on the broad assumptions being made at this point in the design, the ALDOT maximum factored resistance for the HP piles can be achieved at tip elevations as listed in Table 4. The tip elevations are estimated at the **Strength Limit State** using a **resistance factor of 0.8 for compression and 0.6 on side resistance for uplift** on the basis that static and dynamic field load tests will be performed to confirm the design.

Table 4: Preliminary Estimated Factored Resistance for Strength Limit State – HP Piles

	ALDOT Standards		
Pile (in)	Tip Elev. (ft)	Compression (kips)	Uplift (Kips)
HP 12x53	-90	200	140
HP 14 x 117	-120	448	290



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SUBSURFACE CROSS-SECTION
Mobile River Bridge - West HLA Borings

DBA Project No.: 15-049D: Phase 02 - Phase 1 Services

Date: 5/23/2016 Scale: 1" = 35' Vertical 1" = 140' Horizontal

LEGEND:

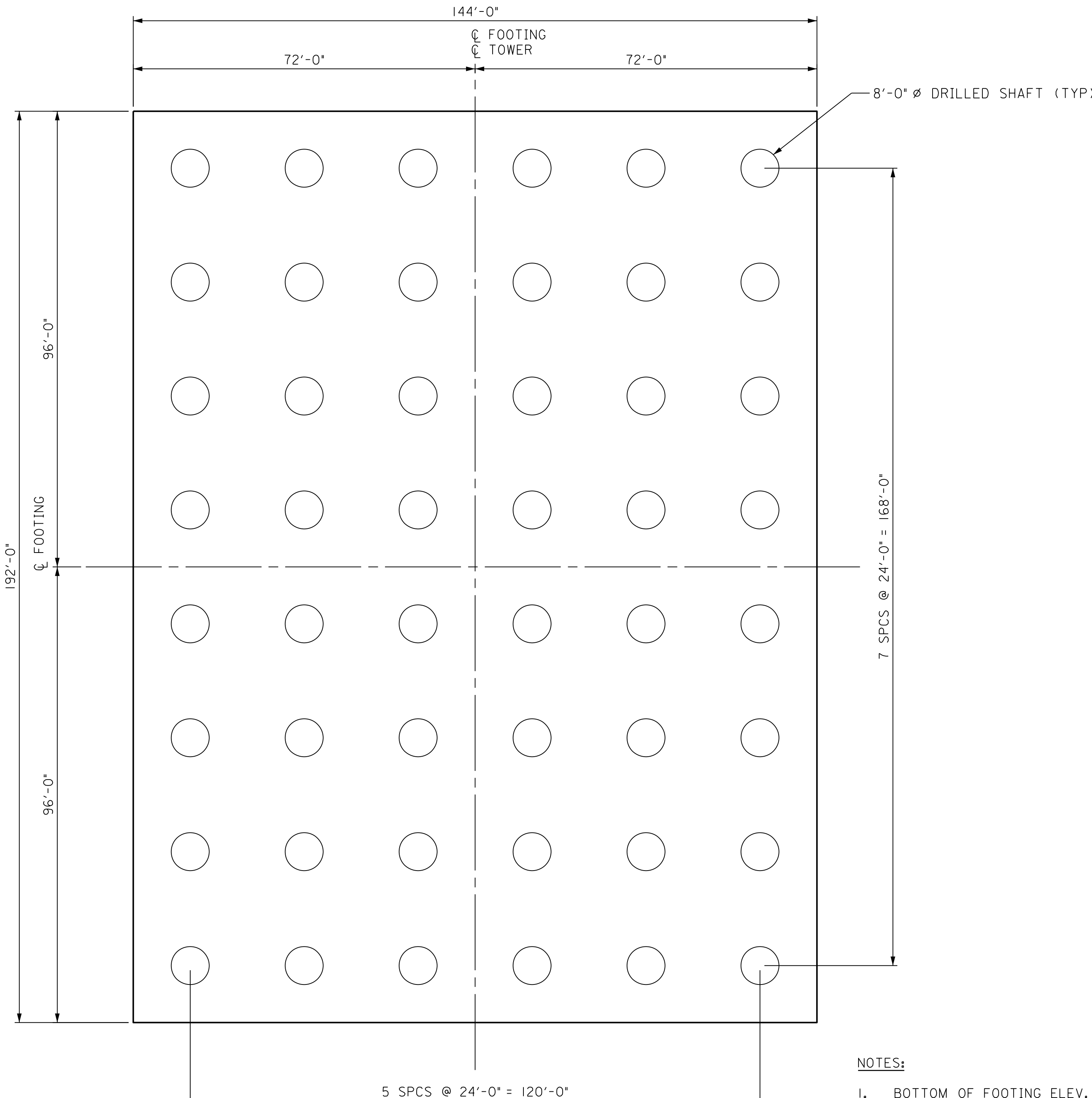
Classification and/or Description | Material Symbol | N (bpf) | Recovery%/RQD% | Qu (psi)

Water level at time of boring
Delayed water level
Cave-in depth
Perched water level.

B.T. = Boring Terminated A.R. = Auger Refusal

NOTES:
Elevations and locations are approximate based on available data.

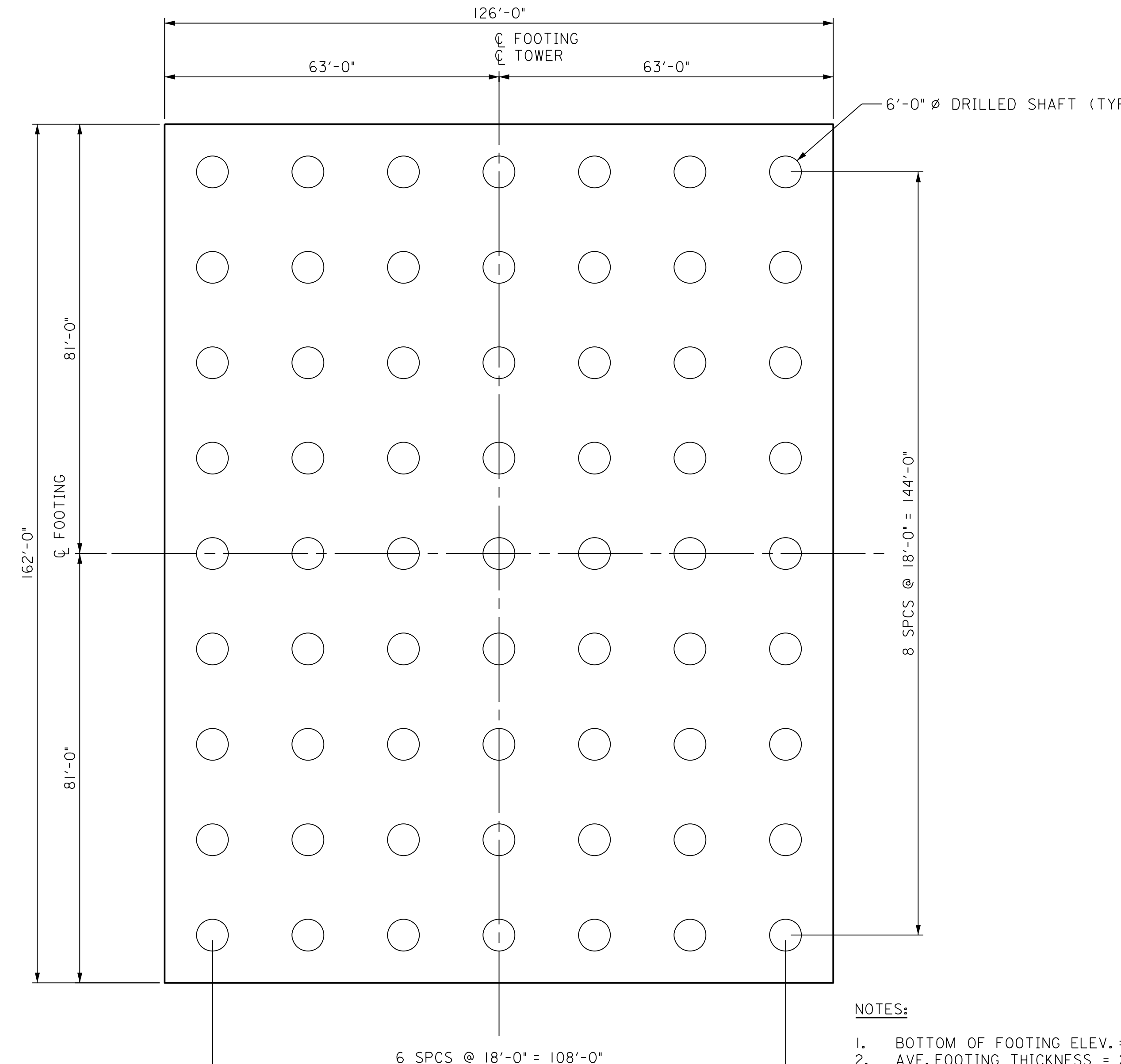
Appendix B – Main Span Conceptual Foundation Design Drawings



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'

6 X 8 LAYOUT
8'-0" ø DRILLED SHAFT
(48 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'

7 X 9 LAYOUT
6'-0" ø DRILLED SHAFT
(63 TOTAL)

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 5

REVISIONS

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(1 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

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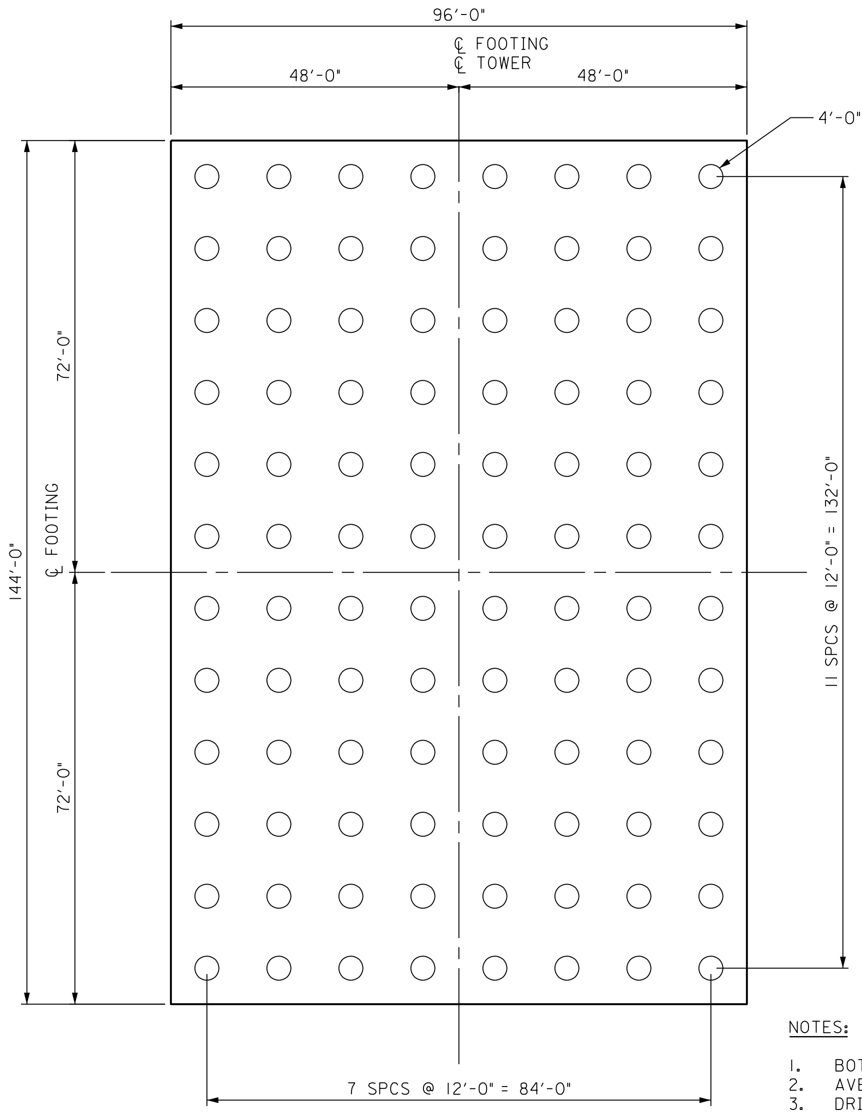
USER: scouture DATE: Wednesday, June 29, 2016 11:02:18 AM
FILE: ALDOT_Headquarters\Mobile_River_Bridge_Phase_1\05.0_Project_Development\05.2_Work_In_Progress\Project_Design\Plans_Assembly\Prelim Structure Type Selection\MRB_MAIN_Concept_Foundations_Edge
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:0.166666

MRB_MAIN_Concept_Foundations_Edge Girder.dgn

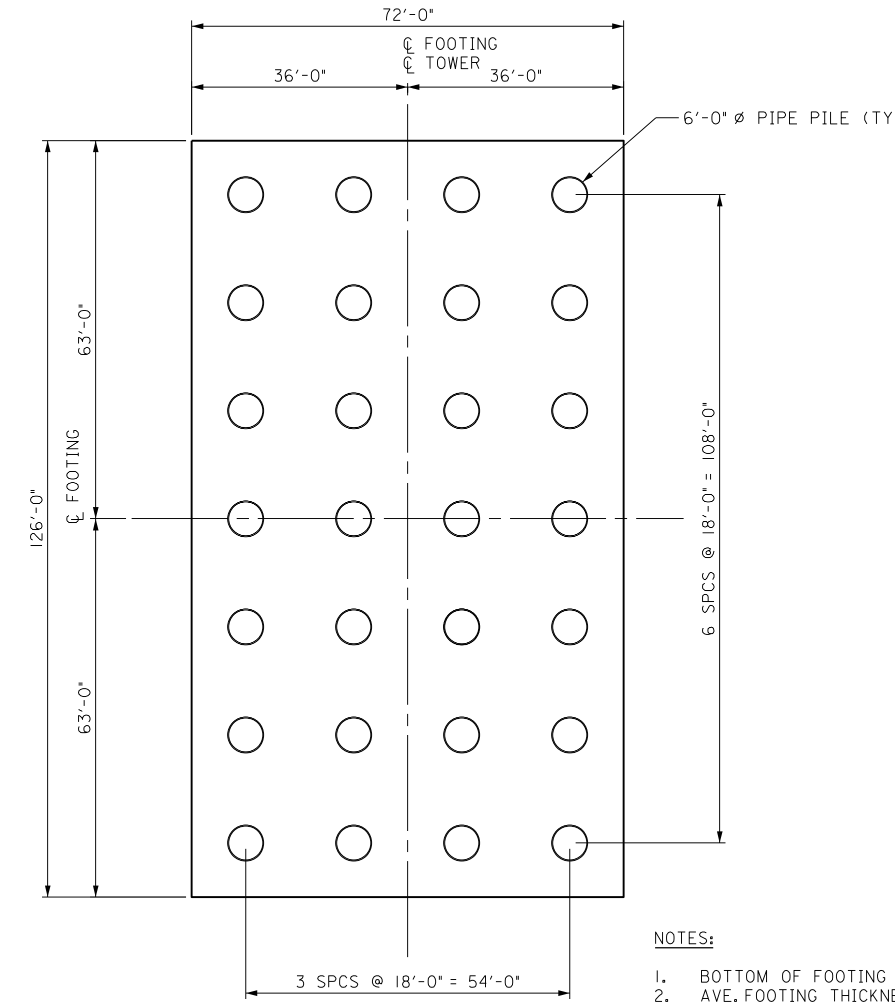
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'

8 X 12 LAYOUT
4'-0" ø DRILLED SHAFT
(96 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PIPE PILE TIP ELEV. = -270'

4 X 7 LAYOUT
6'-0" ø PIPE PILES
(28 TOTAL)

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 5

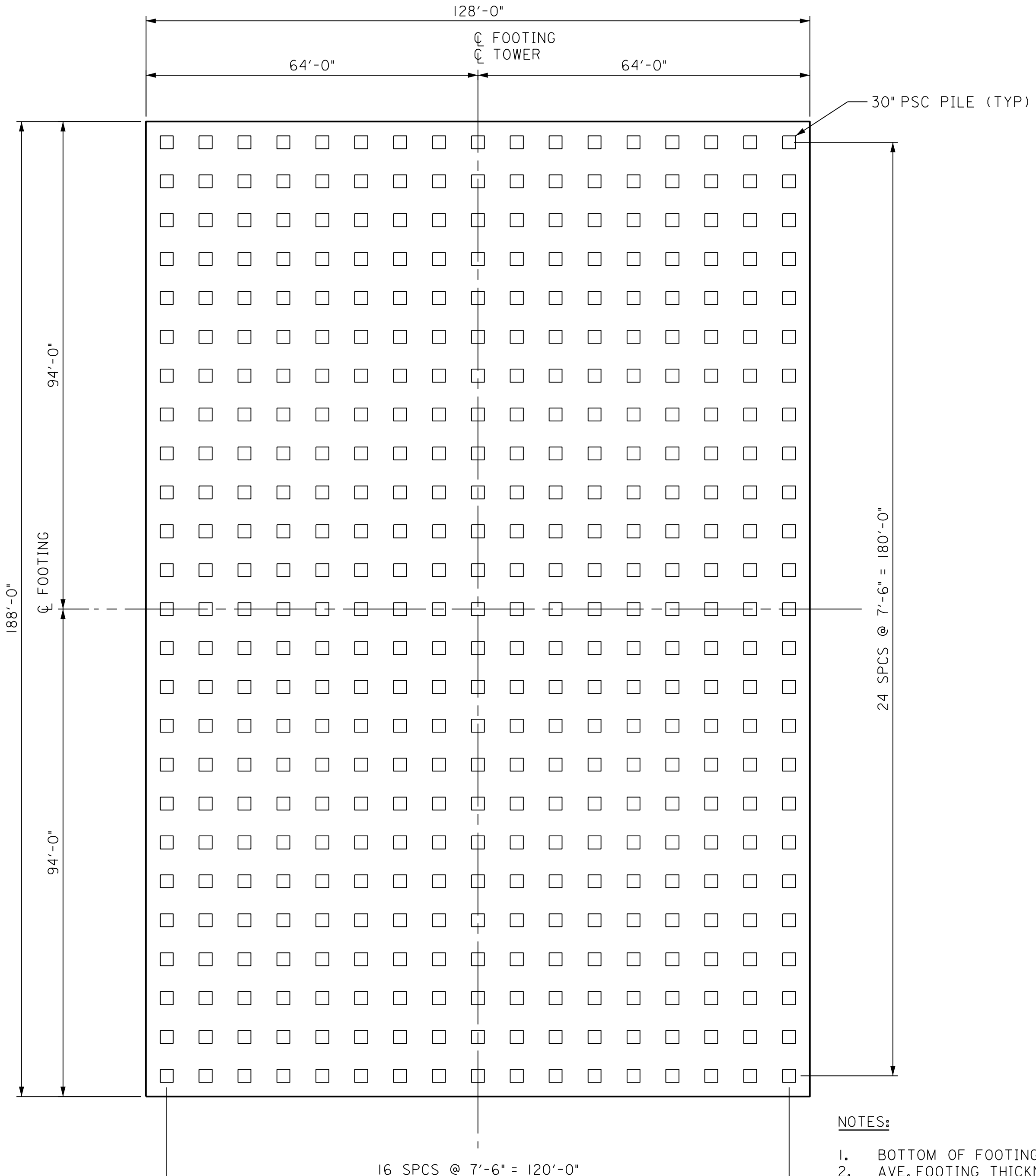
REVISIONS

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

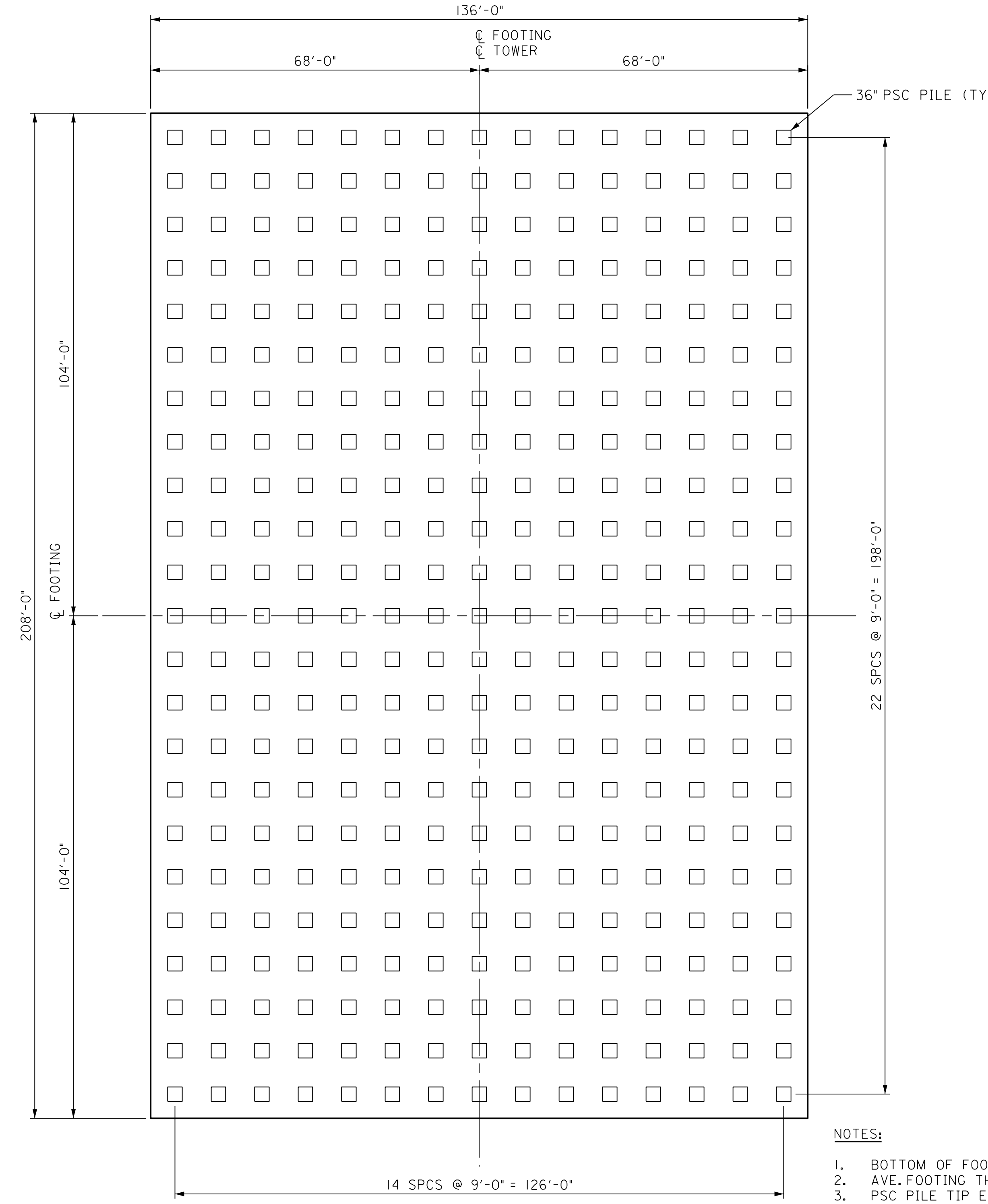


NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PSC PILE TIP ELEV. = -65'

17 X 25 LAYOUT

30" PSC PILES
(425 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PSC PILE TIP ELEV. = -65'

15 X 23 LAYOUT

36" PSC PILES
(345 TOTAL)

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 5

REVISIONS

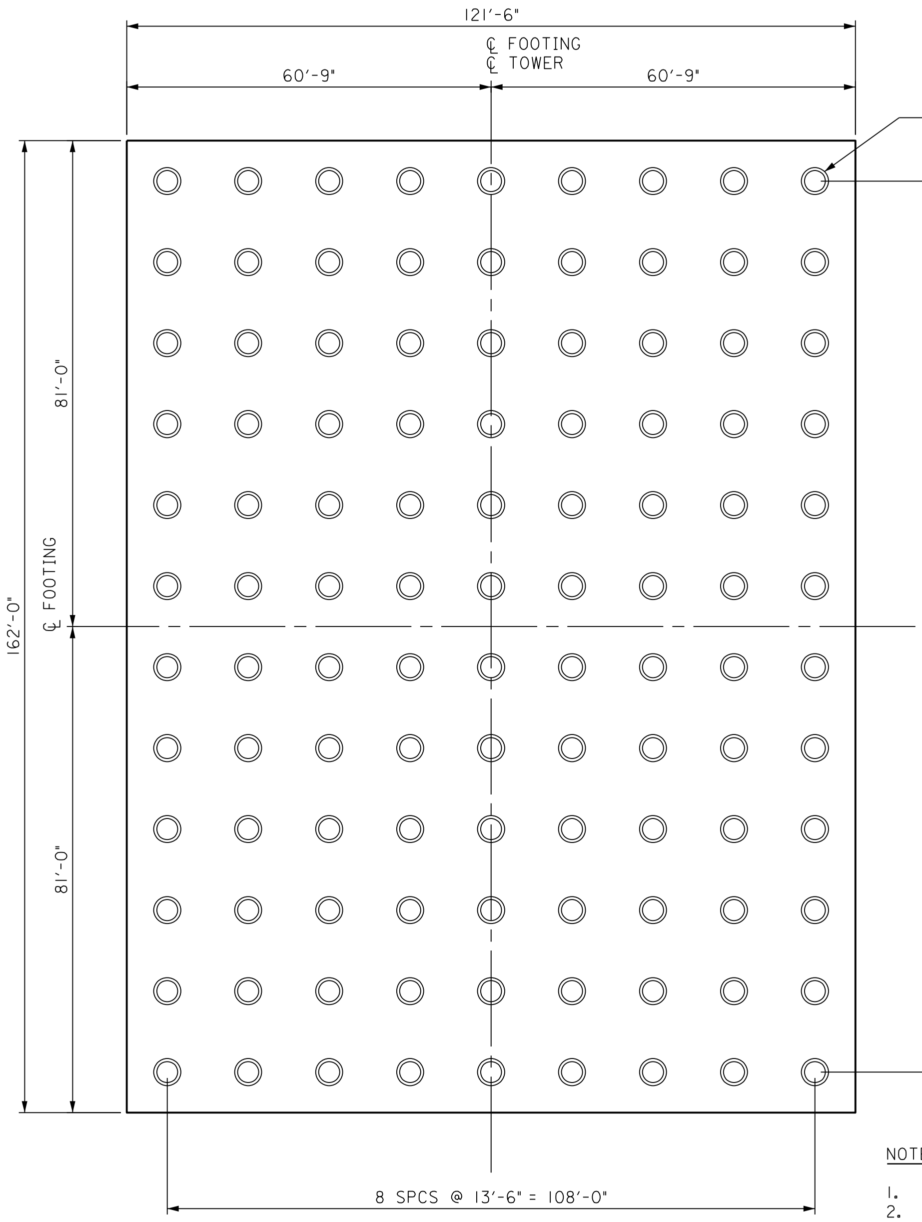
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

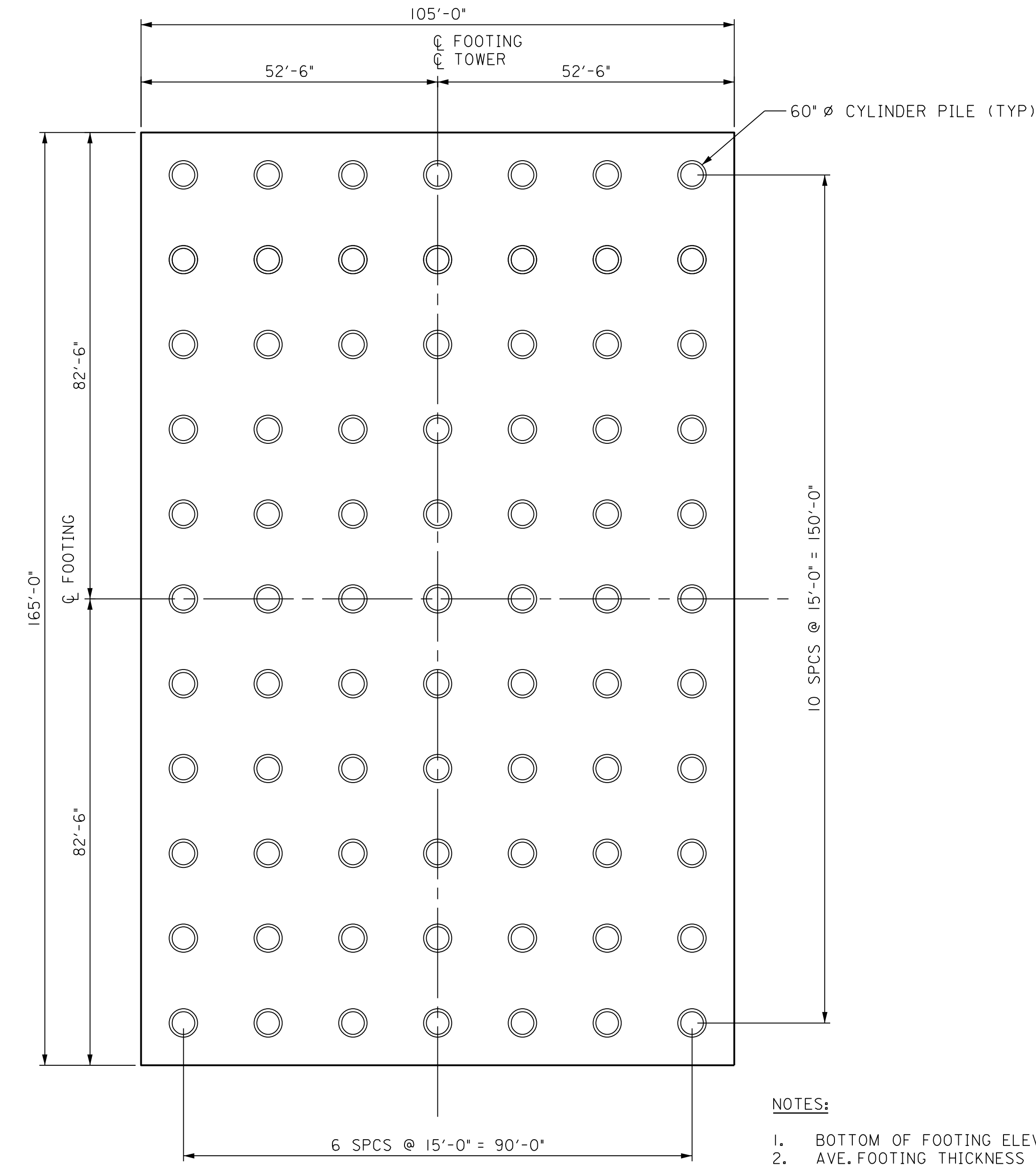


NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -110'

9 X 12 LAYOUT

54" ø CYLINDER PILES
(108 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -120'

7 X 11 LAYOUT

60" ø CYLINDER PILES
(77 TOTAL)

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 5

REVISIONS

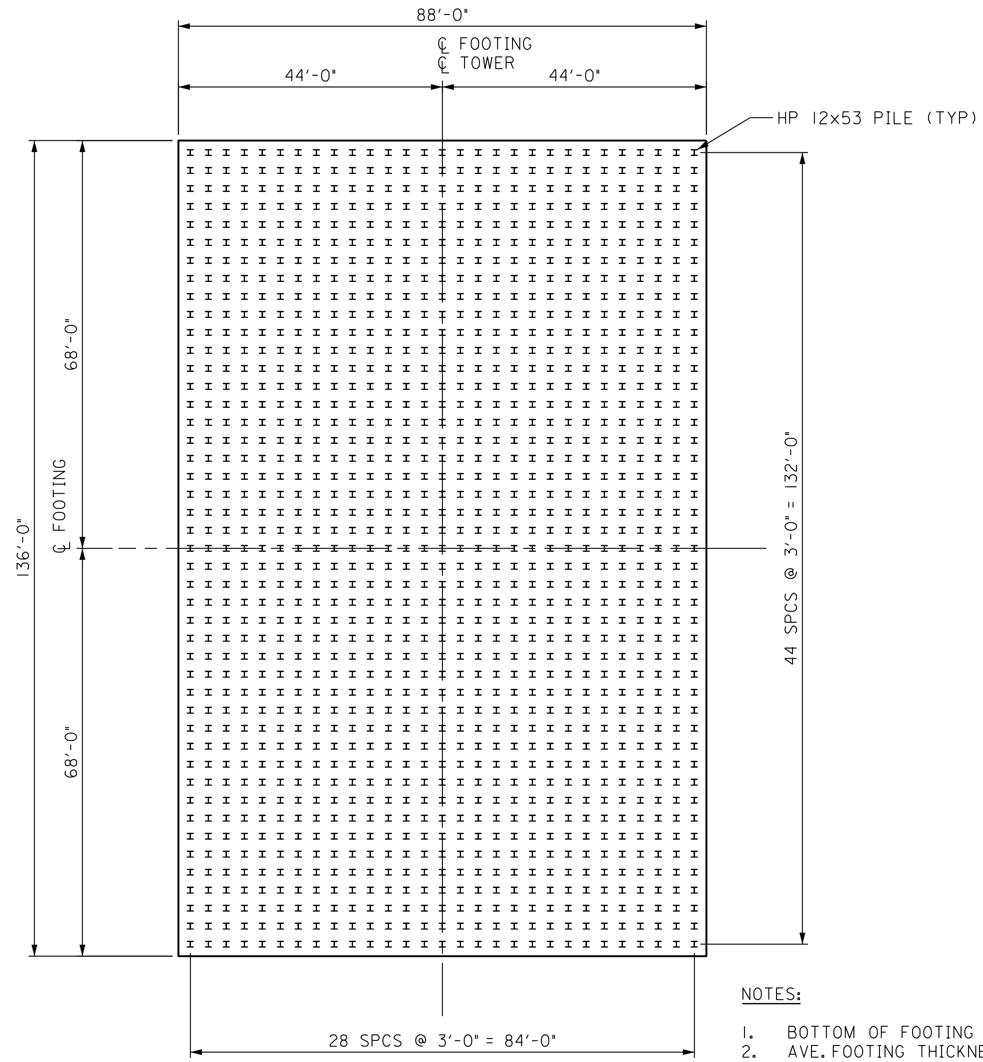
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

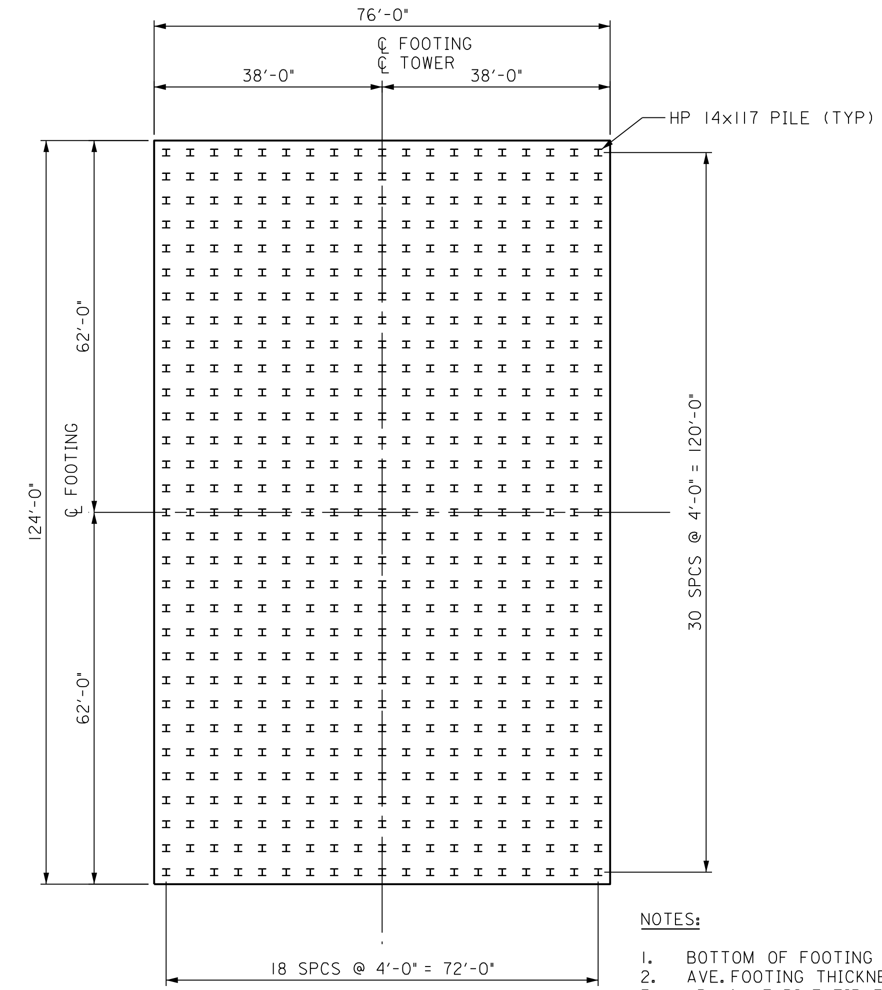


NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 12x53 PILE TIP ELEV. = -90'

29 X 45 LAYOUT

HP 12x53 PILES
(1305 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 14x117 PILE TIP ELEV. = -120'

19 X 31 LAYOUT

HP 14x117 PILES
(589 TOTAL)

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 5

REVISIONS

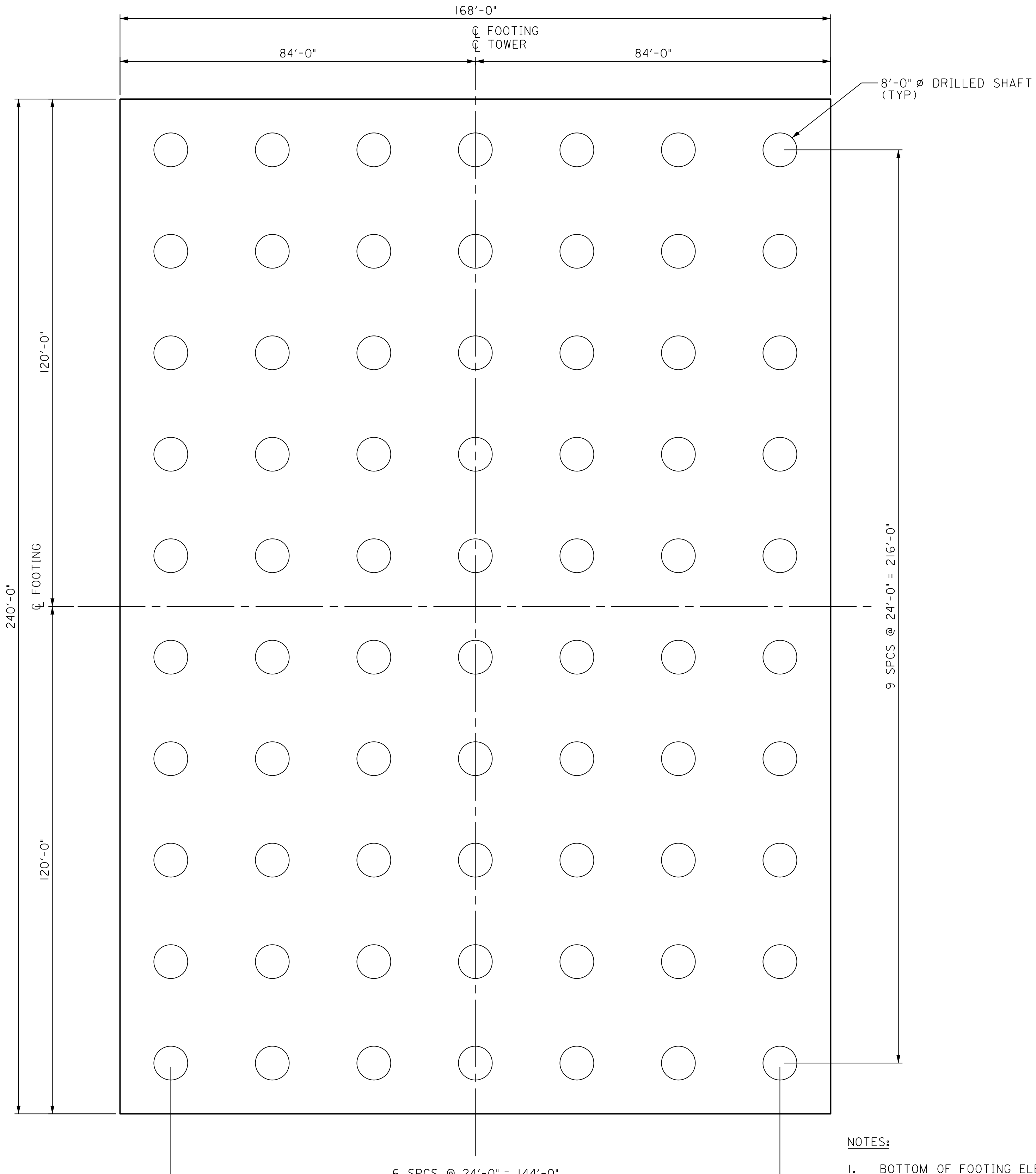
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(5 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

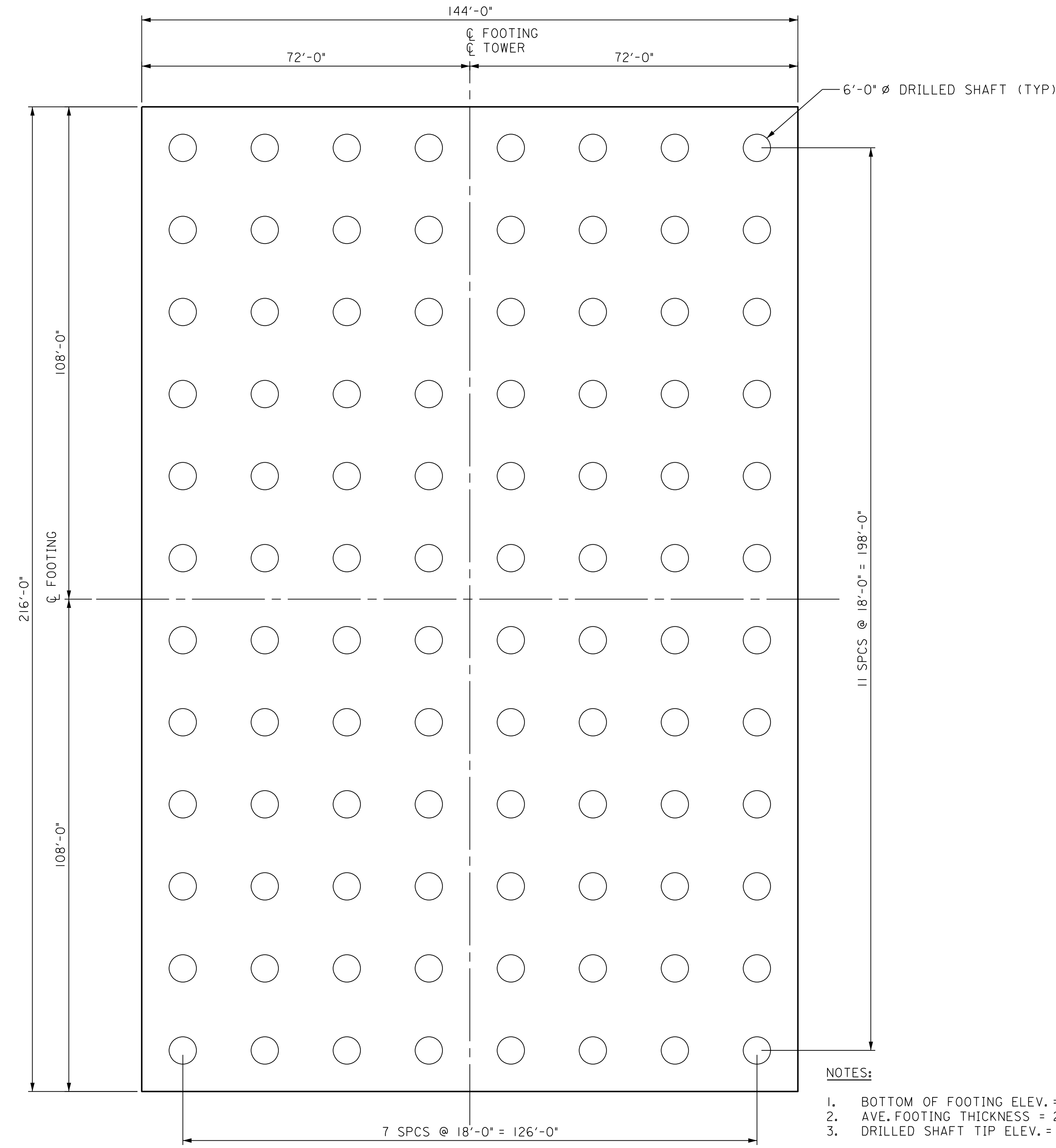
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



7 X 10 LAYOUT
8'-0" ø DRILLED SHAFT
(70 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'



8 X 12 LAYOUT
6'-0" ø DRILLED SHAFT
(96 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'

MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 5

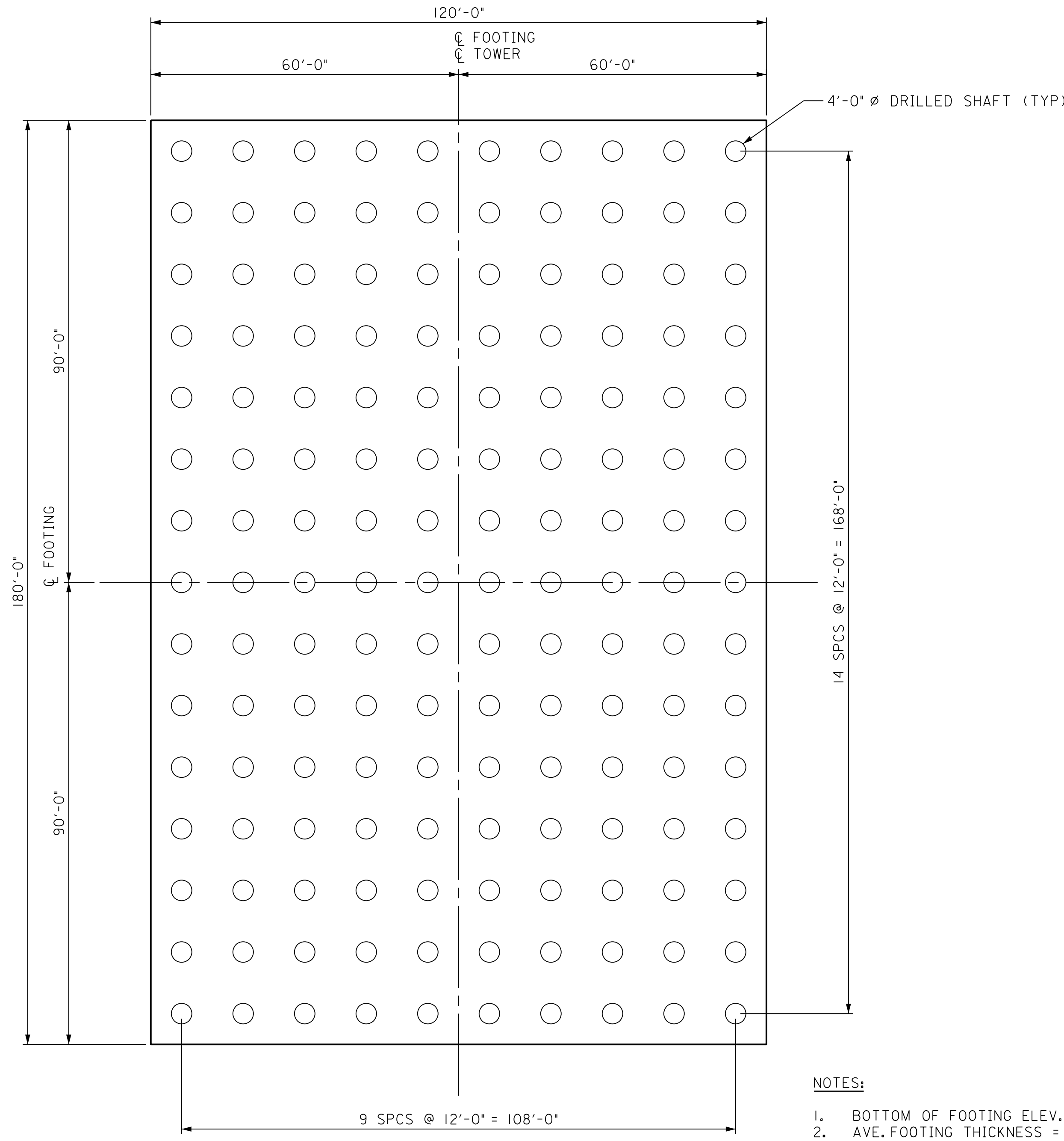
REVISIONS

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(1 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

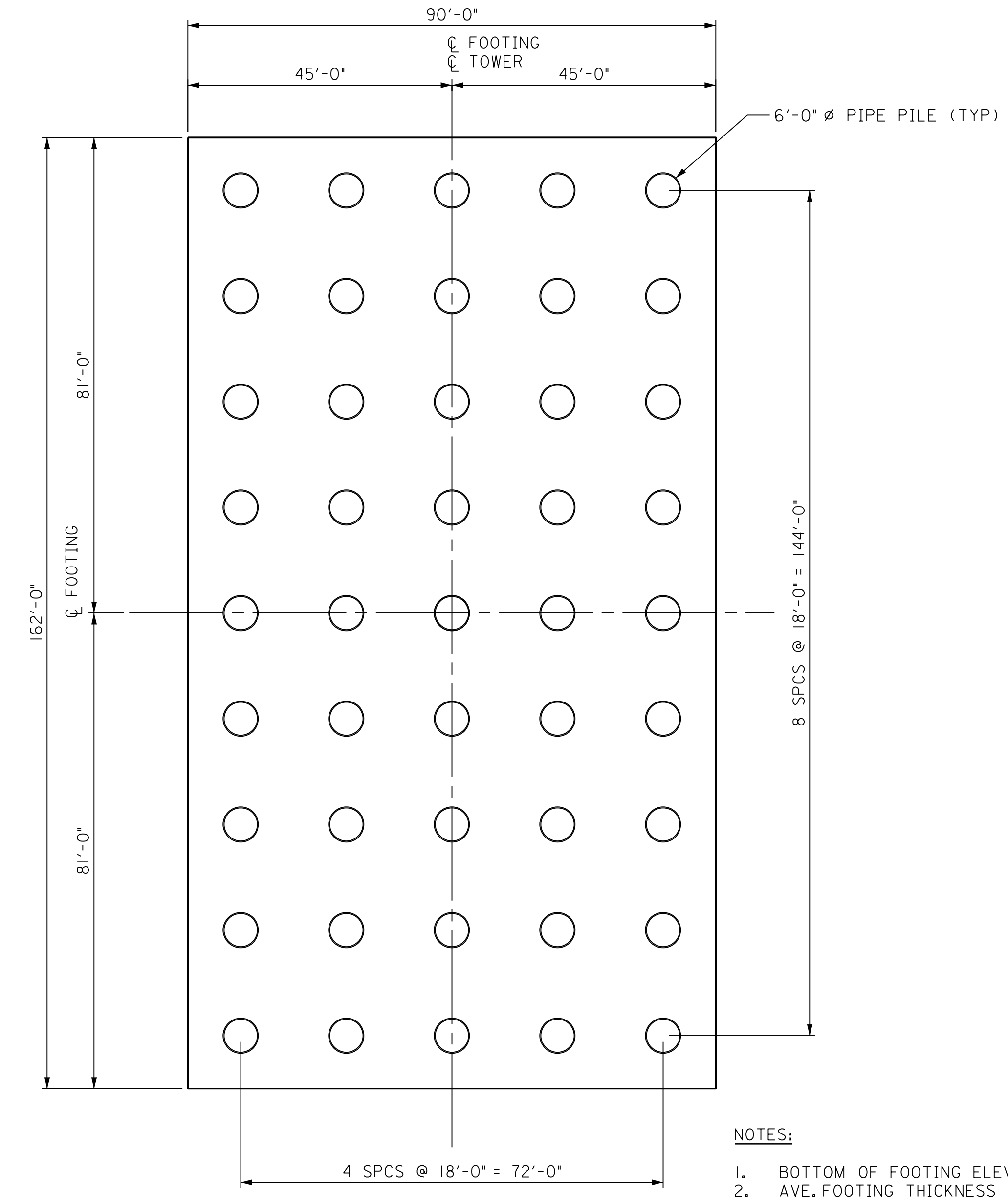
BIN:



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. DRILLED SHAFT TIP ELEV. = -160'

10 X 15 LAYOUT
4'-0" ø DRILLED SHAFT
(150 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PIPE PILE TIP ELEV. = -270'

5 X 9 LAYOUT
6'-0" ø PIPE PILES
(45 TOTAL)

MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 5

REVISIONS

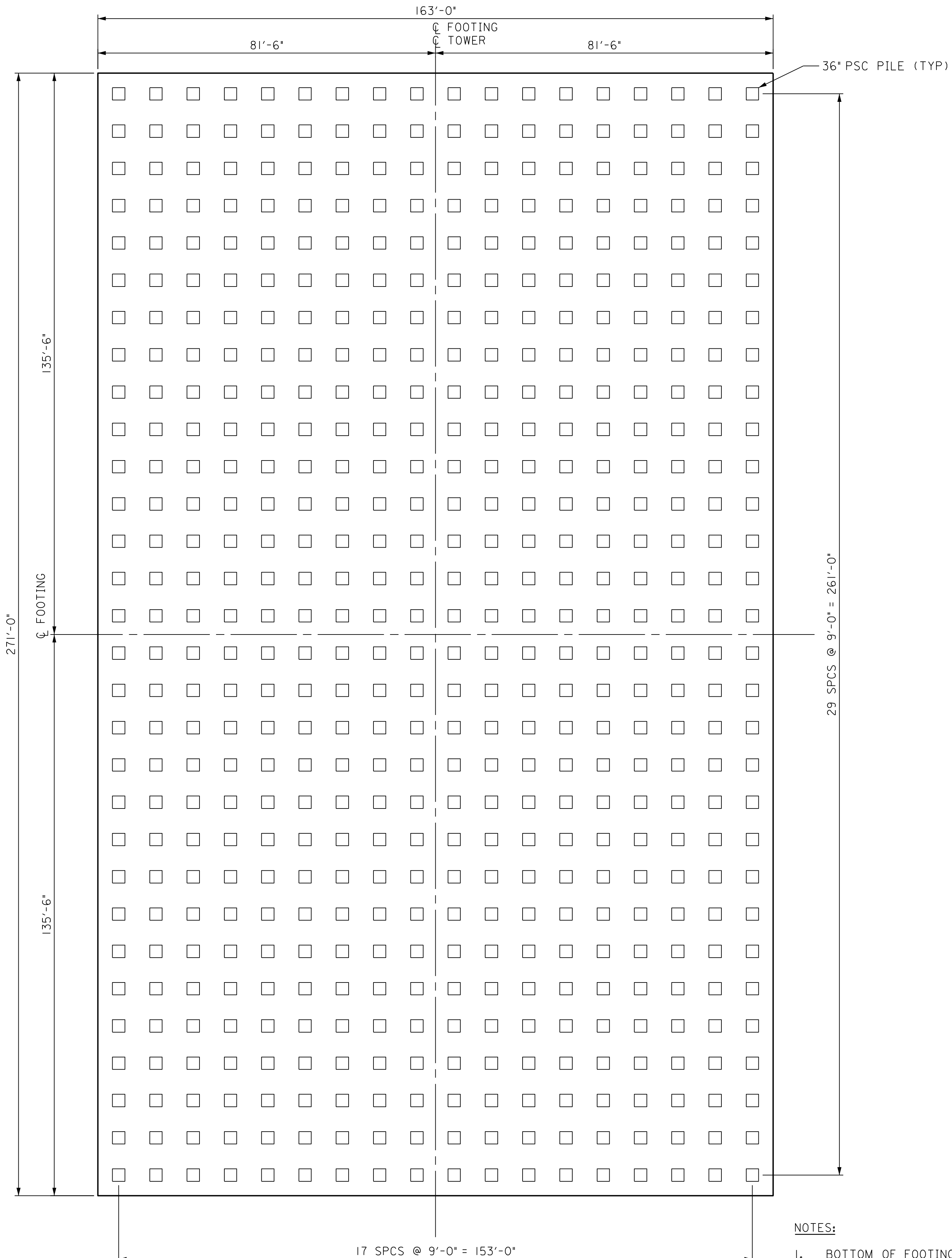
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

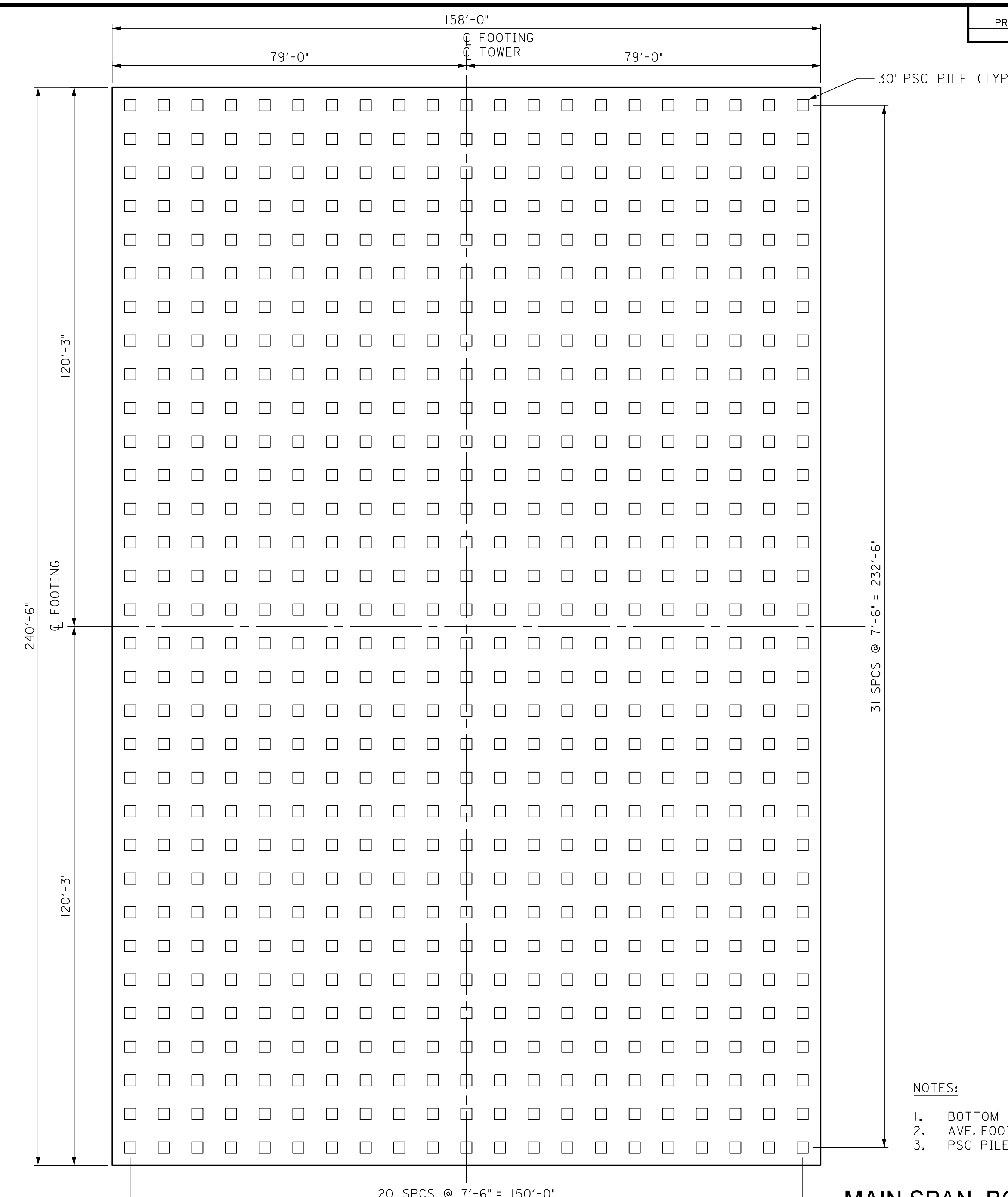


18 X 30 LAYOUT

36" PSC PILES
(540 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PSC PILE TIP ELEV. = -65'



21 X 32 LAYOUT

30" PSC PILES
(672 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. PSC PILE TIP ELEV. = -65'

MAIN SPAN, BOX SECTION SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 5

REVISIONS

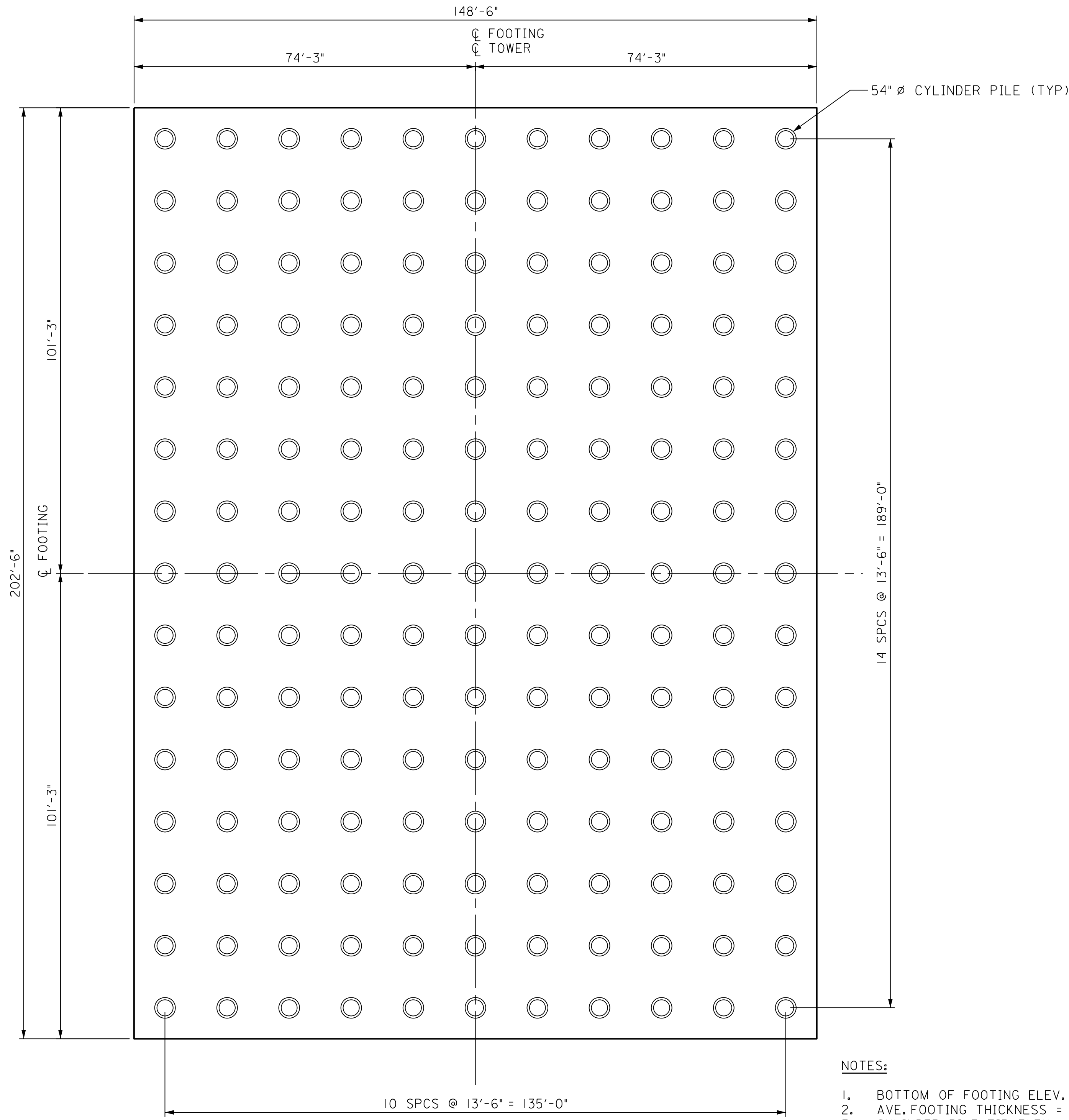
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

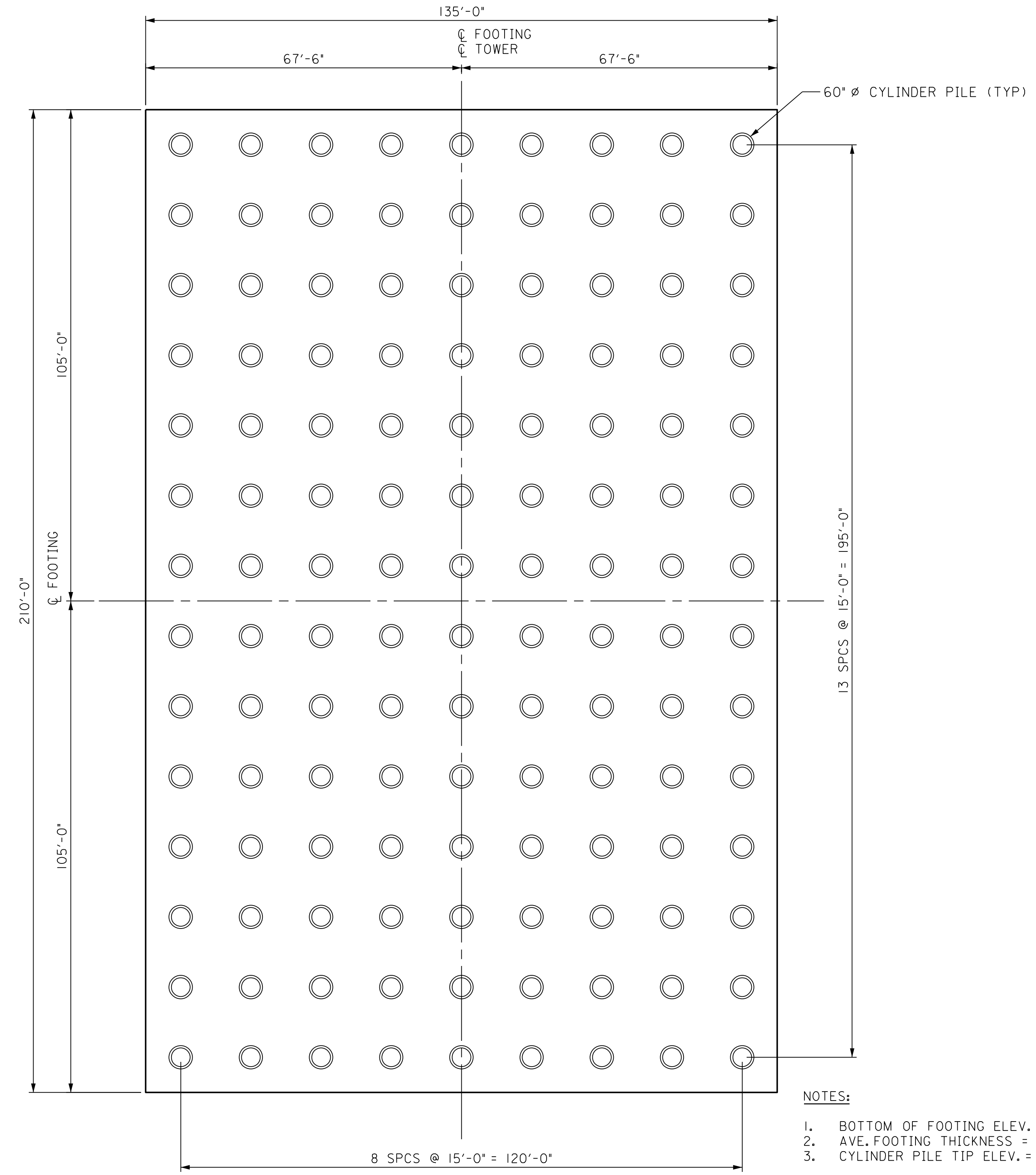
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -110'

11 X 15 LAYOUT
54" ø CYLINDER PILES
(165 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -120'

9 X 14 LAYOUT
60" ø CYLINDER PILES
(126 TOTAL)

MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 5

REVISIONS

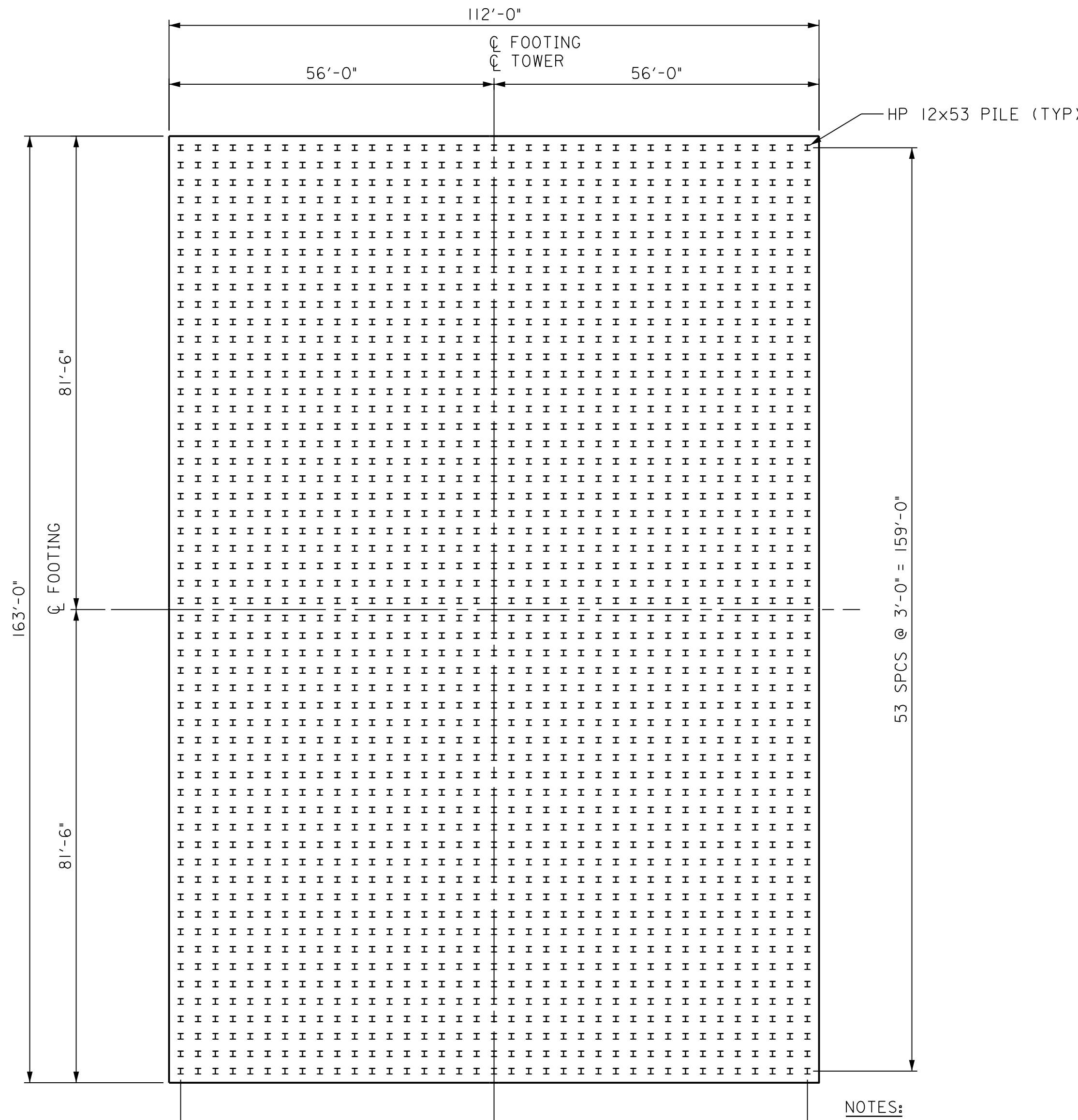
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

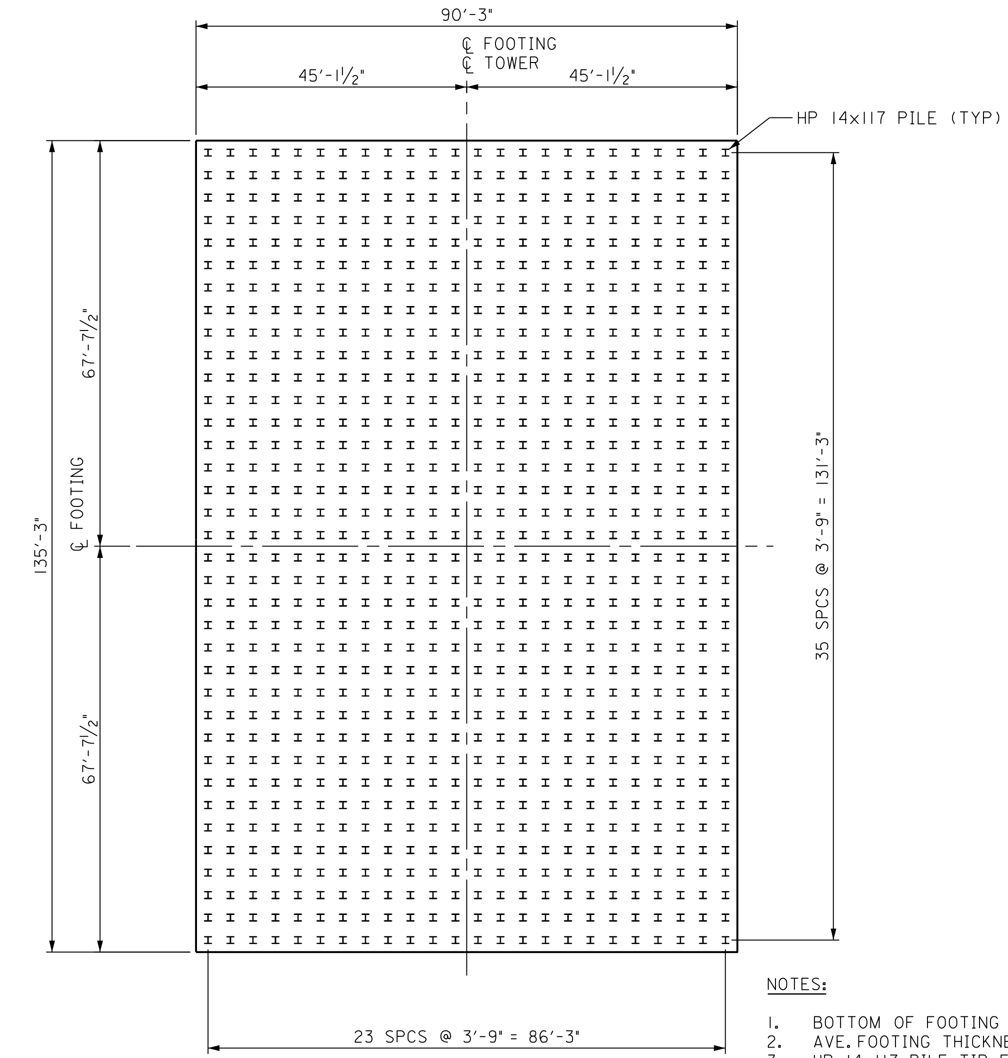


NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 12x53 PILE TIP ELEV. = -90'

37 X 54 LAYOUT

HP 12x53 PILES
(1998 TOTAL)



NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 14x117 PILE TIP ELEV. = -120'

24 X 36 LAYOUT

HP 14x117 PILES
(864 TOTAL)

MAIN SPAN, BOX SECTION SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 5

REVISIONS

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(5 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

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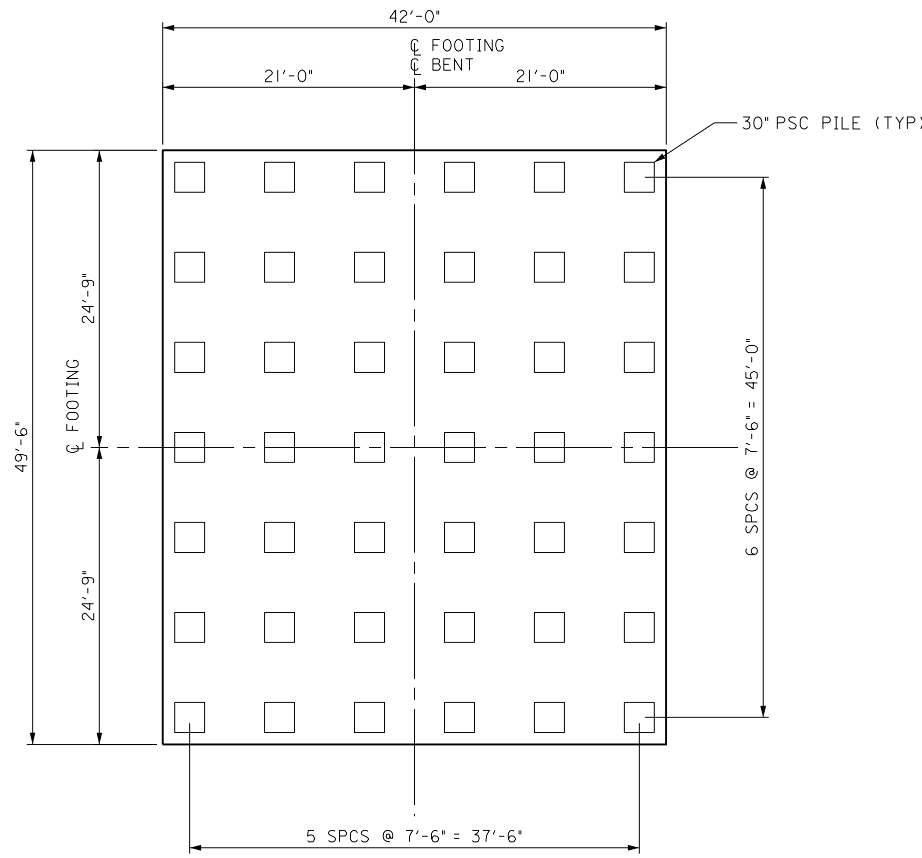
MRB_MAIN_Concept_Foundation_Box.dgn

Appendix C - High Level Approach Spans Conceptual Foundation Design Drawings

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

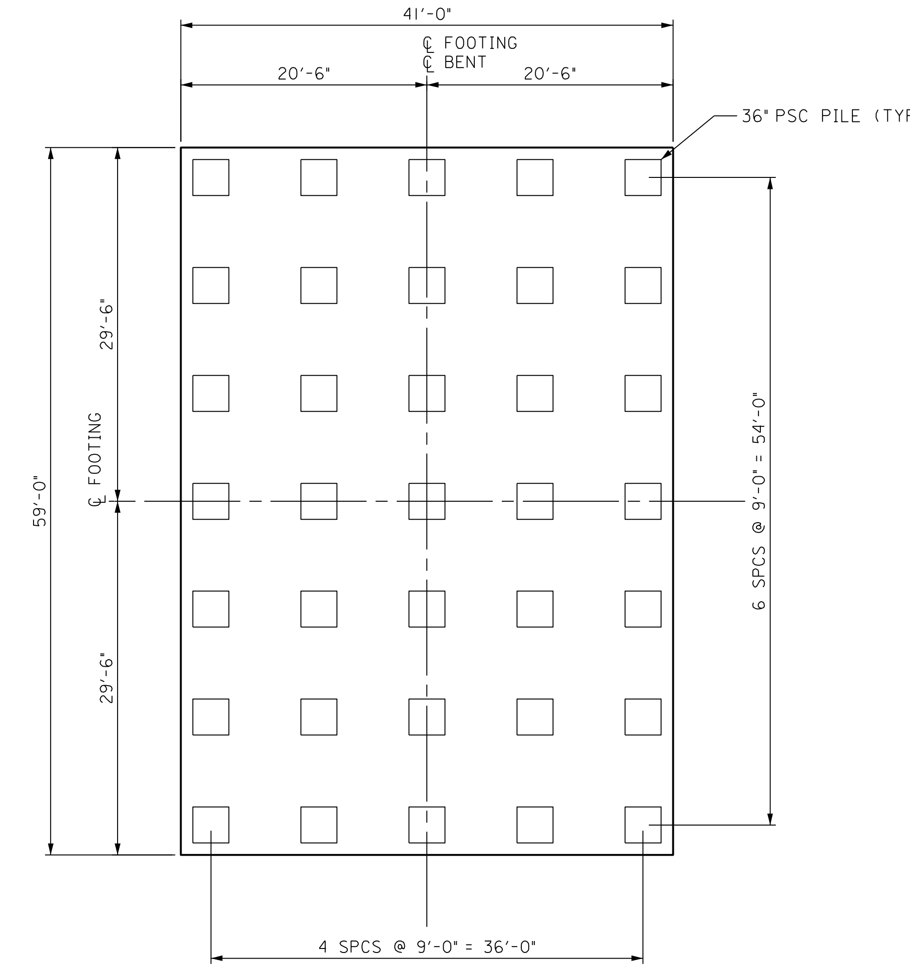


6 X 7 LAYOUT

30" PSC PILES
(42 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. PSC PILE TIP ELEV. = -65'

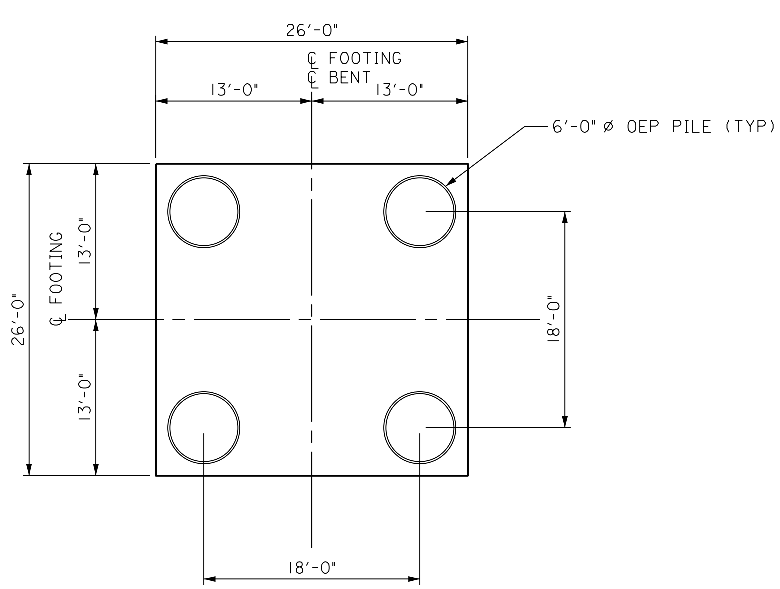


5 X 7 LAYOUT

36" PSC PILES
(35 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 12'
3. PSC PILE TIP ELEV. = -65'



2 X 2 LAYOUT

6'-0" ⌀ OEP PILES
(4 TOTAL)

NOTES:

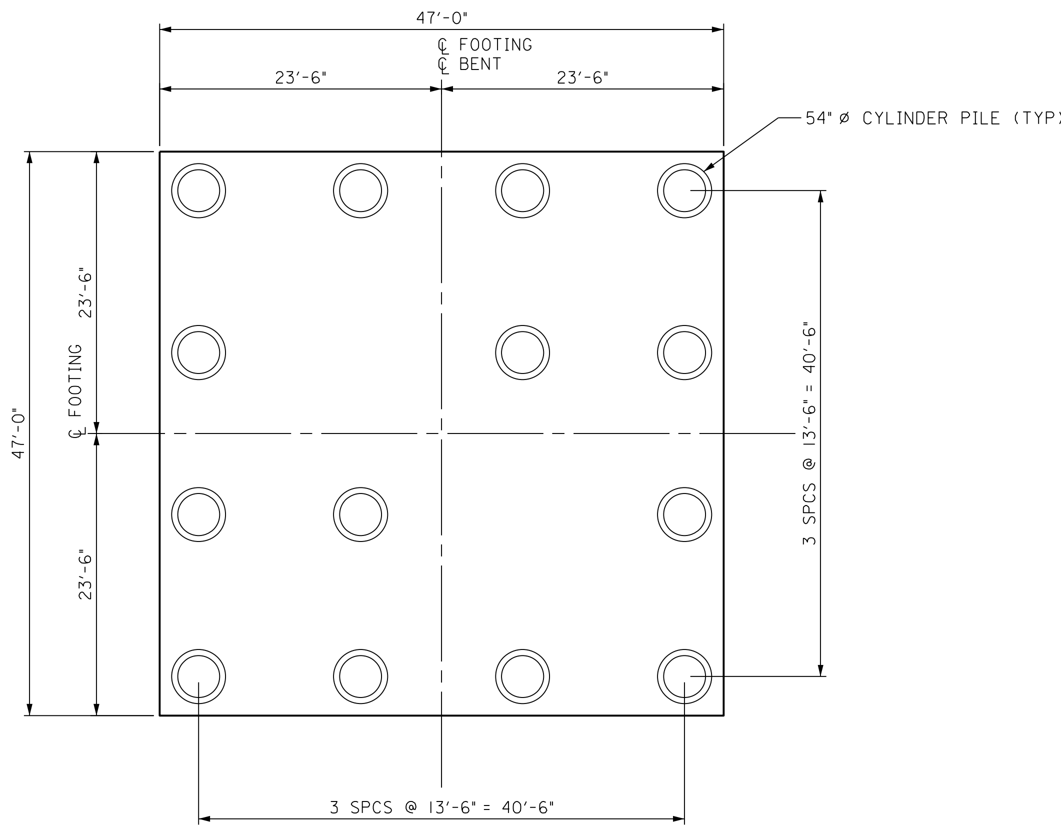
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. PIPE PILE TIP ELEV. = -270'

OPTION 7: SIMPLE SPAN LG-78 BEAMS

COLUMN HEIGHT = 150'

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 3	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS		
REVISIONS	HIGH LEVEL APPR FOUNDATION OPTIONS (1 OF 3)		
	ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
	COMPUTED BY:	CHECKED BY:	DATE DRAWN:
BIN:	VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

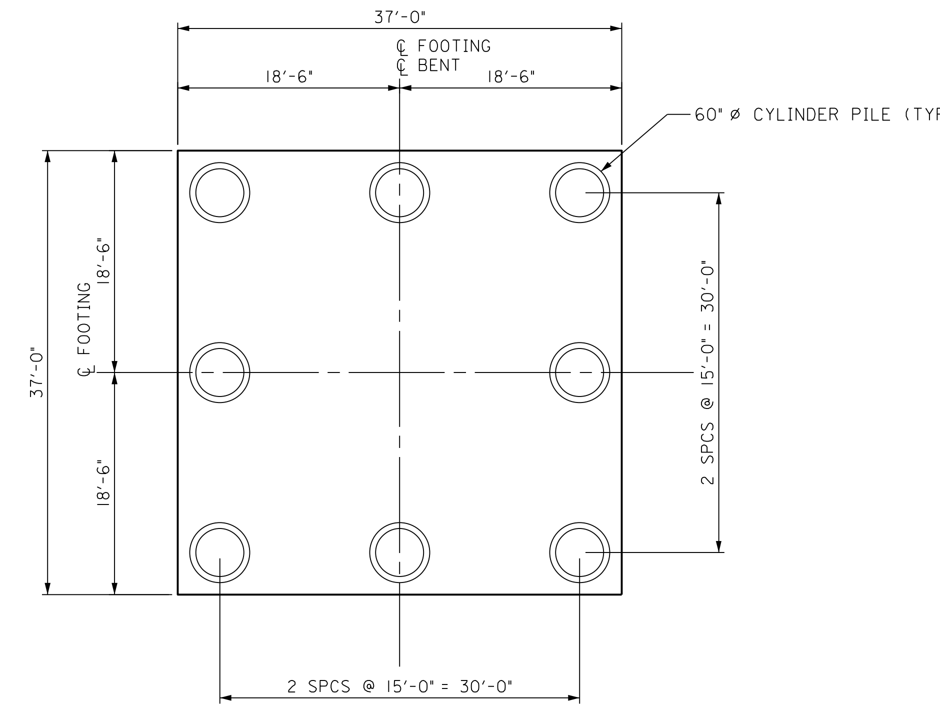


4 X 4 LAYOUT

54" ø CYLINDER PILES
(14 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 15'
3. CYLINDER PILE TIP ELEV. = -110'



3 X 3 LAYOUT

60" ø CYLINDER PILES
(8 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 11'
3. CYLINDER PILE TIP ELEV. = -120'

OPTION 7: SIMPLE SPAN LG-78 BEAMS

COLUMN HEIGHT = 150'

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 3

REVISIONS

BIN:

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(2 OF 3)

ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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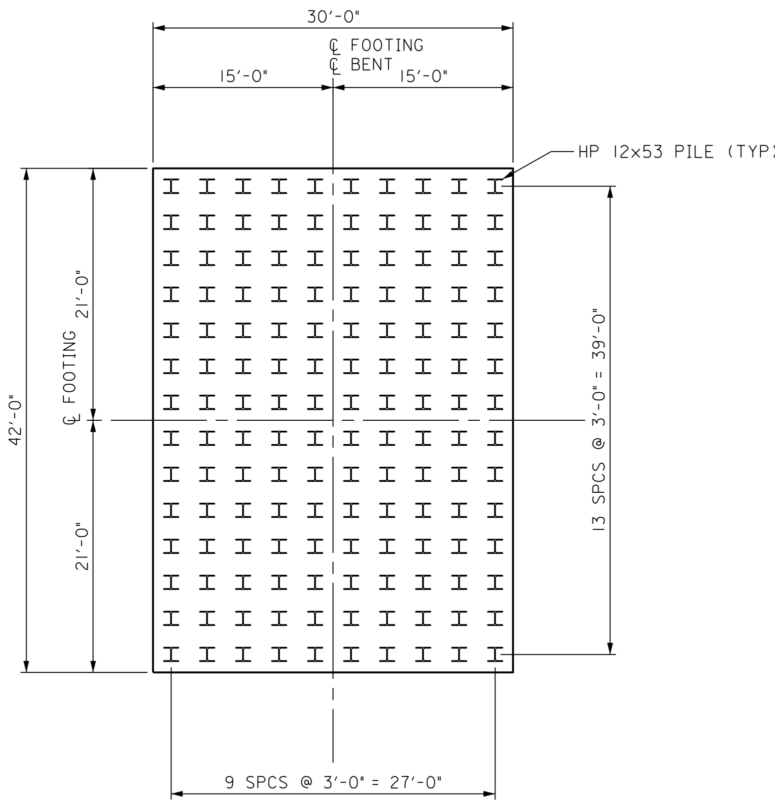
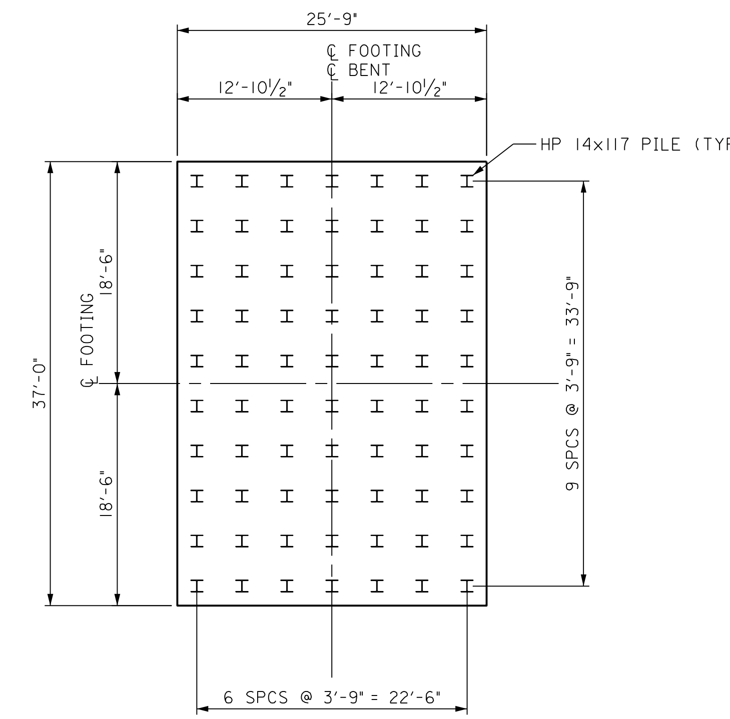
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



10 X 14 LAYOUT
HP 12x53 PILES
(140 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. HP 12x53 PILE TIP ELEV. = -90'



7 X 10 LAYOUT
HP 14x117 PILES
(70 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. HP 14x117 PILE TIP ELEV. = -120'

OPTION 7: SIMPLE SPAN LG-78 BEAMS

COLUMN HEIGHT = 150'

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 3

REVISIONS

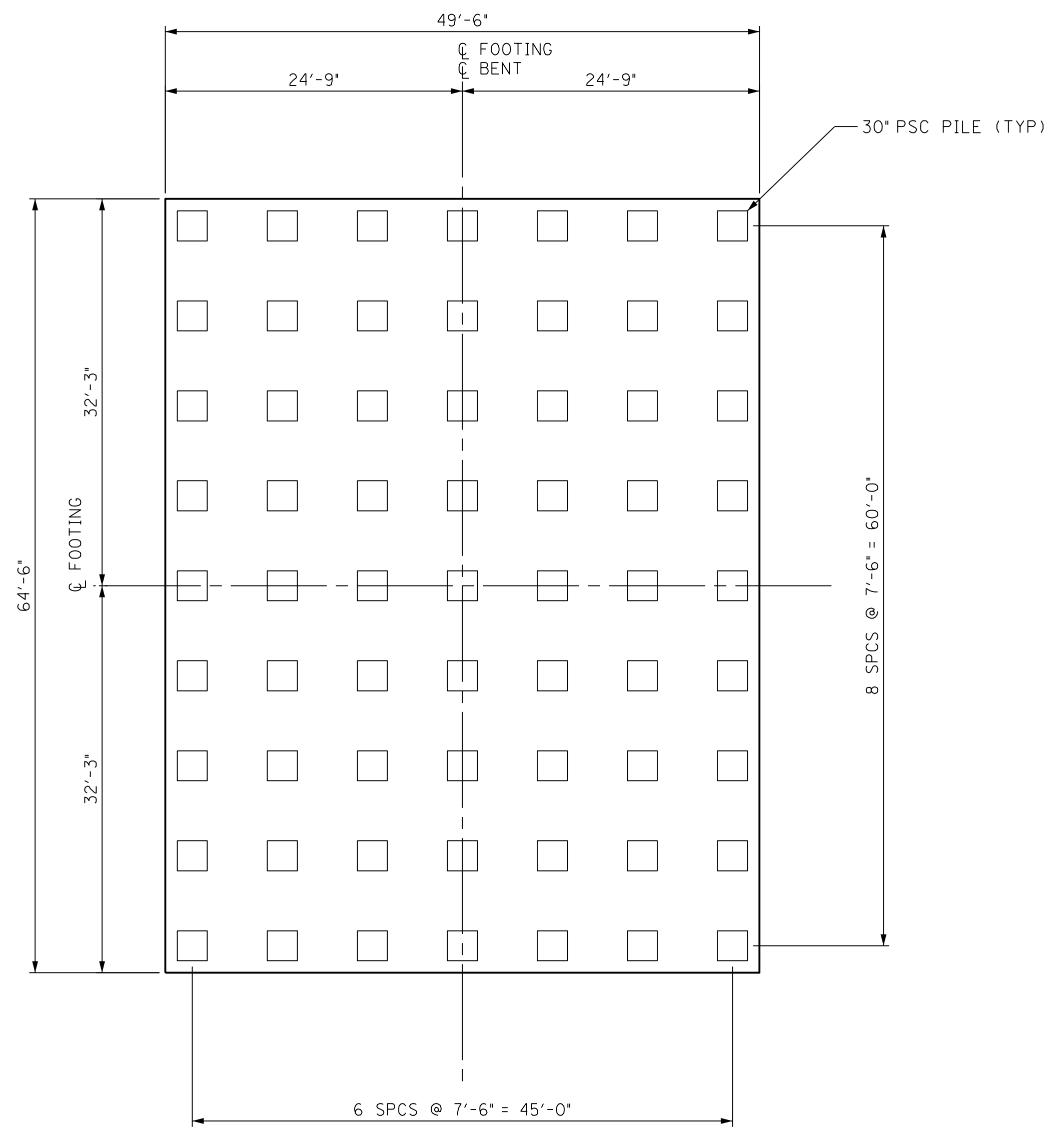
MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(3 OF 3)

ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

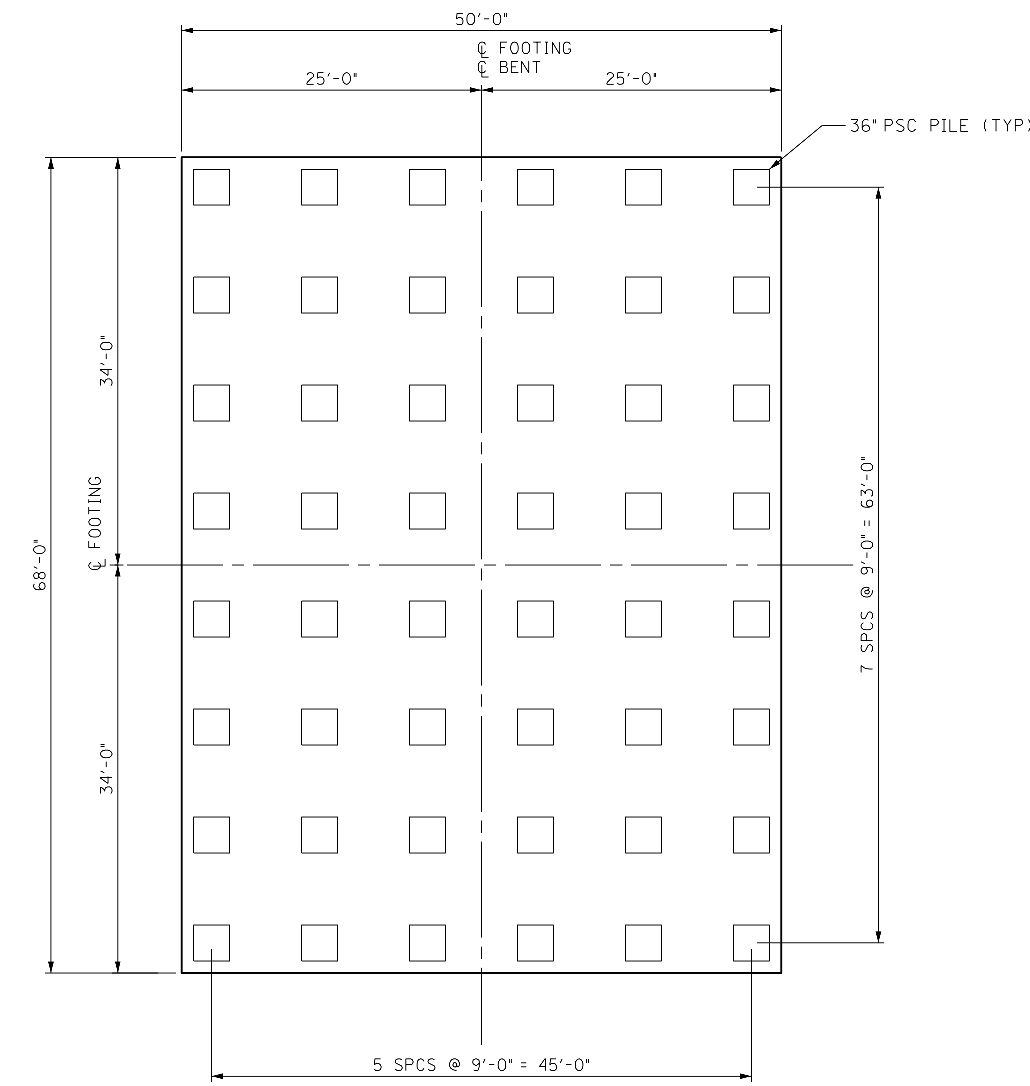
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



7 X 9 LAYOUT
30" PSC PILES
(63 TOTAL)

NOTES:

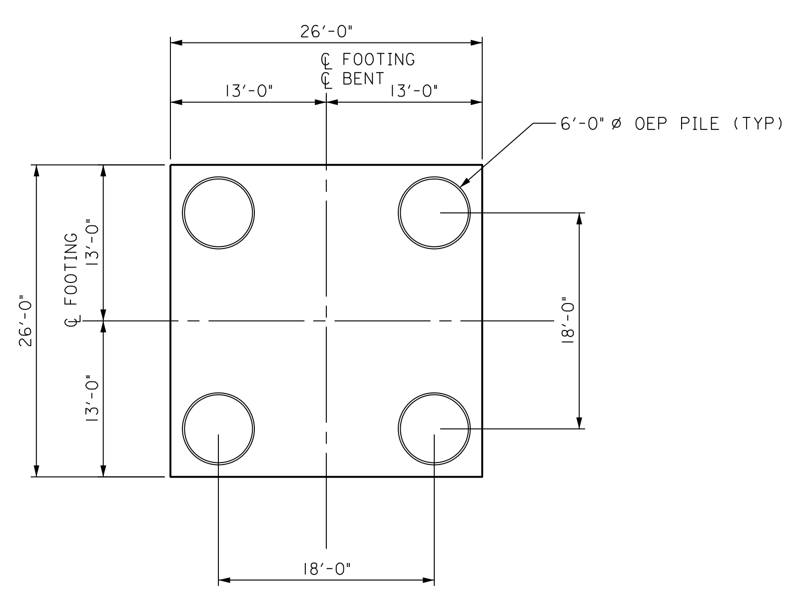
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 12'
3. PSC PILE TIP ELEV. = -65'



6 X 8 LAYOUT
36" PSC PILES
(48 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 14'
3. PSC PILE TIP ELEV. = -65'



3 X 3 LAYOUT
6'-0" ø OEP PILES
(4 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 14'
3. PIPE PILE TIP ELEV. = -270'

OPTION 7: SIMPLE SPAN LG-78 BEAMS

COLUMN HEIGHT = 200'

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 3

REVISIONS

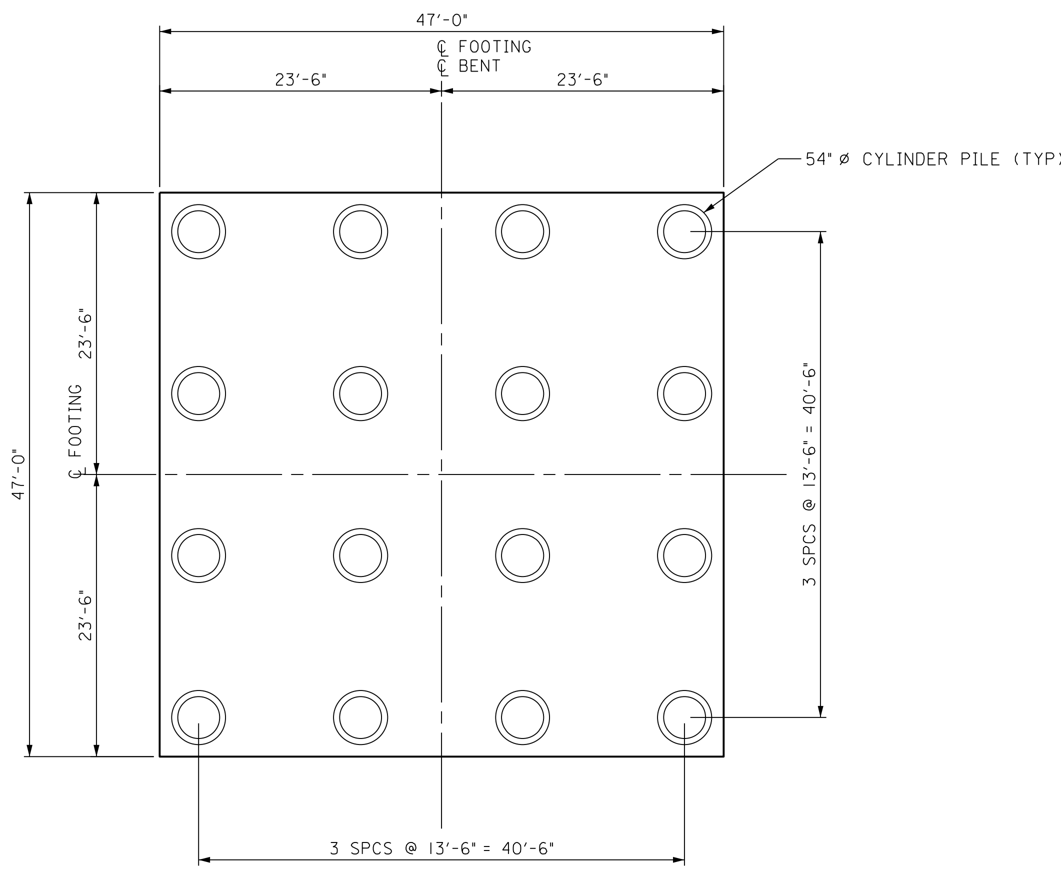
MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(1 OF 3)

BIN:

ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

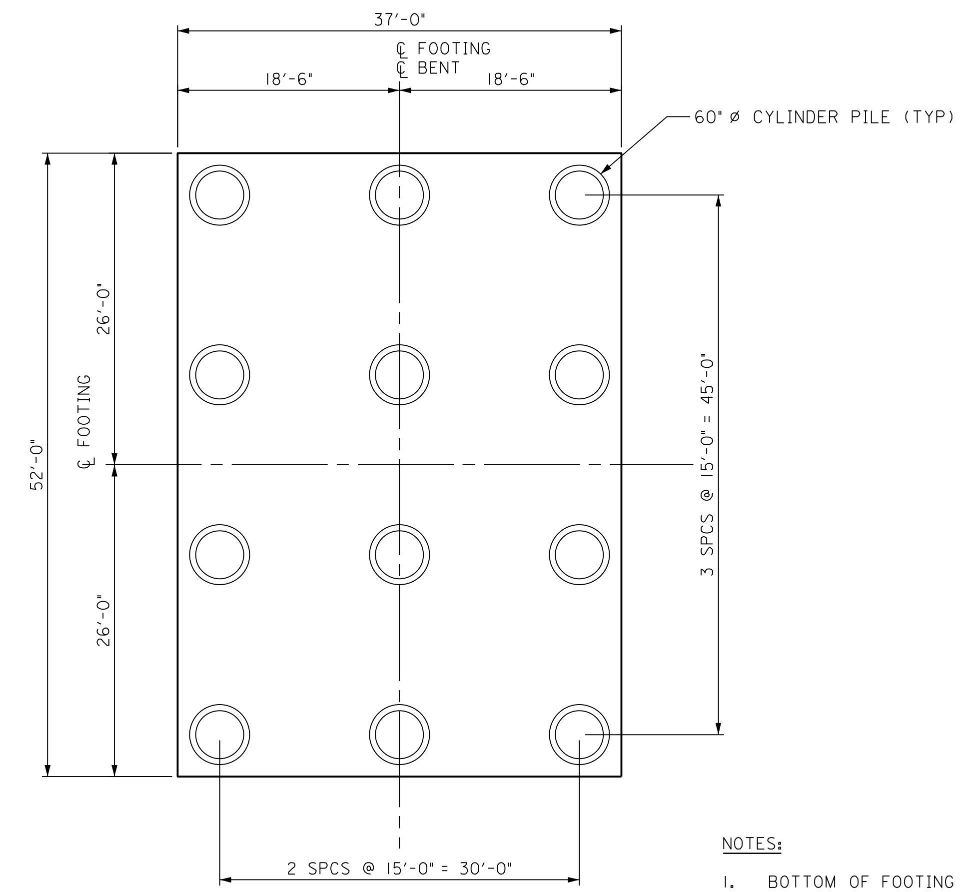


4 X 4 LAYOUT

54" ø CYLINDER PILES
(16 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 13'
3. CYLINDER PILE TIP ELEV. = -110'



3 X 4 LAYOUT

60" ø CYLINDER PILES
(12 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 16'
3. CYLINDER PILE TIP ELEV. = -120'

OPTION 7: SIMPLE SPAN LG-78 BEAMS

COLUMN HEIGHT = 200'

PRESTRESSED BULB TEE LG-78
200' HEIGHT ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 3

REVISIONS

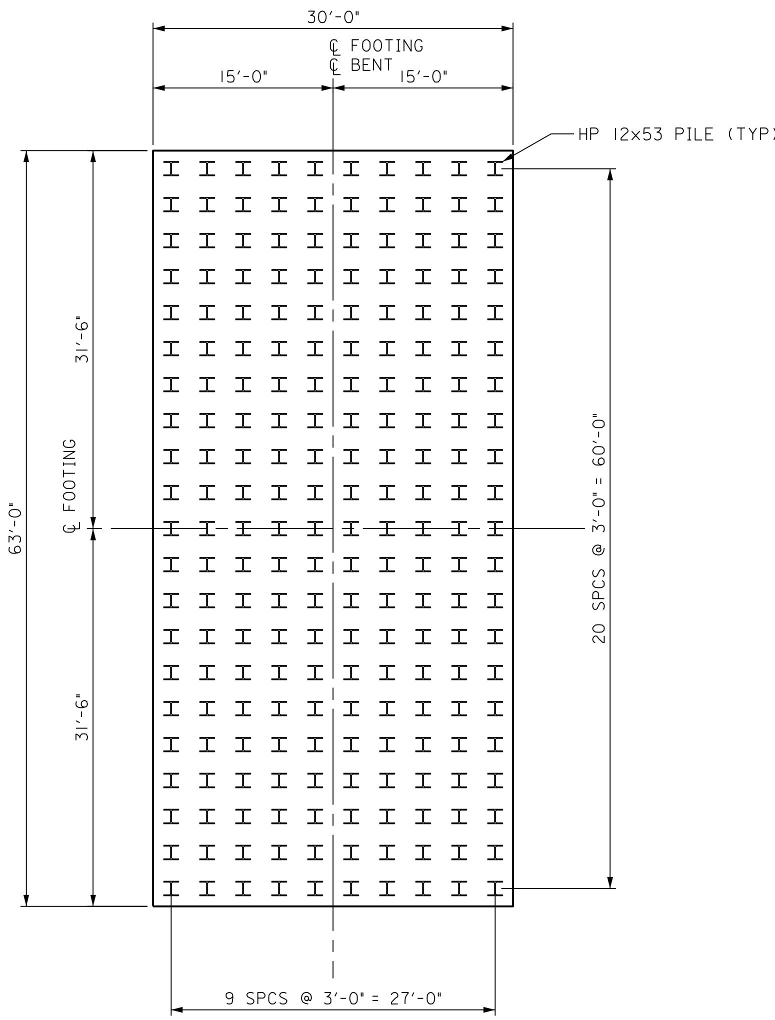
MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(2 OF 3)

ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

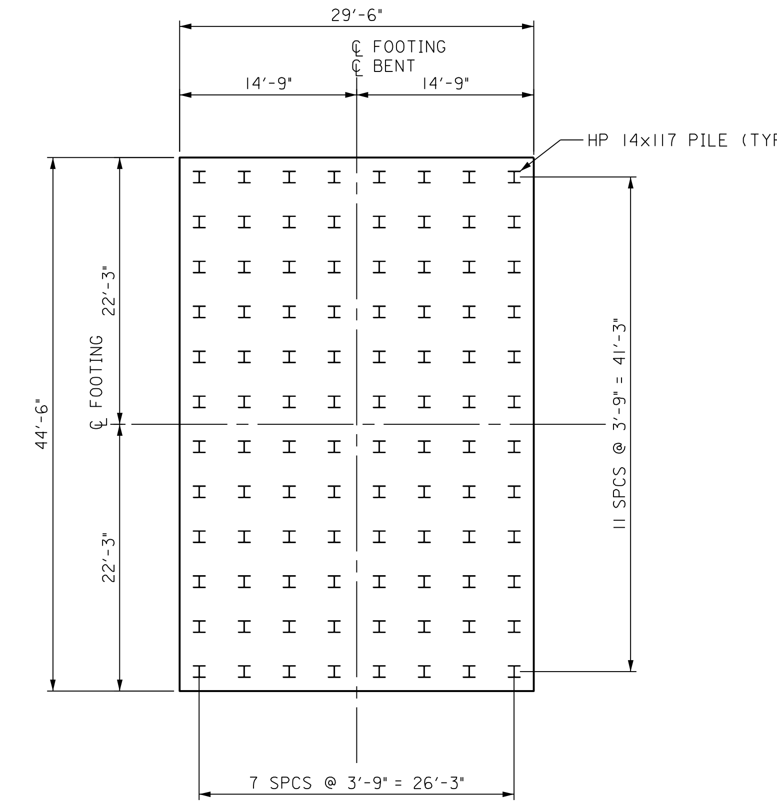


10 X 21 LAYOUT

HP 12x53 PILES
(210 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 17'
3. HP 12x53 PILE TIP ELEV. = -90'



8 X 12 LAYOUT

HP 14x117 PILES
(96 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 12'
3. HP 14x117 PILE TIP ELEV. = -120'

OPTION 7: SIMPLE SPAN LG-78 BEAMS
COLUMN HEIGHT = 200'

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 3

REVISIONS

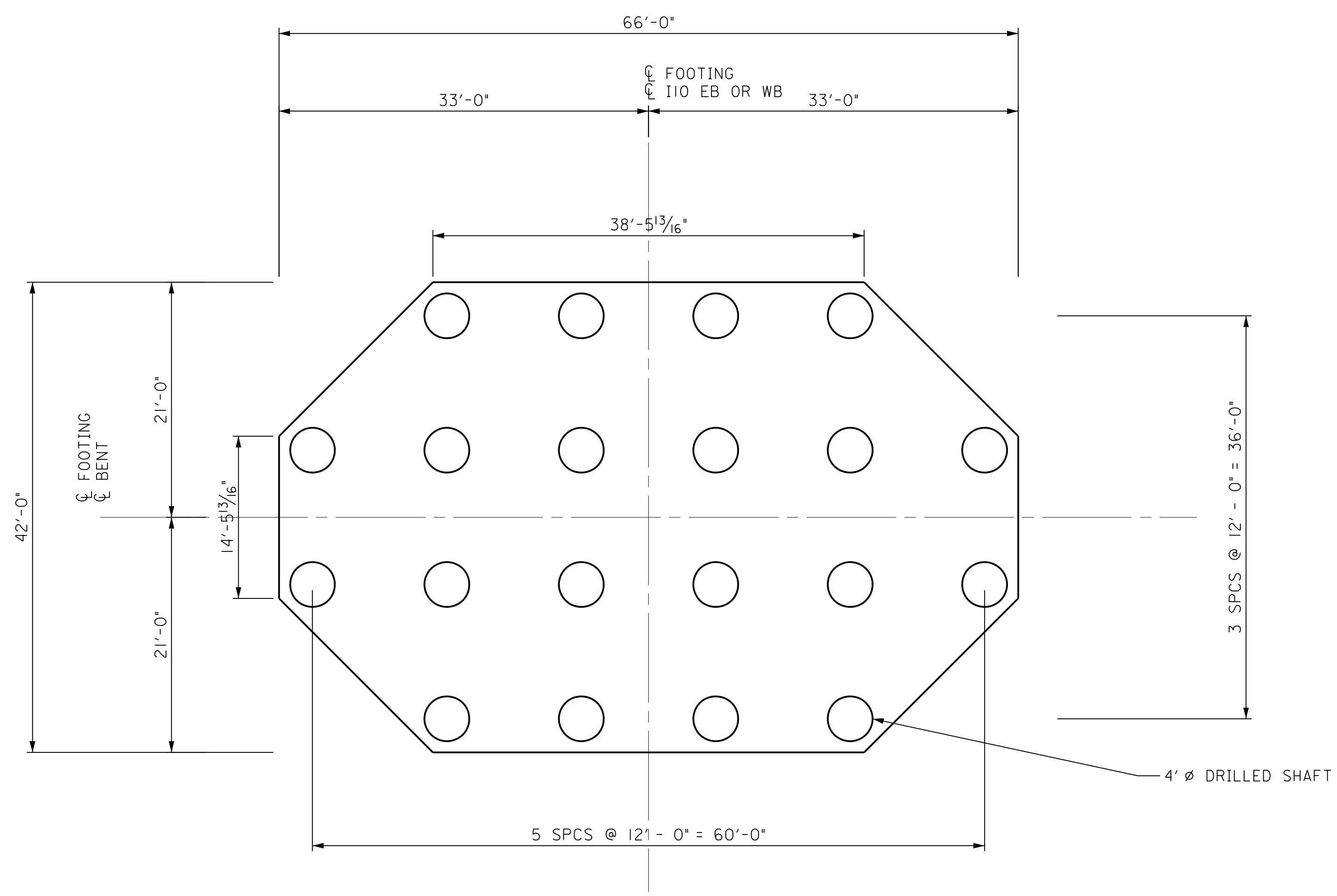
MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(3 OF 3)

ESTIMATED QUANTITIES	DESIGNED BY: RH	DRAWN BY: JRO
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

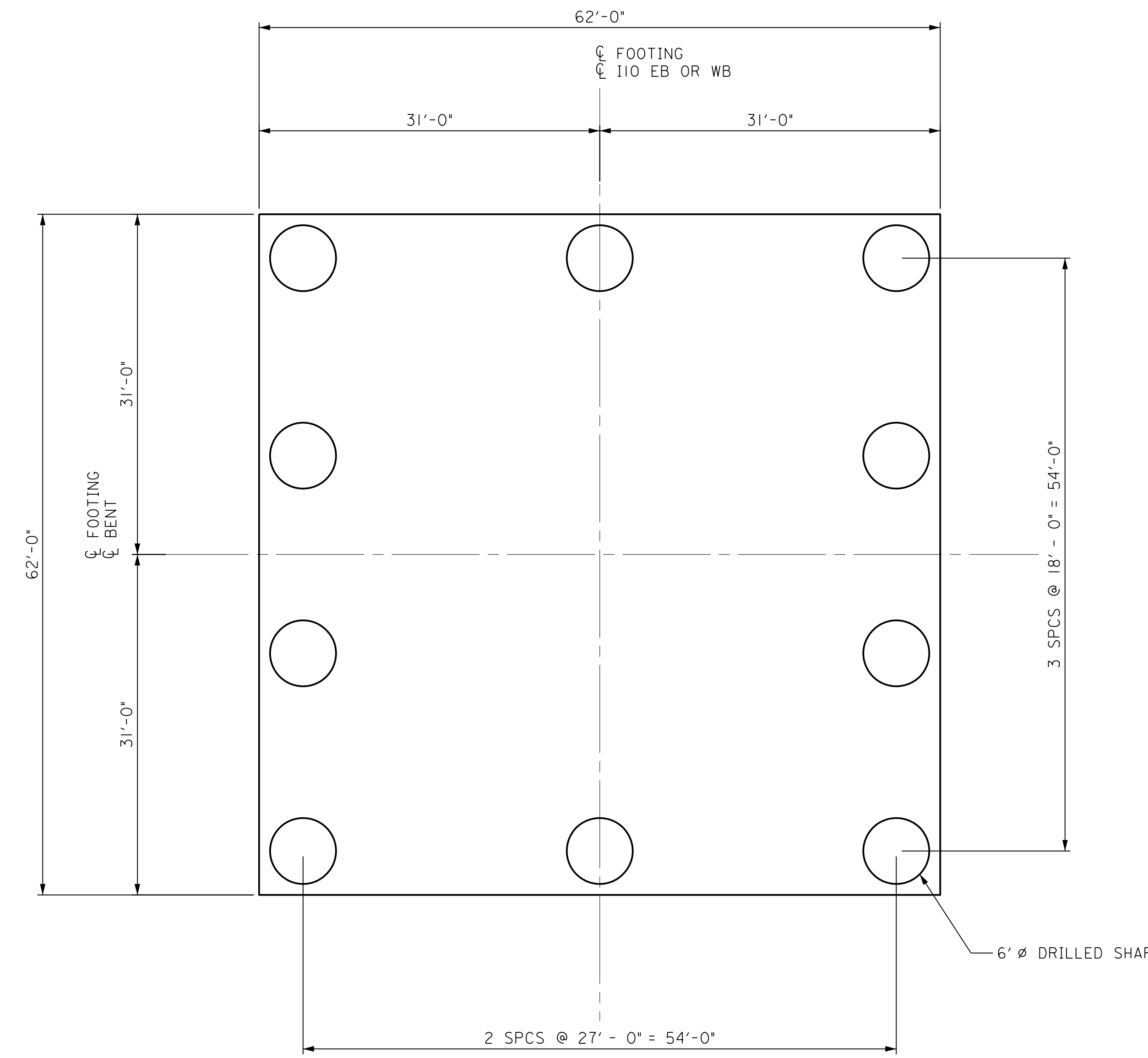


6 X 4 LAYOUT

4FT DRILLED SHAFTS
(20 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -160'



3 X 4 LAYOUT

6FT DRILLED SHAFTS
(10 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -160'

OPTION 1: BALANCED CANTILEVER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(1 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

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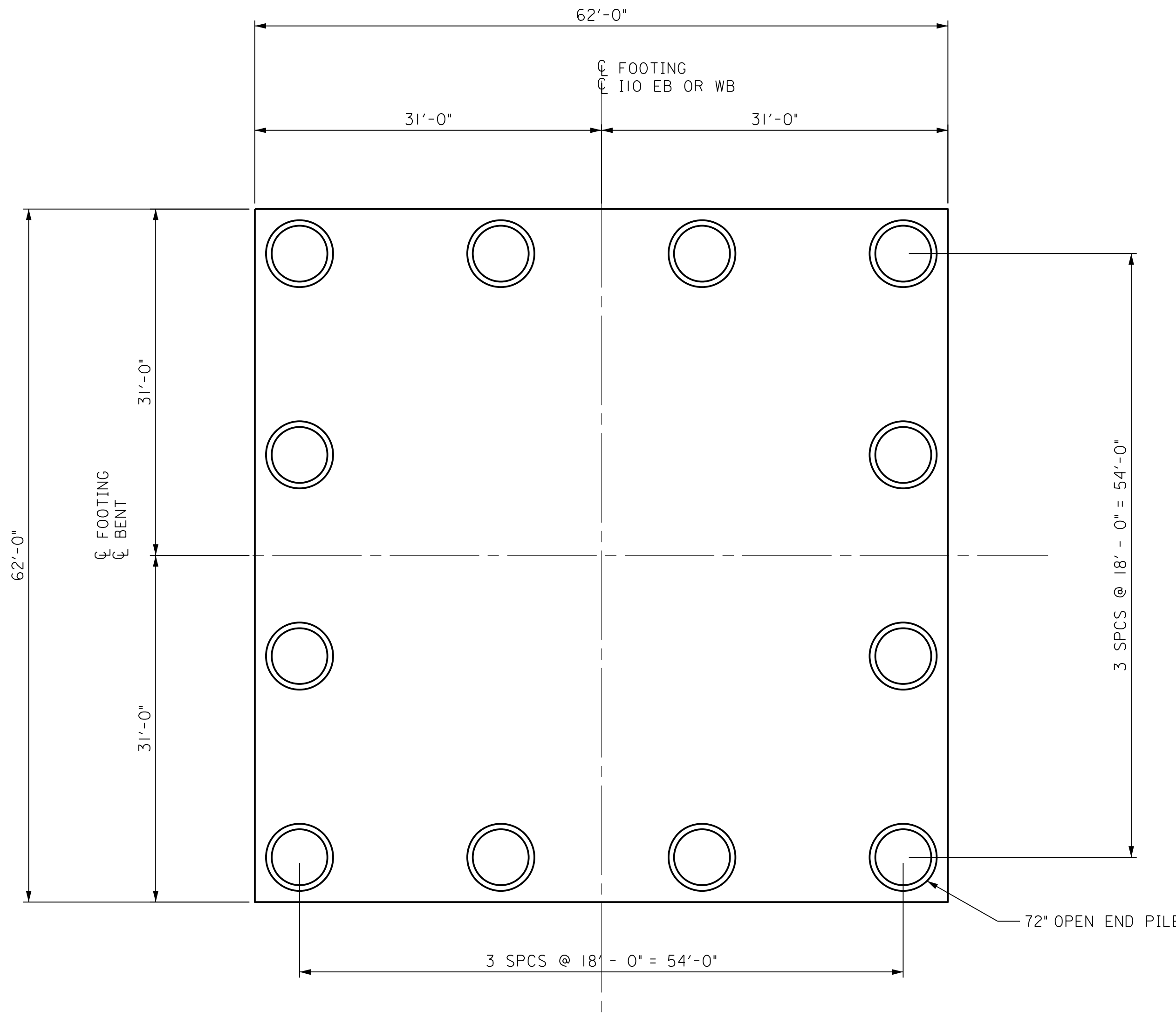
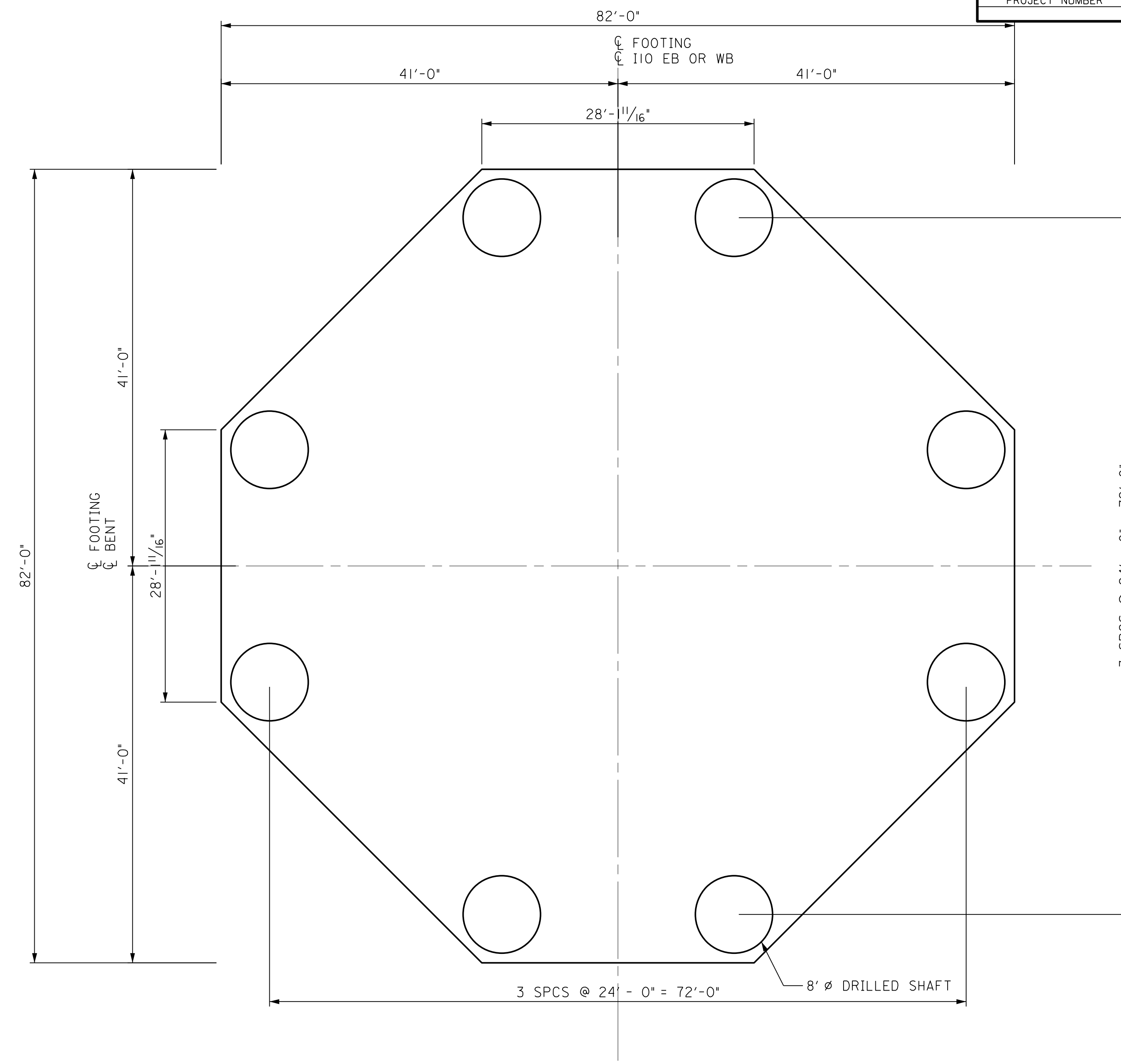


4X4 LAYOUT

OEP PILES
(12 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -270'



OCTAGON LAYOUT

8FT DRILLED SHAFTS
(8 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -160'

OPTION 1: BALANCED CANTILEVER

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(2 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

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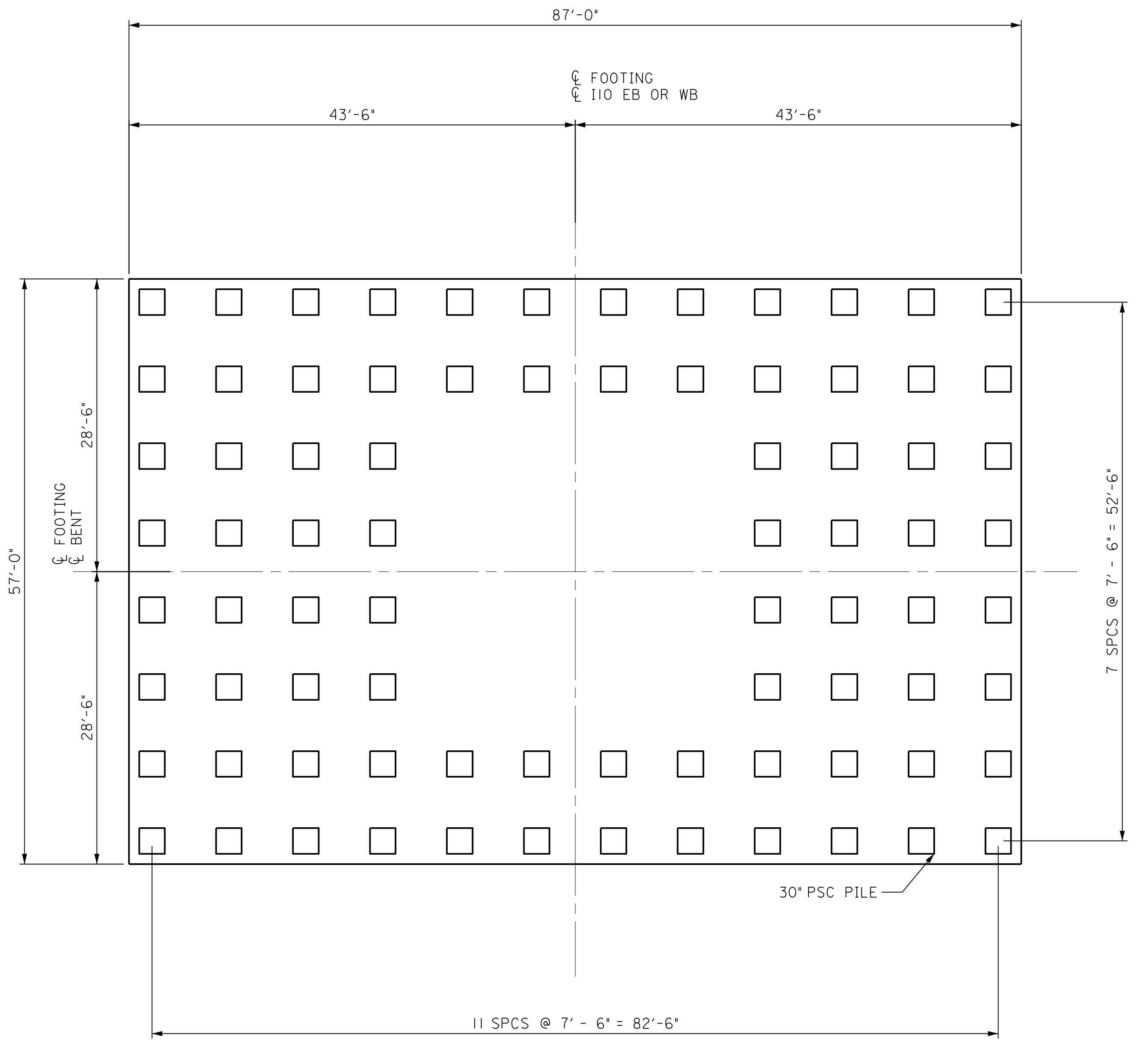
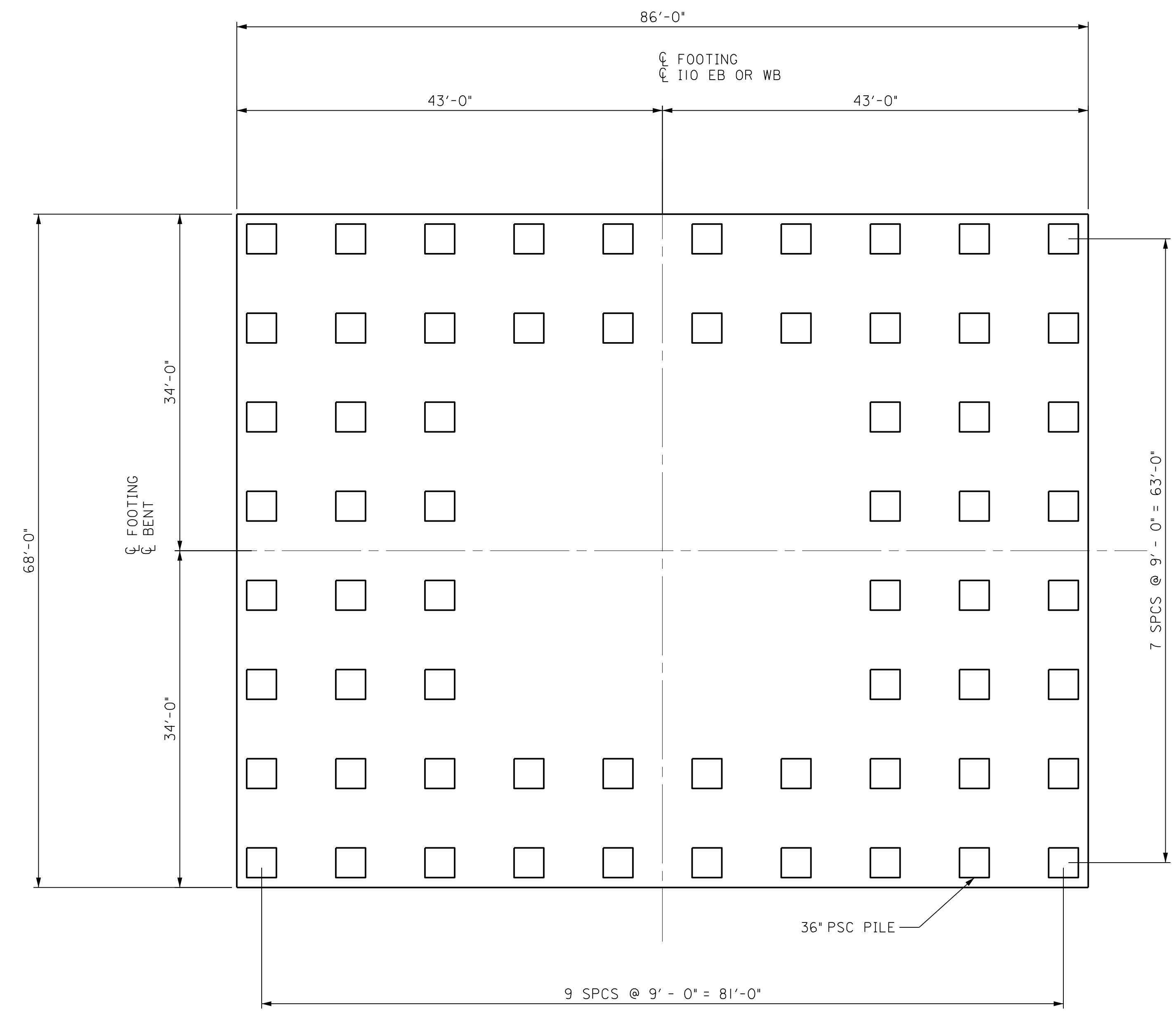


10X8 LAYOUT

30" PSC PILES
(80 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -65'



8X8 LAYOUT

36" PSC PILES
(64 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -65'

OPTION 1: BALANCED CANTILEVER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(3 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

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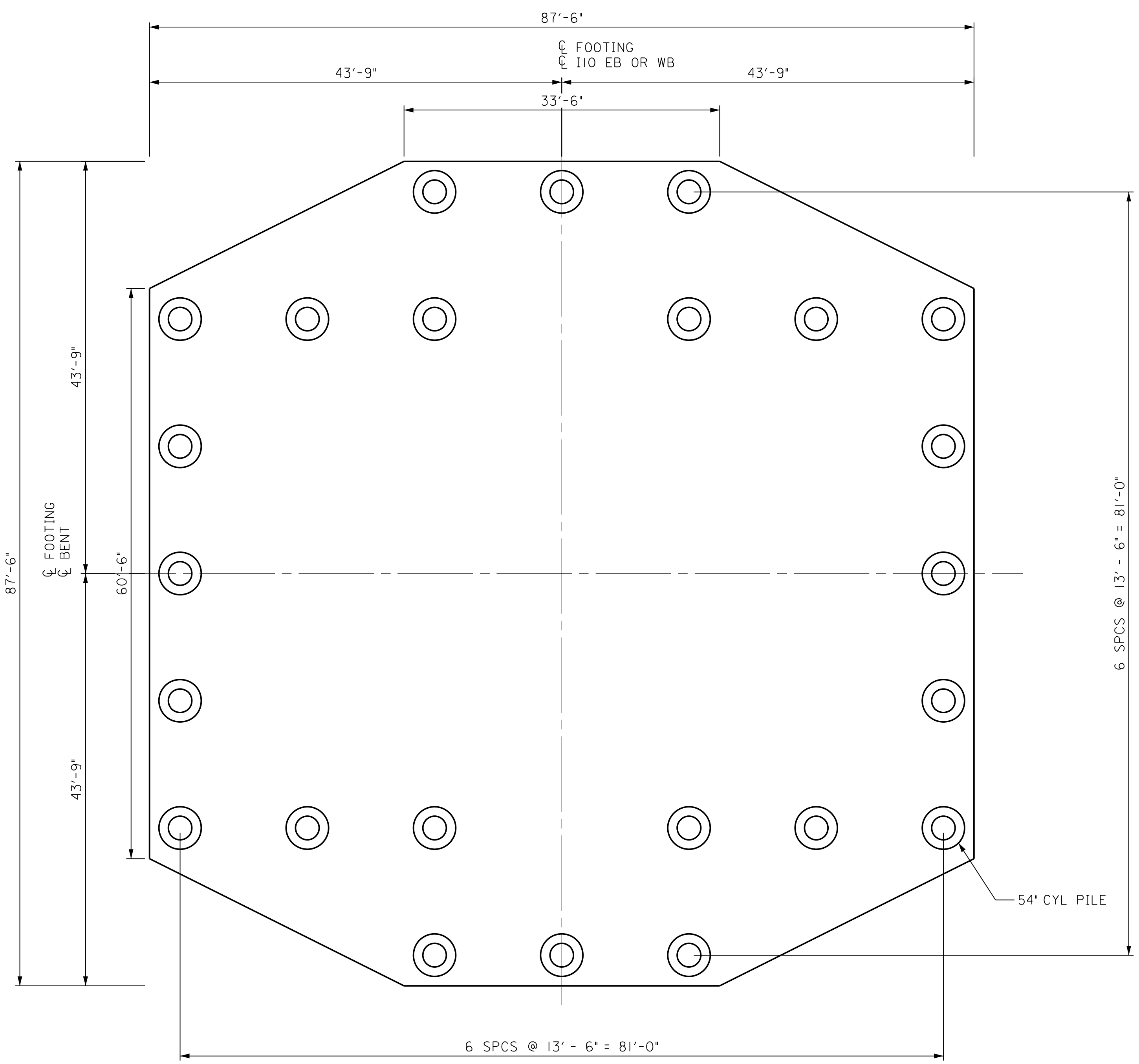
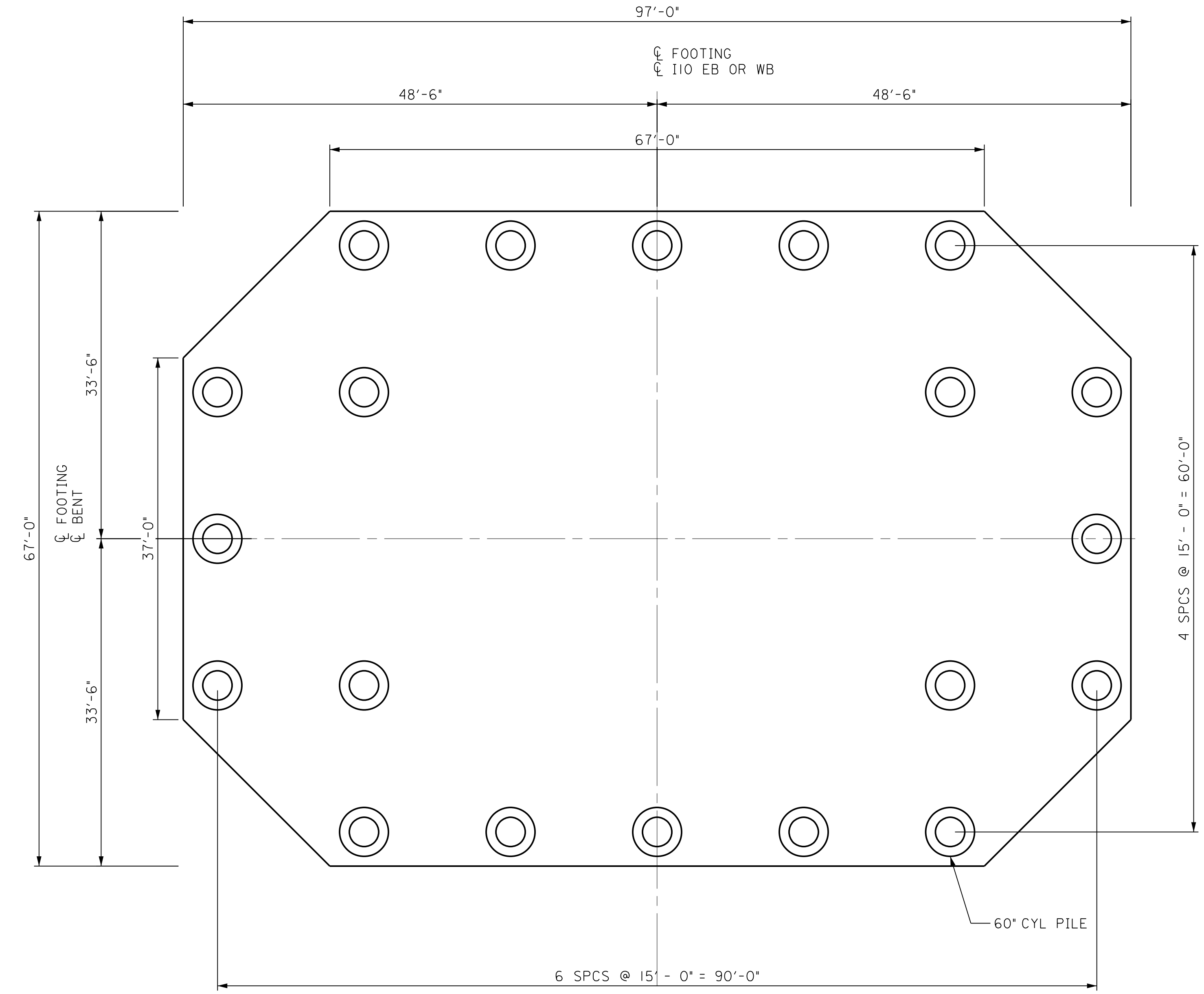


OCTAGON LAYOUT

54" CYL PILES
(24 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -110'



OCTAGON LAYOUT

60" CYL PILES
(20 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -120'

OPTION 1: BALANCED CANTILEVER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(4 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
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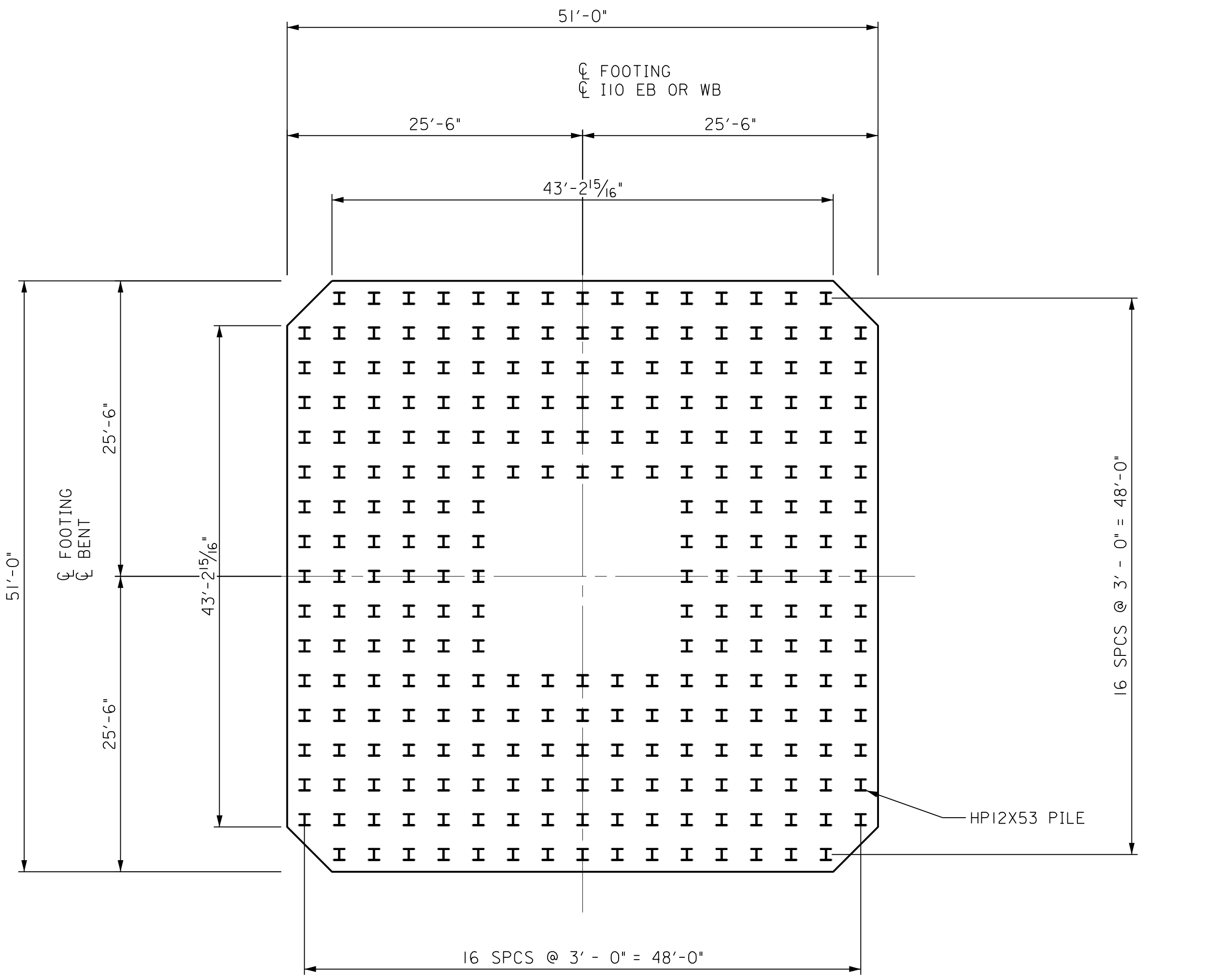
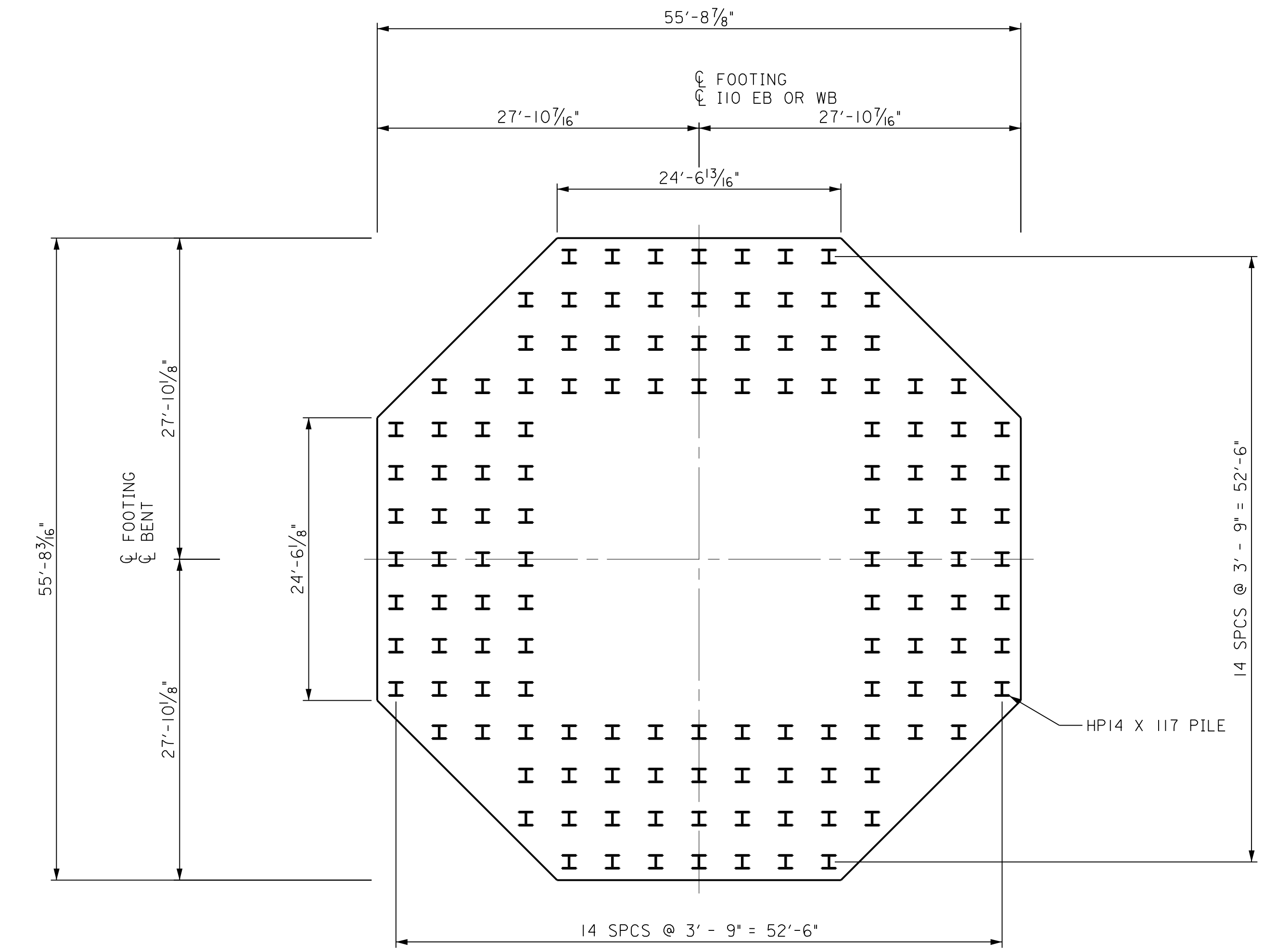


17x17 LAYOUT

HP12X53 PILES
(260 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -90'



OCTAGON LAYOUT

HP14X117 PILES
(132 TOTAL)

NOTES:

1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 10'
3. TIP ELEV. = -120'

OPTION 1: BALANCED CANTILEVER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

HIGH LEVEL APPR FOUNDATION OPTIONS
(5 OF 5)

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

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Appendix D - Final Report on Vibration Due to Pile Driving at the Mobile River Bridge

Final Report on Vibrations Due to Pile Driving at the Mobile River Bridge Site

Research Project 930-839R

INVESTIGATION OF PILE SETUP (FREEZE) IN ALABAMA

Development of a Setup Prediction Method and Implementation into LRFD Driven Pile Design

<u>Addendum:</u> Pile Driving Vibration Monitoring of the Future Mobile River Bridge Project

John Cleary, Ph.D., P.E. (P.I. for Vibration Monitoring)
Eric Steward, Ph.D. (Co. P.I. for Vibration Monitoring)
Andrew Gillis (Graduate Student)
Department of Civil Engineering
University of South Alabama
Mobile, AL 36688

June 12, 2015

ACKNOWLEDGEMENT

The Alabama Department of Transportation sponsored this project. Their support for this study is greatly appreciated.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Alabama DOT or the University of South Alabama. This report does not constitute a standard, specification, or regulation. Comments contained in this paper related to specific testing equipment and materials should not be considered an endorsement of any commercial product or service; no such endorsement is intended or implied.

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ABSTRACT

All projects have some amount of inherent risk; one such risk associated with construction projects is the potential for ground vibrations that could damage nearby structures. Research has been conducted on the effects of vibrations on structures; however, the expected levels of vibration are dependent on several factors including the soil conditions at the construction site. Therefore, site-specific investigations are often recommended.

After concerns were raised by the Alabama Department of Transportation (ALDOT) about damage potential at a project site in South Alabama, an addendum was added to a research project related to investigating pile setup in Alabama soils. The purpose of the addendum was to investigate ground vibrations from pile driving at a project site near the Mobile River in Mobile, Alabama.

An investigation and vibration monitoring program was developed for four pile sizes that are often used by the Alabama Department of Transportation (ALDOT). The piles included thirty-six inch square and twenty-four inch square concrete piles, as well as, two steel H-Piles. The piles were driven using typical installation techniques and the vibration levels at various distances from the piles were monitored.

The investigation found that the largest vibrations were observed while driving the thirty-six inch concrete pile. The maximum vibrations observed had a magnitude of 0.82 inches per second at fifty feet from the pile. The vibrations at 150 feet from the pile had dissipated to 0.15 inches per second. The results of the monitoring program and a literature review determined that an allowable vibration level of 0.5 inches per second for modern structures and 0.1 inches per second for potentially sensitive structures should be established for construction activity at or near the location of the project site. Additionally, a survey distance of 150 feet for modern structures and 250 feet for potentially sensitive structures is recommended.

INTRODUCTION

Background

The following report contains the analysis of ground vibrations generated during a pile driving research study located at the Mobile River Bridge Project Site. The project site, owned by the Alabama Department of Transportation (ALDOT), is located on the Mobile River just south of the Alabama Cruise Terminal, Figure 1. The study consisted of monitoring ground vibrations during the installation of four driven piles; two precast concrete piles and two steel H-piles. The study was conducted in response to concerns raised by ALDOT related to possible damage of nearby structures from ground-borne vibrations. The primary objective of this project was to determine the distance that pile driving operations can be conducted with minimal risk to nearby structures. To accomplish this, the vibration levels at various distances from the driven piles were determined and a prediction equation for other distances was developed. This study was conducted by researchers from the Department of Civil Engineering at the University of South Alabama between August 15, 2013 and August 27, 2013.

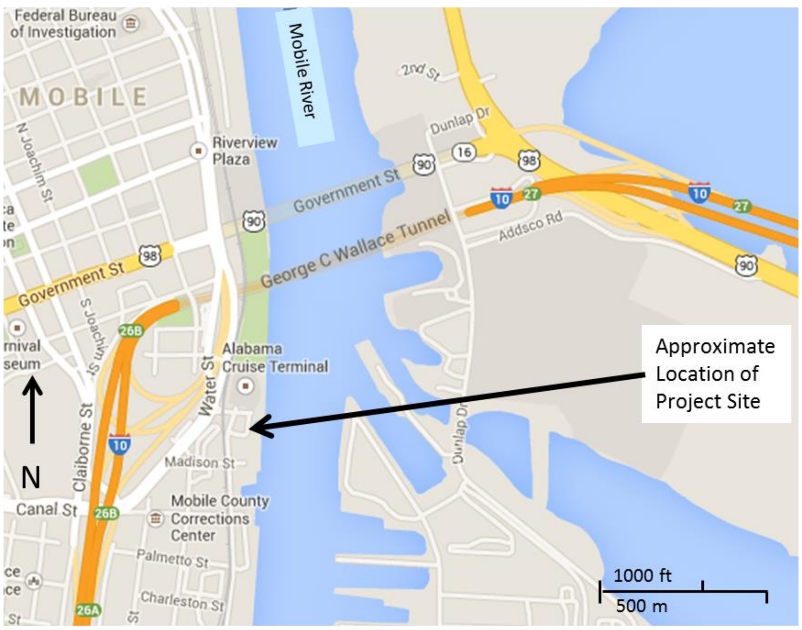

Figure 1: Location of project site, Mobile, AL **(Google 2013)**

Objective

This project consisted of several objectives. The first was to determine the vibration levels from typical piles used by ALDOT. The second objective was to develop a methodology to predict vibrations at any distance from the pile. The third and final objective of the project was to develop guidelines on allowable vibrations for the project site.

Scope

The scope if this report is limited to the vibrations portion of the larger project: *Investigation of Pile Setup (Freeze) In Alabama: Development of a Setup Prediction Method and Implementation into LRFD Driven Pile Design; Addendum: Pile Driving Vibration Monitoring of the Future Mobile River Bridge Project* (Research Project 930-839R).

The vibrations portion of the project was limited to the aforementioned location near the Mobile River. The project included monitoring vibrations during pile installation and restrikes, analysis of vibration data, development of vibration prediction methodology, and vibration limit recommendations.

Report Organization

The report is organized into five main sections: Introduction, Literature Review, Experimental Design, Results, and Conclusions. Each section contains sub sections as needed.

LITERATURE REVIEW

Construction Vibrations

Ground vibrations are commonly generated from several sources including roadway traffic, railroad traffic, and construction activity. Vibrations can be measured and quantified using several different parameters including: displacement, velocity, and acceleration. Ground vibrations are typically measured by the velocity of the ground surface and reported as Peak Particle Velocity or PPV. Typical units of PPV are inches per second (in/sec) in the US system or millimeters per second (mm/sec) in the SI system of units. Typical construction activity that generates vibrations includes: pile driving, heavy equipment operation, concrete breaking (jackhammers), and truck/equipment traffic. Although the level of vibrations generated from these sources can vary widely, some typical vibration levels have been included in Table 1.

Table 1: Typical ground vibrations from construction equipment (Hanson, Towes and Lance 2006)

Equipment		**PPV (in/sec)** (Distance = 25 ft.)
Pile Driver (impact)	upper range	1.518
	typical	0.644
Pile Driver (vibratory)	upper range	0.734
	typical	0.170
Bulldozer	large	0.089
	small	0.003
Caisson Drilling		0.089
Loaded Trucks		0.076
Jackhammer		0.035

Table 1 shows that under typical conditions, pile driving has the potential to create large vibration levels, relative to other construction activity. The pile installation method, however, can affect the level of vibrations. High displacement piles are typically driven using an impact hammer and low displacement piles are sometimes driven using a vibratory hammer. Research has shown that the vibration magnitudes from vibratory hammers are typically smaller than from impact hammers. Additionally, installation techniques such as pre-boring and jetting can reduce vibration levels from impact pile driving (Woods 1997).

The mechanism of vibration formation is the transfer of energy from the pile driving hammer to the pile and then to the surrounding soil. The transfer of energy comes from two main sources. The first is the skin friction that is developed along the surface of the pile and the second is the displacement of the soil at the pile tip. For high displacement piles, the main source of energy transfer is at the pile tip. Several factors can affect the magnitude of vibrations including pile size, pile type, soil type, and the hammer energy. The most important factor in determining vibration levels is the distance from the pile, since vibrations will mitigate or dampen with distance from the source (Dowding 1996).

Damage Thresholds

Vibrations generated from construction activity can cause several concerns at adjacent structures that range from annoyance to structural damage. Several studies have been conducted to determine the relationship between vibration levels, human perception, and structural damage. Table 2 contains a summary of a study reported by Hendriks (2002) for continuous vibrations. The study concluded that vibration levels that are large enough to "annoy people" are at threshold levels for architectural damage to structures that contain plaster walls or ceilings. Since these levels are below levels of even minor structural damage, the perception of building occupants can sometimes lead to discrepancies in the effects of vibrations. The values listed in Table 2 are generally conservative when compared to pile driving vibrations since they were developed for continuous vibrations. Pile driving operations develop discontinuous vibrations that can reduce the damage potential (Hendriks 2002).

Table 2: Continuous vibration levels and effects (Hendriks 2002)

Vibration Level (Peak Particle Velocity)	Human Reaction	Building Effects
0.006-0.019 in/sec	Threshold of perception;	Vibrations unlikely to cause damage
0.08 in/sec	Vibration readily perceptible	Recommended upper level for ruins and ancient monuments
0.1 in/sec	Continuous vibrations begin to annoy people	Virtually no risk of "architectural" damage to normal buildings
0.2 in/sec	Vibrations annoying to people in buildings	Threshold at which there is a risk of "architectural" damage to normal dwelling- houses with plaster wall and ceilings
0.4-0.6 in/sec	Vibrations considered unpleasant by people subjected to continuous vibrations	Vibrations at a greater level than normally expected from traffic, but would cause "architectural" damage and possible minor structural damage

In addition to the many studies to determine the effect of vibrations on structures, several State and Federal Agencies, as well as, International Organizations have developed guidelines on permissible vibration levels due to construction activity. Much of the early work related to vibrations was performed by the United States Bureau of Mines (USBM) in the 1970's and 80's (Siskind, et al. 1980). This research focused on vibrations from blasting operations. Figure 2 shows the recommended vibration limits for blasting as a function of frequency. The limits range from 0.2 to 2.0 inches per second (in/sec).

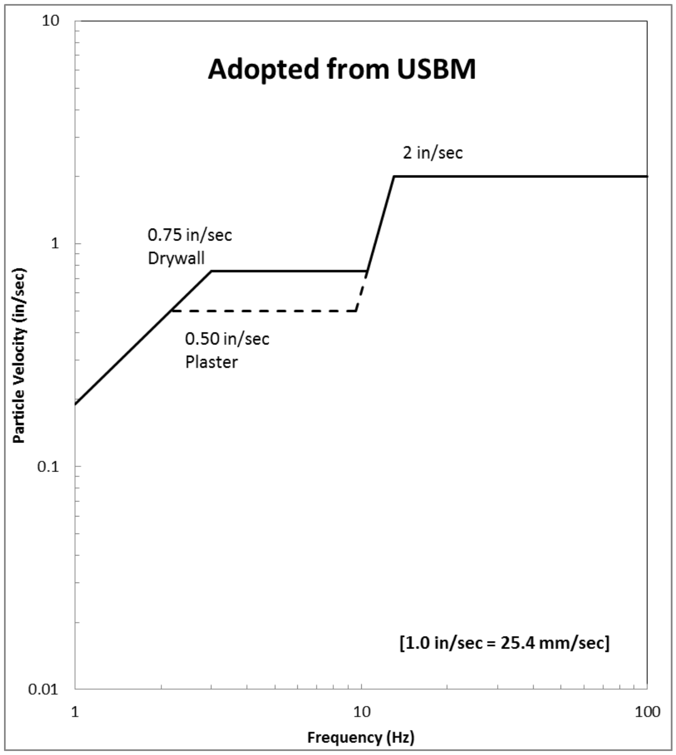

Figure 2: Vibration limits from the USBM (Siskind, et al. 1980)

The American Association of State Highway and Transportation Officials (AASHTO) and the Federal Transit Administration (FTA) have developed guidelines for vibration limits that range from 0.1 to 1.5 in/sec depending on the structure type as shown in Table 3.

Table 3: AASHTO and FTA criteria for construction vibrations

Organization/Jurisdiction	**Comments**	**PPV (in/sec)**
American Association of State Highway and Transportation Officials (AASHTO 1990)	Residential buildings, plastered walls	0.2-0.3
	Residential buildings in good repair with gypsum board walls	0.4-0.5
	Engineered structures, without plaster	1.0-1.5
	Historic sites or other critical locations	0.1
Federal Transit Administration (FTA 2006)	Reinforced-concrete, steel or timber	0.5
	Engineered concrete and masonry	0.3
	Non-engineered timber and masonry	0.2
	Buildings extremely susceptible to vibration damage	0.12

The vibration criteria developed by the various states also have a wide range of values as shown in Table 4. If the table is carefully analyzed, the vibration limits can be divided into several categories including: modern structures, sensitive structures, and miscellaneous structures. The range of vibration limits for modern structures is from 0.4 to 1.0 in/sec and sensitive structures have a range of 0.08 to 0.2 in/sec. These vibration limits correlate well to the AASHTO and FTA limits. A thorough review of construction vibration limits can be found in several reports including: (Tao and Zhang 2012), (Wilson Ihrig & Associates 2012), and (Cleary 2013).

Table 4: State criteria for construction vibrations

Organization/Jurisdiction	Comments	PPV (in/sec)
California Department of Transportation (Caltrans 2002)	Upper level for possible damage Threshold for damage to plaster Ruins and ancient monuments	0.4-0.6 0.20 0.08
Florida DOT (FDOT 2010)	All construction Fresh concrete	0.5 1.5
Iowa DOT (Iowa DOT n.d.)	Project specific specification	0.2
Louisiana Department of Transportation and Development (Tao and Zhang 2012)	General scenario - New requirements - Old requirements Historic structures or loose sandy soil	 0.5 0.2 0.1
New Hampshire DOT (NHDOT 2010)	Modern Homes Older Homes	0.75 0.50
New York City DOT (New York City DOT 2009)	Piles driven adjacent to subway structures (may be lowered)	0.5
Rhode Island DOT (RIDOT 2010)	Lower limits may be applied by engineer	1.0

Dynamic Settlement

In addition to structural damage and human perception, dynamic settlement can occur due to construction vibrations. Research has shown that if loose cohesionless soils (loose sands) are present, relatively low vibration levels can cause densification (Dowding 1996). This densification can lead to settlement related damage in adjacent structures. Loose sands are typically defined as having a relative density less than 40% (Tao and Zhang 2012). Dynamic settlement has occurred in some soils at vibration levels as low as 0.1 in/sec. If loose sands are located on or near a project site, then special considerations for construction vibrations need to be considered.

Vibration Prediction

Since it is typically unrealistic for most construction projects to conduct full scale testing to determine the expected levels of vibrations and since only a discrete number of locations are measured during testing, several methods have been developed to predict vibration levels. The first prediction equations were developed as early as 1912 by Golitsin who developed a simple equation to predict the peak particle displacement of ground vibrations from earthquakes. The equation, as reported by (Bayraktar, et al. 2013) is as follows,

$$A_2 = A_1\sqrt{r_1/r_2}\,e^{-\gamma(r_2-r_1)}, \tag{1}$$

where A_1 is the peak particle displacement of ground vibrations at a distance r_1 from the source, A_2 is the peak particle displacement of ground vibrations at a distance r_2 from the source, and γ is a vibration attenuation coefficient.

More recently, several methods have been developed to predict the peak particle velocity (PPV) from construction activity, pile driving in particular. Hendriks (2002) reported several equations to predict the propagation of construction vibrations. The first equation presented by Hendriks was first reported by Richart, et.al. (1970), who cited Bornitz (1931),

$$V = V_o(D_o/D)^{0.5}e^{\alpha(D_0-D)} \tag{2}$$

where V is the peak particle velocity at distance D, V_o is the peak particle velocity at reference distance D_o, and α is a vibration attenuation parameter that must be determined experimentally.

Hendriks (2002) also reported a simplified equation for pile driving vibrations that is similar to an equation reported by Woods (1997) as follows,

$$V = V_o(D_o/D)^k \tag{3}$$

where V is the peak particle velocity at distance D, V_o is the peak particle velocity at reference distance D_o, and k is a vibration attenuation parameter that must be determined experimentally.

Several researchers have found that a better correlation with predicted and measured vibrations could be determined by including the energy of the pile driving hammer in the equation. This approach is often referred to as the "scaled-distance" approach. One commonly used equation was developed by Wiss and reported by Bayrakter, et al. (2013),

$$v = k\left[D/\sqrt{W_t}\right]^{-n} \tag{4}$$

where W_t is the energy of the source, v is the peak particle velocity at distance D, k is the intercept value of the peak particle velocity at a scaled distance of $D/(W_t)^{1/2}$ equal to one, and n is a vibration attenuation parameter that must be determined experimentally.

The previous equations are relatively accurate at predicting ground vibrations when compared to experimental data, however, they all require testing to determine the soil parameters. Jones & Stokes (2004) performed an extensive literature review and determined that the following equation, with the assumed values shown, could be used to predict pile driving vibrations without experimental evaluations:

$$PPV_{Impact\ Pile\ Driver} = PPV_{Ref}(25/D)^{n}\left(E_{equip}/E_{ref}\right)^{0.5} \tag{5}$$

where $PPV_{Impact\ Pile\ Driver}$ is the peak particle velocity at distance D in feet, PPV_{Ref} is equal to 0.65 in/sec for a reference pile driver at 25 feet, E_{ref} is equal to 36,000 ft-lb (rated energy of reference pile driver), E_{equip} is the rated energy of impact pile driver in foot-pounds, and n is a vibration attenuation parameter with a recommended value of 1.1.

Jones and Stokes also provided a table, Table 5, with suggested “n” values based on the soil type.

Table 5: Suggested “n” values based on soil class: Adopted from (Jones & Stokes 2004)

Soil Class	Description of Soil	Suggested Value of “n”
I	Weak or soft soils: loose soils, dry or partially saturated peat and muck, mud, loose beach sand, and dune sand, recently plowed ground, soft spongy forest or jungle floor, organic soils, top soil. (shovel penetrates easily)	1.4
II	Competent soils: most sands, sandy clays, silty clays, gravel, silts, weathered rock. (can dig with shovel)	1.3
III	Hard soils: dense compacted sand, dry consolidated clay, consolidated glacial till, some exposed rock. (cannot dig with shovel, need pick to break up)	1.1
IV	Hard, competent rock: bedrock, freshly exposed hard rock. (difficult to break with hammer)	1.0

EXPERIMENTAL DESIGN

Overview

The main objective of this research was to determine the distance from nearby structures that pile driving operations can be conducted with minimal risk to those structures. It is important to note that these guidelines were developed for typical piles used by ALDOT at the project site. The project was divided into two phases, collecting data during pile driving and analyzing the data. The information related to the project site, the test piles, the pile driving equipment, and the data collection equipment is located below.

Project Site

The project site is located on the west bank of the Mobile River, just south of the Alabama Cruise Terminal. The soil profile at the site consists primarily of sandy soils to a depth of 90 feet below the ground surface with a clay layer located at an approximate depth of 90 to 110 feet. Table 6 contains a summary of the soil layers that were defined by a standard penetration test (SPT) conducted at the project site. Appendix A contains the details of the soil investigations conducted by an ALDOT drill crew and Southern Earth Sciences.

Table 6: Soil profile at site location

Depth (ft.)	Basic Material	Average Blow Count	Consistency
0-23.5	Sand	12	Loose to Medium
23.5-89.5	Sand	31	Medium to Dense
89.5-108.5	Clay	28	Stiff to Very Stiff
108.5-115	Sand	27	Medium

Figure 3 contains a plan view of the project site. The dashed line in the figure represents the approximate property boundary. Note that the pile locations are approximate and the drawing is not to scale. The arc lines shown in the drawing represent the approximate distance from the piles to where the monitoring equipment was located.

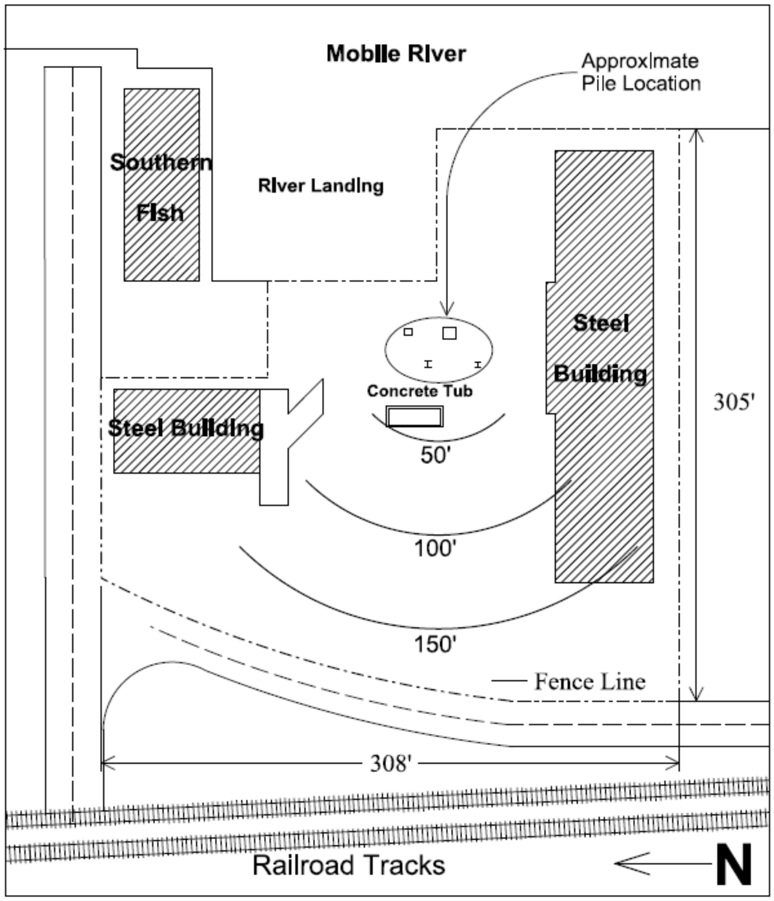

Figure 3: Plan view of Mobile River Bridge Project Site

Four test piles were driven for this project, two prestressed precast concrete piles (PPC) and two steel H-Piles. Table 7 contains descriptions of the piles and Appendix B contains the details of the two pile driving hammers utilized on this project. The piles were installed using typical techniques including pile jetting or vibration followed by driving with a diesel hammer. The concrete piles were jetted to a depth of approximately 30 feet and driven to the final elevation using a Delmag Model D-62-22 diesel hammer. A vibratory driver was used to drive the steel HP 14 to 55 feet and the HP 12 to 15 feet. The steel piles were then driven to the final elevation using an APE Model D30-42 diesel hammer.

Table 7: Pile descriptions

Pile	Cross Section	Material	Length
#1	24” Square	Precast Concrete	81 ft
#2	36” Square	Precast Concrete	89 ft
#3	HP14x117	Steel	106 ft
#4	HP12x53	Steel	70 ft

Vibration Monitoring

Data collectors were placed at various locations throughout the pile installation and testing process. The data collectors utilized for this project were Minimate Plus tri-axial geophones manufactured by Instantel. Each tri-axial geophone unit contains three geophones oriented on three mutually perpendicular axes. The units come with software allowing data collection and analysis in several configurations. For this research, the units were configured to collect histogram data during two-second intervals. When configured in this way the data collector measures all vibrations over the interval, but only records the maximum PPV and the frequency that it occurred at for each geophone over the two second interval.

The geophones were placed at predetermined distances from each pile during installation. Three of the data collectors were located at approximately 50, 100, and 150 feet. A fourth data collector, which had two geophone units attached to it, was located at various distances throughout testing to collect additional information. Table 8 contains a detailed account of the location of each data collector during testing.

During the initial driving of the 36-inch PPC pile, geophone number three was located at the edge of the project site near Southern Fish and Oyster, an adjacent property owner. The fourth data collector had one geophone unit placed at 100 feet from the pile and the other geophone unit was attached to the brick façade of a building that was located on the project site. Please note that the 30-day restrike was at 32-days for the 36-inch concrete pile and 31-days for the 24-inch concrete pile.

Table 8: Geophone location during testing

		Geophone Unit				
Initial Drive	**Pile Type**	**#1**	**#2**	**#3**	**#4a**	**#4b**
Aug. 19, 2013	36” PCP	50 ft	150 ft	69 ft	100 ft	Building
Aug. 20, 2013	24” PCP	99.5 ft	142 ft	n/a	n/a	n/a
Aug. 21, 2013	HP 12	53 ft	101 ft	144 ft	n/a	n/a
Aug. 21, 2013	HP 14	58 ft	106 ft	146 ft	n/a	n/a
24 Hour Restrike						
Aug. 22, 2013	HP 12	50 ft	150 ft	100 ft	n/a	n/a
Aug. 22, 2013	HP 14	50 ft	150 ft	100 ft	n/a	n/a
3-Day Restrike						
Aug. 22, 2013	36” PCP	50 ft	n/a	100 ft	n/a	n/a
Aug. 23, 2013	24” PCP	50 ft	150 ft	100 ft	n/a	n/a
7-Day Restrike						
Aug. 26, 2013	36” PCP	50 ft	150 ft	100 ft	75 ft	125 ft
Aug. 27, 2013	24” PCP	50 ft	150 ft	100 ft	75 ft	125 ft
30-Day Restrike						
Sept. 20, 2013	36” PCP	50 ft	150 ft	100 ft	n/a	n/a
Sept. 20, 2013	24” PCP	55 ft	155 ft	105 ft	n/a	n/a
Sept. 20, 2013	HP 12	50 ft	150 ft	100 ft	n/a	n/a
Sept. 20, 2013	HP 14	50 ft	150 ft	100 ft	n/a	n/a

RESULTS

Vibration Levels

Vibrations were monitored during installation and restrikes on the 36-inch concrete pile at three, seven, and thirty days. A communication error occurred between the ALDOT personnel, the pile driving contractor, and the research team during the installation of the 24-inch concrete pile which resulted in the start of driving prior to the installation of the vibration monitors. Due to this error, the 24-inch concrete pile only had vibrations monitored during the final stage of driving and at all restrikes. The steel piles were monitored during installation and during the one day and thirty day restrikes.

Baseline vibration data was collected at the project site by monitoring vibration levels due to railroad activity from a pair of railroad tracks located adjacent to the project site, Figure 3. The approximate distance from the tracks to the data collectors was determined and the vibration levels from train activity were evaluated. Due to the relatively low vibration levels recorded during train activity, baseline data was not collected for truck traffic.

The vibration data collected from the project site was analyzed and the peak particle velocity (PPV) from each pile was recorded. Table 9 contains a summary of the results. The largest recorded vibration during this study occurred while driving the 36-inch concrete pile and resulted in a PPV of 0.82 inches per second at a distance of 50 feet.

Table 9: Maximum PPV (in/sec) during pile driving operations

Vibration Source	Horizontal Distance from Pile		
	50 feet	100 feet	150 feet
36" Concrete Pile	0.82	0.28	0.15
HP14x117	0.18	0.09	0.11
HP12x53	0.23	0.07	0.08
Railroad Activity	0.03[1]	0.02[1]	0.02[1]

[1]The approximate distances were 60, 110, and 160 feet

Figure 4 shows the maximum PPV for the 36-inch concrete pile, the H-Piles, and railroad activity observed during testing. Since the maximum vibrations occurred during the beginning of the driving process, the 24-inch concrete pile was not included in this figure. The figure confirms that the largest vibrations recorded were associated with the installation of the 36-inch concrete pile.

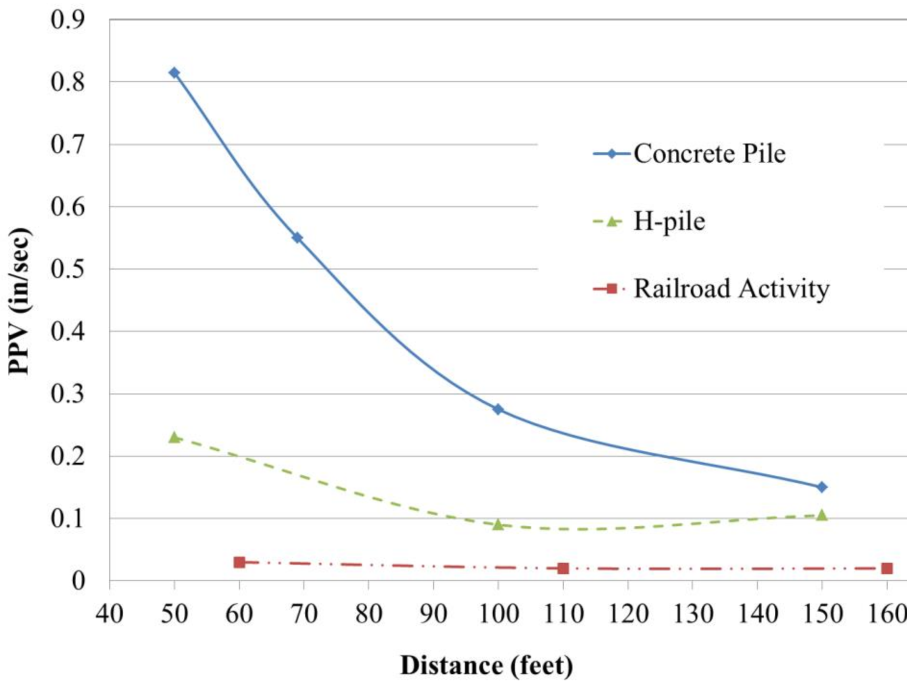

Figure 4: Maximum recorded vibration levels during pile installation

During the driving of the 36-inch concrete pile, one of the geophones was attached to the brick façade of a building that was located on the project site. The building was located to the south of the piles, Figure 3, and was approximately 90 feet from the 36-inch concrete pile. The brick façade was located on the west end of the building and was approximately 140 feet from the pile. The data from this geophone was analyzed and it was determined that the vibration levels were below the threshold for detection, 0.005 in/sec. This indicates that the ground vibrations did not have enough energy to cause vibrations in the building. Additionally, crack width monitors were installed on the outside wall of the building. The crack widths and lengths were monitored throughout the project and it was determined that there were no changes in any of the cracks.

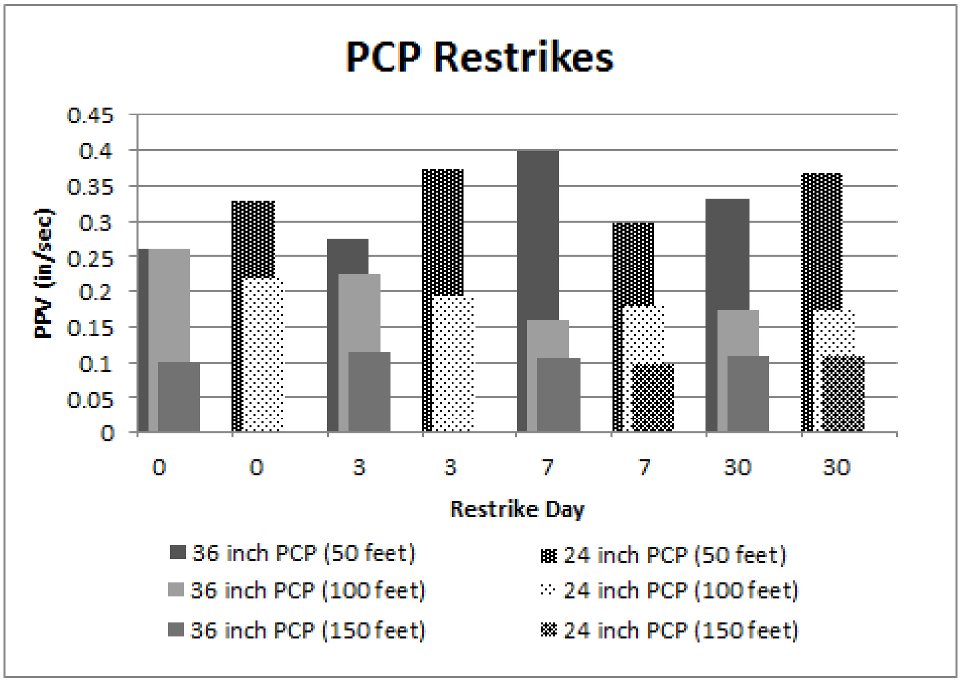

Figure 5: Bar chart of restrikes on precast concrete piles (PCP)

An analysis was performed to compare the vibrations between the 24- and 36-inch concrete piles since data was not collected throughout the driving of the 24-inch pile. Figure 5 shows a bar chart of the vibration levels for each of the concrete piles during the restrikes, note that day zero is at the end of drive. Figure 6 shows the same data in the form of a data plot. The data indicates that the vibration levels for the 24- and 36-inch concrete piles are similar and that the maximum vibrations, near the start of driving, would be expected to be approximately equal for each concrete pile.

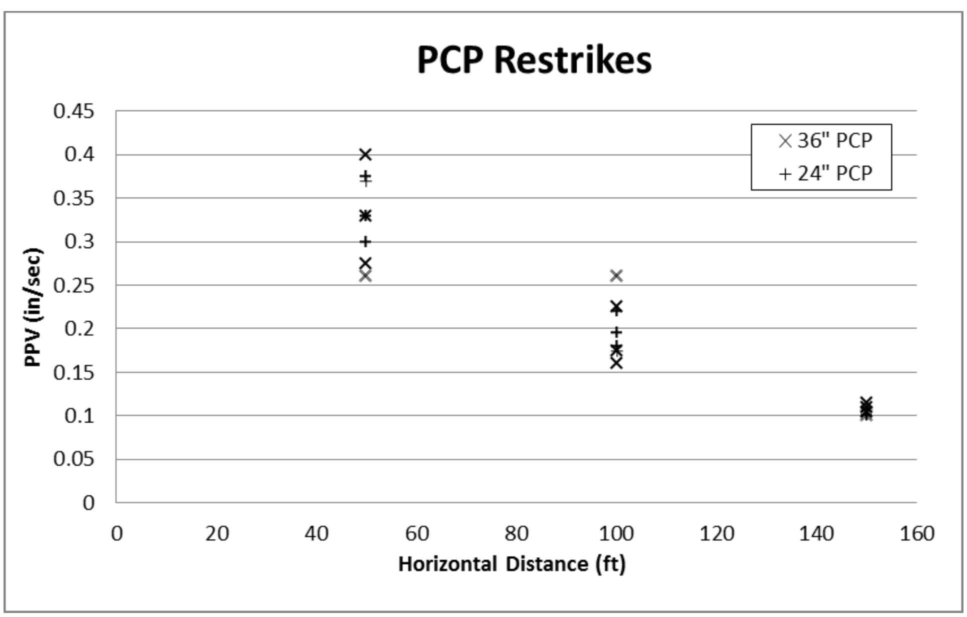

Figure 6: Data plot of restrikes on precast concrete piles (PCP)

Prediction Equation

The second major objective of this project was to develop a methodology to predict the vibration level at various distances from the pile location. Since the primary use of this research is for determining the vibration levels for piles typically used by ALDOT located at or near the project site, two prediction equations were developed. The equations are based on the maximum peak particle velocities while driving the 36-inch concrete pile and the H-piles. Both equations are based on Equation 3, as presented by Hendriks (2002), where the vibration attenuation parameter (*k*) was determined with the experimental data. Equation 6 was developed to predict vibrations for 36 inch concrete pile,

$$PPV = 0.15\left(\frac{150}{d}\right)^{1.6}, \tag{6}$$

and Equation 7 was developed to predict vibrations for the H-piles,

$$PPV = 0.23\left(\frac{50}{d}\right)^{1.6}, \tag{7}$$

where, in both equations, *PPV* is the peak particle velocity at distance (*d*) in inches per second and *d* is the distance from the pile in feet.

Figure 7 shows a plot of the experimental data and the peak particle velocities based on the prediction equation. The results indicate that the prediction equation model fit the experimental data well. However, due to the unusual increase in vibration magnitude at 150 feet for the H-piles, the prediction equation under-predicts the vibration magnitude at 150 feet. It was also noted that the soil attenuation parameter (*k*) for both equations was determined to be 1.6. This was expected since the parameter is primarily dependent on the soil properties and less dependent on the pile type or hammer energy.

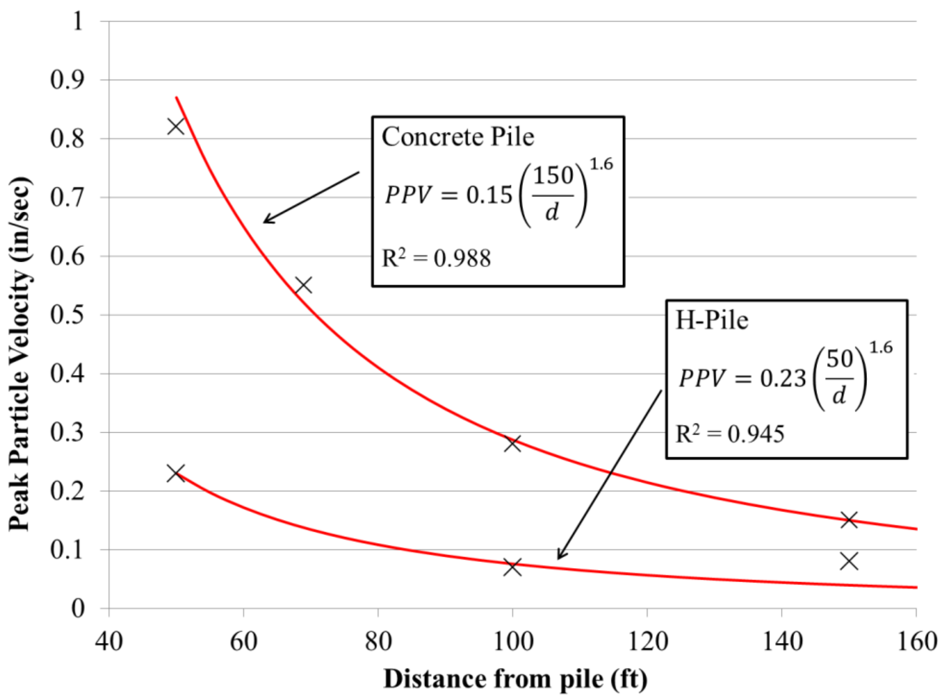

Figure 7: Peak particle velocity versus distance

CONCLUSIONS

The experimental data shows that the largest vibrations occurred during the installation of the 36-inch concrete pile, which was recorded as 0.82 inches per second. According to the research presented in Table 2 (Hendriks 2002), a vibration level of 0.82 inches per second has the potential to cause structural damage to an adjacent structure. However, this vibration was recorded at a distance of 50 feet from the pile; the vibration level at 100 feet from the pile was reduced to 0.275 inches per second. This vibration level could cause potential architectural damage to buildings constructed with plaster, but would not likely cause structural damage. At 150 feet the vibration levels were reduced to 0.15 inches per second, a level that would have little to no risk of damage to adjacent structures.

Based on the experimental data and a thorough review of the literature, it is recommend that a maximum vibration level of 0.5 inches per second for modern structures and 0.1 inches per second for potentially sensitive structures be allowed for construction activity at or near the location of the project site. These vibration levels are the allowable levels at the location of the structure. To determine if any structures should be surveyed and monitored for potential vibration damage, a survey distance of 150 feet for modern structures and 250 feet for potentially sensitive structures should be established. The monitoring distances should be measured from the source of the vibration. The ground vibration prediction equation that was developed would estimate a peak particle velocity of 0.15 inches per second at 150 feet and 0.07 inches per second at 250 feet. The survey distances are well beyond the distance where the prediction equation would estimate vibration levels of 0.5 and 0.1 inches per second and therefore would represent conservative survey distances to ensure adjacent structures are not damaged.

Recommendations for Future Research

The research presented in this report contains detailed analysis for a particular location in the state of Alabama; however, data has not been collected and analyzed for other regions of the state with differing soil conditions. A state wide research project should be initiated to determine vibration propagation and attenuation criteria for soil conditions located throughout the state. This data could be used to develop prediction equations that could be used in project planning. Additionally, the results of this research could be used to develop model vibration specifications for the state of Alabama.

In addition to the research mentioned above, it is recommended that a vibration monitoring program be developed for any large scale construction projects in urban environments. These programs could be used not only to ensure the construction activity is not damaging nearby structures, but to ensure the public that the DOT is proactive in preventing damage.

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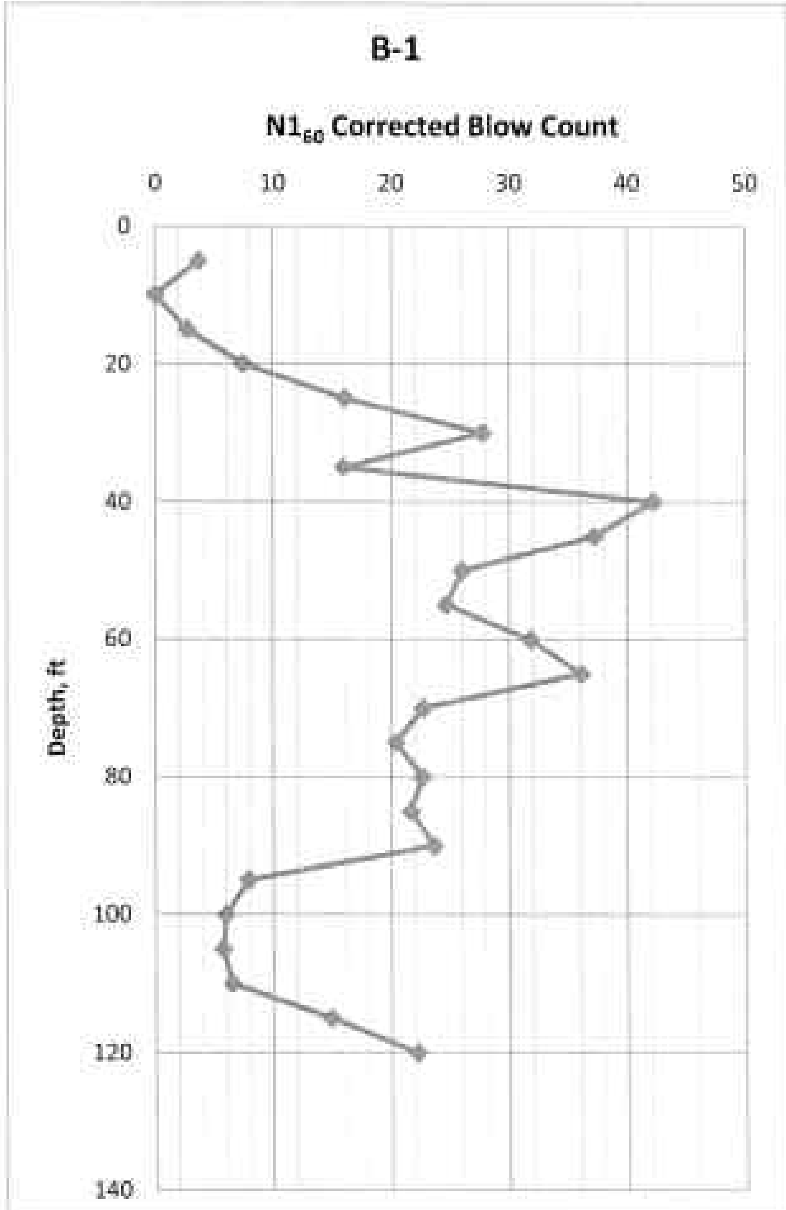
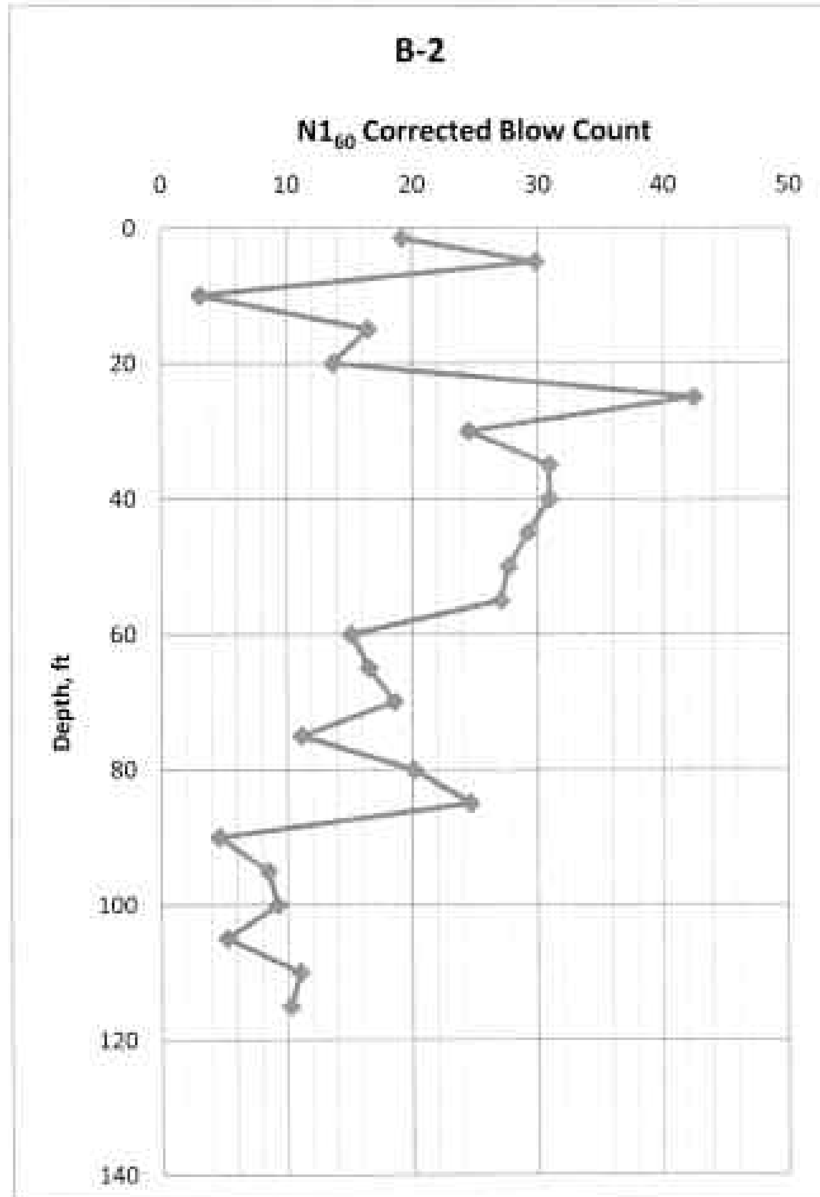
Appendix A: Soil Reports

Two soil investigations were performed at the site. The first was a Standard Penetration Test (SPT), which was performed at two locations. The first location, labeled B-1 in the documents that follow, was located at a property owned by ALDOT that is several hundred feet to the west of the project site. This location was an alternate location for testing. The second location, labeled B-2, was at the project site in the vicinity of where the test piles were installed. The SPT test was performed by an ALDOT drill crew.

The second soil investigation performed was a Seismic Cone Penetration Test (SCPT). Two locations were also investigated, both on the project site. The first test was performed at the location of the test piles and the second was located at 100 to 120 feet from the test piles. The results of both investigations are included here. The SCPT was conducted by Southern Earth Sciences.


B-1
N1$_{60}$ Corrected Blow Count
0
10
20
30
40
50
0
20
40
60
80
100
120
140
Depth, ft


B-2
N1$_{60}$ Corrected Blow Count
0
10
20
30
40
50
0
20
40
60
80
100
120
140
Depth, ft

Project# DPI-0030 (005) Division 9th

Station ________ Offset ________ Ft ________

Ground Elev. 0.0 Water Elev. in Hole ________

Depth of Strata BOR# B-1 Visual BOR Loc. ____

From	To	Consistency or Density	Approx. Moisture	Color	Basic Matl.
0.0	0.3	Asphalt			
0.3	18.5	V. Loose	Moist	Br	br. sand
18.5	23.5	Loose	Moist	Br	SAND
23.5	38.5	Med	Moist	TAN	SAND
38.5	63.5	Dense	Moist	TAN	sand
63.5	68.5	Very Dense	Moist	TAN	Sand
68.5	93.5	Dense	Moist	Tan	Sand
93.5	108.5	Stiff	Moist	Gray	Clay
108.5	118.5	HARD	Moist	Gray	Clay
118.5	120.0	DENSE	Moist	Gray	SAND

Remarks by Driller INStalled well monitor

GPS Cord. 42' 1"

LAT. WAter -10' 3" 24Hrs

LONG. ________

County Mobile Date 8-8-12

________ C/L Driller Young/Evans

Type Drill Used SE 9050 Total Hole Depth 120.0

Identification CME 550x 2.25 Hollow Stem

Other Pertinent Components	Sample No.	Penetration or Sample Elev. From	To	"N" Blow .5	1.0	1.5	"N" Value
*	1-A	3.5	5.0	1	1	1	2
W/clay *	1-B	8.5	10.0	W	O	H	WOH
*	1-C	13.5	15.0	WOH	1	1	2
	1-D	18.5	20.0	1	2	4	6
	1-E	23.5	25.0	5	5	9	14
	1-F	28.5	30.0	10	12	14	26
	1-G	33.5	35.0	9	7	9	16
W/Sand *	1-H	38.5	40.0	26	23	22	45
W/Sand *	1-I	43.5	45.5	23	23	19	42
org matl	1-J	48.5	50.0	11	14	17	31
	1-K	53.5	55.0	9	16	15	31
	1-L	58.5	60.0	18	20	22	42
*	1-M	63.5	65.0	20	23	27	50
*	1-N	68.5	70.0	14	16	17	33
	1-O	73.5	75.0	7	15	16	31

* Samples in JARS

Project# DPT-0030 (005) Division 9th

Station ____ Offset ____ Ft ____

Ground Elev. ____ Water Elev. in Hole ____

Depth of Strata BOR# B-1 Visual BOR Loc. ____

From	To	Consistency or Density	Approx. Moisture	Color	Basic Matl.

Remarks by Driller ____

GPS Cord. ____

LAT. ____

LONG. ____

County Mobile Date 8-8-12

____ C/L Driller Young/Evans

Type Drill Used SE 9050 Total Hole Depth 120.0

Identification CME 550X 2.25 Hollow stem

Other Pertinent Components	Sample No.	Penetration or Sample Elev. From	To	"N" Blow .5	1.0	1.5	"N" Value
	1-P	78.5	80.0	12	17	19	36
	1-Q	83.5	85.0	12	18	18	36
	1-R	88.5	90.0	17	22	19	41
*	1-S	93.5	95.0	3	6	8	14
	1-T	98.5	100.0	5	5	6	11
	1-U	103.5	105.0	4	5	6	11
*	1-V	108.5	110.0	3	6	7	13
*	1-W	113.5	115.0	6	15	16	31
*	1-X	118.5	120.0	13	21	27	48

1-S has 4 jar samples of Clay

Project# DPI-0030 (005) Division 9th

Station ______ Offset ______ Ft ______

Ground Elev. 0.0 Water Elev. in Hole ______

Depth of Strata BOR# B-2 Visual BOR Loc. ______

From	To	Consistency or Density	Approx. Moisture	Color	Basic Mat'l.
0.0	0.2	Topsoil	—		
0.2	3.5	Loose	Moist	Br	Sand
3.5	8.5	Med	Moist	Br	Sand
8.5	13.5	V Loose	Moist	Br	Sand
13.5	23.5	Med	Moist	Gray	Sand
23.5	28.5	Dense	Moist	Tan	Sand
28.5	33.5	Med	Moist	Tan	Sand
33.5	58.5	Dense	Moist	Tan	Sand
58.5	78.5	Med	Moist	Tan	Sand
78.5	89.5	Dense	Moist	Tan	Sand
89.5	93.5	Stiff	Moist	Gray	Clay
93.5	103.5	V Stiff	Moist	Gray	Clay
103.5	108.5	Stiff	Moist	Gray	Clay
108.5	115.0	Med	Moist	Gray	Sand

Remarks by Driller ______

GPS Cord. ______

LAT. ______

LONG. ______

County Mobile Date 8-9-12

______ C/L Driller Turner / Evans

Type Drill Used SE 9050 Total Hole Depth 115.0

Identification CME 550X 2.25 Hollow Stems

Other Pertinent Components	Sample No.	Penetration or Sample Elev. From	To	"N" Blow .5	1.0	1.5	"N" Value
*	2-A	0.0	1.5	4	4	4	8
w/ Gravel *	2-B	3.5	5.0	5	7	9	16
w/ org. mat'l *	2-C	8.5	10.0	1	1	1	2
*	2-D	13.5	15.0	2	5	7	12
	2-E	18.5	20.0	5	4	7	11
*	2-F	23.5	25.0	16	18	19	37
	2-G	28.5	30.0	10	11	12	23
*	2-H	33.5	35.0	7	15	16	31
	2-I	38.5	40.0	7	13	20	33
	2-J	43.5	45.0	9	14	19	33
w/ Sand &	2-K	48.5	50.0	7	15	18	33
org mat'l	2-L	53.5	55.0	10	16	18	34
*	2-M	58.5	60.0	10	10	10	20
	2-N	63.5	65.0	6	12	11	23
	2-O	68.5	70.0	12	17	10	27

* Jar Sample

Project# ______ Division ______

Station ______ Offset ______ Ft ______

Ground Elev. ______ Water Elev. in Hole ______

Depth of Strata BOR# B-2 Visual BOR Loc. ______

From	To	Consistency or Density	Approx. Moisture	Color	Basic Matl.

Remarks by Driller ______

GPS Cord. ______

LAT. ______

LONG. ______

County ______ Date 8-9-12

______ C/L Driller ______

Type Drill Used ______ Total Hole Depth 115.0

Identification

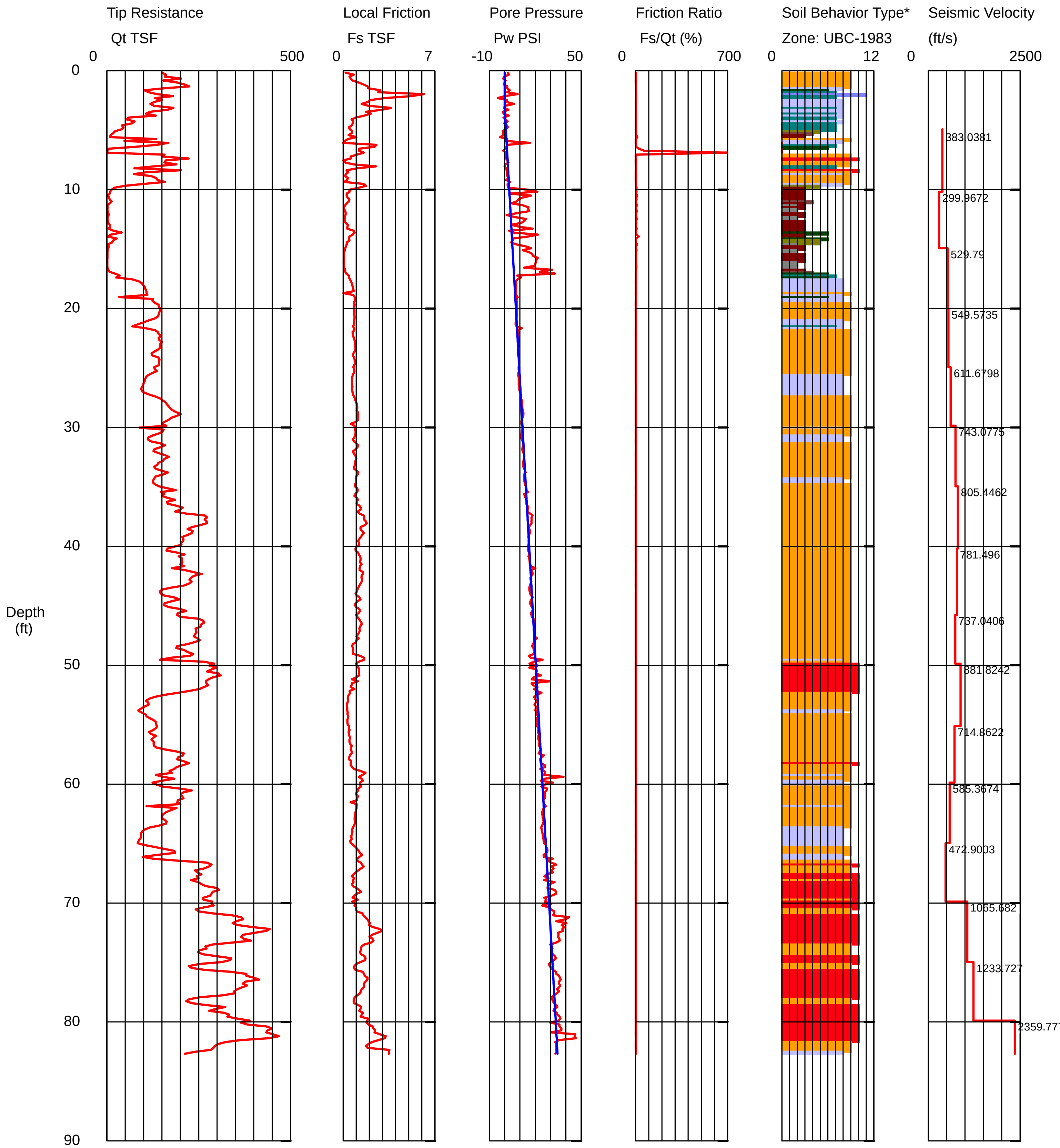
Other Pertinent Components	Sample No.	Penetration or Sample Elev. From	To	"N" Blow .5	1.0	1.5	"N" Value
	2-P	73.5	75	6	7	10	17
*	2-Q	78.5	80.0	6	11	21	32
	2-R	83.5	85.0	12	18	23	41
*	2-S	88.5	90.0	4	4	4	8
*	2-T	93.5	95.0	6	7	8	15
*	2-U	98.5	100.0	1	9	8	17
*	2-V	103.5	105.0	2	4	6	10
*	2-W	108.5	110.0	5	9	13	22
	2-X	113.5	115.0	6	9	13	22

2-T has 3 Jars
2-U has 2 Jars
2-V has 3 Jars

Southern Earth Sciences

Operator: Mike Wright
Sounding: SCPT-1
Cone Used: DDG0892

CPT Date/Time: 8/14/2013 9:08:56 AM
Location: Test Pile Evaluation
Job Number: 13-000



Maximum Depth = 82.68 feet

Depth Increment = 0.164 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

Groundwater measured at 3.1'

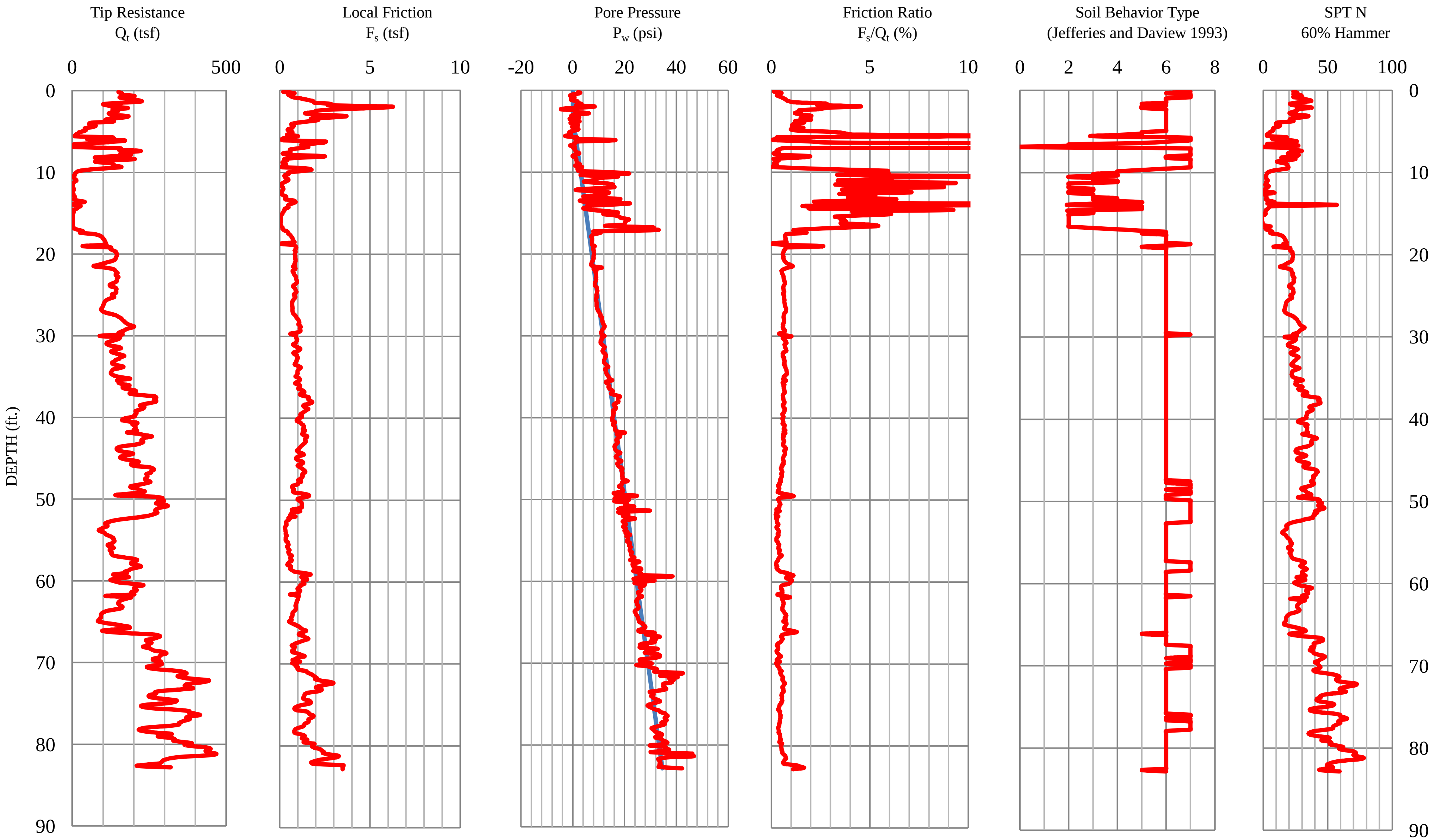
N30.68546 W88.03791

*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST LOG

SOUTHERN EARTH SCIENCES, INC.
Geotechnical, Environmental & Construction Materials Testing

Project Name:	Test Pile Evaluation	**Cone Used:**	DDG0892	**Groundwater Level:**	3.1 feet
Project No.:	13-000	**Operator:**	Mike Wright	**Elevation:**	Unknown
Sounding:	SCPT-1	**CPT Date:**	8/14/2013	**Lat/Long:**	N30.68546 W88.03791



Baseline Data:

	Q_t (tsf)	F_s (tsf)	P_w (psi)
Initial Baseline:	0	0	0
Final Baseline:	-0.602	0.002	-0.172

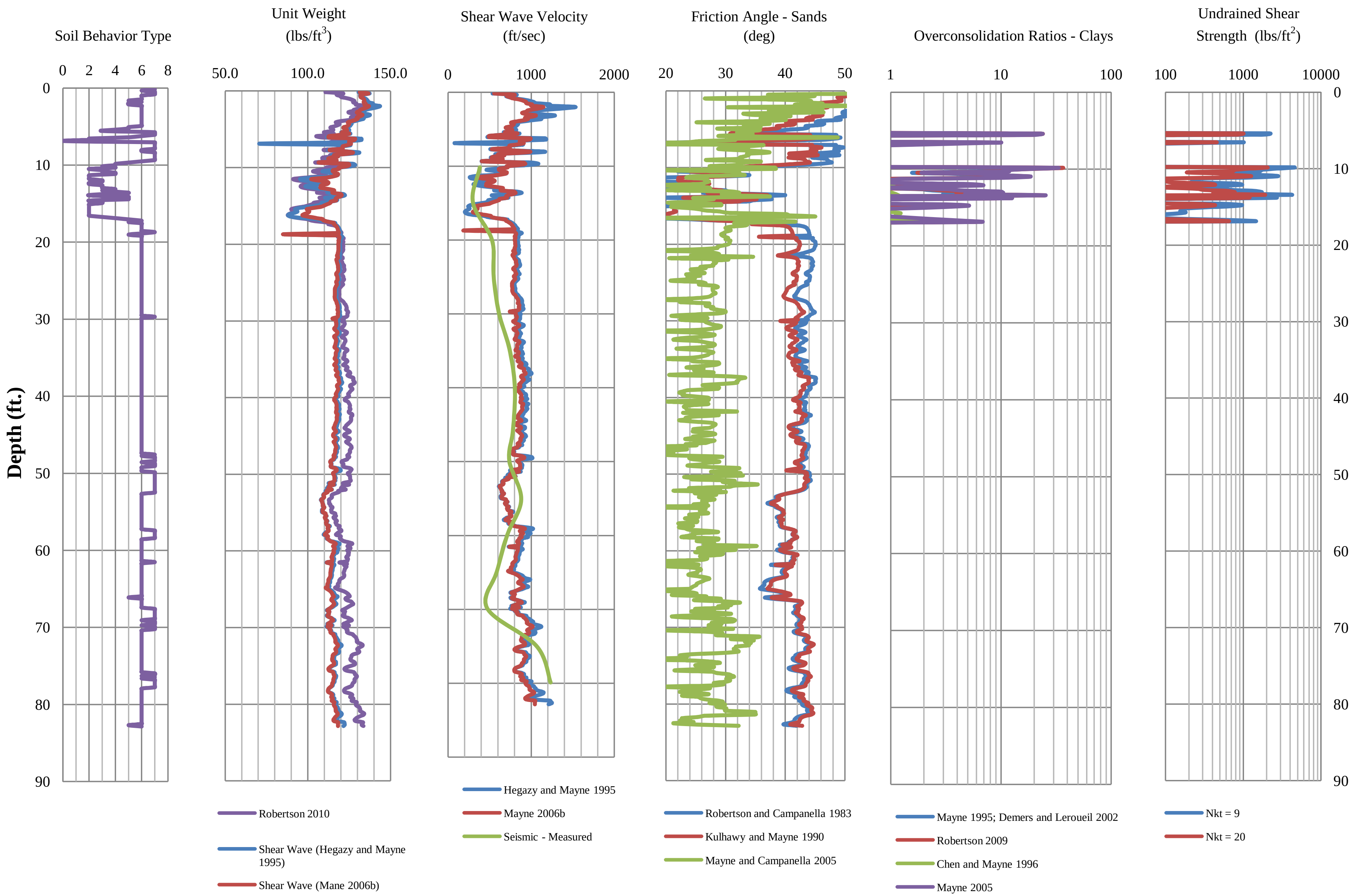
SPT N, SOIL BEHAVIOR TYPE, OR ZONE NUMBER FROM CPT CLASSIFICATION INDEX, Ic

Organic Clay Soils = 2, Clays = 3, Silt Mixtures = 4, Sand Mixtures = 5, Sands = 6, Gravelly Sands = 7

CONE PENETRATION TEST LOG

SOUTHERN EARTH SCIENCES, INC.
Geotechnical, Environmental & Construction Materials Testing

Project Name:	Test Pile Evaluation	**Cone Used:**	DDG0892	**Groundwater Level:**	3.1 feet
Project No.:	13-000	**Operator:**	Mike Wright	**Elevation:**	Unknown
Sounding:	SCPT-1	**CPT Date:**	8/14/2013	**Lat/Long:**	N30.68546 W88.03791

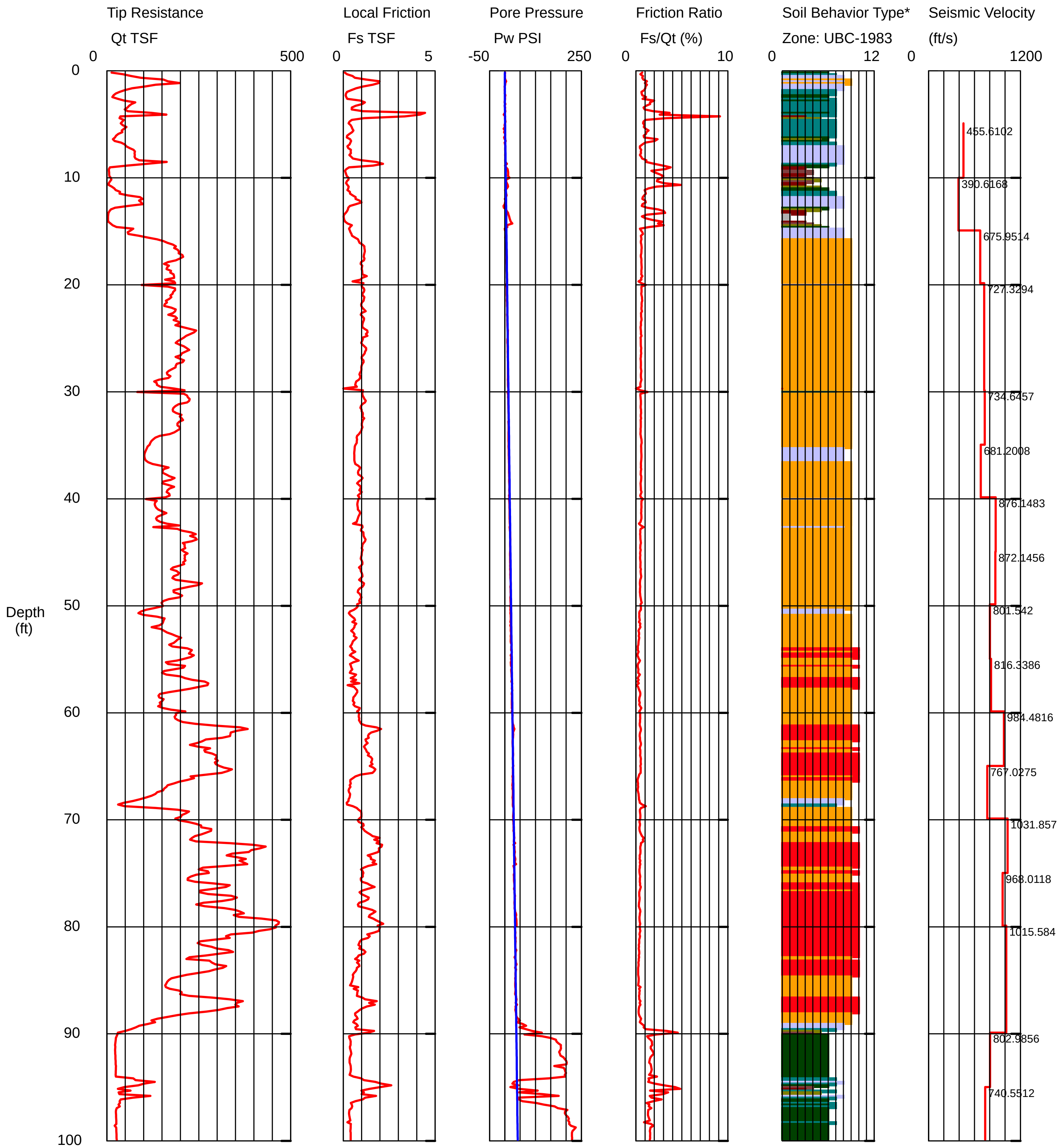


PARAMETERS ABOVE ARE BASED UPON EMPIRICAL CORRELATIONS AND SHOULD BE CONSIDERED APPROXIMATE. IT IS RECOMMENDED THAT CALCULATED PARAMETERS BE CORRELATED BY SPECIFIC LABORATORY DATA AND/OR LOCAL EXERIENCE.

Southern Earth Sciences

Operator: Mike Wright
Sounding: SCPT-2
Cone Used: DDG0892

CPT Date/Time: 8/14/2013 10:35:15 AM
Location: Test Pile Evaluation
Job Number: 13-000



Maximum Depth = 99.90 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Groundwater measured at 3.2'

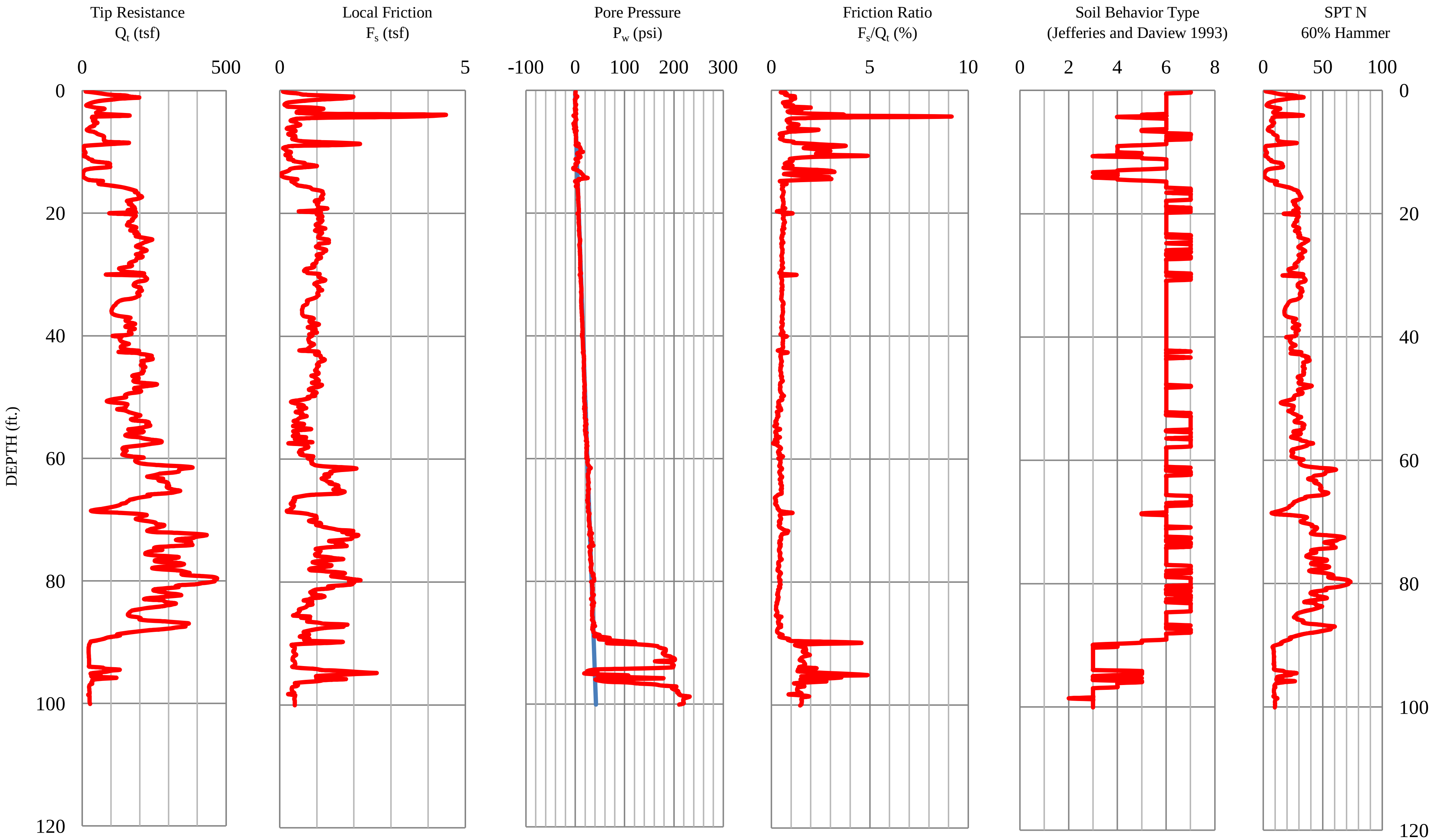
N30.68541 W88.03821

*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST LOG

SOUTHERN EARTH SCIENCES, INC.
Geotechnical, Environmental & Construction Materials Testing

Project Name:	Test Pile Evaluation	**Cone Used:**	DDG0892	**Groundwater Level:**	3.2 feet
Project No.:	13-000	**Operator:**	Mike Wright	**Elevation:**	Unknown
Sounding:	SCPT-2	**CPT Date:**	8/14/2013	**Lat/Long:**	N30.68541 W88.03821



Baseline Data:

	Q_t (tsf)	F_s (tsf)	P_w (psi)
Initial Baseline:	0	0	0
Final Baseline:	0.357	0.012	0.210

SPT N, SOIL BEHAVIOR TYPE, OR ZONE NUMBER FROM CPT CLASSIFICATION INDEX, Ic

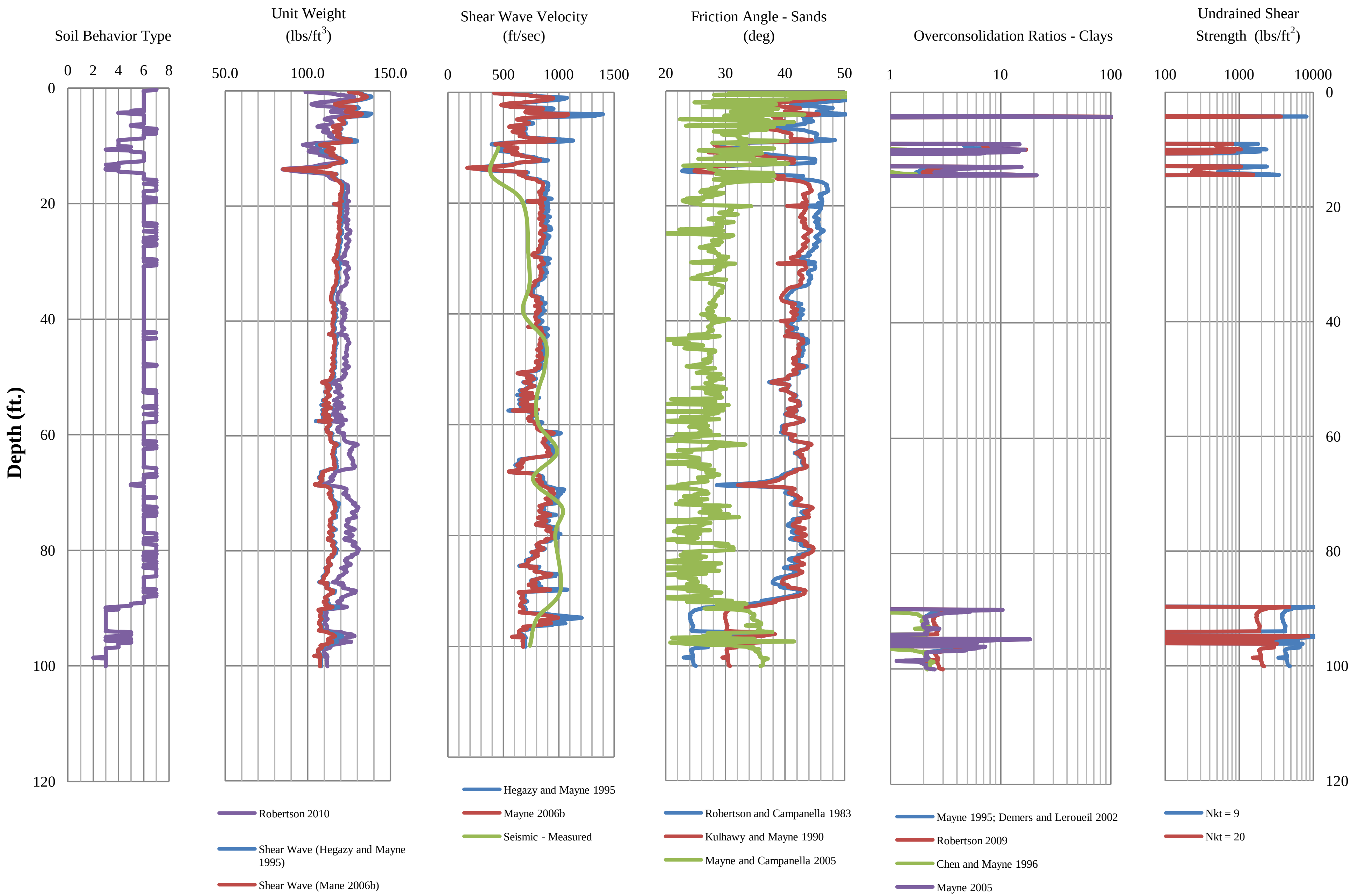
Organic Clay Soils = 2, Clays = 3, Silt Mixtures = 4, Sand Mixtures = 5, Sands = 6, Gravelly Sands = 7

CONE PENETRATION TEST LOG

SOUTHERN EARTH SCIENCES, INC.

Geotechnical, Environmental & Construction Materials Testing

Project Name:	Test Pile Evaluation	**Cone Used:**	DDG0892	**Groundwater Level:**	3.2 feet
Project No.:	13-000	**Operator:**	Mike Wright	**Elevation:**	Unknown
Sounding:	SCPT-2	**CPT Date:**	8/14/2013	**Lat/Long:**	N30.68541 W88.03821



PARAMETERS ABOVE ARE BASED UPON EMPIRICAL CORRELATIONS AND SHOULD BE CONSIDERED APPROXIMATE. IT IS RECOMMENDED THAT CALCULATED PARAMETERS BE CORRELATED BY SPECIFIC LABORATORY DATA AND/OR LOCAL EXERIENCE.

Appendix B: Pile Driving Hammer Information

	Fuel Setting #1	Fuel Setting #2	Fuel Setting #3	Fuel Setting #4
Concrete Piles used Delmag Model D-62-22 Single Acting Diesel Hammer				
36 in PCP				
Setting Usage	Down to 43 feet	43 to 45 feet	45 to 48 feet	48 feet to end Restrikes
Rated Energy	78,960 ft. lbs.	109,725 ft. lbs.	138,960 ft. lbs.	165,000 ft. lbs
24 in PCP				
Setting Usage	Down to 61 feet	61 feet to end Restrikes	N/A	N/A
Rated Energy	78,960 ft. lbs.	109,725 ft. lbs.		
Steel Piles used APE Model D30-42 Single Acting Diesel Hammer				
HP 14				
Setting Usage	N/A	N/A	Entire depth Restrikes	N/A
Rated Energy			66,977 ft. lbs.	
HP 12				
Setting Usage	N/A	Entire depth Restrikes	N/A	N/A
Rated Energy		55,070 ft. lbs		

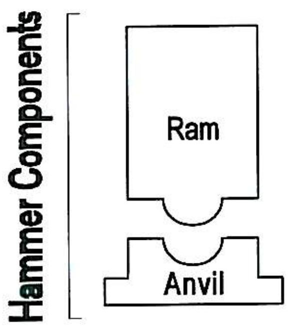
FORM C-14 **ALABAMA DEPARTMENT OF TRANSPORTATION**

Revised 08-07-95 **PILE AND DRIVING EQUIPMENT DATA FORM**

Project Number	County	Division
USA Test Pile & Vibration	Mobile	9th

Pile Driving Contractor or Subcontractor	Bridge Identification Number
Jordan Pile Driving Inc.	N/A

Details of access method to pile top for dynamic testing are: ☐ Attached ☒ Not Applicable



Hammer

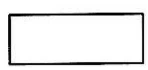
Manufacturer: Delmag Model: D-62-22

Type: S.A. Diesel Serial No.: 238

Rated Energy: 165,000 (ft.-lbs.) at 11.3 (ft.) Length of Stroke

Modifications: Adjustable Fuel Pump

Pump Setting 1	78,960 ft. lbs.
Pump Setting 2	109,725 ft. lbs.
Pump Setting 3	136,950 ft. lbs.
Pump Setting 4	165,000 ft. lbs.



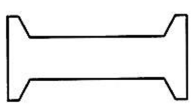
Capblock (Hammer Cushion)

Material: Aluminum & Micarta Alternating

Thickness: 6 (in.) Area: 381 (in.^{2})

Modulus of Elasticity - E : 450 KSI (P.S.I.)

Coefficient of Restitution - e : 0.8

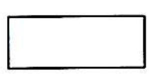


Pile Cap

Helmet ✓
Bonnet
Anvil Block
Drivehead

Weight : 10,000 (lbs.)

Note: Should include weight of striker plate.



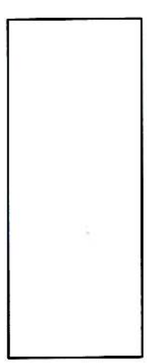
Pile Cushion

Cushion Material: Plywood

Thickness: 10 (in.) Area: 576 (in.^{2})

Modulus of Elasticity - E : 45 KSI (P.S.I.)

Coefficient of Restitution - e : 0.5



Pile

Pile Type: 36" x 36" & 24" x 24" Prestressed Concrete Test Pile

Length (in Leads): 89' & 81' (ft.)

Weight / Ft: 936 & 510 (lbs./ft.)

Wall Thickness: NA (in.) Taper: NA

Cross Sectional Area: 489 & 898 (in^2)

Design Pile Capacity: (Tons)

Description of Splice: N/A

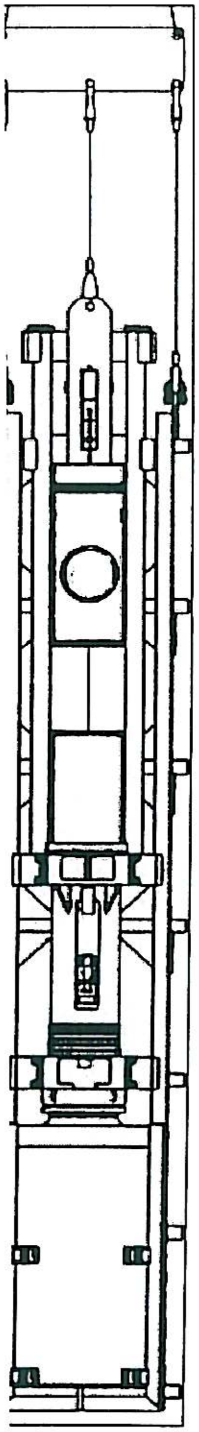
Tip Treatment Description: N/A

Note: If mandrel is used to drive this pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: Date:

Davis Daniel

Model D62-22 Diesel Hammer



Maximum obtainable energy	203,216 ft-lbs
Maximum obtainable stroke	178 inches
Pump setting 1: (minimum)	78,956 ft-lbs
Pump setting 2:	109,749 ft-lbs
Pump setting 3:	137,186 ft-lbs
Pump setting 4: (maximum)	164,250 ft-lbs
Stroke at rated energy	135 inches
Energy at rated stroke	165,000 ft-lbs
Speed (blows per minute)	36-50
Ram	13,700 lbs
Anvil	2,833 lbs
Hammer weight (includes trip device)	29,491 lbs
Typical operating (weight with drive cap)	32,963 lbs
Fuel tank (runs on diesel or bio-diesel)	25.86 gal
Oil tank	8.32 gal
Weight	1100 lbs
Diameter	25 inches
Thickness	8 inches
Type	Monocast MC 901
Diameter	25 inches
Thickness	2 inches
Elastic-modulus	285 kips per square inch
Coeff. of restrituion	0.8
Weight (fits 8 by 26 inch leads)	1,350 lbs
Diesel or Bio-diesel fuel	5.28 gal/hr
Lubrication oil	0.84 gal/hr
**Grease twice per day	
Length overall	232.6 inches
Length over cylinder extension	272.0 inches
Impact block diameter	27.9 inches
Width over bolts	32.6 inches
Hammer width overall	31.5 inches
Width for guiding- face to face	22.0 inches
Hammer center to pump guard	19.3 inches
Hammer center to bolt center	15.0 inches
Hammer depth overall	38.2 inches
Minimum clearance for leads	19.7 inches

FORM C-14 **ALABAMA DEPARTMENT OF TRANSPORTATION**

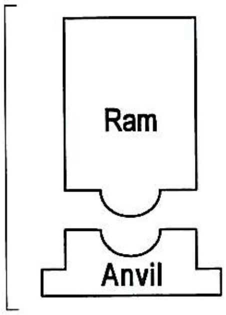
Revised 08-07-95 **PILE AND DRIVING EQUIPMENT DATA FORM**

Project Number	County	Division
USA Test Pile & Vibration	Mobile	9th

Pile Driving Contractor or Subcontractor	Bridge Identification Number
Jordan Pile Driving Inc.	N/A

Details of access method to pile top for dynamic testing are: ☐ Attached ☒ Not Applicable

Hammer Components



Hammer

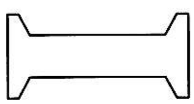
Manufacturer: APE Model: D30-42
Type: S.A. Diesel Serial No.:
Rated Energy: 74,419 (ft.-lbs.) at 11.25 (ft.) Length of Stroke

Modifications: Adjustable Fuel Pump

Pump Setting 1	37,209 ft. lbs.
Pump Setting 2	55,070 ft. lbs.
Pump Setting 3	66,977 ft. lbs.
Pump Setting 4	74,419 ft. lbs.

Capblock (Hammer Cushion)

Material: Aluminum & Micarta Alternating
Thickness: 4 (in.) Area: 398 $(in.^2)$
Modulus of Elasticity - E : 285 (P.S.I.)
Coefficient of Restitution - e : 0.8



Pile Cap

Helmet ✓
Bonnet
Anvil Block
Drivehead

Weight : 1,704 (lbs.)
Note: Should include weight of striker plate.

Pile Cushion

Cushion Material: N/A
Thickness: N/A (in.) Area: N/A $(in.^2)$
Modulus of Elasticity - E : N/A (P.S.I.)
Coefficient of Restitution - e : N/A

Pile

Pile Type: HP 12 x 53 & HP 14 x117
Length (in Leads): 70' & 106' (ft.)
Weight / Ft: 53 & 117 (lbs./ft.)
Wall Thickness: N/A (in.) Taper: NA
Cross Sectional Area: (in^2)
Design Pile Capacity: (Tons)
Description of Splice: Mechanical

Tip Treatment Description:

Note: If mandrel is used to drive this pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: Date:

Davis Daniel

APE Model D30-42 Single Acting Diesel Impact Hammer

D30-42 Finishing Dolphin Piles.

Optional Variable Throttle Control.

Drive Base Assembly.

MODEL D30-42 (3.0 metric ton ram)

SPECIFICATIONS

Stroke at maximum rated energy	135 in (343 cm)
Maximum rated energy (Setting 4)	74,419 ft-lbs (100.47 kNm)
Setting 3	66,977 ft-lbs (90.42 kNm)
Setting 2	55,070 ft-lbs (74.34 kNm)
Minimum rated energy (Setting 1)	37,209 ft-lbs (50.23 kNm)

(Variable throttle allows for infinite fuel settings)

Maximum obtainable stroke	157 in (381 cm)
Maximum obtainable energy	86,546 ft-lbs (117 kNm)
Speed (blows per minute)	34-53

WEIGHTS

Ram	6,615 lbs (3,000 kg)
Anvil	1,358 lbs (616 kg)
Anvil cross sectional area	367.94 in² (2373.80 cm²)
Hammer weight (includes trip device)	13,571 lbs (6,154 kg)
Typical operating (weight with DB26 and H-beam insert)	16,223 lbs (7,357 kg)

CAPACITIES

Fuel tank (runs on diesel or bio-diesel)	17.4 gal (65 liters)
Oil tank	5 gal (19 liters)

CONSUMPTION

Diesel or Bio-diesel fuel	2.6 gal/hr (9.84 liters/hr)
Lubrication	0.26 gal/hr (1 liters/hr)
Grease	8 to 10 pumps every 45 minutes of operation time.

STRIKER PLATE FOR DB 26

Weight	628 lbs (284 kg)
Diameter	22.5 in (57.15 cm)
Area	398 in² (2567.74 cm²)
Thickness	6 in (15.24 cm)

CUSHION MATERIAL

Type/Qty	Micarta / 2 each
Diameter-DB26	22.5 in (57.15 cm)
Thickness	1 in (25.4 mm)

Type/Qty	Aluminum / 3 each
Thickness	1/2 in (12.7 mm)
Diameter	22.5 in (57.15 cm)
Total Combined Thickness	3.5 in (8.89 cm)
Area	398 in² (2567.74 cm²)
Elastic-modulus	285 ksi (1,965 mpa)
Coeff. of restitution	0.8

DRIVE CAP

DB 26:	1,076 lbs (488 kg)

INSERT WEIGHT

H-Beam insert for 12" (305 mm) and 14" (355 mm):	948 lbs (430 kg)
Large pipe insert for sizes 12" to 24" diameter:	1,830 lbs (830 kg)

MINIMUM BOX LEAD SIZE/OPERATING LENGTH

Minimum box leader size	8 in x 26 in (20.32 cm x 66 cm)
Operating length as described above	354 in (900 cm)

AMERICAN PILEDRIVING EQUIPMENT, INC. APE

Corporate Offices
7032 South 196th
Kent, Washington 98032 USA
(800) 248-8498 & (253) 872-0141
(253) 872-8710 Fax

Visit our WEB site:
www.apevibro.com
e-mail: ape@apevibro.com

Note: All specifications are subject to change without notice 08/20/2012

Appendix E - July 28, 2016 Presentation

02 West Side Foundation Study

General Presentation Outline –

West Side Foundation Study

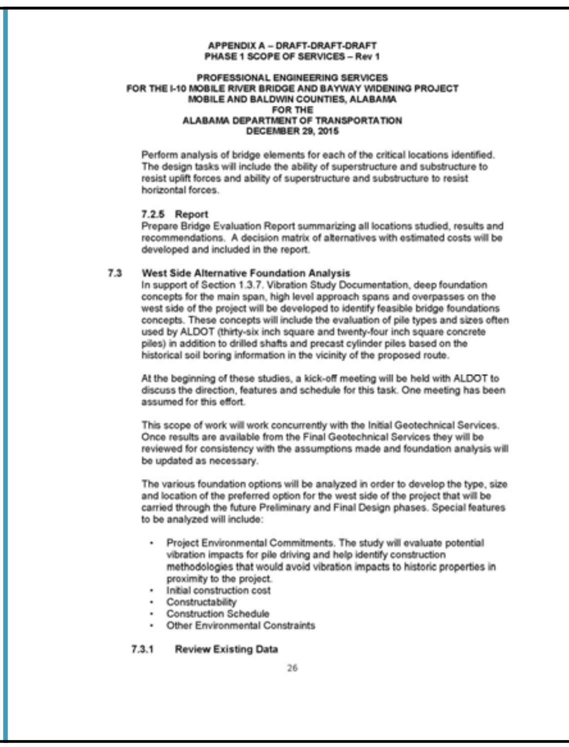
1. Purpose of Study
2. Scope of Work
3. Foundation Types Studied
4. Design Assumptions
5. Design Approach
6. Design Concepts
7. Constructibility
8. Construction Costs
9. Vibration Due to Pile Driving

1. Purpose of Study

- Per DEIS, *"ALDOT will conduct a study to evaluate potential vibration impacts for pile driving and to help identify construction methodologies that would avoid vibration impacts to historic properties in proximity to the project"*

2. Scope of Work

- Cable-Stayed Concepts over the Navigational Channel
- 4-Lane Bridges
- Various Pylon and Cable-Stayed Configurations



APPENDIX A – DRAFT-DRAFT-DRAFT
PHASE 1 SCOPE OF SERVICES – Rev 1

PROFESSIONAL ENGINEERING SERVICES
FOR THE I-10 MOBILE RIVER BRIDGE AND BAYWAY WIDENING PROJECT
MOBILE AND BALDWIN COUNTIES, ALABAMA
FOR THE
ALABAMA DEPARTMENT OF TRANSPORTATION
DECEMBER 29, 2015

Perform analysis of bridge elements for each of the critical locations identified. The design tasks will include the ability of superstructure and substructure to resist uplift forces and ability of superstructure and substructure to resist horizontal forces.

7.2.5 Report
Prepare Bridge Evaluation Report summarizing all locations studied, results and recommendations. A decision matrix of alternatives with estimated costs will be developed and included in the report.

7.3 West Side Alternative Foundation Analysis
In support of Section 1.3.7. Vibration Study Documentation, deep foundation concepts for the main span, high level approach spans and overpasses on the west side of the project will be developed to identify feasible bridge foundations concepts. These concepts will include the evaluation of pile types and sizes often used by ALDOT (thirty-six inch square and twenty-four inch square concrete piles) in addition to drilled shafts and precast cylinder piles based on the historical soil boring information in the vicinity of the proposed route.

At the beginning of these studies, a kick-off meeting will be held with ALDOT to discuss the direction, features and schedule for this task. One meeting has been assumed for this effort.

This scope of work will work concurrently with the Initial Geotechnical Services. Once results are available from the Final Geotechnical Services they will be reviewed for consistency with the assumptions made and foundation analysis will be updated as necessary.

The various foundation options will be analyzed in order to develop the type, size and location of the preferred option for the west side of the project that will be carried through the future Preliminary and Final Design phases. Special features to be analyzed will include:

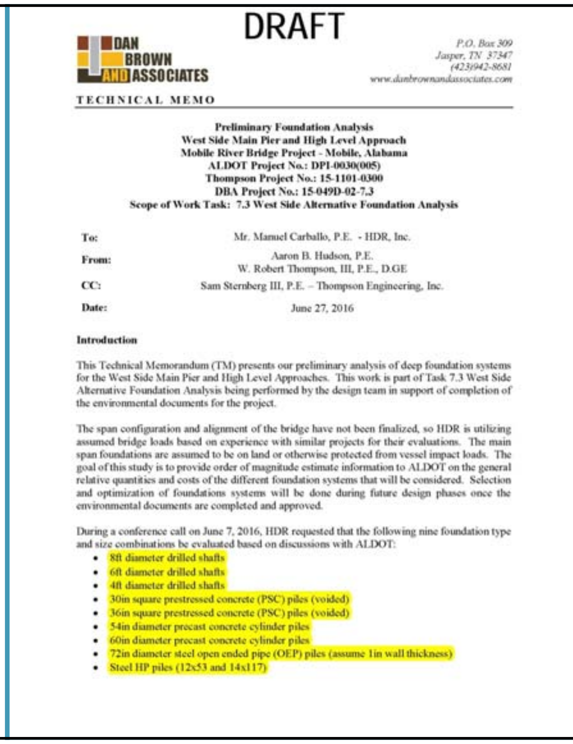
- Project Environmental Commitments. The study will evaluate potential vibration impacts for pile driving and help identify construction methodologies that would avoid vibration impacts to historic properties in proximity to the project.
- Initial construction cost
- Constructability
- Construction Schedule
- Other Environmental Constraints

7.3.1 Review Existing Data

26

3. Foundation Types Studied

- 4ft., 6ft., and 8ft. Drilled Shafts
- 30in. and 36in. Square PSC
- 54in. and 60in. Diameter Precast Cylinder Piles
- 72in Diameter Steel Open Ended Pipe Pile
- 12x53 and 14x117 Steel HP Piles



DRAFT

DAN BROWN AND ASSOCIATES

P.O. Box 309
Jasper, TN 37347
(423)942-8681
www.danbrownandassociates.com

TECHNICAL MEMO

Preliminary Foundation Analysis
West Side Main Pier and High Level Approach
Mobile River Bridge Project - Mobile, Alabama
ALDOT Project No.: DPI-0030(005)
Thompson Project No.: 15-1101-0300
DBA Project No.: 15-049D-02-7.3
Scope of Work Task: 7.3 West Side Alternative Foundation Analysis

To: Mr. Manuel Carballo, P.E. - HDR, Inc.
From: Aaron B. Hudson, P.E.; W. Robert Thompson, III, P.E., D.GE
CC: Sam Sternberg III, P.E. – Thompson Engineering, Inc.
Date: June 27, 2016

Introduction

This Technical Memorandum (TM) presents our preliminary analysis of deep foundation systems for the West Side Main Pier and High Level Approaches. This work is part of Task 7.3 West Side Alternative Foundation Analysis being performed by the design team in support of completion of the environmental documents for the project.

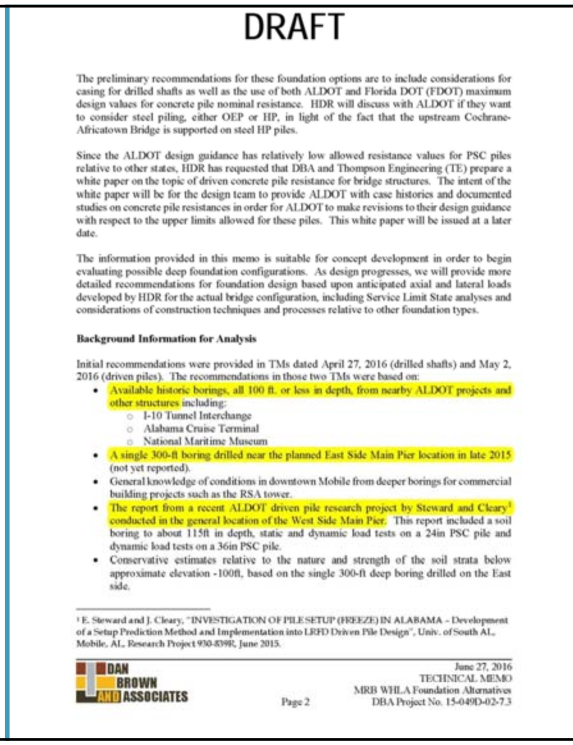
The span configuration and alignment of the bridge have not been finalized, so HDR is utilizing assumed bridge loads based on experience with similar projects for their evaluations. The main span foundations are assumed to be on land or otherwise protected from vessel impact loads. The goal of this study is to provide order of magnitude estimate information to ALDOT on the general relative quantities and costs of the different foundation systems that will be considered. Selection and optimization of foundations systems will be done during future design phases once the environmental documents are completed and approved.

During a conference call on June 7, 2016, HDR requested that the following nine foundation type and size combinations be evaluated based on discussions with ALDOT:

- 8ft diameter drilled shafts
- 6ft diameter drilled shafts
- 4ft diameter drilled shafts
- 30in square prestressed concrete (PSC) piles (voided)
- 36in square prestressed concrete (PSC) piles (voided)
- 54in diameter precast concrete cylinder piles
- 60in diameter precast concrete cylinder piles
- 72in diameter steel open ended pipe (OEP) piles (assume 1in wall thickness)
- Steel HP piles (12x53 and 14x117)

3. Foundation Types Studied

- Available Historic Borings (100ft. Or Less)
- A Single 300ft. Boring near East Tower
- ALDOT Report “Investigation of Pile Setup (Freeze) in Alabama”



DRAFT

The preliminary recommendations for these foundation options are to include considerations for casing for drilled shafts as well as the use of both ALDOT and Florida DOT (FDOT) maximum design values for concrete pile nominal resistance. HDR will discuss with ALDOT if they want to consider steel piling, either OEP or HP, in light of the fact that the upstream Cochrane-Africatown Bridge is supported on steel HP piles.

Since the ALDOT design guidance has relatively low allowed resistance values for PSC piles relative to other states, HDR has requested that DBA and Thompson Engineering (TE) prepare a white paper on the topic of driven concrete pile resistance for bridge structures. The intent of the white paper will be for the design team to provide ALDOT with case histories and documented studies on concrete pile resistances in order for ALDOT to make revisions to their design guidance with respect to the upper limits allowed for these piles. This white paper will be issued at a later date.

The information provided in this memo is suitable for concept development in order to begin evaluating possible deep foundation configurations. As design progresses, we will provide more detailed recommendations for foundation design based upon anticipated axial and lateral loads developed by HDR for the actual bridge configuration, including Service Limit State analyses and considerations of construction techniques and processes relative to other foundation types.

Background Information for Analysis

Initial recommendations were provided in TMs dated April 27, 2016 (drilled shafts) and May 2, 2016 (driven piles). The recommendations in those two TMs were based on:

- Available historic borings, all 100 ft. or less in depth, from nearby ALDOT projects and other structures including:
    - I-10 Tunnel Interchange
    - Alabama Cruise Terminal
    - National Maritime Museum
- A single 300-ft boring drilled near the planned East Side Main Pier location in late 2015 (not yet reported).
- General knowledge of conditions in downtown Mobile from deeper borings for commercial building projects such as the RSA tower.
- The report from a recent ALDOT driven pile research project by Steward and Cleary[1] conducted in the general location of the West Side Main Pier. This report included a soil boring to about 115ft in depth, static and dynamic load tests on a 24in PSC pile and dynamic load tests on a 36in PSC pile.
- Conservative estimates relative to the nature and strength of the soil strata below approximate elevation -100ft, based on the single 300-ft deep boring drilled on the East side.

[1] E. Steward and J. Cleary, "INVESTIGATION OF PILE SETUP (FREEZE) IN ALABAMA – Development of a Setup Prediction Method and Implementation into LRFD Driven Pile Design", Univ. of South AL, Mobile, AL, Research Project 930-839R, June 2015.

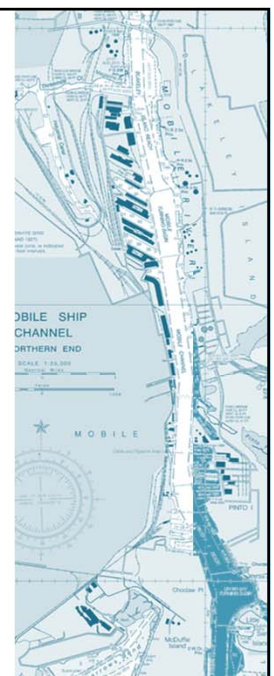
DAN BROWN AND ASSOCIATES — Page 2 — June 27, 2016, TECHNICAL MEMO, MRB WHLA Foundation Alternatives, DBA Project No. 15-049D-02-7.3

4. Design Assumptions

- Estimated loads for each main span bridge type at towers and anchor piers.
- Capacities based on limited borings and historical data.
- No uplift Allowed
- ALDOT SDM limits the axial capacities for precast piles

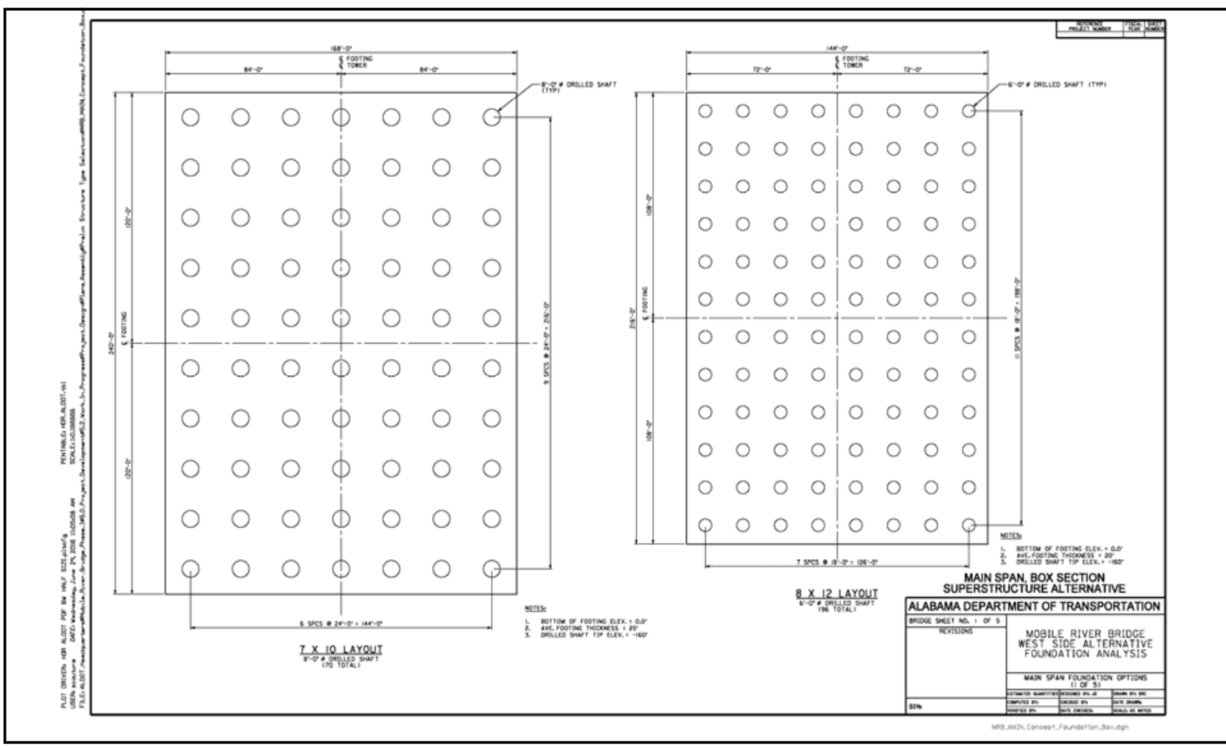
5. Design Approach

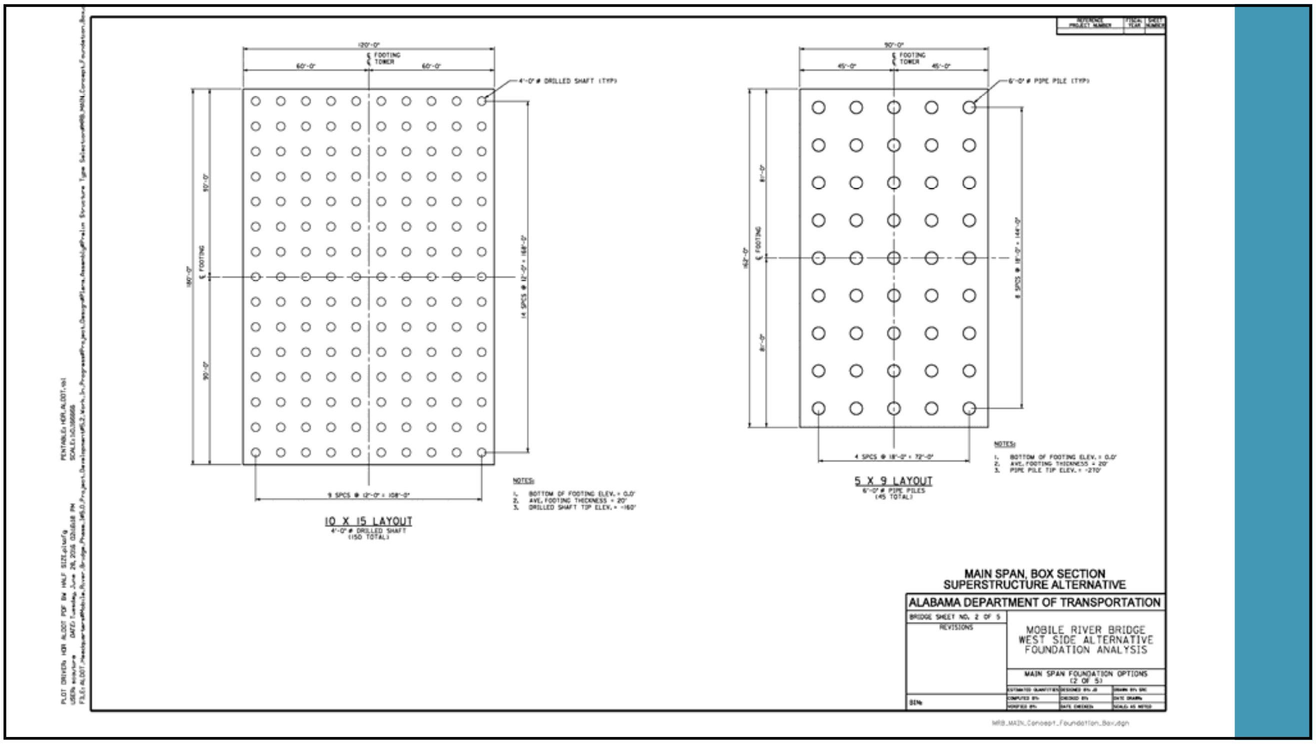
- For Main Span and High-Level Approach Spans Develop Foundation concepts for the Heaviest and Lightest Superstructure Type
- Prepare preliminary quantity take-offs for each foundation Concept and assign unit Costs to develop Estimated Construction Costs

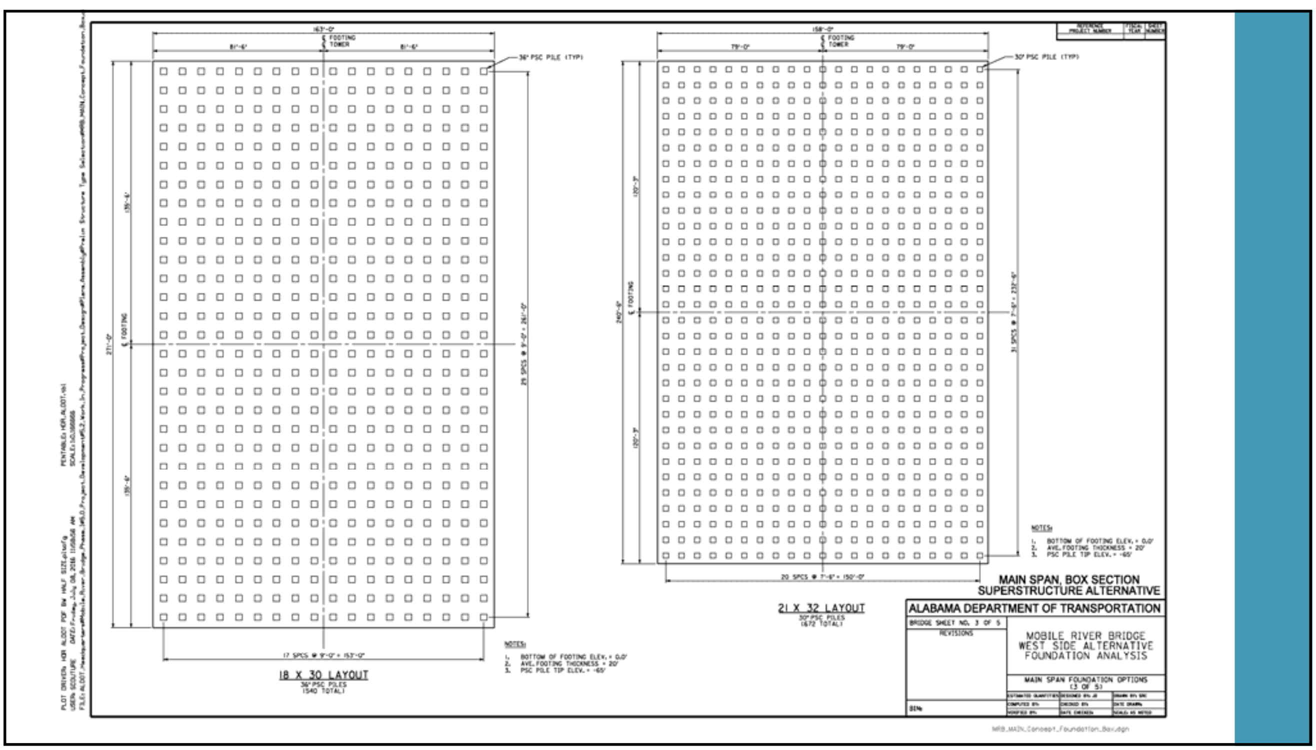


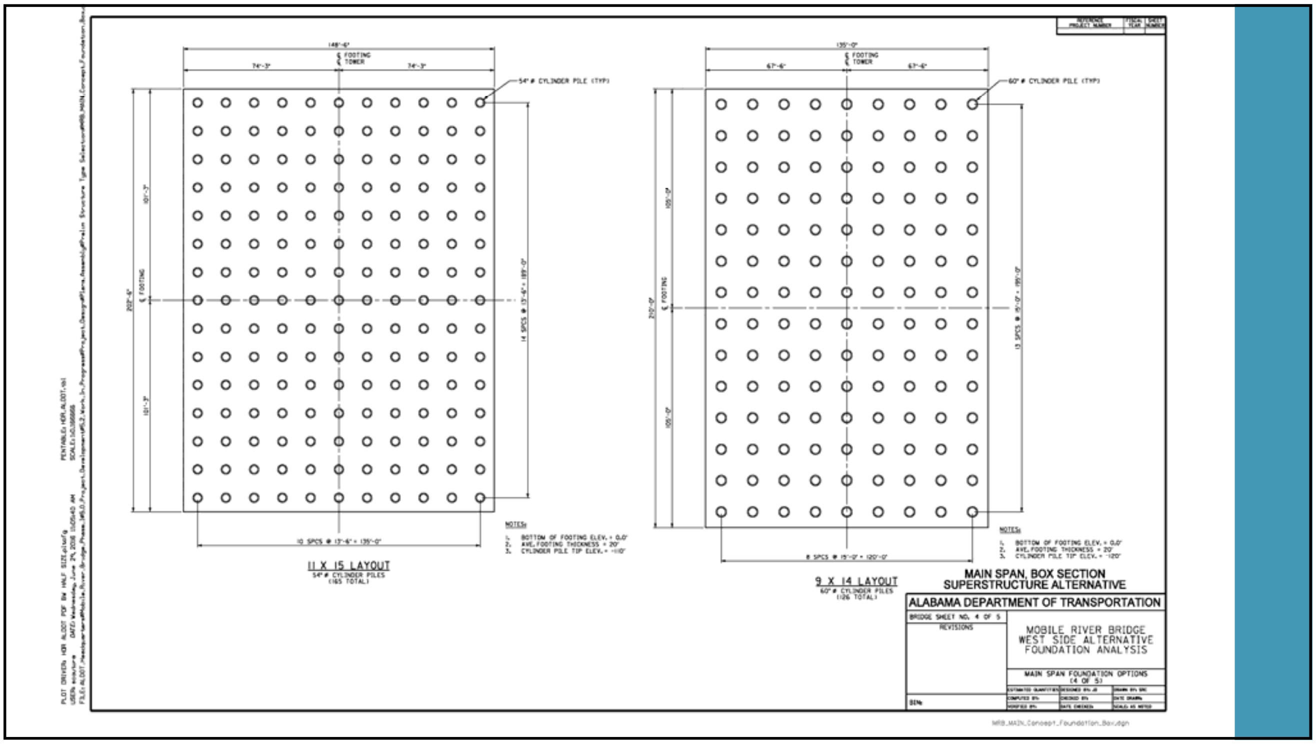
5. Design Concepts

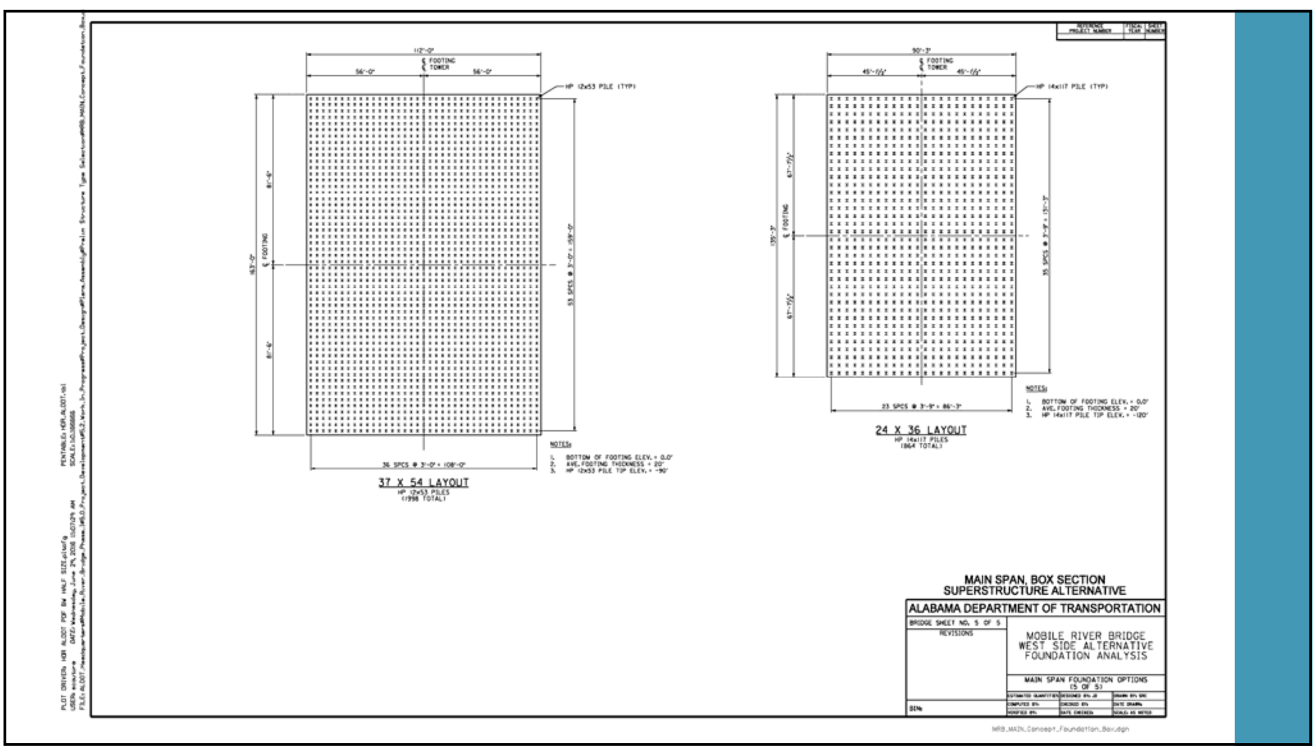
- Main Span Concrete Superstructure




10 X 15 LAYOUT
4'-0" Ø DRILLED SHAFT
(150 TOTAL)
5 X 9 LAYOUT
6'-0" Ø PIPE PILES
(45 TOTAL)
NOTES:
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 2 OF 5
REVISIONS
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

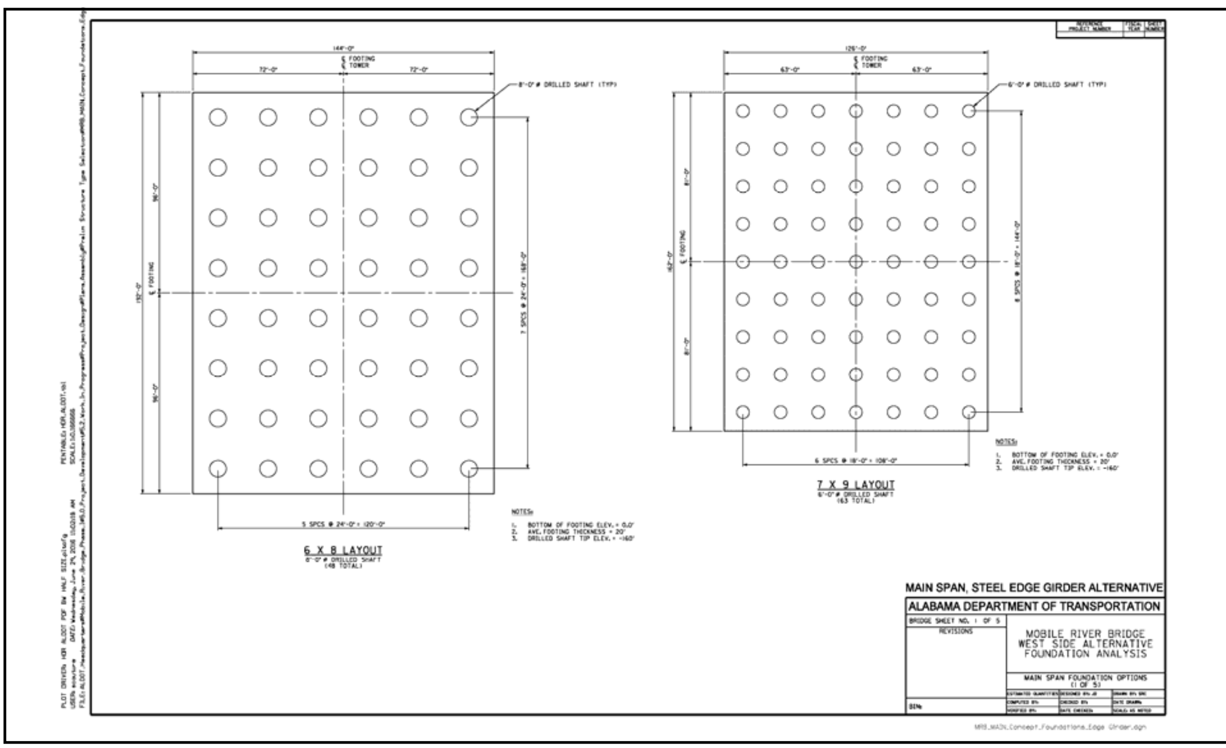

18 X 30 LAYOUT
36" PSC PILES
(540 TOTAL)
21 X 32 LAYOUT
30" PSC PILES
(672 TOTAL)
NOTES:
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 3 OF 5
REVISIONS
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)

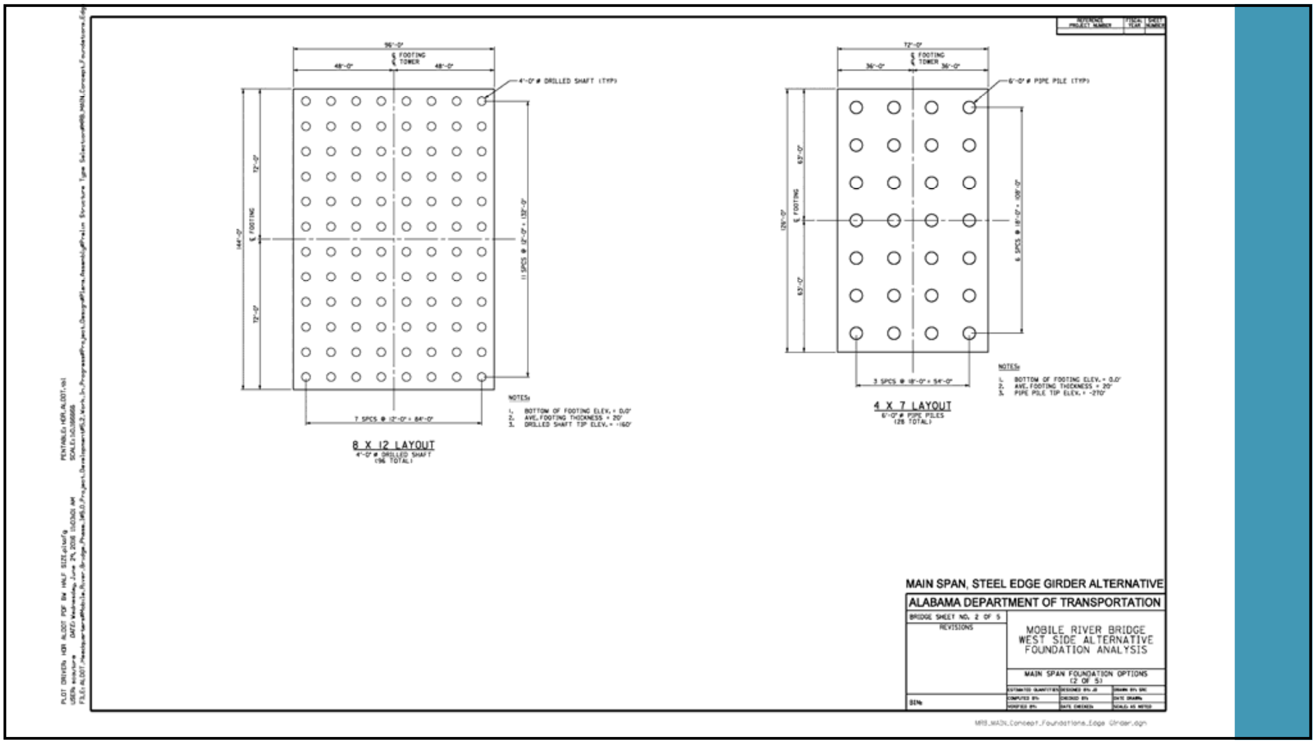

11 X 15 LAYOUT
54" ø CYLINDER PILES
(165 TOTAL)
10 SPCS @ 13'-6" = 135'-0"
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -110'
9 X 14 LAYOUT
60" ø CYLINDER PILES
(126 TOTAL)
8 SPCS @ 15'-0" = 120'-0"
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -120'
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 4 OF 5
REVISIONS
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)

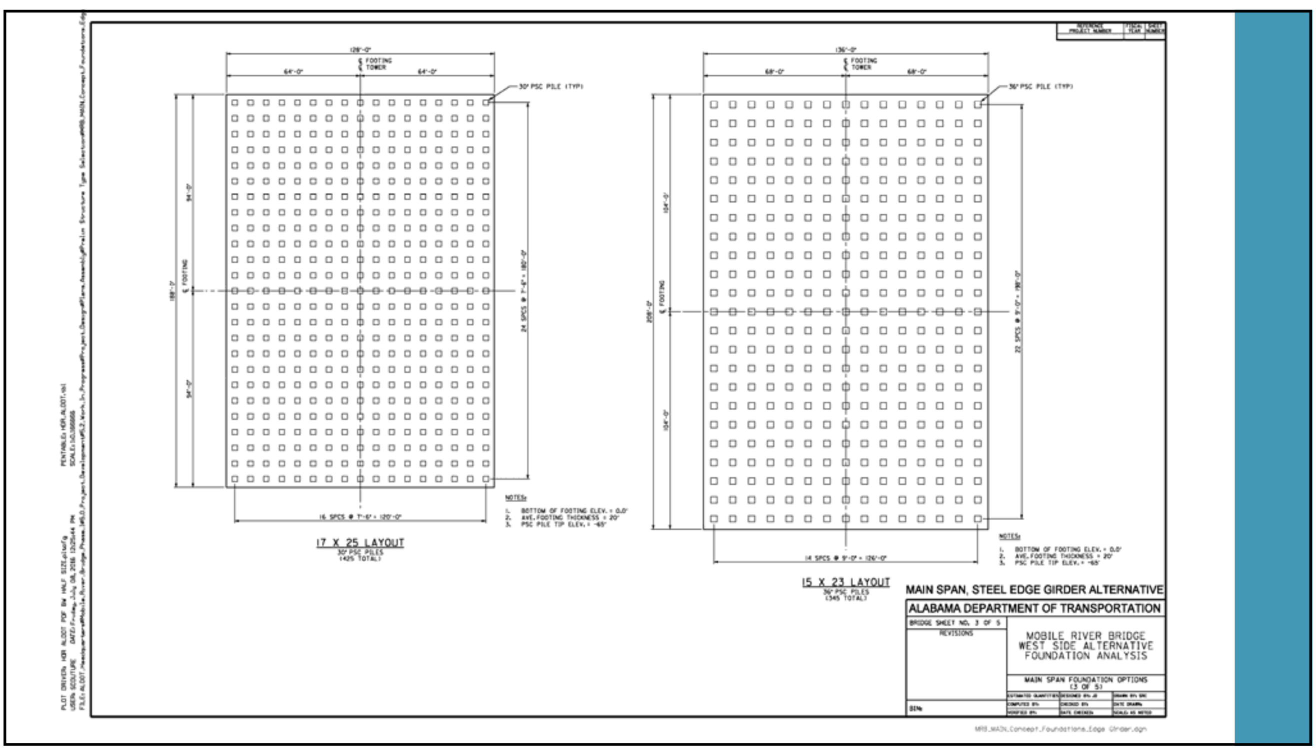

37 X 54 LAYOUT
HP 12x53 PILES
(1998 TOTAL)
36 SPCS @ 3'-0" = 108'-0"
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 12x53 PILE TIP ELEV. = -90'
24 X 36 LAYOUT
HP 14x117 PILES
(864 TOTAL)
23 SPCS @ 3'-9" = 86'-3"
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 14x117 PILE TIP ELEV. = -120'
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 5 OF 5
REVISIONS
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(5 OF 5)

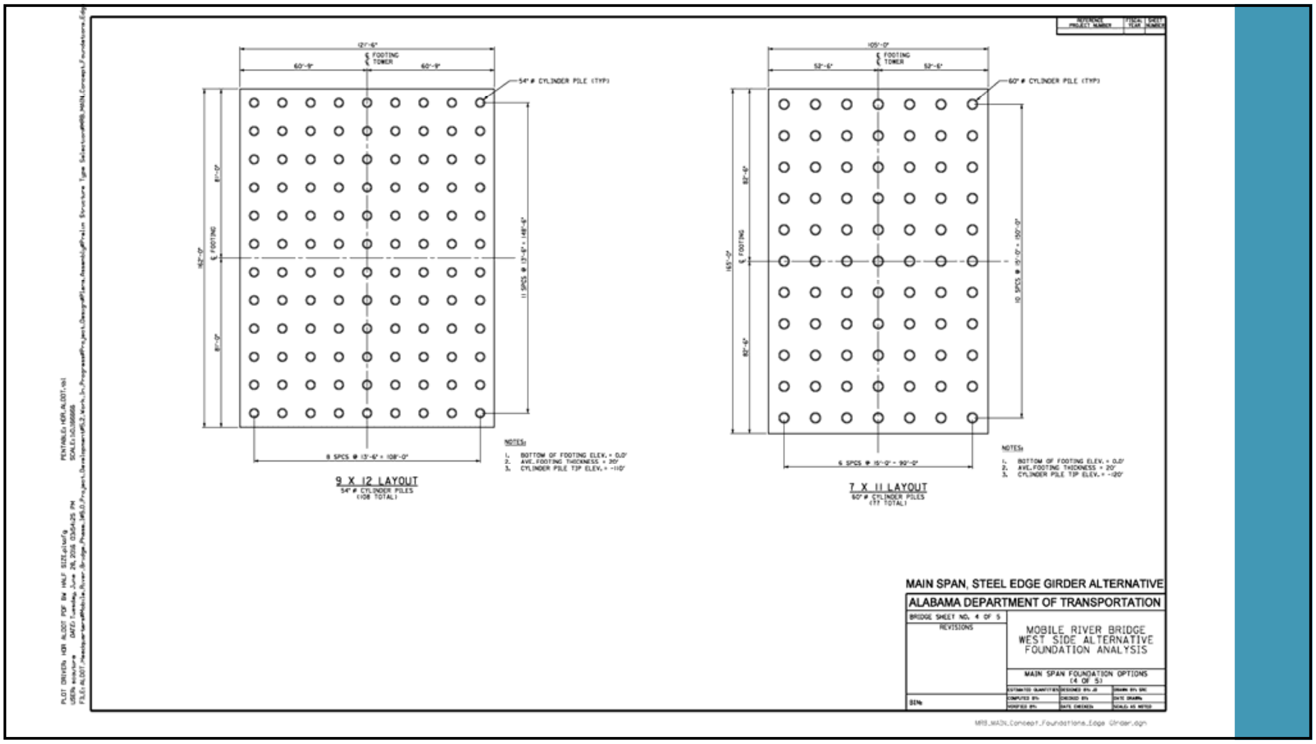
5. Design Concepts

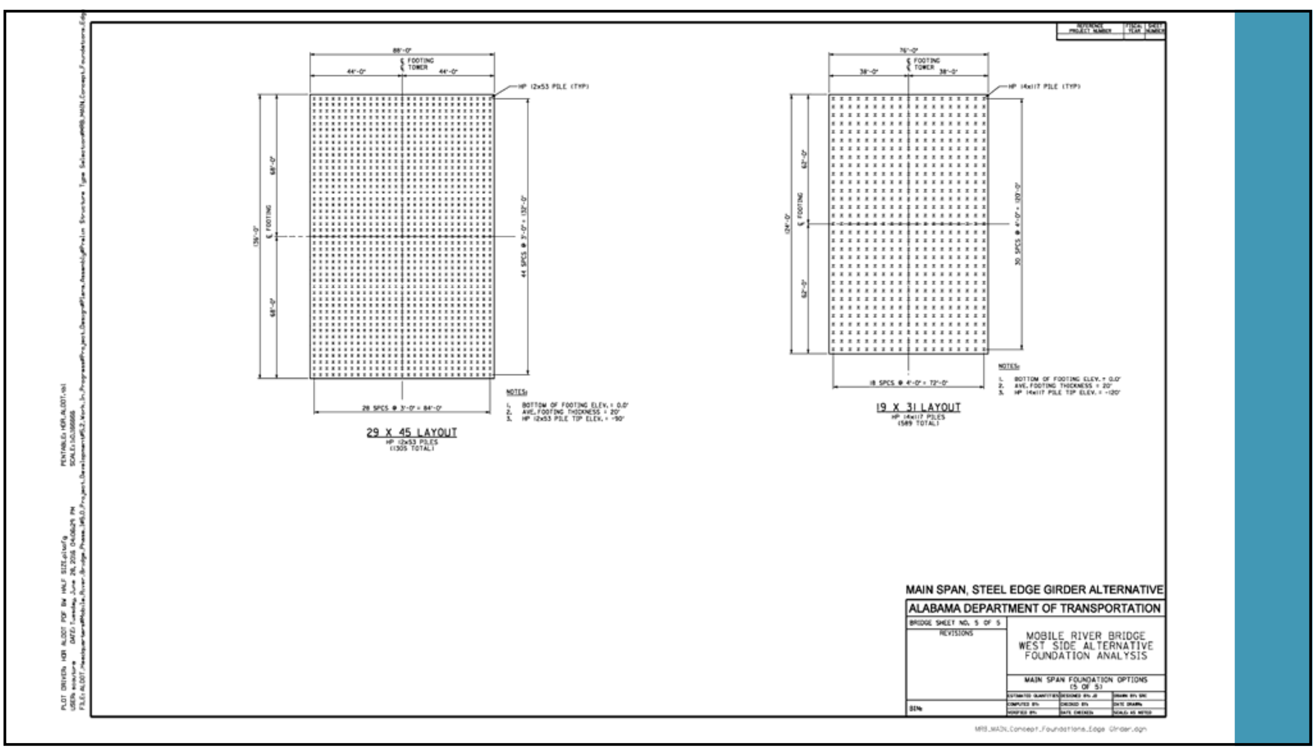
- Main Span Steel Superstructure




8 X 12 LAYOUT
4'-0" ø DRILLED SHAFT
(96 TOTAL)
4 X 7 LAYOUT
6'-0" ø PIPE PILES
(28 TOTAL)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)


17 X 25 LAYOUT
30" PSC PILES
(425 TOTAL)
15 X 23 LAYOUT
36" PSC PILES
(345 TOTAL)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)


9 X 12 LAYOUT
54" ø CYLINDER PILES
(108 TOTAL)
7 X 11 LAYOUT
60" ø CYLINDER PILES
(77 TOTAL)
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -110'
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. CYLINDER PILE TIP ELEV. = -120'
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 4 OF 5
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)


29 X 45 LAYOUT
HP 12x53 PILES
(1305 TOTAL)
19 X 31 LAYOUT
HP 14x117 PILES
(589 TOTAL)
NOTES:
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 12x53 PILE TIP ELEV. = -90'
1. BOTTOM OF FOOTING ELEV. = 0.0'
2. AVE. FOOTING THICKNESS = 20'
3. HP 14x117 PILE TIP ELEV. = -120'
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
BRIDGE SHEET NO. 5 OF 5
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(5 OF 5)

7. Constructibility

Drilled Shafts

- PROS:
  - Reduced Vibration Impacts
  - Smaller footprint of footing has lower potential to impact existing utilities and disturbed area
  - Savings on schedule due to fewer number of shafts
  - Noise-level is less than pile driving, potential to be able to work night shifts
  - 8ft. And 6ft. Shafts - Large pool of experienced drilling contractors, though not necessarily local
- CONS:
  - Limits number of potential local subcontractors
  - Re-handling of spoil removals either by thousands of truckloads of material through the city or by muck barges
  - Potential disturbance and removal of hazardous waste within the spoils
  - Potential for anomalies/voids within the drilled shaft concrete

7. Constructibility

6' Steel Pipe Piles

- PROS:
  - Provides one of the smallest disturbed area footprints, in turn there is less potential impact on existing utilities
  - Since non-displacement pile, densification and heave should be limited, resulting in less variation of pile lengths (will be driven to set tip)
- CONS:
  - Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends
  - Specialty equipment required to install these large diameter piles is not typically owned by local contractors
  - Large lead time required for procurement of steel pipe
  - Limited number of material suppliers for this diameter

7. Constructibility

Precast Square Piles

- PROS:
  - Material is readily available, minimal lead time
- CONS:
  - Risk of Vibration Impact to Adjacent Properties
  - Additional effort required to re-drive piles within a confined cofferdam due to pile heave or densification of soil within footing due to displacement from driving.
  - Large footprint of footing has higher potential to impact existing utilities and disturbed area
  - Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends
  - Risk of piles breaking or damaged during shipping and handling
  - Risk of pile damage due to improper driving or poor hammer performance

7. Constructibility

Cylinder Piles

- PROS:
  - Material is locally available, minimal lead time
- CONS:
  - Risk of Vibration Impact to Adjacent Properties
  - Limited number of suppliers, pricing controlled by the precaster not the contractor
  - Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends
  - Risk of piles breaking or damaged during shipping and handling
  - Risk of pile damage due to improper driving or poor hammer performance

7. Constructibility

Steel HP Piles

- PROS:
  - Readily Available
  - Installed easily using conventional equipment owned by local contractors
  - Non-Displacement, no spoils to remove
- CONS:
  - Risk of Vibration Impact to Adjacent Properties
  - Negative schedule impact due to noise levels requiring work only during certain hours of the day and/or weekends
  - Not recommended in Corrosive Environments (used on Cochrane Bridge)

8. Construction Costs – Main Span Towers

Foundation Type	Steel Superstructure	Concrete Superstructure
8’ Dia. Drilled Shafts	$ 22.7M	$ 32.9M
6’ Dia. Drilled Shafts	$ 20.3M	$ 30.9M
4’ Dia. Drilled Shafts	$ 18.9M	$ 29,4M
6’ Dia. Steel Pipe Piles	$ 11.0M	$ 17.5M
30” PSC	$ 12.2M	$ 19.0M
36” PSC	$ 13.5M	$ 20.7M
54” Concrete Cylinder Piles	$ 10.3M	$ 15.6M
60” Concrete Cylinder Piles	$ 9.1M	$ 14.7M
HP 12x53 Piles	$ 10.9M	$ 16.5M
HP 14x117 Piles	$ 8.5M	$ 11.8M

8. Construction Costs – Anchor Piers

Foundation Type	Steel Superstructure	Concrete Superstructure
8' Dia. Drilled Shafts	$ 5.5M	$ 6.9M
6' Dia. Drilled Shafts	$ 6.3M	$ 7.6M
4' Dia. Drilled Shafts	$ 6.1M	$ 7.4M
6' Dia. Steel Pipe Piles	$ 5.3M	$ 5.2M
30" PSC	$ 3.7M	$ 4.1M
36" PSC	$ 3.6M	$ 3.9M
54" Concrete Cylinder Piles	$ 3.3M	$ 3.4M
60" Concrete Cylinder Piles	$ 3.1M	$ 3.3M
HP 12x53 Piles	$ 5.4M	$ 5.2M
HP 14x117 Piles	$ 4.1M	$ 4.3M

8. Construction Costs – High-Level Approach Spans

Foundation Type	Bulb-Ts Superstructure	Segmental Superstructure
8' Dia. Drilled Shafts	$ 1.8M	$ 3.1M
6' Dia. Drilled Shafts	$ 2.1M	$ 2.7M
4' Dia. Drilled Shafts	$ 2.2M	$ 3.3M
6' Dia. Steel Pipe Piles	$ 1.2M	$ 3.9M
30" PSC	$ 1.3M	$ 1.5M
36" PSC	$ 1.2M	$ 1.6M
54" Concrete Cylinder Piles	$ 1.0M	$ 1.9M
60" Concrete Cylinder Piles	$ 1.0M	$ 1.8M
HP 12x53 Piles	$ 1.6M	$ 1.7M
HP 14x117 Piles	$ 1.1M	$ 1.5M

9. Vibration due to Pile Driving

- Investigate Ground Vibrations from Pile Driving at the Project Site
- Piles Investigated included:
  - 36" PSC
  - 24" PSC
  - Two Steel HP Piles
- Vibration Levels at Various Distances from the Piles were Monitored
- Distance of 150 ft. for Modern Structures
- Distance of 250 ft. for Sensitive Structures

Final Report on Vibrations Due to Pile Driving at the Mobile River Bridge Site

Research Project 930-839R

INVESTIGATION OF PILE SETUP (FREEZE) IN ALABAMA

Development of a Setup Prediction Method and Implementation into LRFD Driven Pile Design

Addendum: Pile Driving Vibration Monitoring of the Future Mobile River Bridge Project

John Cleary, Ph.D., P.E. (P.I. for Vibration Monitoring)
Eric Steward, Ph.D. (Co. P.I. for Vibration Monitoring)
Andrew Gillis (Graduate Student)
Department of Civil Engineering
University of South Alabama
Mobile, AL 36688

June 12, 2015

9. Vibration due to Pile Driving

Appendix F – West High Level Approach Corrosion Test Results

thompson
ENGINEERING

TECHNICAL MEMORANDUM

October 3, 2016

Subject: **Geotechnical Consulting Services – Mobile River Bridge**
West High Level Approach Corrosion Test Results
Mobile River Bridge Project, Mobile, Alabama
ALDOT Project No.: DPI-0030 (005)
Thompson Project No: 16-1101-0110

Attention: Mr. Manuel F. Carballo, P.E., S.E.

Geotechnical Reviewers: Sam Sternberg III, P.E. (Thompson), Cameron L. Crigler, P.E. (Thompson)

Mr. Carballo,

Thompson Engineering Inc. is pleased to present the summary of the results for the electrochemical tests requested by ALDOT at the joint meeting on July 28, 2016 between HDR, Thompson and ALDOT.

Field Exploration and Laboratory Testing

At the request of ALDOT, Thompson remobilized to the site and drilled two (2) additional borings at MB-1 and WHLA-3 to a depth of 30 feet below existing ground surface. Using mud rotary drilling techniques, a Diedrich D50 geotechnical drilling rig equipped with an automatic standard penetration test (SPT) hammer was utilized to advance the borings. All borings were grouted to grade upon field work completion. The field exploration took place between August 17 and 18, 2016

Samples were obtained using the split barrel sampling technique in accordance with AASHTO T-206. Recovered samples were initially examined and visually classified in the field by Thompson Engineers and were then placed on ice. Samples were transported to Test America and Thompson for electrochemical testing.

Electrochemical Testing

Electrochemical series soil tests (total chlorides, total sulfates, resistivity and pH) were performed on the samples. The following soil limit conditions for corrosive environments for steel piles have been provided by ALDOT and FHWA.

ALDOT:

- Resistivity less than 3,000 ohm-cm
- pH less than 5 or greater than 10
- Chlorides greater than 50 ppm
- Sulfates greater than 100 ppm

FHWA[1] *(Publication NHI-05-042):*

- Resistivity less than 2,000 ohm-cm
- pH less than 4.5
- Resistivity is between 2,000 and 5,000 ohms-cm and chlorides are greater than 100 ppm or sulfate greater than 200 ppm
- Sulfates greater than 100 ppm

The electrochemical test results are summarized in **Table 1** below. In addition, electrochemical test results from the previously approved *Bridge Foundation Report for the I-10 Interchange Modifications from Texas Street to West Tunnel Entrance*, dated March 24, 2014, specifically, BR-1 and BR-12, have been included in the table below.

Table 1: Summary of Electrochemical Test Results

Boring No.	Sample No./ Depth(ft.)	pH	Chlorides (ppm)	Sulfates (ppm)	Resistivity (ohms-cm)	ALDOT Classification	FHWA Classification
MB-1	S-1A / 3.0-5.0	7.1	<24	<24	2,500	Non-Aggressive	Non-Aggressive
	S-1B / 5.0-7.0	7.3	<27	<27	3,000	Non-Aggressive	Non-Aggressive
	S-2A / 8.0-10.0	7.2	<30	<30	1,500	Aggressive	Aggressive
	S-2B / 10.0-12.0	7.2	<32	<32	1,500	Aggressive	Aggressive
	S-3A / 18.0-20.0	8.3	<26	<26	6,775*	Non-Aggressive	Non-Aggressive
	S-3B / 20.0-22.0	7.4	<26	28		Non-Aggressive	Non-Aggressive
	S-4A / 28.0-30.0	8.1	<24	<24	8,950*	Non-Aggressive	Non-Aggressive
	S-4B / 30.0-32.0	8.6	<25	30		Non-Aggressive	Non-Aggressive
WHLA-3	S-1A / 3.0-5.0	7.2	<25	<25	5,000	Non-Aggressive	Non-Aggressive
	S-1B / 5.0-7.0	8.1	<23	<23	5,900	Non-Aggressive	Non-Aggressive
	S-2A / 8.0-10.0	7.7	<24	<24	4,900*	Non-Aggressive	Non-Aggressive
	S-2B / 10.0-12.0	6.8	<25	<25		Non-Aggressive	Non-Aggressive
	S-3A / 18.0-20.0	7.2	<24	<24	8,850*	Non-Aggressive	Non-Aggressive
	S-3B / 20.0-22.0	7.2	<24	31		Non-Aggressive	Non-Aggressive
	S-4A / 28.0-30.0	8.2	<24	42	7,200*	Non-Aggressive	Non-Aggressive
	S-4B / 30.0-32.0	8.3	<23	33		Non-Aggressive	Non-Aggressive

BR-1	S-2, S-3, S-4	6	<63	140	1,042	Aggressive	Aggressive
	S-6, S-7, S-8, S-9	6	<12	<25	5,468	Non-Aggressive	Non-Aggressive
	S-10, S-11, S-12	7	<12	<25	4,427	Non-Aggressive	Non-Aggressive
BR-12	S-2, S-3	6	<11	58	11,000	Non-Aggressive	Non-Aggressive
	S-6, S-7, S-8	6	<61	130	9,000	Aggressive	Non-Aggressive
	S-11, S-12, S-14	6	<12	<24	8,000	Non-Aggressive	Non-Aggressive
	S-14, S-15, S-16, S-17	7	<12	<23	12,000	Non-Aggressive	Non-Aggressive

Note: *(*) Samples were combined for resistivity testing.*

Conclusion

The test result summary information above is provided in support of the conceptual design and foundation considerations for the West High Level Approach. During the design phase of this project, additional electrochemical tests will be performed at additional high level approach foundation locations.

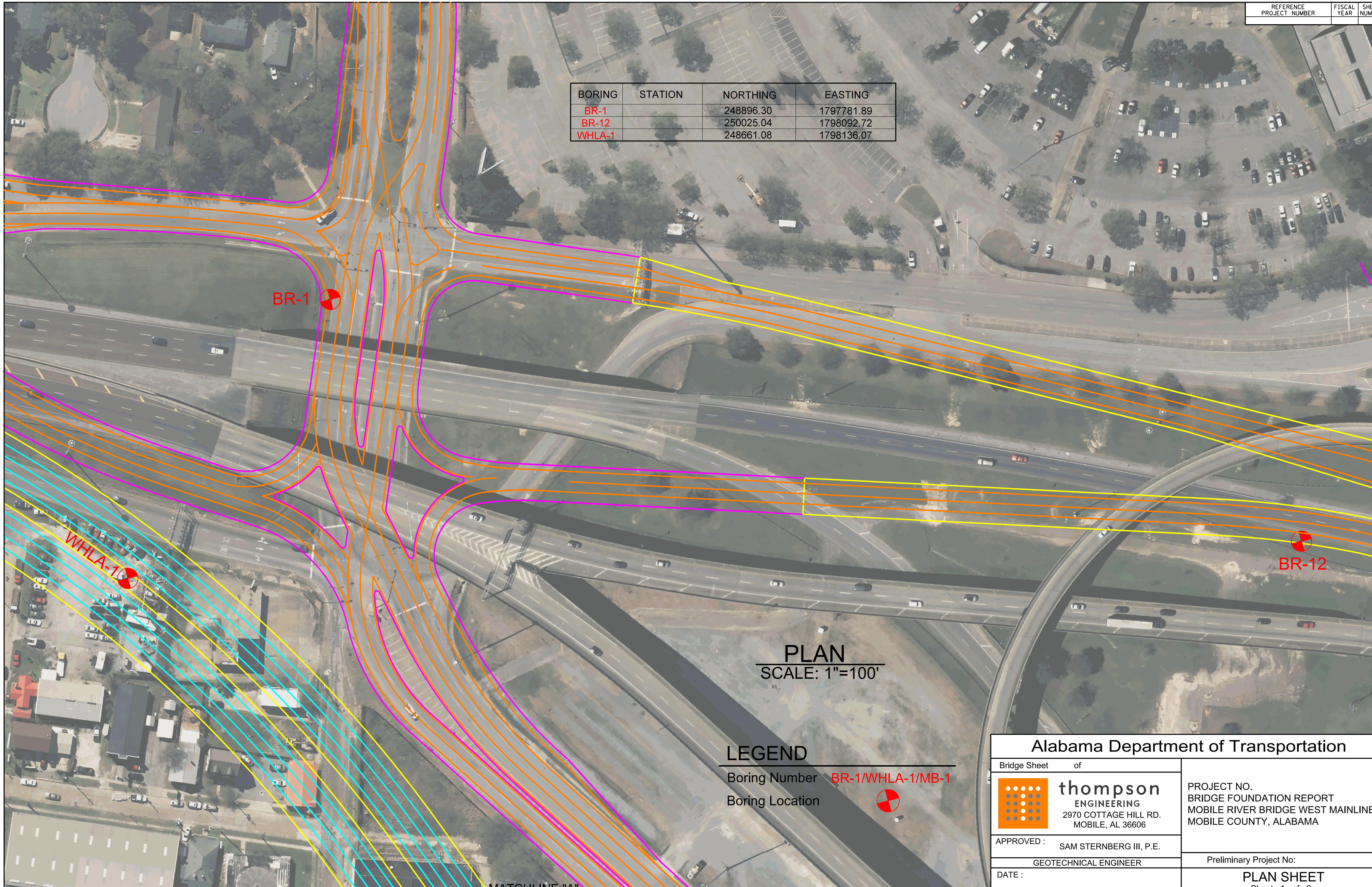
Thompson Engineering appreciates the opportunity to continue our services to Mobile River Bridge Design team with project-related geotechnical consultation. If any questions arise regarding this submittal, kindly advise.

Reference

[1]Hannigan, P.J, G.G. Goble, G.E. Likins, and F. Rausche. *Design and Construction of Driven Pile Foundations – Volume I*. Publication FHWA-NHI-05-042. National Highway Institute: FHWA. U.S. Department of Transportation, 2006.

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

BORING	STATION	NORTHING	EASTING
BR-1		248896.30	1797781.89
BR-12		250025.04	1798092.72
WHLA-1		248661.08	1798136.07



PLAN
SCALE: 1"=100'

LEGEND
Boring Number BR-1/WHLA-1/MB-1

Boring Location

Alabama Department of Transportation

Bridge Sheet of

thompson ENGINEERING
2970 COTTAGE HILL RD.
MOBILE, AL 36606

APPROVED : SAM STERNBERG III, P.E.

GEOTECHNICAL ENGINEER

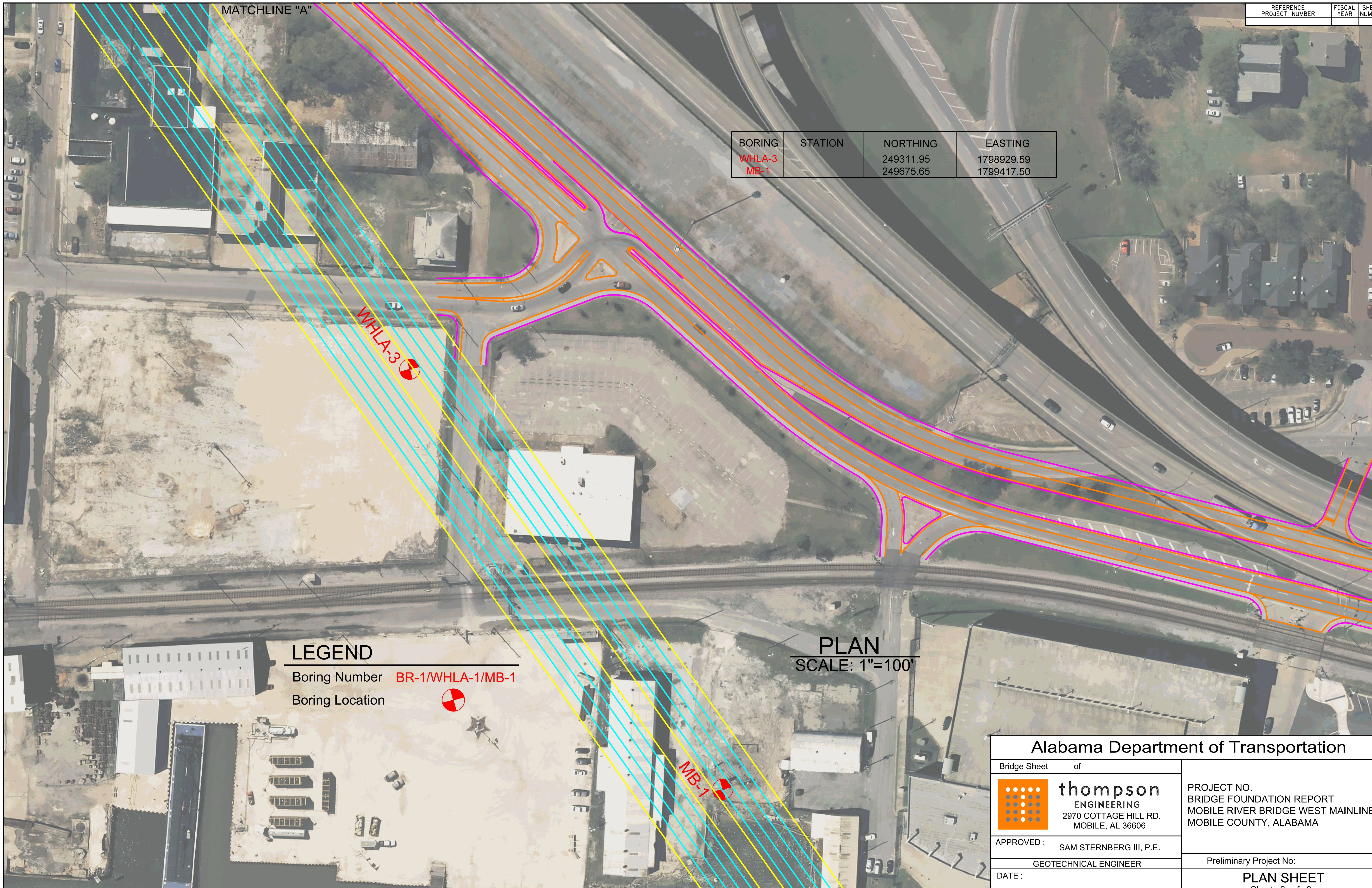
DATE :

PROJECT NO.
BRIDGE FOUNDATION REPORT
MOBILE RIVER BRIDGE WEST MAINLINE
MOBILE COUNTY, ALABAMA

Preliminary Project No:

PLAN SHEET
Sheet 1 of 2

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



BORING	STATION	NORTHING	EASTING
WHLA-3		249311.95	1798929.59
MB-1		249675.65	1799417.50

PLAN

SCALE: 1"=100'

LEGEND

Boring Number BR-1/WHLA-1/MB-1

Boring Location

Alabama Department of Transportation

Bridge Sheet of

thompson ENGINEERING
2970 COTTAGE HILL RD.
MOBILE, AL 36606

APPROVED : SAM STERNBERG III, P.E.

GEOTECHNICAL ENGINEER

DATE :

PROJECT NO.
BRIDGE FOUNDATION REPORT
MOBILE RIVER BRIDGE WEST MAINLINE
MOBILE COUNTY, ALABAMA

Preliminary Project No:

PLAN SHEET
Sheet 2 of 2

Appendix G – Sacrificial Thickness Sample Projects

Mobile River Bridge
Steel Pipe Pile Sacrificial Thickness Project Examples

Project Name	*Location*	*Project Status*	*Project Photograph*	*Pile Condition*	*Service Life (Years)*	*Additional Thickness Location*	*Sacrificial Thickness (in.)*	*Corresponding Corrosion Rate (in./year)*	*Environmental Classification*	*Basis for Corrosion Rates*
Tappan Zee	New York, New York	Under Construction		In Water Partially Buried	100	Above Long Term Scour	3/8	0.00375	Severely Corrosive	FHWA and Caltrans
						Below Long Term Scour	1/8	0.00125		
Gerald Desmond	Long Beach, California	Under Construction		In Land Completely Buried	100	Length of Pile	0.1625	.001 + 1/16”	Corrosive	Caltrans

Appendix F:

Inspection Truck
Clearances

ASPEN A-62

Bridge Inspection Unit

New Technologies Lead the Industry

Design...... Through extensive research utilizing customer input, FEA engineering and repetitive motion testing, the Aspen A-62 was designed from the ground up resulting in greater versatility, simplicity and reliability under all types of conditions. This model has been extended to access areas over, around and under some of your widest bridges to provide you with the pinnacle of "no-compromise" performance and efficiency.

Maneuverability...... Two rotating turntables plus multiple articulating and telescoping booms allow the 40"x 60" rotating platform to deploy off of either side of the truck to access all of your structures. The Aspen A-62 is truly in a league of its own.

Stabilization...... Outriggers are not required and all counterweights stay within the width of the truck body. The redesigned sliding counterweight is installed under the truck bed and operates to either side while the Aspen A-62 is deployed to the opposite side. In the transport mode, the counterweight stays in the center of the truck for improved road handling. A torsion-box subframe and hydraulic axle locks unitize the chassis and truck axles, allowing the vehicle to travel while the unit is fully deployed.

Reliable

Adjustable Turntable

Turntable no. 2 comes equipped with an automatic leveling system, providing smooth platform movement. In addition, the operator can make manual adjustments of +/- 5 degrees to compensate for the slope of the bridge. With the elimination of the leveling arms, the Aspen A-62 has more vertical clearance to get over high fences.

Innovative

Telescoping & Rotating Platform

- 180 degree rotating platform
- Telescoping 4th boom
- Over 13' vertical reach

Durable

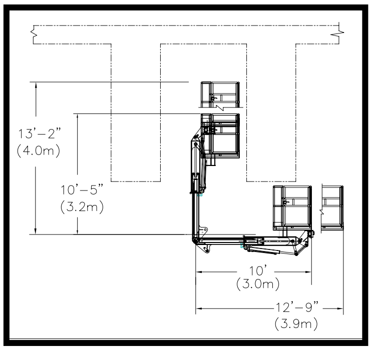
Advanced Control System

As the most advanced microprocessor control system available, the "Plus 1" Graphical Terminal Interface includes a color display depicting unit operation and individual function performance while monitoring the unit's parameters, hydraulics and enabling simple troubleshooting. Wireless controls are included at both operator stations.

Platform Features

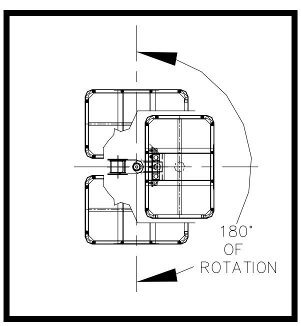
Telescoping 4th boom

The Aspen A-62 has the capability of providing inspectors with over 13’ (4m) of vertical reach. This fully hydraulic feature will enable you to get up and behind your deepest girders for a close-up inspection.



Rotating Platform

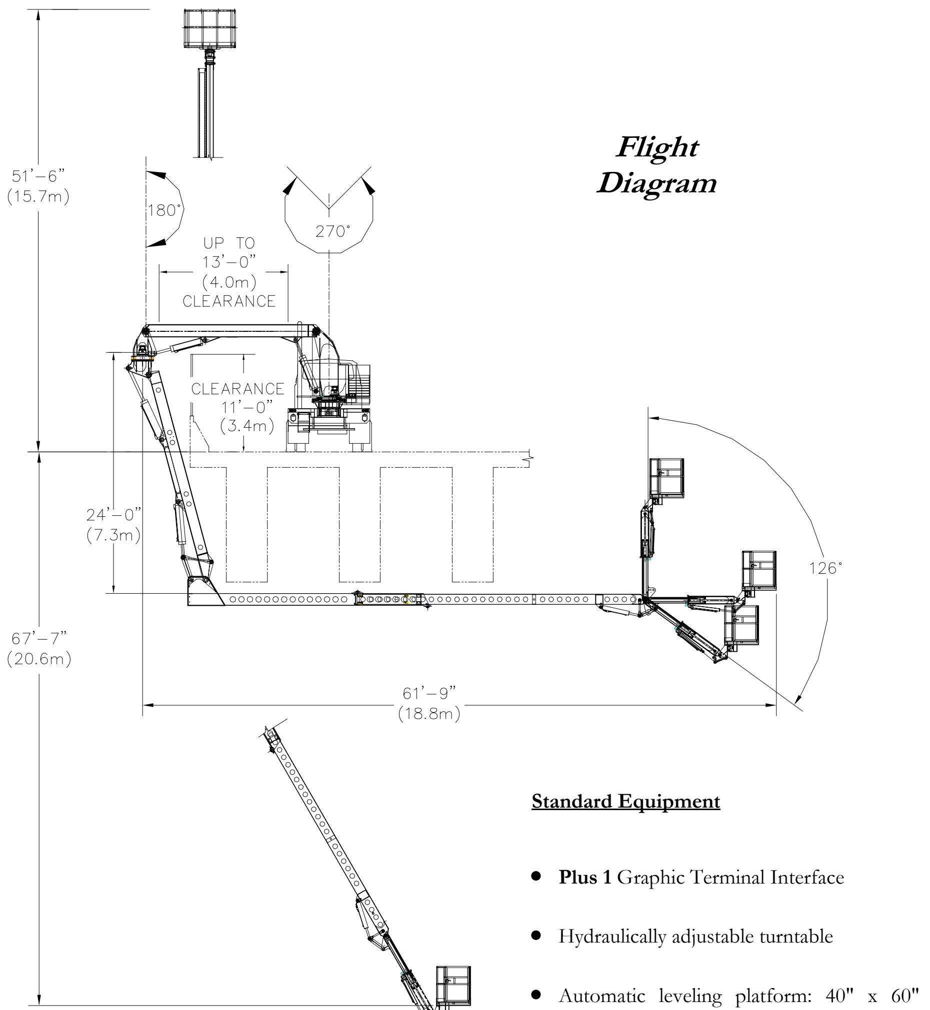
The added flexibility of a 180 degree rotating platform makes maneuvering into working positions easy, increasing productivity and efficiency.



Aspen Aerials, Inc.
4303 West 1st Street
Duluth, MN 55807

Phone: 218-624-1111
Toll Free: 800-888-2773
Fax: 218-624-1714
Web site: www.aspenaerials.com

Specifications



Horizontal Underbridge Reach	**61’-9” / 18.8m**
Vertical Reach Down	**67’-7” / 20.6m**
Vertical Reach Up	**51’-6” / 15.7m**
Boom no. 1 movement	**+30 to - 35 degrees**
Boom no. 2 movement	**+ 0 to -105 degrees**
Boom no. 3 movement	**+90 to - 60 degrees**
Boom no. 4 movement	**+90 to - 36 degrees**
Space Required on Bridge	**102” / 2.5m**
Basket Capacity	**600 lbs. / 272 kg.**
Basket Size	**40” x 60” x 42” 1010mm x 1520mm**
Overall Length	**40’ / 12.2m ***
Overall Height	**13’-3” / 4.0m ***

**** May vary depending on chassis***

Available Options:

- Air Compressor and air line in platform
- Generator with electrical outlet in platform
- Platform Heaters & Floodlights
- Remote Controls

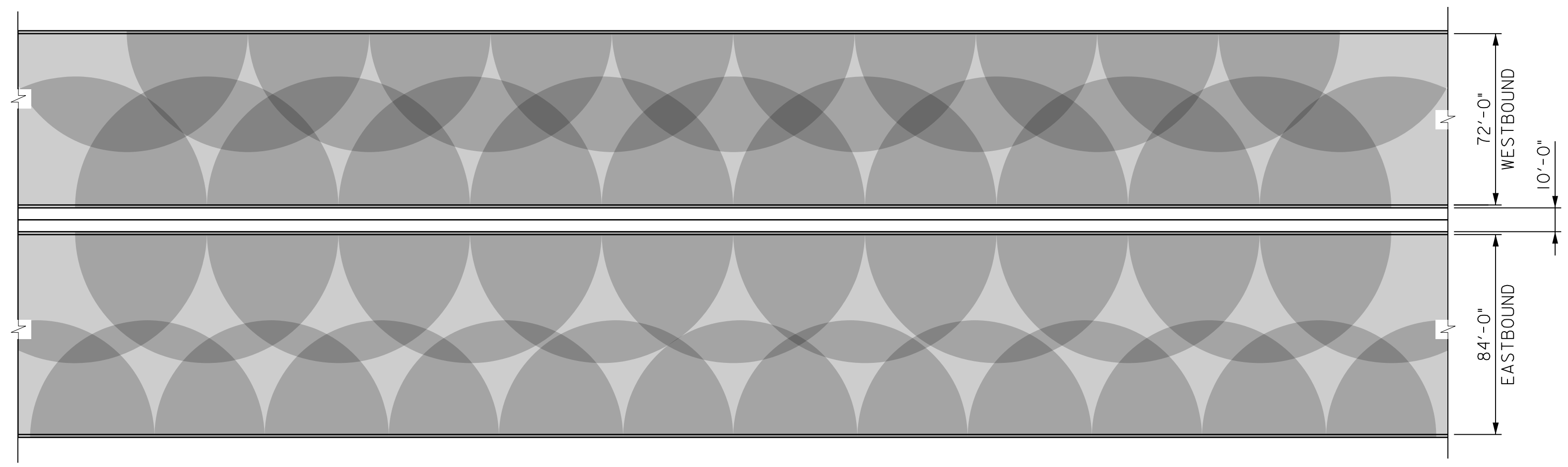
Standard Equipment

- **Plus 1** Graphic Terminal Interface
- Hydraulically adjustable turntable
- Automatic leveling platform: 40" x 60" (1016mm x 1524mm), 600lb. (272kg.) capacity w/access gate
- 2 sets of controls: one set in the platform and one set at the pedestal
- 4 articulating booms (3rd & 4th booms telescope); 2 rotating turrets, rotating platform
- Stability interlocks with monitoring system
- Two power systems: power take-off from the truck’s transmission and a back-up diesel power unit
- 12-volt intercom system between platform,

Various chassis configurations available

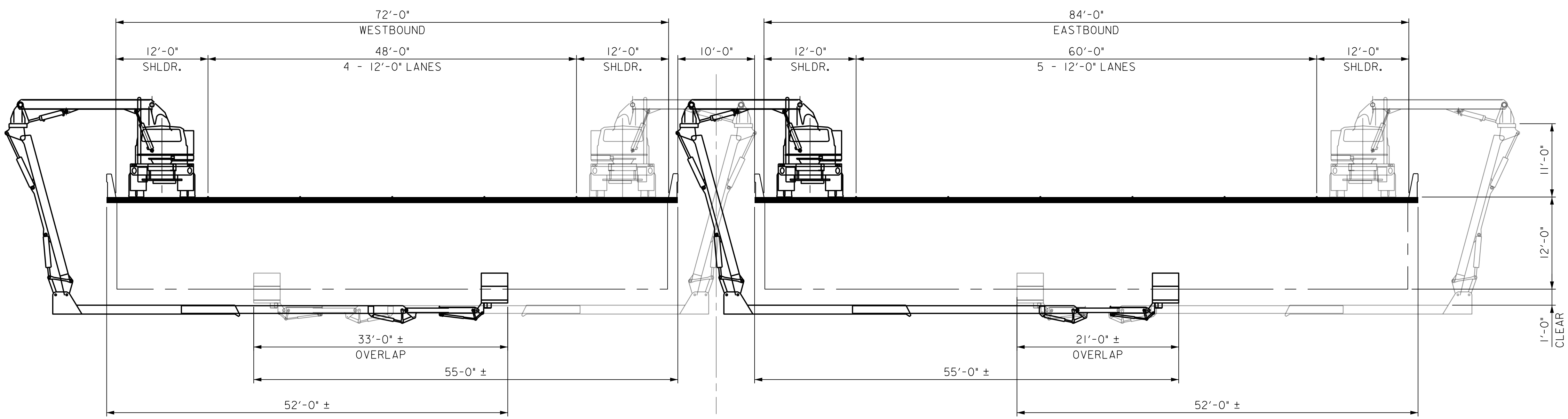
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

DRAFT



PARTIAL PLAN - APPROACH STRUCTURE

SCALE: 1"=30'



TYPICAL SECTION

SCALE: 1/8" = 1'-0"

OPTION 1 - A62 INSPECTION TRUCK

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 3

REVISIONS

PROJECT NO.
MOBILE RIVER BRIDGE
INSPECTION TRUCK
CLEARANCES

INSPECTION ACCESS (1 OF 3)
APPROACH STRUCTURE

ESTIMATED QUANTITIES	DESIGNED BY: MWL	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE: 06/16/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

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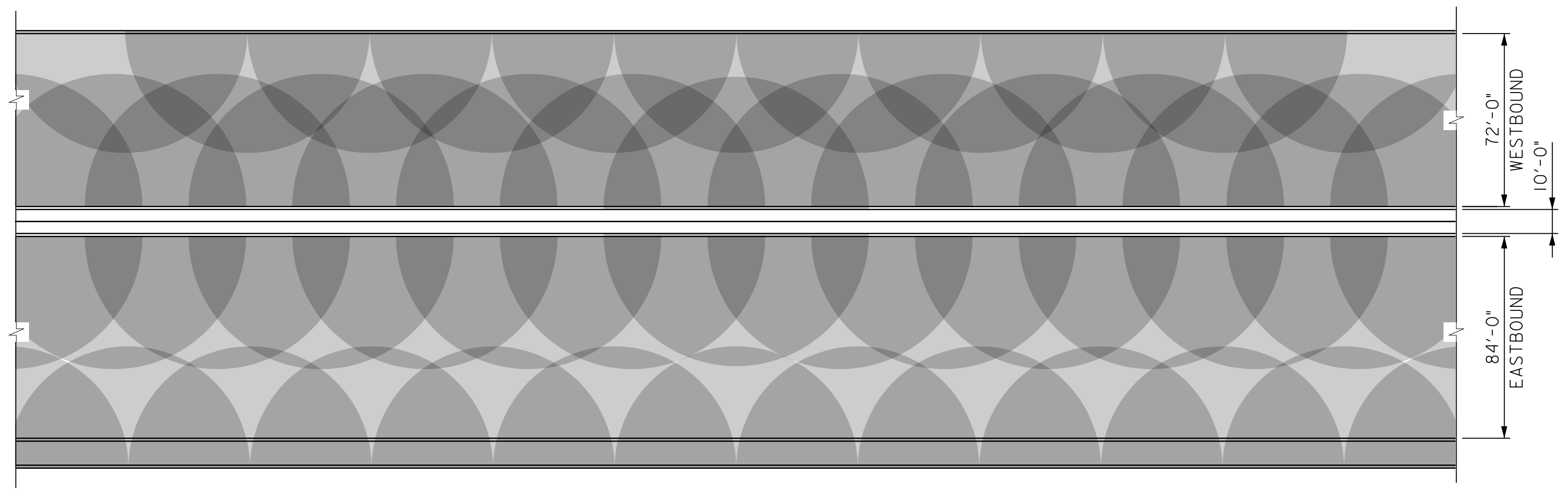
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

DRAFT



PARTIAL PLAN - APPROACH STRUCTURE

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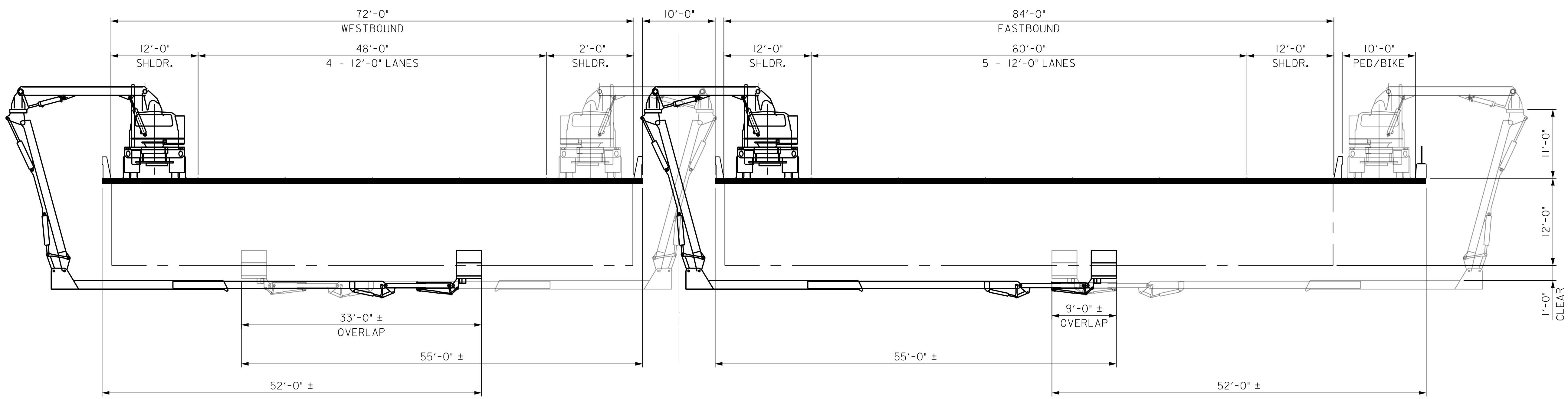

TYPICAL SECTION

SCALE: 1/8" = 1'-0"

OPTION 2 - A62 INSPECTION TRUCK

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 3

REVISIONS

PROJECT NO.
MOBILE RIVER BRIDGE
INSPECTION TRUCK
CLEARANCES

INSPECTION ACCESS (2 OF 3)
APPROACH STRUCTURE

ESTIMATED QUANTITIES	DESIGNED BY: MWL	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE: 06/16/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

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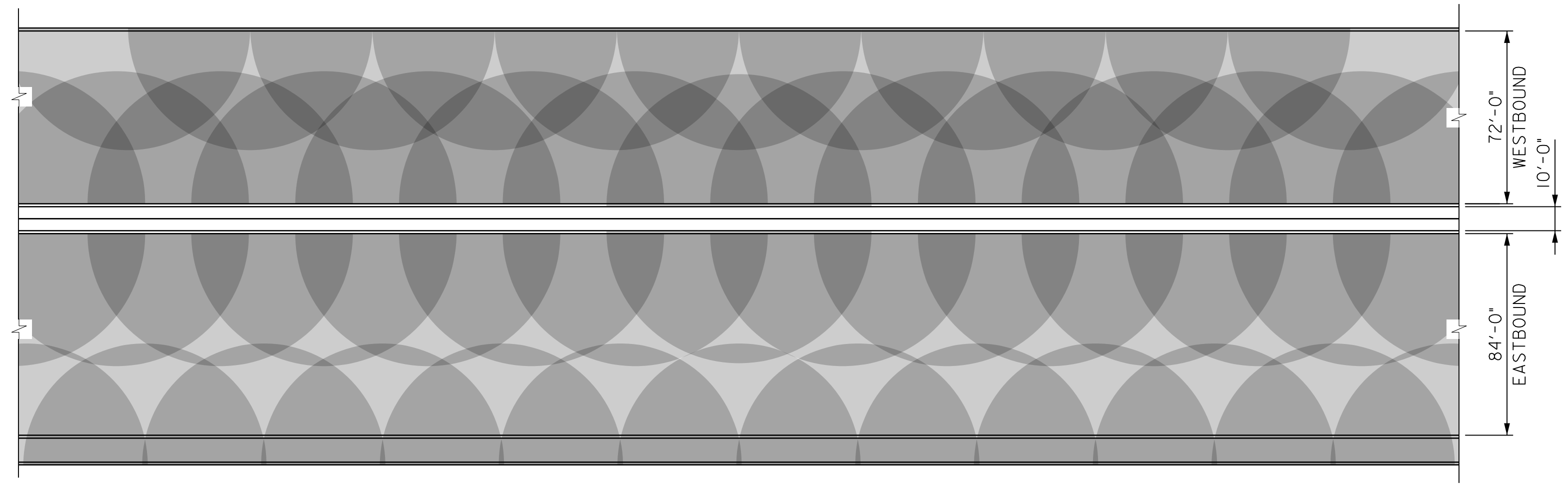
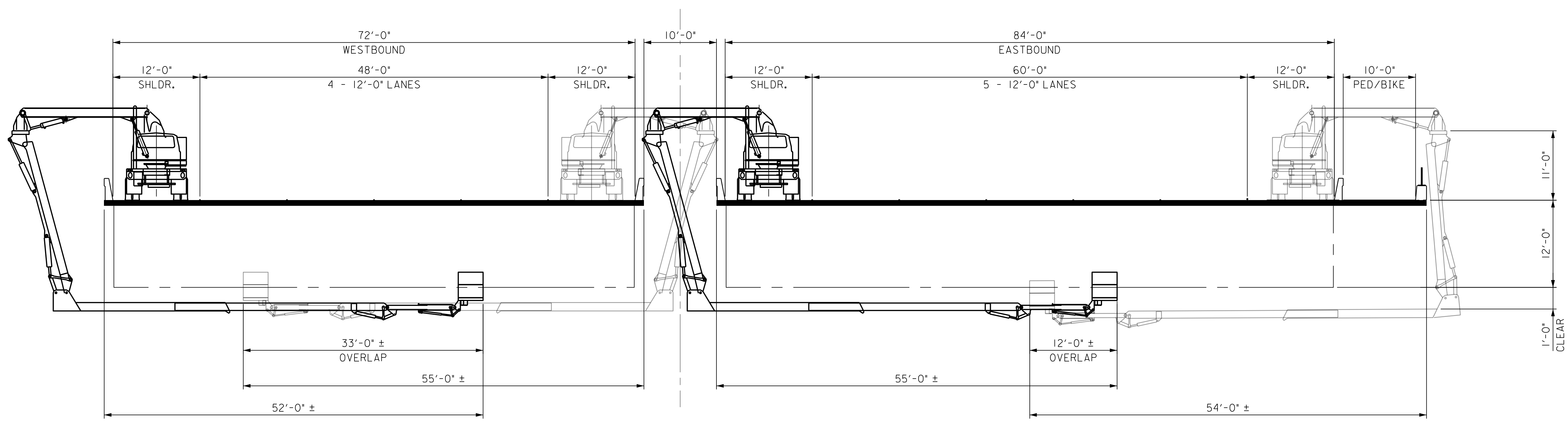
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



PARTIAL PLAN - APPROACH STRUCTURE

SCALE: 1"=30'



TYPICAL SECTION

SCALE: 1/8" = 1'-0"

OPTION 3 - A62 INSPECTION TRUCK

DRAFT

ALABAMA DEPARTMENT OF TRANSPORTATION		
BRIDGE SHEET NO. 3 OF 3	PROJECT NO. MOBILE RIVER BRIDGE INSPECTION TRUCK CLEARANCES	
REVISIONS	INSPECTION ACCESS (3 OF 3) APPROACH STRUCTURE	
ESTIMATED QUANTITIES	DESIGNED BY: MWL	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE: 06/16/2016
BIN: VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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MRB_Approach_Inspection.dgn

Appendix G:

Step 1:

Comparative Cost Estimate Unit Costs

Step 1 – Unit Costs

Superstructure Unit Costs		
Item Description	Units	Unit Price
Steel Reinforcement For Bridge Superstructure (CIP Deck)	Pound	$0.95
Steel Reinforcement For Bridge Superstructure (Precast Segment)	Pound	$1.01
Bridge Concrete Superstructure (CIP Deck)	Cubic Yard	$664.00
Precast Concrete Segments (Span-by-Span)	Cubic Yard	$867.00
Precast Concrete Segments (Medium Cantilever)	Cubic Yard	$1,012.00
Precast Concrete Segments (Long Cantilever)	Cubic Yard	$1,044.00
Bridge Concrete Superstructure (CIP Closure)	Cubic Yard	$1,176.00
Post Tensioning Superstructure Strand (Longitudinal Span-by-Span and Girders)	Pound	$2.91
Post Tensioning Superstructure Strand (Longitudinal Cantilever)	Pound	$3.48
Post Tensioning Superstructure Strand (Transverse)	Pound	$4.43
Precast Curved PCI U72 Girders	Linear Foot	$1,328.00
Precast Curved PCI U96 Girders	Linear Foot	$1,645.00
Pretensioned-Prestressed Concrete Girders, Type LG-78	Linear Foot	$323.00
Pretensioned-Prestressed Concrete Girders, Type FUB-72	Linear Foot	$443.00
Pretensioned-Prestressed Concrete Girders, Type BT-72	Linear Foot	$285.00
Disk Bearings (U72)	Each	$4,428.00
Disk Bearings (U96)	Each	$5,693.00
Disk Bearings (Span-by-Span)	Each	$6,958.00
Disk Bearings (Medium Cantilever)	Each	$15,180.00
Disk Bearings (Long Cantilever)	Each	$18,975.00
Elastomeric Bearing (LG-78 and BT-72)	Each	$1,265.00
Elastomeric Bearing (FUB-72)	Each	$2,530.00
Structural Steel, (armor plate, studs)	Linear Foot	$759.00
Steel Finger Joints (Span-by-Span and Girders)	Linear Foot	$759.00
Steel Finger Joints (Medium Cantilever)	Linear Foot	$2,530.00
Steel Finger Joints (Long Cantilever)	Linear Foot	$2,846.00

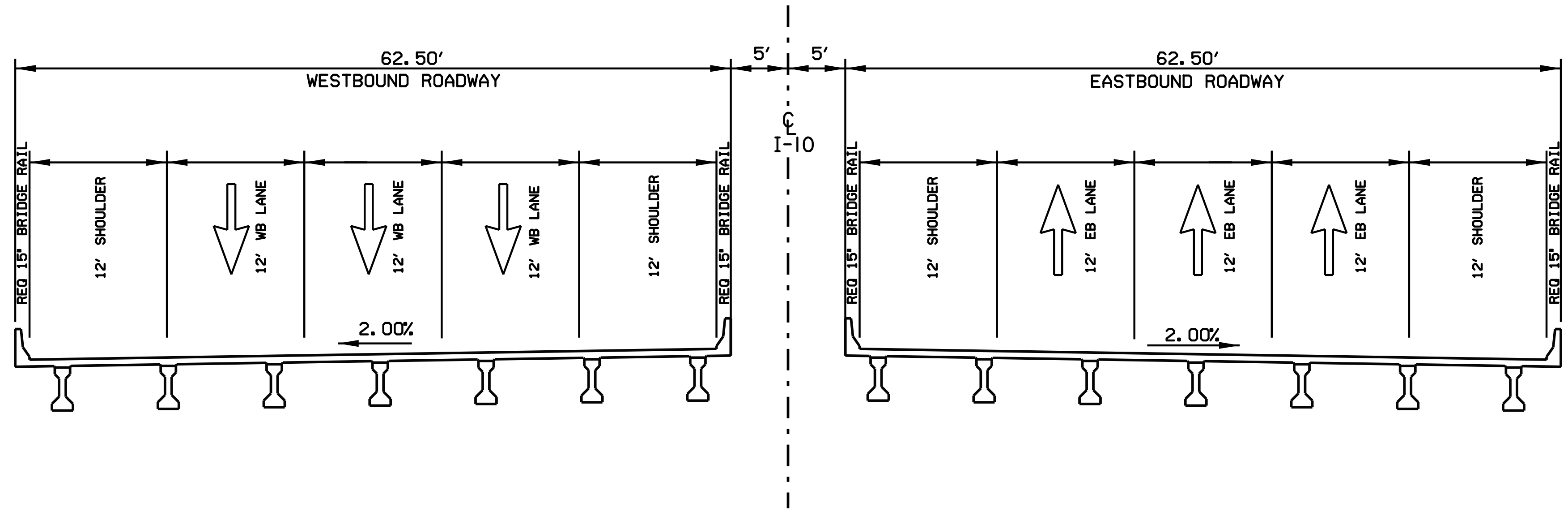
Substructure Unit Costs		
Steel Reinforcement (Grade 60) (Pier)	Pound	$0.89
Steel Reinforcement (Grade 60) (Pile Cap)	Pound	$0.89
Concrete Piling Furnished (54 Inch Diameter)(Spun Cast Cylinder)	Linear Foot	$253.00
Concrete Piling Driven (54 Inch Diameter)(Spun Cast Cylinder)	Linear Foot	$95.00
Bridge Substructure Concrete (Pier)	Cubic Yard	$759.00
Bridge Substructure Concrete (Pile Cap)	Cubic Yard	$411.13
Steel Reinforcement (Grade 60) (Pier)	Pound	$0.89

Miscellaneous		
Concrete Railing	Linear Foot	$95.00
Segmental Erection Equipment (Segment Lifters or Beam and Winch)	Each	$1,265,000.00
Segmental Erection Equipment (Overhead Gantry - Cantilever)	Each	$4,427,500.00
Segmental Erection Equipment (Overhead Gantry - Span-by-Span)	Lump Sum	$2,530,000.00

Appendix H:

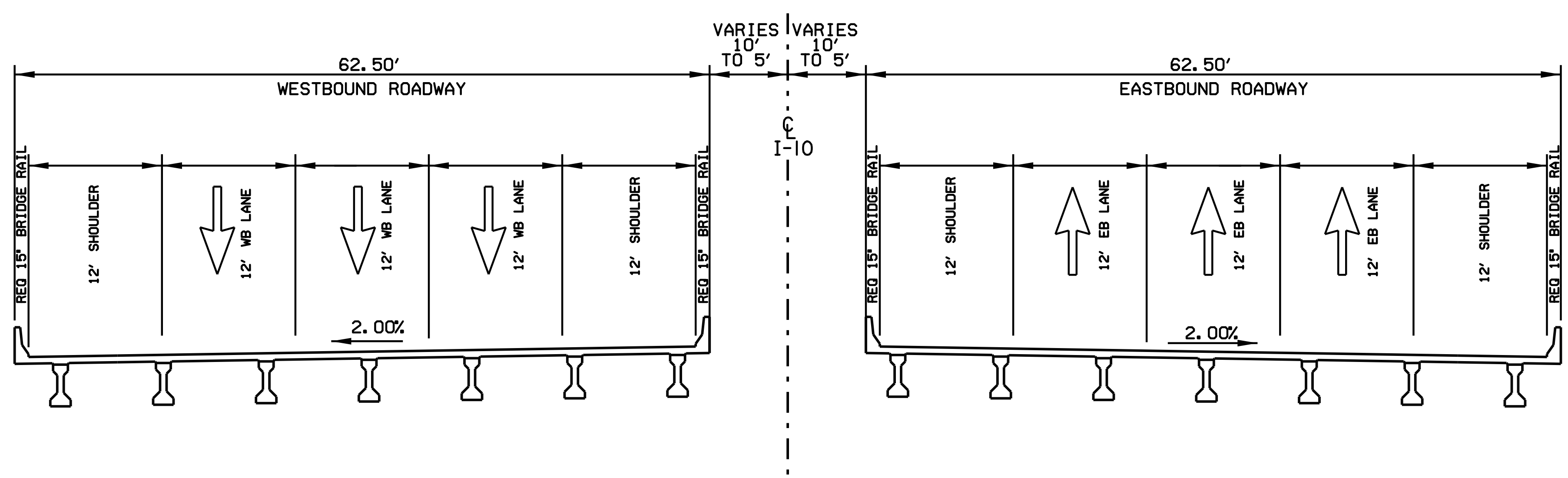
Project Cross-Sections

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



TYPICAL SECTION BAYWAY MAINLINE

STA 585+00 TO STA 598+00

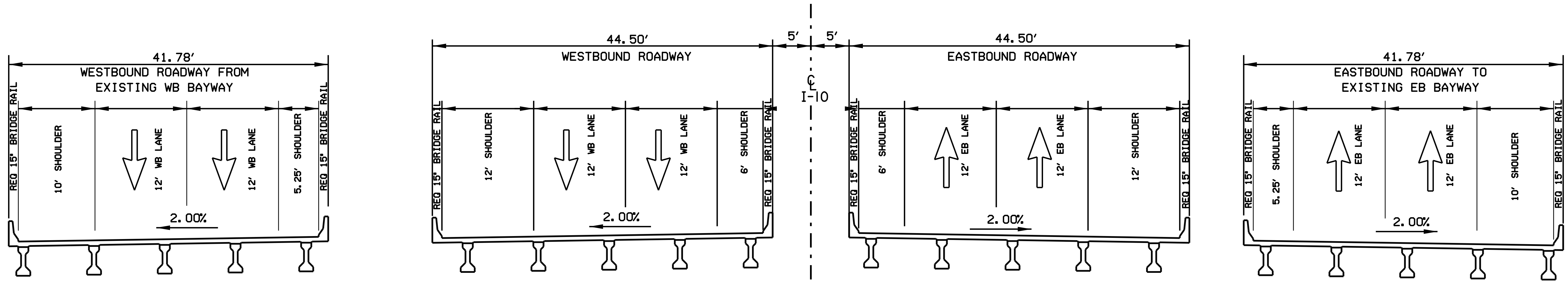


TYPICAL SECTION BAYWAY MAINLINE

STA 570+00 TO STA 585+00

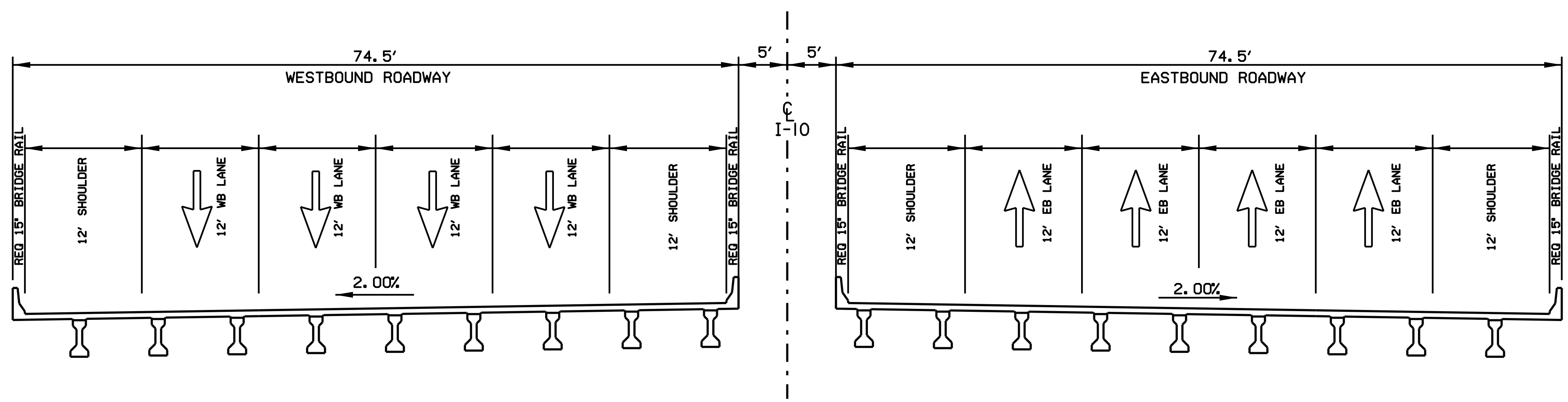
RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



TYPICAL SECTION BAYWAY MAINLINE

STA 636+00 TO STA 666

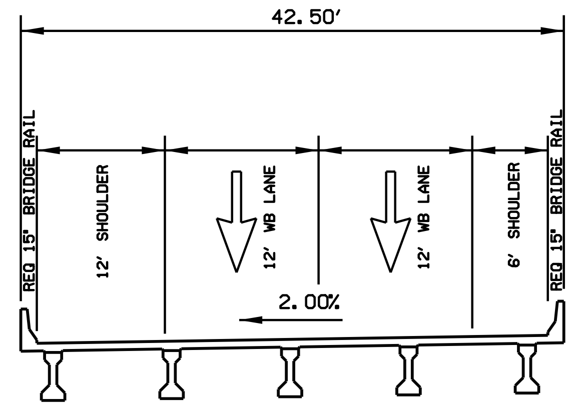


TYPICAL SECTION BAYWAY MAINLINE

STA 598+00 TO STA 636+00

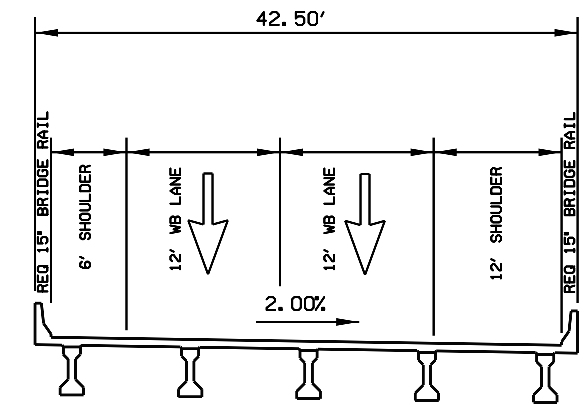
RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



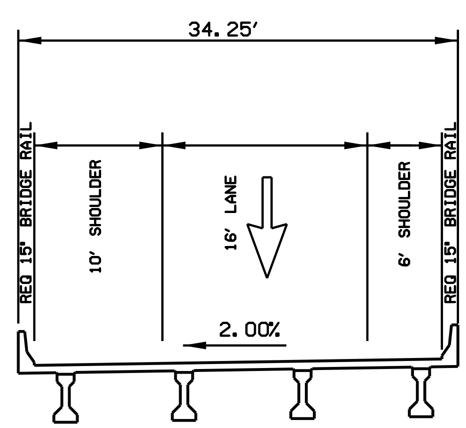
TYPICAL SECTION DOUBLE LANE RAMP

GWT EBR STA 568+75 TO STA 581+00



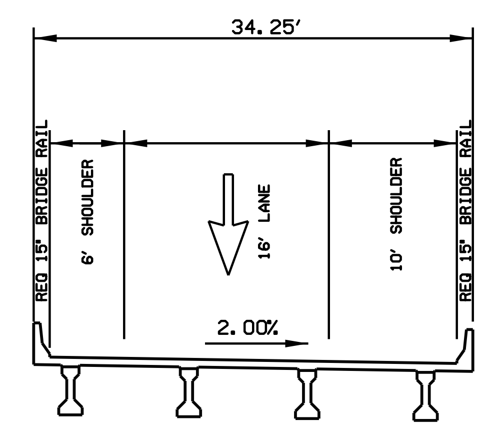
TYPICAL SECTION DOUBLE LANE RAMP

GWT WBR STA 548+30 TO STA 551+75
GWT WBR STA 562+50 TO STA 597+00
CW WB RP 2 STA 550+00 TO STA 562+50



TYPICAL SECTION SINGLE LANE RAMP

GWT EBR STA 548+85 TO STA 568+75
GWT EBR STA 580+00 TO STA 598+00
GW EB RP 1 STA 549+35 TO STA 556+75
CW EB RP 1 STA 560+00 TO STA 568+75

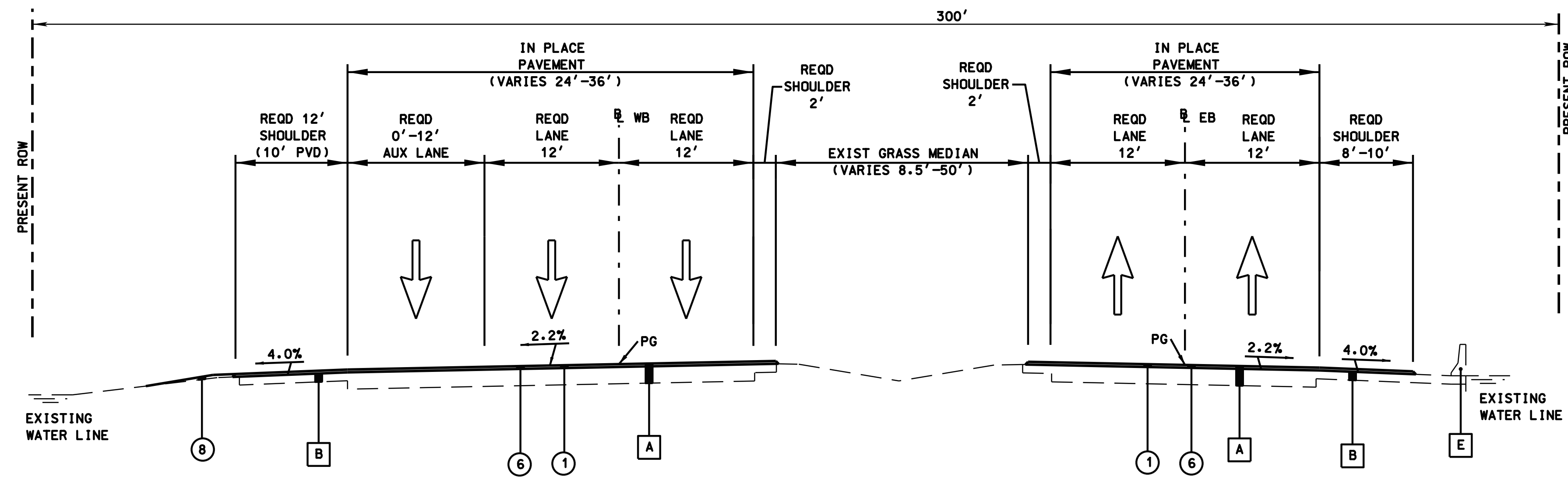


TYPICAL SECTION SINGLE LANE RAMP

GWT WBR STA 551+75 TO STA 562+50
CW WB RP 2 STA 550+00 TO STA 562+50

RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



MID BAY INTERCHANGE
TYPICAL SECTION
NTS
US 90
STA TO

EXISTING MATERIAL LEGEND

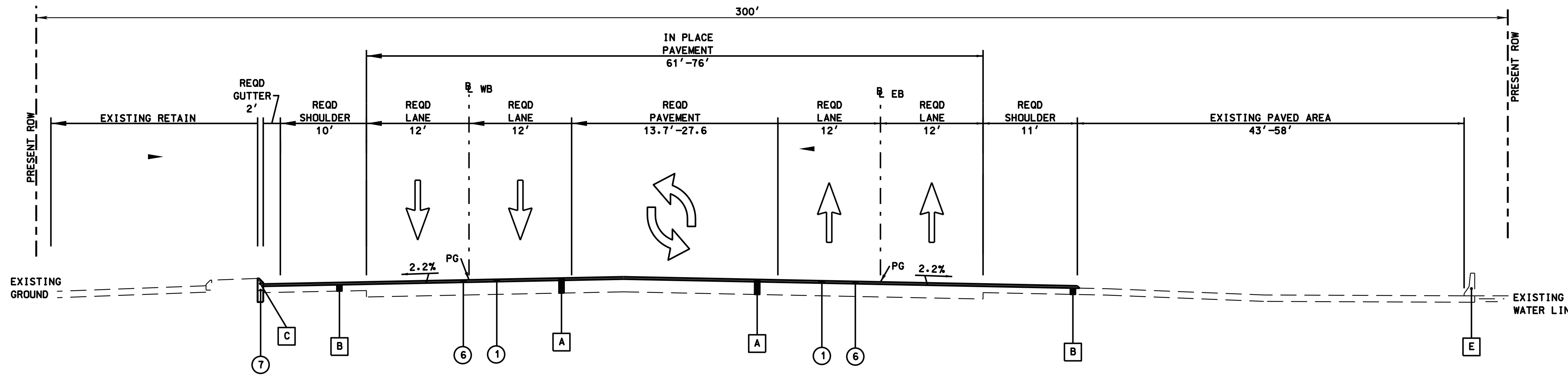
A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
E	IN-PLACE BARRIER RAIL (RETAIN)

REQUIRED MATERIAL LEGEND

1		SUPER PAVEMENT
6	408A-052	PLANING EXISTING PAVEMENT (APPROX. 0.00" - 1.00")
7	623B-000	CONCRETE CURB, TYPE N
8	210A-000 AND/OR 210-000	UNCLASSIFIED EXCAVATION AND/OR BORROW EXCAVATION
9	614A-000	SLOPE PAVING

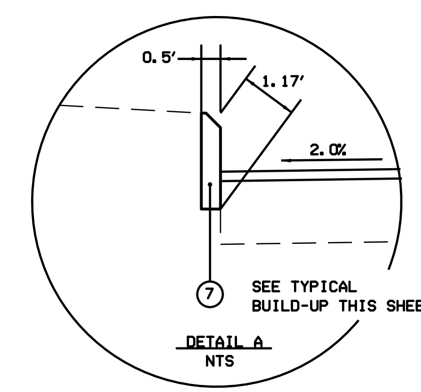
RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



MID BAY INTERCHANGE
TYPICAL SECTION

NTS
US 90
STA TO



EXISTING MATERIAL LEGEND

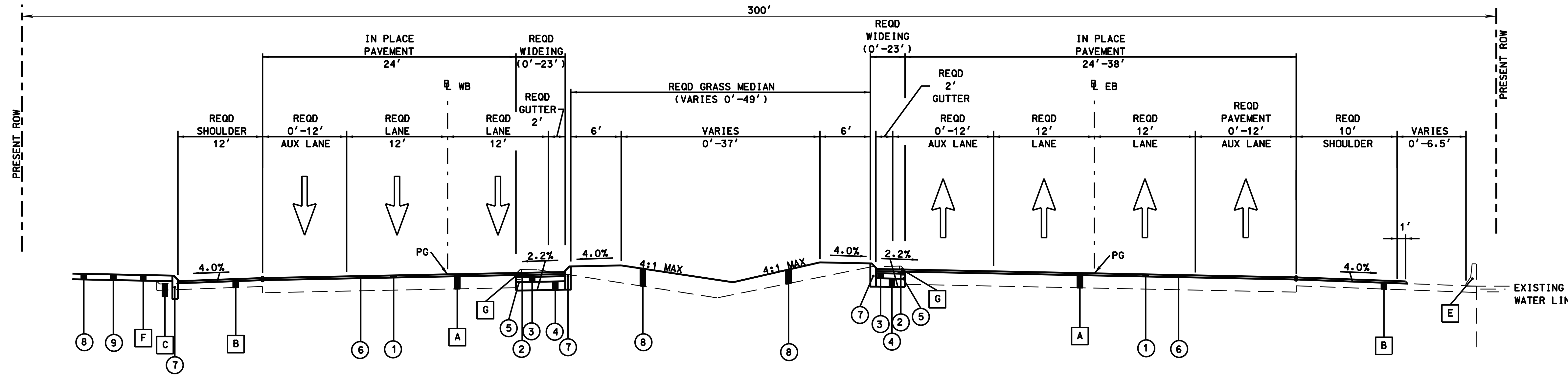
A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
E	IN-PLACE BARRIER RAIL (RETAIN)
G	IN-PLACE CURB (REMOVE)

REQUIRED MATERIAL LEGEND

1		SUPER PAVEMENT
6	408A-052	PLANING EXISTING PAVEMENT (APPROX. 0.00" - 1.00")
7	623B-000	CONCRETE CURB, TYPE N
8	210A-000 AND/OR 210-000	UNCLASSIFIED EXCAVATION AND/OR BORROW EXCAVATION
9	614A-000	SLOPE PAVING

RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



MID BAY INTERCHANGE
TYPICAL SECTION

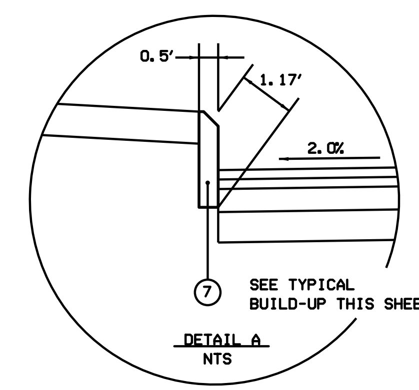
NTS
US 90
STA TO

EXISTING MATERIAL LEGEND

A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
C	IN-PLACE ASPHALT PAVEMENT ROADWAY (REMOVE)
E	IN-PLACE BARRIER RAIL (RETAIN)
F	IN-PLACE SLOPE PAVING (REMOVE)
G	IN-PLACE CONCRETE CURB (REMOVE)

REQUIRED MATERIAL LEGEND

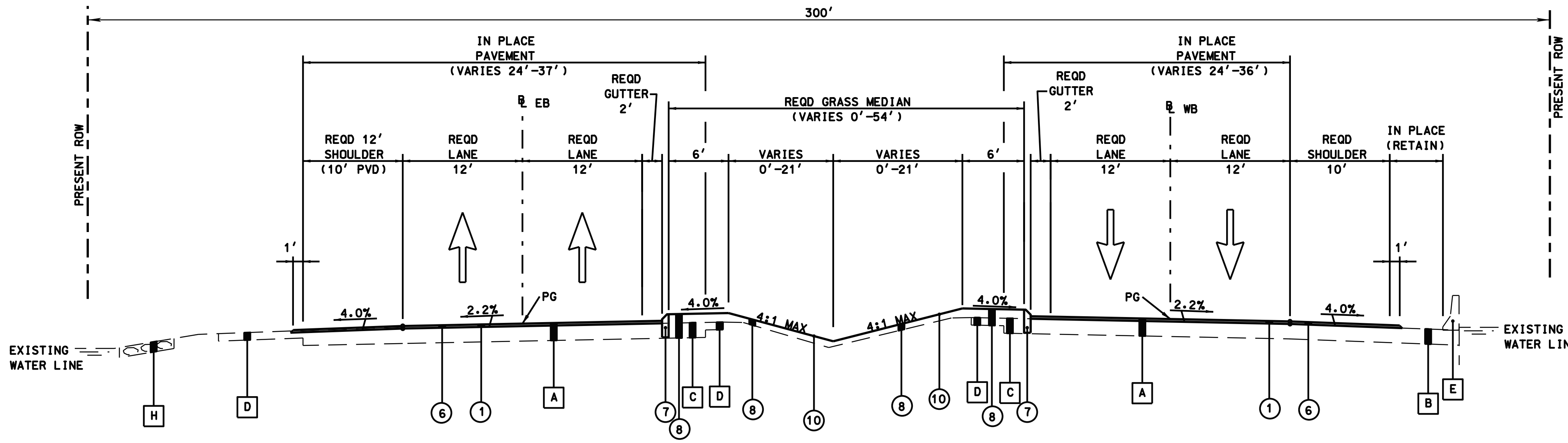
1		SUPER PAVEMENT
2		SUPER PAVEMENT UBL
3		SUPER PAVEMENT LBL
4		AGGREGATE BASE LAYER
5	401A-000	BITUMINOUS TREATMENT "A"
6	408A-052	PLANING EXISTING PAVEMENT (APPROX. 0.00" - 1.00")
7	623B-000	CONCRETE CURB, TYPE N
8	210A-000 AND/OR 210-000	UNCLASSIFIED EXCAVATION AND/OR BORROW EXCAVATION
9	614A-000	SLOPE PAVING
10	614A-000	SLOPE PAVING



DETAIL A
NTS

RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



MID BAY INTERCHANGE
TYPICAL SECTION

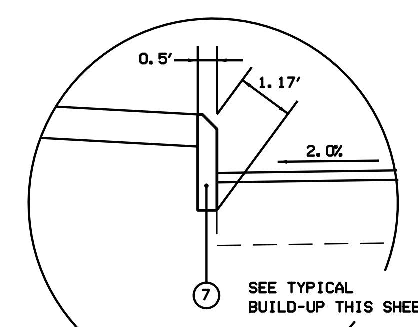
NTS
US 90
STA TO

EXISTING MATERIAL LEGEND

A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
C	IN-PLACE ASPHALT PAVEMENT ROADWAY (REMOVE)
D	IN-PLACE ASPHALT PAVEMENT SHOULDER (REMOVE)
E	IN-PLACE CONCRETE BARRIER RAIL (RETAIN)
H	IN-PLACE RIPRAP (RETAIN)

REQUIRED MATERIAL LEGEND

1		SUPER PAVEMENT
6	408A-052	PLANING EXISTING PAVEMENT (APPROX. 0.00" - 1.00")
7	623B-000	CONCRETE CURB, TYPE N
8	210A-000 AND/OR 210-000	UNCLASSIFIED EXCAVATION AND/OR BORROW EXCAVATION
9	614A-000	SLOPE PAVING
10	650A-000	TOPSOIL



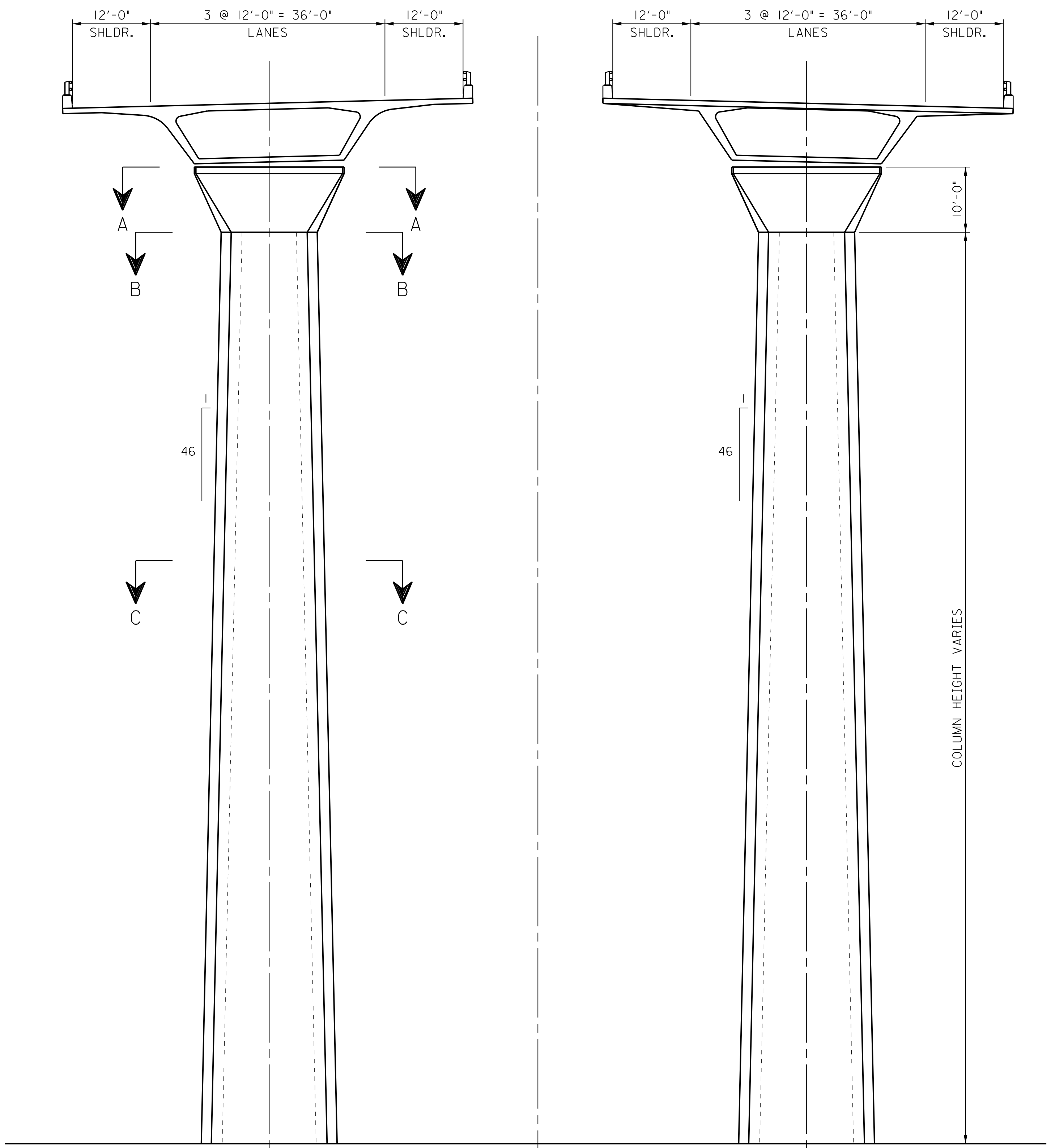
DETAIL A
NTS

RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

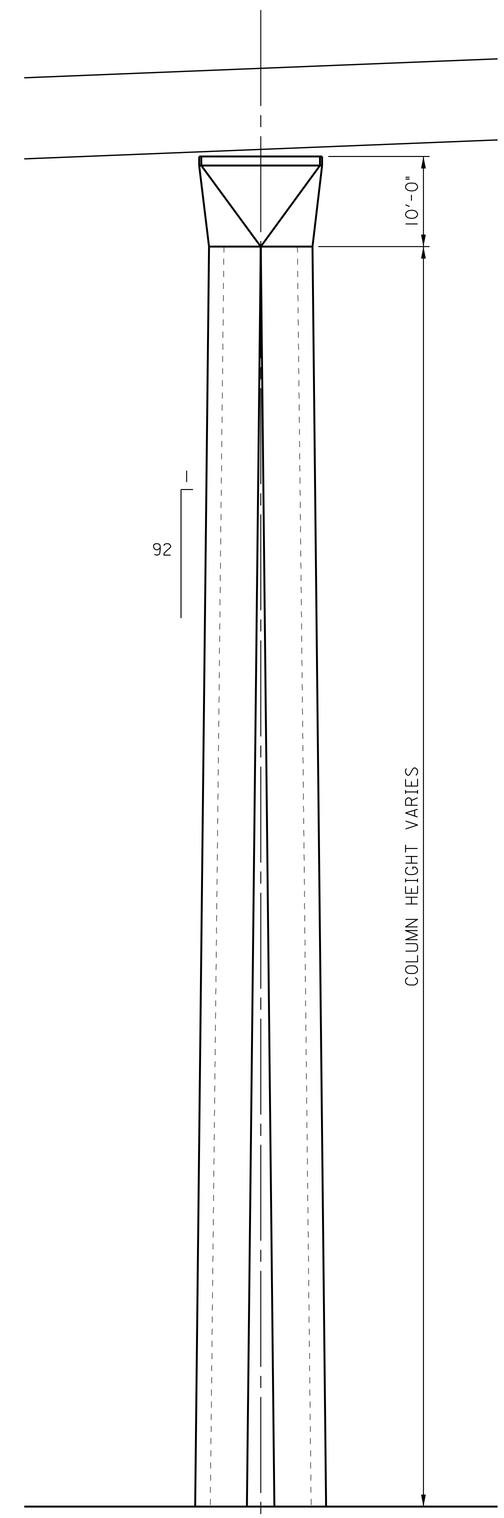
Appendix I:

Typical Piers

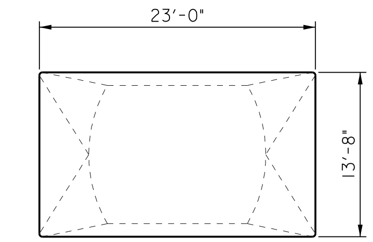
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



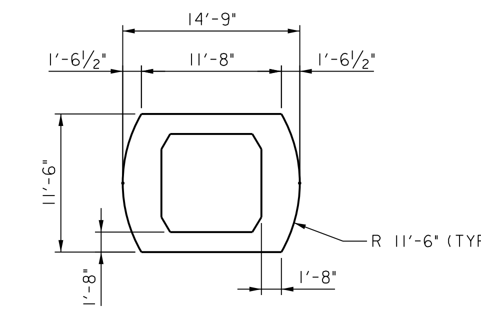
ELEVATION



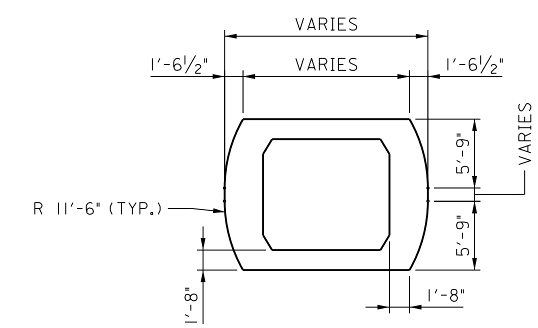
LONGITUDINAL ELEVATION



SECTION A-A



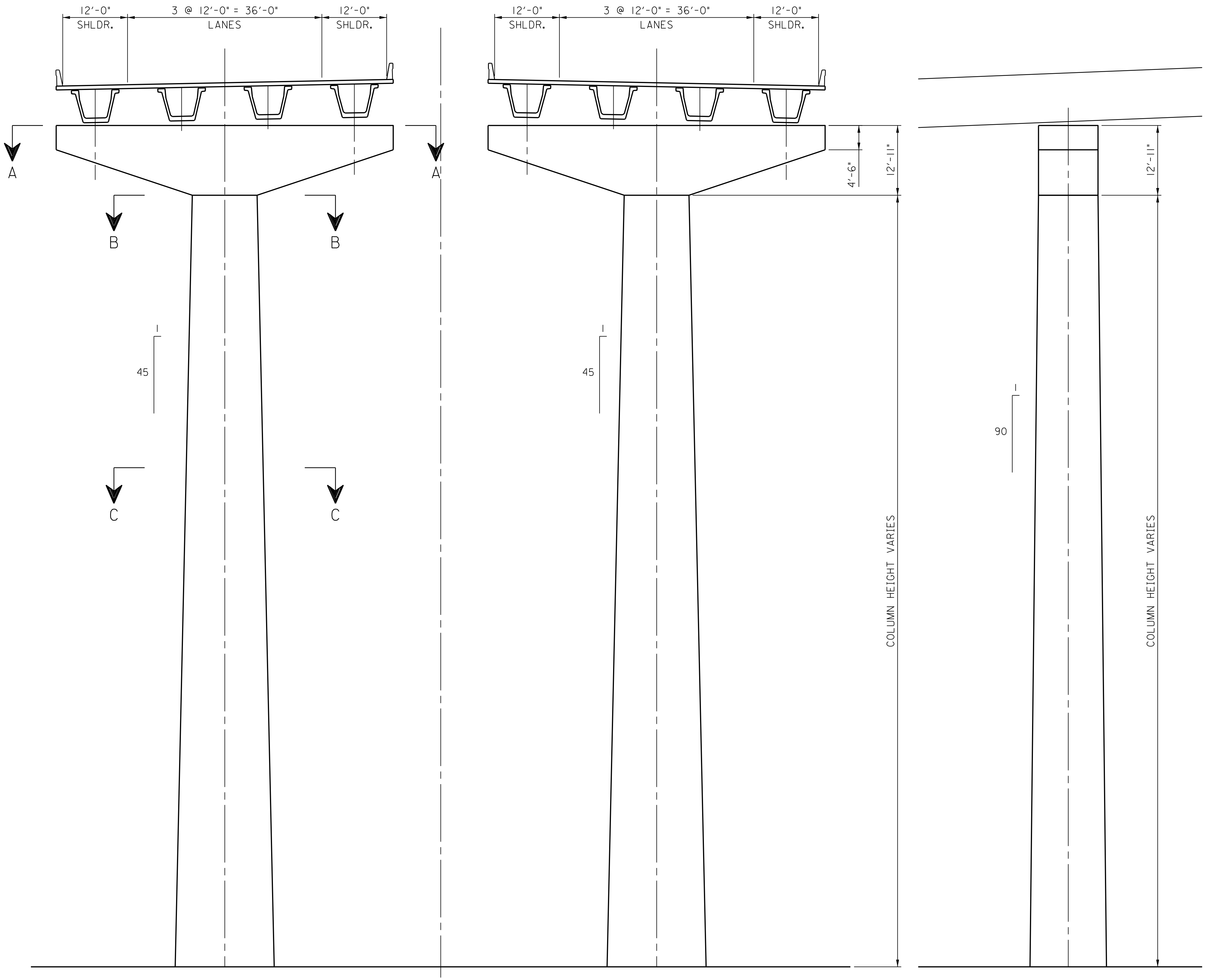
SECTION B-B



SECTION C-C

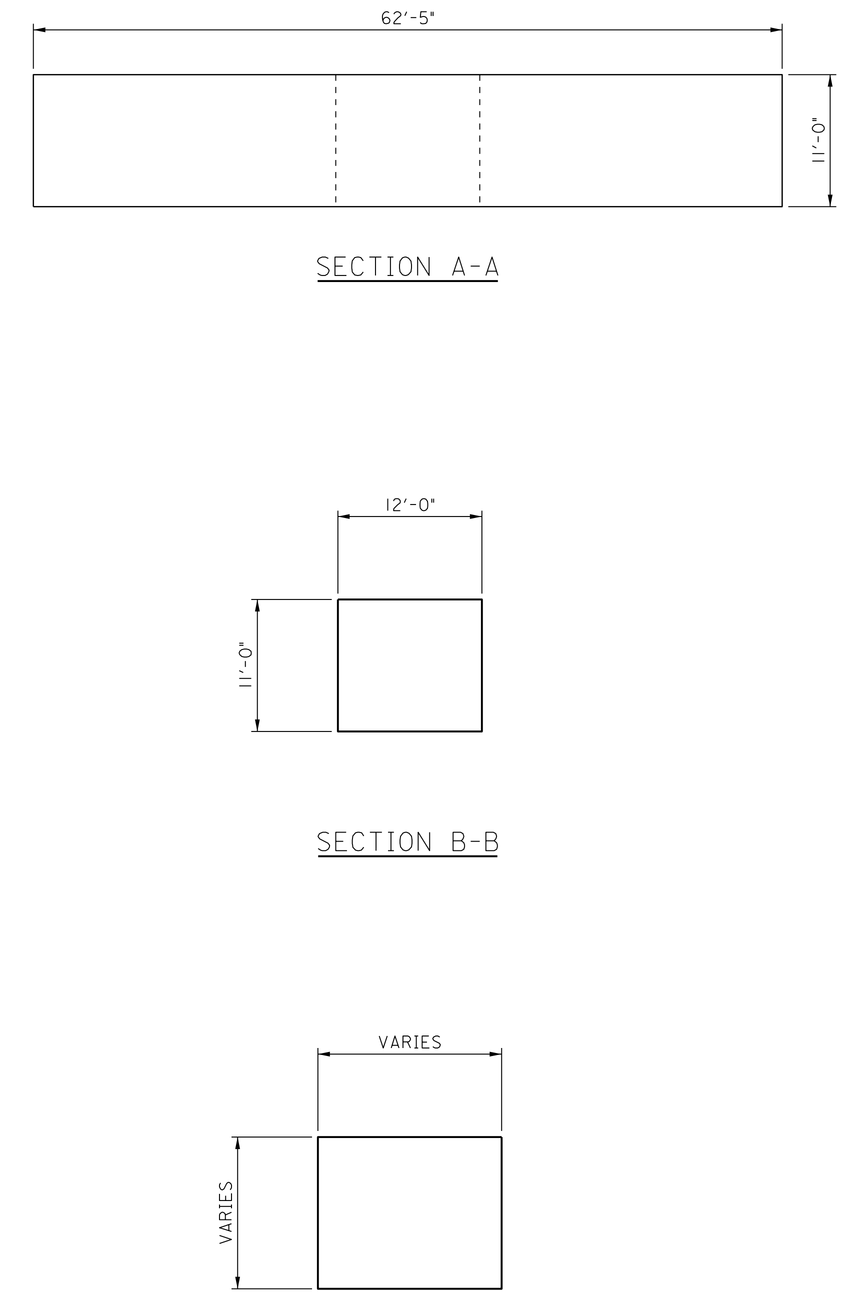
ALABAMA DEPARTMENT OF TRANSPORTATION			
BRIDGE SHEET NO. OF	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH		
REVISIONS	PRECAST SEGMENTAL ALTERNATIVE		
	ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
	COMPUTED BY:	CHECKED BY:	DATE DRAWN:
BIN:	VERIFIED BY:	DATE CHECKED:	SCALE:

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



ELEVATION

LONGITUDINAL ELEVATION



SECTION A-A

SECTION B-B

SECTION C-C

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
HIGH LEVEL APPROACH

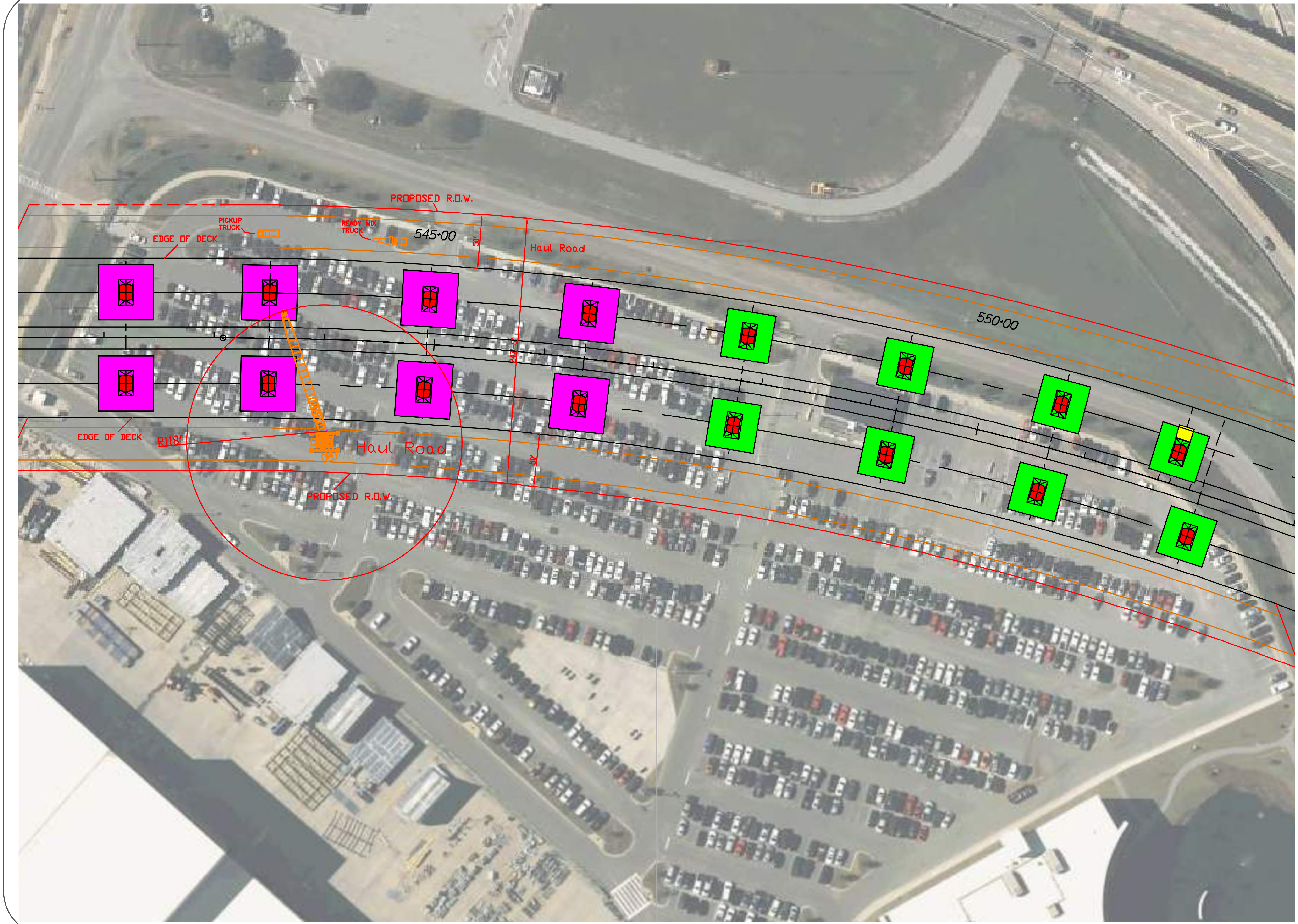
U-BEAM ALTERNATIVE

ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

BIN:

Appendix J:

Precast Segmental Construction Steps



General Notes

No.	Revision/Issue	Date

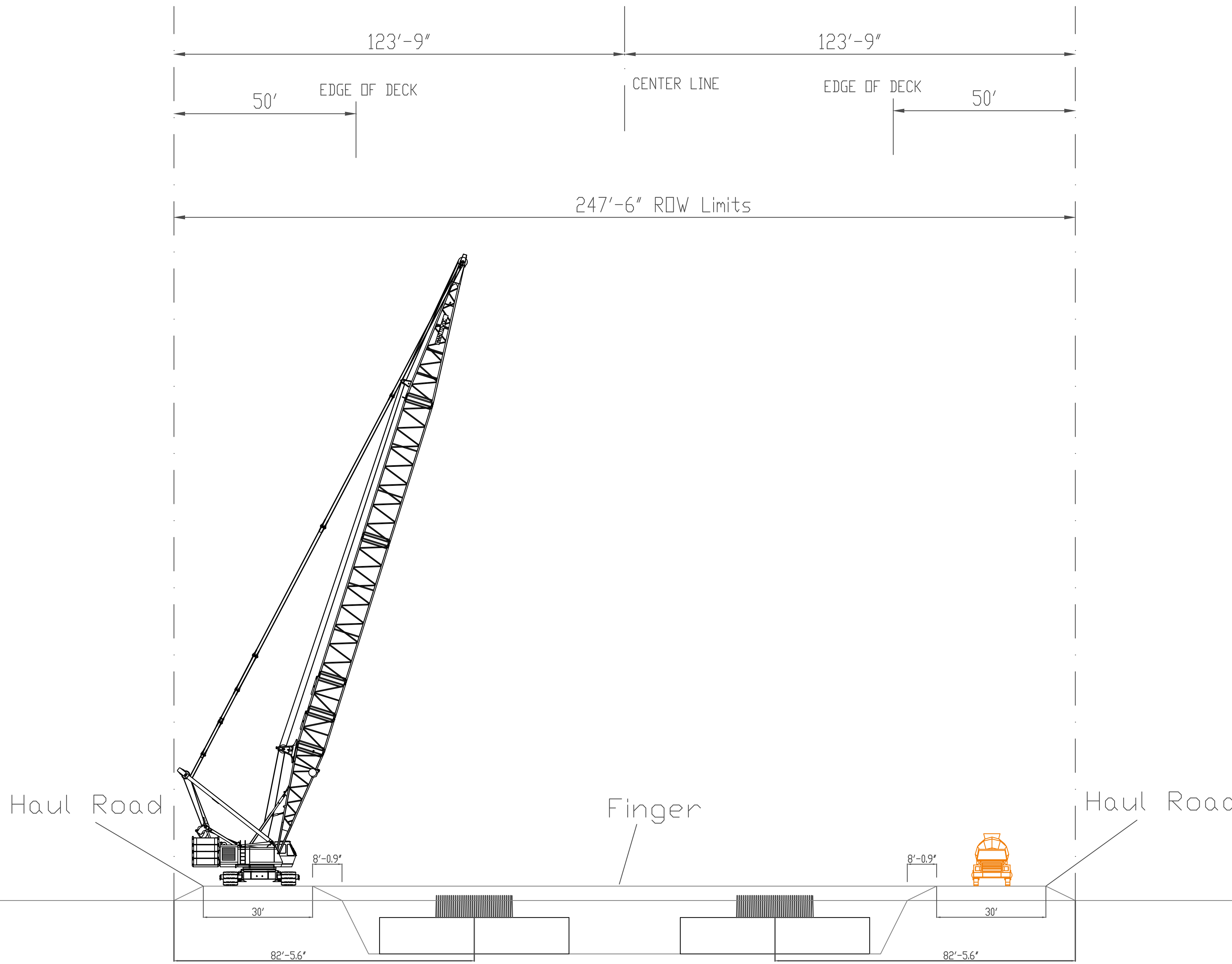
Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

Mobile River Bridge
Precast Segmental
Crane Pick Drawings
Plan View

Project ACS-244	Sheet 01
Date 04/17/2017	
Scale As Noted	



Pier 36 Looking West

General Notes

Use 200' boom length Manitowoc 999 for pouring of footing, column, and cap

No.	Revision/Issue	Date

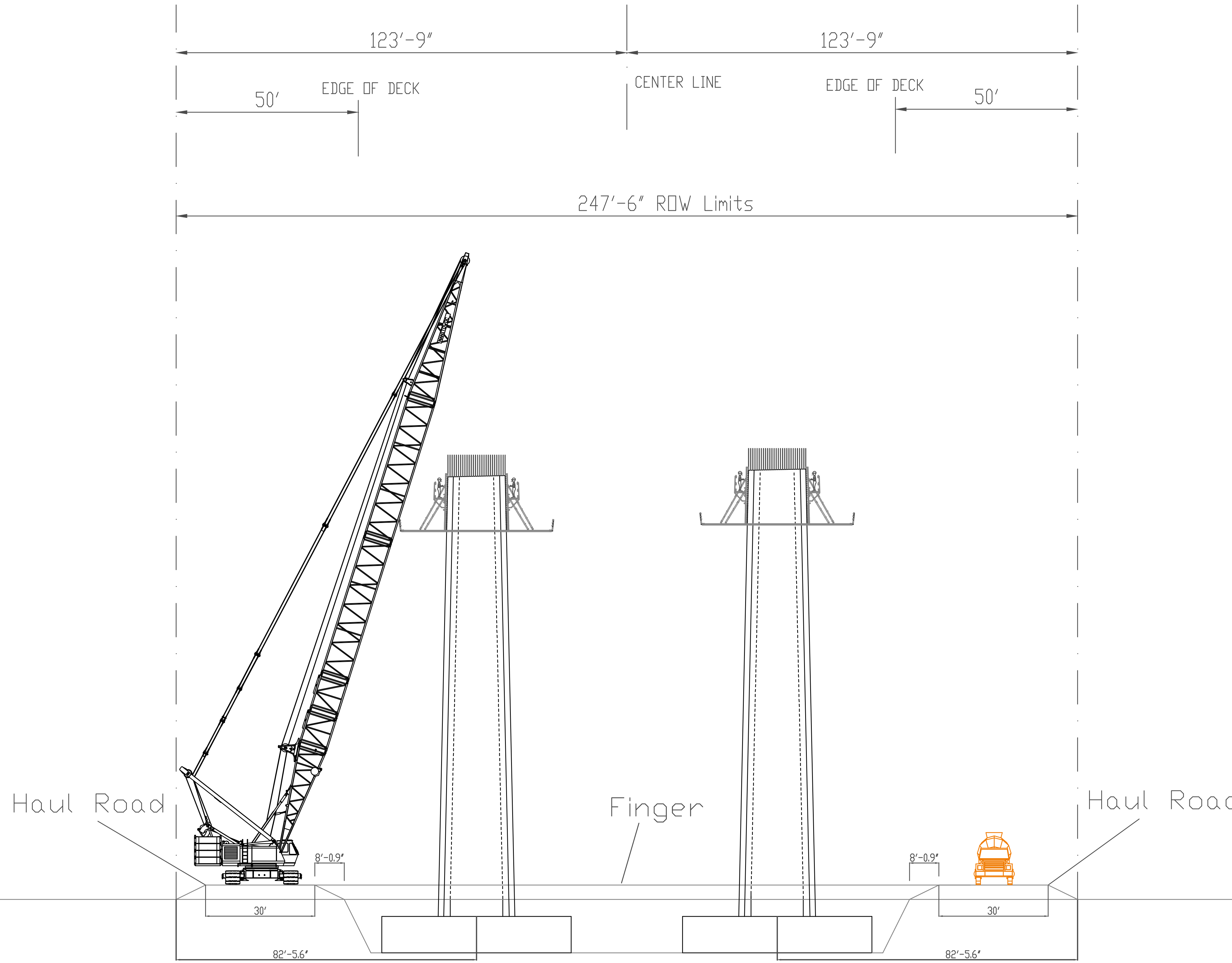
Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

Mobile River Bridge
Precast Segmental
Crane Pick Drawings
Elevation View

Project: ACS-244
Date: 04/17/2017
Scale: As Noted
Sheet: 02



Pier 36 Looking West

General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

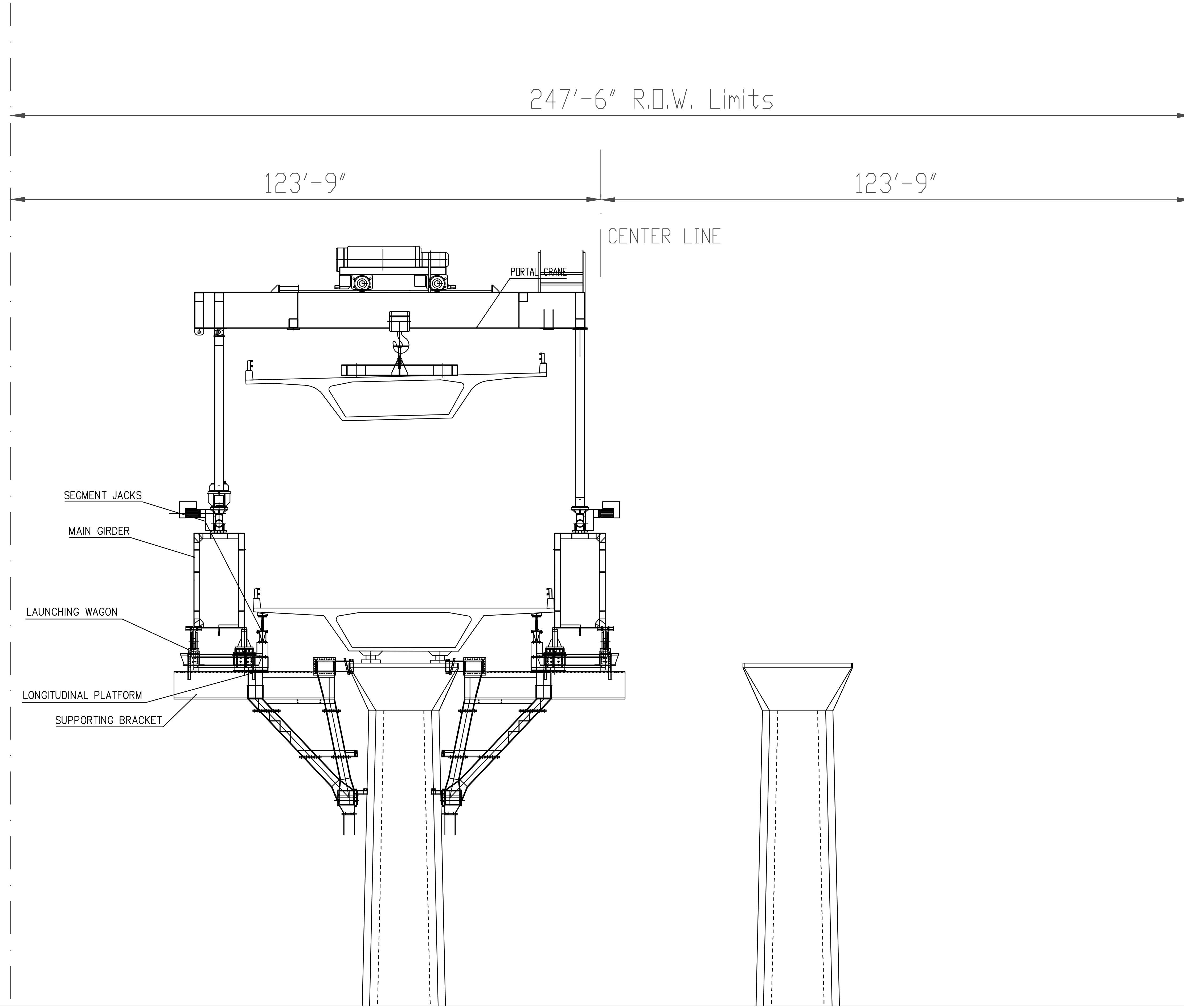
Mobile River Bridge
Precast Segmental
Underslung Truss
Elevation View

Project: ACS-244

Date: 04/17/2017

Scale: As Noted

Sheet: 03



General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

Mobile River Bridge
Precast Segmental
Underslung Truss
Elevation View

Project ACS-244

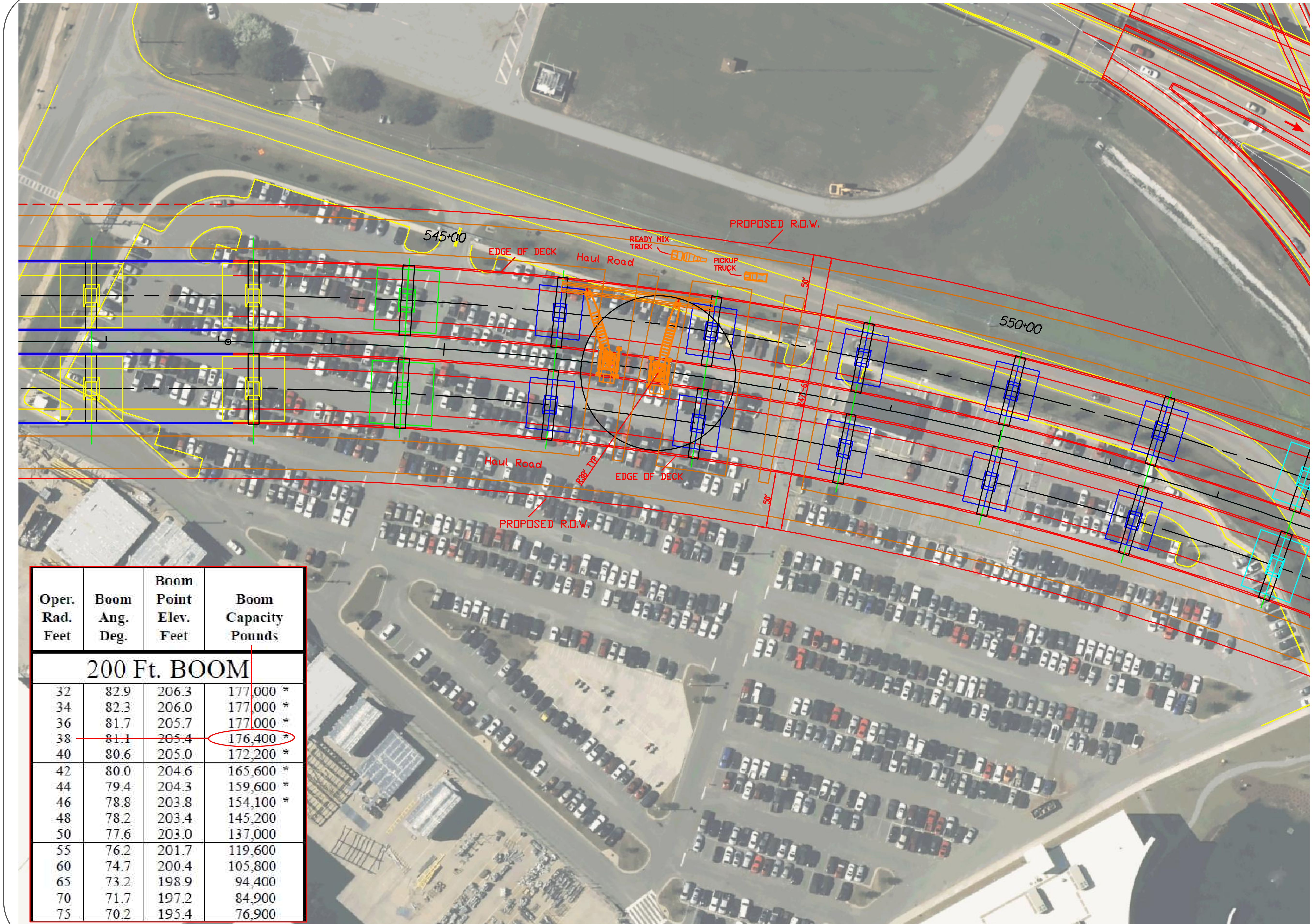
Date 04/17/2017

Scale As Noted

Sheet 04

Appendix K:

U-Beams Construction Steps



Oper. Rad. Feet	Boom Ang. Deg.	Boom Point Elev. Feet	Boom Capacity Pounds
200 Ft. BOOM			
32	82.9	206.3	177,000 *
34	82.3	206.0	177,000 *
36	81.7	205.7	177,000 *
38	81.1	205.4	176,400 *
40	80.6	205.0	172,200 *
42	80.0	204.6	165,600 *
44	79.4	204.3	159,600 *
46	78.8	203.8	154,100 *
48	78.2	203.4	145,200
50	77.6	203.0	137,000
55	76.2	201.7	119,600
60	74.7	200.4	105,800
65	73.2	198.9	94,400
70	71.7	197.2	84,900
75	70.2	195.4	76,900

General Notes

Girder Total Weight:
FUB72 Girder = 1,540 PLF * 136 LF = 209,440 LBS
Rigging = 6,000 LBS
Teardrop = 4,400 LBS * 2 = 8,800 LBS
Party Lines = (209,440 LBS/31000 Line/LBS) = 7 Lines * 2.34 LBS/LF * 200 FT = 3,276 LBS / 2 Cranes = 1,638 LBS per Crane
Total = 227,516 LBS
Girder Length = 136'

Crane 1:
Assume Manitowoc 999 w/ 200' Boom at a 38' Radius = 176,400 LBS

Crane 2:
Assume Manitowoc 999 w/ 200' Boom at a 38' Radius = 176,400 LBS

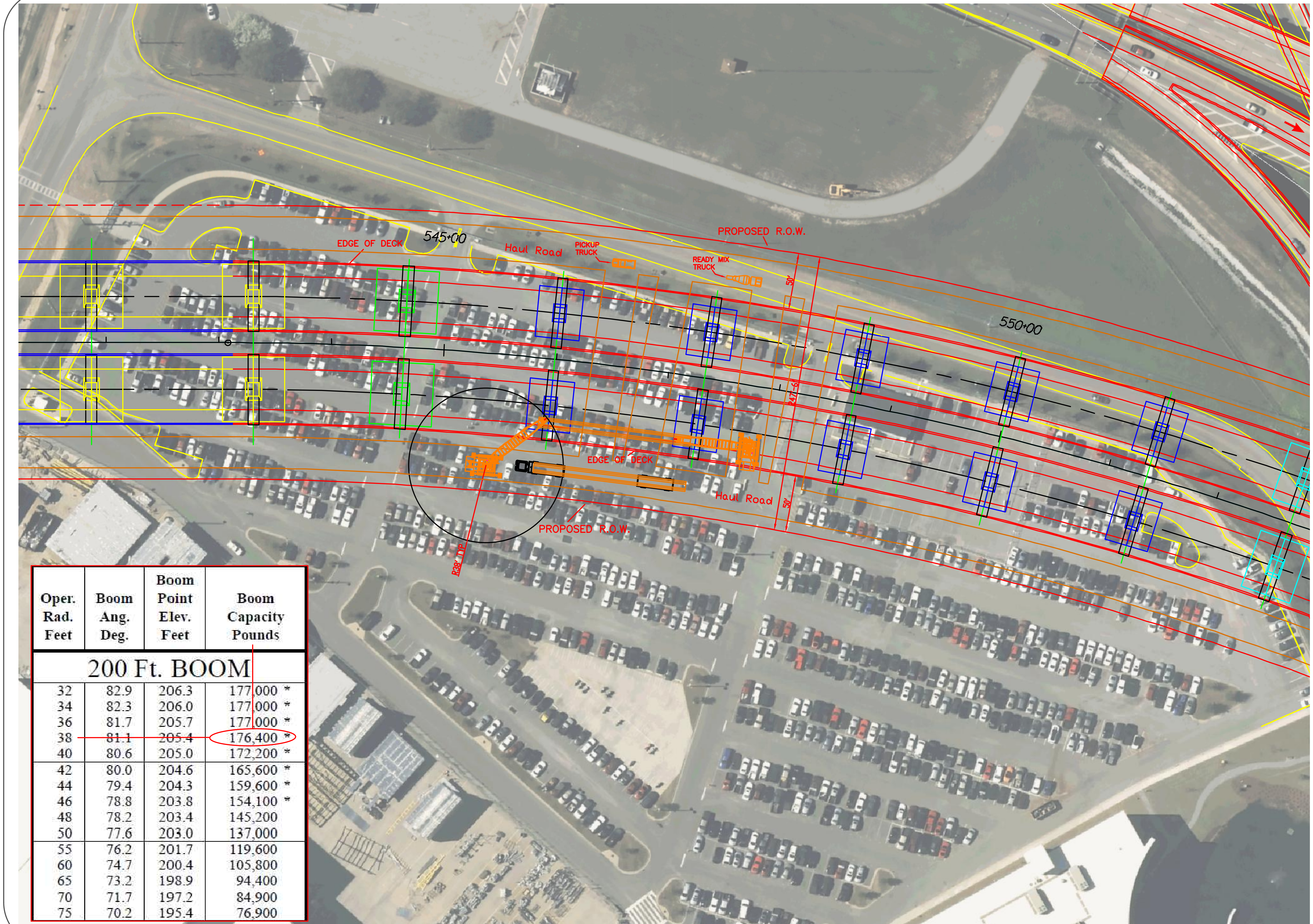
Crane Totals w/ Safety:
176,400 + 176,400 = 352,800 LBS * .75 = 264,600 LBS

No.	Revision/Issue	Date

Firm Name and Address
ARMENI CONSULTING SERVICES, LLC

Project Name and Address
Mobile River Bridge
FUB72 – Tub Girder Option
Crane Pick Drawings
Farthest Pick – Plan View

Project ACS-244
Date 04/17/2017
Scale As Noted
Sheet 01



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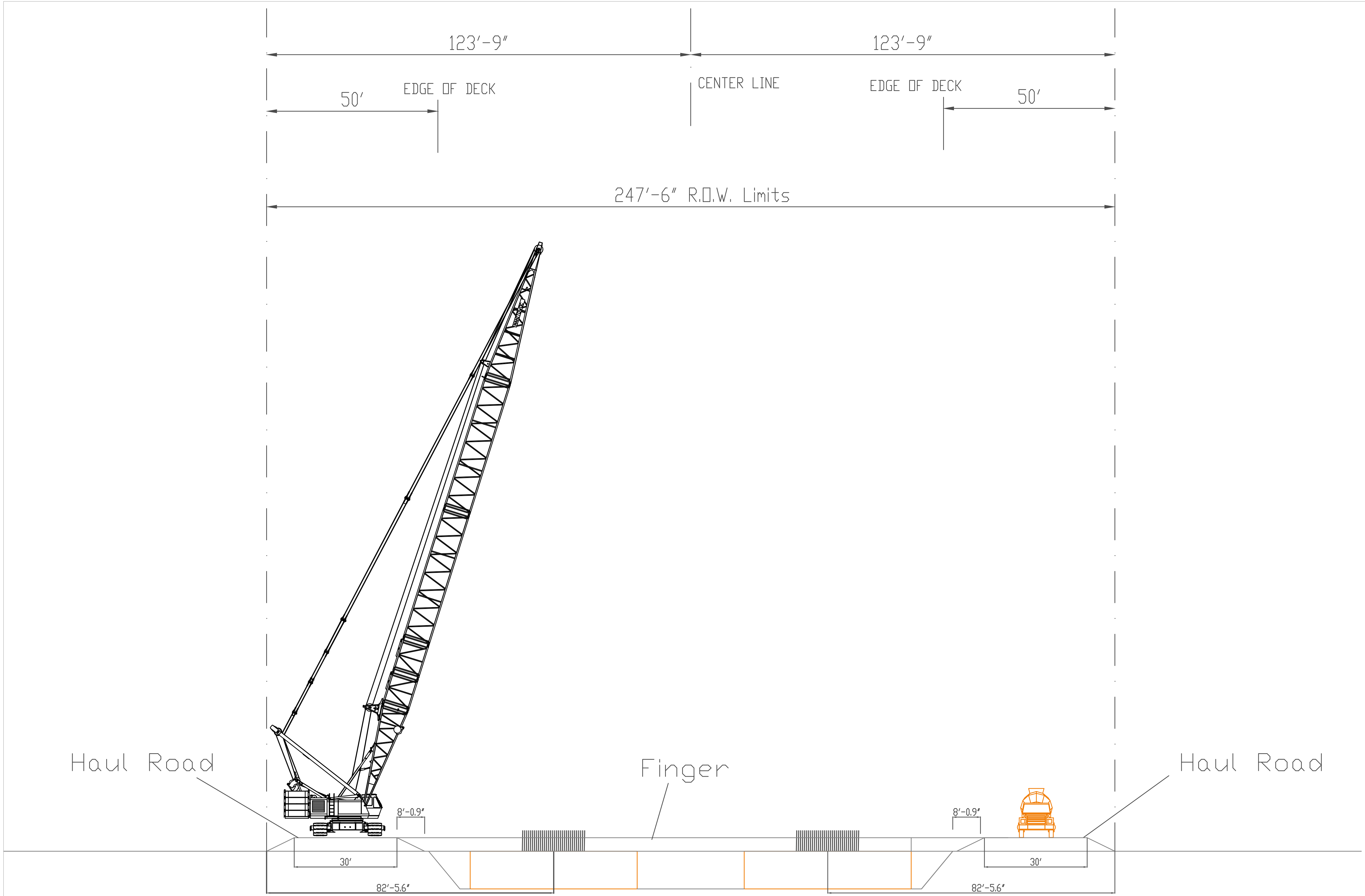
No.	Revision/Issue	Date

Firm Name and Address
ARMENI CONSULTING SERVICES, LLC

Project Name and Address
Mobile River Bridge
FUB72 – Tub Girder Option
Crane Pick Drawings
Plan View – Closest Pick

Project ACS–244
Date 04/17/2017
Scale As Noted
Sheet 02



Pier 9 Looking West

General Notes

No.	Revision/Issue	Date

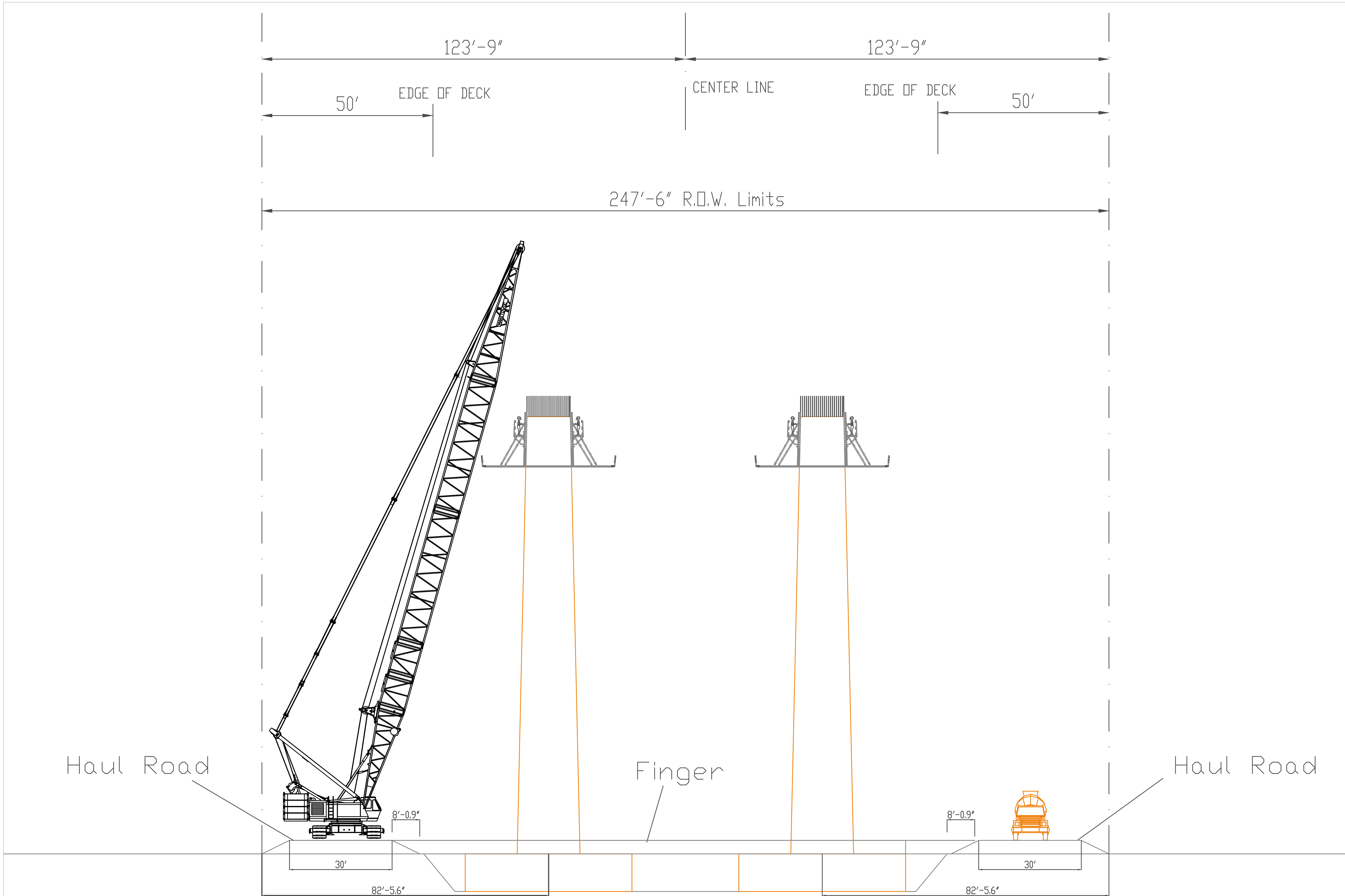
Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

Mobile River Bridge
FUB72 — Tub Girder Option
Crane Pick Drawings
Elevation View

Project: ACS-244	Sheet: 03
Date: 04/17/2017	
Scale: As Noted	



Pier 9 Looking West

General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

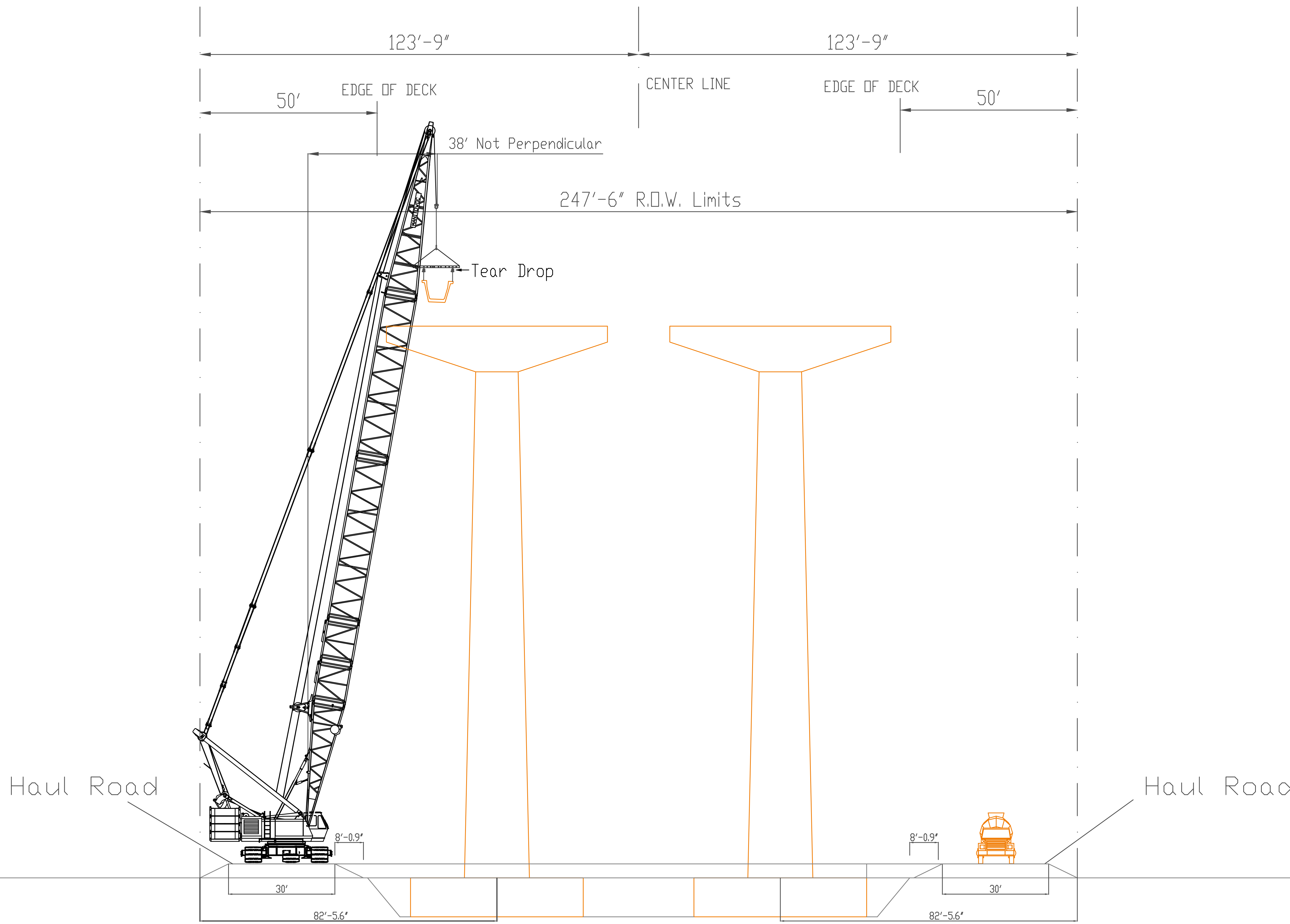
Mobile River Bridge
FUB72 – Tub Girder Option
Crane Pick Drawings
Elevation View – Lift

Project: ACS-244

Date: 04/17/2017

Scale: As Noted

Sheet: 04



General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

Mobile River Bridge
FUB72 – Tub Girder Option
Crane Pick Drawings
Elevation View – Lift

Project: ACS–244

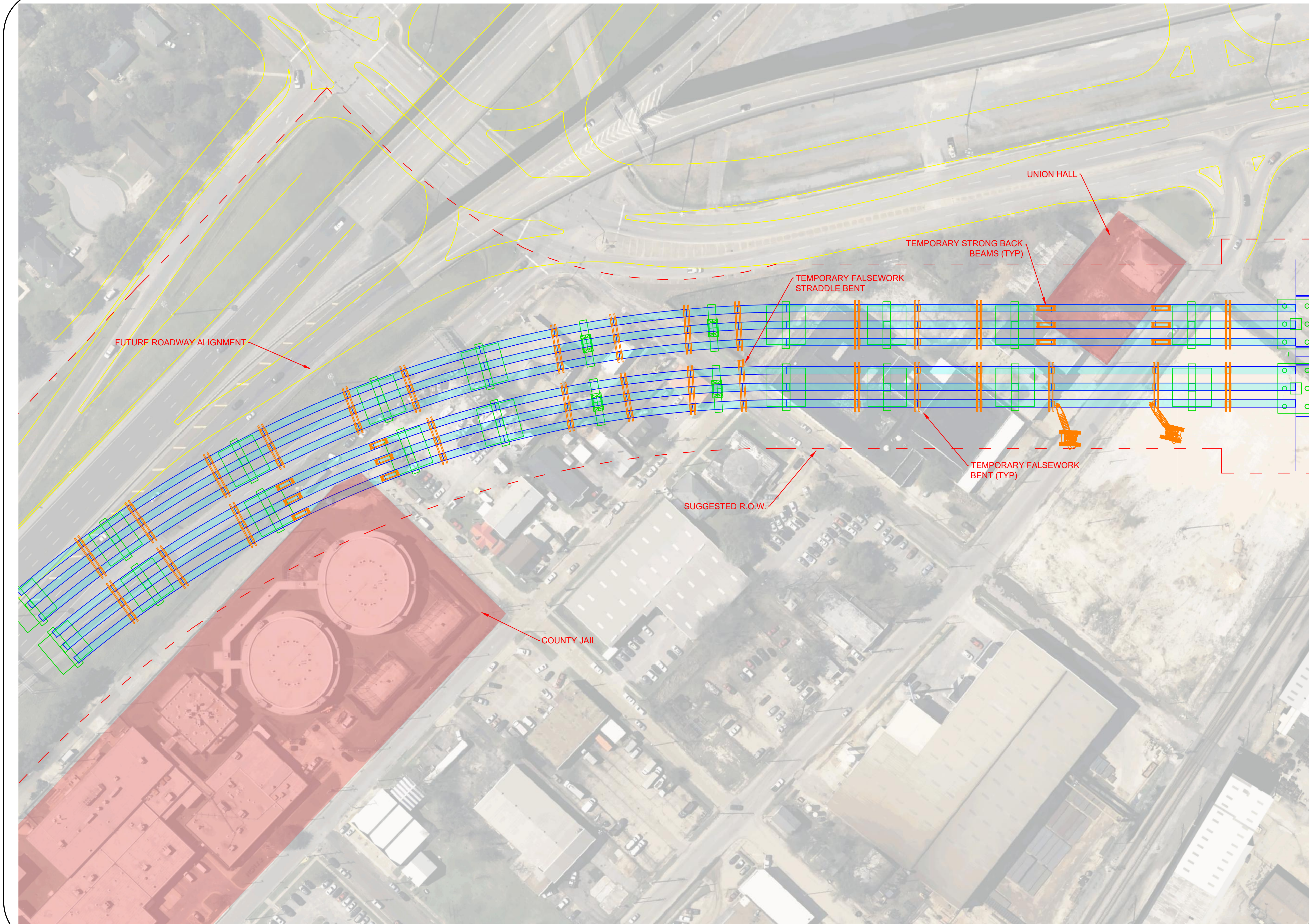
Date: 04/17/2017

Scale: As Noted

Sheet: 05

Appendix L:

U-Beams Shoring Plans



General Notes

No.	Revision/Issue	Date

Firm Name and Address

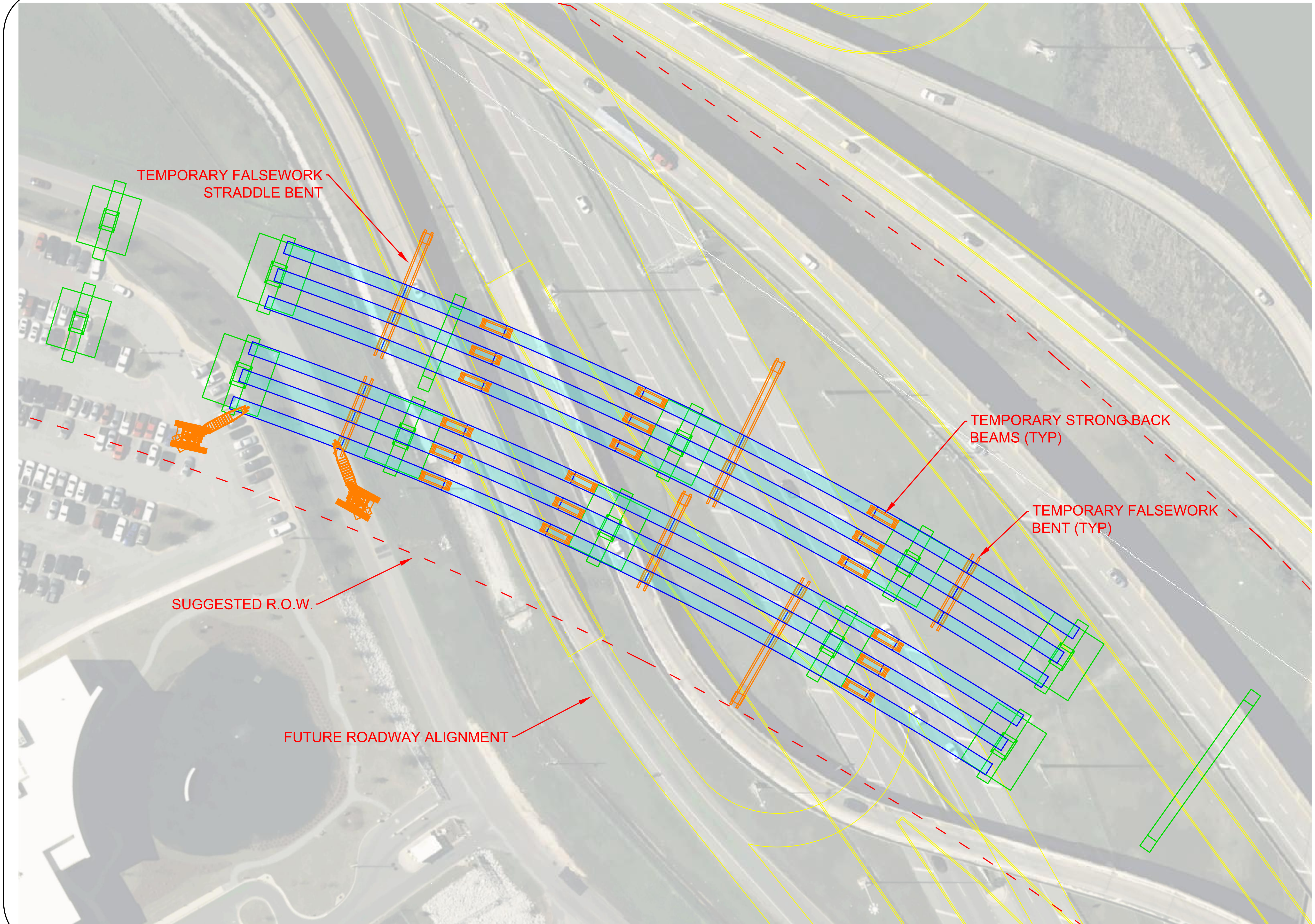
ARMENI CONSULTING SERVICES, LLC

Project Name and Address

MOBILE RIVER BRIDGE

U–BEAM FALSEWORK LOCATION STUDY – WEST

Project ACS–244	Sheet 02
Date 05/15/2017	
Scale As Noted	



General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

MOBILE RIVER BRIDGE

U-BEAM FALSEWORK
LOCATION STUDY

Project ACS-244	Sheet 01
Date 05/09/2017	
Scale As Noted	

Appendix M:

Step 2:

Comparative Cost Estimate Unit Costs

Step 2 – Unit Costs

Superstructure Unit Costs		
Item Description	Units	Unit Price
Steel Reinforcement For Bridge Superstructure (CIP Deck)	Pound	$0.95
Steel Reinforcement For Bridge Superstructure (Precast Segment)	Pound	$1.01
Bridge Concrete Superstructure (CIP Deck)	Cubic Yard	$664.13
Precast Concrete Segments	Cubic Yard	$866.53
Bridge Concrete Superstructure (CIP Closure)	Cubic Yard	$1,492.70
Bridge Concrete Superstructure (CIP Stich Pour Gore Areas)	Cubic Yard	$1,157.48
Elastomeric Bearing	Each	$2,530.00
Post Tensioning Superstructure Strand (Longitudinal)	Pound	$2.91
Post Tensioning Superstructure Strand (Transverse)	Pound	$4.43
Post Tensioning Superstructure Bar (1.375 Dia.)	Pound	$10.12
Pretensioned-Prestressed Concrete Girders, Type FUB-72	Linear Foot	$442.75
Precast Curved PCI U72 Girders	Linear Foot	$1,454.75
Disk Bearings	Each	$4,427.50
Elastomeric Bearings	Each	$3,795.00
Structural Steel, (armor plate, studs)	Linear Foot	$759.00
Modular Expansion Joints	Linear Foot	$759.00

Substructure Unit Costs		
Steel Reinforcement (Grade 60) (Pier)	Pound	$0.89
Steel Reinforcement (Grade 60) (Pile Cap)	Pound	$0.89
Steel Piling Furnished And Driven (HP 14x117)	Linear Foot	$84.76
Bridge Substructure Concrete (Pier)	Cubic Yard	$759.00
Bridge Substructure Concrete (Pile Cap)	Cubic Yard	$411.13
Post Tensioning Substructure Strand	Pound	$2.91
Post Tensioning Substructure Bar (1.375 Dia.)	Pound	$10.12

Miscellaneous		
Concrete Railing	Linear Foot	$94.88
Segmental Erection Equipment (Gantry)	Lump Sum	$2,530,000.00
Structural Excavation	Cubic Yard	$25.30