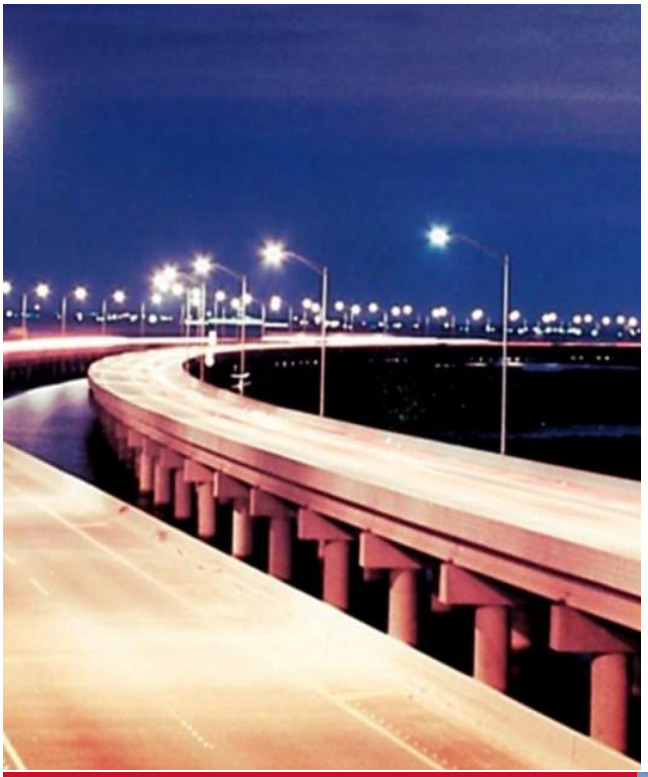

I-10 Mobile River Bridge and Bayway Widening

Preliminary Bridge Type Study – Main Span

Technical Memorandum

Mobile, Alabama
August 2, 2017

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1.0 Introduction

The I-10 Mobile River Bridge and Bayway 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 increase the capacity of the 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 main span unit over the Mobile River (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 project. The study will provide the basis for comparison of effectiveness of bridge alternatives.

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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. (Refer to the High Level Approach Spans Preliminary Bridge Type Study technical memorandum submitted on May 30, 2017, submitted under separate cover.)

2. Cable-stayed main span unit that crosses the Mobile River.

Figure 1 - Bridge Layout

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 cable-stayed main span unit, including 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.

3.0 Bridge Type Evaluation Process Methodology

As part of the I-10 Mobile River Bridge and Bayway project, a bridge type selection process was initiated in 2016 for the main span unit. 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 the bridge concepts and include:

  - Alignment
  - Construction Cost
  - Maintenance and Durability
  - Constructability
  - Structural Limitations
  - Vessel Collision
  - Austal Property
  - 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. The key design criteria and evaluation guidelines were used to select and to develop the preliminary bridge concepts.

  Once a consensus was reached on the design criteria and evaluation guidelines, several preliminary bridge 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 alternatives approved in Step 1
  - Further refine for conformance to the design parameters and 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 duration
  - Complete type, size and location report.

This report documents the work accomplished through the end of Step 2. During Step 1, seven preliminary bridge concepts were developed and evaluated, and on July 28, 2016 were presented to ALDOT. Three concepts 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 July 28, 2016 meeting can be found in Appendix A of this technical memorandum. Step 3 will be accomplished in a future phase of work.

4.0 Step 1 - Preliminary Bridge Concepts

In coordination with ALDOT, a preliminary bridge design criteria document was prepared for the various bridge alternatives and it provides the necessary technical framework for the study (See 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 – Main Span Length and Air Draft

To recommend structure types for the Mobile River Bridge main span unit, initial assumptions must be made as to span arrangements, air draft clearances and structure types to allow for an initial screening of concepts and to prepare comparative costing of the various options.
The Mobile Harbor Federal Navigation Channel is a federally authorized navigation project in the vicinity of the proposed I-10 Mobile River bridge crossing. The navigation channel provides a vital link to maritime facilities, including the Alabama Cruise Terminal that would be just north of the recommended B' alternative.

Crossing the Mobile River Channel results in important considerations in the bridge study area. Requirements of the main span's dimensions are largely influenced by navigational requirements and the ability of the structure to allow existing and future vessels to pass underneath. To accommodate passage of current and future vessels ALDOT prepared an air draft report titled "I-10 Mobile River Bridge Determination of Appropriate Air Draft Clearance" dated December 2012 (See Appendix C). The report established a 215-foot air draft clearance as ideal to maintain and promote the economic viability of the maritime industries, especially the cruise industry.

The main span of the new bridge will have to be approximately 850 feet long (skewed with respect to the navigation channel) to span the navigation channel; however, a longer main span in the range of 1,200-foot to 1,450-foot in length was selected for further study. A longer main span would allow placement of the west tower foundation landward behind the bulkhead line and the east tower foundation within shallow water. Piers in shallow water or land eliminate or minimize scour action, vessel impact loading and protection countermeasures and impacts to the USACE, United States Coast Guard (USCG), and adjacent properties. The bridge span length that would be necessary to clear-span the river was confirmed to be cost prohibitive.

Once the initial span range of 1,200-foot to 1,450-foot was established, appropriate structure types for those spans were selected to allow for a more detailed comparison of the various concepts. Figure 2 depicts the range of different bridge types as a function of span length and Figure 3 depicts the relative cost of bridge types.

As seen in Figure 2, the steel truss, through- arch, and cable-stayed options are typically the most appropriate structure types for the channel span. With the important consideration of cost, the cable-stayed option is traditionally the most economical structure type in this span range.

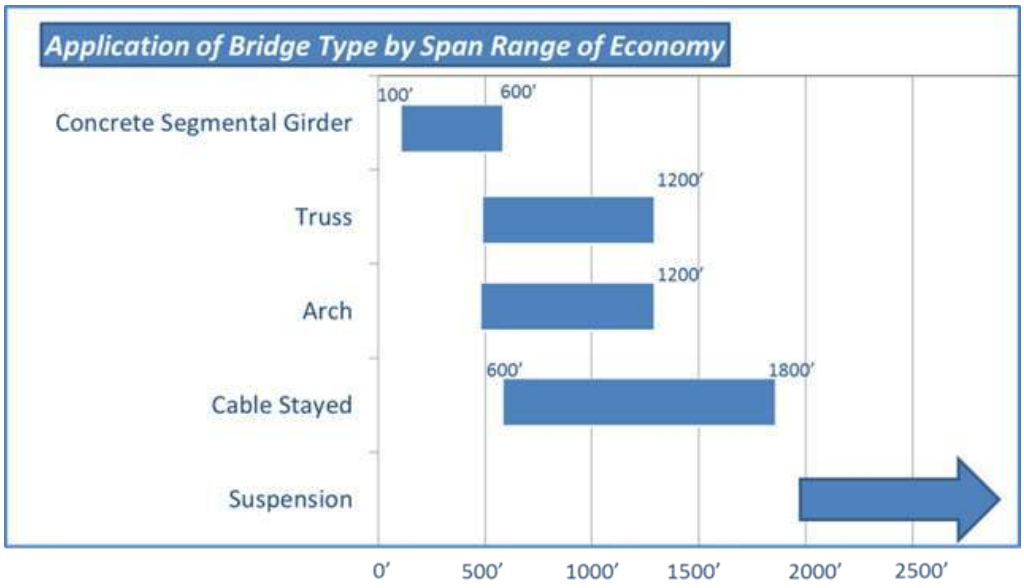


Figure 2 - *Application of Bridge Type by Span Range*

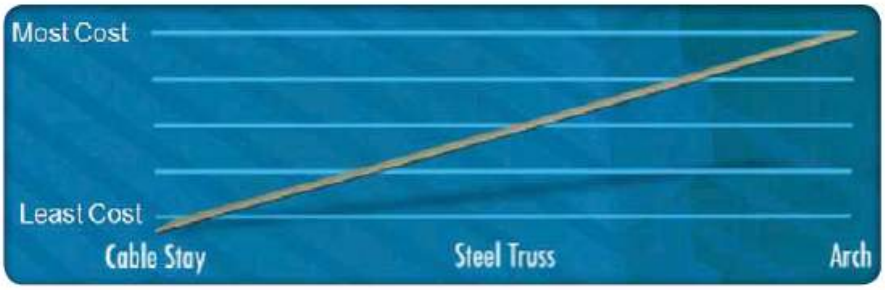


Figure 3 – Relative Cost of Bridge Type

Due to the required span length, bridge height, structural, and economic considerations a cable-stayed bridge is necessary to meet these requirements. For the resulting span range under consideration, the following seven preliminary bridge concepts were prepare and evaluated:

- Precast Concrete Segmental
- Steel Edge Girder – Split Deck
- Cast-in-Place Concrete Edge Girder – Split Deck
- Steel Edge Girder
- Steel Double Deck
- Precast Concrete Segmental – Split Deck
- Cast-in-Place Segmental – Split Deck

Refer to Appendix D for the preliminary span arrangements and cross-sections for each concept.

Similar to the high level approach span studies, main span unit bridge types were studied to determine favorable span arrangements and structure depth which meet project requirements and 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 locations of piers are

constrained to particular locations due to conflicts with the features and facilities crossed by the bridge.

4.1.2 Foundations

Concurrently with the main span 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 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.

For the main span unit the following alternatives were selected:

- Steel edge girders (lightest alternative)
- Segmental box girder (heaviest alternative)

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

Preliminary factored strength demands for the main span unit foundations were determined using this approach. The preliminary foundation loads accounted for:

- Superstructure type
- Span length
- Bridge deck width
- Wind area
- Design wind speed – a site specific Wind Climate Analysis was prepared by RWDI for this initial design phase to provide preliminary recommendations on wind loadings for the main span structure. Refer to Appendix E for a copy of the Wind Climate Analysis report.
- Vessel Collision loads – during Step 1 vessel collision loads were not applied to the preliminary concepts. Vessel collision loads were applied to the three concepts advanced to Step 2.

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

4.1.3 Substructure – Tower and 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. For uniformity during this initial screening step, the same main span unit length was used on all concepts.
- Placed towers in shallow water (near Austal Pier 5) and/or land to minimize foundation scour, vessel collision loads and improve constructability.

4.1.4 Substructure – Tower and Pier Types

Bridge towers are among the most complicated parts of bridge construction and because of their load-bearing function, they are generally also its most time-critical element. Depending on the type of bridge and on its architectural design, various different shapes of towers will be found.

Pier and tower configurations shown in Appendix D were determined based on their location, clearance requirements, anticipated gravity and lateral loads and superstructure type. Towers vary in height from approximately 510 feet to approximately 532 feet

The following pier and tower types were considered:

- Multi-column transition piers – used for transition to the high level approach spans.
- Single mast tower – used for precast segmental and split deck steel edge girder concepts.
- Diamond tower – used for steel edge girder concept.
- Double mast tower – used for steel double deck concept.

- Triple mast tower – used for split deck concrete edge and box girder concepts.

To illustrate additional feasible variations and architectural enhancements to these structural forms several illustrations were prepared; these can be found in Appendix G.

4.2 Bridge Evaluation

4.2.1 Bridge Evaluation Criteria

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

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 1 - Bridge Evaluation Criteria and Description

Qualitative and quantitative methods were used to evaluate the various structure concepts. With the exception of cost and construction duration, a qualitative scoring scale was used to assign a score based on quantifiable numbers and engineering judgment to establish a relationship between the concepts. A scale of good, fair and poor was used to rank each concept.

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

Table 2 - Sample Scoring Criteria

Multiple evaluation matrices were prepared and presented to ALDOT for the various concepts.

4.2.2 Constructability

This criterion was primarily used to rate the bridge types and their impact to the construction schedule, required construction access, active rail lines, existing and proposed traffic lanes, established construction methods and the need for cofferdams. The bridge types that have the shorter construction duration, least impact to traffic, and require the least amount of construction falsework/shoring were rated higher.

	Constructability					
Bridge Alternative	Construction Access	Disruption to Existing Traffic	Established Construction Methods	Impacts to Existing Utilities	Cofferdams	Overall
1. Precast Concrete Segmental	●	●	●	◒	◒	●
2. Steel Edge Girder - Split Deck	●	●	●	◒	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●	●
4. Steel Edge Girder	●	◒	◒	◒	●	◒
5. Steel Double Deck	●	○	○	◒	◒	○
6. Precast Concrete Segmental – Split Deck	●	●	●	◒	◒	●
7. CIP Concrete Segmental – Split Deck	●	●	●	◒	◒	●

Table 3 – Constructability Evaluation of Alternatives

4.2.3 Maintenance and Durability

Long-term maintenance typically includes routine maintenance and if necessary replacement and repair of wearing surfaces, bearings, expansion joints, structural steel coatings, superstructure tie-downs and stay cable system.

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 vehicle can reach the under deck portion of the bridge. Appendix H includes these study drawings.

The bridge types that provide good access, and minimize the use of bearings and tie-downs were rated higher.

	Maintenance and Durability								
Bridge Alternative	Accessibility	Tie-Downs	Brgs.	Deck Joints	Painting of Steel	Deck Replacement	Future Widening	Stay Cable Replacement	Overall
1. Precast Concrete Segmental	●	●	●	●	●	○	○	●	●
2. Steel Edge Girder - Split Deck	●	○	◒	●	◒	●	●	●	◒
3. CIP Concrete Edge Girder – Split Deck	●	●	◒	●	●	○	○	●	●
4. Steel Edge Girder	●	○	◒	●	◒	◒	◒	●	◒
5. Steel Double Deck	◒	○	◒	◒	○	◒	○	●	○
6. Precast Concrete Segmental – Split Deck	●	●	●	●	●	○	○	●	●
7. CIP Concrete Segmental – Split Deck	●	●	●	●	●	○	○	●	●

Table 4 – 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 Mobile's maritime traffic during construction, context design and aesthetics, and avoidance of vibration impacts to historic properties. The bridge types that best comply with this criteria were rated higher. For aesthetics, with the exception of the double deck option, all options were rated high.

	Environmental Commitments					
Bridge Alternative	**Contextual Design**	**Impacts to Maritime Traffic**	**Permitting**	**ROW**	**Superst. Construction Vibrations**	**Overall**
1. Precast Concrete Segmental	●	●	●	◒	●	●
2. Steel Edge Girder - Split Deck	●	●	●	◒	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●	●
4. Steel Edge Girder	●	◒	●	◒	●	◒
5. Steel Double Deck	◒	○	●	●	●	◒
6. Precast Concrete Segmental – Split Deck	●	●	●	◒	●	●
7. CIP Concrete Segmental – Split Deck	●	●	●	◒	●	●

Table 5 – Environmental Commitments Evaluation of Alternatives

4.2.5 Design Considerations

This criterion assesses compliance of alternatives with project specific criteria such as foundation sizes, air-draft clearances, future sidewalk, and tower height. The bridge types that best comply with this criteria were rated higher.

	Design Considerations				
Bridge Alternative	**Foundations Sizes**	**Navigational Air-Draft**	**Tower Height**	**Accom. Future Sidewalk**	**Overall**
1. Precast Concrete Segmental	◒	◒	●	◒	◒
2. Steel Edge Girder - Split Deck	●	●	●	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●
4. Steel Edge Girder	●	●	●	●	●
5. Steel Double Deck	◒	○	●	◒	○
6. Precast Concrete Segmental – Split Deck	◒	◒	●	◒	◒
7. CIP Concrete Segmental – Split Deck	◒	●	●	◒	●

Table 6 – 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.

During the bridge type study phase, two bridge related comparative construction cost estimates were prepared. Since limited structural design was performed during Step 1, the first 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. The intent of this initial comparative cost estimate was to aid in the 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.

For each of the seven 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. Comparative construction costs are based on 2017 costs.

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

Using the unit costs provided in Appendix I, the comparative cost estimate for each of the seven alternatives under consideration was developed. Table 7 summarizes the comparative construction cost for the seven alternatives in 2017 dollars. These cost estimates include a 15% contingency; however, due to the preliminary nature of the alternatives cost comparison, life-cycle costs are not included.

Bridge Alternative	Cost ($) M
1. Precast Concrete Segmental	305
2. Steel Edge Girder – Split Deck	280
3. CIP Concrete Edge Girder – Split Deck	325
4. Steel Edge Girder	295
5. Steel Double Deck	465
6. Precast Concrete Segmental – Split Deck	350
7. CIP Concrete Segmental – Split Deck	325

Table 7 - Comparative Construction Cost Estimates (2017 $)

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 that have the shorter construction duration were rated higher.

Bridge Alternative	Approx. Total Duration (Mos)
1. Precast Concrete Segmental	40-45
2. Steel Edge Girder – Split Deck	40-45
3. CIP Concrete Edge Girder – Split Deck	40-45
4. Steel Edge Girder	40-45
5. Steel Double Deck	42-50
6. Precast Concrete Segmental – Split Deck	40-45
7. CIP Concrete Segmental – Split Deck	40-45

Table 8 – Construction Schedule Evaluation of Alternatives

5.0 Step 1 – Recommendations

During Step 1 of the bridge type evaluation process, seven preliminary bridge concepts for the main span unit were developed, evaluated and presented to ALDOT on July 28, 2016. As a result of this meeting, three preliminary bridge concepts were selected to be carried forward for further study. The three preliminary bridge concepts represented all feasible bridge types and engineering solutions that addressed the ALDOT's criteria and best met the Step 1 objectives. As directed by ALDOT all structural steel alternatives were eliminated from further consideration due to concerns with life-cycle costs associated with structural steel in a coastal environment.

The bridge concepts selected for further development in Step 2 of the bridge type evaluation process are:

- Precast Concrete Segmental Box Girders (Alternative #1)
- Cast-in-Place Concrete Edge Girders (Alternative #3)
- Cast-in-Place Concrete Box Girders (Alternative #7)

The Alternatives were not renumbered between Steps 1 and 2 for consistency and to avoid any confusion when referencing to the different bridge types.

Criteria	Main Span Bridge Alternatives						
	1. Precast Concrete Segmental	2. Steel Edge Girder – Split Deck	3. CIP Concrete Edge Girder – Split Deck	4. Steel Edge Girder	5. Steel Double Deck	6. Precast Concrete Segmental – Split Deck	7. CIP Concrete Segmental – Split Deck
Initial Construction Cost ($) M	305	280	325	295	465	350	325
Construction Schedule (Mos.)	40-45	40-45	40-45	40-45	42-50	40-45	40-45
Constructability	●	●	●	◒	○	●	●
Maintenance and Durability	●	◒	●	◒	○	●	●
Environmental Commitments	●	●	●	◒	◒	●	●
Design Considerations	◒	●	●	●	○	◒	●
Overall	●	◒	●	◒	○	●	●

Table 9 – Selected Bridge Alternatives for Step 2 (Alternatives 1, 3 and 7)

6.0 Step 2 – Develop Three Bridge Alternatives

During Step 2 of the bridge type evaluation process, the bridge types that were selected 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 J for project cross-sections.

6.1.2 Foundations

For the main span unit, the West Side Foundation Alternatives Evaluation report (See Appendix F) concluded that 6 foot diameter steel pipe piles filled with reinforced concrete are considered the most likely foundation type for these elements. This foundation type provides the lowest construction risks; construction costs and pile driving vibrations for the main span unit.

However, as noted in Section 4.1.2, during Step 1 preliminary foundations were prepared for only the lightest and heaviest superstructure types:

- Steel edge 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. In addition, during preparation of the report, four lanes of traffic were used for the development of concepts for comparison of pile types. Therefore, it was necessary to develop and/or revise steel pipe pile foundation concepts for the bridge alternatives considered in Step 2. The process of developing feasible deep foundation concepts involved the following general steps:

- Develop preliminary foundation demands for the bridge types under consideration that include gravity loads, wind loads and vessel collision loads.
- Using the foundation demands and pile capacities provided, size the foundations elements for each alternative.

As discussed in Section 4.1.2, RWDI prepared a project specific wind study report that includes initial recommendations for wind loading to be used for preliminary purposes. In addition to the wind study report included in Appendix E, the team also prepared a Vessel Collision Analysis to better establish the likelihood and magnitude of the vessel collision loading on the main span foundations. Due to the size of the report it was submitted under separate cover to ALDOT; however, on August 8, 2016 a summary progress report was provided to ALDOT and a copy can be found in Appendix K.

6.1.3 Substructure

Pier and tower locations and shapes used during Step 1 were reviewed and revised as necessary using the latest horizontal alignment and engineering analysis. The main technical tasks consisted of:

- East Tower Location: An exhibit illustrating the proposed east tower location near (Pier 5 of the Austal property) was prepared for ALDOT's review and approval. Locating the east tower at this location has several advantages including: shortens the required main span length, places the tower foundation in shallow water, and the river bottom at this location acts as a natural barrier against vessel collision. Refer to Appendix L for the exhibit approved by ALDOT.

- East Tower Access Study: Armeni Consulting Services, LLC prepared a series of study drawings to demonstrate that construction access is feasible for this tower location. Refer to Appendix M for the study drawings. Following this study, it was determined that

the access or right-of-way needed to construct the towers would be the responsibility of the contractor.

- Tower Design Evolution (Superstructure Stability): Tower configurations were evaluated for their impact on the aerodynamic behavior of the superstructure. From previous wind tunnel testing of similar structures it is known that the Precast Segmental Dual Box Girder cross-section (Alternative #1) has a high tolerance to wind loading due to its high torsional resistance and that due to their inherent flexibility, edge girder superstructures (Alternative #3) tend to be more sensitive to wind instability. Alternative #7 consists of cast-in-place box girders with a torsional resistance somewhere between that of Alternatives #1 and #3.

  On this basis, the tower configuration study focused on Alternatives #3 and #7. During preliminary design, an indication of acceptable aerodynamic behavior is the ratio of first torsional mode frequency to first heave mode frequency > 1.5; to calculate these parameters 3-D computer models were developed for the main span units of Alternatives #3 and #7.

  To establish a baseline case, the triple-mast tower shape assumed in Step 1 was used. This tower configuration resulted in a ratio of 1.22 which was considered unacceptable. Next, a sensitivity study was performed to better understand the effects of:

  - Transverse struts located at different upper tower heights.
  - Angling the cable in the transverse direction.
  - Canting the upper tower legs inwards and outwards of the bridge deck.
  - Adding back span piers.
  - Making the superstructure monolithic with the tower.

  After analyzing numerous tower configurations, the study concluded that the most effective solution for superstructure stability of Alternatives #3 and #7 consists of towers with the upper legs canted inwards commonly referred to as a diamond or inverted-y towers. It is important to recognize; however, that once sectional models are developed during final design of the project it may be possible to explore other tower shapes and/or means of providing additional stability to the superstructure cross-section. This analysis was beyond the scope of Step 2.

- FAA Requirements (Tower Height): FAA requirements need to be considered due to the project's proximity to the Mobile Aeroplex at Brookley. Since the tower height is likely to be higher than 499 feet AGL it will be considered an obstruction per Title 14 – Aeronautics and Space Code of Federal Regulation (CFR) Part 77.17. To receive a "no-hazard" determination especially for structures taller than 499 feet it needs to be shown to the FAA that the bridge towers, lights, etc. do not cause aviation hazard. During the next phase of the project it will be necessary to apply for the necessary FAA permits.

In summary, the following pier, tower and stay cable layouts were used for the bridge alternatives refined in Step 2:

- Multi-column transition piers – used for transition to the high level approach spans on all bridge alternatives.

- Single mast tower with central plane(s) of stays –Precast Concrete Segmental (Alternative #1).
- Diamond and Inverted-Y tower with Outboard Planes of Stays – Cast-in-Place Edge and Concrete Girders (Alternatives #3 and #7).

6.1.4 Span Arrangement

Unlike Step 1 where a constant main span length was applied to all concepts, in Step 2 it was desired to optimize the span length for the three remaining alternatives. Optimization of the main span units consisted of the following tasks:

- For Alternative #1 (Precast Concrete Segmental with Single Mast Tower), the west tower foundation was rotated so that it is oriented parallel to the river's bulkhead thus permitting the back span and consequently the main span to be shortened. This however, required a review of the vessel collisions loads on the west tower. The vessel collision analysis was updated and revised loads were reviewed to assess their impact on the foundations.

- For all three alternatives, the Team received ALDOT permission for a permanent closure of Madison Street. This allowed placement of transition and/or intermediate piers at or near this location. Piers placed at this location resulted in a better and more economical span layout for all three alternatives.

- Preparation of a back span location study to optimize the back span length for each alternative. The back span length influences the behavior of the main span unit in several ways:

    - Uplift forces need to be resisted at the back span piers.
    - Live load stress range on stay cables.
    - Structure bending moments.
    - Aerodynamic behavior.

    In general, the benefit of reduced uplift with a longer span is out-weighed by other effects such as:

    - A longer main span unit means a higher construction cost.
    - Increase of live load force range on stay cables.
    - Increased bending moments.

A copy of the study can be found in Appendix N of this memorandum.

On the basis of the additional analysis and studies discussed in this section, the bridge layouts for each of the remaining alternatives were revised and used to calculate updated quantities and construction costs.

The main span unit span layouts by bridge alternative are:

- Alternative #1 (Precast Concrete Segmental): five span unit with 152-560-1350-560-152 foot spans.

- Alternatives #3 and #7 (Cast-in-Place Edge and Concrete Box Girders): three span unit with 585-1380-585 foot spans.

Bridge Layouts for Alternatives #1, #3, and #7 can be found in Appendix O.

6.2 Constructability

All three alternatives considered in Step 2 were evaluated with respect to their erection schemes and potential impacts to the project study area. Impacts to the project study area include impacts to the environment, existing roads, proposed roads, railroads and the navigation channel and are generally created by the construction and location of new bridge foundations, any temporary falsework/ shoring towers, large construction equipment such as cranes, and temporary closures of the navigational channel to accommodate construction activities.

In addition to the revised bridge layouts and cost estimates, another result of the constructability review of alternatives performed in Step 2 was the development of a variant of Alternative #1 (Precast Concrete Segmental). Alternative #1 as presented in Step 1 would be constructed utilizing temporary piers for the back spans and uni-directional construction from the towers for the main span over the navigational channel. After further constructability and construction costs review of Alternative #1, it became apparent that constructing Alternative #1 in a balanced cantilever fashion would have the following benefits:

- Eliminates the need for tall and expensive temporary piers for construction of the back spans.
- Temporary piers needed for construction of the east back span would partially block access to existing industrial facilities.
- Reduction of temporary works could potentially reduce main span construction cost.

The variant of Alternative #1 will be referred to as Alternative #1A in this study and basically consists of an identical structural scheme as Alternative #1. However, due to the use of balanced cantilever construction some of the key structural members sizes and quantities were adjusted and captured in the construction unit costs estimates presented in Section 6.5.

All four alternatives were also evaluated based on their ability to maintain navigational 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 all taken into consideration. Structure type alternatives that utilized top down construction methods to minimize disruptions to traffic were preferred.

For Construction Sequence drawings prepared for each alternative refer to Appendix O.

6.3 Maintenance and Durability

An important aspect of cable-stayed bridges is that provisions need to be made for maintenance access of the superstructure, substructure and stay cable system. Maintenance elements need to be accounted for in the design and provided for and installed as part of the project and be fit for use once the bridge is complete. Different strategies have been used in the past to provide access such as:

- Maintenance travelers capable of accessing the full underside of the bridge
- Snooper trucks
- Maintenance walkways (cat-walks) that are continuous for the full length of the main span unit
- Maintenance ladders primarily used inside the towers
- Maintenance Elevators
- Maintenance lighting
- Access hatches along the bottom soffit of the box girder

The specific maintenance access provisions will vary with superstructure type and tower shape; however, for all alternatives under consideration maintenance access provisions are feasible.

Deck replacement is typically not required for cable-stayed bridges; instead, a bridge design service life is selected by bridge Owners. The bridge would be designed to provide a structure that meets the required design service life either by selecting materials with reduced corrosion potential, by selecting materials and details which resist degradation or by other means acceptable to the bridge Owner. In many cases, a detailed Corrosion Protection Plan for the main span unit and the approach spans is also prepared.

Structure type alternatives that minimize superstructure cracking and prevent chlorides from penetrating the concrete and reaching the reinforcing steel through the use of post-tensioning or other means were rated higher.

6.4 Context Sensitive Design and Aesthetics

Revised architectural concepts of the tower shapes considered on all four Step 2 alternatives (#1, #1A, #3 and #7) were developed and presented to ALDOT. These revised tower concepts have been included in Appendix P.

6.5 Comparative Construction Cost Estimates

A second cost estimate was prepared for the three alternatives carried forward from Step 1 plus Alternative #1A developed in Step 2. Major construction item quantities were computed or revised and a15% 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 Q, the comparative cost estimate for each of the alternatives under consideration was developed. Table 10 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 cost comparison, life-cycle costs are not included.

Bridge Alternative	Cost ($) M
1. Precast Concrete Segmental	**277**
1A. Precast Concrete Segmental	**273**
3. CIP Concrete Edge Girder – Split Deck	**275**
7. CIP Concrete Segmental – Split Deck	**281**

Table 10 - Comparative Construction Cost Estimates (2017 $)

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 main span unit 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, least impact to navigational channel and probable lowest 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.

The analysis of the structure type alternatives concludes that a cast-in-place edge girder superstructure (Alternative #3) supported by diamond or inverted-y shape towers is considered the most likely structure type for the main span unit.

This structure type provides the following benefits:

- Can be built from above without disrupting the navigation channel.
- Since this structure type is built from above it also has minimal impact on the environment, utilities and railroads.
- Construction cost is similar to the other alternatives.
- Does not require fabrication and delivery of large precast elements on the navigational channel or in an urban area.
- High aesthetic value to meet the DEIS commitments.
- Easily accommodates geometric constraints and number of lanes.
- Shallowest superstructure depth of all alternatives – allows optimization (lowering) in the future of the bridge profile which translates to construction cost savings for the main span unit and high level approach spans.
- Construction duration similar to other alternatives.

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, main span length and construction sequence over the navigational channel. 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.

	Main Span Bridge Alternatives			
Criteria	1. Precast Concrete Segmental	1A. Precast Concrete Segmental	3. CIP Concrete Edge Girder – Split Deck	7. CIP Concrete Segmental – Split Deck
Initial Construction Cost ($) M	277	273	275	281
Construction Schedule (Mos.)	47	48	50	50
Constructability	◒	◒	●	●
Maintenance and Durability	●	●	●	●
Environmental Commitments	●	◒	●	●
Design Considerations	●	●	●	●
Overall	●	●	●	●

Table 11 – Recommended Alternative for Step 3 (Alternative 3)

On January 19, 2017 a meeting was held with ALDOT to present the results of Step 2. Four alternatives reflecting feasible bridge types were identified with Alternative #3 selected as best meeting the objectives of Step 2 and selected to be carried forward into Step 3. A copy of the PowerPoint presentation and meeting minutes for the January 19, 2017 meeting can be found in Appendix R of this technical memorandum.

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:

July 28, 2016 Meeting

- PowerPoint Presentation
- Meeting Attendance
- Meeting Minutes

Attendance List

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 ERICKSON	ALDOT-SW Region	251-470-8364	erickson@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	colquett@dot.state.al.us
RANDALL MULLINS	" "	6004	MULLINSR@ " "
Patt Hickox	HDR	251-295-6874	patrick.hickox@hdrinc.com
on the phone:	✓ Justin Doonik,	Bradley Touchstone, ✓	
	✓ Jake Perkins,	Mike Lamont ✓,	
	✓ Barth Hendricks (MM)		
Don Arkle			

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

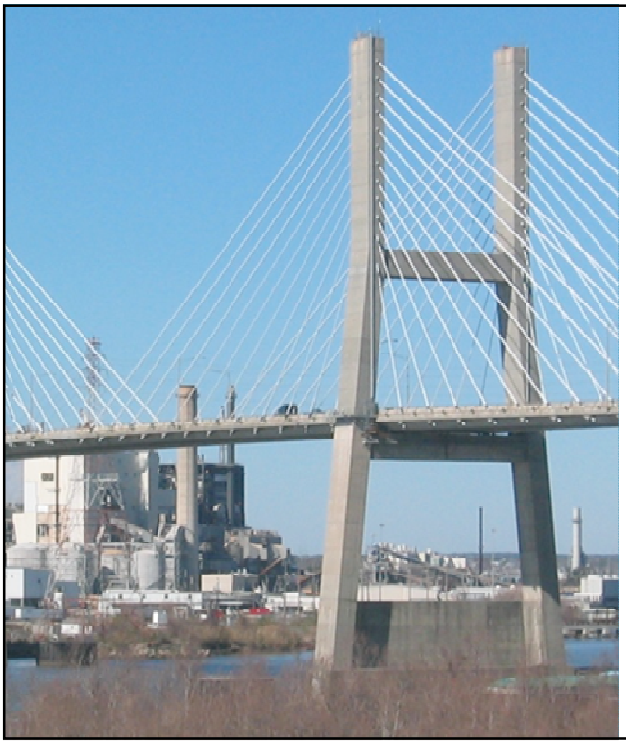
b. ALDOT expressed concern over constructability issues associated with drilled shaft and eliminated all drilled shaft options for future considerations.
c. Tim Colquett will consider steel piling pending the results from studying the corrosiveness of the soil. Sam Sternberg will take samples and send 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.

Mobile River Bridge

Main Span Bridge Type Selection Process and West Side Foundation Study

HDR

July 28, 2016



Agenda

01 **Main Span Bridge Type Selection Process**

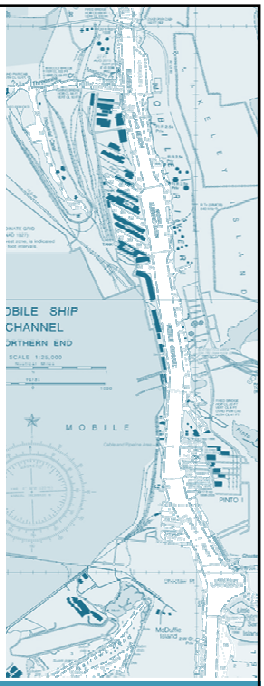
02 **West Side Foundation Study**

01 Main Span Bridge Type Selection Process

General Presentation Outline –

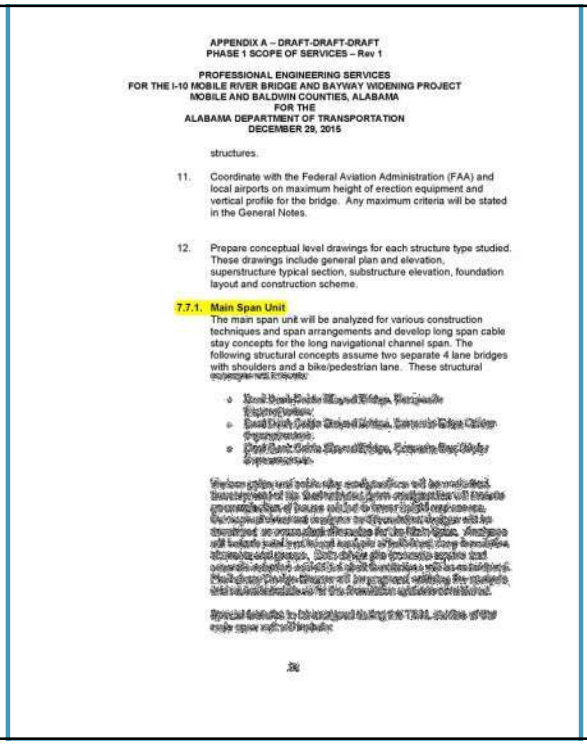
Main Span Bridge Type Selection Process

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



1. 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

structures.

11. Coordinate with the Federal Aviation Administration (FAA) and local airports on maximum height of erection equipment and vertical profile for the bridge. Any maximum criteria will be stated in the General Notes.

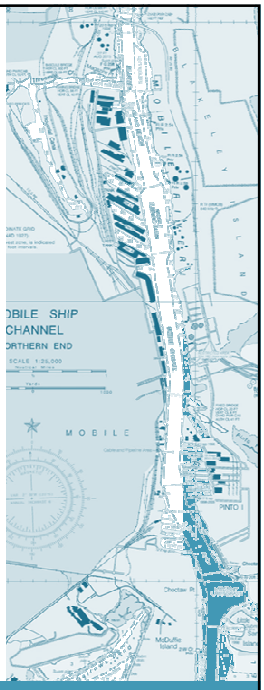
12. Prepare conceptual level drawings for each structure type studied. These drawings include general plan and elevation, superstructure typical section, substructure elevation, foundation layout and construction scheme.

7.7.1. Main Span Unit
The main span unit will be analyzed for various construction techniques and span arrangements and develop long span cable stay concepts for the long navigational channel span. The following structural concepts assume two separate 4 lane bridges with shoulders and a bike/pedestrian lane. These structural

2. Bridge Type Selection Process

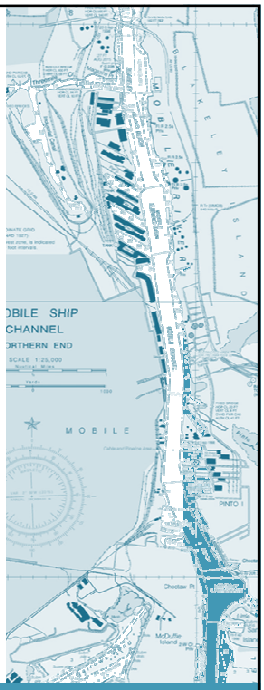
- Three Step Process which Involves Developing and Analyzing Numerous Bridge Concepts
- **Step 1 – Develop Preliminary Bridge Concepts**
- Step 2 - Develop 3 Bridge Type Alternatives
- Step 3 – Develop Preferred Bridge Alternative for TS&L

We are Here



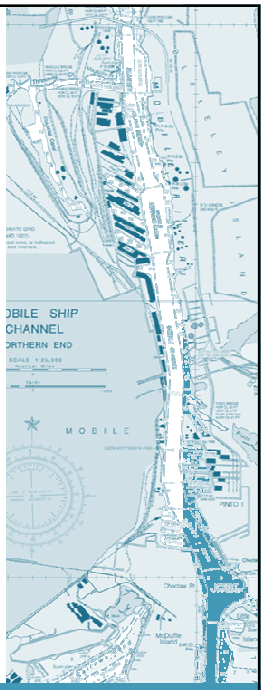
2. Bridge Type Selection Process

- Step 1 – Develop Preliminary Bridge Concepts
  - Feasible Bridge Types
  - In Coordination with ALDOT develop Key Design Criteria and Guidelines to be Used to Evaluate Bridge Concepts:
    - Vertical and Horizontal Clearance
    - Construction Cost
    - Maintenance and Durability
    - Navigational Limitations
    - Constructibility
    - Structural Limitations
    - Environmental Commitments



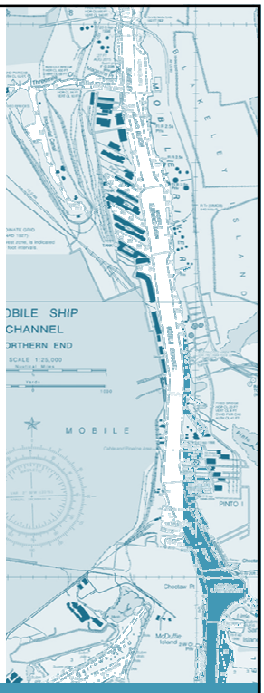
2. Bridge Type Selection Process

- Step 1 – Develop Preliminary Bridge Concepts
  - Desired Outcome:
    - Design Parameters (Mandatory Requirements)
    - Design Guidelines (Desirable Objectives); and
    - 3 Preliminary Bridge Concepts for Further Consideration in Step 2



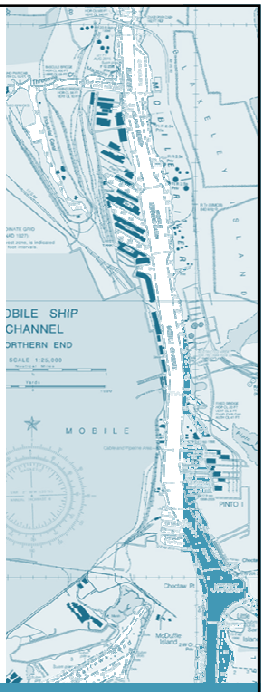
2. Bridge Type Selection Process

- Step 2 - Develop 3 Bridge Type Alternatives
  - Perform Conceptual Engineering on 3 Alternatives Approved in Step 1
  - Further Refine for Conformance to the Design Parameters and Best Meeting the Design Guidelines
  - Prepare Cost Estimates
  - Prepare and Present Updated Evaluation Matrix
  - Desired Outcome:
    - Select Final Bridge Alternative



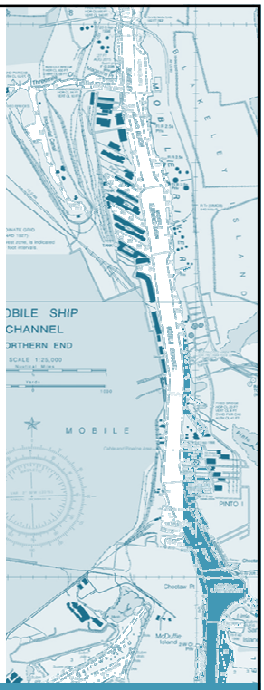
2. Bridge Type Selection Process

- Step 3 – Develop Preferred Bridge Alternative for TS&L
  - Refine Final Bridge Alternative
  - Prepare Cost Estimate
  - Construction Time
  - Complete Bridge Type Selection Report

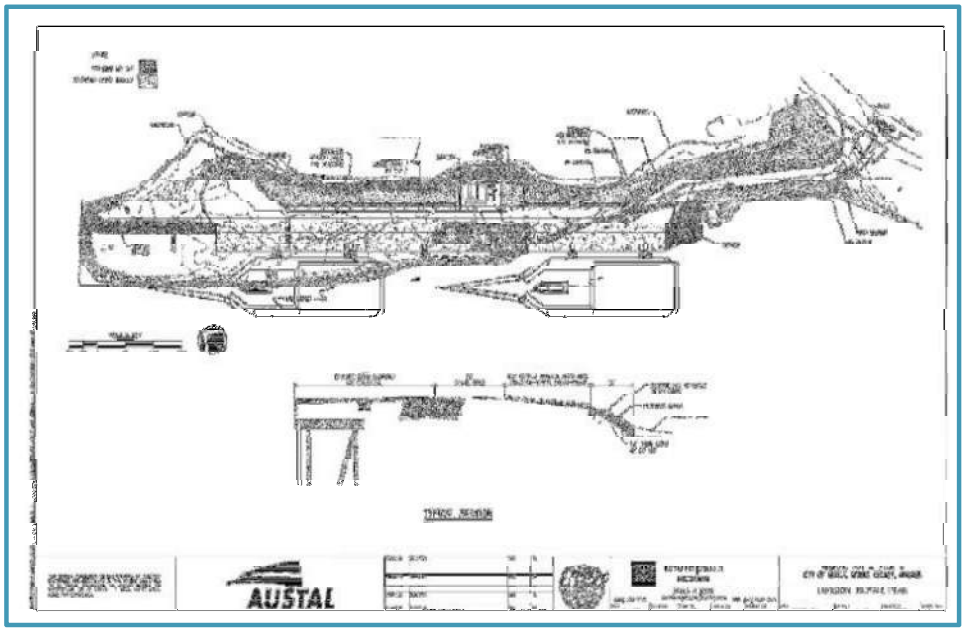


3. Design Guidelines/Constraints

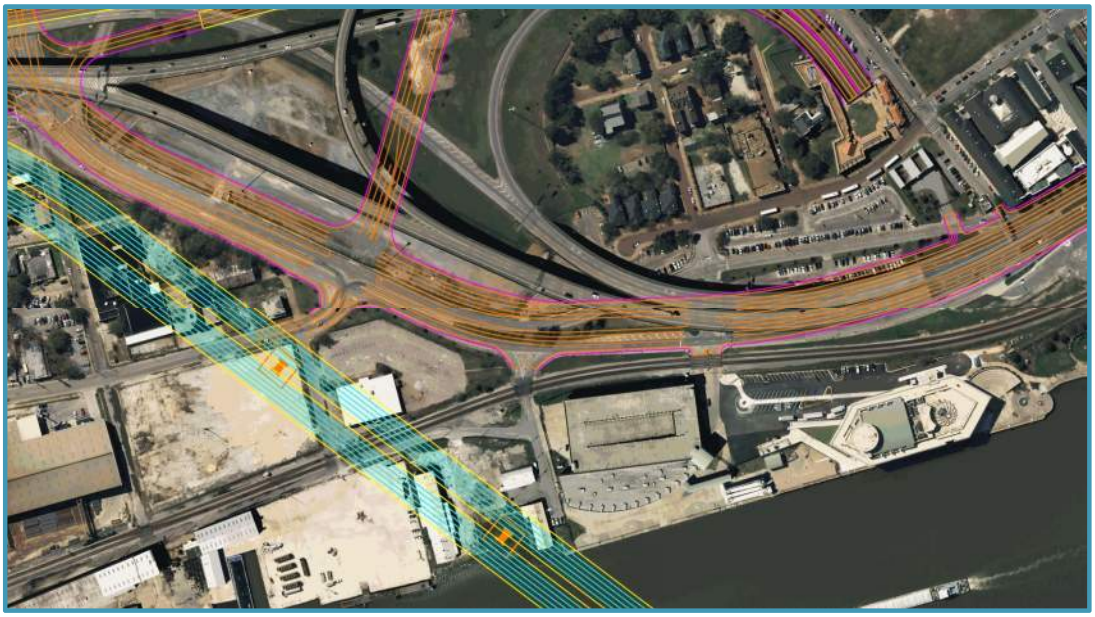
- Bridge Structural Configuration
  - Each Roadway is Composed of four 12-foot-wide Lanes and two 12-foot-wide shoulders
  - A minimum Vertical Clearance of 215-feet
  - A minimum Horizontal Clearance of 600-feet.
- Have no Permanent Effect on River Navigation
- Vessel Collision Impact Resistance
- Pier Placement Locations – Span Length Driver
- Tower Heights – FAA Restrictions
- DEIS Environmental Commitments



3. Design Guidelines/Constraints – AUSTAL Property



3. Design Guidelines/Constraints – West Side



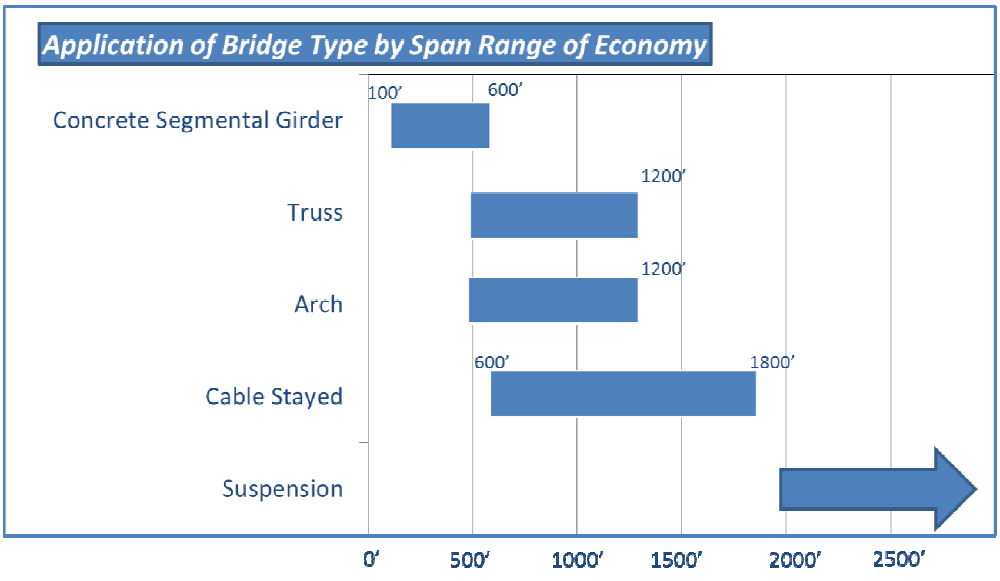
Mobile River Bridge
Bridge Types – Evaluation Matrix

Main Span Criteria

Construction Cost	Constructability	Maintenance and Durability	Construction Schedule	Environmental Commitments	Design Considerations
Initial Construction Cost	Construction Access Disruption to Existing Traffic / Maintenance of Traffic Established Construction Methods Impacts to Existing Utilities Impacts to Railroads Cofferdams	Accessibility Tie-Downs Bearings Deck Joints Painting of Steel Deck Replacement Flexibility for Future Widening Stay Cable Replacement		Contextual Design and Aesthetics Impacts to Mobile's Maritime Traffic during Construction Permitting ROW Construction Vibrations	Foundation Sizes Navigational Air-Draft Clearance Accommodates Future Sidewalk

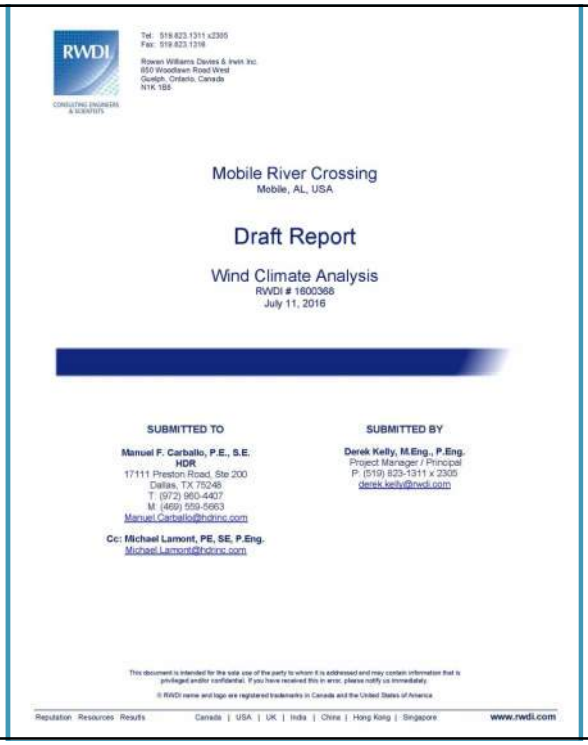
4. Superstructure Types Considered

- Cable-Stayed Bridges - Most Economical Structure Type for this Span Range



5. Design Approach

- Develop Feasible Structural Concepts
- Establish Preliminary Span Layout
- Perform Vessel Collision Impact Studies
- Perform Site Specific Wind Analysis



RWDI

Mobile River Crossing
Mobile, AL, USA

Draft Report

Wind Climate Analysis
RWDI # 1600368
July 11, 2016

SUBMITTED TO

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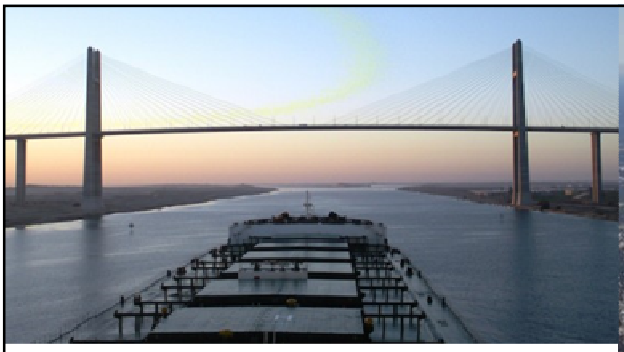
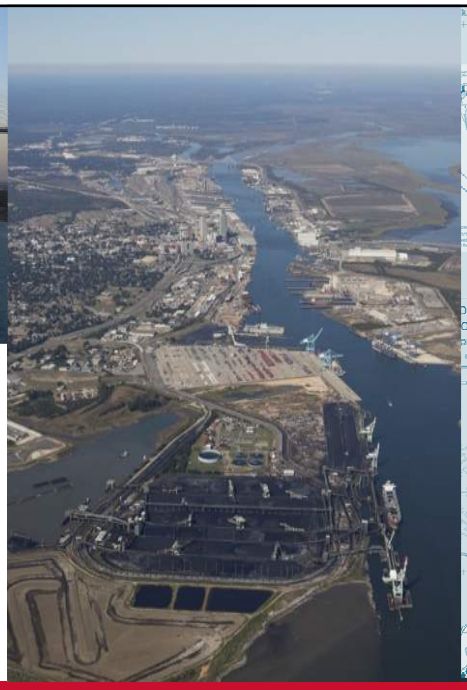


Mobile River Bridge

Vessel Collision Analysis

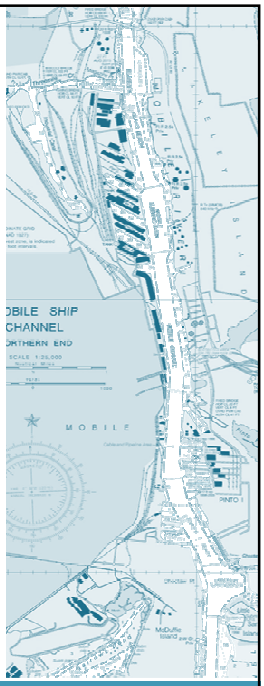
June 24, 2016

HDR



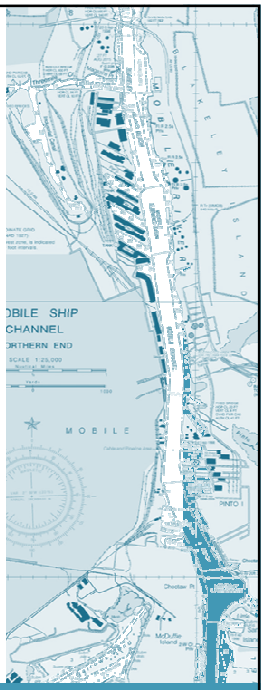
General Presentation Outline

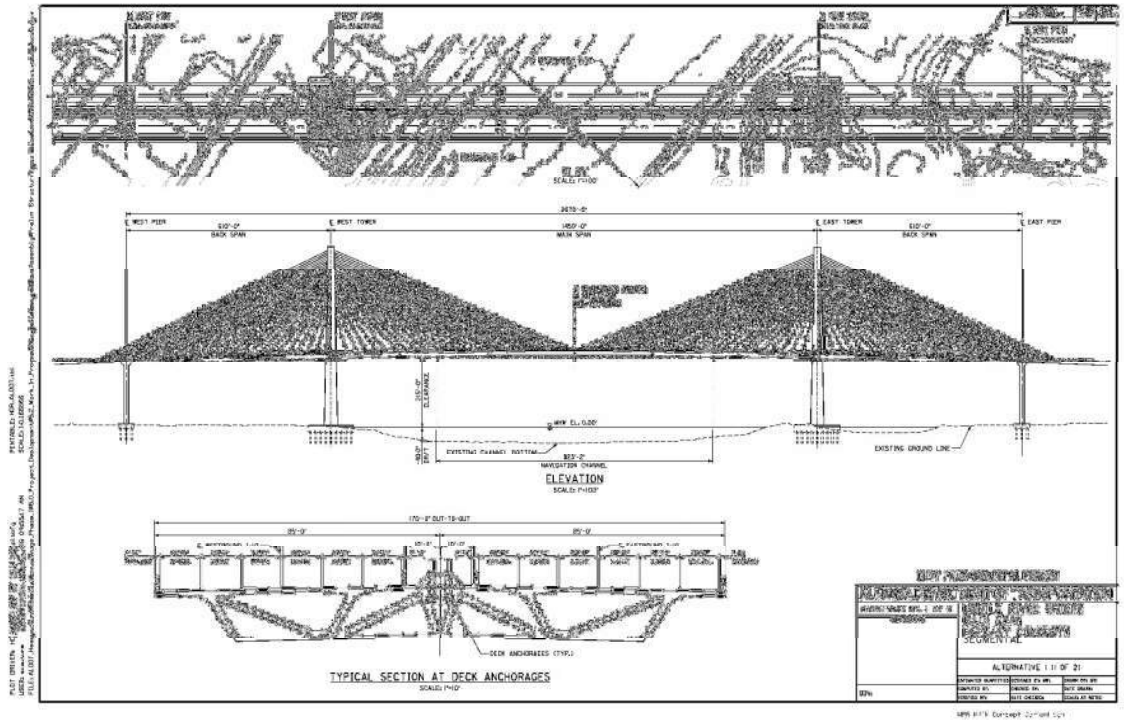
1. Purpose of Analysis
2. Studies to Date
3. AASHTO Vessel Collision Analysis Approach/Methodology
4. Mobile River Bridge Site Vessel traffic
5. Application of Loads to Structure
6. Considerations for Loading
7. Acceptance Frequency (AF) Analysis for Site
8. Future Fleet Considerations
9. Summary and Recommendations
10. Comments/Input

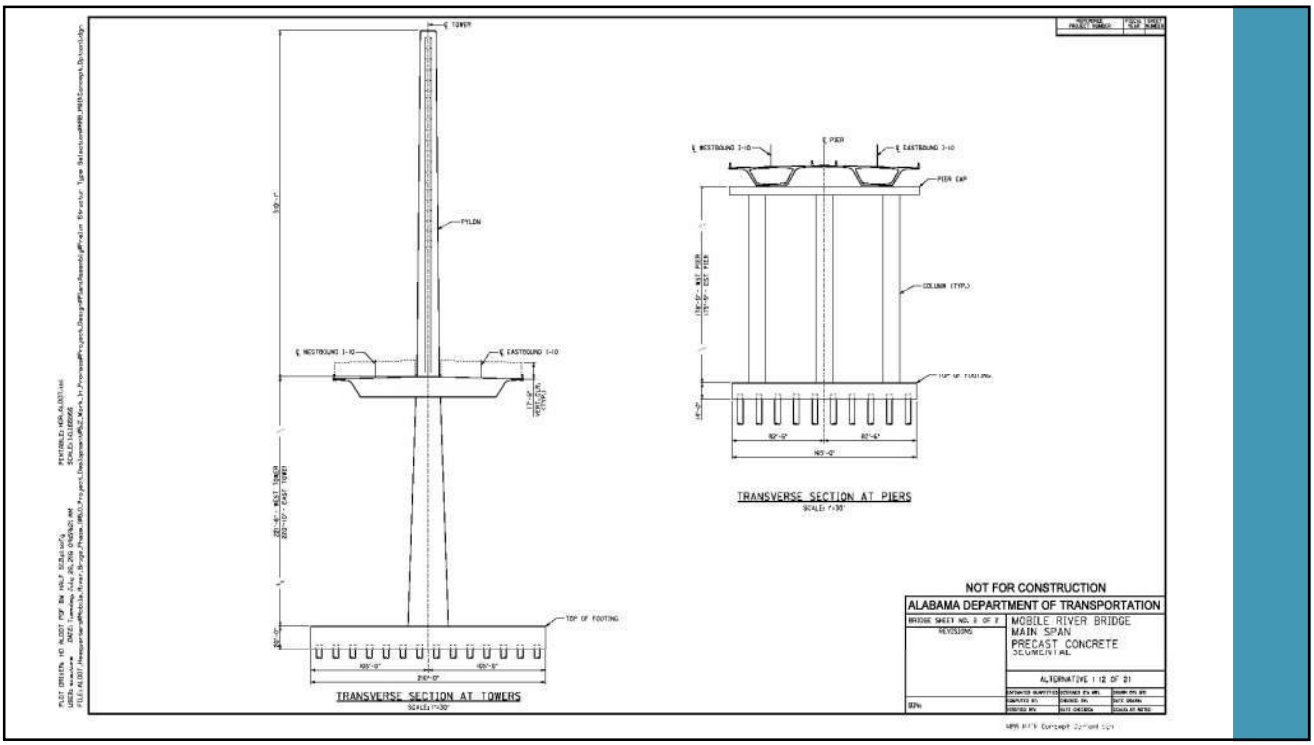


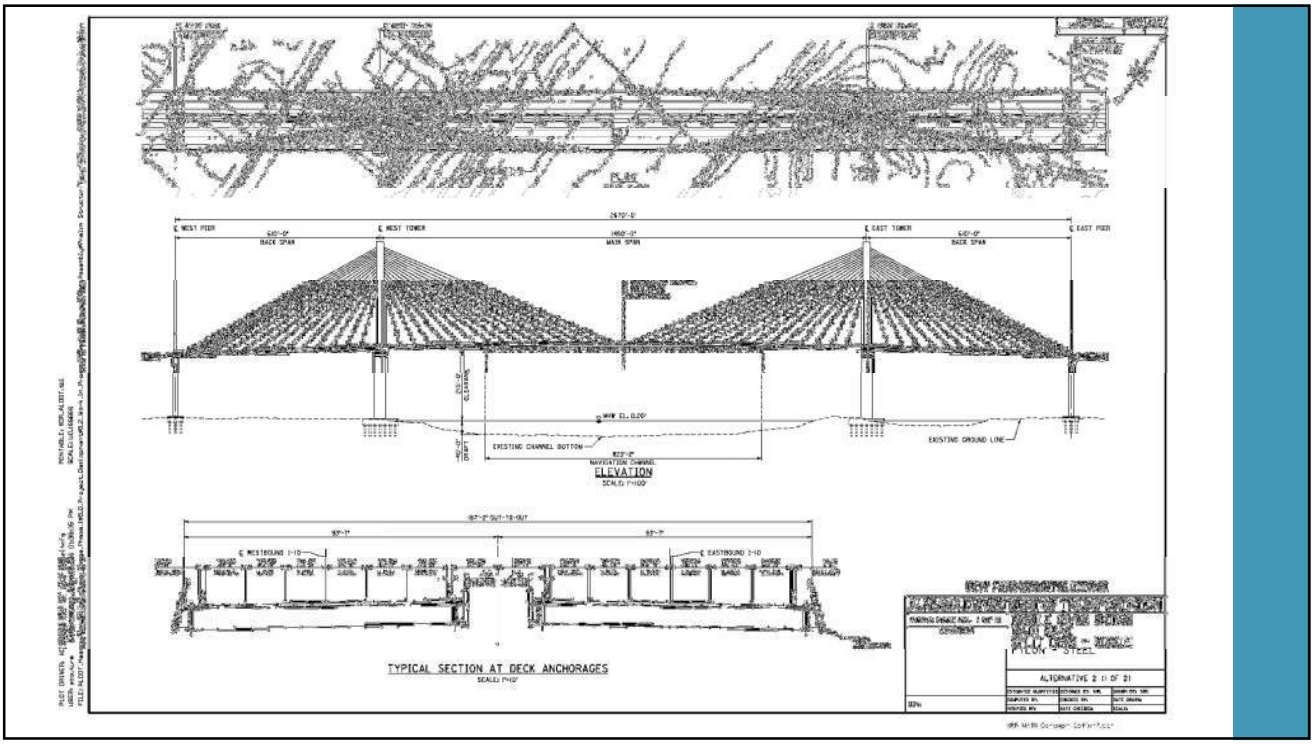
6. Main Span Bridge Alternatives

1. Precast Concrete Segmental
2. Steel Edge Girder – Split Deck
3. Cast-in-Place Concrete Edge Girder – Split Deck
4. Steel Edge Girder
5. Steel Double Deck
6. Precast Concrete Segmental – Split Deck
7. Cast-in-Place Segmental – Split Deck






TOWER
PYLON
WESTBOUND I-10
EASTBOUND I-10
TOP OF FOOTING
TRANSVERSE SECTION AT TOWERS
PIER
PIER CAP
COLUMN (TYP.)
TRANSVERSE SECTION AT PIERS
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE SEGMENTAL
ALTERNATIVE 1 12 OF 21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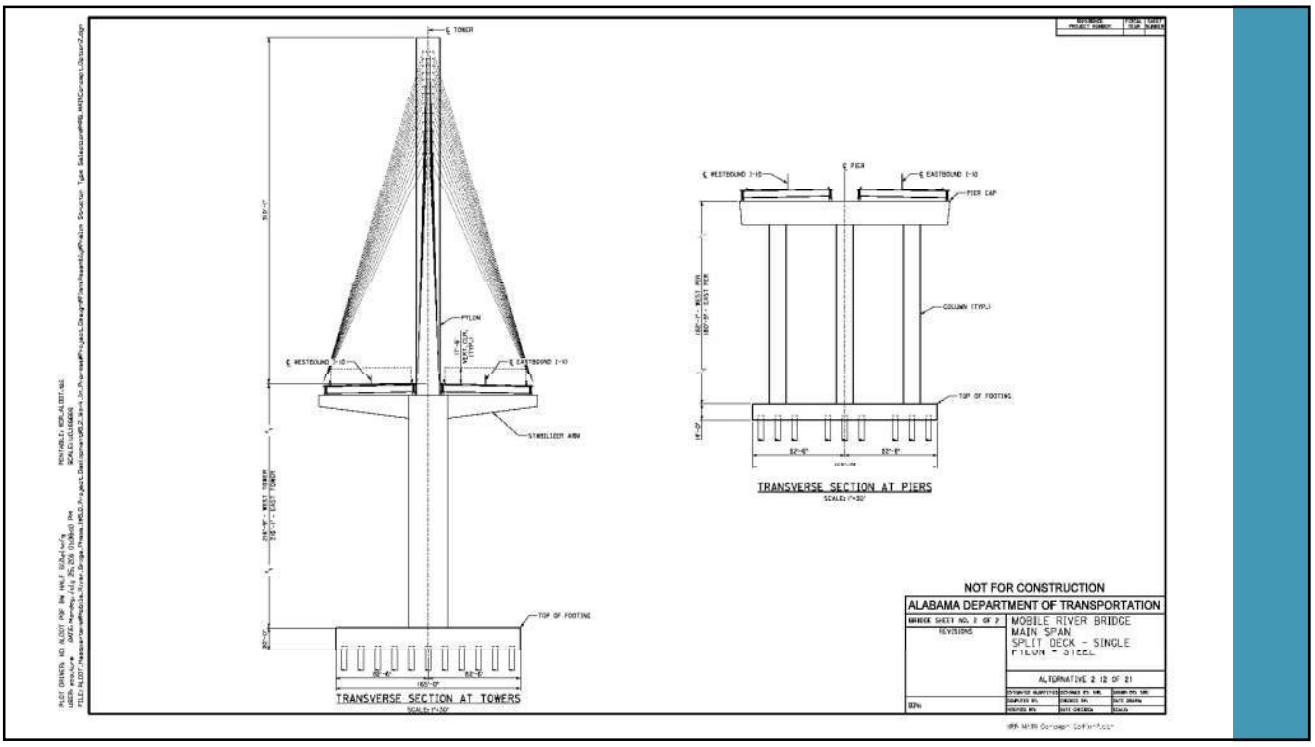
WEST TOWER
EAST TOWER
EAST PIER
BACK SPAN
MAIN SPAN
EXISTING CHANNEL BOTTOM
EXISTING GROUND LINE
NAVIGATION CHANNEL
ELEVATION
WESTBOUND I-10
EASTBOUND I-10
TYPICAL SECTION AT DECK ANCHORAGES
PYLON STEEL
ALTERNATIVE 2 11 OF 21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PYLON
STABILIZER ARM
WESTBOUND I-10
EASTBOUND I-10
TOP OF FOOTING
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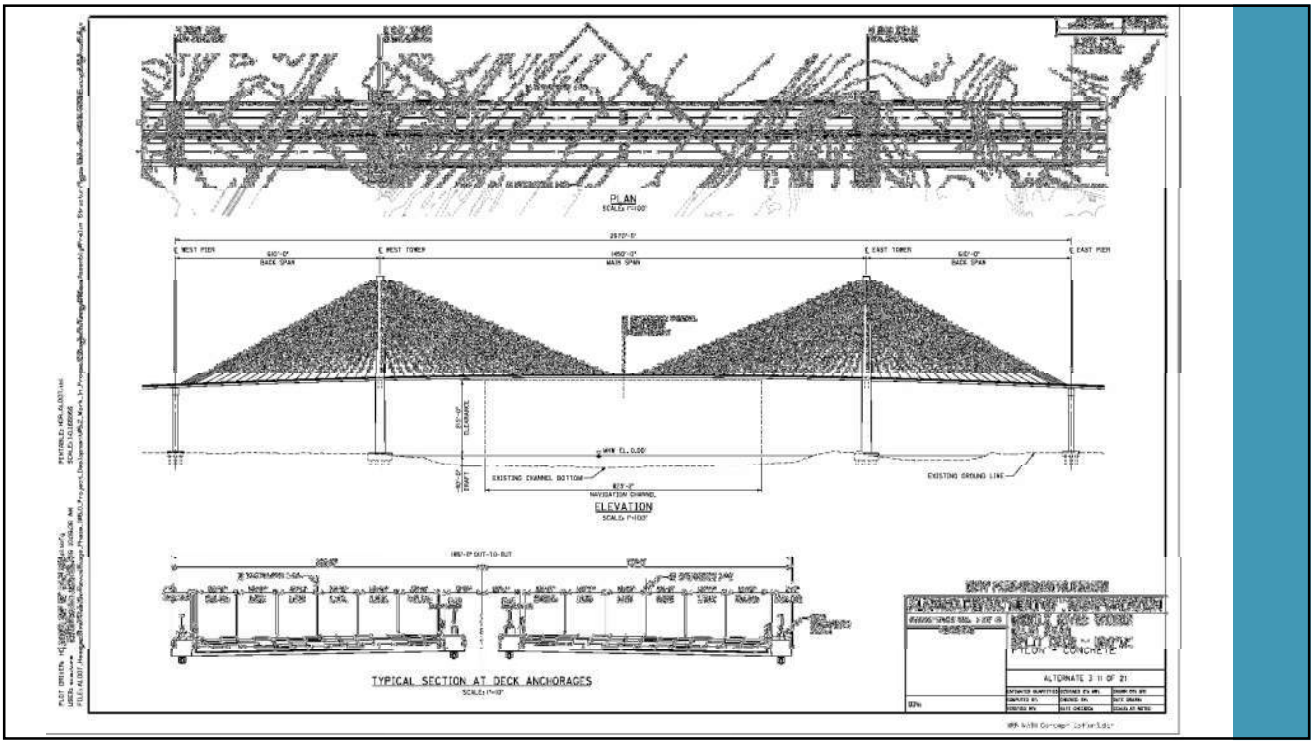
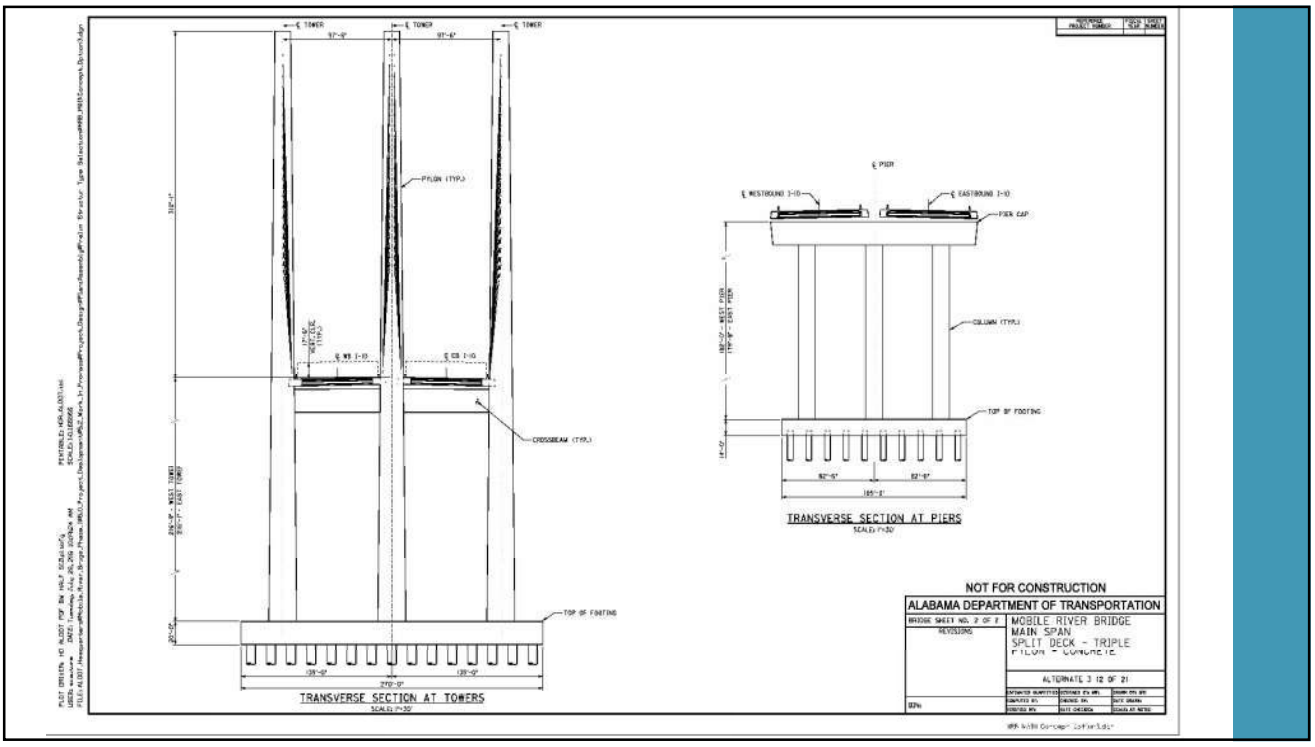
PIER
PIER CAP
COLUMN (TYP.)
TRANSVERSE SECTION AT PIERS
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - SINGLE
ALTERNATIVE 2 12 OF 21


PLAN
WEST PIER
WEST TOWER
EAST TOWER
EAST PIER
MAIN SPAN
EXISTING CHANNEL BOTTOM
EXISTING GROUND LINE
NAVIGATION CHANNEL
ELEVATION
TYPICAL SECTION AT DECK ANCHORAGES
PYLON - CONCRETE
ALTERNATE 3 11 OF 21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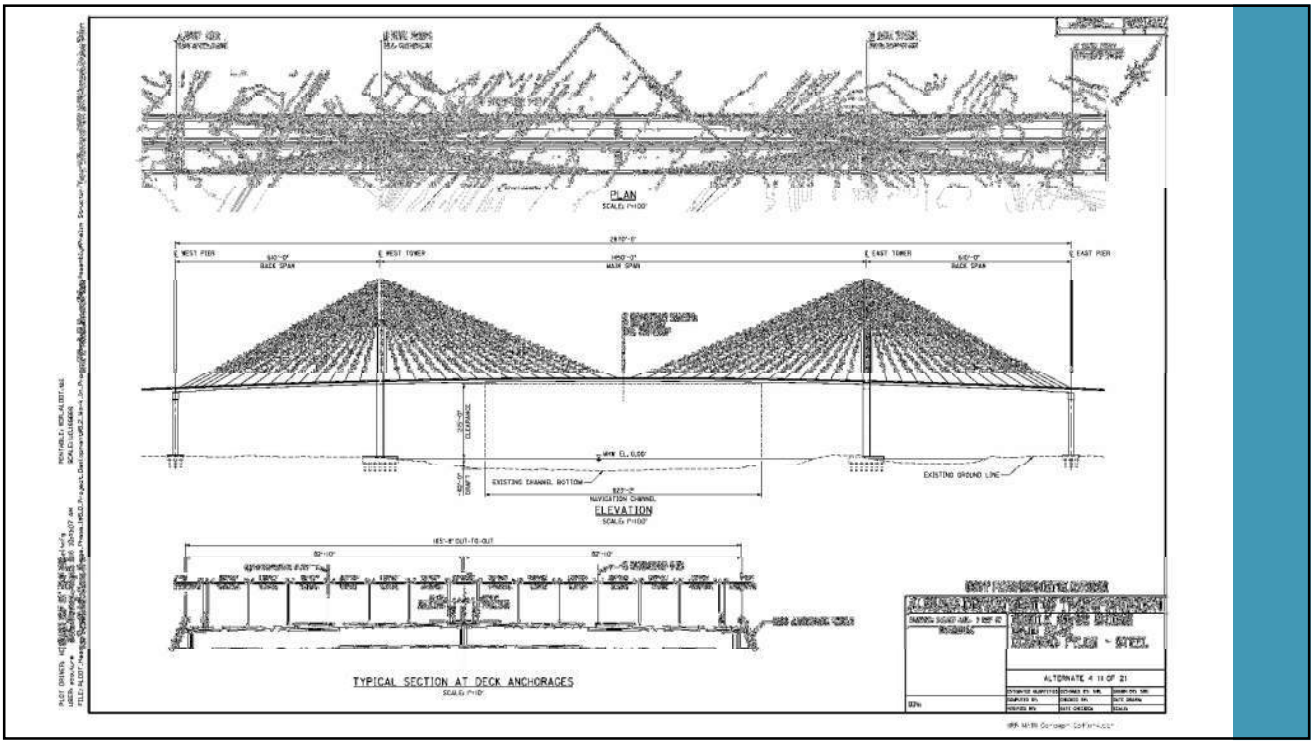
TRANSVERSE SECTION AT PIERS
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - TRIPLE
ALTERNATE 3 (2 OF 2)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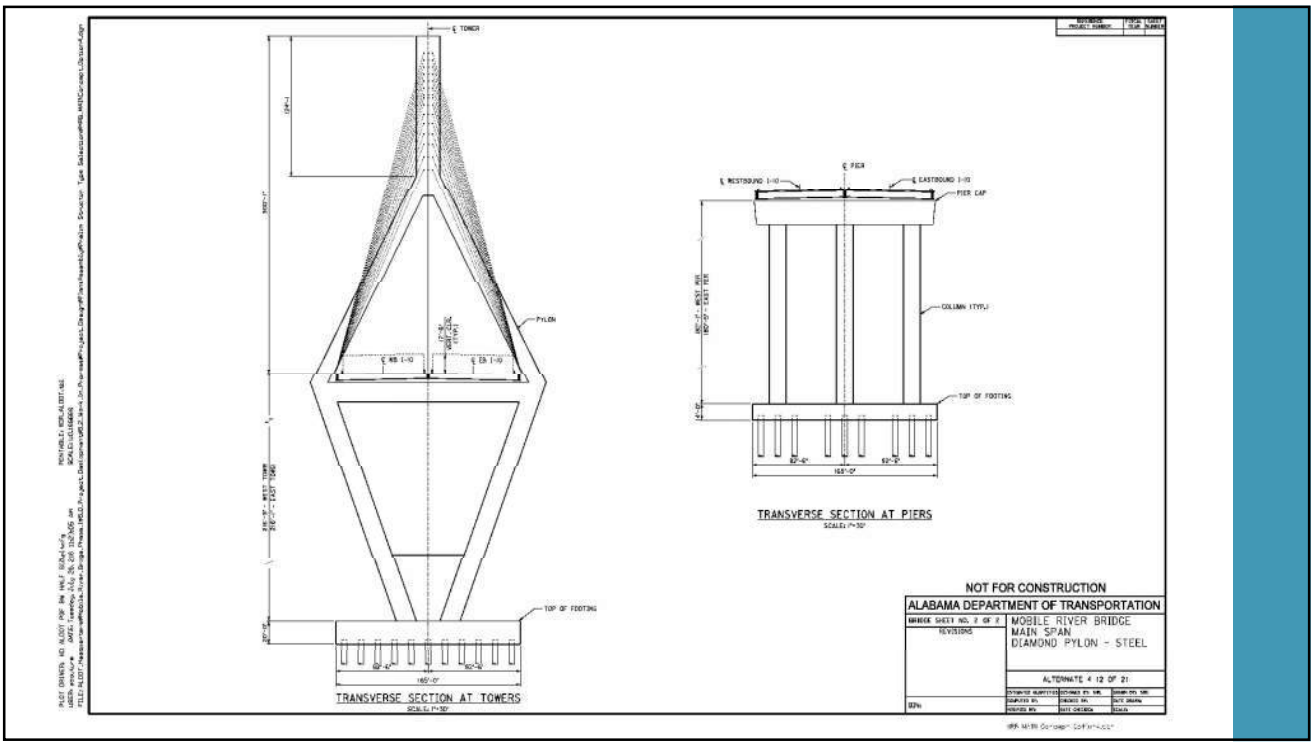
ELEVATION
TYPICAL SECTION AT DECK ANCHORAGES
ALTERNATE 4 (1 OF 2)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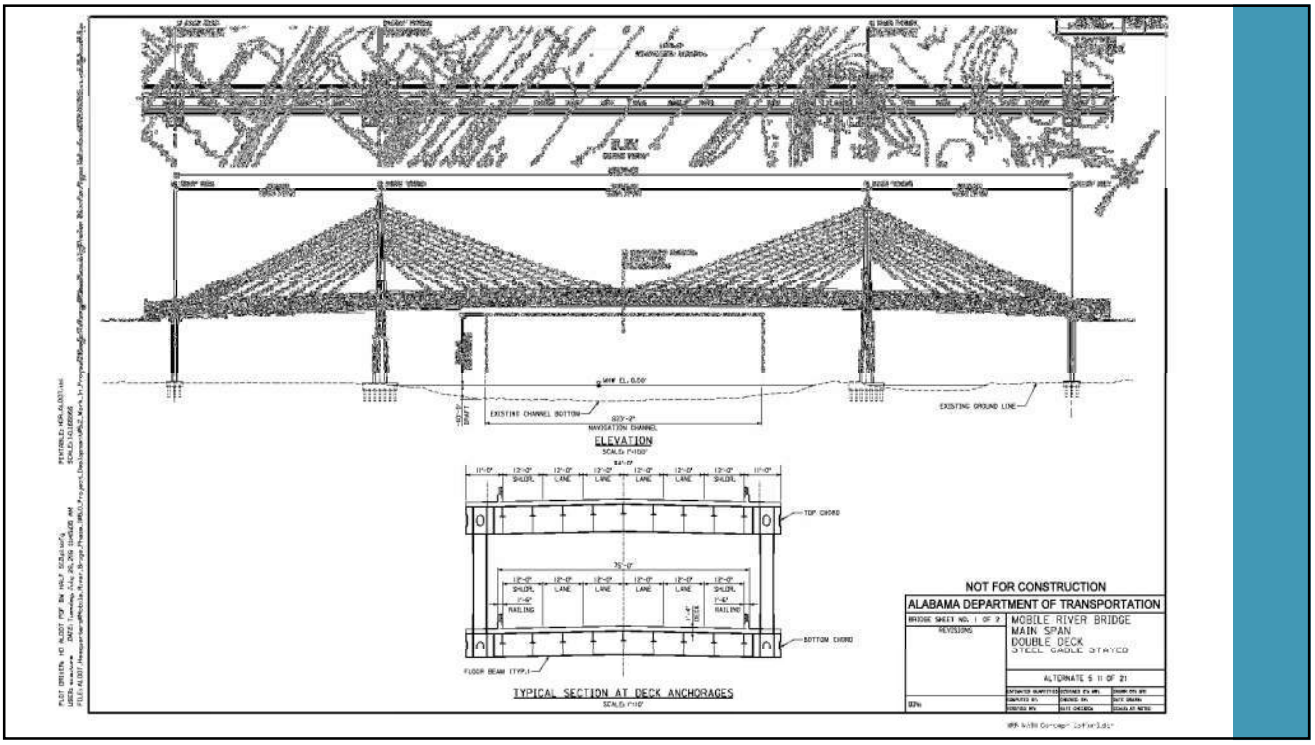
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
DIAMOND PYLON - STEEL
ALTERNATE 4 12 OF 21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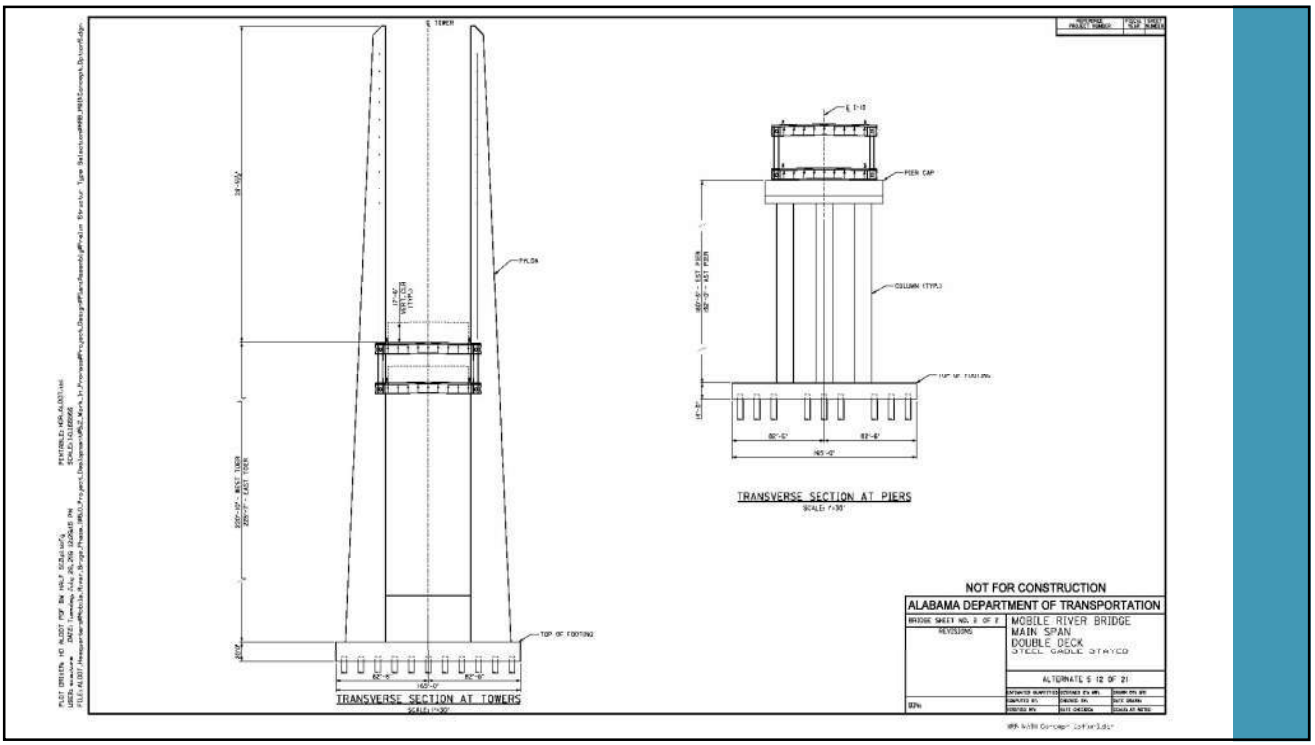
MOBILE RIVER BRIDGE
MAIN SPAN
DOUBLE DECK
STEEL CABLE STAYED
ALTERNATE 5 11 OF 21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TRANSVERSE SECTION AT PIERS
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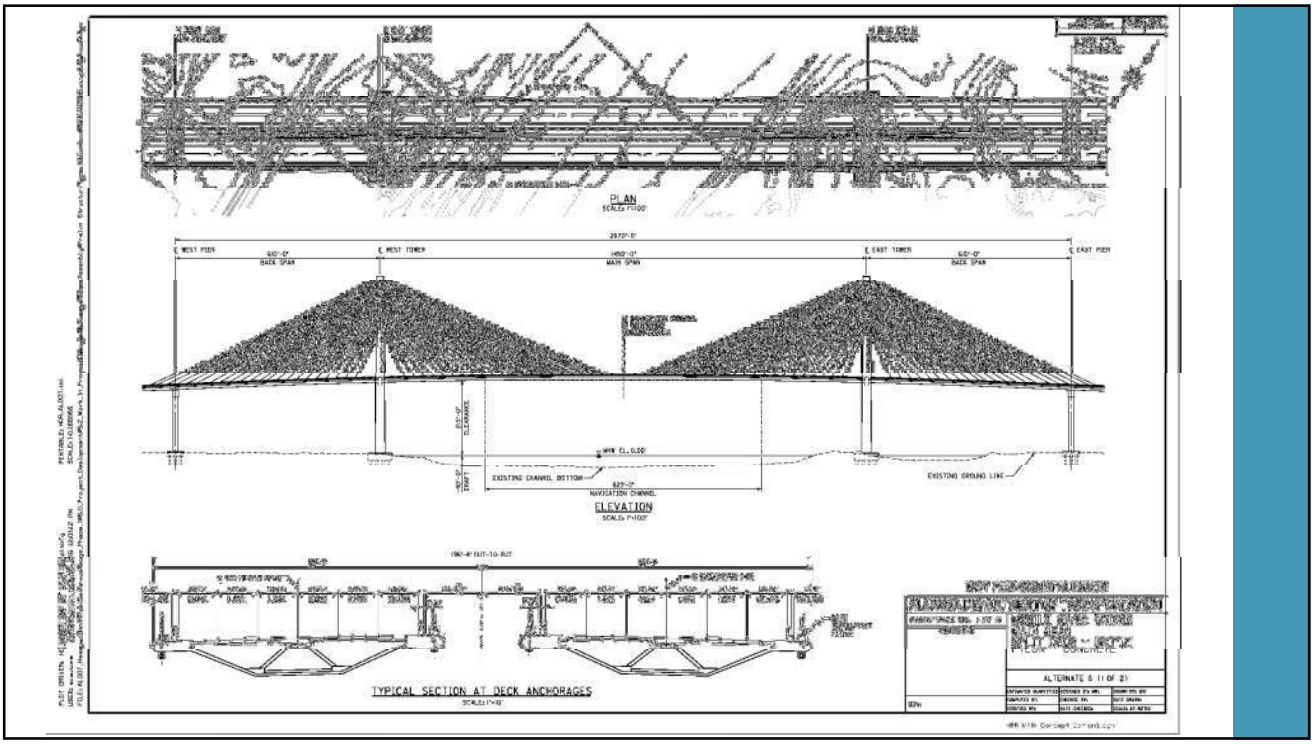
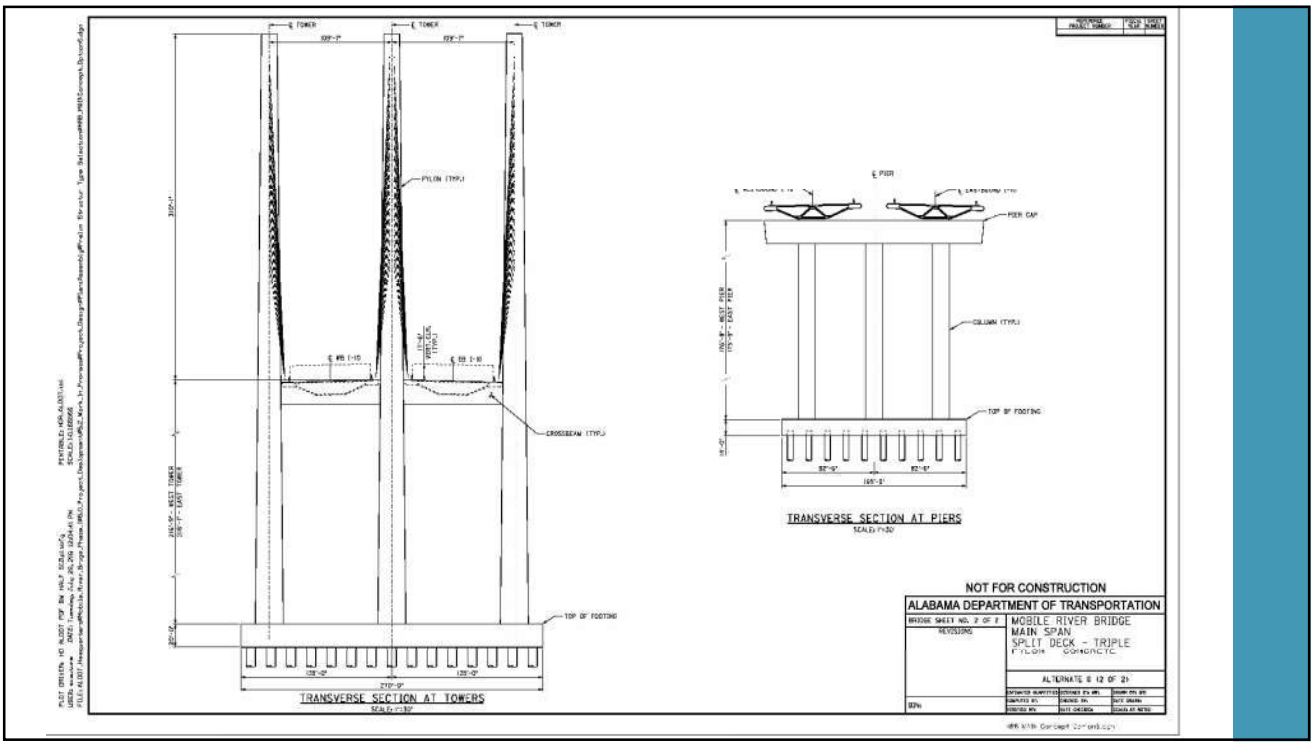
MOBILE RIVER BRIDGE
MAIN SPAN
DOUBLE DECK
STEEL CABLE STAYED
ALTERNATE S 12 OF 21


PLAN
ELEVATION
TYPICAL SECTION AT DECK ANCHORAGES
ALTERNATE S 11 OF 21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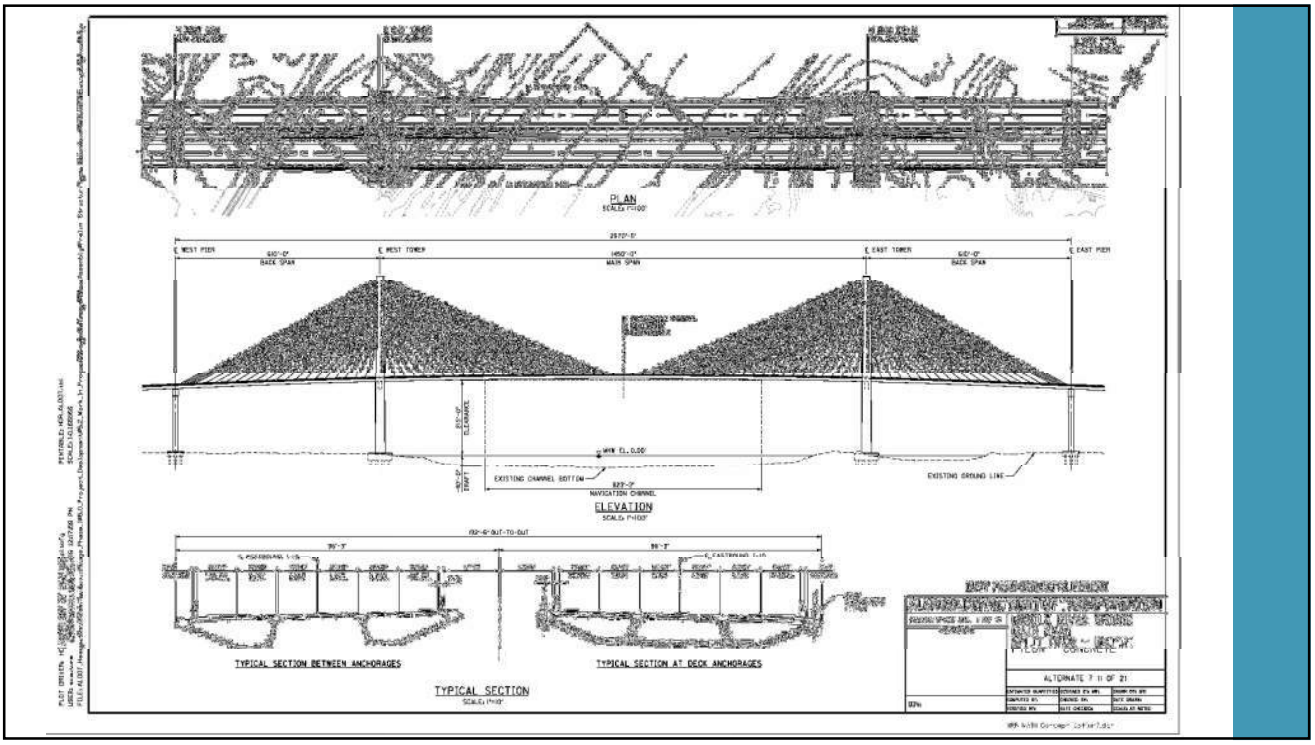
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - TRIPLE
CONCRETE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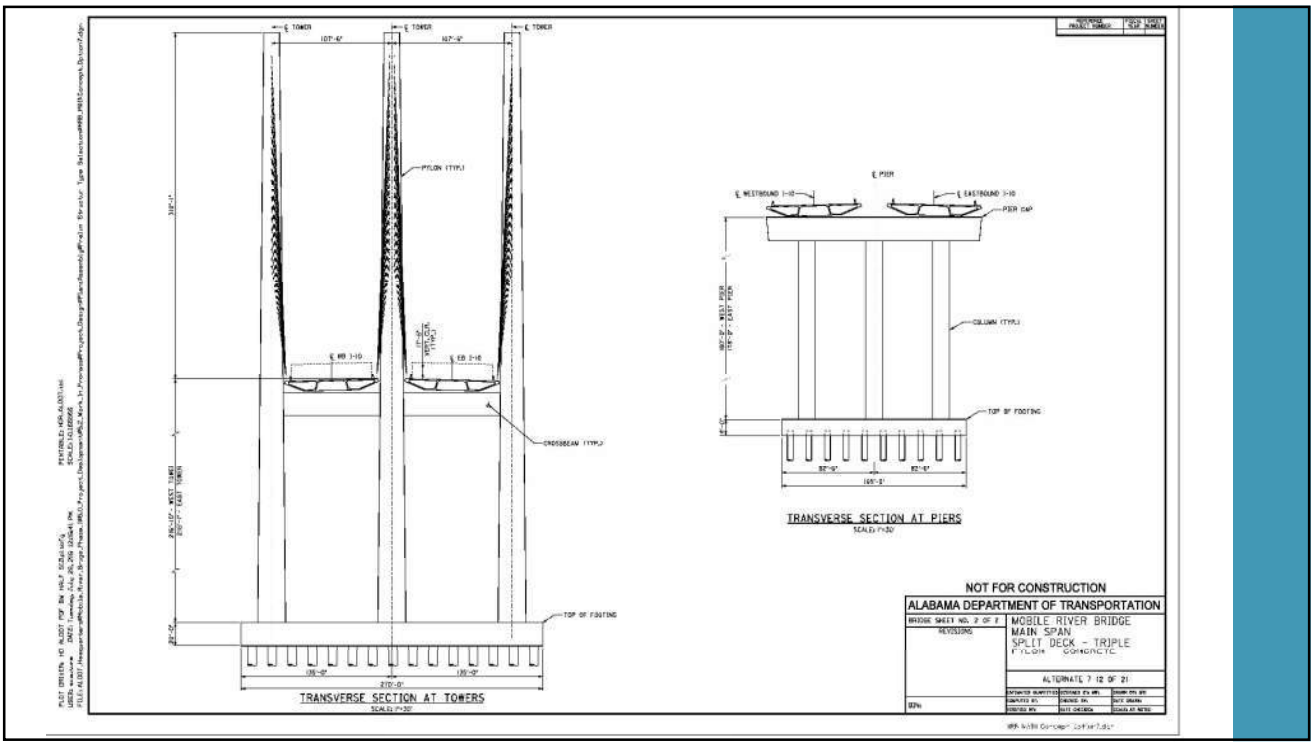
ELEVATION
TYPICAL SECTION
TYPICAL SECTION BETWEEN ANCHORAGES
TYPICAL SECTION AT DECK ANCHORAGES



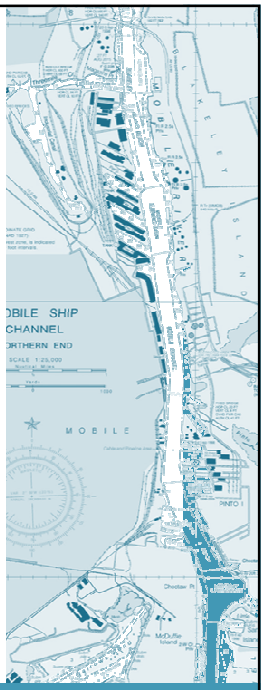
7. Main Span Concepts Evaluation

- 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.

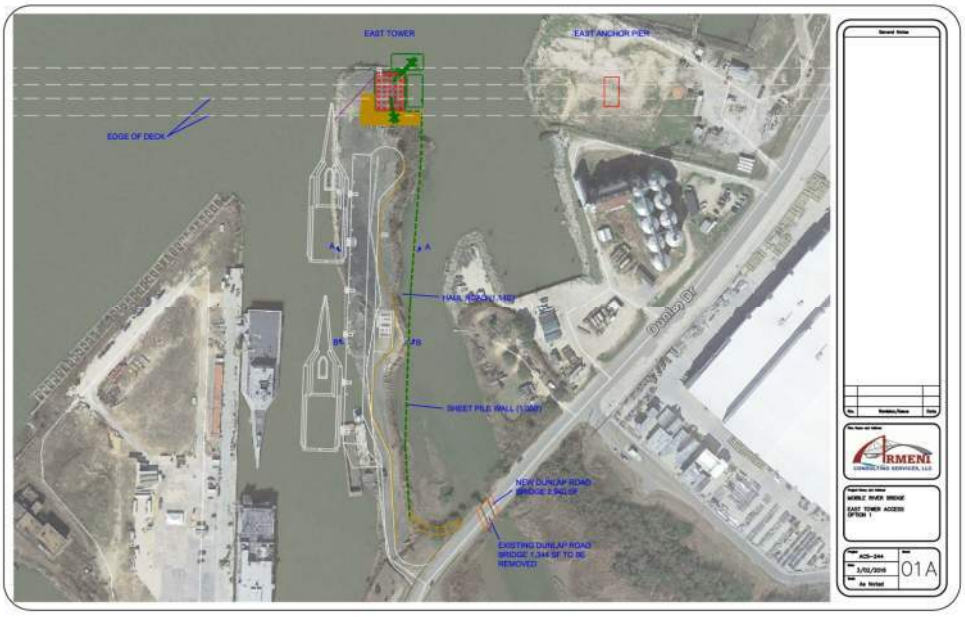
Criteria	●	◒	○
Constructability	Good	Fair	Poor
Maintenance and Durability	Good	Fair	Poor
Environmental Commitments	Good	Acceptable	Not Acceptable
Design Considerations	Good	Fair	Poor

7. Main Span Alternatives Evaluation – Constructibility

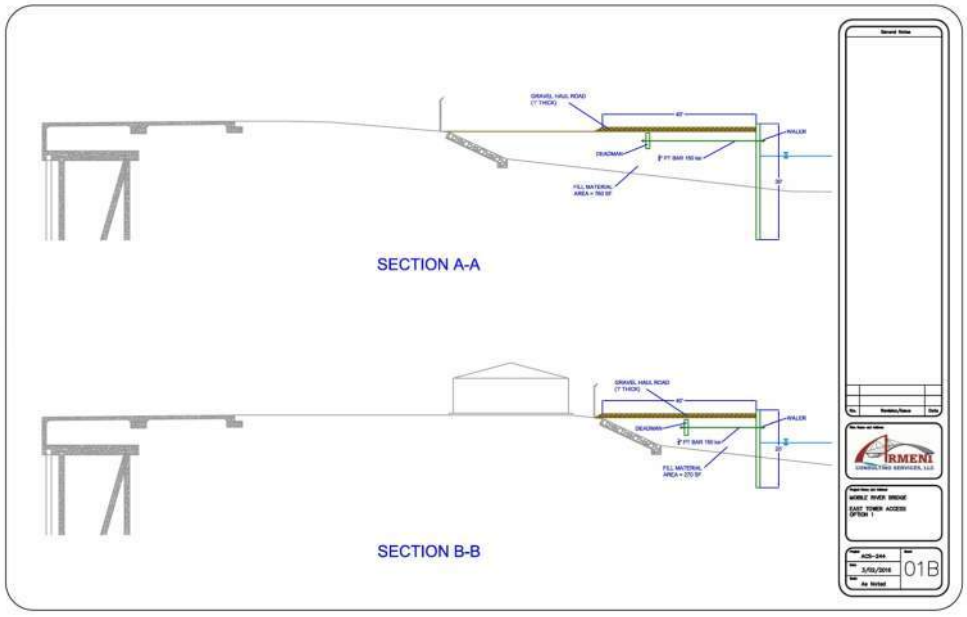
- Construction Access
  - How is Equipment and Material Delivered?
- Disruption to Existing Traffic/Maintenance of Traffic
  - Impacts to Existing Traffic During Construction
- Established Construction Methods
- Impacts to Existing Utilities
- Impacts to Railroads
  - Impact to Railroad Traffic During Construction
- Cofferdams
  - Large and Costly Items



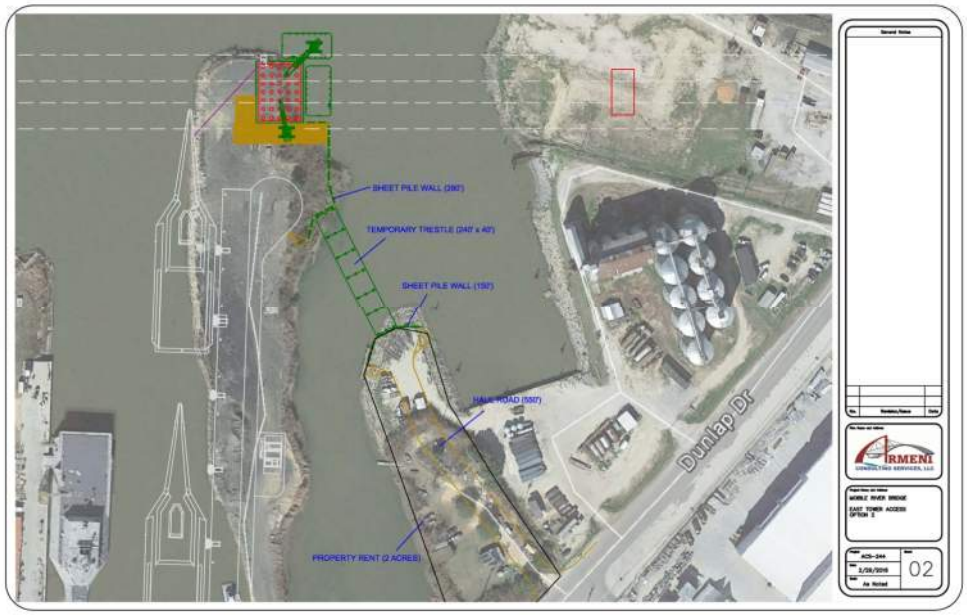
7. Main Span Alternatives Evaluation – Construction Access



7. Main Span Alternatives Evaluation – Construction Access



7. Main Span Alternatives Evaluation – Construction Access



7. Main Span Alternatives Evaluation – Construction Access

7. Main Span Alternatives Evaluation – Cofferdams

Waterborne foundation construction can be costly.

Cofferdams are temporary Watertight enclosures pumped dry to permit construction work below the waterline.

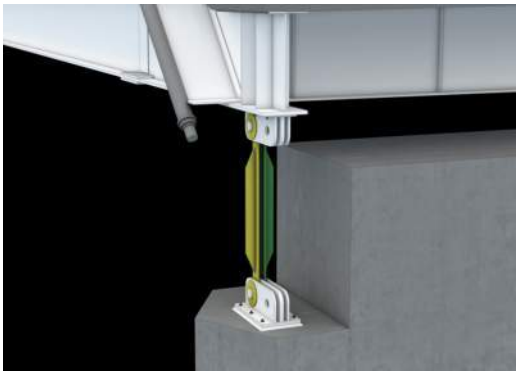
7. Main Span Concepts Evaluation – Constructability

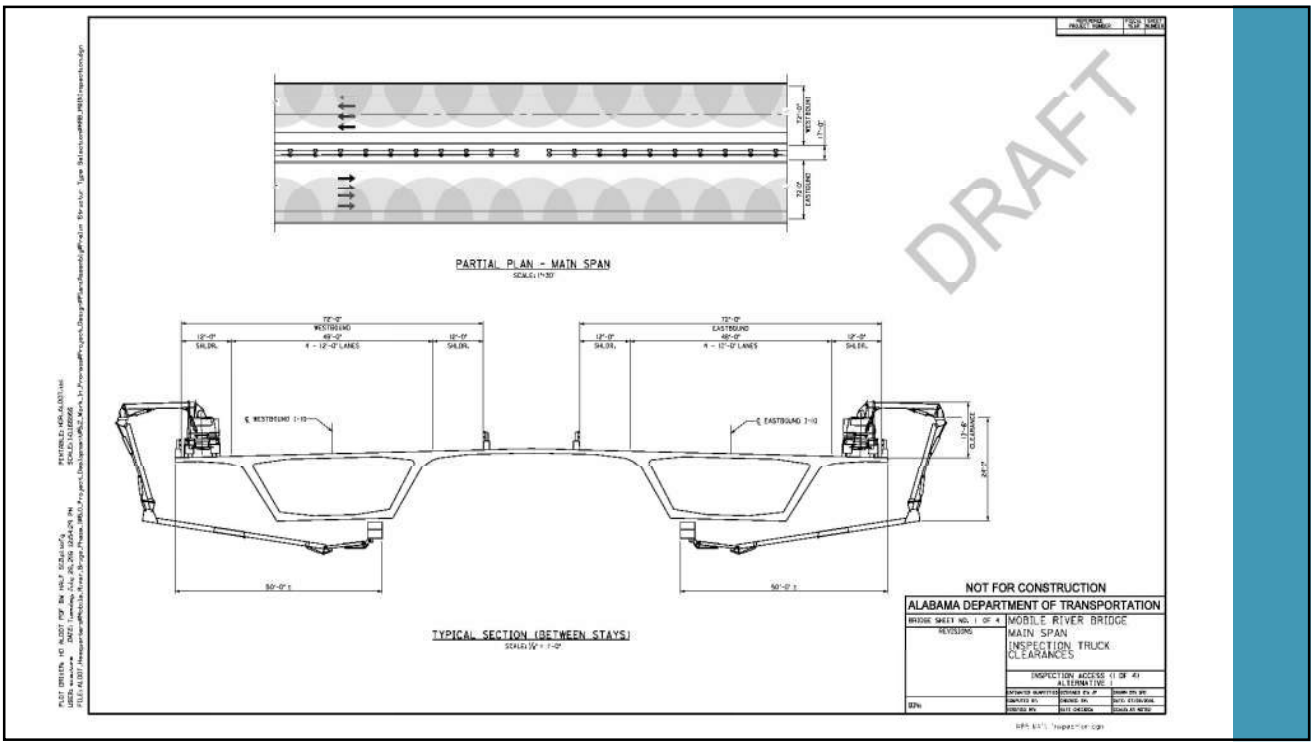
Good = ● Acceptable = ◒ Not Acceptable= ○

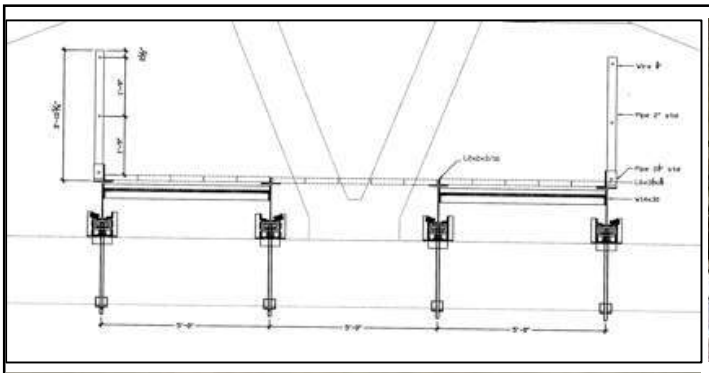
	Constructability					
Bridge Alternative	Construction Access	Disruption to Existing Traffic	Established Construction Methods	Impacts to Existing Utilities	Cofferdams	Overall
1. Precast Concrete Segmental	●	●	●	◒	◒	●
2. Steel Edge Girder - Split Deck	●	●	●	◒	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●	●
4. Steel Edge Girder	●	◒	◒	◒	●	◒
5. Steel Double Deck	●	○	○	◒	◒	○
6. Precast Concrete Segmental – Split Deck	●	●	●	◒	◒	●
7. CIP Concrete Segmental – Split Deck	●	●	●	◒	◒	●

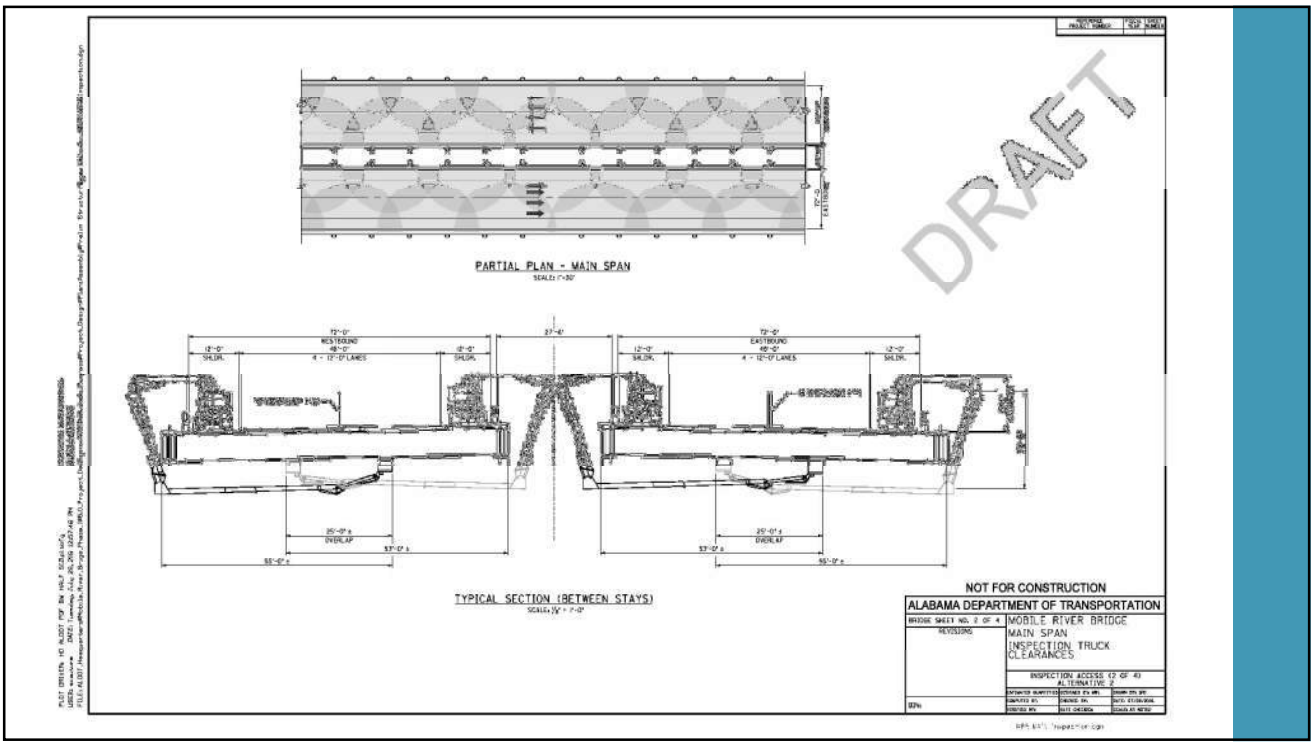
7. Main Span Alternatives Evaluation –
Maintenance and Durability

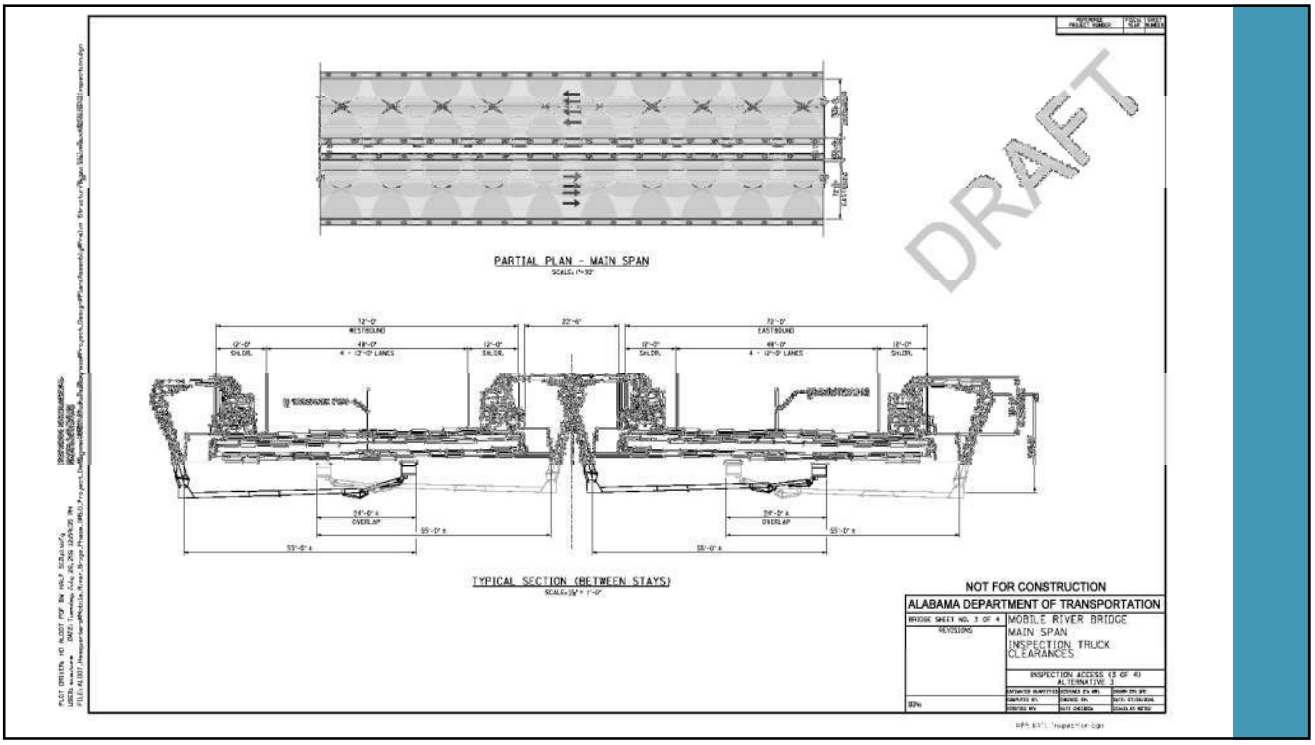
- Maintenance and Inspection Access
- Tie-Downs – To Resist Uplift at Anchor Piers
- Bearings
- Deck Joints
- Painting of Steel
- Deck Replacement
- Future Widening
- Stay Cable Replacement




DRAFT
PARTIAL PLAN - MAIN SPAN
TYPICAL SECTION (BETWEEN STAYS)
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
INSPECTION TRUCK
CLEARANCES




PARTIAL PLAN - MAIN SPAN
TYPICAL SECTION (BETWEEN STAYS)
DRAFT
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
INSPECTION TRUCK CLEARANCES


PARTIAL PLAN - MAIN SPAN
TYPICAL SECTION (BETWEEN STAYS)
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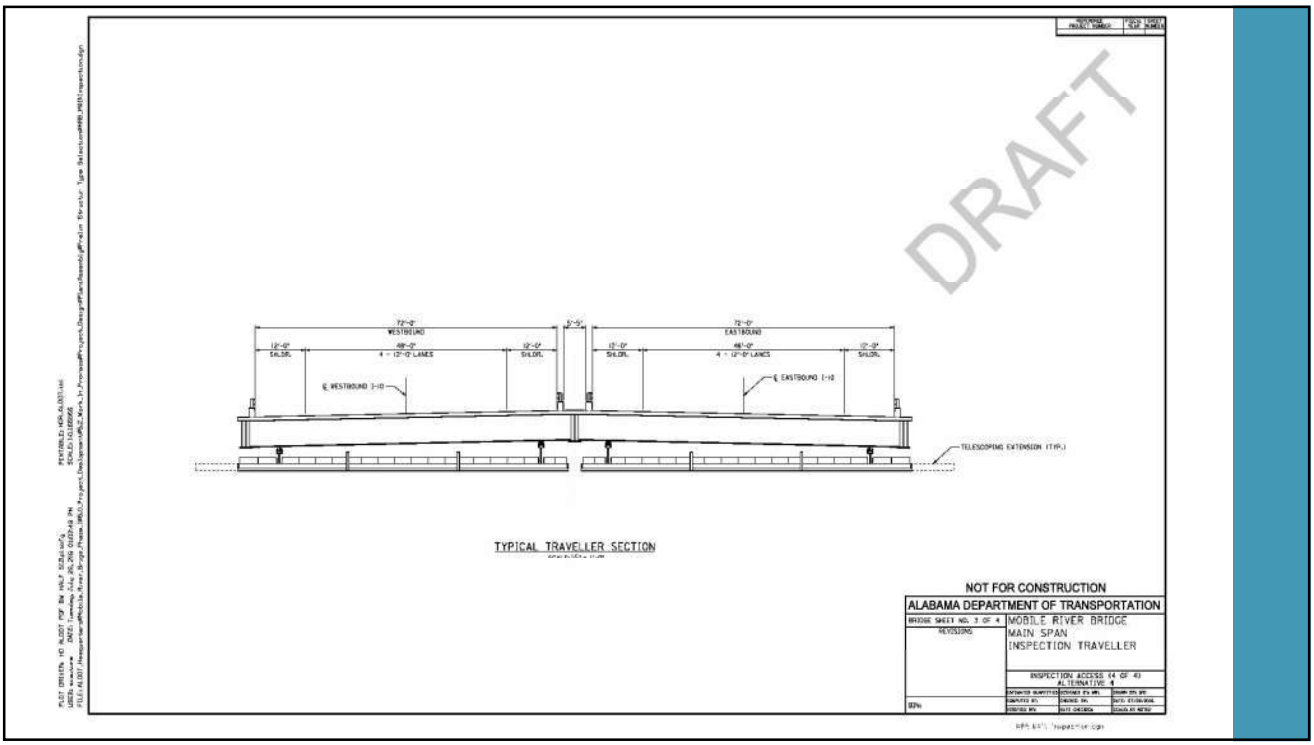
NOT FOR CONSTRUCTION
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
MAIN SPAN
INSPECTION TRUCK CLEARANCES



7. Main Span Concepts Evaluation – Inspection Traveler

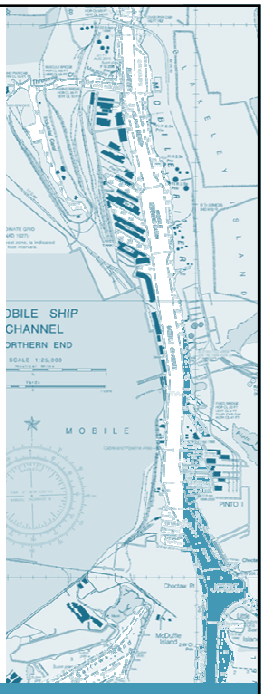
7. Main Span Concepts Evaluation – Maintenance and Durability

Good = ● Fair = ◒ Poor = ○

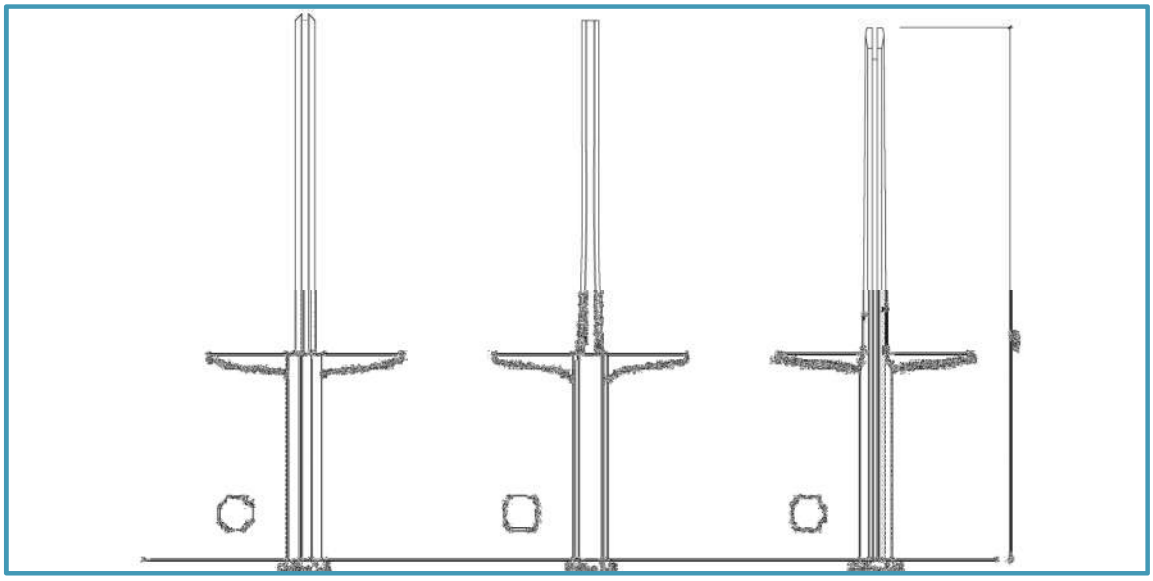
	Maintenance and Durability								
Bridge Alternative	Accessibility	Tie-Downs	Brgs.	Deck Joints	Painting of Steel	Deck Replacement	Future Widening	Stay Cable Replacement	Overall
1. Precast Concrete Segmental	●	●	●	●	●	○	○	●	●
2. Steel Edge Girder - Split Deck	●	○	◒	●	◒	●	●	●	◒
3. CIP Concrete Edge Girder – Split Deck	●	●	◒	●	●	○	○	●	●
4. Steel Edge Girder	●	○	◒	●	◒	◒	◒	●	◒
5. Steel Double Deck	◒	○	◒	◒	○	◒	○	●	○
6. Precast Concrete Segmental – Split Deck	●	●	●	●	●	○	○	●	●
7. CIP Concrete Segmental – Split Deck	●	●	●	●	●	○	○	●	●

7. Main Span Concepts Evaluation – Environmental Commitments

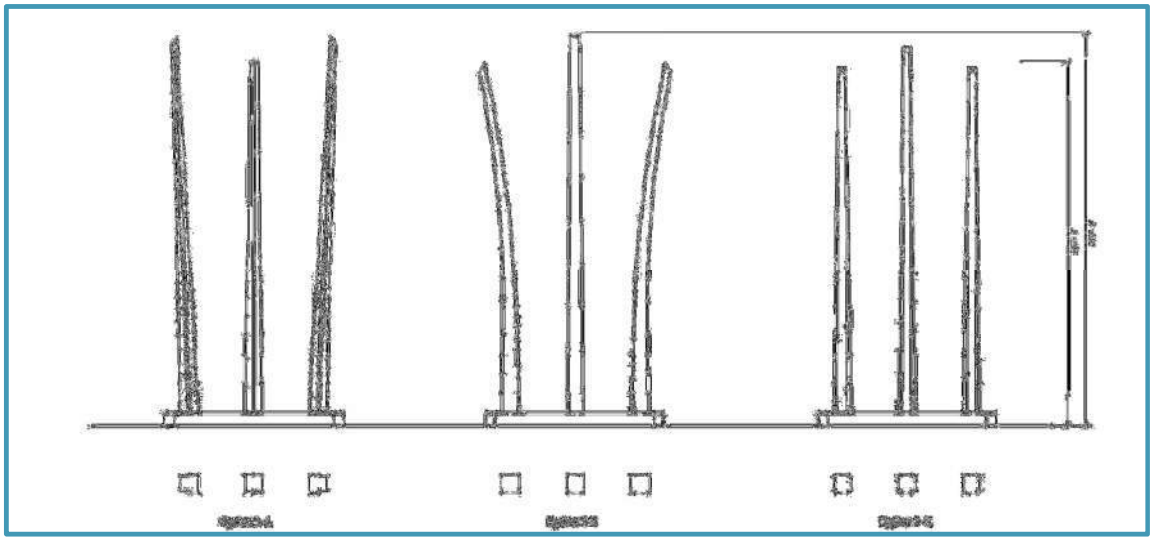
- Contextual Design and Aesthetics
- Impacts to Mobile's Maritime Traffic During Construction
- Permitting
- ROW
- Construction Vibrations



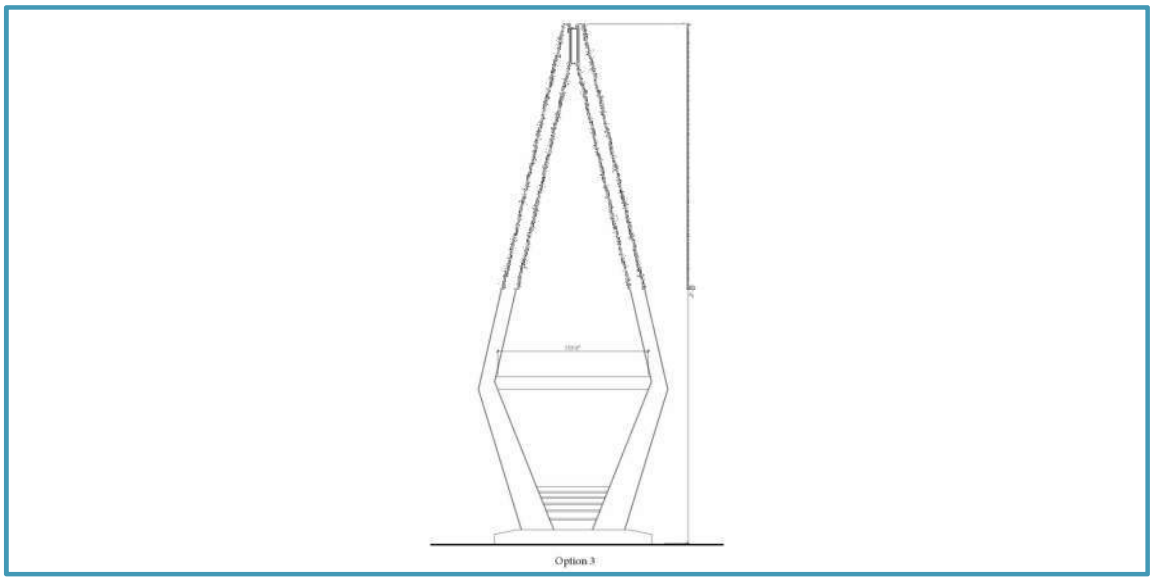
7. Main Span Concepts Evaluation – Contextual Design



7. Main Span Concepts Evaluation – Contextual Design



7. Main Span Concepts Evaluation – Contextual Design



Keeping the Navigation Channel Open – Steel Superstructure Construction

Keeping the Navigation Channel Open –
Precast Concrete Construction

Keeping the Navigation Channel Open – Cast-in-Place Construction

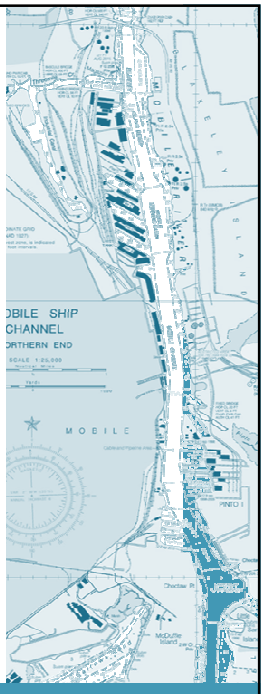
7. Main Span Concepts Evaluation – Environmental Commitments

Good = ● Fair = ◒ Poor = ○

Bridge Alternative	Environmental Commitments					
	Contextual Design	Impacts to Maritime Traffic	Permitting	ROW	Superst. Construction Vibrations	Overall
1. Precast Concrete Segmental	●	●	●	◒	●	●
2. Steel Edge Girder - Split Deck	●	●	●	◒	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●	●
4. Steel Edge Girder	●	◒	●	◒	●	◒
5. Steel Double Deck	◒	○	●	●	●	◒
6. Precast Concrete Segmental – Split Deck	●	●	●	◒	●	●
7. CIP Concrete Segmental – Split Deck	●	●	●	◒	●	●

7. Main Span Concepts Evaluation – Design Considerations

- Foundation Types and Sizes
- Navigational Air-Draft Clearance
  - Structure Depth
- Proximity to Brookley Airport –
  - Pylon Height
- Accommodates Future Sidewalk



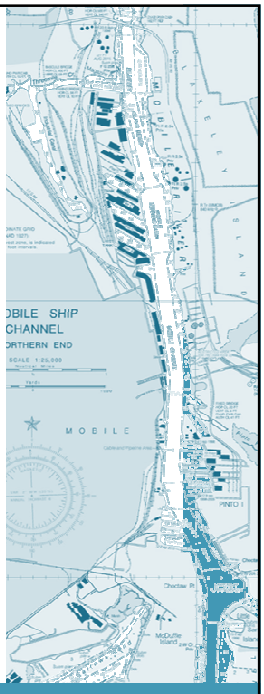
7. Main Span Concepts Evaluation – Design Considerations

Good = ● Fair = ◒ Poor = ○

	Design Considerations				
Bridge Alternative	Foundations Sizes	Navigational Air-Draft	Tower Height	Accom. Future Sidewalk	Overall
1. Precast Concrete Segmental	◒	◒	●	◒	◒
2. Steel Edge Girder - Split Deck	●	●	●	●	●
3. CIP Concrete Edge Girder – Split Deck	●	●	●	◒	●
4. Steel Edge Girder	●	●	●	●	●
5. Steel Double Deck	◒	○	●	◒	○
6. Precast Concrete Segmental – Split Deck	◒	◒	●	◒	◒
7. CIP Concrete Segmental – Split Deck	◒	●	●	◒	●

7. Main Span Concepts Evaluation – Construction Schedule

Bridge Alternative	Approx. Total Duration (Mos)
1. Precast Concrete Segmental	40-45
2. Steel Edge Girder – Split Deck	40-45
3. CIP Concrete Edge Girder – Split Deck	40-45
4. Steel Edge Girder	40-45
5. Steel Double Deck	42-50
6. Precast Concrete Segmental – Split Deck	40-45
7. CIP Concrete Segmental – Split Deck	40-45



7. Main Span Concepts Evaluation – Initial Construction Cost

Bridge Alternative	Cost ($) M
1. Precast Concrete Segmental	305
2. Steel Edge Girder – Split Deck	280
3. CIP Concrete Edge Girder – Split Deck	325
4. Steel Edge Girder	295
5. Steel Double Deck	465
6. Precast Concrete Segmental – Split Deck	350
7. CIP Concrete Segmental – Split Deck	325

- 2017 Construction Costs
- Include 15% Contingency

8. Main Span Concepts Evaluation – Recommendations

Good = ● Fair = ◒ Poor = ○

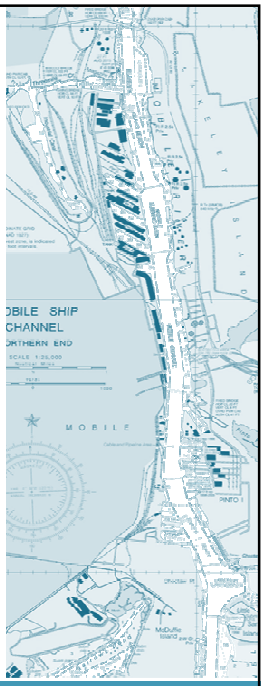
	Main Span Bridge Alternatives						
Criteria	1. Precast Concrete Segmental	2. Steel Edge Girder – Split Deck	3. CIP Concrete Edge Girder – Split Deck	4. Steel Edge Girder	5. Steel Double Deck	6. Precast Concrete Segmental – Split Deck	7. CIP Concrete Segmental – Split Deck
Initial Construction Cost ($) M	305	280	325	295	465	350	325
Construction Schedule (Mos.)	40-45	40-45	40-45	40-45	42-50	40-45	40-45
Constructability	●	●	●	◒	○	●	●
Maintenance and Durability	●	◒	●	◒	○	●	●
Environmental Commitments	●	●	●	◒	◒	●	●
Design Considerations	◒	●	●	●	○	◒	●
Overall	●	◒	●	◒	○	●	●

02 West Side Foundation Study

General Presentation Outline –

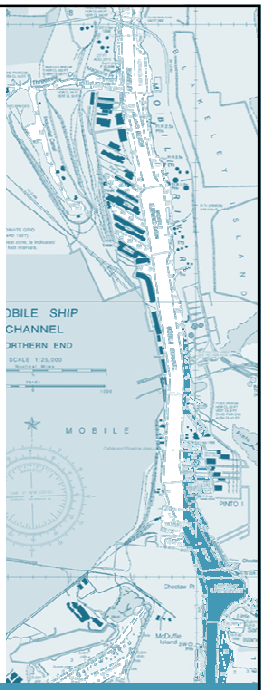
West Side Foundation Study

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



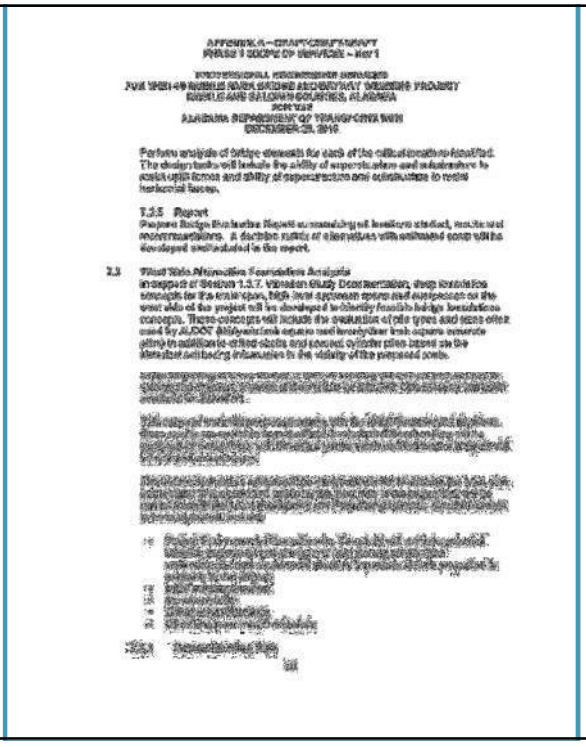
1. Purpose of Study

- 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"*



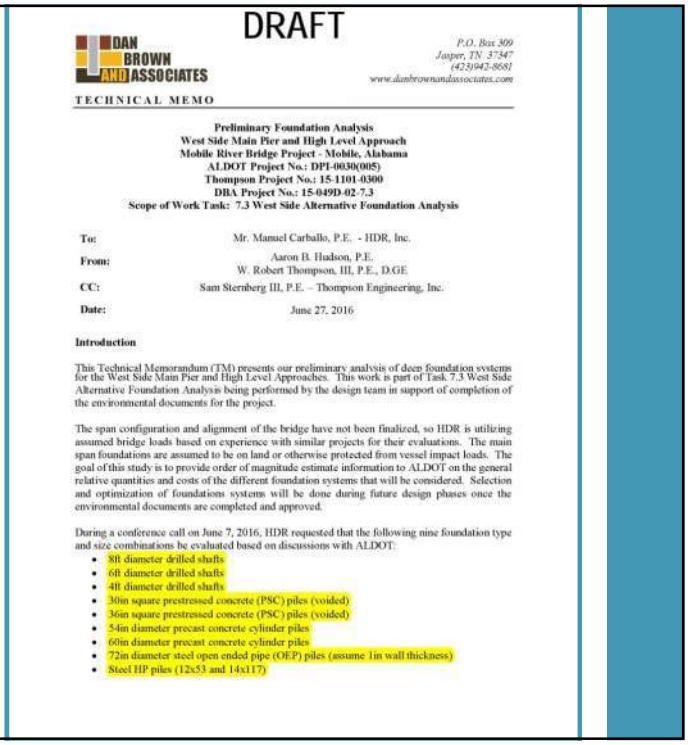
2. Scope of Work

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



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



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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.

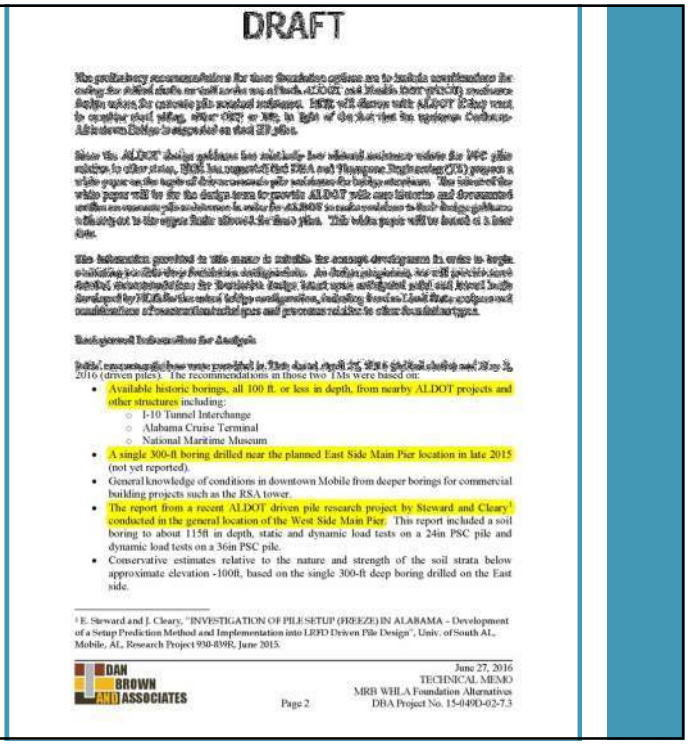
The span configuration and alignment of the bridge have not been finalized, so HDR is utilizing assumed bridge loads based on experience with similar projects for their evaluations. The main span foundations are assumed to be on land or otherwise protected from vessel impact loads. The goal of this study 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”



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- 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.

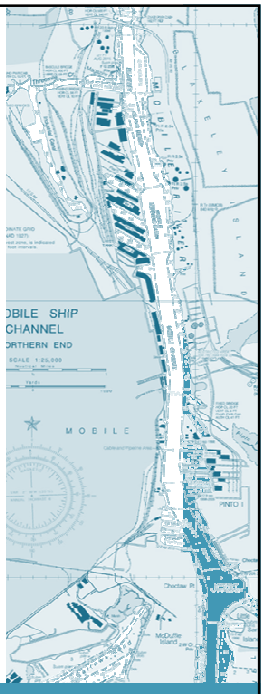
DAN BROWN AND ASSOCIATES
Page 2
June 27, 2016
TECHNICAL MEMO
MRB WHLA Foundation Alternatives
DBA Project No. 15-049D-02-7.3

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

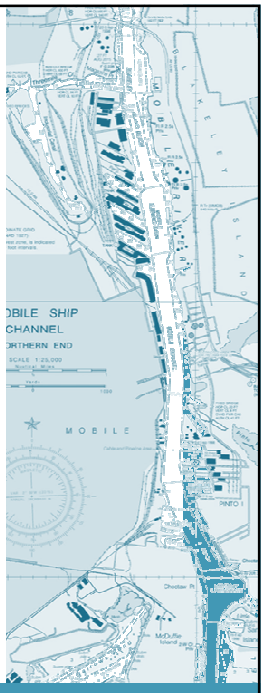
5. Design Approach

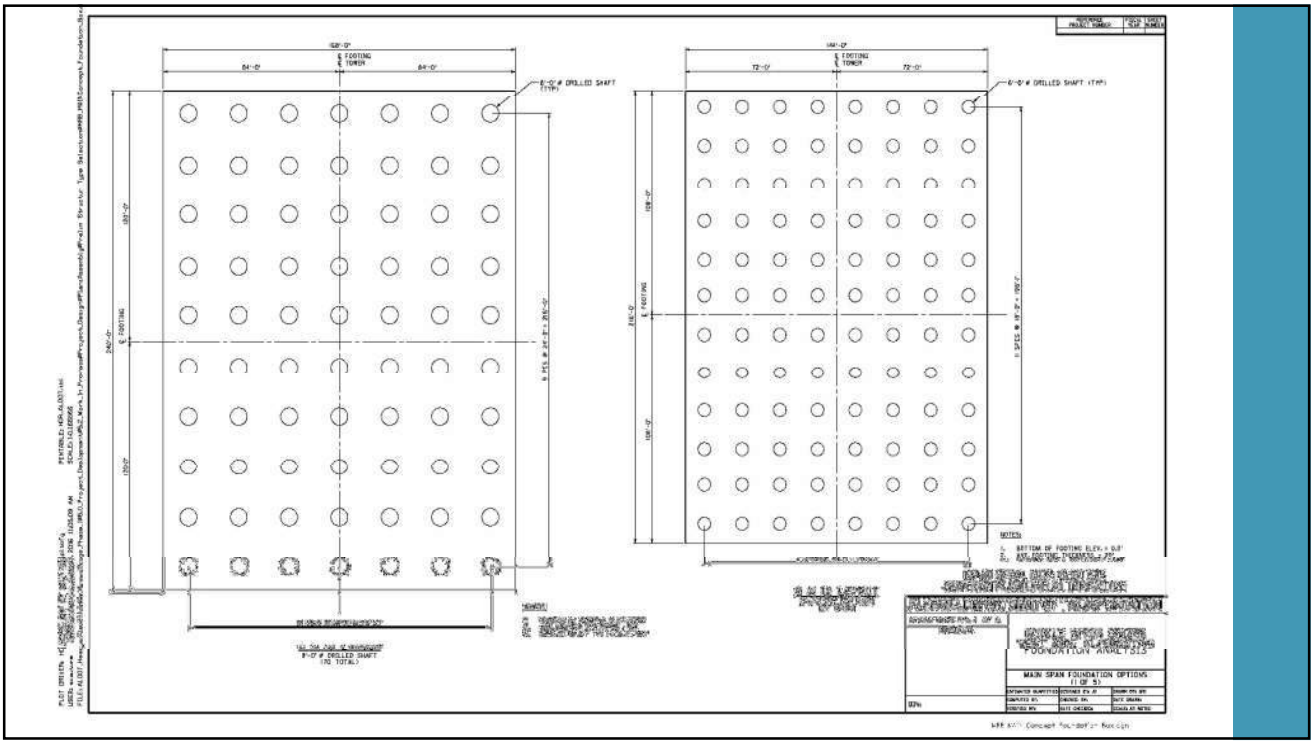
- For Main Span and High-Level Approach Spans Develop Foundation concepts for the Heaviest and Lightest Superstructure Type
- Prepare preliminary quantity take-offs for each foundation Concept and assign unit Costs to develop Estimated Construction Costs

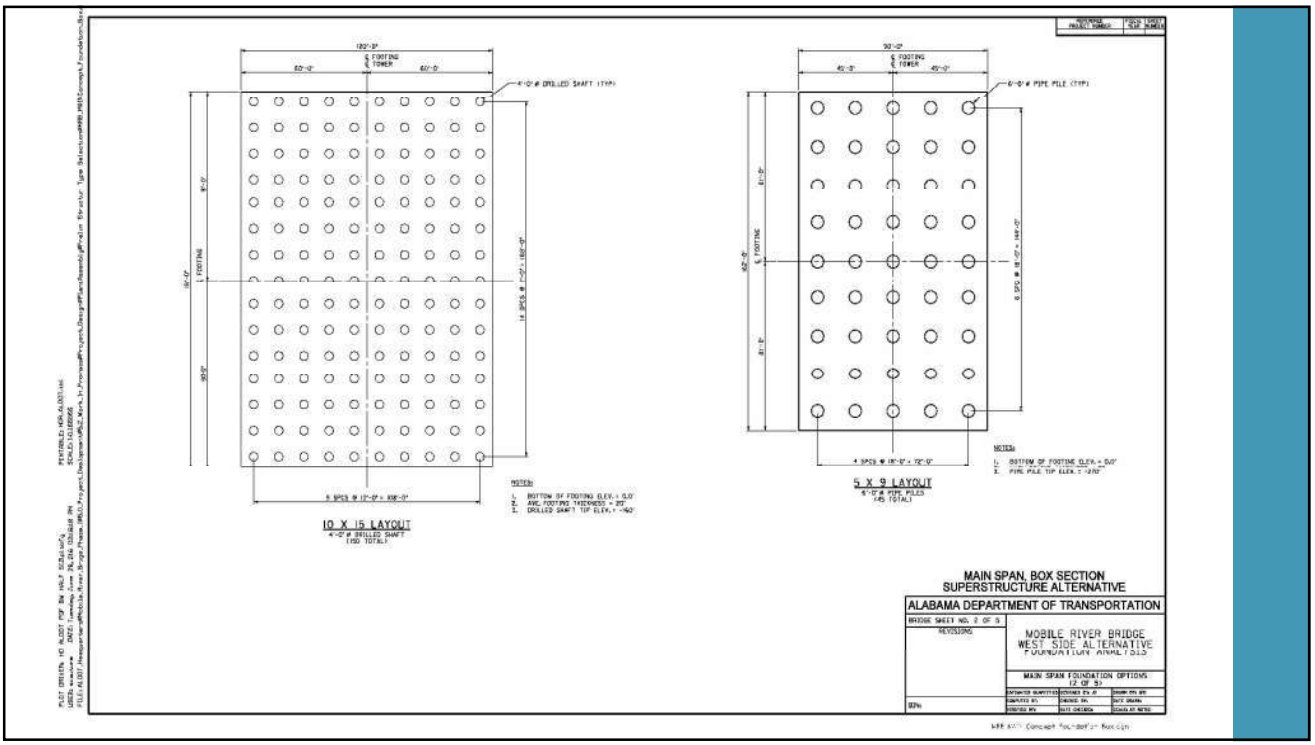


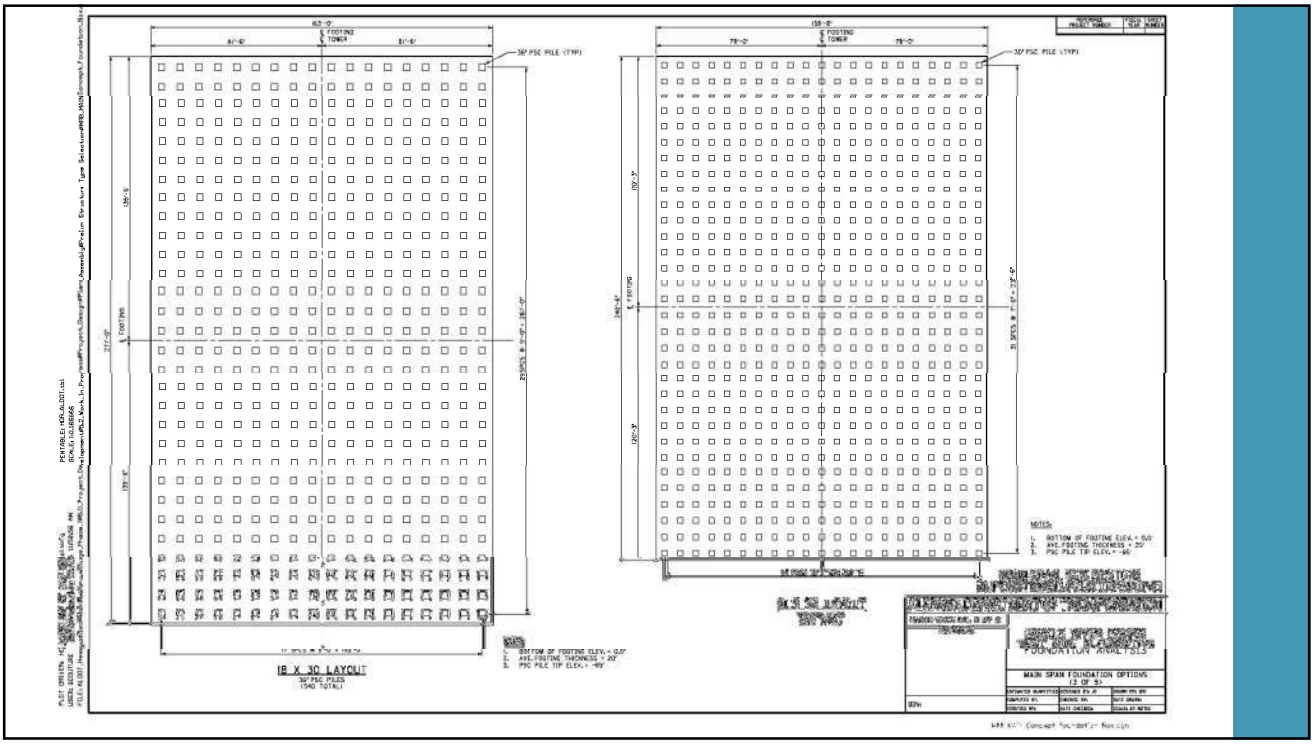
5. Design Concepts

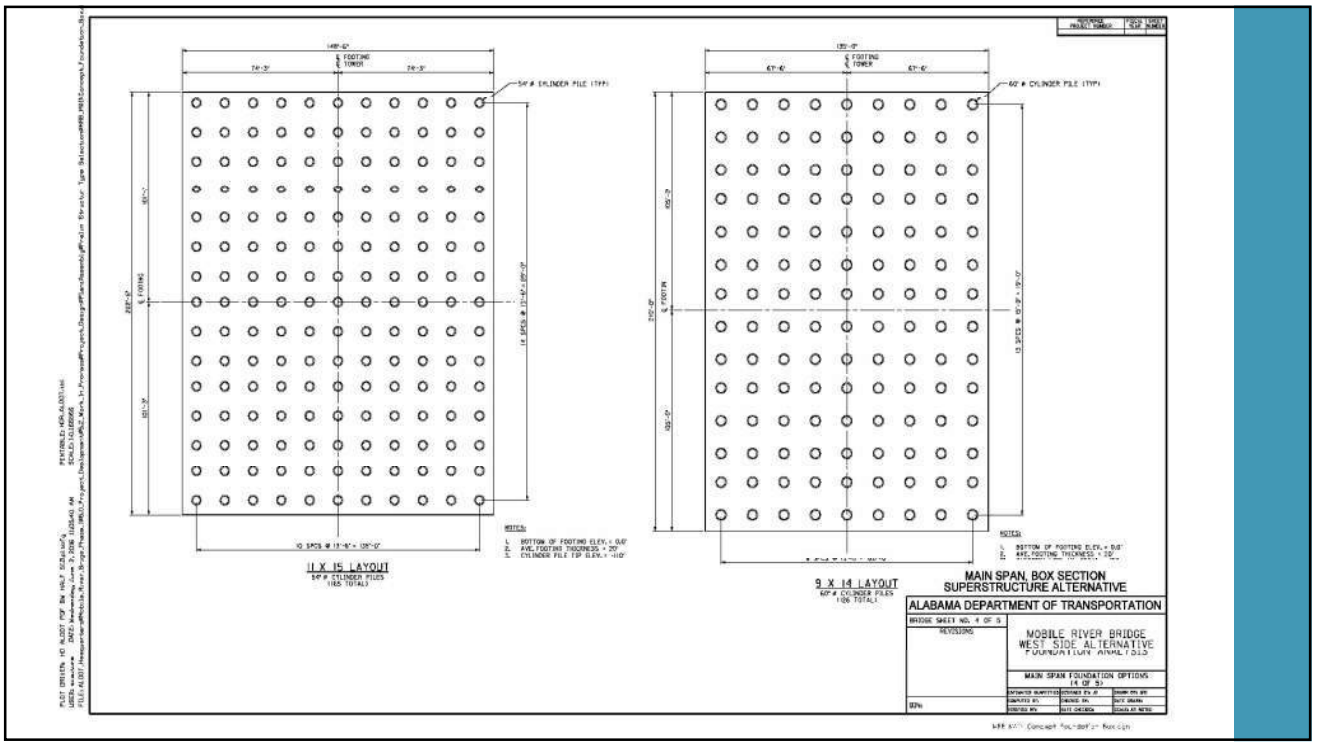
- Main Span Concrete Superstructure

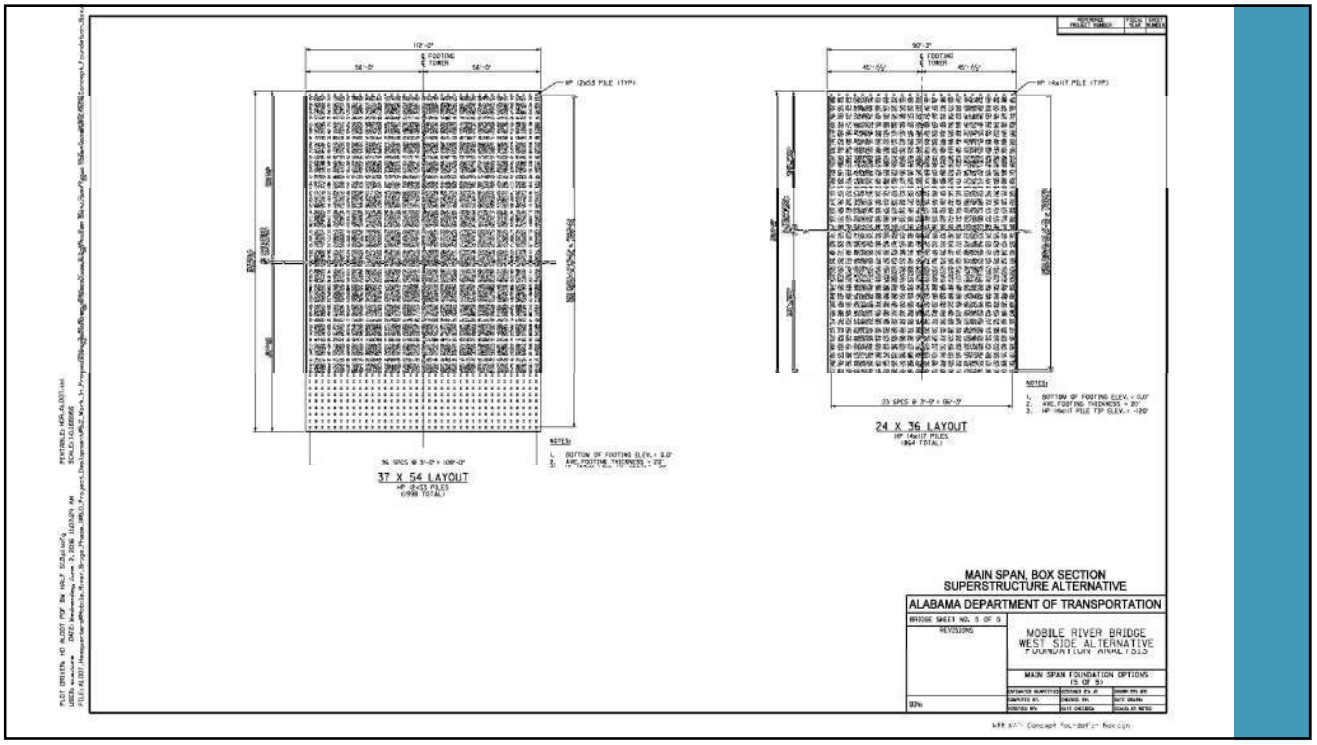





10 X 15 LAYOUT
5 X 9 LAYOUT
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

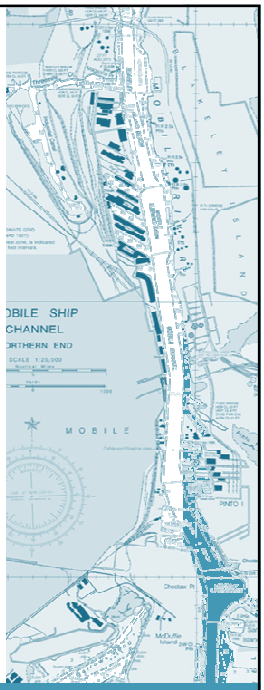

18 X 30 LAYOUT
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

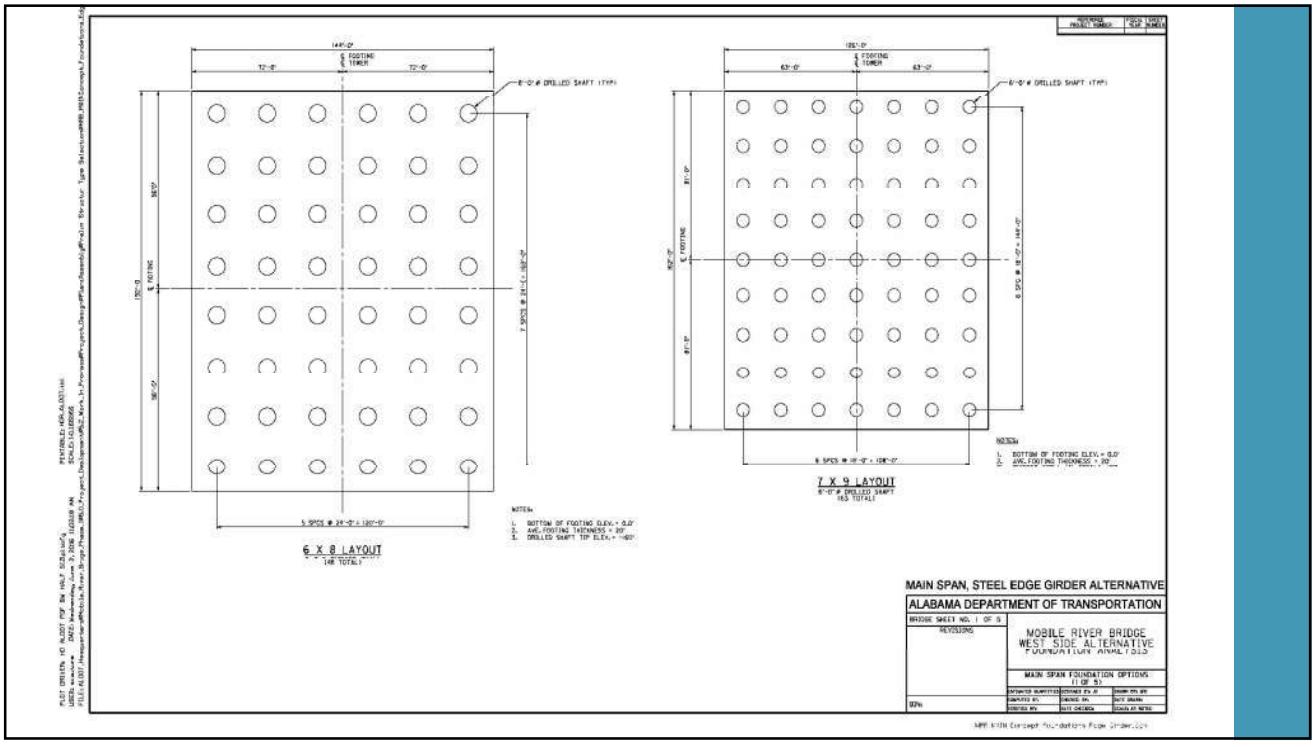

11 X 15 LAYOUT
9 X 14 LAYOUT
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
14 OF 51

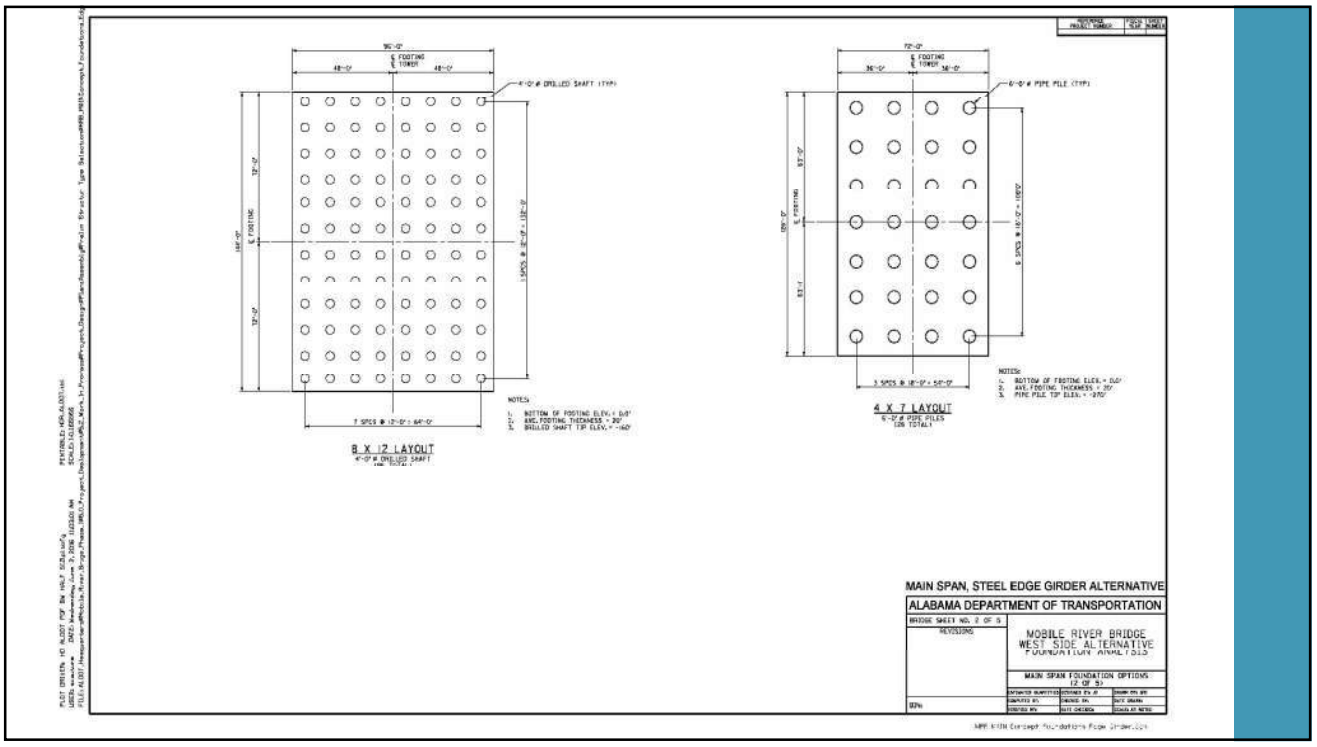

37 X 54 LAYOUT
24 X 36 LAYOUT
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
15 OF 51

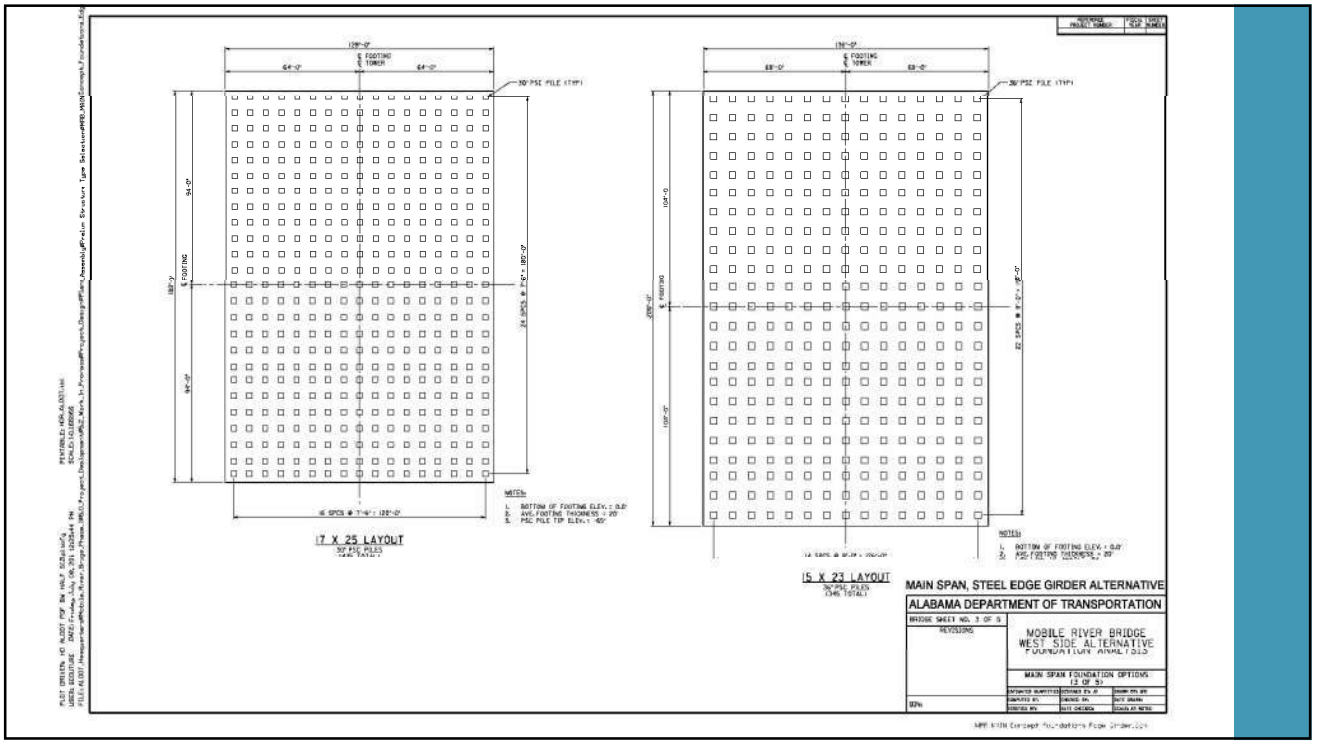
5. Design Concepts

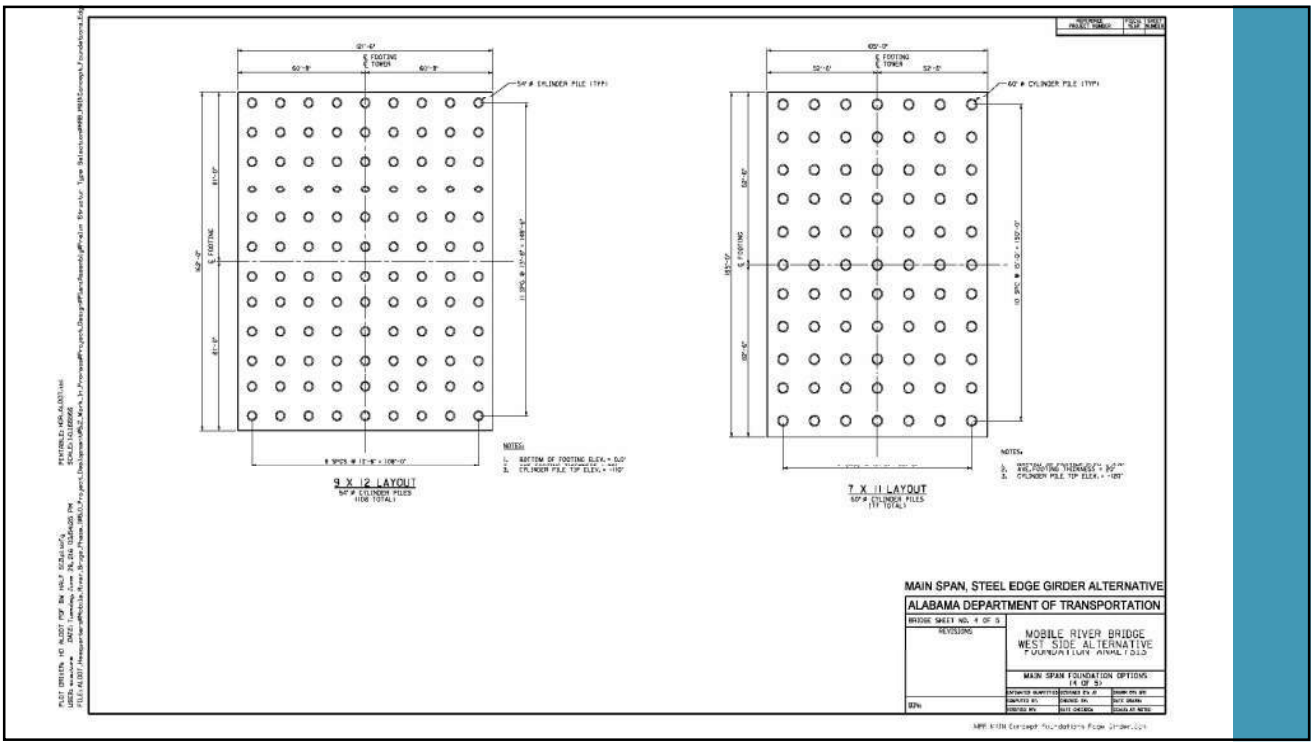
- Main Span Steel Superstructure

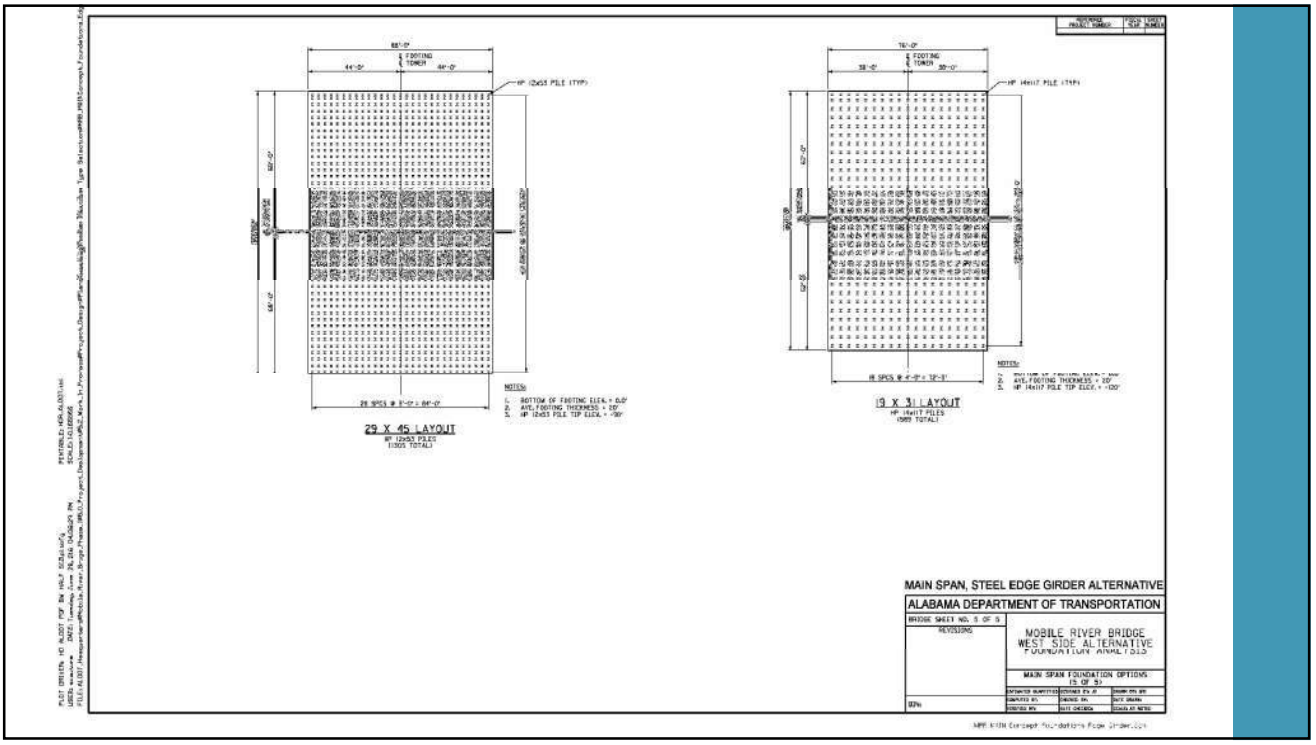





8 X 12 LAYOUT
4 X 7 LAYOUT
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)


17 X 25 LAYOUT
15 X 23 LAYOUT
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)


9 X 12 LAYOUT
54" ø CYLINDER PILES
(108 TOTAL)
7 X 11 LAYOUT
54" ø CYLINDER PILES
(77 TOTAL)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
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)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
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

10. Next Step

- Finalize Foundation Layout for Preferred West Side Mainline Alignment and Interchanges
- Evaluate potential vibration Impacts to other Structures in close Proximity to High-Level Approaches
- Constructibility Review of preferred pile Options

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

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

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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

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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.

2.0 Specifications, Codes and Design Directives

	Reference	Revision Date
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	
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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	

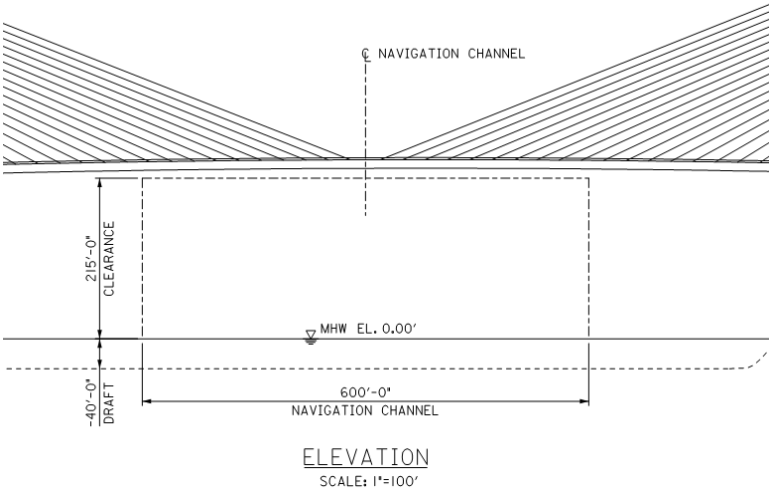


Figure 1

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	

6.0 Material Properties

Reference	Revision Date

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

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

ALDOT SDM Section 5

* 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").

6.1.2 Coefficient of thermal expansion is taken as 0.0000060 per degree F. — LRFD 5.4.2.2

6.1.3 Coefficient of shrinkage is taken as 0.0002. — LRFD 5.4.2.3

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

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		

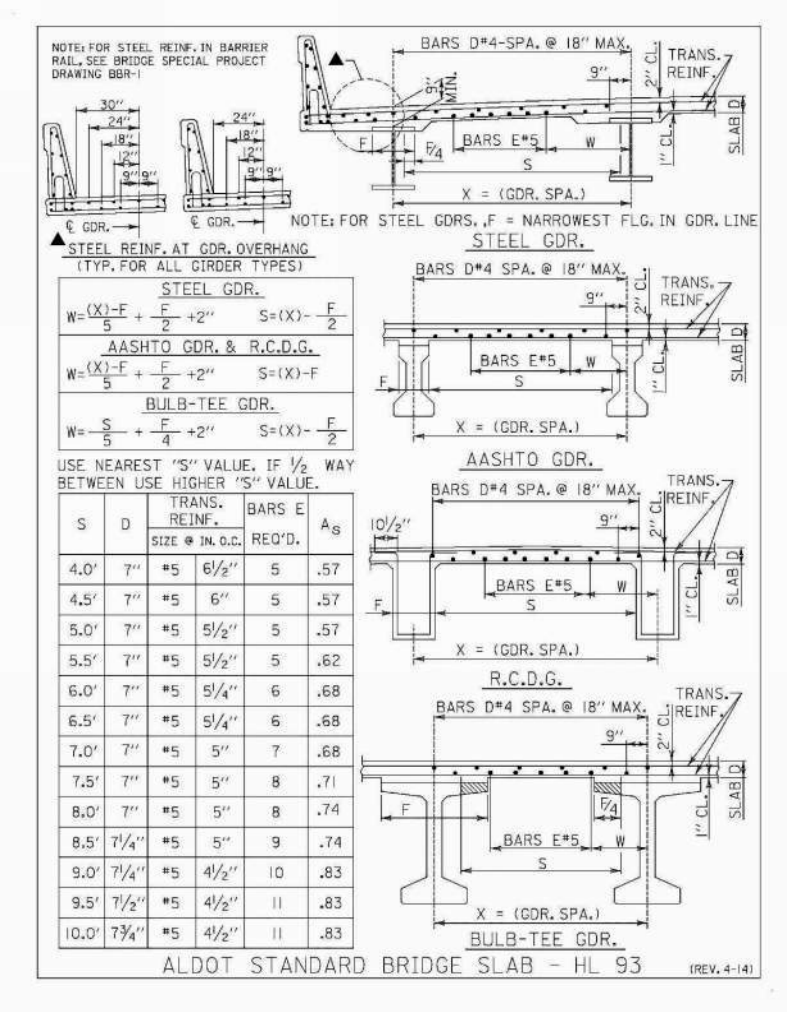
	Strands	Bars	Comments	Reference	Revision Date
Specification	A416 Grade 270 Low-Relaxation	A722 Grade 150 Type II	ASTM		
Size	0.6" strand	1 3/8" bar			
Ultimate Tensile Strength (f_{pu})	270 ksi	150 ksi		LRFD 5.4.4.1	
Yield Strength	90% f_{pu} = 243 ksi	80% f_{pu} =120 ksi		LRFD 5.4.4.1	
Modulus of Elasticity	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**



S	D	TRANS. REINF. SIZE @ IN. O.C.	BARS E REQ'D.	A_s
4.0'	7"	#5	6½"	5	.57
4.5'	7"	#5	6"	5	.57
5.0'	7"	#5	5½"	5	.57
5.5'	7"	#5	5½"	5	.62
6.0'	7"	#5	5¼"	6	.68
6.5'	7"	#5	5¼"	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'	7¼"	#5	5"	9	.74
9.0'	7¼"	#5	4½"	10	.83
9.5'	7½"	#5	4½"	11	.83
10.0'	7¾"	#5	4½"	11	.83

7.3.2.2 Finished Grade Deck Elevations for Steel Girder Spans — ALDOT SDM Section 9

Check point elevations for steel girder spans shall be as shown

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	

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		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.		

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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	

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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		

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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		

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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	

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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

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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

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

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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

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	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.		

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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		

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			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	

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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.		

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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.		

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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

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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.

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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		

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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

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	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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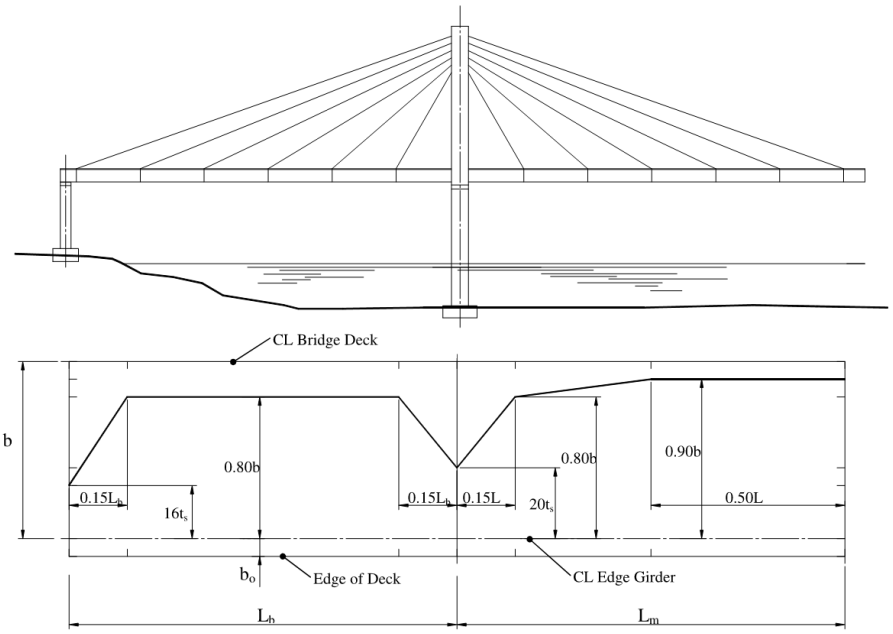
satisfy all relevant clauses of AASHTO LRFD.

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

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	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:		

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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		

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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

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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.

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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: $\phi = 0$.		
7.9.14.2.2 Axial uplift resistance: $\phi = 0.5$ (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: $\phi = 0.8$		
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

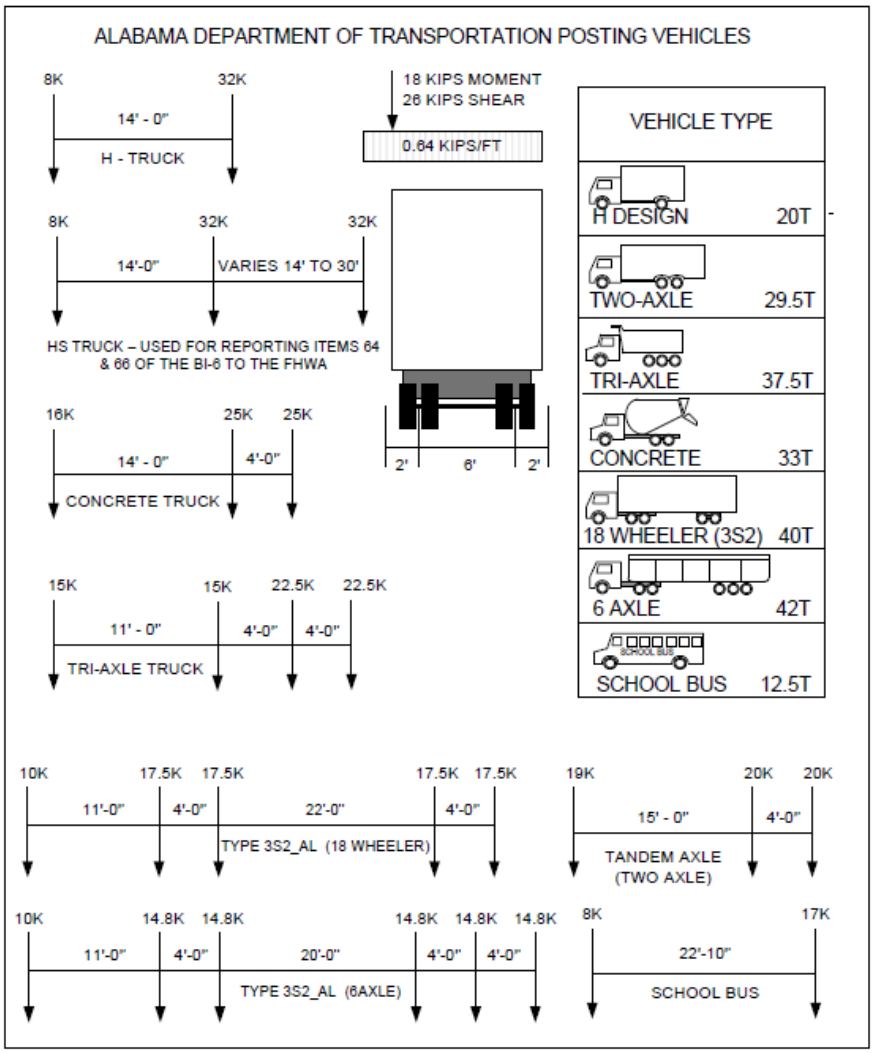
8.2.1 Transverse diaphragms do not require load rating.

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:

Air Draft Study

I-10 Mobile River Bridge Determination of Appropriate Air Draft Clearance

Project DPI-0030(005)
I-10 Mobile River Bridge and Bayway Widening
Mobile and Baldwin Counties, Alabama

December 2012

U.S Department of Transportation
Federal Highway Administration

VOLKERT

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Appendix A – Correspondence

List of Acronyms

ADABB	Air Draft Analysis on the Bayonne Bridge
ADC	Air Draft Clearance
ALDOT	Alabama Department of Transportation
ASPA	Alabama State Port Authority
BCR	Benefit to Cost Ratio
CCL	Carnival Cruise Lines
EA	Environmental Assessment
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
I-10	Interstate 10
KTMH	Keel-to-Mast Height
MACC	Mobile Area Chamber of Commerce
MHW	Mean High Water
MRB	Mobile River Bridge
MRBMEIA	Mobile River Bridge Maritime Economic Impact Analysis
NCL	Norwegian Cruise Lines
PANYNJ	Port Authority of New York and New Jersey
PC	Panama Canal
RCL	Royal Caribbean Cruise Lines
TEU	Twenty-Foot Equivalent Unit
USCG	U.S. Coast Guard
USACE	U.S. Army Corps of Engineers

1.0 INTRODUCTION

The determination of the appropriate Air Draft Clearance (ADC) for the proposed I-10 Mobile River Bridge (MRB) represents an important decision with both short and long term implications for the maritime industry in Mobile, Alabama. The ADC of the bridge will establish a constraint to the passage of vessels upstream of the proposed bridge in the future. This constraint will have an effect on the maritime industry, including limitations on the cruise industry by restricting the size of vessel that can use facilities upstream of the bridge. The size of a cruise ship calling on the Alabama Cruise Terminal has a direct impact on the potential profitability of the cruise industry in Mobile.

ADC can be defined as the vertical dimension of a vessel's Keel-to-Mast Height (KTMH) plus a vertical space for to allow passage below the lowest bridge deck component (**Figure 1**). ADC can be affected by such factors as tides, mean high water (MHW), vessel drafts, channel depths, river stage, and sea level rises. Several other factors influence ADC, including weather (temperature, fog, wind speed and direction, and precipitation), currents, salinity, ship traffic (passing), day or night operations, tug assistance, etc. The acceptable ADC, including safety clearance, should consider adverse conditions rather than ideal physical or operating conditions.

A 190-foot ADC was established in the Feasibility Study conducted for the proposed I-10 MRB project in 1997. The Alabama Department of Transportation (ALDOT) approved the 190-foot ADC in 2000, and the Federal Highway Administration (FHWA) approved the Environmental Assessment (EA) with a 190-foot ADC in 2003. Since completion of the Feasibility Study and receipt of these approvals, conditions related to the size and types of ships using the Port of Mobile have changed, and a fresh look at the appropriate ADC is warranted. Major changes in conditions along the Mobile River since 2003 include the construction and opening of the Alabama Cruise Terminal, a $30 million facility, and the resulting emergence of the cruise ship industry in Mobile; the Mobile Container Terminal, a $300 million world-class intermodal container terminal; the major expansion of the Austal shipbuilding facilities; and the sale of both Bender Shipbuilding & Repair

Company, Inc., and Atlantic Marine, Inc. (see **Figure 2**).

The purpose of this analysis is to document existing conditions that may affect a determination regarding an appropriate ADC for the proposed I-10 MRB; to evaluate current and projected trends in vessel sizes, types, and shapes; and to determine an appropriate ADC that would allow the Port of Mobile to remain competitive in the cruise industry, container and cargo shipping industry, and other maritime industries that currently operate or may choose to locate along the Mobile River in the future. A bridge permit from the U.S. Coast Guard (USCG) required for construction. The USCG will determine if the bridge, "allows reasonable needs of navigation" (USACE, 2012).

Various resources and studies are utilized in this analysis to illustrate the following points:

1) ADC can be a constraint to maritime industries;
2) Bridges in other ports around the U.S. are being raised to increase ADC to accommodate taller ships;
3) The City of Mobile is pursuing a cruise ship class with a 207-foot ADC;
4) Cruise ships can navigate with less than their preferred 15-foot clearance (some as low as 5 feet);
5) Container and cargo ships can navigate with a minimum of 3 feet of clearance;
6) Trends in vessel size indicate that ships will continue to get larger in the future; and
7) Various types of vessels and cargo may traverse to port facilities upstream of the proposed I-10 MRB in the future.

2.0 EXISTING CONDITIONS

At present, there are no vertical restrictions over the Mobile Harbor Navigation Channel south of the Cochrane-Africatown USA Bridge (Cochrane Bridge) across the northern portion of the harbor. The Cochrane Bridge, located upstream of the 40-foot deep portion of the Mobile Harbor Federal Navigation Channel, has an ADC of 140 feet over MHW (**Figure 2**).

The Federal Mobile Harbor Navigation Channel is authorized for a navigation depth of -55 feet by the Water Resources Development Act of 1986, to Mile 1, one mile south (downstream) of Mile 0. Mile 0 is the Bankhead Tunnel. The navigation depth upstream of Mile 1 is -40 feet. The Bankhead and I-10 Wallace Tunnels under the Mobile River constrain the channel depth to -40 feet at their respective locations (see **Figure 2**).

3.0 PROPOSED I-10 MRB ALTERNATIVES

Four Alternatives (A, B, B', and C) are currently being considered for the I-10 MRB. The locations of these alternatives are shown on **Figure 2**. All four of the alternatives would be located between Mile 0 and Mile 1 over the 40-foot-deep navigation channel. All four alternatives would affect the class of cruise ship that could use the Alabama Cruise Terminal located on the west side of the Mobile River (see **Figure 2**).

4.0 EFFECT OF ADC ON MOBILE MARITIME INDUSTRY

The primary issue associated with identifying an appropriate air draft clearance for the proposed I-10 Mobile River Bridge centers around the need to accommodate cruise ships at the Alabama Cruise Terminal. Secondary concerns include future port development and accommodation of cargo/container ships, as well as government marine vessels.

4.1 Effect of ADC on Cruise Industry

Three primary cruise lines operate in the South Atlantic/Gulf Coast markets: Carnival Cruise Lines (CCL), Norwegian Cruise Lines (NCL), and Royal Caribbean Cruise Lines (RCL). These three lines currently have 57 vessels in service or soon to be in service and range in size from 848 to 1,181 feet long with air drafts between 150 and 208 feet. The current trend in cruise ship design is to develop longer cruise vessels without corresponding increases in air draft (Martin, 2012).

In 2000, the City of Mobile and Mobile Area Chamber of Commerce formed the Cruise Industry Task Force to recruit a major cruise line to the Port of Mobile. The Task Force is comprised of various stakeholders and representatives with a vested interest in the viability of the Port of Mobile. In 2002, the City of Mobile was successful in recruiting CCL to call on the Port of Mobile, conducting a series of "trial" runs to determine whether the demand for cruise ships departing from Mobile would make the commitment to have Mobile as a port of call worth the investment. The trial cruises were successful, and, in 2004, the City invested approximately $30 million in the development of a new cruise terminal along the Mobile River. As the only cruise terminal in Alabama, the facility is appropriately named the Alabama Cruise Terminal. Additional investments totaling nearly $15 million were also made for the dock area that supports the terminal (**Appendix A**, MACC letter to ALDOT, April 3, 2012).

From 2004 through October 2011, CCL operated continuous cruise service from the Alabama Cruise Terminal. In 2007, the Port of Mobile was Carnival's "Port of the Year" (Alabama Cruise Terminal website, 2010). The initial cruise ship in Mobile was the Holiday, which has an air draft of approximately 157 feet and accommodates approximately 1,452 passengers and 660 crew members. The Holiday was profitable and successful during its operation, which ended in 2008, when the Elation began calling on the Port of Mobile. The Elation is a Fantasy-class ship which has an air draft of approximately 177 feet and accommodates approximately 2,052 passengers and 920 crew members. The cruises, which included multi-day cruises to the Western

Caribbean, were frequently sold out and have been noted by Carnival Cruise Lines as being profitable (**Appendix A**, MACC letter to ALDOT, April 3, 2012).

Due to market conditions in Mexico and high fuel prices, Carnival Cruise Lines decided to discontinue cruise service from the Port of Mobile (**Appendix A**, MACC letter to ALDOT, April 3, 2012). The last cruise to sail from Mobile occurred in October 2011.

The Cruise Industry Task Force (CITF), along with officials from the City of Mobile and an independent consultant to the City of Mobile, is optimistic about the likelihood of the cruise industry returning to the Alabama Cruise Terminal at some point in the near future. According to the Mobile Area Chamber of Commerce, the City is "actively pursuing all of the major cruise lines that currently work out of the Gulf" as well as "several who have an interest to establish new services in the Gulf region." In addition, it is possible that CCL will choose to return to Mobile rather than allow its competitors to dominate the proven profitable market that is served by the Alabama Cruise Terminal. Letters from the City of Mobile and the Mobile Area Chamber of Commerce are contained in **Appendix A** of this report.

By letter dated March 23, 2012, Mayor Samuel Jones noted that the "repositioning of ships leaving a temporary vacancy in a port is not at all unusual in the cruise industry." Mayor Jones also stated that "replacing the ship is a process which may take time but in the end will result in the reestablishment of the cruise industry in Mobile" (**Appendix A**, City of Mobile letter, March 23, 2012). As a result, the determination of an appropriate air draft clearance for the proposed I-10 MRB should take into consideration the long-term accommodation of existing and future cruise ships. Efforts made by the City's consultant responsible for recruiting cruise industry to Mobile indicate that a major cruise line has stated that they are interested in further discussions with Mobile about placing one of its current ships within a market that would include Mobile once its new ships are in service at other larger ports.

4.1.1 Clearance Required for Safe Passage

With regard to adequate clearance for safe passage of cruise ships beneath a bridge, the U.S. Coast Guard (USCG) does not specify a minimum clearance. The decision whether or not to attempt passage under a bridge is left to the vessel master's decision. Cruise lines have indicated a preference for a 15-foot clearance; however, depending upon the circumstances, the master of a vessel may accept a clearance of less than 15 feet. Coordination with the Jacksonville Port Authority indicated that the CCL's Fascination, which passes under the Dames Point Bridge in Jacksonville, Florida, has operated with a ± 5-foot clearance for the last decade (Jacksonville Port Authority, 2011). This example shows that CCL is currently operating cruise ships with much less than a preferred 15-foot clearance.

The CCL Fantasy-class Elation, which called on Mobile until October 2011, is 855 feet long, has an air draft of 177 feet, and carries 2,052 passengers and 920 crew members. Cruise ships typically have a water draft between 25 and 30 feet. Elation has a water draft of 25 feet, 9 inches. The Elation used the Pier C Turning Basin or the Three-Mile Creek Turning Basin to turn around (see **Figure 2**). The Freedom, a CCL Conquest-class cruise ship, is 953 feet long, has a draft of 27 feet, and has an air draft of 207 feet. This vessel carries 2,974 passengers and 1,150 crew members. The City of Mobile and CITF believe a Conquest-class cruise ship, or its equivalent, is the next logical cruise ship for Mobile (**Appendix A**, MACC letter to ALDOT, April 3, 2012).

Figure 3 illustrates the clearance for the Elation and the Freedom with an I-10 MRB with an ADC of 215 feet. The Freedom could require special conditions and likely tug assistance to turn around in the Three-Mile Creek Turning Basin. The Harbormaster for the ASPA stated that the ships that have a length of approximately 950 feet (similar to the Freedom) could likely turn in the Three-Mile Creek Turning Basin. He noted that container ships, 965 feet long, turned two to three times a week in the Three-Mile Creek Turning Basin prior to completion of the Pinto Island Turning Basin (**Appendix A**, Terry Gilbreath, Harbormaster, email to N.D. "Skeeter" McClure, Volkert, Inc., March

3, 2011).

As shown on **Figure 3**, the clearance for the Elation at the edge of the channel (greatest restriction) would be 38 feet with a 215-foot ADC. The clearance for the Freedom would be 8 feet. Because of the four percent grade of the proposed I-10 MRB, the maximum ADC at the center of the navigation channel would be 220 feet, providing a greater clearance for ships traveling near the center of the navigation channel (**Figure 3**). If the Freedom navigated in the middle third of the 600-foot navigation channel, it would have a clearance of approximately 13 feet.

The minimum clearance for the current cruise ship, Elation, would be 13 feet with a 190-foot ADC. Cruise ships with larger air drafts, such as the Freedom which has a 207-foot air draft, could not pass under a bridge with a 190-foot ADC, even under ideal conditions.

Incremental ADCs between 190 feet and 215 feet would not allow sufficient clearance for larger cruise ships to call on the Alabama Cruise Terminal. For example, the clearance for a cruise ship such as the Freedom at the edge of the channel for a bridge with an ADC of 210 feet would be only 3 feet (210'-207'=3). However, an ADC of 215 feet would provide a clearance of 8 feet (215'-207'=8), which is greater than the clearance the Fascination currently has under the Dames Point Bridge.

Measures to monitor air gaps and assist in safe passage of vessels under obstructions have been developed and implemented and could be installed on the proposed I-10 MRB should they be deemed necessary. The National Oceanic and Atmospheric Administration (NOAA) PORTS has installed air gap sensors on bridges to assist vessel masters in knowing the clearance between the water surface and the bridge. These sensors have been placed on various bridges through the U.S. (including the Dames Point Bridge in Jacksonville, Florida; the Bayonne Bridge and the Verrazzano Narrows Bridge in New York; the Crescent City Connection and the Huey P. Long Bridge in New

Orleans; and the Gerald Desmond Bridge in Long Beach, California) to monitor air gaps in conjunction with tide levels and currents. These sensors assist bar pilots and vessel operators in navigating under vertical obstructions (NOAA, 2011).

4.1.2 Cruise Ship Fleets

The Mobile Area Chamber of Commerce has noted the necessity to provide an air draft clearance of 215 feet to accommodate cruise ships that would have up to 207-foot air drafts. In their April 3, 2012 letter, the Mobile Area Chamber of Commerce stated that, prior to the economic decline resulting in CCL's repositioning their ships to a different market, the cruise line indicated that they had plans to bring a Conquest-class cruise ship (with a 207-foot air draft) to the Alabama Cruise Terminal. By letter dated April 3, 2012, the Mobile Area Chamber of Commerce states, "Our option to handle those ships goes away, along with most of our chances to participate in the cruise industry, if we do not have the minimum height of 215 feet" (**Appendix A**). Based on past market experiences in the cruise industry and trends in cruise ship designs and sizes, it is reasonable and prudent to anticipate that the types of ships that will call on the Port of Mobile in the future would be comparable in size and class to the Conquest class.

The types and sizes of cruise ships currently in service that could be reasonably expected to call on the Port of Mobile have air drafts ranging from approximately 150 feet to 210 feet (**Table 1**). Discussions between the Mobile Area Chamber of Commerce and cruise lines indicate that a bridge with a 215-foot air draft clearance would be a "safe and approved clearance" to allow the operation of cruise ships (**Appendix A**, MACC letter to ALDOT, March 20, 2012).

Table 1: Cruise Ships that could call on the Alabama Cruise Terminal

Cruise Line	Class	Vessel	Year in Service	Length (feet)	Air Draft (feet)	Passengers	Crew
CCL	Conquest	Conquest	2002	953	207	2,974	1,150
CCL	Conquest	Freedom	2006	952	207	2,974	1,150
CCL	Conquest	Glory	2003	953	207	2,974	1,150
CCL	Conquest	Liberty	2005	952	207	2,974	1,160
CCL	Conquest	Valor	2004	953	207	2,974	1,180
CCL	Destiny	Destiny	1996	892	208	2,642	1,040
CCL	Destiny	Triumph	1999	893	208	2,758	1,100
CCL	Destiny	Victory	2000	893	208	2,758	1,100
CCL	Fantasy	Ecstasy	1991	855	177	2,052	920
CCL	Fantasy	Elation	1998	855	177	2,052	920
CCL	Fantasy	Fantasy	1990	855	177	2,052	920
CCL	Fantasy	Fascination	1994	855	177	2,052	920
CCL	Fantasy	Imagination	1995	855	177	2,052	920
CCL	Fantasy	Inspiration	1996	855	177	2,052	920
CCL	Fantasy	Paradise	1998	855	177	2,052	920
CCL	Fantasy	Sensation	1993	855	177	2,052	920
CCL	Holiday	Celebration	1987	733	161	1,486	670
CCL	Holiday	Holiday	1985	728	157	1,452	660
CCL	Spirit	Legend	2002	963	172	2,124	930
CCL	Spirit	Miracle	2004	963	172	2,124	934
CCL	Spirit	Pride	2001	963	172	2,124	930
CCL	Spirit	Spirit	2001	960	172	2,124	930
NCL		Norwegian Crown	1988	616	134	1,052	461
NCL		Norwegian Dawn	2002	965	171	2,244	1,100
NCL		Norwegian Dream	1993	754	150	2,100	700
NCL		Norwegian Jewel	2005	965	171	2,376	1,154
NCL		Norwegian Majesty	1992	679	147	1,462	570
NCL		Norwegian Pearl	2006	965	171	2,466	1,010
NCL		Norwegian Spirit	1998	880	189	1,996	965
NCL		Norwegian Star	2001	965	171	2,244	1,100
NCL		Norwegian Sun	2001	848	180	2,002	950
NCL		Norwegian Wind	1993	754	150	1,730	700
RCL	Mega	Majesty of the Seas	1992	880	173	2,356	812

Cruise Line	Class	Vessel	Year in Service	Length (feet)	Air Draft (feet)	Passengers	Crew
RCL	Mega	Monarch of the Seas	1991	880	173	2,390	858
RCL	Mega	Sovereign of the Seas	1988	880	173	2,292	840
RCL	Radiance	Brilliance of the Seas	2001	963	174	2,110	859
RCL	Radiance	Jewel of the Seas	2004	963	174	2,110	859
RCL	Radiance	Radiance of the Seas	2001	96 3	174	2,112	857
RCL	Radiance	Serenade of the Seas	2003	963	174	2,110	891
RCL	Vision	Enchantment of the Seas	1997	917	164	1,950	760
RCL	Vision	Grandeur of the Seas	1996	917	164	1,950	760
RCL	Vision	Legend of the Seas	1995	867	164	1,804	726
RCL	Vision	Rhapsody of the Seas	1997	915	171	1,998	765
RCL	Vision	Splendour of the Seas	1996	867	164	1,804	720
RCL	Vision	Vision of the Seas	1998	915	171	2,000	742
RCL	Voyager	Adventure of the Seas	2001	1,020	208	3,114	1,185
RCL	Voyager	Explorer of the Seas	2000	1,021	208	3,114	1,185
RCL	Voyager	Mariner of the Seas	2003	1,021	208	3,114	1,185
RCL	Voyager	Navigator of the Seas	2002	1,021	208	3,114	1,185
RCL	Voyager	Voyager of the Seas	1999	1,021	208	3,114	1,176
RCL		Empress of the Seas	1990	692	152	1,602	668
RCL		Freedom of the Seas	2006	1,112	210	3,600	1,360

CCL = Carnival Cruise Line; RCL = Royal Caribbean Cruise Line; NCL = Norwegian Cruise Line
Source: Martin, 2010

As shown in **Table 1**, a bridge with a 215-foot ADC could accommodate nearly all of the cruise ships likely to call on Mobile. It is noted that those ships that are greater than 1,000 feet in length may require special measures to turn (such as backing down the Mobile River to the Pinto Island Turning Basin), but the ships would not be precluded from passing under the proposed I-10 MRB based on ADC.

4.1.3 Economic Benefits of Cruise Industry

There are numerous benefits for increasing the ADC from 190 feet to 215 feet, including safety, fewer constraints on future utilization of the navigation channel, and potential economic benefits to the region. According to the City of Mobile, cruise service from Mobile attracted an average of approximately 182,000 passengers per year. Passenger and crew spending contributed nearly $13 million to the local economy and nearly $142 million throughout the state of Alabama in 2010 (**Appendix A**, City of Mobile letter to ALDOT, March 23, 2012). These economic impacts clearly demonstrate the need for the Port of Mobile to remain competitive in the cruise industry, as well as the City's need to pay off the debt incurred in constructing the Alabama Cruise Terminal.

An analysis was conducted to determine the potential monetary benefits associated with an ADC that could accommodate a larger cruise ship, such as the Freedom from the Conquest class compared to a Fantasy-class ship such as the Elation, which called on Mobile until October 2011.

Table 2 shows the increase in potential economic benefits for the Freedom compared to the Elation. Available draft, an ADC of 215 feet, and existing turning basin dimensions would accommodate the Freedom, if a business decision were made to use this size ship. The projected increase in potential benefits could not be realized if a bridge with an ADC of 190 feet were constructed.

Table 2: Increase in Economic Benefits between a Fantasy Class (FC) and Conquest Class (QC) Cruise Ship in Mobile

Ship	Passengers	Crew	Total Jobs	Economic Impact Category ($ million per year)			
				Total Income and Consumption	State/Local Taxes	Local Purchases	Value to Regional Economy
Elation (FC)	2,052	920	607	24.1	2.3	6.4	32.9
Freedom (QC)	2,974	1,150	778	29.6	2.9	8.4	40.9
Increase	922	230	171	5.5	0.6	2.0	8.0

Source: Martin, 2010 and 2011

The $8.0 million for potential value to the regional economy represents the annual net benefits for being able to accommodate the larger cruise ship. The benefits are the same for all four Build Alternatives. While an ADC of 190 feet or 210 feet could accommodate a number of ships, including some cruise ships, these ADCs are not preferred. Constructing a bridge with an ADC that would not accommodate a Conquest-class or similar size ship would prevent the local, state, and regional economy from realizing the potential economic benefits quantified in **Table 2**.

4.2 Effects of ADC on Container Ships/Marine Cargo Activity

A secondary consideration in determining an appropriate ADC is the ability to accommodate container and cargo ships that currently call on the Port of Mobile and those that are projected to call on the Port of Mobile.

The Port of Mobile is also home to public and private marine cargo terminals located along the Mobile River. The ASPA's public terminals include 27 general cargo and container berths that handled 2.8 million tons of forest products, steel, frozen poultry, and other cargo in 2009. The ASPA also operates a bulk material handling plant that handled 0.7 million tons of coal and iron ore in 2009 and McDuffie Terminals which handled 16.1 million tons of coal in 2009. A total of 766 vessels called on the Port of Mobile in 2009. Private terminals along the Mobile River include the Alabama Bulk Terminal, Gulf Coast Asphalt, Plains Oil Terminal,

Mobile Marine Terminal, Mobile River Terminal, Shell Chemical, Trigeant, and Vulcan Materials, among others. These private terminals handle bulk products, such as ore, coke, and petroleum. A total of 669 vessels called on private terminals in 2011 (**Appendix A**, ASPA letter to ALDOT, April 18, 2012).

According to information provided by Captain Terry Gilbreath, Harbormaster, a total of 1,443 vessels called on the Port of Mobile in 2011 (ASPA, 2012b). As shown in **Table 3**, the Port of Mobile has seen a steady increase in vessel usage since 2009 (when a global decrease in shipping occurred).

Table 3: Vessels Calling on the Port of Mobile

Year	Total Vessels in Port
2011	1,443
2010	1,368
2009	1,351
2008	1,571
2007	1,368

Source: ASPA, 2012b (Appendix A)

An expansion of the Panama Canal (PC) is currently underway that will allow larger ships to transit the PC. The expansion is scheduled to be complete in 2015. Container ships are being designed to take advantage of the PC expansion. A February 2012 publication prepared by the National Association of Development Organizations with support from the Federal Highway Administration, entitled, "Freight Transportation and Economic Development: Planning for the Panama Canal Expansion," emphasizes the need for ports to be prepared for the potential of larger vessels (NADO, 2012).

An article from the January 2010 Alabama Seaport outlines the ASPA's strategy to capitalize on the expansion. Jimmy Lyons, Director and CEO of the ASPA, stated, "We have been laying the groundwork to take advantage of the Panama Canal expansion for quite some time. We opened the Mobile Container Terminal and secured funding for a new turning basin with our eyes to the south and the Far East. The Port of

Mobile is strategically positioned to be a key player in trade through the canal" (Alabama Seaport, 2010).

The article includes the dimensions of the Post-Panamax container vessels. The Mobile Harbor Navigation Channel and turning basin near the Mobile Container Terminal (renamed APM Terminal Mobile) currently have a depth of 45 feet and could be deepened to their authorized depth of 55 feet. Economic and environmental studies would be required to justify any increase of the depth of the navigation channel beyond 45 feet. The PC expansion would allow vessels to have a 50-foot draft. For various reasons, ships do not always load to take advantage of this maximum draft capacity (USACE, 2012). For example, container ships could light load to accommodate the existing 45-foot channel in the Mobile Harbor.

It should be noted that while the ASPA expects vessels from the PC to call on the Port of Mobile, vessels from other parts of the world that are not restricted to the 201-foot ADC of the PC may also choose to call on the Port of Mobile. Therefore, it is important to consider the potential heights of vessels that may originate from other parts of the world in the determination of an appropriate ADC for the proposed I-10 MRB.

In February 2011, Maersk Line signed a contract with a Korean shipbuilding firm for ten 18,000 twenty-foot equivalent unit (TEU) container ships. These ships will be 1,312 feet long with a KTMH of 239.5 feet and an ADC of approximately 189.5 feet (assuming a 50-foot draft). While these very large ships are not expected to call on the Port of Mobile, they illustrate the overall trend for larger container ships. In addition, as larger container ships come into the fleet, there is a cascading effect. Smaller ships that are replaced by larger, newer ships are assigned to smaller ports, such as Mobile. Cascading typically increases average vessel size for each trade service (USACE, 2012).

The U.S. Army Corps of Engineers (USACE), New York District, conducted an Air Draft

Analysis on the existing Bayonne Bridge (ADABB) for the Port Authority of New York and New Jersey (PANYNJ) (USACE, 2009). The ADABB and its results are used in this analysis to provide background information on issues involved and assisted in formulating an approach to identify an appropriate ADC for a proposed new bridge over a navigation channel that does not currently have ADC constraints. The USACE utilized the Emma Maersk, with a TEU capacity of 12,508, for the ADABB for the PANYNJ. This container ship has a length of 1,306 feet and a KTMH of 251 feet. The ADC for this ship, assuming a 50-foot draft, would be 201 feet. The ADC of the proposed jacked-up or new replacement for the Bayonne Bridge is 215 feet, providing a clearance of 14 feet from the top of the ship to the vertical height of the bridge. This ship is used as a surrogate for this analysis to demonstrate ADC requirements associated with larger container ships. It is recognized that while this ship would not be able to turn in the turning basins in the Mobile Harbor at this time, other container ships that may choose to call on the Port of Mobile in the future could have similar height and draft dimensions and shorter lengths than the Emma Maersk. **Figure 4** shows the Emma Maersk under an I-10 MRB with an ADC of 215 feet.

A new larger turning basin (Pinto Island Turning Basin) was recently constructed at approximately Mile 2 across from the Mobile Container Terminal (APM Terminal Mobile) (see **Figure 2**). The Pinto Island Turning Basin measures 1,175 feet by 715 feet. The new turning basin can turn ships up to 1,100 feet long. This turning basin currently has a navigation depth of 45 feet. It is authorized to be deepened to 55 feet, if needed and justified should ships with larger drafts begin to call on the Port of Mobile. Energy efficiency, fuel consumption, and emission reductions are drivers toward larger ships. Navigation features will tend to be improved and expanded to accommodate larger vessels as they become available if economically justified. For example, the ASPA and the USACE are currently evaluating modifications to the Pinto Island Turning Basin to increase the length of ships that can be turned and to improve the ability to turn larger ships efficiently.

By resolution dated June 24, 2008, the Alabama State Port Authority (ASPA), advocated the following regarding ADC for the I-10 MRB: "In the event a new bridge is to be constructed over the Mobile River, then the new bridge should have a vertical clearance of at least 215 feet, but no less than 195 feet." A 215-foot ADC would accommodate container ships expected to call on the Port of Mobile while also allowing high air draft cruise ships to pass underneath the bridge. By letter dated January 11, 2012, Mr. Jimmy Lyons confirmed that the position of ASPA has not changed (**Appendix A**).

Additional coordination with the ASPA specifically addressing the ADC issues and turning basin dimensions provides insight on the ASPA's views on the ADC. In an e-mail dated March 3, 2011, Captain Terry Gilbreath, Harbormaster, stated: "As for the air draft, the ASPA will support the higher the better. It may be a hundred years before another bridge is built crossing the Mobile River and we have no idea what vertical clearance needs will be at that point, but we would support the proposed vertical clearance of 215 feet" (**Appendix A**, ASPA email , March 3, 2011).

In a letter dated April 18, 2012, Captain Gilbreath provided additional background information on his experience with ports and navigation features, as well as explaining his role as Harbormaster for the Port of Mobile. In closing, he reaffirmed that, in the future, many ships will require a vertical clearance of at least 215 feet (**Appendix A**).

4.3 Government Vessel Activity

The U.S. Navy and the USCG were contacted for their views regarding potential impacts of bridge heights on government vessels that might call on the Port of Mobile. Each agency indicated that the vessels that would potentially call on Mobile for repair or port-of-call activities would not be impacted by the proposed ADC (Martin, 2012).

In addition, the Navy was contacted to determine if there were restrictions for placement of a bridge near a shipbuilding company constructing Navy vessels. Austal

USA began operations on the east bank of the Mobile River in 1999, and its operations have expanded considerably in the last decade. Austal now occupies three specific parcels of land involved in the fabrication, assembly, and final outfitting of high speed aluminum vessels for the U.S. Navy: a 115-acre site, 300 feet off-river, encompassing the Module Manufacturing Facility; a 15-acre site on the Mobile River encompassing the Assembly Bay Yard; and a 35-acre site along the Mobile River to the south encompassing the new Outfitting Yard. Austal's business plan projects that its employment will increase from 1,800 to over 4,000 personnel between 2010 and 2014 (Martin, 2012).

In 2008, the Navy awarded Austal a contract to build 10 Joint High Speed Vessels. The Navy Long Range Shipbuilding Plan is to build 41 of these vessels over the 30-year span of the Plan. In 2010, the Navy awarded Austal a contract to build 10 Littoral Combat Ships. The Navy Long Range Plan is to build 55 of these vessels. These programs establish a long-term steady prospect to provide vessels to the Navy on a continuing basis. The Navy has stated there currently are no Federal Regulations that would prevent a shipyard located under or in proximity to a bridge from bidding on or being awarded a Navy contract (Martin, 2012). The Littoral Combat Ships being constructed by Austal are approximately 417 feet long, with a 111-foot air draft clearance, and a 14.7-foot draft (Austal, 2012). The proposed 215-foot ADC of the proposed I-10 MRB would not affect the passage of Littoral Combat Ships from Austal's facilities to the Gulf of Mexico and is not expected to restrict vessels currently under contract or those expected to be constructed by Austal.

5.0 COMPARISON OF ADC CONSTRAINTS FOR OTHER PORTS

The Port of Mobile competes with and will continue to compete with the Ports of New Orleans, Savannah, Charleston, and Tampa, which currently have bridges and ADC constraints. The Port of Mobile also competes with ports that do not have ADC constraints, such as Gulfport and Houston. **Table 4** shows height obstructions to port facilities by bridges at various large ports handling container and cargo ships. In

addition, the ports of New Orleans, Jacksonville, and Tampa accommodate cruise ships. Knowing the height restrictions that exist at other ports with which the Port of Mobile competes, as well as those with which it does not currently compete, assists in the determination of an appropriate ADC to keep the Port of Mobile competitive and economically vital by accommodating as many vessels as possible while providing safe and efficient movement of people and goods via I-10. The fact that ports in Long Beach and New York/New Jersey are willing to expend substantial amounts of money to raise existing bridges to accommodate larger ships indicates that ports recognize the need to remain competitive in the shipping market.

Table 4: Port Facilities with Significant Height Obstructions to Large Ships

Location	Obstruction	Height of Restriction (MHW)
San Francisco/Oakland	Golden Gate Bridge	225 feet
Oakland	Oakland Bay Bridge	220 feet
New York & New Jersey	Verrazano Narrows Bridge	219 feet
Panama Canal	Bridge of the Americas	201 feet
Los Angeles	Vincent Thomas Bridge	185 feet
Savannah	Talmadge Bridge	185 feet
Charleston *	Arthur Ravenel, Jr. Bridge	185 feet
Jacksonville *	Dames Point Bridge (Napoleon Bonaparte Broward Bridge)	175 feet
Tampa *	Sunshine Skyway Bridge	175 feet
New Orleans	Crescent City Connection (most downstream bridge on MS River)	170 feet
Long Beach	Gerald Desmond Bridge	156 feet**
New York & New Jersey	Bayonne Bridge	151 feet***

Source: USACE, 2009

* Southeastern bridges were added to table for comparison purposes.

** Scheduled to be increased to 200 feet

*** Scheduled to be increased to 215 feet

6.0 BENEFIT TO COST ANALYSIS FOR 190'ADC vs. 215' ADC

An ADC of 190 feet was previously proposed for the I-10 MRB prior to the construction of the Alabama Cruise Terminal and the APM Terminal Mobile. As shown on **Figures 5, 6, and 7**, a bridge with an ADC of 190 feet would severely limit the passage of tall ships. An ADC of 190 feet cannot accommodate a CCL Conquest-class cruise ship.

6.1 Preliminary Construction Cost

Table 5 shows a cost comparison between an I-10 MRB with an ADC of 190 feet and an ADC of 215 feet. The maximum increase in bridge cost is 3.6% for a 25-foot increase in ADC.

Table 5: Cost Comparison of I-10 MRB at 190-Foot ADC and 215-Foot ADC

Alternative	190 feet ($ million)	215 feet ($ million)	Increase ($ million)	% Increase
A	640.3	661.4	21.1	3.3
B	646.5	669.6	23.1	3.6
B'	647.9	671.2	23.3	3.6
C	659.0	677.9	18.9	2.9

6.2 Benefit to Cost Ratio (BCR)

For this analysis, the increase in construction cost to increase the ADC from 190 feet to 215 feet for the four alternatives was converted to an average annual cost using a Present Value Worth Equations Formula Analysis Calculator. A project life of 50 years was assumed with an interest rate of 4.125%, the 2011 PL-566 Water Resource Discount Rate prescribed for USACE Benefit to Cost Ratio (BCR) calculations. **Table 6** contains the results of the BCR analysis.

Table 6: Benefit to Cost Ratio (BCR) for a Bridge with an Increase in ADC from 190 Feet to 215 Feet

Alternative	Incremental Construction Cost ($M)	Average Annual Cost ($M)	Average Annual Benefits ($M)[1]	BCR	Net Benefits ($M)[2]
A	21.1	1.00	8.0	8.0	7.0
B	23.1	1.10	8.0	7.3	6.9
B'	23.3	1.11	8.0	7.2	6.9
C	18.9	0.90	8.0	8.9	7.1

[1] See **Table 2**

[2] Average Annual Benefits minus Average Annual Cost

A BCR ranging from 7.2 to 8.9 demonstrates a strong positive economic incentive for an ADC of 215 feet. The potential net benefit ($6.9 to $7.1 million per year) also supports the higher ADC from a regional economic viewpoint.

The BCR calculated above only considers the potential benefits associated with a larger cruise ship. Other economic benefits will likely accrue due to the 215-foot ADC that would further increase the positive economic impacts to the region.

7.0 ADDITIONAL COORDINATION RELATED TO ADC

Coordination with the USCG has been ongoing throughout the development of the proposed project. The USCG has not provided any indication that they would object to the 215-foot ADC. The USCG has indicated that the proposed bridge should not have a vertical clearance less than that of the Cochrane-Africatown Bridge, which has an ADC of 140 feet. The USCG was provided two packages of information on the proposed bridge on June 24, 2003 and May 22, 2008. In addition, USCG participated in Public Involvement and Agency coordination meetings that presented the proposed I-10 Mobile River Bridge with an ADC of 215 feet, as follows:

Meeting	Date
Public Involvement Meeting	September 2, 2010
Agency Coordination Meeting	February 2, 2012
Section 106 Consulting Parties Meeting	July 26, 2012

Additional coordination was conducted regarding bridge safety clearances required by USCG in November 30, 2012.

The Federal Aviation Administration (FAA) was invited by the FHWA to serve as a Cooperating Agency on the development of the EIS for the proposed project. The FAA declined the invitation to serve as a Cooperating Agency and indicated that permits would be required for the proposed bridge related to lighting requirements for the tops of the bridge pylons. The RSA Tower, with height of 745 feet, is located in close proximity to the proposed bridge. The top of the proposed bridge pylons would be approximately 515 feet high and would not be located within flight paths.

8.0 SUMMARY AND CONCLUSIONS

At present, the Mobile Harbor Navigation Channel has no vertical restrictions downstream of the Cochrane-Africatown Bridge. The proposed I-10 MRB represents a

decision with long-term implications. Two ADCs, 190 feet and 215 feet were evaluated. The difference in cost is approximately 3.6 percent. The ADC of a bridge becomes a constraint that can limit the passage of vessels and suppress economic efficiency and growth of maritime enterprises as well as compromising economic viability and the economic vitality of these enterprises. The ADC can also present safety considerations that can be exacerbated during adverse conditions. Adequate protection of vessels, passengers/crews, and the bridge from possible collisions, damage, and threats to human lives represents an important consideration.

An ADC of 190 feet would create serious restrictions and safety concerns for tall ships, particularly cruise ships with air drafts ranging up to 210 feet, which are being actively pursued by the City of Mobile and are expected to return to Mobile. An ADC of 215 feet would accommodate a cruise ship of the CCL Conquest class (2,974 passengers) (or similar size ship) with a clearance of 8 feet. An ADC of 190 feet would not accommodate larger cruise ships or container ships. A BCR analysis for increasing the ADC from 190 feet to 215 feet for cruise ships demonstrated potential BCR from 7.2 to 8.9 and net benefits from $6.9 to $7.1 million per year, depending upon the alternative. Potential economic benefits resulting from larger cruise ships being able to call on the Alabama Cruise Terminal would be lost if the ADC was 190 feet.

As demonstrated by projects occurring at the PANYNJ and the Port of Long Beach, ship sizes are increasing, and existing bridges that restrict navigation are being replaced or modified, at high costs, to achieve greater ADCs. The Bayonne Bridge will be raised from an ADC of 151 feet to 215 feet for an estimated cost of $1.5 billion at the PANYNJ, and the ADC of the Gerald Desmond Bridge will be increased from 156 feet to 200 feet for an estimated cost of $800 million at the Port of Long Beach. The need to raise the proposed I-10 MRB in the future to accommodate larger ships can be avoided by providing an adequate ADC when it is initially constructed.

Based upon the potential to preclude future navigation options for taller cruise ships

and other marine vessels, safety considerations, a relatively small investment cost, and a high BCR, an ADC of 215 feet is justified for the I-10 MRB. An ADC of 215 feet would allow the Port of Mobile to remain competitive in the cruise industry and container cargo shipping with other ports that are unobstructed, such as Gulfport and Houston, as well as those that are currently obstructed, such as New Orleans, Savannah, Charleston, Jacksonville, and Tampa.

Based upon this analysis and coordination with the ASPA, City of Mobile, Mobile Area Chamber of Commerce, and other maritime interests, it has been demonstrated that an ADC of 215 feet is ideal to maintain and promote the economic viability of the maritime industries, especially the cruise industry, now utilizing the Mobile Harbor Navigation Channel. An ADC of 215' also eliminates impediments to future growth and expansion of these maritime industries.

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FIGURES

I-10 MOBILE RIVER BRIDGE
DESCRIPTION OF BRIDGE, SHIP, AND NAVIGATION FEATURES
FOR AIR DRAFT CLEARANCE 215 FEET

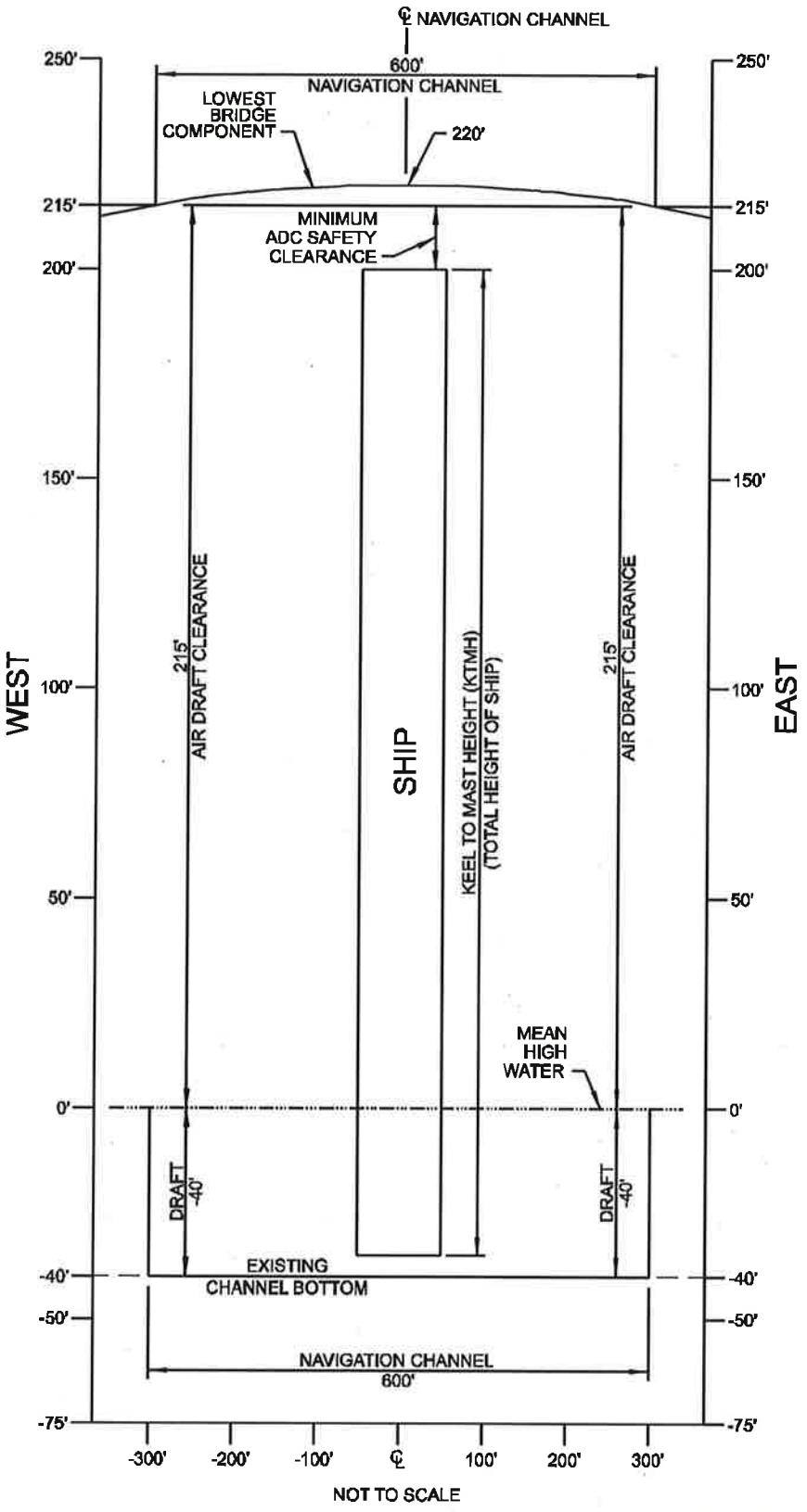


FIGURE 1

VOLKERT

Note: This map is for presentation use only and not to be used for construction purposes.

VOLKERT

N

N.T.S.

Legend

Alternate A — Alternate C

Alternate B — Alternative B Prime

Path: \\12-envir-serv\envir\PROJECTS\900\911602 Mobile River I-10 Bridge EIS\GIS\Map_Documents\Figures\Alternatives_Turning_Basin.mxd

I-10 Mobile River Bridge Alternatives and Mobile Harbor Navigation Features

Figure 2

I-10 MOBILE RIVER BRIDGE
215 FEET AIR DRAFT CLEARANCE FOR TWO CARNIVAL CRUISELINE SHIPS

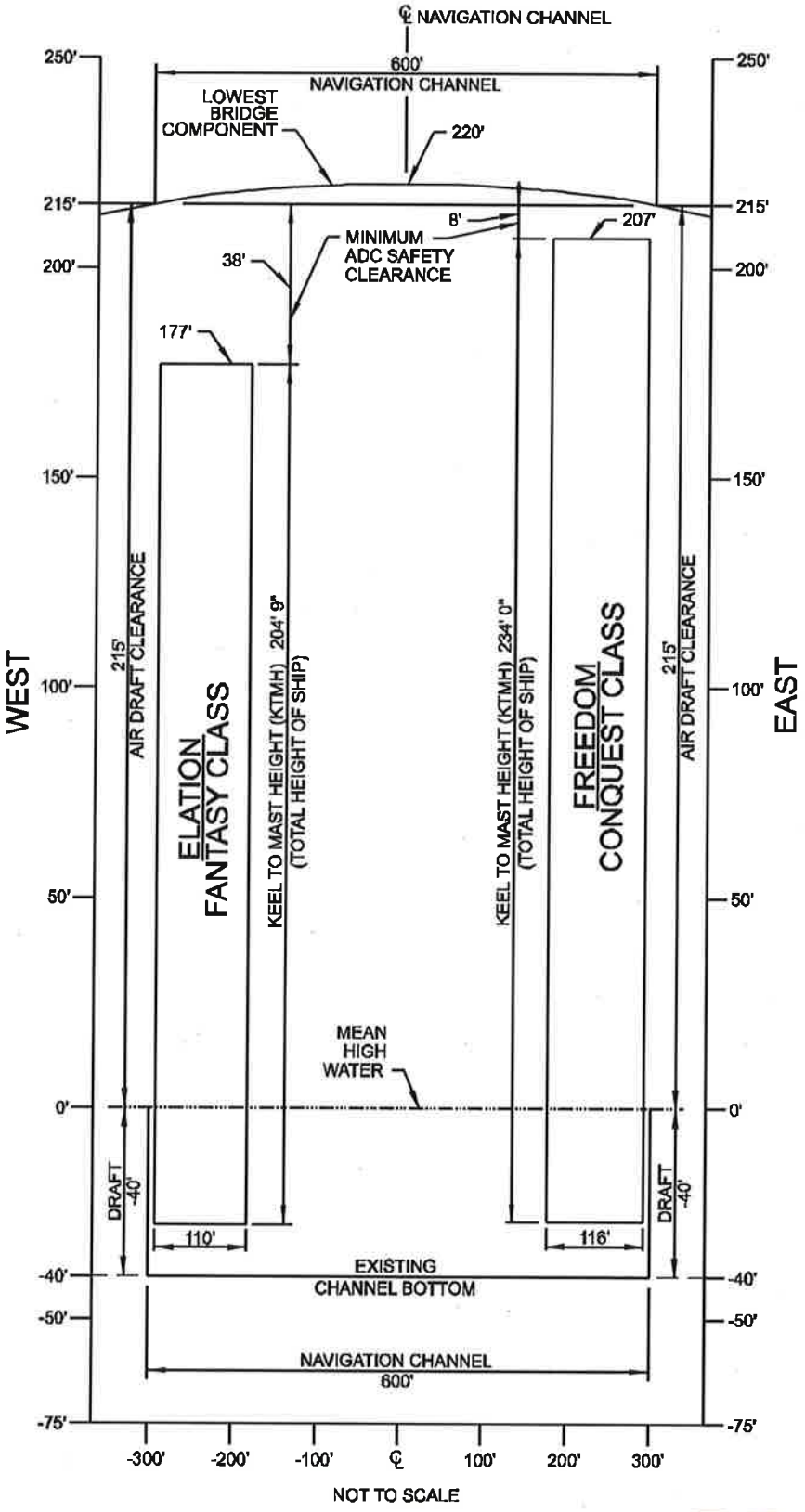


FIGURE 3

VOLKERT

I-10 MOBILE RIVER BRIDGE
215 FEET AIR DRAFT CLEARANCE FOR
12,508 TEU CONTAINER SHIP EMMA MAERSK

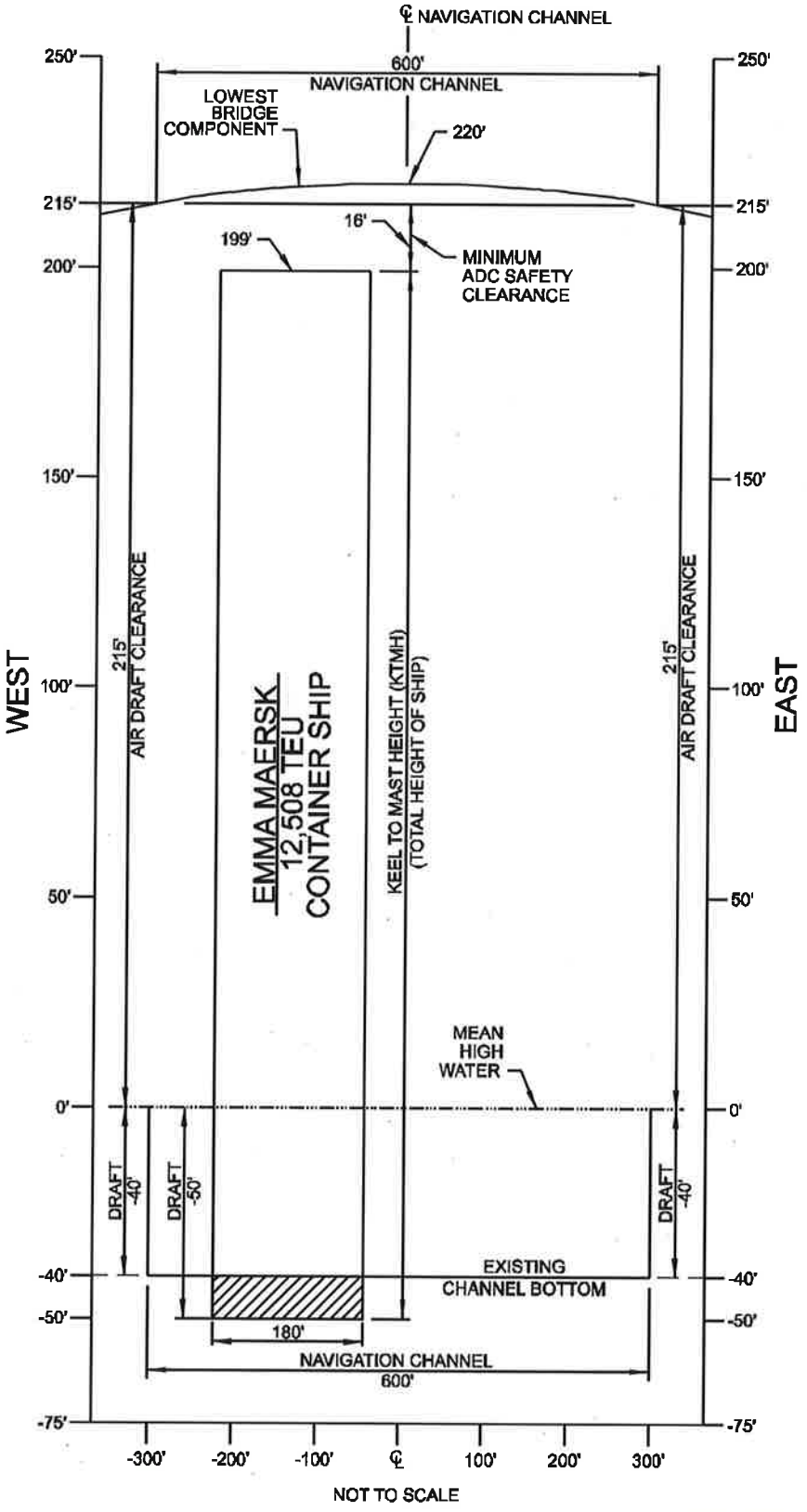


FIGURE 4

VOLKERT

I-10 MOBILE RIVER BRIDGE
DESCRIPTION OF BRIDGE, SHIP, AND NAVIGATION FEATURES
FOR AIR DRAFT CLEARANCE 190 FEET

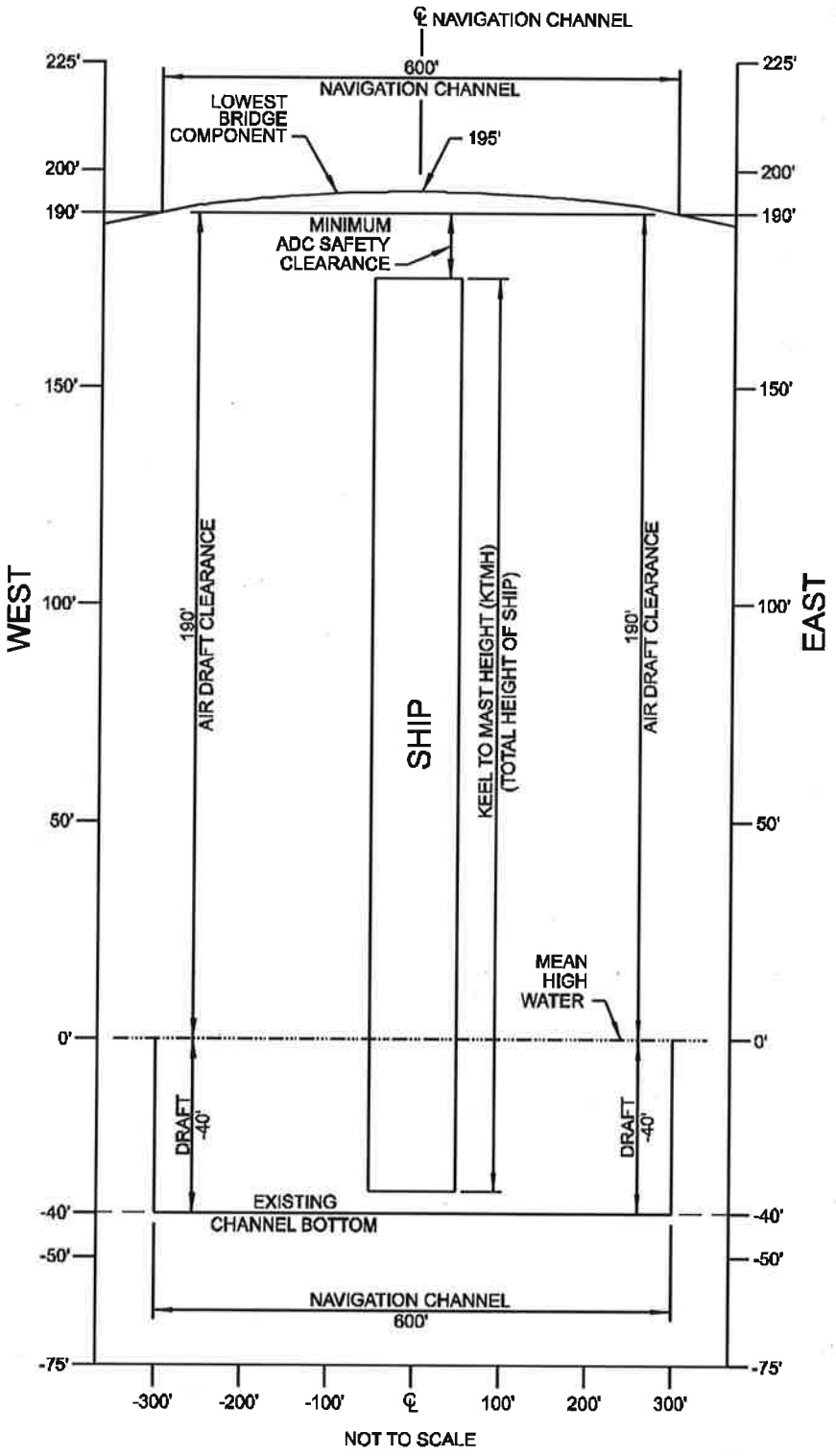


FIGURE 5

VOLKERT

I-10 MOBILE RIVER BRIDGE
190 FEET AIR DRAFT CLEARANCE FOR TWO CARNIVAL CRUISELINE SHIPS

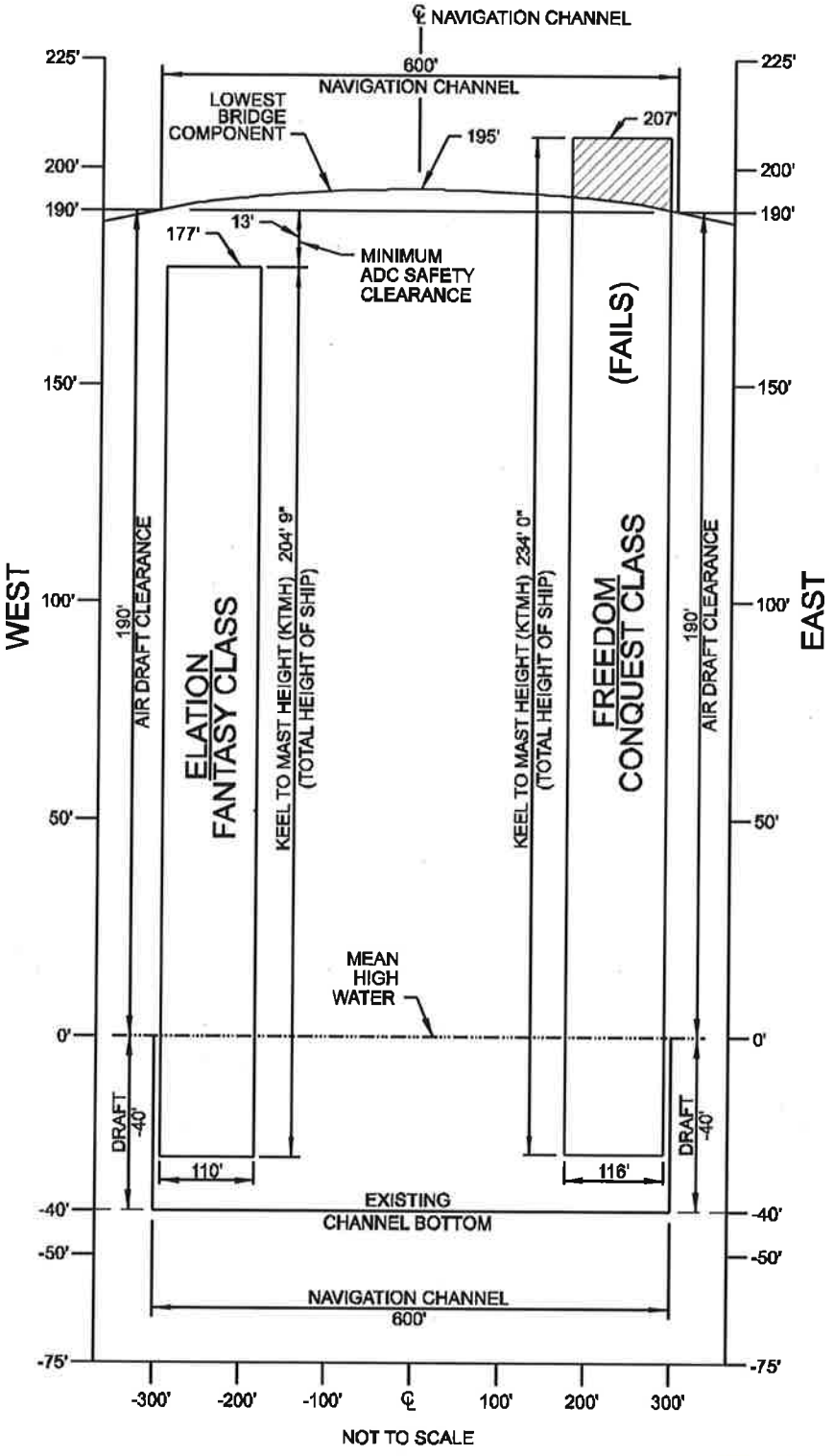
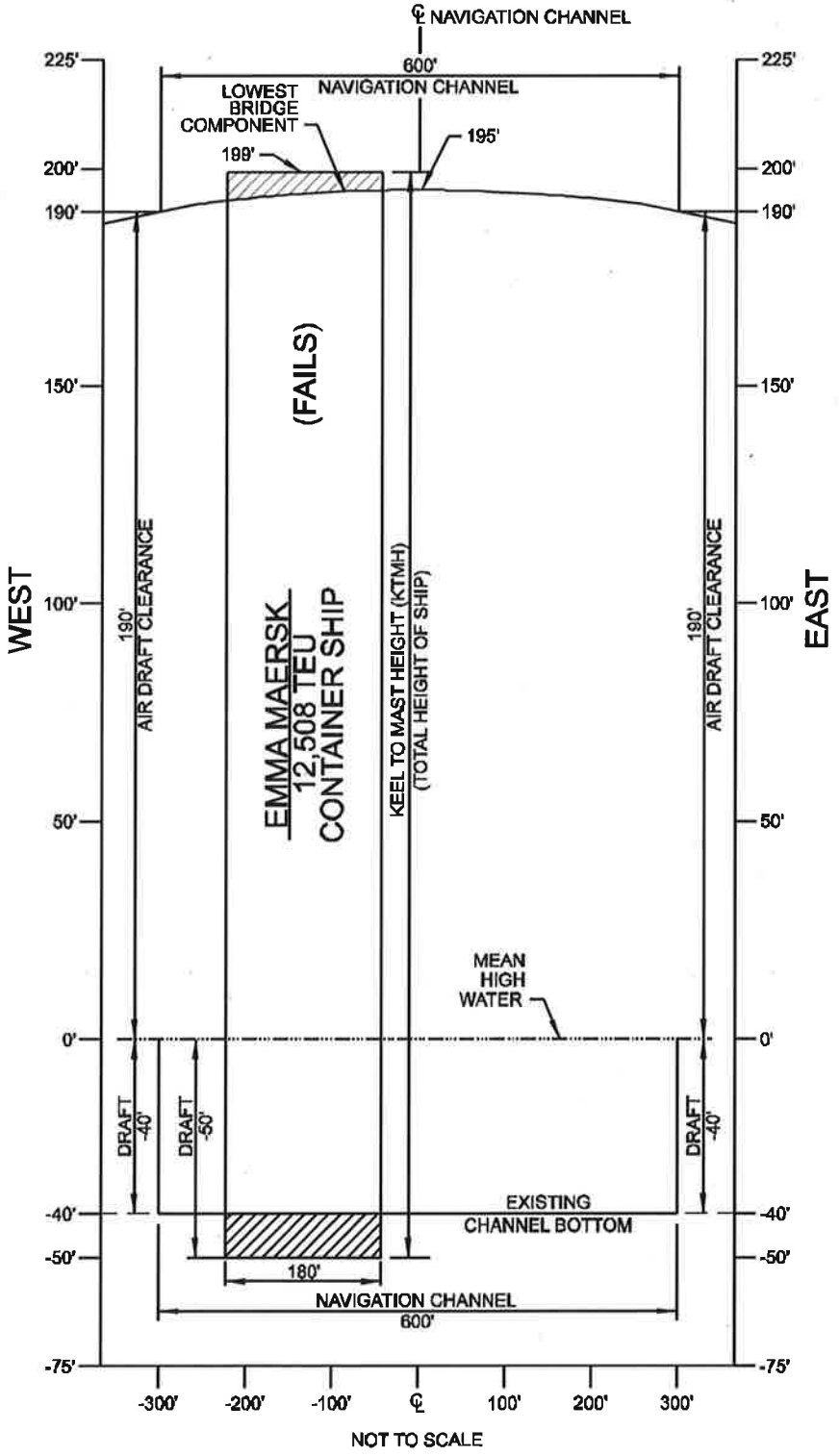


FIGURE 6

VOLKERT

I-10 MOBILE RIVER BRIDGE
190 FEET AIR DRAFT CLEARANCE FOR
12,508 TEU CONTAINER SHIP EMMA MAERSK



VOLKERT

FIGURE 7

APPENDIX A
CORRESPONDENCE

Skeeter McClure

From:	Terry Gilbreath [tgilbreath@asdd.com]
Sent:	Thursday, March 03, 2011 9:06 AM
To:	McClure, N. D. (Skeeter)
Cc:	Buddy Covington; Missi Shumer; david webber; Kenny Nichols; Smitty Thorne; Jimmy Lyons; Judith Adams; Jerald Kichler; Bob Harris
Subject:	RE: Air Draft Clearance (ADC) Evaluation for I-10 Mobile River Bridge(MRB)

Good morning Skeeter,

We presently have the CARNIVAL ELATION using the Port of Mobile on a frequent scheduled basis. The ELATION is 855 feet long, 103 feet wide and carries about 28 feet of draft. She turns most of the time in the River C turning basin which is just off our C pier and if there is congestion or adverse atmospheric conditions, she will turn in the Three Mile Creek turning basin which is just below the Cochran Bridge. The turn at Three Mile Creek turning basin would require that there be no ships at either the Plains Terminal or Vulcan Terminal. Because of the configuration of her azimuthal pod engines and bow thrusters, she does not normally require a tug, but may have one standing by if the atmospherics are adverse. I believe that turning the ship in the new Mobile turning basin would be our last resort as the ship would not want to back up or down the river. The Master of the ship, in consult with the bar pilot onboard, will have the final say, but don't believe that the ship would want to back up or down the river for a distance of over one nautical mile.

So, to answer your questions:

1. The Conquest Class could turn in the Three Mile Creek turning basin. We bring in 965 foot container ships that presently turn near Mobile Container Terminal but prior to the new turning basin would turn at Three Mile Creek turning basin. This was completed 2 -3 times a week for over a year. The turn in Three Mile Creek turning basin required close coordination with the Plains Terminal (on the west side of the turning basin) and the Vulcan Terminal (on the east side of the turning basin). If a big ship is turning at Three Mile Creek, we cannot have a ship at either of those two terminals. I would have to keep close eye on departure and arrival dates and times at those terminals and coordinate with the pilots to minimize delays to any vessel. We only had a couple of times that there was a conflict and we were able to work out equitable solutions for all involved with minimal delay.
2. It depends on atmospheric conditions and types of engines on the Conquest class, but they may not require a tug at all. If the current is really strong or excess winds, the pilot may order up a couple of tugs as standby.
3. We now turn 965 foot container ships in the new lower turning basin several times per week. I don't believe the pilots (or the Cruise Companies) would be real fond of either backing up the river or backing down the river for a little more than one nautical mile.
4. Same as answer #2, except that if the ship was backing up or down the river, they would more than likely take a couple of extra tug for control.

As for the air draft, the Alabama State Port Authority will support the higher the better. It may be a hundred years before another bridge is built crossing the Mobile River and we have no idea what vertical clearance needs will be at that point, but we would support the proposed vertical clearance of 215 feet.

Let me know if you need any additional assistance.

Best regards,

Terry D. Gilbreath
Harbor Master
Captain, U. S. Coast Guard (Retired)
Alabama State Port Authority
Office: (251) 441-7074
Cell: (251) 510-7399
http://www.asdd.com/

From: McClure, N. D. (Skeeter) [mailto:smcclure@volkert.com]
Sent: Wednesday, March 02, 2011 9:32 AM
To: Terry Gilbreath

Cc: Buddy Covington; Missi Shumer; david webber; Kenny Nichols
Subject: Air Draft Clearance (ADC) Evaluation for I-10 Mobile River Bridge(MRB)

Captain Gilbreath:

I spoke with you on 2/23/11 on the above subject. The Carnival Cruise Line currently uses a Fantasy class cruise ship in Mobile. We are evaluating the next larger class cruise ship of Carnival Cruise Lines, the Conquest class, in our ADC evaluation. The dimensions I have are: Length 953 ft, Air Draft 207 ft, Draft 27 ft. I have the following questions for you:

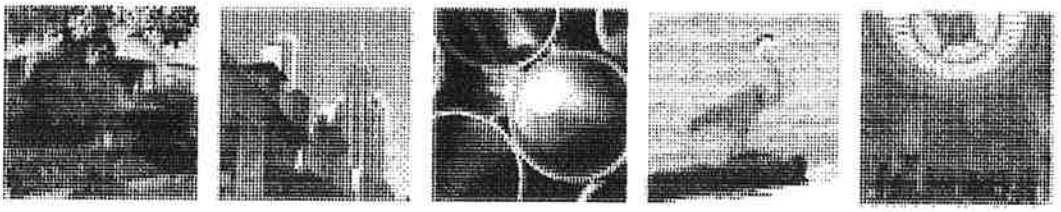
1. Can this size ship turn in the Turning Basin downstream of the Cochrane Bridge?
2. If so, are special accommodations or tug assistance required?
3. Can this size ship turn in the new Turning Basin near Pinto Island and back up to the Cruise terminal?
4. If so, are special accommodations or tug assistance required?

Also, I would appreciate any observations or recommendations you may have regarding an adequate ADC for the I-10 MRB. Please call me at 342-1070 if you want to discuss this matter. Thank you in advance for your consideration and input.

Best Regards! Skeeter

N.D. "Skeeter" McClure, IV, P.E., D.WRE
Manager
Environmental Services
Volkert, Inc.
3809 Moffett Road
Mobile, Alabama 36618
office: 251-342-1070 ext 107
cell: 251-604-3183
fax: 251-316-3854
smcclure@volkert.com

The information contained in this email, including any accompanying documents or attachments, is from Volkert, is intended only for the use of the individual or entity named above, and is privileged and confidential. If you are not the intended recipient, be aware that any disclosure, dissemination, distribution, copying or use of the contents of this message is strictly prohibited. If you have received this message in error, please notify Volkert immediately at our corporate office (251)342-1070. Thank you for your cooperation.



MOBILE AREA
CHAMBER OF COMMERCE

March 20, 2012

Mr. Vince E. Calametti PE
Alabama Department of Transportation
1701 N. Beltline Highway
Mobile, AL 36618

Re: I-10 Mobile River Bridge Height

Gentlemen:

The Cruise Industry Task Force continues to work on recruiting one or more cruise lines to Mobile. Carnival Cruise Line sailed out of Mobile as a homeport for more than 7 years, before suspending service this past fall. Since then, representatives of the City of Mobile, The Task Force, and private firms have continued a dialogue that promises opportunities with one or more cruise lines in the future.

The City's commitment to the cruise industry is strongly and continually evidenced by those efforts and the $32 million cruise terminal that sits ready and able to handle these future customers. That coupled with the successful numbers generated on past services (185,000 passengers per year) gives us great confidence in the future of cruising from Mobile.

Carnival Cruise line, for example, still remains one of our main targets for a return to Mobile. If that effort is successful, their Conquest class vessels are the most likely long term ships to serve our markets. That class ship carries an air draft of approximately 208 feet. The U.S. Coast Guard has no set figure for clearance above the ship's actual height, only that the safety of the individual situation will rule. They recommend/prefer a 15 foot clearance. That would require up to 223 feet as ideal. Discussions with them have led us to believe that 215 feet (at mean high tide) would be a safe and approved clearance.

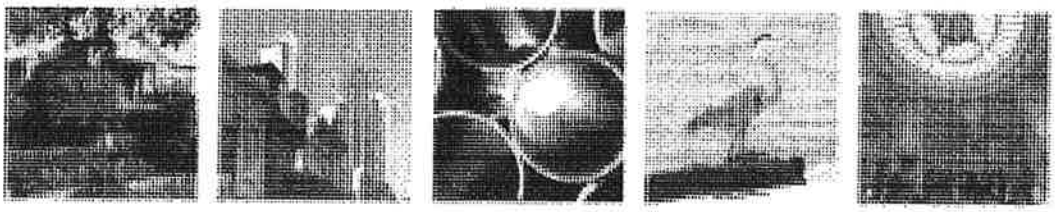
Based on the above, we believe that a minimum height of 215 feet is critical to the future of the cruise industry in Mobile. This is an industry that has worked here before, and we are confident, as market conditions evolve, will be here again.

Respectfully,

Michael Lee
Chairman
Cruise Industry Task Force

POST OFFICE BOX 2187 ▪ MOBILE, ALABAMA 36652-2187 ▪ 251.433.6951 ▪ FAX 251.432.1143 ▪ WWW.MOBILECHAMBER.COM

UNITED STATES CHAMBER OF COMMERCE ACCREDITED



MOBILE AREA
CHAMBER OF COMMERCE

April 3rd, 2012

Mr. Vince E. Calametti PE
Alabama Department of Transportation
1701 N. Beltline Highway
Mobile, AL 36618

Re: I-10 Mobile River Bridge Height

Gentlemen:

Further to my letter of March 20th regarding the need for a minimum bridge height of 215 feet, I want to advise you of the efforts going into the recruitment of another cruise line for our city and state.

The Cruise Task Force was originally formed around 2000 to recruit a major cruise line to the Port of Mobile. The task force was and is currently composed of a cross section of organizations and shareholders in the maritime, tourism, economic development industries, and public officials. The group led efforts over several years to introduce our city to the industry, and to explore who might be a good fit for our market. After over 2 years of travels and meetings with the lines in Miami, Atlanta, and Seattle, in 2002 the City was successful in attracting a trial run of eight voyages by Carnival Cruise Lines. Those trials sold out quickly and convinced Carnival to homeport a year round ship in Mobile. That ship's success led eventually to replacement with two newer, larger ships during a run that lasted from 2004 to 2011. Making this possible was the City's investment of approximately $30 million in a state of the art cruise terminal on the Mobile River (not including the approximate additional $15 million spent on the dock area supporting the terminal).

Market conditions in Mexico, coupled with the extremely high price of fuel, caused Carnival to move our ship into a different market, sailing from Port Canaveral to the Bahamas and nearby islands. Currently, those markets are commanding a higher ticket price and require about half of the fuel consumption of the Mobile Mexico runs.

Carnival was quick to advise us that, even under these conditions, they had continued to make money in Mobile, attracting over 185,000 passengers per year. Their decision was a purely business one, in that the return was better in the other market, and that they had no competition at this time in Mobile. They believe that their passengers will simply move to New Orleans on their other ships, given no option.

These factors make us optimistic that the market will return to Mobile. Other lines have seen Carnival's success, and the number of passengers that chose to cruise from Mobile. And, by their own admission, the vessel was profitable in Mobile. The Cruise Task Force is actively pursuing all of the other major cruise lines that currently work out of the Gulf. We are also talking to several who have an interest to establish new services in the Gulf region. Some of the major players are well known, Royal Caribbean, Norwegian Caribbean, Princess, and MSC for example. All compete directly with Carnival, and have watched Mobile closely during the time Carnival operated here. Some have expressed strong interest and negotiations continue. Carnival is also another possibility, returning to Mobile rather than leaving it to its competitors.

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Much is no doubt driven by market conditions as discussed above. But these are ever changing, particularly the fuel cost factor. With our city's large investment in infrastructure, the world class facility that represents, and our past history of success, we are confident that cruise ships will return to Mobile. Mobile was voted the "New Port of the Year" by the cruise industry leading publication, *Porthole Magazine.* The Mobile vessels were consistently rated 1st or 2nd in the Carnival fleet in customer satisfaction, do in no small part to convenient set up of the new Mobile terminal.

In the last few weeks and months, we have continued to meet with these large cruise lines and are very optimistic that, as more ships come into service (buildings have continued even during the downturn giving each line more ships that will need more ports of call). As I have said above, Carnival remains one of our major targets for all the reasons stated. They had already offered us a Conquest class vessel, as the next logical step above Mobile's past ships, before the downturn changed their plans. It is logical and prudent to assume that Mobile's cruise future will involve those vessels (with their 208 foot air draft). Our option to handle those ships goes away, along with most of our chances to participate in the cruise industry, if we do not have the minimum height of 215 feet.

Results may not be immediate. It could be another year or two before ships return to Mobile, if our past experience is any indication. But they will return, and will again be an important and long term part of our regional economy.

Respectfully,

Michael Lee
Chairman
Cruise Industry Task Force

ALABAMA STATE PORT AUTHORITY

April 18, 2012

Mr. Vince E. Calametti, P.E.
Alabama Department of Transportation
1701 Beltline Highway
Mobile, AL 36618

Dear Mr. Calametti,

As a follow up on a meeting that I had with Mr. Skeeter McClure and Mr. David Webber of Volkert, Inc on April 6, 2012, I am sending you this letter with additional information regarding the role of the Harbormaster in the Port of Mobile and information regarding number of vessels arriving at the Alabama State Port Authority for the last few years.

I was hired as the new Harbormaster for the Port of Mobile in August 2008 where I overlapped with the previous Harbormaster until January 2009 where I took over the position permanently. I had served previously in the U. S. Coast Guard for over 25 years having retired as a Captain. I had previously served a tour in Mobile from 1999 – 2002 as the Executive Officer of the Marine Safety Office and as such was the designated Alternate Federal Captain of the Port. I then served as the Federal Captain of the Port in Louisville, KY from July 2002 – July 2004 and then as Captain of the Port in Morgan City, LA from July 2004 until July 2007. I then served as the Budget and Personnel Director for the Eighth Coast Guard District in New Orleans, LA until my retirement in August 2008.

My role as Harbormaster for the port of Mobile is codified in the Alabama State Code Section 33-3-4

> ***Harbor Master and deputy harbor masters - Appointment; duties.***
>
> *...It shall be the duty of the Harbor Master with the assistance of his or her deputy harbor masters, subject to the supervision of the Alabama State Port Authority, to perform all the duties and render all the services imposed upon the port authority by this chapter and all similar duties and services pertaining to the harbor and port required by other laws to be done and performed by the port authority, and, subject to such supervision, to enforce all rules and regulations promulgated by the port authority pursuant to its police powers and its powers to maintain a general and special supervision over the harbor and port and all vessels and other watercraft in or about the harbor or port as set forth in this chapter or any other laws pertaining thereto...*

I work frequently with the local Coast Guard on issues of Waterways Management, emergency response, casualty investigations, Aids to Navigation and dredging. I am sure that I will be working with the local Coast Guard Waterways Division and the Coast Guard Bridge Administration in New Orleans on this bridge project as we move toward completion. I also work daily with the Mobile Maritime community from shipping agents to pilots to tug boat companies, etc. I expect to be in this job for several years and hope to be able to see the bridge project come to fruition.

One of my duties as Harbormaster is to keep track of all foreign flag vessels entering and exiting the Port of Mobile and we keep very detailed records of the same. Prior to the completion of the new turning basin near Pinto Island, we would have numerous large vessels that were doing cargo operations at McDuffie, the cruise terminal or the shipyard that would have to transit up the Mobile River and turn at the Three Mile Creek turning basin. The biggest ships that we have presently that turn at Three Mile Creek turning basin are large tank ships that are going to the Plains or Shell Terminal. Some of these ships have been 950 feet long by 158 feet wide. Since we have opened the new turning basin, the ships that are calling on McDuffie Island Coal Terminal, Pinto Island Terminal and the Mobile Container Terminal are able to turn in that location and would not be impacted by a new bridge since they would be turning below the bridge.

I have attached a table below that shows the number of ships making calls in the Port of Mobile and the number of vessels that use the ASDD docks and then other docks.

Year Total	Total Vessels in Port	Vessels at docks other than ASDD	Total vessels at ASDD
2011	1443	669	774
2010	1368	582	786
2009	1351	585	766
2008	1571	568	1003
2007	1368	443	925

The tables show that we are making steady increase in vessel usage since 2009 (which correlates to global decreases in shipping in that year). I believe that both the increases are attributable to the increases in traffic due to the new Container Terminal and also increase at our Pinto Island Terminal.

In regards to Air Draft Clearance requirements for the new bridge, the Alabama State Port Authority had stated "the new bridge should have a vertical clearance of at least 215 feet, but no less than 195 feet." This statement is still correct and I firmly believe that in the next 100 years, you will find that some ships will require the 215 feet clearance and we should build the bridge to meet needs for the foreseeable future to include the turning of larger cruise ships in the port.

I look forward to working with you in the future and would glad to assist in this important project for the Mobile area.

Sincerely,

Terry D. Gilbreath
Captain, U. S. Coast Guard (Retired)
Harbormaster
Alabama State Port Authority

CITY OF MOBILE

SAMUEL L. JONES
MAYOR

OFFICE OF THE CITY COUNCIL
COUNCIL MEMBERS

REGGIE COPELAND, SR.
PRESIDENT-DISTRICT 5
FREDRICK D. RICHARDSON, JR.
VICE PRESIDENT-DISTRICT 1
WILLIAM C. CARROLL, JR.
DISTRICT 2
JERMAINE A. BURRELL
DISTRICT 3
JOHN C. WILLIAMS
DISTRICT 4
BESS RICH
DISTRICT 6
GINA GREGORY
DISTRICT 7

CITY CLERK
LISA C. LAMBERT

March 23, 2012

Mr. John R. Cooper
Director
Alabama Department of Transportation
1409 Coliseum Drive
Montgomery, AL 36110

Re: Alabama Cruise Terminals/Proposed I-10 Mobile River Bridge

Dear Mr. Cooper:

I am writing to you regarding the impact that the height of the proposed bridge will have on the cruise industry in Mobile. Additionally, I wanted to let you know that even though we are currently without a ship, we are aggressively recruiting a replacement ship from any number of different cruise lines.

Maintaining and expanding cruise line service is a viable part of our strategic plan for Mobile. The last ship embarking from Mobile had a total passenger count per year of over 182,000. Passenger and crew spending to the local merchants totaled $13,000,000.

This single ship contributed substantially to the State economy as well. Our economic benefit studies show that total expenditures related to the cruise industry throughout the State of Alabama reached $142,000,000 in 2010. The spending by the cruise lines, their passengers and crew generated an estimated 2,380 jobs paying $87,000,000 in wage income throughout the Alabama economy.

As for whether the cruise industry has gone dormant in Mobile, the answer is an emphatic "no". We have retained a consultant in the cruise industry to actively market Mobile. Only two weeks ago, he reported contacts with three cruise lines, one of which indicated that after additional ships now under construction came into service, there would be possibility of further discussions with Mobile about repositioning one of the current ships to a market that could include Mobile. The point being that we are not taking anything for granted. We are actively recruiting another ship and we do expect that to happen.

P. O. BOX 1827 • MOBILE, ALABAMA 36633-1827

I should mention that the repositioning of ships leaving a temporary vacancy in a port is not at all unusual in the cruise industry. This has happened to other major U. S. cities. Replacing the ship is a process which may take time but in the end will result in the re-establishment of the cruise industry in Mobile.

I should also mention that we should look long term. We know that the type of ship that Mobile can currently handle requires the higher bridge elevation. We should also make you aware that with the success and growth of the facilities managed by the Alabama State Port Authority, expansion of the turning basins is inevitable and will result in the capability of larger ships, including cruise ships, using Mobile.

We would be happy to provide whatever other information you may require or answer any questions about this matter. As always, we appreciate your support of this community.

Very truly yours,

SAMUEL L. JONES
Mayor

SLJ/bb

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ALABAMA STATE PORT AUTHORITY

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DIV ENGINEER	✓		
ADMINISTRATION			
CONSTRUCTION			
COUNTY TRANS			
DISTRICT ENGRS			
EQUIPMENT			
MAINTENANCE			
MATERIALS			
PLANNING			
SPECIAL PROJ			✓

January 11, 2012

Mr. John R. Cooper
Director
Alabama Department of Transportation
1409 Coliseum Boulevard
Montgomery, Alabama 36110

Dear John:

I want to extend my appreciation for your meeting with the business community to discuss the proposed Interstate 10 Bridge over the Mobile River. As you know, traffic, safety and projected utilization demands require an additional river crossing. The Alabama State Port Authority has been proactive in the Alabama Department of Transportation's study and design process, and the Port Authority is an ardent supporter of the department in bringing this project to an expedited conclusion. I think we would all agree alleviating our ongoing issues at this Interstate 10 choke point will benefit Alabama commerce and its economic vitality.

I have enclosed, as information, a copy of the Port Authority's Board of Directors' position and action regarding the proposed I-10 bridge, as well as a letter to your predecessor outlining key reasons for an expedited bridge project. My board, my staff and I stand ready to assist your department in moving this project forward. Should you have any questions, please do not hesitate to give me a call.

Sincerely,

James K. Lyons
Director / Chief Executive Officer

Enclosure

RECEIVED
JAN 31 2012
ALDOT

RECEIVED
JAN 27 2012
ALDOT Ninth Division
Division Engineer

Alabama State Port Authority • P.O. Box 1588 • Mobile, AL 36633-1588 • www.asdd.com

RESOLUTION NO. 2008-28
OF THE BOARD OF DIRECTORS OF
THE ALABAMA STATE PORT AUTHORITY
REGARDING NEW BRIDGE OVER MOBILE HARBOR

WHEREAS, the Alabama State Port Authority (the "Authority") is an agency of the State of Alabama (the "State") with responsibility for the promotion, development, supervision, control, management and direction of the state docks and all harbors, seaports, and river ports within the State or its jurisdiction; and

WHEREAS, the operation and continued growth and development of the state docks facilities in the Port of Mobile (the "State Docks") is dependent upon the development of a new route over the Mobile River that will alleviate traffic through the Wallace Tunnel on I-10 as soon as possible (the "New Route"); and

WHEREAS, the operation and continued growth and development of the State Docks is further dependent upon the development of the New Route in a manner that will not adversely affect the operations of the State Docks or the maritime interests it serves; and

WHEREAS, consistent with the foregoing, the Director has recommended that a letter be sent in the form attached hereto as Exhibit A to all interested stakeholders, specifically including the Alabama Department of Transportation, expressing the Authority's position on the development of the New Route and urging that a final decision on such New Route be made as soon as possible so that construction of the New Route can be completed as soon as possible; and

WHEREAS, the Legal and Planning Committee of the Board of Directors has concurred in the Director's recommendation and recommended approval thereof by the Board of Directors;

NOW, THEREFORE, BE IT RESOLVED, that the Board of Directors hereby authorizes and directs the Director to send a letter to in the form attached hereto as Exhibit A to all interested stakeholders, specifically including the Alabama Department of Transportation, expressing the Authority's position on the development of the New Route and urging that a final decision on such New Route be made as soon as possible so that construction of the New Route can be completed as soon as possible and to take such action as is necessary to convey the Authority's position as expressed herein.

Done this 24th day of June, 2008 in Mobile, Alabama.

ALABAMA STATE PORT AUTHORITY
BOARD OF DIRECTORS

By _____
Tim Parker
Its Chairman

ATTEST:

By _____
Larry R. Downs
Its Secretary-Treasurer

EXHIBIT A

June 24, 2008

Mr. Duncan Joseph (Joe) McInnes
Director
Alabama Department of Transportation
1409 Coliseum Boulevard
Montgomery, Alabama 36610

Dear Joe:

For more than 10 years, Metropolitan Area Mobile has been debating possible solutions that would alleviate traffic on Interstate 10 via the Wallace Tunnel. The establishment of an additional route across the Mobile River is necessary to improve safety, eliminate congestion and expand surface infrastructure to accommodate Mobile's increased vehicular and freight movements along Interstate 10. The Alabama State Port Authority's utmost concern regarding the final placement of the proposed I-10 bridge over Mobile River is that any such placement does not impede waterborne commerce so important to the overall economy of our great state. As you are aware, the Port of Mobile statewide creates approximately 92,000 direct and indirect jobs with a tax impact of over $356,000 annually generating $10.3 billion in economic impact.

Since 2000, the Port Authority has invested hundreds of millions of dollars in seaport infrastructure to serve Alabama's growing economy. The $300 million container terminal at Choctaw Point is the seaport's single largest investment since the state docks inception in 1928. The new container port will provide Alabama manufacturers and shippers with infrastructure and services essential to containerized shipping. Containerized shipping is vital to logistics and supply chain networks utilized by Alabama's automotive, aviation, durable goods, refrigerated and frozen foods, retail, and distribution business sectors. The new terminal will open this September and will be accompanied by significant increases in freight movements on both Interstate 10 and Interstate 65.

Today, the Port Authority's Board of Directors approved a Resolution urging the Alabama Department of Transportation to identify a new route over the Mobile River to alleviate traffic through Interstate 10 Wallace Tunnel. A copy of that Resolution has been enclosed for your review. We believe the best and most effective compromise on the placement of a bridge over the Mobile River would be for the Department of Transportation to take the following steps:

a. Eliminate the proposed "Route C" from the project plans;
b. Expeditiously review current data on the proposed northern route's capability to meet the project's stated purpose and need to alleviate congestion through the I-10 tunnel;
c. Establish a new route that shall pass no farther north than "Route A and no farther south than "Route B" in the event that the suggested Northern Route fails to meet the purpose and need to alleviate congestion through the I-10 Tunnel.
d. In the event a new bridge is to be constructed over Mobile River, then the new bridge should have a vertical clearance of at least 215 feet, but no less than 195 feet.
e. In the event a new bridge is to be constructed over Mobile River, then the grade change on the approach should begin at least 300 ft. northward of the I-10 Virginia Street Exit.

The Port Authority's Board of Director's further encourages the Alabama Department of Transportation to expedite all necessary study and review, so that a new Interstate 10 Bridge route can be constructed.

The efficient movement of both goods and services can only serve Alabama's economic interests. The public seaport facilities, along with expanded and improved interstate and highway infrastructure, are essential to sustainable growth. We stand ready to support the Alabama Department of Transportation in its endeavor to resolve the congestion and safety issues at the Wallace Tunnel.

Sincerely,

James K. Lyons
Director & CEO

ALABAMA DEPARTMENT OF TRANSPORTATION

1409 Coliseum Boulevard, Montgomery, Alabama 36130-3050

Bob Riley
Governor

Joe McInnes
Transportation Director

June 24, 2003

Bridge Administration
Eighth Coast Guard District
Hale Hoggs Federal Building
500 Camp Street
New Orleans, LA 70130

Attention: Mr. David Frank

Re: Alabama Department of Transportation
Project: DPI-0030 (005), I-10 Mobile River Bridge
and Bayway Widening
Mobile and Baldwin Counties

Dear Sir:

Enclosed for your information is 3 copies of the Environmental Assessment for the subject project. This document is being forwarded for your information.

Sincerely,

Don T. Arkle, Chief
Design Bureau

By: John Shill for:
Alfedo Acoff, Coordinator
Environmental Technical Section

AA/JS/jn

Enclosures

cc: Mr. Ronnie Poiroux
Mr. Don Arkle
Mr.John Shill
ETS File

VOLKERT
& ASSOCIATES, INC.

www.volkert.com

3809 Moffett Road (36618)
P.O. Box 7434
Mobile, Alabama 36670-0434
251.342.1070
Fax 251.342.7962
volkert@volkert.com

May 22, 2008

Mr. David Frank
Bridge Administrator
Eighth Coast Guard District
500 Poydras Street
Room 1313
New Orleans, Louisiana 70130-3310

Subject: Contract ID No. #205
Supplemental Agreement #3
Project Nos. DPI-0030(005)
I-10 Mobile River Bridge EIS
Mobile and Baldwin Counties
Volkert Project No. 911602.12

Dear Mr. Frank:

As you are aware, an Environmental Impact Statement (EIS) is currently being prepared for the proposed I-10 Mobile River Bridge and Bayway Widening EIS. At this time, three build alternatives are under consideration, one of which was evaluated in the approved Environmental Assessment (EA) for the subject project prior to its elevation to an EIS. The U.S. Coast Guard served as a Cooperating Agency on the EA for the proposed project and has agreed to continue to serve as a Cooperating Agency on the EIS (see enclosed letter dated October 5, 2005). Enclosed is a package of information regarding coordination with the U.S. Coast Guard to date, as well as some additional information regarding potential clearances and proposed pier and pylon locations for the proposed bridge.

We will continue to coordinate with the U.S. Coast Guard regarding clearance requirements, proposed pier and pylon locations, and other considerations related to the proposed I-10 Mobile River Bridge as the corridor study and EIS develops. We appreciate your interest in this project and look forward to receiving any comments on the information provided. Should you have any questions or comments, please contact Mr. Skeeter McClure or Mr. David Webber at (251) 342-1070.

Sincerely yours,
VOLKERT & ASSOCIATES, INC.

Buddy Covington
Environmental Manager

Office Locations:
Birmingham, Foley, Huntsville, Mobile, Alabama • Gainesville, Orlando, Pensacola, Tampa, Florida • Atlanta, Georgia • Collinsville, Illinois
Baton Rouge, Louisiana • D'Iberville, Mississippi • Raleigh, North Carolina • Chattanooga, Tennessee • Alexandria, Virginia • Washington, D.C.

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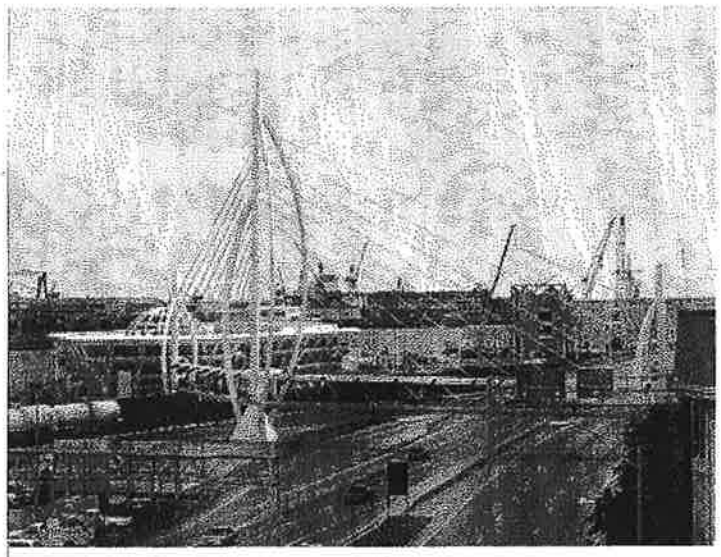
Everything Alabama

Mobile Mayor Sam Jones, others headed to Miami to court cruise line

Published: Tuesday, July 19, 2011, 6:40 AM Updated: Tuesday, July 19, 2011, 6:40 AM

By **Robert McClendon, Press-Register**



This rendering shows GulfQuest, an interactive maritime museum under construction on the Mobile River waterfront at the foot of Government Street. Mobile Mayor Sam Jones is joining a group heading to Miami to try to recruit a new cruise line for the city. The group plans to tout the GulfQuest maritime museum, which they see as a vital part of their marketing arsenal. (Courtesy GulfQuest.org)

MOBILE, Alabama -- Mobile Mayor **Sam Jones** and a small contingent of tourism industry representatives will head to Miami Thursday to court a new cruise line for the city.

Jones declined through a spokeswoman to specify which line he and the others were targeting, citing "the competitive nature of the industry."

The Miami area serves as home to nearly all of the significant cruise companies. They include Carnival Cruise Lines, Royal Caribbean International and Norwegian Cruise Line.

Carnival, which currently runs the Elation from the Alabama Cruise Terminal,**announced earlier this year** that it would pull out of Mobile in October.

Joining Jones on the recruiting expedition will be David Randel, acting president of the **Mobile Bay Convention and Visitors Bureau**; Sheila Gurganus, manager of the Alabama Cruise Terminal; and Tony Davis, chief financial officer of PCH Hotels and Resorts, which owns two major downtown hotels.

Jones has said that the group hopes to tout the GulfQuest maritime museum, which they see as a vital part of their marketing arsenal.

Randel described the $52 million museum as "the anchor" of the group's pitch.

A-17

This morning, the City Council will consider whether to issue $3 million in new debt to complete funding for the museum construction, under way on the west bank of the Mobile River next to the Cruise Terminal.

A cruise industry insider contacted Monday by the Press-Register said that Mobile's effort to leverage the museum into a new cruise ship sounded like "a long shot."

"In today's market, there is one thing that cruise lines are focused on and that's cost, cost, cost," said Oivind Mathisen, editor and publisher of **Cruise Industry News**, a trade publication.

Cruise lines take into account the quality of a port city's tourism offerings, he said, but only in so far as they will help fill ships at a high ticket price.

'Every little bit helps'

When Carnival announced its intention to leave Mobile, it blamed the high fuel costs of sailing out of the northern Gulf of Mexico and an inability to raise ticket prices. The company said the decision had nothing to do with Mobile's appeal as a tourist destination.

Mathisen said that Norfolk, Va., is the only other American cruise city that he knows of that paired its terminal with an adjacent museum.

The Mobile museum is surely an asset, he said, but it's hard to say if it will actually contribute to the city's viability as a cruising port.

Robert Jumonville, director of the Port of New Orleans, offered a different perspective, however. The appeal of his city as a tourist destination has undoubtedly played a role in its success as cruising port, he said.

Currently host to two cruise ships, the port is expecting to add two more ships this fall, one of which is the Elation.

carnival-elation.JPG

The Carnival Elation sits in dock in 2009. (Press-Register/Mike Kittrell)

"Every little bit helps," Jumonville said. "If you think it's an attraction to the U.S. traveling public, it will help."

Mobile borrowed $21.2 million to buy the cruise terminal and build a new gangway. Unless the city finds another ship, it will face a $2.25 million hole in its budget, a combination of lost parking revenues and continued debt service on the city-owned facility.

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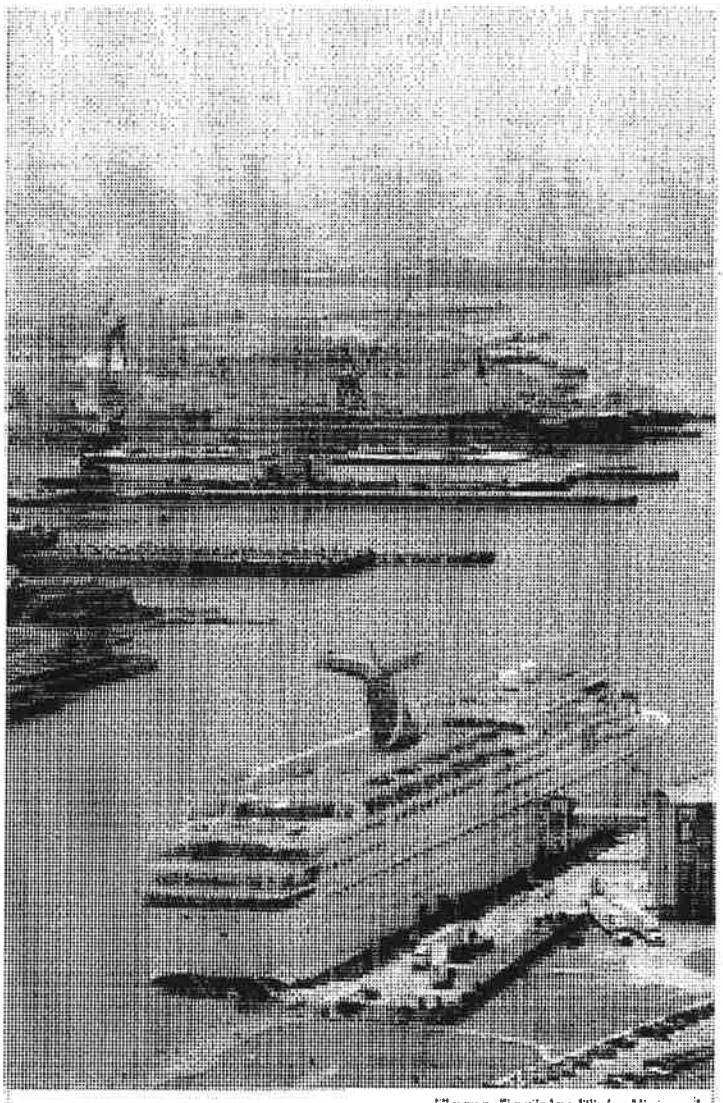
Everything Alabama

Mayor: No deal for cruise line in Mobile

Published: Monday, October 03, 2011, 6:11 PM Updated: Monday, October 03, 2011, 7:02 PM

By **Dan Murtaugh, Press-Register**

MOBILE, Alabama -- Mayor Sam Jones just shot down a news report claiming that the city of Mobile has reached an agreement with Norwegian Cruise Lines to begin sailing out of the **Alabama Cruise Terminal** later this month.



(Press-Register/Beth Hansen)

The Alabama Cruise Terminal as seen on May 16, 2009.

"It is not true," Jones said. "I wish it was, but it's not."

Carnival Cruise Lines, the only tenant at the terminal, has announced it is **leaving** the Mobile market later this month.

Jones said the city is not even in negotiations with a cruise line. Jones made presentations to Norwegian and other cruise lines earlier this year in Miami. The next step would be for one of those lines to ask the city to submit a formal proposal that would be the starting point for negotiations, but that hasn't happened yet, Jones said.

"I'm optimistic they will, but as of today we have not heard from any of the folks we sent presentations out to," Jones said.

WPMI-TV15 published a **story** this afternoon, citing a restaurant owner and an unnamed source "inside city hall" who both said an agreement had been reached between the city and Norwegian.

Carnival began service from Mobile in 2004 with the 1,452-passenger Holiday. That ship, which was the smallest in Carnival's fleet, was consistently booked above the listed capacity.

Last year Carnival brought in the larger Fantasy, with a listed capacity of 2,052 passengers. In May, the company replaced the Fantasy with the Elation, which is the same size. The Elation is newer than the Fantasy but does not have all of the old ship's upgrades.

In March, Carnival announced that the Elation will move to New Orleans Oct. 22. There, it replaces a sister ship, the Ecstasy, that Carnival will move to Port Canaveral on Florida's Atlantic Coast.

City officials have been scrambling to find another cruise line to take Carnival's place, because parking revenue from cruise passengers pays down the $25.9 million in debt remaining on the city-owned cruise terminal.

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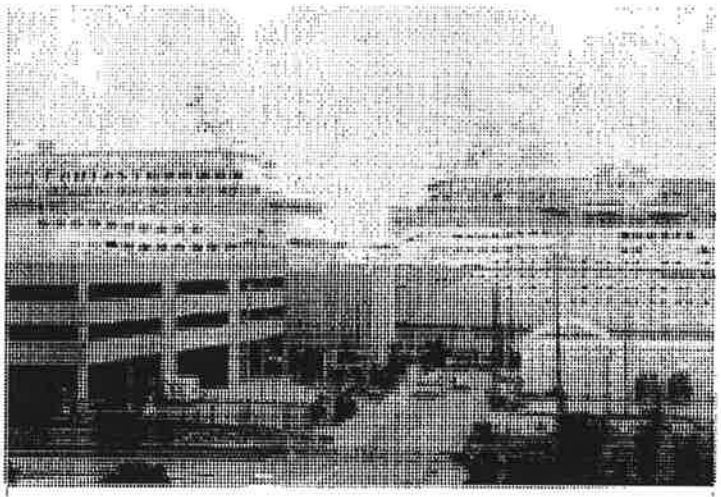
Everything Alabama

Cruise ship leaves behind unknown economic impact in its wake

Published: Saturday, October 22, 2011, 8:55 AM Updated: Saturday, October 22, 2011, 2:14 PM

By **Dan Murtaugh, Press-Register**

MOBILE, Alabama -- When Carnival's Elation leaves the Alabama Cruise Terminal today, on the way to its new home in New Orleans, it will leave in its wake an economic impact in the millions of dollars.



(Joshua Dahl/Correspondent)

The Carnival Elation passes the Carnival Fantasy as the Elation arrives at its new port, the Alabama Cruise Terminal, in Mobile, AL on Saturday, May 15, 2010.

The most obvious victim is the city of Mobile. The city owns the cruise terminal and uses proceeds from passenger parking to pay $2 million a year in debt service.

Hotels will also feel an immediate impact. Cruise passengers booked 15,000 room nights a year, according to tourism officials.

Some passengers and crew members also ate at restaurants, shopped at stores and visited attractions in Mobile, but how much money they spent is harder to know.

Elation will head to New Orleans, where it replaces a sister ship, Ecstasy, that Carnival will move to Port Canaveral on Florida's Atlantic Coast.

Carnival announced in March that it would leave the Mobile market, saying fuel costs for Gulf of Mexico cruises were more expensive than for Caribbean cruises and that trips from Mobile commanded below-average rates compared to other Gulf ports.

Alabama Real Estate Holdings, an arm of the Retirement Systems of Alabama, borrowed $20 million to build the cruise terminal south of downtown Mobile in 2004 after Carnival announced it would home port a ship here.

In 2008, the city borrowed $18.6 million to buy the terminal, then the next year paid $2.6 million to build a

new gangway for the terminal to allow a larger ship to dock there. Last year it paid $970,000 for a pie-shaped parcel of land, about the size of a football field, at the corner of Government and Water streets for overflow parking from cruises.

City spokeswoman Barbara Drummond said the city will slash operating costs at the terminal when the cruise ship leaves, only spending as much as necessary to maintain the facility.

Mobile Mayor Sam Jones has lobbied other cruise companies to use the terminal, but so far has had no success. Drummond said that until another cruise ship comes to Mobile, the city may try to lease parking space to downtown businesses or use it for big events like Mardi Gras.

Parking revenue from the past year will cover the debt payment this fiscal year, she said. But if the city can't find new revenue streams from the terminal before next October, she said, it will have to dip into its general fund to pay off the building.

"It's 2013 when we'll be impacted financially," she said.

The impact will be more sudden for area hotels. David Randel, president of the Mobile Bay Convention & Visitors Bureau, said 15 hotels offer a stay-and-cruise package that lets customers leave their cars at the hotel during the cruise provided they stay the night before. The package gets about 15,000 takers a year, he said, and it was gaining in popularity.

"That's what personally hurts me the most," he said. "People were coming in a day early and starting to enjoy this great city."

Randel said many of the 900 crew members on board the ship also ventured into the city on the five-hour breaks they had between trips. Most of them took a bus out to Bel Air Mall, he said.

"They'd come back filled with packages," he said.

David Rasp, who owns the Royal Scam, the restaurant closest to the Cruise Terminal, said cruise customers had only a modest impact on his business.

"I'm not saying it's not going to have an effect," he said of the cruise ship departing, "I just don't know how measurable it's going to be."

The impact on other local attractions is similarly murky. According to data from University of South Alabama economics professor Semoon Chang, attendance is up by nearly 3,000 a month at the Gulf Coast Exploreum since Carnival arrived in October 2004, but it has dropped at both the USS Alabama Battleship Park and at Bellingrath Gardens.

Gretchen Faust Jaspering, who took over as executive director at the Exploreum earlier this month, said she

hadn't been there long enough to have a definitive view on the cruise ship's impact.

She did note, though, that when the Exploreum opened its Kangaroo Kraze exhibit last week, a few of the cruise ship employees decided to check it out.

"A couple of Australian dancers came over to get a taste of their home," she said.

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Staten, Liz

From: Copper, Adam <acopper@springsgov.com>
Sent: Wednesday, June 7, 2017 3:12 PM
To: Staten, Liz
Cc: Kelley, Brian
Subject: ATBP - Scope and Fee

Liz,

I was hoping you would be able to meet and go over a project and provide a scope and fee. The project is retrofitting a pond in ATBP to be full spectrum and adding an additional pond on the north end of the park. Separate second scope and fee will be providing underground detention on a lot just north of the USOM off Sierra Madre.

Let me know what time would work for you. We want to move quickly on the ATBP, so let me know if you have the availability.

Thanks,

Adam Copper, P.E.
Senior Engineer
City of Colorado Springs – Public Works
Capital Improvements Program Team
Direct (719) 385-5436
acopper@springsgov.com

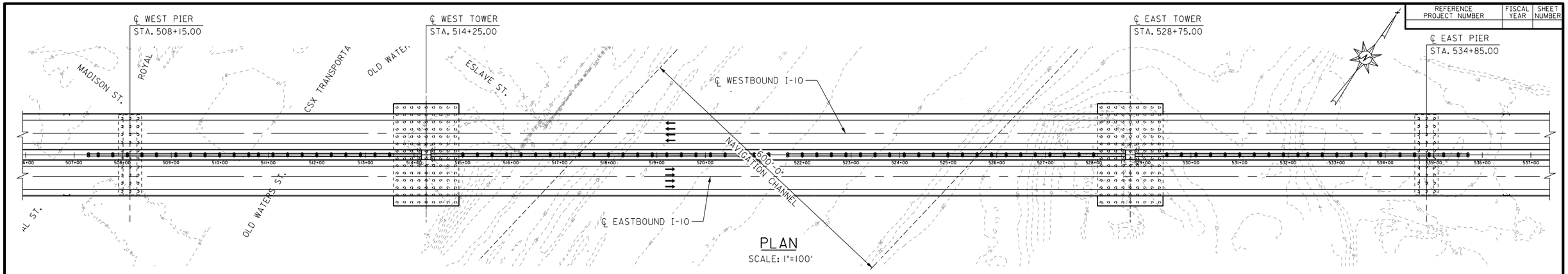
COLORADO SPRINGS
OLYMPIC CITY USA

Appendix D:

Step 1:

Bridge Layouts

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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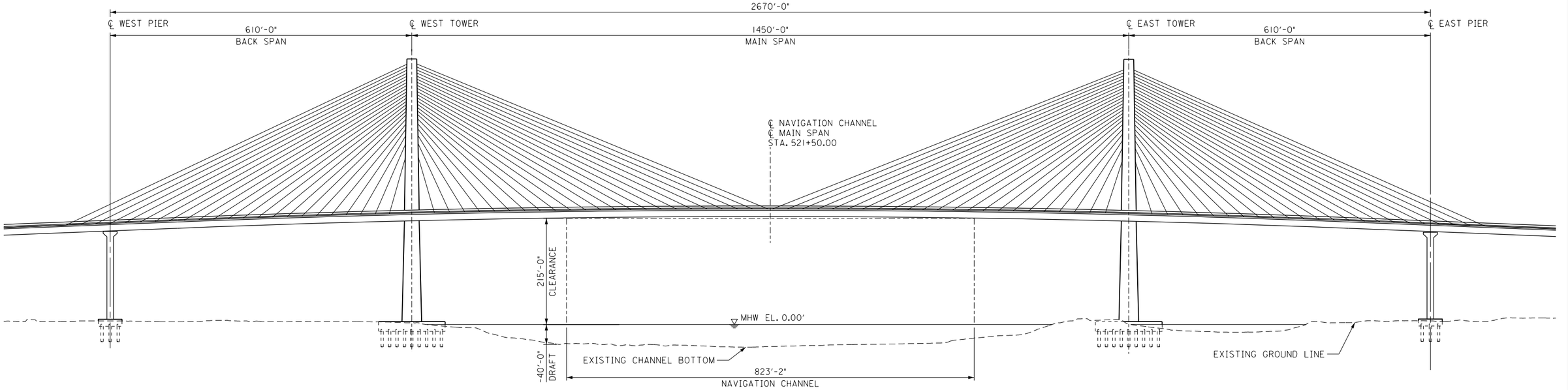

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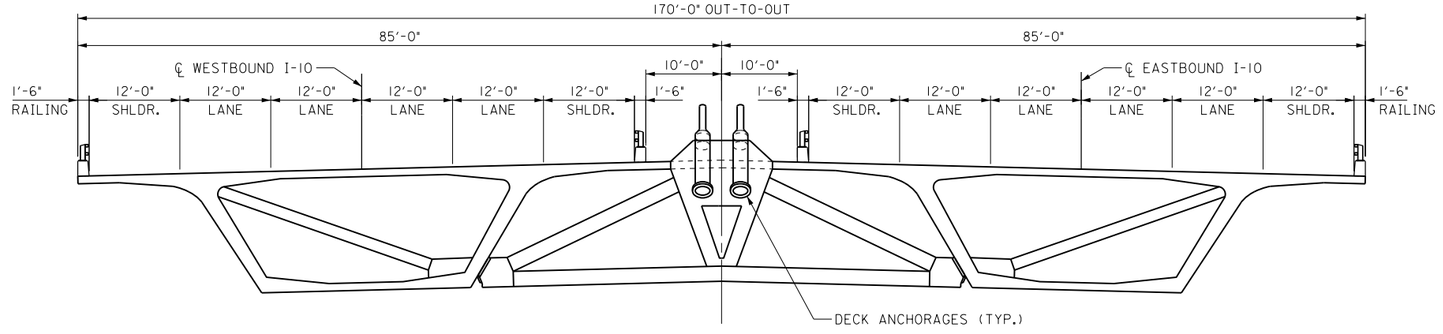

TYPICAL SECTION AT DECK ANCHORAGES
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NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE SEGMENTAL

ALTERNATIVE 1 (1 OF 2)

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VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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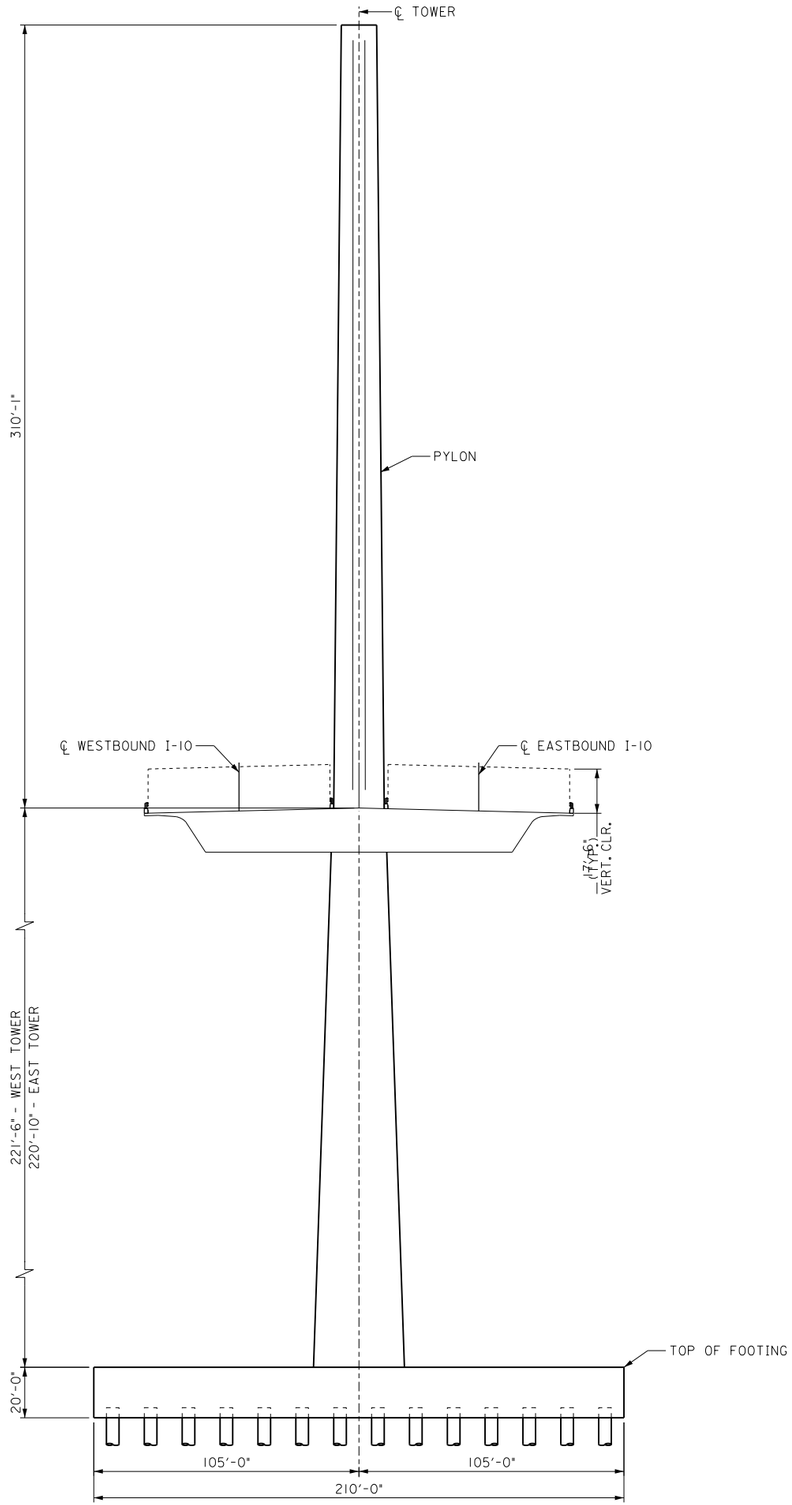

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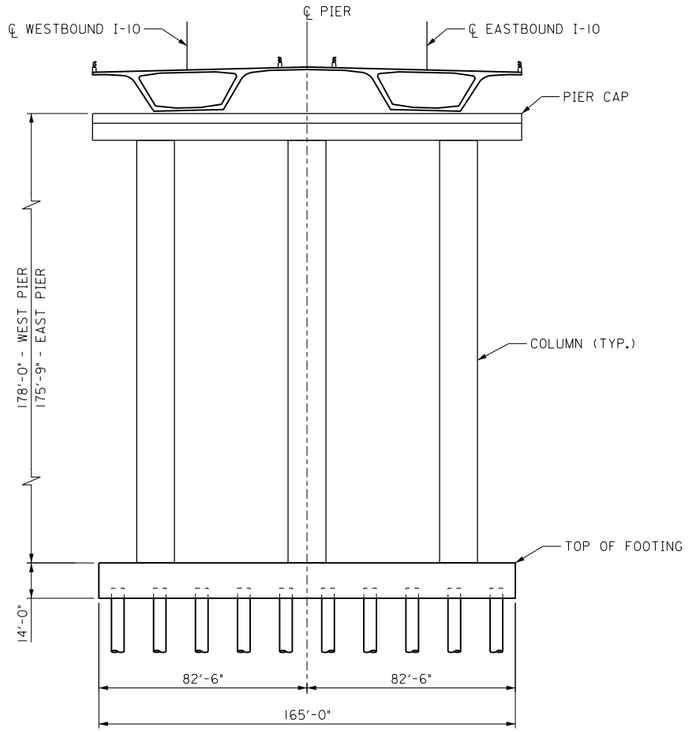

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REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

ALTERNATIVE 1 (2 OF 2)

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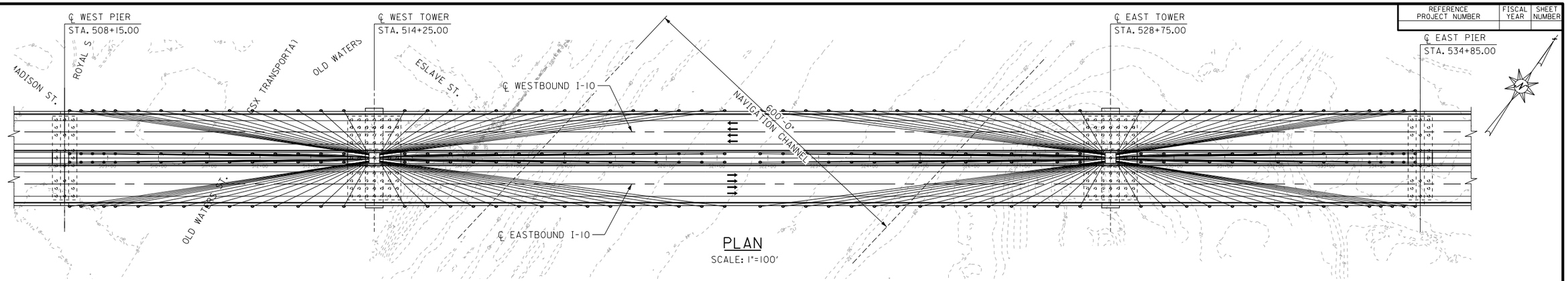

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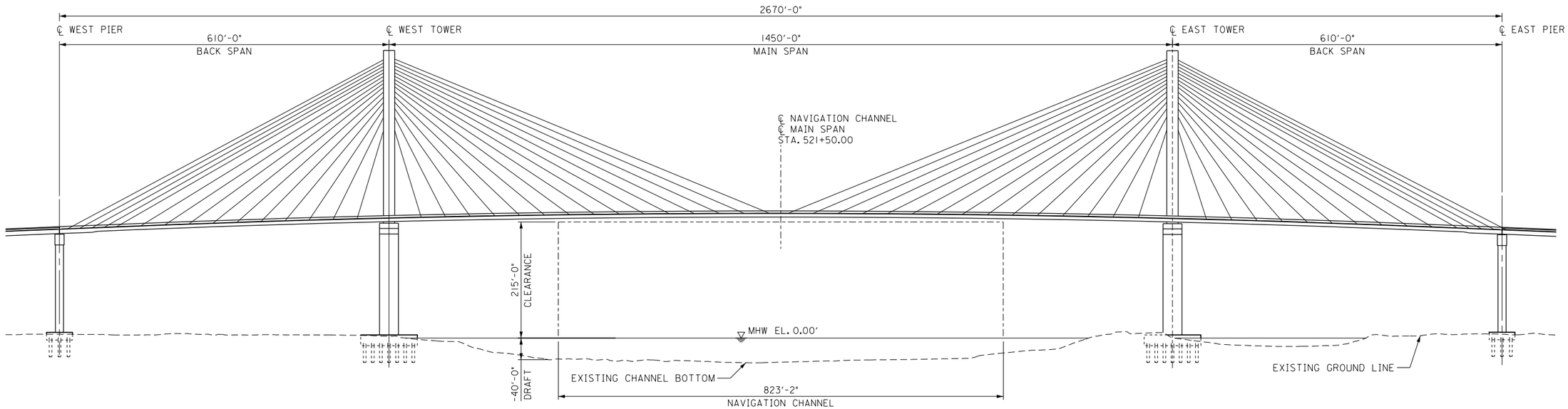

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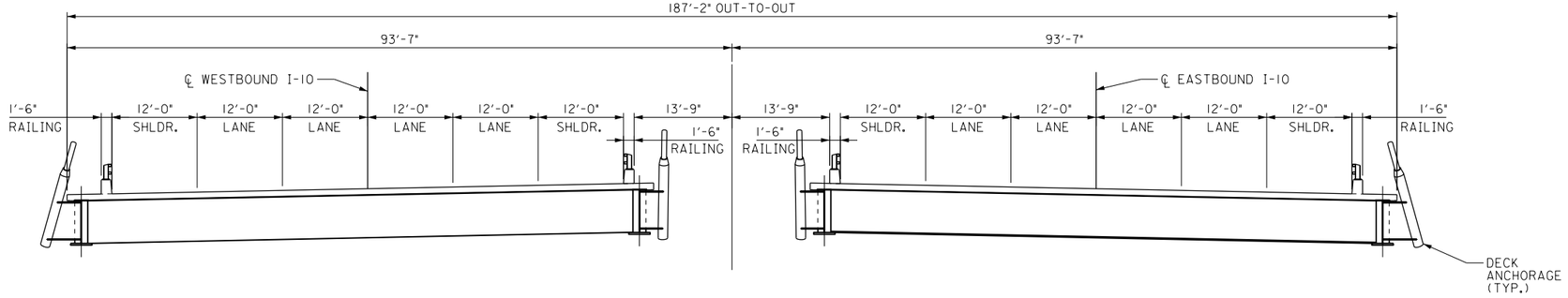

TYPICAL SECTION AT DECK ANCHORAGES
SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - SINGLE
PYLON - STEEL

ALTERNATIVE 2 (1 OF 2)

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VERIFIED BY:	DATE CHECKED:	SCALE:

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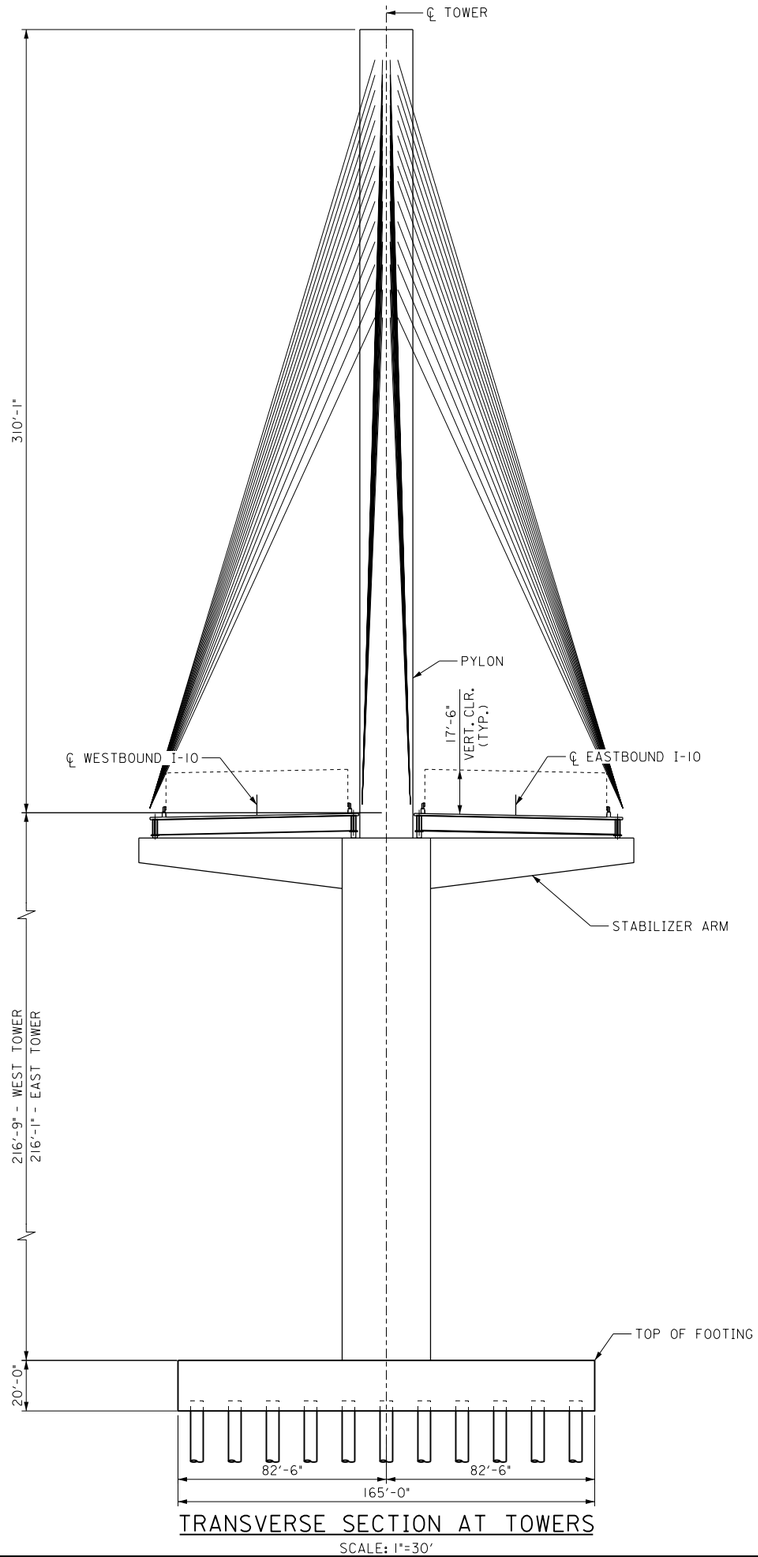

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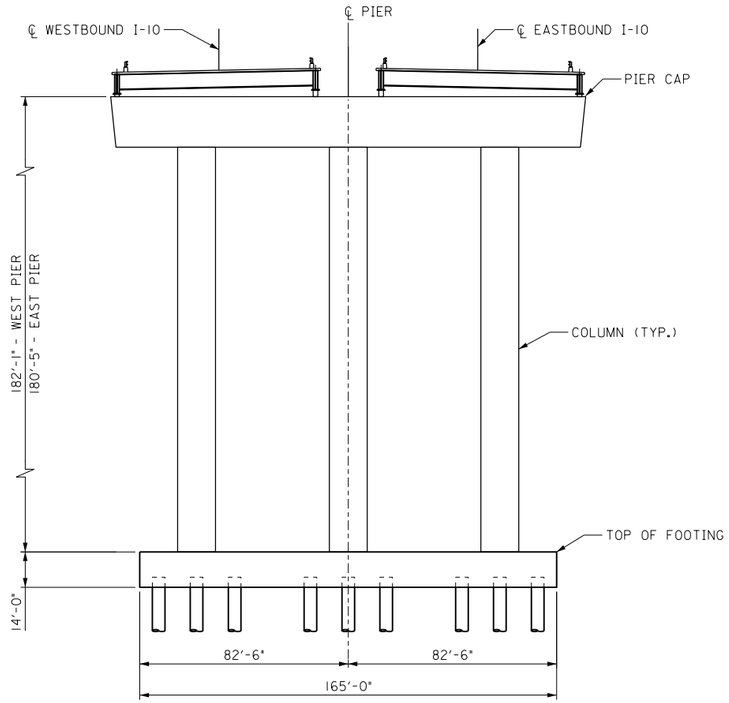

TRANSVERSE SECTION AT PIERS

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NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - SINGLE
PYLON - STEEL

ALTERNATIVE 2 (2 OF 2)

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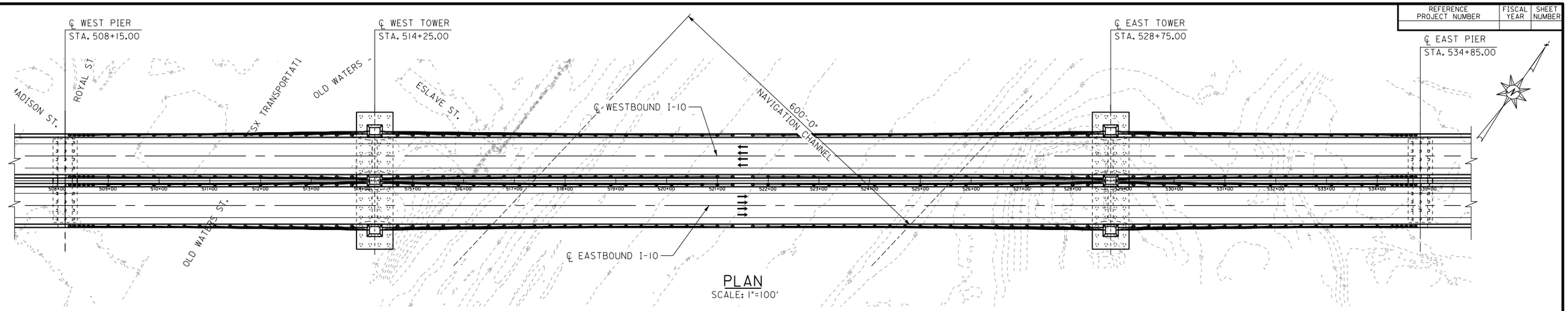

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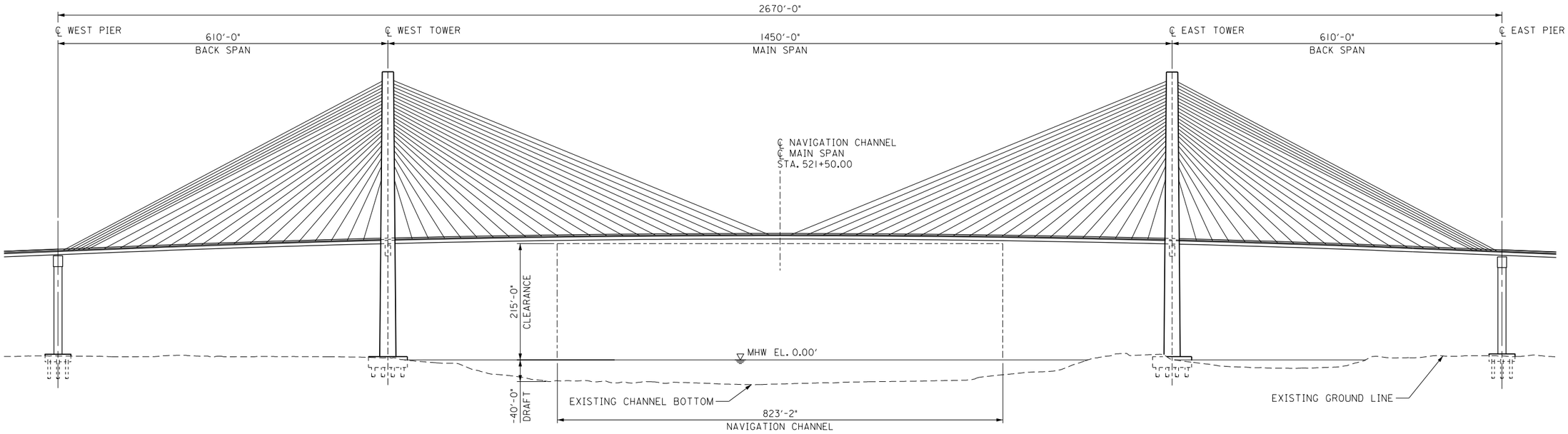

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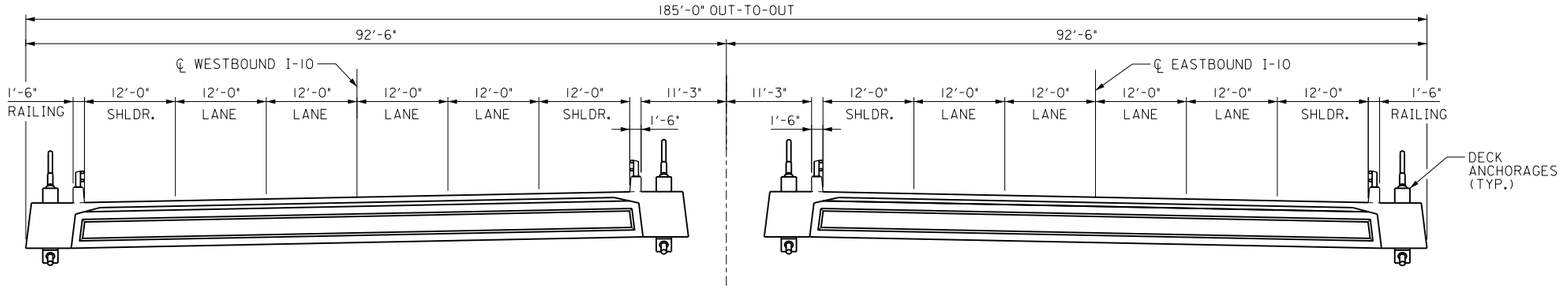

TYPICAL SECTION AT DECK ANCHORAGES
SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - TRIPLE PYLON - CONCRETE
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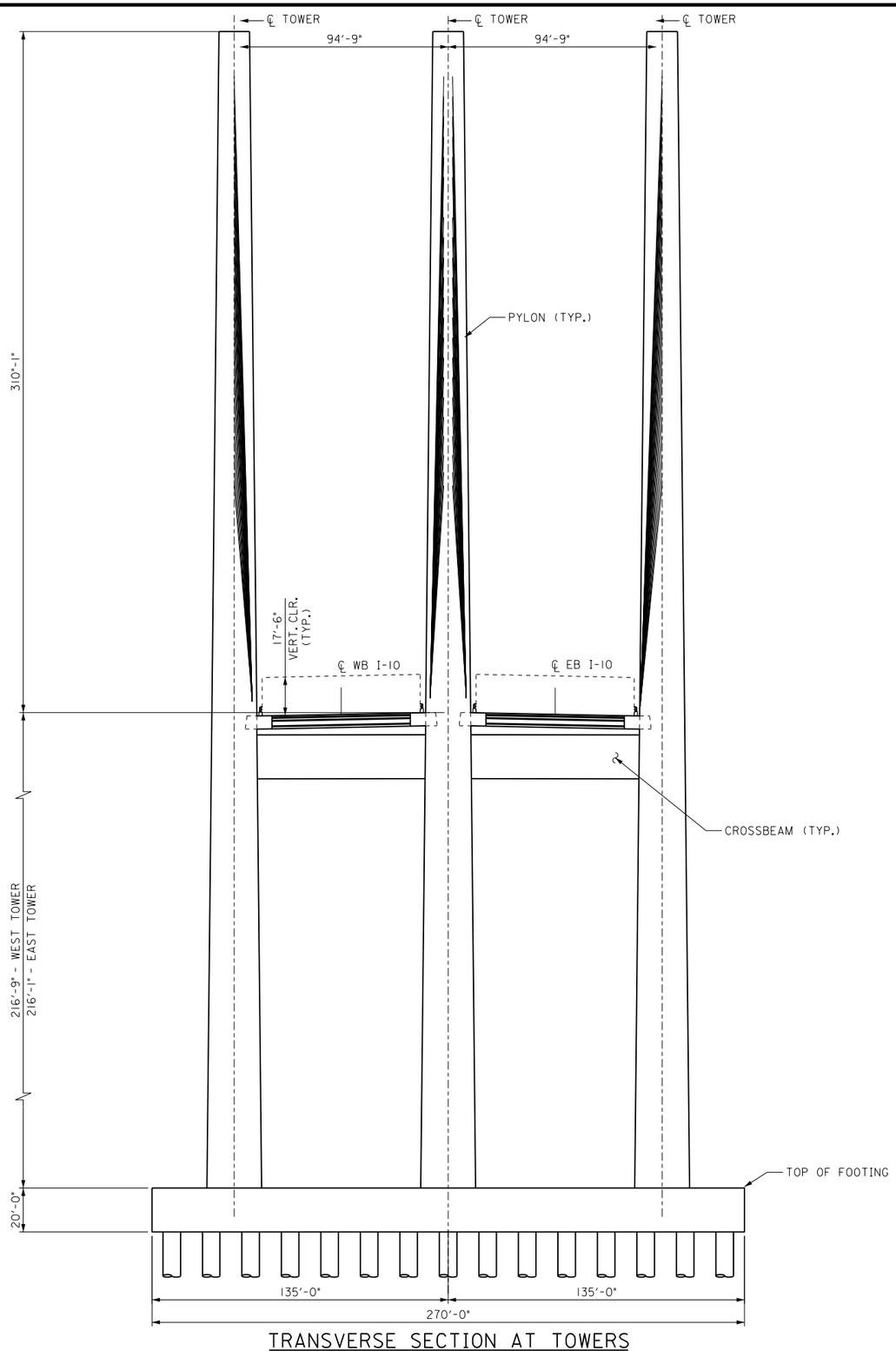

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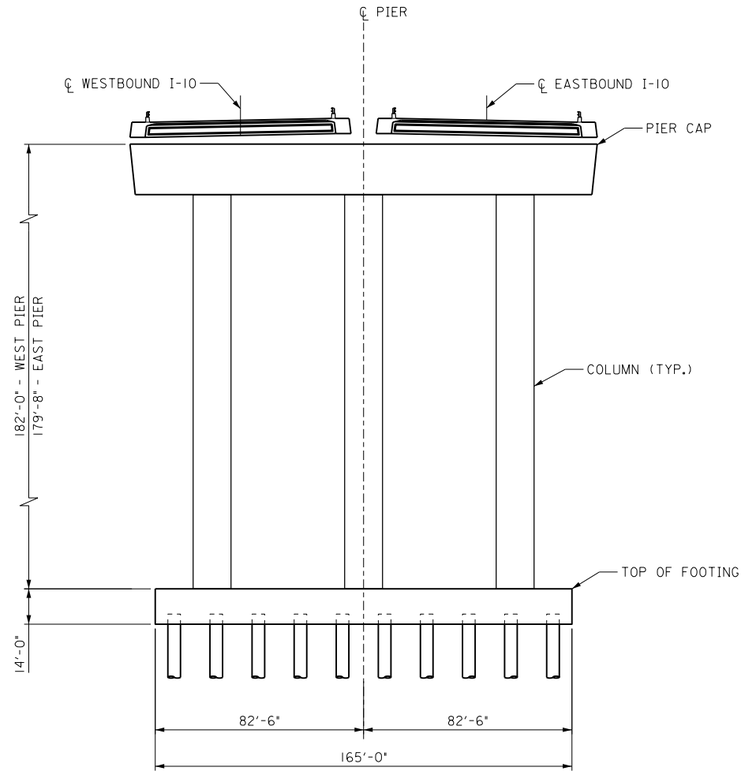

TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - TRIPLE
PYLON - CONCRETE

ALTERNATE 3 (2 OF 2)

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

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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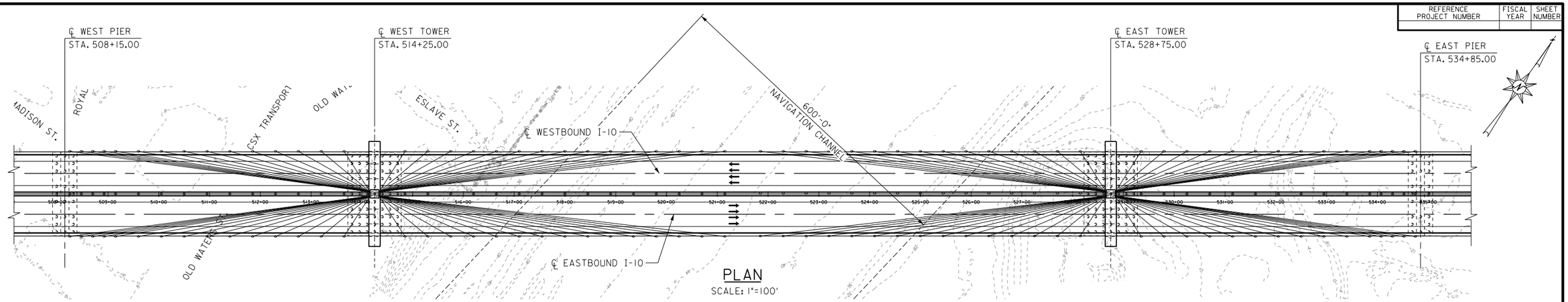

PLAN
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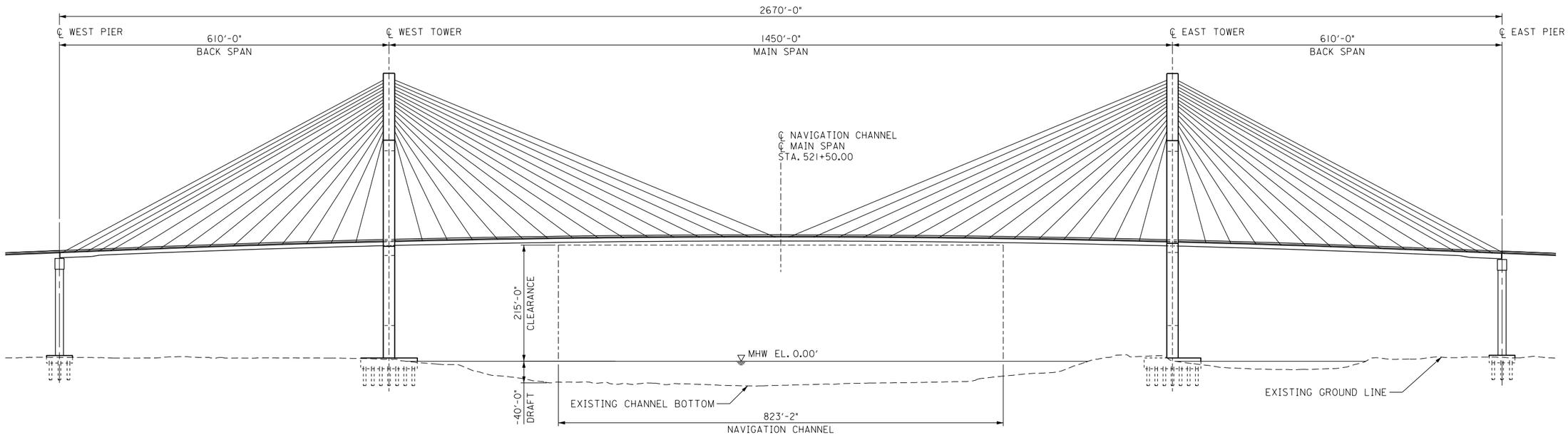

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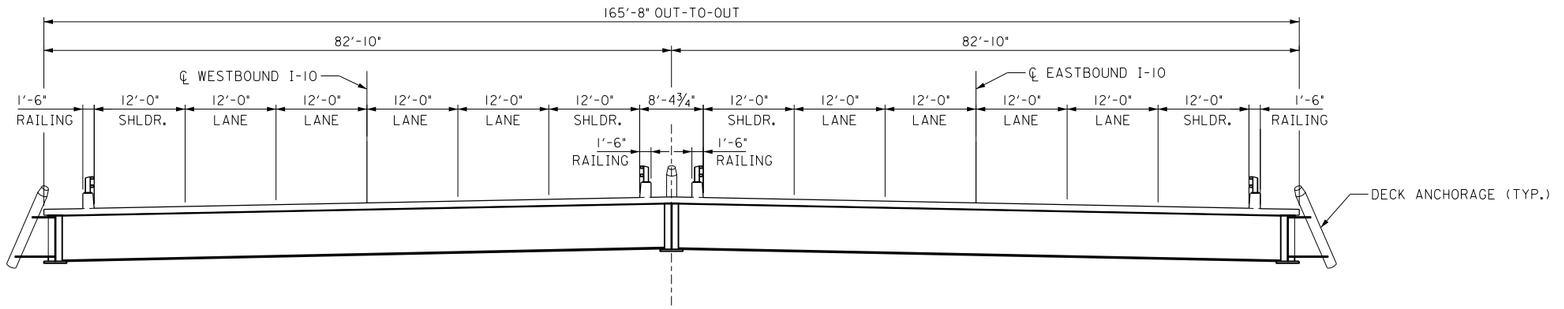

TYPICAL SECTION AT DECK ANCHORAGES
SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
DIAMOND PYLON - STEEL

ALTERNATE 4 (1 OF 2)

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

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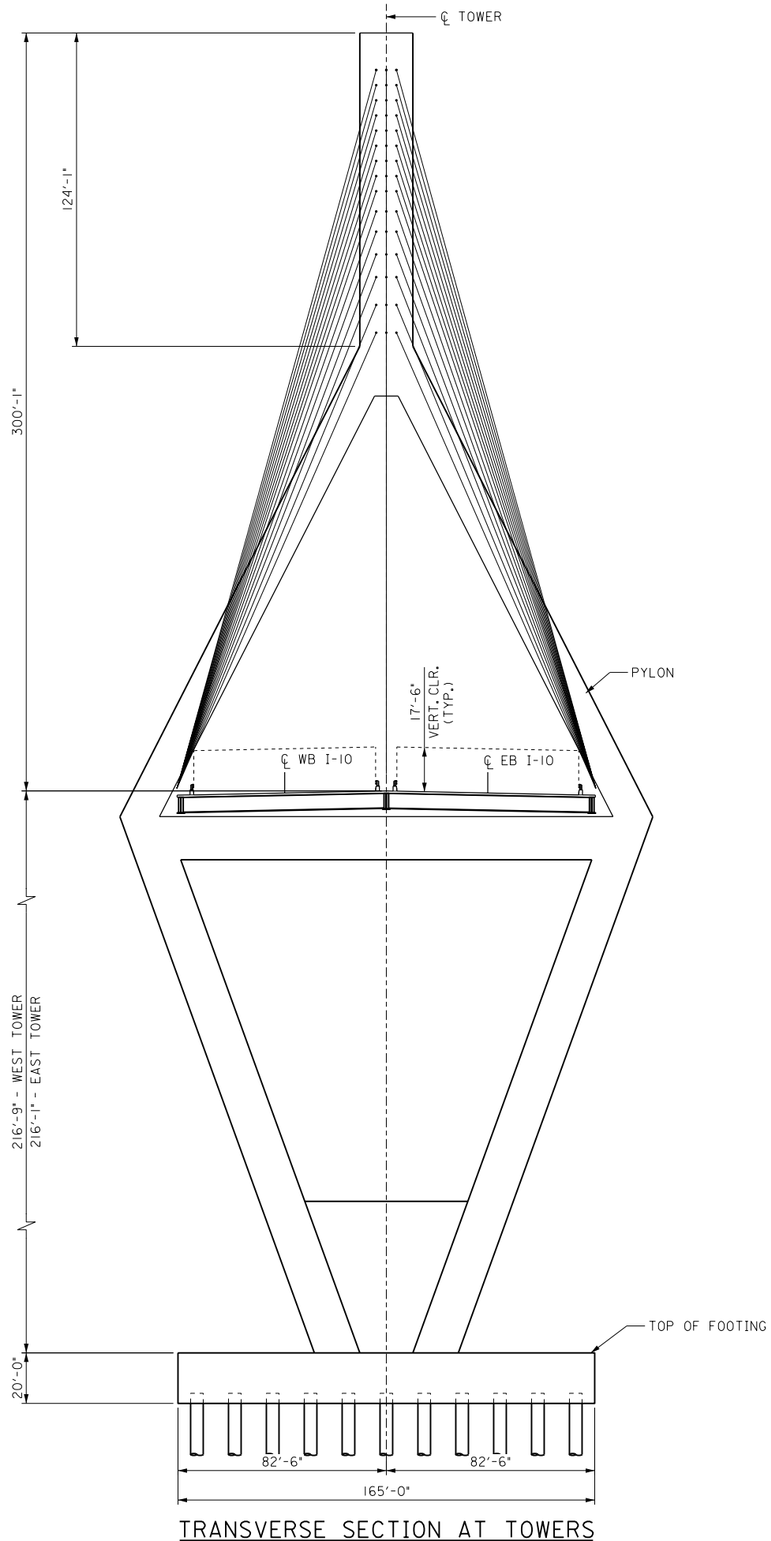

TRANSVERSE SECTION AT TOWERS

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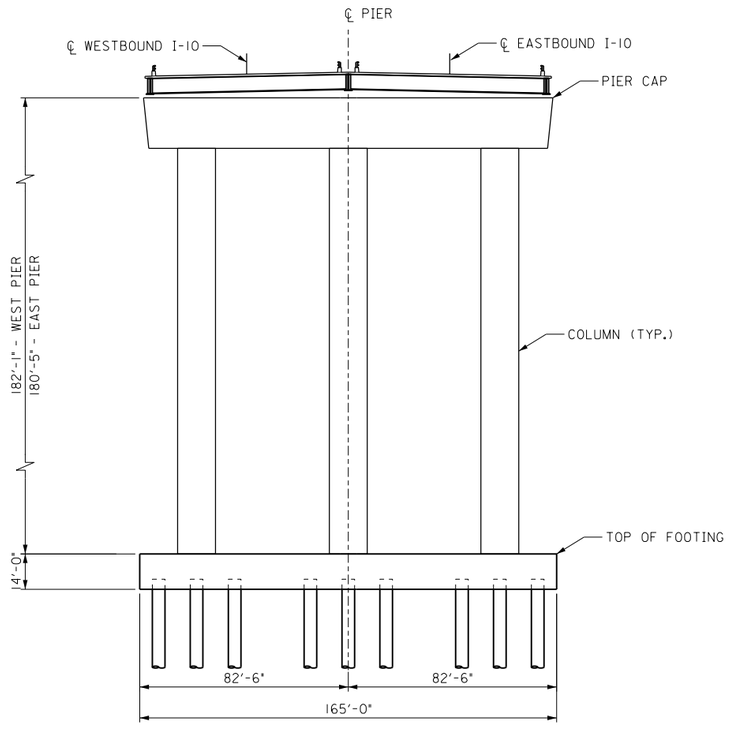

TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
DIAMOND PYLON - STEEL

ALTERNATE 4 (2 OF 2)

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

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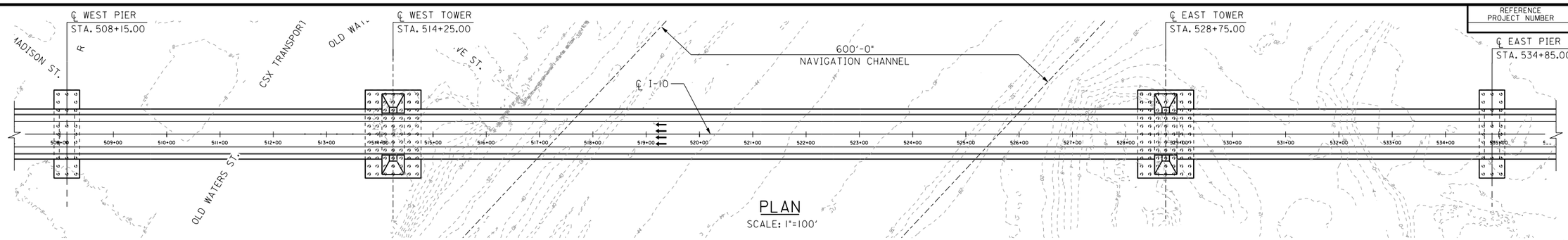
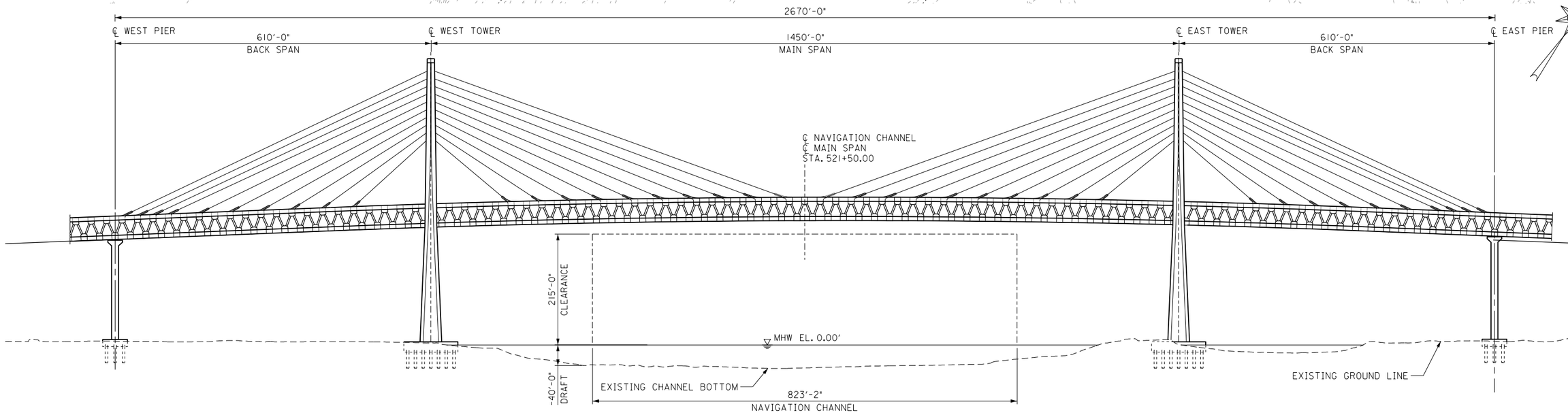


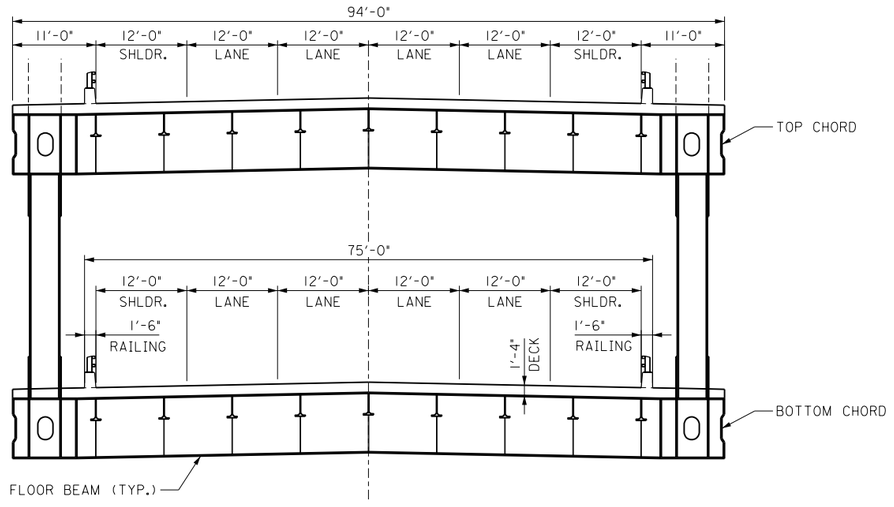
PLAN

SCALE: 1"=100'



ELEVATION

SCALE: 1"=100'



TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
DOUBLE DECK
STEEL CABLE-STAYED

ALTERNATE 5 (1 OF 2)

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

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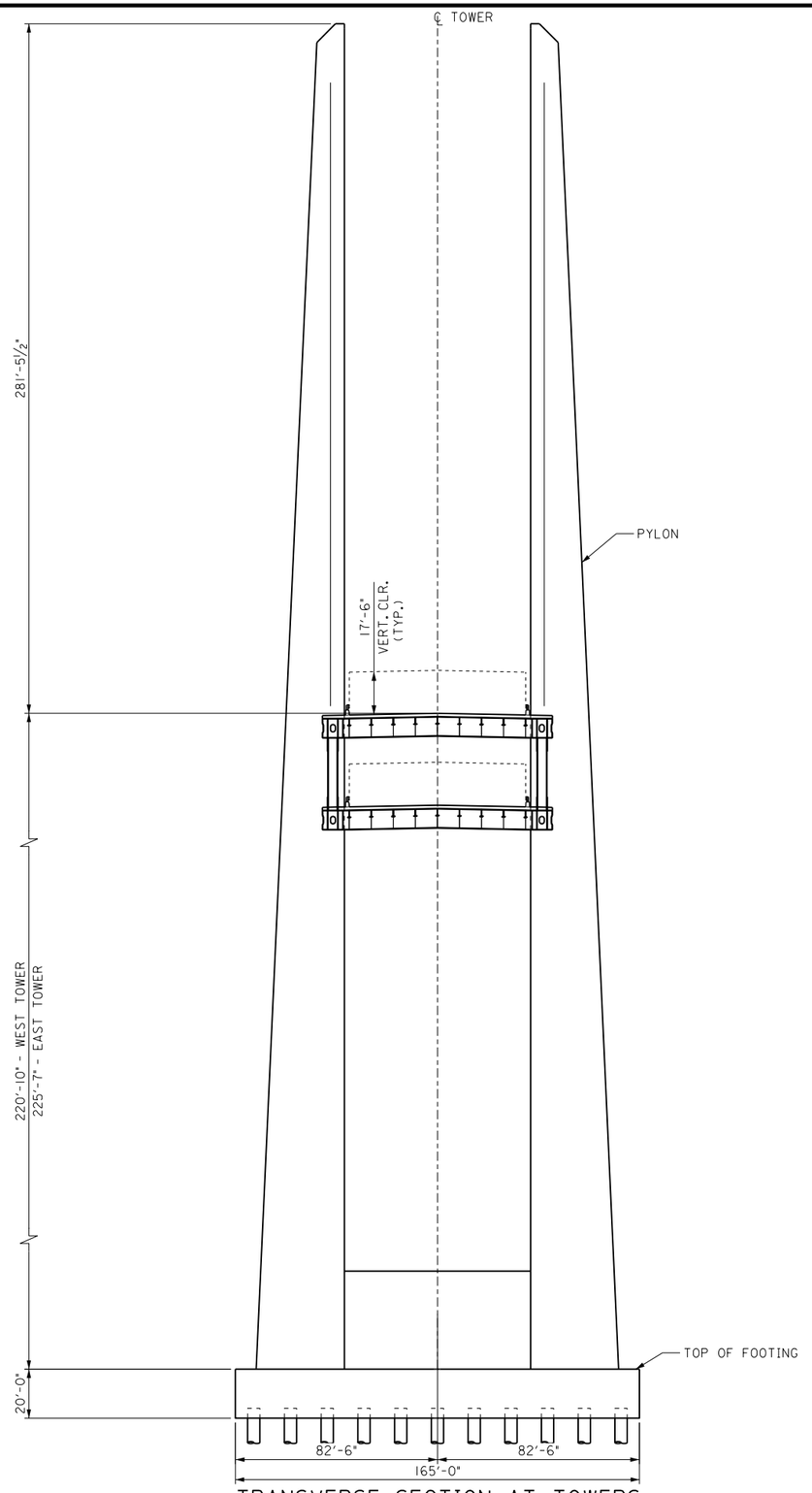
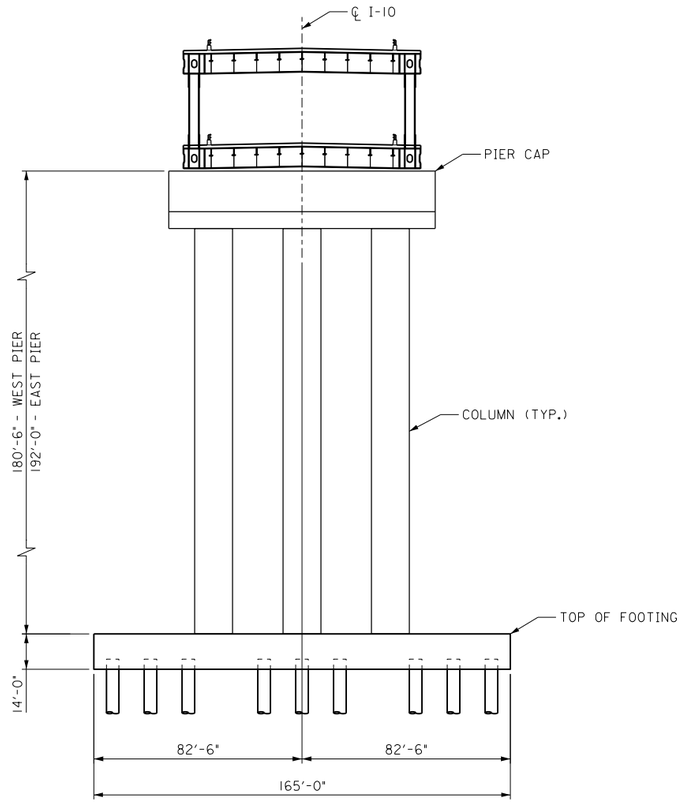


TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
DOUBLE DECK
STEEL CABLE-STAYED

ALTERNATE 5 (2 OF 2)

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

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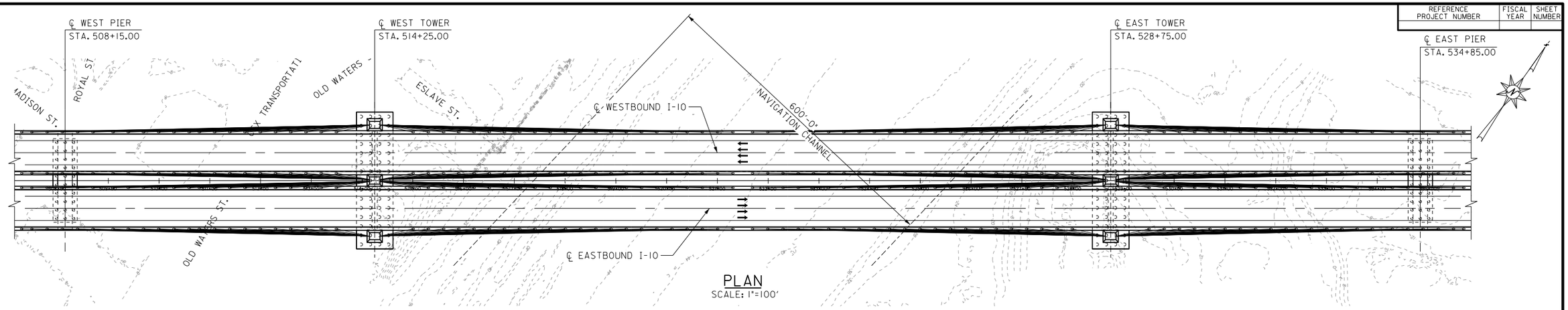

PLAN
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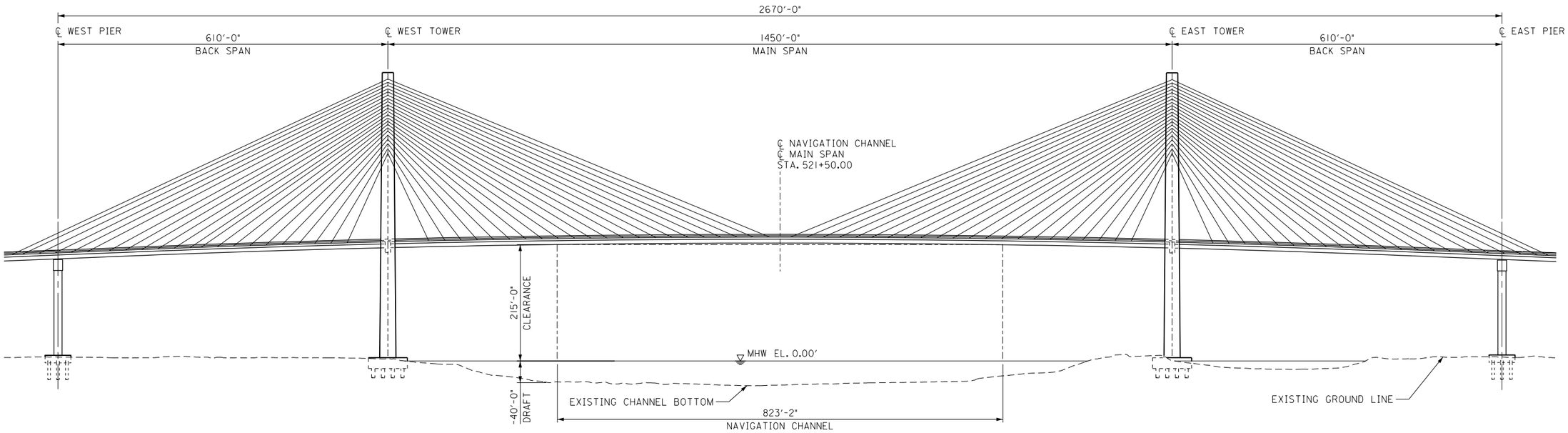

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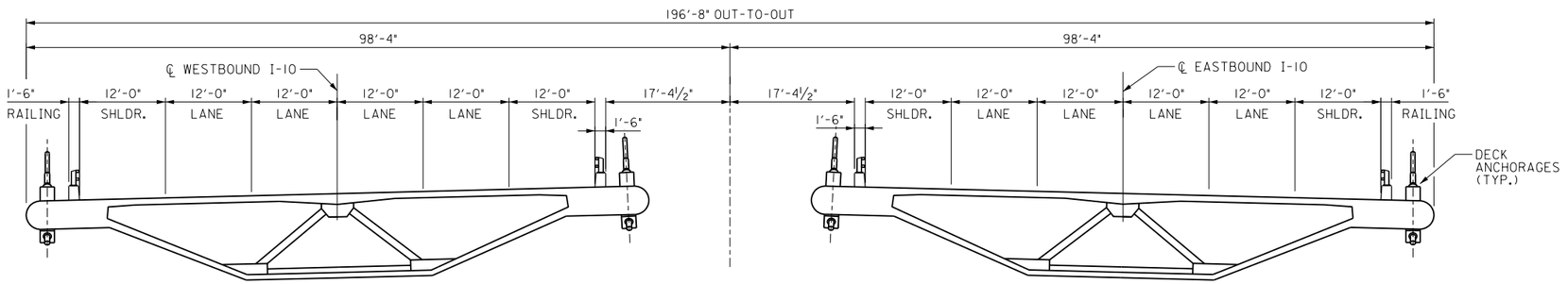

TYPICAL SECTION AT DECK ANCHORAGES
SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - TRIPLE PYLON - CONCRETE
REVISIONS	ALTERNATE 6 (1 OF 2)
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: MWL / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

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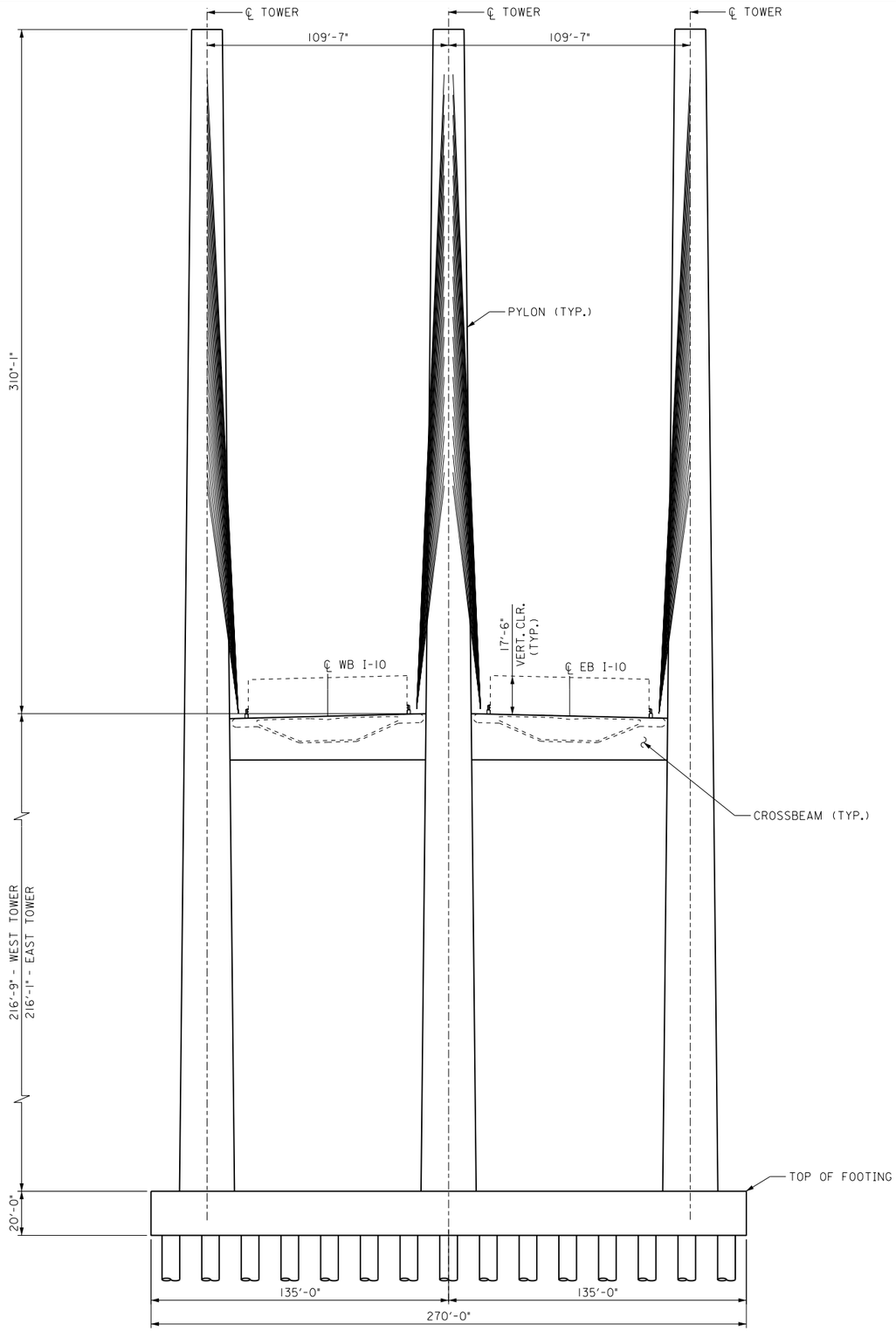
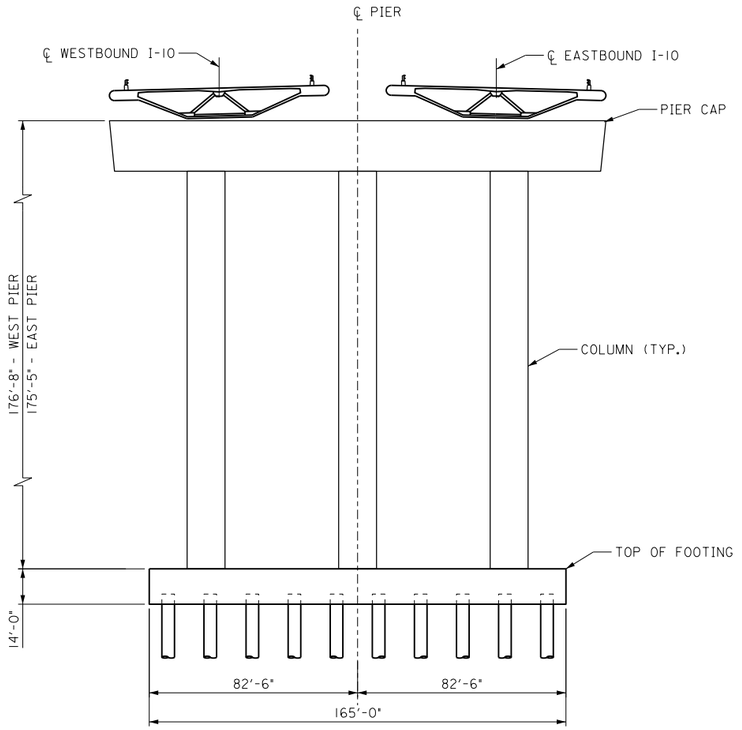


TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - TRIPLE
PYLON - CONCRETE

ALTERNATE 6 (2 OF 2)

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

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
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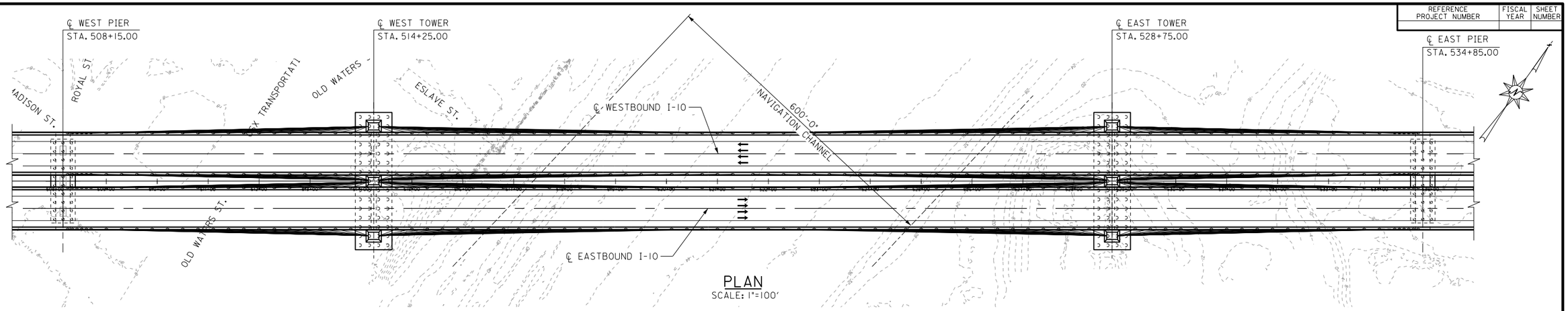
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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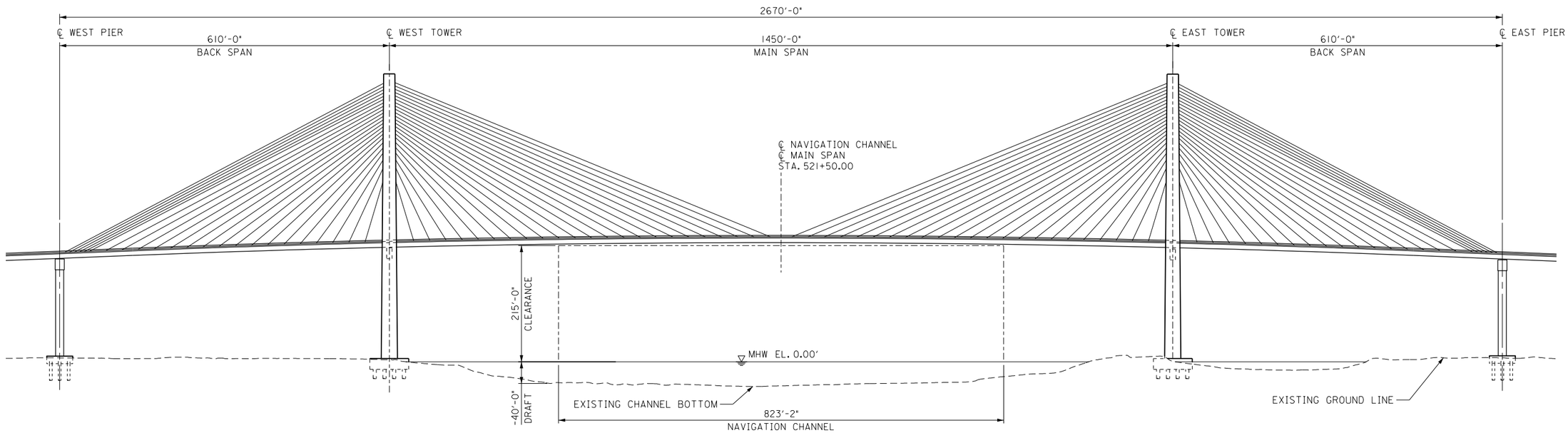

ELEVATION
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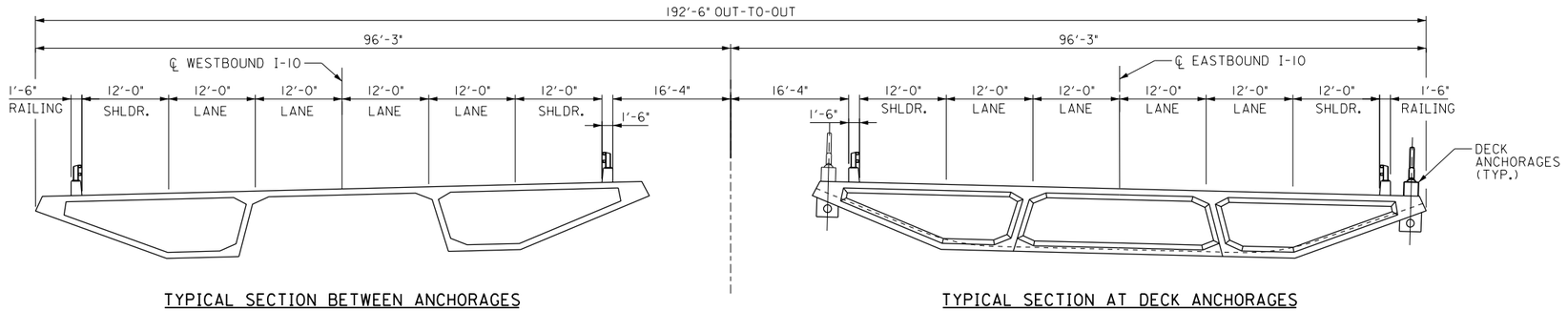

TYPICAL SECTION BETWEEN ANCHORAGES

TYPICAL SECTION AT DECK ANCHORAGES

TYPICAL SECTION
SCALE: 1"=10'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 2

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - TRIPLE
PYLON - CONCRETE

ALTERNATE 7 (1 OF 2)

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

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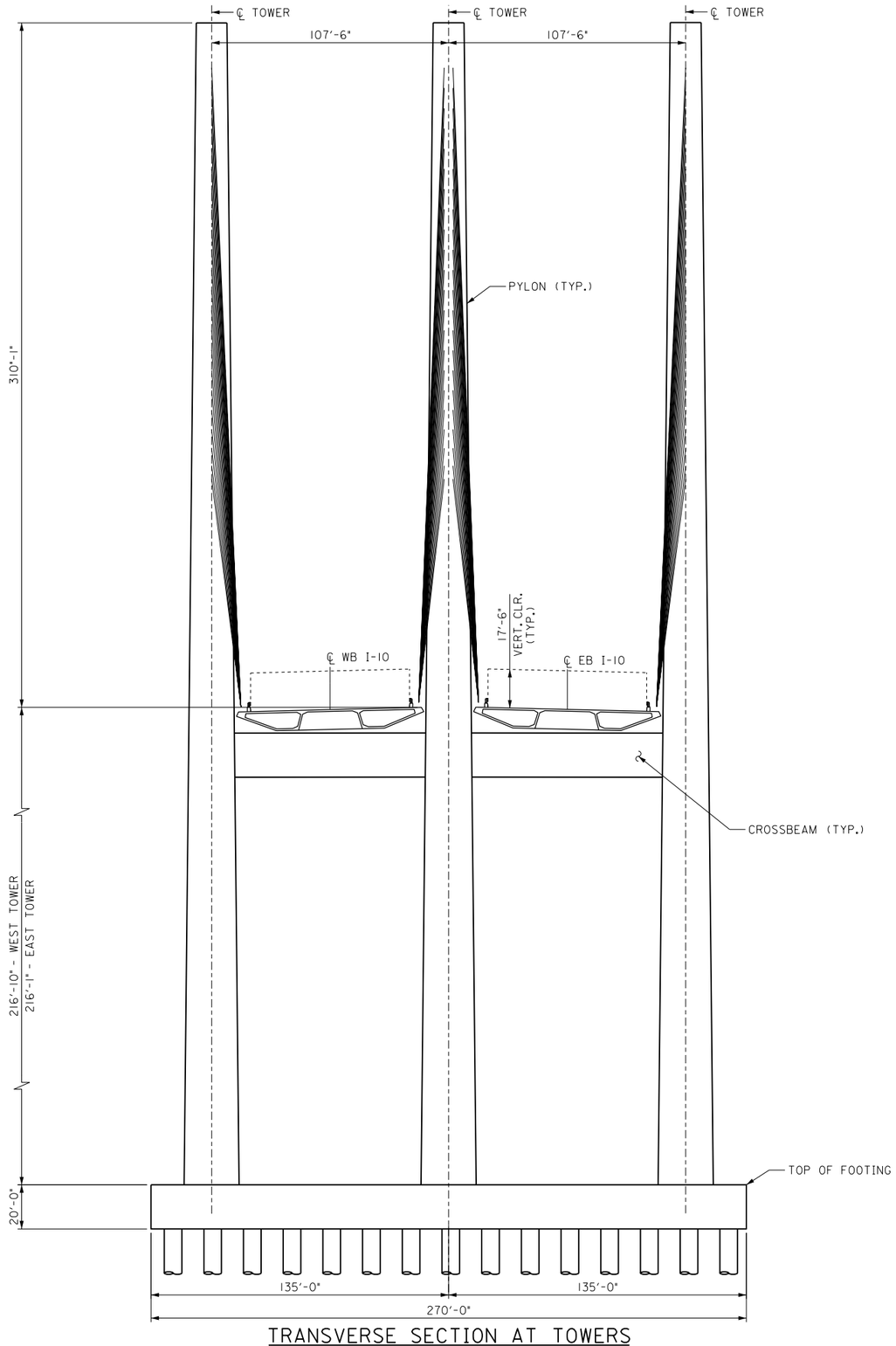
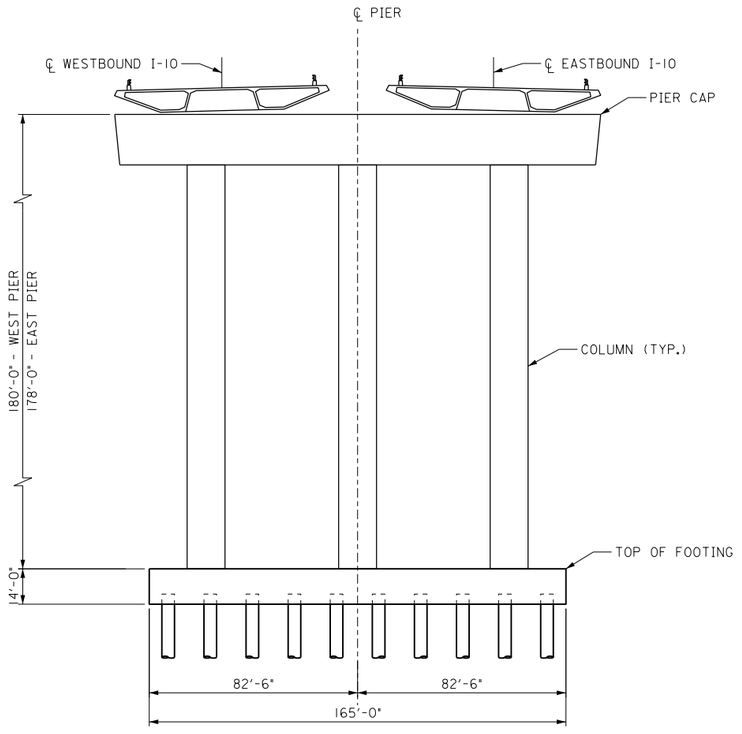


TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 2	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - TRIPLE PYLON - CONCRETE
REVISIONS	ALTERNATE 7 (2 OF 2)
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: MWL / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 24, 2017 12:01:07 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

Appendix E:

Wind Studies Report

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650 Woodlawn Road West
Guelph, Ontario, Canada
N1K 1B8

Mobile River Crossing

Mobile, AL, USA

Draft Report

Wind Climate Analysis

RWDI # 1600368
July 11, 2016

SUBMITTED TO

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VERSION HISTORY

RWDI Project #1600368	Wind Engineering Study of Mobile River Crossing Bridge	
Report	**Releases**	**Dated**
1. Wind Climatology	1st Release to all parties for reviewing	June 30, 2016
2. Project Team	Mike Gibbons, M.E.Sc. Valerie Sifton, P.Eng. Derek Kelly, M.Eng., P.Eng.	Project Coordinator Wind Engineering Specialist Project Manager

EXECUTIVE SUMMARY

Rowan Williams Davies & Irwin Inc. (RWDI) was retained by HDR Inc. to conduct wind engineering studies of the proposed new Mobile River Crossing Bridge project, to be built over the Mobile River in Mobile, Alabama. The current study focuses on:

- The determination of the project required design wind speeds for wind loading and aerodynamic stability verification; and,
- The determination of site specific turbulence properties.

The primary facts and findings of this wind climate study are:

- Recommended wind speeds at deck level of 240 ft
  - 82 mph, mean hourly wind speed for a return period of 20 years, applicable for design during construction of the bridge;
  - 113 mph, mean hourly wind speed for a return period of 100 years, applicable for the design of the completed bridge;
  - 141 mph, 10-min mean wind speed for a return period of 1,000 years, applicable for stability verifications during construction; and,
  - 163 mph, 10-min mean wind speed for a return period of 10,000 years, applicable for stability verifications of the completed bridge.

Wind speeds for design do not include any wind directionality reductions whereas for stability verifications, reductions due to wind climate directionality for extreme winds were included.

Site specific turbulence properties were derived as required for the stability and wind loads studies.

1. INTRODUCTION

Rowan Williams Davies and Irwin Inc. (RWDI) was retained by HDR Inc. to undertake comprehensive wind engineering studies for the proposed Mobile River Crossing Bridge project. The proposed bridge will consist of the construction of a cable-stayed bridge over the Mobile River, with a main span length between 1250 ft and 1450 ft.

2. WIND CLIMATE ANALYSIS

2.1 Overview

This section of the report presents the analysis of the wind climate and wind turbulence properties for the Mobile River Crossing Bridge project. The results presented will be used for the testing and analyses of aerodynamic stability and design wind loads. This study also includes recommended design wind criteria following project requirements and current design practice in North America.

2.2 Wind Climate and Site Analysis

Below are established and recommended design wind criteria required for stability analysis and derivation of wind loads for this bridge. References to relevant documents including ASCE 7-10 Standard are also given. Wind speeds were adjusted for the height of the deck as described later in the report.

2.2.1 Sources of Data

The wind statistics used to determine the design wind speeds and directionality at the bridge site were based on the surface wind measurements taken between 1980 and 2015 at the Mobile Regional Airport, located approximately 12 miles to the west of the proposed bridge site. This airport station contains sufficiently long records to perform a comprehensive statistical analysis for normal winds. Figure 2-1 shows the location of the airport in relation to the Mobile River Crossing Bridge. To enhance our knowledge of extreme wind events, in addition to the historical data, a site specific computer simulation of hurricanes was also obtained. This hurricane simulation was provided by Applied Research Associates (ARA), of Raleigh, NC and was generated using the Monte Carlo Simulation Technique. ARA provided data of hurricane passages both at the surface, deck height and upper levels, corresponding to 33 ft, 240 ft and 2000 ft heights, respectively. The wind speeds at deck level were combined with locally recorded data of surface winds, scaled up to the deck level of 240 ft, to develop the wind climate model for this study. The hurricane study is based on a simulation of 300,000 storm years occurring in the North Atlantic basin and considers all storms that came within a 155 mile radius of the project site.

2.2.2 Local Terrain

Care was taken in the analysis of the hourly wind records from the Mobile Regional Airport to account for the effects of the upwind terrain surrounding the airport. Ideally, airport anemometers are installed in the type of unobstructed open terrain that is used as a reference condition by the building codes. It is important to take into account the site effects for every wind direction so as to correct the recorded wind speeds to as close as possible to their true values representing a standard open terrain.

Prior to conducting any analysis of the surface wind speed observations, the effect of upwind terrain roughness and land cover characteristics at the wind instrument, and at the bridge site, were assessed for each wind direction.

The worldwide recognized wind engineering reference ESDU describes a method[1,2] based on earlier work of Deaves and Harris[3] for evaluating changes in the mean velocity profile following changes in ground roughness. This is particularly suited to translate wind speeds between two sites that experience the same winds, but where upwind conditions at the two sites vary. It can also be used to adjust a measured wind speed value to that of the required reference wind exposure such as standard exposures as defined in the Design Codes.

This method was applied to determine the velocity profiles at the location of each airport's wind instrument, and at the Mobile River Crossing site. Photographs of the airports and their surroundings, and satellite imagery were all used to assess the ground roughness changes for each wind direction.

For comparison to code values, the wind speeds were adjusted by wind direction based on the exposure of the anemometer to produce wind speeds that are equivalent to standard open terrain (i.e. Exposure C in ASCE 7-10[4]).

For evaluation of the wind conditions at the bridge site, the wind speeds from the airport were transposed to the bridge site. This was accomplished by using the wind speeds recorded at the airport anemometer, along with the full wind velocity profile for each wind direction. The wind records were then scaled up to gradient height, which is the height above which the roughness elements on the earth's surface have no impact on the wind speed or turbulence. This establishes a regional wind condition independent of terrain.

1 Engineering Sciences Data Unit, Strong Winds in the atmospheric boundary layer, Part 1: Mean Hourly Speeds, ESDU 82026, London ,UK, 1982.
2 Engineering Sciences Data Unit, Strong Winds in the atmospheric boundary layer, Part 2: Discrete Gust Speeds, ESDU 83045, London ,UK, 1983
3 Deaves, D.M. and Harris, R.I., A Mathematical Model of the Structure of Strong Winds, *Construction Industry Research and Information Association (U.K.) Report #76,* 1978
4 ASCE STANDARD, ASCE/SEI-7-10, American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, 2010.

The gradient height wind speed was then scaled down to the deck height of 240 ft on a direction by direction basis, based on the terrain conditions and wind profiles at the bridge site. Figure 2-2 illustrates this method of how measured wind speeds are translated from the anemometer location to the bridge site at deck level.

2.2.3 Analysis

The design wind speeds for the bridge site were determined applying the following:

(i) The joint probability of wind speed and direction for the site was determined based on the available meteorological data and hurricane simulation. The statistical analyses of this wind data produced a mathematical model based on the Weibull probability distribution, which is discussed in Section 2.2.4;

(ii) This mathematical model was used to evaluate wind speed as a function of return period and also to evaluate the component of the wind velocity normal to the bridge span as a function of return period. This method employed the Upcrossing technique, which is discussed further in Section 2.2.5; and

(iii) Extreme Value Analysis (EVA) using a Fisher-Tippet Type I distribution was conducted on all wind records collected at the three airports, and similarly to the results of the hurricane simulation.

Results contained in this report are discussed as mean-hourly (i.e., 1-hour mean) speeds, which are applicable for structural design, or as 10-minute mean speeds. In this study, 10-minute mean speeds are given since this is the typical time for an aerodynamic instability to develop on a long-span bridge. To relate the mean-hourly wind speed to the 10-minute mean, the relationship shown in the Durst curve (in Figure C26-5.1 of ASCE 7-10) provided as Figure 2-3 was assumed. According to this curve, a 1-hour mean wind speed can be converted to a 10-minute mean speed by multiplying by a factor of 1.067. Similarly, a 3-second gust speed is higher by a factor of 1.52 than the 1-hour mean speed. Adjustments for other terrain conditions were made using ESDU methodology[5].

2.2.4 Joint Probability of Wind Speeds and Directions

A mathematical model of the joint probability of wind speed and direction was fitted to the meteorological wind data assuming a Weibull type probability distribution. This distribution expresses the probability of the wind speed at a given elevation exceeding a value U as

[5] Engineering Sciences Data Unit, Characteristics of the Atmospheric Turbulence Data Near the Ground: Part III, Variations in Space and Time for Strong Winds, ESDU 86010, London ,UK, 1986.

$$P_\theta(U) = A_\theta \exp\left[-\left(\frac{U}{C_\theta}\right)^{K_\theta}\right], \quad (2\text{-}1)$$

where P_θ is the probability of exceeding the wind speed U in the angle sector θ;

θ is the central angle of an angle sector, measured clockwise from true North; and

A_θ, C_θ, K_θ are coefficients selected to give best fit to the data.

Note that A_θ is the fraction of time the wind blows from within the angle sector θ. The size of angle sectors used in this analysis was 10 degrees. To provide additional flexibility in curve fitting for normal winds, two Weibull curves were fitted, one to lower velocities and one to higher velocities, with blending expressions being used to provide smooth transitions. This "double" fitting was applied in modeling the data from the three meteorological stations for normal winds, whereas single fit Weibull parameters derived for the ARA data were applied to hurricane winds.

From the probability distributions given by Equation (2-1), the overall probability of wind speed was obtained by summing over all wind directions for both the extra-tropical wind conditions derived from the historical record, and the hurricane wind conditions derived from the Monte Carlo simulation.

$$P(U) = (1 - f_H) \cdot P_E(U) + f_H \cdot P_H(U) = \sum_\theta [(1 - f_H) P_{\theta E}(U) + f_H P_{\theta H}(U)], \quad (2\text{-}2)$$

where the subscript E refers to extra-tropical or non-hurricane winds, the subscript H refers to hurricane winds, and f_H. is the fraction of time that there are hurricane winds.

At gradient height the wind speeds are well above the earth's surface roughness effects. The height used for determining gradient speed was 2000 ft. Since the anemometer at the airport station is near ground level at the bottom of the planetary boundary layer, it is affected by ground roughness. These ground roughness effects were assessed using the methods given in ESDU combined with information on the local terrain roughness gathered from topographic maps and other site information. Factors were developed to convert the anemometer records to wind speeds at gradient height and then to the bridge site.

Whereas the 36 years of data (1980-2015 from the Mobile Regional Airport) are assumed sufficient in representing synoptic wind events, they may not contain enough extreme records of hurricanes. Therefore hurricane winds were simulated numerically and applied to improve the predictions at higher return periods. By combining both normal winds and hurricane simulation data, a function representing wind speed vs.

return period was generated and presented in Figure 2-4. In this figure, wind speeds are given as 3-sec gusts at an elevation of 33 ft in standard open terrain.

2.2.5 Upcrossing Method to Determine Directionality Effects on Design Winds

By adapting random noise theory to meteorological data (Rice[6]), it can be shown that the return period, R, in years of a given gradient wind speed, U, can be determined from the Weibull distribution discussed in Section 2.2.4 by

$$\mathrm{R} = \left[\frac{1}{2}\overline{|\dot{\mathrm{U}}|} \cdot \mathrm{p(U)} \cdot \mathrm{T_A}\right]^{-1}, \tag{2-3}$$

where $\overline{|\dot{\mathrm{U}}|}$ is the average of the absolute rate of change of the hourly values of U with time, T_A is the total number of hours in a year, i.e., $T_A \approx 8766$, and $p(U)$ is the probability density of the Weibull summed across all directions, and relates to the cumulative Weibull distribution in given in equation 2-1 by:

$$\mathrm{p_\theta(U)} = \frac{\mathrm{P_\theta(U)}}{\mathrm{dU}}, \tag{2-4}$$

$$\mathrm{p(U)} = \sum_\theta \mathrm{p_\theta(U)}. \tag{2-5}$$

Since we have two separate Weibull models representing the extra-tropical *(E)* and hurricane *(H)* conditions, equation (2-3) becomes:

$$R = \left[\tfrac{1}{2}\overline{|\dot{U}_E|} \cdot p_E(U) \cdot (T_A - T_H) + \tfrac{1}{2}\overline{|\dot{U}_H|} \cdot p_H(U) \cdot T_H\right]^{-1}. \tag{2-6}$$

Where T_H is the average number of hurricane hours per year. Equation 2-6 was used to determine the return periods for a series of selected wind speeds. The wind speed corresponding to a required return period (e.g., 20, 100, 1,000 years etc.) could then be determined by interpolation. This method is called the Upcrossing Method and was applied to the meteorological data from the three stations and the ARA Hurricane simulation to determine directional reduction factors discussed further in Section 2.3.2.

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6 Rice, S.O., Mathematical Analysis of Random Noise, *The Bell System Technical Journal*, Vol. 23, 1944.

2.2.6 Extreme Value Analyses to Determine Design Wind Speeds

The meteorological data available from the Mobile Regional Airport (1980-2015) were again employed in this analysis. The peak mean-hourly wind speeds based on a nominal 4-day epoch were extracted using the Method of Independent Storms (Cook[7]). These speeds were fit to a Fisher-Tippet Type I distribution, which is given by

$$P(\hat{U}) = \exp\left(-exp\left(-a(\hat{U}-b)\right)\right), \quad (2\text{-}7)$$

where $P(\hat{U})$ is the probability that the annual peak velocity will not exceed the value, $\hat{U}$ is the peak velocity, $1/a$ is dispersion and b is the mode.

As described by Harris[8], traditional methods of extreme value analysis typically include fitting monthly or annual maxima to an extreme value distribution. Harris also states that using the Method of Independent Storms in lieu of the monthly or annual maxima increases the number of wind speed maxima used in the extreme value fitting considerably, which ultimately provides greater accuracy in the prediction of the extreme wind speeds.

The size of the hurricane simulation dataset allows for an extreme value analysis of the winds to be conducted without fitting the wind speed data to a particular statistical distribution. Each wind speed is given a rank based on its magnitude. The wind speed probability distribution, $P_t(v > V)$, is given as:

$$\mathrm{P_t(u > U) = 1 - \sum_{x=0}^{\infty} P(u > U|x) \cdot p_t(x)}, \quad (2\text{-}8)$$

where t denotes the return period in years, $p_t(x)$ is a Poisson process defined as the probability that x storms occur during the time period t, and $P(u > U|x)$ is the probability that velocity u is less than U given that x storms occur.

2.3 Results

According to the wind map on Figure 26.5-1A of the ASCE 7-10 Standard, the basic speed for this region is 163 mph 3-sec gust, corresponding to a return period of 1,700 years. Figure 2-4 shows the 3-sec gust wind speeds in open terrain at elevation 33 ft recommended for the bridge site, the extreme value analysis of the data from Mobile Regional Airport, the extreme value analysis of the ARA hurricane simulation and

[7] Cook, N.J., Towards Better Estimation of Extreme Winds, *J. Wind Eng. Ind. Aerodyn.*, Vol. 9, pp 295-323, 1982.
[8] Harris, R.I., XIMIS, a penultimate extreme value method suitable for all types of wind climate, J. Wind Eng. Ind. Aerodyn., Vol 97, pp 271-286, 2009.

the ASCE 7-10 recommended speeds at return periods ranging from 10 to 1,700 years. The proposed curve for design in this study is based on the combined probability of the historical and hurricane extreme value analyses. It agrees well with the values from ASCE 7-10 for all return periods considered. Based on the analysis completed by RWDI and described herein, it is recommended that the combined historical and hurricane distribution be used as the basis for design. This recommendation is in accordance with 26.5.3 of ASCE 7-10.

2.3.1 Wind Speeds at the Bridge Site

Figure 2-5 shows various wind speeds at a deck level of 240 ft as a function of return period. This figure presents the following information:

- mean-hourly speeds at deck level for return periods from 1 to 10,000 years derived from the meteorological data described in Section 2.2, uncorrected for wind directionality; and,
- the 10-minute mean wind speed for return periods from 1,000 and 10,000 years with and without wind directionality.

Mean-hourly speeds are to be used for derivation of design loads whereas the 10-min speeds are to be applied for flutter stability verifications.

2.3.2 Wind Directionality Effects

The relative probabilities of exceeding various mean wind speeds from within each of thirty six direction sectors are shown in Figure 2-6. The curves show the probability of exceeding wind speeds with 20, 100, 1,000 and 10,000 year return periods as a function of wind direction. Also the probability of “all winds” (i.e., of any speed) is shown. The proposed bridge main span axis is oriented approximately parallel to the southwest/northeast direction. Therefore, winds normal to the spans would blow from approximately north-west and south-east. Figure 2-6 shows that the most probable directions for strong winds (e.g., once in 100 years) are from the east-northeast. Since the loading of individual structural components varies differently with wind direction, it is difficult to develop a generally applicable directionality reduction factor for all structural components. This, combined with the above-mentioned alignment of strong winds, indicated to us that no directionality reduction can be applied to the winds applicable for bridge design and this is in accordance with current design practice of North America.

Figure 2-6 shows that the most probable directions for very high return periods (e.g., once in 10,000 years) are from the east-northeast and south. There is evidence (Irwin and Schuyler[9]) that flutter instability is essentially a function of the wind velocity component normal to the span. Therefore, directionality reduction factors may be applied to the wind speeds for stability assessment of the bridge.

2.3.3 Terrain at the Bridge Site

The terrain surrounding the bridge site is a combination of open water, suburban and urban terrain. As an approximate approach to assess the terrain effects, the ESDU method was used. The suburban and urban areas were taken as having roughness lengths in the range of z_0 = 0.7 ft to 1.0 ft. The roughness lengths of the water fetches were determined as in the range of 0.01 ft to 0.03 ft depending on wind speed. In terms of the traditional power law formula, in which mean velocity varies with height to the power of an exponent α, the value of the exponent ranges from 0.13 to 0.17.

2.3.4 Wind Speeds at Deck Height

For structural design of major bridges, a return period of 100 years is typically used. As described in the previous section, the 100-year mean-hourly speed was estimated to be 113 mph at the deck level (Table 2-1). In accordance with typical practices in North America, for the construction phase a 20-year return period is typically referenced for which the estimated mean-hourly speed is 82 mph.

For flutter instability of the completed bridge, a very long return period needs to be considered because, if flutter occurs, there is a very high probability of structural failure. Thus the recommended return period is 10,000 years. The typical wind speed averaging time used in bridge studies in North America for assessing aerodynamic instabilities has been 10 minutes. Therefore, the recommended 10,000-year speed is a 10-minute mean value. Using a ratio of 1.067 to scale mean-hourly speeds to 10-minute mean speeds, the design speed for flutter is thus calculated to be 163 mph including directionality effects. For construction, a shorter return period is justifiable due to the shorter length of the construction period, and 1,000 years is recommended. The 1,000-year design flutter speed, arrived at by a similar approach, is 141 mph including directionality effects.

2.3.5 Turbulence Properties at the Bridge Sites

The same ESDU methodology used in determining the wind speeds at the deck level was also applied when estimating the turbulence intensities and length scales at the site. The power law exponent (α), the

[9] Irwin, P.A. and Schuyler, G.D., Experiments on a Full Aeroelastic Model of Lions' Gate Bridge in Smooth and Turbulent Flow. National Research Council of Canada, NAE Report LTR-LA-206, 1977.

range of roughness length values used (z_0) in determining turbulence properties, turbulence intensities (I_u, I_w, I_v) and length scales (${}^xL_u$, ${}^xL_w$, ${}^yL_u$, ${}^yL_w$, and ${}^zL_w$, parameters required for the buffeting response of long-span bridges to strong winds) are given in Table 2-2.

TABLES

Table 2-1: Recommended wind speeds at bridge site

Wind Speed Applicable for	Return Period (years)	Mean Wind Speed (mph) at Deck Level 240 ft and Averaging Time		Corresponding 3-sec Gust Speed (mph) at 33 ft Open Terrain
Design during construction	20	82	1 h	93
Design of completed bridge	100	113	1 h	126
Stability during construction	1,000	141*	10 min	147*
Stability of completed bridge	10,000	163*	10 min	170*

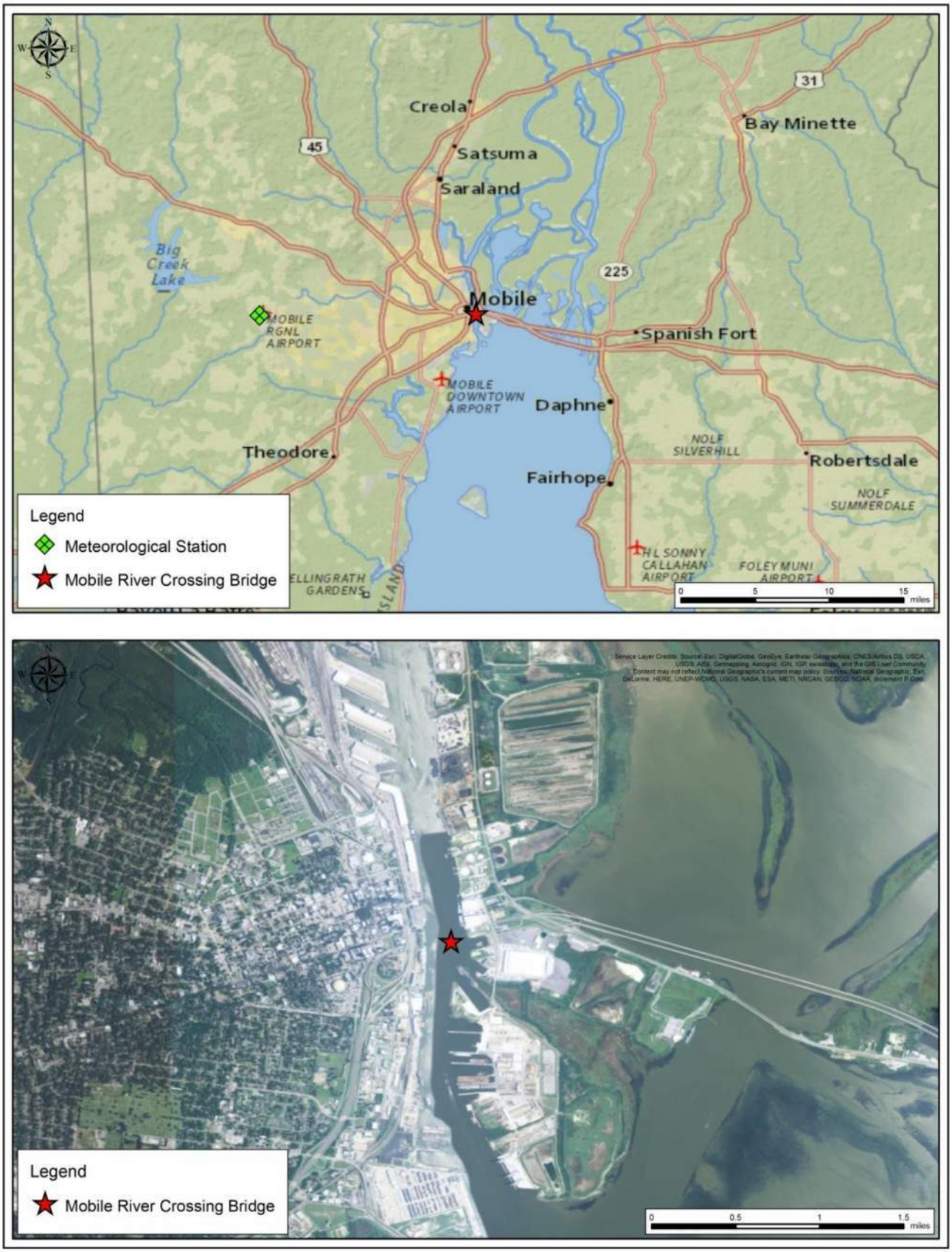
*Includes reduction due to extreme wind climate directionality

Table 2-2: Turbulence properties at deck level of 240 ft

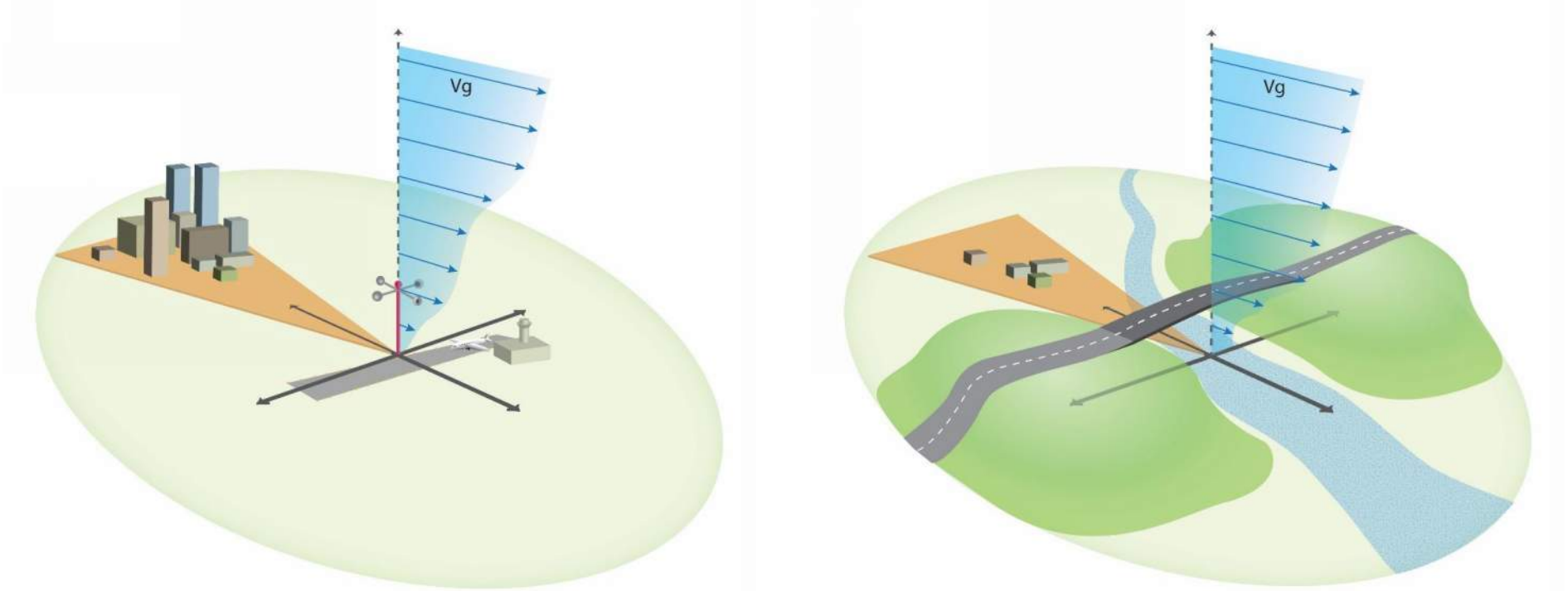
Direction (°CW from N)	z_0 (ft)	α	I_u (%)	I_v (%)	I_w (%)	${}^xL_u$ (ft)	${}^xL_w$ (ft)	${}^yL_u$ (ft)	${}^yL_w$ (ft)	${}^zL_u$ (ft)
150	0.10	0.140	14.0	10.9	7.7	1847	154	511	86	317
330	0.20	0.119	15.3	11.9	8.4	1857	155	513	86	317

Notes:
1. α - power law constant of wind profile
2. $I_{u,v,w}$ - longitudinal, horizontal-across-wind, and vertical turbulence intensities
3. ${}^{x,y,z}L_{u,v,w}$ - turbulence length scales

FIGURES



Location of the Mobile Bridge and Airport	Figure No. 2-1	RWDI
Mobile Bridge – Mobile, Alabama, USA Project #1600368	Date: June 30, 2016	

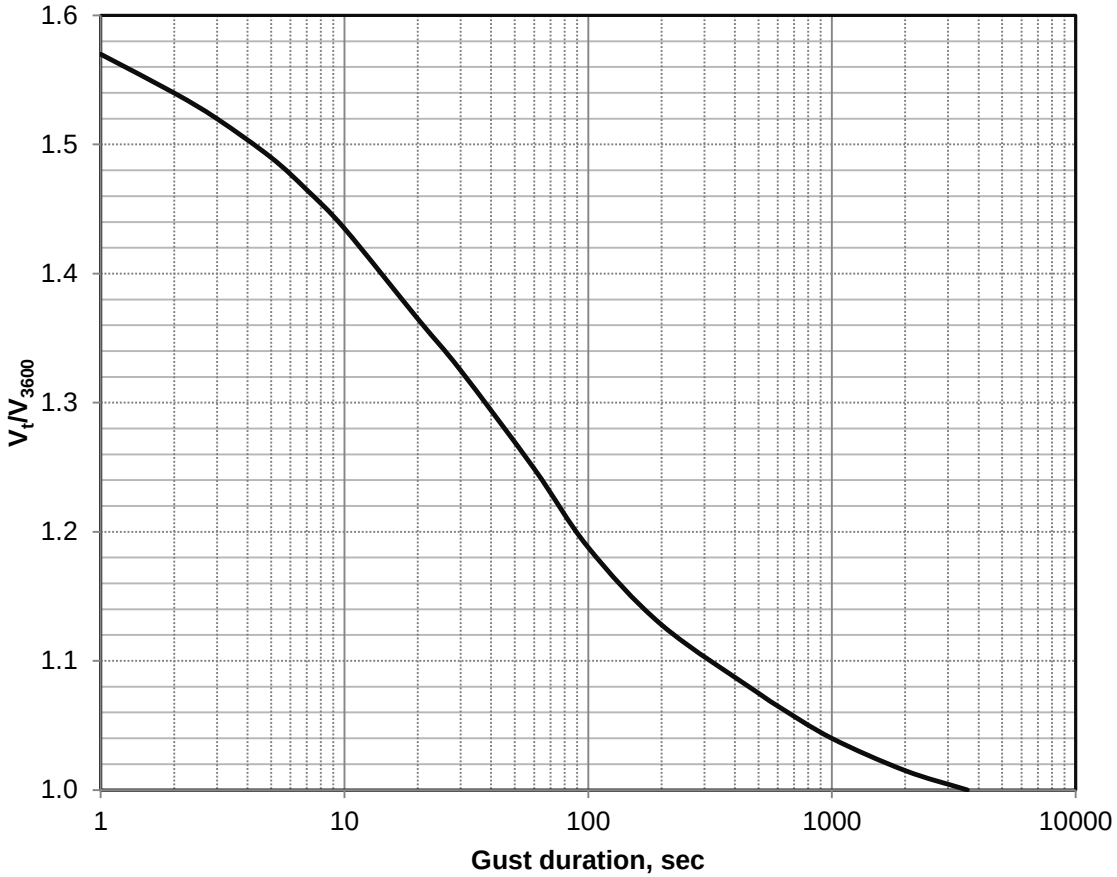


(a) Wind Profile at the Wind Measurement Location

(b) Wind Profile at the Bridge Site

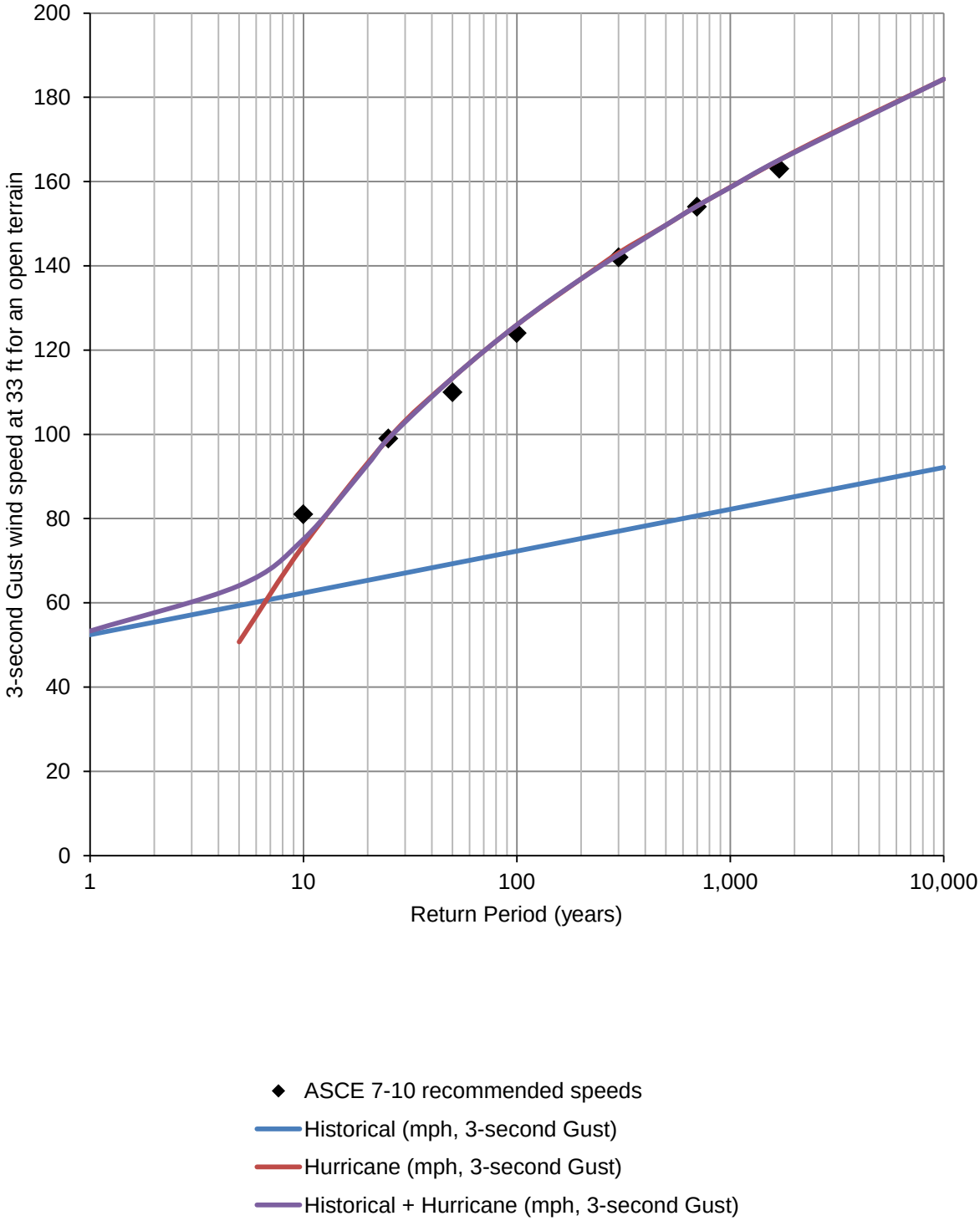
The upwind terrain at the airport or wind measurement site (a) influences the wind speed profile differently than at the bridge site (b), up to gradient height, which is the height beyond which the surface roughness has any influence on the wind speed or turbulence. The ESDU method described in Section 2.2 of this report calculates the wind speed profile based on the changes in the upwind terrain and their relative distance to the measurement location, up to gradient height. The gradient height wind speed can then similarly scaled down to the bridge deck height based on the upwind terrain at the bridge site. Note that these figures are meant to be illustrative in nature and not representative of the specific project site.

Translating Measured Wind Speeds to a Different Location	Figure No. 2-2	RWDI
Mobile Bridge – Mobile, Alabama, USA Project #1600368	Date: June 30, 2016	

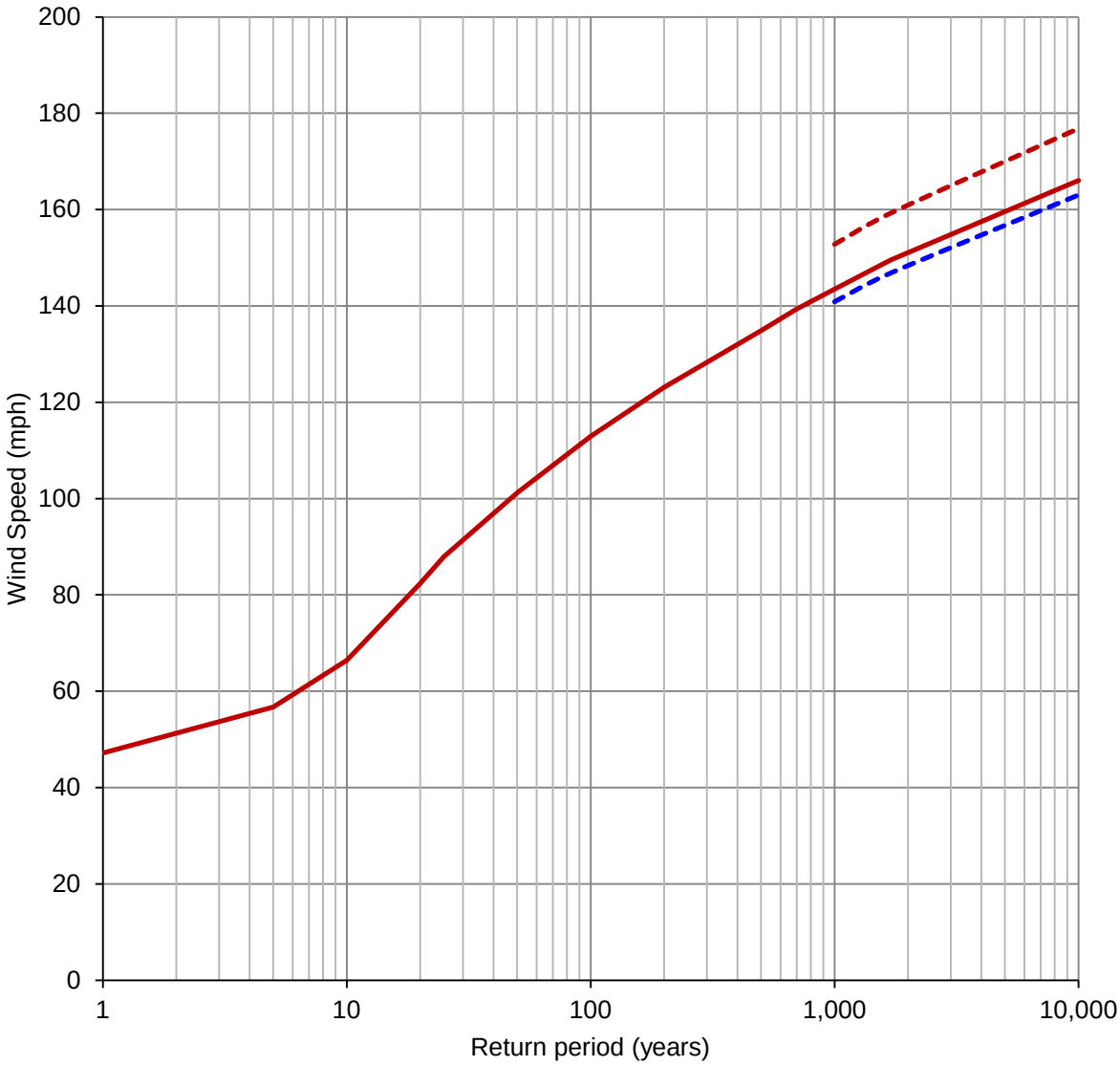


Notes:
1. From Figure C26-5.1 of ASCE 7-10
2. Gust to mean hourly speed ratios are only valid for open terrain

Gust Speed to Mean Speed Ratio vs Speed Averaging Time	Figure No. 2-3	RWDI
Mobile Bridge – Mobile, Alabama, USA Project #1600368	Date: June 30, 2016	



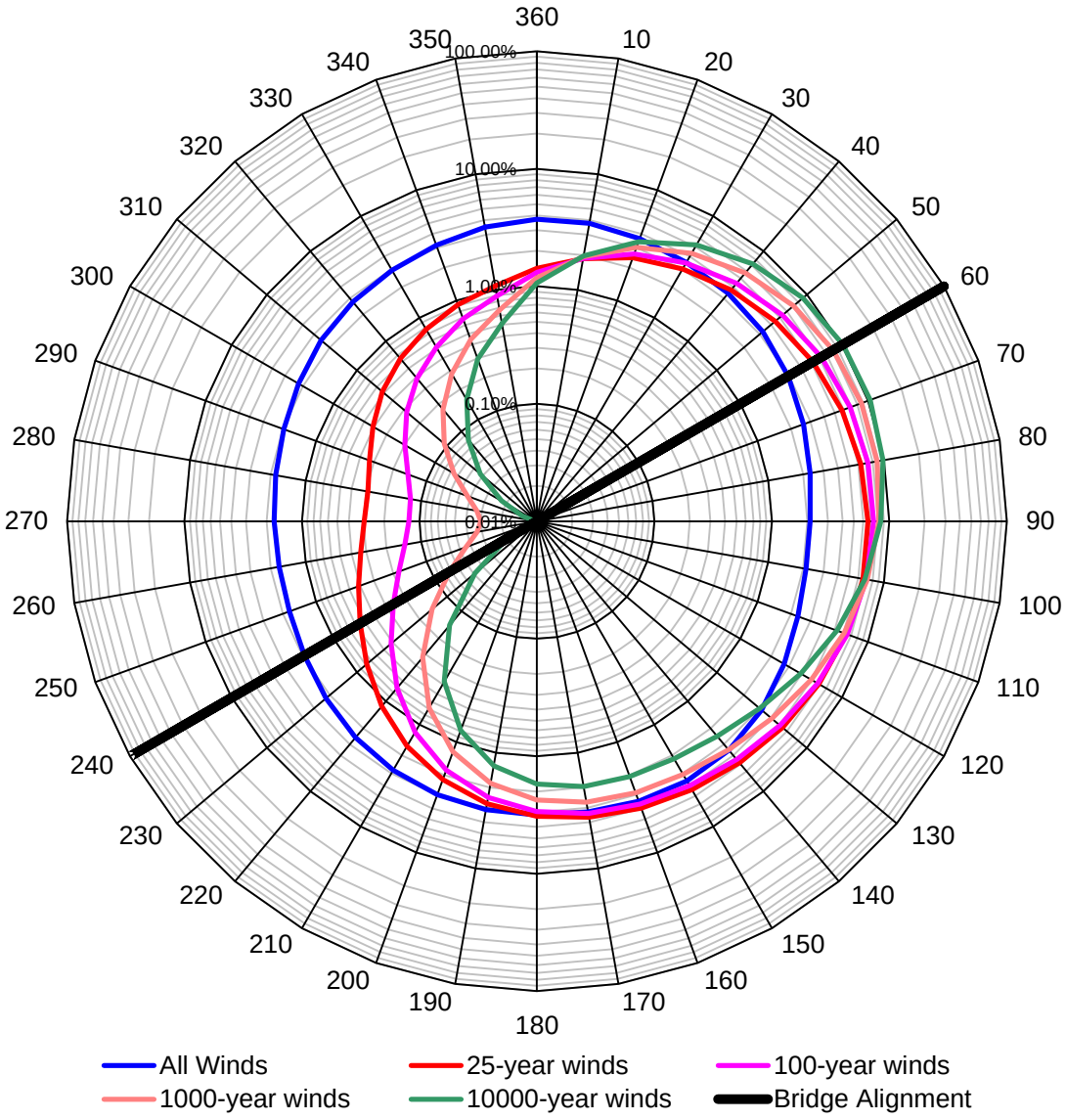
3-second Gust wind speed at 33 ft for an open terrain Wind Engineering Study Mobile Bridge – Mobile, Alabama, USA Project #1600368	Figure No. 2-4 Date: June 30, 2016	RWDI



Mean Wind Speed for Various Return Periods
Wind speeds at elevation 240 ft above grade
Wind Engineering Study
Mobile Bridge – Mobile, Alabama, USA
Project #1600368

Figure No. 2-5

Date: June 30, 2016



Directional distribution of hourly mean winds at the project site
Probability (%) of the wind direction for certain return periods
Wind Engineering Study
Mobile Bridge – Mobile, Alabama, USA Project #1600368

Figure No. 2-6

Date: June 30, 2016

RWDI

Appendix F:

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.

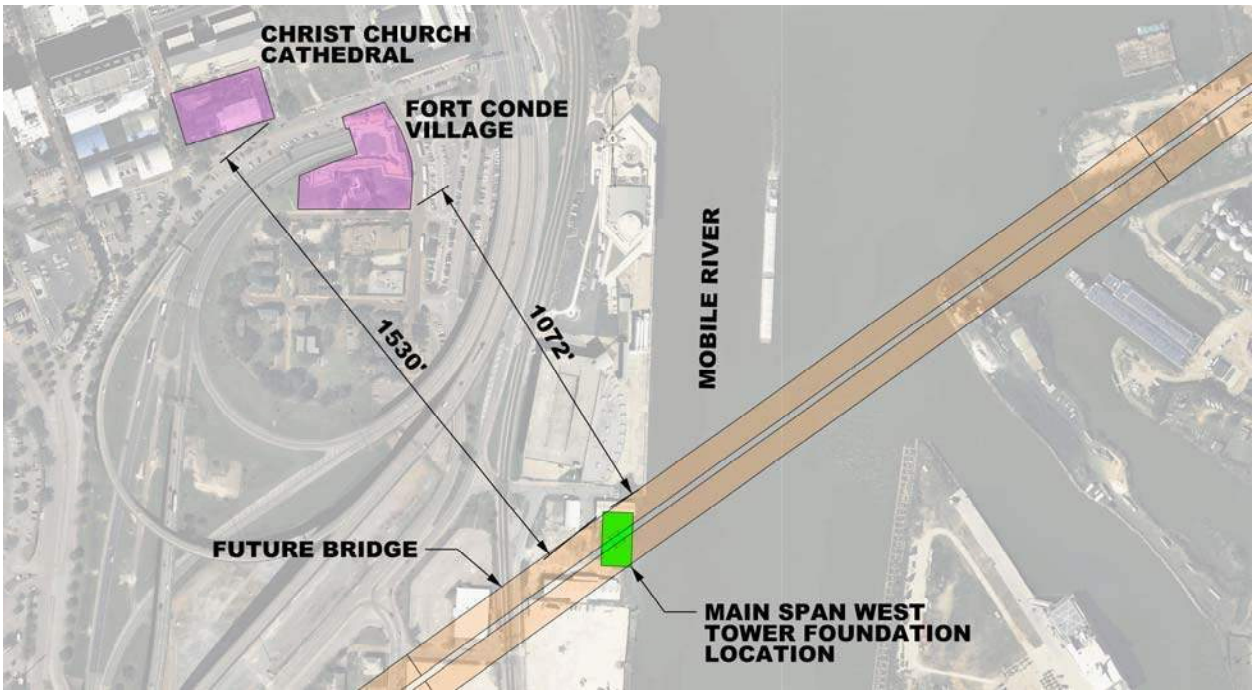


Figure 1– Distance to Historic Properties

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).

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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.

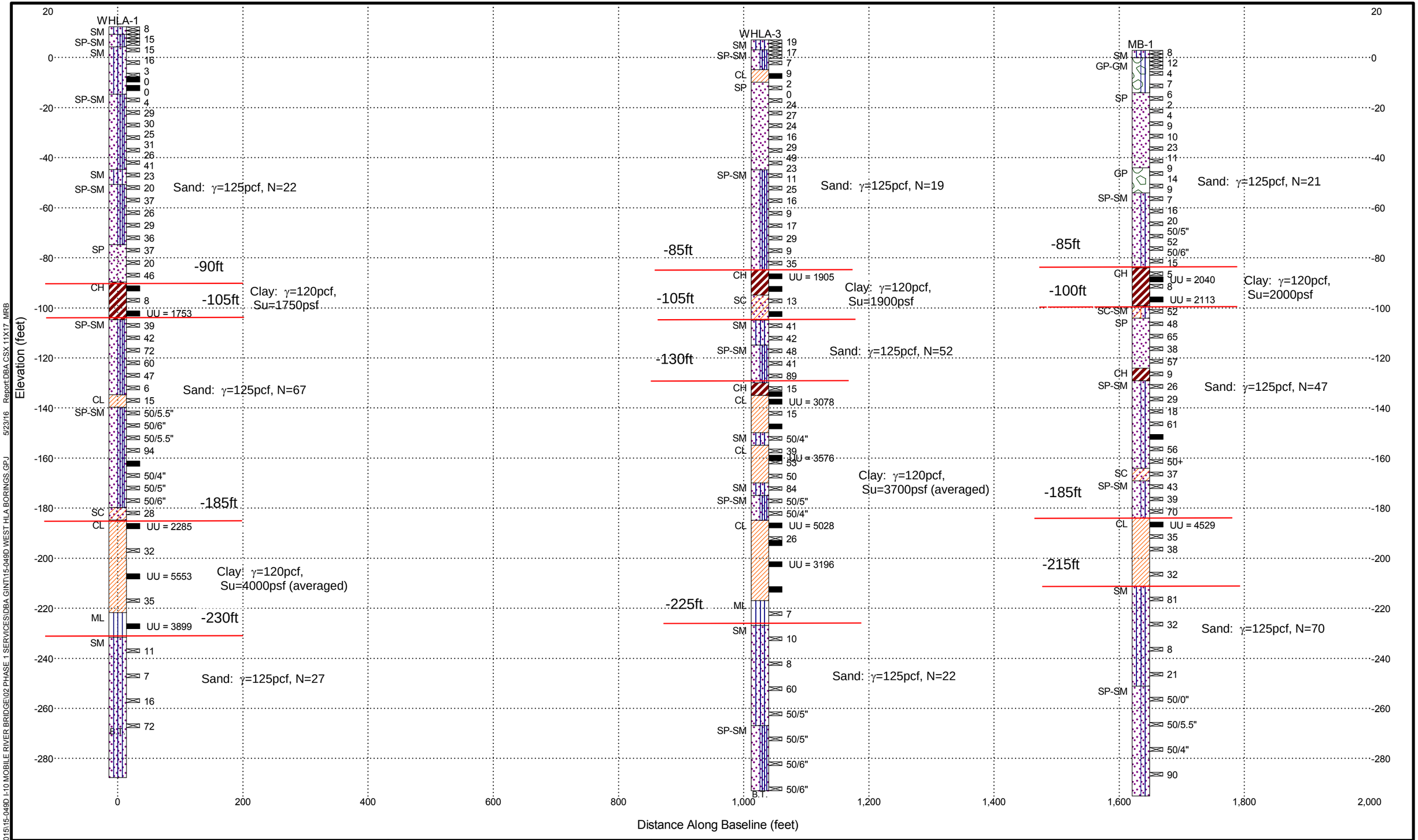
HP Piles

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
Qu (psi)
Material Symbol
N (bpf)
Recovery%/RQD%

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.

Z:\SHARED\PROJECTS\2015\15-049D I-10 MOBILE RIVER BRIDGE\02 PHASE 1 SERVICES\DBA GINT\15-049D WEST HLA BORINGS.GPJ 5/23/16 Report:DBA CSX 11X17 MRB

Appendix B – Main Span Conceptual Foundation Design Drawings

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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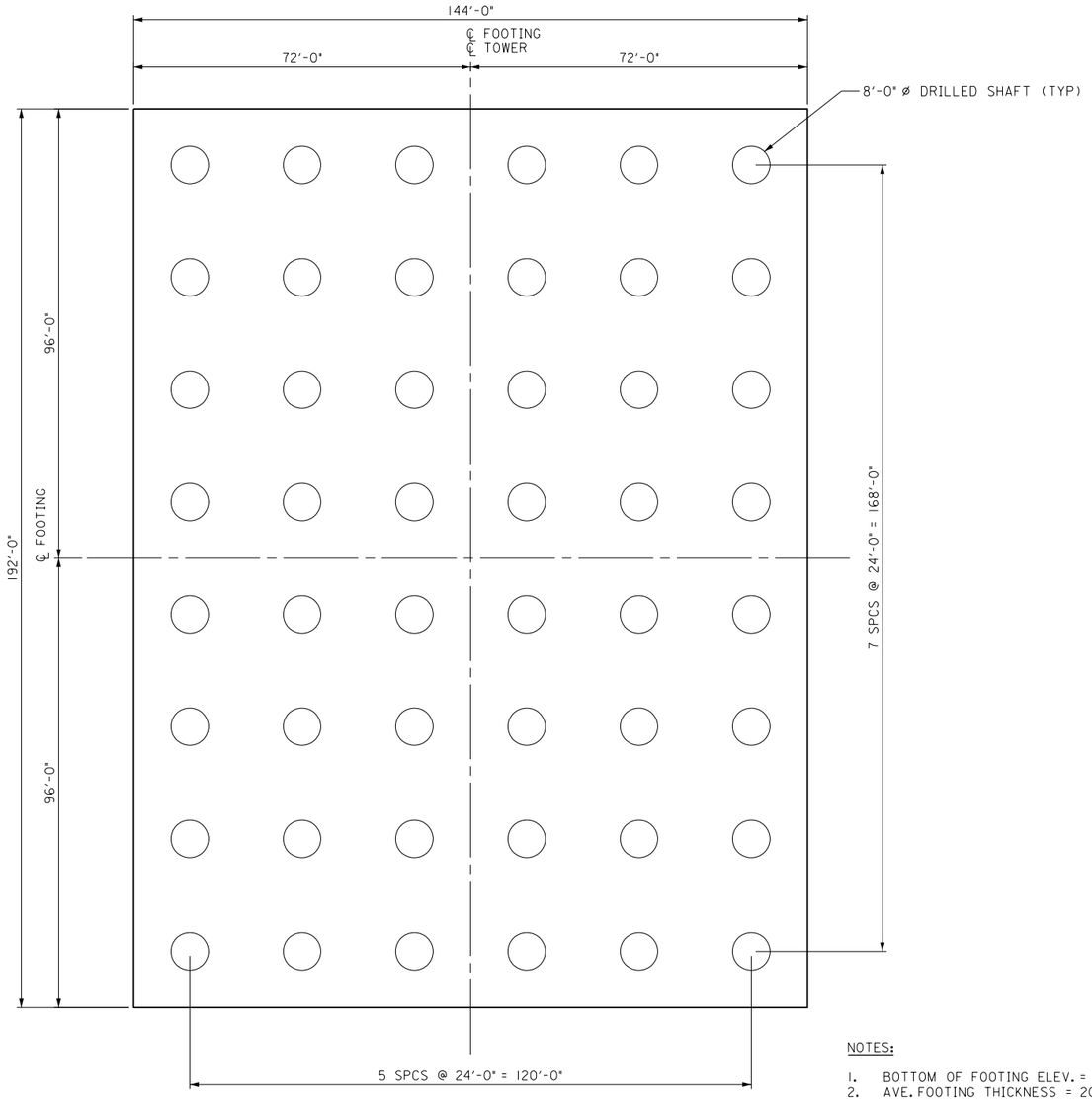
SCALE: 1:0.166666

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

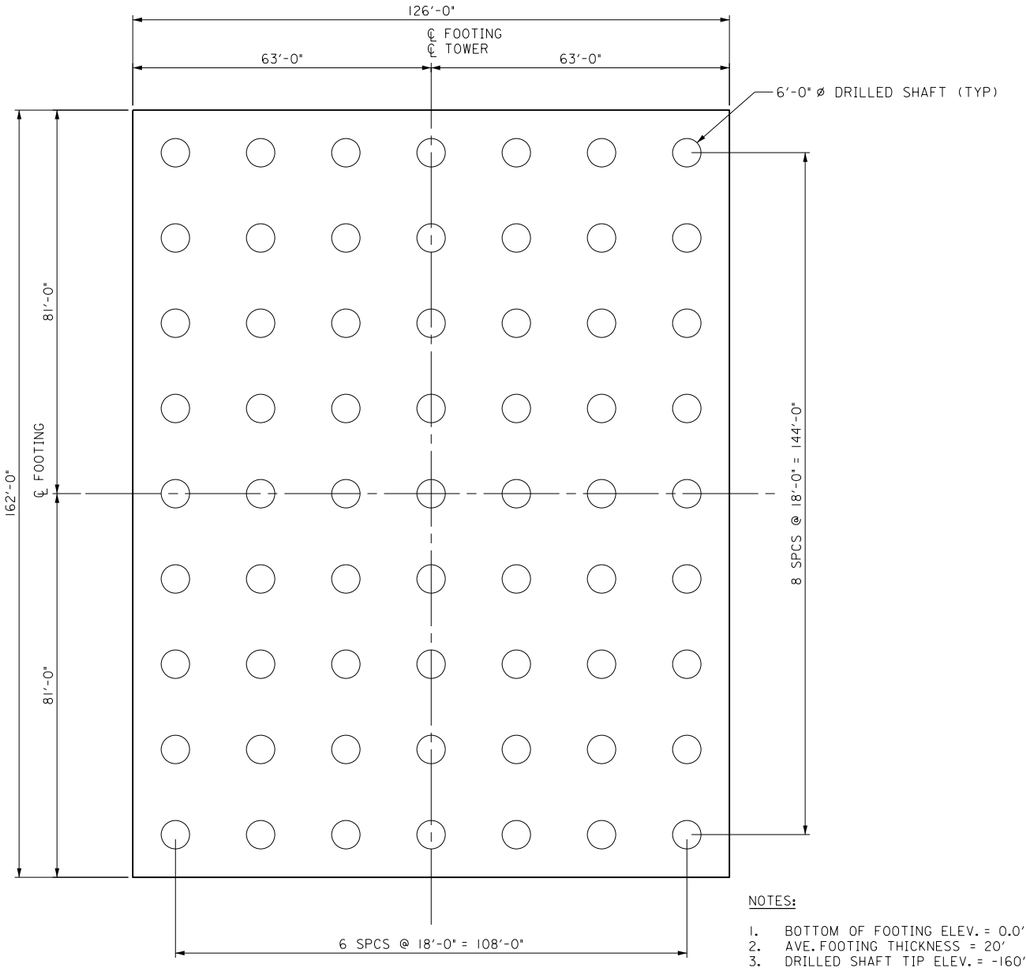


NOTES:

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

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)

MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 5	MOBILE RIVER BRIDGE WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS
REVISIONS	MAIN SPAN FOUNDATION OPTIONS (1 OF 5)
	ESTIMATED QUANTITIES / DESIGNED BY: JD / DRAWN BY: SRC
BIN:	COMPUTED BY: / CHECKED BY: / DATE DRAWN:
	VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

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USER: scouture
FILE: ALDOT_Headquarters@Mobile_River_Bridge_Phase_I@5.0_Project_Development@5.2_Work_In_Progress@Project_Design@Plans_Assembly@Prelim Structure Type Selection@MRB_MAIN_Concept_Foundations_Edge Girder.dgn
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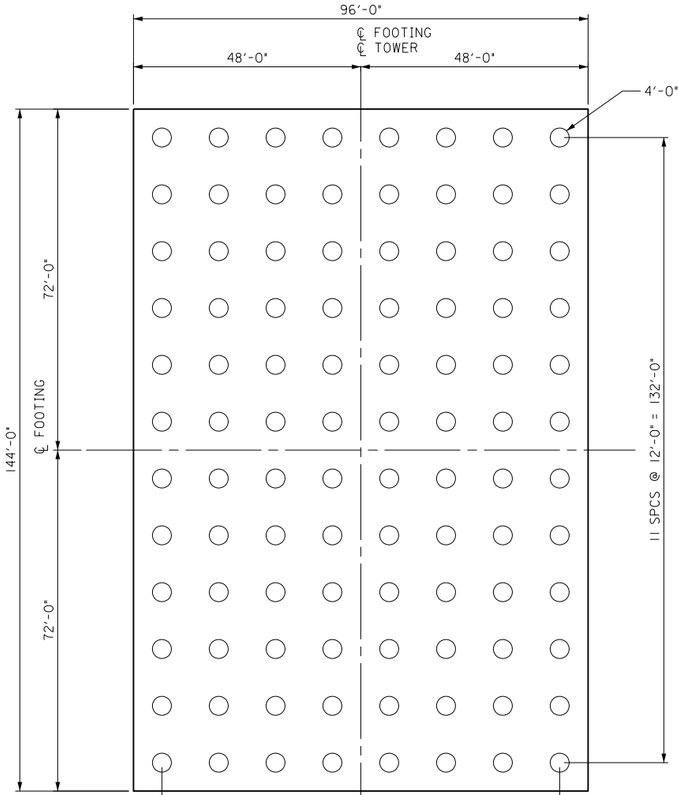
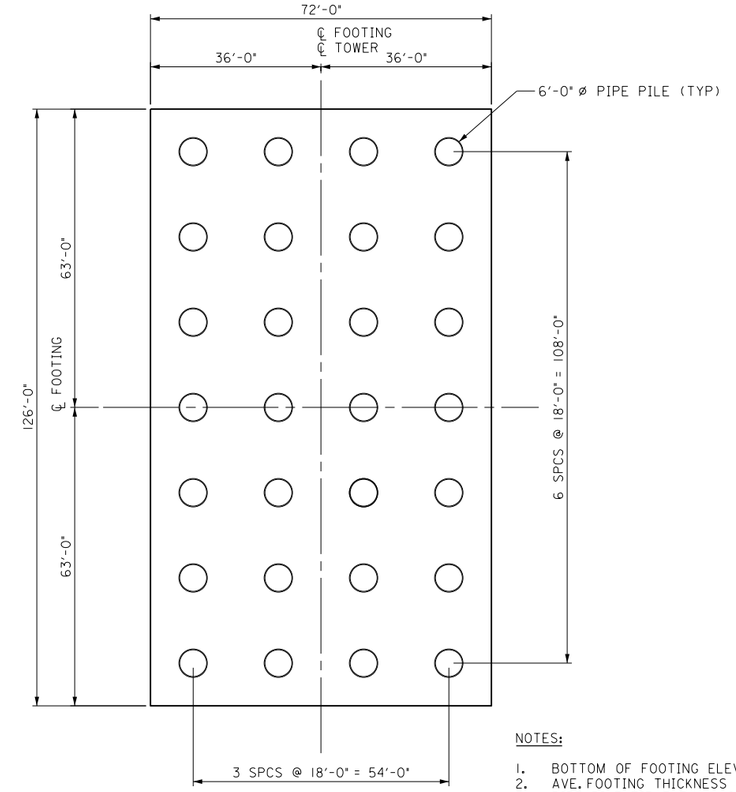
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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

BIN:

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

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.plotcfg
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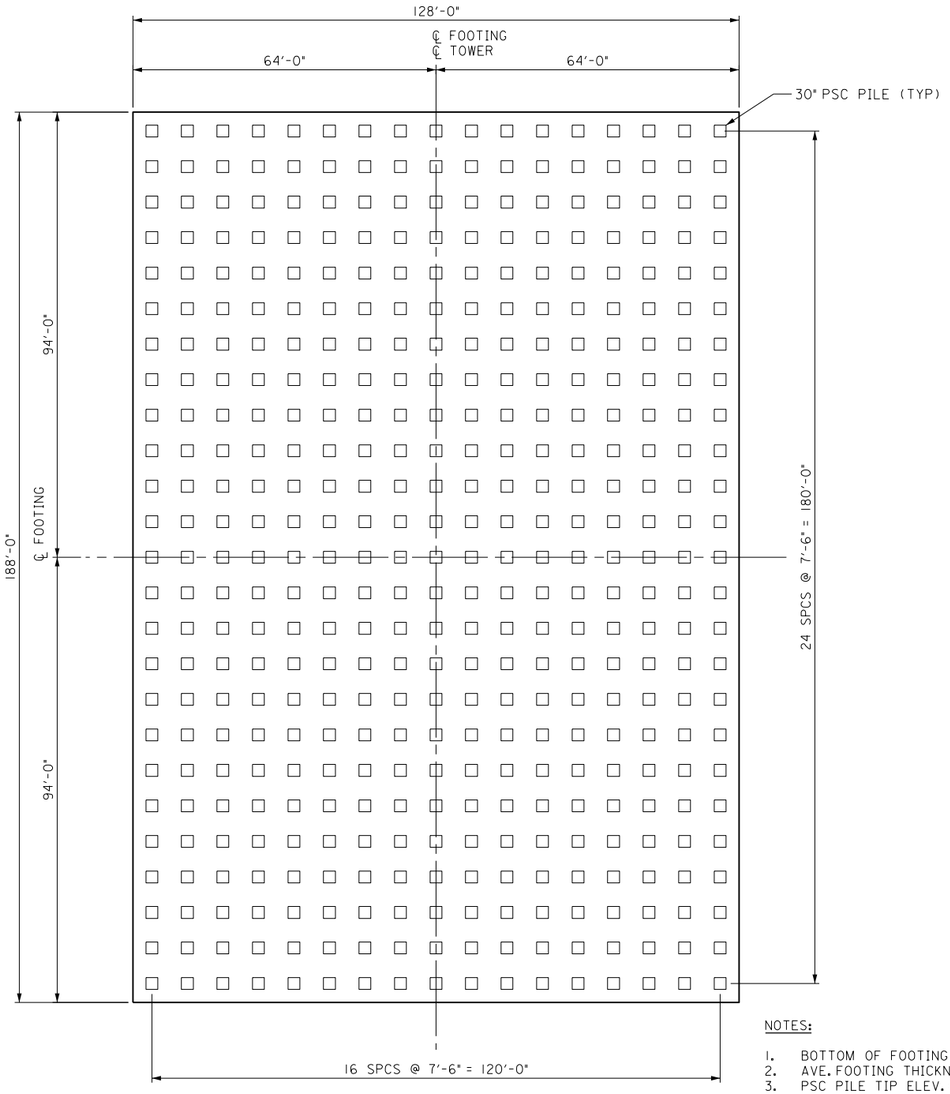
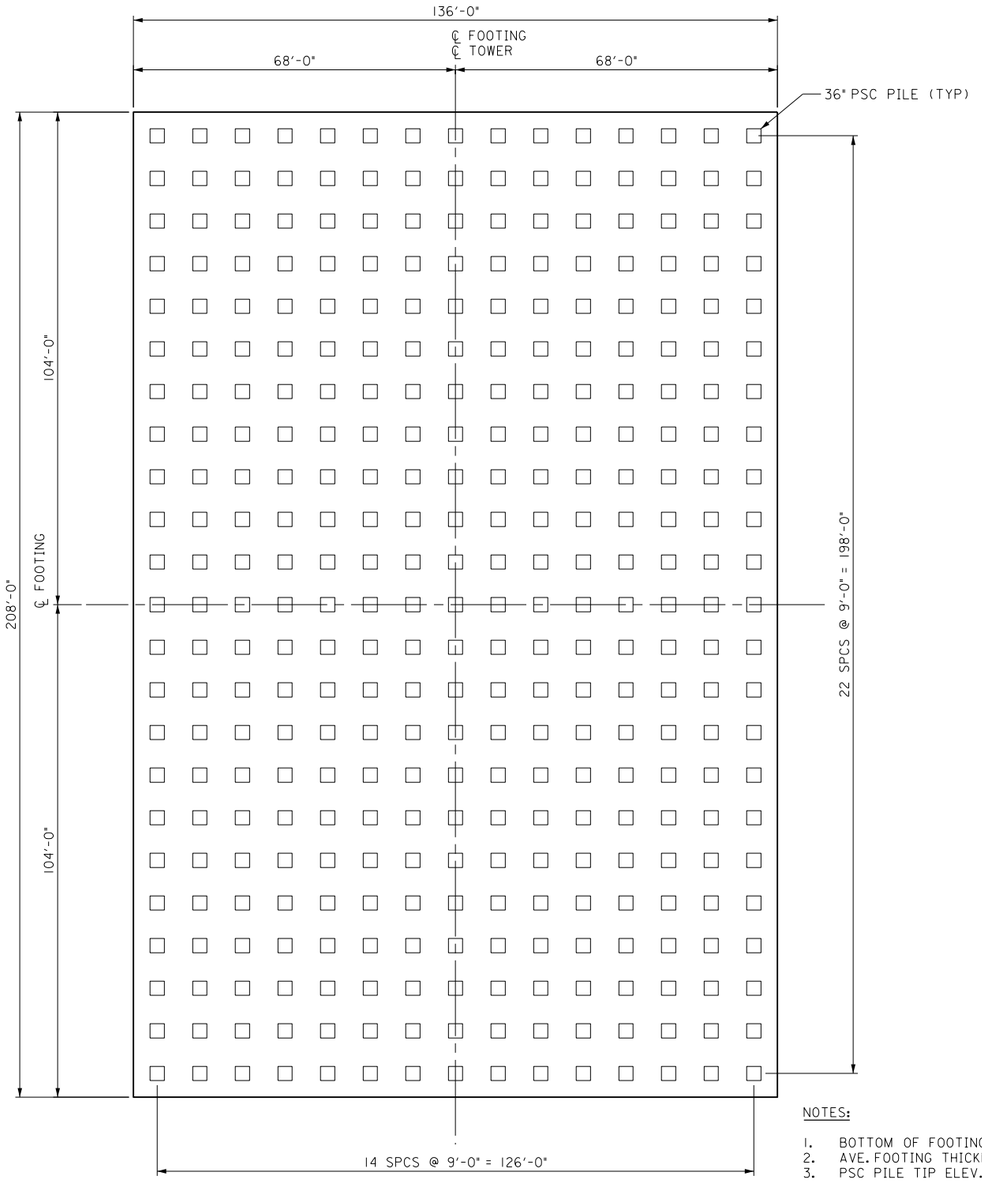
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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

BIN:

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

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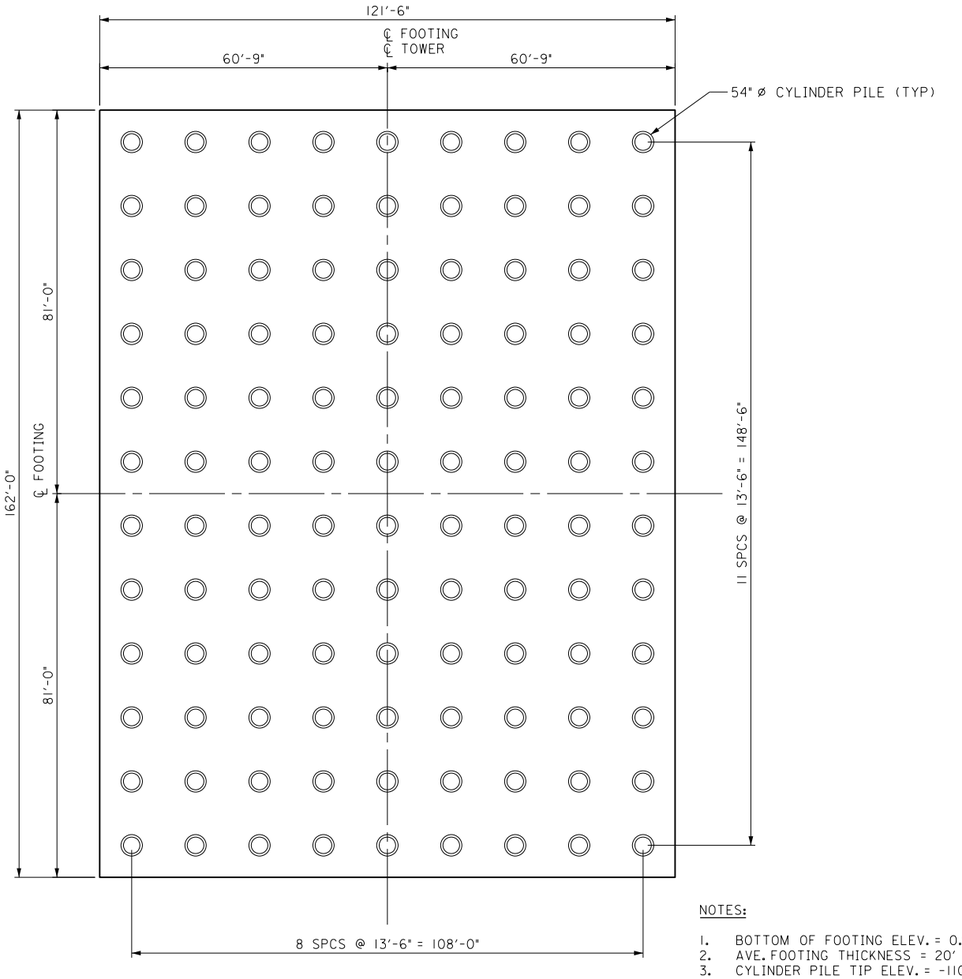
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

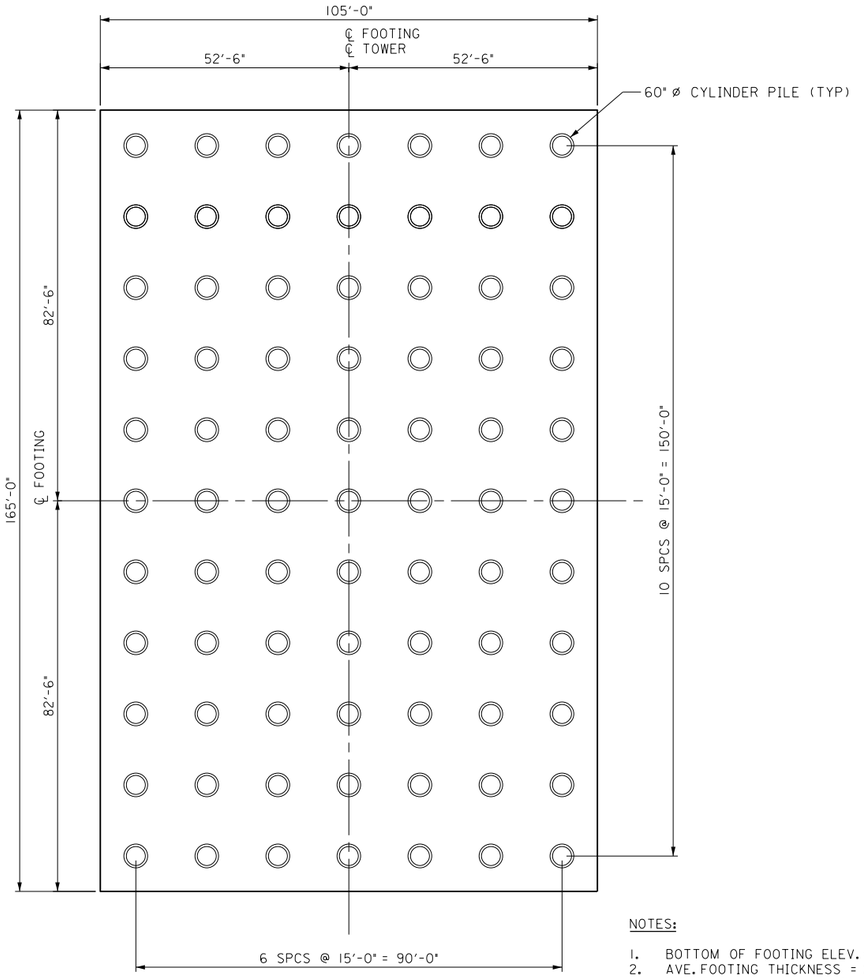


NOTES:

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

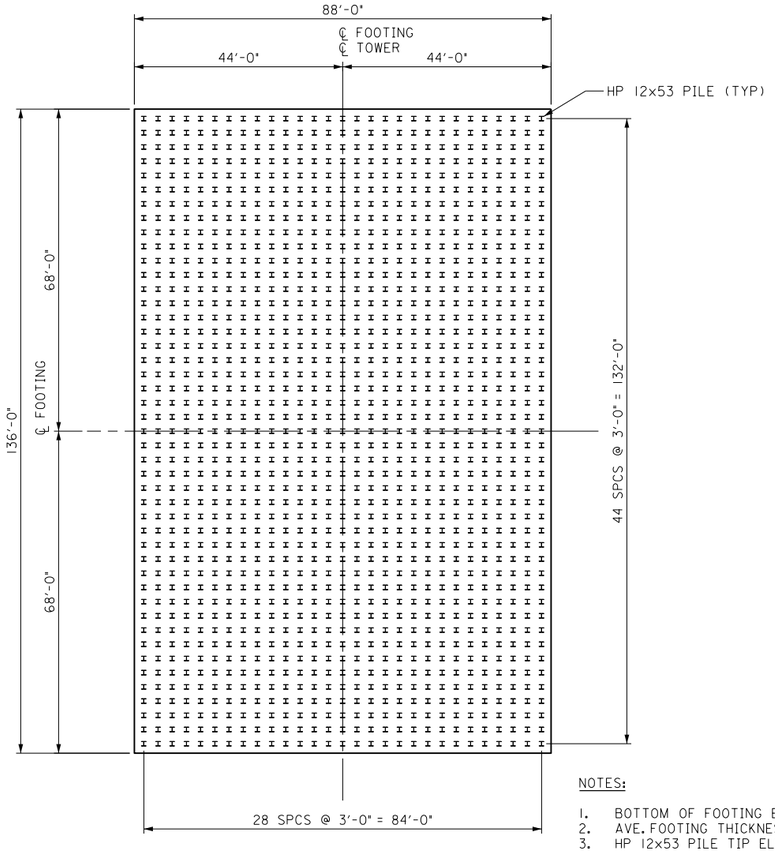
REVISIONS

BIN:

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)

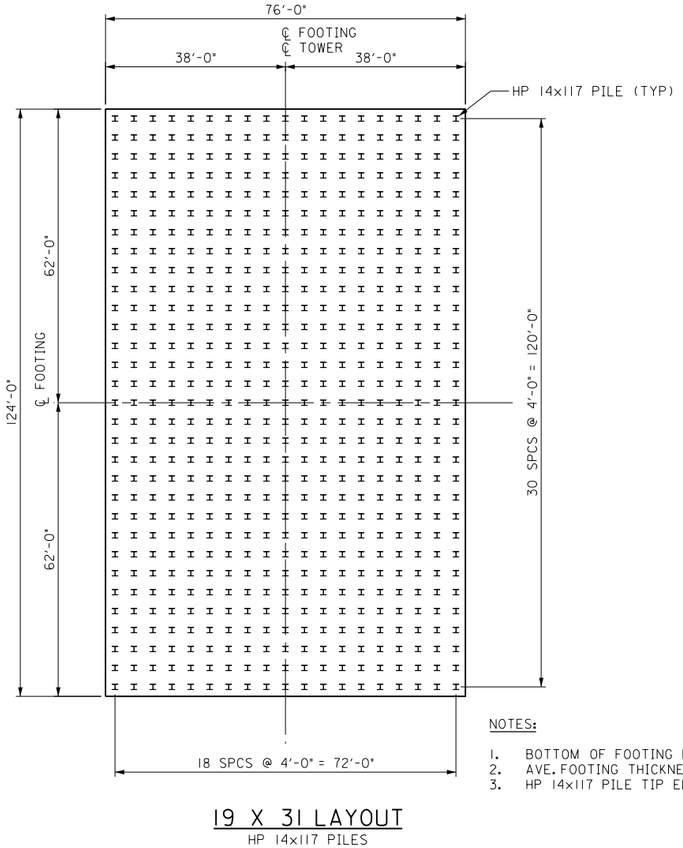
ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED



29 X 45 LAYOUT
HP 12x53 PILES
(1305 TOTAL)

NOTES:

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



19 X 31 LAYOUT
HP 14x117 PILES
(589 TOTAL)

NOTES:

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

REVISIONS

BIN:

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

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

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PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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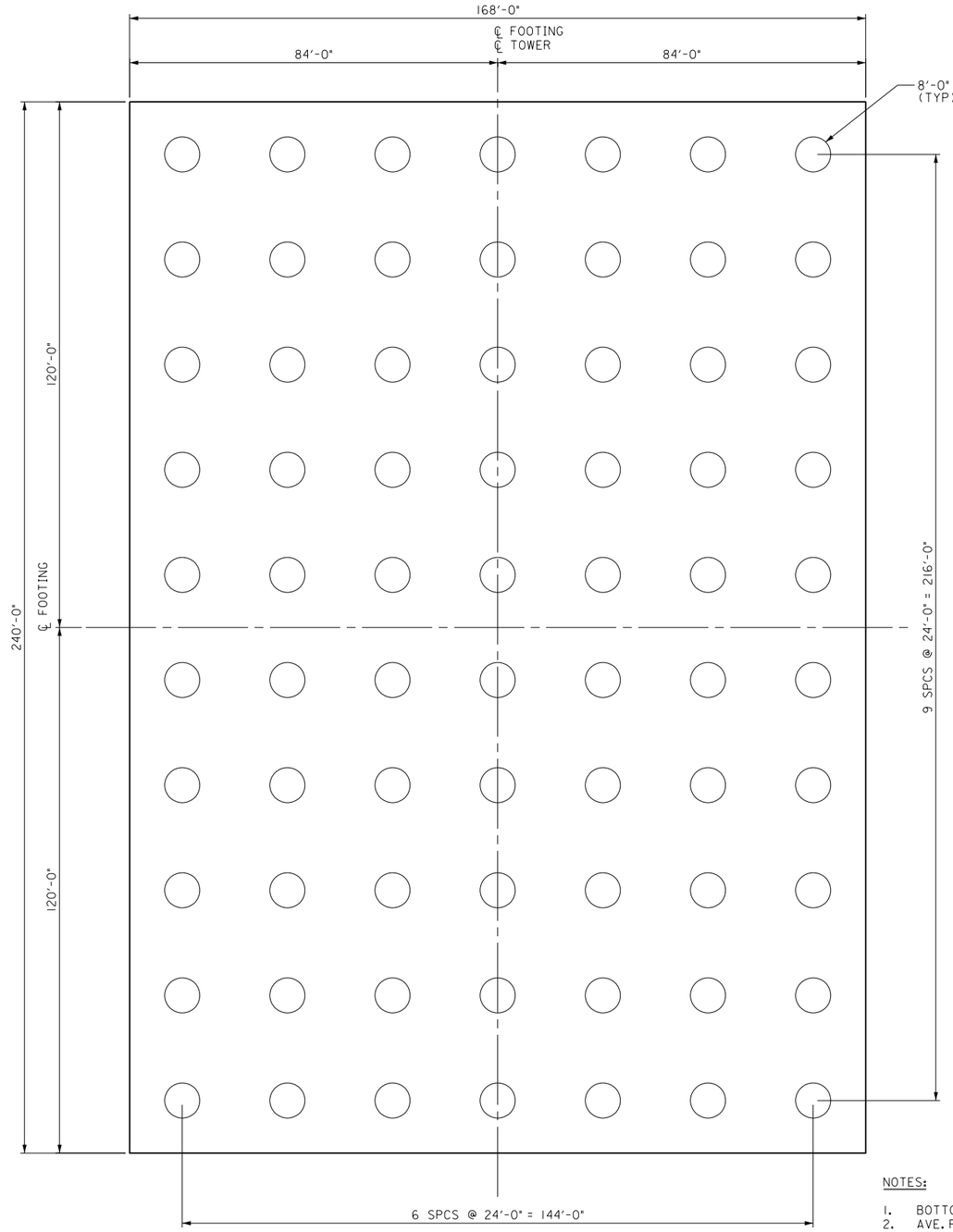
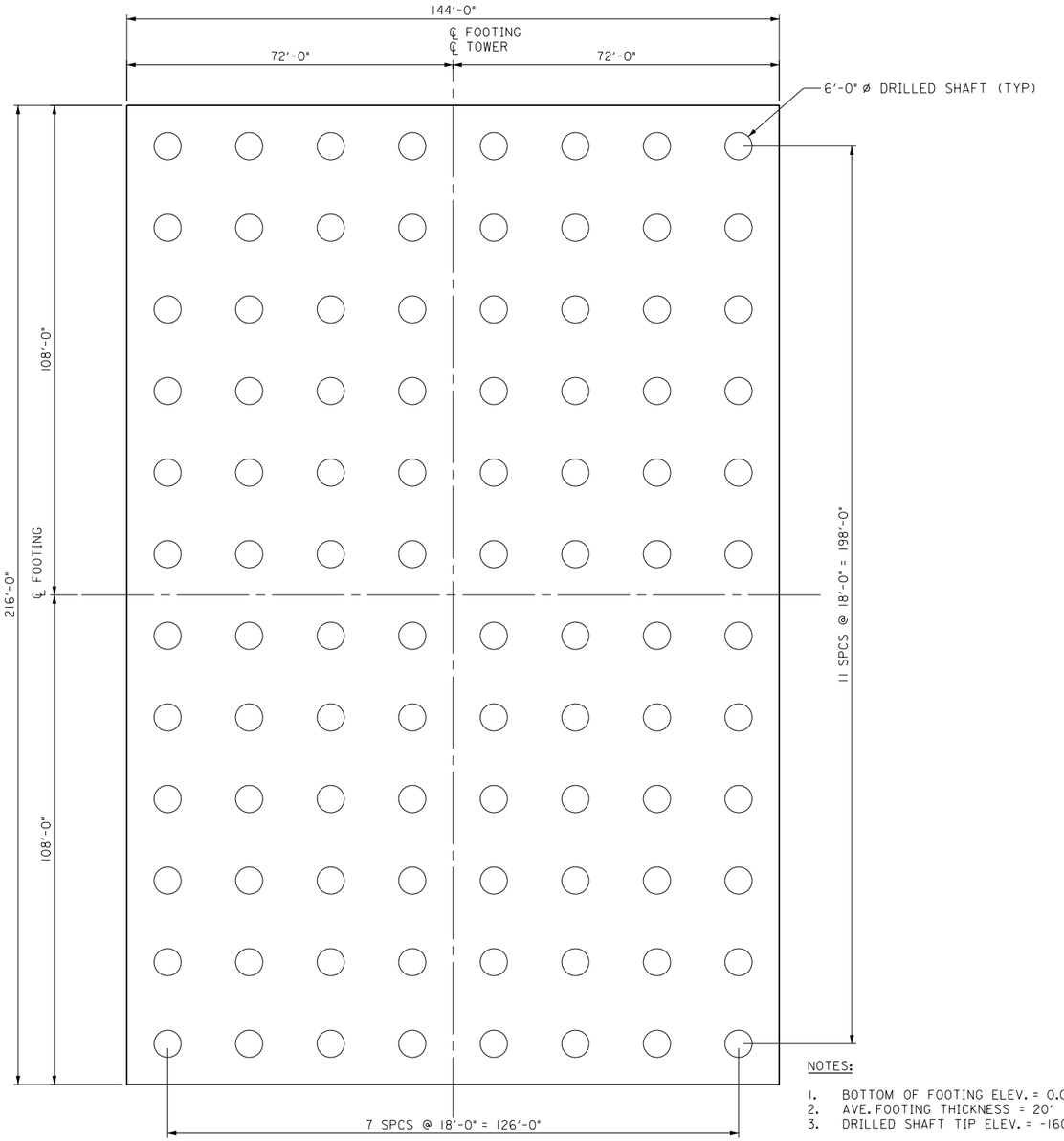
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER



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

BIN:

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

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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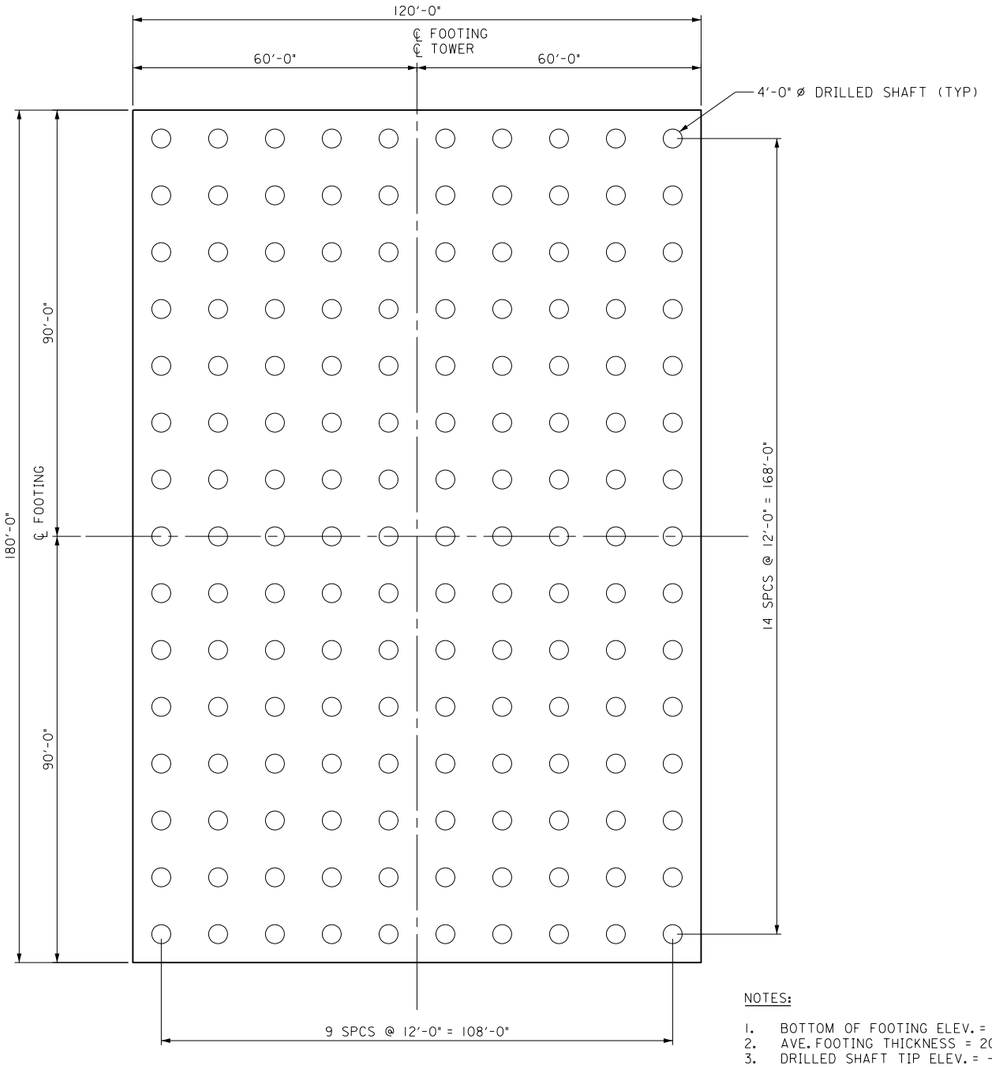
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

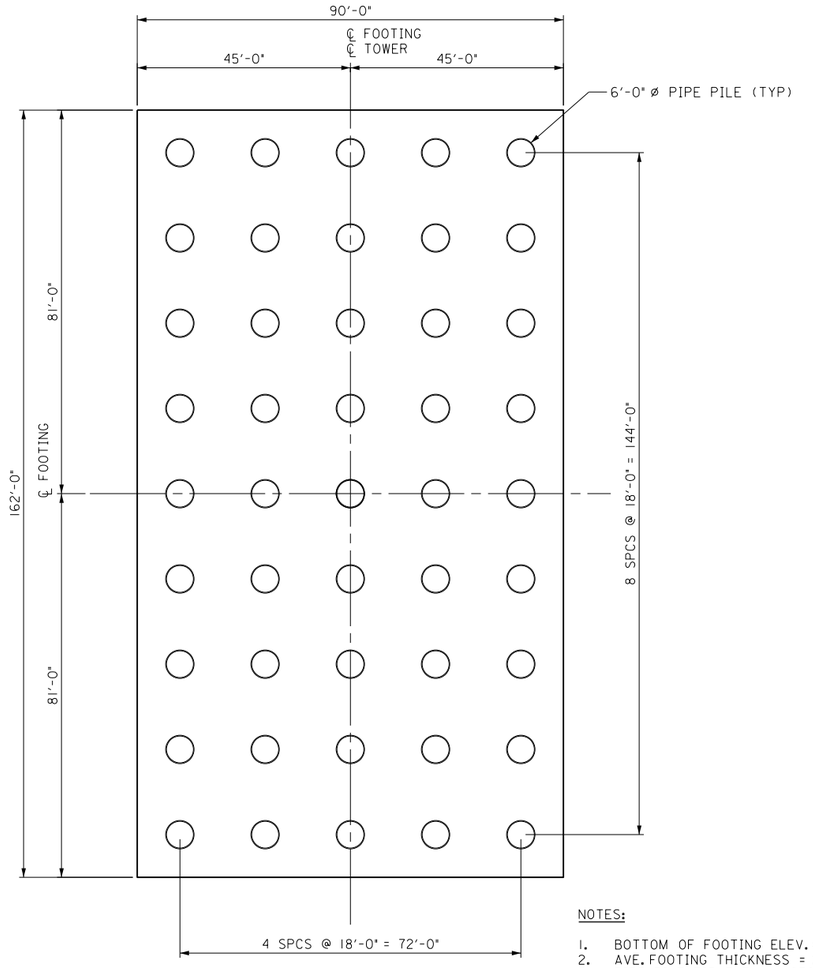


NOTES:

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

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

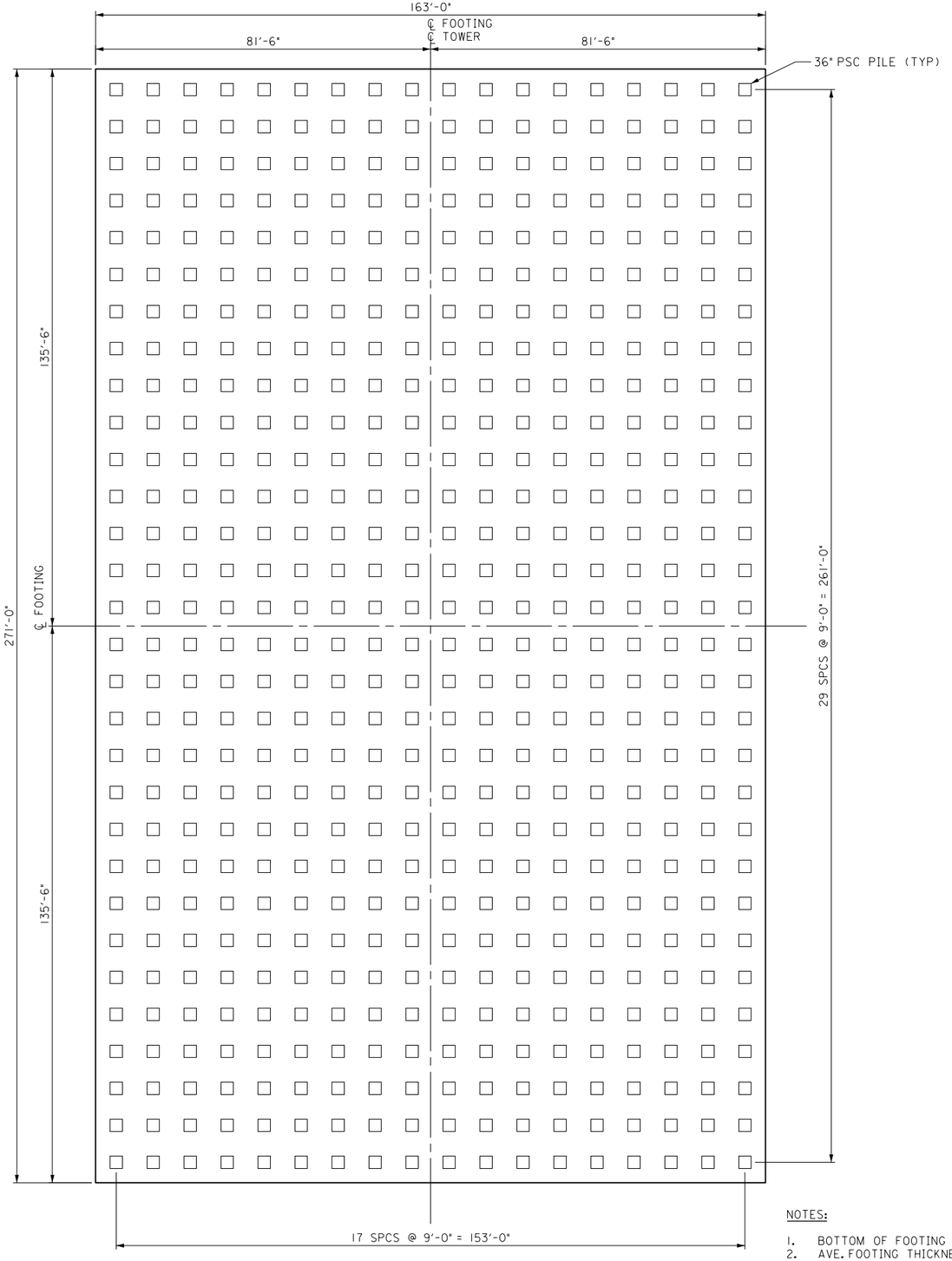
REVISIONS

BIN:

MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS

MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

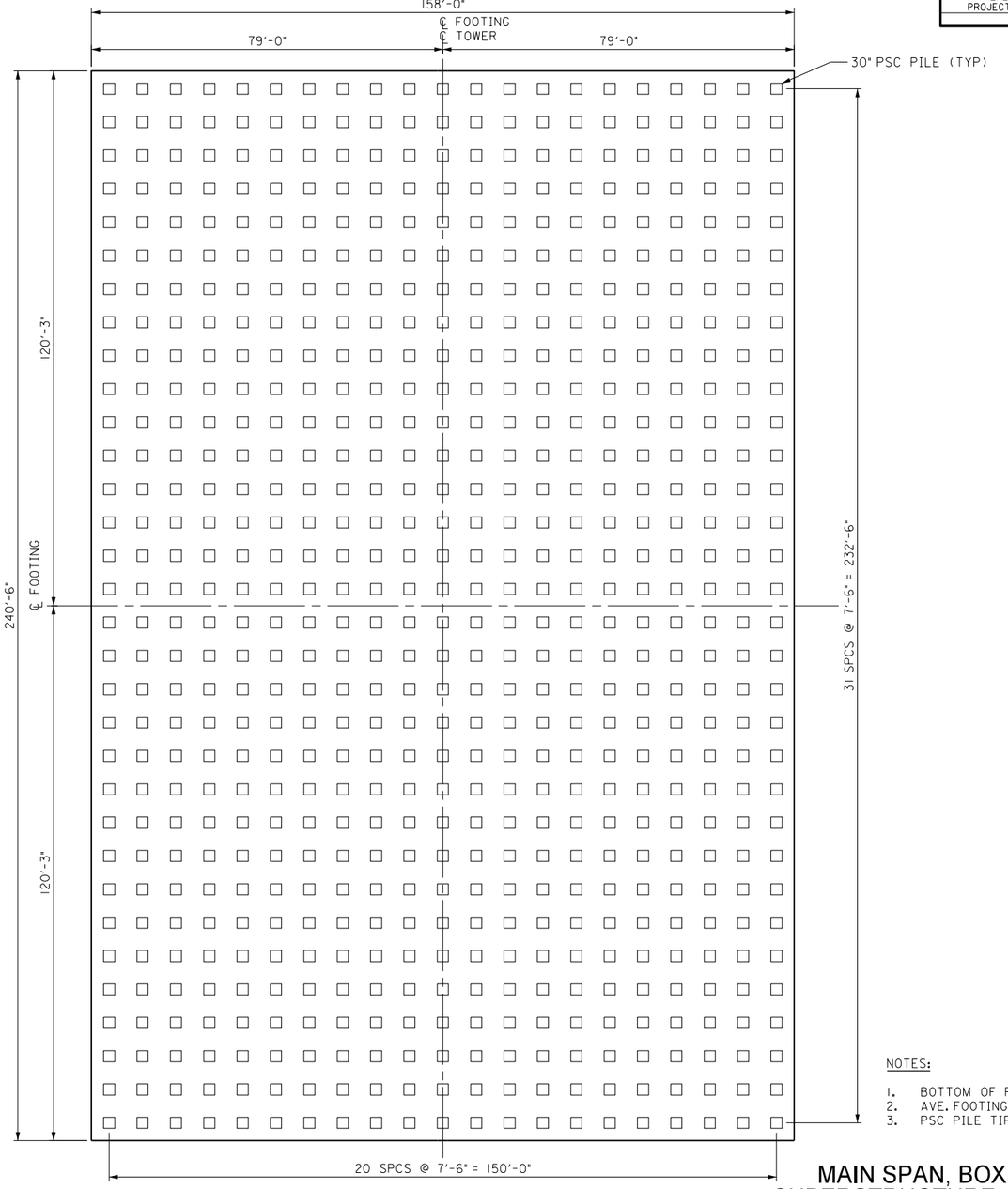
ESTIMATED QUANTITIES	DESIGNED BY: JD	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED



NOTES:

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

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)

MAIN SPAN, BOX SECTION SUPERSTRUCTURE ALTERNATIVE

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 5

REVISIONS

BIN:

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

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PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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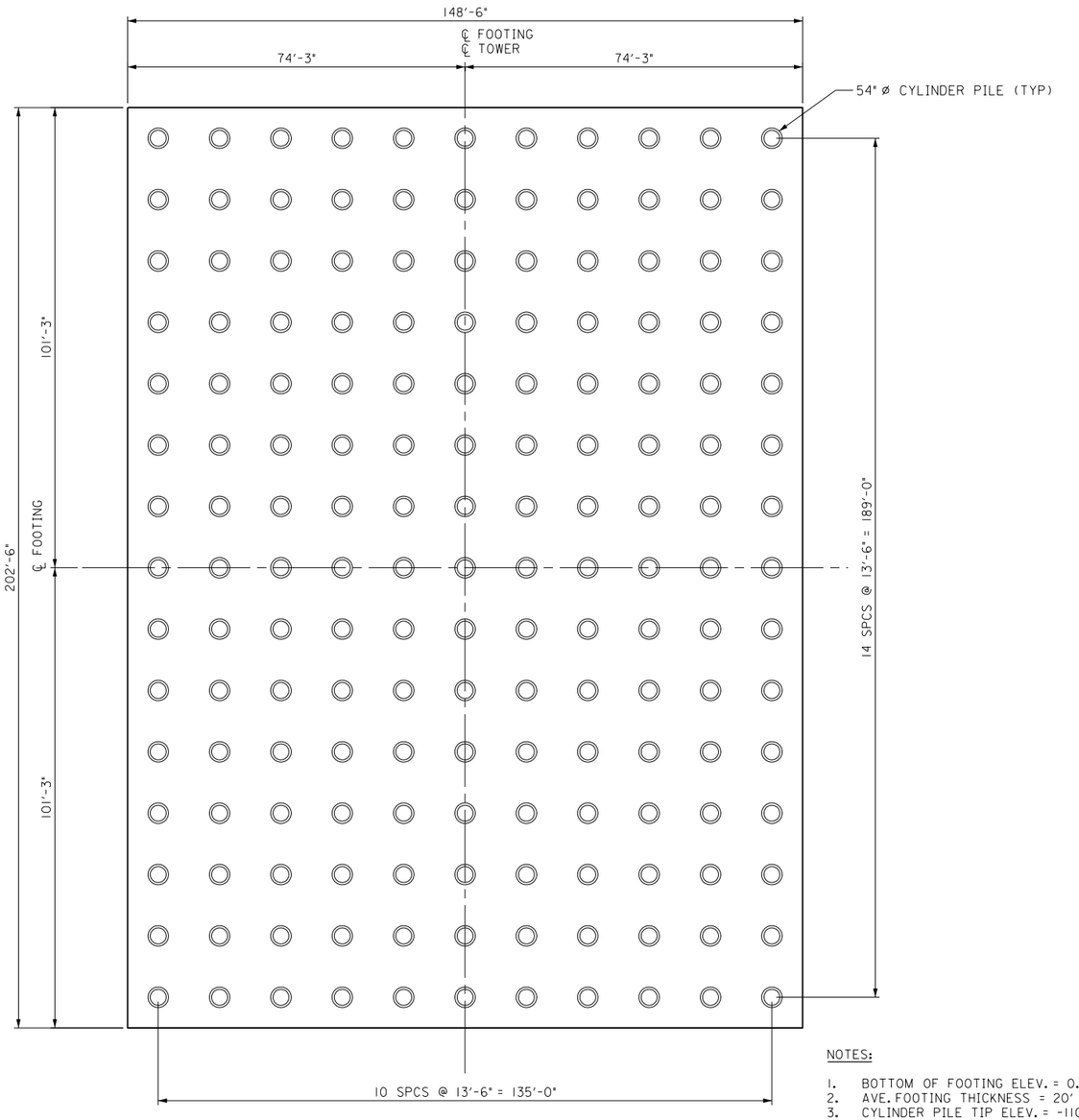
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

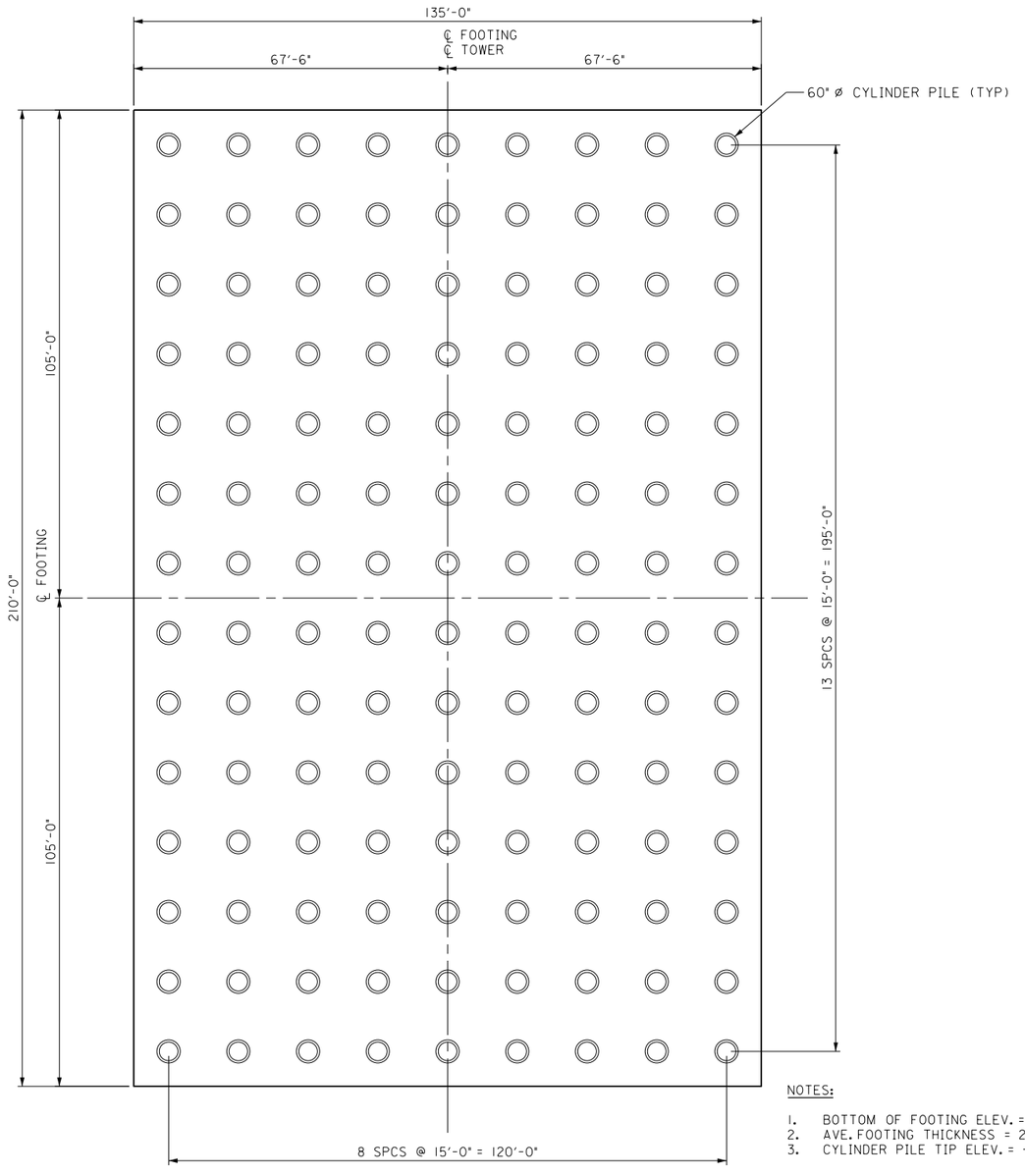


NOTES:

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

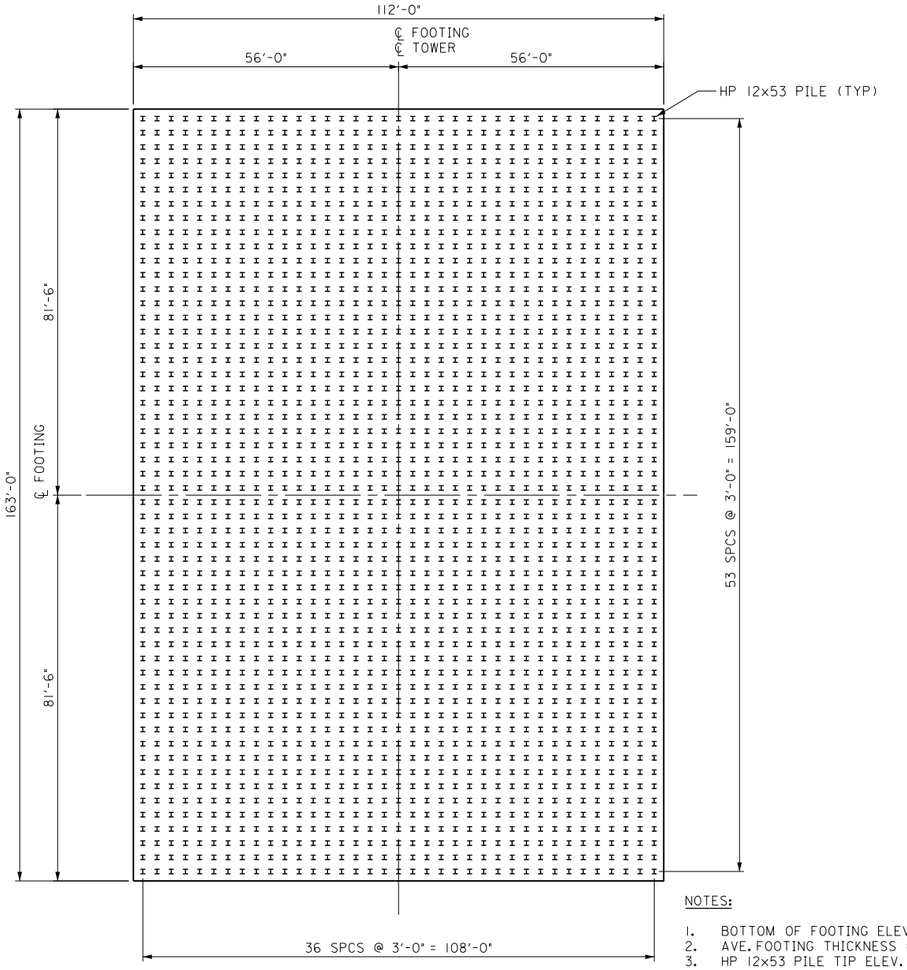
REVISIONS

BIN:

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

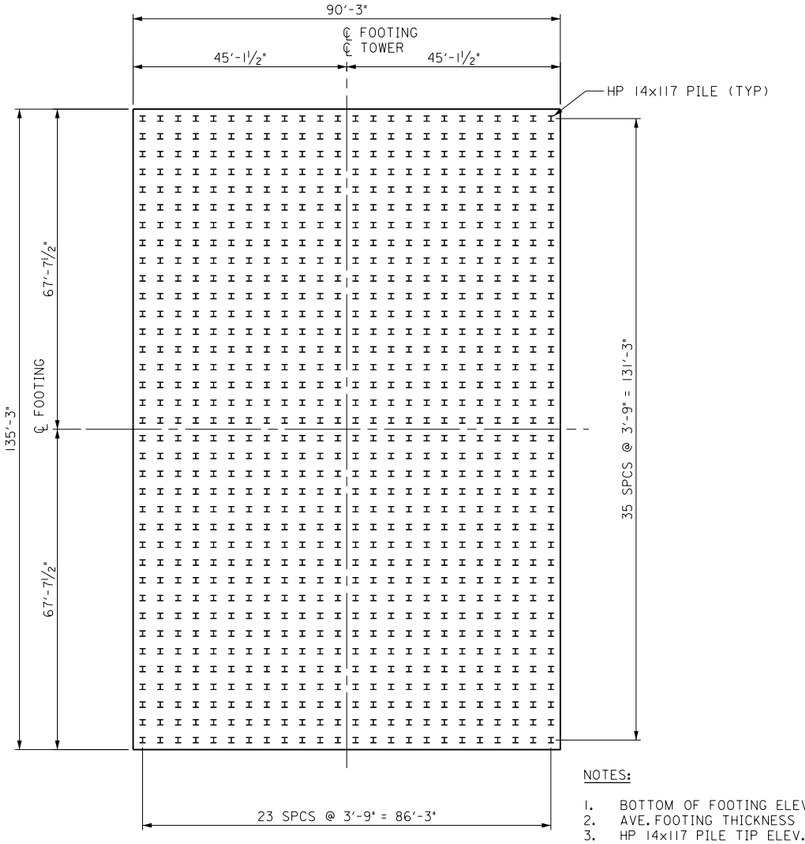


NOTES:

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)

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

MAIN SPAN, BOX SECTION SUPERSTRUCTURE ALTERNATIVE

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 5

REVISIONS

BIN:

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

MRB_MAIN_Concept_Foundation_Box.dgn

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: scouture
FILE: ALDOT_Headquarters@Mobile_River_Bridge_Phase_I@5.0_Project_Development@5.2_Work_In_Progress@Project_Design@Plans_Assembly@Prelim Structure Type Selection@MRB_MAIN_Concept_Foundation_Box.dgn
PENTABLE: HDR_ALDOT.tbl
DATE: Wednesday, June 29, 2016 11:07:29 AM
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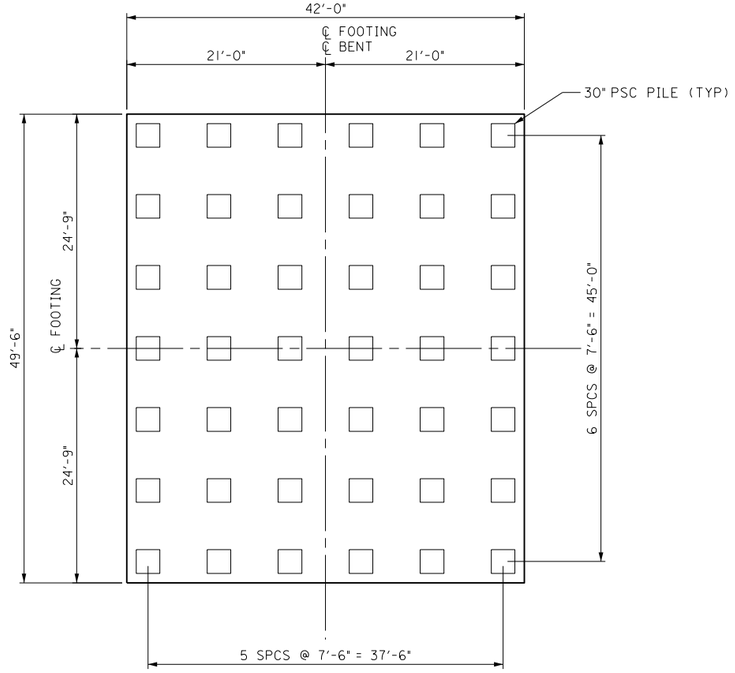
Appendix C - High Level Approach Spans Conceptual Foundation Design Drawings

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

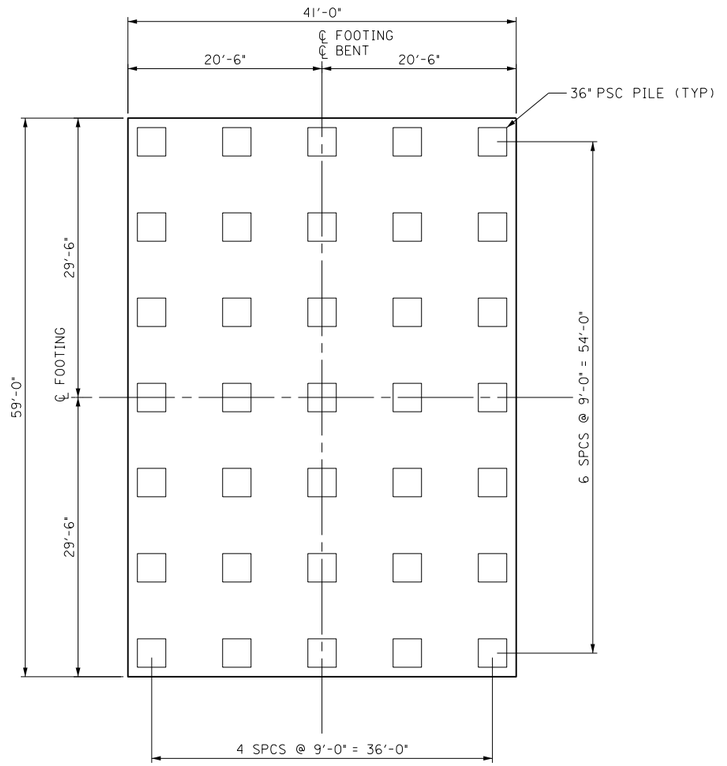


6 X 7 LAYOUT

30" PSC PILES
(42 TOTAL)

NOTES:

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

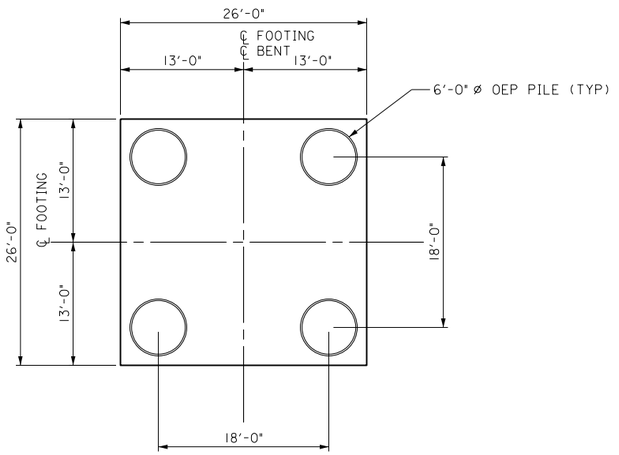


5 X 7 LAYOUT

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" ø 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)
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: RH / DRAWN BY: JRQ COMPUTED BY: / CHECKED BY: / DATE DRAWN: VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

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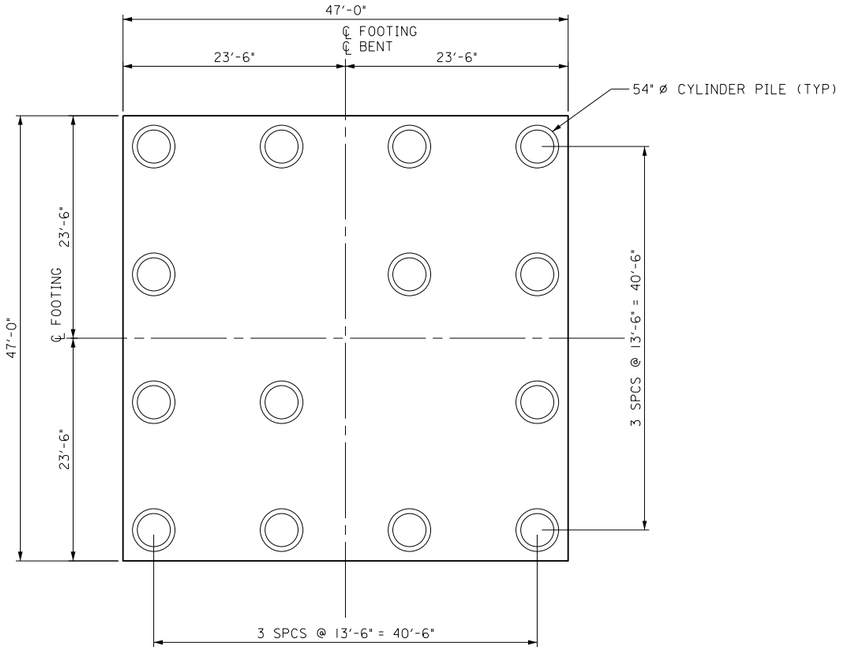
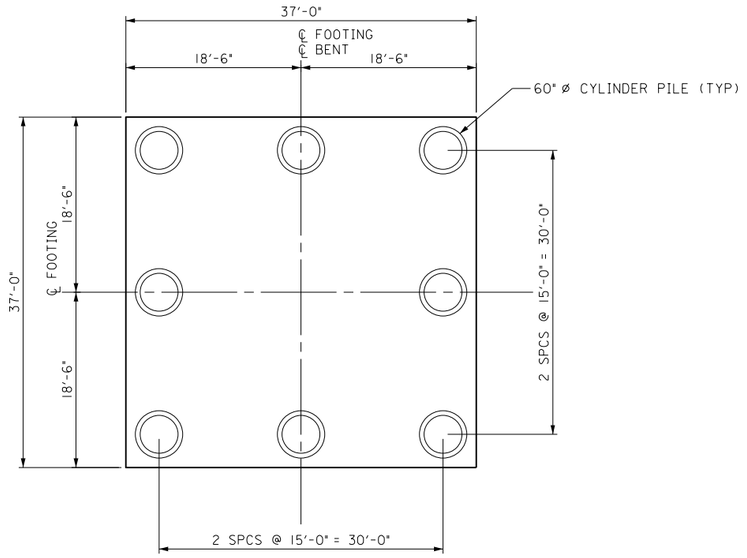


4 X 4 LAYOUT
54" ø CYLINDER PILES
(14 TOTAL)

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: JRQ
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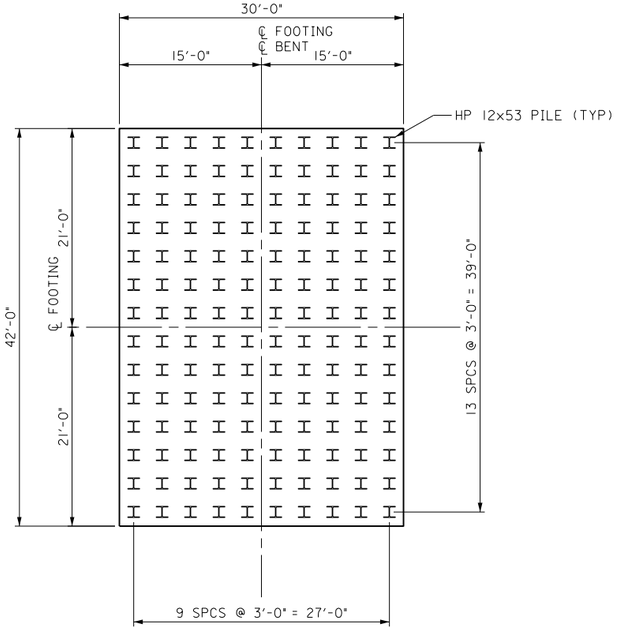
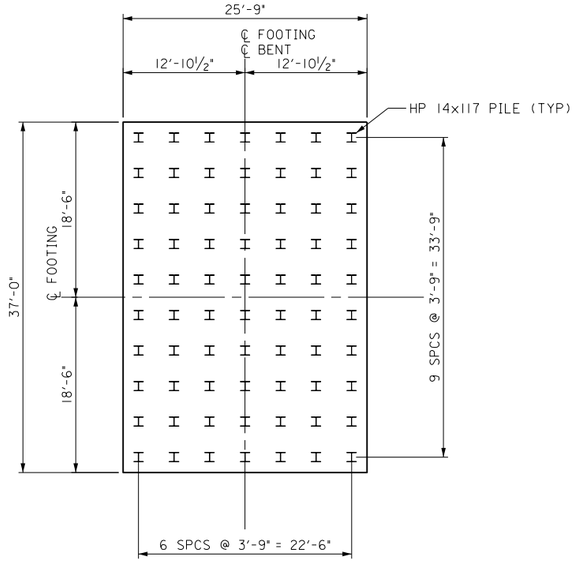


10 X 14 LAYOUT

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	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS
REVISIONS	HIGH LEVEL APPR FOUNDATION OPTIONS (3 OF 3)
	ESTIMATED QUANTITIES / DESIGNED BY: RH / DRAWN BY: JRQ
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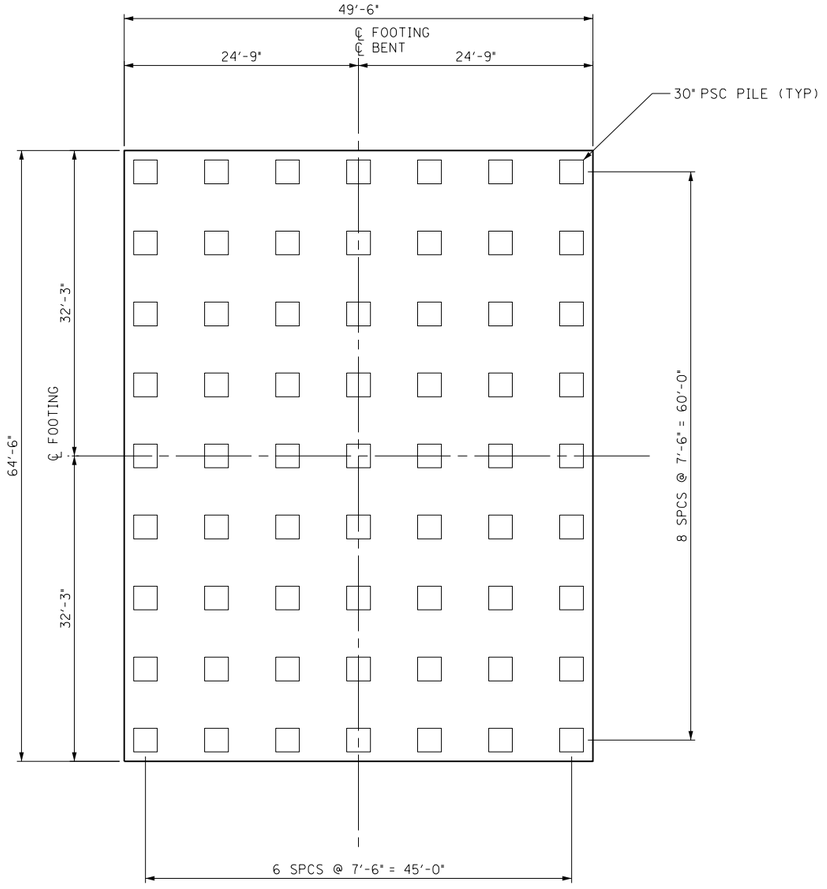
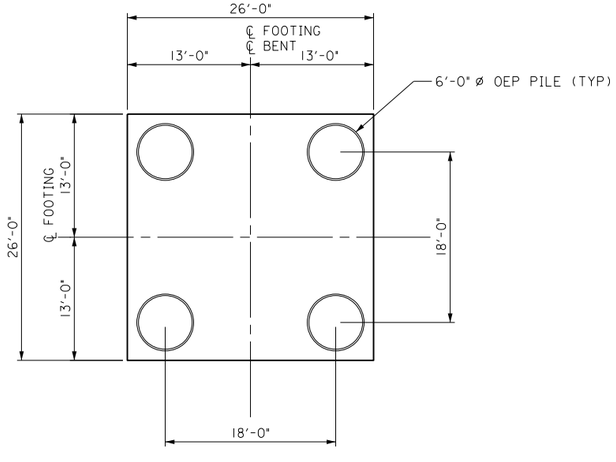


7 X 9 LAYOUT

30" PSC PILES
(63 TOTAL)

NOTES:

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

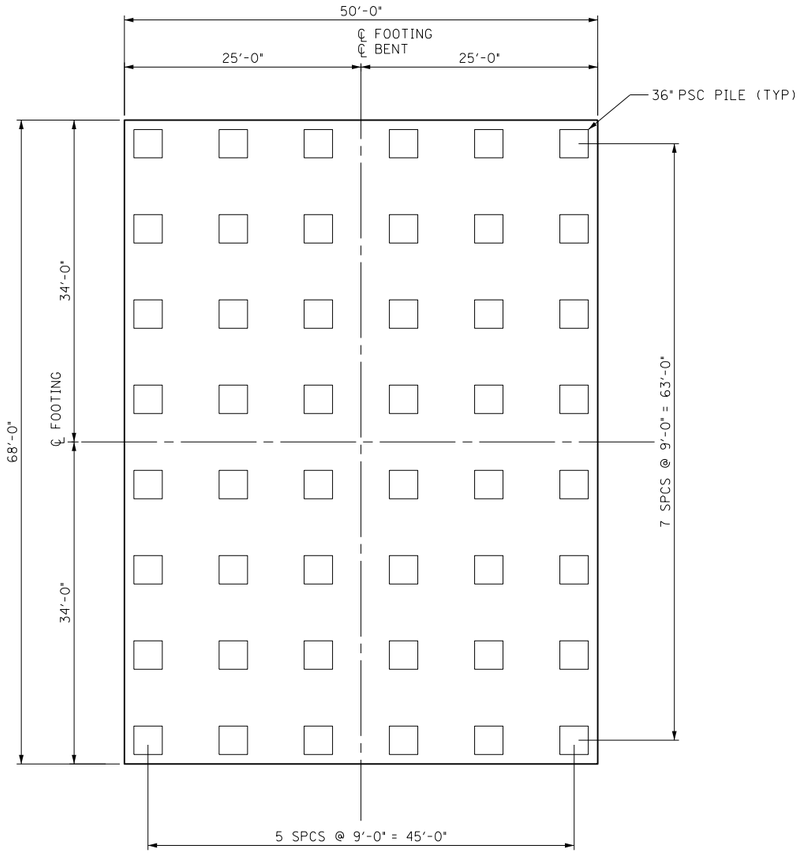


3 X 3 LAYOUT

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'



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'

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

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)
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: RH / DRAWN BY: JRQ COMPUTED BY: / CHECKED BY: / DATE DRAWN: VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

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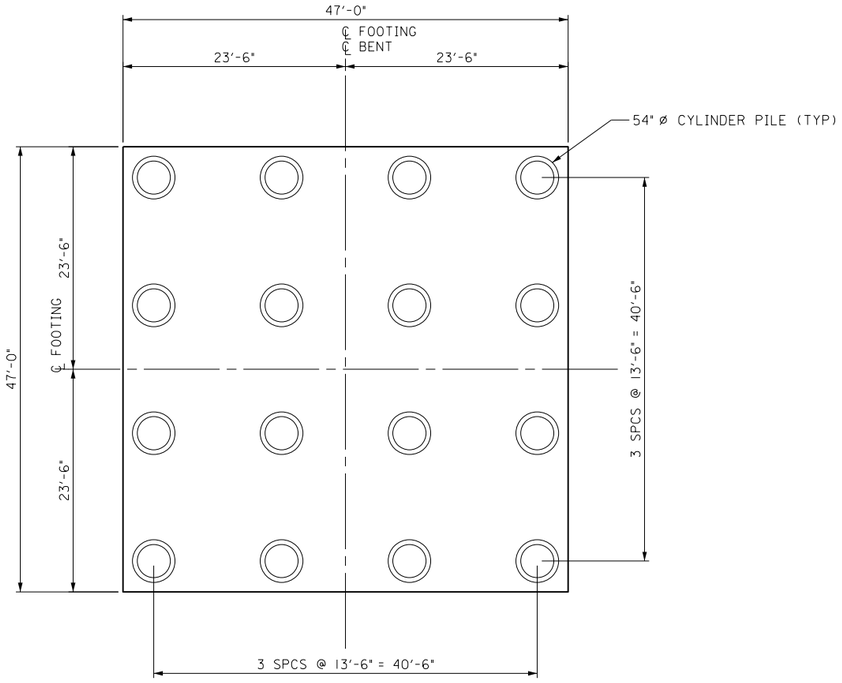
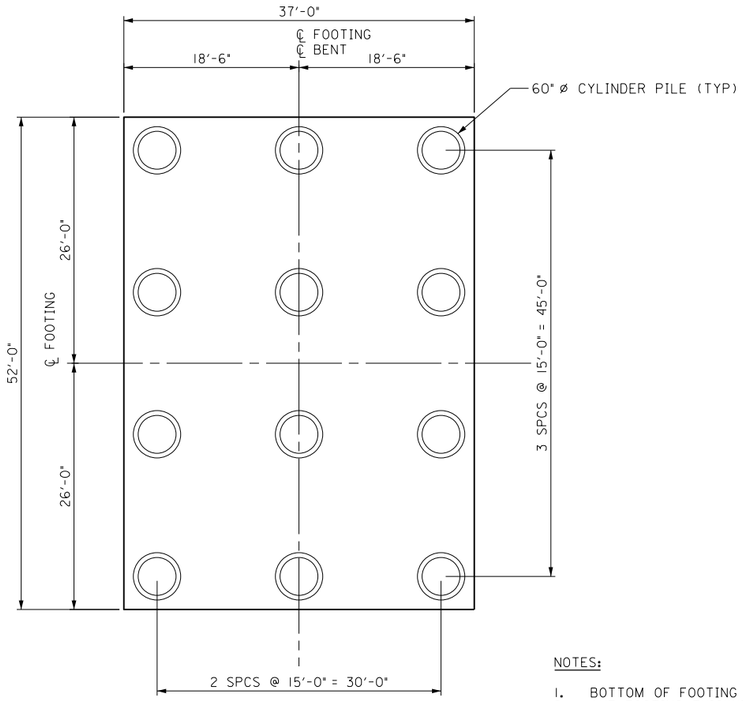


4 X 4 LAYOUT
54" ø CYLINDER PILES
(16 TOTAL)

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'

PRESTRESSED BULB TEE LG-78
200' HEIGHT ALTERNATIVE

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

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 3	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS
REVISIONS	HIGH LEVEL APPR FOUNDATION OPTIONS (2 OF 3)
	ESTIMATED QUANTITIES / DESIGNED BY: RH / DRAWN BY: JRQ
BIN:	COMPUTED BY: / CHECKED BY: / DATE DRAWN:
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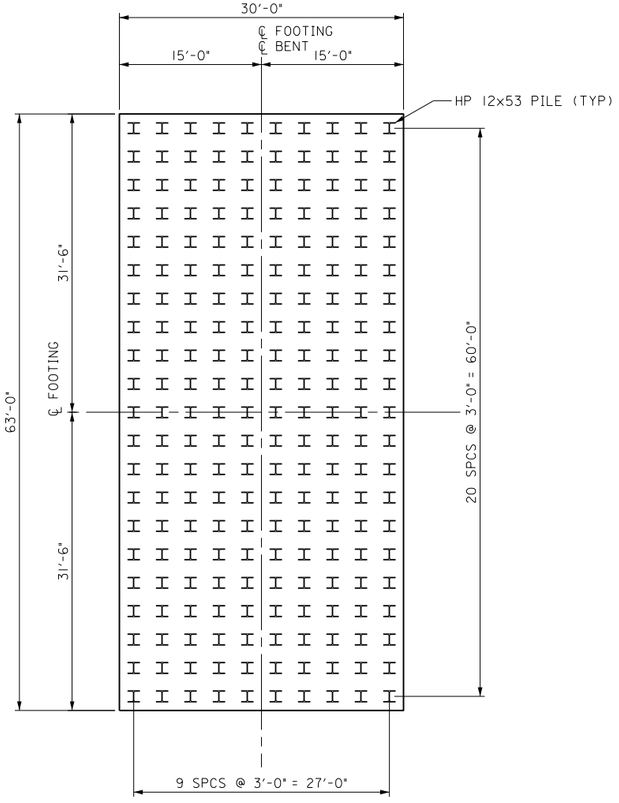
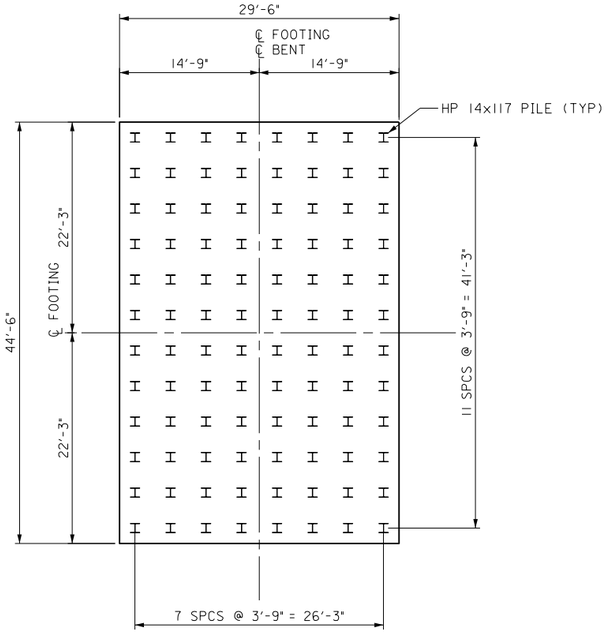


10 X 21 LAYOUT

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	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS
REVISIONS	HIGH LEVEL APPR FOUNDATION OPTIONS (3 OF 3)
	ESTIMATED QUANTITIES / DESIGNED BY: RH / DRAWN BY: JRQ
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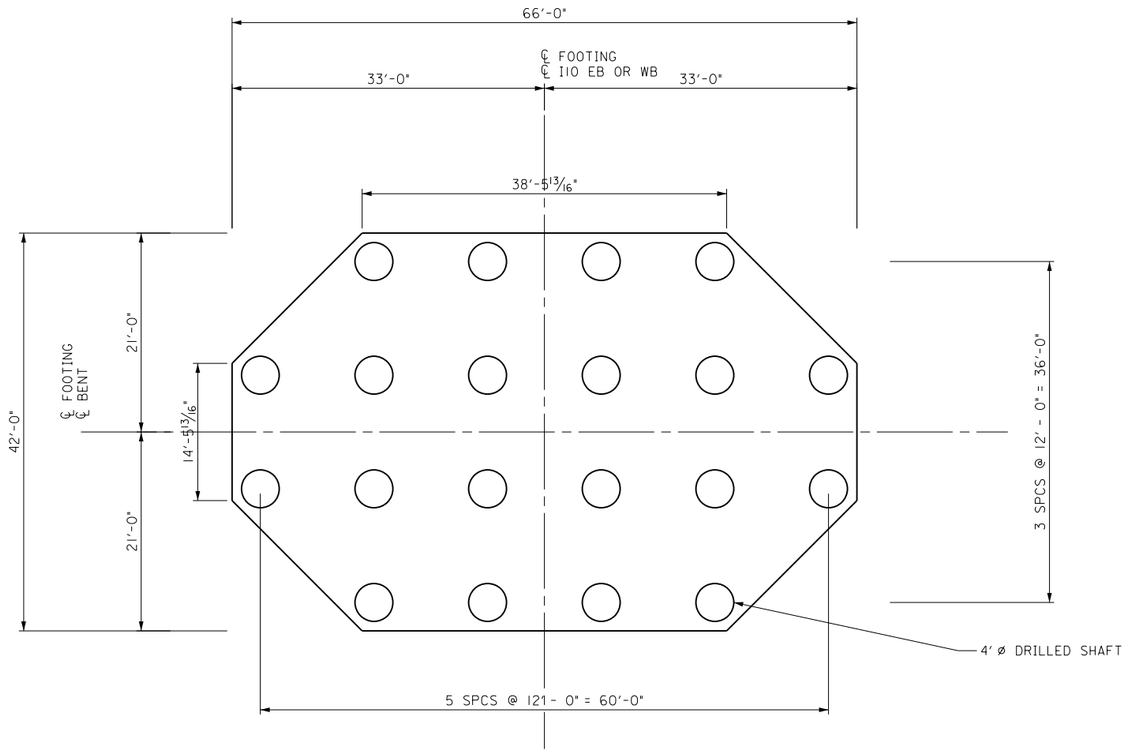
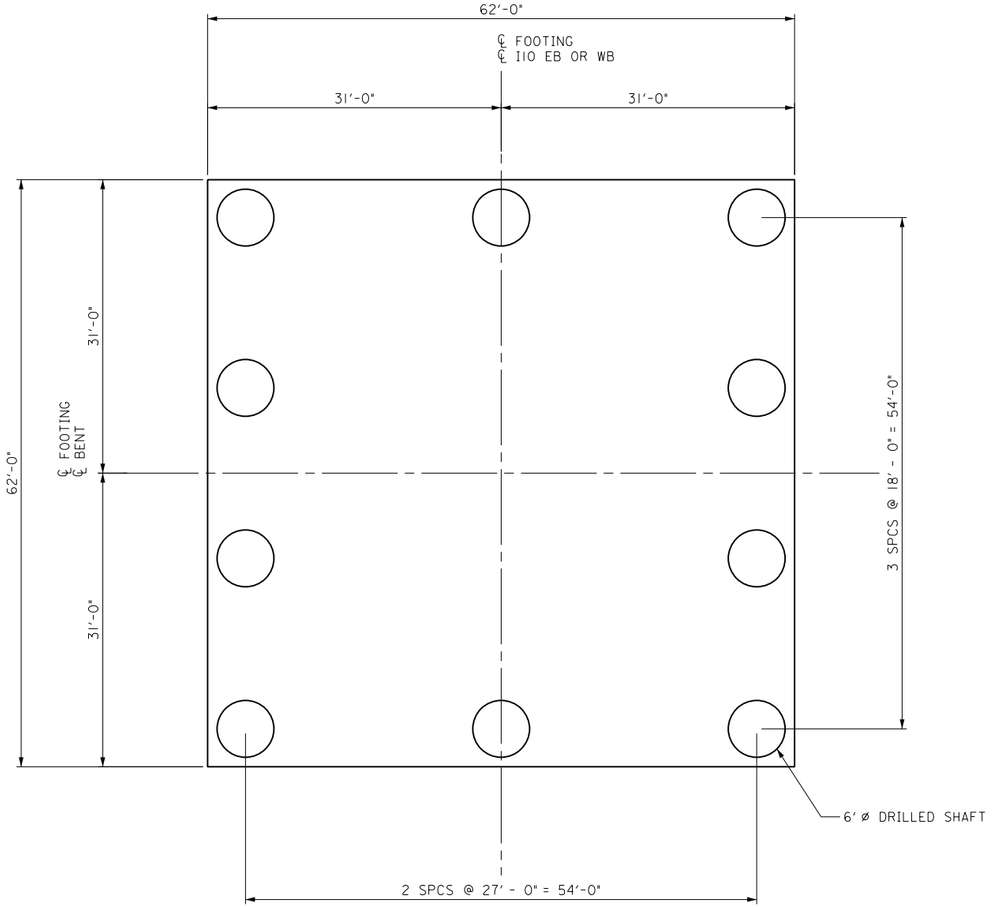


6 X 4 LAYOUT

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

BIN:

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:

PLOT DRIVER: HDR ALDOT PDF CL HALF SIZE.pltcfg
USER: JSPANIER DATE: Friday, September 02, 2016 11:09:57 AM
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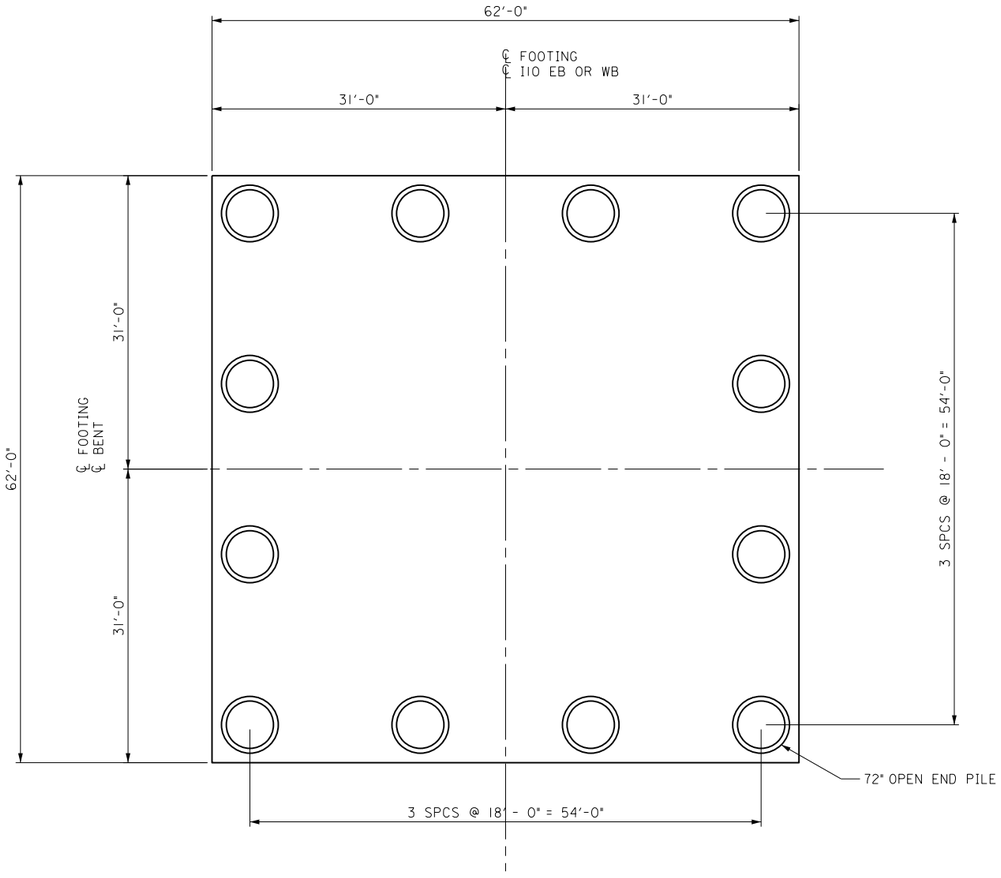
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

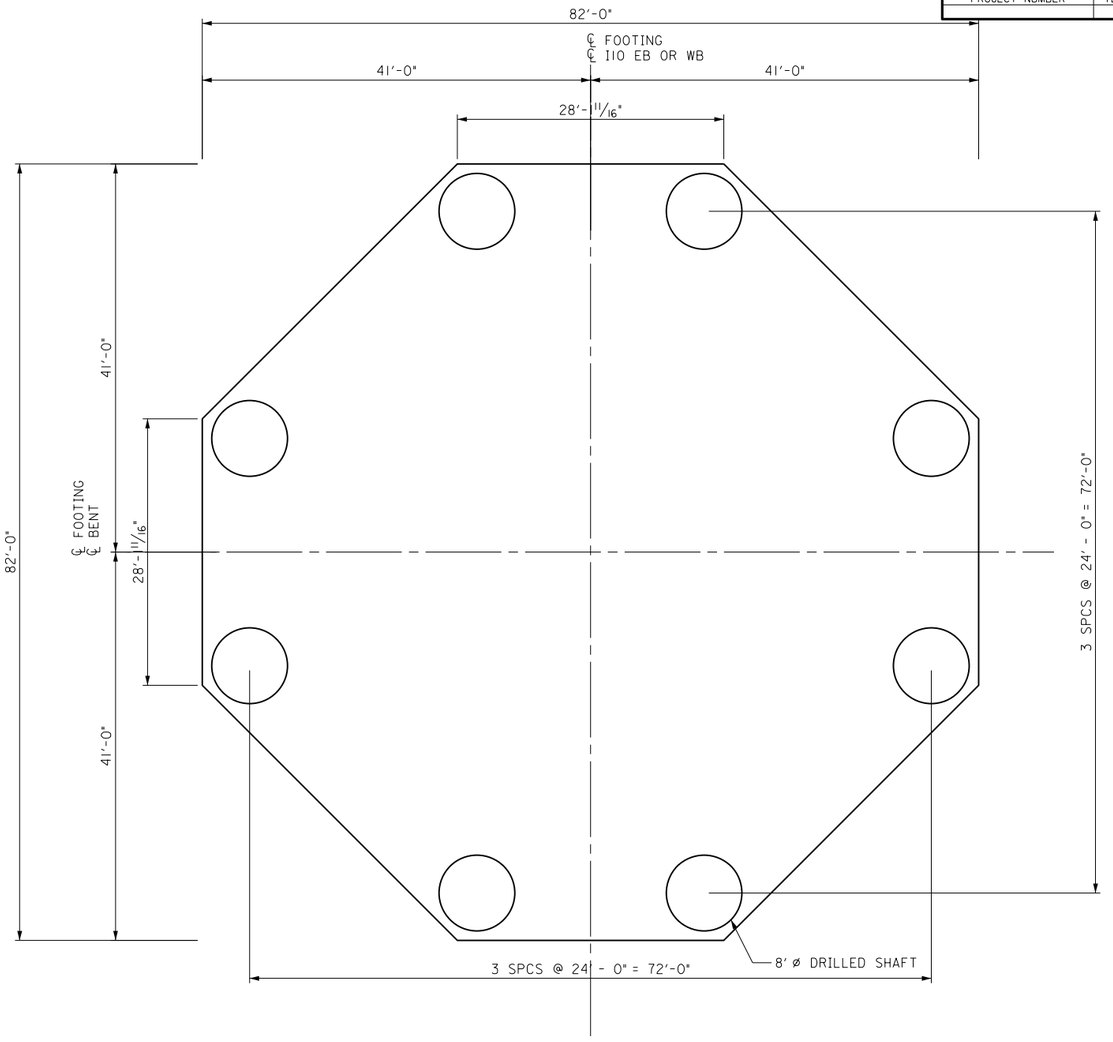


4X4 LAYOUT

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

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

BIN:

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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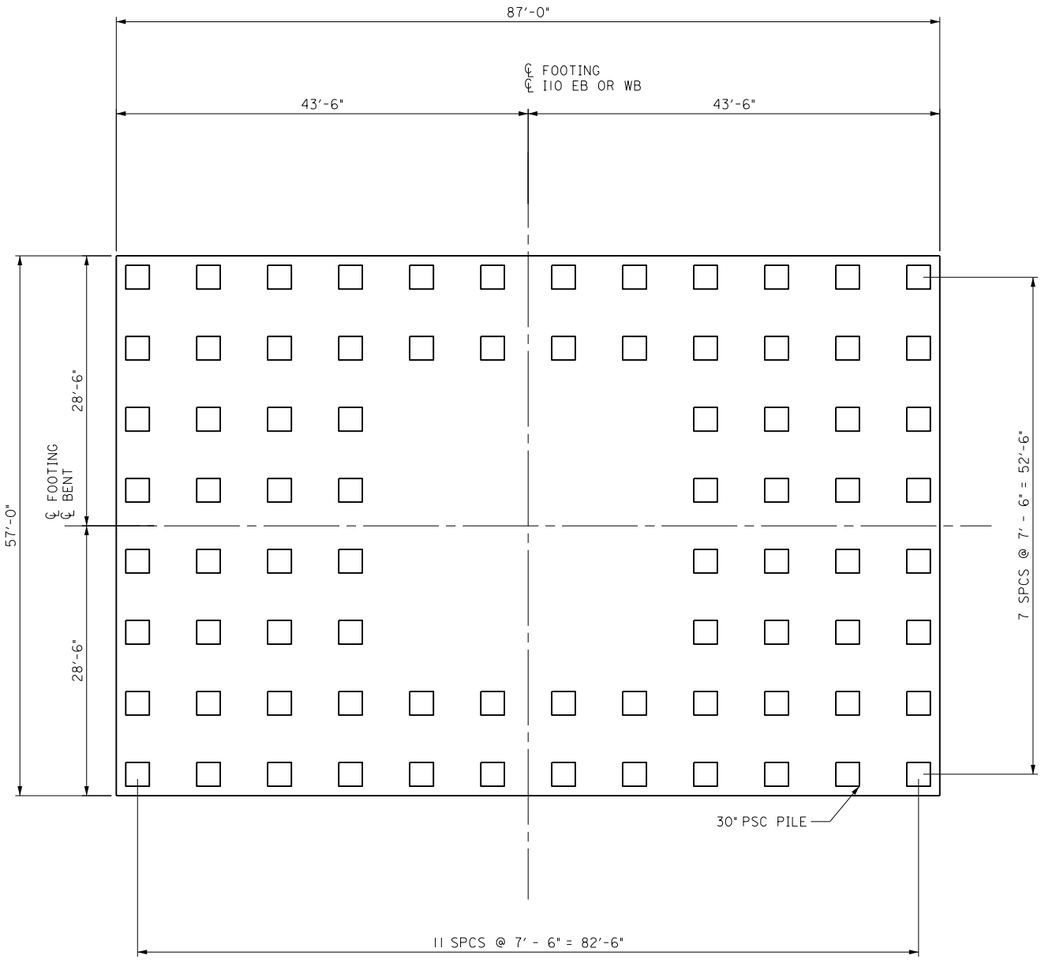
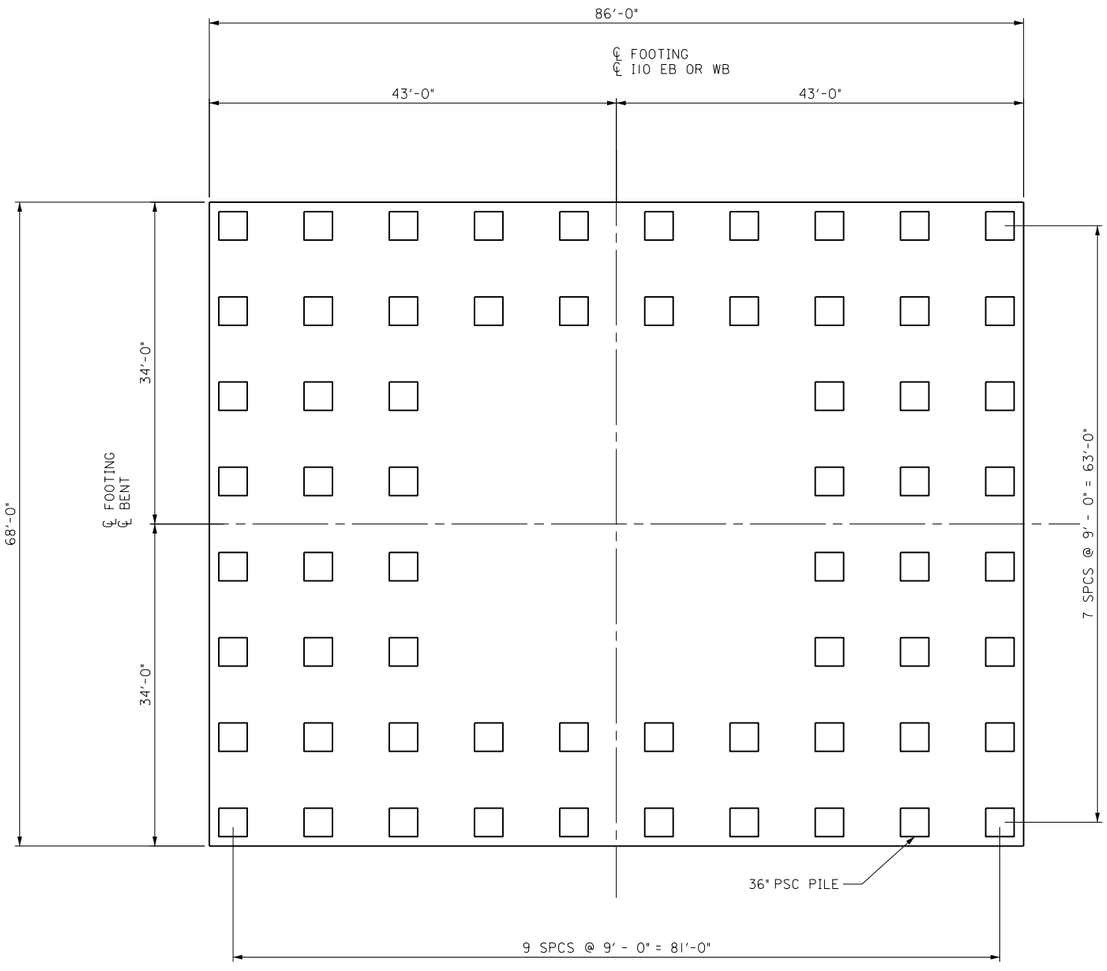


10X8 LAYOUT

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

BIN:

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:
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VERIFIED BY:	DATE CHECKED:	SCALE:

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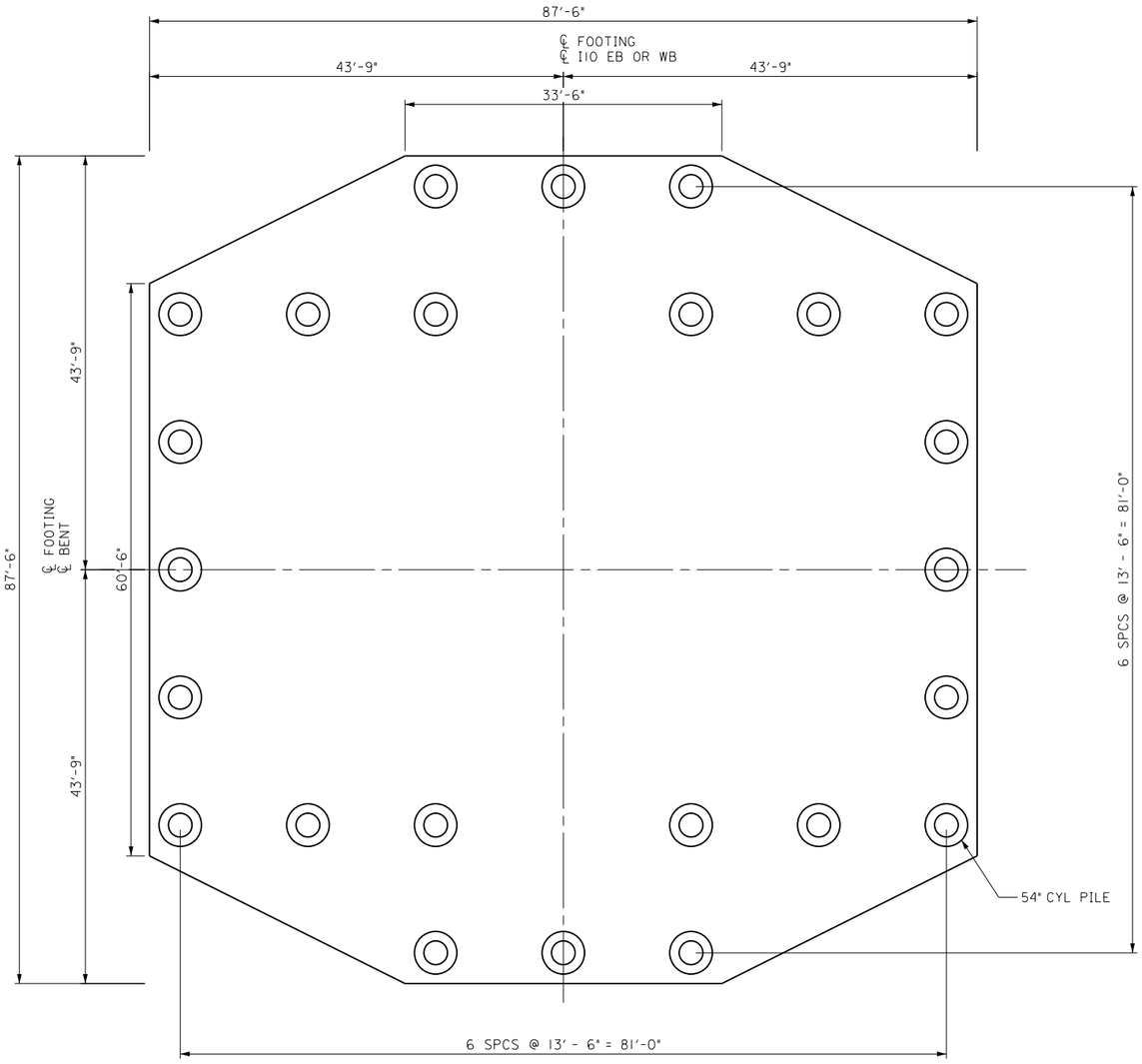
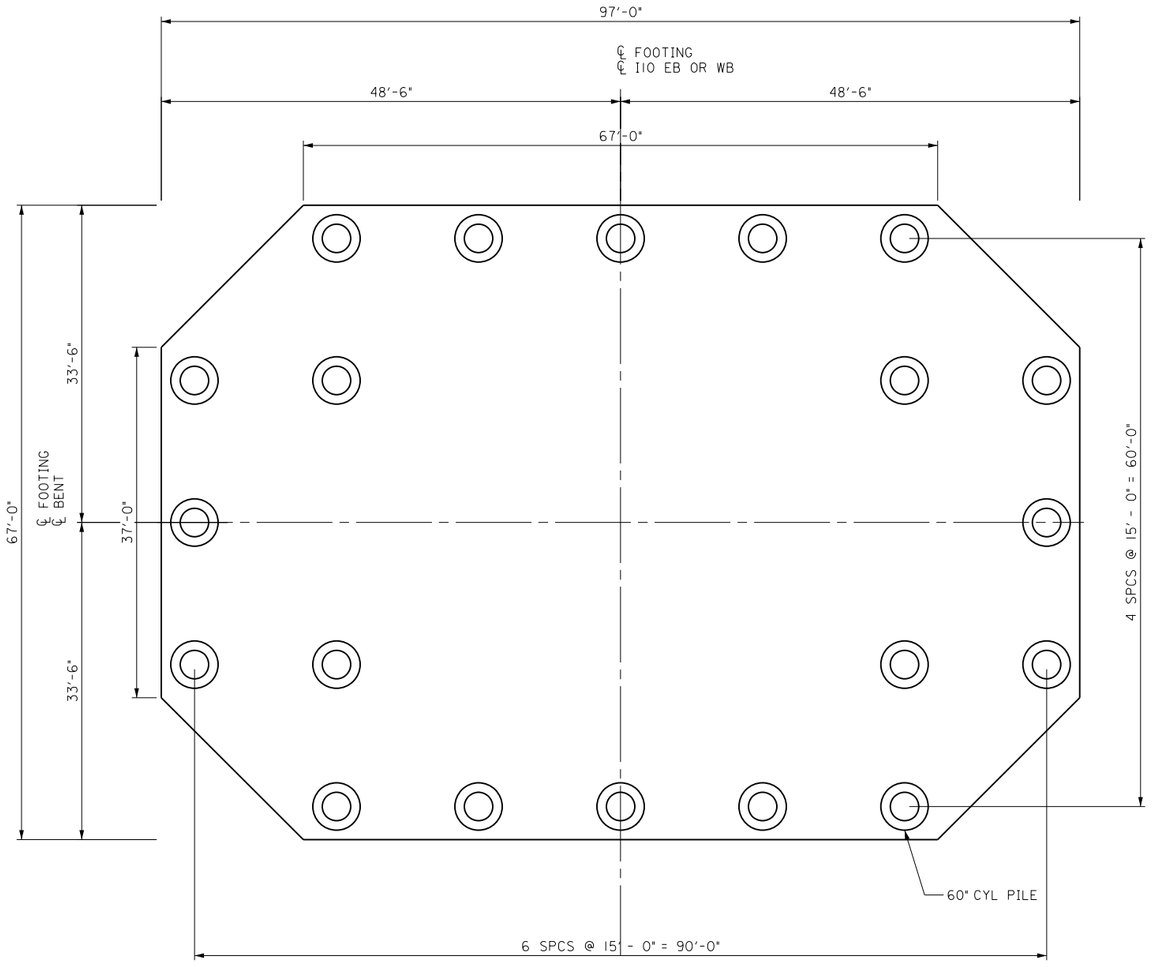


OCTAGON LAYOUT

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

BIN:

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:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

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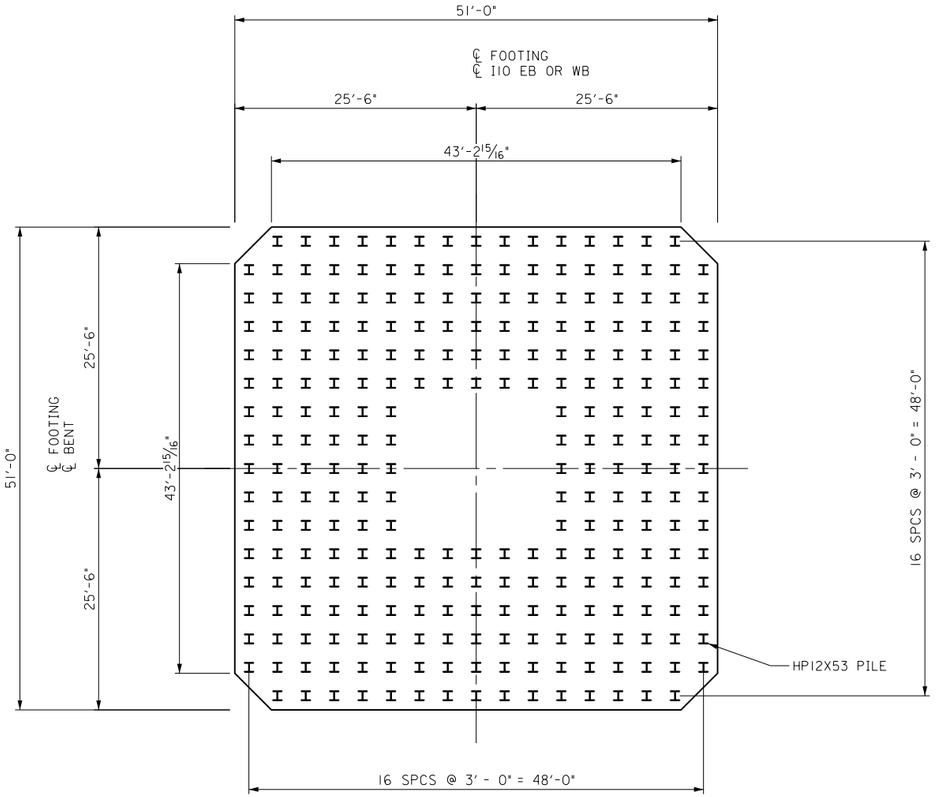
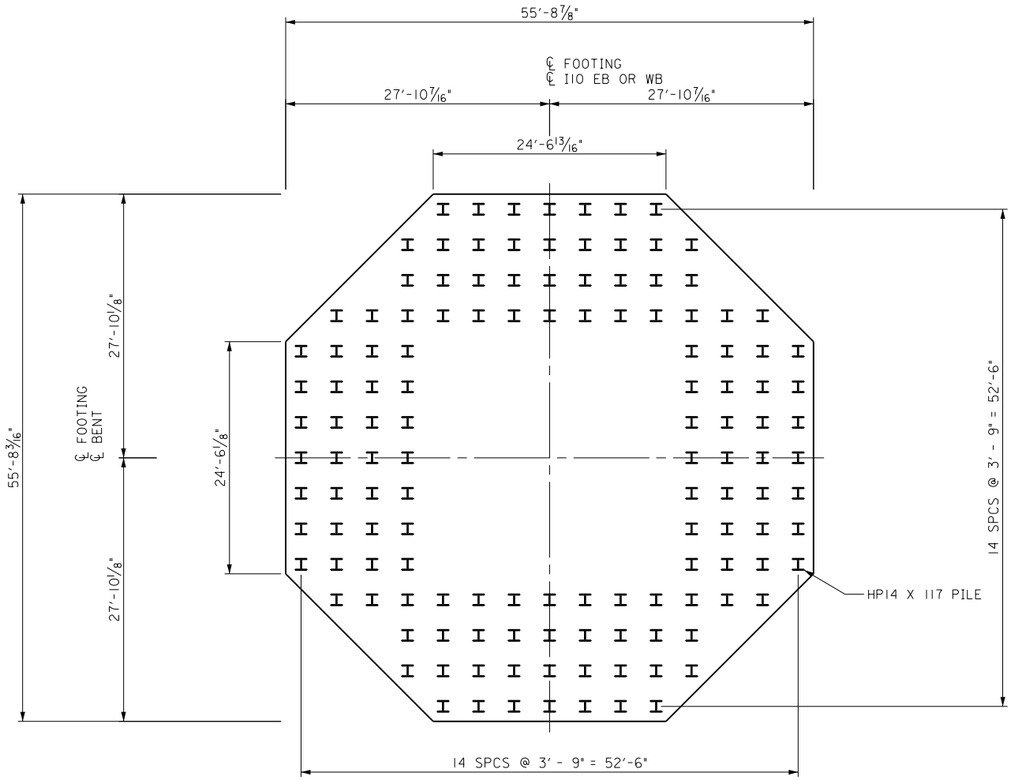


17x17 LAYOUT

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	MOBILE RIVER BRIDGE HIGH LEVEL APPROACH WEST SIDE ALTERNATIVE FOUNDATION ANALYSIS
REVISIONS	HIGH LEVEL APPR FOUNDATION OPTIONS (5 OF 5)
	ESTIMATED QUANTITIES: / DESIGNED BY: / DRAWN BY:
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DATE: Friday, September 02, 2016 11:09:58 AM
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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.

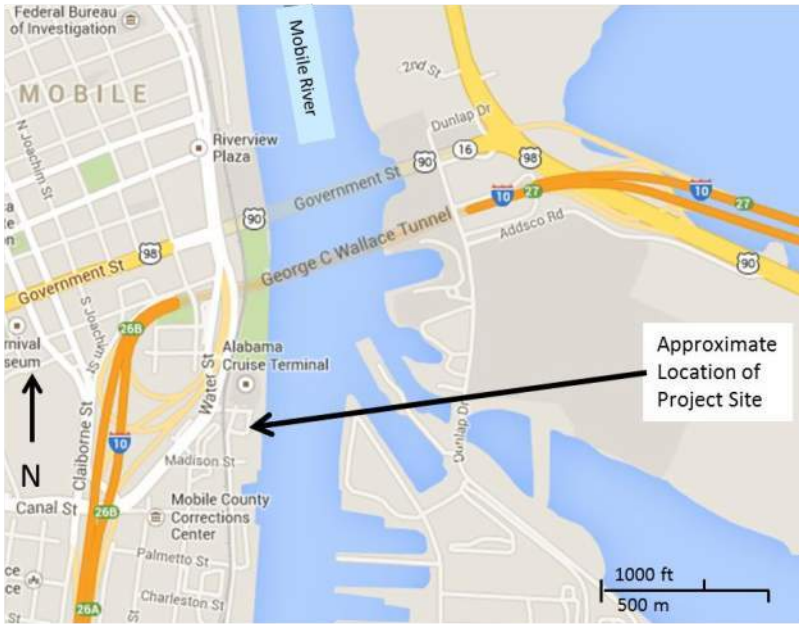

Figure 1: Location of project site, Mobile, AL **(Google 2013)**

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).

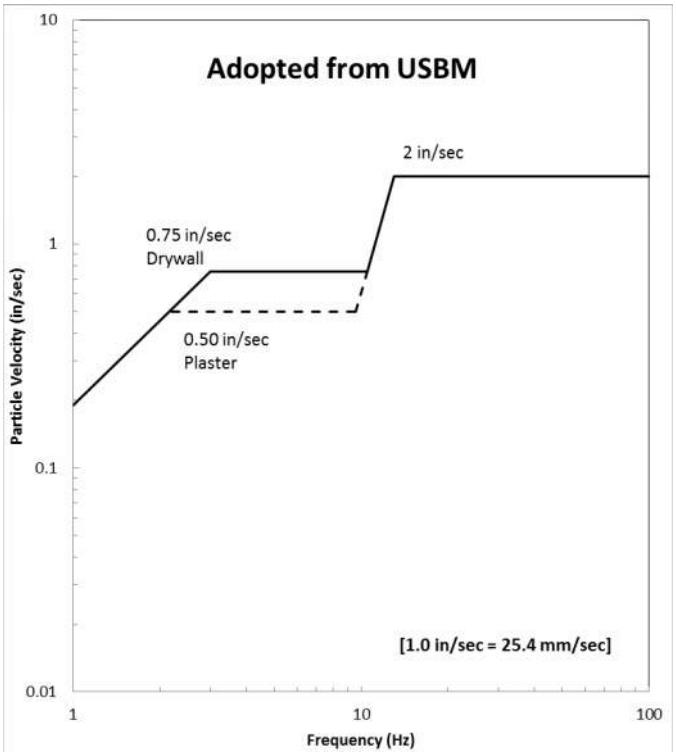

Figure 2: Vibration limits from the USBM (Siskind, et al. 1980)

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:

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

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.

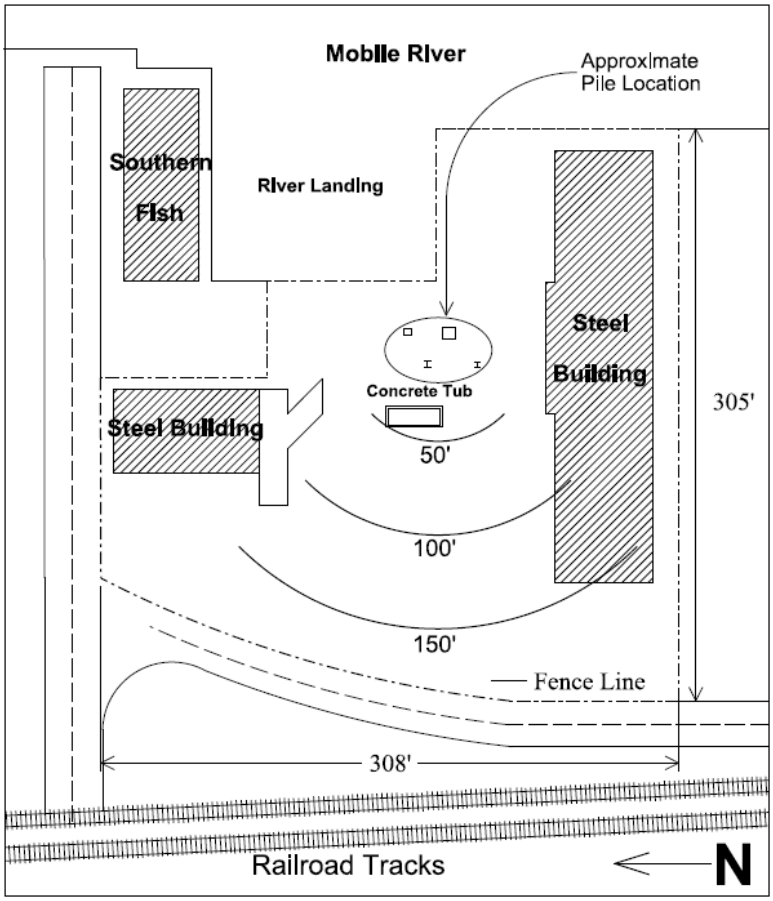

Figure 3: Plan view of Mobile River Bridge Project Site

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.

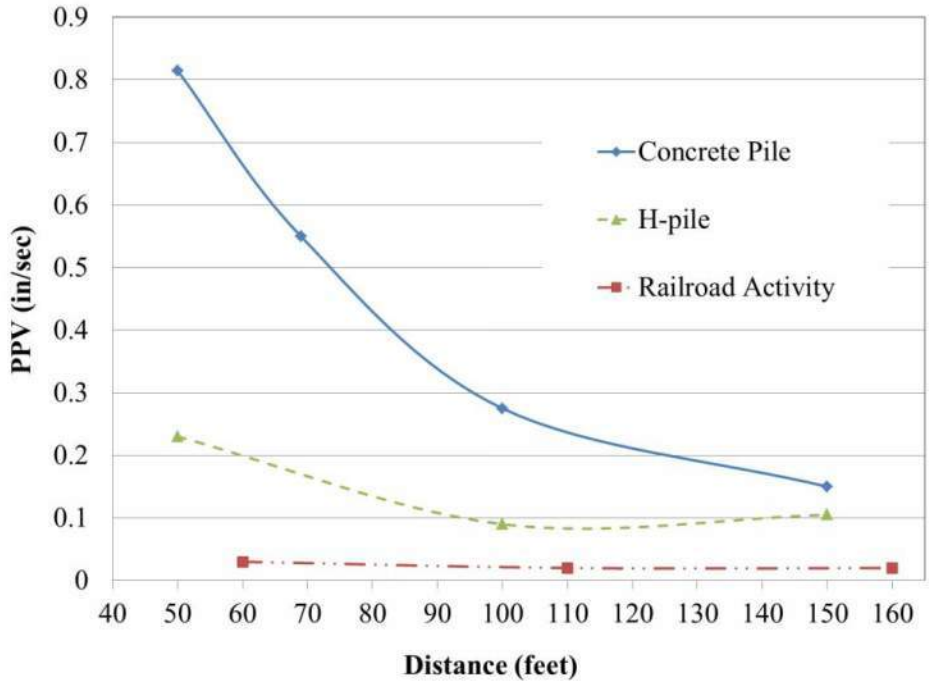

Figure 4: Maximum recorded vibration levels during pile installation

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.

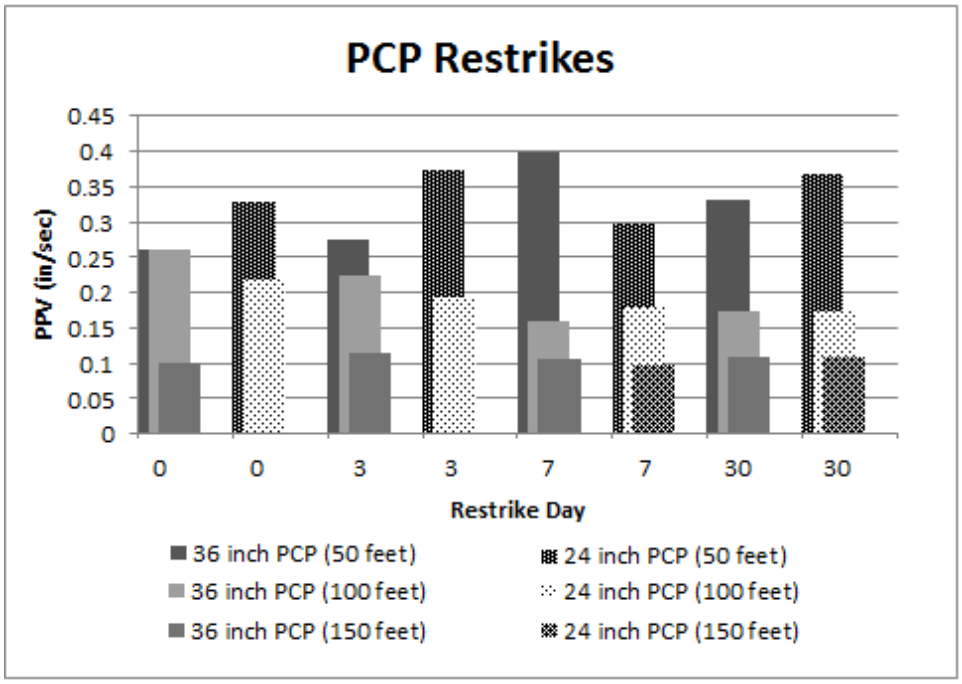

Figure 5: Bar chart of restrikes on precast concrete piles (PCP)

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.

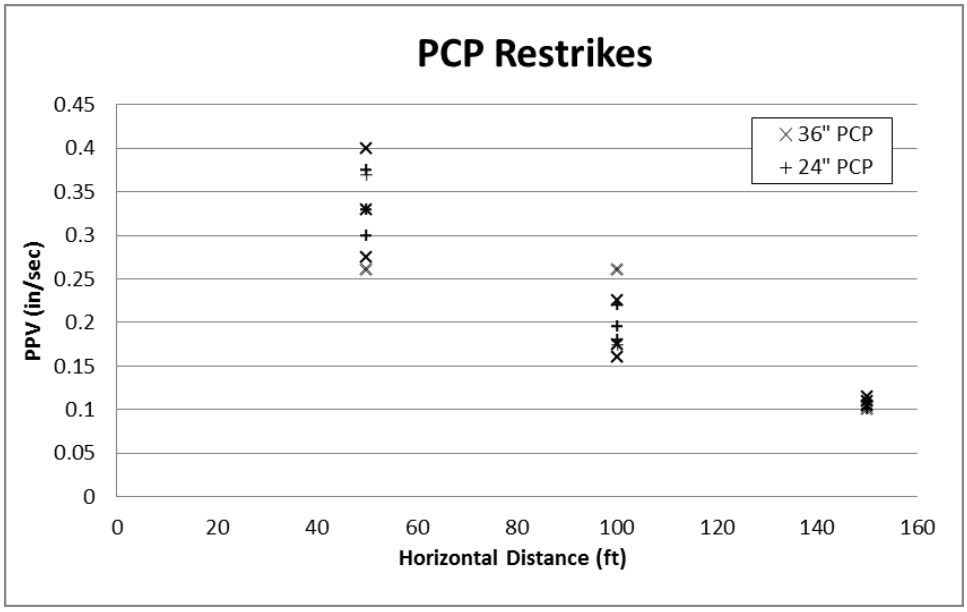

Figure 6: Data plot of restrikes on precast concrete piles (PCP)

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.

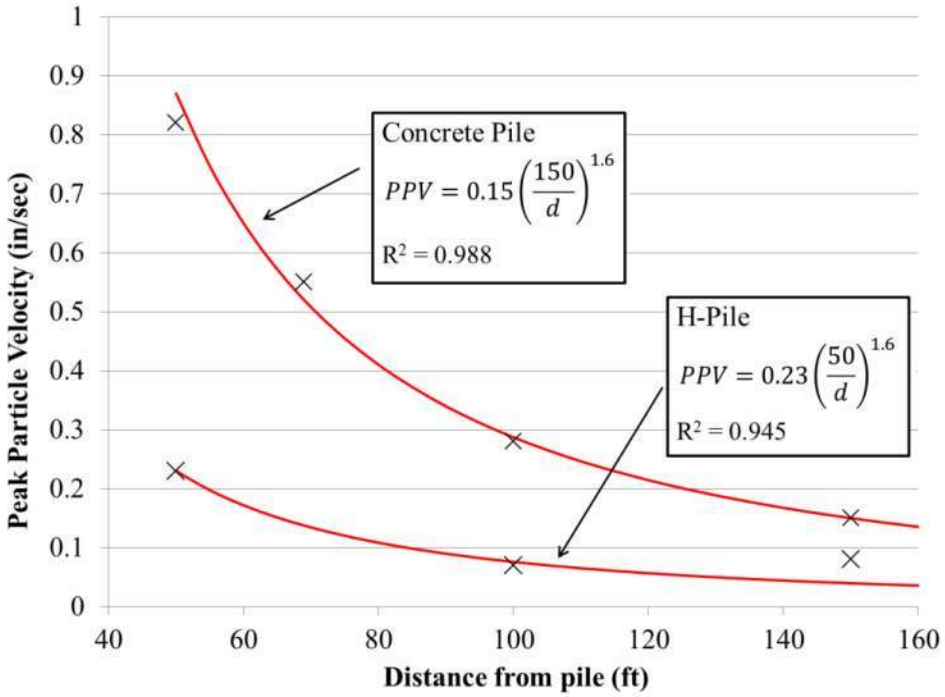

Figure 7: Peak particle velocity versus distance

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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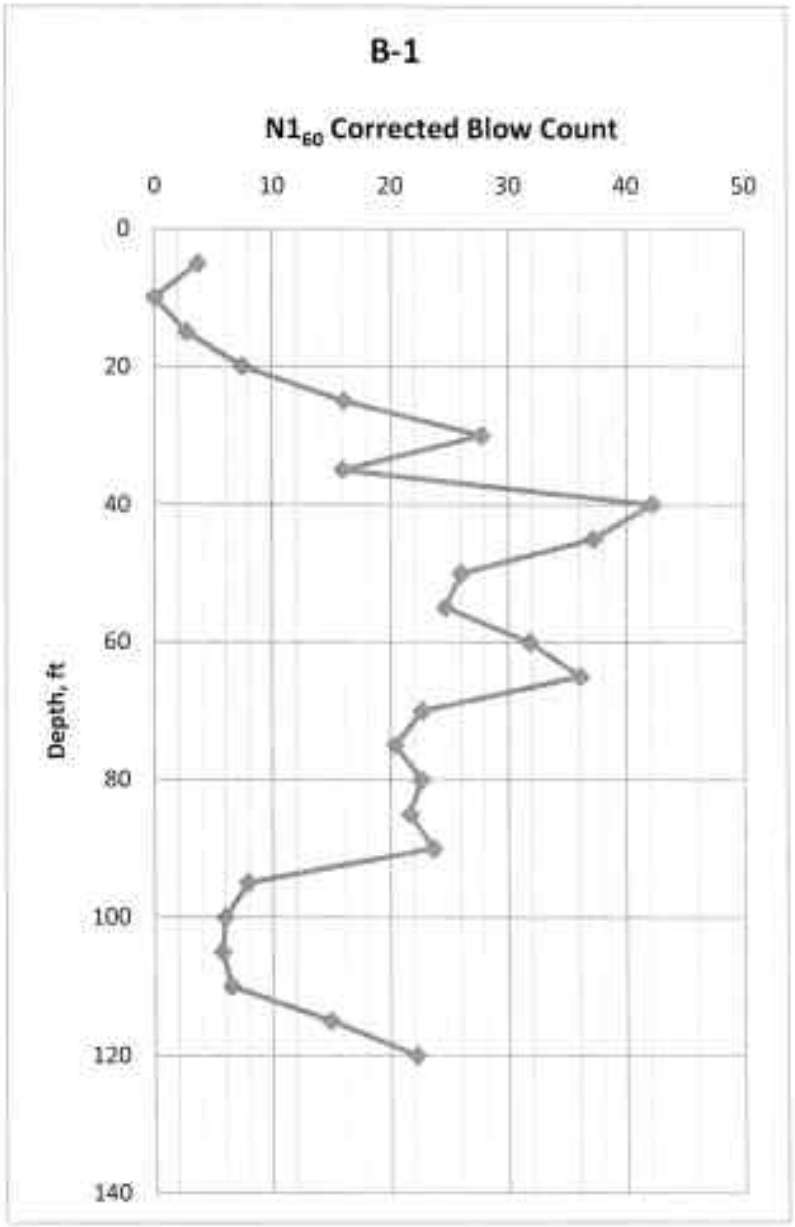
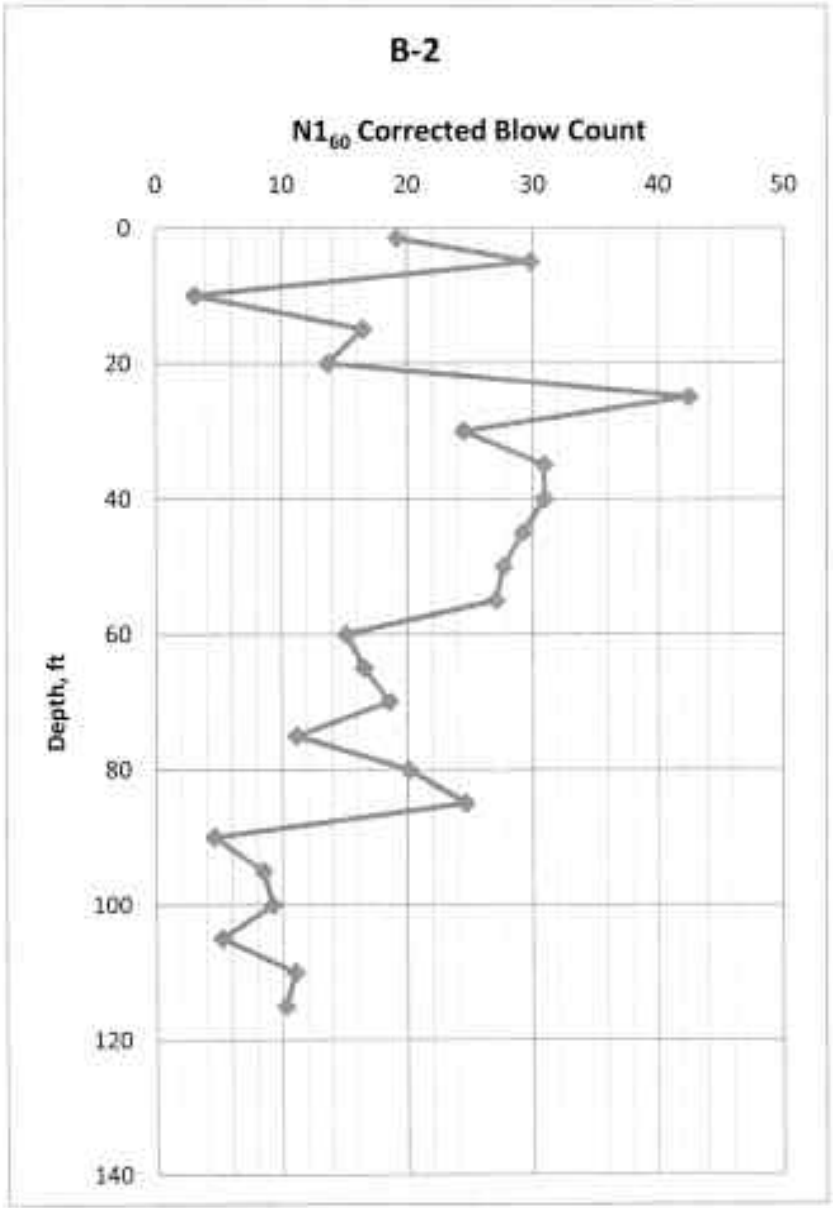
Appendix A: Soil Reports

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
Depth, ft
0
20
40
60
80
100
120
140


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

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' 11"

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	WH	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
More 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 8-6-12

_____ 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 A JAR Samples of Clay

Project# DP1-0030 (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 Matl.
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. Matl. *	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 & Org matl	2-K	48.5	50.0	7	15	18	33
	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'

Identification

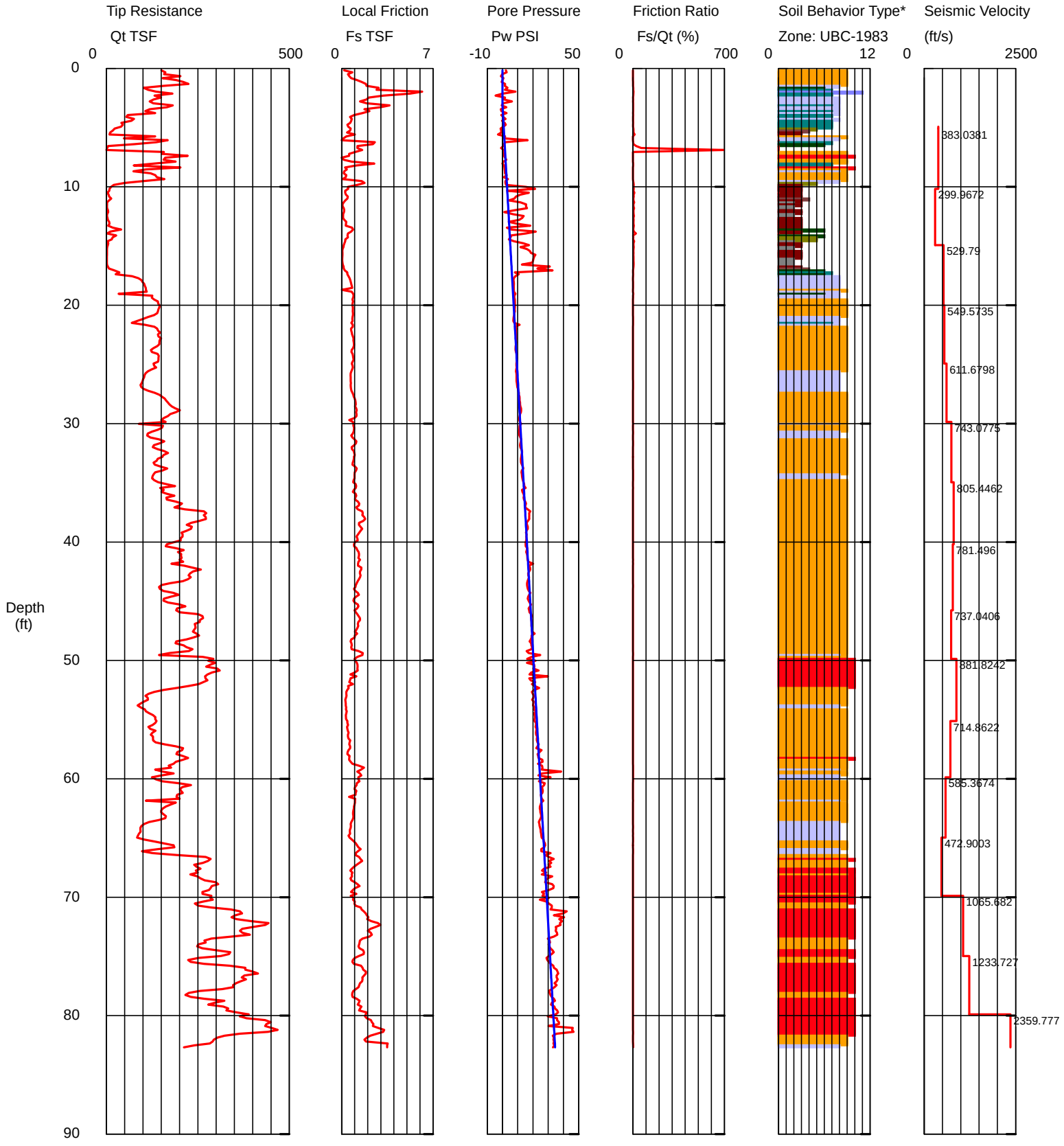
Other Pertinent Components	Sample No.	Penetration or Sample Elev. From	To	"N" Blow .5	1.0	1.5	"N" Value
	2-P	73.5	75	6	7	10	17
*	2-Q	78.5	80.0	6	11	21	32
	2-R	83.5	85.0	12	18	23	41
*	2-S	88.5	90.0	4	4	4	8
*	2-T	93.5	95.0	6	7	8	15
*	2-U	98.5	100.0	1	9	8	17
*	2-V	103.5	105.0	2	4	6	10
*	2-W	108.5	110.0	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'

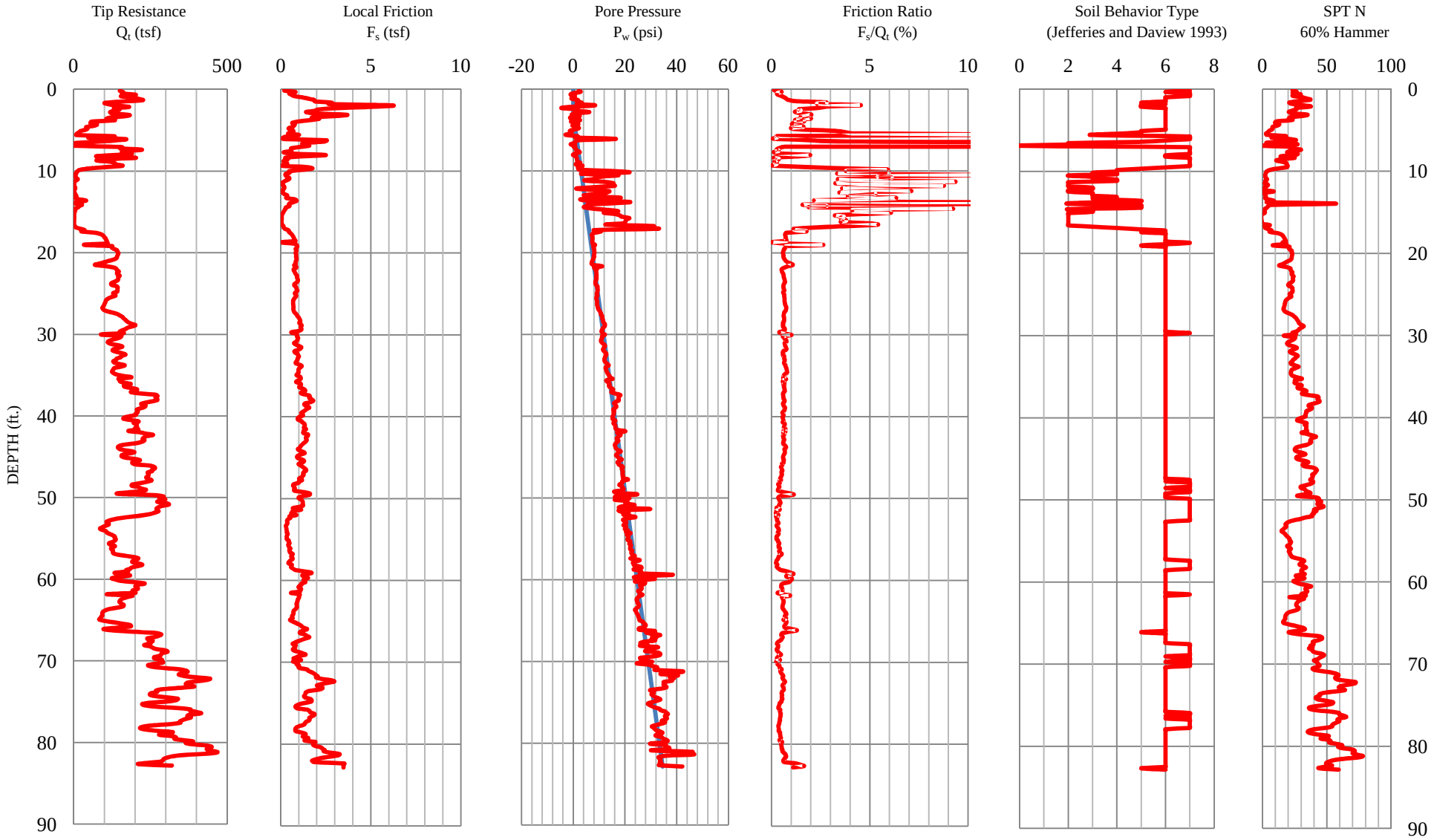
N30.68546 W88.03791

*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

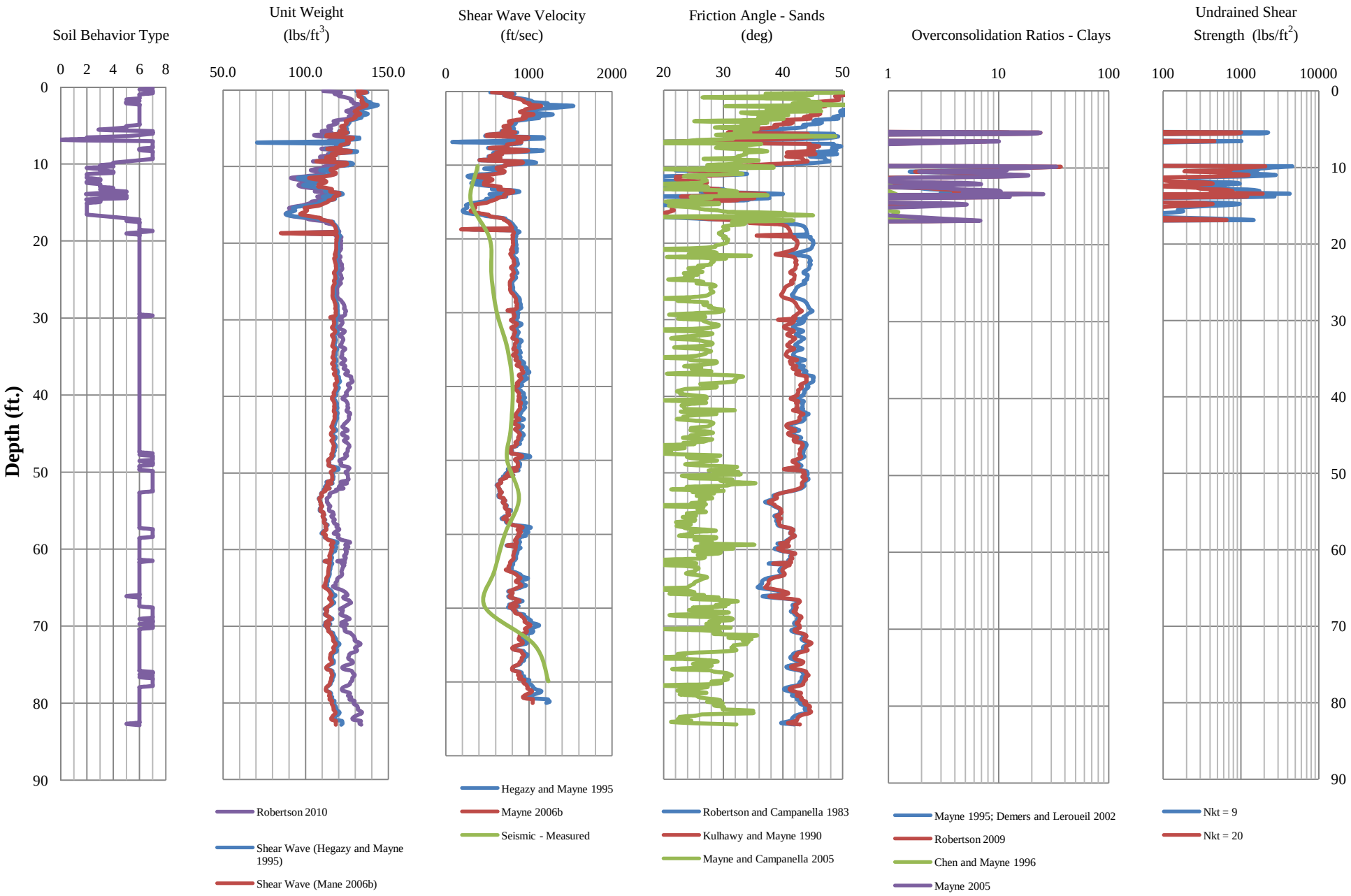
SPT N, SOIL BEHAVIOR TYPE, OR ZONE NUMBER FROM CPT CLASSIFICATION INDEX, Ic

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

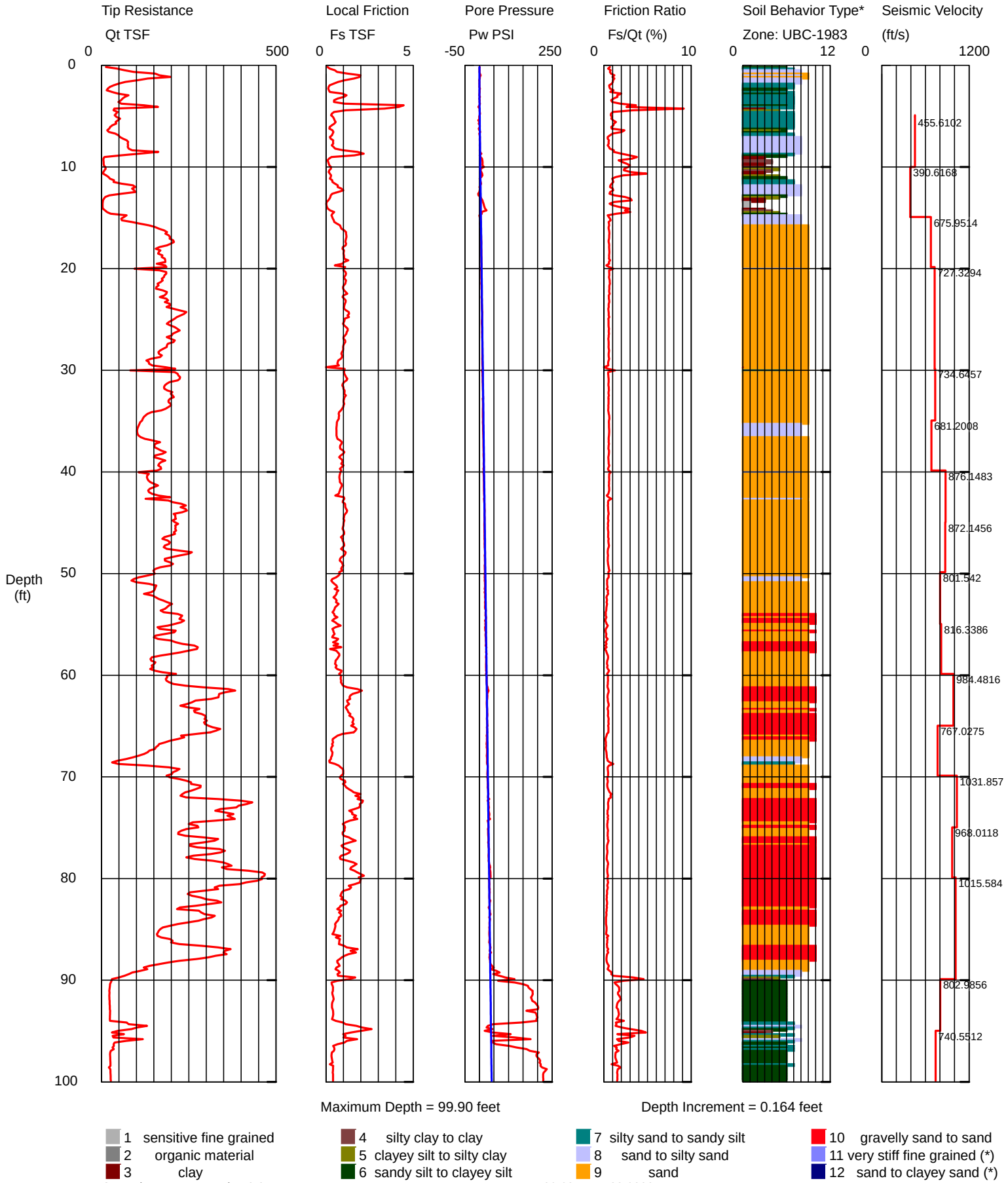


PARAMETERS ABOVE ARE BASED UPON EMPIRICAL CORRELATIONS AND SHOULD BE CONSIDERED APPROXIMATE. IT IS RECOMMENDED THAT CALCULATED PARAMETERS BE CORRELATED BY SPECIFIC LABORATORY DATA AND/OR LOCAL EXERIENCE.

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'

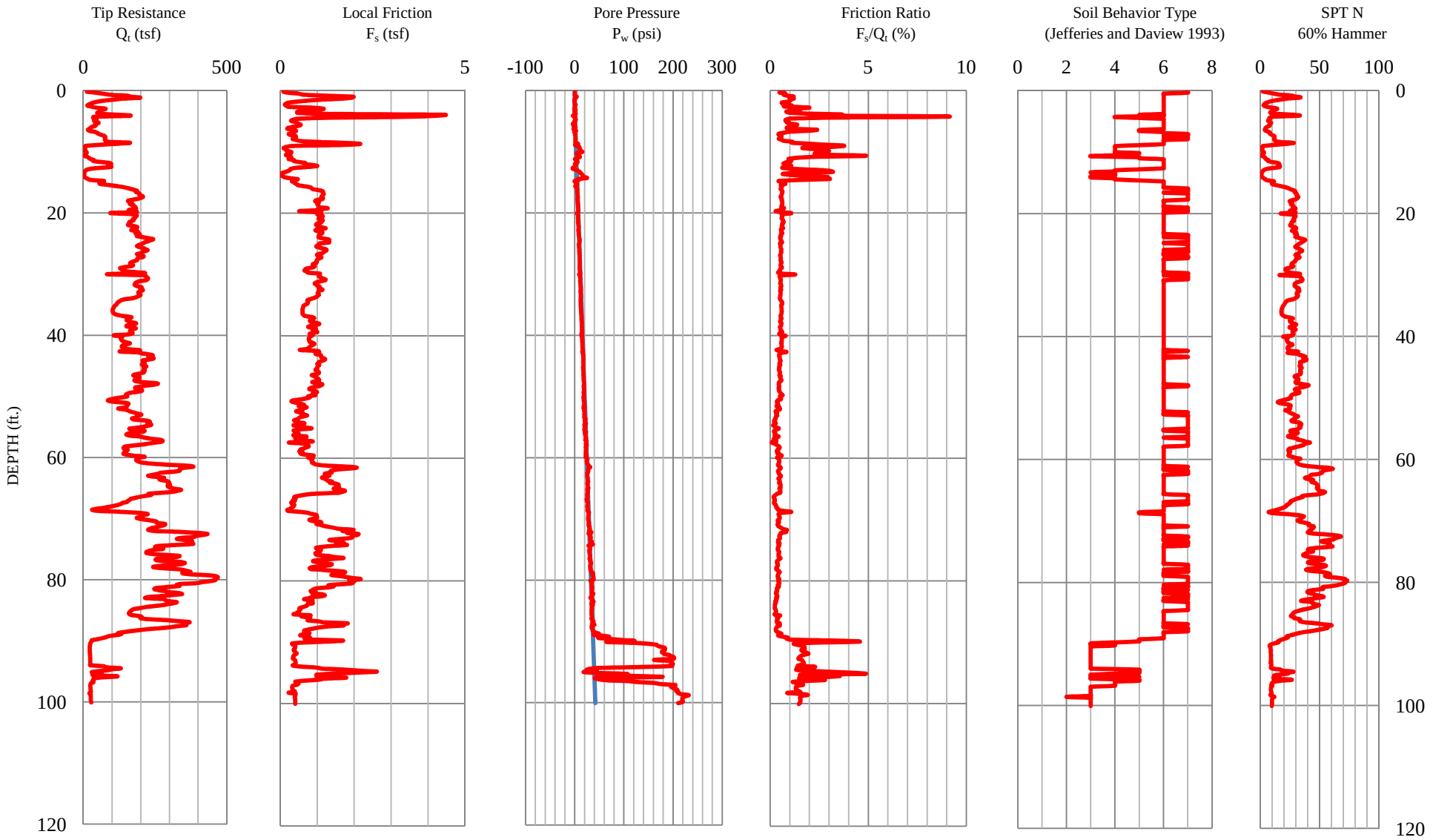
N30.68541 W88.03821

*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

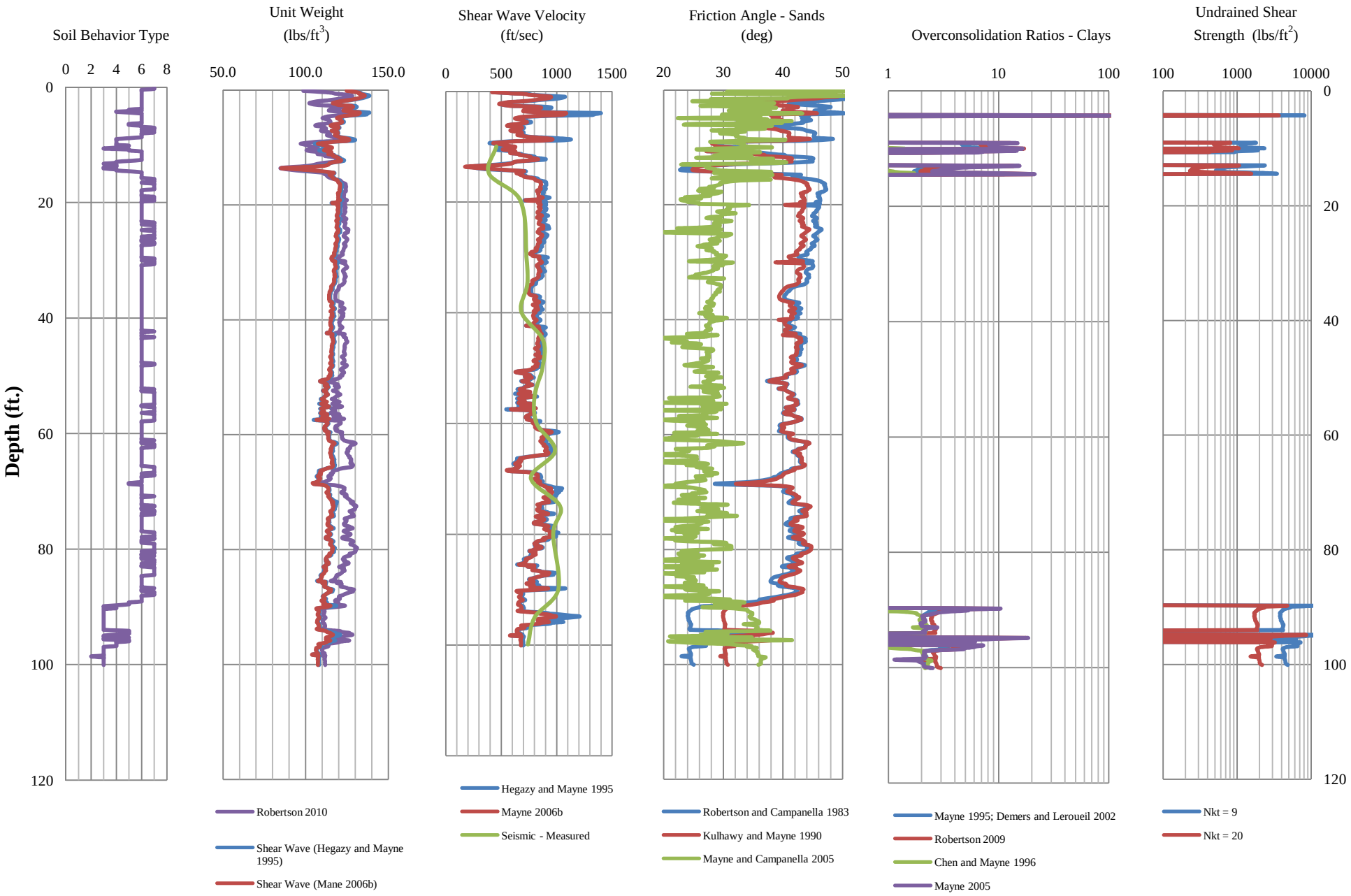
SPT N, SOIL BEHAVIOR TYPE, OR ZONE NUMBER FROM CPT CLASSIFICATION INDEX, Ic

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**

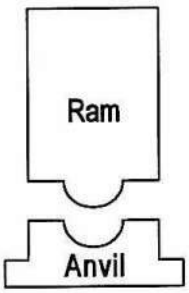
Revised 08-07-95 PILE AND DRIVING EQUIPMENT DATA FORM

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 Components



Hammer

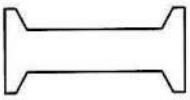
Manufacturer: Delmag Model: D-62-22
Type: S.A. Diesel Serial No.: 238
Rated Energy: 165,000 (ft.-lbs.) at 11.3 (ft.) Length of Stroke

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

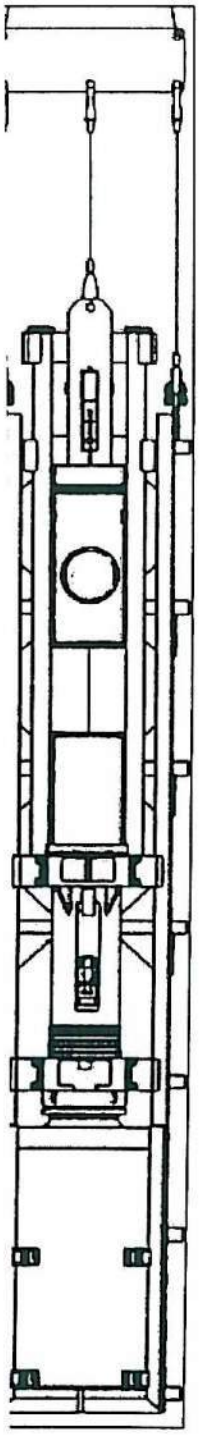
Pile

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 (in²)
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: Davis Daniel Date:



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**

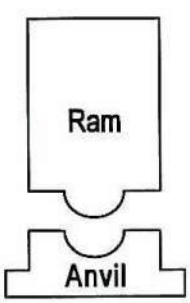
Revised 08-07-95 PILE AND DRIVING EQUIPMENT DATA FORM

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

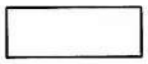


Hammer

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

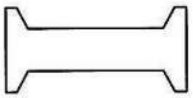
Modifications: Adjustable Fuel Pump

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)

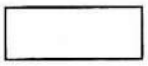
Material: Aluminum & Micarta Alternating
Thickness: 4 (in.) Area: 398 (in.²)
Modulus of Elasticity - E : 285 (P.S.I.)
Coefficient of Restitution - e : 0.8



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: (in²)
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.

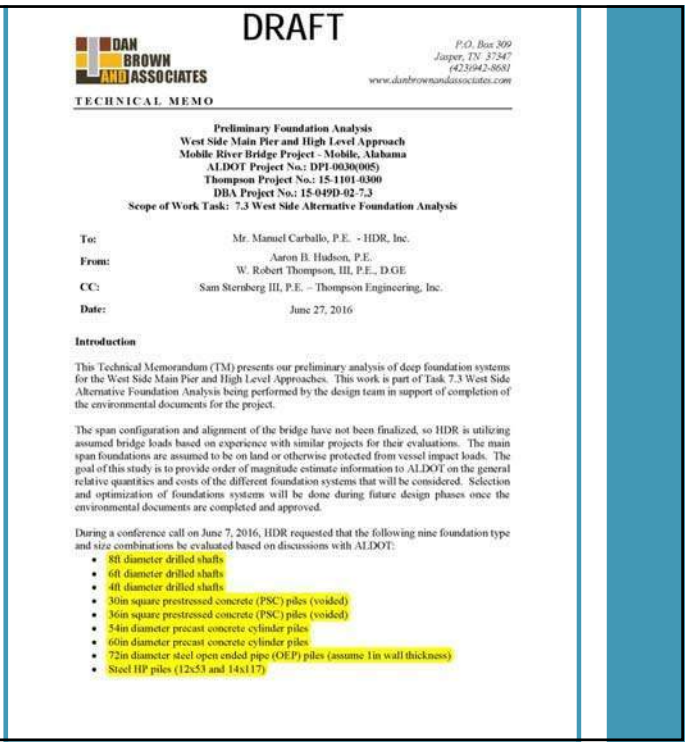
The various foundation options will be analyzed in order to develop the type, size and location of the preferred option for the west side of the project that will be carried through the future Preliminary and Final Design phases. Special features to be analyzed will include:

- 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

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



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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.

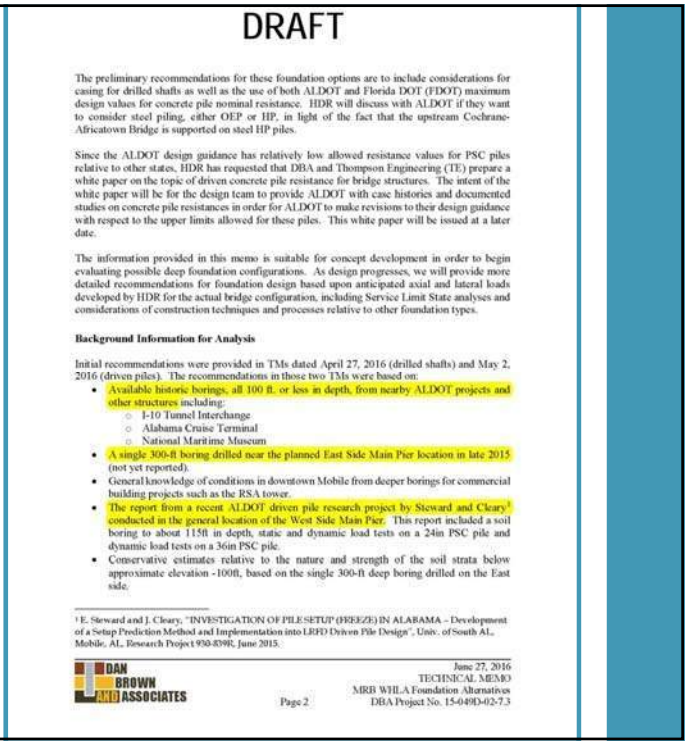
The span configuration and alignment of the bridge have not been finalized, so HDR is utilizing assumed bridge loads based on experience with similar projects for their evaluations. The main span foundations are assumed to be on land or otherwise protected from vessel impact loads. The goal of this study 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”



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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.

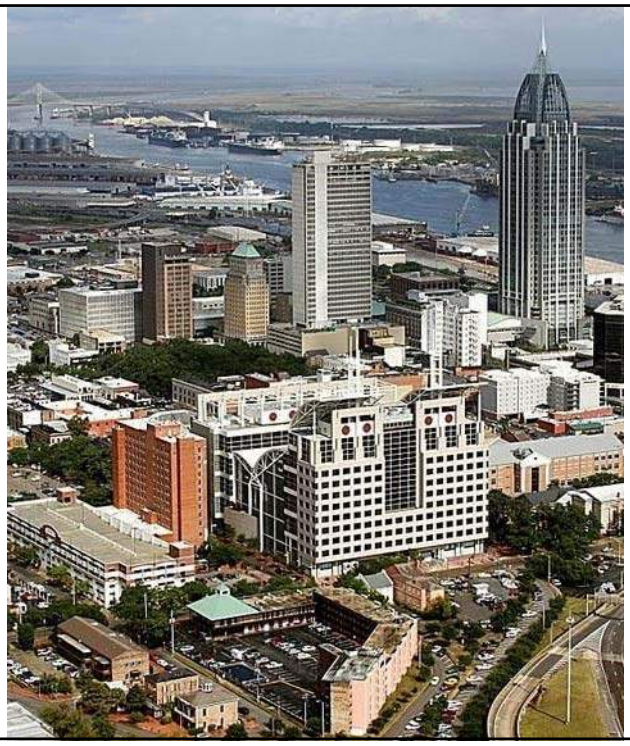
DAN BROWN AND ASSOCIATES

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

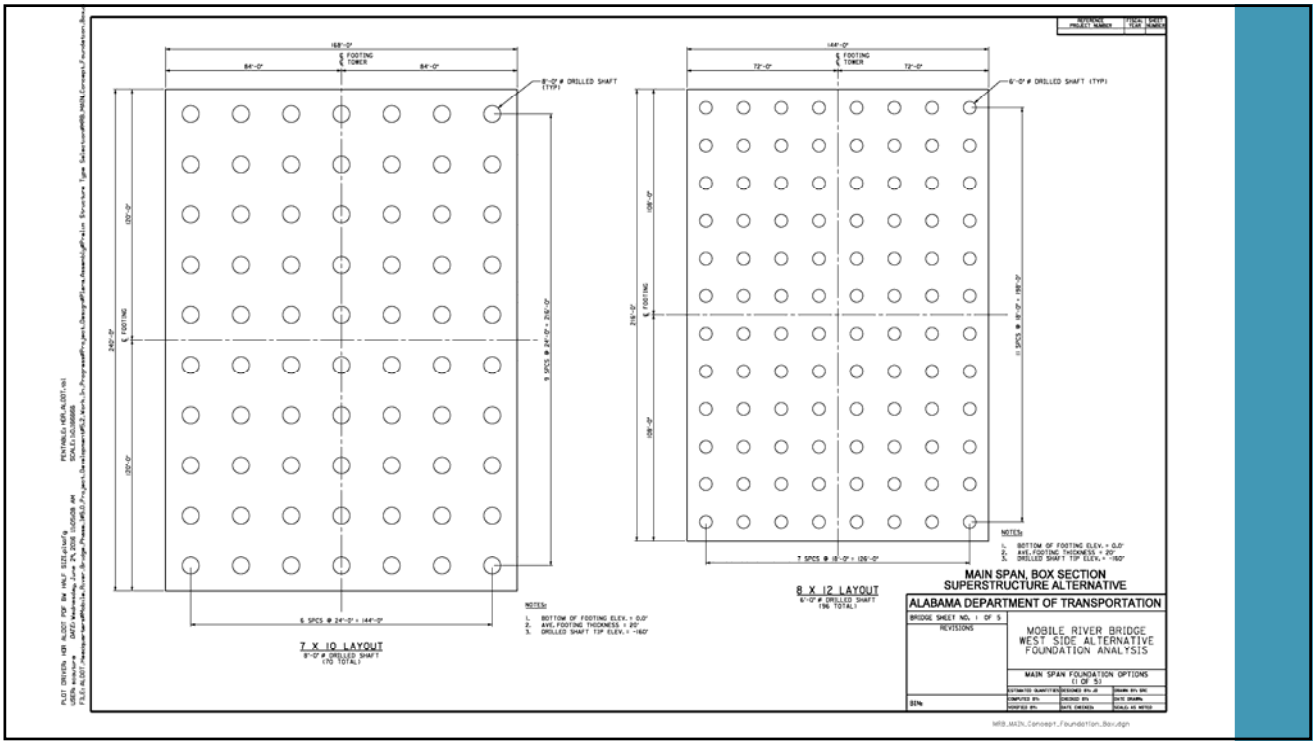


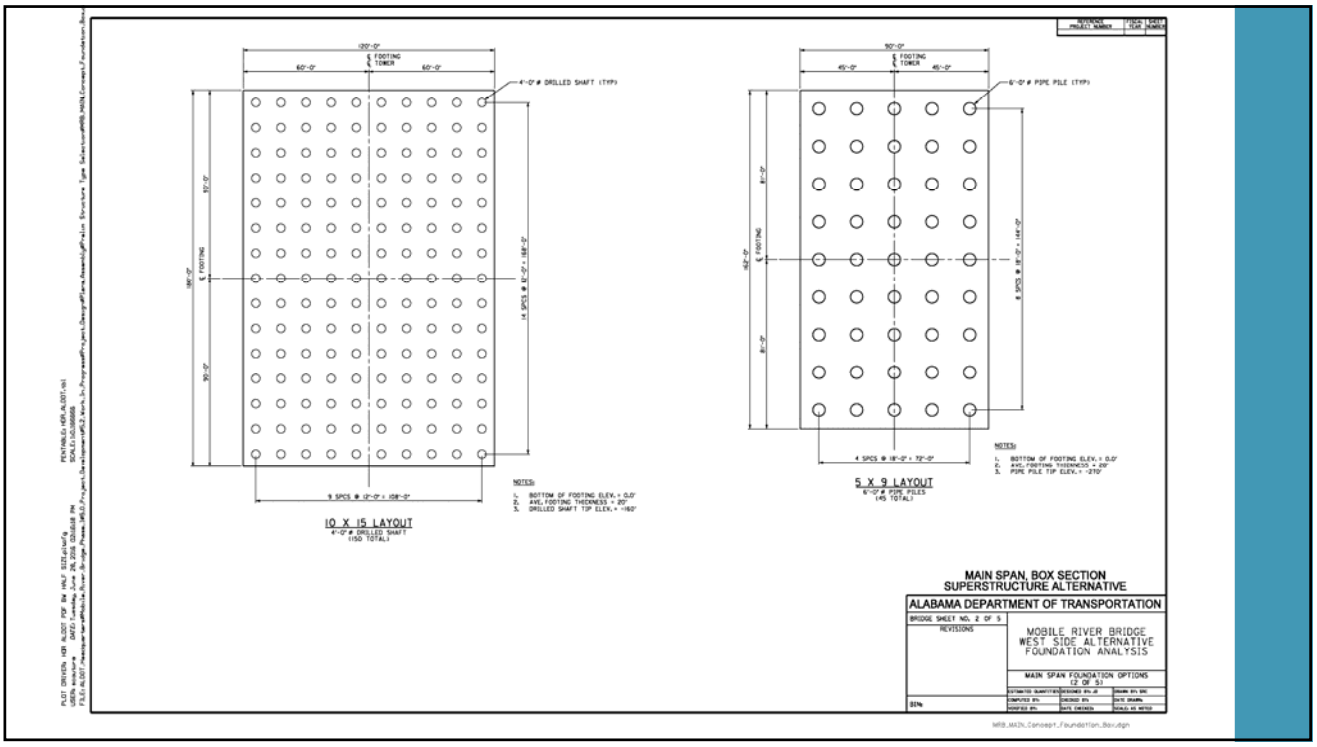
5. Design Approach

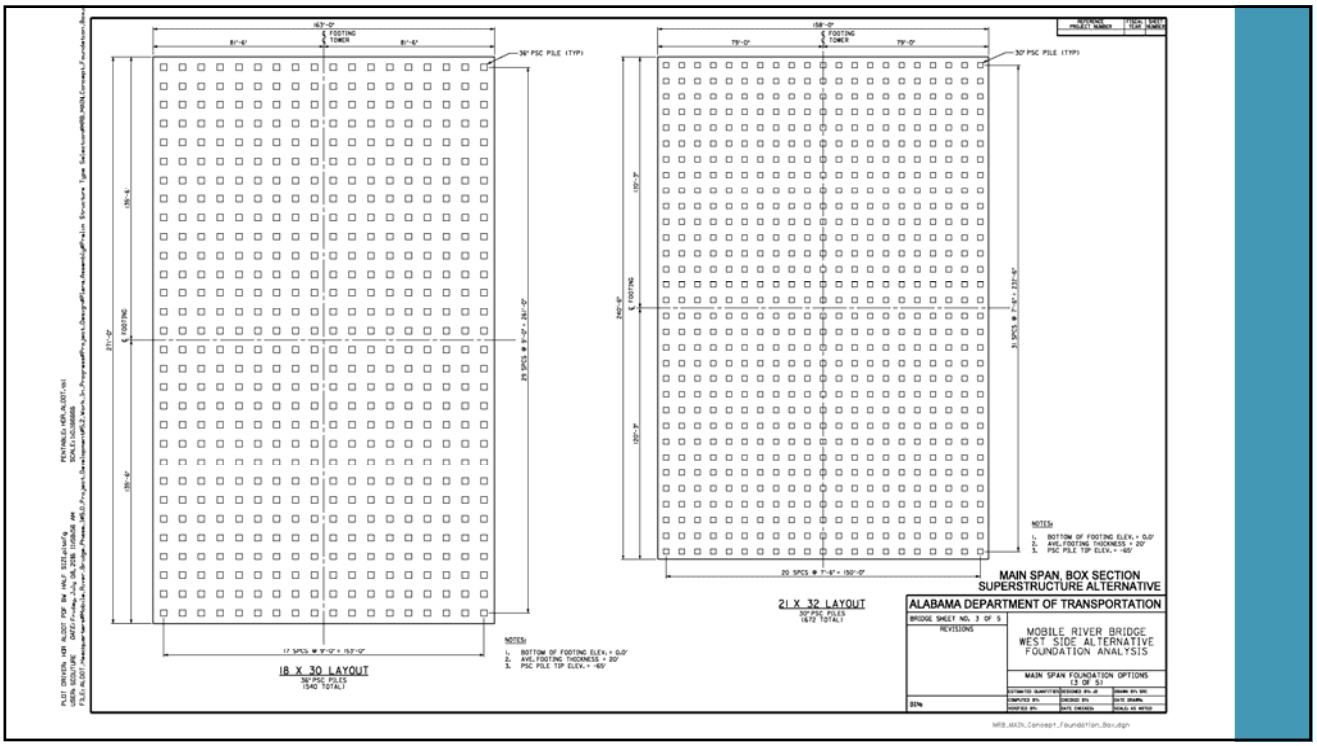
- For Main Span and High-Level Approach Spans Develop Foundation concepts for the Heaviest and Lightest Superstructure Type
- Prepare preliminary quantity take-offs for each foundation Concept and assign unit Costs to develop Estimated Construction Costs

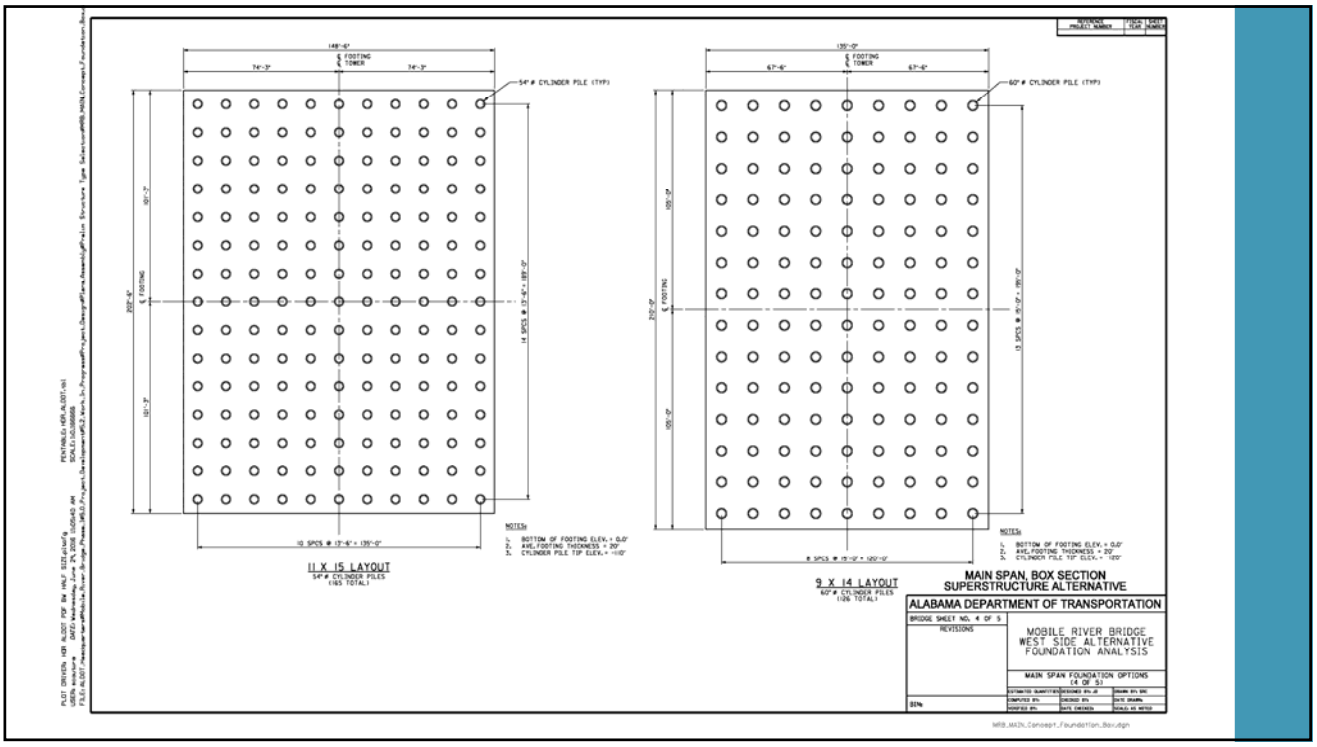
5. Design Concepts

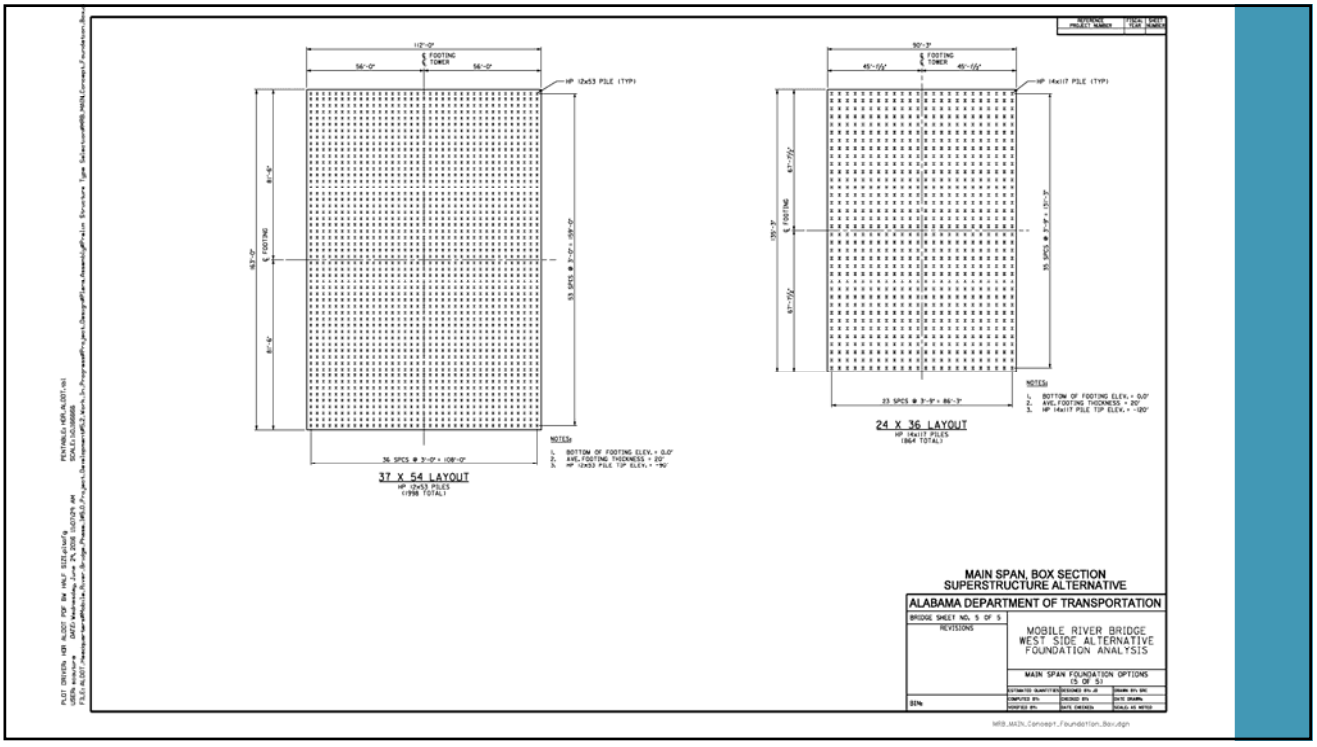
- Main Span Concrete Superstructure




10 X 15 LAYOUT
4'-0" ø DRILLED SHAFT
(150 TOTAL)
5 X 9 LAYOUT
6'-0" ø PIPE PILES
(45 TOTAL)
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(2 OF 5)

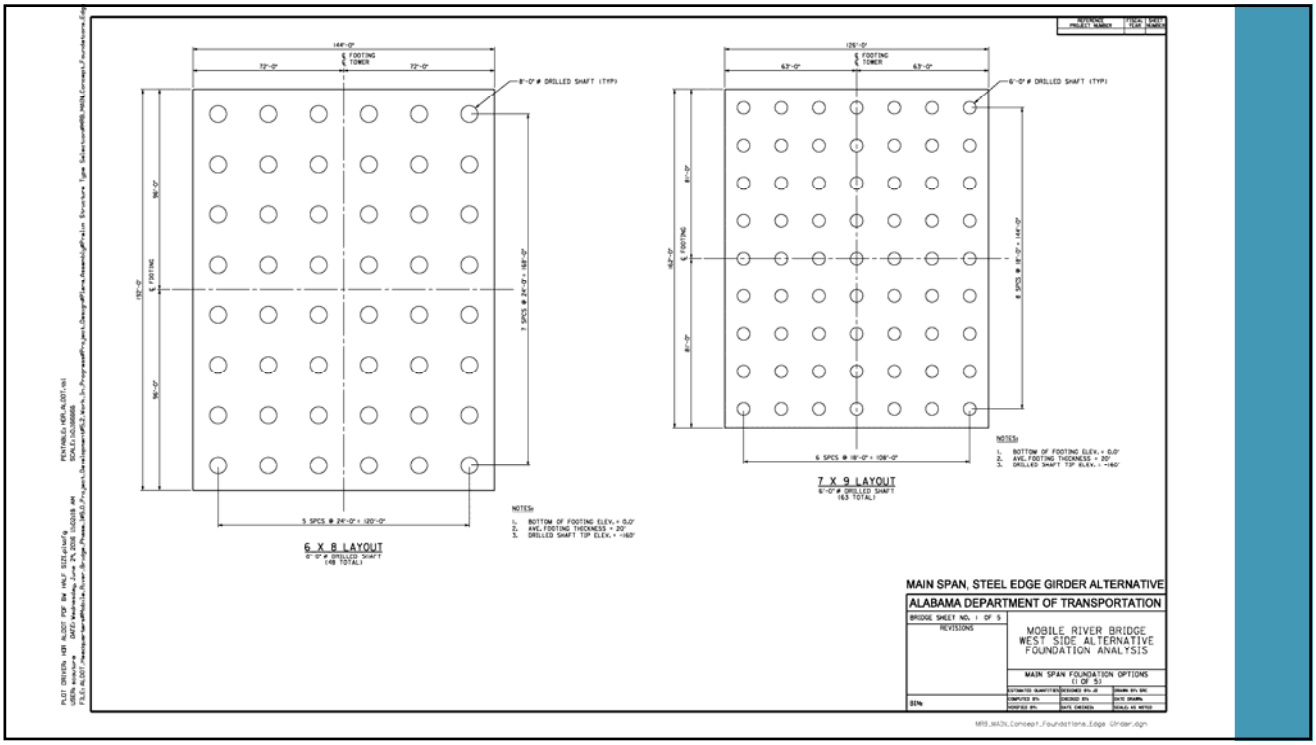

18 X 30 LAYOUT
36" PSC PILES
(540 TOTAL)
21 X 32 LAYOUT
30" PSC PILES
(672 TOTAL)
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(3 OF 5)

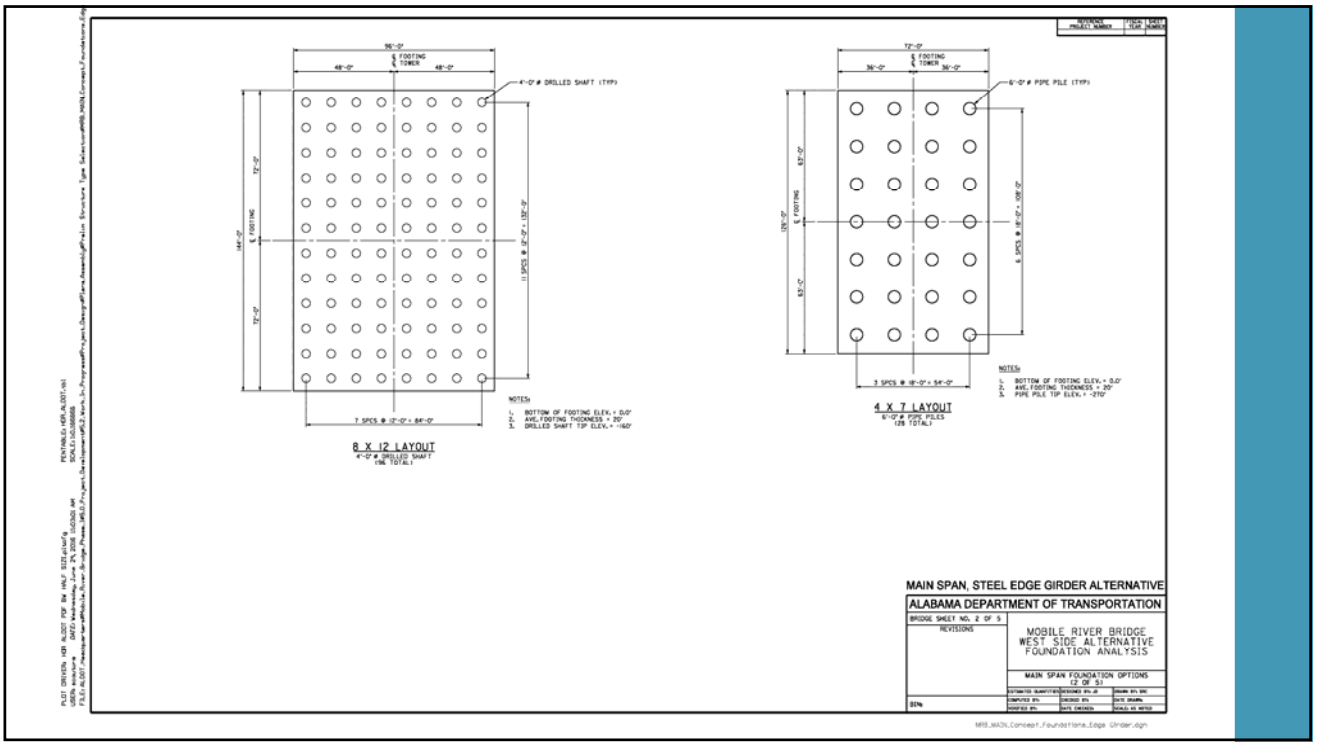

11 X 15 LAYOUT
54" ϕ CYLINDER PILES
(165 TOTAL)
9 X 14 LAYOUT
60" ϕ CYLINDER PILES
(126 TOTAL)
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(4 OF 5)

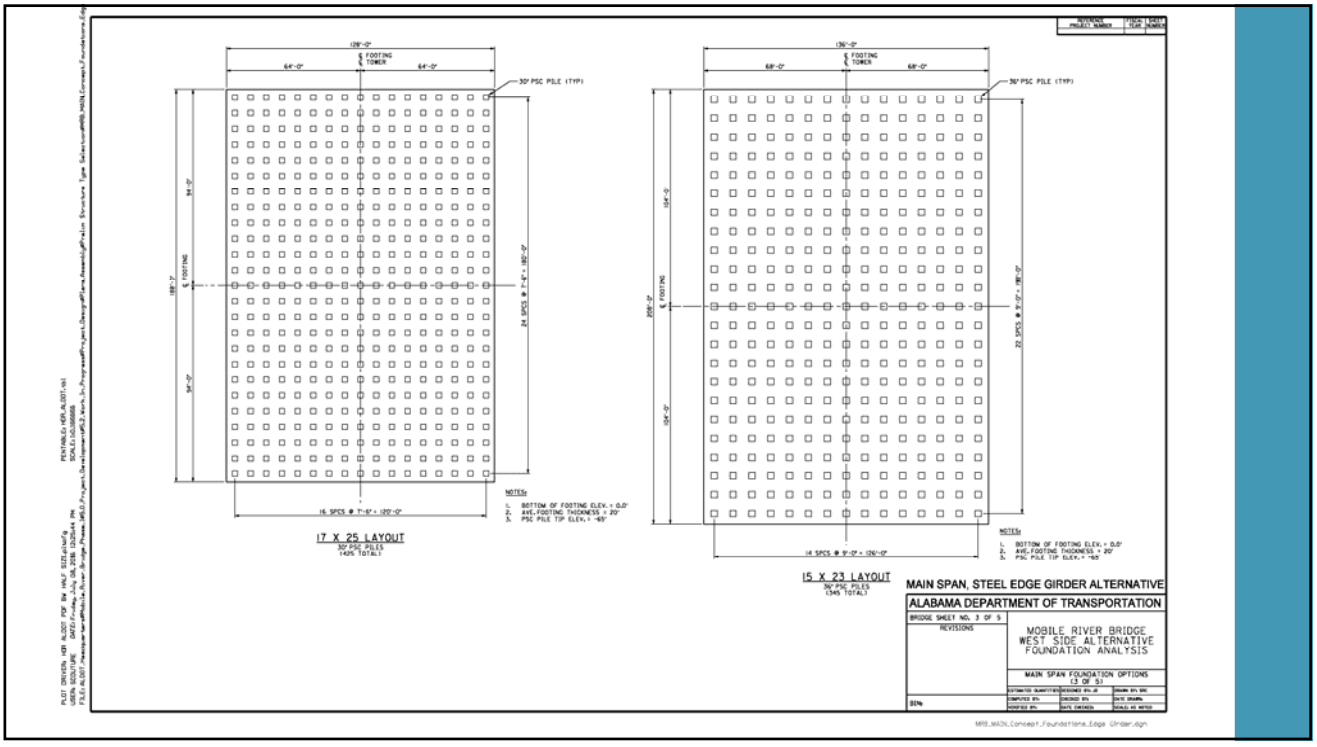

37 X 54 LAYOUT
HP 12x53 PILES
(1998 TOTAL)
24 X 36 LAYOUT
HP 14x117 PILES
(864 TOTAL)
MAIN SPAN, BOX SECTION
SUPERSTRUCTURE ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
MOBILE RIVER BRIDGE
WEST SIDE ALTERNATIVE
FOUNDATION ANALYSIS
MAIN SPAN FOUNDATION OPTIONS
(5 OF 5)

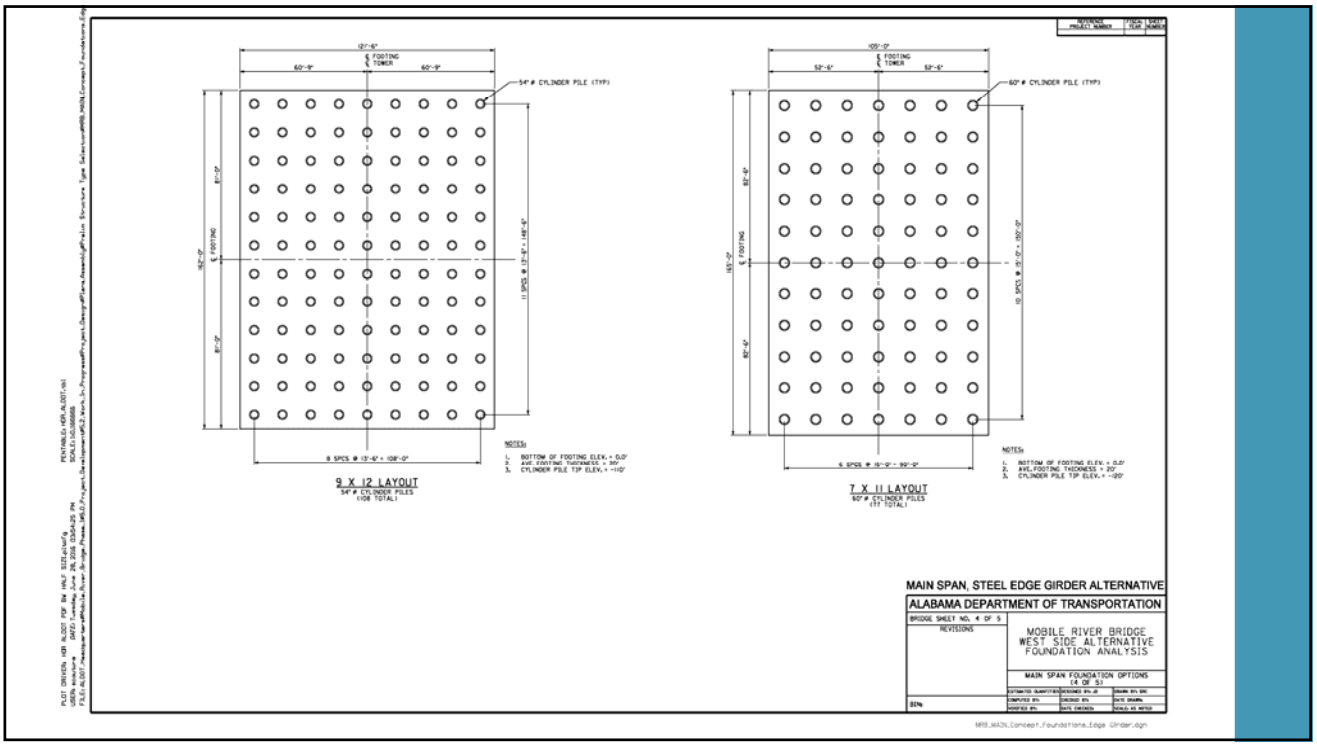
5. Design Concepts

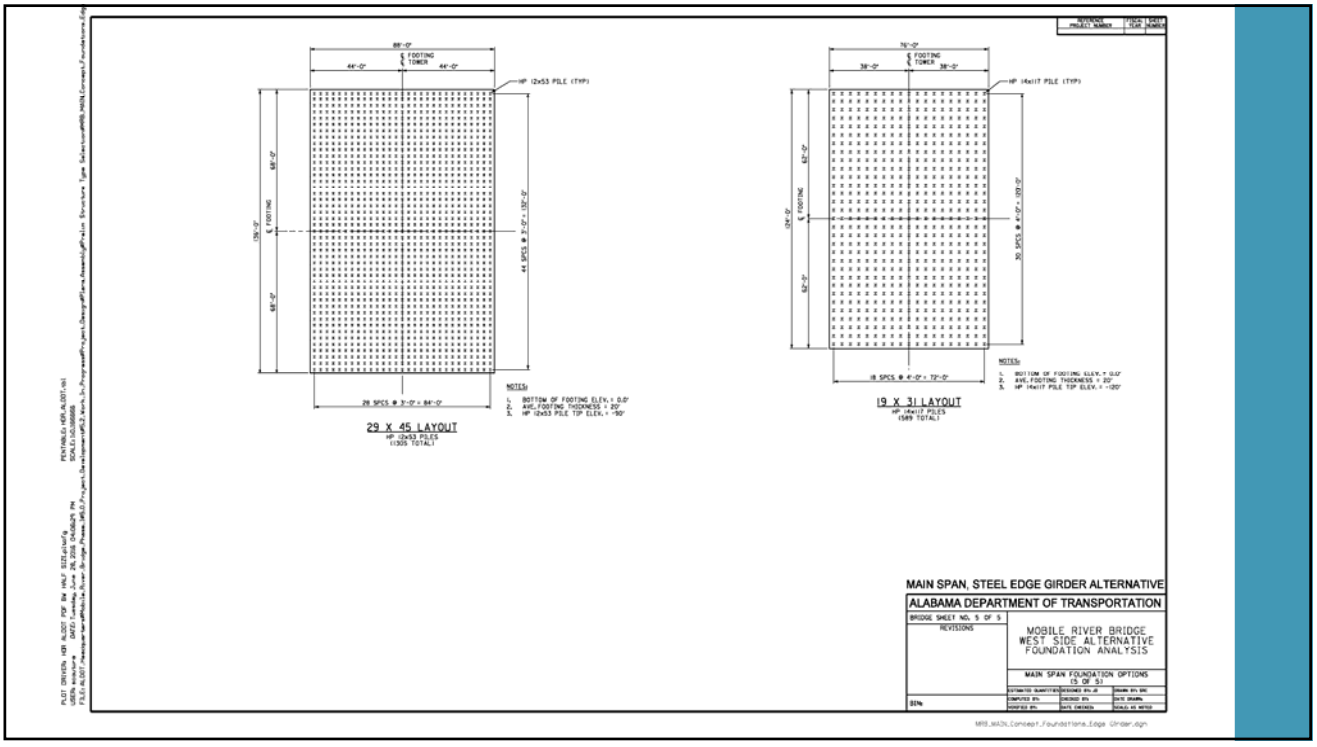
- Main Span Steel Superstructure




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
1405 TOTAL
15 X 23 LAYOUT
30" PSC PILES
1345 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)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
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)
MAIN SPAN, STEEL EDGE GIRDER ALTERNATIVE
ALABAMA DEPARTMENT OF TRANSPORTATION
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

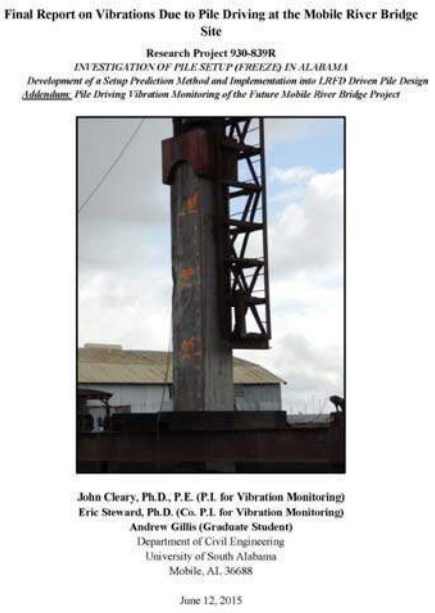
Foundation Type	Steel Superstructure	Concrete Superstructure
8’ Dia. Drilled Shafts	$ 5.5M	$ 6.9M
6’ Dia. Drilled Shafts	$ 6.3M	$ 7.6M
4’ Dia. Drilled Shafts	$ 6.1M	$ 7.4M
6’ Dia. Steel Pipe Piles	$ 5.3M	$ 5.2M
30” PSC	$ 3.7M	$ 4.1M
36” PSC	$ 3.6M	$ 3.9M
54” Concrete Cylinder Piles	$ 3.3M	$ 3.4M
60” 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.

2970 Cottage Hill Road, Ste. 190
Mobile, AL 36606
251.666.2443 ph. / 251.666.6422 fax
www.thompsonengineering.com

A THOMPSON HOLDINGS, INC. COMPANY

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.

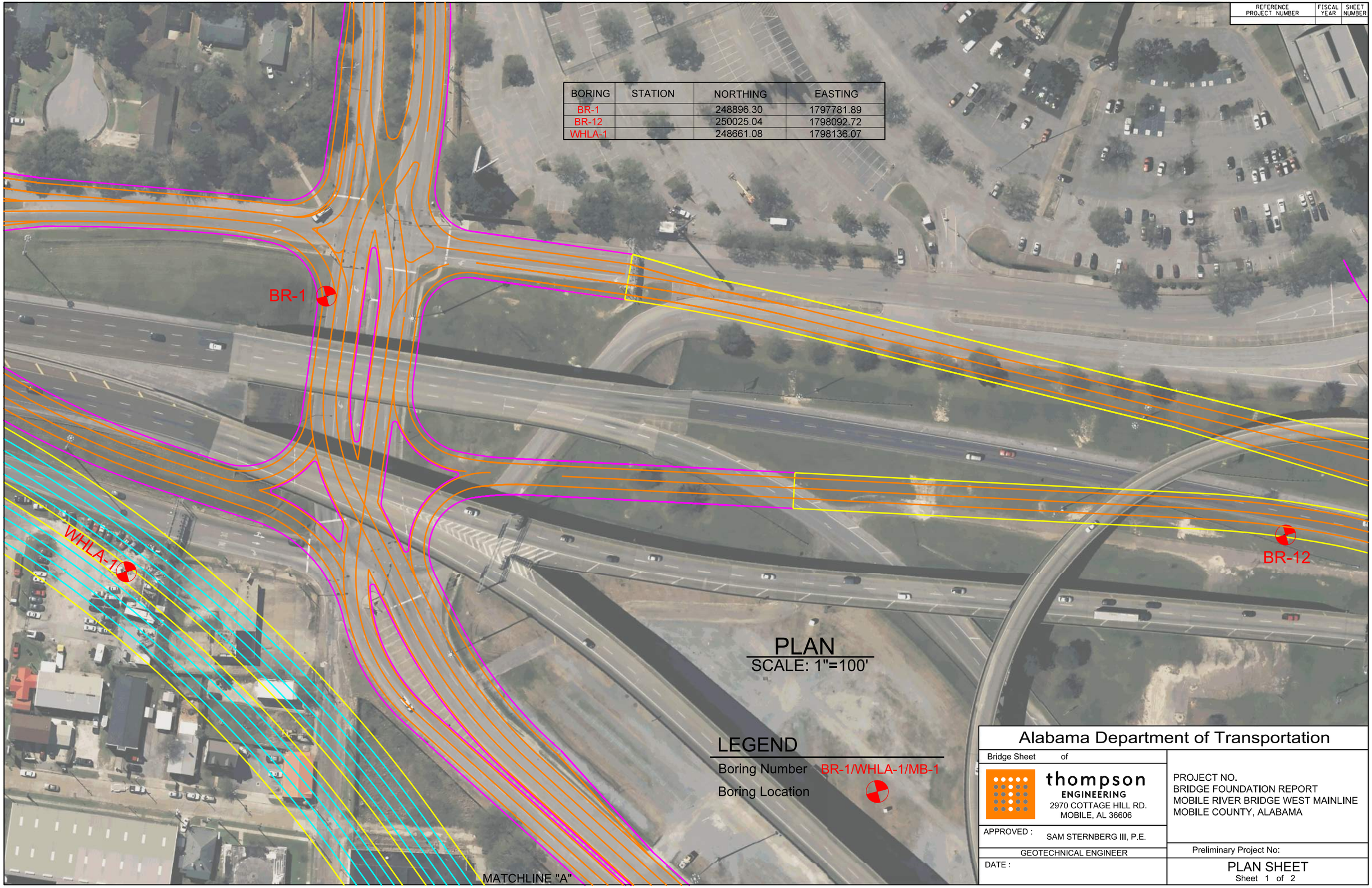
Thompson Engineering appreciates the opportunity to continue our services to Mobile River Bridge Design team with project-related geotechnical consultation. If any questions arise regarding this submittal, kindly advise.

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

MATCHLINE "A"

BORING	STATION	NORTHING	EASTING
WHLA-3		249311.95	1798929.59
MB-1		249675.65	1799417.50

WHLA-3

MB-1

LEGEND

Boring Number BR-1/WHLA-1/MB-1

Boring Location

PLAN

SCALE: 1"=100'

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

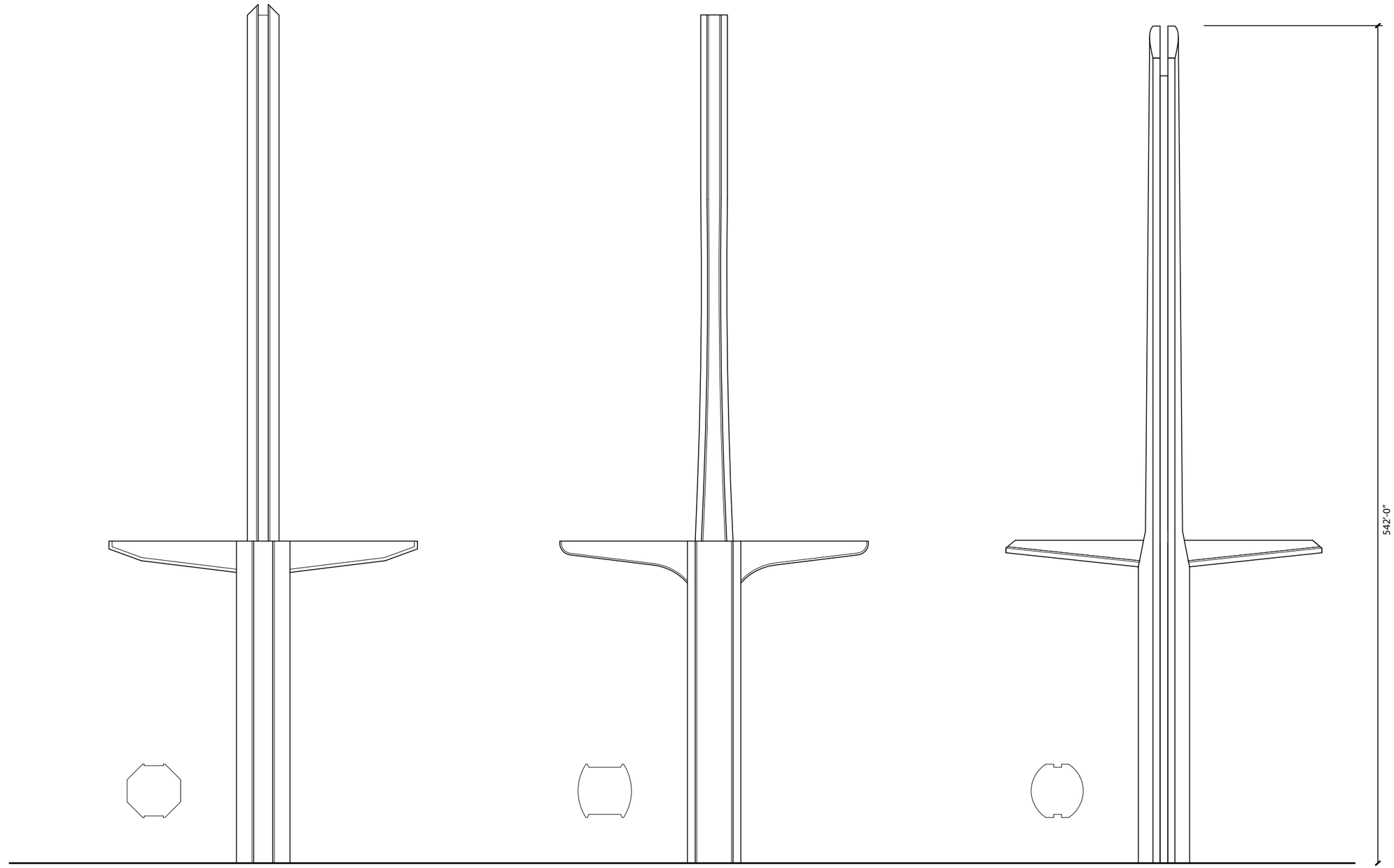
Mobile River Bridge
Steel Pipe Pile Sacrificial Thickness Project Examples

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 G:

Step 1:

Tower Aesthetic Concepts



Option 1-A

Option 1-B

Option 1-C

TOUCHSTONE ARCHITECTURE™

title: Single Pylon Concepts

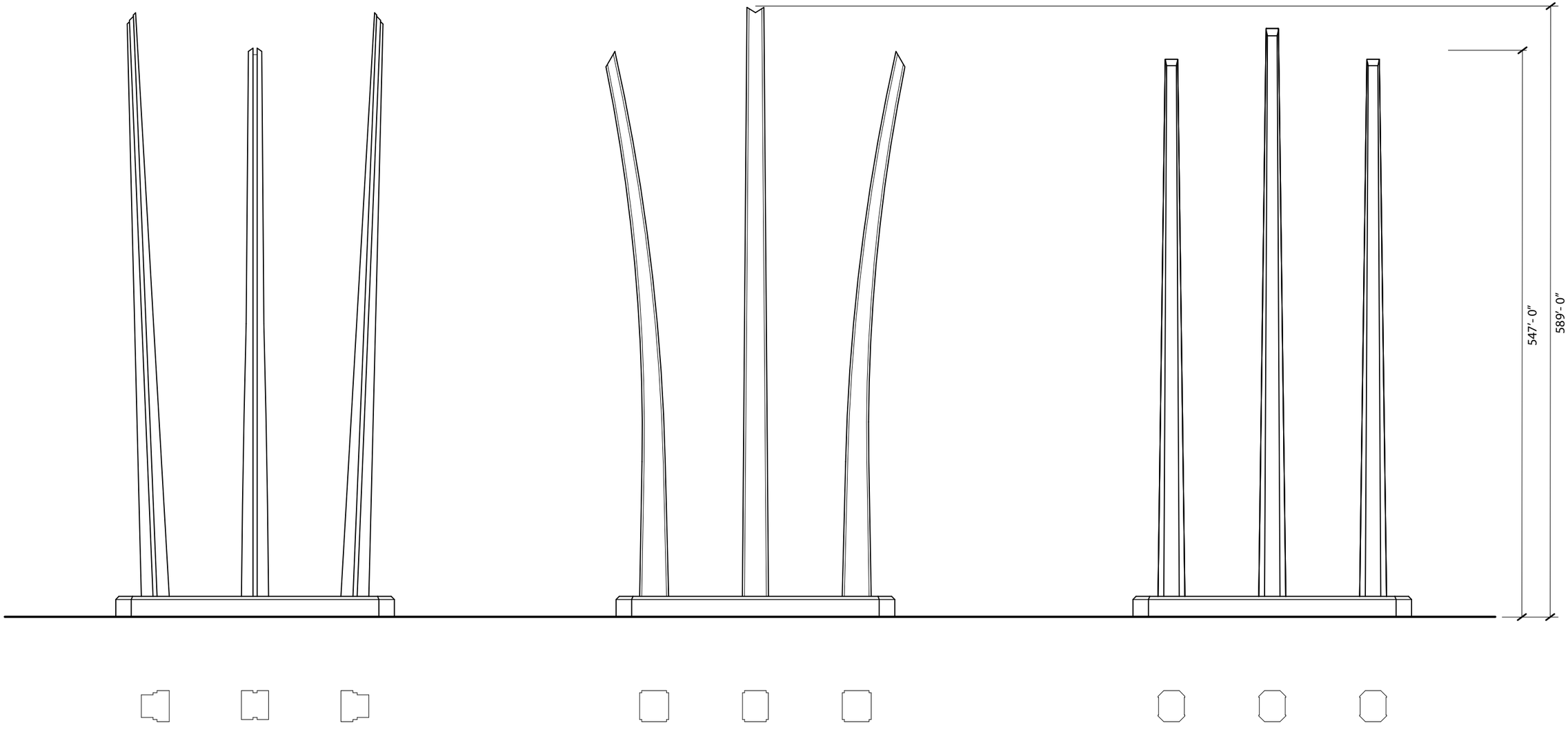
date: 26 July 2016

MOBILE RIVER BRIDGE - MOBILE, AL

This architectural rendering conveys design intent only and may not be relied upon for dimensional or illumination accuracy.

sheet: a101

© Touchstone Architecture 2016



Option 2-A

Option 2-B

Option 2-C

TOUCHSTONE ARCHITECTURE™

title: Triple Pylon Concepts

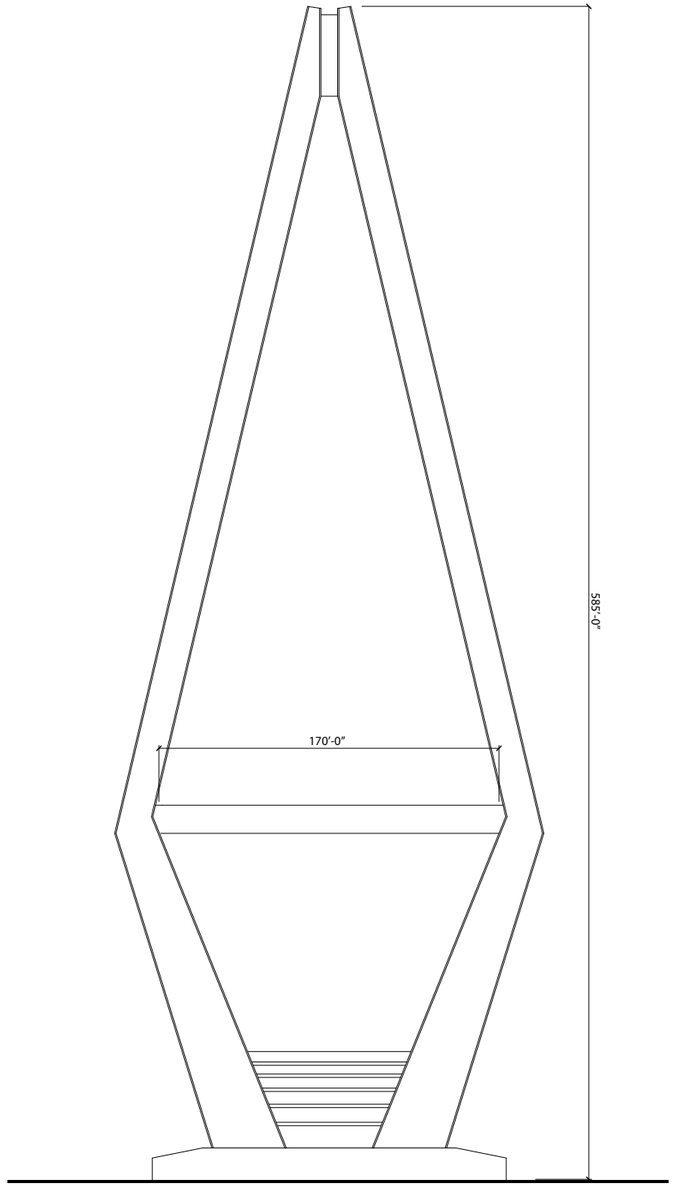
date: 26 July 2016

MOBILE RIVER BRIDGE - MOBILE, AL

This architectural rendering conveys design intent only and may not be relied upon for dimensional or illumination accuracy.

sheet: a102

© Touchstone Architecture 2016



Option 3

title: Diamond Tower Concept

date: 26 July 2016

MOBILE RIVER BRIDGE - MOBILE, AL

TOUCHSTONE ARCHITECTURE™

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sheet: a103

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Appendix H:

Inspection Access

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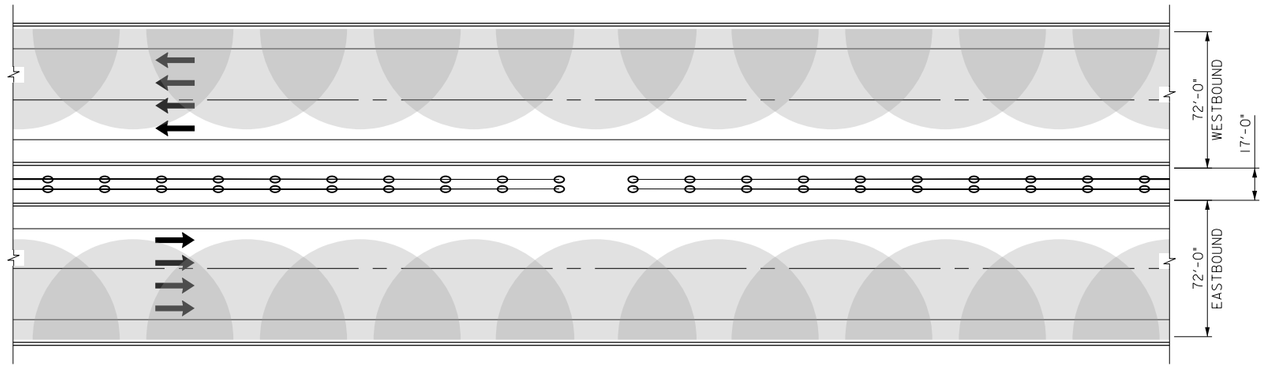

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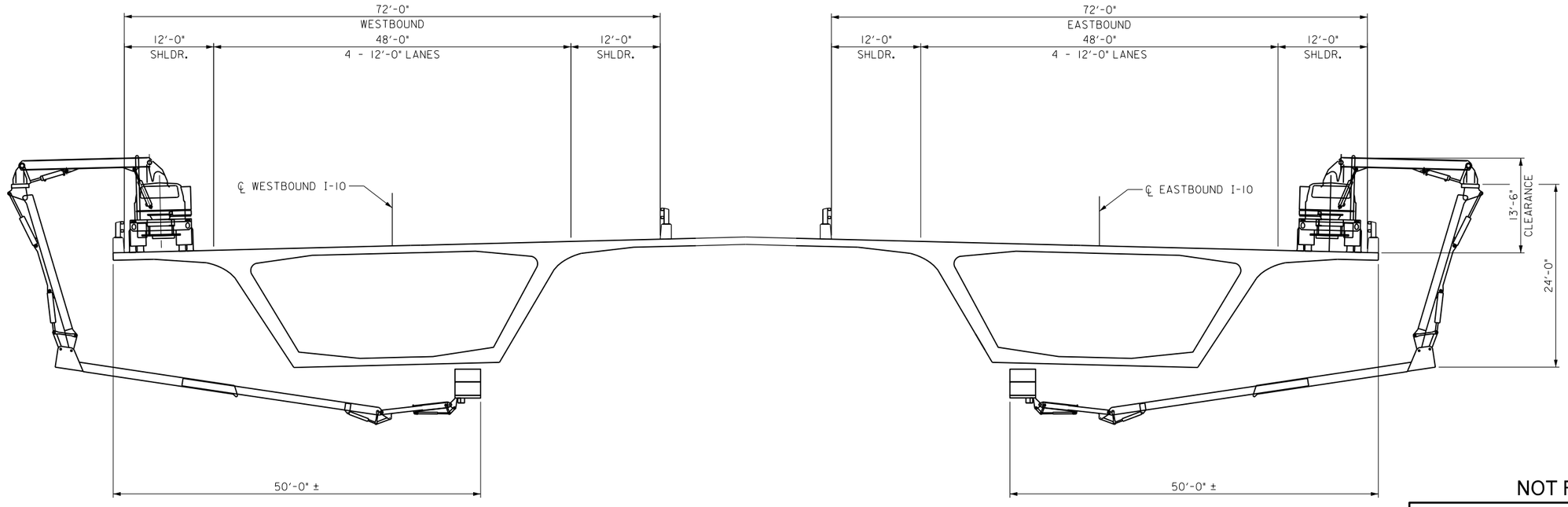

TYPICAL SECTION (BETWEEN STAYS)

SCALE: 1/8" = 1'-0"

DRAFT

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 4

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
INSPECTION TRUCK
CLEARANCES

INSPECTION ACCESS (1 OF 4)
ALTERNATIVE 1

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COMPUTED BY:	CHECKED BY:	DATE: 07/08/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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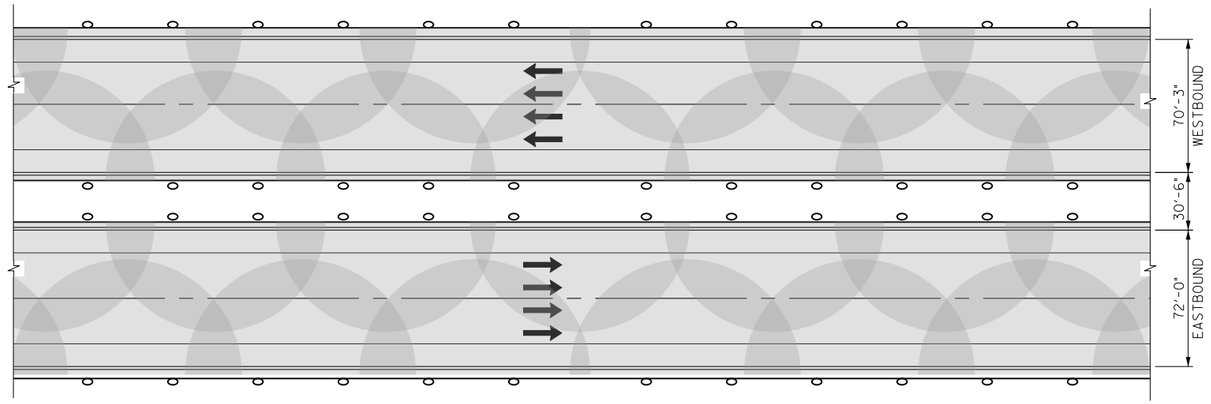

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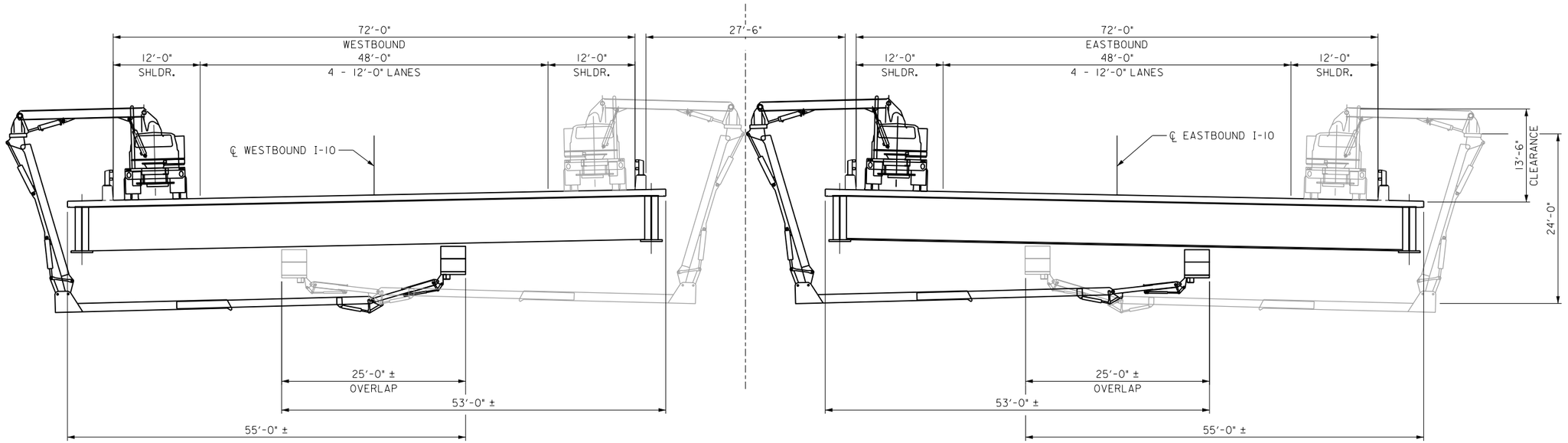

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ALABAMA DEPARTMENT OF TRANSPORTATION		
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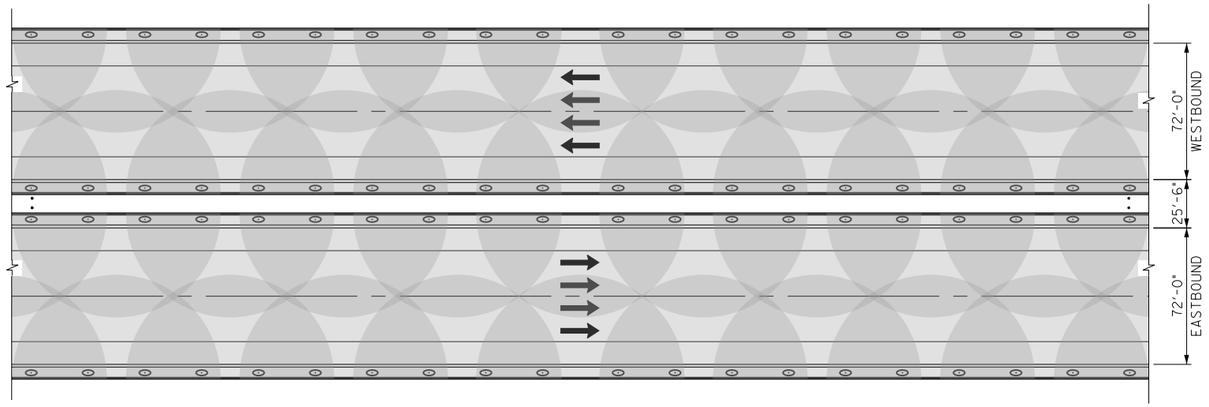

PARTIAL PLAN - MAIN SPAN

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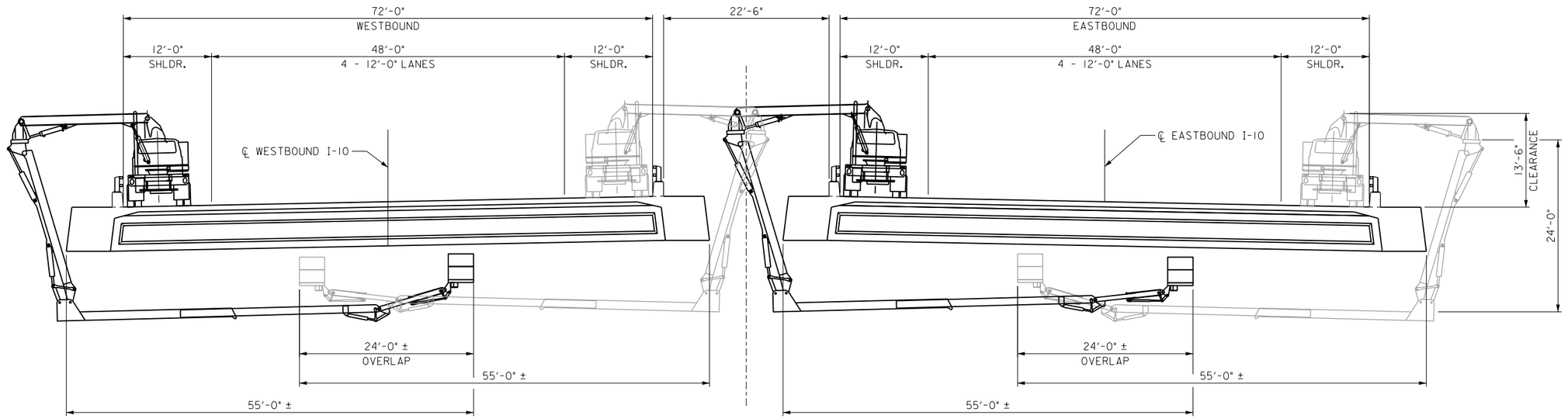

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SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

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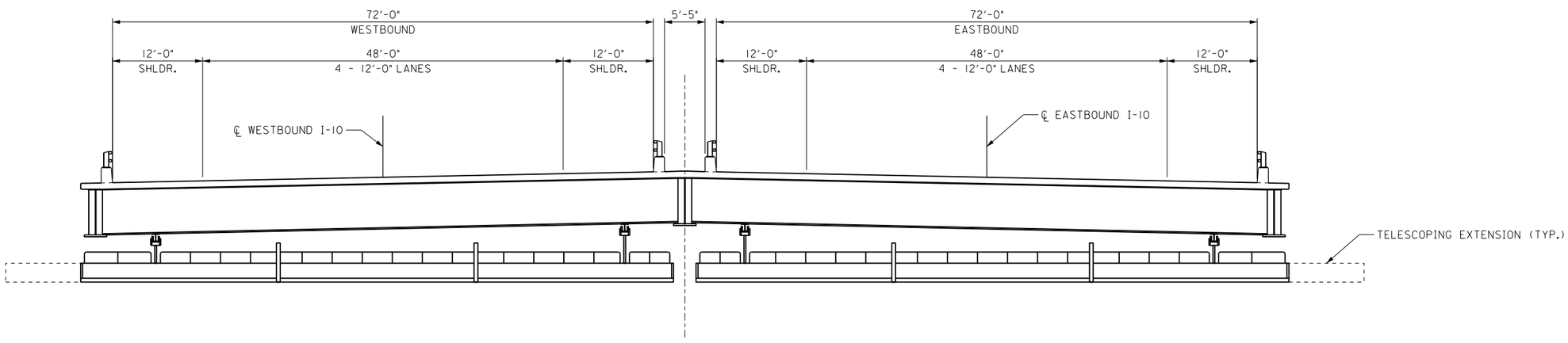

TYPICAL TRAVELLER SECTION

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NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 4

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
INSPECTION TRAVELLER

INSPECTION ACCESS (4 OF 4)
ALTERNATIVE 4

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VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

Appendix I:

Step 1:

Unit Costs

Step 1 Unit Costs

Pay Item	Unit	Unit Cost
Structural Excavation	CY	$40.00
60" Cylinder Piles	LF	$600.00
Seal Concrete	CY	$400.00
Cofferdam	SF	$60.00
Footing Concrete	CY	$550.00
Substructure Concrete	CY	$900.00
Tower Concrete	CY	$1,400.00
Stay Cables	LB	$12.00
Post-Tensioning - Substructure	LB	$7.00
Structural Steel - Substructure	LB	$3.00
Post-Tensioning - Superstructure	LB	$7.00
Structural Steel - Superstructure	LB	$2.50
Concrete Box Girder	CY	$1,550.00
Barriers	LF	$140.00
Bearings	EA	$15,000.00
Modular Expansion Joint	LF	$4,500.00
Overlay	SF	$12.00
Erection Equipment	LS	$4,000,000.00
Footing Concrete Rebar Density	LB/CY	200
Substructure Concrete Rebar Density	LB/CY	350
Tower Concrete Rebar Density	LB/CY	350
Deck Concrete Rebar Density	LB/CY	400
Reinforcement - Black	LBS	$1.25
Reinforcement - Epoxy	LBS	$1.50
Contingency		15%

Appendix J:

Project Cross-Sections

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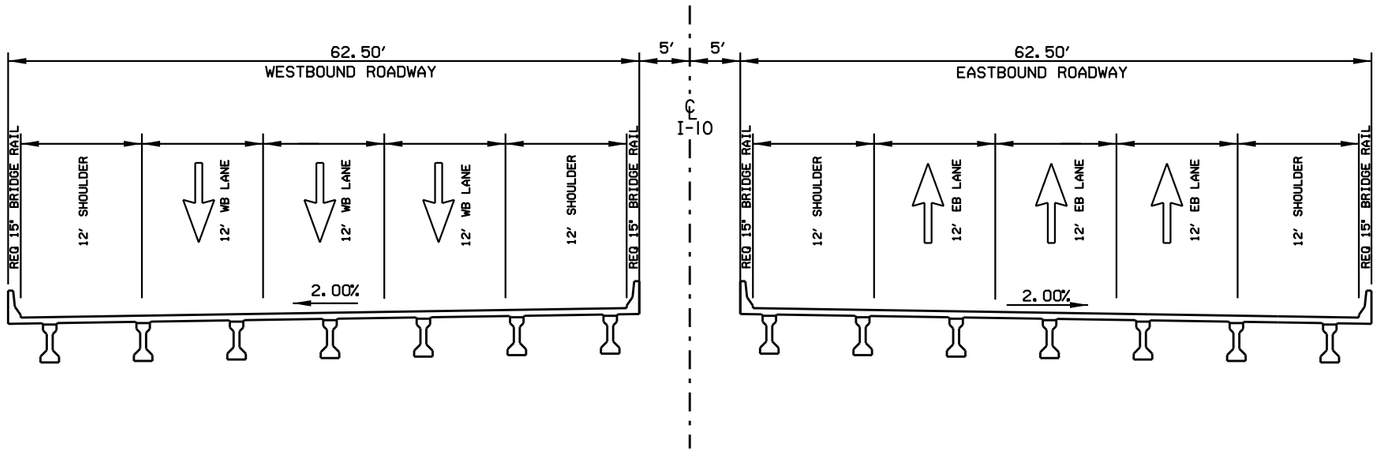
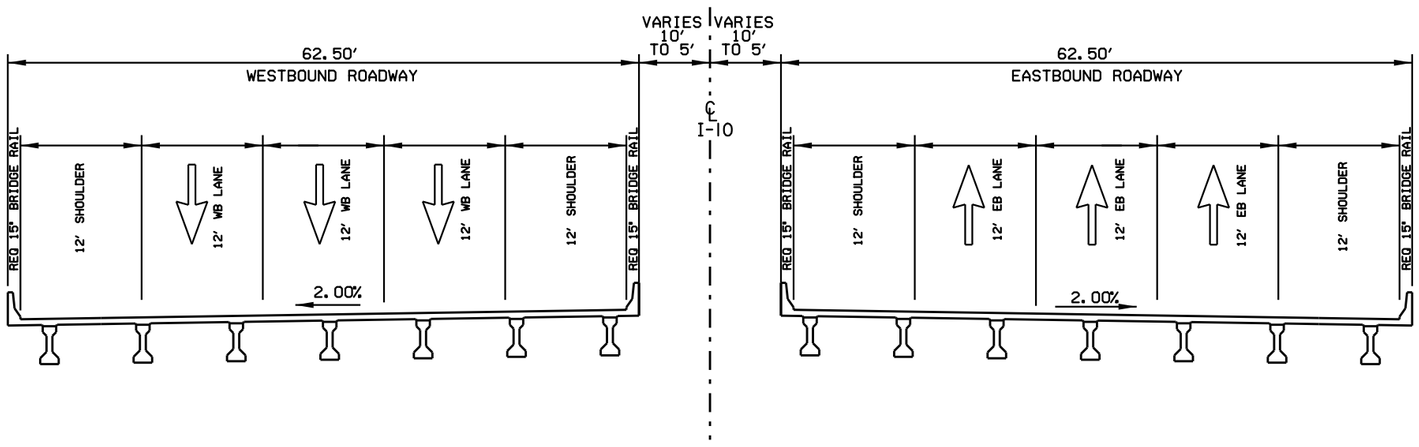


TYPICAL SECTION BAYWAY MAINLINE

STA 585+00 TO STA 598+00



TYPICAL SECTION BAYWAY MAINLINE

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DATE:	DATE:	DATE:					

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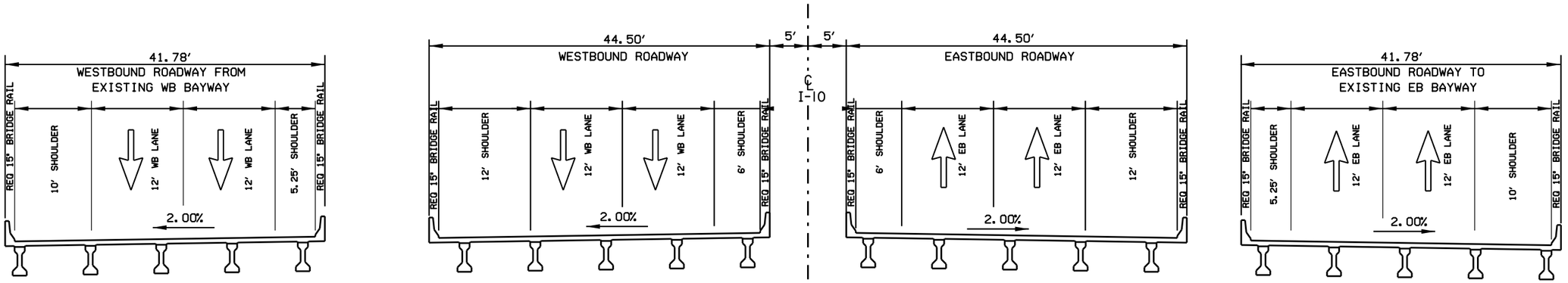

TYPICAL SECTION BAYWAY MAINLINE

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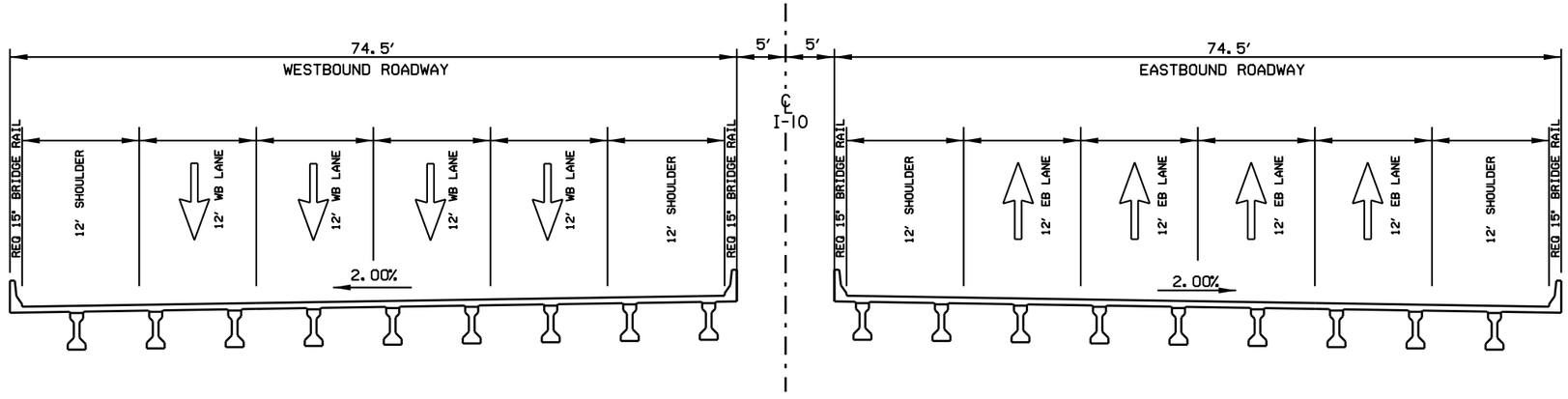

TYPICAL SECTION BAYWAY MAINLINE

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DATE:	DATE:	DATE:					

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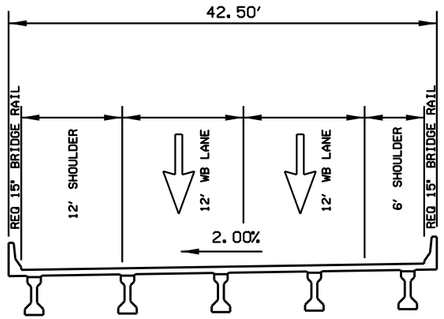
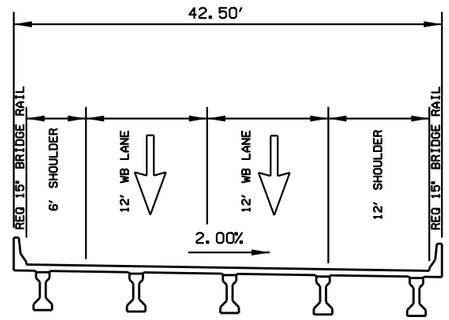


TYPICAL SECTION DOUBLE LANE RAMP

GWT EBR STA 568+75 TO STA 581+00



TYPICAL SECTION DOUBLE LANE RAMP

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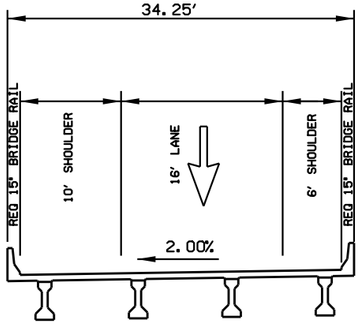
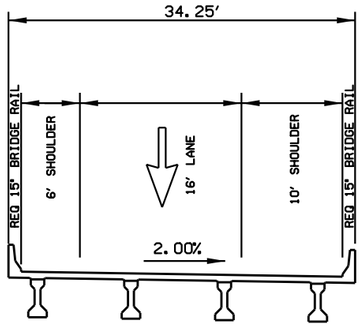
GWT WBR STA 562+50 TO STA 597+00
CW WB RP 2 STA 550+00 TO STA 562+50



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

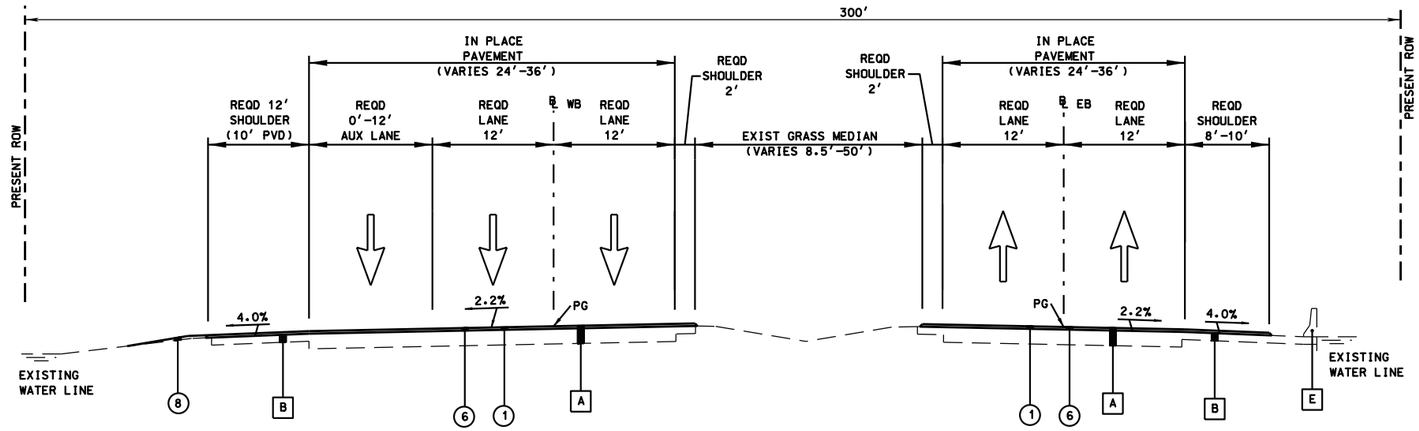


TYPICAL SECTION SINGLE LANE RAMP

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

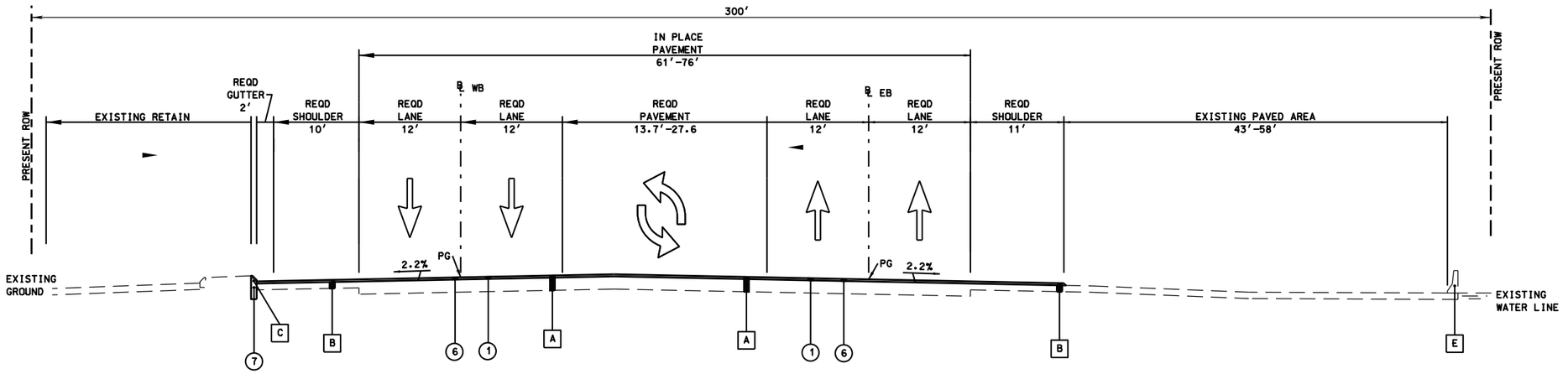
A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
E	IN-PLACE BARRIER RAIL (RETAIN)

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

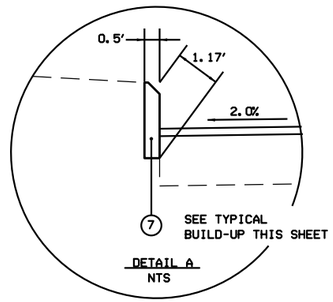
RESPONSIBLE PE:	SUPERVISOR:	DESIGNER:	PLAN SUBMITTAL	ALABAMA DEPARTMENT OF TRANSPORTATION	NOT TO SCALE	SHEET TITLE	ROUTE
DATE:	DATE:	DATE:					

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



MID BAY INTERCHANGE
TYPICAL SECTION

NTS
US 90
STA TO



DETAIL A
NTS

EXISTING MATERIAL LEGEND

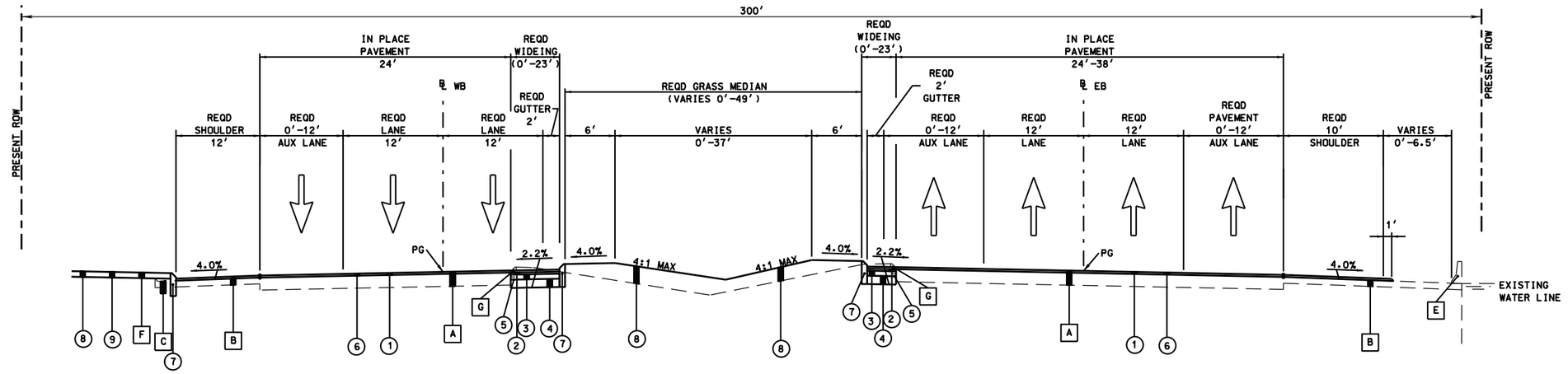
A	IN-PLACE ASPHALT PAVEMENT ROADWAY (RETAIN)
B	IN-PLACE ASPHALT PAVEMENT SHOULDER (RETAIN)
E	IN-PLACE BARRIER RAIL (RETAIN)
G	IN-PLACE CURB (REMOVE)

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

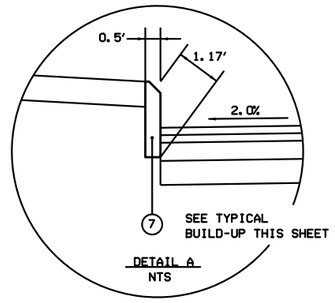
NTS
US 90
STA TO

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

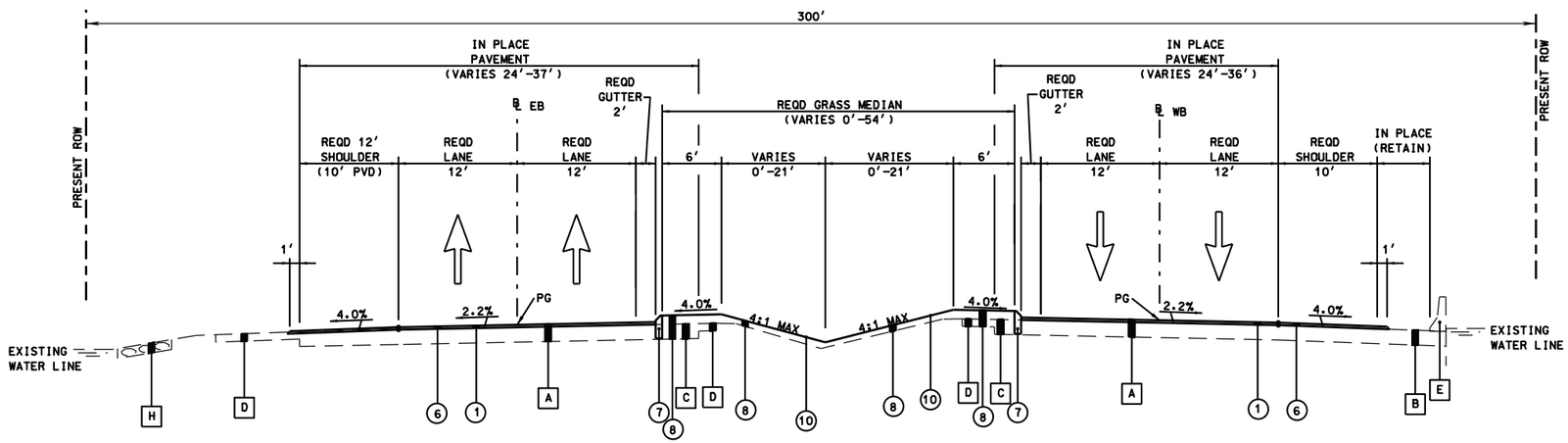
1		SUPER PAVEMENT
2		SUPER PAVEMENT UBL
3		SUPER PAVEMENT LBL
4		AGGREGATE BASE LAYER
5	401A-000	BITUMINOUS TREATMENT "A"
6	408A-052	PLANING EXISTING PAVEMENT (APPROX. 0.00" - 1.00")
7	623B-000	CONCRETE CURB, TYPE N
8	210A-000 AND/OR 210-000	UNCLASSIFIED EXCAVATION AND/OR BORROW EXCAVATION
9	614A-000	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

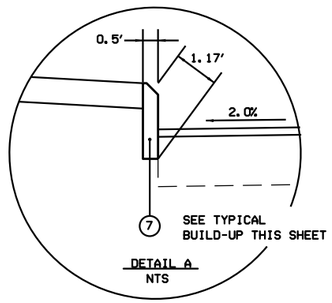
NTS
US 90
STA TO

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 K:

Vessel Collision Summary Report

I-10 Mobile River Bridge and Bayway Widening

Proposed Bridge Vessel Impact Recommendations for the I-10 Mobile River Bridge

Mobile, Alabama
August 8, 2016

1 Introduction

The purpose of this memorandum is to provide a summary of our initial findings and recommendations for the magnitude and application of vessel collision loads for the proposed main span over the Mobile River as presented to ALDOT on June 24, 2016 (Refer to Attachment A for a copy of the presentation). In accordance with the requirements of *AASHTO LRFD Bridge Design Specifications, provisions of Section 3.14* apply to all bridge components in a navigable waterway crossing with water depths over two feet, and requires such structures to be assessed and designed for vessel impacts. The design shall account for the recommended loads by either resisting the demand, or adequately protecting the structure.

The primary requirement of *AASHTO 3.14* is for a site-specific vessel collision study to be conducted; in order to ascertain the parameters necessary for a probabilistic assessment, considering site geometry and vessel traffic, and to determine a risk-based loading requirement for design. In accordance with 3.14.2, it shall remain the responsibility of the owner to establish and/or approve;

- The bridge's operational classification (I.e. Critical/Essential or Typical) – Recommendation provided in this Technical Memorandum
- The vessel traffic density in the waterway – Recommendation provided in this Technical Memorandum
- The design velocity of vessels for the bridge – Recommendation provided in this Technical Memorandum
- The degree of damage that the bridge components, including protective systems, are allowed to sustain. – Analysis is currently under way, recommendations will be provided under separate cover.

2 Vessel Collision Assessment (VCA) Approach

The proposed main span crossing of the Mobile River Bridge is the focus of this vessel collision study, which has been conducted to establish the appropriate vessel collision loading at the site. The assessment evaluated available historical commercial vessel traffic data between 2009 and 2014 and considered future growth rates in order to establish a *future fleet* that would be utilized for design and analysis. Air draft impacts were not assessed as this has previously been addressed in previous reports[1].

The AASHTO approach utilizes various factors that rely upon site-specific information and vessel traffic that is computed to calculate an Acceptance Criteria (AF) which is required to meet the minimum stipulation for the bridge type. The inverse of the AF is the return period.

For the assessment of the proposed Mobile River Bridge, our recommendation is to use a classification of "Critical" or "Essential". AASHTO defines a "Critical" or "Essential" bridge as a bridge that may be damaged to an extent acceptable to the Owner;

- **"*Critical*" or "*Essential*", having a minimum return period of 10,000 years**
    - **This corresponds to a 1% in 100 years exceedance rate, or an AF of 0.001.**

[1] Draft Environmental Impact Statement,
Appendix C, Air Draft Clearance Report, December 2012

The AF is computed for all bridge components located within a stipulated range and includes both primary piers of the proposed Mobile River Bridge. The analysis evaluates the input parameters and outputs a minimum design loading (demand) that meets an anticipated future fleet and the minimum return period.

Key assumptions:

- The siting of the proposed main span bridge and location of the main span piers is such that some bank/channel protection is available to reduce vessel impact vulnerability. The West pier has some bank protection in both directions of vessel traffic; while the East pier has bank protection in the inbound direction, and is somewhat protected in the outbound direction by a shallow channel region to the North.
- Passenger/ Cruise Vessels have been incorporated into the commercial vessel by adjusting their weight ratios to match the impact loading of similarly-sized tanker and bulk container vessels.

3 Mobile River Site Vessel Traffic

The Mobile Bay is a high-density vessel traffic region servicing major public and private cargo and tanker facilities. Using publically available real time AIS data from NOAA (marinecadastre.org), actual vessel traffic for the available years (2009 to 2014) for the bridge site was analyzed and reduced to produce a vessel traffic distribution list for use in the vessel collision analysis.

The site vessel traffic data shows a steady level of large vessel traffic, with 2009 having the most vessels and largest distribution of passenger (cruise) ships.

As a result, it is our recommendation to use 2009 as the base year for the current vessel data traffic (density) and velocities, and also as the base for the future condition.

Figure 1 Mobile River Vessel Traffic (2009)

Considering the 2009 base year, approximately 750 tankers and cargo ships (of the over 4000 recorded vessels) directly influence the vessel collision results due to their large size. Smaller vessels including barge and tow traffic are included in the overall traffic data, but due to their relative size are not included in the vessel fleet classes.

Each vessel's data was captured, including the vessel name, length (LOA), beam (Width), transiting draft (depth), and velocity. All of these properties were carried through the data analysis to assist with fleet classification and associated speeds that represent this bridge site.

The following tables present the fleet class, characteristics and vessel class speeds for each direction utilized in the AASHTO analysis for 2009 (considered the base year).

2009 Vessel Fleet ***Vessel Class Speed, Knots***

Class	*DWT Class, tonnes*	*Inbound*	*Outbound*	*Average*
V-1	160,000-180,000	4.3	4.1	4.2
V-2	140,000-159,999	4.2	4.8	4.5
V-3	120,000-139,999	4.0	5.0	4.5
V-4	100,000-119,999	4.5	5.1	4.8
V-5	80,000-99,999	4.4	4.8	4.6
V-6	60,000-79,999	4.7	5.0	4.9
V-7	40,000-59,999	4.7	5.3	5.0
V-8	20,000-39,999	5.1	4.9	5.0
V-9	10,000-19,999	5.4	6.1	5.8
V-10	1,000-9,999	5.6	6.5	6.0

Table 1 Fleet Class Characteristics

4 Future Growth Rates

In order to assess the bridge's vulnerability to vessel collision over the 75 year life of the bridge, a future fleet definition is necessary in order to determine the demand. Although the past data analyzed was finite and highly accurate, the future data is less certain and necessitates an estimation based upon growth rates in order to present a future vessel fleet for evaluation. After considering a range of growth models in order to evaluate the sensitivity of the results, including research produced by Florida DOT and predictions generated from a nearby Gulf port, we are recommending a compound growth rate of 2% across the fleet. The resulting future vessel traffic numbers are considered to be conservative, upper-bound values given the current traffic trends. The following chart presents the 2009 and future (+75-year) projection for each class of vessel DWT

The application of the future fleet compound growth rates produces a fleet distribution illustrated in the figure below.

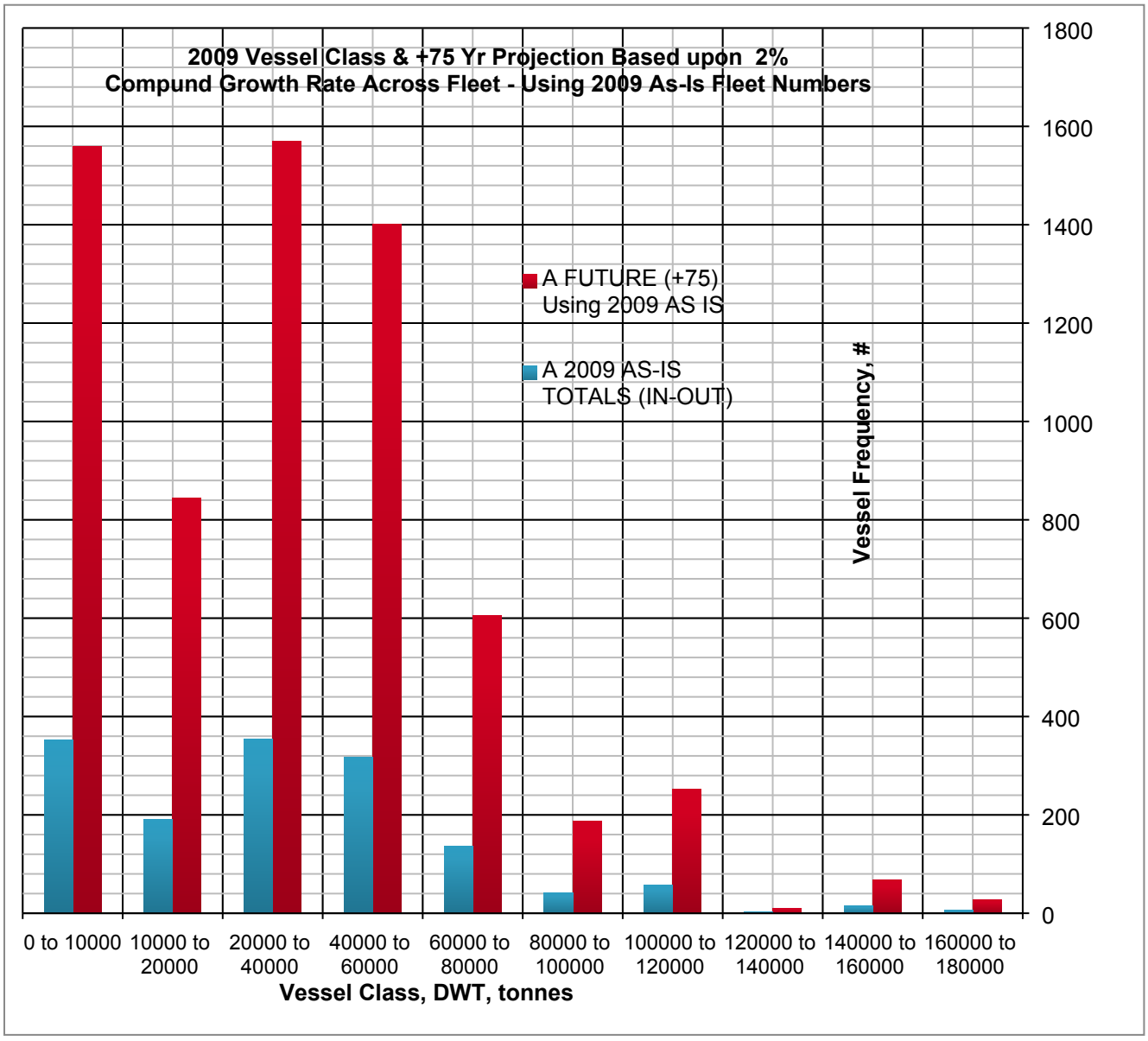


Figure 2 2009 Vessel Class and 75Year Projections

5 Analysis Results and Force Recommendations

The following table summarizes the AASHTO vessel collision analysis results, indicating the lateral pier design force for both the current traffic data and the future fleet condition based upon the aforementioned growth factors. **The recommended design vessel collision force, using the traffic values for the future fleet, and the vessel speeds from the 2009 fleet, is 17,000 kips.**

	2009 ACTUAL AS-IS (CASE A)			FUTURE with AS-IS (CASE A2)		
	No Rate Increase, AS-IS			2% Across the Fleet Increase		
DWT Class	IN	OUT		IN	OUT	
160000-180000	**3**	**3**		**14**	**14**	
140000-159999	**8**	**7**		**36**	**31**	
120000-139999	**0**	**0**		**5**	**5**	
100000-119999	**28**	**29**		**124**	**129**	
80000-99999	**22**	**20**		**98**	**89**	
60000-79999	**69**	**68**		**305**	**301**	
40000-59999	**157**	**160**		**694**	**707**	
20000-39999	**179**	**176**		**791**	**778**	
10000-19999	**97**	**94**		**429**	**416**	
1000-9999	**178**	**175**		**787**	**773**	
Pier Location	Pier #					
		Design Vessel/Ps			***Design Vessel/Ps***	
1E-DN	P6	50,000	*DWT*		70,000	
		14,601	*Ps, kips*		16,661	
1W-DN	P5	50,000	*DWT*		70,000	
		15,022	*Ps, kips*		17,046	
1E-UP	P2	70,000	*DWT*		110,000	
		15,731	*Ps, kips*		18,662	
1W-UP	P1	50,000	*DWT*		92,754	
		13,342	*DWT*		16,991	
Hp to meet AF	**H_p**	13,000			**17,000**	
(AF>10,000 = OK)	**AF**	10,530			10,386	

Table 2 Vessel Collision Analysis Results

6 Degree of Damaged and Protective Measures

Once the vessel collision force is calculated it will be applied to the bridge structural model to assess the level of damage and to identify potential mitigation strategies such as reinforcing of key structural elements and/or provide supplementary protective measures. Table 3 below uses five cable-stayed bridges over a major navigation channel to illustrate the various protection strategies that have been used previously on similar bridges.

This analysis is underway way and the result will be presented to ALDOT under separate cover.

Vessel Collision Design Similar Projects							
Project Name	***Project Photo***	***Project Location***	***Project Description***	***Design Load (main pier) (kips)***	***Vessel Speed***	***Protection System***	***Notes***
Tappan Zee Bridge Replacement		New York	Cable-Stay – 1,200 ft. Main Span (Under Construction)	23,000	10 knots	No Protection	Capacity of bridge with one pile removed at service level is evaluated
Port Mann Bridge		Vancouver, B.C.	Cable-Stay – 1,542ft. Main Span (Completed 2012)	16,900	Not Readily Available	No Protection	Designed for Ultimate Capacity
CCHB (Proposal)		Corpus Christi, Texas	Cable-Stay – 1,520ft. Main Span	18,700	7 knots	Grounding considered, Permanent Cofferdam to protect slender piles	Designed for Ultimate Capacity
Veterans' Glass City Skyway		Toledo, Ohio	Cable-Stay - 612.5ft. Main Span (Completed 2007)	23,000	Not Readily Available	No Protection	Designed for Essentially Elastic criteria
Sunshine Skyway		Tampa, Florida	Cable-Stay - 1,200ft. Main Span (Completed 1987)	12,000	Not Readily Available	Dolphin and Rock Islands	Designed for Ultimate Capacity

Table 3 Vessel Collision Design of Similar Projects

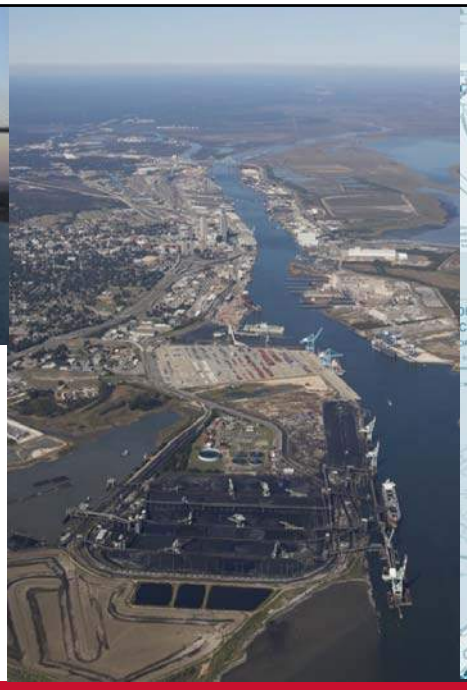
Appendix A – June 24, 2016 Presentation

Mobile River Bridge

Vessel Collision Analysis

June 24, 2016

HDR



General Presentation Outline

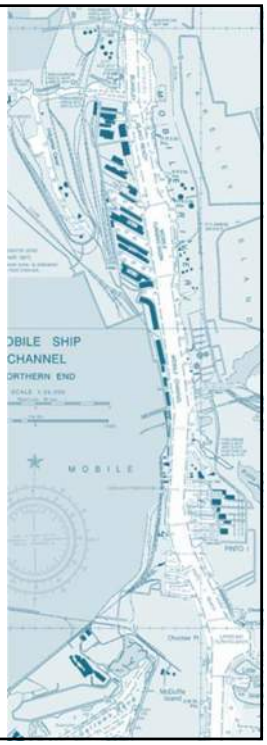
1. Purpose of Analysis
2. Studies to Date
3. AASHTO Vessel Collision Analysis Approach/Methodology
4. Mobile River Bridge Site Vessel traffic
5. Application of Loads to Structure
6. Considerations for Loading
7. Acceptance Frequency (AF) Analysis for Site
8. Future Fleet Considerations
9. Summary and Recommendations
10. Comments/Input

01 Purpose of Analysis

- Purpose of Analysis
  - AASHTO requirement;
    - *3.14.1All bridge components in a navigable waterway crossing, located in design water depths over 2.0 ft shall be designed for vessel impact*
    - Design to resist vessel collision forces AND/OR adequately protect

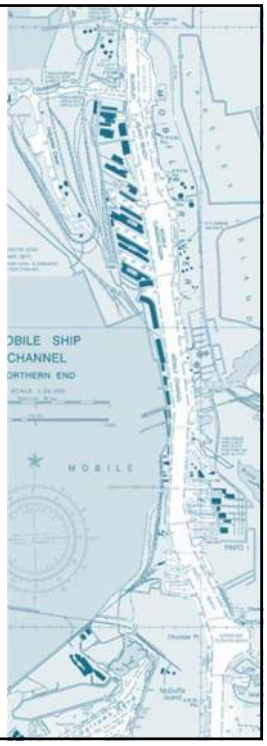
3.14.2—Owner's Responsibility

The Owner shall establish and/or approve the bridge operational classification, the vessel traffic density in the waterway, and the design velocity of vessels for the bridge. The Owner shall specify or approve the degree of damage that the bridge components, including protective systems, are allowed to sustain.



Scope:

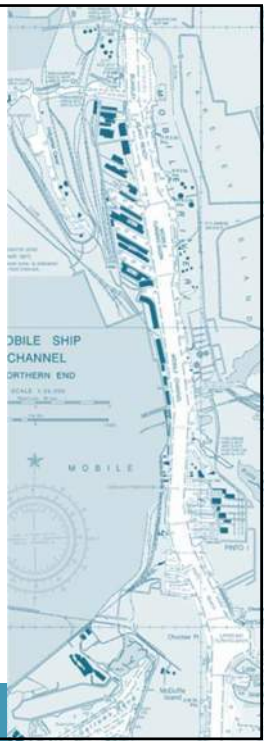
- Main Span Focus
  - Assess Current snapshot of Commercial Vessel Traffic the operational classification (critical or typical)
  - The vessel traffic density
  - Understand Future Commercial Vessel Traffic Scenario
  - Foundation and Pier Vulnerability
  - The design velocity of vessels for the bridge
    - Tankers
    - Bulk Containers
    - Passenger vessels
    - Barges and tows
  - Vessels not Included in Analysis:
    - Naval Vessels
    - Recreational vessels
    - < 1,000 DWT
    - Security Considerations
  - Impacts due to Air Draft Not Included
    - Assumed addressed



02 Studies to Date

Studies to Date

- Draft Environmental Impact Statement
  - FHWA-AL-EIS-14-01-D
  - July 2014
- Appendix C:
  - » Air Draft Clearance Report (December 2012) : 215 ft.

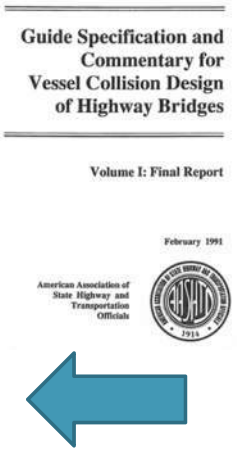


03 AASHTO Vessel Collision Analysis Approach/Methodology

Guide Specifications for Vessel Collision

3 Original Methods:

- MI – Simplified, conservative approach, good when barges are predominant and only in certain prescriptive cases can it be used.
- MII – Probabilistic (expected approach) uses actual traffic data, vessel characteristics and details with the site geometry to assess the probable “design vessel” which meets an “acceptance criteria”.
- MIII – Risk + Economic approach to better determine the costs of disruption to the bridge, and other potential factors as a means to understand the protection needs.


Guide Specification and Commentary for Vessel Collision Design of Highway Bridges

Volume I: Final Report

February 1991

American Association of State Highway and Transportation Officials

Current AASHTO 3.14 Requires Method II

Loading Combinations

Extreme Event II

- CV – Vessel Collision

3-14 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, SEVENTH EDITION, 2014

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time: EQ	BL	IC	CT	CV
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL, IM & CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL, IM & CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Acceptance Criteria (AF)

- "Critical" or "Essential" is Assumed
  - 10,000 year Return Period (1/AF)
  - 0.01 in 100 years (1% in 100 years)
  - AF = 0.0001
- Calculated for the entire bridge within 6 x LOA
- Statistics and probabilities managed with site specific conditions

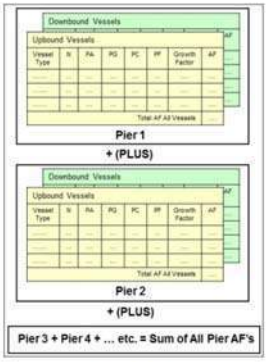


Figure 4 – Computation of Annual Frequency of Collapse (AF)

4.8.2 Design Vessel Acceptance Criteria

The design vessels shall be selected based on the bridge operational classification, vessel, bridge, and waterway characteristics in accordance with the following acceptance criteria for the total bridge:

- *Critical/Essential Bridges*. The acceptable annual frequency of collapse, *AF*, of critical/essential bridges shall be equal to, or less than, 0.01 in 100 years ($AF = 0.0001$).
- *Typical Bridges*. The acceptable annual frequency of collapse, *AF*, of typical bridges shall be equal to, or less than, 0.1 in 100 years ($AF = 0.001$).

The acceptable annual frequency of bridge collapse for the total bridge as determined above shall be distributed over the number of pier and span elements located within the waterway, or within the distance

4.8.3 Annual Frequency of Collapse

The annual frequency of bridge element collapse shall be computed by:

$$AF = (N)(PA)(PG)(PC)(PF) \qquad (4.8.3\text{-}1)$$

where:

AF = annual frequency of bridge element collapse due to vessel collision;

N = annual number of vessels classified by type, size, and loading condition which can strike the bridge element;

PA = probability of vessel aberrancy;

PG = geometric probability of a collision between an aberrant vessel and a bridge pier or span;

PC = probability of bridge collapse due to a collision with an aberrant vessel; and

PF = adjustment factor to account for potential protection of the piers from vessel collision due to upstream or down stream land masses, or other structures, that block the vessel.

AF shall be computed for each bridge element and vessel classification. The summation of all element *AF*s equals the annual frequency of collapse for the entire bridge structure.

Probability of Aberrancy (PA)

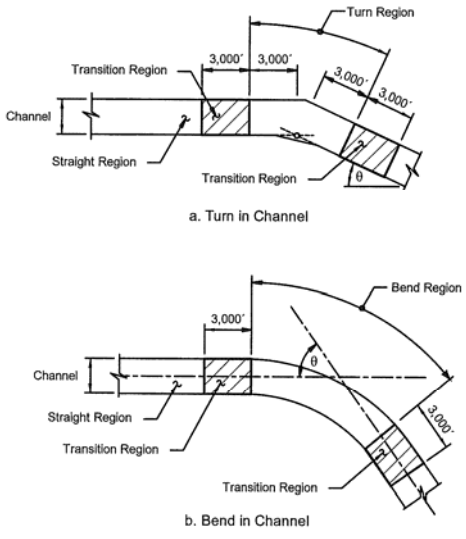


Figure 4.8.3.2-1—Waterway Regions for Bridge Location

$$PA = BR(R_B)(R_C)(R_{XC})(R_D) \qquad (4.8.3.2\text{-}1)$$

BR = A historical accident data value specific to bridges or barges.

R_b= Factor for Channel Characteristics, Straight, transition, or a bend/turn region

Others to account for currents and traffic density.

Geometric Probability (PG)

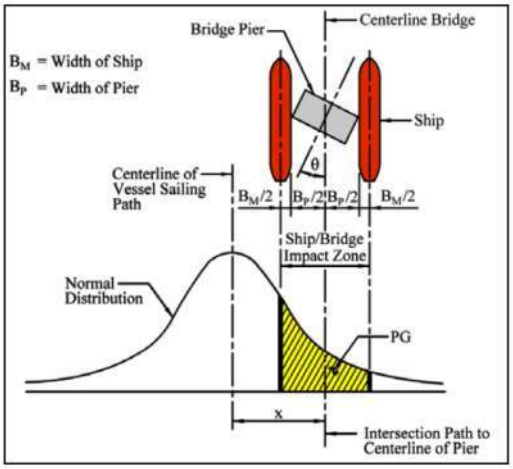


Figure 5 – Geometric Probability of Pier Collision

PG → A geometric probability associated with the possibility that an aberrant vessel (losing control) could hit a pier or bridge element.

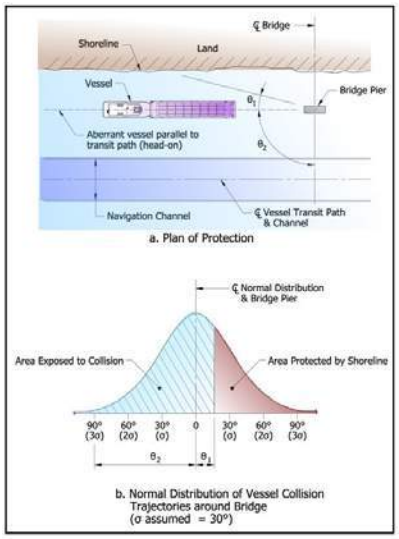
Based upon a normal distribution using the mean as the CL transit path, the std Dev is equal to the LOA of the vessel design fleet, and extends 3 x LOA either side of the CL transit path.

Protection Factor (PF)

This Adjusts the AF for full or partial protection such as that provided by artificial islands, dolphins, piling systems or other, including the river banks.

In some cases and for particular directions, a wharf , waterway feature, or parallel bridge may protect the bridge element in question, affecting the AF.

Full protection means PF= 0.0, and none is 1.0



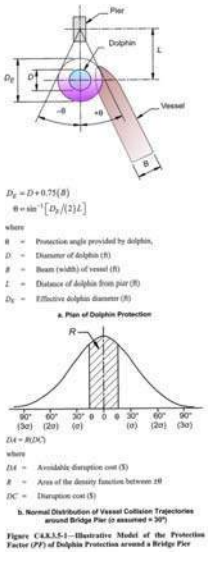
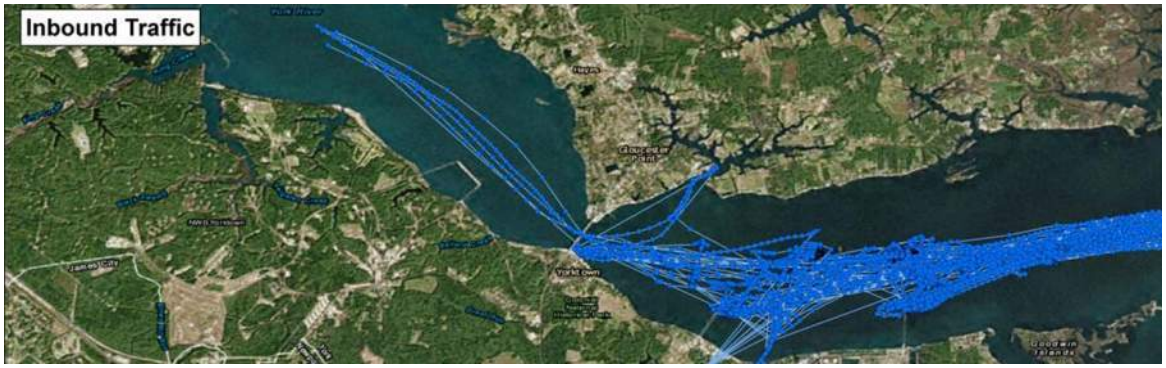


Figure 6 – Protection Factor Illustration

Vessel Frequency (N)

- The number of vessels for that class, adjusted for the physical and geometric possibility that each vessel could collide with that particular bridge element:
- Depth at location may reduce fully loaded vessels for that direction
- Airspace clearance (Vertical Nav Envelope) effects to that class (Perhaps large low vessels won't hit the superstructure).



Example NOAA Real Time Data

Probability of Collapse (PC)

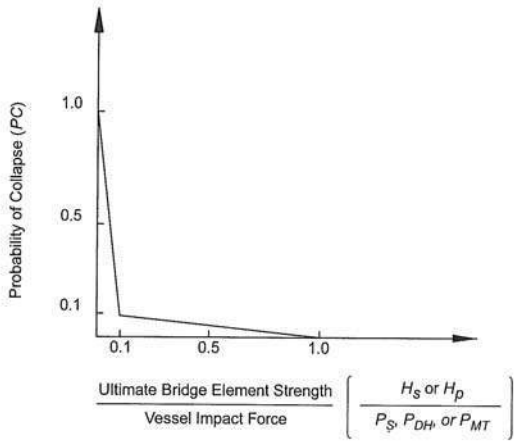


Figure 4.8.3.4-1—Probability of Collapse Distribution

PC includes a combination of effects based upon impact speed, vessel size and mass:

- Hp
- Ps

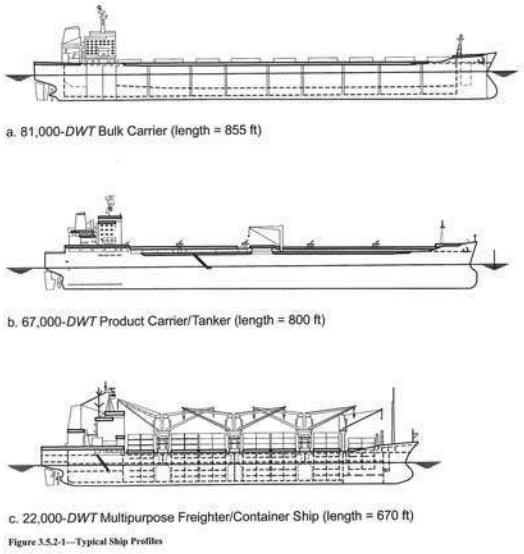
Computed based upon analysis of Ps (Vessel impact loading) with the Ultimate Pier Resistance (to be determined).

Iterative Process:

The Ultimate pier lateral resistance (Hp) required so that the AF is met for the bridge: Iterated until met.

Probability of Collapse (PC) (cont'd)

Sample Ship Impact Forces (P_s)



a. 81,000-*DWT* Bulk Carrier (length = 855 ft)

b. 67,000-*DWT* Product Carrier/Tanker (length = 800 ft)

c. 22,000-*DWT* Multipurpose Freighter/Container Ship (length = 670 ft)

Figure 3.5.2-1—Typical Ship Profiles

AASHTO Guide Specification C3.9.1-1

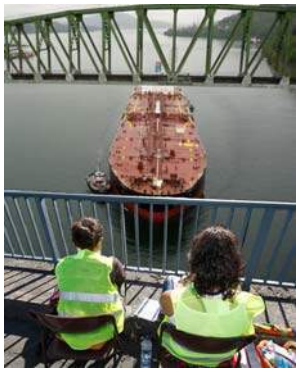
Vessel Type	Displacement Tonnage (Carrying Capacity), Tons	Typical AASHTO Impact Forces, Kips (10 knots Assumed)
Barges	3,000 to 6,000	2,000
Ships:		
Small Ship	3,000	6,900
Medium	10,000	12,300
Large	40,000	25,800
Extra Large	100,000	41,300

Where to Obtain "N" – Number of Vessels?

- NOAA – Real Historical Data from 2009 to 2014
  - Vessel rich (I.e. tankers, cruise, cargo)
  - Key to information for Mobile, AL.
- WSCS/USAC – Passed the Point historical data
  - Barge Information rich
  - Less critical information (Large Vessels Govern)
- Movable Bridge Logs
- Interviews with Barge Operators
- Lock Gate Records
- The *Tired* and True Approach

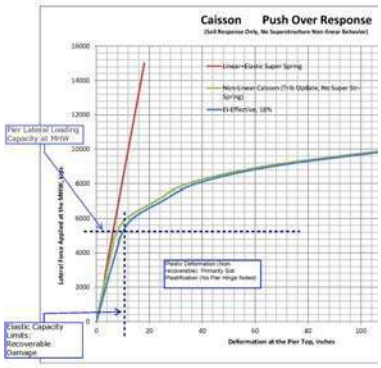
MarineCadastre.gov

Welcome to the US Army Corps of Engineers
Navigation Data Center

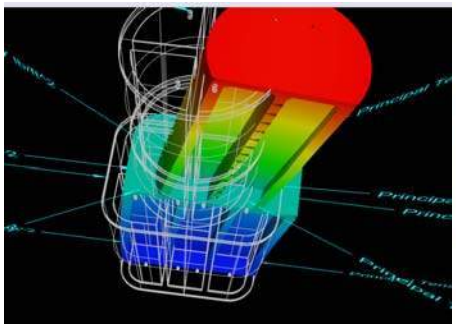


Ultimate Pier Capacity (Hp):

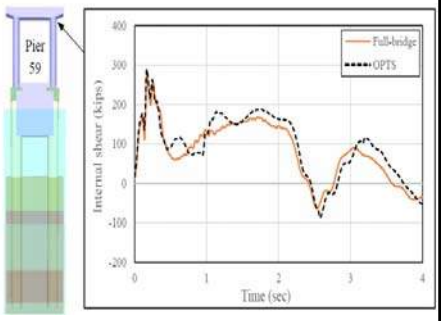
- The AASHTO analysis iterates to the required demand that the bridge is then designed for.
- The new bridge can be allowed to plastify to absorb damage:
  - AASHTO 3.14.13 allows for inelastic behaviour and redistribution of force effects as long a “no collapse” criteria is met
  - Owner approval for damage state required (By AASHTO 3.14.2)
    - Fully Elastic – All damage to vessel, bridge has minor non-structural damage, remains in full serviceability
    - Partial damage – Damage to vessel and structure



Push Over Response of a bridge Foundation



Stress Analysis and Response of a bridge Support (Pier)

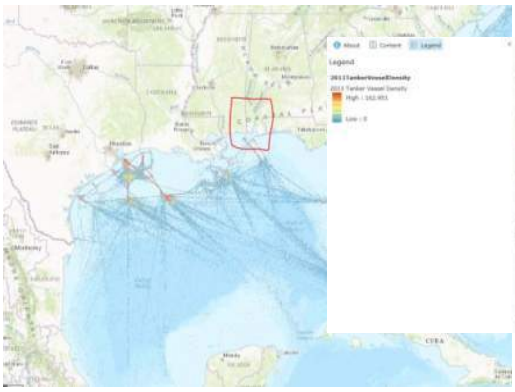


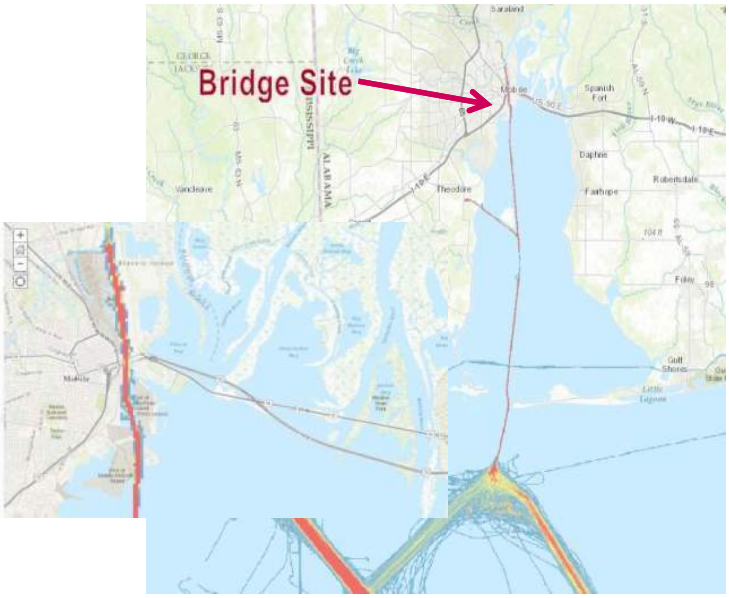
Time-history Response of a bridge Foundation to dynamic/impact loading

04 Mobile River Bridge Site Vessel Traffic

Mobile Bay: High Density, Large Vessels

- Major link for ASPA and other maritime facilities
- Future Cruise Terminal
- Maritime Industry
- Public and private cargo terminals
- 25.1 Million tons of Cargo (2011)*
- Long term ship building - Austal USA

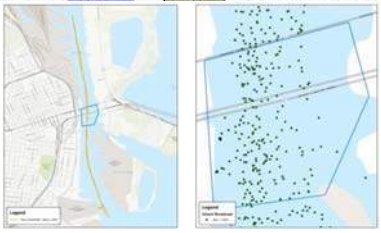




NOAA Data: heat maps

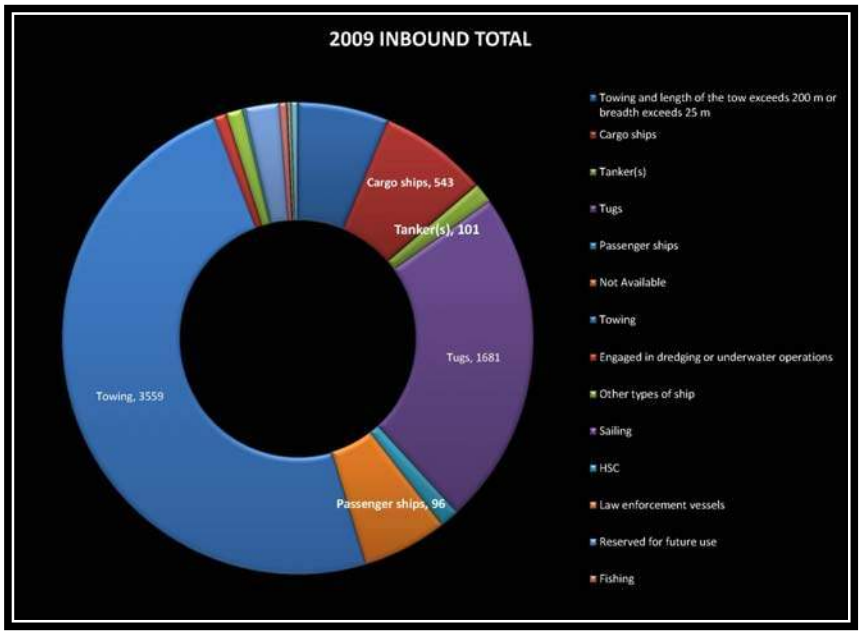
Mobile Bay: High Density, Large Vessels

Unique MMSI	In # mo	Vessel Name	VesselType	Length	Length	Width	Width	dwt
235058391	3	CMA CGM SWORDFISH	Cargo ships	294	964.32	32	104.96	needed
235059882	9	CMA CGM TARPON	Cargo ships	294	964.32	32	104.96	
235052035	2	CMA CGM WHITE SHARK	Cargo ships	294	964.32	32	104.96	
235012140	2	CMA CGM ORCA	Cargo ships	294	964.32	32	104.96	
235012510	3	CMA CGM BLUE WHALE	Cargo ships	293	961.04	32	104.96	
235012300	3	CMA CGM DOLPHIN	Cargo ships	293	961.04	32	104.96	
235054096	2	CMA CGM MARLIN	Cargo ships	293	961.04	32	104.96	
235054102	2	CMA CGM KINGFISH	Cargo ships	293	961.04	32	104.96	
367685000	1	MAERSK UTAH	Cargo ships	292	957.76	32	104.96	
303540000	6	SEALAND RACER	Cargo ships	292	957.76	32	104.96	
368346000	6	SEA LAND EAGLE	Cargo ships	292	957.76	32	104.96	
338401000	8	SEALAND METEOR	Cargo ships	292	957.76	32	104.96	
366196000	7	SEALAND CHAMPION	Cargo ships	291	954.48	32	104.96	
368991000	4	REGULATOR	Cargo ships	288	944.64	32	104.96	
235005690	1	CAPE OSPREY	Cargo ships	280	918.4	45	147.6	
538003322	1	OVERSEAS YELLOWSTONE	Tanker(s)	280	918.4	40	131.2	
259975000	1	CATHERINE KNUTSEN	Tanker(s)	277	908.56	44	144.32	
538001641	2	GENMAR ORION	Tanker(s)	274	898.72	48	157.44	
538090158	1	SKS SENNE	Tanker(s)	274	898.72	48	157.44	
538001210	1	NORDIC JUPITER	Tanker(s)	274	898.72	48	157.44	
259833000	1	SKS SINNI	Tanker(s)	274	898.72	48	157.44	
636014185	1	ARIADNI	Tanker(s)	274	898.72	50	164	
308329000	1	ZENITH SPIRIT	Tanker(s)	272	892.16	48	157.44	
259820000	1	WILANA	Tanker(s)	271	888.88	44	144.32	
538090184	1	HELLESPONT TRINITY	Tanker(s)	270	885.6	46	150.88	
235010440	1	ZIM XIAMEN	Cargo ships	260	852.8	32	104.96	
247185700	8	DRUMMOND VOYAGER	Cargo ships	258	846.24	42	137.76	
636091513	9	ZIM JAMAICA	Cargo ships	254	833.12	32	104.96	
636091515	2	ZIM IBERIA	Cargo ships	254	833.12	32	104.96	
240773000	5	CMA CGM LOTUS	Cargo ships	253	829.84	32	104.96	
366204000	3	PLESS	Cargo ships	250	820	32	104.96	
538003233	1	OVERSEAS ACADIA	Tanker(s)	250	820	45	147.6	



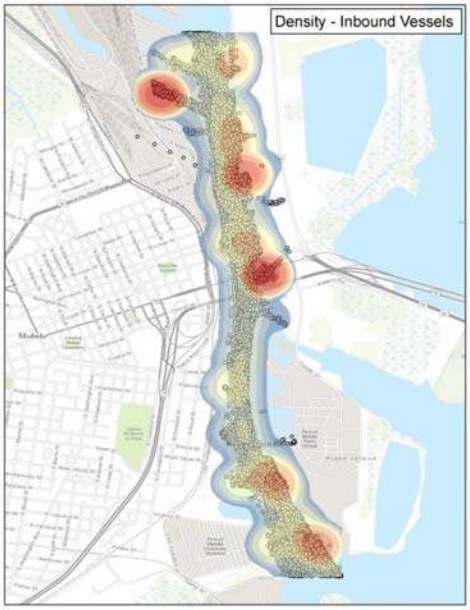
NOAA Site Specific Data Shown

Mobile Bay: High Density, Large Vessels



NOAA Reduced Data Source at Bridge Site

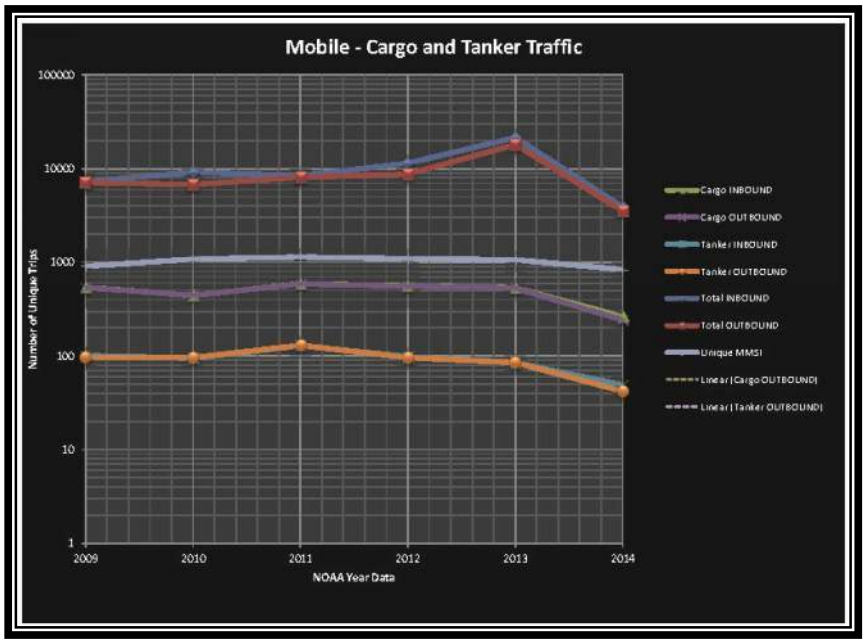
Mobile Bay: High Density, Large Vessels



NOAA Data Source at Bridge Site: All Vessels

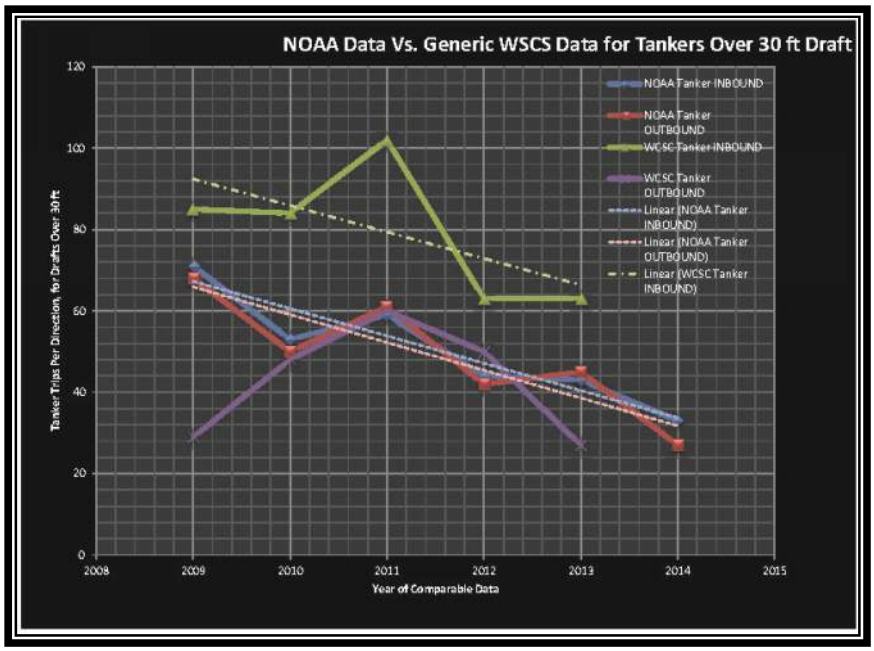
NOAA Reduced Data Source at Bridge Site:
> 656 ft . & >3.5 knots

Mobile Bay: High Density, Large Vessels



NOAA Reduced Data Source at Bridge Site

Mobile Bay: High Density, Large Vessels



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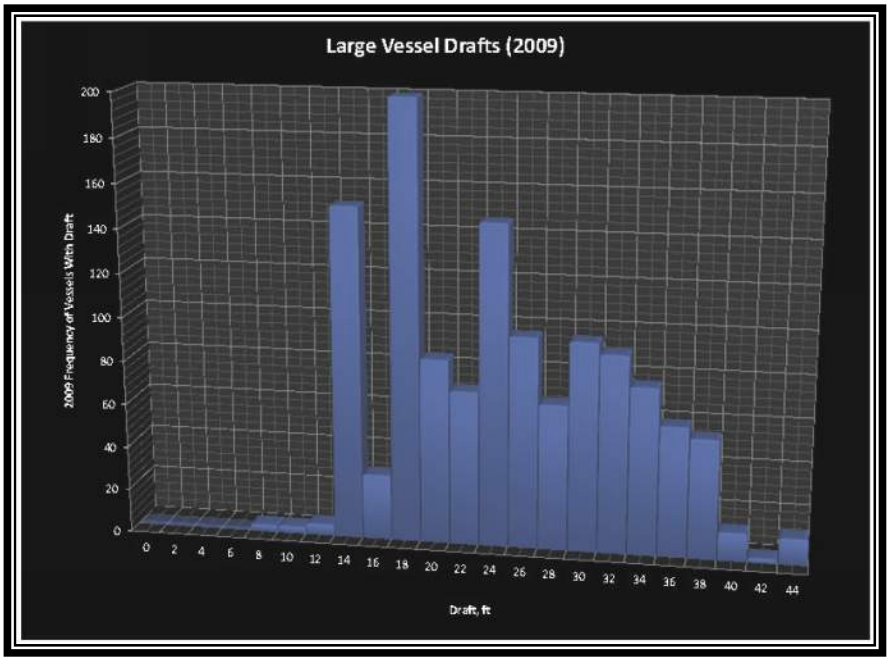
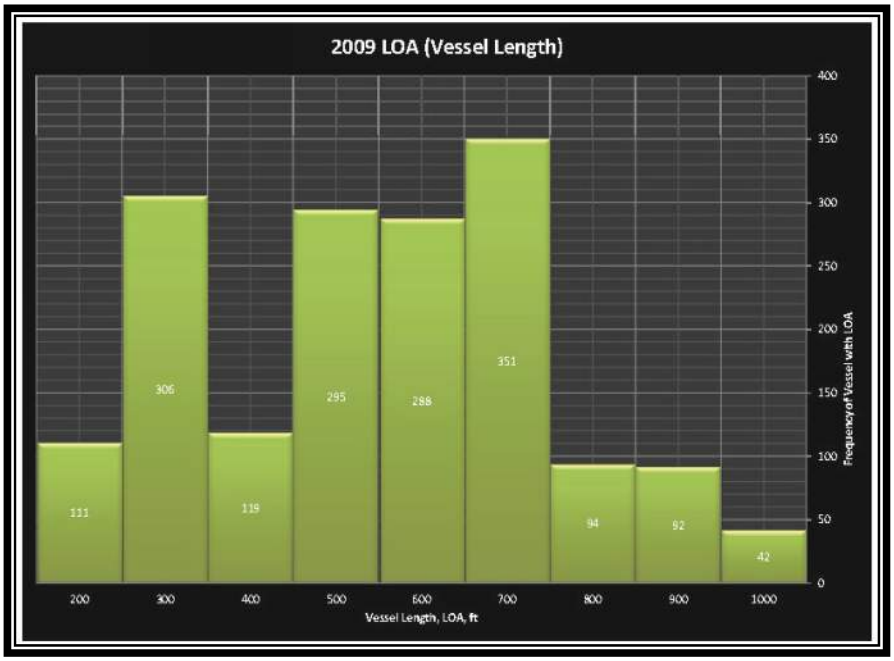
Mobile Bay: High Density, Large Vessels



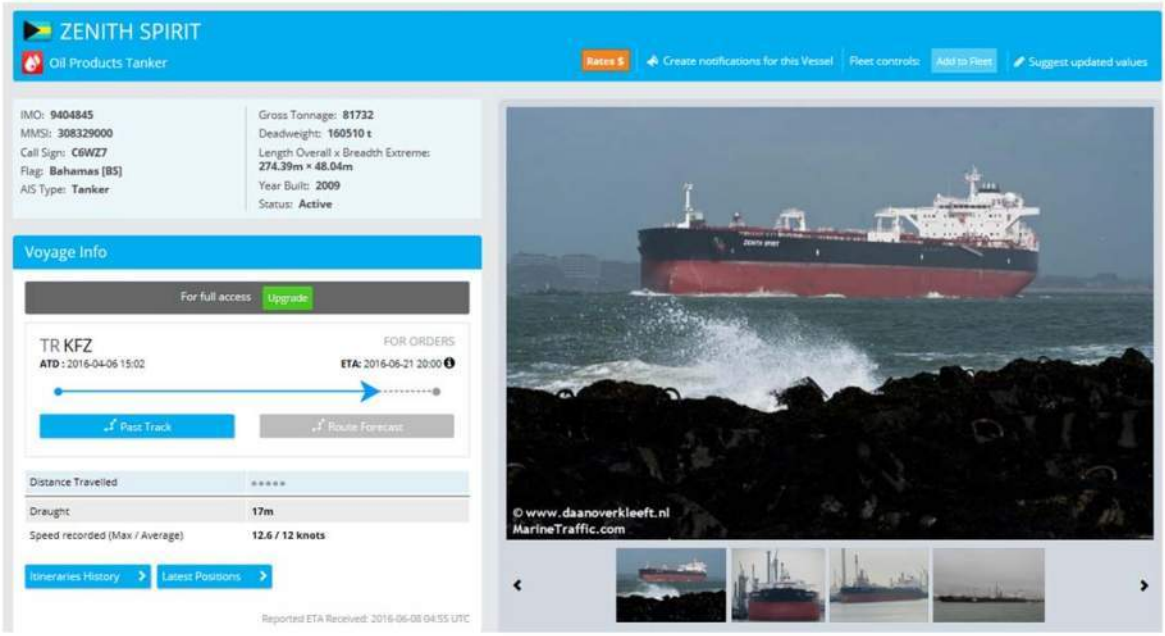
NOAA Reduced Data Source at Bridge Site

Mobile Bay: High Density, Large Vessels



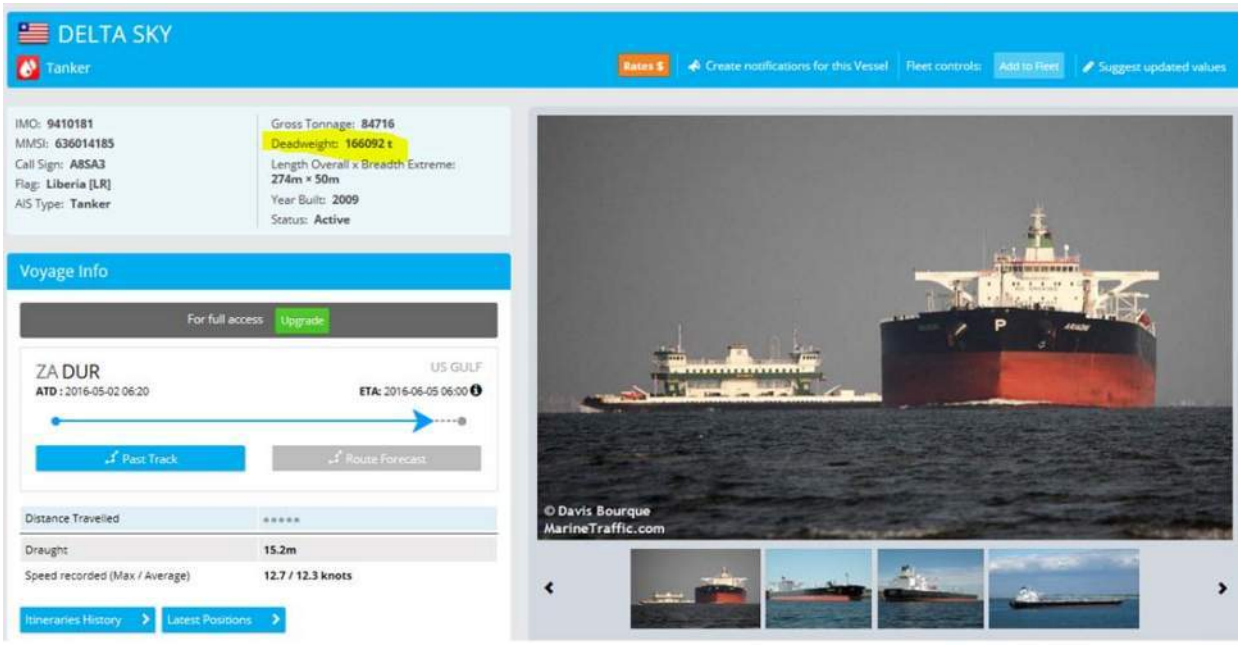
NOAA Reduced Data Source at Bridge Site

Mobile Bay: High Density, Large Vessels



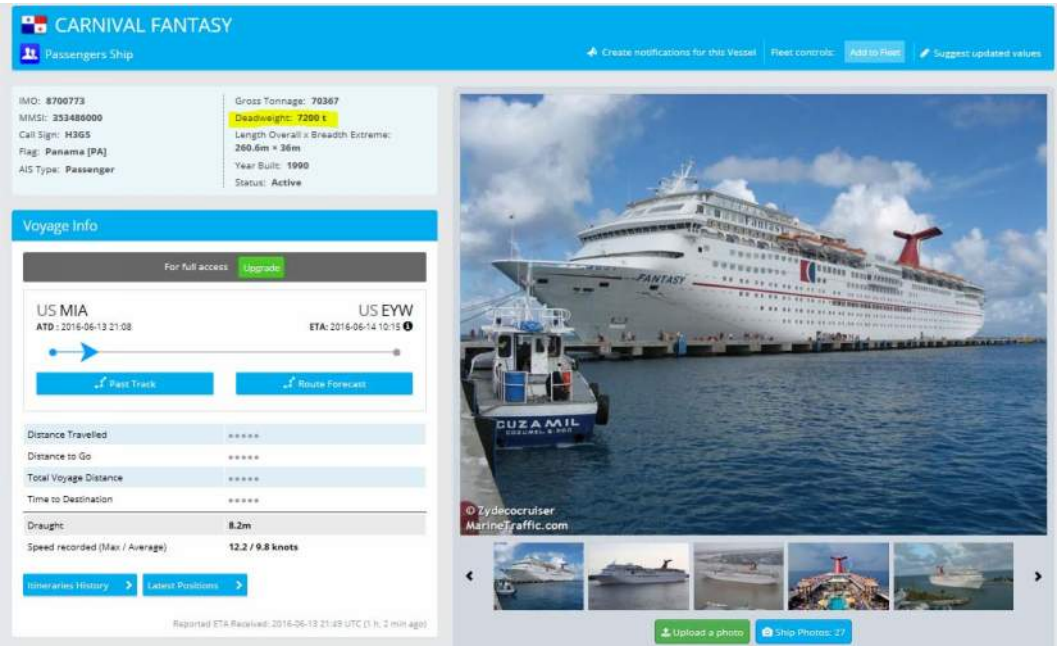
Source: Marinetraffic.com

Mobile Bay: High Density, Large Vessels



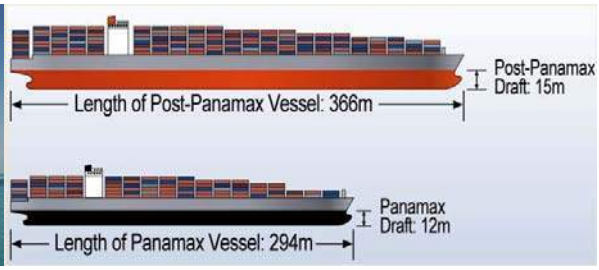
Source: Marinetraffic.com

Mobile Bay: High Density, Large Vessels



Source: Marinetraffic.com

Mobile Bay: High Density, Large Vessels



Panamax:

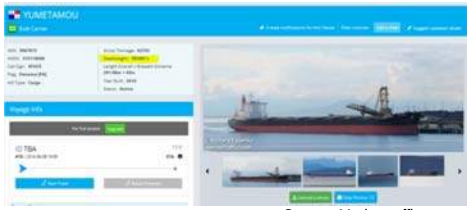
- 52,000 DWT
  - 30,000 kip @10 kt
  - LOA = 980 ft
  - Beam = 106 ft
  - Draft = 40ft

Post Panamax:

- 120,000 DWT
  - 45,000 kip @ 10 kt
  - LOA = 1,400ft***
  - Beam = 180ft
  - Draft = 50 ft

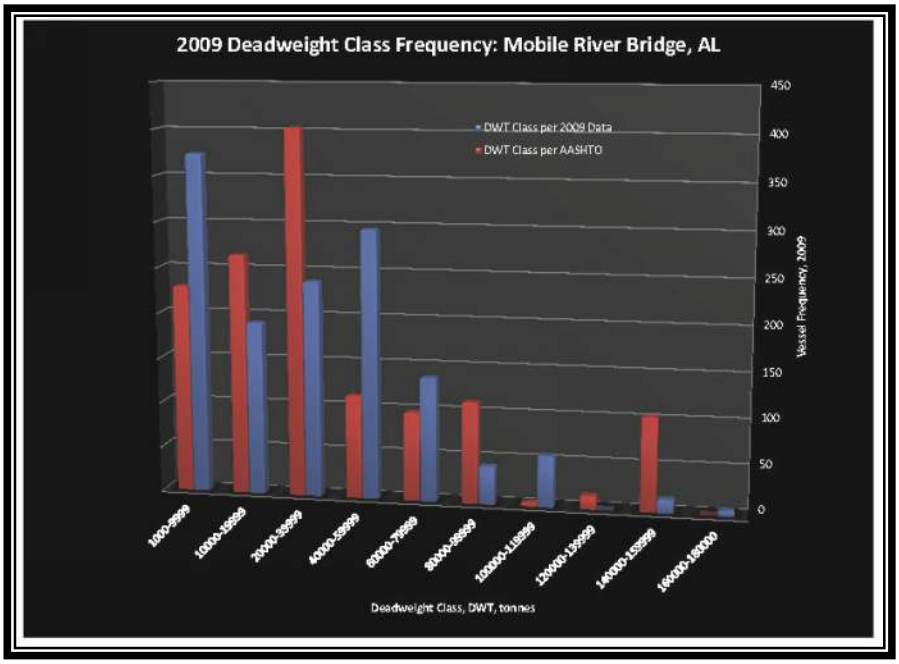
<u>Mobile River:</u>

- Largest Vessels (Oil Tankers)
  - 182,000 DWT (2013) – 55,000 kip @10 kt
  - 167,000 DWT (2009) – 53,000 kip @ 10 kt



Source: Marinetraffic.com

Mobile Bay: High Density, Large Vessels



NOAA Reduced Data Source at Bridge Site

05 Application of Loads to Structure

Applying the Design Load to Structure

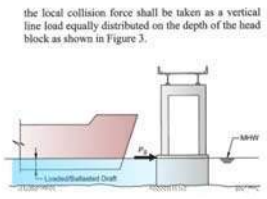


Figure 3.15.1-1—Ship Impact Concentrated Force on Pier (for Foundation Design and Overall Stability)

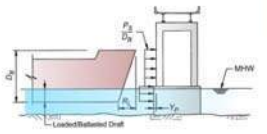


Figure 3.15.1-2—Ship Impact Line Load for Local Collision Force on Pier (for Structure Check and Design)

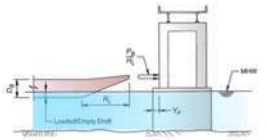


Figure 3.15.1-3—Barge Impact Line Load for Local Collision Force on Pier (for Structure Check and Design)

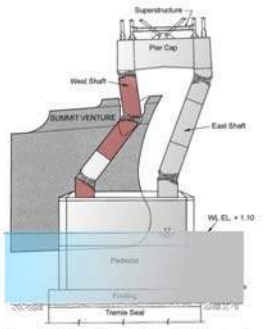


Figure C3.15.1-1—Collapse of Pier 25 of the Sunshine Skyway Bridge Subsequent to Impact by the Bow Overhang of the M/V Summit Venture

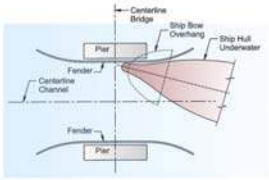


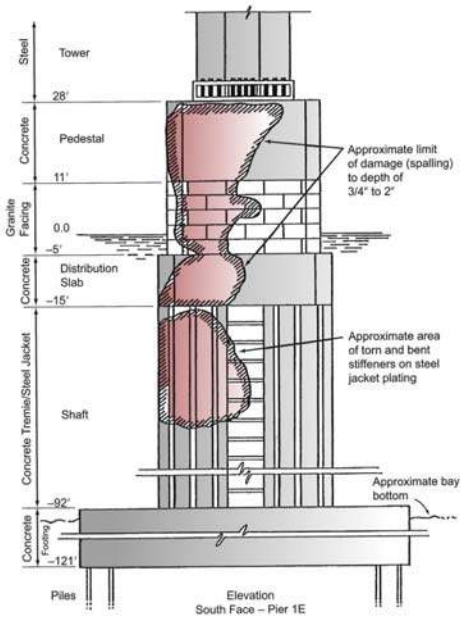
Figure C3.15.1-2—Plan of Ship Bow Overhang Impacting Pier Behind Fender

- Once Design Vessel Is determined at each location (Could differ for each pier, superstructure), how applied:
- Extreme Event II Loading Combination
- For Substructure:
  - Equivalent Static Force applied as:
  - 100% at MHW in Transverse Direction
  - 50% at MHW, Longitudinal Direction
  - Not Concurrent.

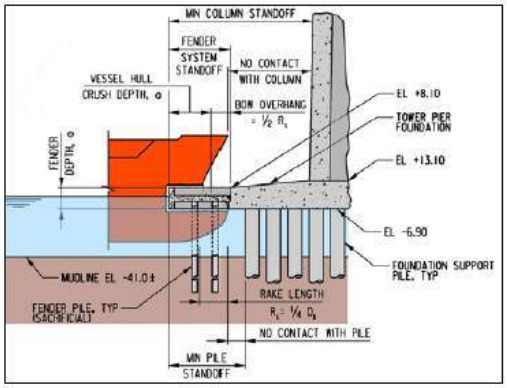
06 Considerations for Loading

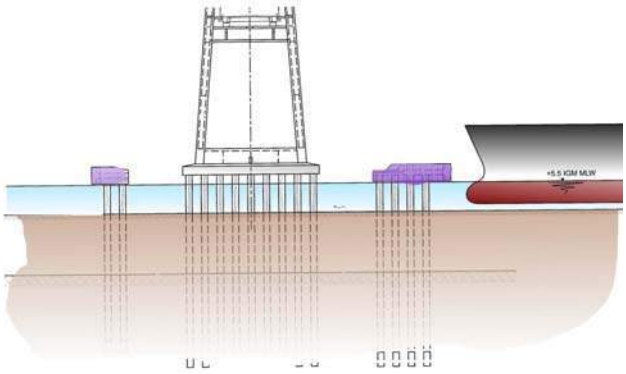
Considerations: Vessel Geometry and Impact Exposure

1. Direct Application to Structure



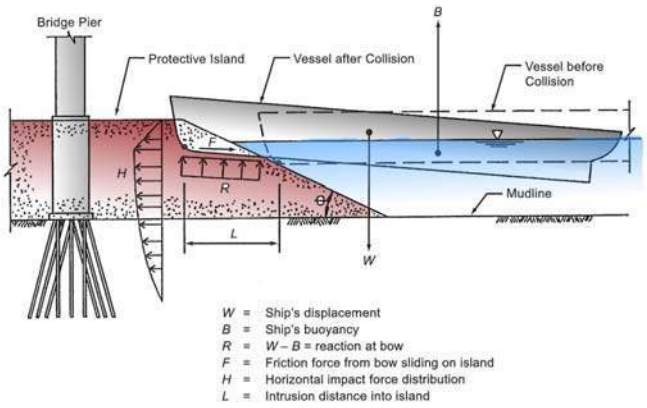
Considerations:





2. Permanent Protection to Structure
   - Artificial Islands
   - Dolphins
   - Other Structure

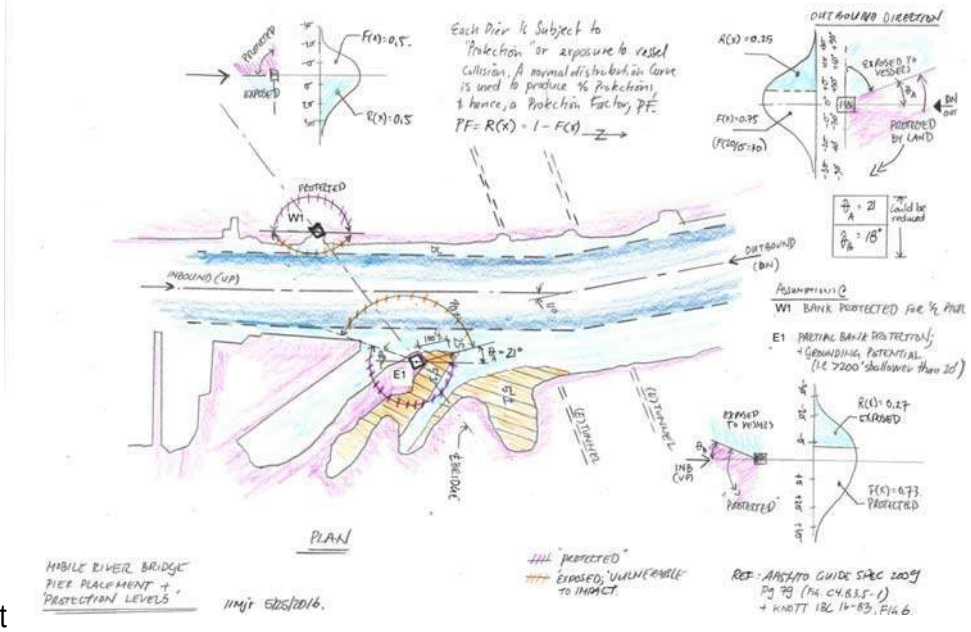
Considerations:



3. Grounding Study As Permanent Protection
   - Artificial Islands
   - Reef
   - Bank Protection

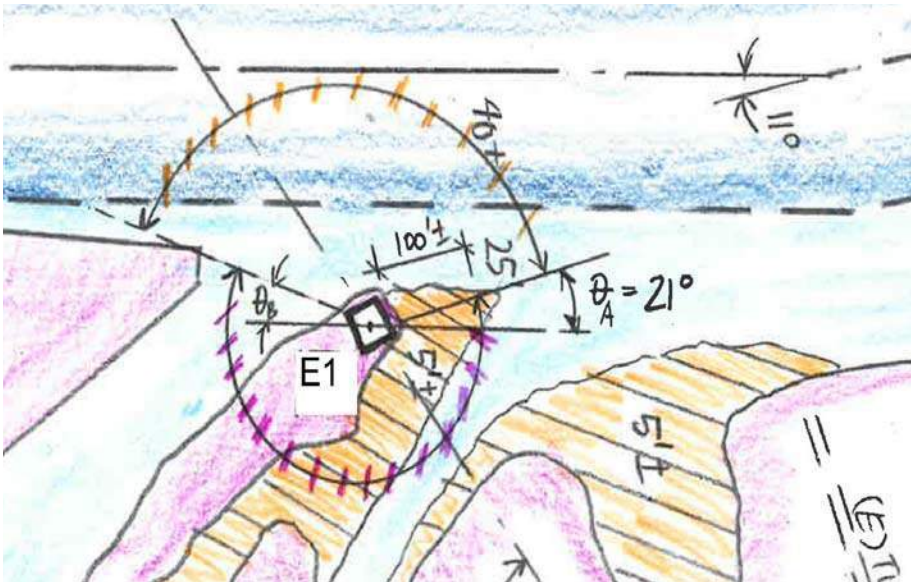
07 Acceptance Frequency (AF) Analysis for Site

AF Analysis for Mobile, AL



Pier Protection Concept

AF Analysis for Mobile, AL



Pier Protection Concept Detail at Eat Pier

AF Analysis (2009 With Bank Protection Shown): Input

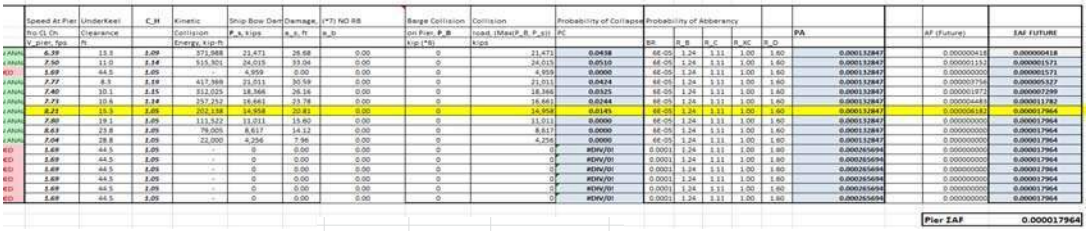
Critical or Typical ?	Critical	AF Limit	0.0001
			Variables
BR	Ship	*0.00006*	*BR_ship*
	Barge	*0.00012*	*BR_barge*
Cross Current, knots	0	*1*	*R_XC*
Current, Parallel, knots	1.12	*1.112*	*R_C*
Channel Tur/Bendn Angle	degrees	11	*THETA*
Region	TURN/BEND	1.24	*R_B*
Channel Density	HIGH	1.60	*R_D*
Protection Factor, %	0%	1.0	*PF*
Design Life	75		
Mean High Water, ft	1.5	MHW=1.5 ft, MHHW=1.6 ft, MLW = 0.1 ft (CHART)	
Growth Factor, %	0%		
Channel Width, ft	**589**		*C*
Mean statistics	**0**		
Minimum Channel Speed, knot	1	1.688	*V_min , fps*
Mud Dpeth Assumption	3	ft	

	Pier Protection			
	Pier Protection Angles			
	Enter θ_1, deg	Enter θ_2, deg		PF
1E-DN	0	90	50.0%	0.5000
1W-DN	90	18	72.6%	0.2743
2E-DN	90	90	99.9%	0.0000
2W-DN	90	90	99.9%	0.0000
1E-UP	0	90	50.0%	0.5000
1W-UP	21	90	75.8%	0.2420
2E-UP	90	90	99.9%	0.0000
2W-UP	90	90	99.9%	0.0000

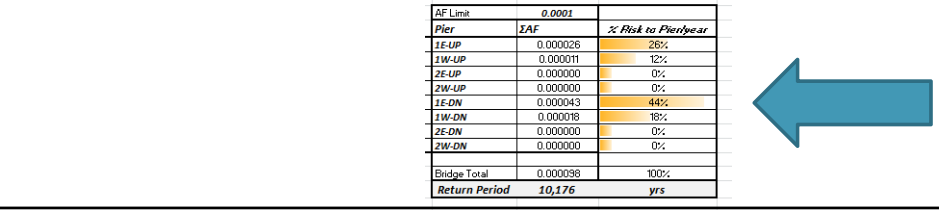
	DWT Class, Tonnes				Avg LOA	Avg BEAM	Avg Draft, Loaded	Avg W	Avg DWT	2009	2009	Total Up and DN	AVG	INBound	Outbound	Average
V	Vessel	V-1	160000-180000	160000	180000	897	160	31.2	244,883	170,000	3	3	6	4	4.1	4.1	4.2
V	Vessel	V-2	140000-159999	140000	159999	858	156	33.5	235,548	154,293	8	7	15	10	4.2	4.8	4.5
V	Vessel	V-3	120000-139999	120000	139999	0	0	0.0	0	130,000	0	0	0	0	0.0	0.0	0.0
V	Vessel	V-4	100000-119999	100000	119999	788	130	36.2	170,030	110,000	28	29	57	38	4.5	5.1	4.8
V	Vessel	V-5	80000-99999	80000	99999	747	122	34.4	144,351	92,754	22	20	42	28	4.4	4.8	4.6
V	Vessel	V-6	60000-79999	60000	79999	795	103	33.9	110,021	70,000	69	68	137	91.33333	4.7	5.0	4.9
V	Vessel	V-7	40000-59999	40000	59999	665	97	29.2	83,199	50,000	147	149	296	197.3333	4.8	5.5	5.1
V	Vessel	V-8	20000-39999	20000	39999	444	68	25.4	50,951	30,000	120	118	238	158.6667	4.9	5.7	5.3
V	Vessel	V-9	10000-19999	10000	19999	497	86	20.7	29,466	15,000	97	94	191	127.3333	5.4	6.1	5.8
V	Vessel	V-10	1000-9999	1000	9999	290	44	15.7	12,331	5,500	188	186	374	249.3333	5.3	6.3	5.9
B	1 barge	B-1L	8000-15000	8000	15000	0	0	0.0	0	11,500			0				
B	1 barge	B-1E	4200-7999	4200	7999	0	0	0.0	0	6,100			0				
B	1 barge	B-2L	4000-4199	4000	4199	0	0	0.0	0	4,100			0				
B	1 barge	B-3L	1000-3999	1000	3999	0	0	0.0	0	2,500			0				
B	1 barge	B-2E	500-999	500	999	0	0	0.0	0	750			0				
B	1 barge	B-3E	0-499	0	499	0	0	0.0	0	250			0				
Tug	1 Tug	T-1	0-5000	0	5000	0	0	0.0	0	2,500			0				
										total	682	674	1356	0	0		
					Averages	*590*	*590*	*598*		*Enter Tab Name*	2009_Data_Summary	2009_Data_Summary					
											2009 - INBOUND	2009 - OUTBOUND					
											UP	DN					

AF Analysis for Mobile, AL

Pier	PG	1W-DN				Vessel Data							COL O	COL V		Geometric Probability				3 x LOA
Pier Data					Protection Factor				*FULL LD*			*Sigma, σ*								
X, ft	*H_p, kips*	*B_p, ft*	*Water Depth, ft*		*PF*				*Draft, ft*	*Beam, ft*	*LOA, ft*	*DWT, tons*	*N, future*	*V_channel, knots*	*V_channel, fps*	*"X1", ft*	*"X2", ft*	*PG*	*PG, % (Not Used)*	
525	13000	105	44.5		0.2	Vessel	V-1	V	31.2	160	897	170,000	3	4.08	6.89	392.6867	657.3133	0.098964395	9.9%	2689.6
525	13000	105	44.5		0.2	Vessel	V-2	V	33.5	156	858	154,293	7	4.83	8.16	394.395	655.605	0.100496456	10.0%	2573.16
525	13000	105	44.5		0.2	Vessel	V-3	V	0.0	0	0	130,000	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	Vessel	V-4	V	36.2	130	788	110,000	29	5.06	8.54	407.4857	642.5143	0.095123418	9.5%	2363.357
525	13000	105	44.5		0.2	Vessel	V-5	V	34.4	122	747	92,754	20	4.84	8.17	411.574	638.426	0.094475532	9.4%	2240.076
525	13000	105	44.5		0.2	Vessel	V-6	V	33.9	103	795	70,000	68	5.02	8.48	420.7568	629.2432	0.084008046	8.4%	2384.029
525	13000	105	44.5		0.2	Vessel	V-7	V	29.2	97	665	50,000	149	5.47	9.23	423.8244	626.1756	0.088737459	8.9%	1996.582
525	13000	105	44.5		0.2	Vessel	V-8	V	25.4	68	444	30,000	118	5.86	9.55	438.4789	611.5231	0.077499987	7.7%	1333.156
525	13000	105	44.5		0.2	Vessel	V-9	V	20.7	86	497	15,000	94	6.10	10.29	429.3191	620.6809	0.087988846	8.8%	1491.825
525	13000	105	44.5		0.2	Vessel	V-10	V	15.7	44	290	5,500	186	6.29	10.62	450.2553	599.7447	0.040943068	4.1%	870.0026
525	13000	105	44.5		0.2	1 barge	B-1L	B	0.0	0	0	11,500	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	1 barge	B-1E	B	0.0	0	0	6,100	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	1 barge	B-2L	B	0.0	0	0	4,100	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	1 barge	B-3L	B	0.0	0	0	2,500	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	1 barge	B-2E	B	0.0	0	0	750	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0
525	13000	105	44.5		0.2	1 barge	B-3E	B	0.0	0	0	250	0	0.00	0.00	472.5	577.5	#NUM!	#NUM!	0

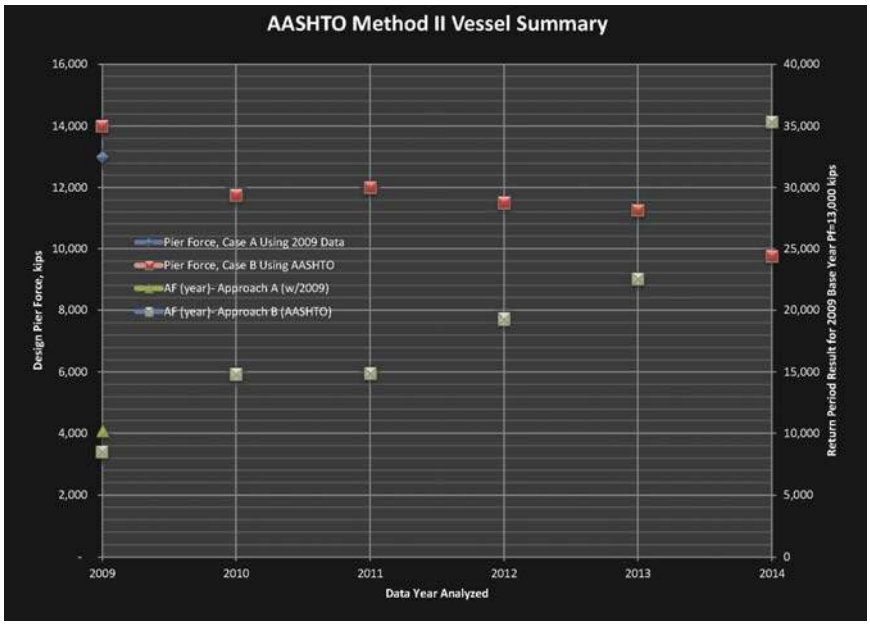


	Speed At Pier	UnderKeel	C_H	Kinetic	Ship Bow Dam	Damage, (*7) NO RB		Barge Collision	Collision	Probability of Collapse	Probability of Abberancy					PA		AF (Future)	ΣAF FUTURE
	Pro-Cl. Ch.	Clearance		Collision	P_a, kips	a_a, ft	a_b	on Pier, P_B	load, (Max(P_B, P_s)))	PC	BR	R_B	R_C	R_XC	R_D				
	V_pier, fps	ft		Energy, kip-ft				kips (*5)	kips										
ANAL	6.39	13.3	1.09	371,988	21,471	28.68	0.00	0	21,471	0.0438	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000000418	0.000000418
ANAL	7.50	11.0	1.14	515,301	24,015	33.04	0.00	0	24,015	0.0510	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000001153	0.000001571
ICED	5.69	44.5	1.05	-	4,959	0.00	0.00	0	4,959	0.0000	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000000000	0.000001571
ANAL	7.27	8.3	1.19	417,369	21,011	30.59	0.00	0	21,011	0.0424	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000003756	0.000005327
ANAL	7.40	10.1	1.15	312,025	18,366	26.16	0.00	0	18,366	0.0325	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000001872	0.000007199
ANAL	7.73	10.6	1.14	257,252	16,661	23.78	0.00	0	16,661	0.0244	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000004481	0.000011782
ANAL	8.21	15.3	1.05	202,138	14,958	20.81	0.00	0	14,958	0.0145	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000006182	0.000017964
ANAL	7.80	19.1	1.09	111,522	11,011	15.60	0.00	0	11,011	0.0000	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000000000	0.000017964
ANAL	8.63	23.8	1.05	79,005	8,617	14.12	0.00	0	8,617	0.0000	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000000000	0.000017964
ANAL	7.04	28.8	1.09	22,000	4,256	7.96	0.00	0	4,256	0.0000	6E-05	1.24	1.11	1.00	1.60	0.000132847		0.000000000	0.000017964
ICED	5.69	44.5	1.05	-	0	0.00	0.00	0	0	#DIV/0!	0.0001	1.24	1.11	1.00	1.60	0.000265694		0.000000000	0.000017964
ICED	5.69	44.5	1.05	-	0	0.00	0.00	0	0	#DIV/0!	0.0001	1.24	1.11	1.00	1.60	0.000265694		0.000000000	0.000017964
ICED	5.69	44.5	1.05	-	0	0.00	0.00	0	0	#DIV/0!	0.0001	1.24	1.11	1.00	1.60	0.000265694		0.000000000	0.000017964
ICED	5.69	44.5	1.05	-	0	0.00	0.00	0	0	#DIV/0!	0.0001	1.24	1.11	1.00	1.60	0.000265694		0.000000000	0.000017964
ICED	5.69	44.5	1.05	-	0	0.00	0.00	0	0	#DIV/0!	0.0001	1.24	1.11	1.00	1.60	0.000265694		0.000000000	0.000017964
																		Pier ΣAF	0.000017964

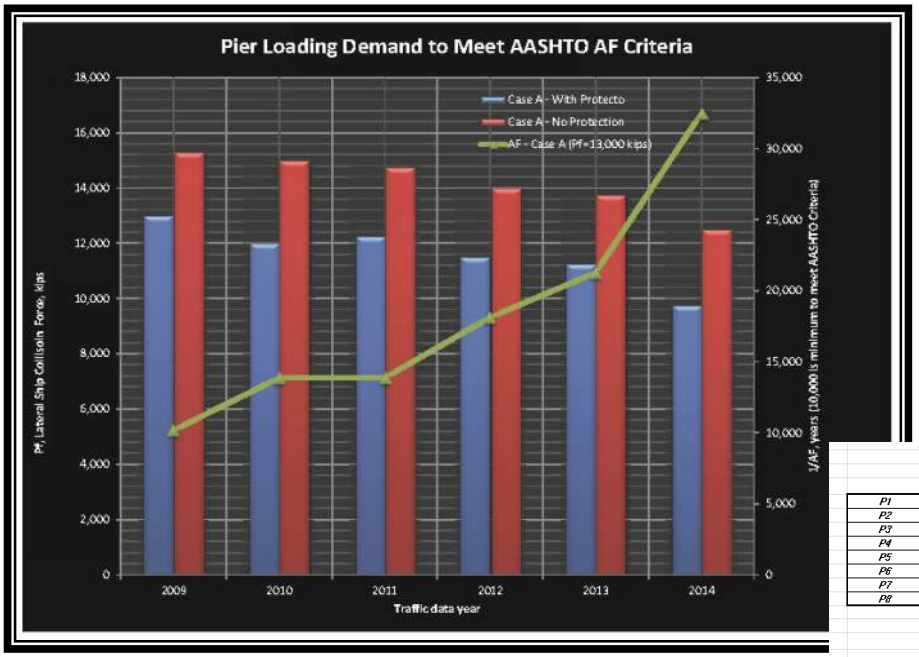


AF Limit	*0.0001*	
Pier	*ΣAF*	*% Risk to Pier/year*
1E-UP	0.000026	26%
1W-UP	0.000011	12%
2E-UP	0.000000	0%
2W-UP	0.000000	0%
1E-DN	0.000043	44%
1W-DN	0.000018	18%
2E-DN	0.000000	0%
2W-DN	0.000000	0%
Bridge Total	0.000098	100%
Return Period	***10,176***	***yrs***

AF Analysis for Mobile, AL



AF Analysis for Mobile, AL

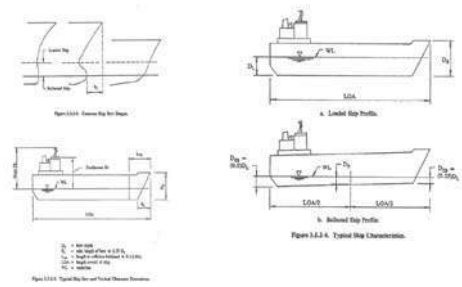


		AF Limit	0.0001	
		Pier	ΣAF	% Risk to Pier/year
P1	1E-UP	0.000026		26%
P2	1W-UP	0.000011		12%
P3	2E-UP	0.000000		0%
P4	2W-UP	0.000000		0%
P5	1E-DN	0.000043		44%
P6	1W-DN	0.000018		18%
P7	2E-DN	0.000000		0%
P8	2W-DN	0.000000		0%
		Bridge Total	0.000098	100%
		Return Period	10,176	yrs

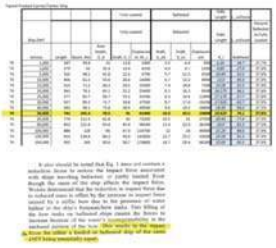
AF Analysis for Mobile, AL

Preliminary Design Summary:

- Largest Vessel
  - 182,000 DWT (2013)
  - 167,000 DWT (2009)
- Design Vessel (No Future Fleet)
  - 50,000 DWT
  - LOA = 665 ft
  - Beam = 97 ft
  - Draft = 30 ft
  - 9.23 fps (5.5 knots)
  - Ps= 15,400 kips

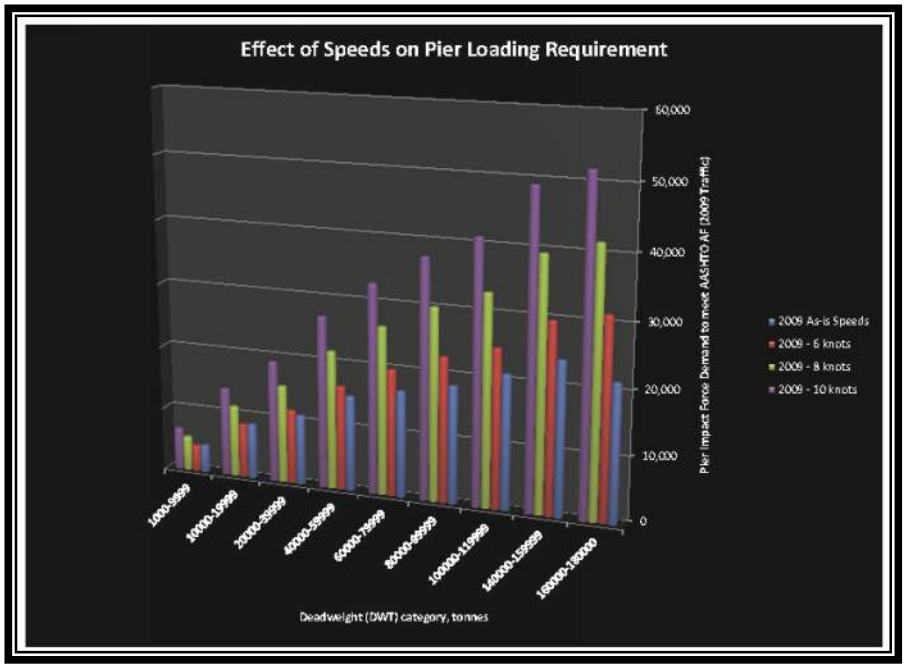


2013 MAX



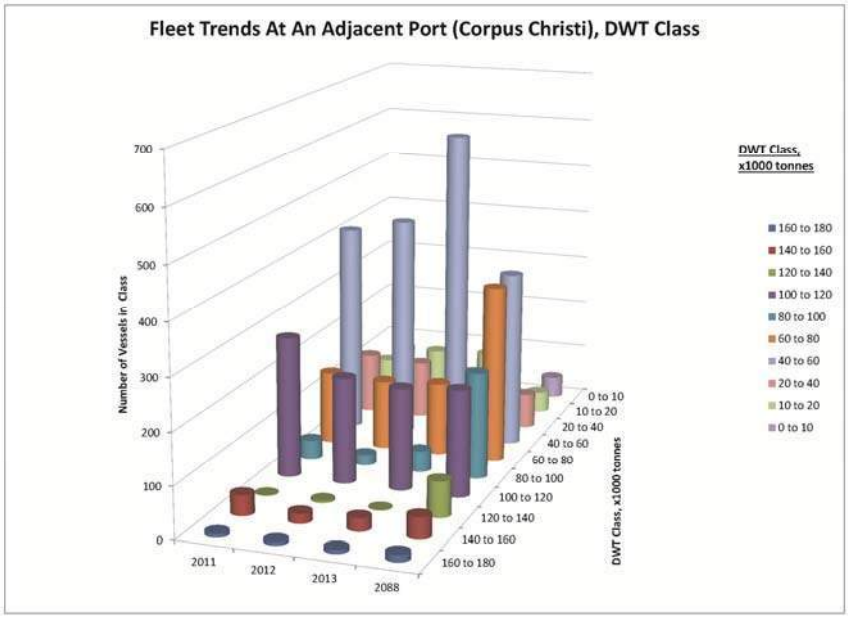
2009 MAX

AF Analysis for Mobile, AL

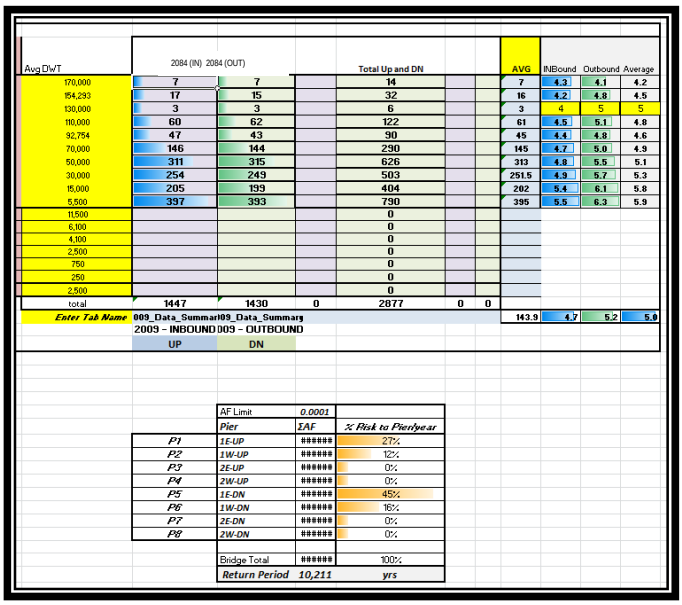


08 Future Fleet Considerations

AF Analysis for Mobile, AL – Future Fleet



AF Analysis for Mobile, AL – Future Fleet



Avg DWT	2084 (IN)	2084 (OUT)		Total Up and DN			AVG	INBound	Outbound	Average
170,000	7	7		14			7	4.3	4.1	4.2
154,293	17	15		32			16	4.2	4.8	4.5
130,000	3	3		6			3	4	5	5
110,000	60	62		122			61	4.5	5.1	4.8
92,754	47	43		90			45	4.4	4.8	4.6
70,000	146	144		290			145	4.7	5.0	4.9
50,000	311	315		626			313	4.8	5.5	5.1
30,000	254	249		503			251.5	4.9	5.7	5.3
15,000	205	199		404			202	5.4	6.1	5.8
5,500	397	393		790			395	5.5	6.3	5.9
11,500				0						
6,100				0						
4,100				0						
2,500				0						
750				0						
250				0						
2,500				0						
total	1447	1430	0	2877	0	0				
Enter Tab Name	009_Data_Summar	009_Data_Summary					143.9	4.7	5.2	5.0
	2009 – INBOUND	009 – OUTBOUND								
	UP	DN								

AF Limit	0.0001	
Pier	IAF	% Risk to Pier/year
P1	1E-UP	######	27%
P2	1W-UP	######	12%
P3	2E-UP	######	0%
P4	2W-UP	######	0%
P5	1E-DN	######	45%
P6	1W-DN	######	16%
P7	2E-DN	######	0%
P8	2W-DN	######	0%
Bridge Total		######	100%
Return Period	10,211	yrs	

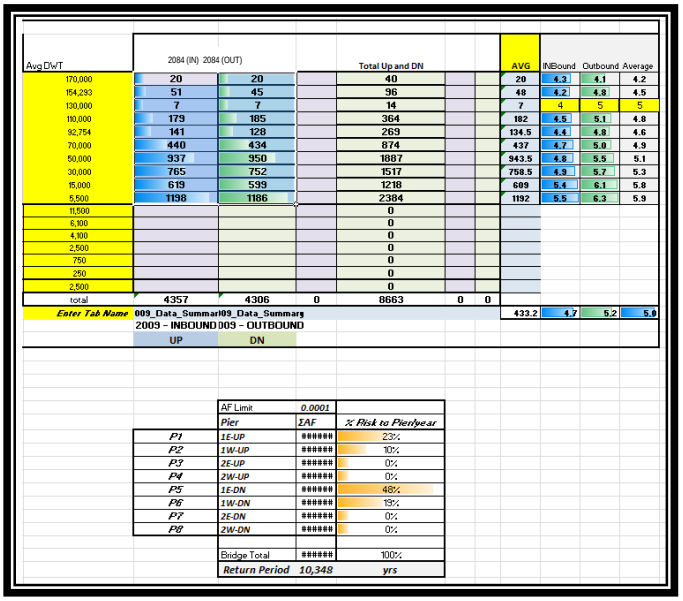
1% Growth Rate On All Fleet Classes

=(N_2009)*(1+1%)^75

<u>Apply a 1% growth rate to Mobile 2009 Fleet</u>

- Ps=15,000 kips
- Design Vessel: 70,000 DWT (16,661 kips)

AF Analysis for Mobile, AL – Future Fleet



Avg DWT	2084 (IN)	2084 (OUT)		Total Up and DN			AVG	INBound	Outbound	Average
170,000	20	20		40			20	4.3	4.1	4.2
154,293	51	45		96			48	4.2	4.8	4.5
130,000	7	7		14			7	4	5	5
110,000	179	185		364			182	4.5	5.1	4.8
92,754	141	128		269			134.5	4.4	4.8	4.6
70,000	440	434		874			437	4.7	5.0	4.9
50,000	937	950		1887			943.5	4.8	5.5	5.1
30,000	765	752		1517			758.5	4.9	5.7	5.3
15,000	619	599		1218			609	5.4	6.1	5.8
5,500	1198	1186		2384			1192	5.5	6.3	5.9
11,500				0						
6,100				0						
4,100				0						
2,500				0						
750				0						
250				0						
2,500				0						
total	4357	4306	0	8663	0	0				
Enter Tab Name	009_Data_Summar	009_Data_Summary					433.2	4.7	5.2	5.0
	2009 – INBOUND	009 – OUTBOUND								
	UP	DN								

AF Limit	0.0001	
Pier	IAF	% Risk to Pier/year
P1	1E-UP	######	23%
P2	1W-UP	######	10%
P3	2E-UP	######	0%
P4	2W-UP	######	0%
P5	1E-DN	######	48%
P6	1W-DN	######	19%
P7	2E-DN	######	0%
P8	2W-DN	######	0%
Bridge Total		######	100%
Return Period	10,348	yrs	

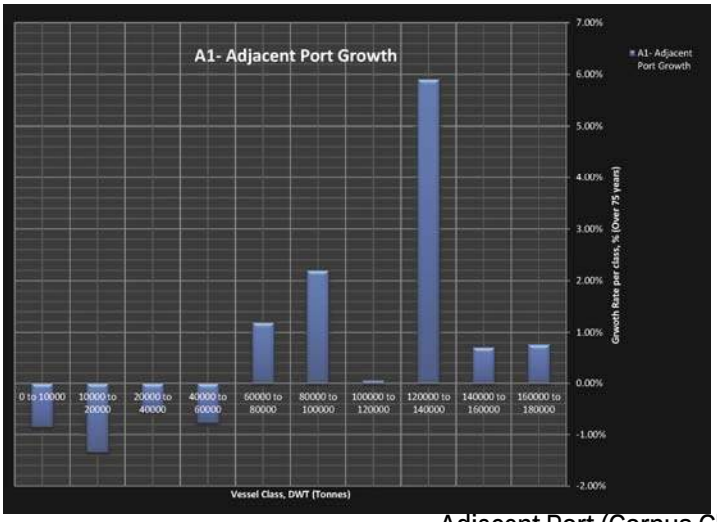
2.5% Growth Rate On All Fleet Classes

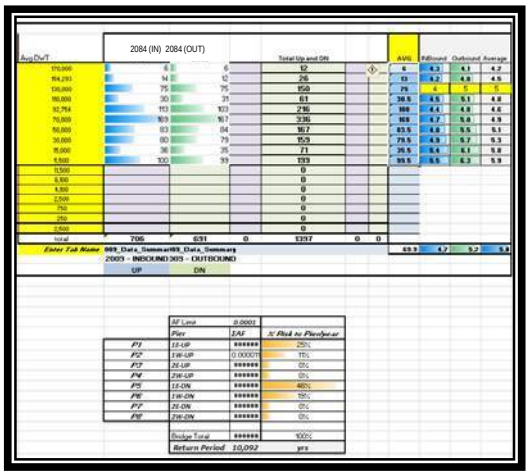
=(N_2009)*(1+1%)^75

<u>Apply a 2.5% growth rate to Mobile 2009 Fleet</u>

- Ps=17,500 kips
- Design Vessel: 90,000 DWT (18,500 kips)

AF Analysis for Mobile, AL – Future Fleet





Variable% Growth Rate On Fleet Classes

Adjacent Port (Corpus Christi) Growth Rates Result Applied to Mobile

- Ps=16,000 kips
- Design Vessel: 70,000 DWT (16,661 kips)

09 Summary and Recommendations

Summary, Recommendations

1. Summary
   - A very busy Port, high density of large vessels
   - Significant amount of site-specific data analyzed to produce a customized solution meeting AASHTO and cost-effectiveness
2. Recommendations
   - Design Vessel for Analyzed Data (50,000 DWT), P_s= 13,000 kips
   - Design Vessel for 1% Growth Data (70,000 DWT), P_s= 15,000 kips
   - Design Vessel for 2.5% Growth Data (90,000 DWT), P_s= 18,500 kips
   - Design Vessel for Global Growth Trend (70,000 DWT), P_s= 16,000 kips

   (Based upon Similar Port Region Growth Rates, Conservative for Pier Protection and Speed Assumptions)

ALDOT Input & Next Steps

1. ALDOT Input

   3.14.2 Owner's Responsibility: To establish/approve

   The operational classification (critical or typical)
   The vessel traffic density
   The design velocity of vessels for the bridge
   The ***degree of damage*** that the bridge components, including protective systems, are allowed to sustain.
   Fleet Growth Level
2. Next Steps
   - Application of design loading into Structural design or super and sub structure approach
   - Bathymetry Verification for Protection and Grounding Levels
   - USACE "passed-the-point" Data to cross check trends over more years
   - Determination of structure type to resist the demand, protection etc.
   - Loading and protection level refinement

10 Comments/Input

Mobile River Bridge: Vessel Collision Analysis

Questions/Input/Comments?

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Appendix L:

East Tower Location

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

0' 50' 100' 200'

MAIN SPAN SUPERSTRUCTURE

BACK SPAN SUPERSTRUCTURE

APPROX. 90'-0"

APPROX. 162'-0"

℄ MOBILE RIVER BRIDGE

LEGEND:

- PROPERTY OWNER: AUSTAL
- PROPERTY OWNER: ORCA INC.
- MAIN SPAN FOOTING

PLAN

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

REVISIONS

MOBILE RIVER BRIDGE
EAST TOWER LOCATION

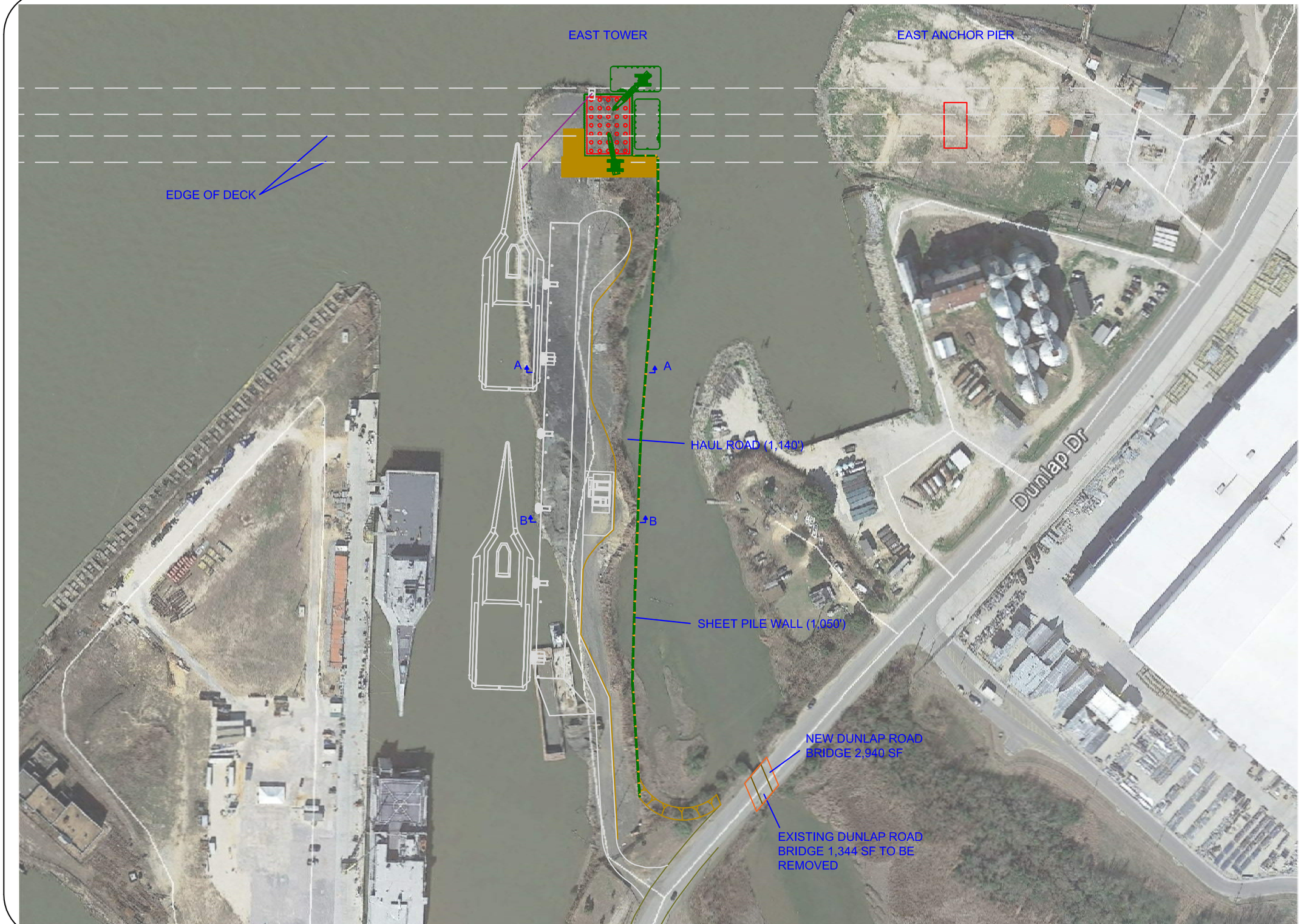
ESTIMATED QUANTITIES	DESIGNED BY:	DRAWN BY:
COMPUTED BY:	CHECKED BY:	DATE DRAWN:
VERIFIED BY:	DATE CHECKED:	SCALE:

BIN:

MRB_HLA_SEQUENCE_PLN.dgn

Appendix M:

East Tower Access Study



General Notes

No.	Revision/Issue	Date

Firm Name and Address

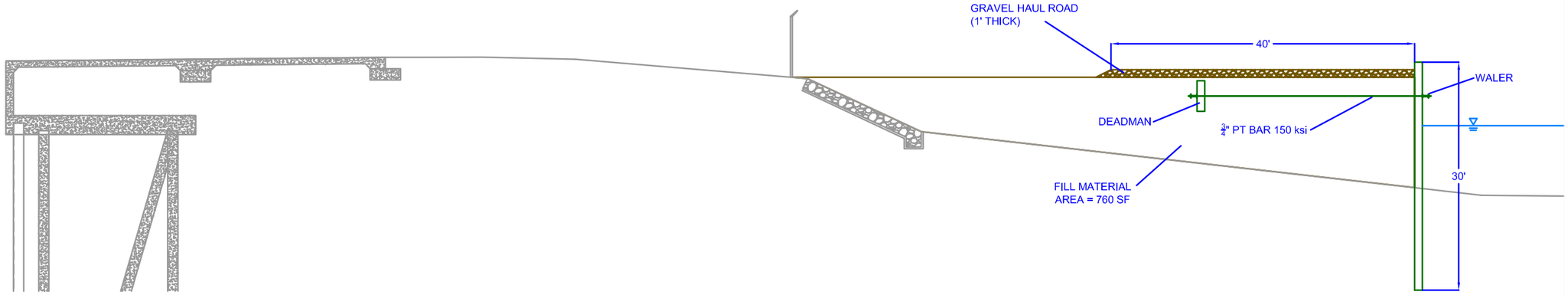
ARMENI CONSULTING SERVICES, LLC

Project Name and Address

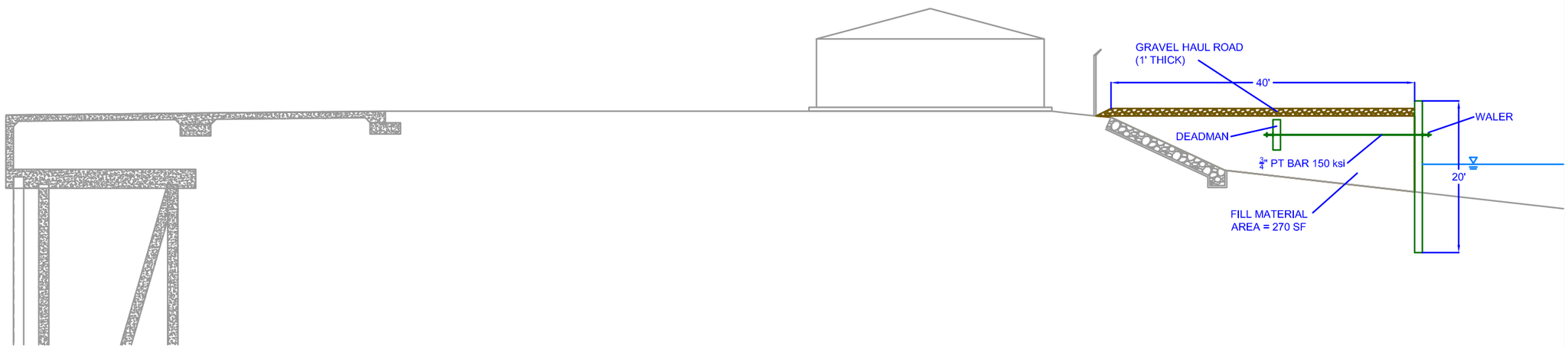
MOBILE RIVER BRIDGE

EAST TOWER ACCESS
OPTION 1

Project	ACS-244
Date	3/02/2016
Scale	As Noted
Sheet	01A



SECTION A-A



SECTION B-B

General Notes

No.	Revision/Issue	Date

Firm Name and Address

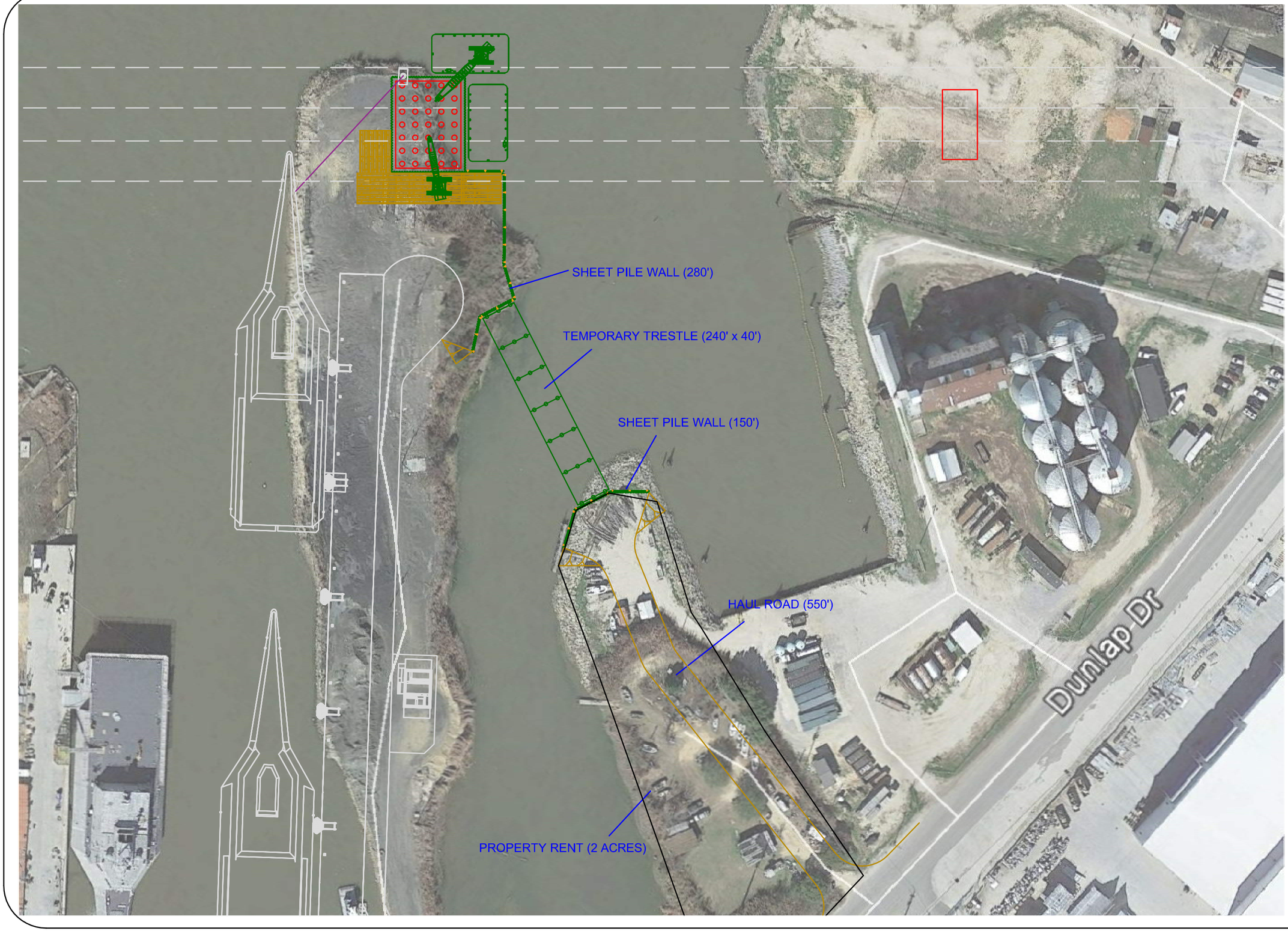
ARMENI CONSULTING SERVICES, LLC

Project Name and Address

MOBILE RIVER BRIDGE

EAST TOWER ACCESS
OPTION 1

Project	Sheet
ACS-244	01B
Date: 3/02/2016	
Scale: As Noted	



General Notes

No.	Revision/Issue	Date

Firm Name and Address

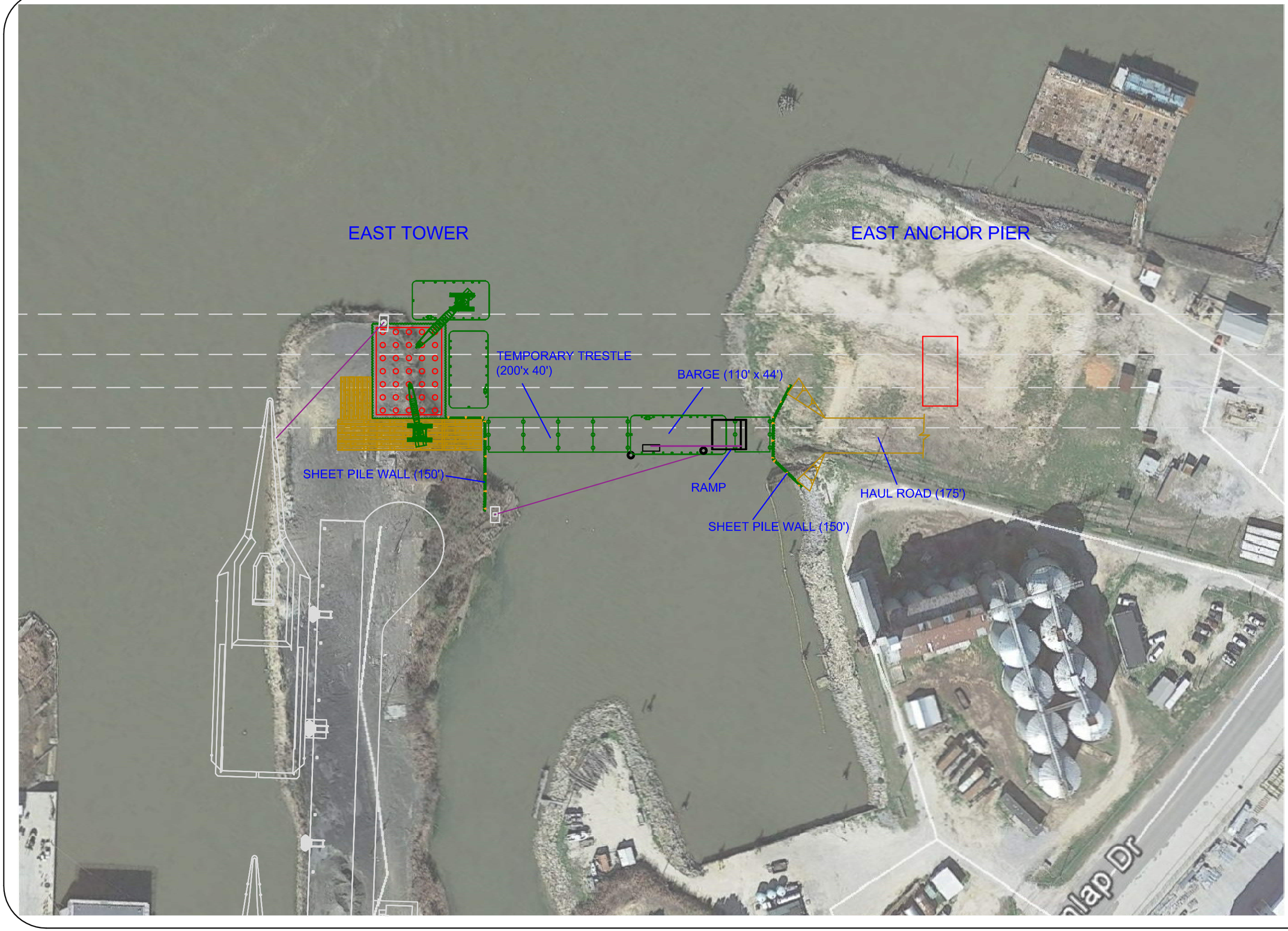
ARMENI CONSULTING SERVICES, LLC

Project Name and Address

MOBILE RIVER BRIDGE

EAST TOWER ACCESS
OPTION 2

Project	Sheet
ACS-244	02
Date: 2/29/2016	
Scale: As Noted	



General Notes

No.	Revision/Issue	Date

Firm Name and Address

ARMENI CONSULTING SERVICES, LLC

Project Name and Address

MOBILE RIVER BRIDGE

EAST TOWER ACCESS
OPTION 3

Project	Sheet
ACS-244	03
Date: 2/29/2016	
Scale: As Noted	

Appendix N:

Back Span Pier Location Study

I-10 Mobile River Bridge and Bayway Widening

Main Span Bridge – Back Span Pier Location Study

Mobile, Alabama

September 18, 2016

1 Introduction

Three different bridge options are under consideration for the main span design of the Mobile River Bridge: a precast concrete box girder bridge, a cast-in-place concrete edge girder bridge, and a cast-in-place box girder bridge. Each of these options has its own unique tower stiffness, deck stiffness, and self-weight loading; but for the purposes of the type selection, the current span layout for the three options is identical (610'-1450'-610').

For a given main span length, the chosen back span arrangement for a cable-stayed bridge influences the structural behavior, including: the magnitude of the uplift force to be resisted at the back span "anchor" pier, the live load stress range of the back span stays, magnitude of the superstructure and tower bending moments, the overall vertical structural stiffness for aerodynamic effects, and the service-level rotations that must be accommodated at the back span expansion joint.

To optimize the back span pier layout for each of the three main span options under consideration, a study has been undertaken to evaluate these effects on each of the three different bridge options, for three different back span configurations.

2 Bridge Types and Layouts Studied

2.1 3 Options

2.1.1 Precast Box Girder Bridge (Alternative 1)

The precast box girder cross-section is shown in *Figure 1*. It has a relatively stiff superstructure, and a single pylon tower. Stays are located in the median between the two decks, connected to delta frames, and are spaced at 24'.

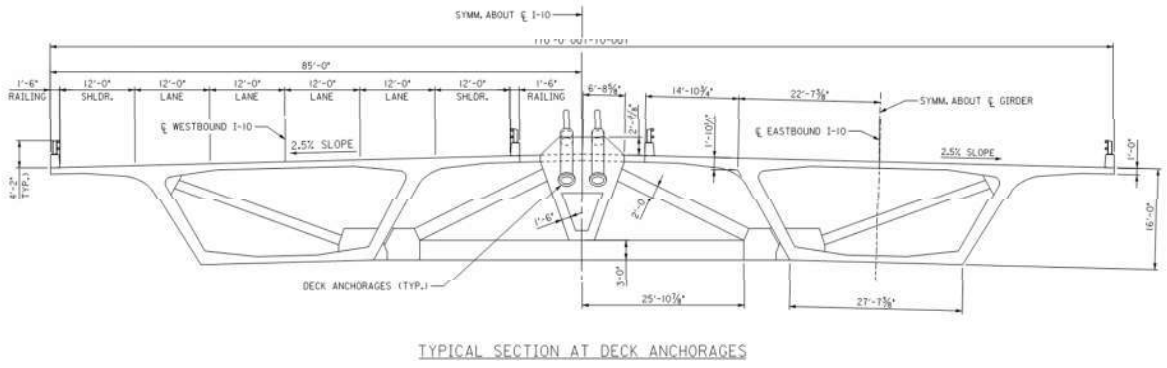


Figure 1 Alternative 1 – Precast Box Girder

2.1.2 Cast-in-place Edge Girder Bridge (Alternative 3)

The cast-in-place edge girder cross-section is shown in *Figure 2*. It has a relatively slender superstructure, and will have either a double-diamond or a triple pylon tower. Stays are located

at the edge of each deck (four planes total), and are spaced at 30’ maximum. Floorbeams are spaced at 15’.

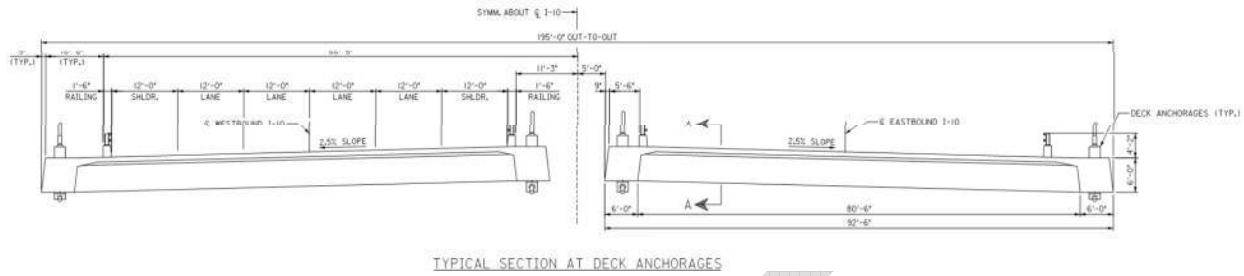


Figure 2 Alternative 3 – Cast-in-place Edge Girder

2.1.3 Cast-in-place Box Girder Bridge (Alternative 7)

The cast-in-place box girder cross-section is shown in *Figure 3*. It has a superstructure stiffness that is between the other two options, and will have either a double-diamond or a triple pylon tower. Stays are located at the edge of each deck (four planes total), and are spaced at 30’ maximum. Floorbeams are spaced at 30’.

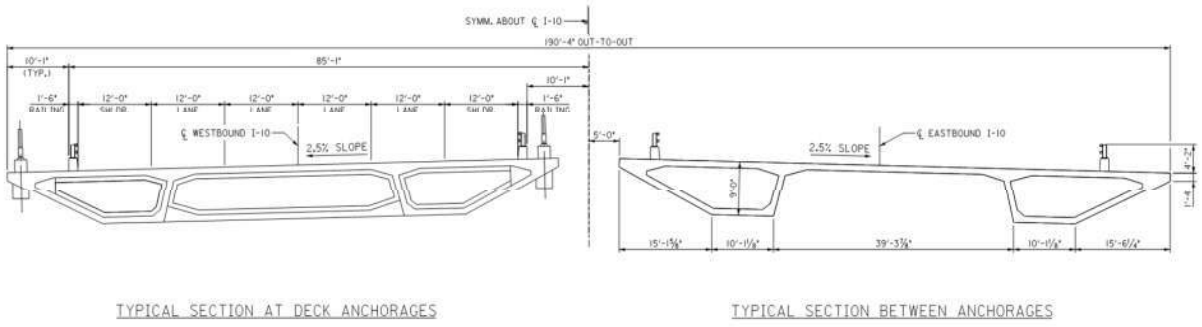


Figure 3 Alternative 7 – Cast-in-Place Box Girder

2.2 Bridge Layouts

2.2.1 Optimum Back Span Ratio for Each Option

At the beginning of the type selection phase, a common back span ratio of 0.42 was selected for the various steel and concrete options under consideration. Now that the options have been narrowed down to 3, we can look at the optimum back span length for each of the three options.

A traditional method of determining the optimum back span length during preliminary design uses the dead-load to live-load ratio for each option, and correlates it to a target live load stress ratio of the back-stay cables. See *Figure 4* (Svensson, 2012).

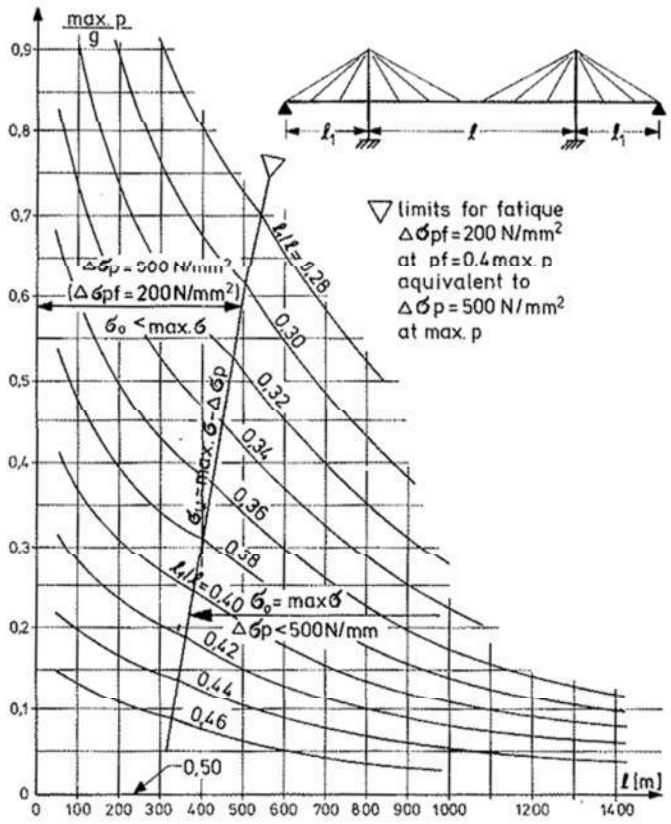


Figure 4 Relation between span ratios and backstay steel stresses **(Svensson, 2012)**

Following this approach, the table below lists the optimum back span length for each of the three options under consideration, assuming a 1450-ft main span and using the graph in Figure 4.

Bridge Option	Uniform DL (kip/ft)	Uniform LL (kip/ft)	Optimum BS Length
Alternative 1	43.5	2.5	680’ (0.47 of MS)
Alternative 3	29.0	2.5	660’ (0.46 of MS)
Alternative 7	28.9	2.5	660’ (0.46 of MS)

Note that this approach only looks at the backstay cable stress, and does not consider effects on other parts of the structure. It also assumes a pure fan-type stay layout, and neglects the stiffness of the deck. Therefore the “optimum” BS length resulting from this method should be taken in this context; and this value should be considered along with other factors when determining the optimum back span configuration. It does suggest that the heavier concrete bridges under consideration may be more suited to a larger BS/MS ratio than the current 0.42 of MS.

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2.2.2 Span Configurations Studied

For the three bridge options under consideration for the Mobile River Bridge, we have looked at 3 span configuration cases:

1) Back span length = 610', main span length = 1450' (BS/MS ratio = 0.42), no intermediate back span piers (Base Case). See *Figure 5*
2) Back span length = 710', main span length = 1450' (BS/MS ratio = 0.49), no intermediate back span piers (Long BS Case). See *Figure 6*
3) Back span length = 710', main span length = 1450', intermediate pier at 500' from tower (BS/MS ratio = 0.49, intermediate pier at 0.34 of main span length (Long BS, Intermediate Pier Case). See *Figure 7*

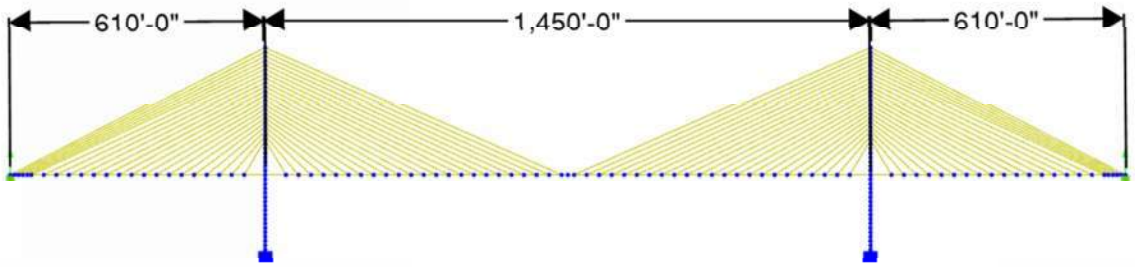

Figure 5 Larsa model - Base Case

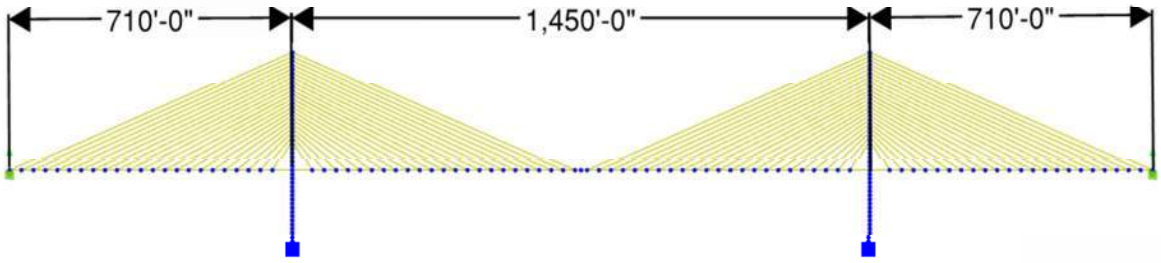

Figure 6 Larsa model - Long BS Case

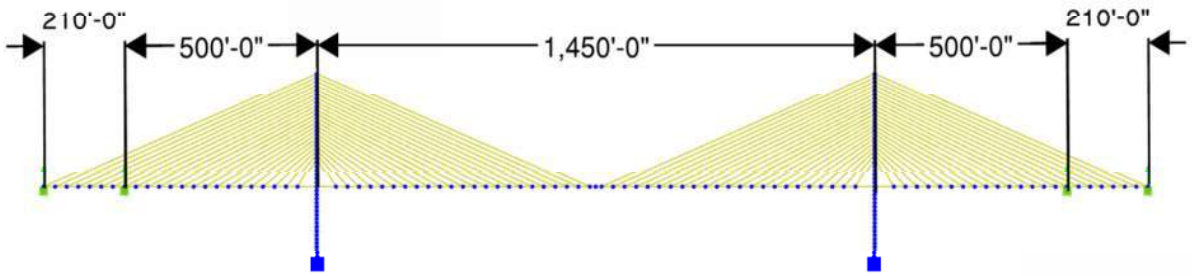

Figure 7 Larsa model - Long BS, Intermediate Pier Case

Each of the models is run with the three options, with the appropriate dead load and member stiffnesses for the deck and the towers for each option. Section properties are as follows:

Section Properties (per deck)	Alternate 1	Alternate 3	Alternate 7
Deck Area	204 ft2	130 ft2	145 ft2
Deck Ixx	7,350 ft4	450 ft4	1,500 ft4
Tower Area	384 ft2	420 ft2	420 ft2
Tower Ixx	74,048 ft4	45,815 ft4	45,815 ft4

Stays are adjusted for each model to minimize dead load deck bending moments prior to applying uniform live load to the back span and the main span separately. For simplicity, each model is fixed at the tower and is supported on rollers at the back span support location; and each model assumes the same stay cable layout across all three bridge options.

3 Results

3.1 Effects Considered

The effect of the back span configuration on the following factors has been evaluated:

- Overall uplift force to be resisted
- Back stay cable force range (fatigue / unloading)
- Superstructure bending moments
- Tower base bending moments
- Frequency value of the first vertical dynamic mode (for aerodynamic effects)
- LL Rotation at expansion joint

3.1.1 Uplift at Anchor Pier

Uplift in the back span of cable-stayed bridges is a function of the ratio of the main span to back span lengths. Uplift results from using an unbalanced span arrangement, which is necessary to

prevent unloading of the back span cables under live loading. Span layout can be chosen to minimize uplift, but it is difficult to efficiently eliminate uplift. Traditionally on cable-stayed bridges, uplift at the back-span piers is resisted using one or a combination of: 1) counterweight, 2) additional back-span length and intermediate back-span piers, or 3) tie-downs.

3.1.2 Back Stay Cable Stress Range

A longer back span can be chose to reduce uplift forces, but a longer back span will cause an increase in the live load stress range in the longest back stays, due to more unloading of these stays when live load is applied in the back span. This can lead to fatigue problems, or potential unloading of the stays.

3.1.3 Superstructure Bending Moments

Longer back span can lead to increased back span (and to a lesser extent, main span) deck bending moments. Intermediate piers can also potentially cause larger negative moments due to the introduction of a hard point in the back span.

3.1.4 Tower Base Bending Moments

Tower base bending moments can increase with a longer back span, because the more flexible span sheds load to the tower when the main span is loaded.

3.1.5 Vertical Frequency

The vertical frequency of the structure is an important characteristic when it comes to aerodynamic effects. In general a stiffer vertical response will lead to better aerodynamic performance, and the chosen back span configuration plays a part in the overall vertical response. A lower vertical frequency can also lead to issues with parametric vibrations of the stays.

3.1.5 Live Load Rotation at Expansion Joint

The length of the back span affects the amount of live load rotation that occurs at the back span expansion joint, if the superstructure is not continuous over the anchor pier. Higher rotations at this joint can lead to detailing issues with the joint, the bearings (may preclude the use of pot bearings), or the tie-downs (if used). In addition, high rotations in this area will lead to higher bending forces in the longer back stay cables.

3.2 Sample Deflected Shapes

See *Figure 8* for deflected shapes for the three configurations with the back span live loading applied; and *Figure 9* for deflected shapes with the main span live loading applied.

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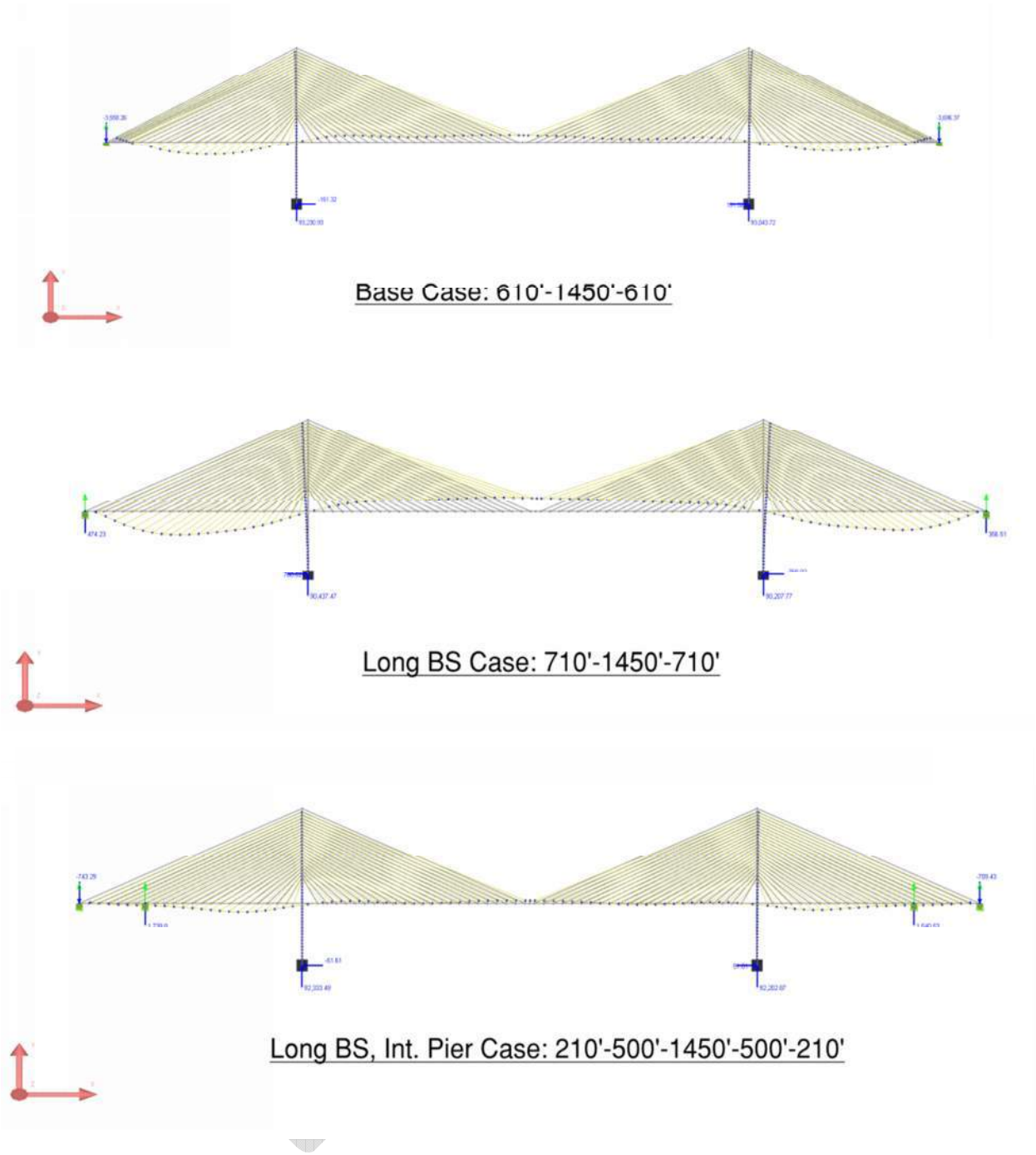

Figure 8 Deflected Shapes – Back Span LL (Alternative 1)

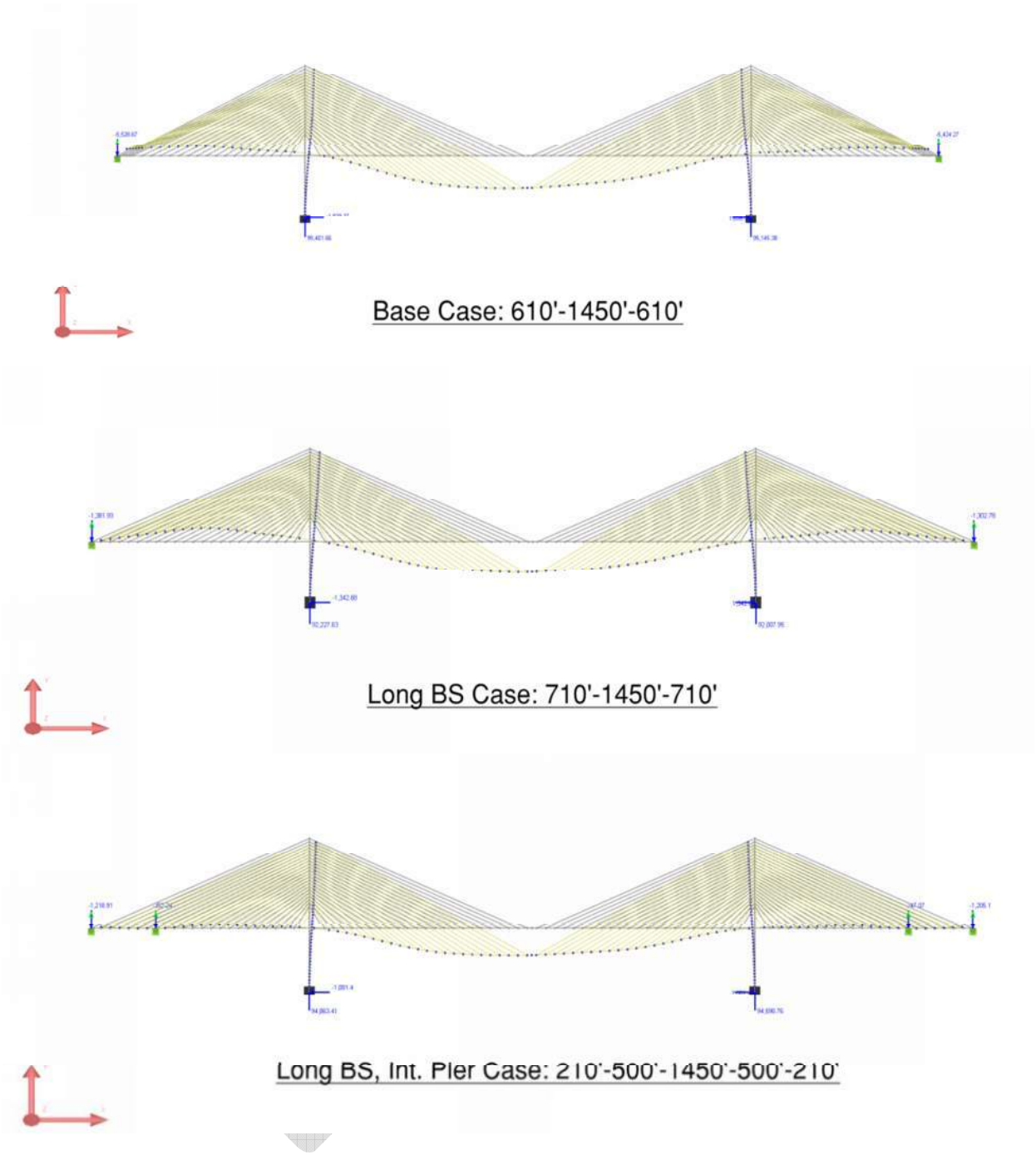

Figure 9 Deflected Shapes – Main Span LL (Alternative 1)

3.3 Tabulated Results

All comparisons are to the base case.

3.3.1 Uplift at Anchor Pier

The results for uplift at the anchor pier are as follows:

Back Span Uplift (DL+LL)	Alternate 1	Alternate 3	Alternate 7
Base Case	5,529 kips	4,192 kips	4,411 kips
Long BS Case	1,382 kips	1,298 kips	1,338 kips
Long BS, Int Pier Case	1,571 kips	1,606 kips	1,614 kips

The Long BS cases, with and without the intermediate pier, result in 60% to 75% reduction in the uplift forces to be resisted at the back span pier(s). Note the tabulated value for uplift for the intermediate pier case includes the uplift at both the anchor pier and the intermediate pier.

3.3.2 Back Stay Cable Force Range

The results for back stay live load force range are as follows:

Back Stay LL Force Range	Alternate 1	Alternate 3	Alternate 7
Base Case	862 kips	1,544 kips	1,259 kips
Long BS Case	1,142 kips	2,256 kips	1,788 kips
Long BS, Int Pier Case	526 kips	797 kips	657 kips

The Long BS case results in an increase in the live load force range of 30% to 50%, and with the intermediate pier, the force range is reduced by 40% to 50%. Thus if a configuration is chose with a longer back span and no intermediate pier, fatigue may govern the back stay size.

3.3.3 Superstructure Bending Moments

The results for deck bending in the back span are as follows:

Back Span Positive Moment (LL)	Alternate 1	Alternate 3	Alternate 7
Base Case	61,615 kip-ft	20,235 kip-ft	30,364 kip-ft
Long BS Case	73,621 kip-ft	25,130 kip-ft	41,119 kip-ft
Long BS, Int Pier Case	39,934 kip-ft	19,703 kip-ft	25,846 kip-ft

Back span positive moments increase by 20% to 30% for the Long BS case, and decrease by up to 35% with the intermediate pier.

Back Span Negative Moment (LL)	Alternate 1	Alternate 3	Alternate 7
Base Case	-43,864 kip-ft	-17,470 kip-ft	-24,358 kip-ft
Long BS Case	-50,358 kip-ft	-21,389 kip-ft	-30,839 kip-ft
Long BS, Int Pier Case	-41,717 kip-ft	-23,454 kip-ft	-31,766 kip-ft

Back span negative moments increase by 15% to 30% for the Long BS case, and increase by up to 35% with the intermediate pier.

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Increased bending moments will lead to more post-tensioning; but it is expected that, in this span range, the increased moments will not require an increase in the deck section.

3.3.4 Tower Base Bending Moments

Results for tower base bending moments are as follows:

Tower Base Moment (LL)	Alternate 1	Alternate 3	Alternate 7
Base Case	266,581 kip-ft	288,326 kip-ft	258,021 kip-ft
Long BS Case	321,414 kip-ft	357,001 kip-ft	317,492 kip-ft
Long BS, Int Pier Case	188,810 kip-ft	193,957 kip-ft	172,627 kip-ft

The Long BS case results in increases in tower base moments of 25% to 30%, while the intermediate pier reduces tower base moments by 30% relative to the base case. A longer back span will result in higher tower demands, both for LL and for wind cases, unless an intermediate pier is incorporated.

3.3.5 Vertical Frequency

The results for vertical frequency of the structure are as follows:

First Vertical Frequency	Alternate 1	Alternate 3	Alternate 7
Base Case	0.233 Hz	0.223 Hz	0.240 Hz
Long BS Case	0.207 Hz	0.186 Hz	0.205 Hz
Long BS, Int Pier Case	0.263 Hz	0.274 Hz	0.290 Hz

The Long BS case results in decreases the vertical frequency of the structure by 10% to 20%, while the intermediate pier increases the vertical frequency by 10 to 25% relative to the base case. The flexibility of Alternative 3 in particular may become an issue if the back span length is increased without adding an intermediate pier.

3.3.5 Live Load Rotation at Expansion Joint

The results for LL rotation at the expansion joint are as follows:

BS Exp Jt Rotation (LL)	Alternate 1	Alternate 3	Alternate 7
Base Case	0.005 rads	0.020 rads	0.011 rads
Long BS Case	0.007 rads	0.027 rads	0.015 rads
Long BS, Int Pier Case	0.000 rads	0.005 rads	0.001 rads

The Long BS case results in increased rotations at the expansion joint, by roughly 40% relative to the base case. Thus a longer back span may require special details for the joints or bearings, and could lead to increased damping requirements for the back stays.

3.4 Cost Estimates

The cost of each back span configuration has been roughly estimated for each bridge option, to give an idea of which span configurations are feasible. For the base case span layout, an estimate is provided for either a tiedown, or counterweight concrete. For the Long BS cases, it is assumed that the cost of resisting the much smaller uplift forces are negligible; the primary cost of these options is the extra bridge length, and the cost of the intermediate pier if provided. The rough estimated costs are tablulated here:

Cost Estimate (Rough Order of Magnitude)	Alternate 1	Alternate 3	Alternate 7
1) Base Option, Tiedown	$ 1,400,000	$ 1,000,000	$ 1,100,000
2) Base Option, Counterweight	$ 1,700,000	$ 1,300,000	$ 1,400,000
3) Long BS Option	$ 17,545,000	$ 17,185,000	$ 17,455,000
4) Long BS, Intermediate Pier	$ 22,600,000	$ 22,600,000	$ 22,600,000

Breakdown of the costs is included in the Appendix.

4 Conclusions / Recommendations

As is clear in the cost estimates, the cost of longer back spans, and thus a longer cable-stayed bridge, is significant. It is recommended that this study be used to tailor the solution for each of the three alternates to best fit the unique characteristics of each option.

In general, the benefit of reduced uplift with a longer back span is mostly out-weighed by the other effects. This doesn't mean that a longer back span is out of the question – increasing the back span ratio from 0.42 to 0.45 or so could take advantage of some of the benefits without necessitating an intermediate pier.

Other considerations (not studied here) are either making the back span integral with the back span pier, or continuous over the back span pier. An integral superstructure at the anchor pier allows the pier cap and pier to serve as the counterweight, without any discrete tie-down elements, and the height of the anchor piers (180'+) makes the pier flexible enough that the moments resulting from an integral pier are manageable. Making the superstructure continuous over the anchor pier eliminates the concerns with excessive joint rotation, and reduces the uplift forces transmitted to the pier; it can result in large bending moments on the approach side, however. This may be mitigated somewhat with either an in-span hinge in the adjacent span, or extending the stay cables partially into the approach span.

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5 Appendix

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Base Case = 0.42 BS/MS					
Long BS Case = 0.49 BS/MS					
Long BS, Int Pier Case = BS Pier @ 0.34, 0.49 of MS					
	43.5	29.0	28.9	k/ft	
	Results (alt 1)	Results (alt 3)	Results (alt 7)		Comments
Tiedown Force (Base Case)	5,529	4,192	4,411	kips	
Tiedown Force (Long BS Case)	1,382	1,298	1,338	kips	100' Longer BS decreases uplift by 70-75%
Tiedown Force (Long BS, Int Pier Case)	1,219	831	866	kips	100' Longer BS w/ int pier decreases uplift by 75-80%
Tower Base LL M (Base Case)	266,581	288,326	258,021	kip-ft	
Tower Base LL M (Long BS Case)	321,414	357,001	317,492	kip-ft	100' Longer BS increases tower LL moments by 20-25%
Tower Base LL M (Long BS, Int Pier Case)	188,810	193,957	172,627	kip-ft	100' Longer BS w/ int pier decreases tower LL moments by 30%
Backspan Neg. LL M (Base Case)	-43,864	-17,470	-24,358	kip-ft	
Backspan Neg. LL M (Long BS Case)	-50,358	-21,389	-30,839	kip-ft	100' Longer BS increases bs negative LL moments by 15-30%
Backspan Neg. LL M (Long BS, Int Pier Case)	-41,717	-23,454	-31,766	kip-ft	100' Longer BS w/ int pier increases bs negative LL moments by 0-35%
Backspan Pos. LL M (Base Case)	61,615	20,235	30,364	kip-ft	
Backspan Pos. LL M (Long BS Case)	73,621	25,130	41,119	kip-ft	100' Longer BS increases bs positive LL moments by 20-35%
Backspan Pos. LL M (Long BS, Int Pier Case)	39,934	19,703	25,846	kip-ft	100' Longer BS w/ int pier decreases bs positive moments by 0-35%
Vertical Frequency (Base Case)	0.233	0.223	0.240	Hz	
Vertical Frequency (Long BS Case)	0.207	0.186	0.205	Hz	100' Longer BS decreases frequency by 10-20%
Vertical Frequency (Long BS, Int Pier Case)	0.263	0.274	0.290	Hz	100' Longer BS w/ int pier increases frequency by 10-25%
Exp Jt Rotation (Base Case)	0.005	0.020	0.011	rads	
Exp Jt Rotation (Long BS Case)	0.007	0.027	0.015	rads	100' Longer BS increases exp jt rotations by 40%
Exp Jt Rotation (Long BS, Int Pier Case)	0.000	0.005	0.001	rads	100' Longer BS w/ int pier decreases exp jt rotations by 80-100%
LL Stay Force Range (Base Case)	862	1,544	1,259	kips	
LL Stay Force Range (Long BS Case)	1,142	2,256	1,788	kips	100' Longer BS increases LL stay force range by 30-50%
LL Stay Force Range (Long BS, Int Pier Case)	526	797	657	kips	100' Longer BS w/ int pier decreases LL stay force range by 40-50%
	Cost Estimate (alt 1)	Cost Estimate (alt 3)	Cost Estimate (alt 7)		
1) Base Option, Tiedown	$ 1,400,000	$ 1,000,000	$ 1,100,000		
2) Base Option, Counterweight	$ 1,700,000	$ 1,300,000	$ 1,400,000		
3) Long BS Option	$ 17,545,000	$ 17,185,000	$ 17,455,000		
4) Long BS, Intermediate Pier	$ 22,600,000	$ 22,600,000	$ 22,600,000		

1) Base Option, Tiedown										
Tiedown Strands	33,494	lb	$ 15	$ 502,409	assume 30' long, 2 piers, 2 decks, double strands for redundancy					
Pier PT	100,482	lb	$ 8	$ 803,855	2 piers, 180' long tendons, 2 decks					
				Total Cost of Tiedown	$ 1,400,000					
2) Base Option, Counterweight										
Counterweight Concrete	5480	CY	$ 300	$ 1,644,000	2 piers, 2 decks					
				Total Cost of Counterweight	$ 1,700,000					
3) Long BS Option										
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Add'l PT in BS	157,500	lb	$ 6	$ 945,000						
			Total Cost (200' of MS Bridge + add'l PT)		$ 17,545,000					
4) Long BS, Intermediate Pier										
	Quantity		Unit Cost	Total						
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Intermediate Piers										
Footing	2	ea	$ 1,000,000	$ 2,000,000						
Piers	2	ea	$ 2,000,000	$ 4,000,000						
				Total Cost (200' of MS Bridge + 2 int piers)	$ 22,600,000					

1) Base Option, Tiedown										
Tiedown Strands	25,519	lb	$ 15	$ 382,788	assume 30' long, 2 piers, 2 decks, double strands for redundancy					
Pier PT	76,558	lb	$ 8	$ 612,461	2 piers, 180' long tendons, 2 decks					
				Total Cost of Tiedown	$ 1,000,000					
2) Base Option, Counterweight										
Counterweight Concrete	4160	CY	$ 300	$ 1,248,000	2 piers, 2 decks					
				Total Cost of Counterweight	$ 1,300,000					
3) Long BS Option										
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Add'l PT in BS	97,500	lb	$ 6	$ 585,000						
			Total Cost (200' of MS Bridge + add'l PT)		$ 17,185,000					
4) Long BS, Intermediate Pier										
	Quantity		Unit Cost	Total						
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Intermediate Piers										
Footing	2	ea	$ 1,000,000	$ 2,000,000						
Piers	2	ea	$ 2,000,000	$ 4,000,000						
				Total Cost (200' of MS Bridge + 2 int piers)	$ 22,600,000					

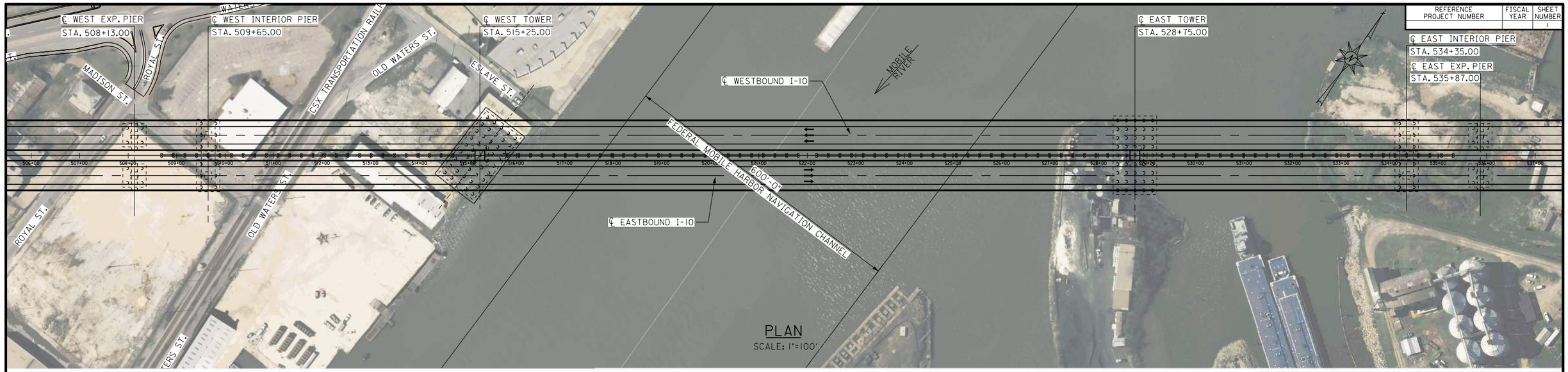
1) Base Option, Tiedown										
Tiedown Strands	26,760	lb	$ 15	$ 401,396	assume 30' long, 2 piers, 2 decks, double strands for redundancy					
Pier PT	80,279	lb	$ 8	$ 642,233	2 piers, 180' long tendons, 2 decks					
			Total Cost of Tiedown		$ 1,100,000					
2) Base Option, Counterweight										
Counterweight Concrete	4360	CY	$ 300	$ 1,308,000	2 piers, 2 decks					
			Total Cost of Counterweight		$ 1,400,000					
3) Long BS Option										
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Add'l PT in BS	142,500	lb	$ 6	$ 855,000						
		Total Cost (200' of MS Bridge + add'l PT)			$ 17,455,000					
4) Long BS, Intermediate Pier										
	Quantity		Unit Cost	Total						
Add'l MS										
MS Bridge (@$700/SF)	200	ft	$ 119,000	$ 23,800,000						
Approach Bridge (@$240/SF)	-200	ft	$ 36,000	$ (7,200,000)						
Intermediate Piers										
Footing	2	ea	$ 1,000,000	$ 2,000,000						
Piers	2	ea	$ 2,000,000	$ 4,000,000						
			Total Cost (200' of MS Bridge + 2 int piers)		$ 22,600,000					

Appendix O:

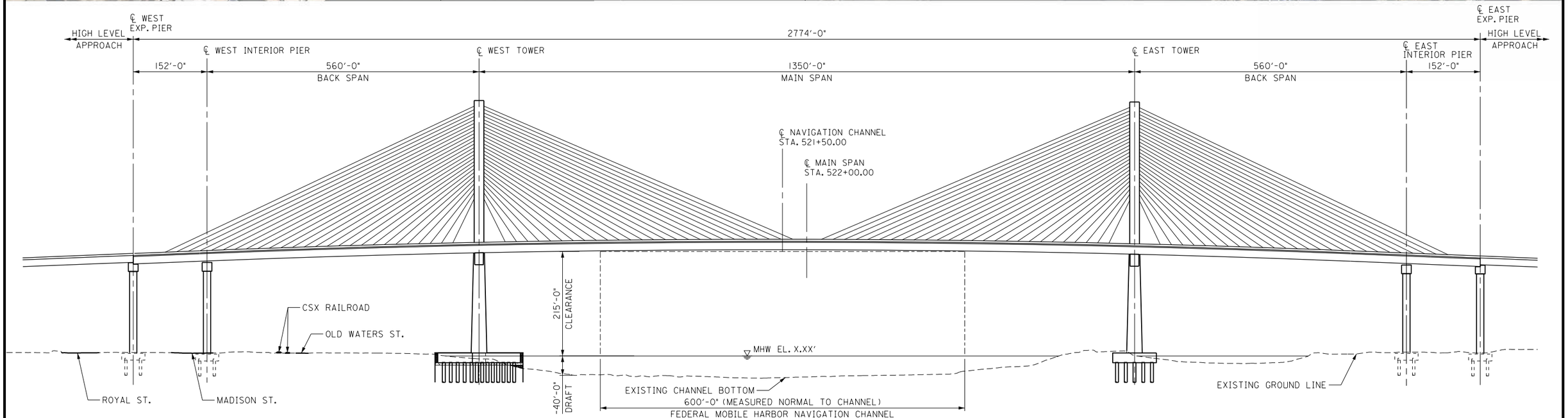
Step 2:

Bridge Layouts

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		1



PLAN
SCALE: 1"=100'



ELEVATION
SCALE: 1"=100'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

PLAN & ELEVATION
ALTERNATIVE 1

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN:8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:03:42 PM
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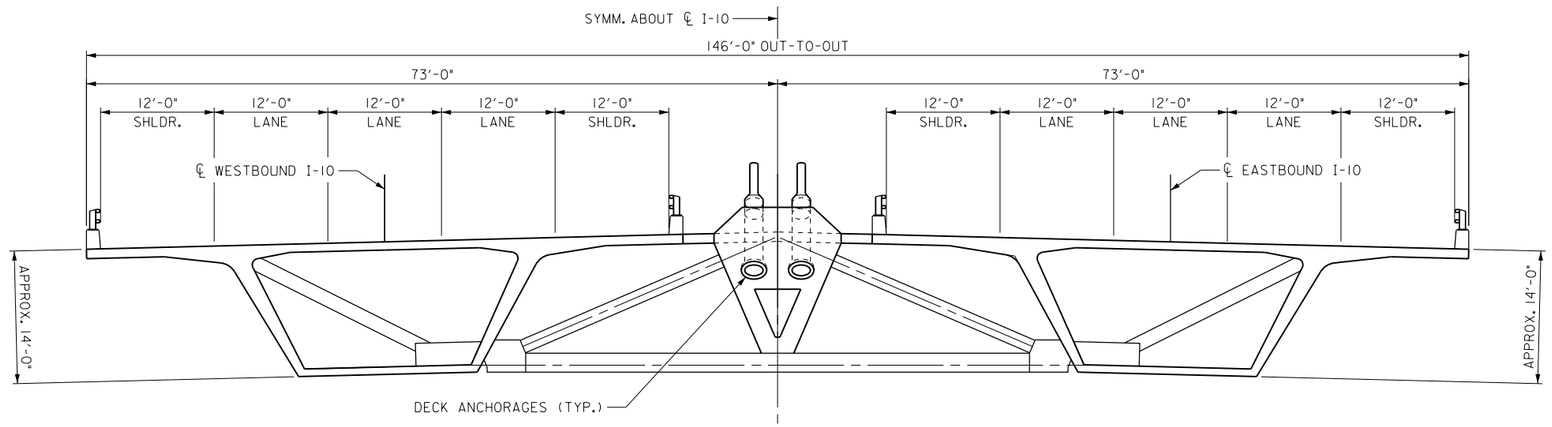
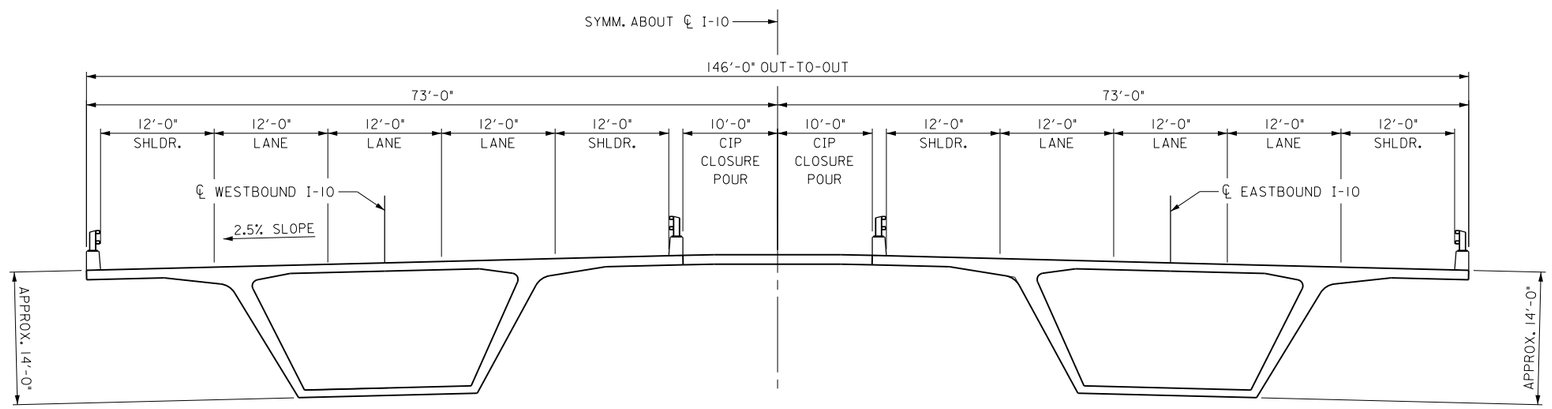
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		2



TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BETWEEN DECK ANCHORAGES

SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

DECK TYPICAL SECTIONS
ALTERNATIVE I

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.plt
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:03:44 PM
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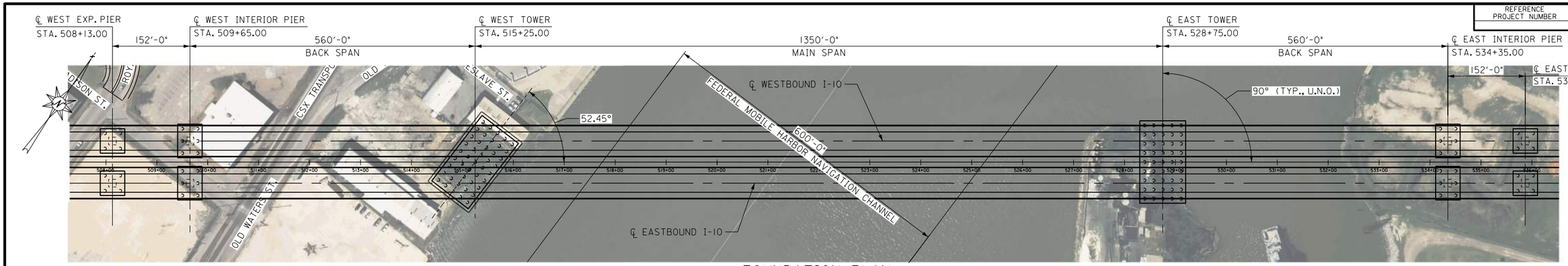
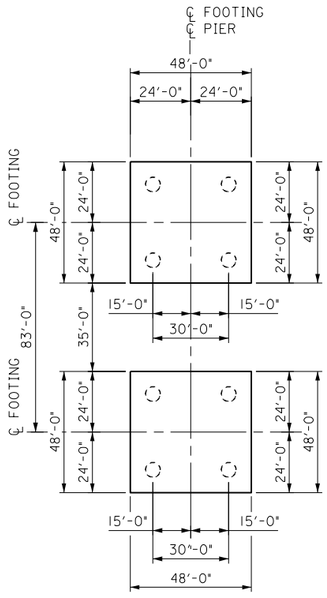
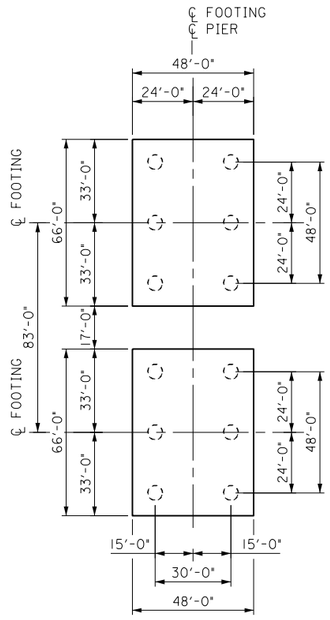
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		3



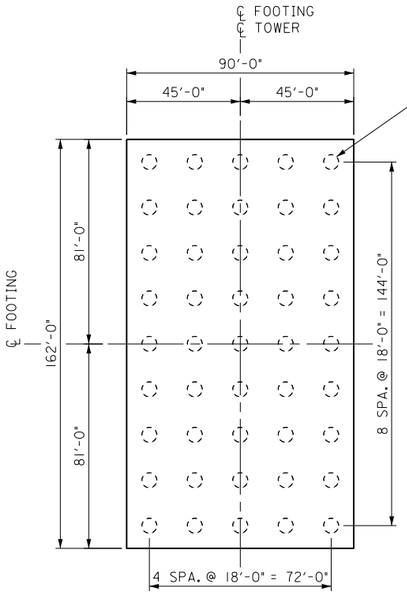
FOUNDATION PLAN
SCALE: 1"=100'



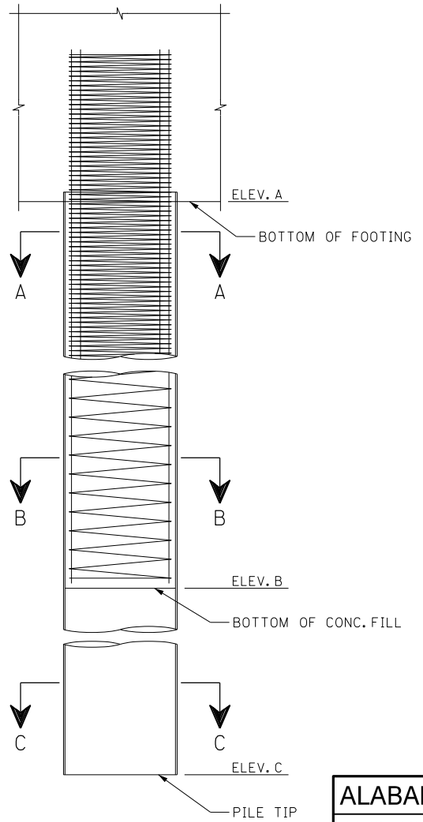
EXPANSION PIER FOOTING PLAN
SCALE: 1"=30'



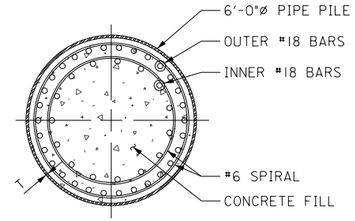
INTERIOR PIER FOOTING PLAN
SCALE: 1"=30'



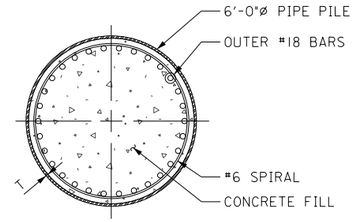
TOWER FOOTING PLAN
SCALE: 1"=30'



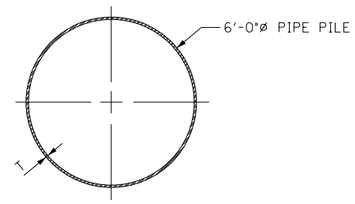
ELEVATION
SCALE: 1/4" = 1'-0"



SECTION A-A
SCALE: 3/8" = 1'-0"



SECTION B-B
SCALE: 3/8" = 1'-0"



SECTION C-C
SCALE: 3/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

FOUNDATION PLAN & PILE DETAILS
ALTERNATIVE I

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:03:46 PM
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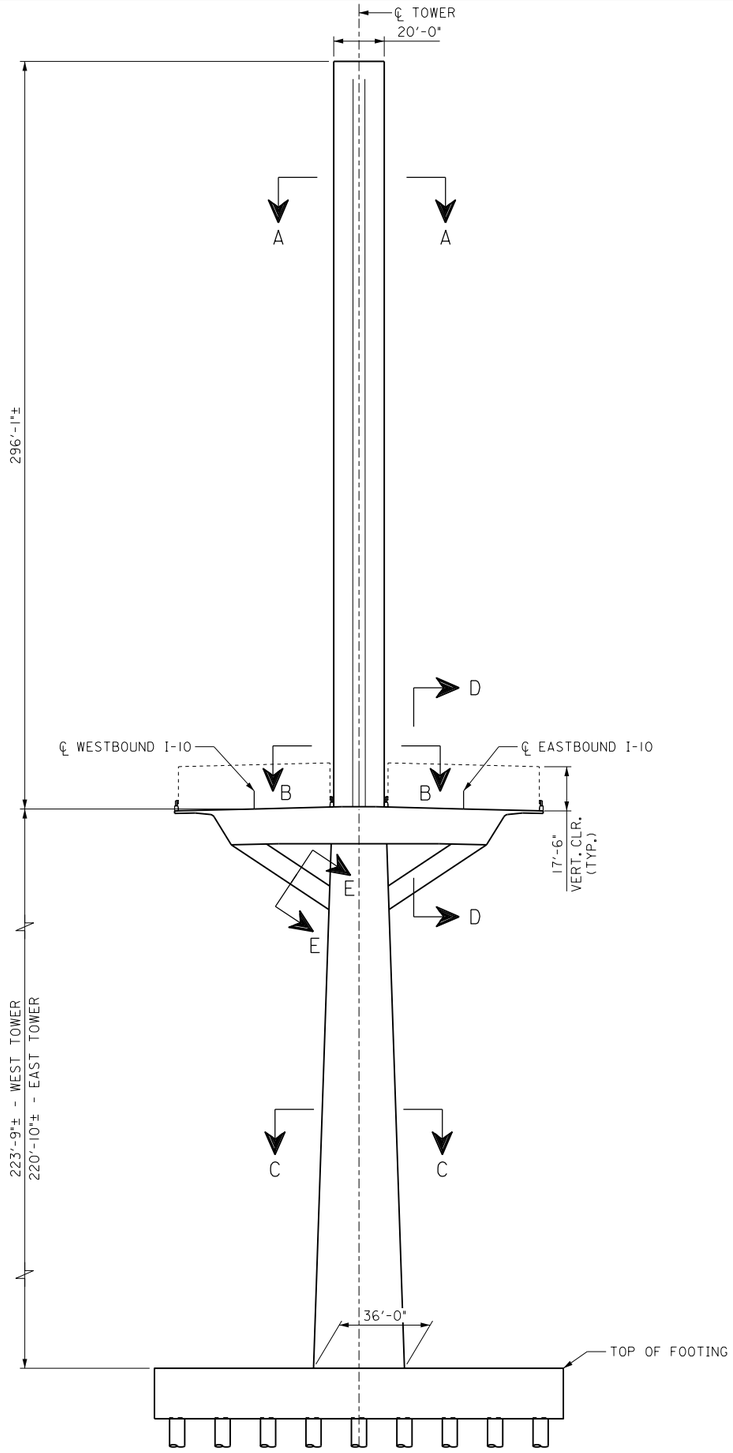
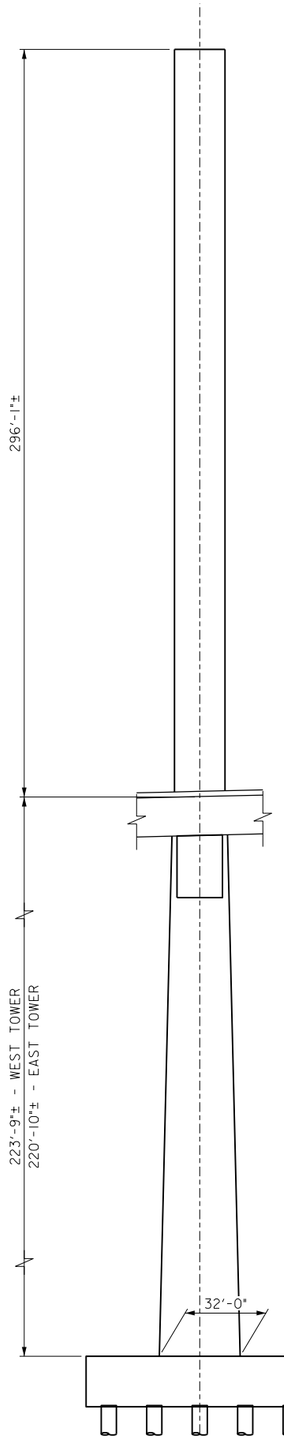
SCALE: 1:1.99992

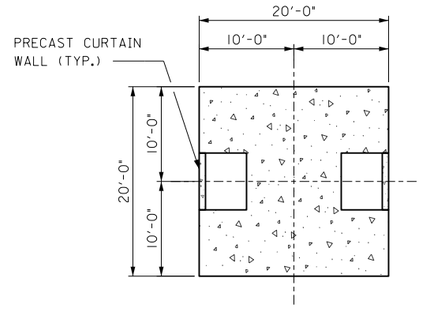
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		4



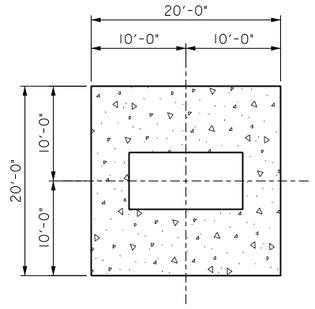
TRANSVERSE SECTION AT TOWERS
SCALE: 1"=30'



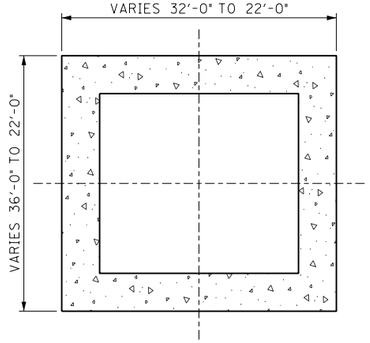
ELEVATION AT TOWERS
SCALE: 1"=30'



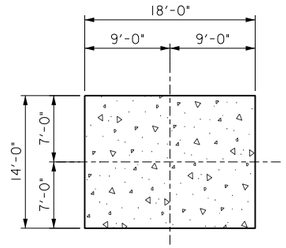
SECTION A-A
SCALE: 1/8" = 1'-0"



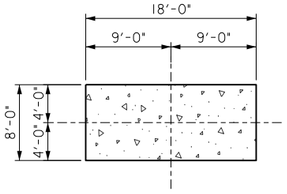
SECTION B-B
SCALE: 1/8" = 1'-0"



SECTION C-C
SCALE: 1/8" = 1'-0"



SECTION D-D
SCALE: 1/8" = 1'-0"



SECTION E-E
SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

TOWER SECTIONS
ALTERNATIVE I

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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DATE: Monday, July 31, 2017 04:03:47 PM
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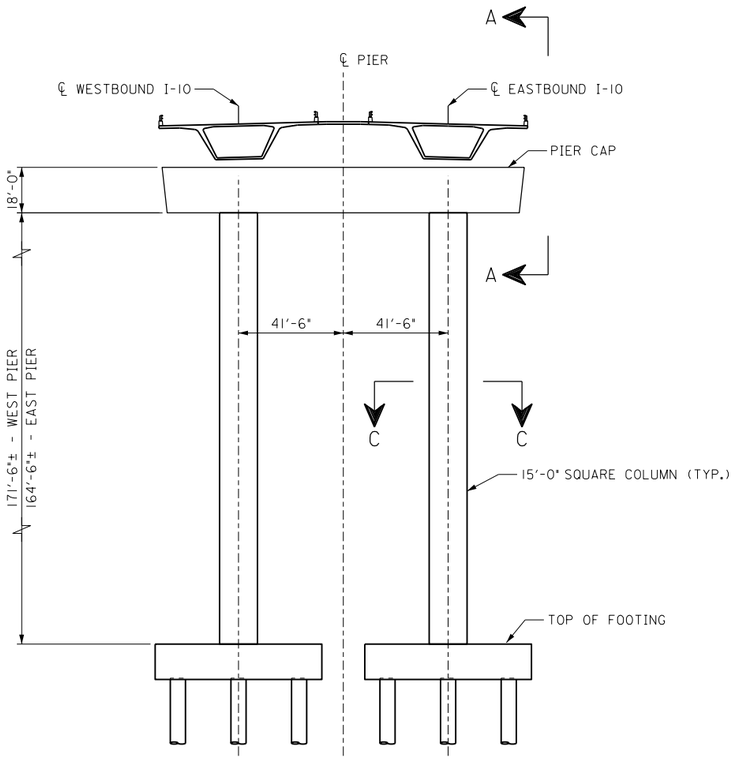
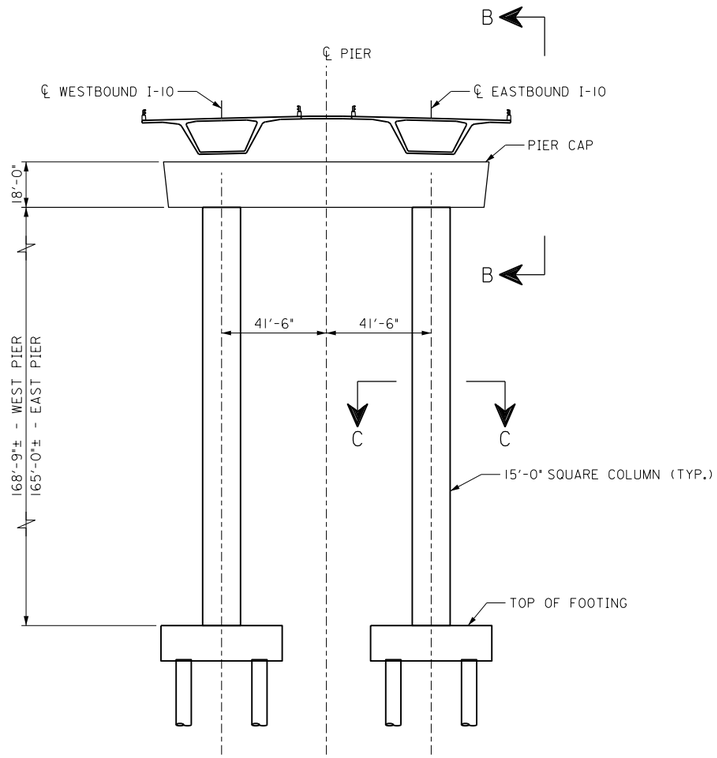
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		5



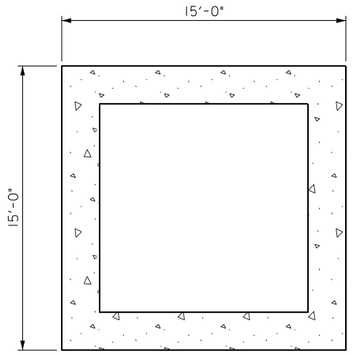
TRANSVERSE SECTION AT INTERIOR PIERS

SCALE: 1"=30'



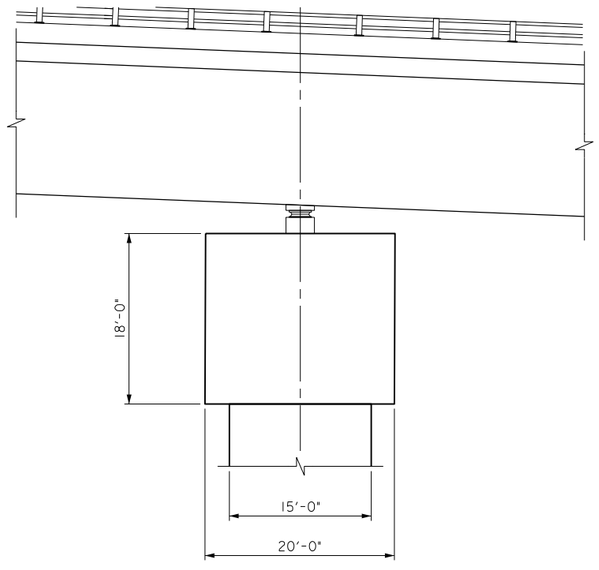
TRANSVERSE SECTION AT EXPANSION PIERS

SCALE: 1"=30'



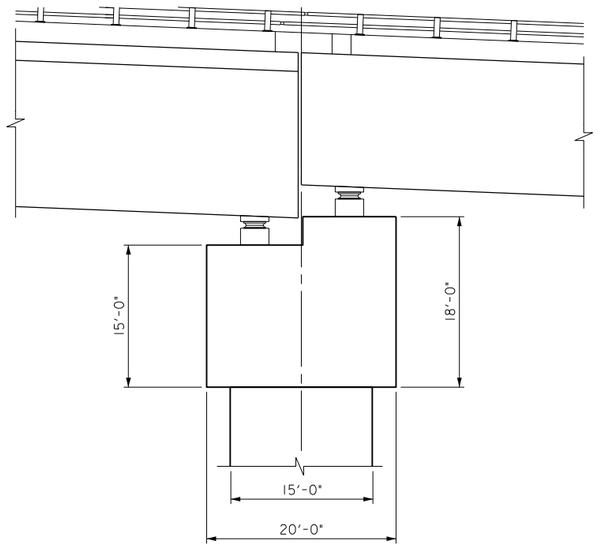
SECTION C-C

SCALE: 1/4" = 1'-0"



VIEW A-A

SCALE: 1/8" = 1'-0"



VIEW B-B

SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 7

REVISIONS

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

BACK SPAN PIER SECTIONS
ALTERNATIVE I

BIN:	ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
	COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
	VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.plt
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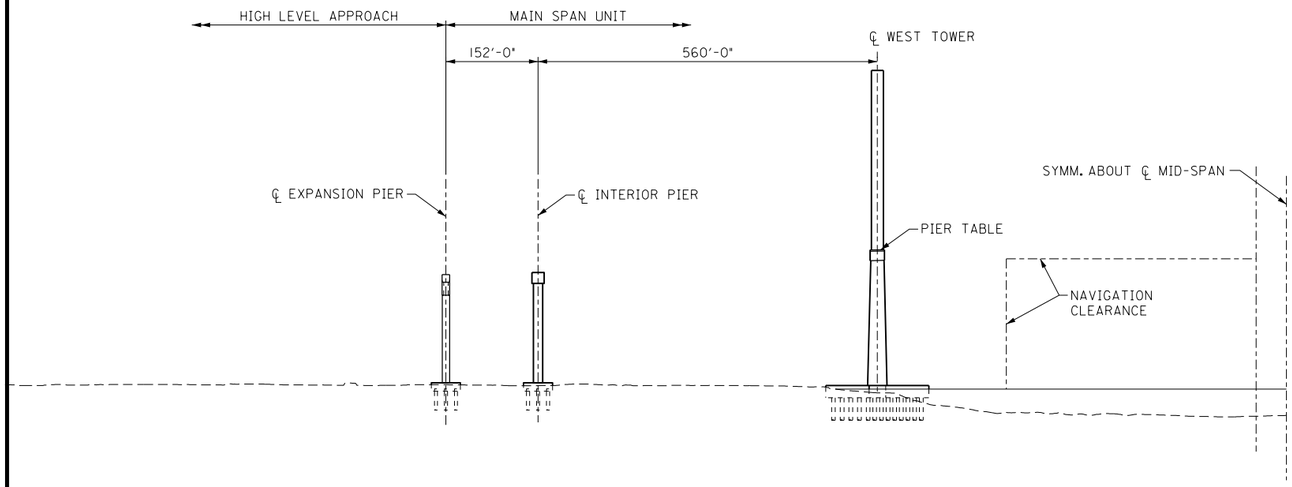
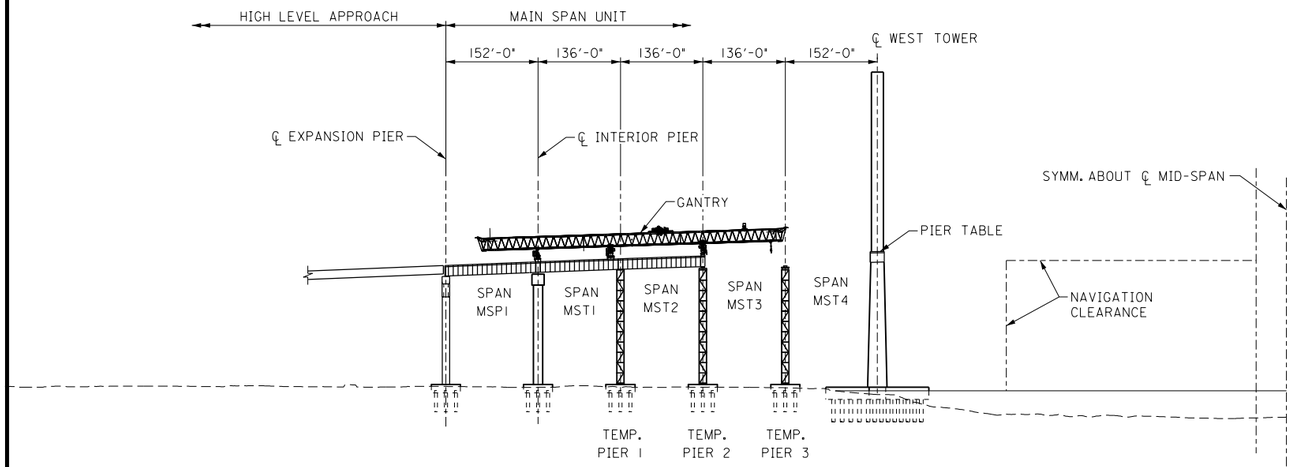
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		6



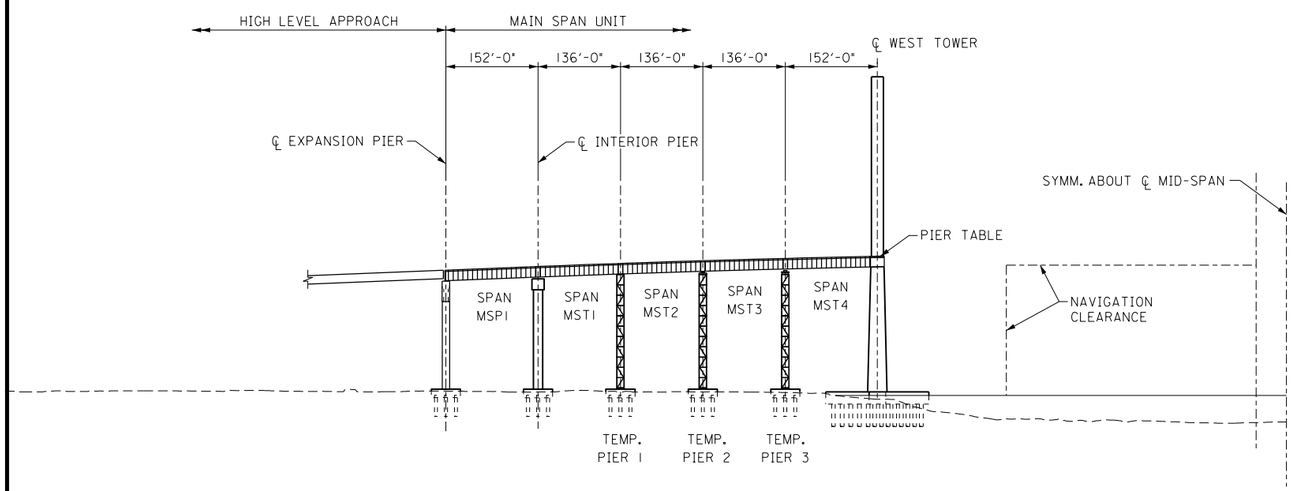
STEP 1

1. CONCURRENTLY WITH HIGH LEVEL APPROACH SPANS, BUILD TOWERS AND PIER TABLE.
2. BUILD EAST AND WEST EXPANSION AND INTERIOR PIERS.



STEP 2

1. ERECT BACK SPAN TEMPORARY PIERS.
2. SET ERECTION GANTRY FOR SPAN MSP1 WESTBOUND.
3. INSTALL WESTBOUND SEGMENTS FOR SPAN MSP1 USING SPAN-BY-SPAN ERECTION METHOD WITH SEGMENTS DELIVERED USING THE PREVIOUSLY COMPLETED HIGH LEVEL APPROACH SPANS.
4. ADVANCE GANTRY TO TEMPORARY SPAN MST1.
5. INSTALL WESTBOUND SEGMENTS FOR MST1 USING SPAN-BY-SPAN ERECTION METHOD.
6. ADVANCE GANTRY TO TEMPORARY SPAN MST2.



STEP 3

1. INSTALL WESTBOUND SEGMENTS FOR TEMPORARY SPAN MST2 USING SPAN-BY-SPAN ERECTION METHOD.
2. ADVANCE GANTRY TO TEMPORARY SPAN MST3.
3. INSTALL WESTBOUND SEGMENTS FOR TEMPORARY SPAN MST3 USING SPAN-BY-SPAN ERECTION METHOD.
4. ADVANCE GANTRY TO TEMPORARY SPAN MST4.
5. INSTALL WESTBOUND SEGMENTS FOR TEMPORARY SPAN MST4 USING SPAN-BY-SPAN ERECTION METHOD.
6. MOVE GANTRY TO SPAN MSP1 EASTBOUND.
7. REPEAT STEPS 2 AND 3 FOR EASTBOUND.
8. ERECT BACKSPAN DELTA FRAMES AND CAST BACK SPAN MEDIAN SLAB.
9. REMOVE ERECTION GANTRY.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 6 OF 7	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL
REVISIONS	ERECTION SEQUENCE (1 OF 2) ALTERNATIVE 1
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: JJP / DRAWN BY: SRC
	COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/17/2016
	VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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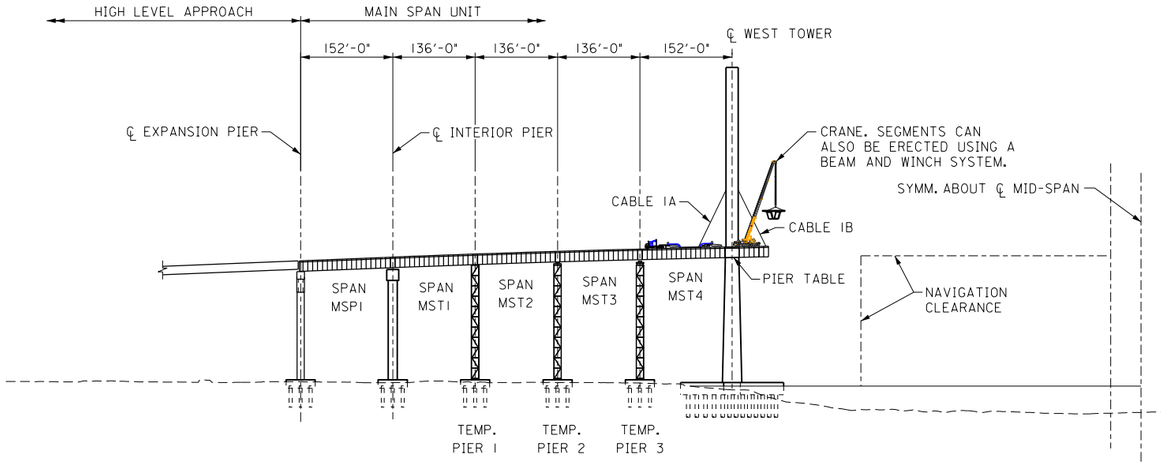
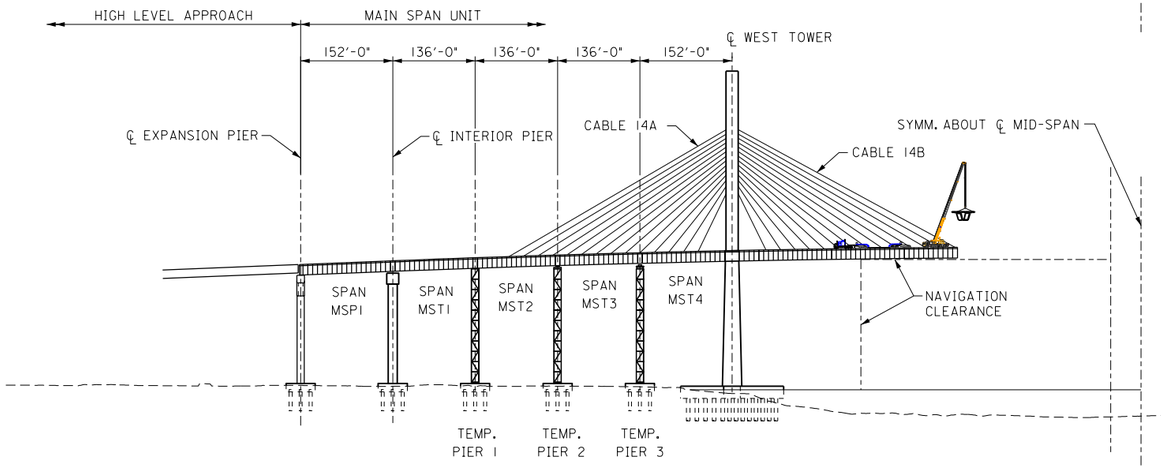
DATE: Monday, July 31, 2017 04:04:01 PM

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		7



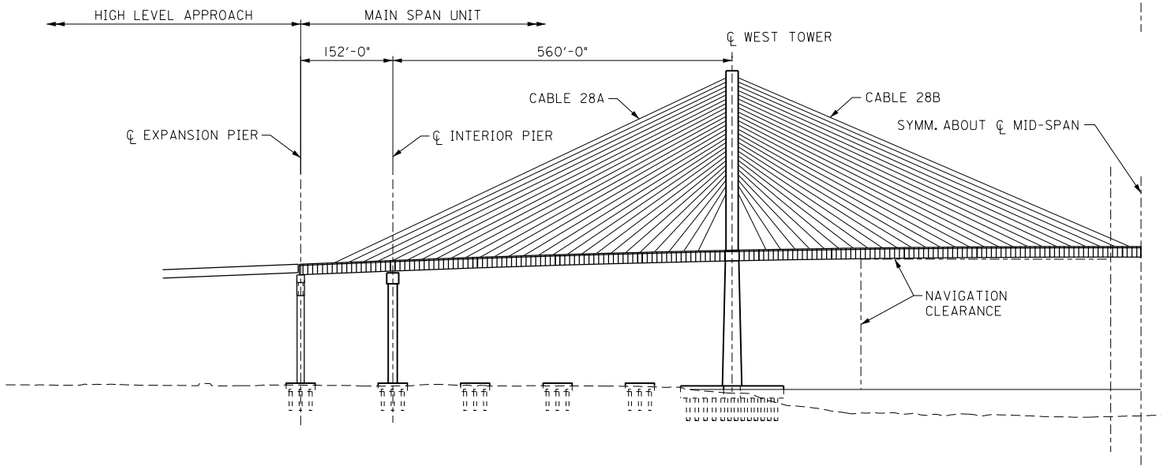
STEP 4

1. ERECT FIRST 3 MAINSPAN SEGMENT PAIRS IN CANTILEVER.
2. ERECT MAINSPAN SEGMENT PAIRS IN CANTILEVER.
   a. ERECT 3 SEGMENT PAIRS.
   b. INSTALL DELTA FRAME AND CAST MEDIAN SLAB.
   c. INSTALL STAY CABLES 1A/B.
   d. REPEAT STEPS a THROUGH c FOR STAY CABLES THROUGH 13A/13B.



STEP 5

1. ERECT MAINSPAN SEGMENT PAIRS IN CANTILEVER.
   a. ERECT 3 SEGMENT PAIRS.
   b. INSTALL DELTA FRAME AND CAST MEDIAN SLAB.
   c. INSTALL STAY CABLES 14 A/B
   d. REPEAT STEPS a THROUGH c FOR STAY CABLES THROUGH 27A/27B.



STEP 6

1. ERECT MAINSPAN SEGMENT PAIRS IN CANTILEVER.
   a. ERECT 3 SEGMENT PAIRS.
   b. INSTALL DELTA FRAME AND CAST MEDIAN SLAB.
   c. INSTALL STAY CABLES 28 A/B.
   d. CAST MAINSPAN CLOSURE.
   e. REMOVE TEMPORARY PIERS.
   f. CAST CONCRETE PARAPETS.

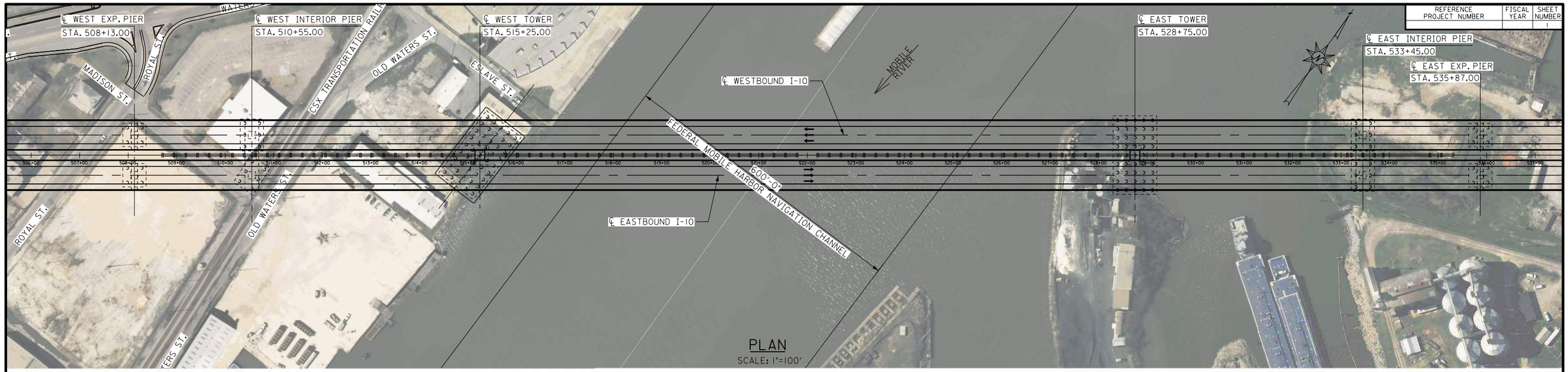
NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

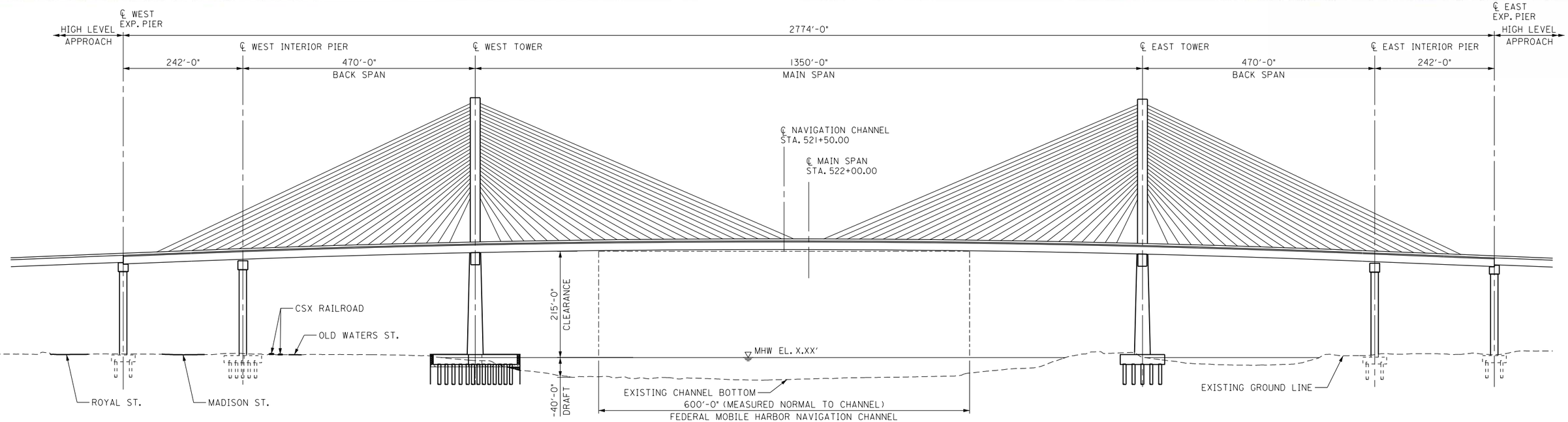
BRIDGE SHEET NO. 7 OF 7	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL
REVISIONS	ERECTION SEQUENCE (2 OF 2) ALTERNATIVE I
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: JJP / DRAWN BY: SRC
	COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/17/2016
	VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
PENTABLE: HDR-ALDOT.tbl
SCALE: 1:1.99992
DATE: Monday, July 31, 2017 04:04:02 PM

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		1



PLAN
SCALE: 1"=100'



ELEVATION
SCALE: 1"=100'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 8

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

PLAN & ELEVATION
ALTERNATIVE IA

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.plotcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:19 PM
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REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
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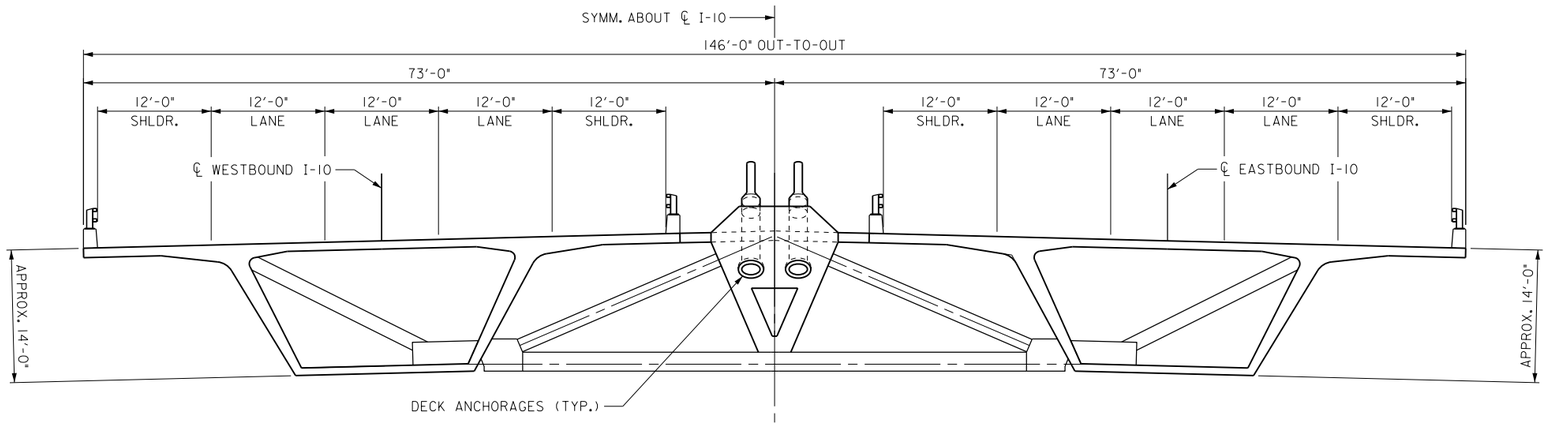
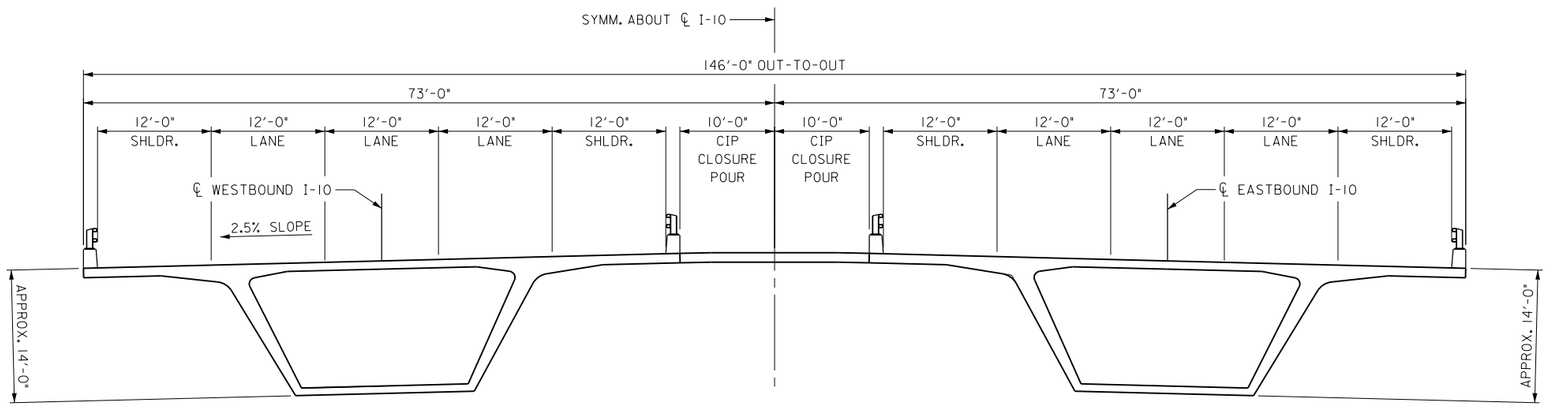


TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BETWEEN DECK ANCHORAGES

SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 8

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

DECK TYPICAL SECTIONS
ALTERNATIVE 1A

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
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DATE: Monday, July 31, 2017 04:04:22 PM
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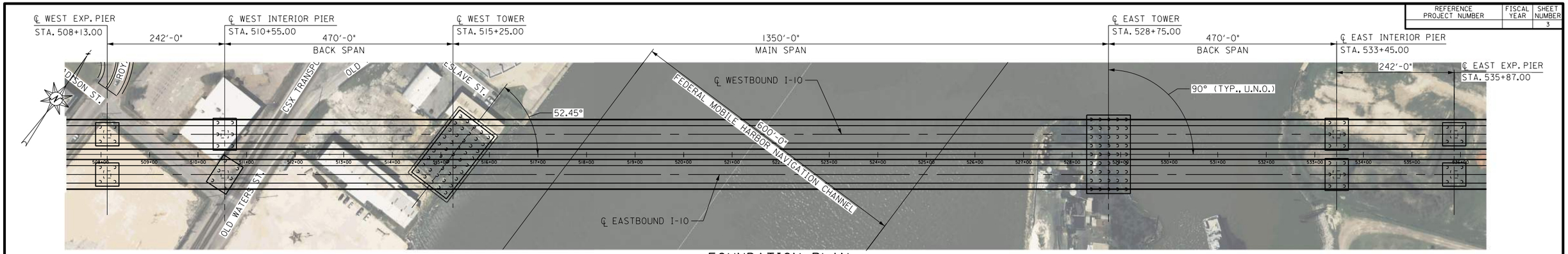
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		3



FOUNDATION PLAN

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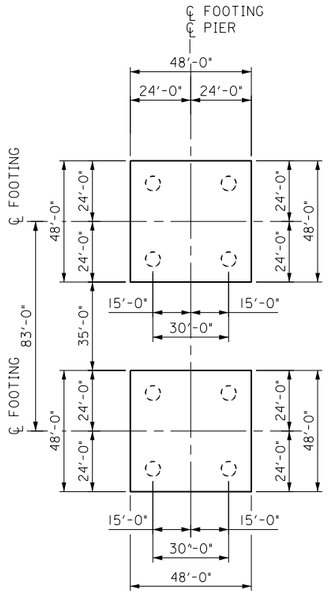
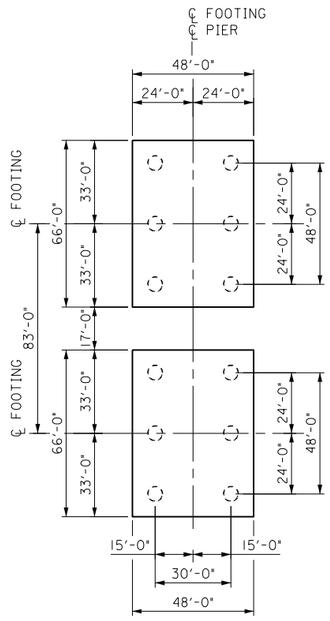


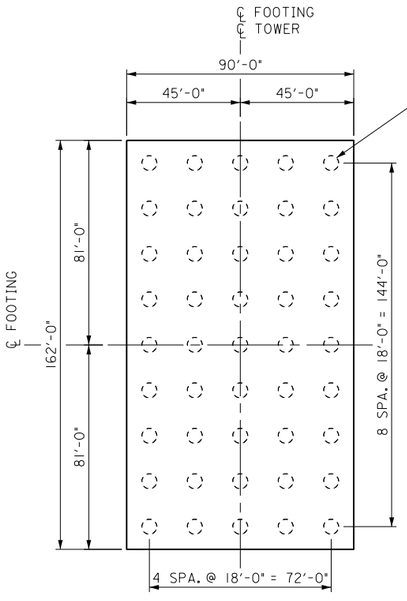
EXPANSION PIER FOOTING PLAN

SCALE: 1"=30'



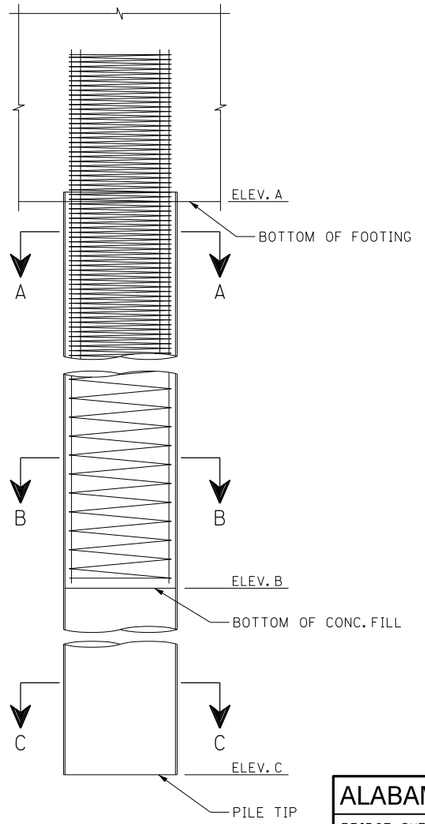
INTERIOR PIER FOOTING PLAN

SCALE: 1"=30'



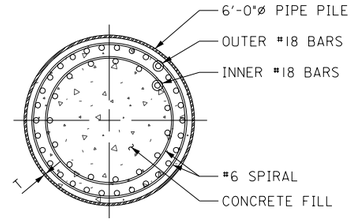
TOWER FOOTING PLAN

SCALE: 1"=30'



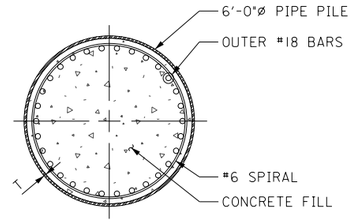
ELEVATION

SCALE: 1/4" = 1'-0"



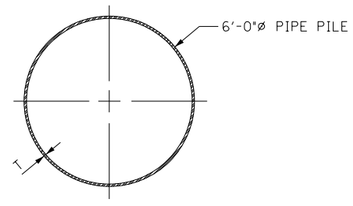
SECTION A-A

SCALE: 3/8" = 1'-0"



SECTION B-B

SCALE: 3/8" = 1'-0"



SECTION C-C

SCALE: 3/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 8

REVISIONS

BIN:

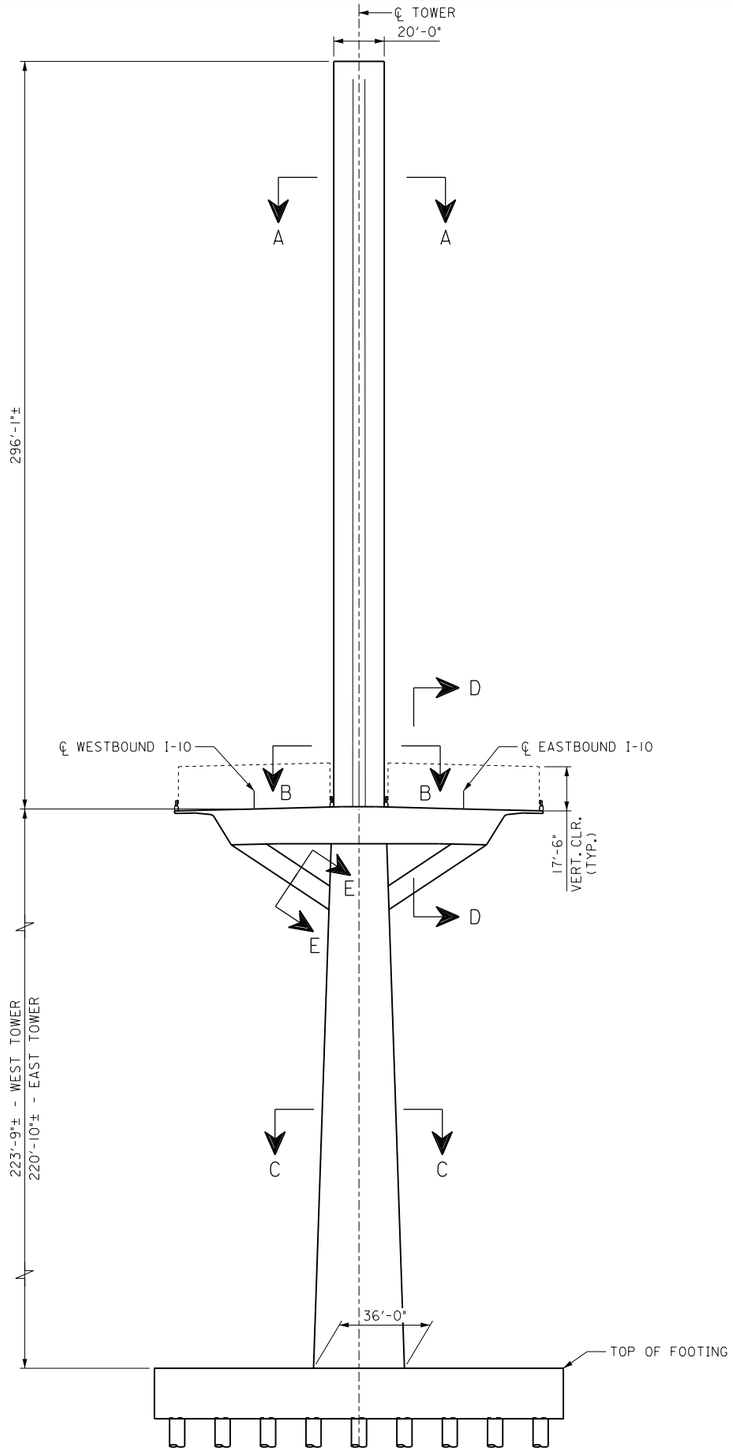
MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

FOUNDATION PLAN & PILE DETAILS
ALTERNATIVE IA

ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

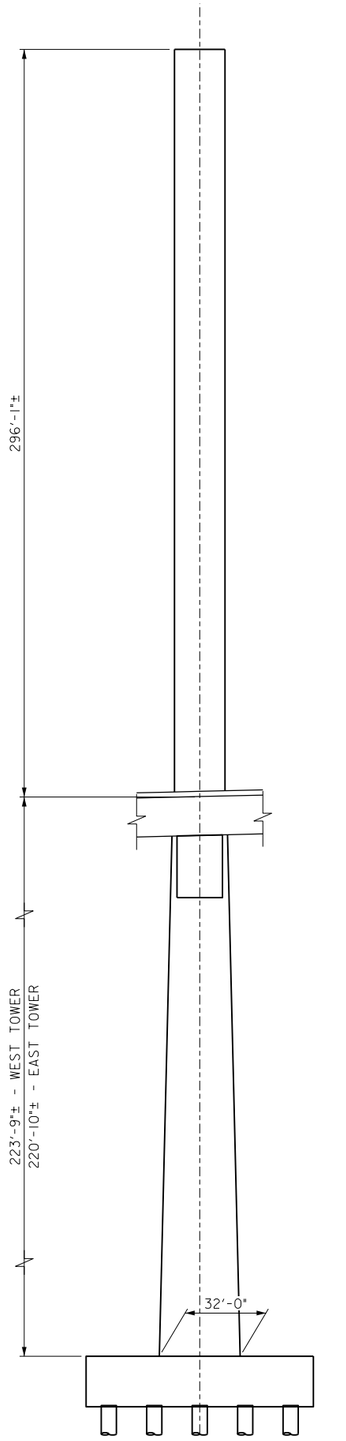
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:24 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		4



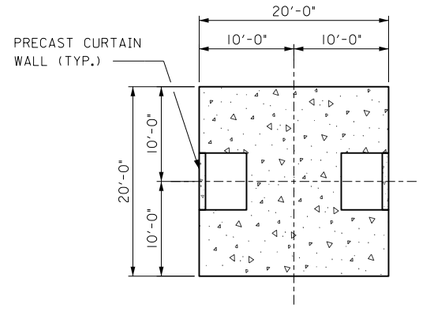
TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



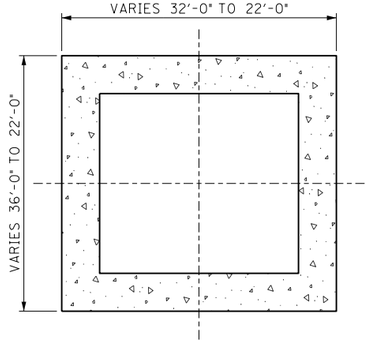
ELEVATION AT TOWERS

SCALE: 1"=30'



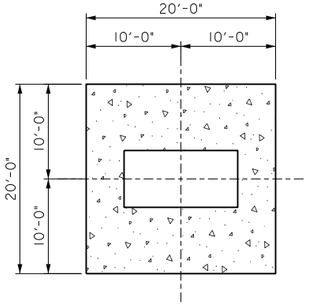
SECTION A-A

SCALE: 1/8" = 1'-0"



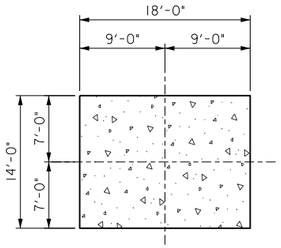
SECTION C-C

SCALE: 1/8" = 1'-0"



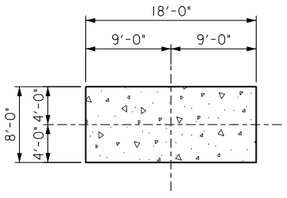
SECTION B-B

SCALE: 1/8" = 1'-0"



SECTION D-D

SCALE: 1/8" = 1'-0"



SECTION E-E

SCALE: 1/8" = 1'-0"

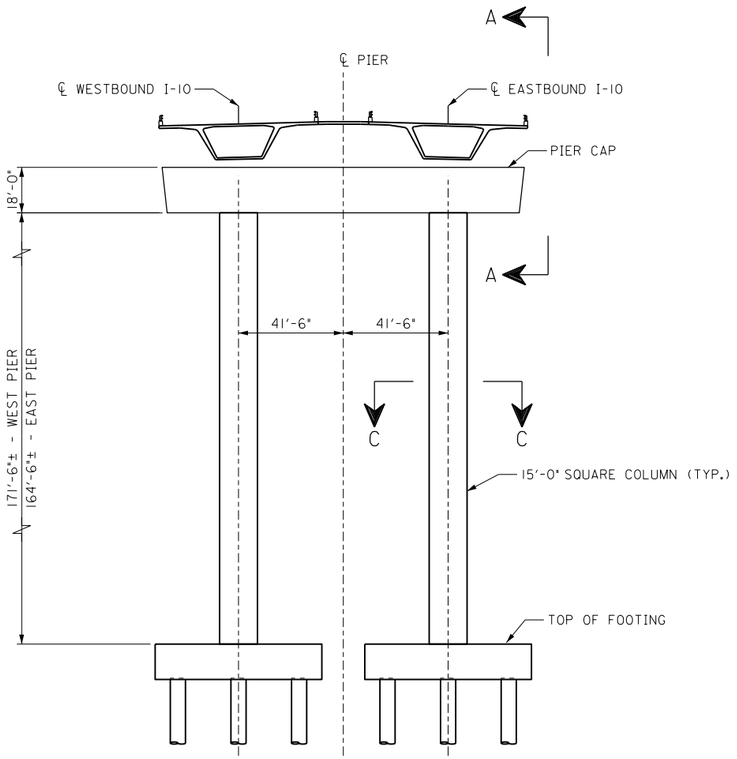
NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 8	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL
REVISIONS	TOWER SECTIONS ALTERNATIVE 1A
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: JJP / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/17/2016 VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

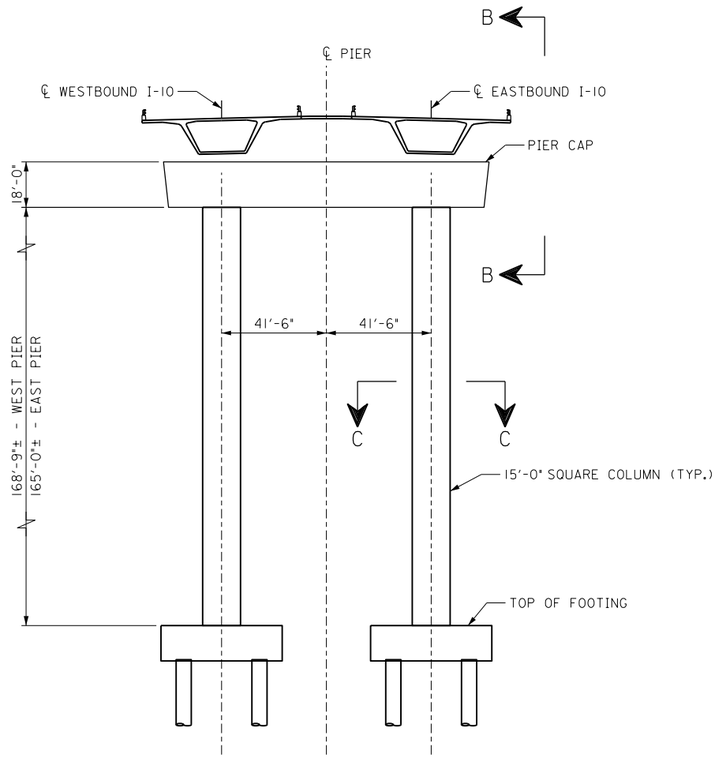
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:25 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		5



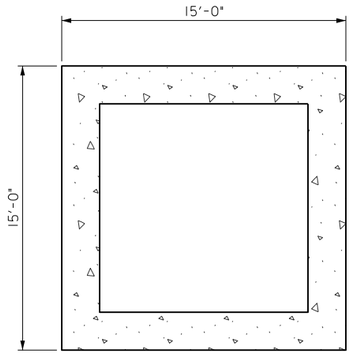
TRANSVERSE SECTION AT INTERIOR PIERS

SCALE: 1"=30'



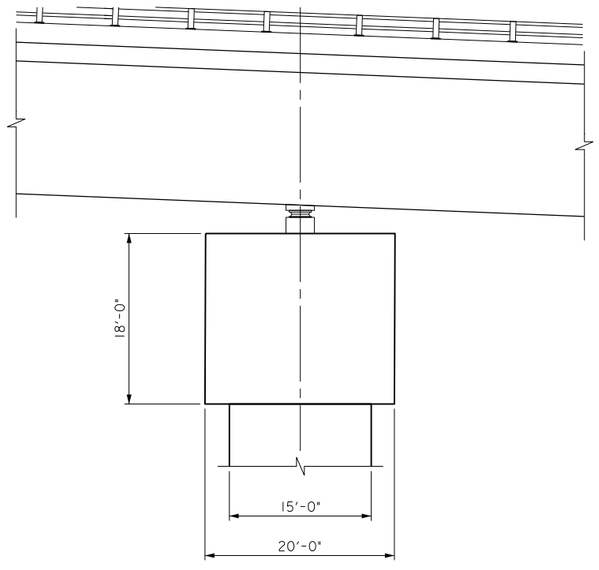
TRANSVERSE SECTION AT EXPANSION PIERS

SCALE: 1"=30'



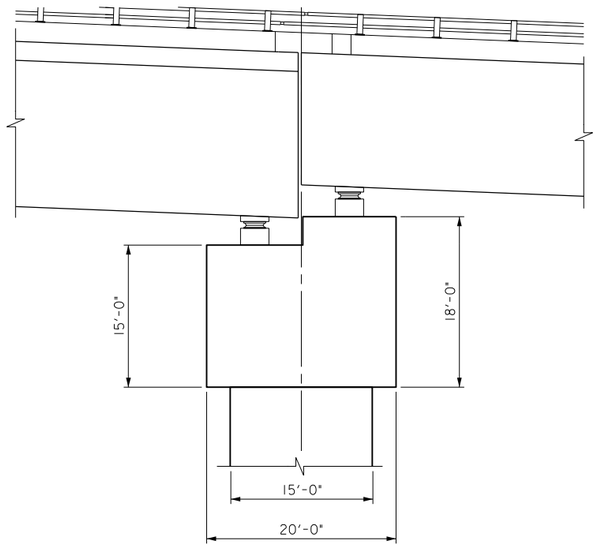
SECTION C-C

SCALE: 1/4" = 1'-0"



VIEW A-A

SCALE: 1/8" = 1'-0"



VIEW B-B

SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 8

REVISIONS

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

BACK SPAN PIER SECTIONS
ALTERNATIVE 1A

	ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
BIN:	COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
	VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:26 PM
PENTABLE: HDR_ALDOT.tbl
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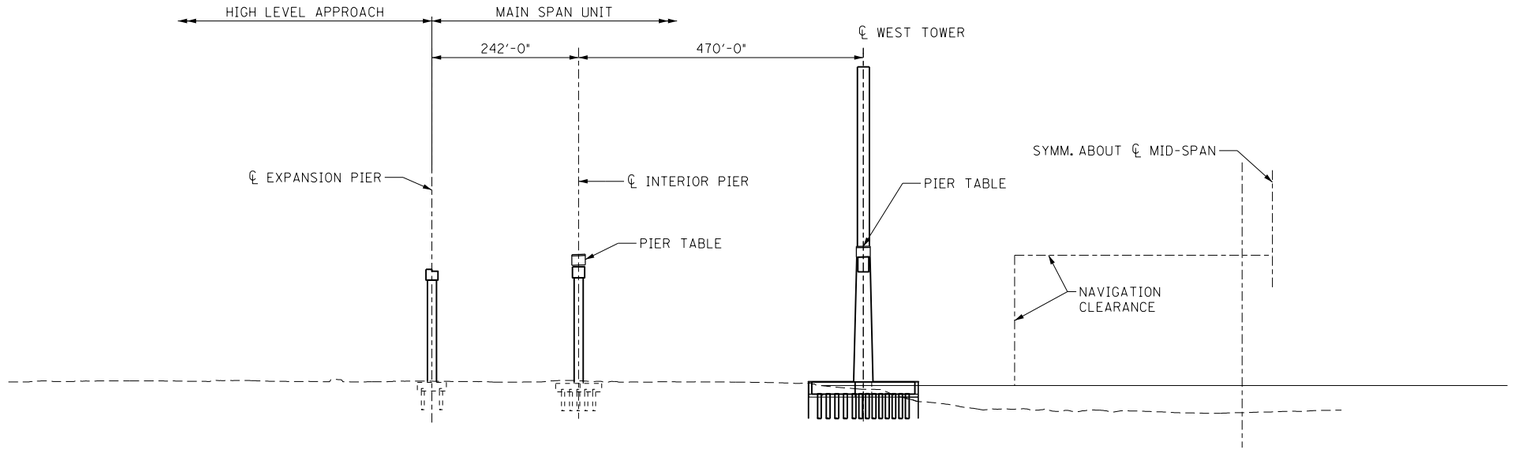
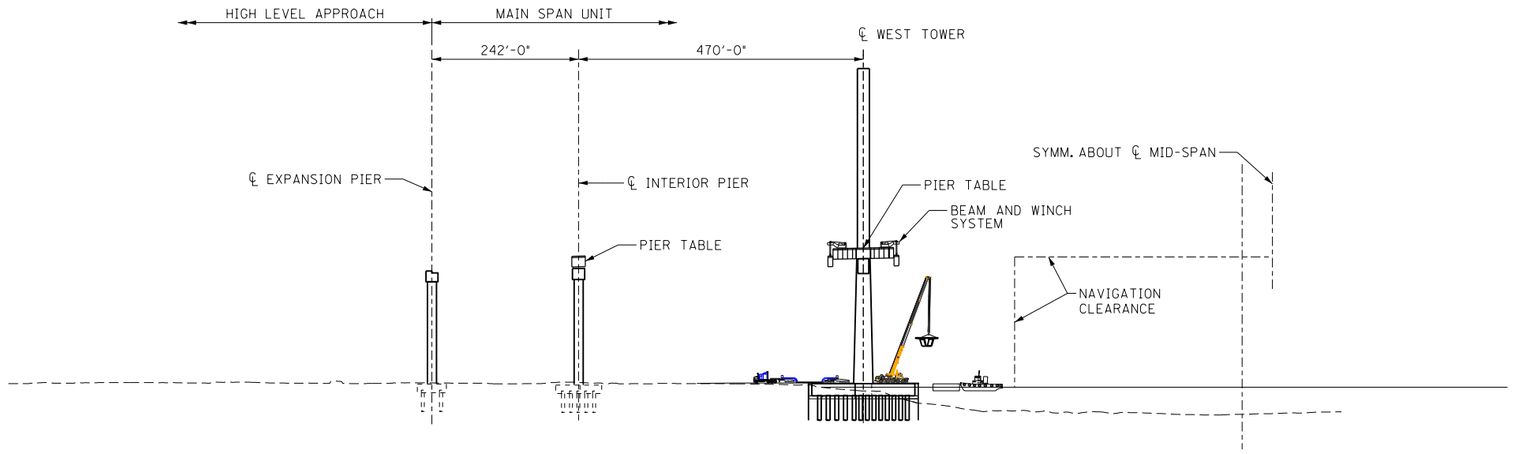
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		6



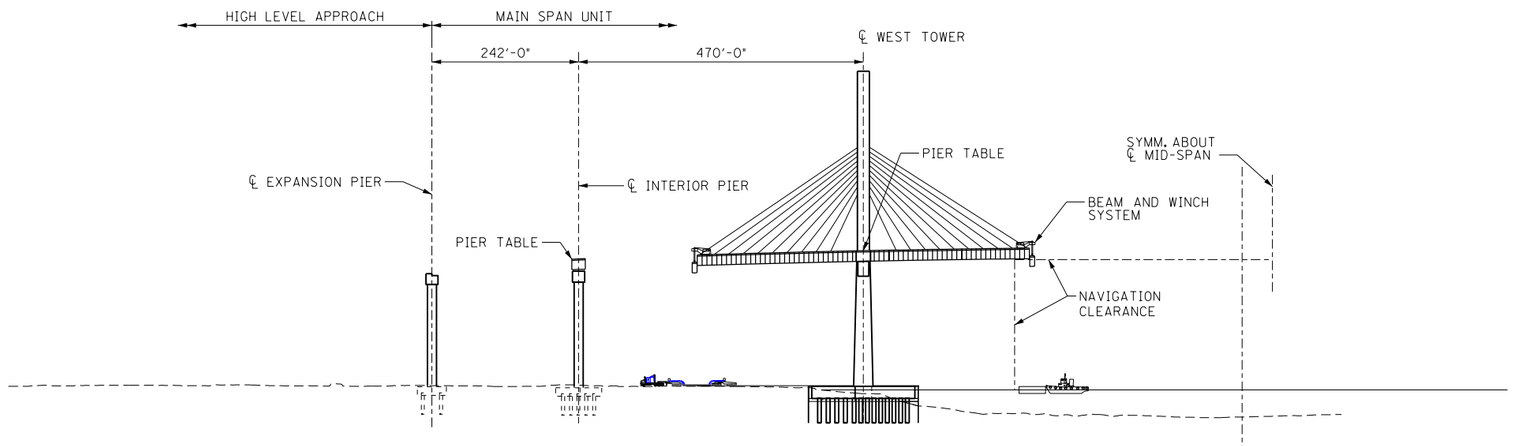
STEP 1

1. CONCURRENTLY WITH HIGH LEVEL APPROACH SPANS, BUILD TOWERS AND PIER TABLES.
2. BUILD EAST AND WEST EXPANSION AND INTERIOR PIERS.



STEP 2

1. INSTALL BEAM AND WINCH SYSTEM ON PIER TABLE.
2. BEGIN LIFTING PRECAST SEGMENTS INTO PLACE EMPLOYING BEAM AND WINCH SYSTEM.
3. DELIVER SEGMENTS FOR WEST BACK SPAN BY TRUCK, AND DELIVER THE REST OF THE SEGMENTS BY BARGE.
4. ERECT MAIN SPAN SEGMENTS IN PAIRS:
   a. ERECT 3 MAIN SPAN SEGMENT PAIRS
   b. INSTALL DELTA FRAME AND CAST MEDIAN
   c. INSTALL STAY CABLE



STEP 3

1. CONTINUE CONSTRUCTION OF SUPERSTRUCTURE CANTILEVER.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 6 OF 8	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL		
REVISIONS	ERECTION SEQUENCE (1 OF 3) ALTERNATIVE IA		
	ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
BIN:	COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
	VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR_ALDOT_PDF_BW_HALF_SIZE.pltcfg
USER: JSPANIER
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DATE: Monday, July 31, 2017 04:04:34 PM
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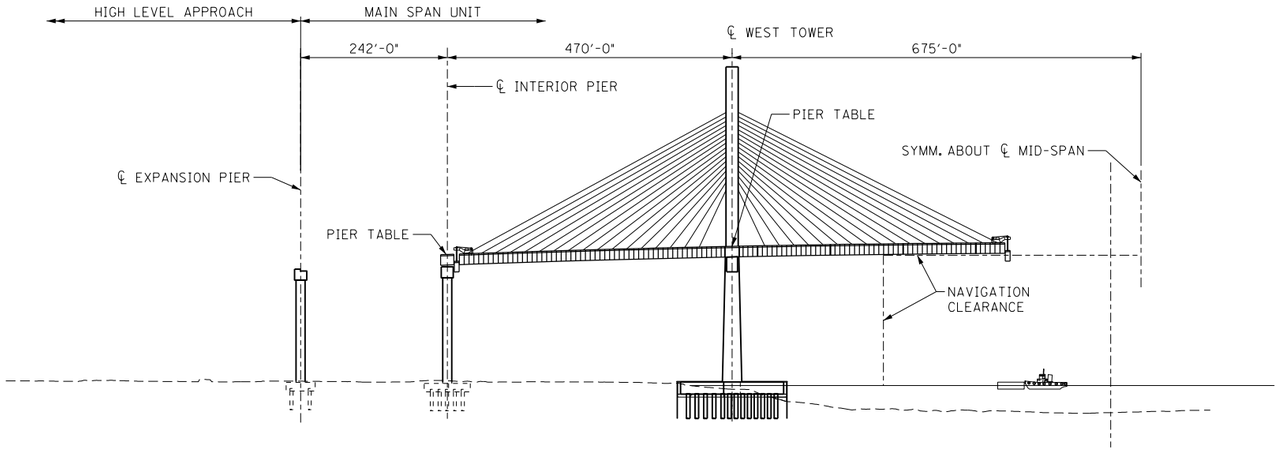
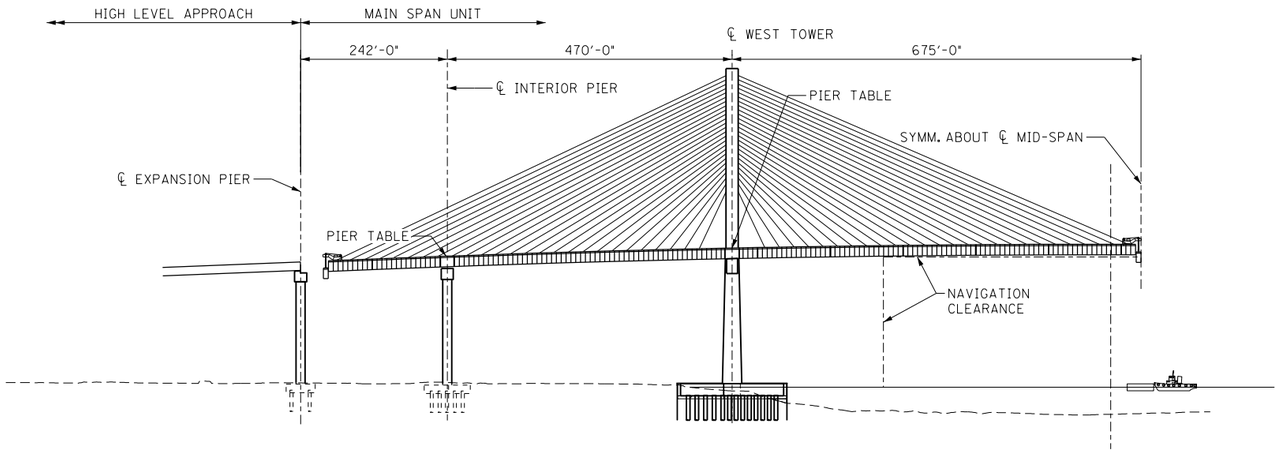
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		7



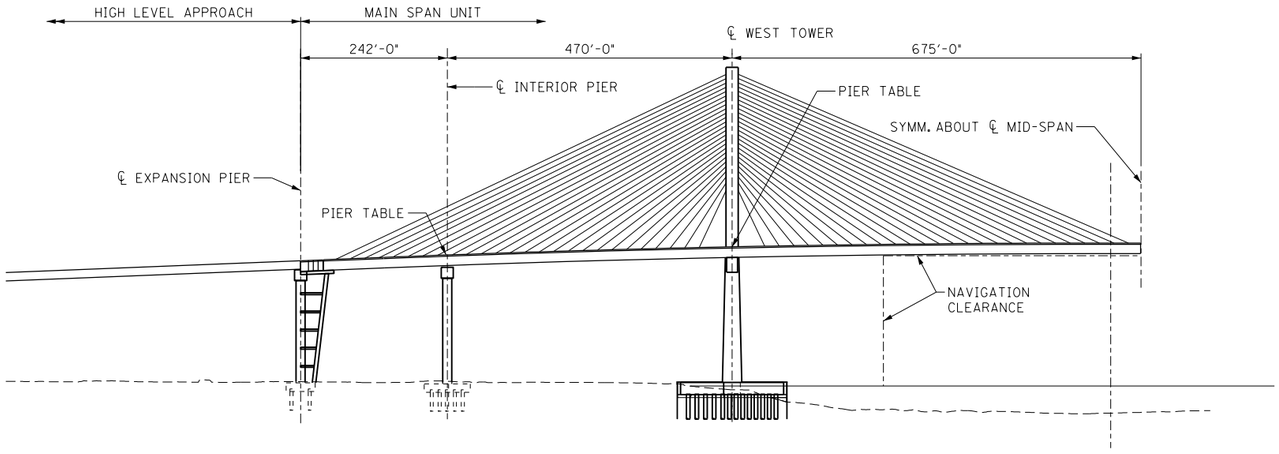
STEP 4

1. CAST CLOSURE SEGMENT BETWEEN BACK SPAN CANTILEVER SEGMENT AND PIER TABLE
2. AFTER CLOSURE SEGMENT CONCRETE HAS REACHED THE SPECIFIED STRENGTH, COUPLE AND STRESS P.T. BARS.



STEP 5

1. ADVANCE BEAM AND WINCH AND CONTINUE CANTILEVER CONSTRUCTION.



STEP 6

1. INSTALL FALSEWORK AND TEMPORARY SUPPORT NEAR EXPANSION JOINT PIER.
2. ERECT SEGMENTS ON FALSEWORK.
3. CAST CLOSURE SEGMENT IN BACK SPAN
4. INSTALL AND STRESS P.T.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 7 OF 8	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL
REVISIONS	ERECTION SEQUENCE (2 OF 3) ALTERNATIVE IA
	ESTIMATED QUANTITIES / DESIGNED BY: JJP / DRAWN BY: SRC
BIN:	COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/17/2016
	VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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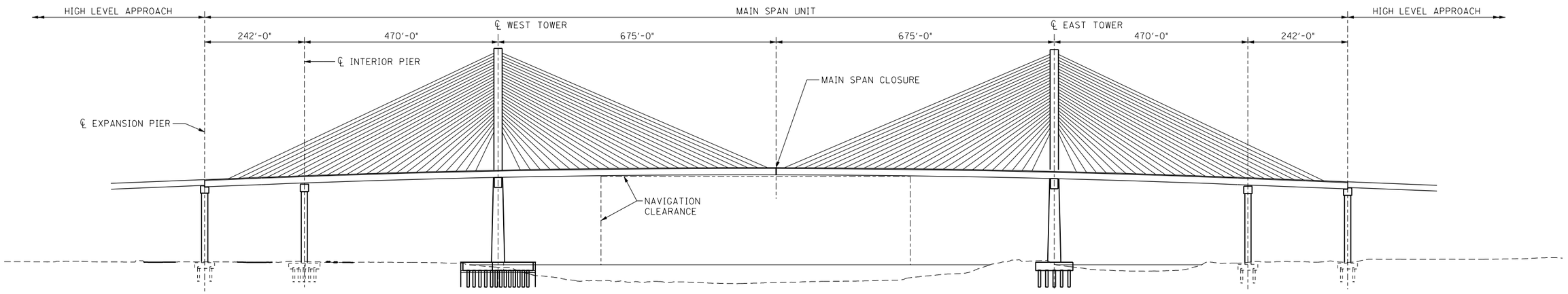
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		8



STEP 7

1. REMOVE FALSEWORK.
2. CAST MAIN SPAN CLOSURE.
3. INSTALL AND STRESS P.T.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 8 OF 8	MOBILE RIVER BRIDGE MAIN SPAN PRECAST CONCRETE SEGMENTAL
REVISIONS	ERECTION SEQUENCE (3 OF 3) ALTERNATIVE IA

BIN:	ESTIMATED QUANTITIES	DESIGNED BY: JJP	DRAWN BY: SRC
	COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/17/2016
	VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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FILE: ø

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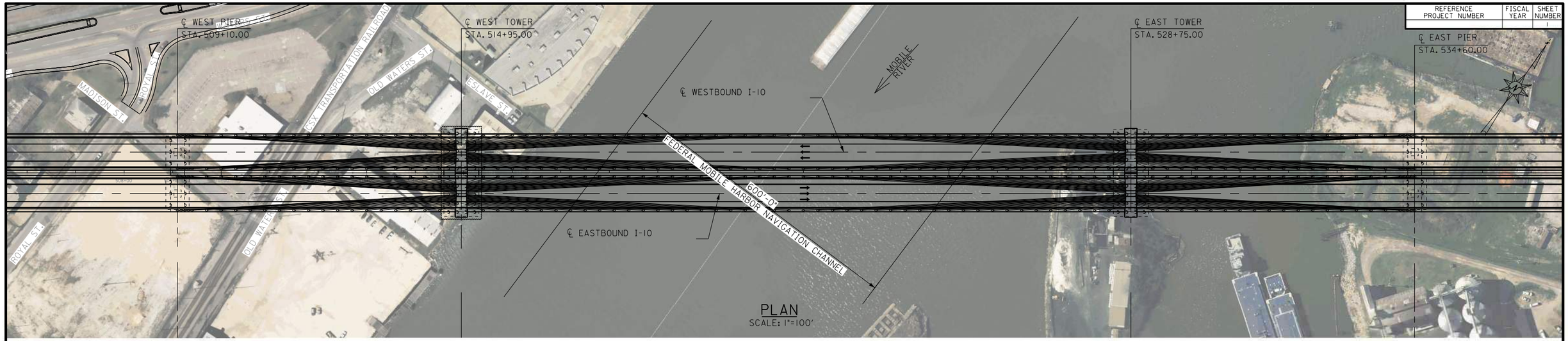
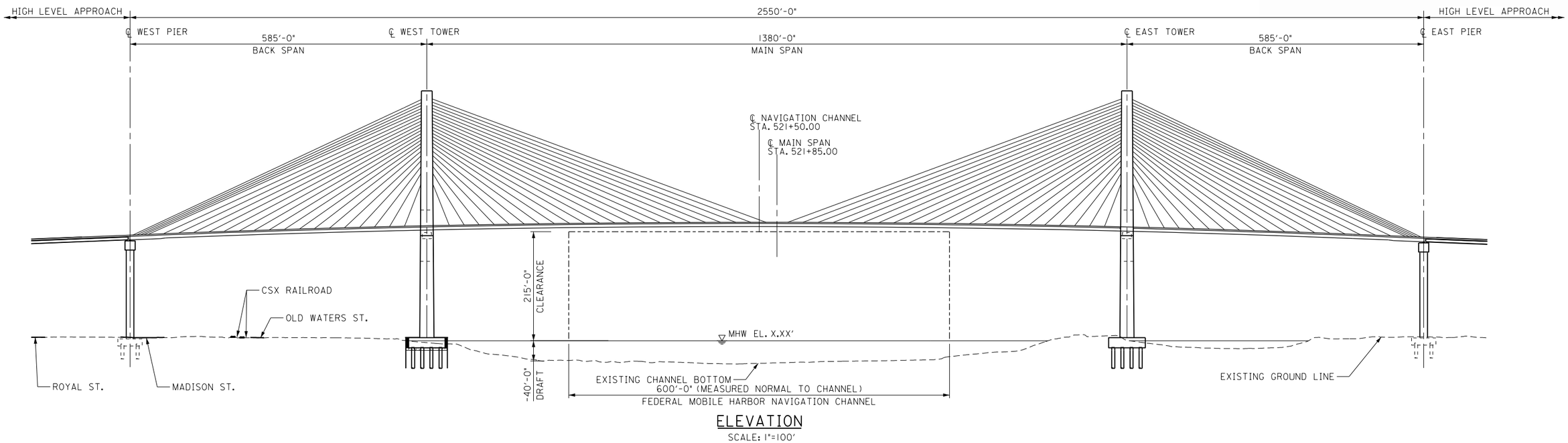
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		1



PLAN
SCALE: 1"=100'



ELEVATION
SCALE: 1"=100'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 7

REVISIONS

BIN:

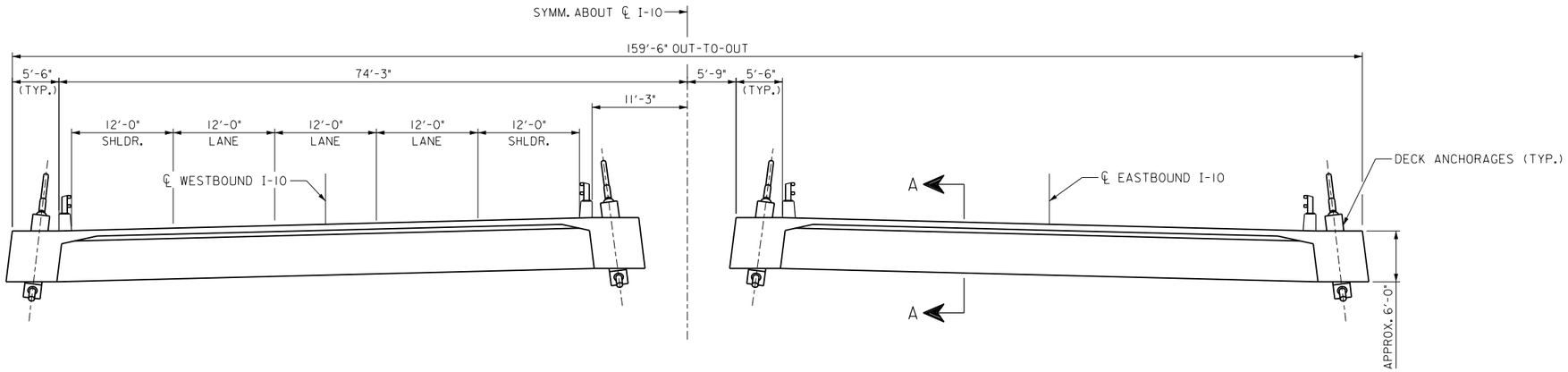
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE DIAMOND - CONCRETE

PLAN & ELEVATION
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

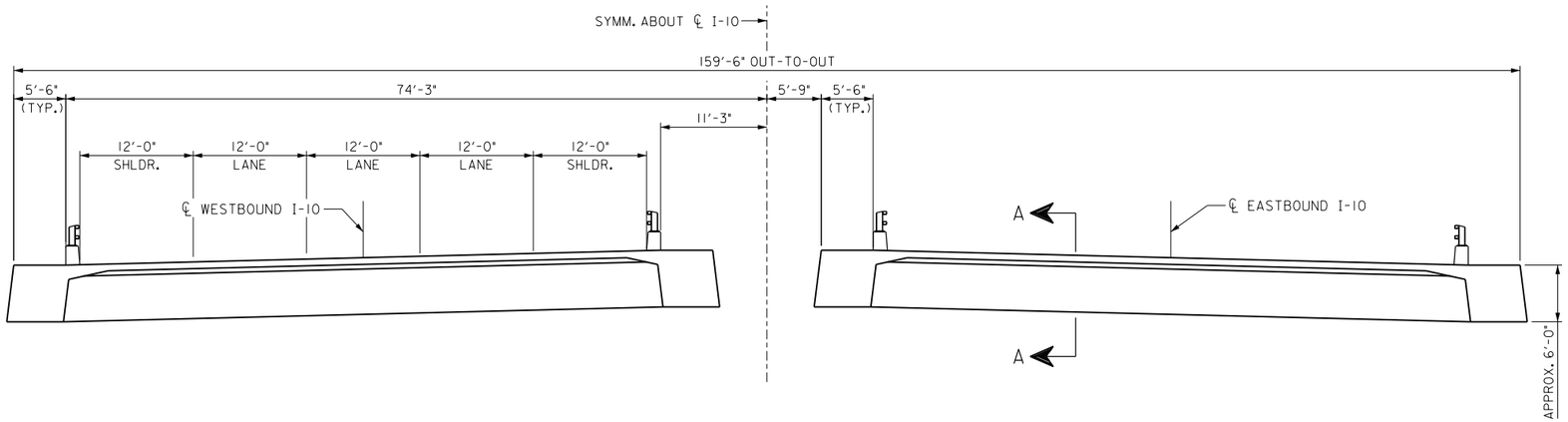
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:52 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		2



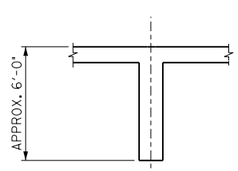
TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BETWEEN DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



SECTION A-A

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE DIAMOND - CONCRETE

DECK TYPICAL SECTIONS ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:04:55 PM
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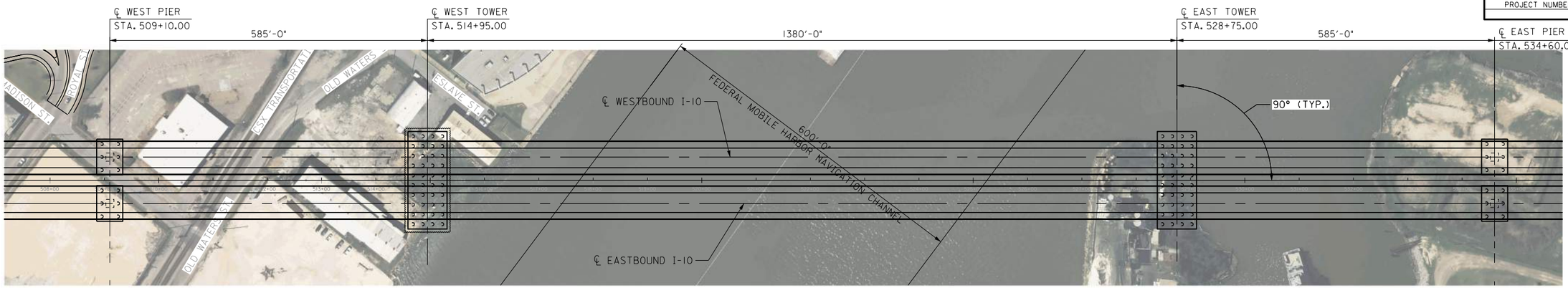
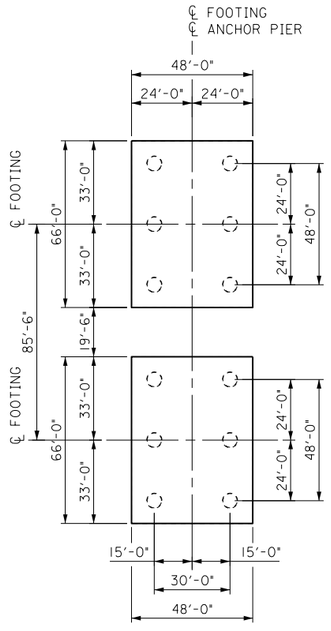
SCALE: 1:1.99992

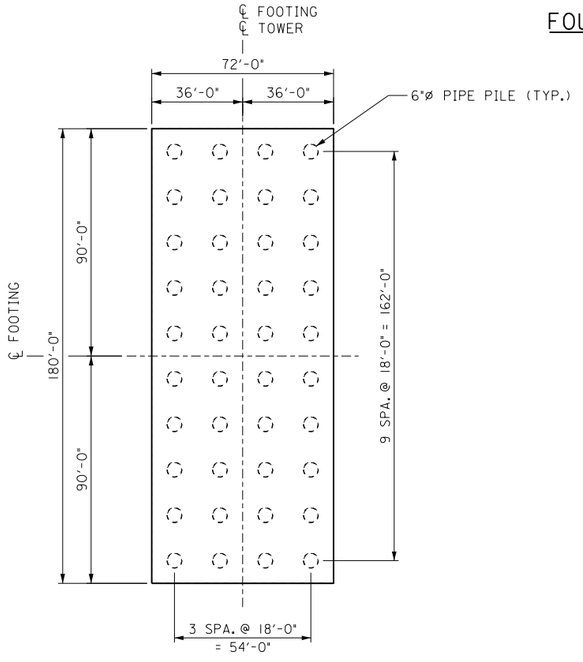
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		3



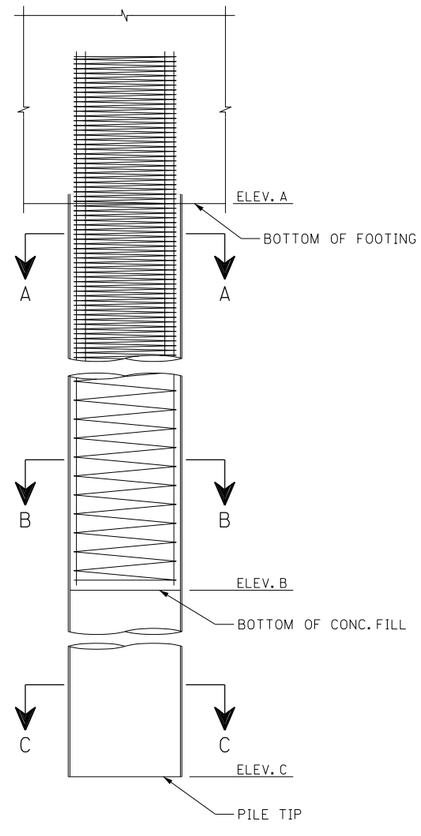
FOUNDATION PLAN
SCALE: 1"=100'



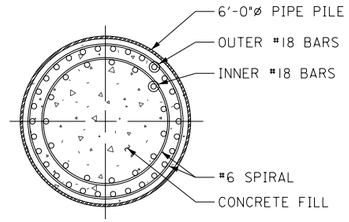
ANCHOR PIER FOOTING PLAN
SCALE: 1"=30'



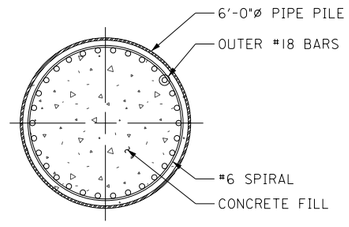
TOWER FOOTING PLAN
SCALE: 1"=30'



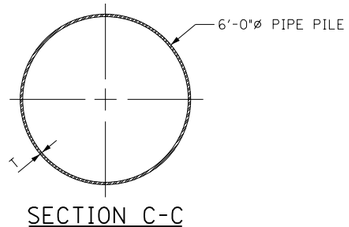
ELEVATION
SCALE: 1/4" = 1'-0"



SECTION A-A
SCALE: 3/8" = 1'-0"



SECTION B-B
SCALE: 3/8" = 1'-0"



SECTION C-C
SCALE: 3/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION		
BRIDGE SHEET NO. 3 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE	
REVISIONS	FOUNDATION PLAN & PILE DETAILS ALTERNATE 3	
	ESTIMATED QUANTITIES	DESIGNED BY: ADM / DRAWN BY: SRC
	COMPUTED BY:	CHECKED BY: / DATE DRAWN: 8/18/2016
BIN:	VERIFIED BY:	DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HOR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
PENTABLE: HOR-ALDOT.tbl
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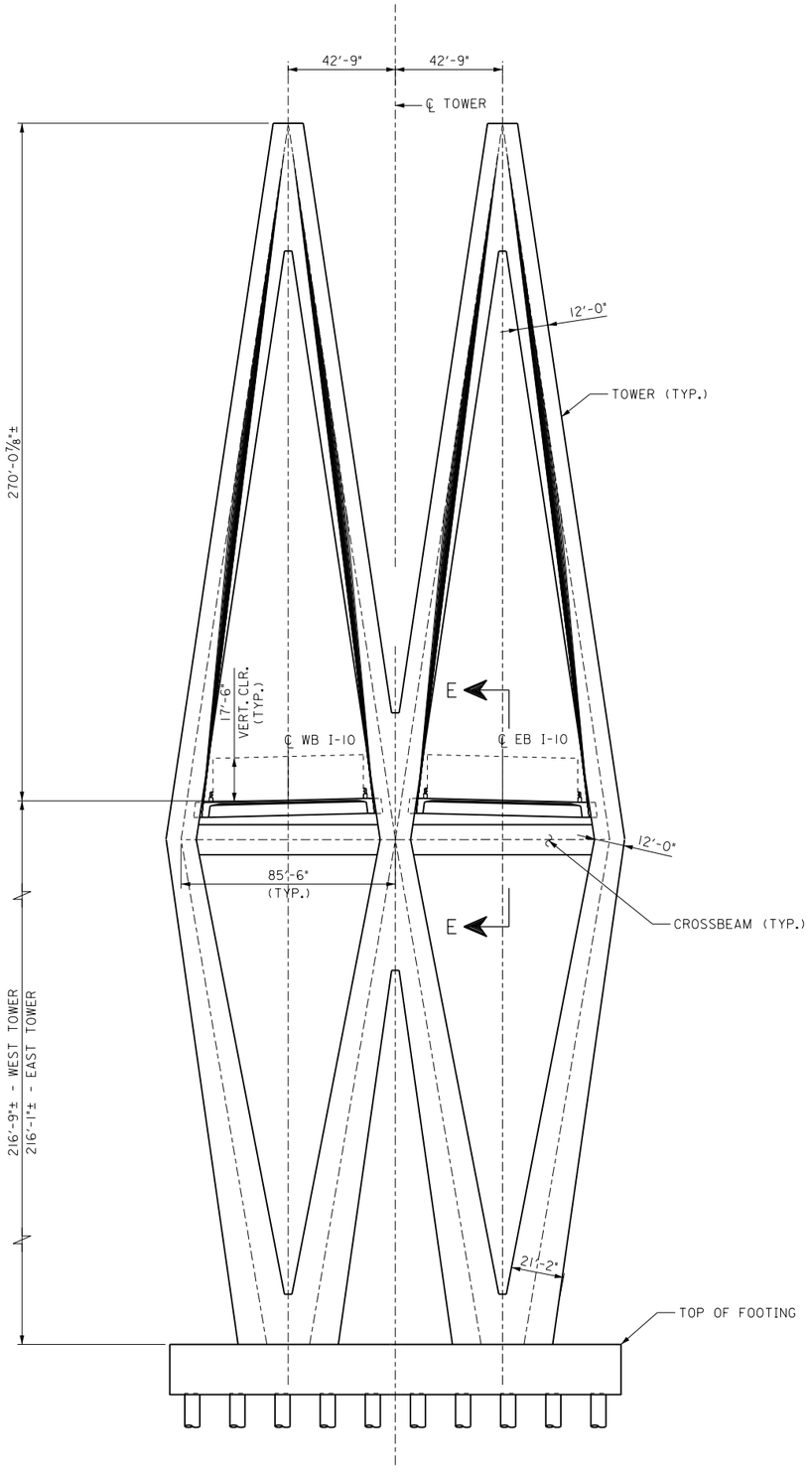
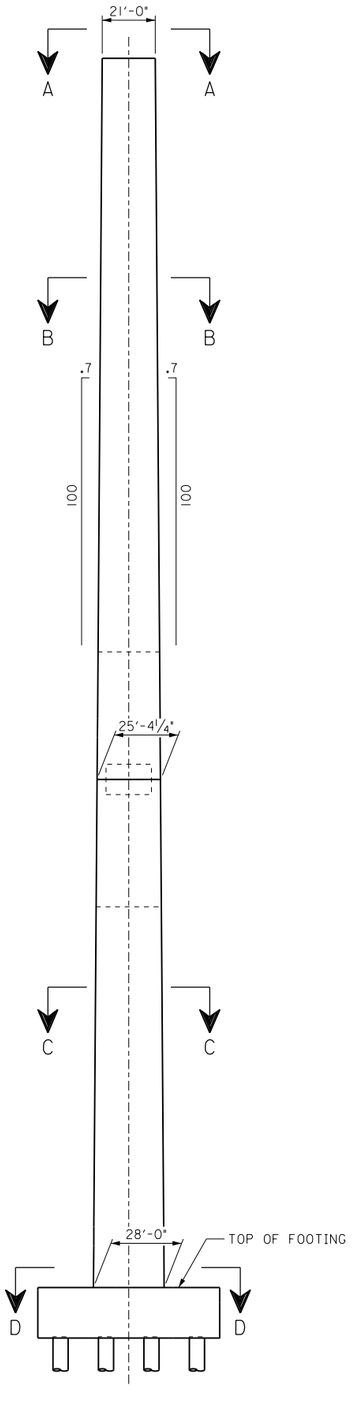
DATE: Monday, July 31, 2017 04:04:57 PM

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		4



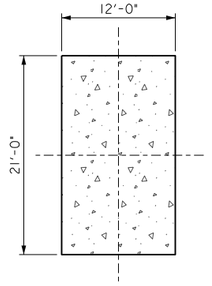
TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



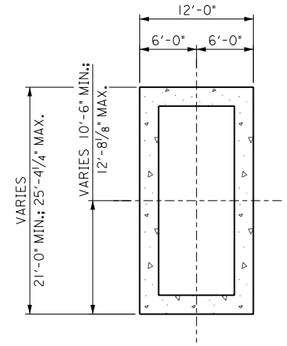
LONGITUDINAL ELEVAION AT TOWERS

SCALE: 1"=30'



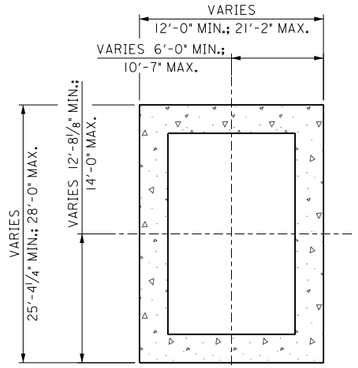
SECTION A-A

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 1/8" = 1'-0"

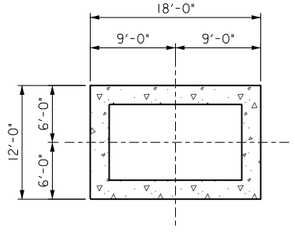


SECTION C-C

SCALE: 1/8" = 1'-0"

SECTION D-D

SCALE: 1/8" = 1'-0"



SECTION E-E

SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 7

REVISIONS

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE
DIAMOND - CONCRETE

TOWER SECTIONS
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

PLOT DRIVER: HDR ALDOT PDF Bw HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:05:02 PM
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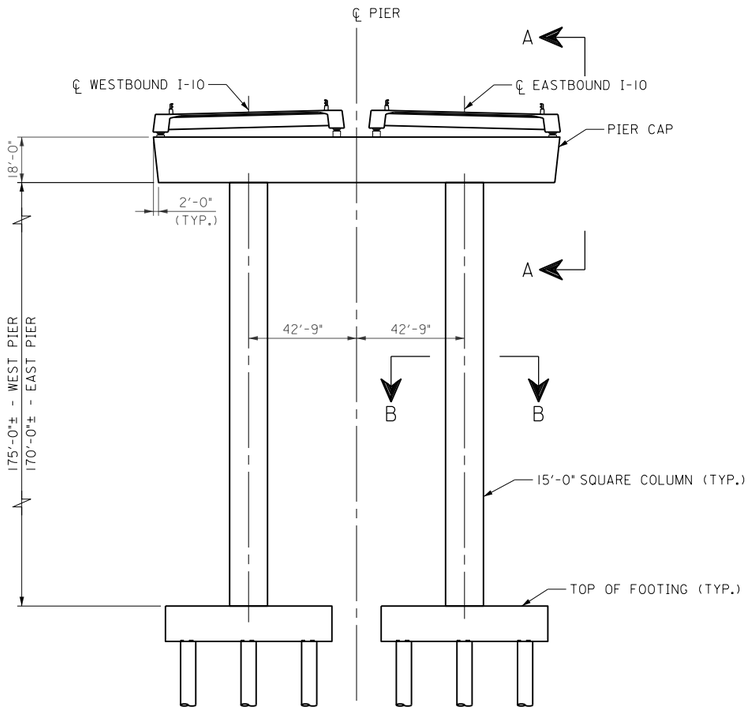
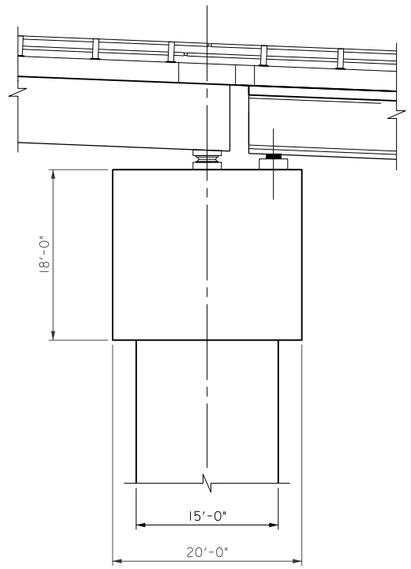
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		5



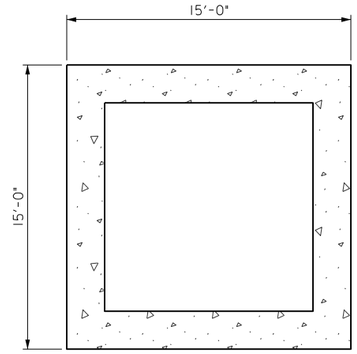
TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'



VIEW A-A

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

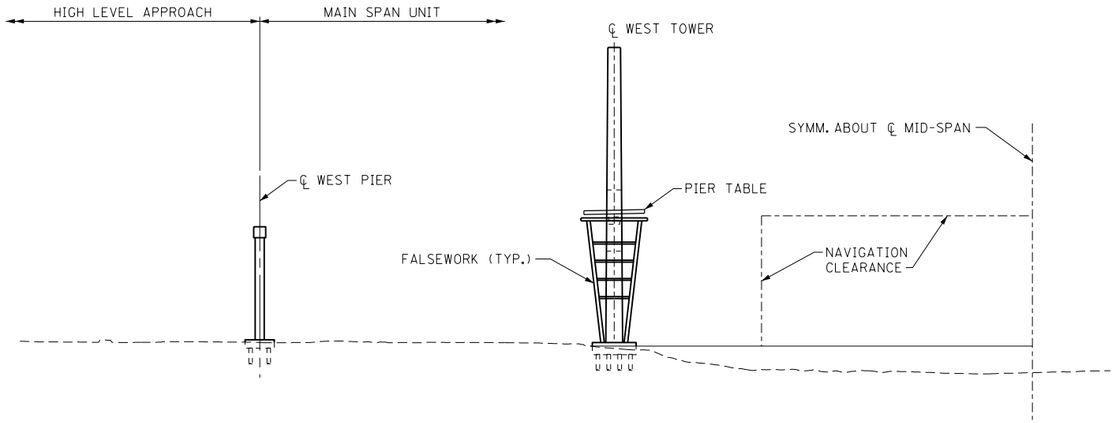
ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE
REVISIONS	BACK SPAN PIER SECTIONS ALTERNATE 3
	ESTIMATED QUANTITIES: DESIGNED BY: ADM — DRAWN BY: SRC
BIN:	COMPUTED BY: — CHECKED BY: — DATE DRAWN: 8/18/2016
	VERIFIED BY: — DATE CHECKED: — SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
DATE: Monday, July 31, 2017 04:05:06 PM
FILE: ø

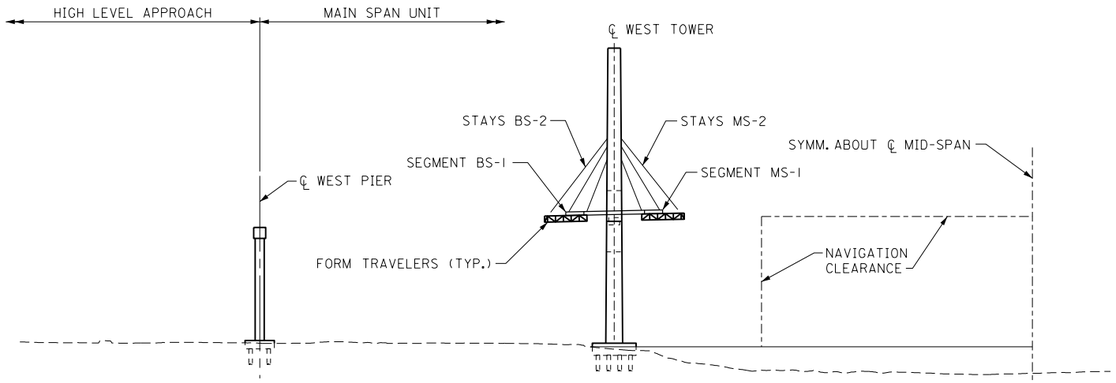
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		6



STEP 1

1. BUILD TOWERS.
2. BUILD BACK SPAN TRANSITION PIERS.
3. BUILD PIER TABLE ON FALSEWORK.

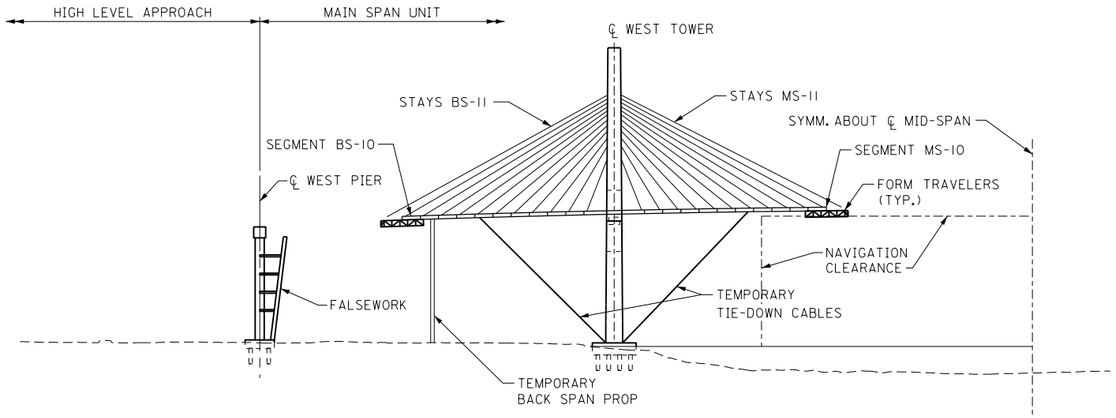


STEP 2

1. INSTALL TEMPORARY FIXITY SUPPORTS ON PIER TABLE.
2. INSTALL STAY CABLES MS-0 AND BS-0.
3. REMOVE PIER TABLE FALSEWORK.

STEP 3

1. INSTALL FORM TRAVELERS ON PIER TABLE, ADVANCED TO SUPPORT SEGMENTS BS-1 AND MS-1.
2. INSTALL STAY CABLES BS-1 AND MS-1 TO SUPPORT FORM TRAVELERS.
3. CAST SEGMENT PAIRS BS-1 AND MS-1.
4. STRESS FLOORBEAM TENDONS IN SEGMENT BS-1 AND MS-1.
5. ADJUST STAY CABLES BS-1 AND MS-1 AND TRANSFER STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. ADVANCE FORM TRAVELERS TO SUPPORT SEGMENTS BS-2 AND MS-2.
7. INSTALL STAY CABLES BS-2 AND MS-2 TO SUPPORT FORM TRAVELERS.



STEP 4

1. REPEAT STEP 3.3 THROUGH STEP 3.7 FOR SEGMENTS BS-2/MS-2 THROUGH BS-17/MS-17.
   a. INSTALL TEMPORARY TIE-DOWN CABLES AFTER CASTING SEGMENTS BS-7/MS-7.
   b. INSTALL TEMPORARY BACK SPAN PROP AFTER CASTING SEGMENTS BS-10/MS-10.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 6 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE	
REVISIONS	ERECTION SEQUENCE (1 OF 2) ALTERNATIVE 3	
	ESTIMATED QUANTITIES	DESIGNED BY: ADM / DRAWN BY: SRC
BIN:	COMPUTED BY:	CHECKED BY: / DATE DRAWN: 8/18/2016
	VERIFIED BY:	DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
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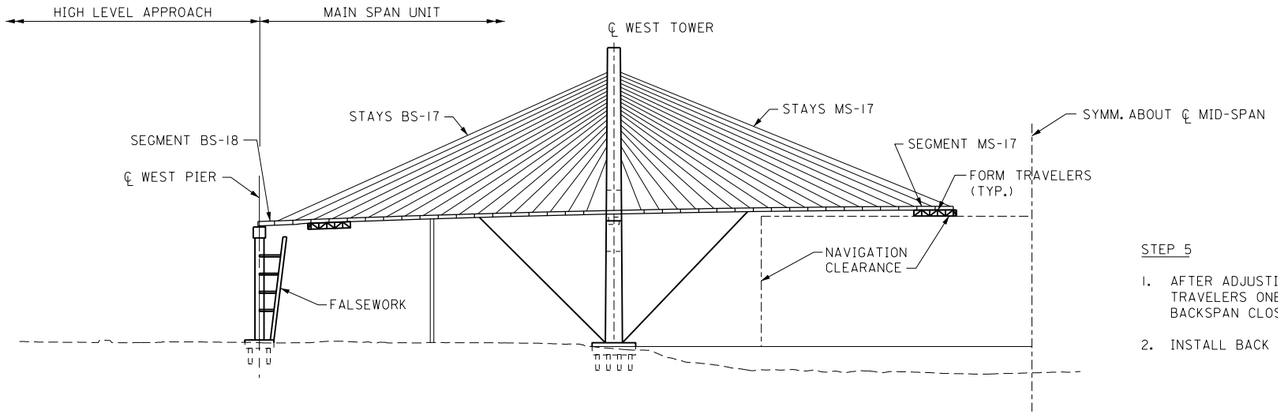
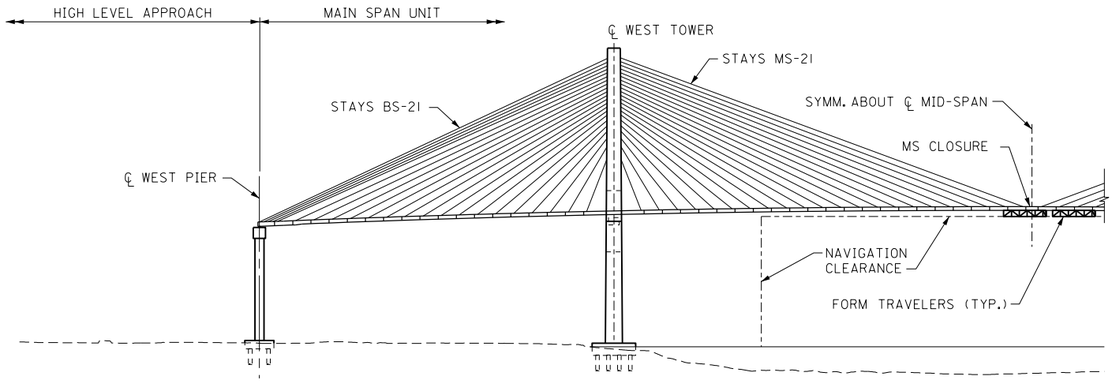
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		7



STEP 5

1. AFTER ADJUSTING STAY CABLES BS-17 AND MS-17, MOVE BACK SPAN TRAVELERS ONE SEGMENT TOWARDS TOWER TO MAKE ROOM FOR BACKSPAN CLOSURE FALSEWORK.
2. INSTALL BACK SPAN CLOSURE FALSEWORK, RELEASE BACK SPAN PROP.



STEP 6

1. ADVANCE MAIN SPAN TRAVELER TO SEGMENT MS-18.
2. INSTALL STAY CABLES BS-18, AND MS-18 TO SUPPORT FORM TRAVELER.
3. CAST SEGMENT PAIR MS-18.
4. STRESS FLOORBEAM TENDONS IN SEGMENT MS-18.
5. ADJUST STAY CABLES BS-18 AND MS-18, AND TRANSFER MS-18 STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. REPEAT STEPS 6.1 THROUGH 6.5 FOR SEGMENTS MS-19 THROUGH MS-21.
7. RELEASE BACK SPAN CLOSURE FALSEWORK.
8. ONCE EAST AND WEST MAIN SPAN CANTILEVERS ARE COMPLETED THROUGH SEGMENT MS-21, MOVE BACK (OR REMOVE) EAST TRAVELER AND ADVANCE WEST TRAVELER TO MAIN SPAN CLOSURE SEGMENT.
9. RELEASE TEMPORARY TIE-DOWN CABLES, AND ADJUST STAY CABLES TO ALIGN TIPS FOR CLOSURE.
10. CAST MAIN SPAN CLOSURE SEGMENT AND REMOVE FORM TRAVELERS.
11. COMPLETE BARRIERS AND OVERLAY.

ERECTION NOTES:

1. STAY CABLE INSTALLATION SHALL BE BALANCED WITHIN 5% FORCE FOR EACH EDGE OF DECK WITHIN A DECK.
2. SEGMENTS ERECTED IN BALANCED CANTILEVER SHALL BE CAST SIMULTANEOUSLY ON MAIN SPAN / BACK SPAN.
3. EASTBOUND/WESTBOUND DECK ERECTION SHALL BE BALANCED ON EACH ERECTION FRONT WITHIN TWO SEGMENT LENGTHS, EXCEPT THAT EACH DECK SHALL BE AT THE SAME STEP WHEN:
   a. TEMPORARY TIE-DOWN CABLES OR PROPS ARE INSTALLED.
   b. CLOSURE SEGMENTS ARE CAST.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 7 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE
REVISIONS	ERECTION SEQUENCE (2 OF 2) ALTERNATIVE 3
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: ADM / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/18/2016 VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
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DATE: Monday, July 31, 2017 04:05:42 PM
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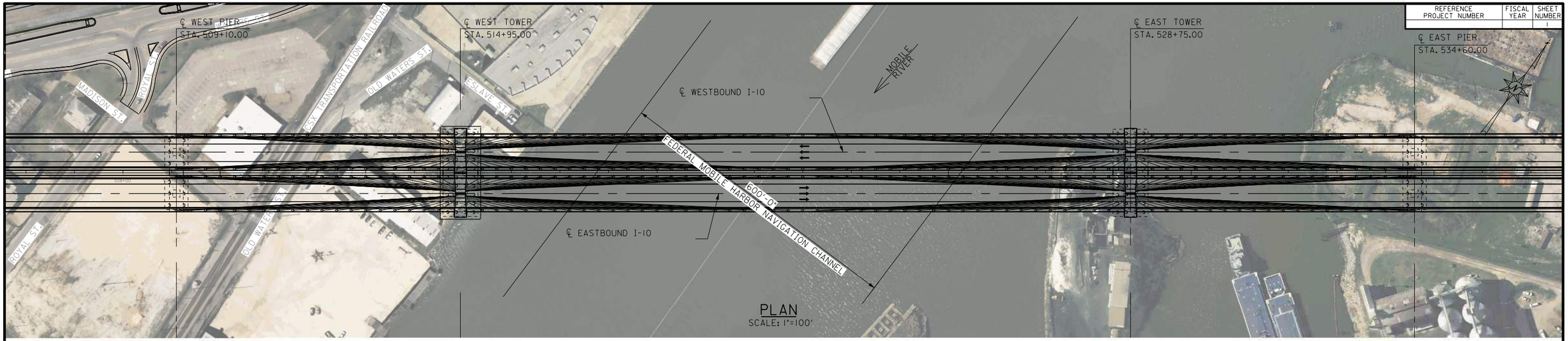
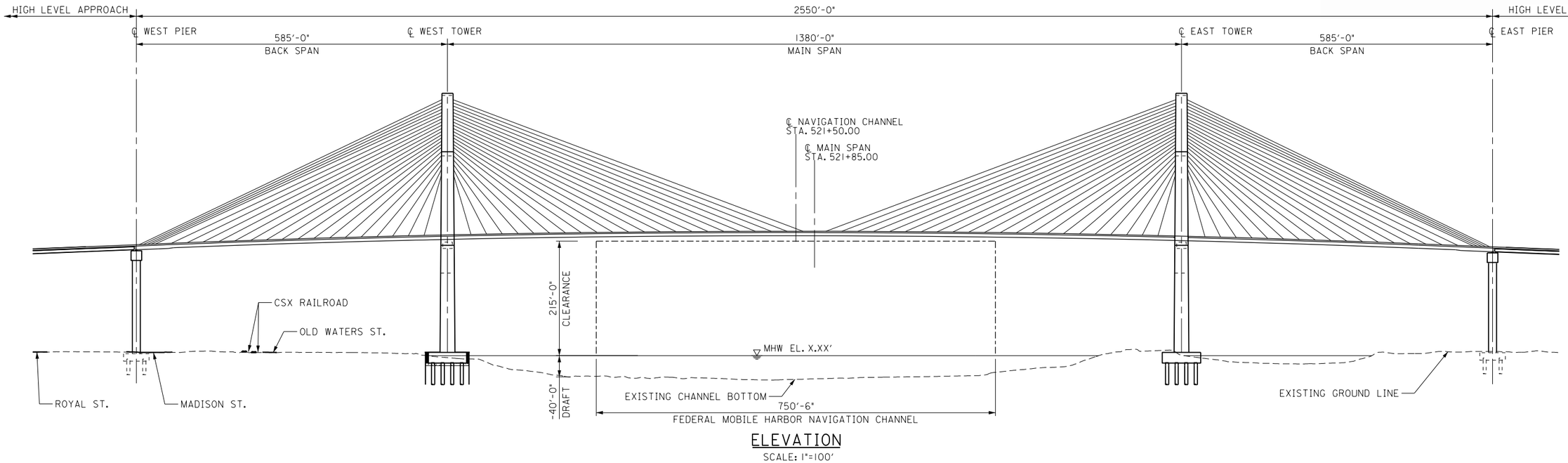
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		1



PLAN
SCALE: 1"=100'



ELEVATION
SCALE: 1"=100'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - INVERTED
"Y" - CONCRETE

PLAN & ELEVATION
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.plt.cfg
USER: JSPANIER
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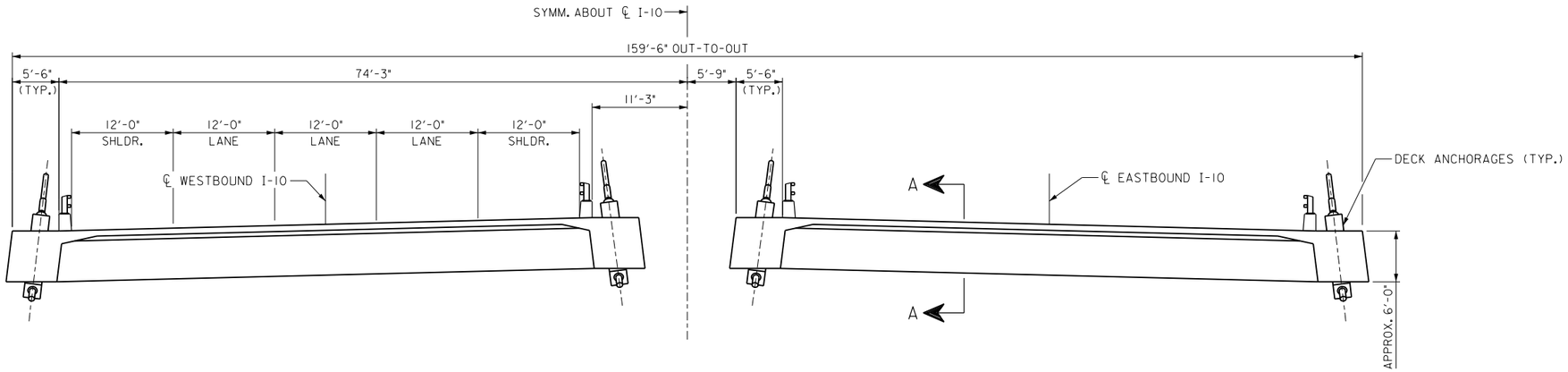
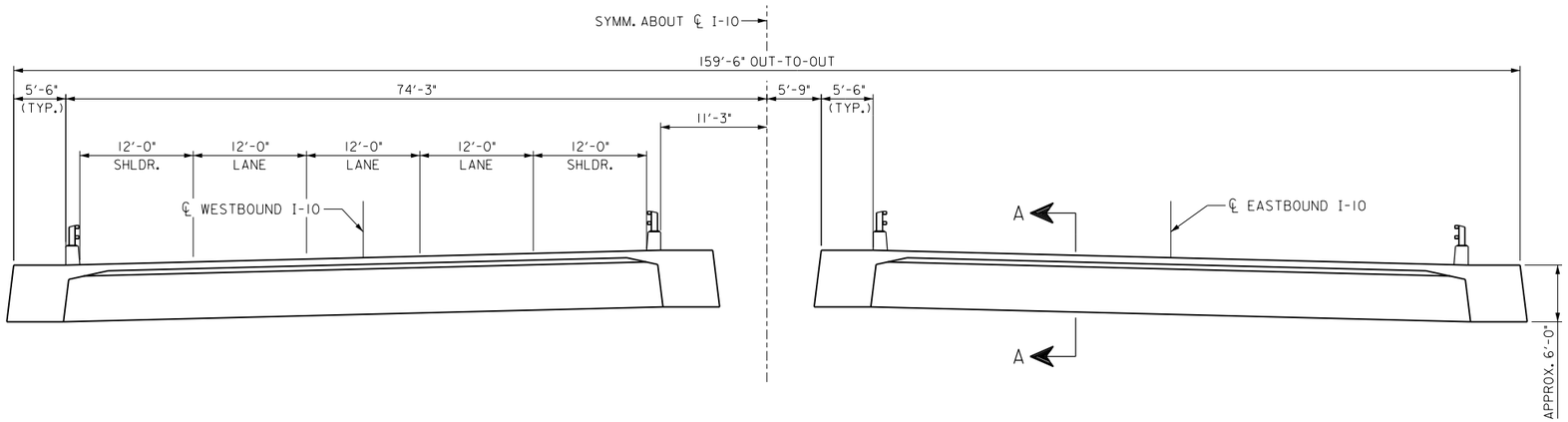
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		2



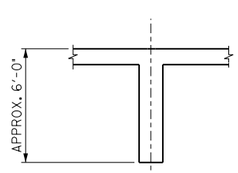
TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BETWEEN DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



SECTION A-A

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - INVERTED
"Y" - CONCRETE

DECK TYPICAL SECTIONS
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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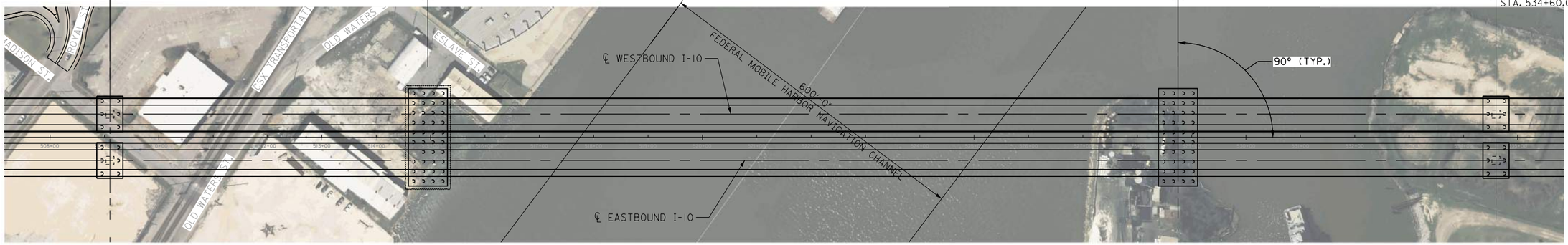
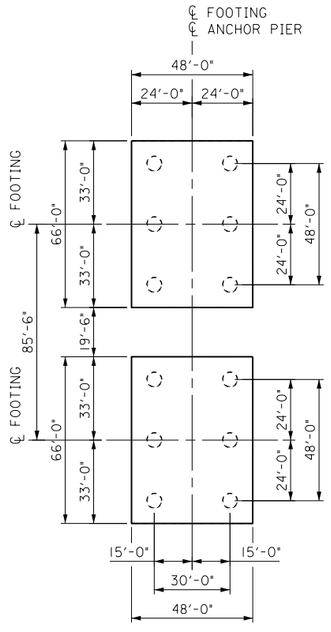
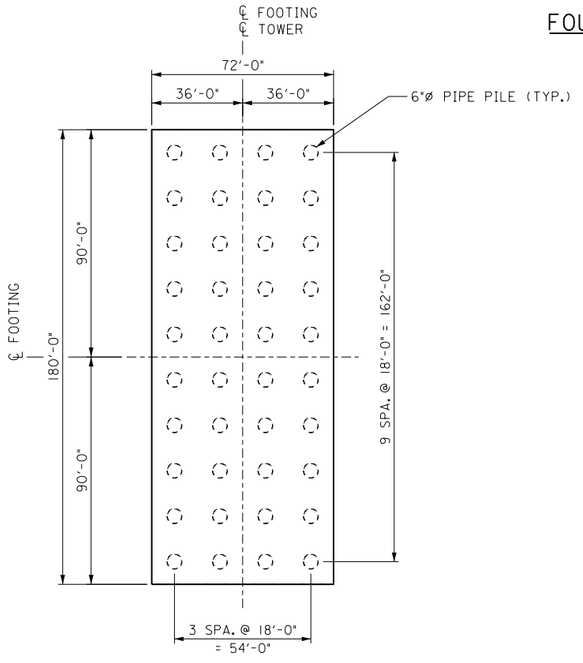
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		3



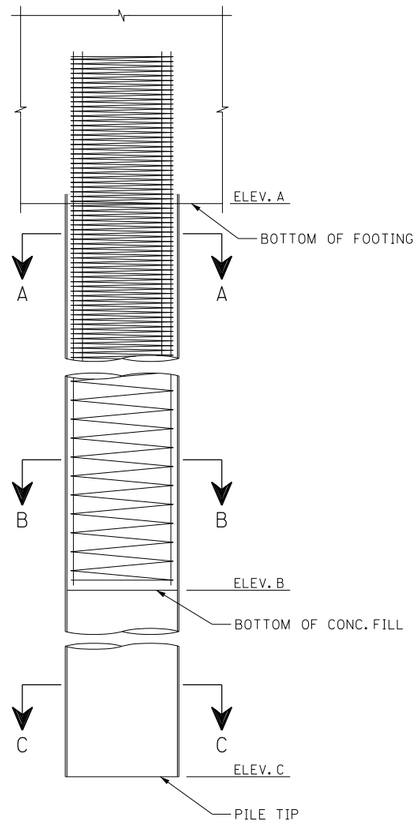
FOUNDATION PLAN
SCALE: 1"=100'



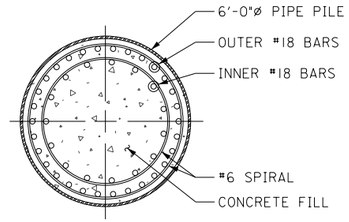
ANCHOR PIER FOOTING PLAN
SCALE: 1"=30'



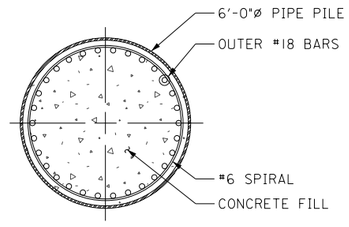
TOWER FOOTING PLAN
SCALE: 1"=30'



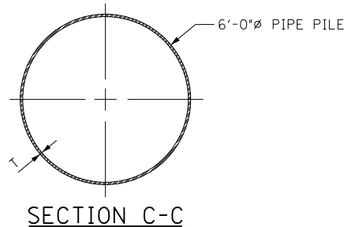
ELEVATION
SCALE: 1/4" = 1'-0"



SECTION A-A
SCALE: 3/8" = 1'-0"



SECTION B-B
SCALE: 3/8" = 1'-0"



SECTION C-C
SCALE: 3/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 7

REVISIONS

BIN:

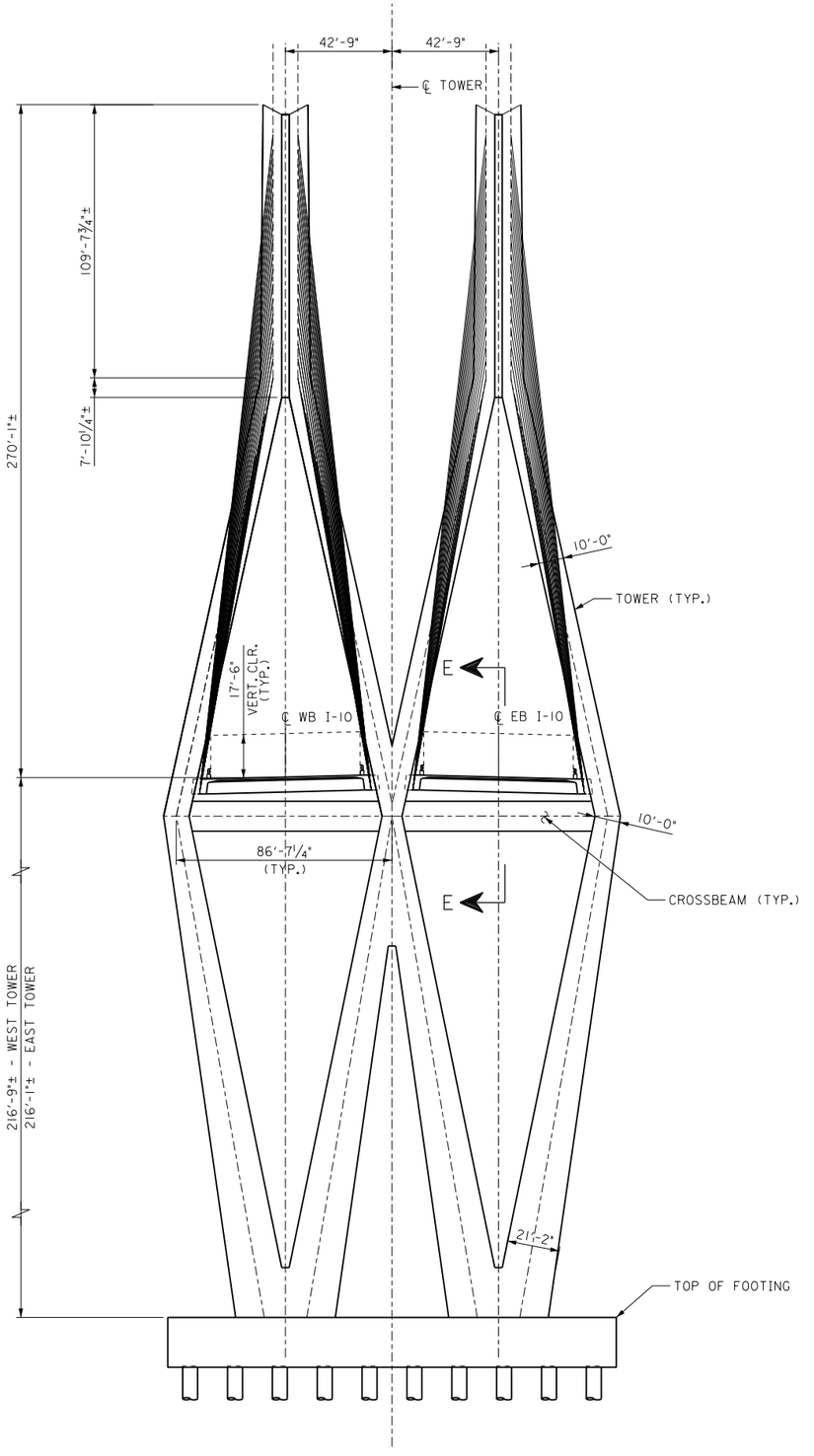
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - INVERTED
"Y" - CONCRETE

FOUNDATION PLAN & PILE DETAILS
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

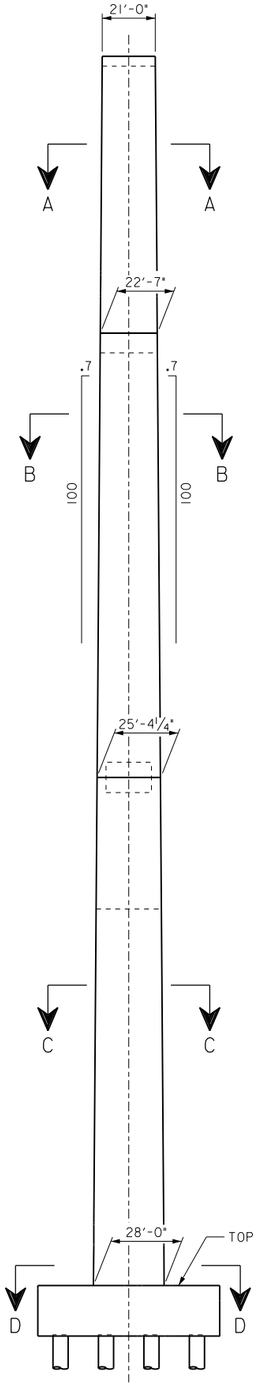
PLOT DRIVER: HOR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
PENTABLE: HOR-ALDOT.tbl
SCALE: 1:1.99992
DATE: Monday, July 31, 2017 04:05:56 PM

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		4



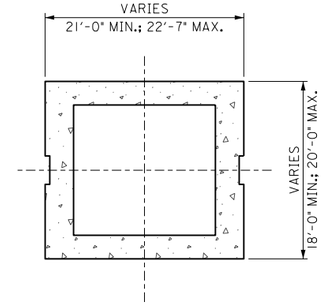
TRANSVERSE SECTION AT TOWERS

SCALE: 1"=30'



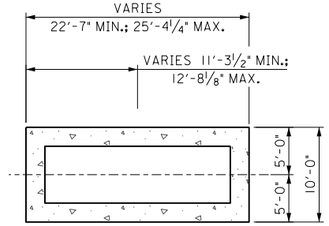
LONGITUDINAL ELEVAION AT TOWERS

SCALE: 1"=30'



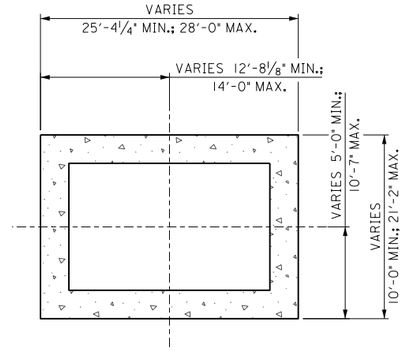
SECTION A-A

SCALE: ⅛" = 1'-0"



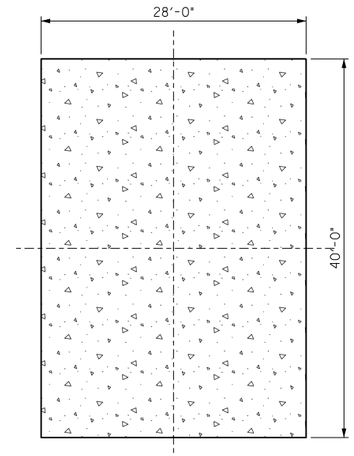
SECTION B-B

SCALE: ⅛" = 1'-0"



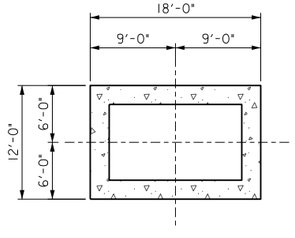
SECTION C-C

SCALE: ⅛" = 1'-0"



SECTION D-D

SCALE: ⅛" = 1'-0"



SECTION E-E

SCALE: ⅛" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 7

REVISIONS

BIN:

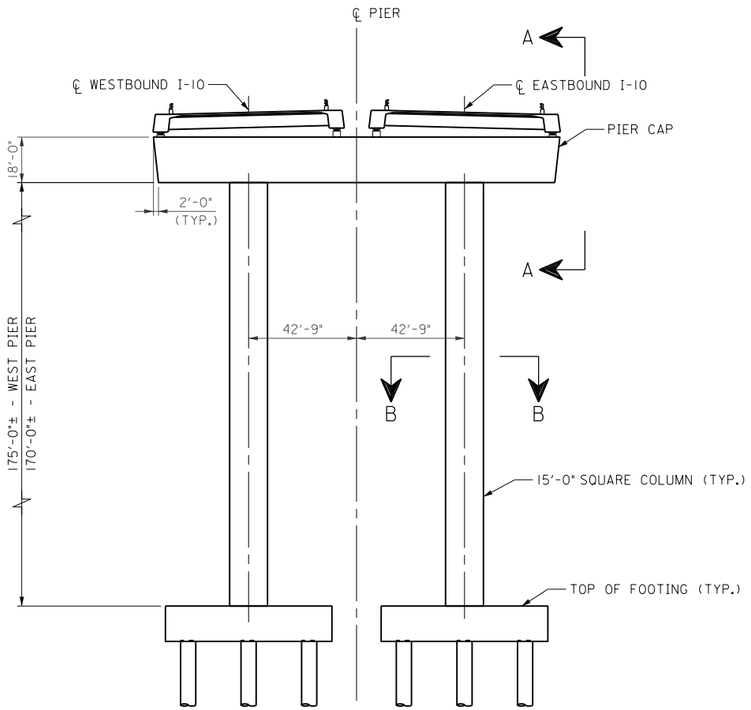
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - INVERTED
"Y" - CONCRETE

TOWER SECTIONS
ALTERNATE 3

ESTIMATED QUANTITIES	DESIGNED BY: ADM	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

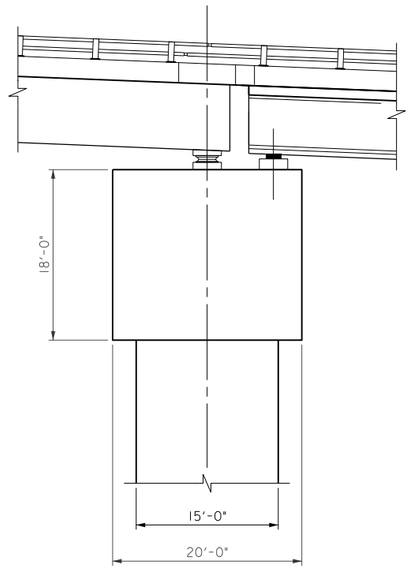
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:05:57 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		5



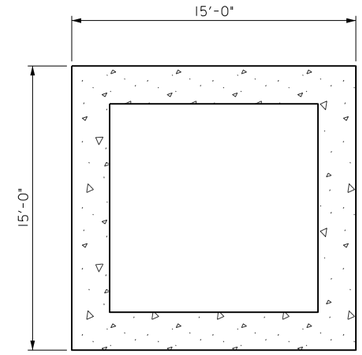
TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'



VIEW A-A

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

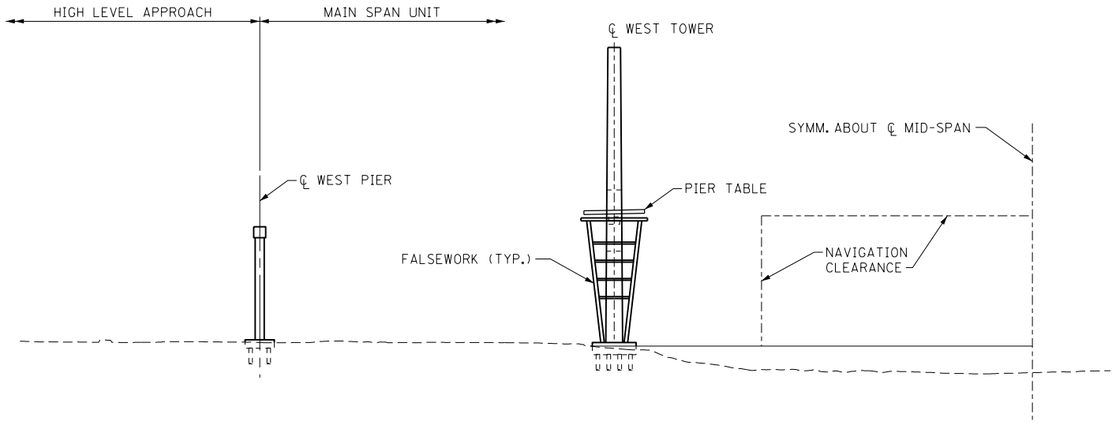
ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - INVERTED "Y" - CONCRETE
REVISIONS	BACK SPAN PIER SECTIONS ALTERNATE 3
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: ADM / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/18/2016 VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
DATE: Monday, July 31, 2017 04:05:57 PM
FILE: ø

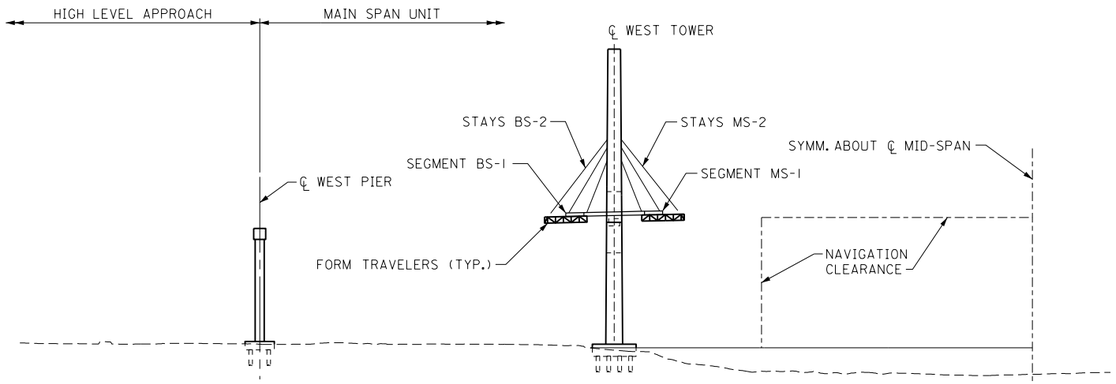
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		6



STEP 1

1. BUILD TOWERS.
2. BUILD BACK SPAN TRANSITION PIERS.
3. BUILD PIER TABLE ON FALSEWORK.

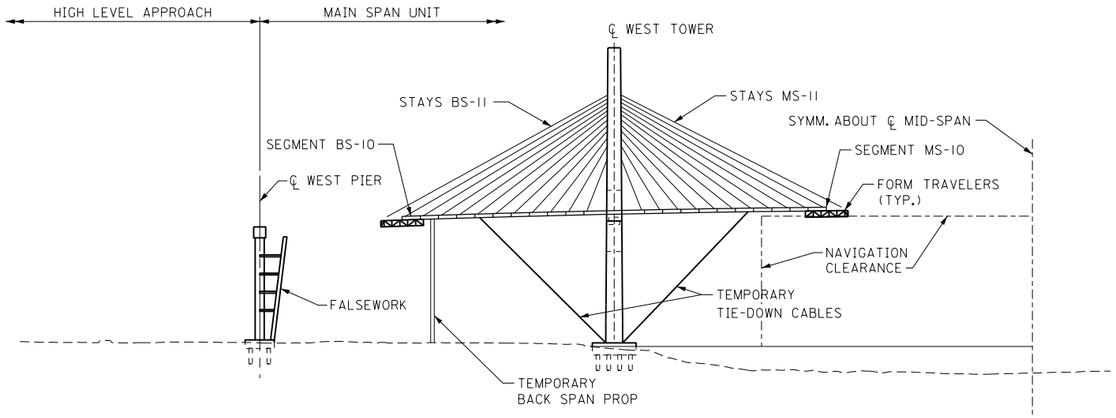


STEP 2

1. INSTALL TEMPORARY FIXITY SUPPORTS ON PIER TABLE.
2. INSTALL STAY CABLES MS-0 AND BS-0.
3. REMOVE PIER TABLE FALSEWORK.

STEP 3

1. INSTALL FORM TRAVELERS ON PIER TABLE, ADVANCED TO SUPPORT SEGMENTS BS-1 AND MS-1.
2. INSTALL STAY CABLES BS-1 AND MS-1 TO SUPPORT FORM TRAVELERS.
3. CAST SEGMENT PAIRS BS-1 AND MS-1.
4. STRESS FLOORBEAM TENDONS IN SEGMENT BS-1 AND MS-1.
5. ADJUST STAY CABLES BS-1 AND MS-1 AND TRANSFER STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. ADVANCE FORM TRAVELERS TO SUPPORT SEGMENTS BS-2 AND MS-2.
7. INSTALL STAY CABLES BS-2 AND MS-2 TO SUPPORT FORM TRAVELERS.



STEP 4

1. REPEAT STEP 3.3 THROUGH STEP 3.7 FOR SEGMENTS BS-2/MS-2 THROUGH BS-17/MS-17.
   a. INSTALL TEMPORARY TIE-DOWN CABLES AFTER CASTING SEGMENTS BS-7/MS-7.
   b. INSTALL TEMPORARY BACK SPAN PROP AFTER CASTING SEGMENTS BS-10/MS-10.

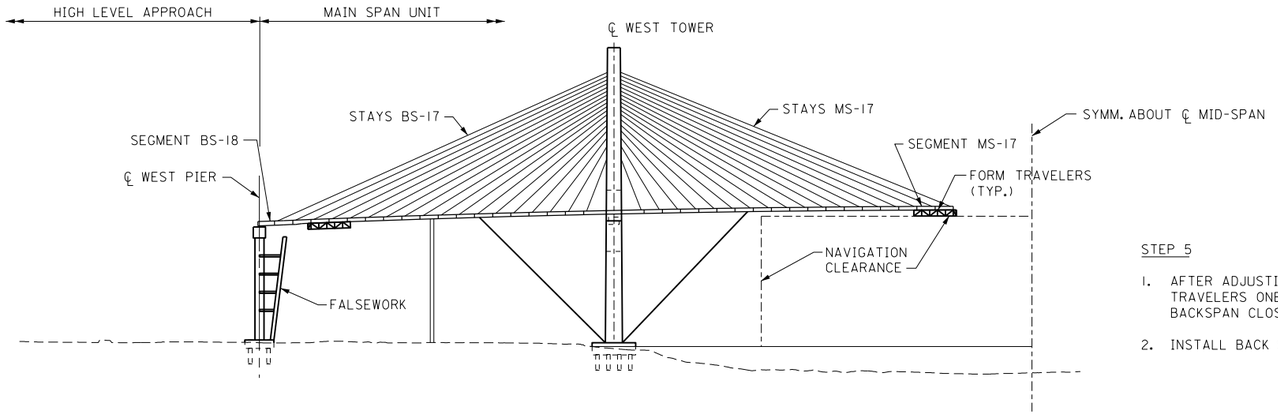
NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 6 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - INVERTED "Y" - CONCRETE
REVISIONS	ERECTION SEQUENCE (1 OF 2) ALTERNATIVE 3
	ESTIMATED QUANTITIES / DESIGNED BY: ADM / DRAWN BY: SRC
BIN:	COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/18/2016
	VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

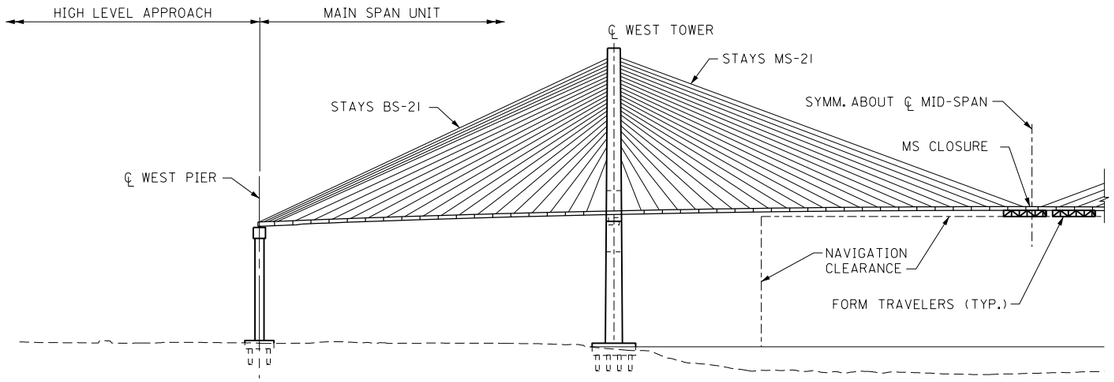
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
PENTABLE: HDR-ALDOT.tbl
SCALE: 1:1.99992
DATE: Monday, July 31, 2017 04:06:06 PM

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		7



STEP 5

1. AFTER ADJUSTING STAY CABLES BS-17 AND MS-17, MOVE BACK SPAN TRAVELERS ONE SEGMENT TOWARDS TOWER TO MAKE ROOM FOR BACKSPAN CLOSURE FALSEWORK.
2. INSTALL BACK SPAN CLOSURE FALSEWORK, RELEASE BACK SPAN PROP.



STEP 6

1. ADVANCE MAIN SPAN TRAVELER TO SEGMENT MS-18.
2. INSTALL STAY CABLES BS-18, AND MS-18 TO SUPPORT FORM TRAVELER.
3. CAST SEGMENT PAIR MS-18.
4. STRESS FLOORBEAM TENDONS IN SEGMENT MS-18.
5. ADJUST STAY CABLES BS-18 AND MS-18, AND TRANSFER MS-18 STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. REPEAT STEPS 6.1 THROUGH 6.5 FOR SEGMENTS MS-19 THROUGH MS-21.
7. RELEASE BACK SPAN CLOSURE FALSEWORK.
8. ONCE EAST AND WEST MAIN SPAN CANTILEVERS ARE COMPLETED THROUGH SEGMENT MS-21, MOVE BACK (OR REMOVE) EAST TRAVELER AND ADVANCE WEST TRAVELER TO MAIN SPAN CLOSURE SEGMENT.
9. RELEASE TEMPORARY TIE-DOWN CABLES, AND ADJUST STAY CABLES TO ALIGN TIPS FOR CLOSURE.
10. CAST MAIN SPAN CLOSURE SEGMENT AND REMOVE FORM TRAVELERS.
11. COMPLETE BARRIERS AND OVERLAY.

ERECTION NOTES:

1. STAY CABLE INSTALLATION SHALL BE BALANCED WITHIN 5% FORCE FOR EACH EDGE OF DECK WITHIN A DECK.
2. SEGMENTS ERECTED IN BALANCED CANTILEVER SHALL BE CAST SIMULTANEOUSLY ON MAIN SPAN / BACK SPAN.
3. EASTBOUND/WESTBOUND DECK ERECTION SHALL BE BALANCED ON EACH ERECTION FRONT WITHIN TWO SEGMENT LENGTHS, EXCEPT THAT EACH DECK SHALL BE AT THE SAME STEP WHEN:
   a. TEMPORARY TIE-DOWN CABLES OR PROPS ARE INSTALLED.
   b. CLOSURE SEGMENTS ARE CAST.

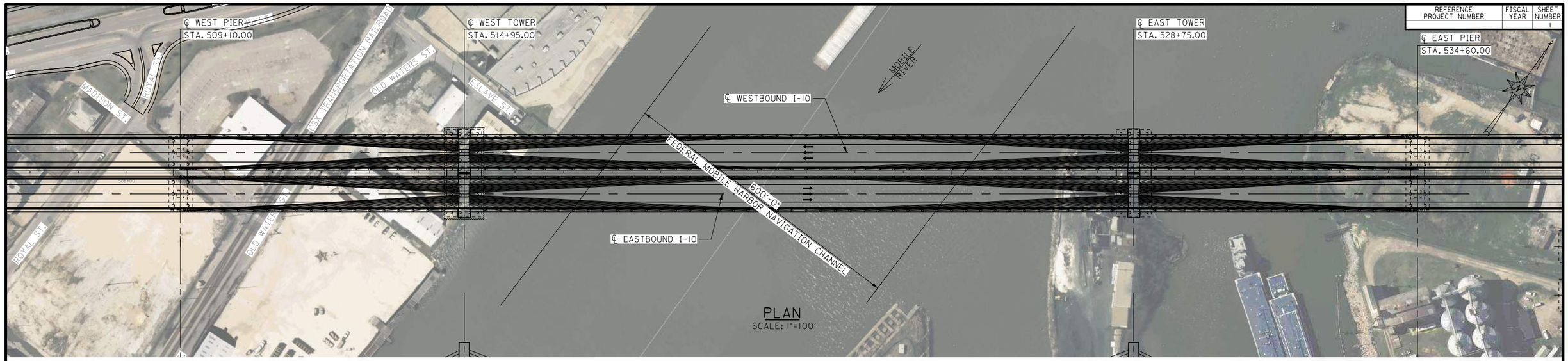
NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

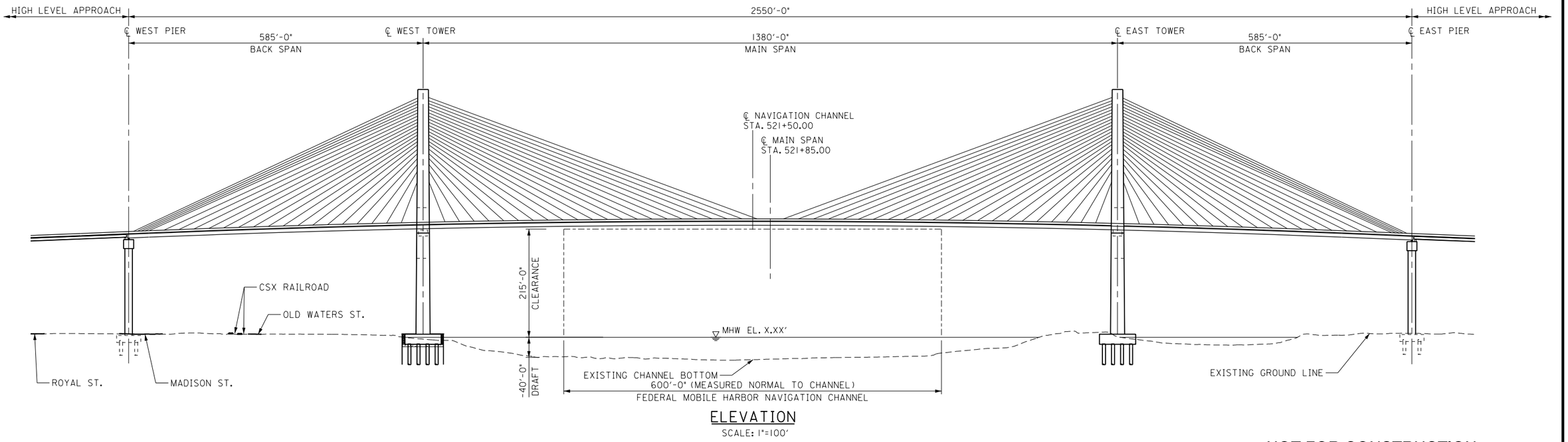
BRIDGE SHEET NO. 7 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - INVERTED "Y" - CONCRETE	
REVISIONS	ERECTION SEQUENCE (2 OF 2) ALTERNATIVE 3	
	ESTIMATED QUANTITIES	DESIGNED BY: ADM / DRAWN BY: SRC
BIN:	COMPUTED BY: / CHECKED BY:	DATE DRAWN: 8/18/2016
	VERIFIED BY: / DATE CHECKED:	SCALE: AS NOTED

PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER DATE: Monday, July 31, 2017 04:06:07 PM
FILE: ø
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:1.99992

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		1



PLAN
SCALE: 1"=100'



ELEVATION
SCALE: 1"=100'

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 7

REVISIONS

BIN:

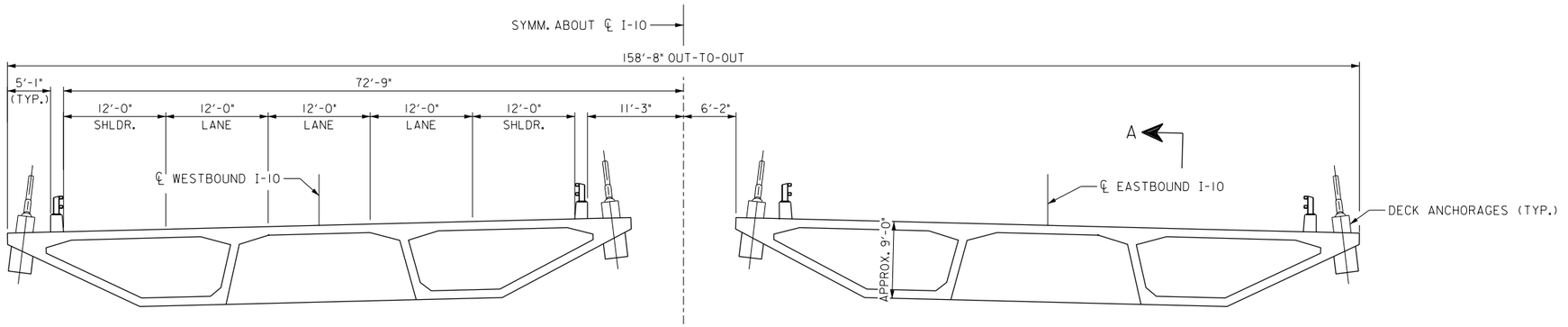
MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE DIAMOND - CONCRETE

PLAN & ELEVATION
ALTERNATE 7

ESTIMATED QUANTITIES	DESIGNED BY: NRH	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

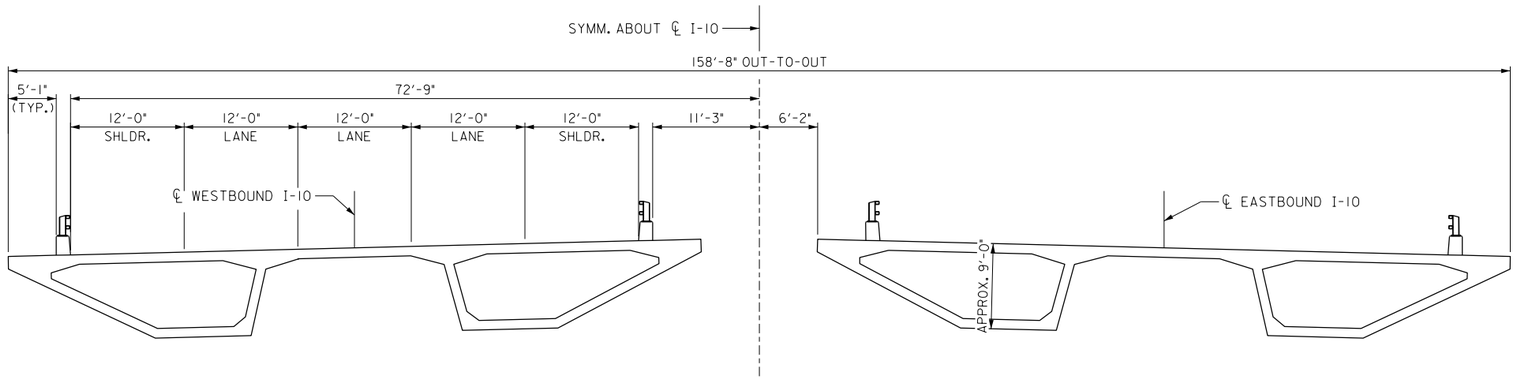
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER
FILE: ø
DATE: Monday, July 31, 2017 04:06:19 PM
PENTABLE: HDR_ALDOT.tbl
SCALE: 1:2

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		2



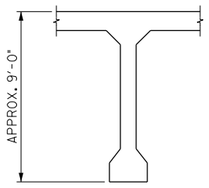
TYPICAL SECTION AT DECK ANCHORAGES

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BETWEEN ANCHORAGES

SCALE: 1/8" = 1'-0"



SECTION A-A

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 2 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE DIAMOND - CONCRETE

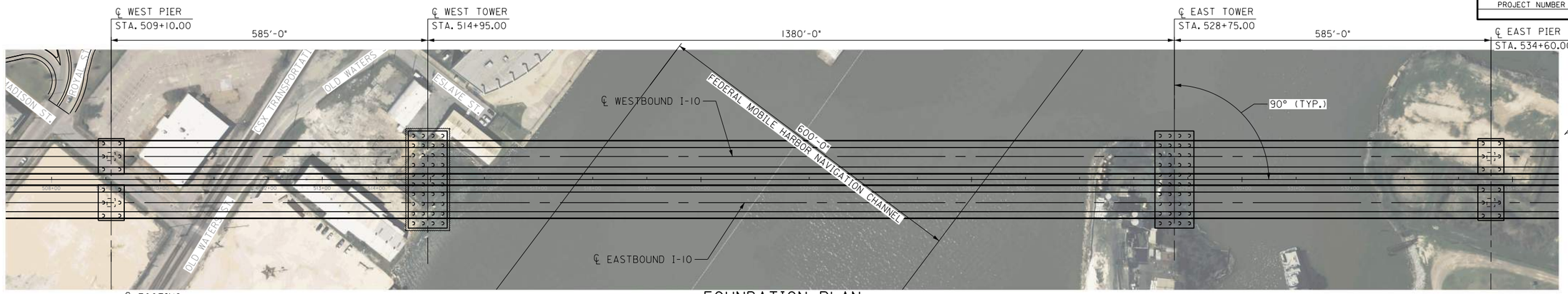
DECK TYPICAL SECTIONS
ALTERNATE 7

ESTIMATED QUANTITIES	DESIGNED BY: NRH	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

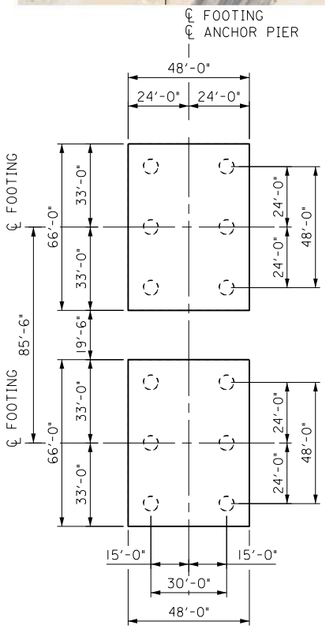
PLOT DRIVER: HDR ALDOT PDF BW HALF SIZE.pltcfg
USER: JSPANIER DATE: Monday, July 31, 2017 04:18:41 PM
FILE: ø

PENTABLE: HDR_ALDOT.tbl
SCALE: 1:2

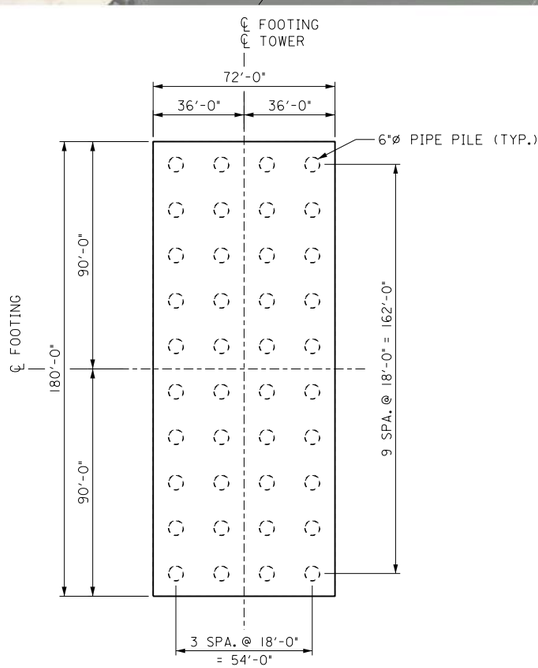
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		3



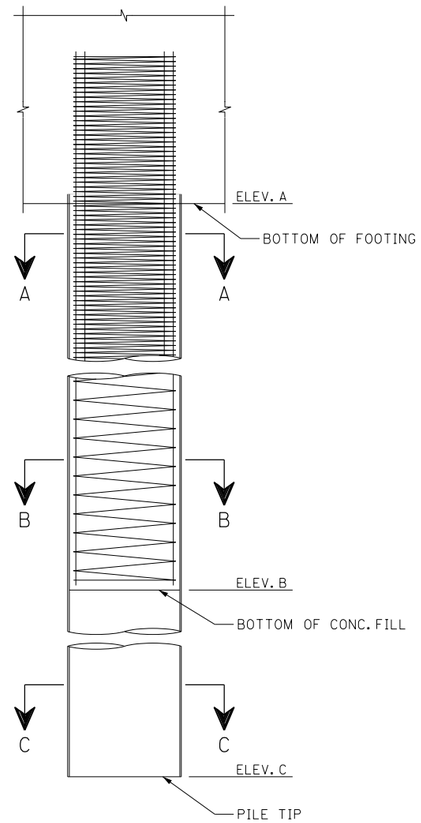
FOUNDATION PLAN
SCALE: 1"=100'



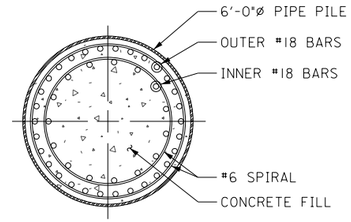
ANCHOR PIER FOOTING PLAN
SCALE: 1"=30'



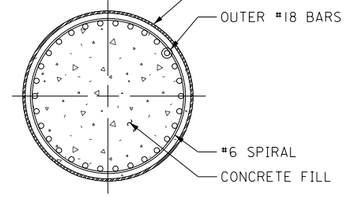
TOWER FOOTING PLAN
SCALE: 1"=30'



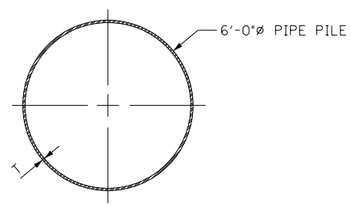
ELEVATION
SCALE: 1/4" = 1'-0"



SECTION A-A
SCALE: 3/8" = 1'-0"



SECTION B-B
SCALE: 3/8" = 1'-0"



SECTION C-C
SCALE: 3/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 3 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE
DIAMOND - CONCRETE

FOUNDATION PLAN & PILE DETAILS
ALTERNATE 7

ESTIMATED QUANTITIES	DESIGNED BY: MWL	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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DATE: Monday, July 31, 2017 04:06:24 PM
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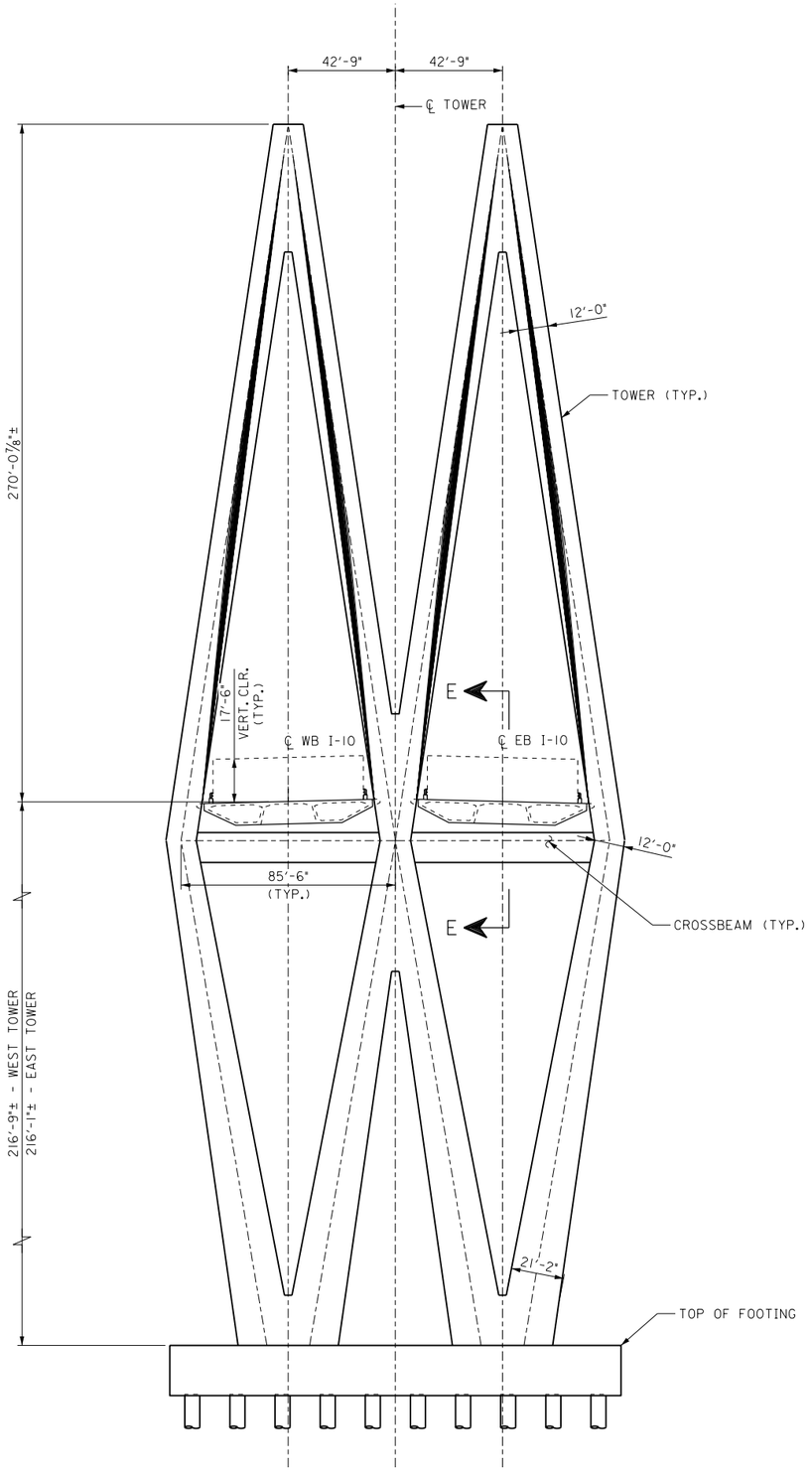
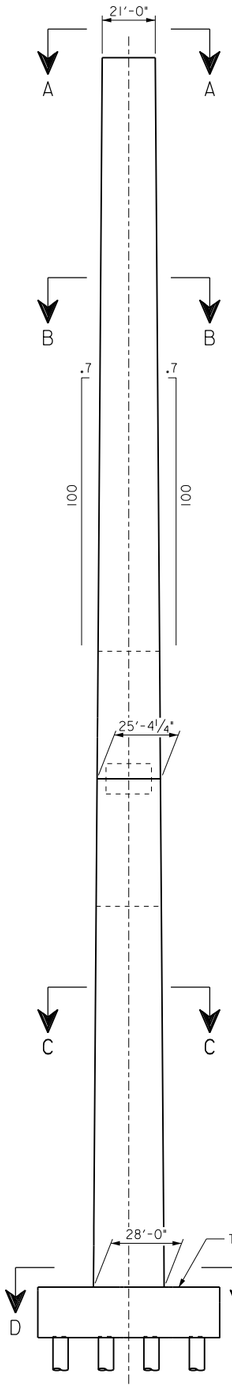
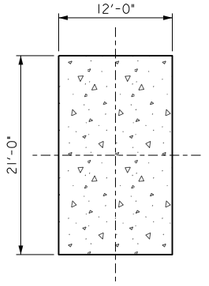
		4

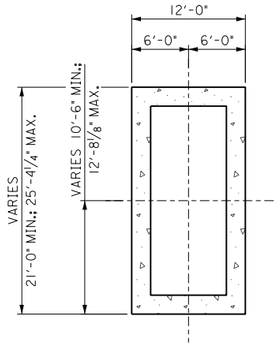




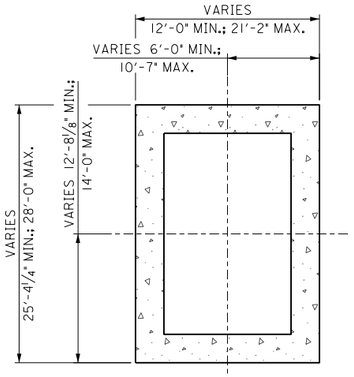
LONGITUDINAL ELEVAION AT TOWERS
SCALE: 1"=30'



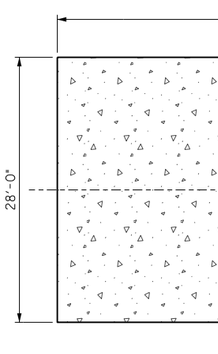
SECTION A-A
SCALE: 1/8" = 1'-0"



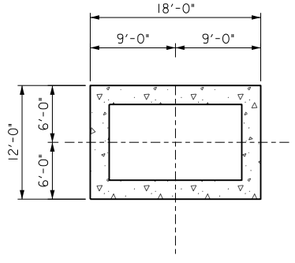
SECTION B-B
SCALE: 1/8" = 1'-0"



SECTION C-C
SCALE: 1/8" = 1'-0"



SECTION D-D
SCALE: 1/8" = 1'-0"



SECTION E-E
SCALE: 1/8" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 4 OF 7

REVISIONS

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE
DIAMOND - CONCRETE

TOWER SECTIONS
ALTERNATE 7

ESTIMATED QUANTITIES	DESIGNED BY: MWL	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

BIN:

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REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
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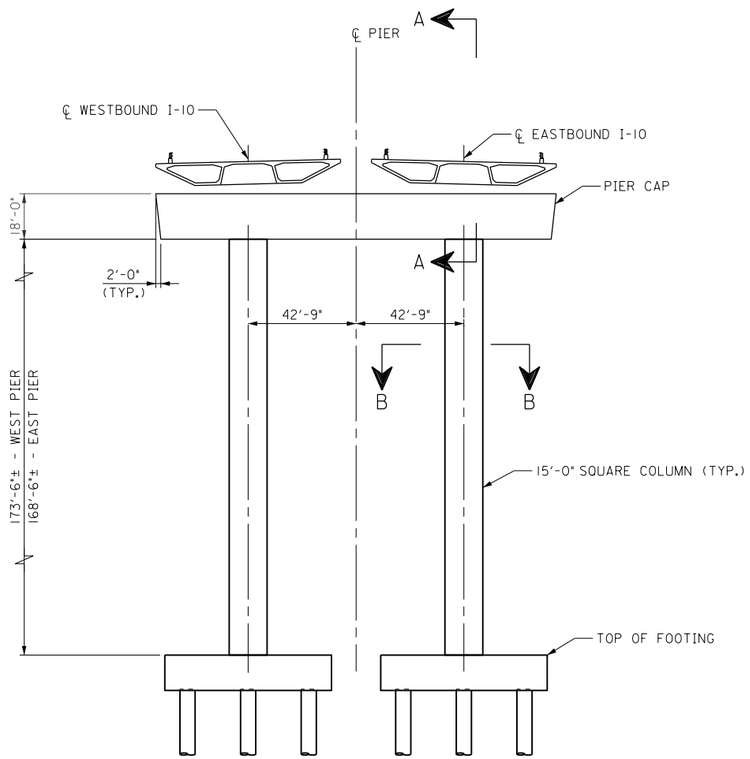
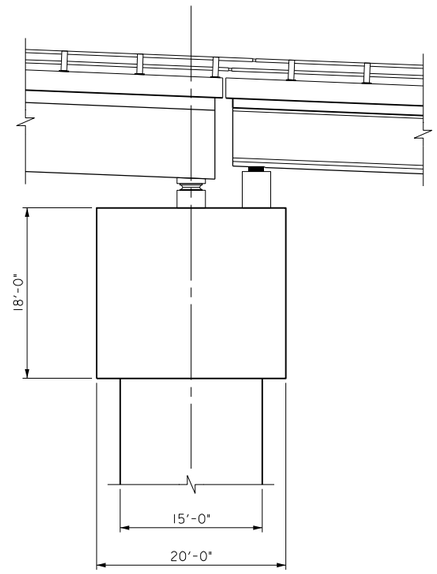


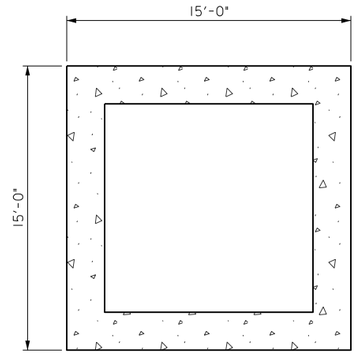
TRANSVERSE SECTION AT PIERS

SCALE: 1"=30'



VIEW A-A

SCALE: 1/8" = 1'-0"



SECTION B-B

SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 5 OF 7

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE
PYLON - DIAMOND

BACK SPAN PIER SECTIONS
ALTERNATE 7

ESTIMATED QUANTITIES	DESIGNED BY: NRH	DRAWN BY: SRC
COMPUTED BY:	CHECKED BY:	DATE DRAWN: 8/18/2016
VERIFIED BY:	DATE CHECKED:	SCALE: AS NOTED

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REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
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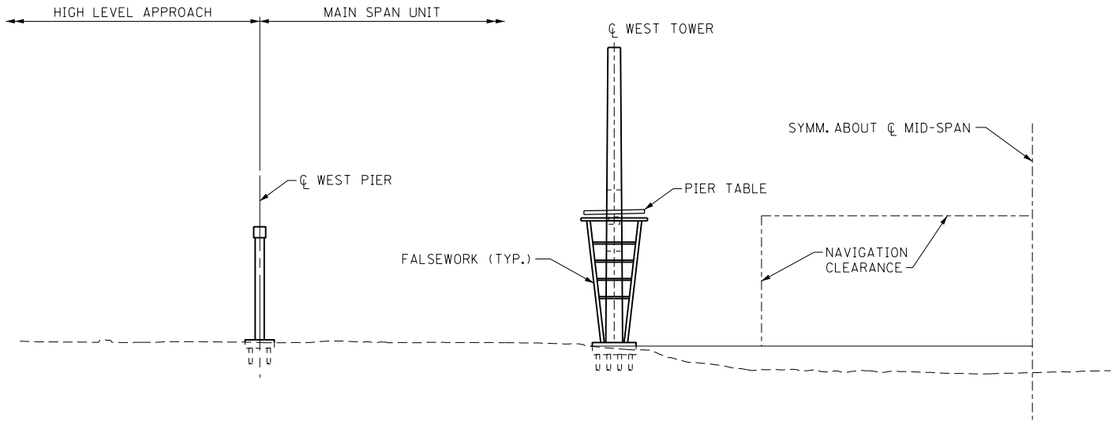
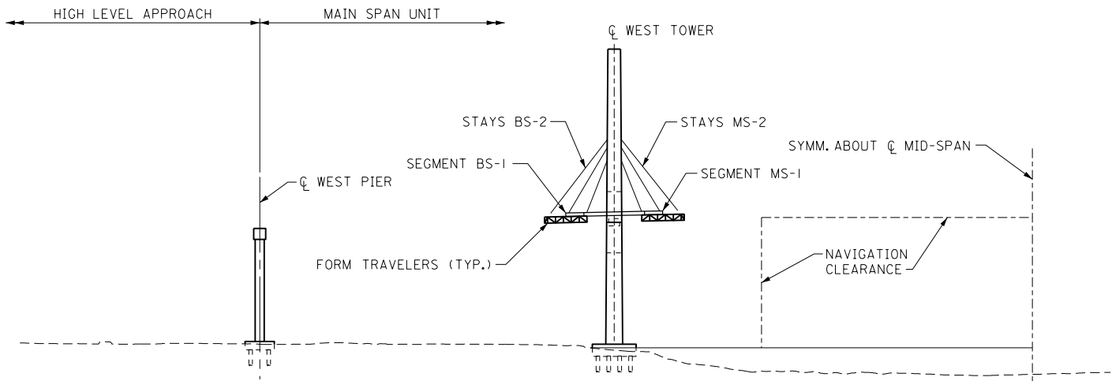


STEP 1

1. BUILD TOWERS.
2. BUILD BACK SPAN TRANSITION PIERS.
3. BUILD PIER TABLE ON FALSEWORK.

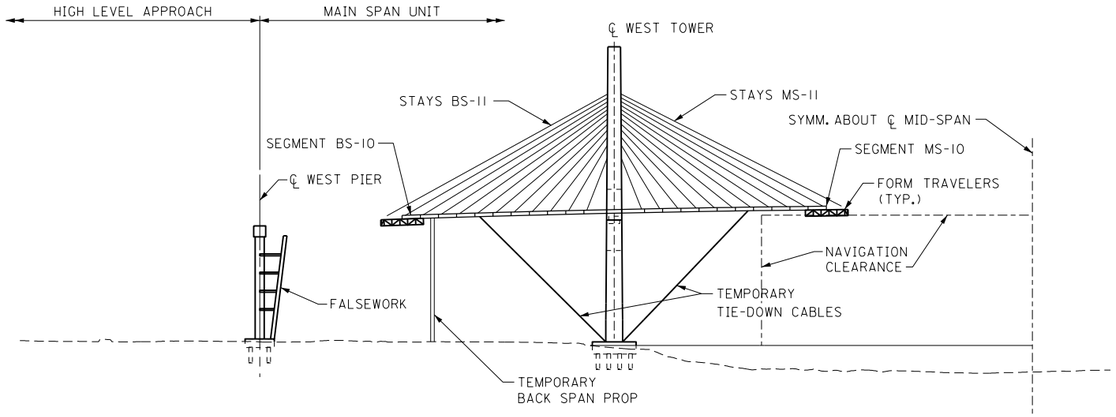


STEP 2

1. INSTALL TEMPORARY FIXITY SUPPORTS ON PIER TABLE.
2. INSTALL STAY CABLES MS-0 AND BS-0.
3. REMOVE PIER TABLE FALSEWORK.

STEP 3

1. INSTALL FORM TRAVELERS ON PIER TABLE, ADVANCED TO SUPPORT SEGMENTS BS-1 AND MS-1.
2. INSTALL STAY CABLES BS-1 AND MS-1 TO SUPPORT FORM TRAVELERS.
3. CAST SEGMENT PAIRS BS-1 AND MS-1.
4. STRESS FLOORBEAM TENDONS IN SEGMENT BS-1 AND MS-1.
5. ADJUST STAY CABLES BS-1 AND MS-1 AND TRANSFER STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. ADVANCE FORM TRAVELERS TO SUPPORT SEGMENTS BS-2 AND MS-2.
7. INSTALL STAY CABLES BS-2 AND MS-2 TO SUPPORT FORM TRAVELERS.



STEP 4

1. REPEAT STEP 3.3 THROUGH STEP 3.7 FOR SEGMENTS BS-2/MS-2 THROUGH BS-17/MS-17.
   a. INSTALL TEMPORARY TIE-DOWN CABLES AFTER CASTING SEGMENTS BS-7/MS-7.
   b. INSTALL TEMPORARY BACK SPAN PROP AFTER CASTING SEGMENTS BS-10/MS-10.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 6 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE	
REVISIONS	ERECTION SEQUENCE (1 OF 2) ALTERNATIVE 7	
	ESTIMATED QUANTITIES	DESIGNED BY: MWL / DRAWN BY: SRC
BIN:	COMPUTED BY:	CHECKED BY: / DATE DRAWN: 8/18/2016
	VERIFIED BY:	DATE CHECKED: / SCALE: AS NOTED

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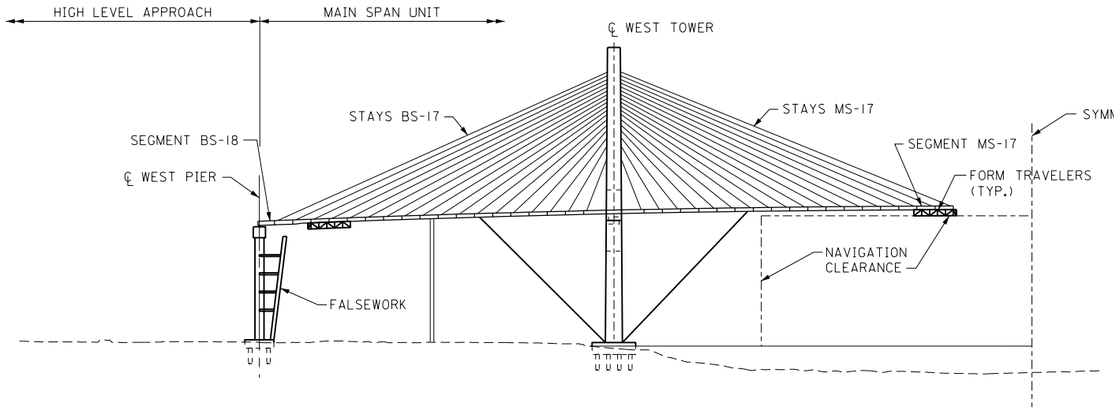
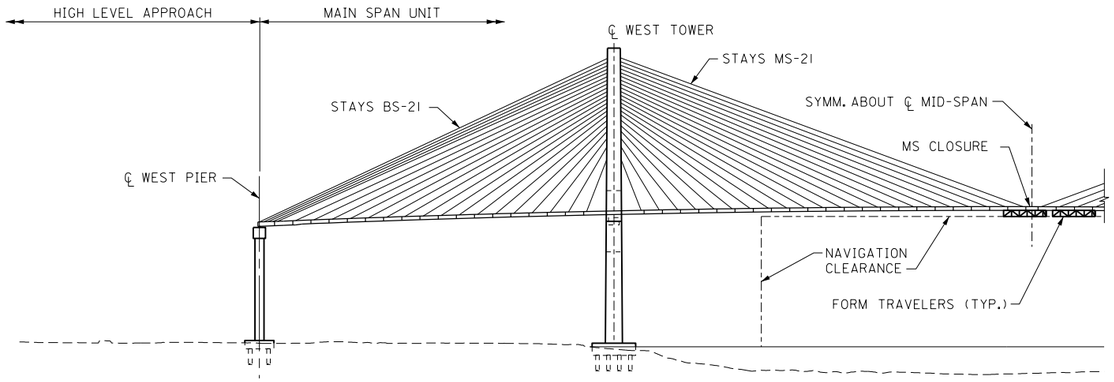
REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER
		7



STEP 5

1. AFTER ADJUSTING STAY CABLES BS-17 AND MS-17, MOVE BACK SPAN TRAVELERS ONE SEGMENT TOWARDS TOWER TO MAKE ROOM FOR BACKSPAN CLOSURE FALSEWORK.
2. INSTALL BACK SPAN CLOSURE FALSEWORK, RELEASE BACK SPAN PROP.



STEP 6

1. ADVANCE MAIN SPAN TRAVELER TO SEGMENT MS-18.
2. INSTALL STAY CABLES BS-18, AND MS-18 TO SUPPORT FORM TRAVELER.
3. CAST SEGMENT PAIR MS-18.
4. STRESS FLOORBEAM TENDONS IN SEGMENT MS-18.
5. ADJUST STAY CABLES BS-18 AND MS-18, AND TRANSFER MS-18 STAY ANCHORAGE TO PERMANENT STRUCTURE.
6. REPEAT STEPS 6.1 THROUGH 6.5 FOR SEGMENTS MS-19 THROUGH MS-21.
7. RELEASE BACK SPAN CLOSURE FALSEWORK.
8. ONCE EAST AND WEST MAIN SPAN CANTILEVERS ARE COMPLETED THROUGH SEGMENT MS-21, MOVE BACK (OR REMOVE) EAST TRAVELER AND ADVANCE WEST TRAVELER TO MAIN SPAN CLOSURE SEGMENT.
9. RELEASE TEMPORARY TIE-DOWN CABLES, AND ADJUST STAY CABLES TO ALIGN TIPS FOR CLOSURE.
10. CAST MAIN SPAN CLOSURE SEGMENT AND REMOVE FORM TRAVELERS.
11. COMPLETE BARRIERS AND OVERLAY.

ERECTION NOTES:

1. STAY CABLE INSTALLATION SHALL BE BALANCED WITHIN 5% FORCE FOR EACH EDGE OF DECK WITHIN A DECK.
2. SEGMENTS ERECTED IN BALANCED CANTILEVER SHALL BE CAST SIMULTANEOUSLY ON MAIN SPAN / BACK SPAN.
3. EASTBOUND/WESTBOUND DECK ERECTION SHALL BE BALANCED ON EACH ERECTION FRONT WITHIN TWO SEGMENT LENGTHS, EXCEPT THAT EACH DECK SHALL BE AT THE SAME STEP WHEN:
   a. TEMPORARY TIE-DOWN CABLES OR PROPS ARE INSTALLED.
   b. CLOSURE SEGMENTS ARE CAST.

NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 7 OF 7	MOBILE RIVER BRIDGE MAIN SPAN SPLIT DECK - DOUBLE DIAMOND - CONCRETE
REVISIONS	ERECTION SEQUENCE (2 OF 2) ALTERNATIVE 7
BIN:	ESTIMATED QUANTITIES / DESIGNED BY: MWL / DRAWN BY: SRC COMPUTED BY: / CHECKED BY: / DATE DRAWN: 8/18/2016 VERIFIED BY: / DATE CHECKED: / SCALE: AS NOTED

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USER: JSPANIER DATE: Monday, July 31, 2017 04:06:36 PM
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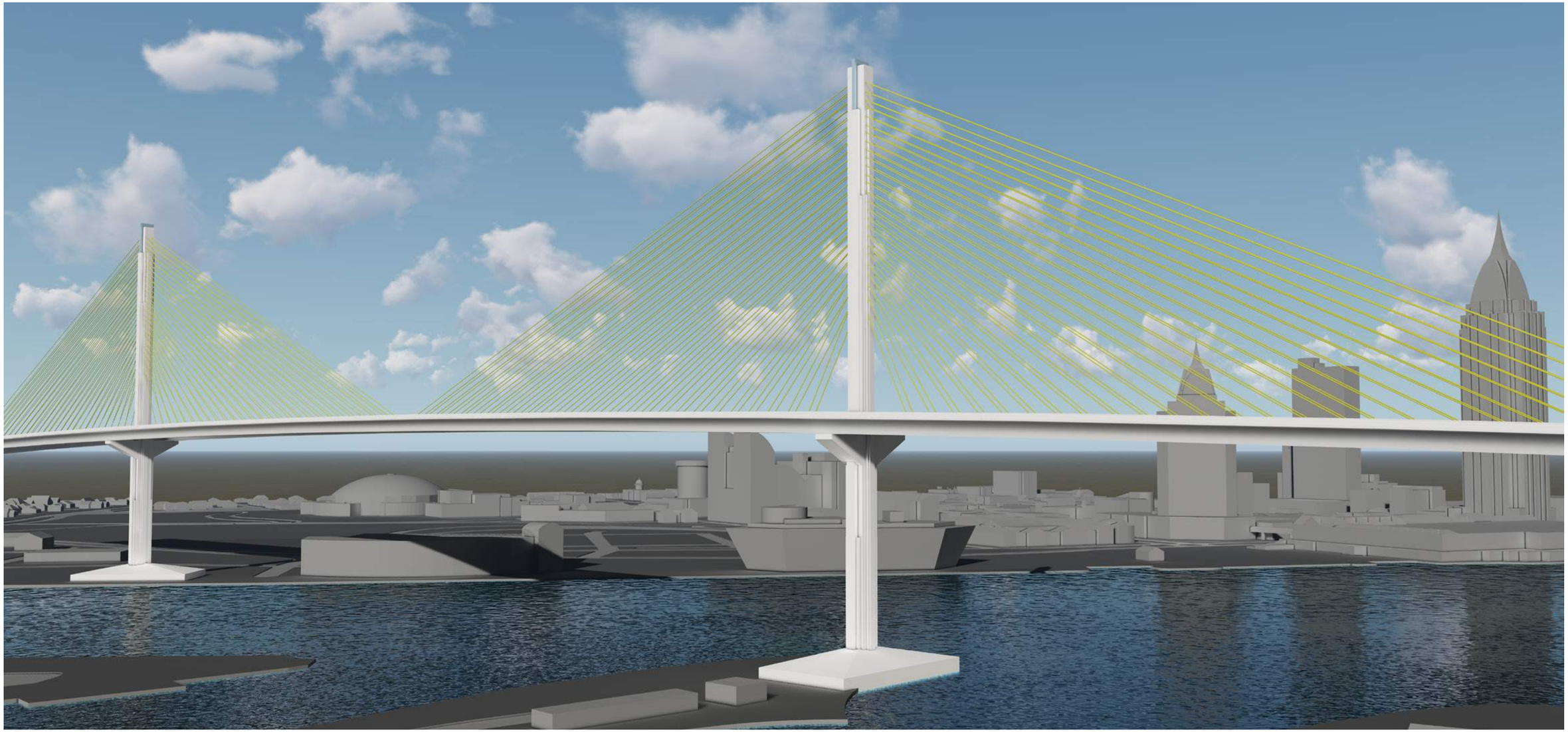
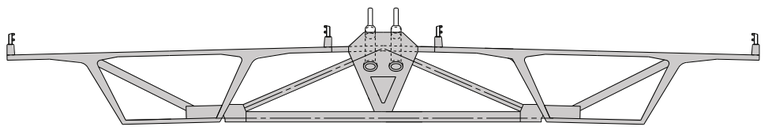
Appendix P:

Step 2:

Tower Aesthetic Concepts





Typical Cross Section at Cable Anchor

TOUCHSTONE ARCHITECTURE™

title: Alt_01 - Art Deco Pylon

date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

This architectural rendering conveys design intent only and may not be relied upon for dimensional or illumination accuracy.

sheet: a101

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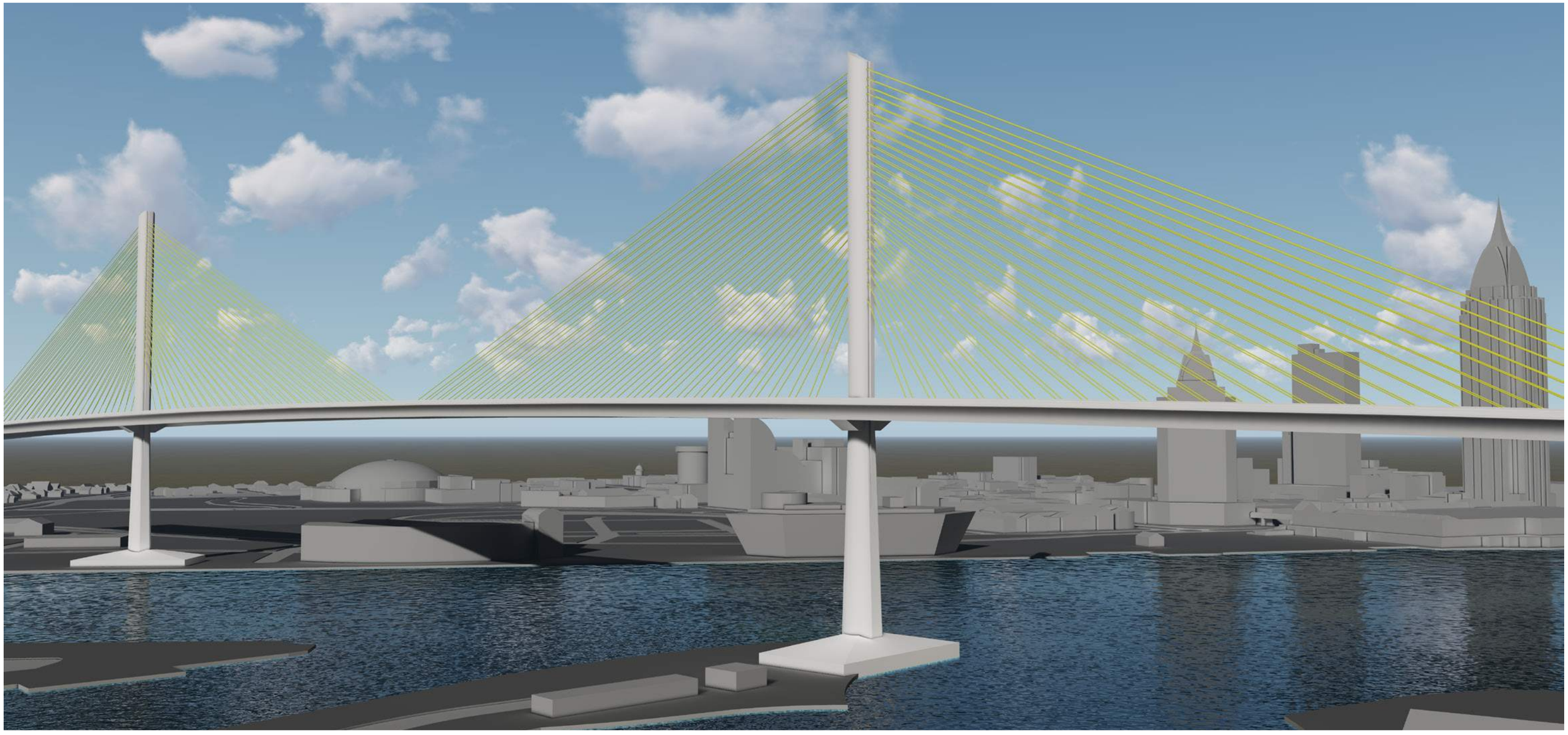

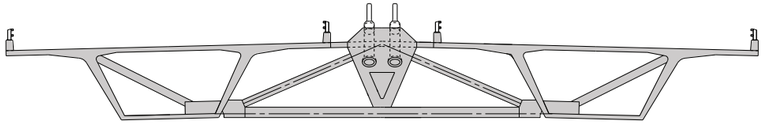

Typical Cross Section at Cable Anchor

TOUCHSTONE ARCHITECTURE™

title: Alt_01 - Oval Pylon

date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

This architectural rendering conveys design intent only and may not be relied upon for dimensional or illumination accuracy.

sheet: a102

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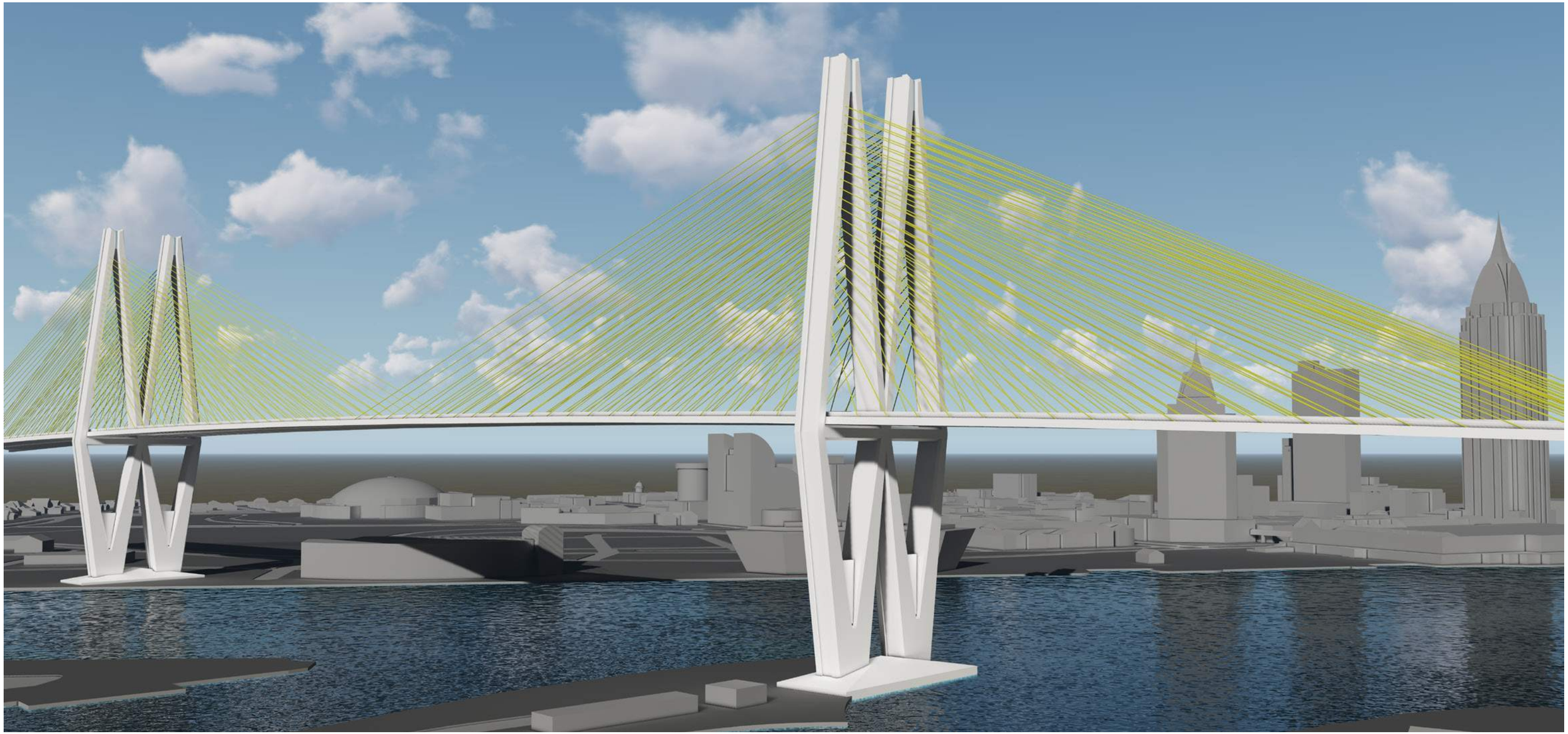
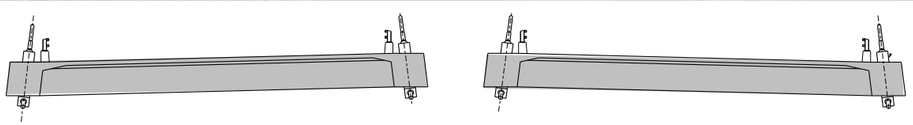
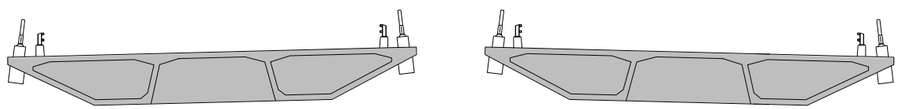




Typical Cross Section “A” at Cable Anchor



Typical Cross Section “B” at Cable Anchor

TOUCHSTONE ARCHITECTURE™

title: Alt_03 Diamond Tower

date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

This architectural rendering conveys design intent only and may not be relied upon for dimensional or illumination accuracy.

sheet: a103

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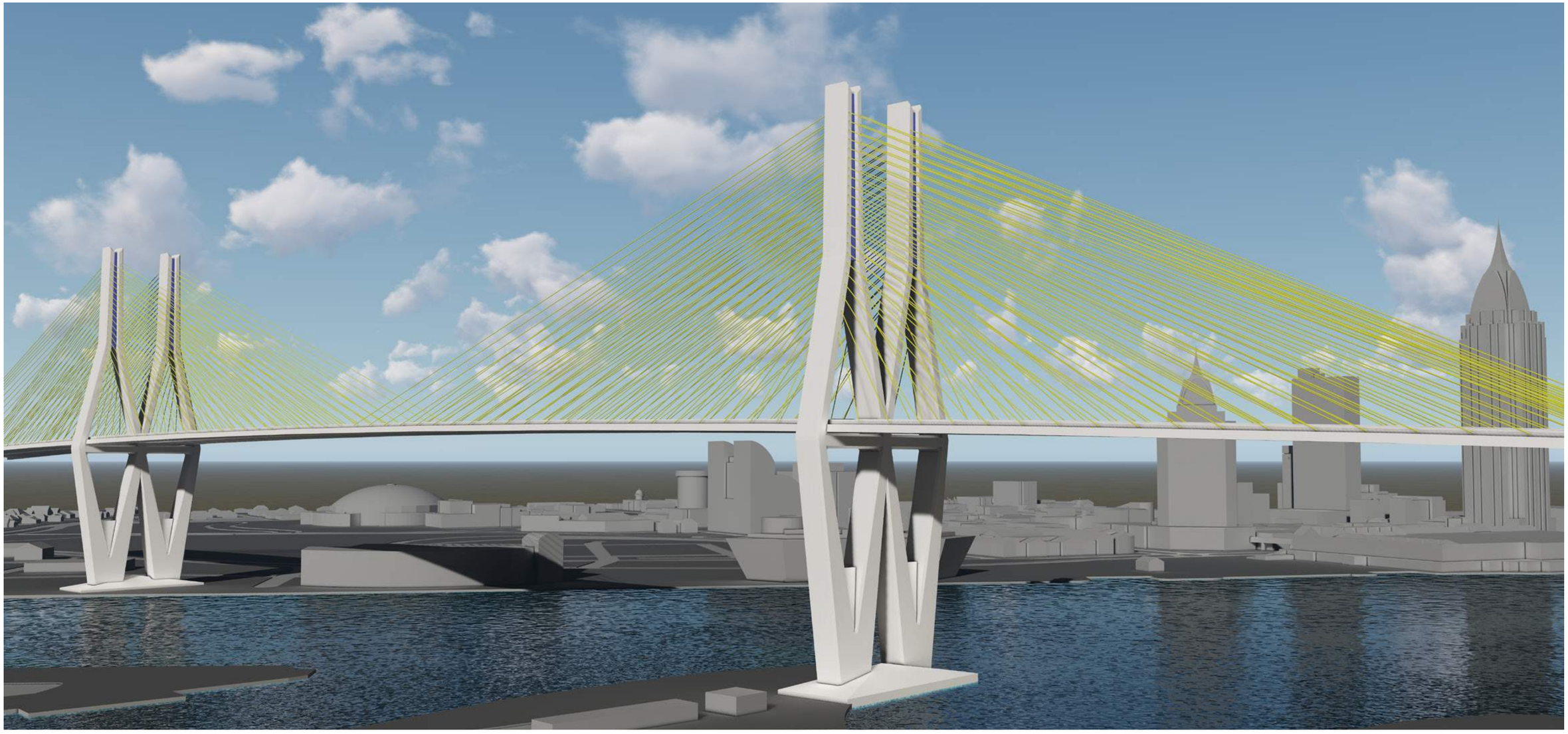
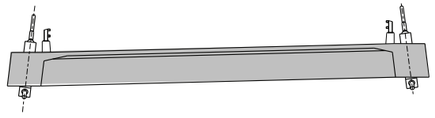
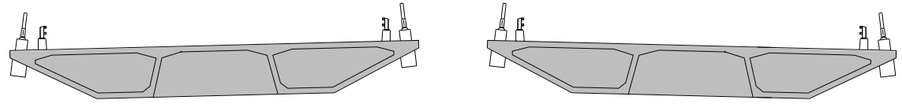




Typical Cross Section “A” at Cable Anchor



Typical Cross Section “B” at Cable Anchor

TOUCHSTONE ARCHITECTURE™

title: Alt_03 Inverted “Y” Tower

date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

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sheet: a104

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Appendix Q:

Step 2:

Unit Costs

Step 2 Unit Costs		Option No.1 - Concrete Box Single Pylon (Span-by-Span)	Option No.1A - Concrete Box Single Pylon (Balanced Cantilever)	Option No. 3 - Twin Concrete Edge Girder (Diamond Pylon)	Option No. 7 - Twin Concrete Cast-in-Place Box (Diamond Pylon)
Pay Item	**Unit**	**Unit Cost**	**Unit Cost**	**Unit Cost**	**Unit Cost**
Cofferdam	SF	$50.00	$50.00	$50.00	$50.00
Pier Protection System	LS	$1,000,000.00	$1,000,000.00	$1,070,000.00	$1,070,000.00
Structural Excavation	CY	$25.00	$25.00	$25.00	$25.00
Seal Concrete	CY	$200.00	$200.00	$200.00	$200.00
Structural Backfill	CY	$15.00	$15.00	$15.00	$15.00
Pile Testing (Statnamic)	EA	$150,000.00	$150,000.00	$150,000.00	$150,000.00
Pile Testing (Static)	EA	$250,000.00	$250,000.00	$250,000.00	$250,000.00
Pile Testing (PDA)	EA	$7,500.00	$7,500.00	$7,500.00	$7,500.00
6' Dia Pipe Piles	LF	$600.00	$600.00	$600.00	$600.00
Reinforcement - Black - Substructure	LBS	$1.25	$1.25	$1.28	$1.28
Reinforcement - Black - Superstructure	LBS	$1.25	$1.25	$1.28	$1.28
Footing Concrete	CY	$275.00	$275.00	$275.00	$275.00
Backspan Pier Concrete	CY	$900.00	$900.00	$900.00	$900.00
Tower Concrete	CY	$1,050.00	$1,050.00	$1,100.00	$1,100.00
Miscellaneous Steel – Tower/Access	LS	$250,000.00	$250,000.00	$500,000.00	$500,000.00
Anchor Box Steel/Cradle (Saddle)	LBS	$3.00	$3.00	$4.50	$4.50
Stay Cables	LB	$10.50	$10.50	$10.50	$10.50
Stay Cable Testing	LS	$400,000.00	$400,000.00	$400,000.00	$400,000.00
Post-Tensioning - Substructure	LB	$6.00	$6.00	$6.00	$6.00
Temporary Cantilever Tie-Downs	LS	$0	$1,000,000.00	$1,000,000.00	$1,000,000.00
Post-Tensioning - Superstructure (Strand)	LB	$5.00	$5.00	$5.00	$5.00
Post-Tensioning - Superstructure (Bars)	LB	$8.00	$8.00	$8.00	$8.00
Structural Steel - Superstructure	LB	$2.50	$2.50	$2.50	$2.50
Concrete - Superstructure	CY	$900.00	$900.00	$1,150.00	$1,150.00
Cast-in-Place Concrete Cosures	CY	$1,800.00	$1,800.00	-	-
Barriers	LF	$140.00	$140.00	$140.00	$140.00
Bridge Drainage System	LS	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Bearings	EA	$20,000.00	$20,000.00	$17,500.00	$17,500.00
Modular Expansion Joint	LF	$4,500.00	$4,500.00	$4,500.00	$4,500.00
Overlay	SF	$12.00	$12.00	$12.00	$12.00
Roadway Lighting	LS	$1,260,000.00	$1,260,000.00	$1,560,000.00	$1,560,000.00
Navigational Lighting System	LS	$125,000.00	$125,000.00	$150,000.00	$150,000.00
**Aesthetic Lighting System	LS	$0	$0	$0	$0
Lighting Protection System	LS	$50,000.00	$50,000.00	$100,000.00	$100,000.00
Striping	LS	$25,000.00	$25,000.00	$25,000.00	$25,000.00
Signs	LS	$50,000.00	$50,000.00	$50,000.00	$50,000.00
Anti-Vandalism/Terrorist Protection	EA	$40,000.00	$40,000.00	$30,000.00	$30,000.00
Special Erection Equip/Temporary Works	EA	$1,300,000.00	$0	$1,250,000.00	$1,250,000.00
Damper/STU	EA	$0	$0	$125,000.00	$125,000.00
Fire Suppression	LS	$3,500,000.00	$3,500,000.00	$3,500,000.00	$3,500,000.00

***Included as a stand-alone item in the overall project cost estimate*

Appendix R:

January 19, 2017 Meeting

- PowerPoint Presentation
- Meeting Attendance
- Meeting Minutes

Meeting Minutes

Project:	I-10 Mobile River Bridge & Bayway Widening
Subject:	Main Span Bridge Type Selection Process – Preferred Alternative
Date:	Thursday, January 19, 2017 10:00 AM
Location:	Alabama Department of Transportation - SWR
Attendees:	Patrick Hickox (HDR) Manuel Carballo (HDR) Jacob Perkins (HDR) Michael Lamont (HDR) Bradley Touchstone (TA) John Armeni (AC) Jimmy Shumock (TE) Steve Flukinger (TE) Sam Sternberg (TE) Kevin Jones (ALDOT) Tim Colquitt (ALDOT) Randall Mullins (ALDOT) Don Arkle (ALDOT) Edwin Perry (ALDOT) Brian Aaron (ALDOT) Stan Biddick (ALDOT) William Adams (ALDOT) Vince Calametti (ALDOT) Edwin Perry (ALDOT) Andrew Wood (ALDOT) Matt Ericksen (ALDOT)

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

A. **Main Span Bridge Type Selection Process**

1. **Type Selection Process Overview**

   No comments or action items from this section of the presentation.

2. **Review Results from Previous Meeting**

3. *No comments or action items from this section of the presentation.*

4. **Further Development of Alternatives**

   ALDOT commented that for the final design, the vessel collision analysis will need to include an updated bathymetry of the Mobile River in the vicinity of the west and east tower foundation locations to confirm the assumptions made for the preliminary design.

   ALDOT asked if we could consider lowering the footing to protect the piling. HDR stated that this could be done but it would not be the most cost effective solution given the location of the footing adjacent to the Mobile River. ALDOT recommended that for the West Tower Permanent Cofferdam Detail (shown on slide 43) that a full concrete skirt be used instead of the rock backfill material shown on the drawing.

This preference will be moved forward in the design. Also, it was noted that the magnitude of the vessel impact did not change from that previously predicted.

The corrosion test results (provided on slide 52) was discussed. Thompson and HDR recommended that we take some additional samples from a boring and the water to better determine the corrosiveness and to finalize the recommendation on the steel pile option thicknesses. ALDOT has some concerns over the service life and the use of coatings for the steel pile option. It was agreed that additional samples and testing would be performed before making a decision on the steel pile option.

ALDOT asked a question regarding changing option 1 and 1A to cast-in-place. This was discussed and HDR will provide additional information in the form a technical memorandum. HDR met with Tim Colquitt and Randall Mullins on January 25, 2017 to review this additional information (see attached technical memorandum).

The group also discussed using a single stay in place of the dual stays shown for options 1 and 1A. In order to maintain a more common longitudinal spacing of the stays, a very large stay cable would be required that would be outside of the typical cross sectional sizes provided the material suppliers within the industry. It was agreed that if option 1 or 1A was determined to be advanced, this would be further evaluated.

5. Main Span Concepts – Refined Evaluation

ALDOT noted that the construction methods shown on slides 83-85 could be used for the other alternatives.

ALDOT expressed some concern over the location of the back span piers and would like to see plan views on aerial photography. See attached for these updated plan sheets.

ALDOT asked about the expected aerodynamic responses for the individual options. HDR responded that all of the options have a similar response.

ALDOT asked about open versus closed traffic railing on the main span unit. HDR will further evaluate this but noted that TL-5 tested open railing is being utilized on bridges and may offer some aerodynamic improvements.

The final options were discussed and it was agreed that Alternative 3 – Cast-In-Place Concrete Edge Girder, Split Deck is the preferred option to be carrier further into design.

6. Next Steps

ALDOT to discuss maintaining the aesthetic lighting with the City of Mobile. HDR to provide a rendering for discussion at the next ETS meeting in Montgomery.

Attendance List

Meeting: Mobile River Bridge– Main Span Bridge Type Selection Process – Preferred Alternative	Time: 10:00 a.m.	Date: January 19, 2017

Name	Firm	Phone	Email
BRADLEY TOUCHSTONE	TOUCHSTONE ARCH	850 545 5227	btouchstone @ touchstonearchitecture.com
JAKE PERKINS	HDR	360-620-2085	jacob.perkins@hdrinc.com
MIKE LAMONT	HDR	360-~~561~~ 481-3219	michael.lamont@hdrinc.com
JOHN ARMENI	ARMENI CONSULTING	404-414-3743	john.armeni@armeniconsulting.com
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KEVIN JONES	ALDOT SW	251 470-8200	joneske@dot.state.al.us
MATT ERICKSON	ALDOT SW	251-470-8374	ericksm@dot.state.al.us
Sam Sternberg	TE	251-509-7786	ssternberg@thompsonengineering.com
Jimmy SHUMOCK	TE	251-666-2445	JSHUMOCK@THOMPSONENGINEERING.COM
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WILLIAM ADAMS	ALDOT/DESIGN	334-242-6488	adamswi@dot.state.al.us
RANDALL MULLINS	ALDOT/BRIDGE	" 242-6004	MULLINSR@ " "
TIM COLQUETT	ALDOT/BRIDGE	" " - 6007	colquettw@ " "
DON ARKLE	ALDOT	334-242-6319	arkled@ " " " "
Stan Biddick	ALDOT/Design	334-242-6833	biddickse@ "
Patrick Hickox	HDR		
Manuel Carballo	HDR		
Steve Flukinger	TE		
Edwin Perry	ALDOT		

Agenda

Project:	I-10 Mobile River Bridge & Bayway Widening
Subject:	Main Span Bridge Type Selection Process – Preferred Alternative
Date:	Thursday, January 19, 2017
Location:	Alabama Department of Transportation -SWR

A. **Main Span Bridge Type Selection Process**

1. **Type Selection Process Overview**
2. **Review Results from Previous Meeting**
3. **Further Development of Alternatives**
4. **Main Span Concepts – Refined Evaluation**
5. **Next Steps**

Mobile River Bridge

Main Span Bridge Type Selection Process – Preferred Alternative

HDR

January 19, 2017

General Presentation Outline –

Main Span Bridge Type Selection Process

1. Type Selection Process Overview
2. Review Results from Previous Meeting
3. Further Development of Alternatives
4. Main Span Concepts - Refined Evaluation
5. Next Steps

01 Type Selection Process Overview

1. 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

Today's Desired Outcome is to Selected one Alternative to Carry to the TS&L Phase

02 Review Results from Previous Meeting

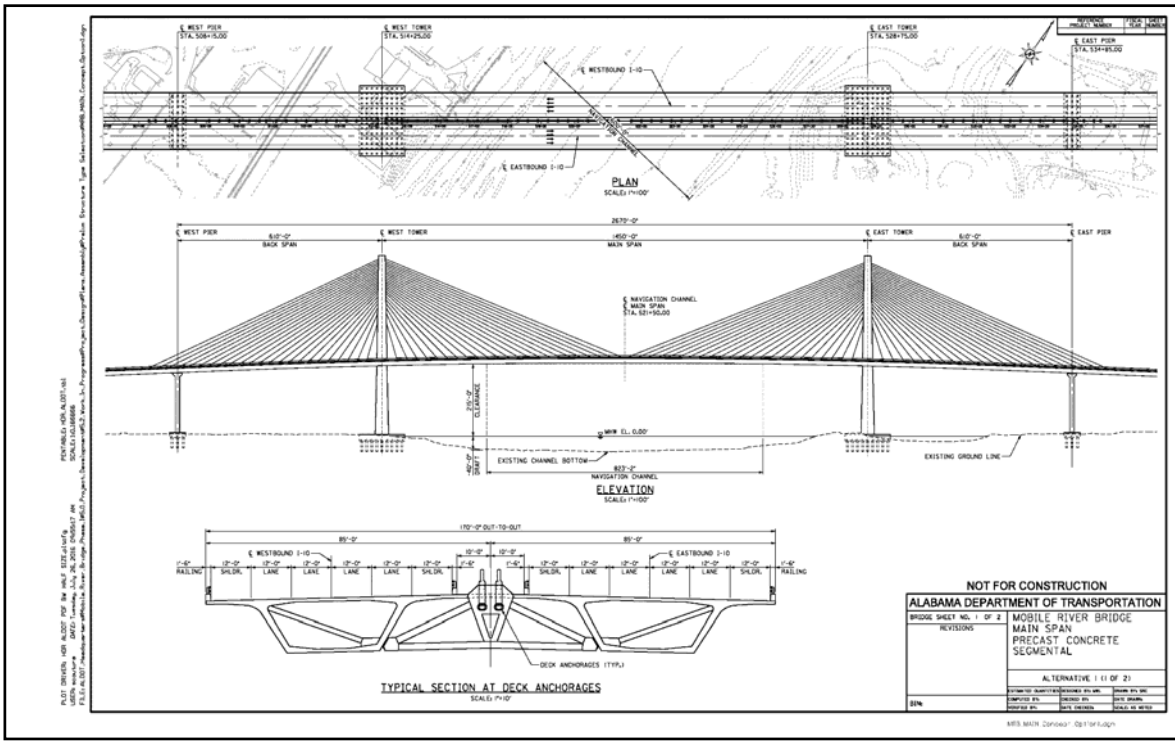
Mobile River Bridge
Bridge Types – Evaluation Matrix

Main Span Criteria

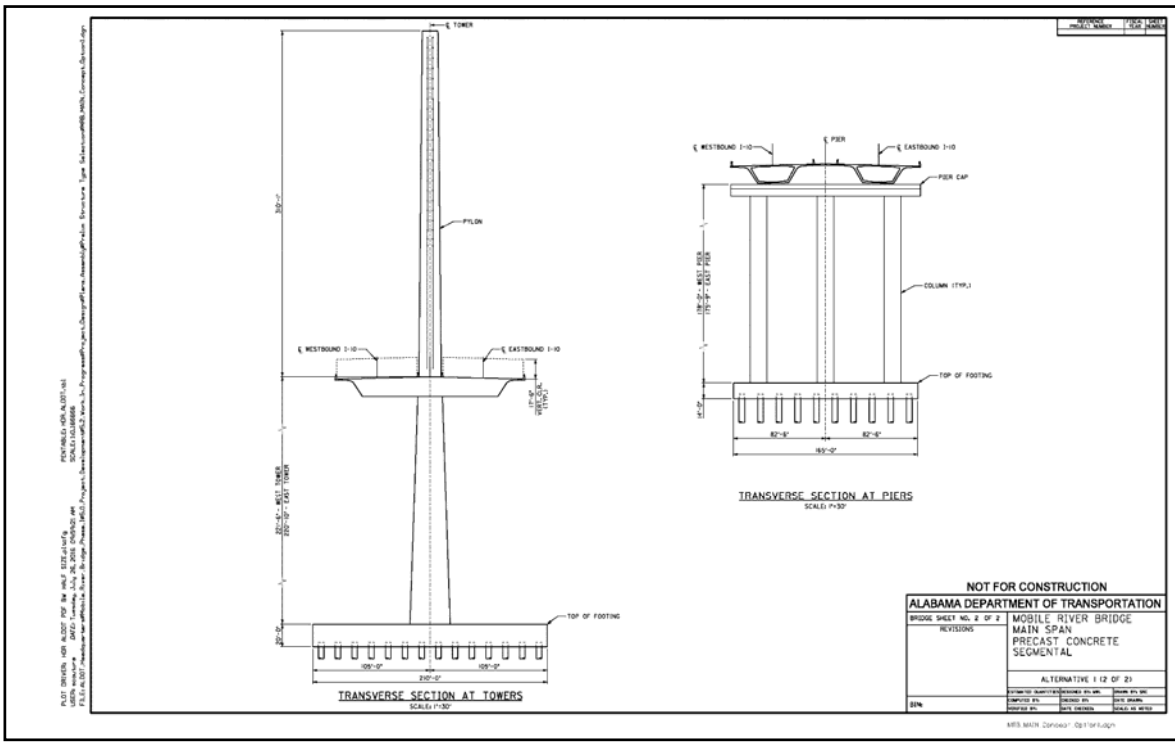
Construction Cost	Constructability	Maintenance and Durability	Construction Schedule	Environmental Commitments	Design Considerations
Initial Construction Cost	Construction Access Disruption to Existing Traffic / Maintenance of Traffic Established Construction Methods Impacts to Existing Utilities Impacts to Railroads Cofferdams	Accessibility Tie-Downs Bearings Deck Joints Painting of Steel Deck Replacement Flexibility for Future Widening Stay Cable Replacement		Contextual Design and Aesthetics Impacts to Mobile's Maritime Traffic during Construction Permitting ROW Construction Vibrations	Foundation Sizes Navigational Air-Draft Clearance Accommodates Future Sidewalk

2. Main Span Bridge Alternatives

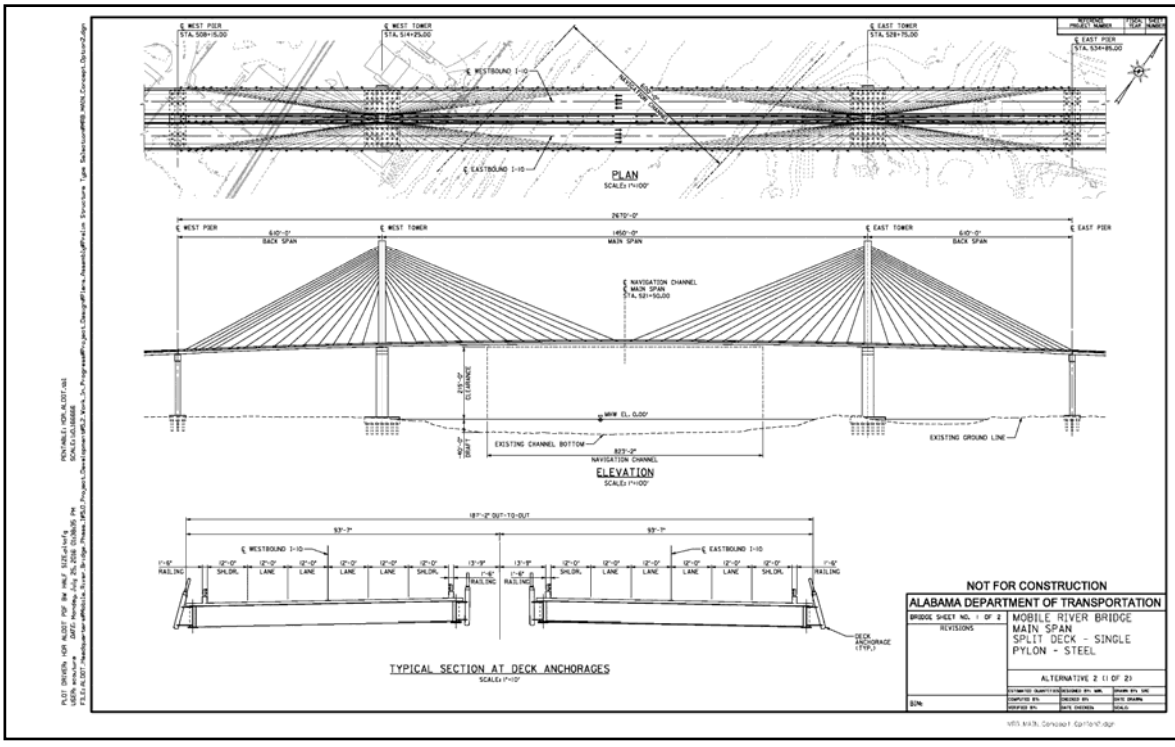
1. Precast Concrete Segmental
2. Steel Edge Girder – Split Deck
3. Cast-in-Place Concrete Edge Girder – Split Deck
4. Steel Edge Girder
5. Steel Double Deck
6. Precast Concrete Segmental – Split Deck
7. Cast-in-Place Segmental – Split Deck



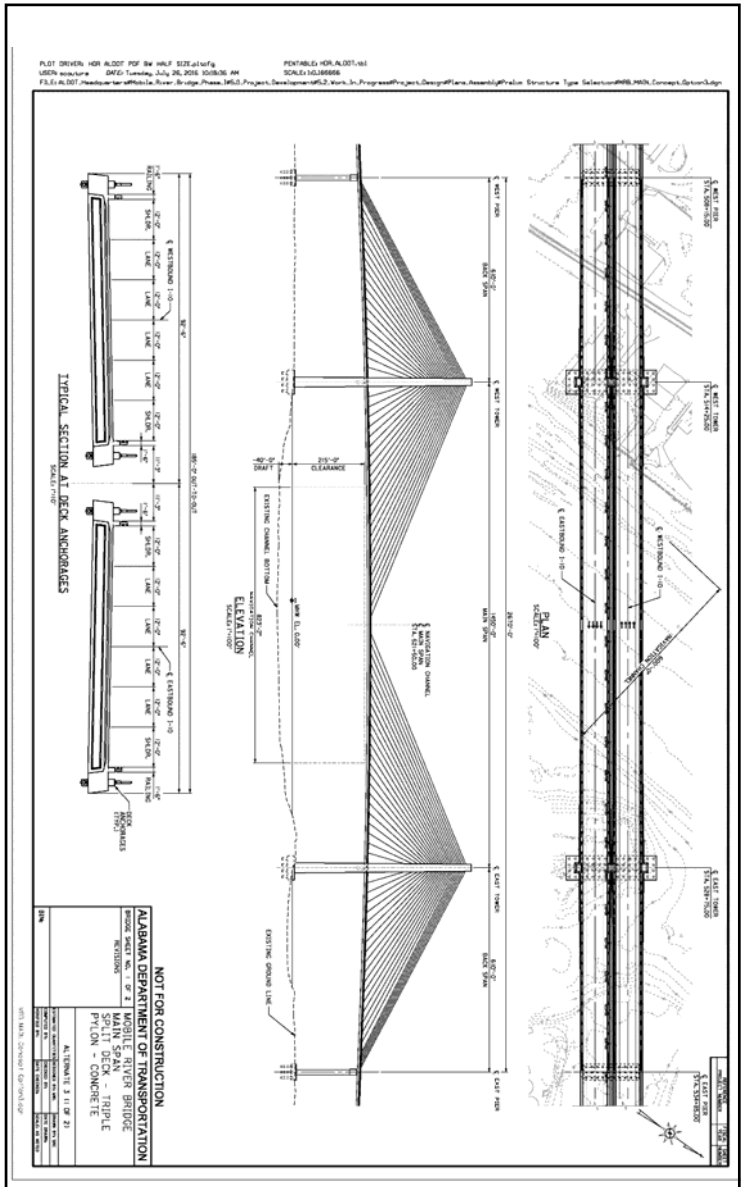
Alternative 1 – Precast Concrete Segmental



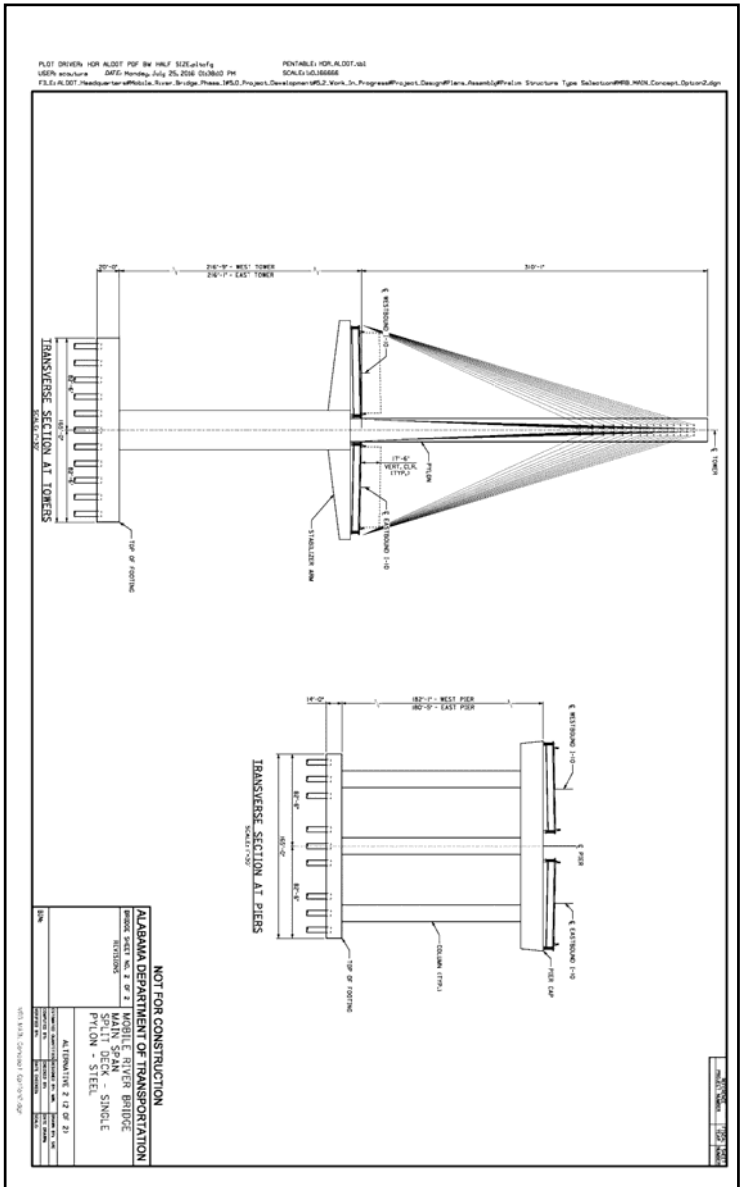
Alternative 1 – Precast Concrete Segmental



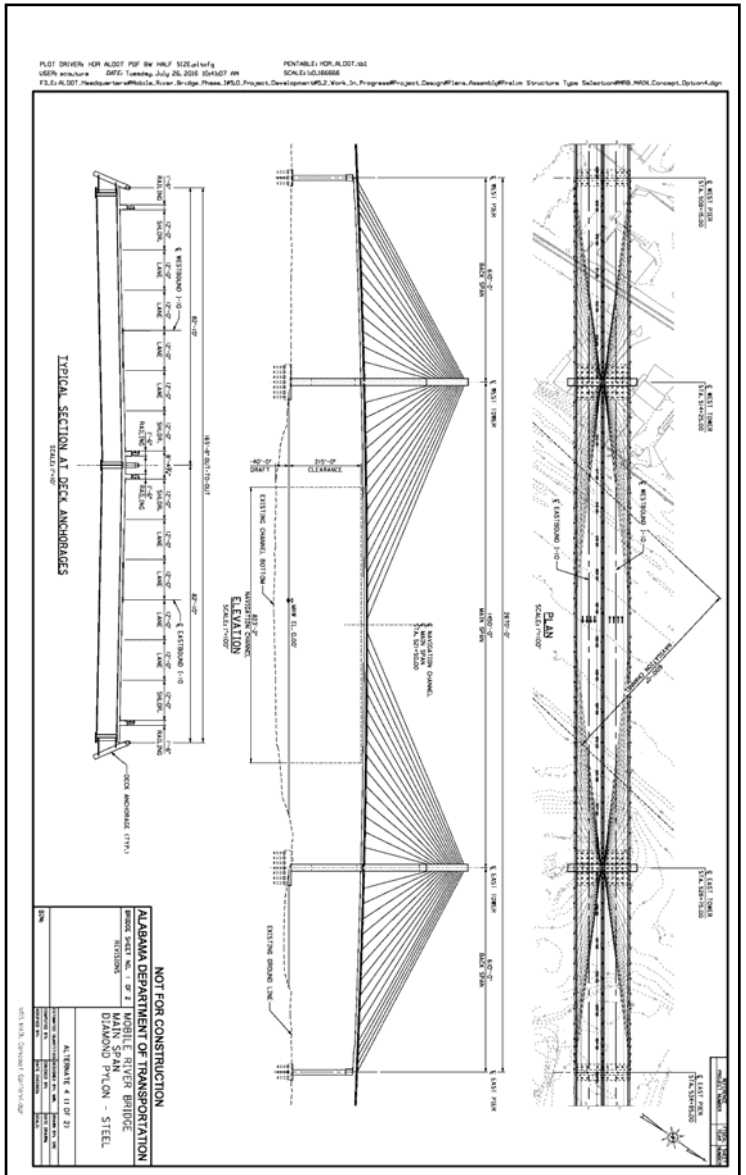
Alternative 2 – Steel Edge Girder (Split Deck)



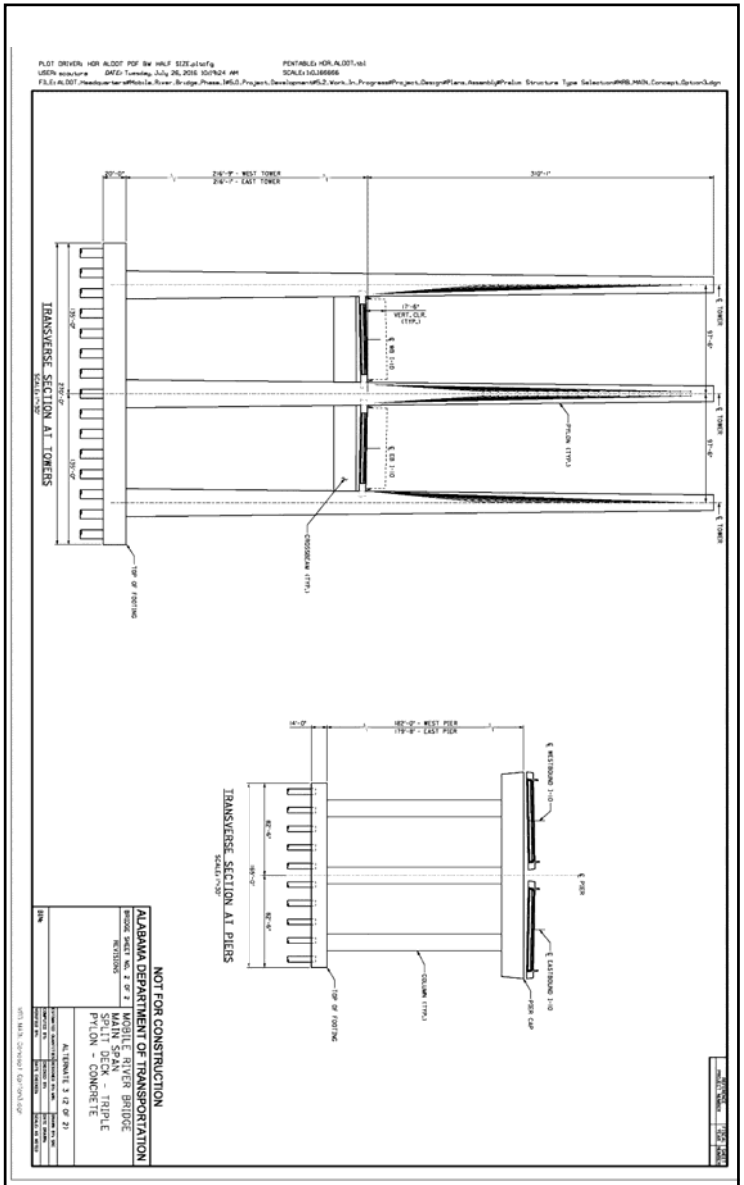
Alternative 3 – Cast-in-Place Concrete Edge Girder (Split Deck)



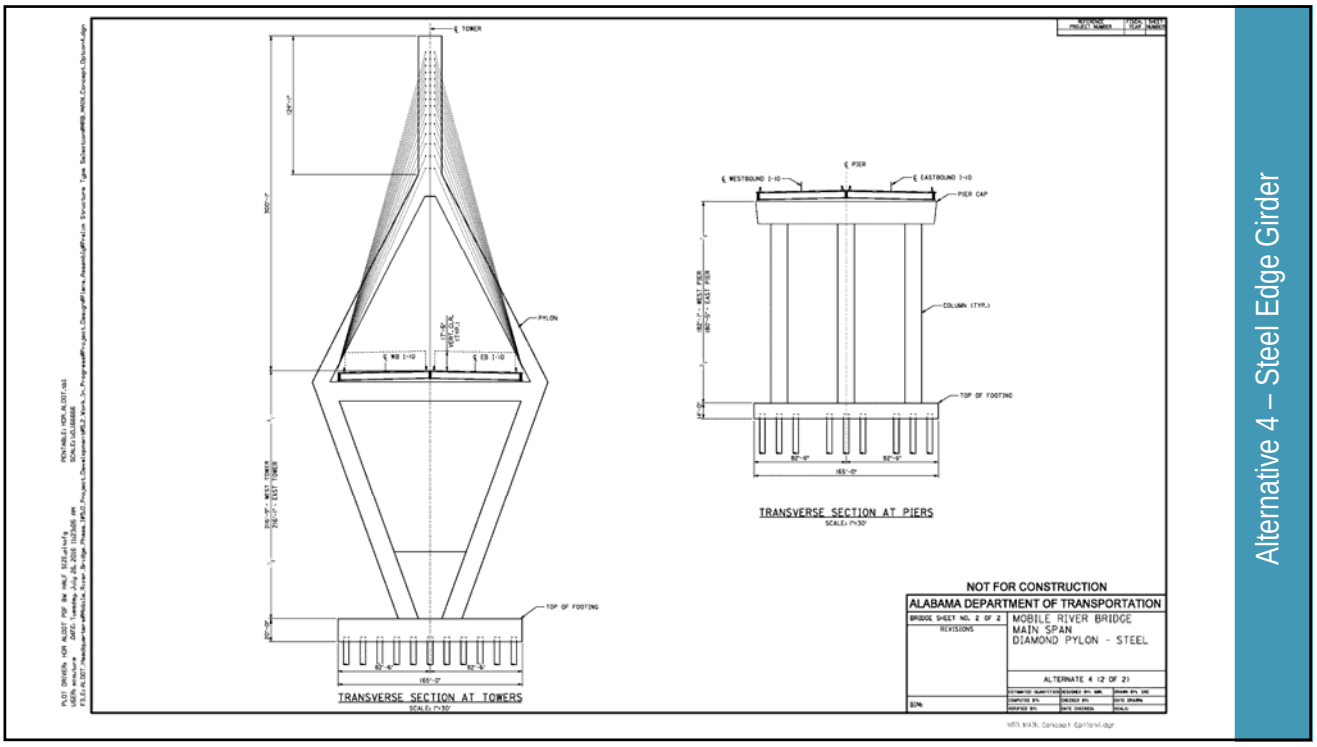
Alternative 2 – Steel Edge Girder (Split Deck)



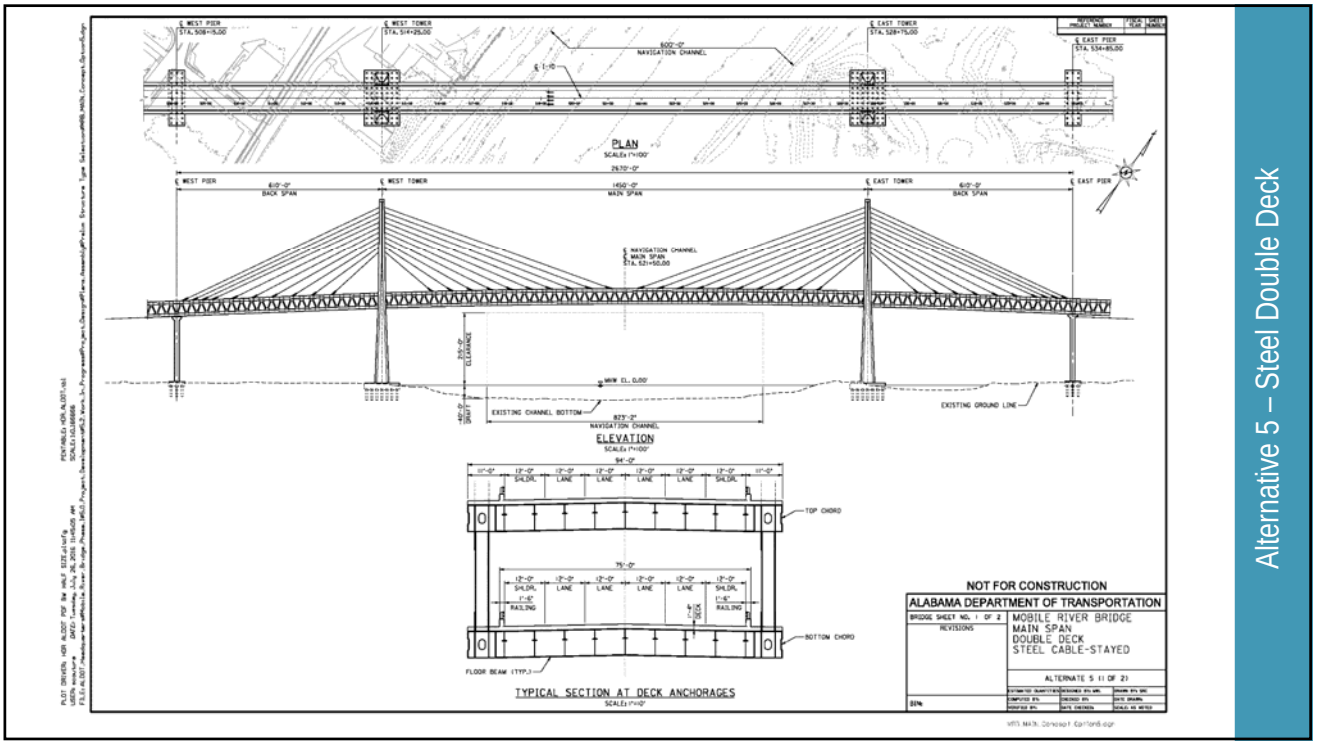
Alternative 4 – Steel Edge Girder



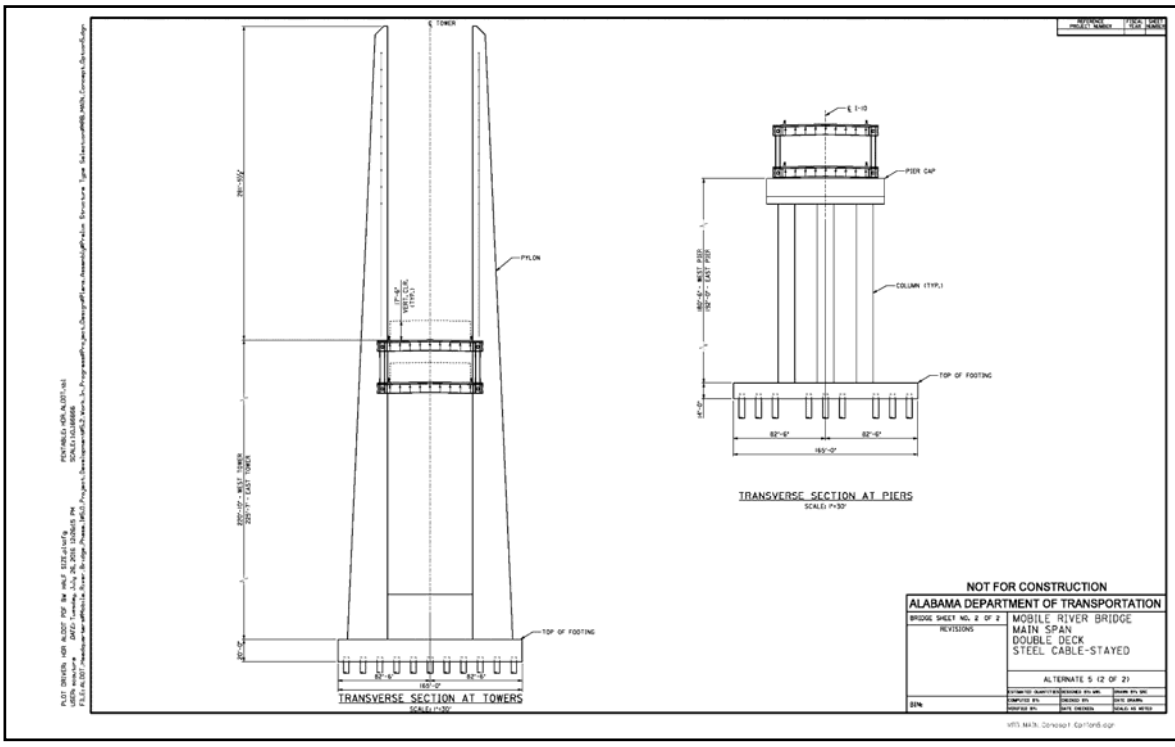
Alternative 3 – Cast-in-Place Concrete Edge Girder (Split Deck)



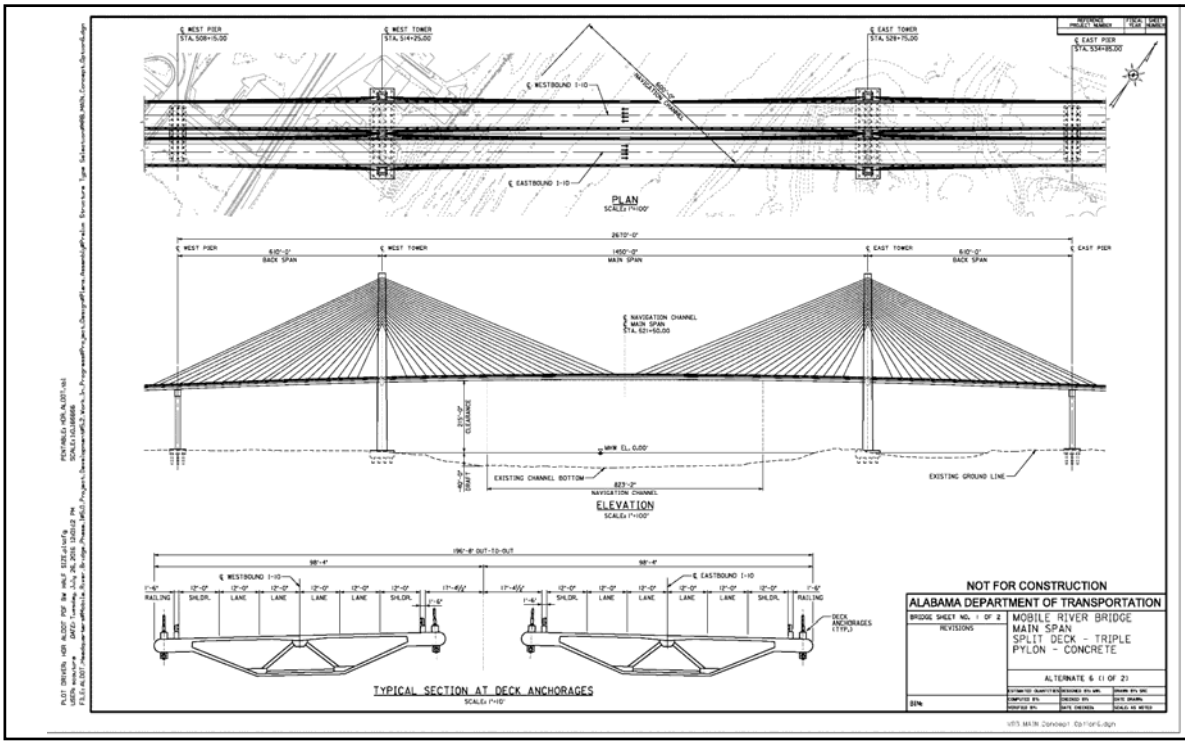
Alternative 4 – Steel Edge Girder



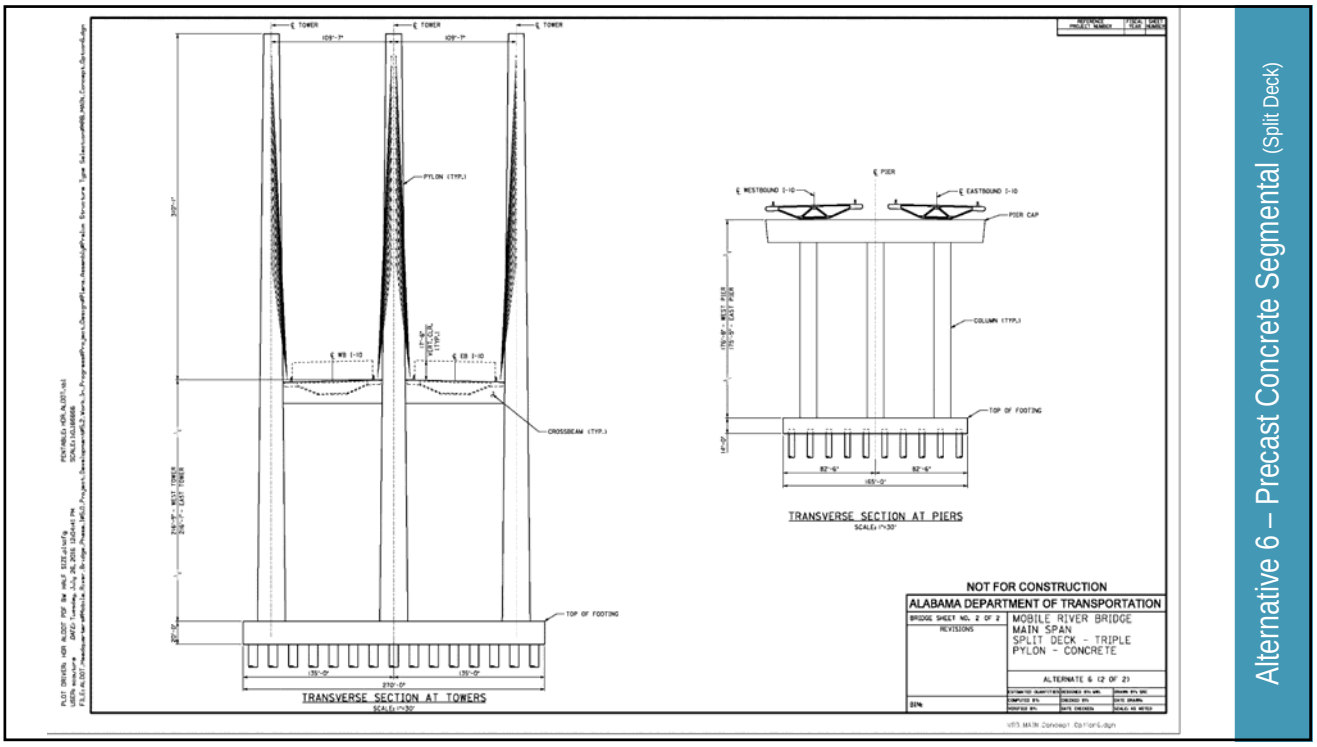
Alternative 5 – Steel Double Deck



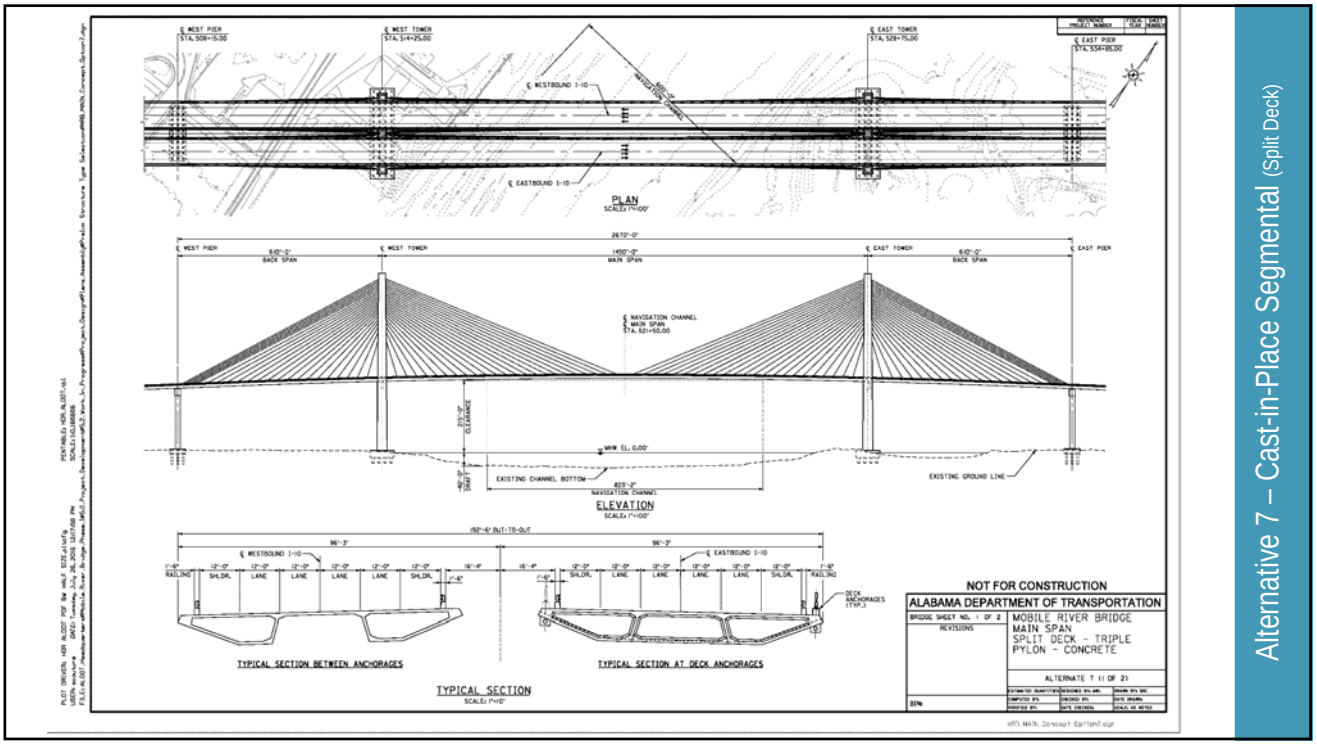
Alternative 5 – Steel Double Deck



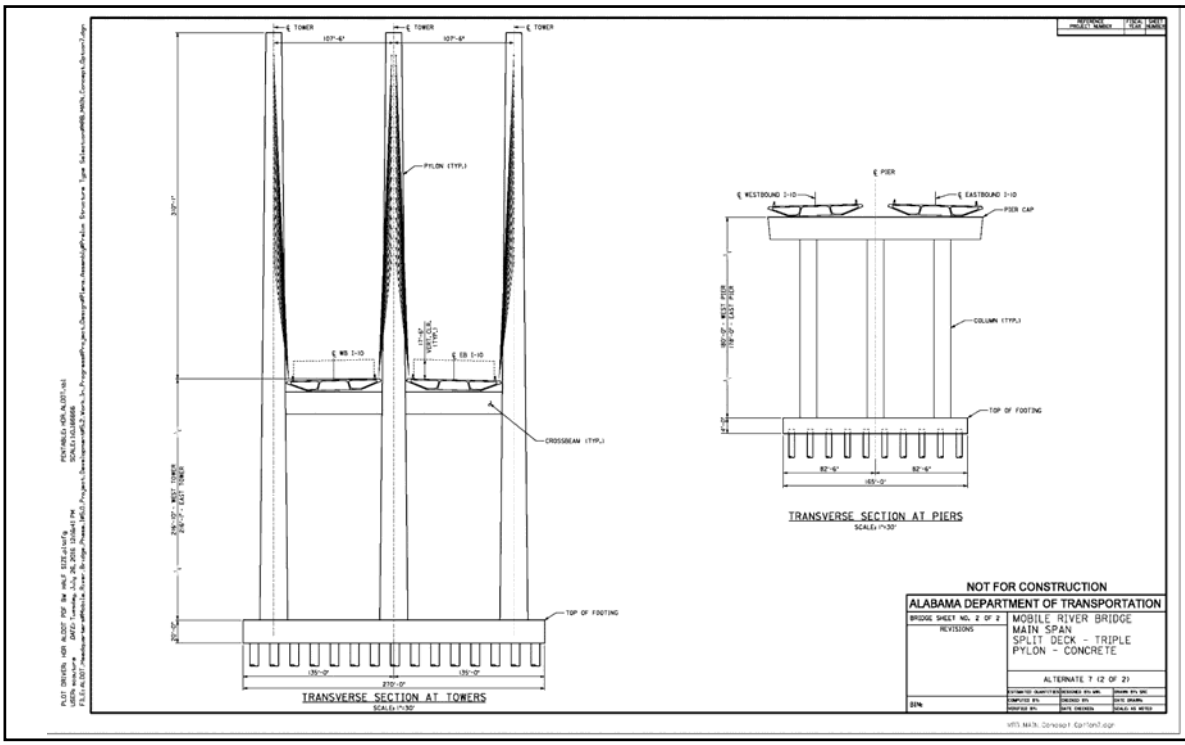
Alternative 6 – Precast Concrete Segmental (Split Deck)



Alternative 6 – Precast Concrete Segmental (Split Deck)



Alternative 7 – Cast-in-Place Segmental (Split Deck)



Alternative 7 – Cast-in-Place Segmental (Split Deck)

2. Main Span Concepts Evaluation – Recommendations

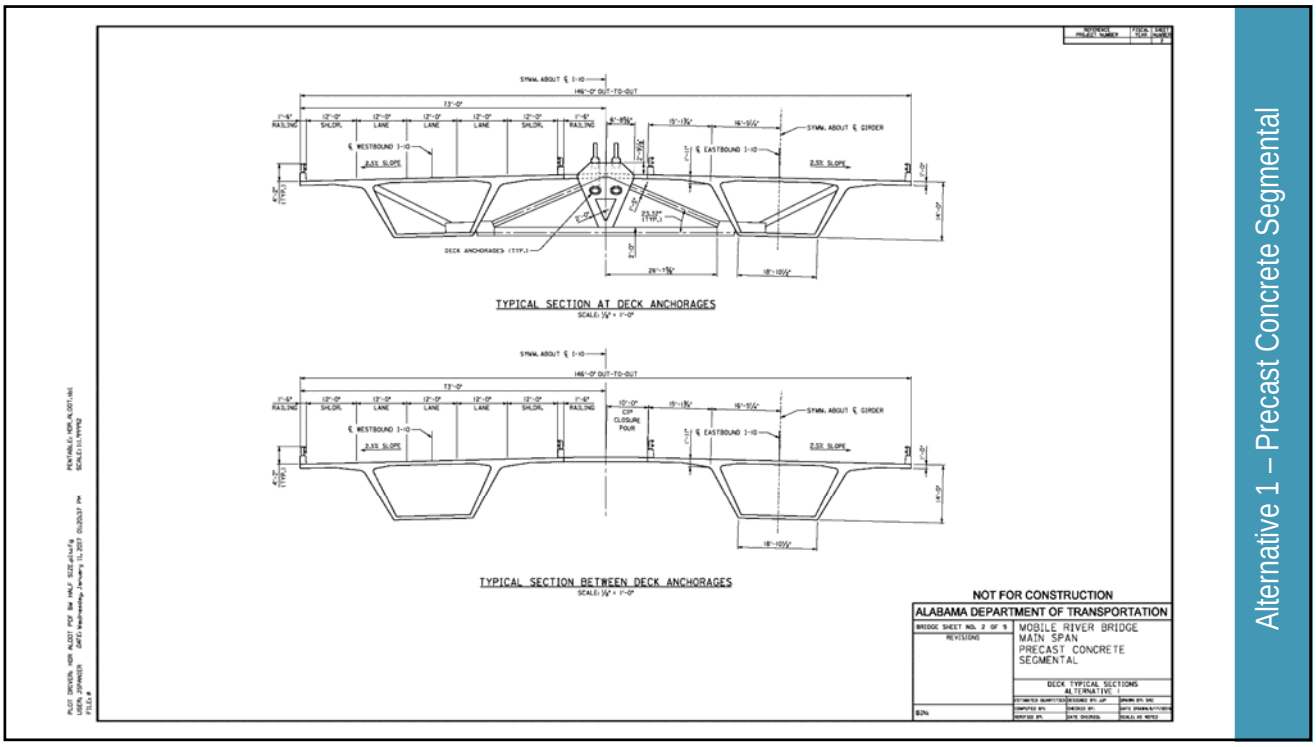
Good = ● Fair = ◒ Poor = ○

	Main Span Bridge Alternatives						
Criteria	1. Precast Concrete Segmental	2. Steel Edge Girder – Split Deck	3. CIP Concrete Edge Girder – Split Deck	4. Steel Edge Girder	5. Steel Double Deck	6. Precast Concrete Segmental – Split Deck	7. CIP Concrete Segmental – Split Deck
Initial Construction Cost ($) M	305	280	325	295	465	350	325
Construction Schedule (Mos.)	40-45	40-45	40-45	40-45	42-50	40-45	40-45
Constructability	●	●	●	◒	○	●	●
Maintenance and Durability	●	◒	●	◒	○	●	●
Environmental Commitments	●	●	●	◒	◒	●	●
Design Considerations	◒	●	●	●	○	◒	●
Overall	●	◒	●	◒	○	●	●

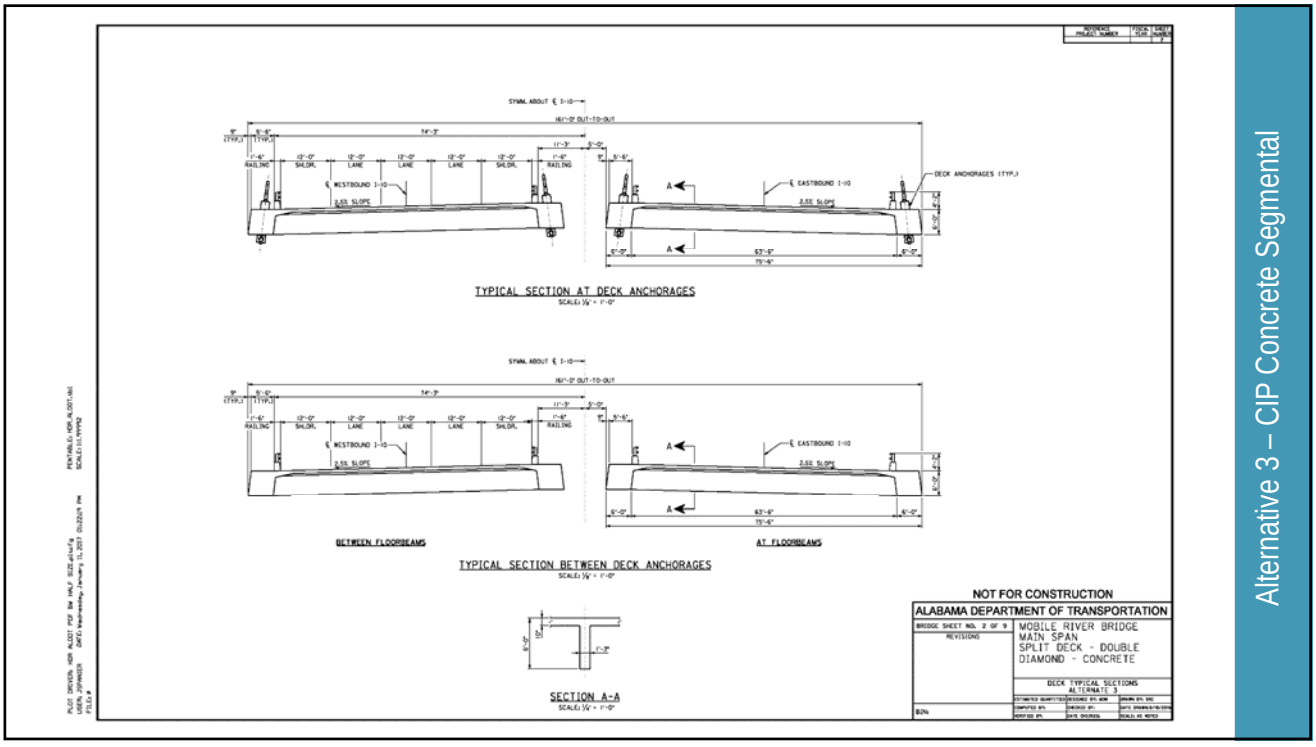
03 Further Development of Alternatives

3. Further Development of Alternatives

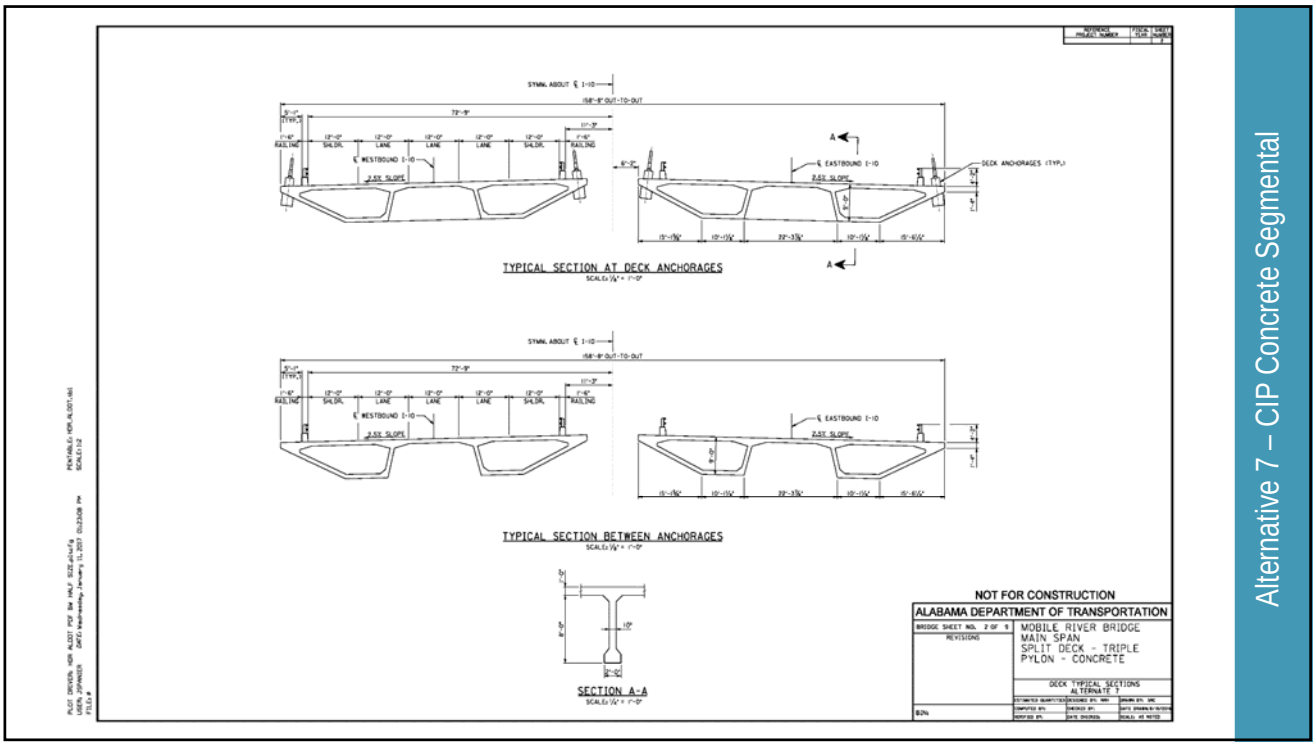
a. Updated Cross-Sections from 4 lanes to 3 Lanes in Each Direction
b. East Tower Location
c. Optimized Main Span Length
d. Vessel Collision Review (Pier Protection)
e. Location of Back Span Pier
f. Tower Design Evolution (Superstructure Stability)
g. Tower Design Evolution (FAA Review)
h. Corrosion Test Results
i. Revised Bridge Layouts
j. Aesthetic Concepts



Alternative 1 – Precast Concrete Segmental



Alternative 3 – CIP Concrete Segmental

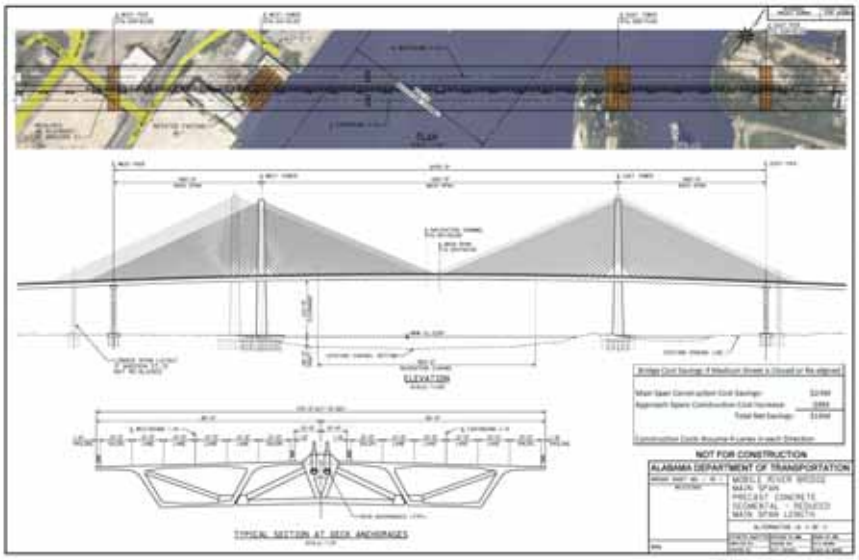


Alternative 7 – CIP Concrete Segmental

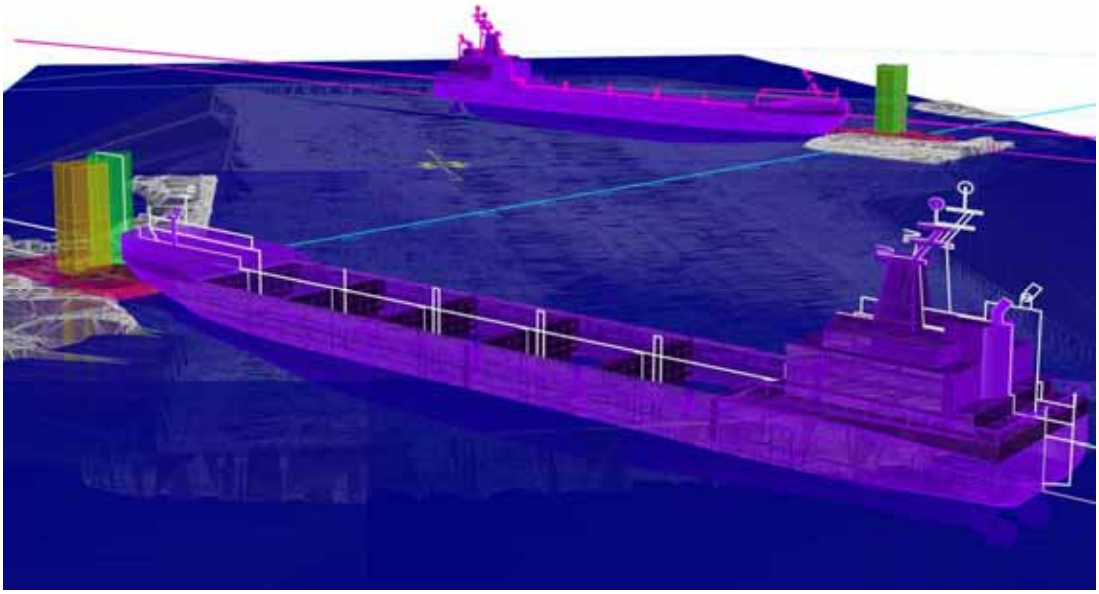
East Tower Location – Received ALDOT Approval

Optimized Main Span Length

- Received ALDOT Approval to place Pier on Madison Street:
  - Allows shortening of Back Span and Main Span
  - Places West Tower Closer to the Bulkhead
  - East Tower Location Unchanged
  - Requires Re-Assessment of Vessel Collision on West Tower

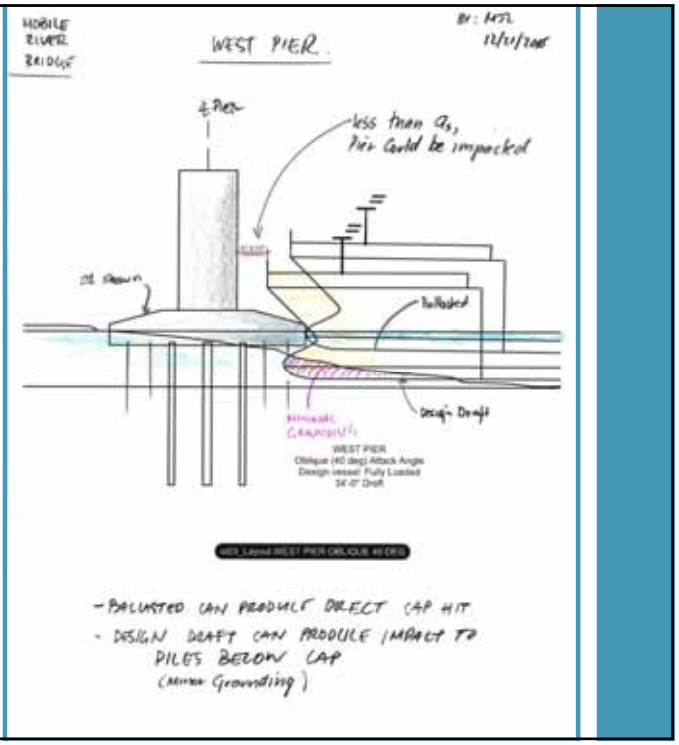


Vessel Collision Review



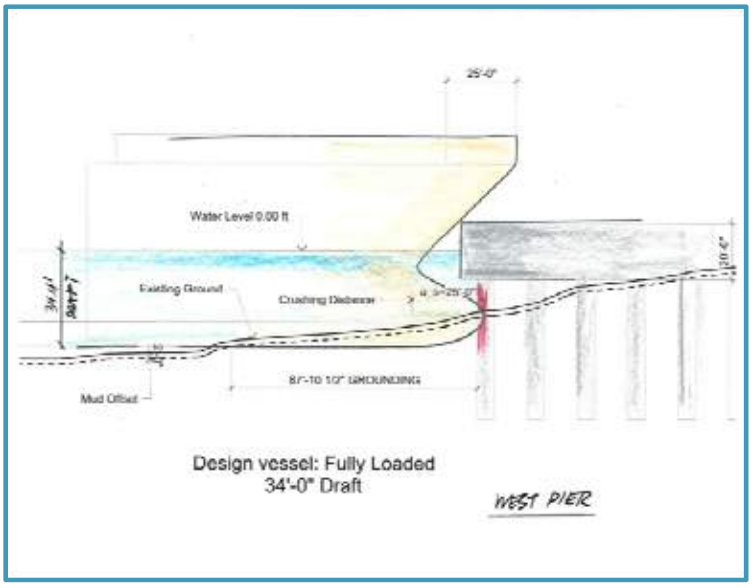
Vessel Collision Review

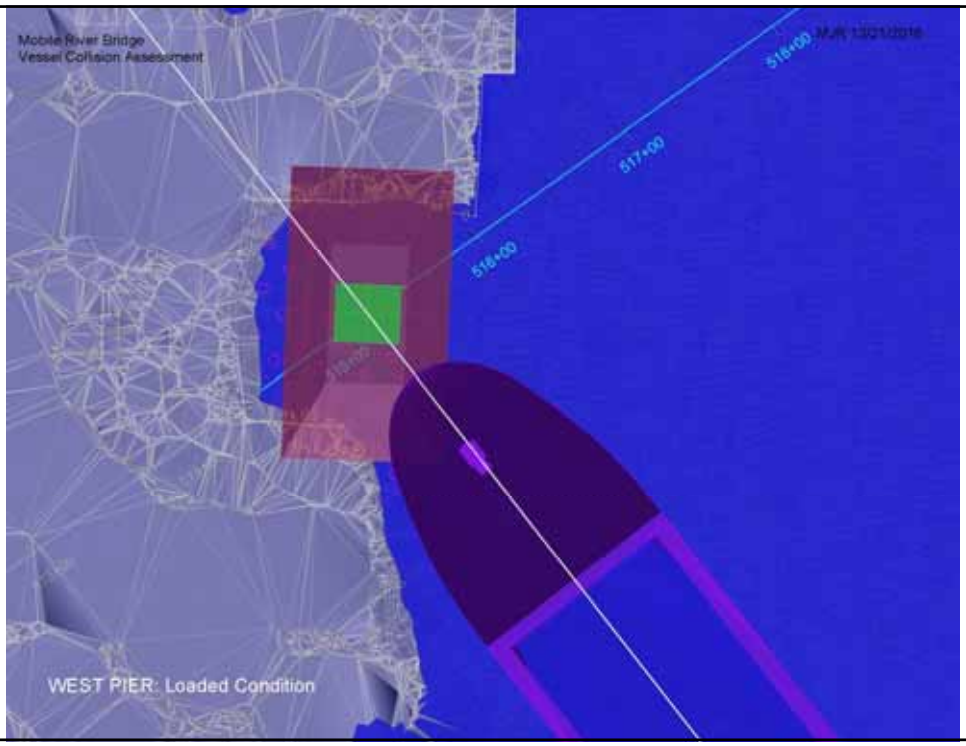
- West Tower:
  - Evaluated Design Vessel's Ballasted Draft and Design Draft



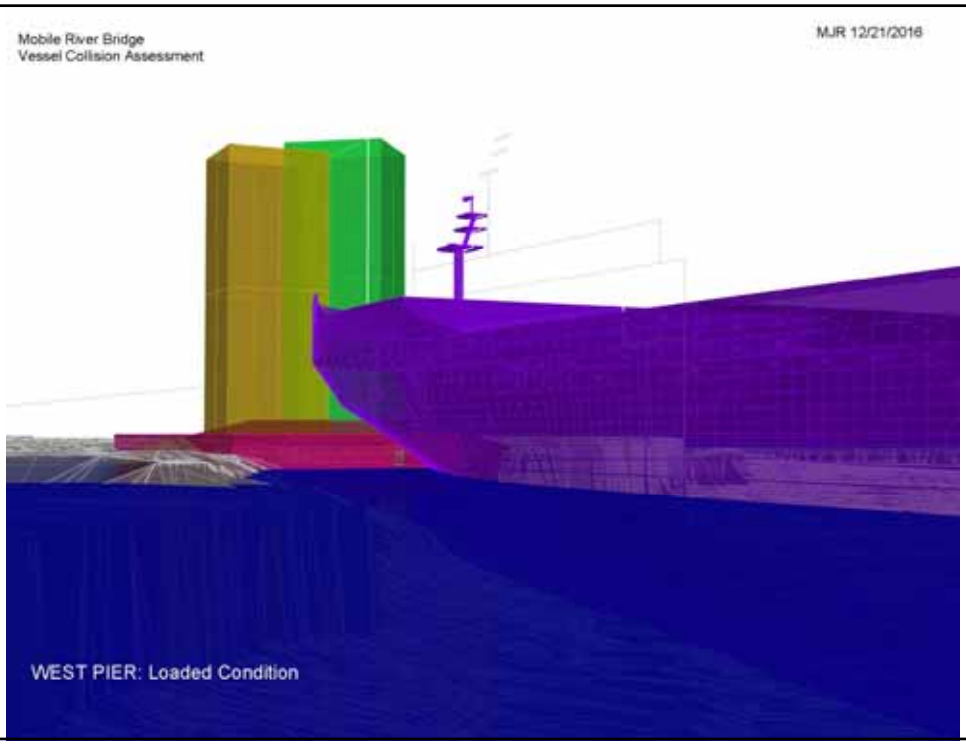
Vessel Collision Review

- West Tower:
  - Design Vessel fully Loaded

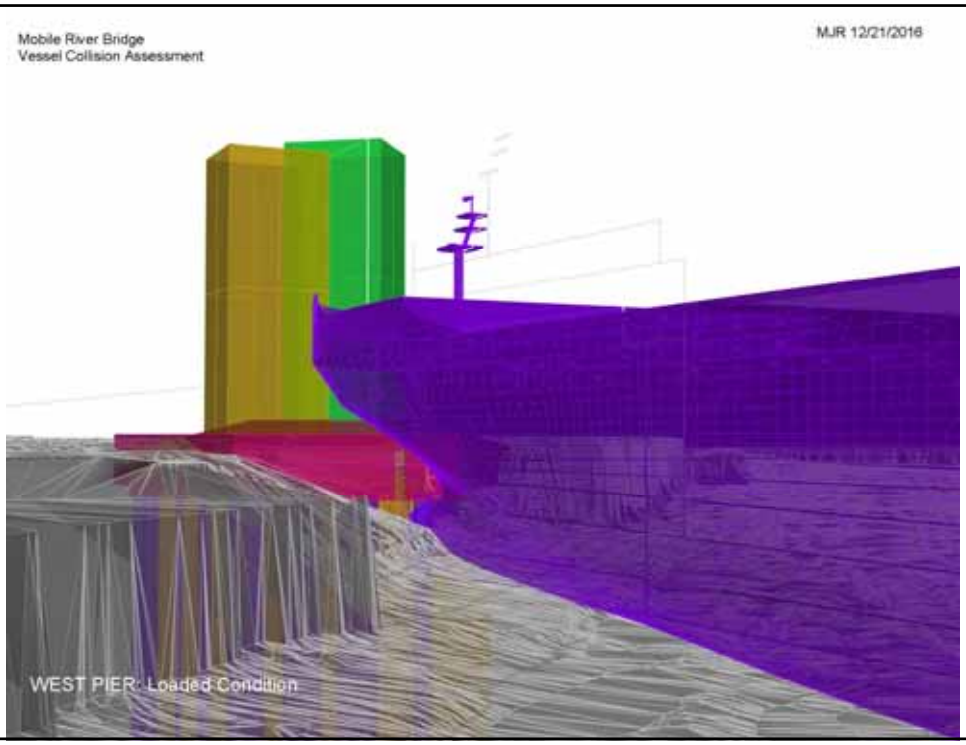




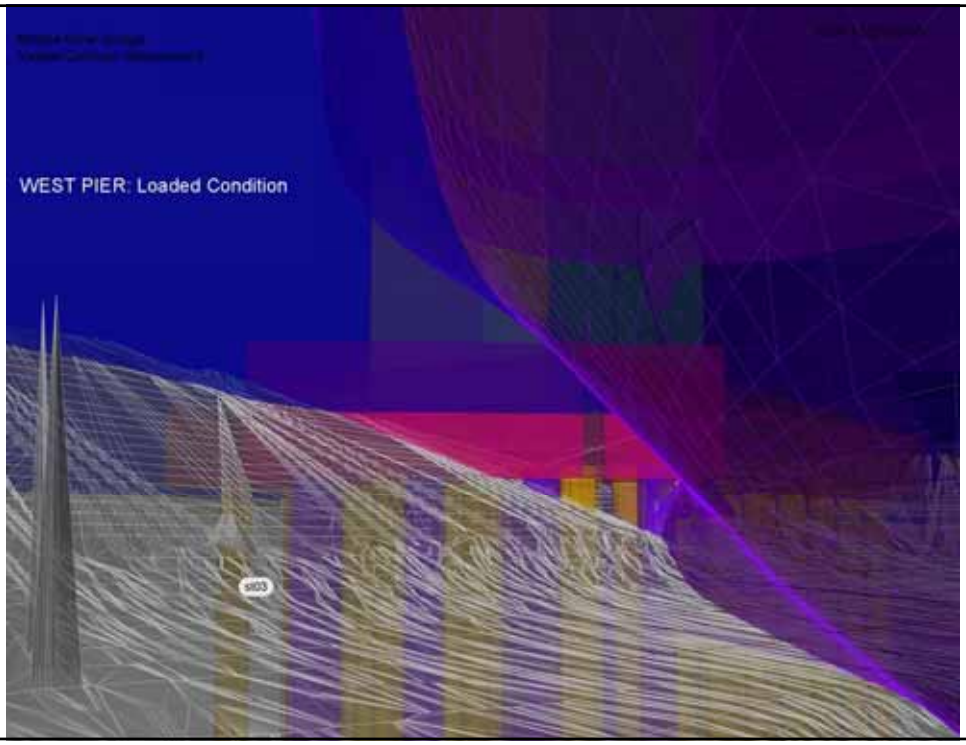
West Pier – Loaded Condition



West Pier – Loaded Condition



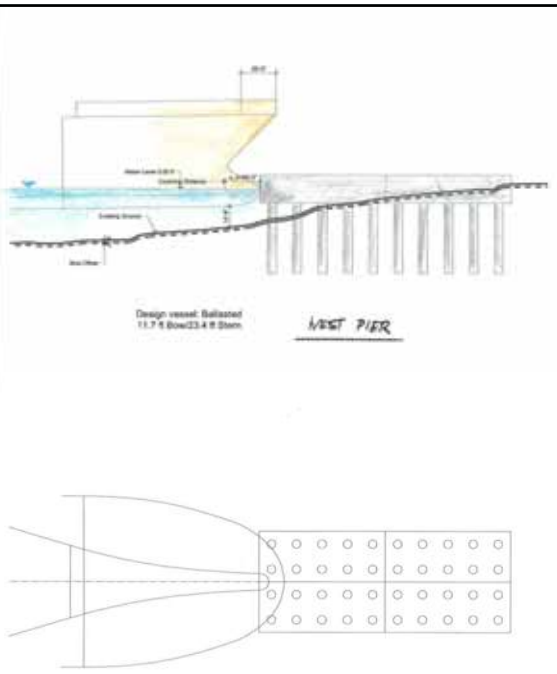
West Pier – Loaded Condition

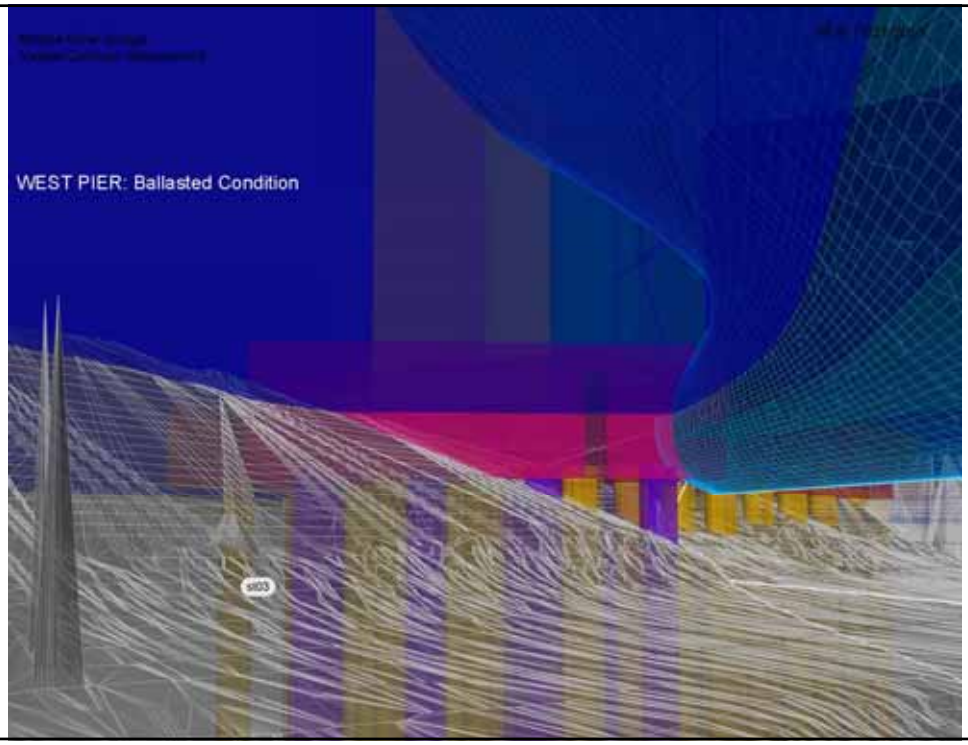


West Pier – Loaded Condition

Vessel Collision Review

- West Tower:
  - Design Vessel Ballasted

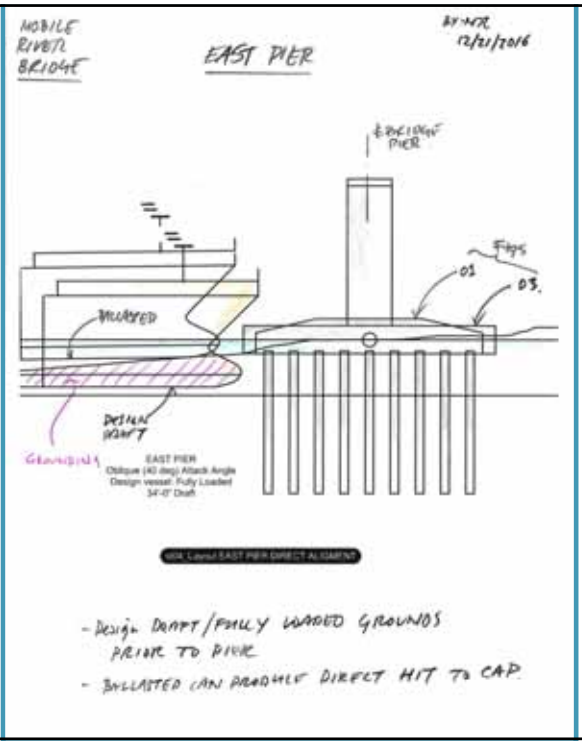




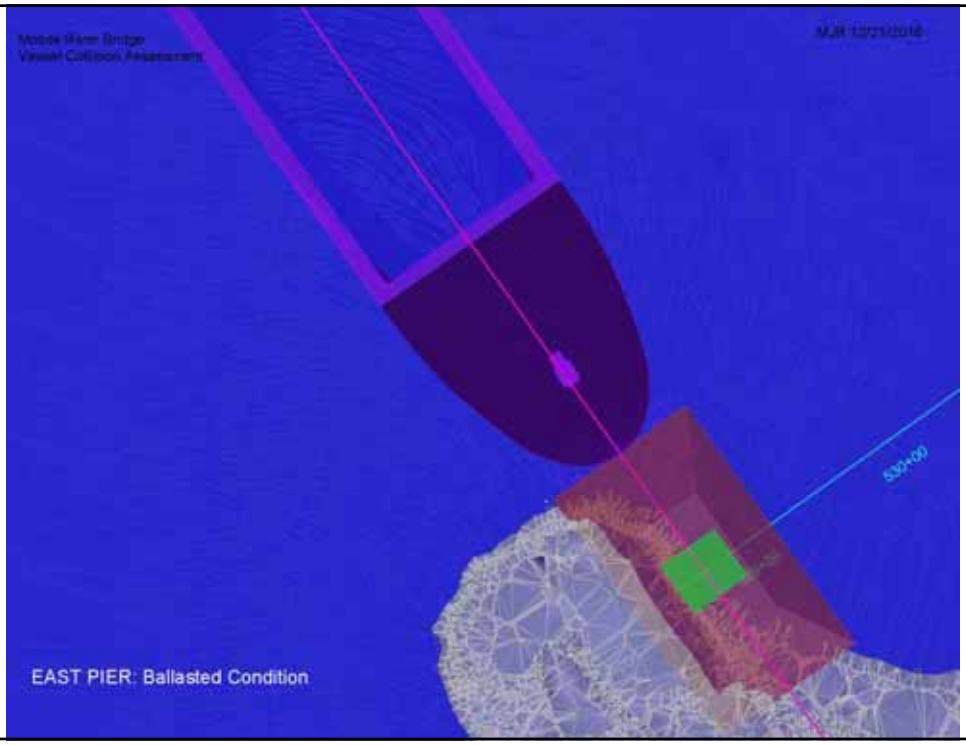
West Pier – Ballasted Condition

Vessel Collision Review

- East Tower:
  - Evaluated Design Vessel's Ballasted Draft and Design Draft

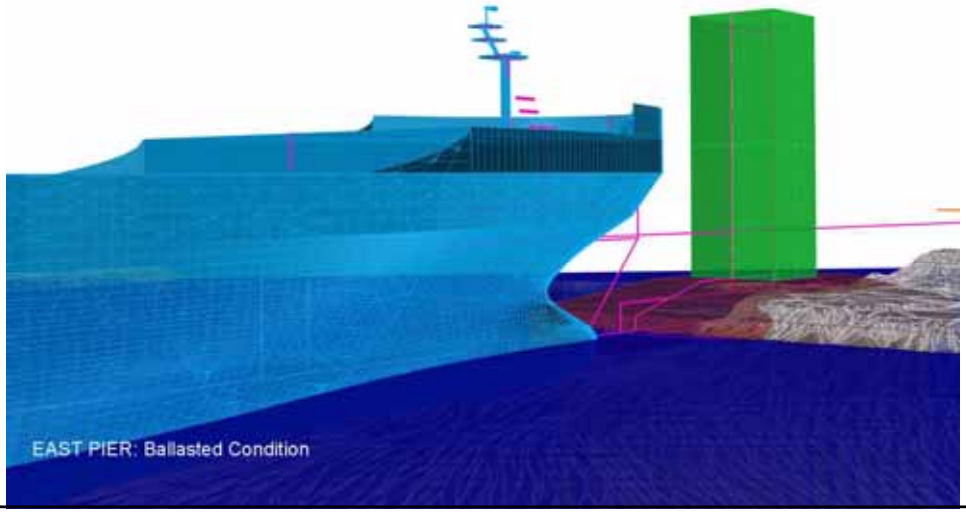


East Pier – Ballasted Condition



Mobile River Bridge
Vessel Collision Assessment

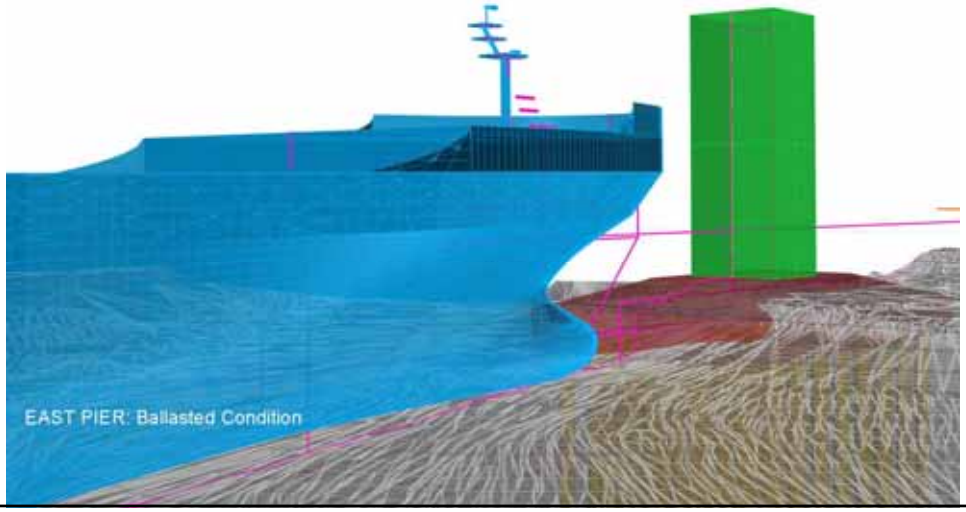
MJR 12/21/2016



East Pier – Ballasted Condition

Mobile River Bridge
Vessel Collision Assessment

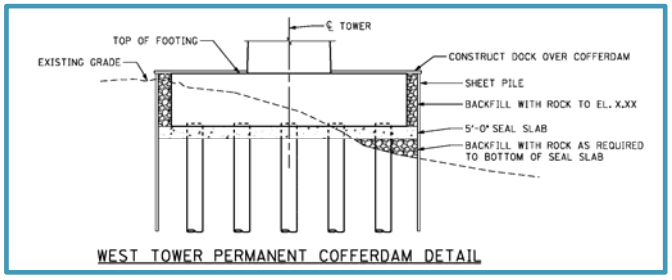
MJR 12/21/2016



East Pier – Ballasted Condition

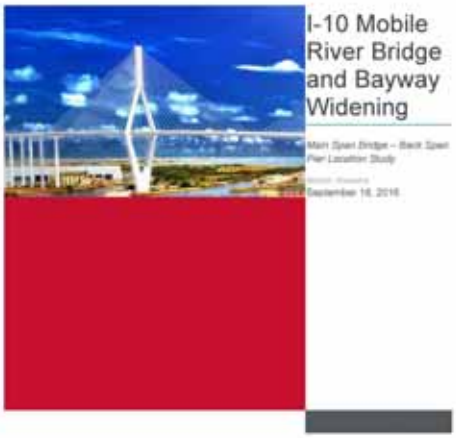
Vessel Collision Review

- Findings using the Design vessel geometry (92,000 DWT, LOA= 870 ft, Design Draft = 34.4', a_s= 25 ft) indicate the following:
  - East Pier:
    -Likely only subject to Ballasted direct hit to the Pile Cap (I.e. no pile damage)
    -Grounding of the design vessel
  - West Pier:
    -Subject to design vessel impacts below the cap
    -Subject to direct cap impacts for ballasted Condition
  - Magnitude of Vessel Collision Load remains unchanged from That Previously reported: 17,000 Kips



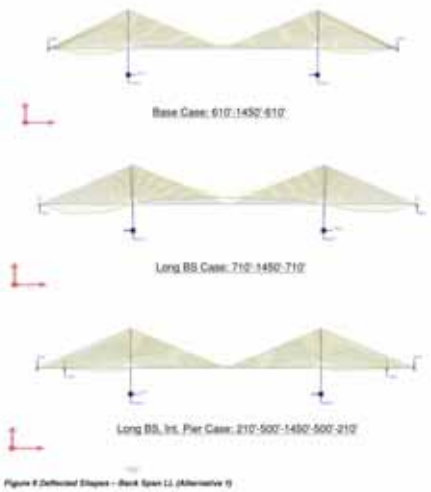
Back Span Location Study

- Back span Length influences the Behavior of the main span unit:
  - Uplift force to be Resisted at Back span Piers
  - Live Load stress range on Stay Cable (important for fatigue considerations)
  - Structure bending Moments
  - Aerodynamic Behavior
- A study was Prepared to Optimize the back span pier layout for Each Alternative



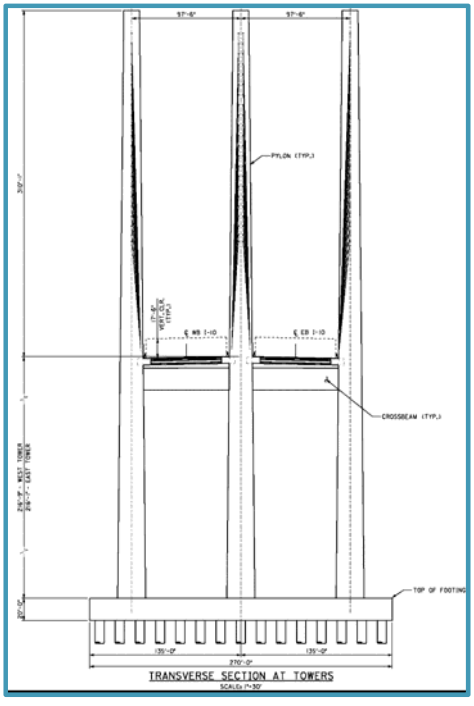
Back Span Location Study

- In General, the Benefit of Reduced Uplift with a Longer span is out-weighed by the Other Effects:
  - Longer Main Span Unit = Higher Construction Cost
  - Increase of Live Load force Range
  - Increase in Bending Moments



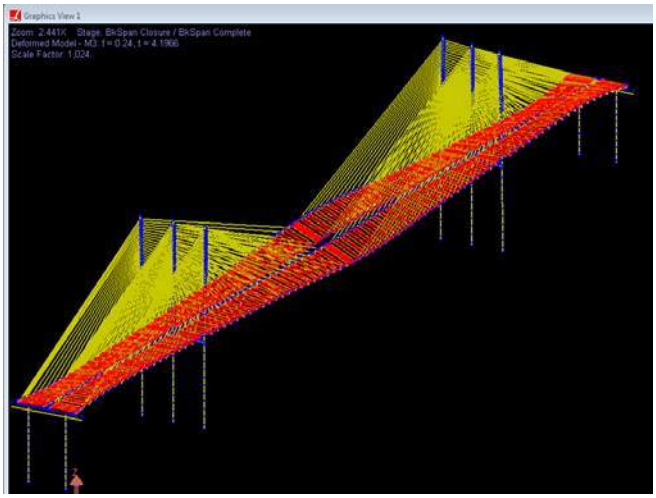
Tower Design Evolution

- Tower Configuration Impacts Structural and Aerodynamic Behavior of Superstructure
- For Alternatives 3 and 7 (Flexible Superstructure) it is Important to Investigate Behavior
- During Preliminary Design an Indication of Acceptable Aerodynamic Behavior is a Ratio of First Torsional Mode Frequency to First Heave Mode Frequency > 1.5



Tower Design Evolution

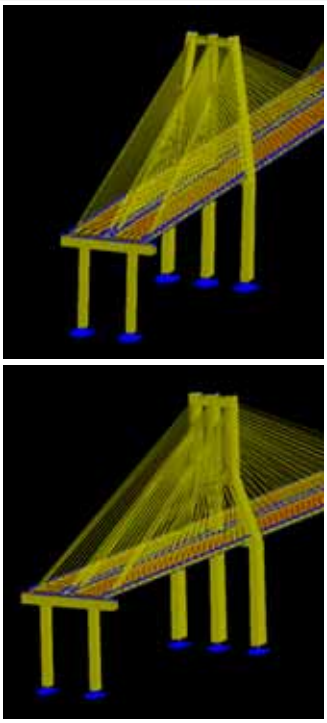
- Developed 3-D model to Calculate Frequencies
- Baseline Model Consisted of 3 Vertical Shafts with no Transverse Struts with Straight Cable Stays
- Ratio of first torsional mode frequency to first vertical heave mode frequency is 1.22 (0.238 / 0.196)



First torsional mode frequency = 0.238 Hz:

Tower Design Evolution

- Prepared Sensitivity Study to better Understand The effects of:
  - Transverse Struts Located at different Upper Tower Heights
  - Angling the Cables in the Transverse Direction
  - Canting Inwards and Outwards the Upper Tower Legs
  - Adding Back Span Intermediate Piers
  - Making the Superstructure Monolithic with the Tower
- For Alternatives 3 and 7 the most Effective Solution is a Tower with the Upper Tower Legs canted Inwards

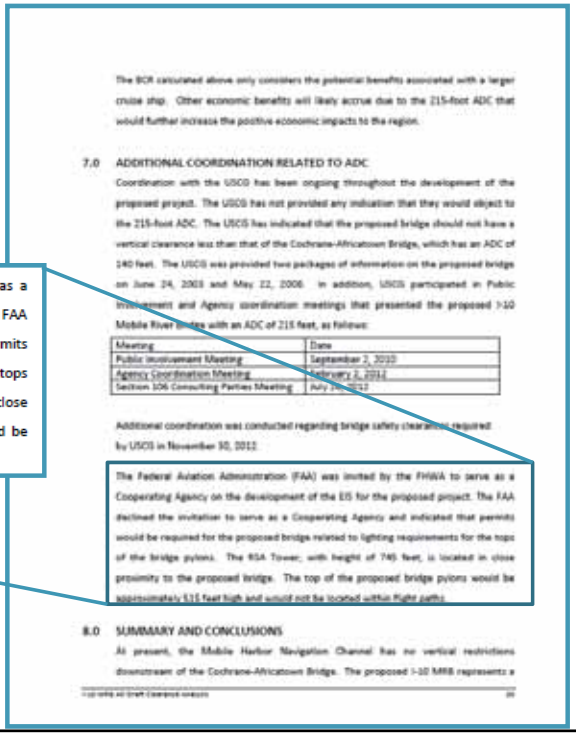


Various Tower Configurations Studies

FAA Requirements

- Coordination with the FAA

The Federal Aviation Administration (FAA) was invited by the FHWA to serve as a Cooperating Agency on the development of the EIS for the proposed project. The FAA declined the invitation to serve as a Cooperating Agency and indicated that permits would be required for the proposed bridge related to lighting requirements for the tops of the bridge pylons. The RSA Tower, with height of 745 feet, is located in close proximity to the proposed bridge. The top of the proposed bridge pylons would be approximately 515 feet high and would not be located within flight paths.



The BCR calculated above only considers the potential benefits associated with a larger cruise ship. Other economic benefits will likely accrue due to the 215-foot ADC that would further increase the positive economic impacts to the region.

7.0 ADDITIONAL COORDINATION RELATED TO ADC

Coordination with the USCG has been ongoing throughout the development of the proposed project. The USCG has not provided any indication that they would object to the 215-foot ADC. The USCG has indicated that the proposed bridge should not have a vertical clearance less than that of the Cochrane-Africatown Bridge, which has an ADC of 140 feet. The USCG was provided two packages of information on the proposed bridge on June 24, 2003 and May 22, 2006. In addition, USCG participated in Public Involvement and Agency coordination meetings that presented the proposed I-10 Mobile River Bridge with an ADC of 215 feet, as follows:

Meeting	Date
Public Involvement Meeting	September 2, 2010
Agency Coordination Meeting	February 2, 2012
Section 106 Consulting Parties Meeting	July [illegible], 2012

Additional coordination was conducted regarding bridge safety clearances required by USCG in November 30, 2012.

The Federal Aviation Administration (FAA) was invited by the FHWA to serve as a Cooperating Agency on the development of the EIS for the proposed project. The FAA declined the invitation to serve as a Cooperating Agency and indicated that permits would be required for the proposed bridge related to lighting requirements for the tops of the bridge pylons. The RSA Tower, with height of 745 feet, is located in close proximity to the proposed bridge. The top of the proposed bridge pylons would be approximately 515 feet high and would not be located within flight paths.

8.0 SUMMARY AND CONCLUSIONS

At present, the Mobile Harbor Navigation Channel has no vertical restrictions downstream of the Cochrane-Africatown Bridge. The proposed I-10 MRB represents a

FAA Requirements

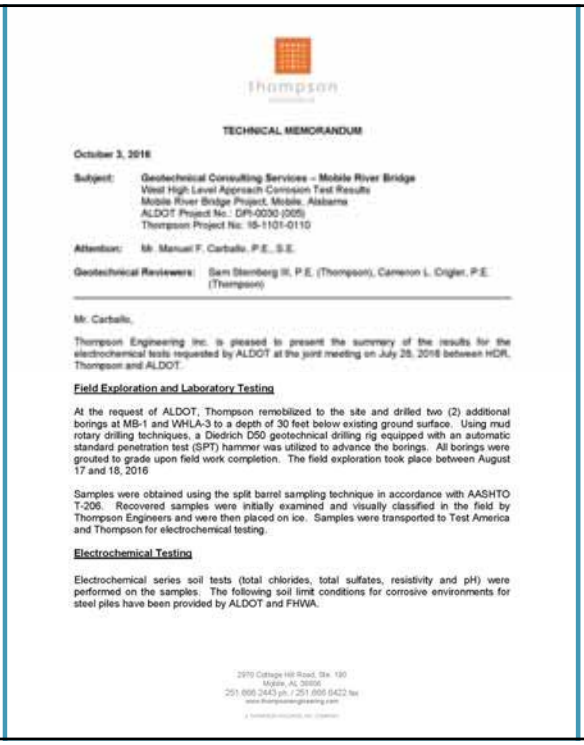
- The project will need an analysis per Title 14-Aeronautics and Space Code of Federal Regulation (CFR) Part 77.9 Construction or Alteration Requiring Notice
- Per Part 77.17 any structure taller than 499 AGL is an obstruction. Structures nearer the airport must be shorter and can be taller the farther they are from the airport
- To receive a "no-hazard" determination especially for structures <u>taller</u> than 499 feet need to show the FAA that the bridge towers, lights, etc. do not cause any aviation hazard

Any construction or alteration that is more than 200 ft. AGL at its site

A height of 499 feet AGL at the site of the object

Corrosion Test Results

- During the September 14, 2016 West Side Foundation meeting ALDOT inquired about the site's Environmental Classification for Steel Piles Corrosion
- Thompson Engineering performed a limited Field and Laboratory Testing Program to Establish an Initial Assessment of the Project Site
- Only Soil Samples on Land were taken and Analyzed



Thompson

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 Stemberg 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

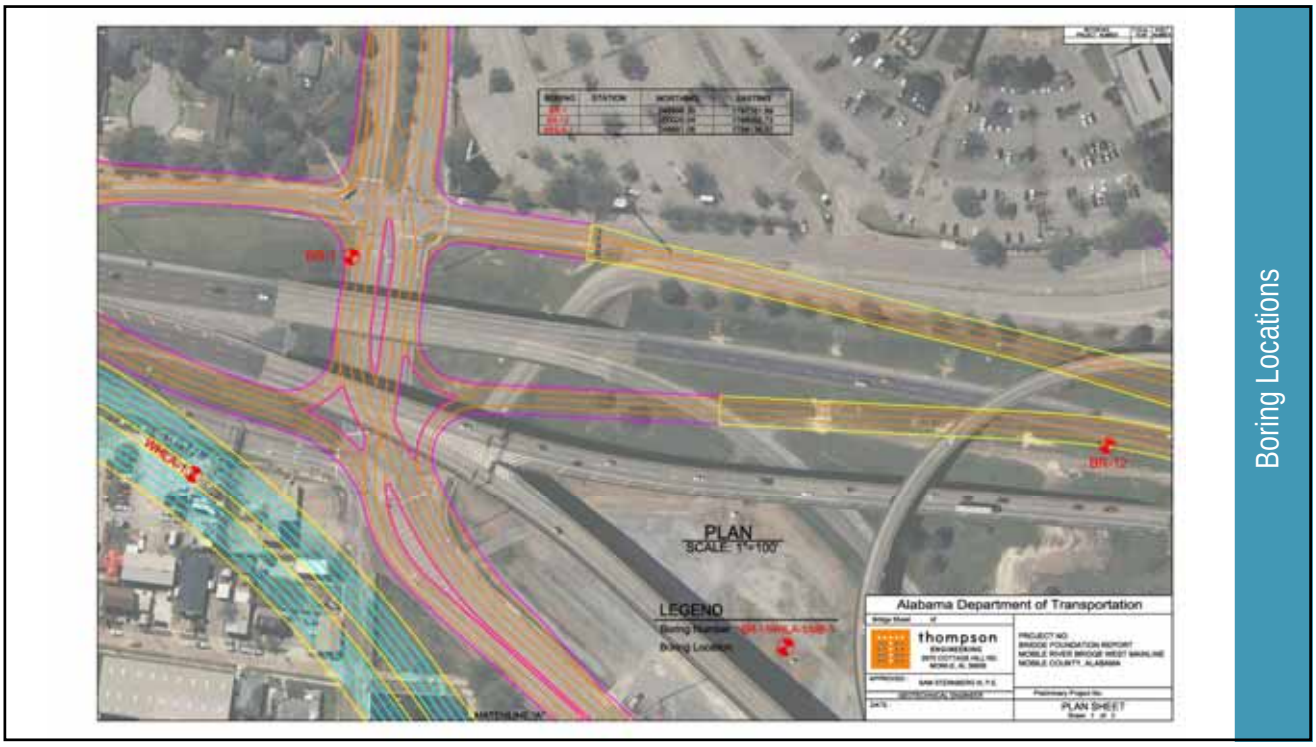
At the request of ALDOT, Thompson remobilized to the site and drilled two (2) additional borings at MB-1 and WHLA-3 to a depth of 30 feet below existing ground surface. Using mud rotary drilling techniques, a Diedrich D50 geotechnical drilling rig equipped with an automatic standard penetration test (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.

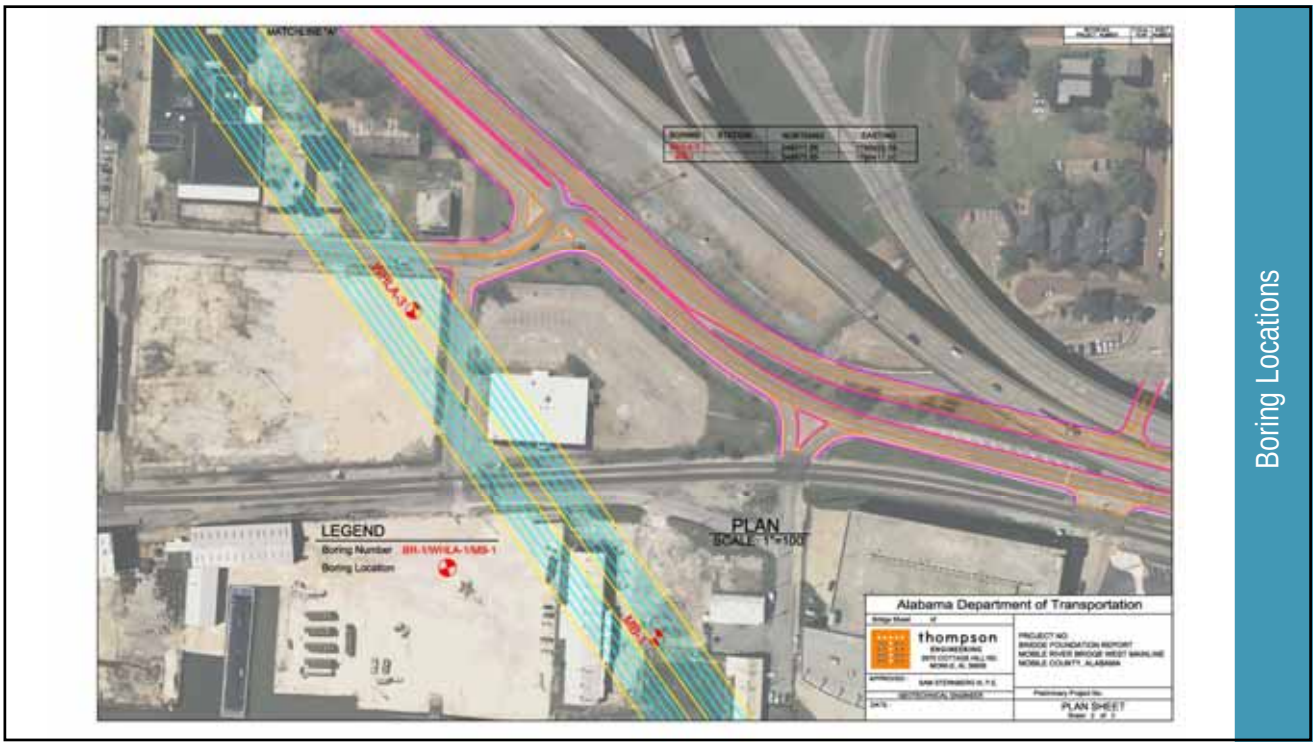
Electrochemical Testing

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.

2970 Cottage Hill Road, Ste. 190
Mobile, AL 36606
251.666.2443 ph / 251.666.6422 fax
www.thompsonengineering.com



Boring Locations



Boring Locations

Corrosion Test Results

- The Samples were Classified in Accordance with ALDOT and FHWA Guidelines
- The Majority of the Samples are classified as Non-Aggressive

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 1,00 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	26		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		Aggressive	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

Corrosion Test Results

- During the Next Phase of Design Additional samples will be Collected including Mobile River water samples
- Scour depth Needs to be Considered for the East Tower (Unknown at this time)
- On the Basis of these Initial Corrosion Test Results Recommend that Steel Piles be further Considered for the Main Span and High Level Approach Spans

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.

Corrosion Mitigation Approach for Steel Piles

- Approach:
  - Establish Environmental Classification (non-corrosive, corrosive)
  - Establish Corrosion Rate
    - Based on Environmental Classification, elevation in soil (surface, buried, partially buried, etc.), physical soil characteristics, water exposure (tidal, submerged, splash zone, etc.)
  - Establish Service Life
  - Calculate Additional Sacrificial Thickness (Corrosion Allowance)

$ESL = T/CR$	Single-Side Corrosion
$ESL = T/2CR$	Double-Sided Corrosion
$ESL = T/(CR_1 + CR_2)$	Diff. Corrosion Rates

Where:

ESL = Estimated Service Life (years)

T = Allowable Loss of Metal (in.)

CR = Corrosion Rate (in./yr.)

Revised Bridge Layouts

- Revised The Bridge Layouts on the Basis of the Analytical Refinements
- The Revised Layouts are the Basis for the Updated Quantities and Construction Costs
- Developed a Variant of Alternative 1 to be Constructed in Cantilever – Named Alternative 1A
- Used Site Specific Wind Analysis Loads for Analytical Studies

Mobile River Crossing

Mobile, AL, USA

Draft Report

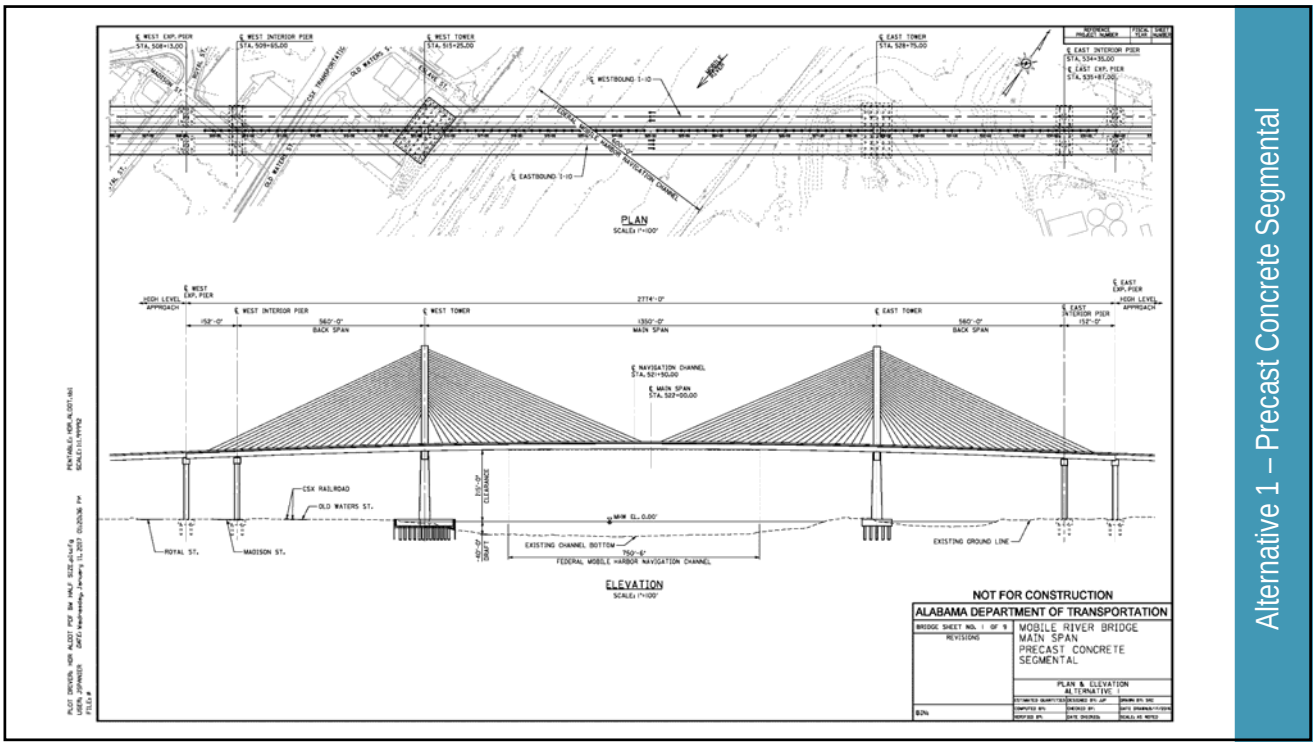
Wind Climate Analysis

RWDI # 1600368

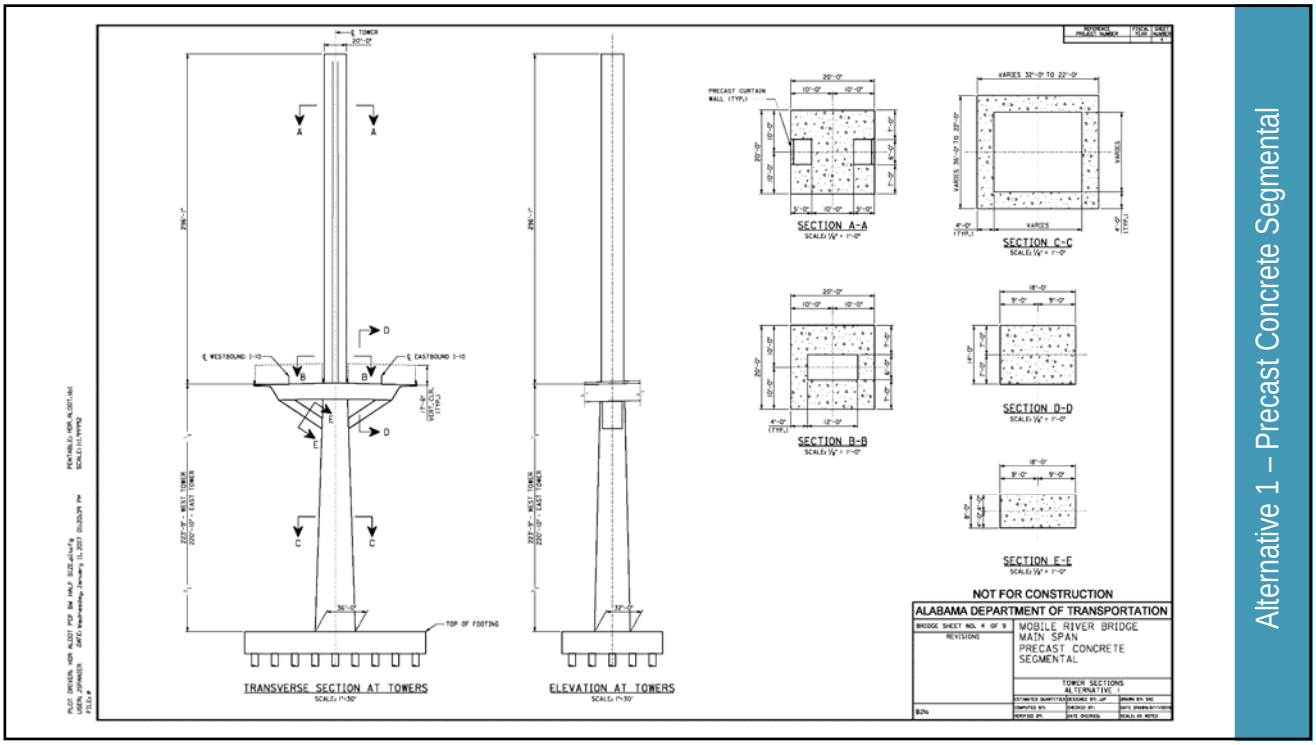
July 11, 2016

SUBMITTED TO

SUBMITTED BY



Alternative 1 – Precast Concrete Segmental



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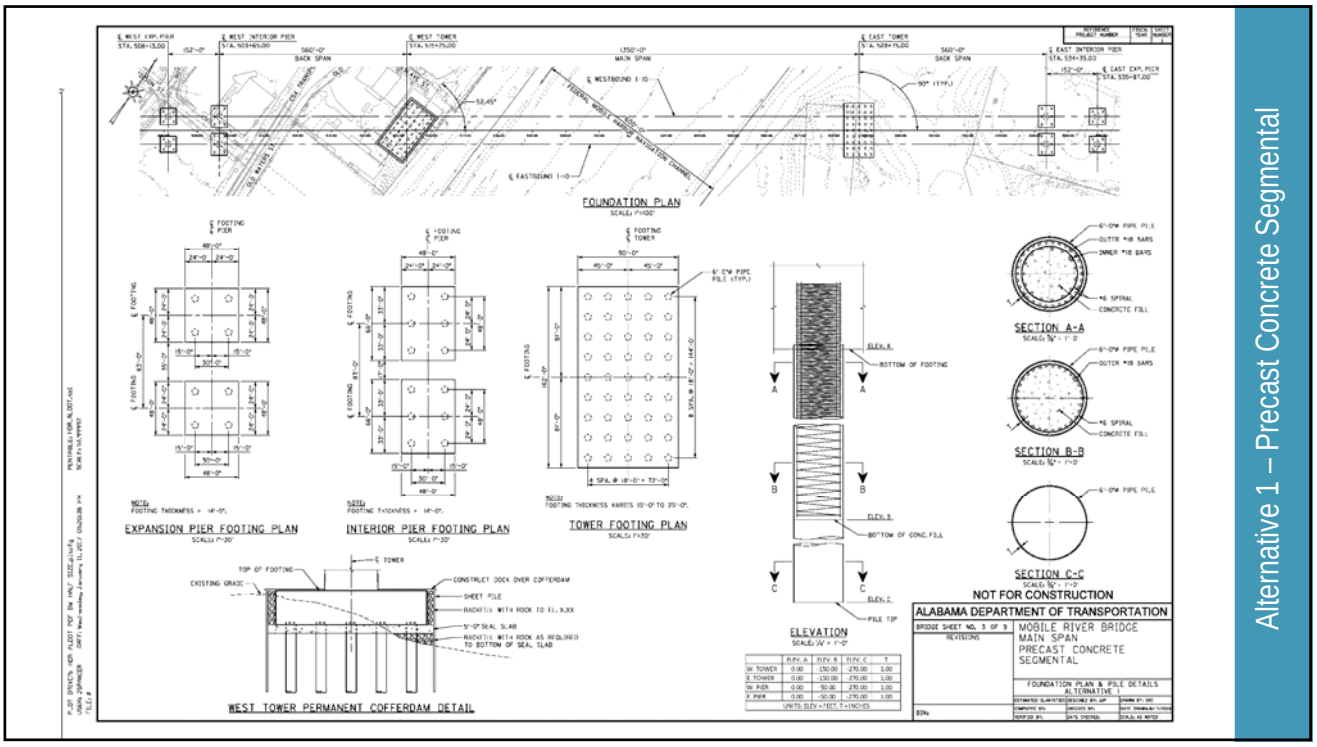

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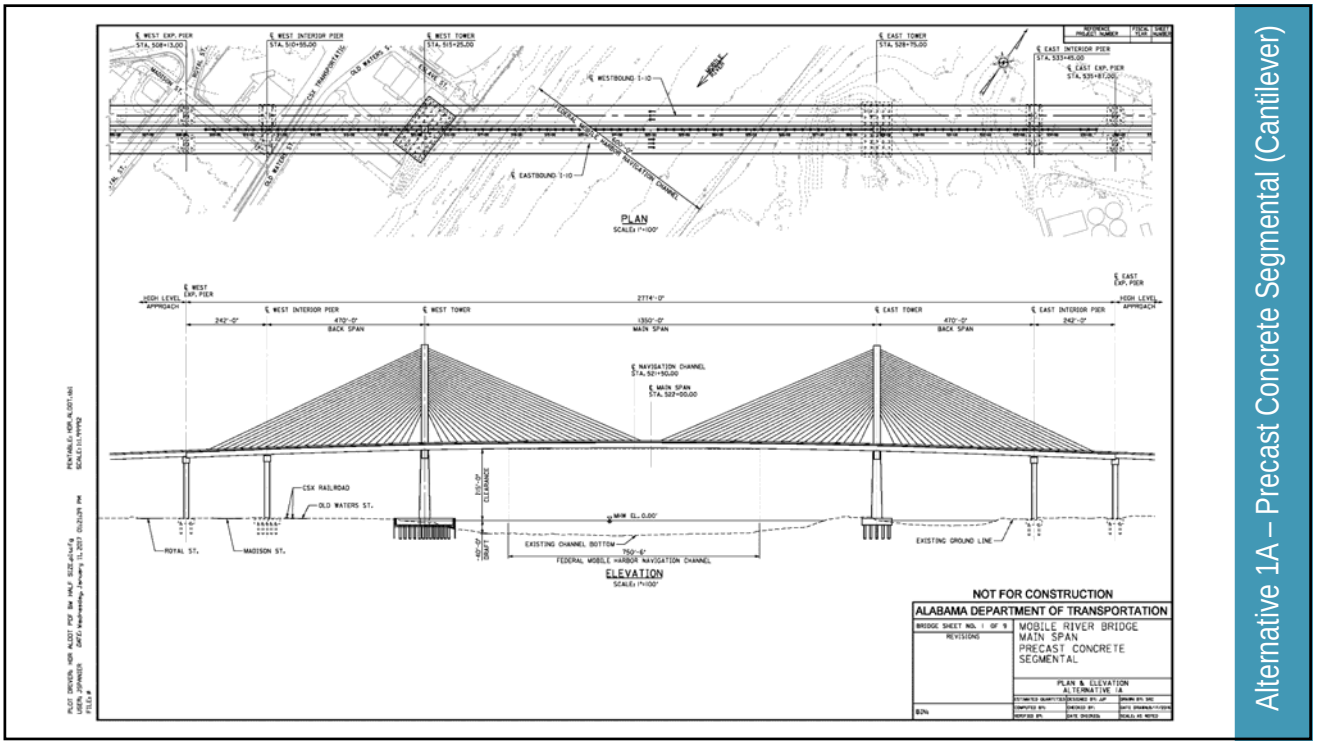
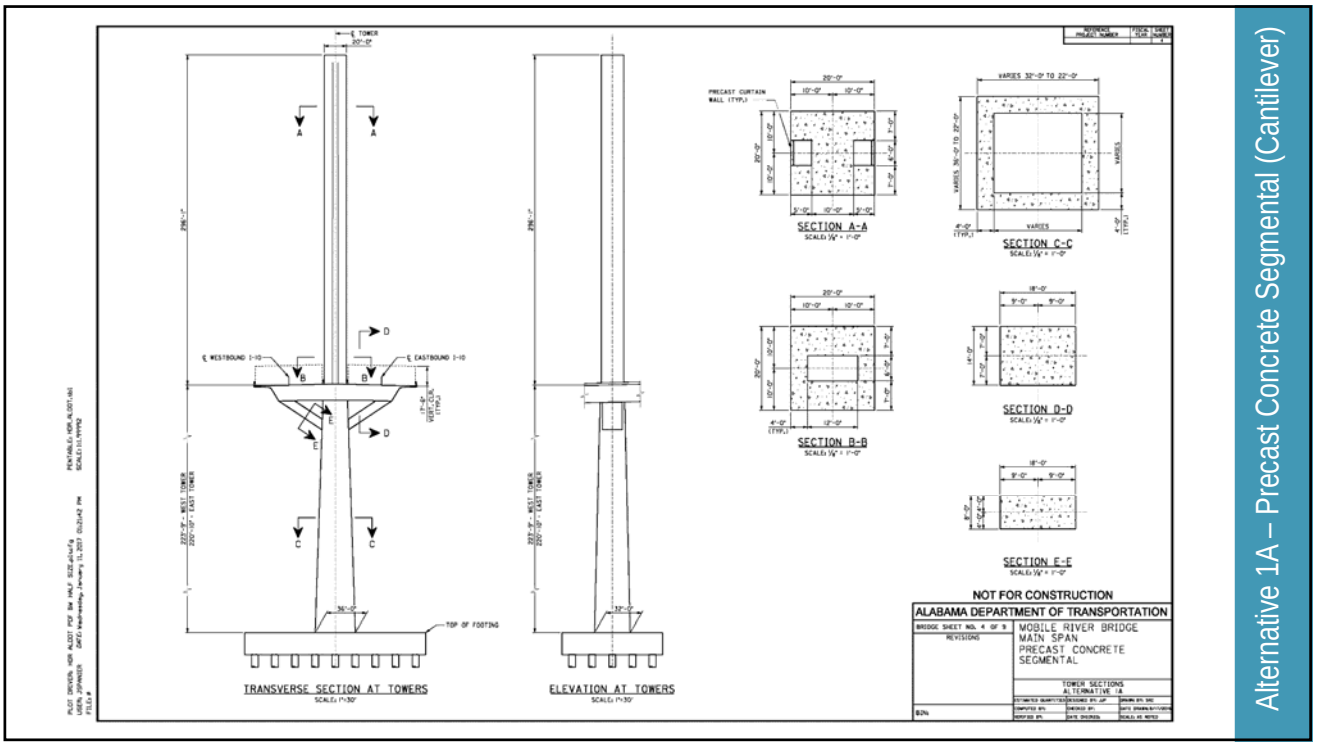
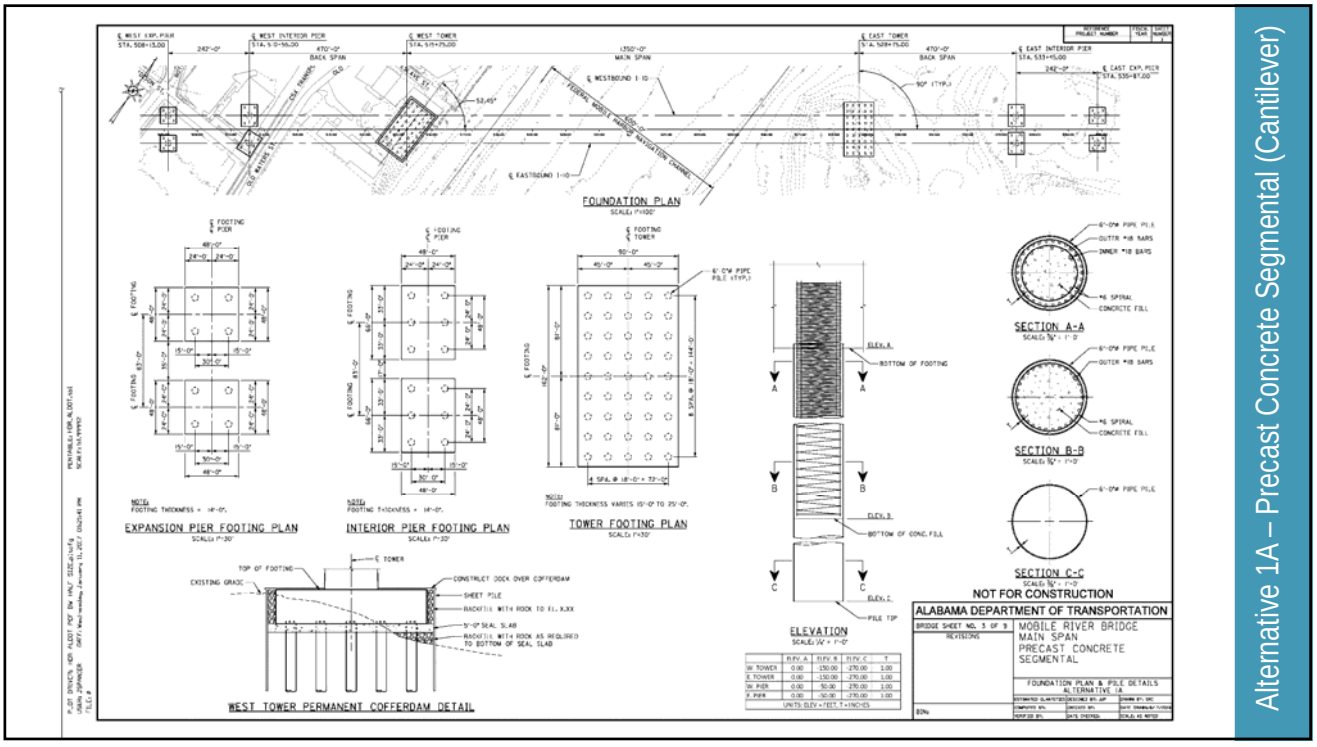


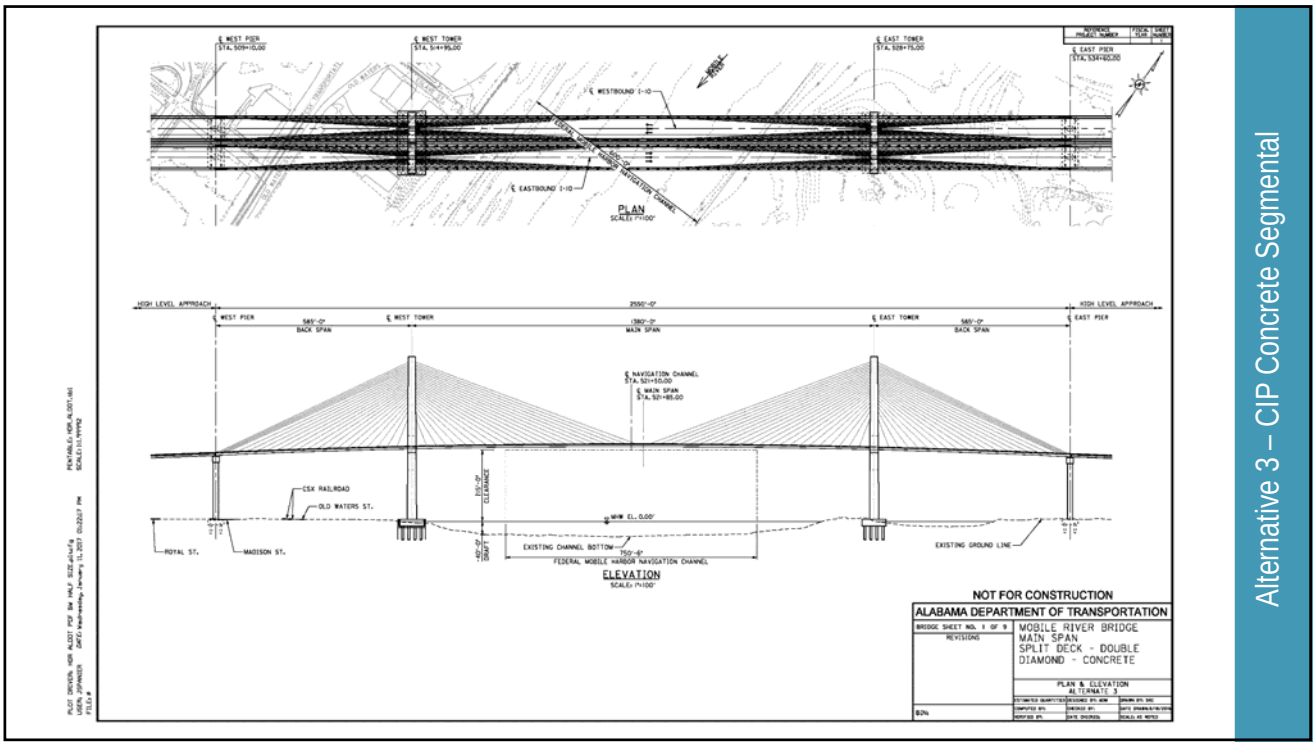
Alternative 1A – Precast Concrete Segmental (Cantilever)



Alternative 1A – Precast Concrete Segmental (Cantilever)



Alternative 1A – Precast Concrete Segmental (Cantilever)



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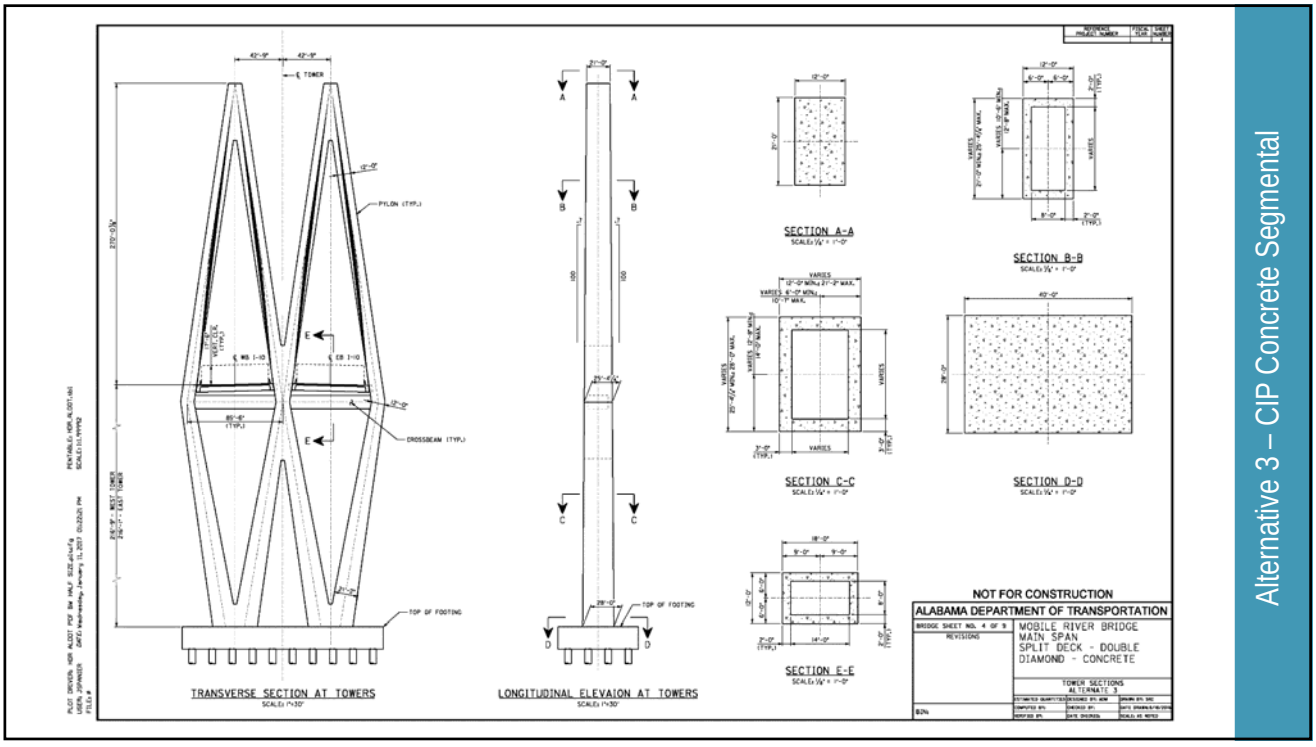

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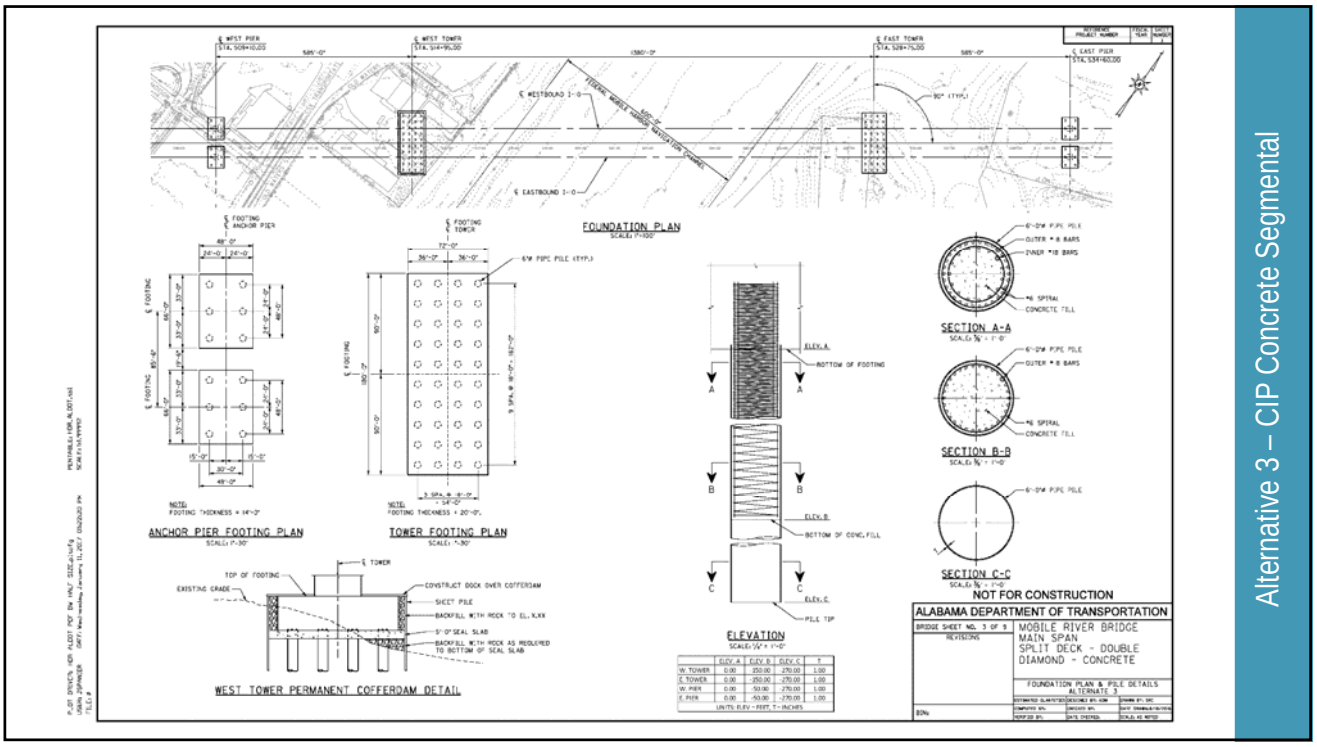

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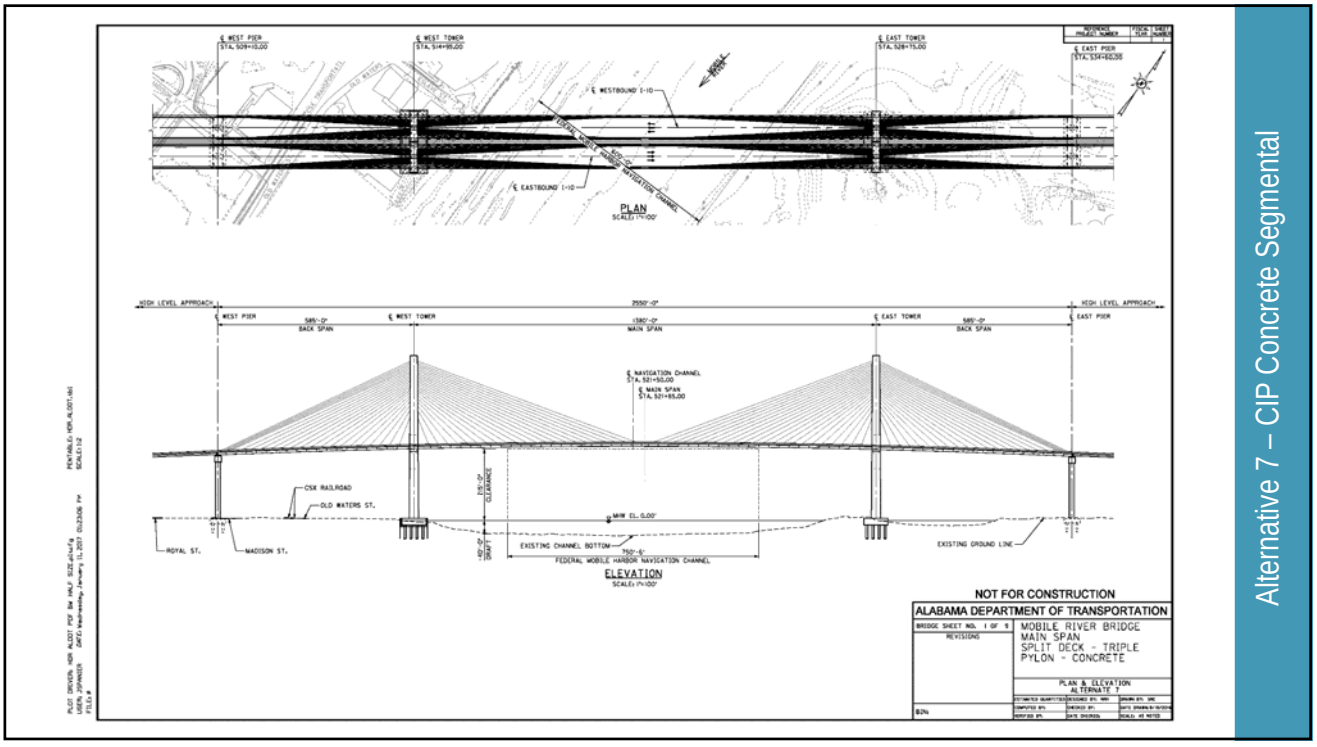

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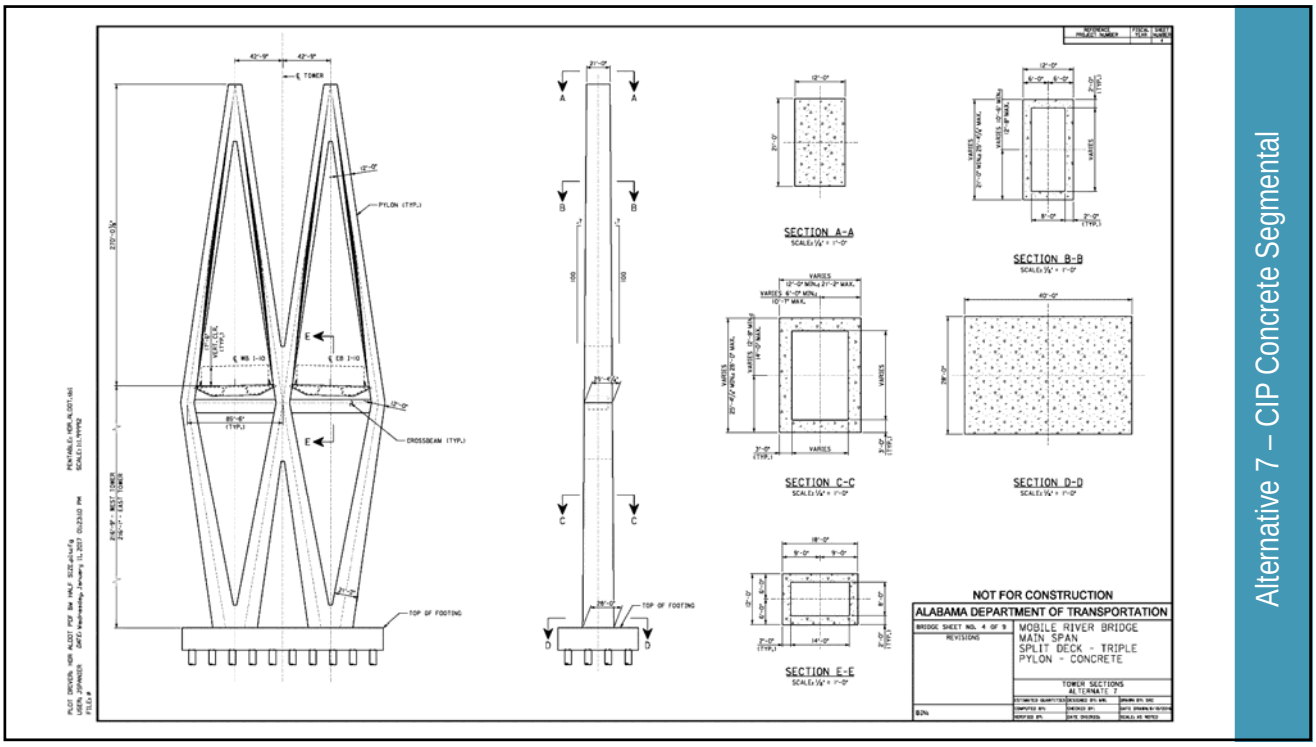

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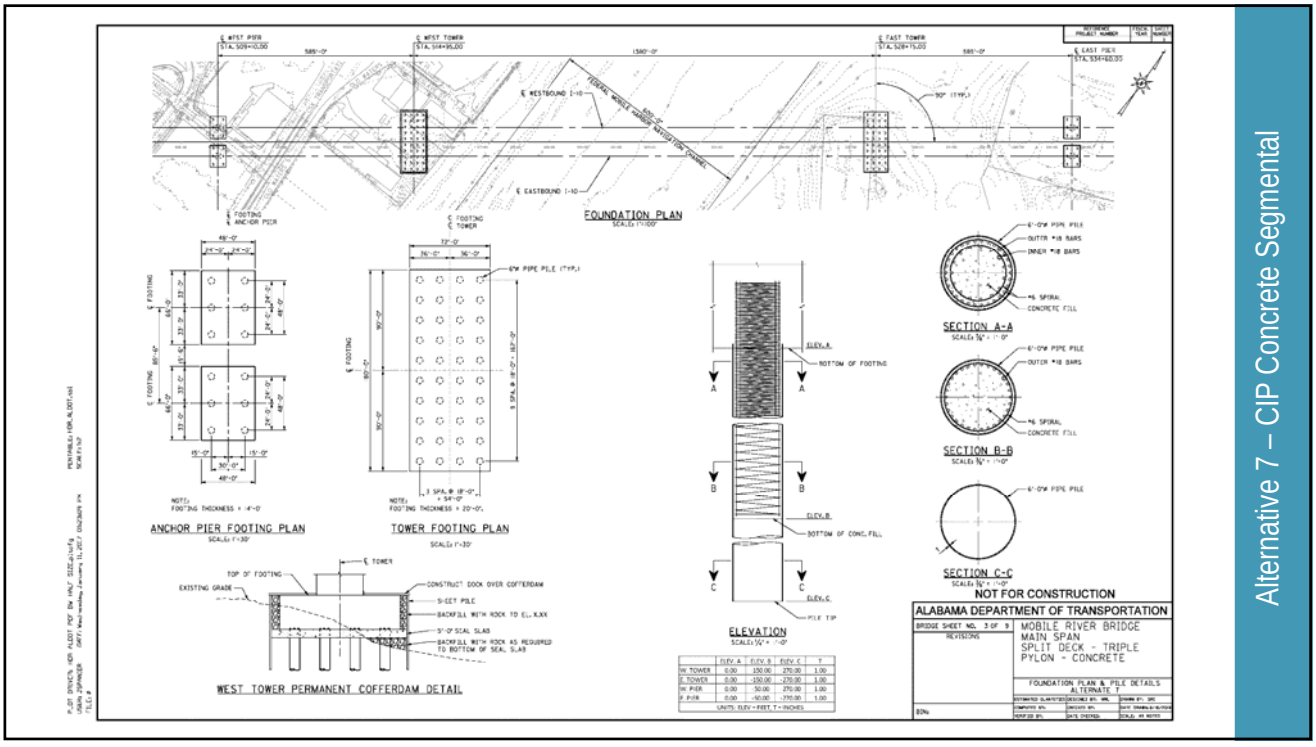

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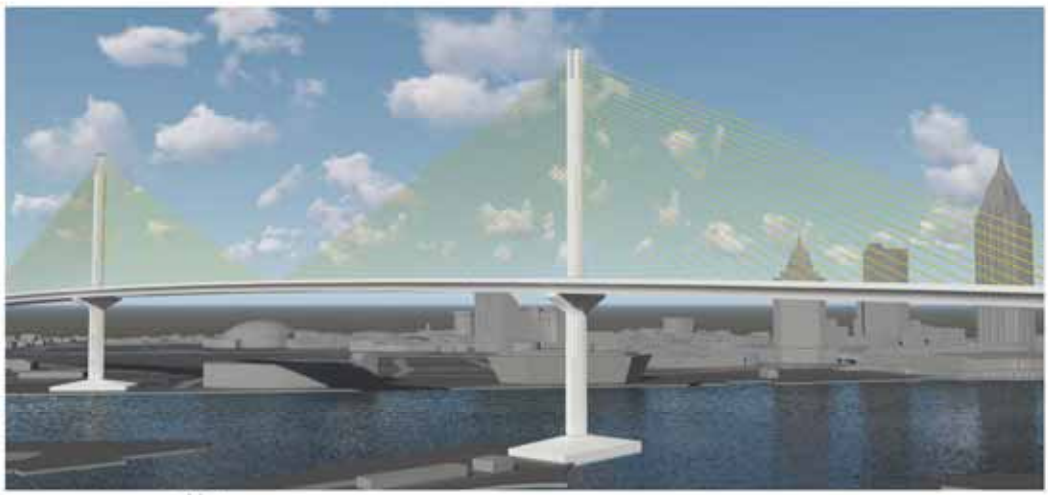


Typical Cross Section at Cable Anchor

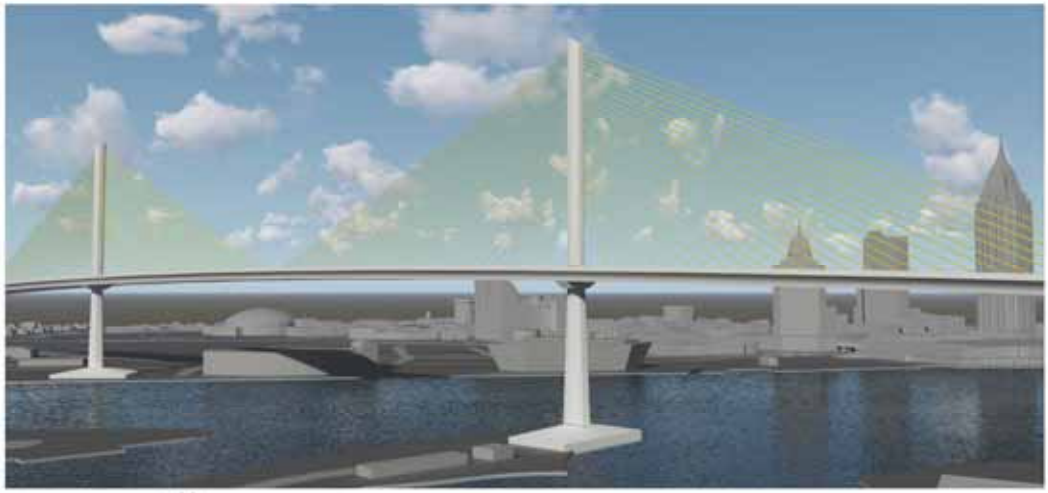
TOUCHSTONE ARCHITECTURE

title: Alt_01 - Art Deco Pylon
date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

sheet: g101

Alternatives 1 and 1A – Precast Concrete Segmental



Typical Cross Section at Cable Anchor

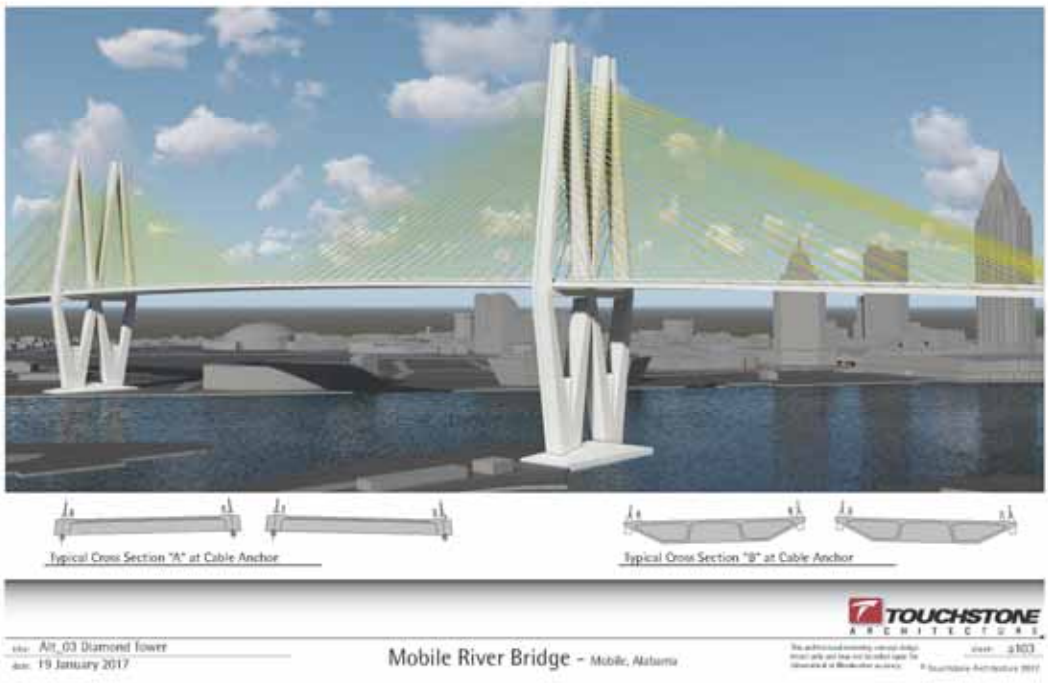
TOUCHSTONE ARCHITECTURE

title: Alt_01 - Oval Pylon
date: 19 January 2017

Mobile River Bridge - Mobile, Alabama

sheet: g102

Alternatives 1 and 1A – Precast Concrete Segmental



Typical Cross Section "A" at Cable Anchor

Typical Cross Section "B" at Cable Anchor

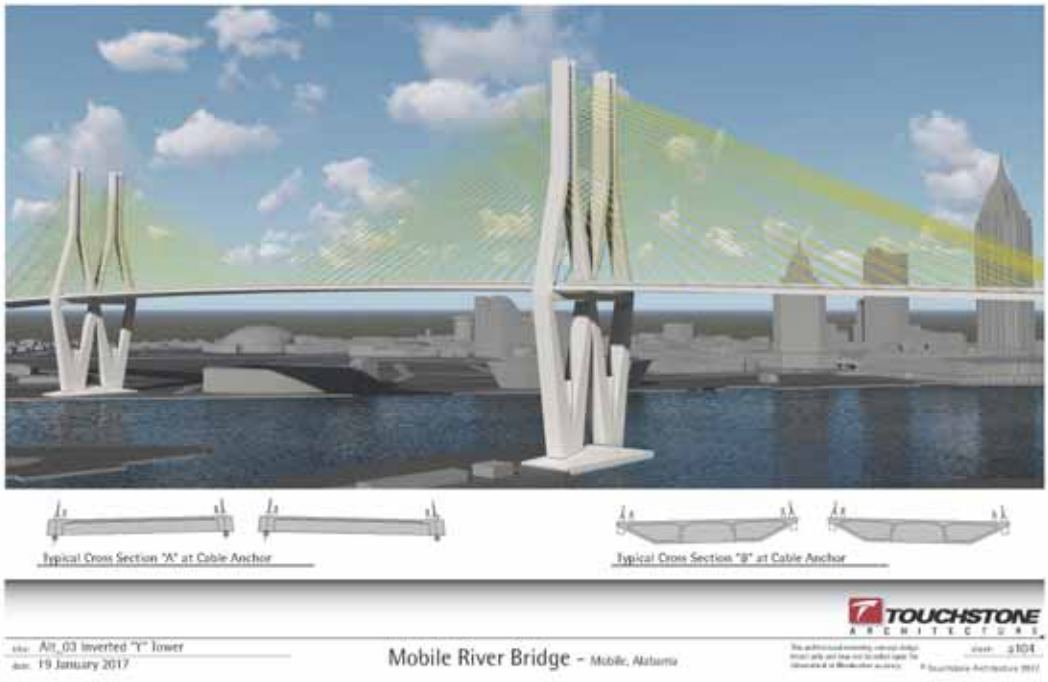
TOUCHSTONE ARCHITECTURE

Alt_03 Diamond Tower
19 January 2017

Mobile River Bridge - Mobile, Alabama

g103

Alternatives 3 and 7 – CIP Concrete Segmental



Typical Cross Section "A" at Cable Anchor

Typical Cross Section "B" at Cable Anchor

TOUCHSTONE ARCHITECTURE

Alt_03 Inverted "Y" Tower
19 January 2017

Mobile River Bridge - Mobile, Alabama

g104

Alternatives 3 and 7 – CIP Concrete Segmental

04 Main Span Concepts - Refined Evaluation

Constructibility

- Prepared Construction Sequence Schemes for the four Alternatives (1, 1A, 3 and 7)
- These Construction Schemes Became the Basis for Estimating Construction Durations

Construction Sequence - Alternative 1 (Precast Concrete Segmental)

Construction Sequence - Alternative 1 (Precast Concrete Segmental)

Setup Precast yard

Deliver Precast Segment to Project Site

Alternative 1 – Precast Concrete Segmental

Construct Back Span Utilizing Temporary Piers

Use Completed Back Span to Deliver Main Span Segments

Alternative 1 – Precast Concrete Segmental

Use Deck Mounted Cranes to Install Precast Segments and Concrete Frames

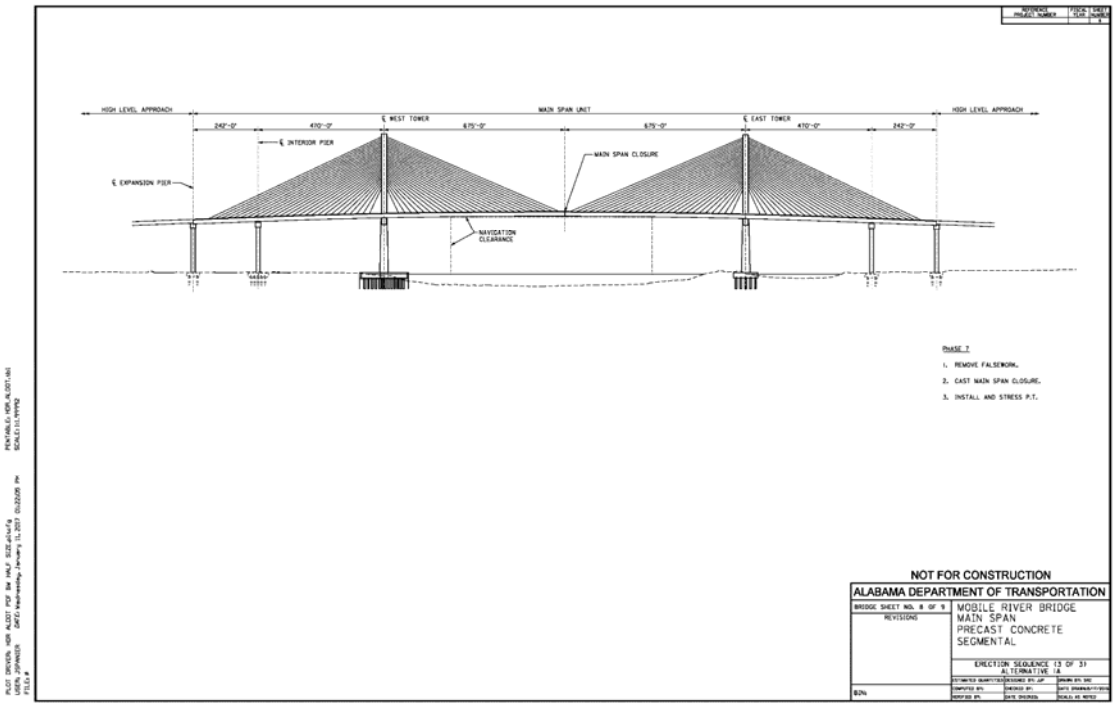
Alternative 1 – Precast Concrete Segmental

Maintain Navigation Channel Open

Alternative 1 – Precast Concrete Segmental

Construction Sequence - Alternative 1A (Precast Concrete Segmental)

Construction Sequence - Alternative 1A (Precast Concrete Segmental)

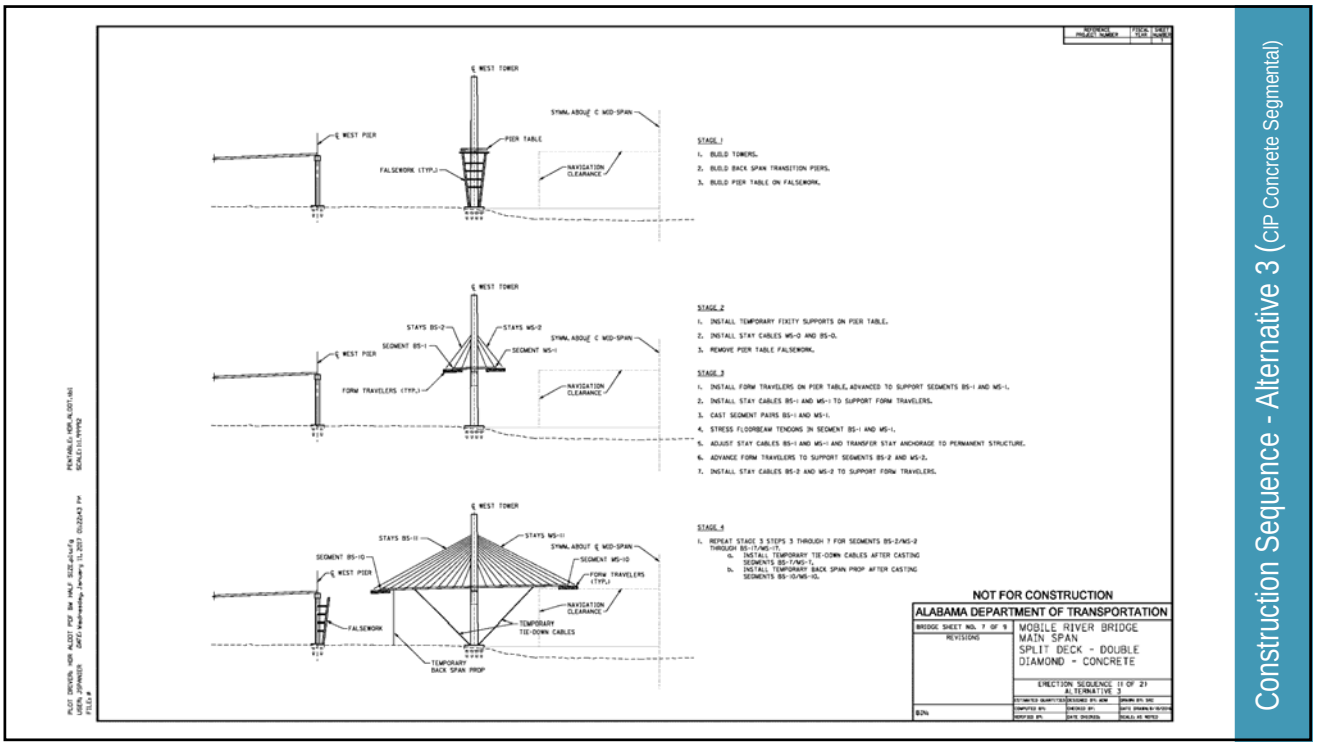


Construction Sequence - Alternative 1A (Precast Concrete Segmental)

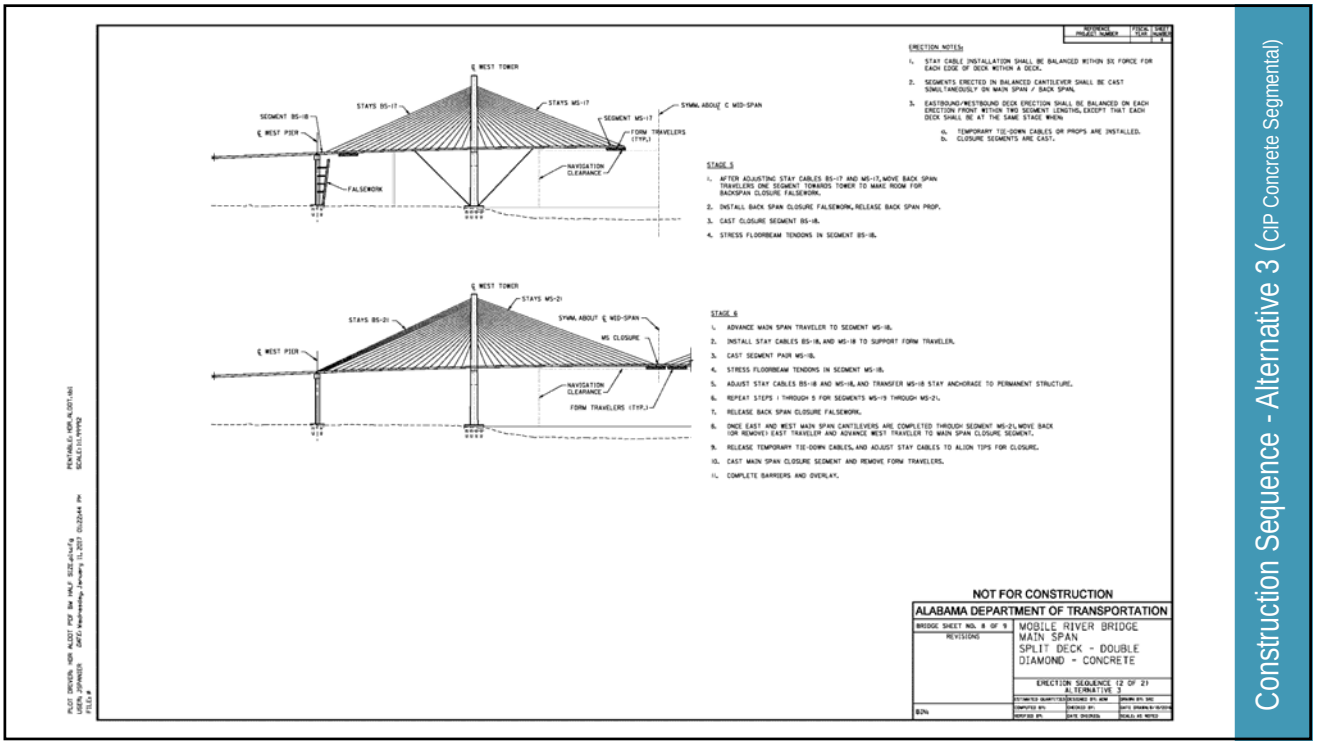
Main Span is Built in Cantilever from the Towers

Precast Segments are Lifted Utilizing a Beam & Winch

Alternative 1A – Precast Concrete Segmental



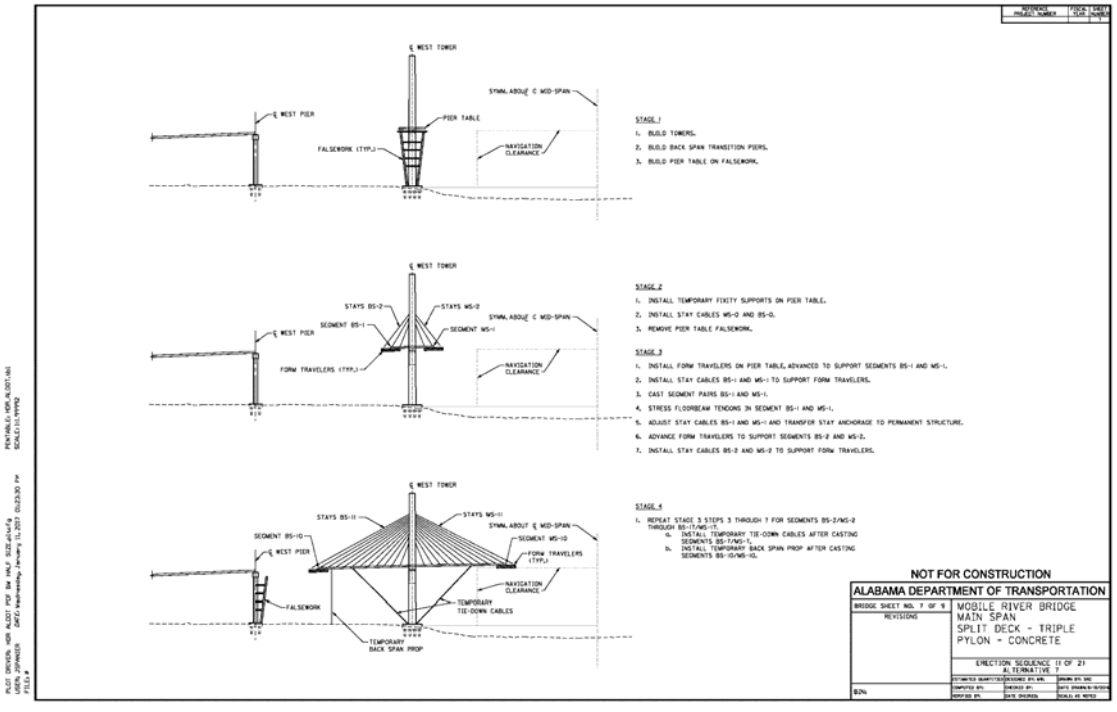
Construction Sequence - Alternative 3 (CIP Concrete Segmental)



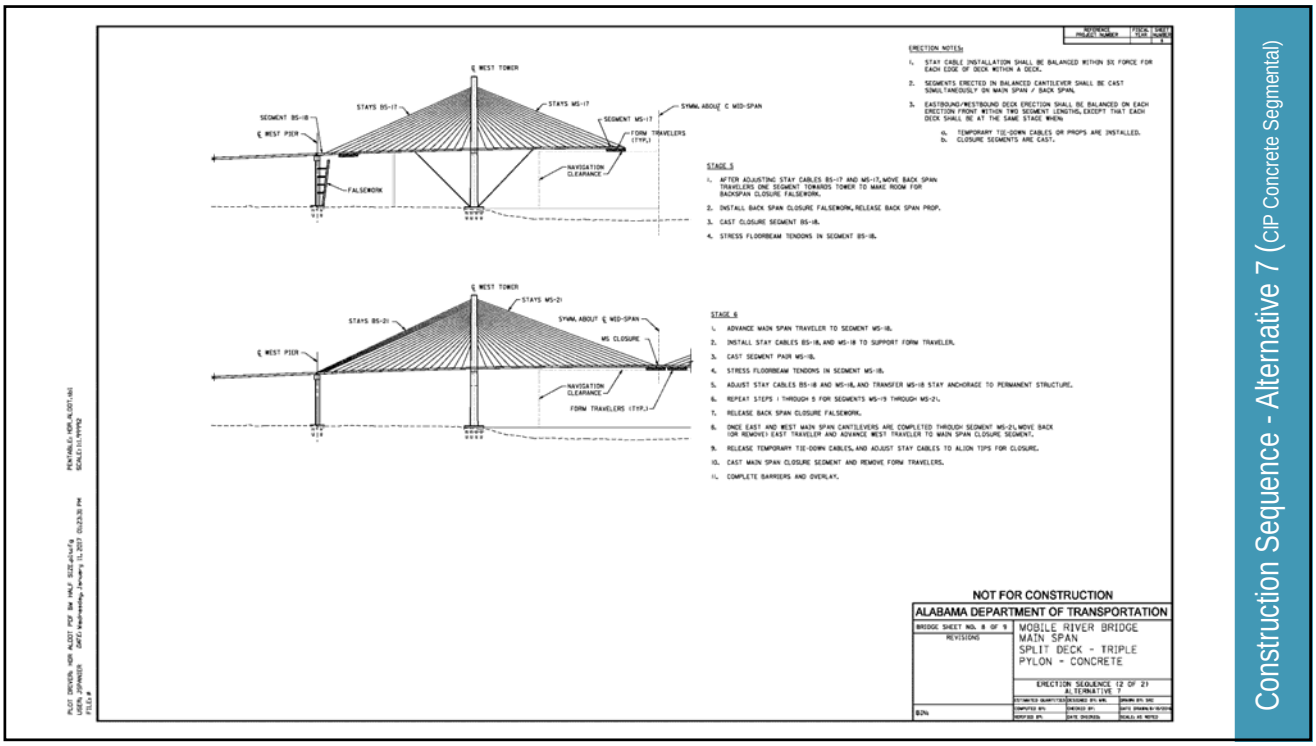
Construction Sequence - Alternative 3 (CIP Concrete Segmental)

Form Travelers used to Build Edge Girders in Cantilever from the Towers

Alternative 3 – CIP Concrete Segmental



Construction Sequence - Alternative 7 (CIP Concrete Segmental)



Construction Sequence - Alternative 7 (CIP Concrete Segmental)

Structure Type and Construction Sequence similar to Cochrane Bridge

Form Travelers used to Build Box Girders in Cantilever from the Towers

Alternative 7 – CIP Concrete Segmental

Main Span Concepts Evaluation – Construction Schedule

- Assumptions
  - 6 Day Work Week
  - Shut down Project for Holidays and inclement Weather on the order of Approximately 26 work days per year
  - West and East Towers constructed Concurrently
  - Superstructure work Beyond pier Table does not begin until Tower is complete
  - Superstructure Construction Proceeds from West and East Towers concurrently

Main Span Concepts Evaluation – Construction Schedule

Bridge Alternative	Approx. Total Duration (Mos)
1. Precast Concrete Segmental	47
1A. Precast Concrete Segmental	48
3. CIP Concrete Edge Girder – Split Deck	50
7.CIP Concrete Segmental – Split Deck	50

Main Span Concepts Evaluation – Initial Construction Cost

- For the Remaining Alternatives Quantities were Updated based on the revised bridge layouts and additional analytical modeling
- Developed a comprehensive list of Pay Items that in Addition to Major Structural Components included Allowances for:
  - Pile Load Test Program
  - Drainage System
  - Aesthetic Lighting
  - Lightning Protection
  - Navigational Lighting System
  - Roadway Lighting
  - Bridge Access

Main Span Concepts Evaluation – Initial Construction Cost

- Assigned Unit Costs based on Historical costs of Cable-Stayed Projects of Similar Scope
- Costs Developed for Ranking of Alternatives. As design Progresses the Cost Estimates will be revised accordingly.
- Cost Estimates do not include:
  - Right-of-Way
  - Maintenance Costs
  - Utility Relocation
  - Engineering Inspection
  - ALDOT Overhead

Main Span Concepts Evaluation – Initial Construction Cost

Bridge Alternative	Cost ($) M	Cost ($/SF)
1. Precast Concrete Segmental	277	684
1A. Precast Concrete Segmental	273	674
3. CIP Concrete Edge Girder – Split Deck	269	706
7. CIP Concrete Segmental – Split Deck	275	737

- 2017 Construction Costs
- Include 15% Contingency
- Include 5% Gross Receipts Tax

Main Span Concepts Evaluation – Initial Construction Cost

- However, to compare apples-to-apples we need to add to Alternatives 3 and 7 the Additional High Level Approach Spans length
  - Alternative 1 Main Span Unit Length = 2774'-0"
  - Alternatives 3 and 7 Main Span Unit Length = 2550'-0"
  - Additional Length = 224'-0"
  - Assume $200/sf for HLA
  - HLA width = 62'-5" (Ea. Direction)
- Additional Cost for Alternatives 3 and 7 = $200/SF x (62'-5" x 2) x 224' = $5.6M

Main Span Concepts Evaluation – Initial Construction Cost

Bridge Alternative	Cost ($) M
1. Precast Concrete Segmental	277
1A. Precast Concrete Segmental	273
3. CIP Concrete Edge Girder – Split Deck	275
7. CIP Concrete Segmental – Split Deck	281

- 2017 Construction Costs
- Include 15% Contingency
- Include 5% Gross Receipts Tax

Mobile River Bridge
Bridge Types – Evaluation Matrix

Main Span Criteria

Construction Cost	Constructability	Maintenance and Durability	Construction Schedule	Environmental Commitments	Design Considerations
Initial Construction Cost	Construction Access Disruption to Existing Traffic / Maintenance of Traffic Established Construction Methods Impacts to Existing Utilities Impacts to Railroads Cofferdams	Accessibility Tie-Downs Bearings Deck Joints Painting of Steel Deck Replacement Flexibility for Future Widening Stay Cable Replacement		Contextual Design and Aesthetics Impacts to Mobile's Maritime Traffic during Construction Permitting ROW Construction Vibrations	Foundation Sizes Navigational Air-Draft Clearance Accommodates Future Sidewalk

Main Span Concepts Evaluation – Recommendations

Good = ● Fair = ◒ Poor = ○

Main Span Bridge Alternatives

Criteria	1. Precast Concrete Segmental	1A. Precast Concrete Segmental	3. CIP Concrete Edge Girder – Split Deck	7. CIP Concrete Segmental – Split Deck
Initial Construction Cost ($) M	277	273	275	281
Construction Schedule (Mos.)	47	48	50	50
Constructability	◒	◒	●	●
Maintenance and Durability	●	●	●	●
Environmental Commitments	●	◒	●	●
Design Considerations	●	●	●	●
Overall	●	●	●	●

05 Next Steps

5. Next Steps

- Sectional Study of Preferred Alternative
- Study Feasible Open Rail Options
- Prepare TS&L Drawings
- Update Construction Cost
- Update Construction Schedule
- Aesthetics Meeting
- Include Scour Results (if Available)

I-10 Mobile River Bridge and Bayway Widening

Development of Main Span Cross-Section
Alternative 1B – Cast-in-Place

Mobile, Alabama
January 25, 2017

1 Introduction

During the Main Span meeting on January 19, 2017 in Mobile, Alabama the question was raised if Alternative 1A could be constructed utilizing cast-in-place construction techniques instead of precast concrete segmental as currently shown for that Alternative. The purpose of this memorandum is to further explore the feasibility of that concept and prepare recommendations for ALDOT.

2 Development of Cross-Sections

Development of cross-sections for concrete cable-stayed bridges is driven by several factors including:

1. Total bridge width
2. Span length
3. Precast or Cast-in-Place concrete construction
4. Stay cable array
5. Desired Aerodynamic shape

Traditionally for either precast or cast-in-place concrete the deck has been suspended by a central plane of stays along the center median (refer to Appendix A - Alternatives 1 and 1A) or multiple planes of stays supporting the deck along the edges of the superstructure (refer to Appendix A – Alternatives 3 and 7). When evaluating the feasibility of constructing Alternative 1A in cast-in-place with form travellers there are two possibilities that need to be considered:

1. Can the cross-section for Alternative 1A as currently shown be cast-in-place.

   The cross-section consists of two precast segmental box girders with a cast-in-place median slab that provides continuity between the two decks. At the stay cable locations, a precast concrete frame is used to anchor the stay cable and provide structural continuity between the two box girders in the transverse direction of the bridge.

   This structural configuration is advantageous as it allows the use of a central plane of stays to support very wide decks; however, it was originally developed to be precast so it has not been optimized for cast-in-place construction. Even though it is possible to cast-in-place the sections in between the stay cables (they consist of two trapezoidal boxes with a median slab) the same cannot be said about the concrete frame. The concrete frame is a large precast element that will require the contractor to lift it from below thus minimizing the benefit of a cast-in-place alternative which is to build the superstructure completely from above without disrupting the navigation channel.

2. Can a cross-section be developed that combines a central plane of stays and cast-in-place construction?

   If the superstructure is supported by a central plane of cables a box girder type cross-section is needed. As the bridge width increases, the cross-section needs to be stiff enough to transfer the majority of the dead and live load transversely to the centrally located supports. The most commonly used cross-section consists of a single cell trapezoidal box girder with internal struts as shown in Figure 1.

Figure 1 - Trapezoidal Box Girder Cross-Section with Internal Struts

The trapezoidal box girder is a closed section that is torsional stiff and capable of efficiently resisting the torsional loads applied by wind and live load on the bridge.

The required amount of torsional rigidity depends on the width of the deck and the length of the main span. As the width of the bridge increases, the effectives of the section decreases. In North and South America there are two cast-in-place concrete cable stayed bridges with a central plane of stays.

The Centennial Bridge in Panama is a 3,452 ft. long concrete cable-stayed bridge that was completed in 2005. The main span is 1,378 ft. in length and the out-to-out deck width is 112.5 ft.. The deck of the cable-stayed main span is composed of cast-in-place concrete single cell box girder with internal steel struts and transverse post-tensioned ribs in the top deck. A constant depth of 15 ft. was maintained for the entire extent of the box girder, and segments are 20 ft. in length. (See Figure 2)

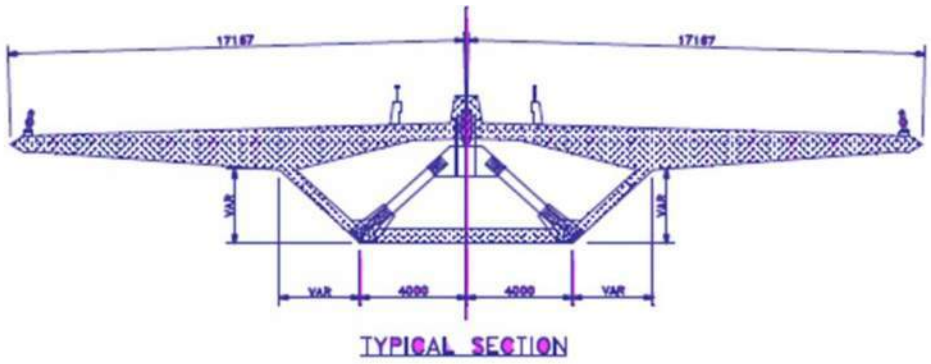


Figure 2 - Centennial Bridge – Panama, Cross-Section at Stay Cable (Dimensions in mm)

The Penobscot Narrows Bridge is a cast-in-place segmental cable-stayed bridge with a central plane of stays. The bridge was completed in 2007 and has a main span length of 1,161 ft. and an out-to-out deck with of 57.5 ft. (See Figure 3)

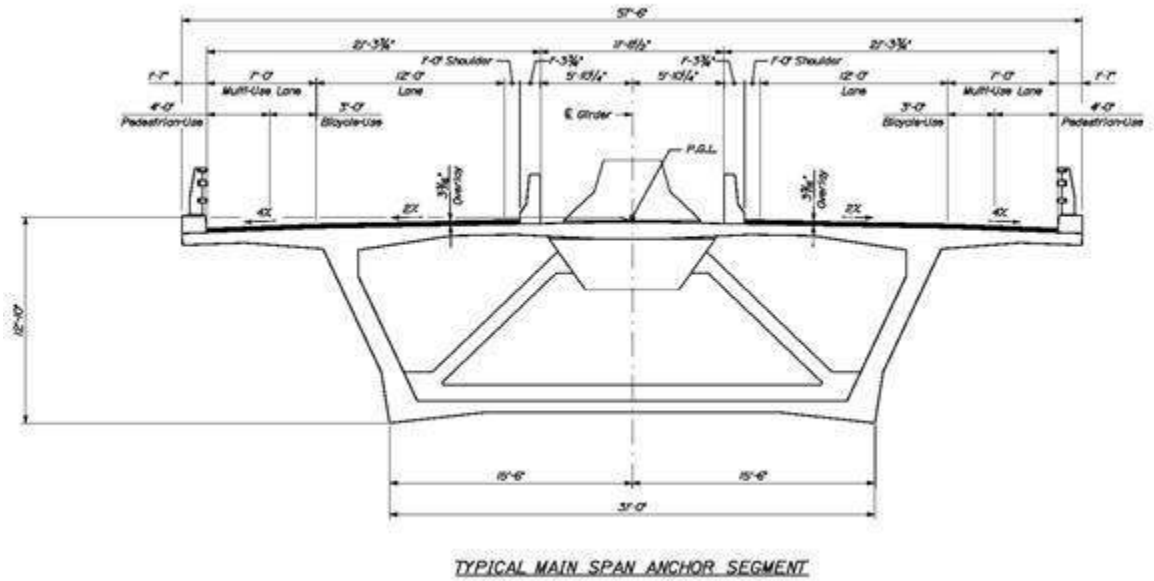


Figure 3 - Penobscot Narrows Bridge – USA, Cross-Section at Stay Cable

Recent North and South America Cast-in-Place Concrete Cable-Stayed Bridges with Central Plane of Stays

Project	Location	Year Built	Main Span Length (ft.)	Deck Width (ft.)	Girder Depth (ft.)	Kips/Ft.
Penobscot Narrows Bridge	U.S.A.	2007	1161	57.5	12	26.6
Centennial Bridge	Panama	2005	1378	112.5	15	40

Table 1 – Cast-in-Place Concrete Bridges with Central Plane of Stays

To establish the feasibility of a cast-in-place box girder with a central plane of stays for the Mobile River Bridge, a conceptual level cross-section was developed and named Alternative 1B.

3 Mobile River Bridge Cross-Section Alternative 1B

As previously discussed, as the deck width increases the effectives and feasibility of a single cell box girder decreases. The cantilever wings get thicker; as the web spacing increases the top slab thickness has to be increased to accommodate the transverse bending of the box, unless the box depth is increased the internal struts inclination may become flatter thus making them more inefficient. In summary, there is a practical and structural limit to the single cell box girder. A more efficient structural form for wide bridge decks is the multi-cell box girder as shown in Figure 4. The cross-section can be shaped to have a high torsional stiffness and be aerodynamically efficient; however, they are typically heavier than a single cell box girder.

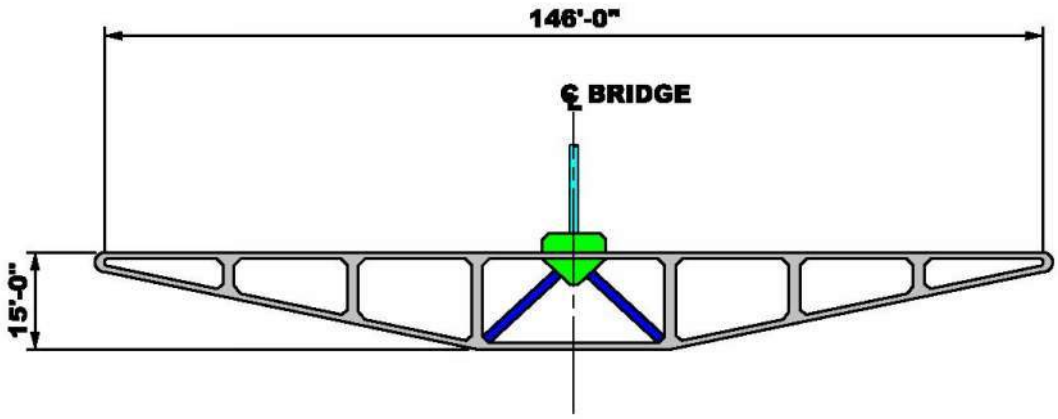


Figure 4 – Mobile River Bridge Multi-Cell Cross-Section Alternative 1B

To help assess the feasibility of a cast-in-place cross-section with central plane of cables, Table 2 was prepared to summarize key indicators for the Mobile River Bridge Alternatives presented on January 19, 2017, Alternative 1B, the Penobscot Narrows Bridge and the Centennial Bridge.

Comparison of Key Metrics

Project	Location	Year Built	Main Span Length (ft.)	Deck Width (ft.)	Girder Depth (ft.)	Kips/Ft.
Penobscot Narrows Bridge	U.S.A.	2007	1161	57.5	12	26.6
Centennial Bridge	Panama	2005	1378	112.5	15	40
Mobile River Bridge - 1A Precast	U.S.A.	N/A	1350	146	14	56.3
Mobile River Bridge – 1B Cast-in-Place	U.S.A.	N/A	1350	146	15	63.5
Mobile River Bridge - 3 Cast-in-Place	U.S.A.	N/A	1350	74 (Split Deck)	6	47.7 (Two Decks)
Mobile River Bridge - 7 Cast-in-Place	U.S.A.	N/A	1380	73.167 (Split Deck)	9	54.6 (Two Decks)

Table 2 – Comparison of Key Metrics

From Table 2 it is important to recognize that the proposed spans lengths for the Mobile River Bridge are well within standard practice for cast-in-place box girder cable stayed bridges. Alternative 1A has a main span length of 1,350 ft. compared to 1,378 ft. of the Centennial Bridge and 1,161 ft. of the Penobscot Narrows Bridge. However, the major difference is in the overall bridge width. The Mobile River Bridge overall width for Alternative 1A is 146 ft. which corresponds to a 150% increase over the Penobscot Narrows Bridge and 30% over the Centennial Bridge.

To assess the cost feasibility of Alternative 1B, Table 2 also lists the weight of the superstructures on a Kips/ft. basis. This is not only an indicator of the difference in superstructure material consumption but also an indicator of the relative impact to tower and foundation costs. As the superstructure gets heavier the towers and foundations will need to be strengthened. Since the Penobscot Narrows Bridge and Centennial Bridge are narrower it would be expected to see a decrease in Kips/ft. when compared to Alternative 1B. For this reason, it is more meaningful to compare Alternative 1B tp Alternatives 1A, 3 and 7. At 63.5 Kips/ft., Alternative 1B is heavier than Alternatives 1A, 3 and 7 by a margin ranging from 13% to 33%. If we assume similar cast-in-place concrete unit costs as Alternatives 3 and 7 then it would be reasonable to expect an increase of approximately $13.4M in superstructure concrete costs only for Alternative 1B over Alternative 1A, $12M over Alternative 3 and $6.5M over Alternative 7.

4 Recommendations

On this basis of this study we recommend that a cast-in-place box girder combined with a central plane of stays not considered any further during this TS&L phase. Specifically noting the following:

1. Casting in place the cross-section shown for Alternative 1A will still require lifting of the heavy concrete frames from below thus requiring access to the navigation channel.

2. Even though the cast-in-place cross-section developed for Alternative 1B (Figure 4) is structurally feasible there appears to be no cost savings for the project.

3. Alternatives 3 and 7 can be built completely from the top; thus satisfying the DEIS commitment to maintain the navigation channel open.

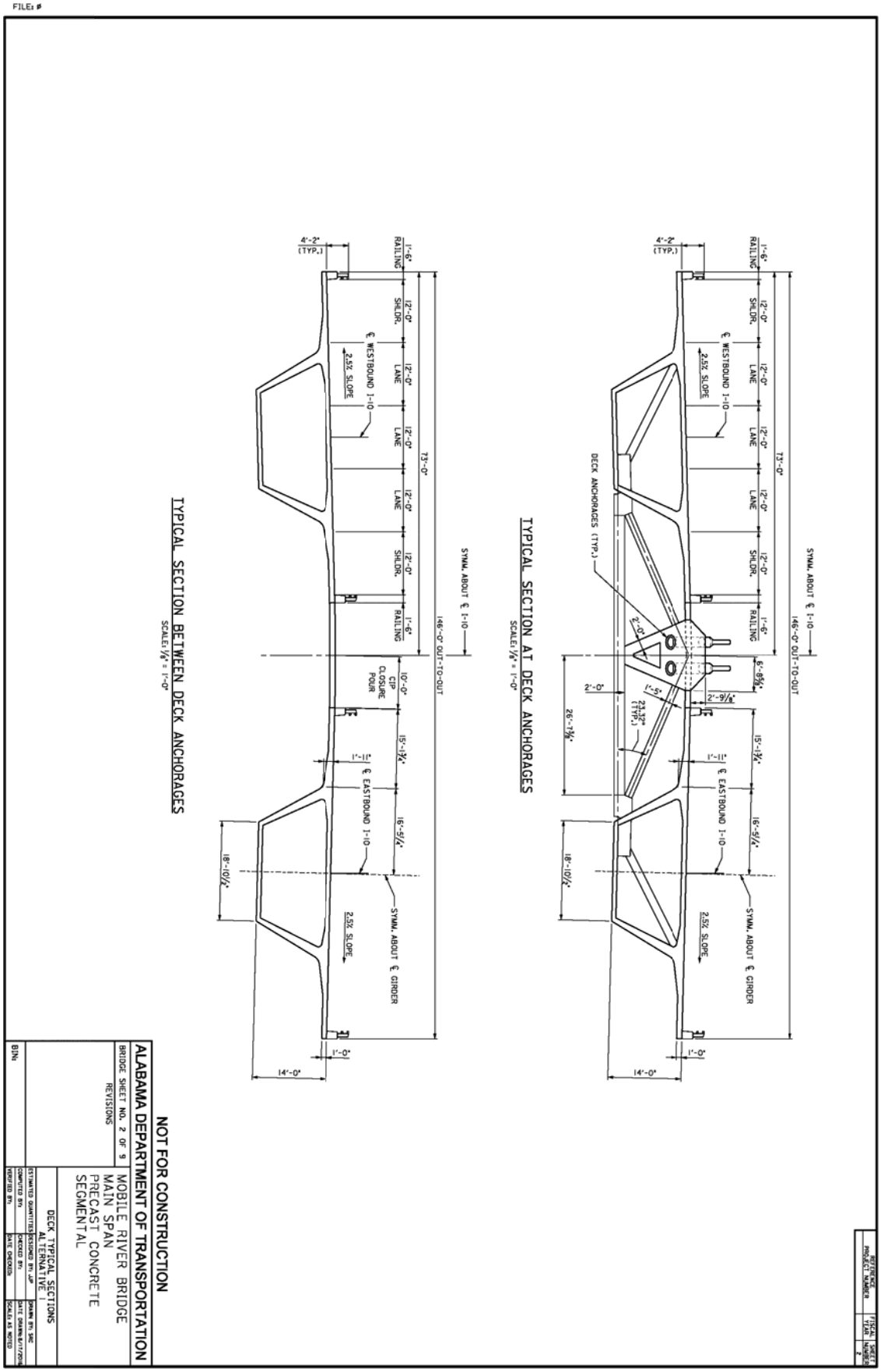
4. From our experience and research, in the US there has not been a concrete cable-stayed bridge constructed that combines the deck width of this project with a central plane of stays.

5. A multi-cell box girder presents challenges for maintenance and inspection access.

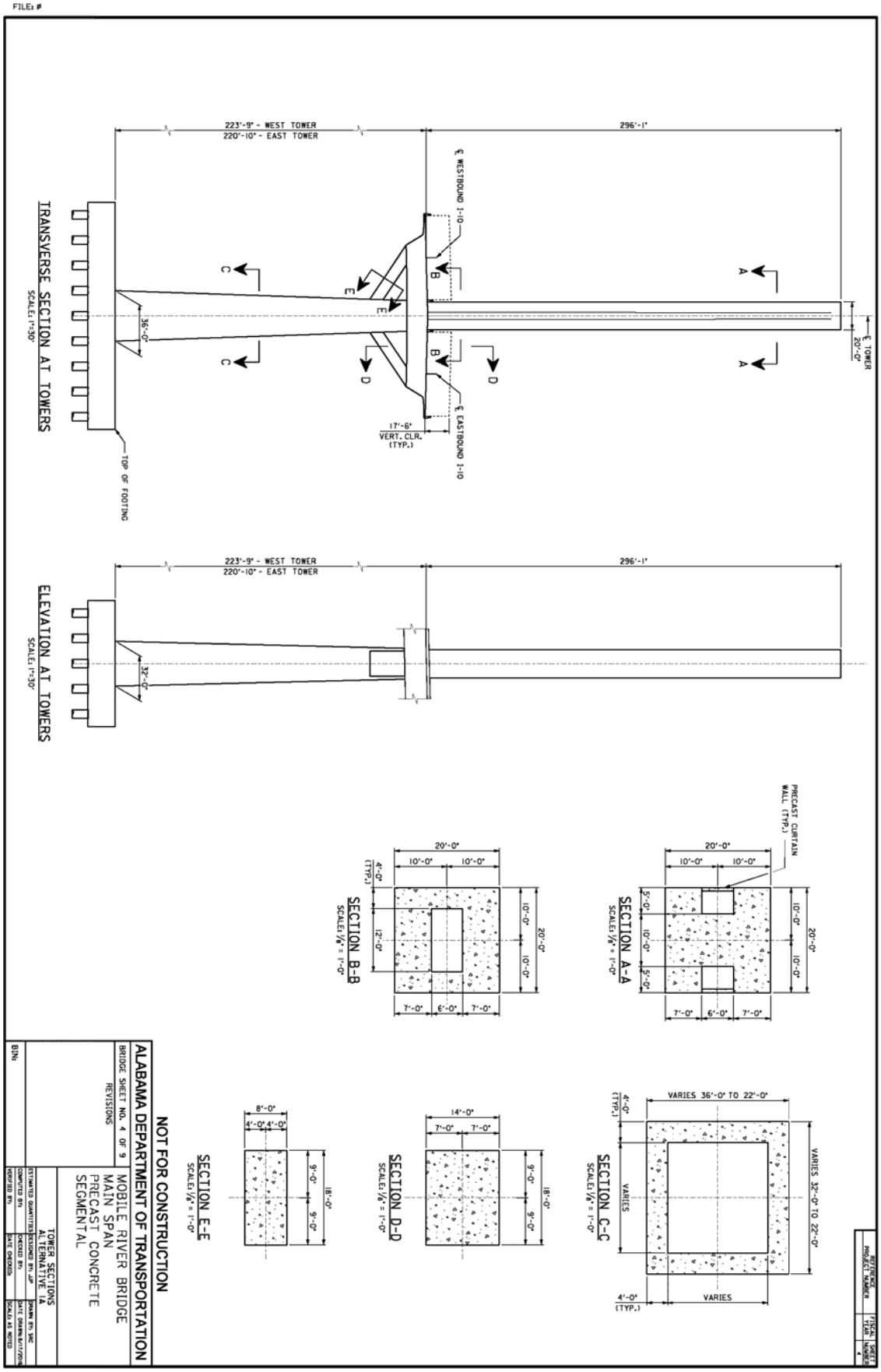
6. A rather large and heavy form traveler will be needed out on the cantilever tip. It would be much heavier than the edge girder (Alternative 3) or shallow box section (Alternative 7)

Appendix A – Alternatives 1A, 3 and 7

Alternative 1A – Precast Concrete Segmental (Cantilever)

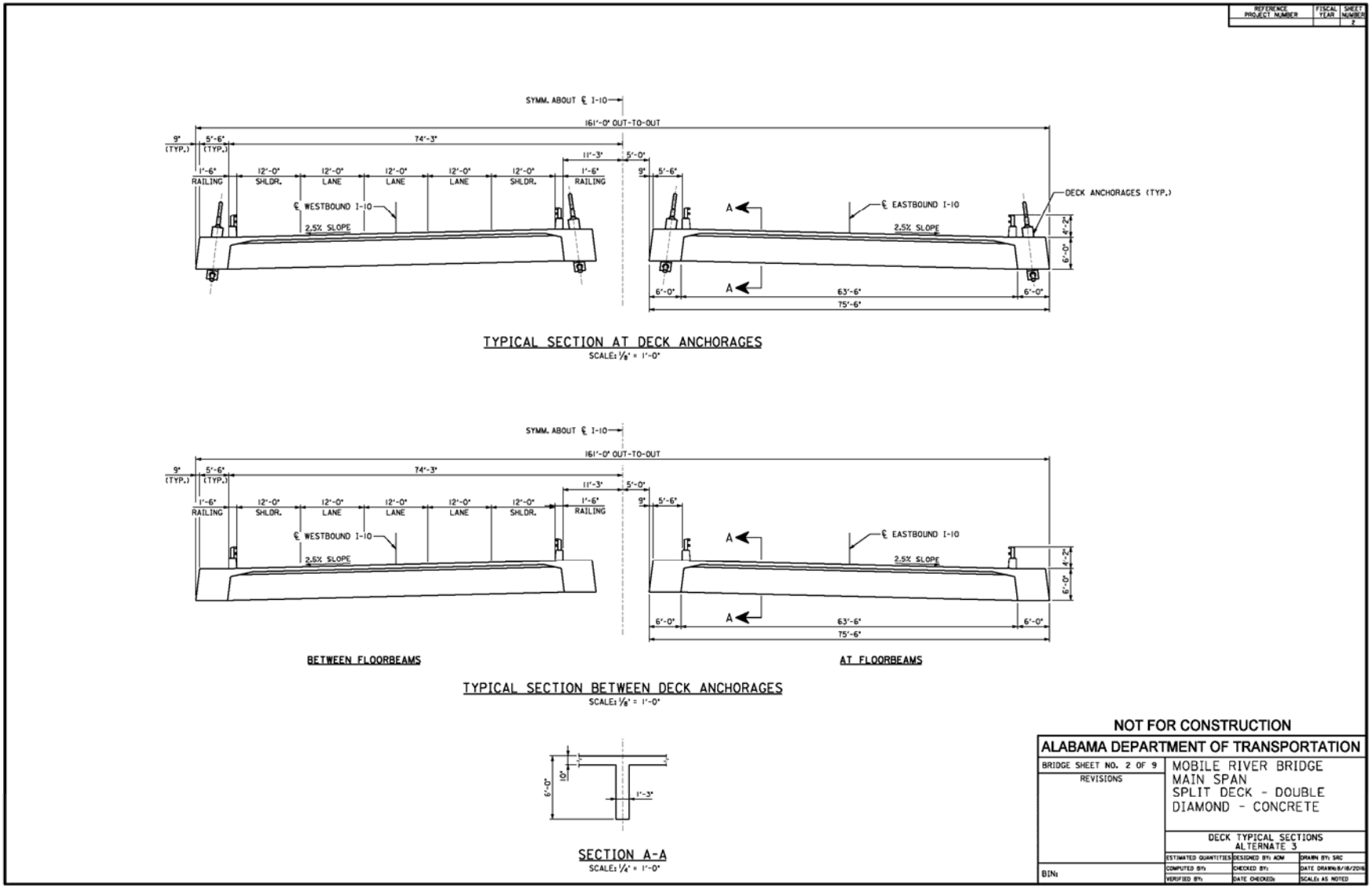


Alternative 1A – Precast Concrete Segmental



Alternative 1A – Precast Concrete Segmental (Cantilever)

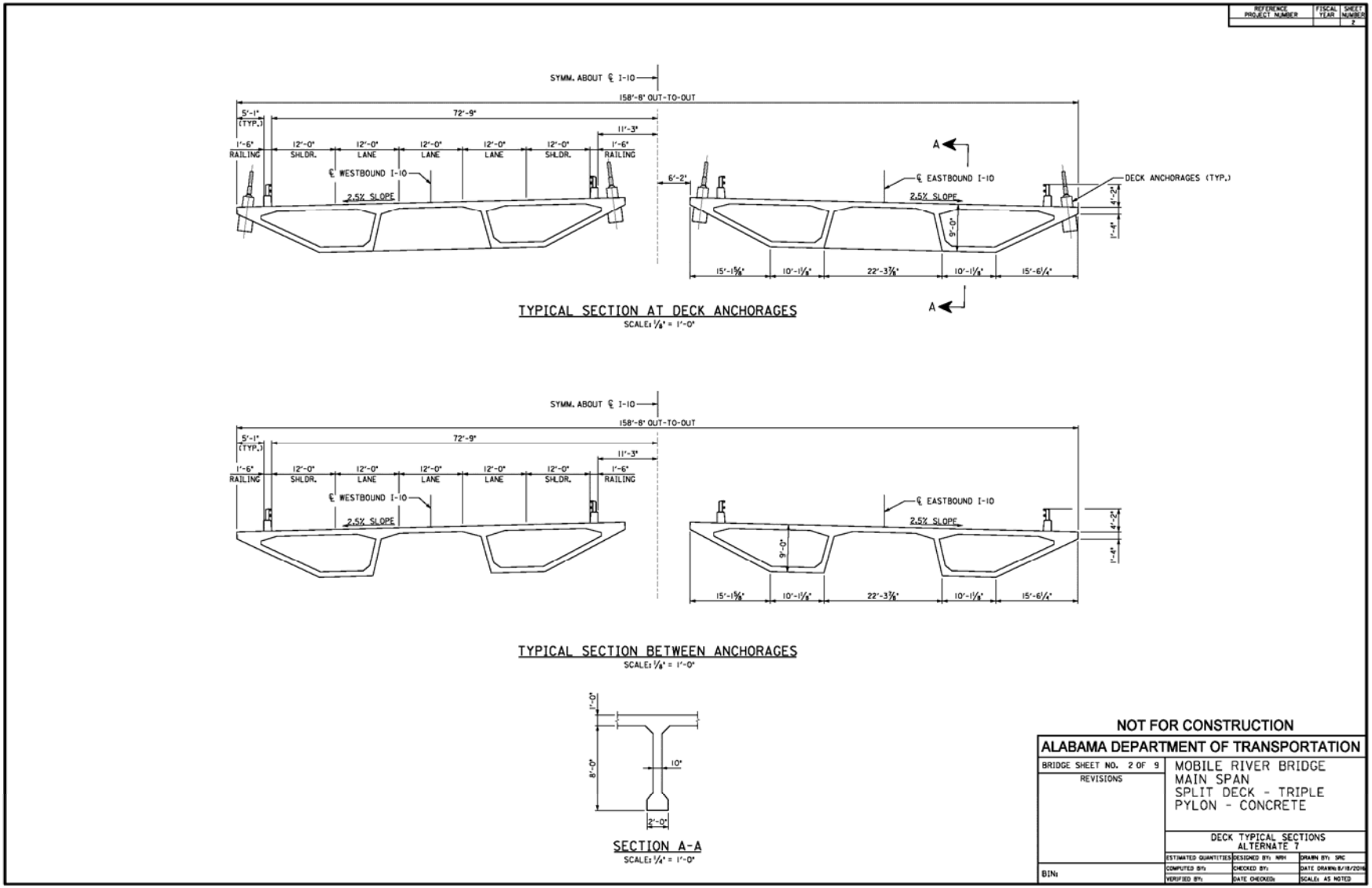
Alternative 3 – CIP Concrete Segmental



Alternative 3 – CIP Concrete Segmental

Alternative 3 – CIP Concrete Segmental

Alternative 7 – CIP Concrete Segmental



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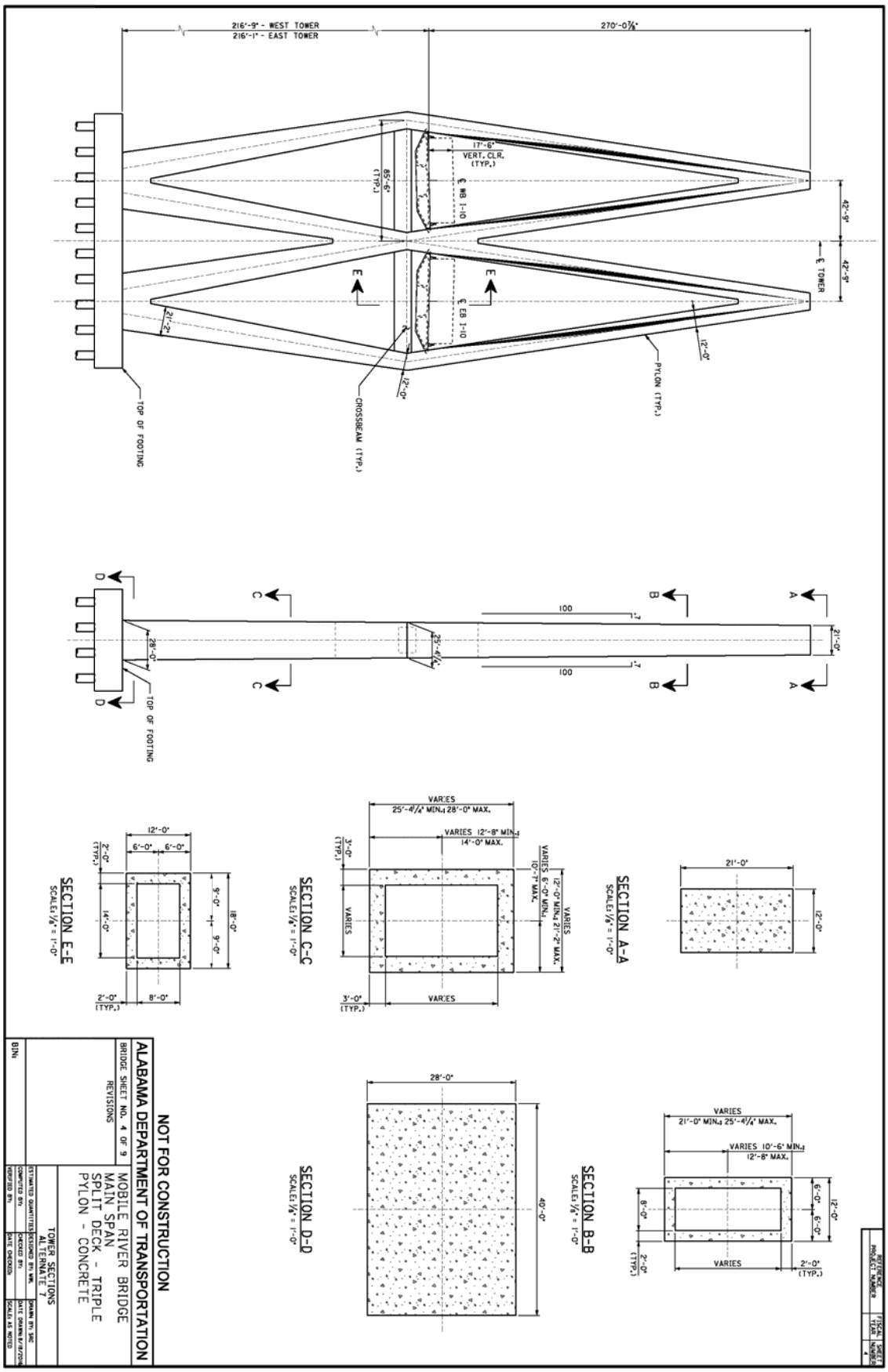

Alternative 7 – CIP Concrete Segmental

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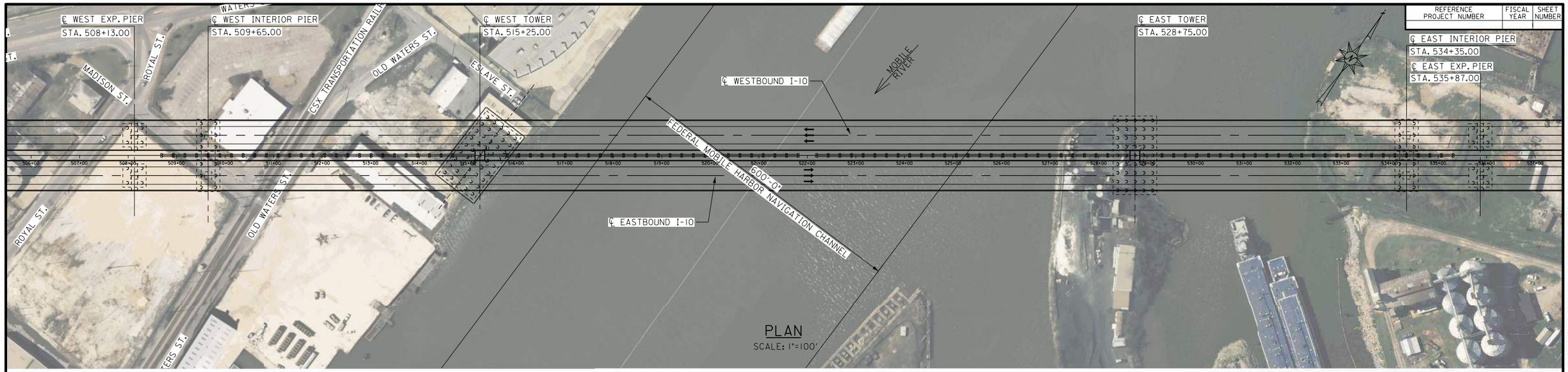

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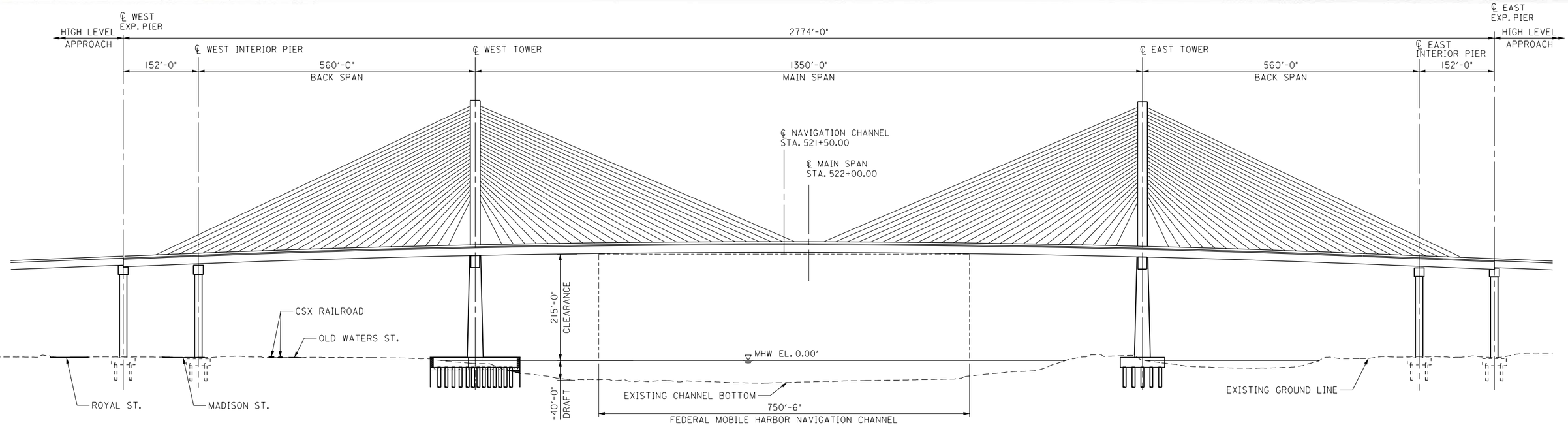

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NOT FOR CONSTRUCTION

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 9

REVISIONS

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

PLAN & ELEVATION
ALTERNATIVE 1

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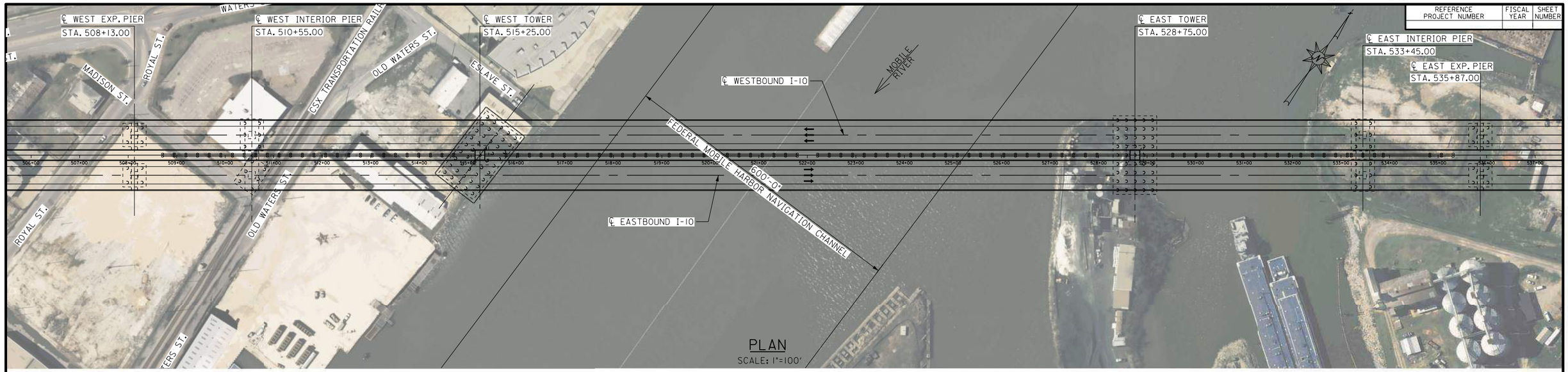

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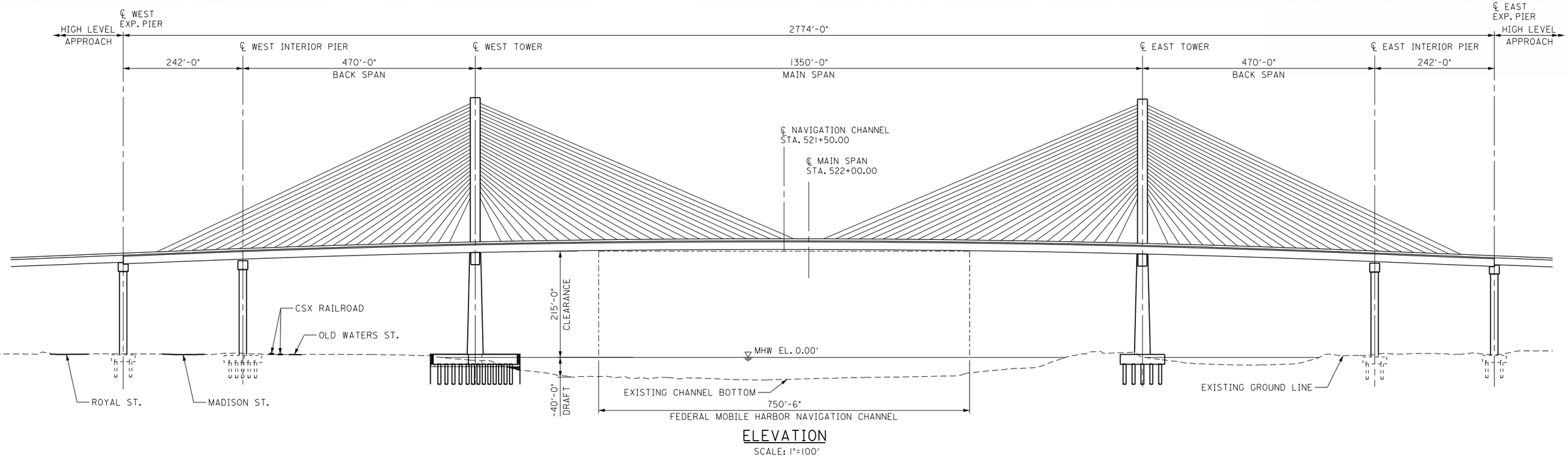

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 9

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
PRECAST CONCRETE
SEGMENTAL

PLAN & ELEVATION
ALTERNATIVE IA

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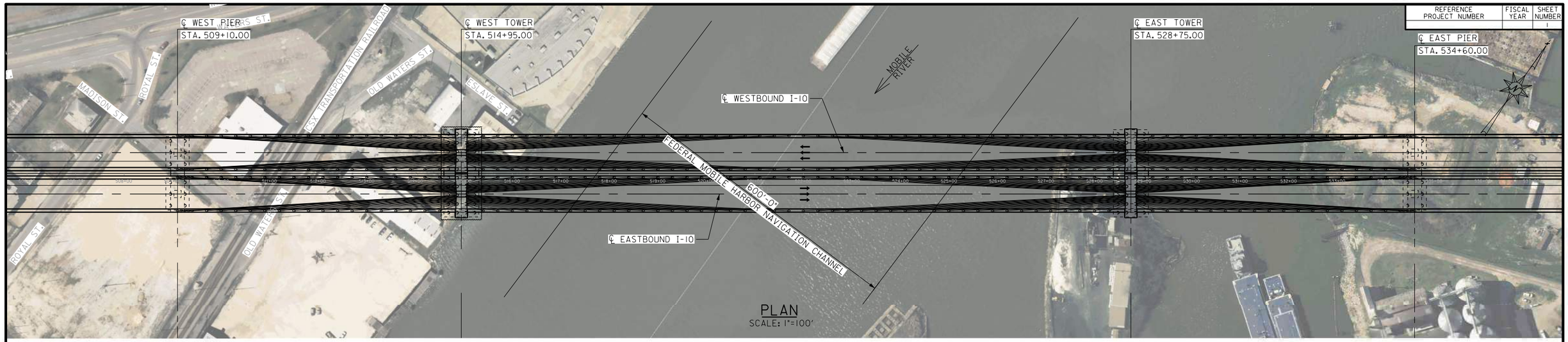

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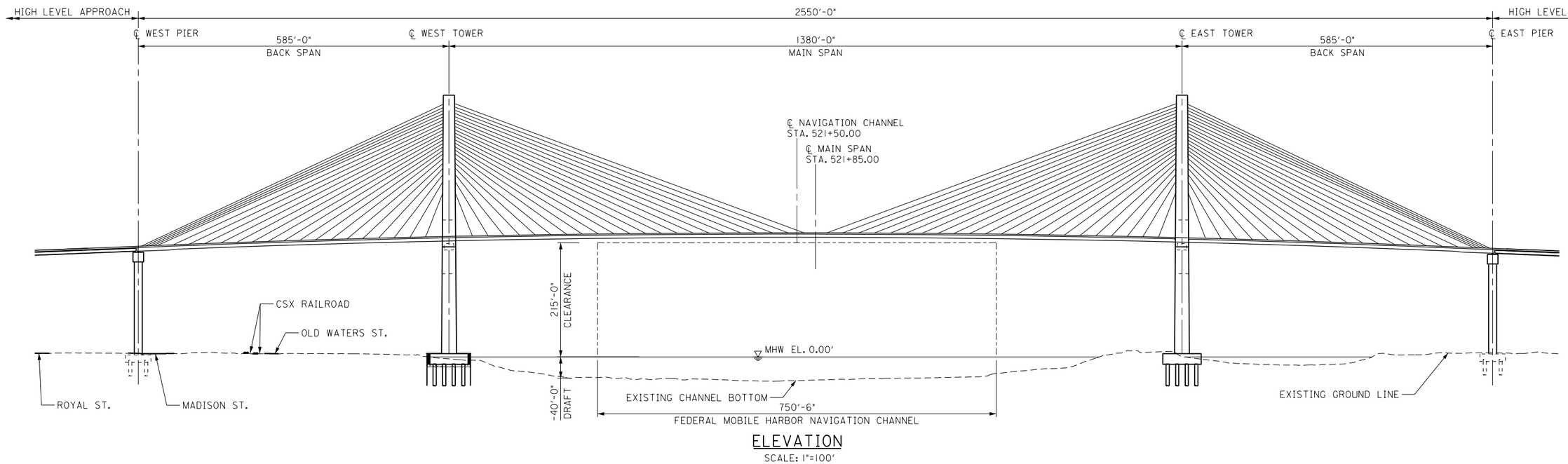

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 9

REVISIONS

BIN:

MOBILE RIVER BRIDGE
MAIN SPAN
SPLIT DECK - DOUBLE DIAMOND - CONCRETE

PLAN & ELEVATION
ALTERNATE 3

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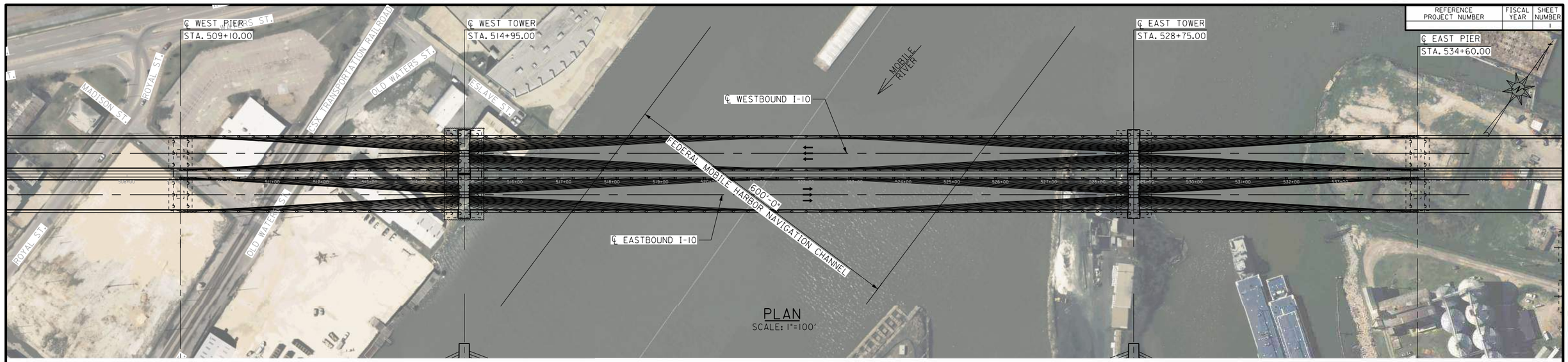

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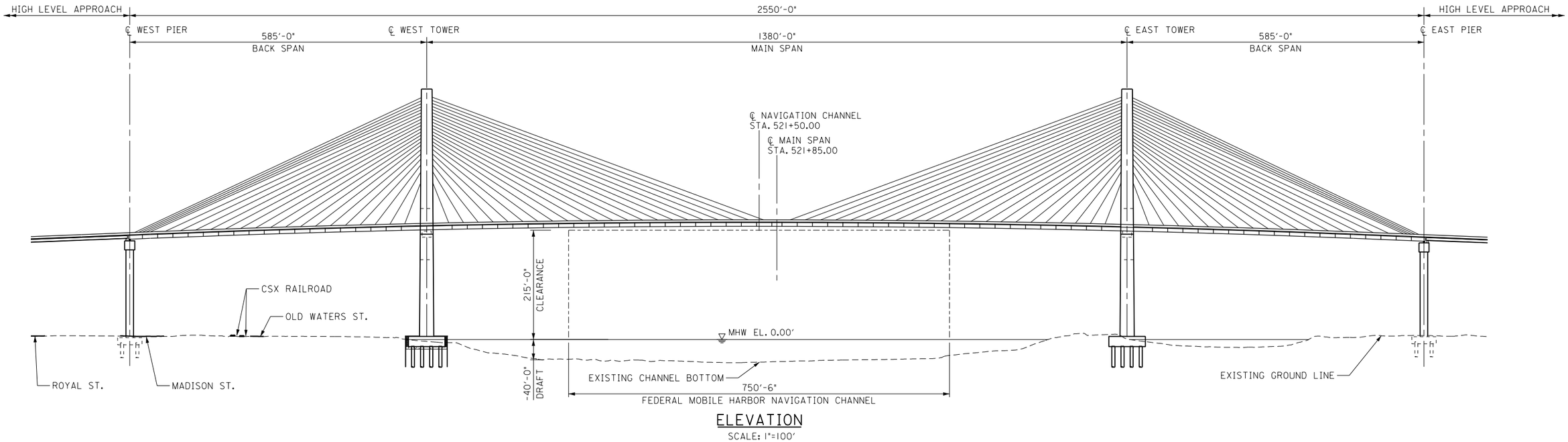

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ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. 1 OF 9

REVISIONS

BIN:

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MAIN SPAN
SPLIT DECK - TRIPLE
PYLON - CONCRETE

PLAN & ELEVATION
ALTERNATE 7

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