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CSX Mobile River Lift Bridge: Redesign and Construction

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Introduction

Following a Value Engineering Review of an existing design for this new vertical lift bridge by HNTB, to replace an existing swing span, a redesign of the structure was performed to examine foundations, superstructure, and construction staging, as well as the mechanical and electrical systems to develop cost savings.

To keep within the tight design schedule, the existing design by others was used as the basis for implementing improvements.

Cost effective redesign was performed by HNTB to lower the estimated construction cost of the bridge while maintaining or improving the design requirements.

The Value Engineering Review determined that potential major savings were present in the following components:

1. Lift Towers
2. Foundations

As part of the Value Engineering Review the design calculations were reviewed. It was determined that the design loads were in excess of the capabilities of the motor and drive system specified. This resulted in changes to the counterweight sheave bearings to reduce the operating loads and allow the originally specified motor to be used.

During construction the contractor submitted a Value Engineering Proposal to use driven pipe piles instead of drilled shafts. The contractor also requested that the lift truss be reanalyzed and redesigned as needed so that the member connections could be detailed for the cambered shape instead of detailed for the condition with dead load (Chicago style). An alternative method of adjusting the operating rope tension was also requested by the Contractor.

Figure 1 shows the plan view of both the original design layout and the final design layout. The final design layout incorporates both the Value Engineering by HNTB to revise the foundations and towers and the Value Engineering by Scott Bridge Co. which modified the foundations to use driven pipe piles instead of drilled shafts.

Figure 2 shows the elevations view of both the original design layout and the final design layout.

Tower Redesign

Both truss style towers and single frame towers have been used on vertical lift bridges. Each has a typical set of advantages and disadvantages.

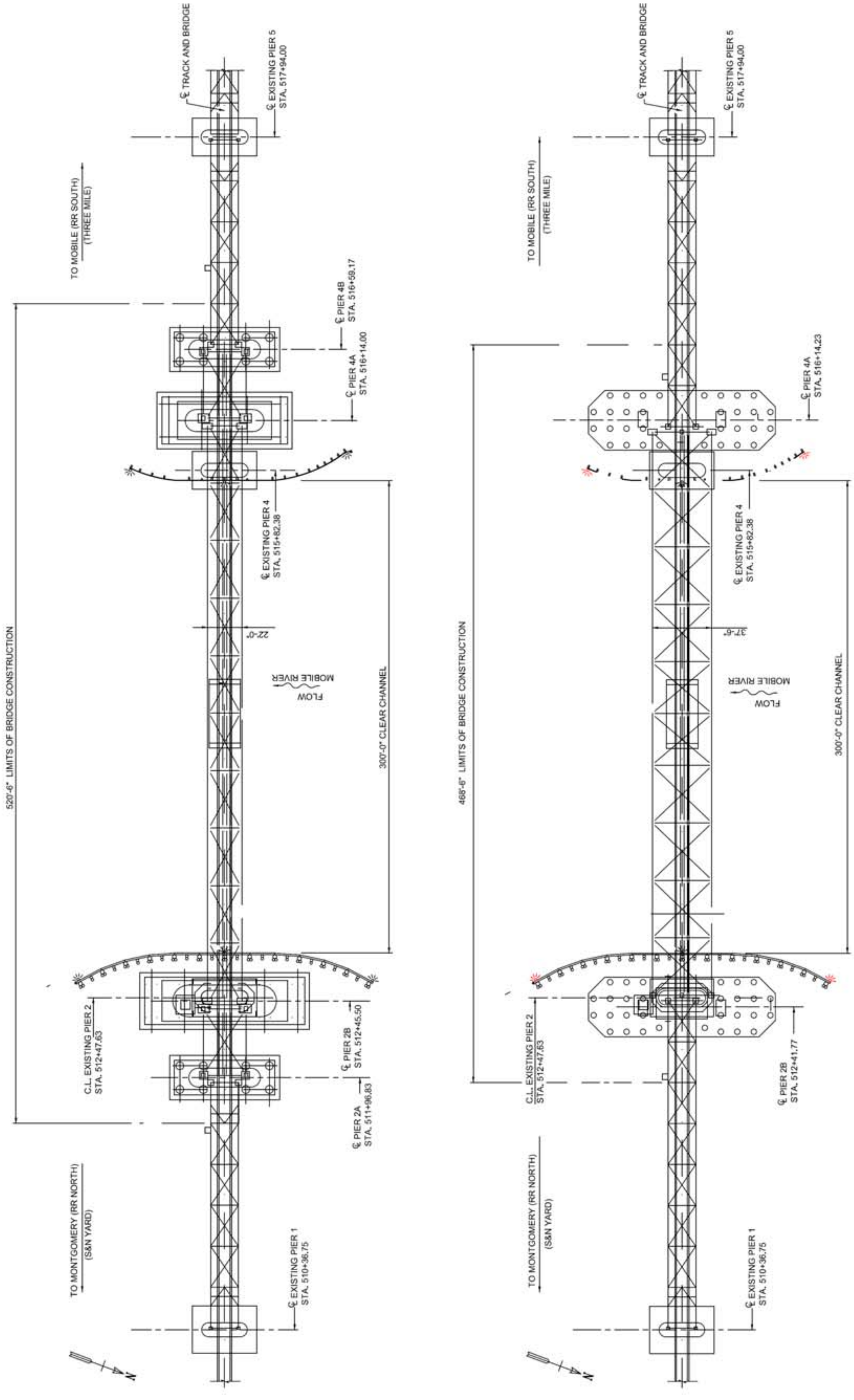

FIGURE 1: Plan view of original design layout (Top) and final design layout (Bottom).

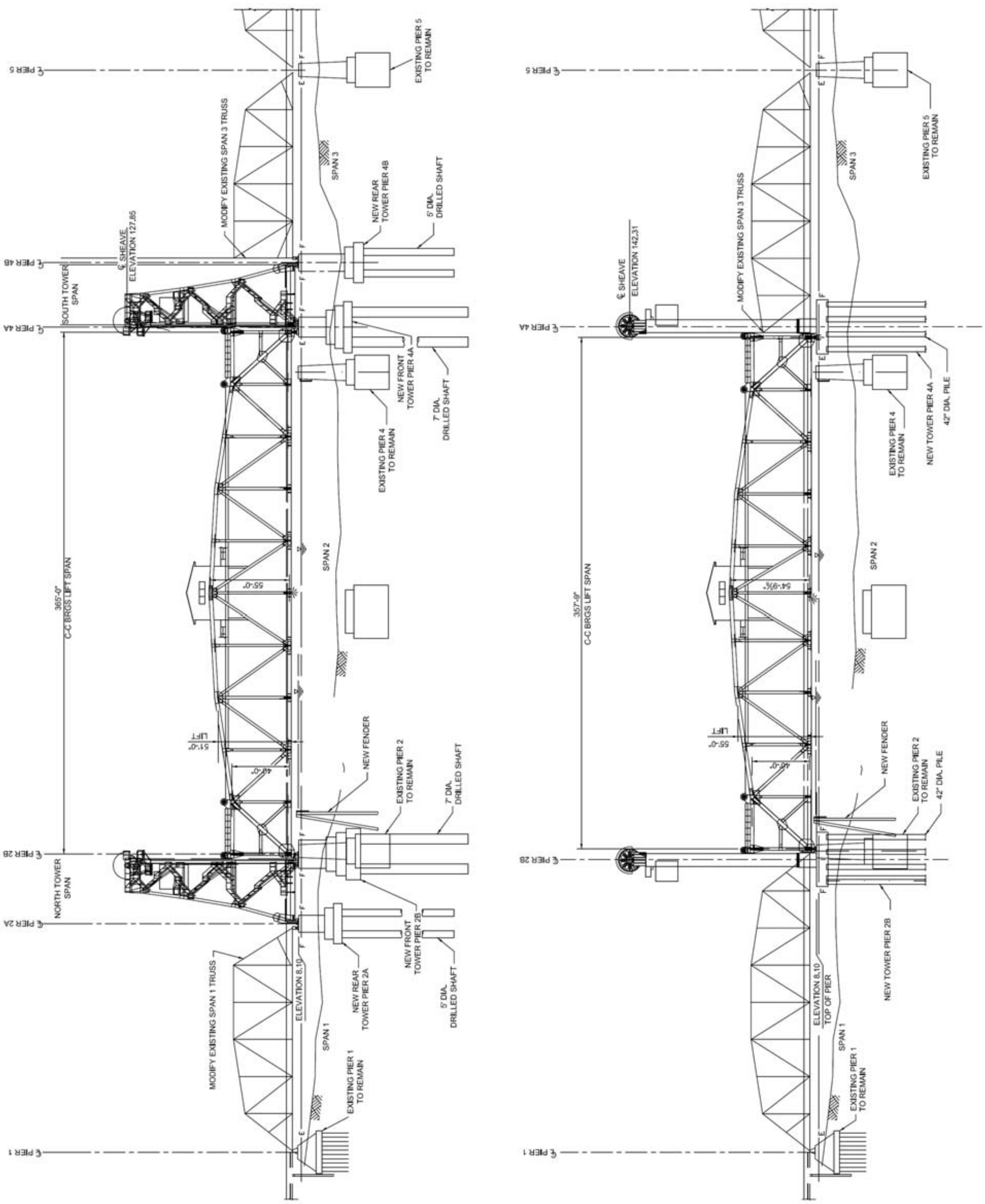

FIGURE 2: Elevation view of original design layout (Top) and final design layout (Bottom).

The original design of the lift towers used a truss approach with three vertical bays. The height of the tower was 119.7 feet from top of foundation to centerline of sheave and the width was 26.5 feet center to center of columns. The towers were designed for a 51 foot lift with the members sized to allow an additional 15 foot addition for increased vertical clearance in the future. In the span longitudinal direction (RR North/South) the front and rear legs had a 42 foot spacing. In the span transverse direction the front columns are connected by diagonal bracing in three bays down to the clearance envelope for the track where a portal frame is used. The majority of the sheave reactions (91% of dead loads) are carried by the front columns.

The redesign of the towers used a single frame on each end with a mid-height strut between columns (see Figure 3). The tower columns are 9 feet deep by 5 feet wide steel boxes with 1 ¼ inch plates with one longitudinal stiffener on the 5 foot sides and 1 ½ inch plates with three longitudinal stiffeners on the 9 foot sides. The towers resist loads in the span longitudinal direction by flexure of the columns with a fixed connection to the pedestals and the foundation. The towers resist loads in the transverse direction by frame action with the mid-height strut providing a tie that reduces both the unbraced length in that direction and the transverse moments.

The height of the tower was initially designed at 130.2 feet for a 51 foot lift matching the original design. Just before bid advertisement the United States Coast Guard (USCG) requested that the design be modified for a 55 foot lift. HNTB quickly revised design calculations and 27 design sheets in 21 calendar days to accommodate this request. The final height of the tower was 134.2 feet from top of foundation to centerline of sheave and the width was 49.7 feet center to center of columns.

	Tower	*Bridge Truss*
Original Design	1,700,000 lb	2,200,000 lb
Redesign	1,750,000 lb	2,378,000 lb

Table 1. Structural Steel Quantities

Each of the tower types presented advantages and disadvantages as follows. The material cost of the steel is not significantly different between the two as seen in Table 1.

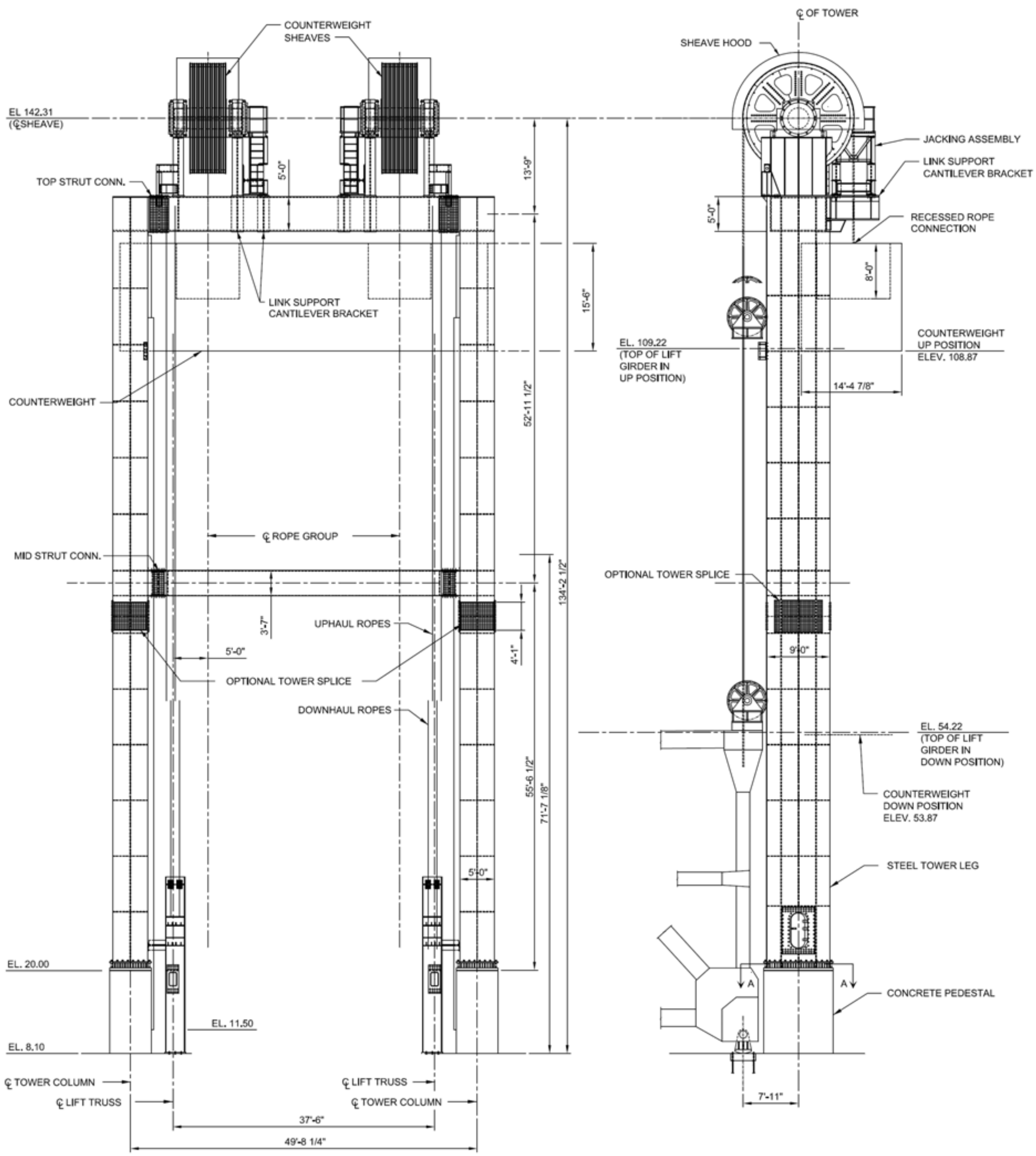

FIGURE 3: Final design of towers.

Redesigned Tower Advantages

The primary advantage of the redesigned towers over the original towers is reduced field work to erect the towers since each tower leg has only one splice and the mid-height strut and top strut are the only connecting members. The cantilevered sections on the top strut to hang the counterweight were installed before the top strut was lifted into place. Minimizing the field work in the erection of the towers was especially important since that work was fouling the active track and the contractor had limited windows of time while fouling. For this reason the cost of erected steel in the low bid of the original design was higher than the cost of erected steel in the low bid for the redesign (see related paper in this proceedings).

The redesigned towers also have an advantage in resisting vessel impact. The redesign scope included determining vessel impact forces based on the barges that are in use on the Mobile River and incorporating those forces into the design. The fender system was retained from the original design for redirecting light impacts but it was shown to be inadequate for either a fully loaded barge or for an unloaded barge travelling at maximum speed. The redesigned foundations and the Value Engineering changes to those foundations during construction are capable of resisting the fully loaded barge impact and preventing damage to the towers. For an unloaded barge the forward bow can be considerably ahead of the portion of the barge at the waterline and it is geometrically possible that the bow could impact a tower before impacting the foundation. For this reason the concrete pedestals for the towers were extended vertically 12 feet so that an unloaded barge would impact the side of the pedestal instead of the steel tower leg. The pedestals were reinforced to resist this vessel collision case.

Since the redesigned towers are not a deep structure there is no need for a tower span. This also means that the modifications to the existing trusses shown in Figure 2 could be reduced with a smaller modification made to Span 3 and no modification required for Span 1.

Redesigned Tower Disadvantages and Solutions

The redesigned tower layout also has a few disadvantages compared to the original design layout. The temporary condition of the counterweight hung off the tower without being connected to the counterweight ropes and balanced by the span weight combined with longitudinal wind load exerts a very large moment at the base of the tower legs. The erection required that the counterweights be filled with concrete in stages while hung on the towers. In order to maximize the amount of time available and ensure safety of the operation the contractor volunteered to supply Grade 105 anchor rods on the tension face of the connection. Using the high strength anchor rods increased the allowable wind load acting on the towers to approximately 78% of the 75 psf design lifetime wind event for this temporary condition.

The single leg towers have geometric disadvantages for clearances that were solved by increasing the tower width and height compared to the original design. The 9 foot depth of the tower legs reduces the clearance between the tower/pedestal faces and the end of both the final vertical lift span and the swing span being replaced compared to the original truss tower with smaller footprint front legs. Because of this the redesigned tower width needed to be almost double the original design tower width to accommodate operation of the swing span during construction. This then resulted in the bridge truss being widened as CSX preferred to not have support trusses at the ends to extend the span guide brackets. This accounts for most of the increase in the bridge truss structural steel shown in Table 1. Some of that increase is due to revisions in the bridge truss design during construction, as described later in this paper. The tower height necessary was greater for the redesign with the same vertical lift height since the removal of the tower spans also means the counterweight cannot be dropped as far. In the redesign layout, it would contact an approach truss if lowered as far as the original design counterweight could be lowered.

Foundation Redesign

As seen in Figure 1 and 2 the original design included an extension of Pier 2 (shown as Pier 2B) and the construction of new Piers 2A, 4A, and 4B. This layout was dictated by the tower type and the design objective of improving the navigation channel clearance to 300 feet. Table 2 shows the total Pier concrete in cubic yards and the total Pier reinforcing steel in pounds for the original design and the redesign. The original design used cofferdams which were eliminated and replaced with floating forms in the redesign. This greatly simplified the work to be done under trafic, allowed the decreased volume of concrete shown in Table 2, resulting in significant cost savings. The redesigned caps for the deep foundation elements (drilled shafts later revised to driven pipe piles) carry the loads as a deep beam from the towers to the deep foundation elements. Because the pile cap is much shallower than that in the original design, much more reinforcing steel is required, however.

	Concrete	*Reinforcing*
Original Design	4,607 CY	411,852 lb
Redesign	2,313 CY	743,000 lb

Table 2. Total Pier Quantities

Counterweight Sheave Bearings

The original machinery design specified use of a variable-frequency flux-vector drive and a 150 horsepower main span drive motor. The counterweight sheave and operating drum bearings were designed as plain bronze sleeve-type.

During the value engineering process, the design loads and calculated power requirements were verified, and found to be suspect

Designing according to the AREMA Manual for Railway Engineering, it was determined that approximately 225 horsepower was required to operate the lift span (including consideration of the motor full load torque factors allowed for starting and acceleration). Increasing the motor size and redesigning the machinery for this increased capacity was quickly discounted as impractical.

The calculations were examined to determine possible areas where loads could be reduced or efficiency could be increased. One of the largest sources of resistance was the friction in the plain counterweight sheave bearings. Upon comparison of the friction factors specified by AREMA, it was determined that use of roller bearings instead of the plain bronze bearings could reduce the design power requirements from approximately 225 horsepower to just over 100 horsepower.

	Coefficient for Starting	Coefficient for Motion	Motor Horsepower Required
Plain Bearings	0.135	0.090	225
Roller Bearings	0.004	0.003	104

TABLE 3: Summary of Counterweight Sheave Bearing Friction Factors and Motor Horsepower Required

No other change or combination of changes offered as significant a reduction in design operating loads. The decision was made to alter the design of the counterweight sheaves to utilize spherical roller bearings.

Implementing this change on a single-track rail bridge presented some challenges. Plain bearings are typically maintained and inspected by removing the bearing cap vertically, allowing access along the longitudinal axis of the shaft. However, roller bearings are mounted from the shaft ends. Further, inspection of large roller bearings is performed by removing a cover plate located on the outboard side of the bearing housing. This requires a fair amount of working space between the pair of sheaves in a tower. Accommodating working space for future replacement of the inboard seals resulted in the need to increase the bearing center-to-center distance even further. See Figures 4 and 5. A 231-series bearing was selected for its narrow profile to maximize the available space.

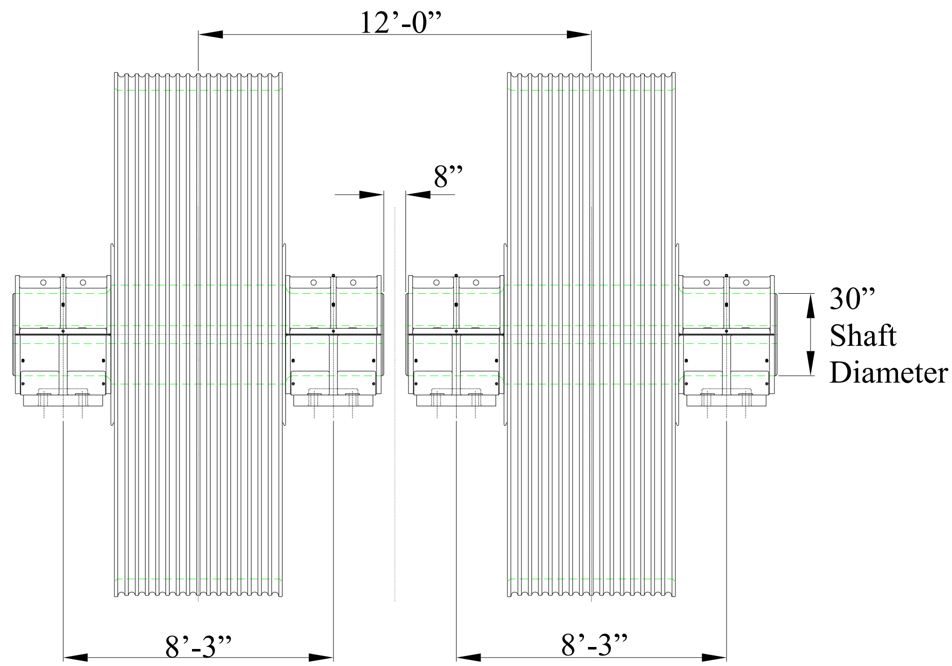

FIGURE 4: Elevation View of sheave pair with plain bronze bearings.

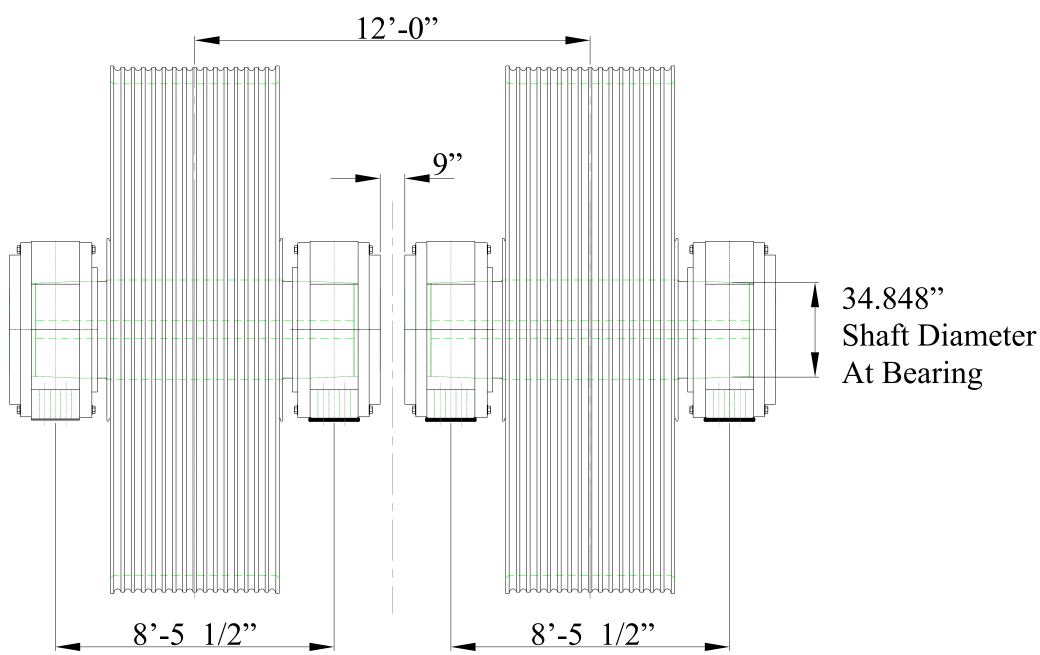

FIGURE 5: Elevation View of sheave pair with roller bearings. Note bearing center distance was increased, as well as work space between inboard bearings.

Fortunately, the structural portion of the value engineering exercise altered the proposed span and tower geometry to facilitate construction of the new towers outside of the existing structure. The lift span was increased in width from 22’-0” to 37’-6” center-of-truss to center-of-truss. This in turn provided ample room to accommodate roller bearings and the associated working area required. This was listed as a disadvantage in the tower redesign discussion due to the structural design implications but is an advantage for the machinery design.

Once the geometric issues were resolved, the question was raised as to whether the machinery could now actually be downsized with the efficiency of the roller bearings. Since the motor horsepower could be reduced by at least 17%, could there be additional savings in use of a smaller motor and thus reducing the capacity of the remainder of the span drive machinery? The most significant material costs identified were the speed reducers and the operating drums/ropes. Discussions took place with two major reducer manufacturers, and both indicated that while there would be some savings associated with slightly lower capacity reducers, the savings would not be significant. Similarly, reducing the diameter of the operating ropes would result in a smaller operating drum, but the savings here, again, were not significant. It was decided that the engineering effort required to significantly redesign the machinery would not result in meaningful savings, so the motor size was kept at 150 horsepower.

This significant increase in machinery efficiency obviously does not come without an associated cost. The natural question is, how much?

The 2009 engineer's estimate for the counterweight sheaves, shafts and bearings was $6.48MM. For this bid item the low bid was $4.2MM, the high bid $6.72MM, and the average was $5.3MM. As a comparison, the 2006 bid for the plain-bearing design was $3.65MM. Depending on the factor applied for escalation, in general the roller bearing design was approximately 30-50% more costly than the sleeve bearing design. This was still significantly less costly than increasing the capacity of the entire operating machinery system and the associated structural, mechanical, and electrical engineering costs to implement those changes. Further, the owner realizes cost savings due to these operating efficiencies every time the bridge is opened.

Modifications During Construction

Driven Pipe Piles

After award of the project the contractor submitted a Value Engineering Proposal to modify the foundations by replacing the six 7 foot diameter drilled shafts at each pier with thirty 42 inch diameter steel pipe piles with partial concrete fill. The dimensions of the caps were increased to fit the new layout and some adjustments were made to the reinforcing. Figure 6 shows a photograph of the bridge during construction with piles being driven on either side of the existing Pier 2.

The existing spread footings at Pier 1 and Pier 2 experienced some uniform settling during the pile driving and some horizontal movement perpendicular to the span. The settling was handled by shimming the bearings of the existing Span 1 truss. The horizontal movement was directional with the location of the piles on either side of Pier 2. The contractor was able to control the horizontal movement by alternating on which side of Pier 2 the pipe piles were driven. The horizontal movement was controlled such that no service interruptions were required during construction and after all piles were driven the approach truss alignment was corrected by jacking to reposition the bearings.

FIGURE 6: Bridge during construction, with piles driven at Pier 2B.

Truss Connection Detailing

The bridge truss was originally designed, and then modified by HNTB for the revised width, assuming that the gusset plate connections would be detailed for the final truss shape with dead load. This approach requires that the truss members initially be forced to fit to the connections but that as the dead load of the structure is imposed the truss assumes its final shape and the moments in the members and connections are removed. The contractor requested that the gusset plate connections be detailed and fabricated so that the trusses could be erected without force fitting the members. HNTB revised the original design to include moments due to the connections being fixed prior to application of the dead load. None of the top chord or bottom chord members required modification for this change. Some diagonal members were modified by using a single access hole in the lower flange instead of matching perforations. None of the gusset plates required modification.

Operating Rope Tension Adjustment

One issue which was the source of much discussion was the operating rope take-ups. These are mounted at the end of each operating rope attached to the tower. They are used to adjust the tension in the operating ropes by taking up slack or relieving tension as required. The original design incorporated a 24" travel machine screw jack. The constraints provided by the geometry of the brackets and take-up rods provided a useful adjustment travel of 5-7/8". The Contractor became concerned that this would not be a sufficient amount of travel to remove the "construction stretch" from the ropes during installation of the lift span. As the lift span float-in had a rather short navigational channel outage, the Contractor did not want to risk potentially spending time during this outage having to take the slack out of the operating ropes at the drum end of the rope. The ropes are 6x25 filler wire construction with a fiber core, prestretched at the manufacturer. Using the guidance provided in the Wire Rope User's Manual, allowing for the more conservative end of the range of approximately 1/2% to 3/4% of the rope length as construction stretch, the construction rope stretch was estimated to be 15 1/4", but this does not account for any benefit gained by pre-stretching the ropes. Again using the Wire Rope User's Manual, the elastic stretch was also estimated. For ropes such as this which as loaded to a small amount of their capacity, the elastic stretch was estimated at 2 13/16". Combining the estimated construction stretch with the estimated elastic stretch yielded a total of 18 1/16".

The take-up rods and brackets were modified in several iterations, ultimately allowing 24" of useful adjustment travel. The Contractor was still not comfortable with this based on advice from his Engineer, but at this point, there was no reasonable way to obtain more travel from this basic system design. The Contractor then proposed use of a wedge-type socket. These are often used in the running ropes of cranes and derricks. In these sockets, a steel "wedge" use used to lock the rope into place in the socket. As the load on the rope is increased the wedge seats more firmly. These were initially not approved by the Engineer because their design inherently is less efficient than the spelter-type socket most commonly used on movable bridges. After further discussion, the operating rope calculations were revisited in more detail. With the reduction in capacity due to the socket design, the wire rope assemblies would be very near their allowable capacity at the maximum design loading condition. Since ultimately the rope assemblies would be within their allowable capacity, with the Railroad's concurrence the use of wedge-type sockets was allowed. Modifying the position of the rope within the socket is the "coarse" adjustment, while fine-tuning is performed with the machine screw jack system.

Conclusion

The CSX Mobile River Bridge was changed out in one day, with the swing span floated out and the lift span floated in, allowing barge traffic to be under way again on the river within 72 hours. The train outage was 40 hours.

Acknowledgements

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