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I Street Vertical Lift Bridge Concept Development and Design

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Introduction

The existing I Street Bridge, connecting the cities of Sacramento and West Sacramento, and spanning over the Sacramento River, was originally built in 1911. It is currently owned and operated by the Union Pacific Railroad. The existing bridge consists of a double deck through truss swing span and simply supported through truss approach spans. The lower deck of the bridge carries railroad traffic and the upper deck carries vehicular and pedestrian traffic. The bridge has a narrow cross section with only two 9 ft traffic lanes and two 5 ft pedestrian walkways on the upper deck.

Figure 1 – Existing I Street Bridge

As part of a large revitalization project to update and develop the railyards district of Sacramento, a need arose for an improved crossing over the Sacramento River. An emphasis was placed on providing bridge alternatives that maintain a "neighborhood friendly" river crossing. Neighborhood friendly was defined as having a "low profile, one easily integrated into the surrounding communities, and that will accommodate not just cars, but pedestrians, bicycles and possibly trolley cars". Due to the project needs, the profile of the new bridge was kept as low as possible while leaving room for the bridge structure to still be located above design high water. Structure types were chosen for the initial study to minimize the distance from the underside of the structure to the profile grade. These requirements necessitated the need for through type superstructures and almost immediately eliminated the fixed bridge alternatives.

The United States Coast Guard defines the navigational channel width as 278 ft. with a minimum low steel elevation of 92.5 ft. NAVD88.

Once the new bridge is built, the existing I Street bridge will no longer be used for vehicular traffic. New bike paths that are part of the river front revitalization will connect to the existing I Street bridge to take the place of the vehicular upper deck. The lower deck will continue to be utilized by the railroad.

Concept Development

During the feasibility stage of the project, it was determined that the fixed bridge alternative was not viable, so the focus was to determine the movable bridge type that would be the most feasible given the project constraints. Swing span and single leaf bascule alternatives were considered, but were quickly eliminated due to the large channel width. The three movable bridge types considered feasible were as follows:

- Double Leaf Bascule
- Four Leaf Bascule
- Vertical Lift

Preliminary conceptual designs were developed for each of the three alternatives to get baseline construction costs. An alternative evaluation matrix was developed which scored each of the alternatives within the following categories:

- Performance
    - Constructability
    - Environmental and Site Impacts
    - Mobility and Connectivity
    - Future Streetcar
- Construction Costs
- Life Cycle Cost Considerations
    - Future Inspection
    - Future Maintenance
- Aesthetics and User Impression
    - Neighborhood-friendly structure
    - Aesthetics

After all alternatives were scored within each category, weightings of each category were applied in several different increments to obtain a sensitivity of the weightings. From this study it was clear that a vertical lift bridge was the preferred movable bridge type to be advanced for further consideration.

Since this bridge is to be in a prominent location as part of a revitalization project, the bridge aesthetics were deemed to be a critical aspect of the project. An architect was brought onto the project team to develop concepts for a vertical lift bridge. In total, seven unique concepts with several sub-concepts were developed. All concepts were evaluated through the project team, the bridge owner, and the local community. At the end, two alternatives separated themselves from the rest of the pack – the Thru alternative and the Spring alternative, see Figure 2. The selection committee was a dead split between the two alternatives. However, it was noted by several committee members that the Spring alternative would be the preferred alternative if the towers could be slimmed down to make the scale of the towers more in line with the slender scale of the network tied arch.

Figure 2 - Thru and Spring Alternatives

The initial larger scale towers were required due to the need for two sheaves per tower which was necessary due to the weight of the structure at the time. When the comment about the large towers was received, Modjeski and Masters investigated methods to help reduce the weight of the lift span so that a single sheave per tower would be possible. At the time, an exodermic deck was assumed due to the desire for a solid surface deck for the roadway, bike path and sidewalk. To reduce the weight of the span, an aluminum solid deck was investigated, and the weight of the lift span was able to be reduced to a level where one sheave per tower was achievable. Thus, the Spring alternative was selected for final design.

Figure 3 - Spring Alternative with Slender Towers

Final Design

The spring alternative that was selected for final design consists of a basket handle network tied arch lift span with two lift span piers each having two independent pylons to support the sheaves. The center span length of the lift span is 308 ft. The cross section of the lift span consists of three 12 ft traffic lanes, two 2 ft. shoulders and two 6 ft. bike lanes centered on the lift span. Beyond the traffic barriers that are adjacent to the bike lanes are two 10 ft. sidewalks and two variable width overlooks. The initial inclination angle of the arch ribs was 25 degrees, and the maximum curb-to-curb width of the bridge was approximately 125 ft. at the center of the lift span. As the design progressed further and as additional architectural features were added to the bridge, the weight and the cost of the lift span kept increasing. As was previously mentioned, an aluminim deck was selected to reduce the span weight. However, this deck type is not the most economical choice not only because of the cost of the deck but also because it was unable to provide composite action with the stringer and floorbeams. Therefore, the steel weight increased which caused the cost of the lift span to rise correspondingly. Significant changes were needed to bring the cost back within budget. These changes consisted of reducing the sidewalk widths from 10 ft. to 8 ft, reducing the variable width overhang and changing the arch inclination

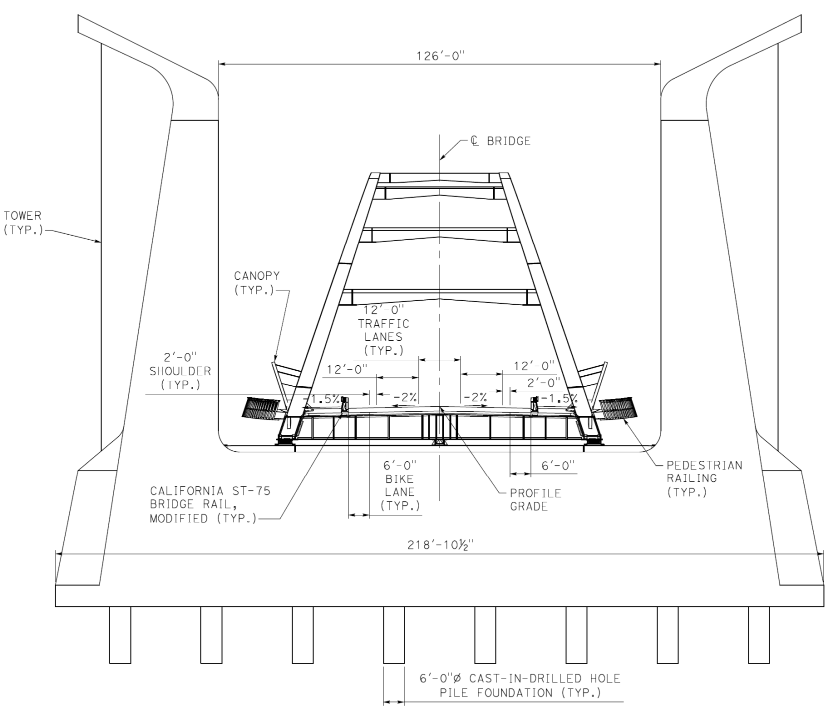


Figure 4 - Typical Section (18.5 degrees)

angle from 25 degrees to 18.5 degrees. These cross-sectional changes reduced the weight enough to allow the deck to be the heavier, but less expensive, exodermic deck. With all the changes made, both the weight of the lift span and the cost of the lift span were able to be reduced from the initial version. The current weight of the lift span is approximately six million pounds, and the maximum curb-to-curb width of the bridge is approximately 109 ft. at the center of the lift span.

Design is in accordance with AASHTO LRFD Bridge Design Specifications and Caltrans Amendments to AASHTO LRFD Bridge Design Specifications. The major differences for the Caltrans Amendments are the requirements to design for an additional Strength II load combination using a P15 Permit truck with a load factor of 1.35 and an impact factor of 25%. Also, an additional Fatigue truck, P9 Permit truck, is to be used for fatigue design with a load factor of 1.00 and an ADTT of 20.

Aesthetic Design Features

In addition to the complex design features of the vertical lift bridge, there are several other aesthetic design features that will be incorporated into the design which consist of the following:

- Shade awnings with a structural steel frame and stretched fabric covering will be provided in the bench/overlook area to provide a refuge from the California sun.
- Raised wooden, black locust, benches are provided where the tied arch hangers interface with the deck. The bench ends before the end of the span and a flat wooden area is provided connecting the pedestrian walkway with the overlook area.
- Open grid aluminum panels will be provided outboard of the wooden bench to provide a see-through deck to the river below.
- Rear faces of the vertical lift towers will have a translucent stretched fabric.
- Color changing lights will be provided along the arch rib and within the vertical lift towers that shine through the stretched fabric. Lights will also be provided in the upper lateral members for street lighting. The bridge railing will have lights within to create an ambient glow for the pedestrian path. The pedestrian railing on the overlook will have lighting below the handrail to light the overlook area.
- Arch rib and upper lateral members will be connected with hidden splice plates to provide a seamless transition between member segments.
- Aesthetic panels will be provided over the knuckle to exaggerate the size of the knuckle and to provide transition control lines that transition to match the approach span concrete barrier.

Network Tied Arch

The network tied arch lift span consists of five tie girder segments, five arch rib segments and two knuckle sections connecting the arch rib and tie girder. The arch is parabolic with a maximum rise of 75 ft. at the center of the span and with an 18.5-degree inclination towards the bridge centerline, see Figure 5. There are twenty hangers per arch span. The tie girders are parallel with the longitudinal profile grade to simplify floorsystem connections.

Tie girders consist of bolted built-up parallelogram sections with horizontal flanges and 18.5-degree inclined webs. Perforations are located at the top of the top flange of the tie girder to allow for passage of the hangers which are reinforced with cover plates. There are also perforations in the bottom flange with reinforcement plates and a lockable door to gain access to the inside of the tie girder. The webs and flanges are connected by bolts to provide internal redundancy since the tie girders are non-redundant fracture critical members. The flanges consist of two plates to minimize the effect of a flange plate fracturing and becoming ineffective for the redundancy design condition. The AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members was used for the redundancy design.

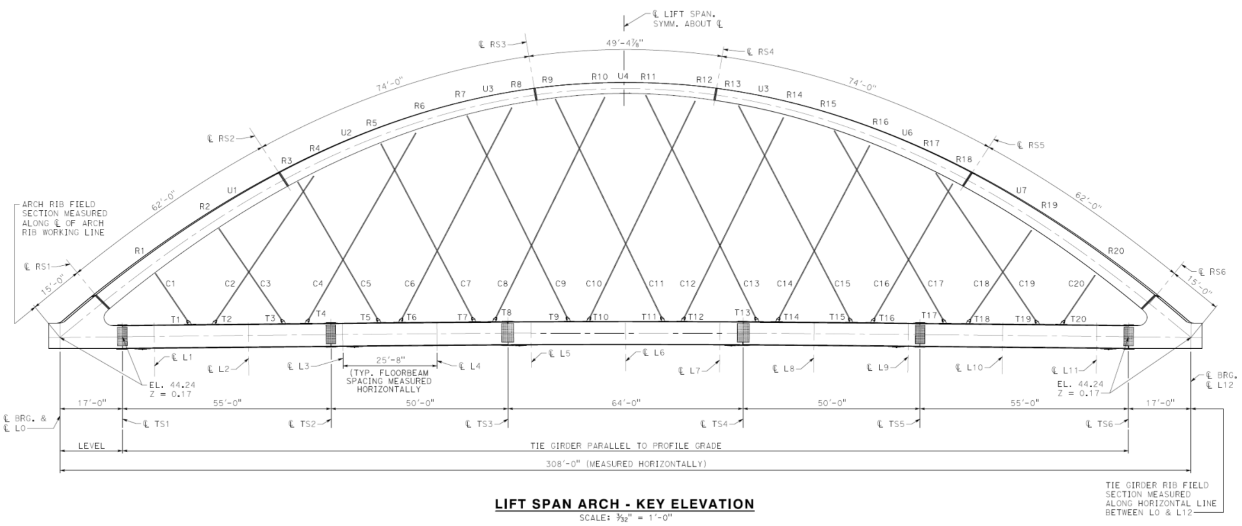


Figure 5 – Network Tied Arch Elevation

The arch ribs also consist of parallelogram sections, but unlike the tie girders, they are welded at the web to flange interface. The arch rib boxes taper in width and depth from each end towards the highest point on the arch. The dimensions of the arch rib at the ends are 56" wide by 60" tall. The smallest section at the center of the bridge is 32" wide by 36" tall. Reinforced access perforations are provided adjacent to the field splice locations for access to make splice connections. However, the length of the arch between splices will not be accessible due to the configuration of the hanger connections and inside dimensions of the arch.

Due to the basket handle configuration of the network tied arch and the tension thrust force imposed on the end floorbeam, it was also designed to provide internal redundancy due to lack of system redundancy. The end floorbeam is a bolted built-up I-shape member. Unlike the interior floorbeams which are simply supported, the end floorbeam is designed to provide a moment connection with the knuckle. The AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members was used for design of this member. Redundancy conditions were evaluated for the bottom flange in the positive moment regions and for the top flange in the negative moment regions near the connection with the knuckles. The moment flange connections with the knuckle were also evaluated for internal redundancy with the flange connection split into two parts for a single flange.

Parallelogram Design

The parallelogram configuration of the arch ribs and tie girders was a challenging design feature. Since the members are not symmetrical about either axis, the principal axis rotation angle was determined to evaluate the sections about their major and minor principal axes. The other complication was that the arch rib varies in out-to-out dimensions along the length of the arch which causes the principal axis to vary along the length of the arch. However, the variation in the principal axis angle was always less than one degree between arch splices, so the difference was minimal. The primary principal axis rotates from 41 to 44 degrees from vertical for the arch rib. The tie girders primary principal axis varies from 31 to 33 degrees, but it remains constant between splices since the section is constant between splices.

LUSAS was used for the 3D global analysis. Local coordinates were defined within the model for the parallelogram sections such that the analysis model would envelope the maximum and minimum stresses about the major and minor principal axes. As mentioned, the principal angle from one end to the other of an arch rib splice section varied because the outside dimensions of the box are tapering from the end to the center of the span. Since the difference in angle from one end to the other was different by less than one degree, the average principal axis rotation angle was used to define the local coordinate system for each individual splice section.

Knuckle

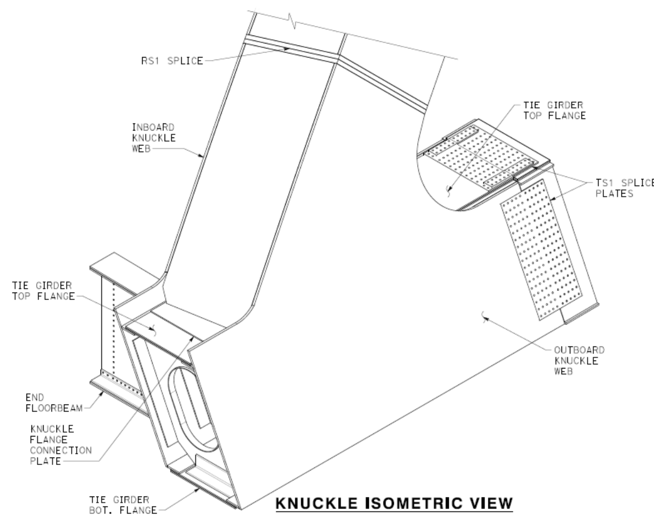
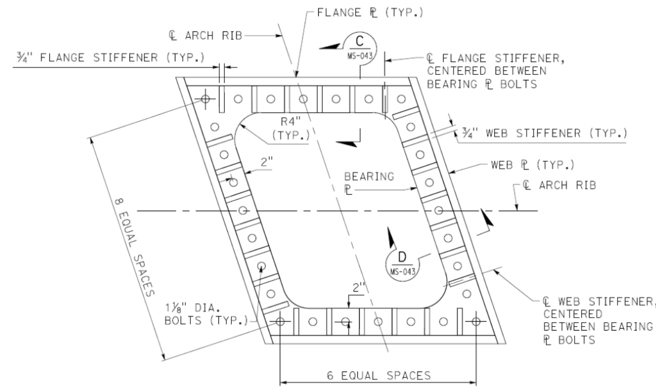


Figure 6 – Knuckle Detail

The knuckle is one of the most congested areas of this entire bridge. It is the interface of the arch rib, tie girder, end floorbeam and lift span bearing. The web of the tie girder and web of the arch rib are to be fabricated from the same planar plate. However, since the tie girders are parallel with the profile grade, it was realized that for the web plate to remain planar, the knuckle section of the tie girder had to remain level. So, the last segment of the tie girder within the knuckle is level up to the first tie girder splice. Connection tab plates are CJP welded to the web plates of the tie girder to provide a location to bolt the tie girder flange plates. The arch rib flange plates are welded to the arch rib web plates. A connection plate is welded to the arch rib flange that is then bolted to the top flange of the tie girder to minimize fatigue and redundancy impacts to the tie girder.

In addition to all the structural member congestion at the arch, aesthetic cover plates will be provided outside of the structural knuckle. The architectural plates will further pronounce the flared size of the arch rib and provide transition lines to the approach concrete barriers. The architectural panels will be detailed as either removable panels or with access hatches to allow for proper inspection of the critical knuckle region.

Hidden Arch Splices



ARCH RIB RS3 SPLICE

Figure 7 – Hidden Arch Splice Section View

A conventional splice connection for a steel box member consists of plates that are bolted to the inboard and outboard faces of each plate building up the box section. The typical splice was deemed not to be desirable for the aesthetics of the bridge. Therefore, a bolted splice was developed to hide the bolted components for the arch rib splices and the upper lateral splices. The system that was developed aligns the bolts parallel with the axis of the arch rib connecting through end bearing plates that are perpendicular to the axis of the bridge. Stiffeners are provided to brace the end bearing plate and to transfer the force from the bolts through a welded shear connection with the web and flange plates. The arch rib members are predominantly in compression and most loading conditions only require the end bearing plate to transmit the force through bearing. However, per AISC guidelines, a design condition was considered where 50% of the required compressive strength was assumed to act in tension which ended up as the governing load case.

Hangers

Hangers on a network tied arch typically consist of either bridge strand or stay cables. Stay cables offer better protection than a bridge strand would because of strands are fully encapsulated and isolated from one another. However, since the strands are individualized, they are required to be tensioned individually which increases the size of the tensioning equipment. With the size necessary for the arch rib, it would not be feasible to fit the jacking equipment inside the box. A hole could be provided for the jacking equipment to protrude outside of the box, but the accessibility at the top of the arch for tightening would not be desirable. Similarly, the tie girder dimensions did not leave ample room to tighten the stay cable strands inside of the box and due to the non-redundant nature of the tie girder, it was preferred to minimize the openings in the tie girder. Therefore, to simplify details and ease of installation, it was decided that bridge stand would be best suited for the hangers. The climate at the project location did not give many concerns for the decrease in protection from the elements.

At the arch rib hanger connection, a shark fin style connection was provided for connection to the bridge strand with an open strand socket end detail. At the tie girder hanger connection, a spreader beam is located inside the tie girder spanning between tie girder webs. The hanger is equipped with a Type 7 socket for tension adjustability. At the interface of the top flange of the tie girder and the hanger, a watertight covering will be provided to keep water out of the tie girder.

Lifting Girder and Span Restraints

The lifting girder is located below the knuckle of the tied arch. A PTFE spherical bearing will be provided between the knuckle and the lifting girder. At the fixed end of the bridge, the spherical bearings will be restrained longitudinally and unrestrained transversely. At the expansion end of the bridge the spherical bearings will be unrestrained both longitudinally and transversely.

At the center of the end floorbeam, a transverse restraint will be provided between the end floorbeam and the lifting girder. A similar detail will be provided at both ends of the bridge except that the expansion end of the bridge will have a longer restraint system to accommodate thermal expansion/contraction and seismic displacements.

A fixed bearing will be provided beneath the lifting girder which is connected to the lift span tower. This bearing will be located directly beneath the spherical bearings located at the top of the lifting girder. The bearing will be designed to have inclined faces with enough height to restrain the lifting girder from rotation about the transverse axis of the bridge. These bearings will be identical on both ends of the lift span.

An additional transverse restraint will be provided to constrain the bottom of the lifting girder to the top of the lift span pier. A portion of this restraint will remain fixed to the lifting girder while the other half will be connected to the lift span tower. This restraint will transmit the transverse loading from the lift span to the lift span tower in the closed position and will also act as a centering device.

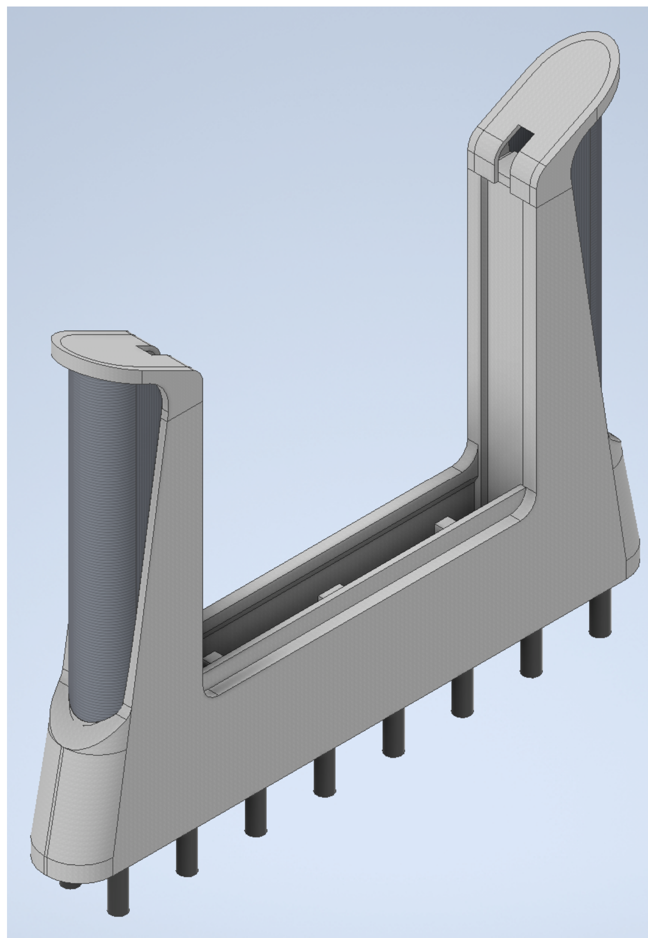


Figure 8 – Lift Span Tower

Lift Span Towers

The vertical lift span towers will each be founded on sixteen 6 ft. diameter drilled shafts with permanent steel casings. The pier walls and towers will comprise of conventionally reinforced cast in place concrete. The back faces of the towers will have a translucent stretched ETFE fabric that allows for aesthetic backlighting within the tower. The machinery and electrical equipment will be located inside the lower part of the pier between the tower legs. Counterweights will be fully enclosed within the tower legs.

On the channel side of the lift span tower, a notch will be provided for parking of an inspection traveler. The inspection traveler will be provided for full width access to the underside of the floorsystem. The traveler will be designed to allow for decoupling from the lift span so that the traveler remains in the parking area on the pier while the lift span operates.

Three columns are located inside the lower part of the pier that cantilever up from the pier foundation. The two outer columns support the vertical and longitudinal load from the lift span bearings. The center column is for support of the transverse lift span loading. These three columns are designed to form plastic hinges at their base to allow for a capacity protected condition of the vertical lift span in the closed position.

Operating Machinery

The lift span machinery will be a modified span drive configuration where the machinery is predominantly located inside the lower portion of the tower, beneath the lifting girder. Two similar sets of machinery will be provided at each lift span tower. The primary reducer and two 100 horsepower motors will be located at the centerline of the bridge. Line shafts will extend out towards the tower legs where the secondary reducer and an operating drum will be located. Counterweight sheaves will be the only machinery components locate at the top of the tower. Deflector sheaves will be fastened to the bottom of the lifting girder, to the bottom of the counterweight and to the top of the pier directly beneath the counterweight which will be used in conjunction with the operating ropes to pull down on the counterweight to open the lift span and to pull down on the lifting girder to close the lift span. Anchor points for both ends of the operating rope will be equipped with take-up mechanisms.

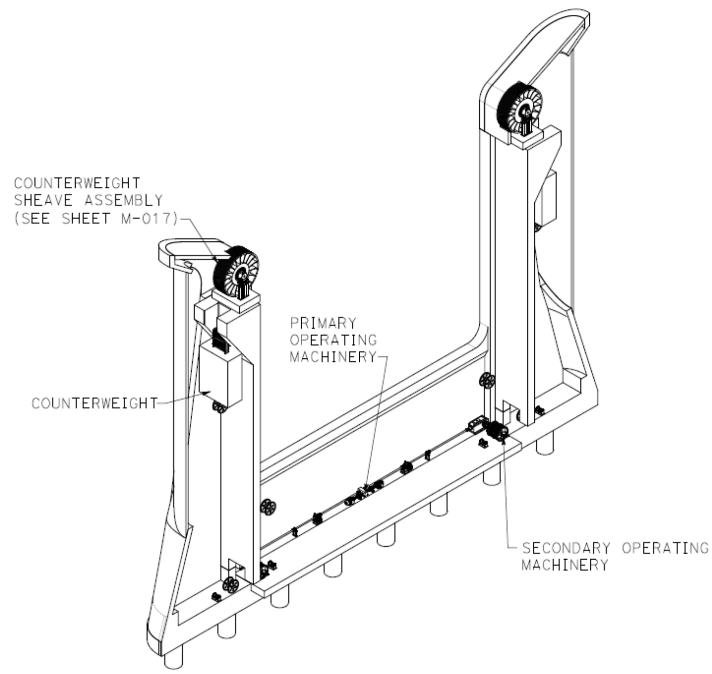


Figure 9 - Modified Span Drive Machinery Layout

A 17 ft diameter sheave will be provided at the top of each tower leg (four total). Twenty-two 2-3/8" diameter ropes per sheave will be used to connect the counterweight to the lifting girder. Spherical roller bearings will be press fit into the sheave hub to minimize the overall width of the sheave assembly which was a critical design feature to ensure the slender lift span tower was achievable.

3D Modeling

Autodesk Inventor is being utilized to model the lift span, towers, and machinery in 3D space which will then be used to generate 2D contract drawings. While not required, a 3D model was deemed necessary for this project due to the complex geometry of the network tied arch lift span. Inventor was chosen to create the model because M&M has been using this software for well over a decade. The project is currently at the 65% level of design, but the visualization of this bridge in the 3D atmosphere has proven to be critical to the project's success thus far. The geometry on this lift span is complex and the 3D modeling has helped to find clashes/conflicts that may not have been found if the drawings were generated using conventional 2D methods.

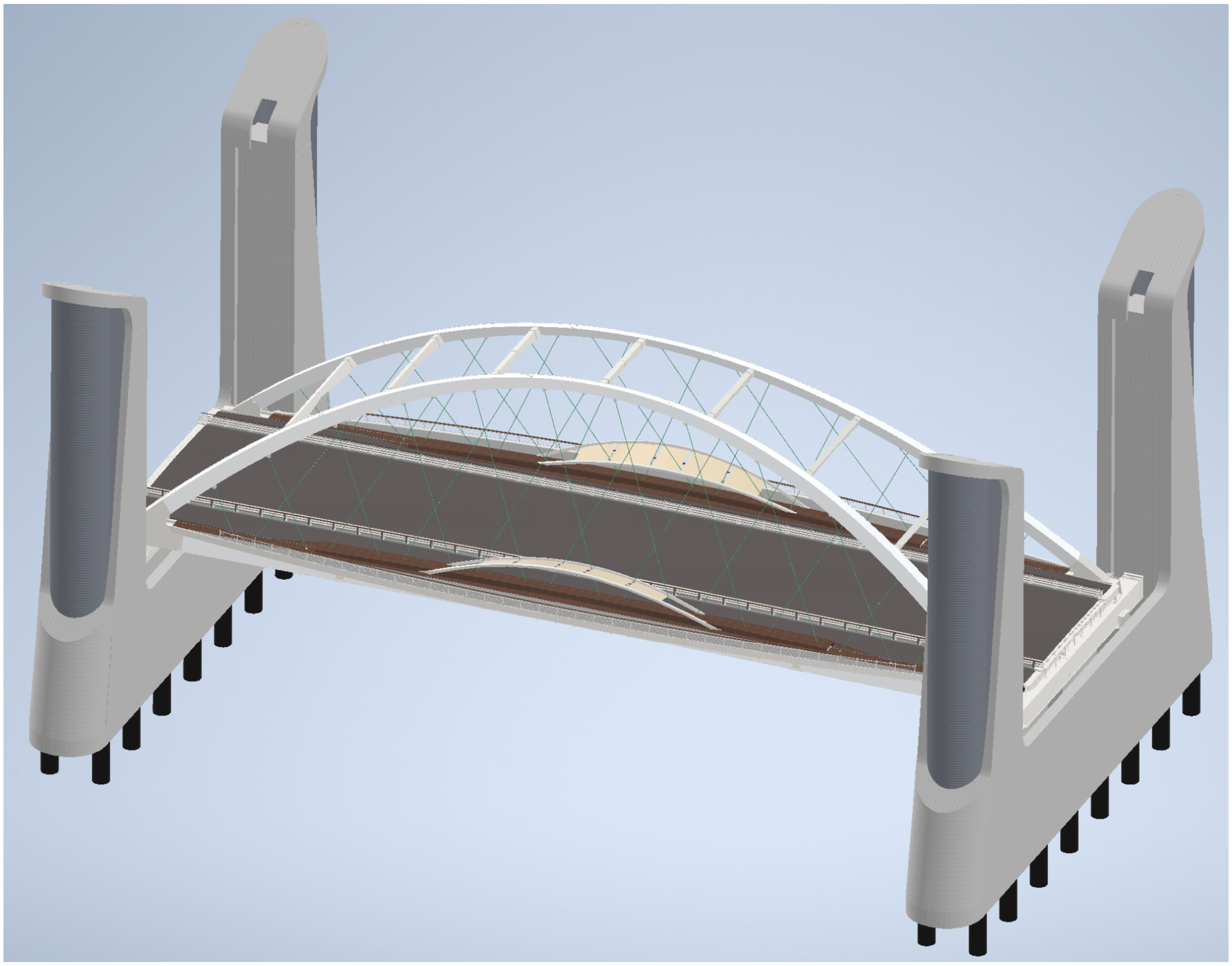


Figure 10 - Inventor Model of Lift Span and Towers

Being that this was one of the most complex structures to be modeled in 3D, there have been some learning curves and obstacles that have had to be overcome.

- First, is that when modeling in 3D, the modeler needs to have a clear direction and understanding of what the final product will look like so that the part or assembly can be efficiently built.

- Second, the 3D model file sizes become very large with many different parts, assemblies, and subassemblies on a complex structure like I Street. The large files can cause a delay in work efficiency due to waiting for files to open or cause a lag if the user's computer specifications are not sufficient.

- Third, the initial generation of 2D drawings is delayed compared to conventional design. While the 3D model helps to very easily create initial drawings, sections and details, the 2D drawing cannot be generated until all the components that make up the detail are generated. For example, a general plan and elevation and a typical section are two of the first drawings that are generated for a project. If those drawings are generated from the 3D model, all the main member components would need to be modeled before those drawings could be generated. Also, there are drawing like a steel framing plan that still require 2D drawings since there is normally exaggerated linework offset from work points to show whether a beam is simply supported or continuous.

- Finally, the best practice seems to be where the modeler is either the designer of the component they are modeling, or at least have a design mentality. With a program like Inventor, there are many different approaches that one can take to model a part or assembly. However, to take full advantage of the parametric qualities, the modeler needs to understand what the control geometry is for the part they are modeling and how some geometry could change if there are future revisions. This is not to say that technicians no longer have a role, but it will require more direction from the engineers during the modeling process.

The path forward with 3D modeling appears to require a shift from the conventional 2D design workflow. For a 3D project to be successful it seems like most, if not all, of the designers and the project managers need to become fluent in 3D modeling or at least be able to navigate and dimension from the 3D model. However, until owners and clients also become proficient in 3D models, submissions will continue to be 2D drawings which will necessitate developing them.

Conclusion

The I Street vertical lift bridge project has blossomed into what will become a stunning signature bridge and a major focal point of not just the riverfront and railyards revitalization project, but for the cities of Sacramento and West Sacramento overall. Many different design aspects and visual aspects make this project unique. This project has been a challenging and rewarding project and we look forward to seeing it through to completion.