Design Challenges of Galveston Causeway Railroad Bridge Replacement

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Abstract

The 125-ft rolling leaf bascule span of the Galveston Causeway Railroad Bridge over the Intracoastal Waterway has been replaced by a 383-ft vertical lift span to increase the navigation channel from 100 to 300 feet, by the Order to Alter the Bridge issued by the United States Coast Guard under the provisions of the Truman-Hobbs Act. The bridge, owned by the County of Galveston and operated by the BNSF Railway, consists of long concrete arch approaches to the movable span and carries a single railroad track across the Galveston Bay. This paper presents the unique design challenges and creative solutions related to the structural aspects of this complicated project. A total of 15 replacement alternatives were developed and evaluated based on track alignment and profile, span arrangement and efficiency, foundation type, arch modification, ease of construction, span change operation, interruption to rail and marine traffic, vessel collision, life-cycle cost, environmental impact, aesthetics, and railroad preferences. The alternative selected was a new vertical lift bridge with piers on H-pile foundations located between the existing arch piers in a track alignment parallel to and 24'-10" from the operating track. The arch spans adjacent to the existing bascule span had to be removed along with the abutments that anchored the arches. The tight construction clearance required the existing bascule span to be operated on one pinion and the live track to be carried by a narrow strip of arches. The new footings were designed to not only support the lift span and towers, but also serve as new abutments for the truncated approach arches. Horizontal struts connecting the arch stems were used to resist arch thrust forces during the footing construction. Because of the extremely tight clearance, the lift span truss is 8 feet narrower than the towers and it required creative span guide arrangement and design. Also, post-tensioned anchor bolts and heavy grillages were employed to resist hurricane wind loads on the 164-ft towers. The lift truss span was also designed to carry its own weight with the wind and a moving utility vehicle on the grid deck comparable to H20 loading in the event of a bottom chord member being severed by a barge impact when the span is in the down position. The intent of this requirement is to prevent span collapse under the severed bottom chord condition and to facilitate repairs. The traditional method of enlarging stringers and floorbeam flanges to carry the forces in the chords would have yielded a rather bizarre, unreasonable, and extremely heavy floor system, and thus increased the structural, mechanical and electrical demand and cost for the entire bridge. Through a holistic approach that included a 3-D analysis to determine the accurate load paths and took advantage of the internal redundancy of the space truss and the presence of the span guides, this special design requirement was met by only marginally increasing the lateral and sway member sizes. The innovative solutions to unique design challenges associated with this replacement bridge helped create an efficient and cost effective structure.

1.0 INTRODUCTION

1.1 Historic Background

The Galveston Causeway over the Intracoastal Waterway in Galveston, Texas is the only rail link between the mainland and the Galveston Island as shown in Figure 1. The Causeway is owned by Galveston County. Original construction was completed in 1912. The Burlington Northern & Santa Fe and Union Pacific, as successors to the Gulf, Colorado & Santa Fe Railway Co., Galveston, Houston & Henderson Railroad Co. and Galveston, Harrisburg & San Antonio Railway Co., are tenants of the county under a 999 year lease dated December 15, 1908. BNSF operates and maintains the Causeway and associated tracks as a joint facility of BNSF and UPRR.
As originally constructed, the Causeway consisted of a 125’ Scherzer rolling lift bascule span on concrete piers carrying two railroad tracks, an interurban track and a 16’ roadway as shown in Figures 2 and 3. Approaches consisted of fourteen 70’ concrete arches on each side and embankment with concrete protection beyond the arches. The 1915 Hurricane severely damaged large portions of the protected embankment. Following that storm twenty-eight 60’ arch spans were added on the Island end and fifty-one 60’ arch spans on the Mainland end. This work was completed in 1922. Prior to construction of a parallel four lane highway bridge in 1939, the two-lane roadway on the original causeway was the only road between the Island and the Mainland. The highway bridge left the original bridge for rail traffic only, although the road was used in several highway bridge closure occasions. The original highway bridge was replaced by a new I-45 Bridge in 2008, which is only a few hundred feet to the west of the railroad bridge. The rail bascule span was replaced in 1988 with a single track span with a grating roadway occupying the same space as the track for railway maintenance vehicles and emergency access as shown in Figure 4. The original causeway was listed on the National Register of Historic Places in 1976.
Figure 2 – Original Causeway

Figure 3 - Original 1912 Bascule Bridge

Figure 4 - 1988 Replacement Bascule Bridge
1.2 Truman-Hobbs Project

The Galveston railroad bridge is uniquely the most hazardous bridge on the entire Gulf Intracoastal Waterway. More accidents occur as a result of collisions with this bridge than any other bridge on the Gulf Coast. The original horizontal clearance for the navigational channel is about 100 feet for both the rail bridge and the old I-45 bridge. The U. S. Coast Guard declared the bridges a hazard to navigation in 2001. The new I-45 bridge completed in 2008 has a 300 feet horizontal navigation clearance, and temporary dolphins were built to guide the vessels to transit between the 100 feet and the 300 feet navigation channels, as shown in Figure 5. By the Order to Alter the Bridge issued by the United States Coast Guard under the provisions of the Truman-Hobbs Act, the existing bascule bridge needed to be altered to meet the 300 foot clear channel width requirement of the expanded waterway.

Figure 5 - Navigation Channel Clearances
1.3 Existing Bascule Bridge and Arch Approaches

The existing railroad bridge consisted of a 125-foot single track steel rolling lift bascule span with concrete arches on each side of bridge as approaches. The approach arch spans were built to accommodate three railroad tracks and a roadway, although only one track is in service after the original three track bascule span was replaced by a single track span in 1988. The track was realigned at the 1988 bascule span to 24'-10" from the original east track, as shown in Figure 6. The arch spans are typically 78'-0" in length each, except those immediately adjacent to the bascule span are slightly longer. Railroad tracks and the roadway are supported by separate arches with railroad arch being thicker than the highway arch. The existing bascule span foundations on either side of the navigation channel served as abutments which resisted the thrust forces transmitted longitudinally through arch spans in each approach. The concrete arches are lightly reinforced and supported by timber piles. The arches are in fair condition. Figure 7 and 8 show the elevation at the bascule bridge and the crown section of a typical arch span.
2.0 REPLACEMENT ALTERNATIVES

At the very beginning, it was determined that a single track vertical lift bridge was the appropriate type for the replacement structure considering the horizontal and vertical clearance requirements. A comprehensive study of replacement alternatives was conducted and yielded a preferred alternative for preliminary and final design.

2.1 Alternative Criteria

All alternatives were developed to meet the following criteria:

- Minimum Horizontal Clearance of 300 feet
- Minimum Vertical Clearance of 73 feet above mean high water for lift span in open position
- Minimum Vertical Clearance of 8 feet above mean high water for lift span in closed position
- Continuous Railroad service during the construction except for prescheduled short track outages
- Continuous service to the Intracoastal marine traffic during the construction except for prescheduled closures

2.2 Alternative Development Parameters

2.2.1 Track Alignment

The following four track alignments were considered:

- Alignment 1 - 50' North of the Old Track No.1
- Alignment 2 - 9' North of the Old Track No.1
- Alignment 3 - Old Track No.1
- Alignment 4 - Current Operating Track, 24'-10" South of the Old Track No. 1

Alignment 1 was chosen so that the new structure will be completely clear of the existing structure. Alignment 2, with the new structure partially within the limits of the existing structure, provided more construction clearance for the new bridge than Alignment 3 and Alignment 4. Alignment 3 used the original alignment for Old Railroad Track No. 1 while Alignment 4 used the current operating track alignment between the original Track No. 2 and the Interurban Track.

2.2.2 Track Profile

The track profiles under consideration were as follows:

- Current Track Profile
- Raised Track Profile over New Lift Bridge

The purpose of raising the rail profile over the new lift span and towers was to facilitate the float-in operation. With the raised rail profile, the low steel of the new lift span would clear the existing arches so that the arches would not need to be removed prior to float-in of the new lift span. This would drastically reduce the rail track outage needed for the entire float-in operation.

2.2.3 Lift Span Length

The following three lift spans were investigated:

- 316'-8" (10 Panels @ 31'-8")
- 336'-0" (12 Panels @ 28'-0")
- 382'-8" (14 Panels @ 27'-4")
The lift span length was primarily determined by the required minimum horizontal clearance and the location of the proposed substructures. 316'-8" was the shortest lift span length that met the 300' minimum horizontal clearance. This span length was used when the new substructures are off the existing structures and have no restriction of their placements. The longest lift span (382'-8"), however, would be used when the substructures are placed between the two existing piers for the ease of construction. When the existing piers would be used to support the new lift span and towers, the lift span length (336'-0") was determined by the locations of the existing Piers 13 and 14.

2.2.4 Span and Tower Width

The following combinations of span and tower widths were used, which were primarily based on the construction clearance and/or structural efficiency.

- 22'-6" Span / 22'-6" Tower
- 22'-6" Span / 28'-6" Tower
- 20'-6" Span / 22'-6" Tower
- 20'-6" Span / 28'-6" Tower

2.2.5 Foundation and Pier Type

The following foundation and pier types were considered:

- Pile Foundation with Pier Shaft – Independent New Pier
- Drilled Shafts with Cap – Independent New Pier
- Combined Pile Foundation with Pier Shaft – New Pier shares Pile Foundation with New Arch Abutment
- Existing Piers – New Towers are supported by existing arch piers

The new bridge would be supported by either the new piers or the existing arch piers. The existing piers would need certain modifications and/or extensions to provide adequate space for lift span and tower bearings. The type of new pier and foundation was primarily chosen based on its constructability.

2.3 Summary of Alternatives

Six alternatives with a total of 15 varied versions were developed based on the above criteria and design parameters. The conceptual layouts for each alternative, which include plan & elevation, alternative features and potential advantages/disadvantages, are presented in Figure 9. For Alternative 3 to 6, each has the following three versions representing three different track alignments and profiles:

- Version A – Alignment 3 / Current Rail Profile
- Version B – Alignment 4 / Current Rail Profile
- Version C – Alignment 4 / Raised Rail Profile

Alternatives 1 and 2, using Alignments 2 and 1 respectively, were the original replacement concepts by US Coast Guard. The difference between Alternatives 2A and 2B is the substructure type. Alternative 2A is of pier on pile foundation type of construction while Alternative 2B uses drilled shaft piers. Those two alternatives employ the shortest lift span, which would result in savings in towers, foundations, and electrical and mechanical components. However, they also required long new approaches, which represent significant cost increases.

Drilled shaft piers constructed between existing arch piers were used for Alternatives 3. For Alternative 4, the lift span and towers would be supported by the pier shafts on the pile foundations shared by the new arch abutments. For Alternative 6, the lift span and towers would be entirely supported by the modified existing arch piers. Alternative 5 also reused the existing arch piers but with rear tower legs supported by the new arch abutment.
Figure 9 – Replacement Alternatives
Table 1 summarizes parameters described above and the following important features for each alternative.

- Approaches – New / Partially New and Partially Reuse Existing / Reuse Existing
- Pier Construction – On or Off Operating Track / On or Off Existing Arch
- Removal of Existing Arch and/or Pier for New Pier Construction
- Arch Pier Modification – Length Extension and/or Width Widening for Seat
- Track Outage for Float in-out Operation

### Table 1 – Summary of Alternatives

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<th>Foundation / Pier / Tower</th>
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<th>Pier Construction</th>
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### 2.4 Evaluation of Alternatives

#### 2.4.1 Critical Issues

The following are factors which were considered to be important to the project:

- **Track Alignment and Profile**
  - Alignment
  - Profile
- **Lift Span and Tower**
  - Structural Efficiency
  - Tower Configuration
  - Mechanical Considerations
  - Electrical Considerations
- **Pier and Foundation**
  - Foundation Type
  - Extension or Modification of Existing Arch Pier
  - New Arch Abutments
  - Cofferdam
  - Additional Vessel Collision Protection
- **Construction**
  - Track Interference
Construction Clearance
- Removal of Existing Arch Pier and Arch Span
- Temporary Retaining Walls
- Temporary Deck Girder Span
- **Float In-out and Track Outage**
  - Operation of Float-in and Float-out
  - Track Outage
  - Removal of Existing Arch
  - Removal of Existing Bascule Span
  - Removal of Rack Frame
- **Cost**
  - Initial Construction Cost for Lift Bridge and Approaches
  - Life-Cycle Cost
- **Environmental Impact**
- **Aesthetics**
- **Railroad’s Preference**

### 2.4.2 Evaluation Method

Each of the alternatives was evaluated for its quality in each of the critical issues outlined above. A numerical scoring method was employed to compare the relative merits of each alternative. The scoring system consists of a raw score and a weighted score for each critical issue. The methodology used provides a rational way to objectively evaluate a series of subjective qualities. It is particularly helpful for assessing the overall feasibility of an alternative in numerous areas of concerns, and the serves as a guide to the relative merits of alternatives, and as a useful aid to decision making.

**Raw Score (from 1 to 4)**

A raw score of 1 to 4 was assigned for Inferior, Fair, Good and Superior with respect to each critical issue under consideration. For cost, the raw score is calculated by the reciprocal of the cost index normalized to the maximum of score of 4.

**Weights (from 1 to 10)**

A weight was assigned in a scale of 1 to 10, from least important (1) to most important (10).

**Weighted Score = Raw Score x Weight**

For critical issue with sub items, the score for the issue is represented by the weighted average of all sub item weighted scores.

**Total Score = \[ \Sigma \text{Weighted Scores} \] x [Railroad’s Preference]**

The total score of an alternative represents the relative strength of the alternative with respect to the critical issues. The alternative with the highest total score is the most favorable among the alternatives under consideration.

### 2.5 Evaluation Results

The detailed scoring matrix was generated with weights, raw score and weighted score for each alternative and critical issue. Table 2 summarizes the scoring matrix with only the total weighted scores for the 15 major issues discussed above.
Table 2 – Scoring Matrix of Alternatives

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Alternative 4A has the highest total score and was overall the most favorable alternative based on the evaluation criteria and methods applied in this evaluation. Alternative 4A featured a structurally efficient and cost effective foundation arrangement with a common steel pile footing between two existing arch piers for both new arch abutment and the lift bridge. The location of the new arch abutment placement is considered to be ideal. Although this Alternative required the longest lift span, it avoided the construction of costly new approaches or the reuse of the existing arch piers as new bridge supports, as called for other Alternatives employing shorter lift span. Using Alignment 3, the Old Track No.1 which is 24'-10” from the current operating track, Alternative 4 in its offline construction increased ease of construction, minimized the track outage required for float in-out operation, and provided the superior track alignment.

As a conclusion, a new vertical lift bridge with piers on pile foundations located between existing arch piers in the Old Track Alignment was considered as the most favorable replacement option for this project.

3.0 DESIGN CHALLENGES AND CREATIVE SOLUTIONS

Once the replacement alternative was decided there were many design challenges. In this paper, we will focus on three distinctive areas unique to this project and the creative solutions to those challenges.

- Span and Tower
- Arch and Foundation
- Severed Bottom Chord Condition for Lift Truss Span

3.1 Span and Tower

3.1.1 Tight Clearances

The width of the lift span was controlled by the clearance to the existing bascule span counterweight and the width of the tower was controlled by the construction clearance to the live track. Figure 10 shows the new structures and the existing structure side by side for the duration of construction. Figure 11 shows the tight clearance between the new tower and the live track.
3.1.2 Bascule Operation on One Pinion

The east rack gears, rack frame and the concrete base of the existing bascule, as shown in Figure 12, had to be removed to clear the vertical lift span. The clearance was only about 8 inches, as shown in Figure 13. This resulted in the existing bascule span needing to be operated on only one of its two drive pinions and rack frames for a short period of time when the new lift span is in place, but not operational.
The contractor came up with creative measures to rebalance the counterweight with confirmation from instrumentation. The actual bascule operation at a slow operating speed on the single pinion was smooth, quiet, and unnoticeable. Although unconventional, the single pinion operation proved to be very successful and significantly minimized the rail traffic outage.

Figure 12 – Existing Bascule Rack Frame and Concrete Base

Figure 13 – Removal of Existing Bascule Rack Frame and Concrete Base
3.1.3 Span Guides

Because the tower is wider than the span, a special span guide arrangement using separate transverse and longitudinal span guide gears for the fixed end of lift span was developed, as shown in Figure 14 for the fixed bearing end of the bottom chord. For the expansion bearing end, only the transverse span guide is present, with the guide plate on the tower column being T shaped for expansion. There are also span guides at the top chord, which are critical components for meeting the design requirements for the special severed chord condition discussed in Section 3.3.

![Figure 14 – Span Guide Arrangement](image)

3.1.4 Span and Tower Design for Wind

Because of the tight clearance, both the lift truss span and the towers are narrower than normal. The site is in a high wind location and the bridge is susceptible to hurricane wind. The span and towers were designed for AREMA wind loads and additional wind loads for hurricane and erection.

- Regular Wind for Span in Open Position
- Hurricane Wind for Span in Closed Position

Additional wind load combinations:

- DL + HW (70 psf) @ 140% Basic Allowable
- DL + E @ 125% Basic Allowable
- DL + E + EW @ 140% Basic Allowable

In order to design for the following additional wind load conditions, the truss span has a robust lateral system (see Figure 15) and the base of the tower required post-tensioned anchor bolts (see Figure 16) with large grillages embedded in the concrete. The bearing anchorage was designed for no tension between the base plate and the concrete for all the load conditions and associated span positions.
3.2 Arch and Foundation

3.2.1 Integrated Foundations

One of the unique features of this project is that the foundations for the new lift span and towers needed to be structurally integrated with the existing concrete arch approach spans. The old bascule span foundations on either side of the navigation channel served as abutments for the arches and resisted the thrust forces from the concrete arch spans. They would need to be removed along with two of the flanking arch spans to provide a wider channel. The foundations, as shown in Figure 17, would have to support not only the new lift span and towers, but also serve as the new abutments for the truncated existing arch spans. Large thrust forces for anchoring the arches and large eccentricity due to the location of the lift
span presented a design challenge for the new foundation. The foundation is of pile footing type with a large number of HP14x117 piles, as shown in Figure 17. The piles had to be driven through the existing concrete arch, and 24" diameter holes were augured from the deck level through the arch fill within pipe sleeves, and then cored through the arch to facilitate the pile driving, as shown in Figure 18. A cofferdam was constructed for each pier. After pile driving, tremie seal placement, and dewatering of the cofferdams, the H-piles were cut off to the required elevation and new mass concrete foundations were then poured in multiple lifts and in dry conditions within the cofferdams between and against each of the existing arch pier stems as shown in Figure 19.

Figure 17 – Pier Foundation Configurations
Figure 18 – Pile Driving through Concrete Arch

Figure 19 – Completed Pier 14A Foundation
3.2.2 Arch Stability and Capacity During Construction

During the foundation construction and tower erection and throughout the different stages of demolition and construction, a longitudinal strut system was used to connect the two arch pier stems of the arch spans at the water level to resist longitudinal thrust during periods after the concrete arch has been partially removed, and before the new pier concrete has been sufficiently placed and cured, as shown in Figure 20. This was critical for maintaining stability and providing sufficient capacity. Transverse structural steel members were also framed into the longitudinal struts to provide additional bracing and to support the steel sheet pile cofferdam walls driven parallel to the bridge to enclose the areas of the new foundations between the existing arch pier stems.

![Figure 20 – Arch Strut System](image)

LUSAS 3-D analysis of the arch was used to investigate the various conditions during the construction, with a sample model shown in Figure 21.

![Figure 21 – LUSAS Arch Model](image)
3.2.3 Temporary Track Stabilization

For construction activities and float in-out operation, a large portion of the arch spans had to be removed with only an 18’ wide strip under the track to remain for the rail traffic. Maintaining the railroad track and carrying traffic over the remaining arch span strip through the work zone was challenging. Although the contract plans show a possible track stabilization method, the contractor developed a system of sheet piles along the edges of arch strips, with micropile rods installed transversely under the track through the arch fill, as shown in Figure 22. The micropile rods engaged longitudinal wales to resist the lateral loads, retain the arch fill and ballast, and stabilize the track. The contractor also used the concept in the contract drawing and installed a truss system below the 18’ arch strip to provide additional vertical capacity, as shown in Figure 23. The arch strips with the temporary track stabilization and supporting truss systems, shown in Figure 24, safely carried the live track and ensured the continuing rail service during the construction.
3.3 Severed Bottom Chord Condition for Lift Truss Span

3.3.1 Requirements

BNSF Railway required the design of the proposed replacement bridge at the Galveston Causeway to incorporate the following features for the special condition of a severed bottom chord:

- Each line of stringers and stringer to floor beam connections shall be capable of carrying the maximum bottom chord force for one truss, increased by the ratio of (top chord to bottom chord distance)/(top chord to mid depth of stringer distance), for each of the following conditions:
  - Dead load plus wind load with the bridge nearly closed.
  - Dead load plus wind and H20 truck loading without impact on roadway combined with stringer stresses resulting from H20 truck loading without impact with bridge fully closed.
- The lateral strength of the end floor beams shall be adequate to transfer the maximum bottom chord force due to above loadings from the inner line of stringers to the end posts of the truss without exceeding the yield point of steel.
- The lateral strength of each interior floor beam shall be adequate to transfer the maximum bottom chord reaction at the panel point due to the above loadings from the truss to the inner line of stringers without exceeding the yield point of steel.

These special features were taken from the special provisions for the existing bridge, a 125-foot single track steel rolling lift bascule span built in 1988. For the proposed 382’-8” vertical lift span, with a resulting force demand 5 times greater than that of the existing, the proposed bridge would require extremely thick and wide floorbeam flange plates and could not reasonably provide the stringer connections as required. Complying with the above special design requirements would have resulted in a rather bizarre, unreasonable and costly structure. Considering that the intent of these requirements is to prevent collapse in the event a bottom chord is severed and to facilitate repairs, a different approach was investigated.
3.3.2 Methodology

Traditionally, bridge truss design is based on 2D truss analysis. However, under the special condition of a severed bottom chord, the 2D analysis cannot accurately reflect the behavior of the entire truss system that includes subsystems of truss, floor, lateral, and sway members. Therefore, 3D analysis is required to evaluate the load redistributions and internal redundancy under such special conditions. SAP2000 was used to perform 3D analysis of the entire lift span truss system. The following factors were considered:

- Different Locations of Severed Bottom Chord
- Different Boundary Conditions
- Different Load Combinations
- Load Redistributions
- Participation of Stringer and Effect of Stringer Relief Joints
- Effect of Span Guide Engagement
- Stresses and Capacities
- Displacements
- Reactions
- Design Implementations

Through a holistic approach that takes advantage of the internal redundancy of the space truss, that intent can be fulfilled with a 3D truss system designed for the forces involved. This will result in minimum member size increases and, thus, a much more cost effective structure.

3.3.3 3-D Model

Space Frame FEM Analysis was employed to investigate the 3D behavior of the lift truss span. To simulate the truss action, both rotational degrees of freedom were released at the ends of the frame members at the truss joints. For the base models, the axial degree of freedom was released for the stringers to eliminate the stringer’s participation in the load redistribution. Separate models were developed to investigate the effect of the stringers in load redistribution, one model with the stringers fully participating and the other with the stringers partially participating due to the presence of relief joints. The accuracy of the analysis results depend on the accuracy of the section properties used in the 3D analysis, especially for the severed chord conditions where the rigidities of the members play a critical role in determining the load redistributions and truss behaviors. To accurately reflect the member rigidities, the elastic section properties were used in the analysis. Figure 25 shows the basic 3D SAP Model of the lift span. All major members were included in the model.

Figure 25 - Basic 3D SAP Model
3.3.4 Severed Chord Cases, Boundary Conditions and Load Combinations

Because of the unsymmetrical boundary conditions, a total of 14 severed chord cases were considered to catch every possible event. It is assumed that only one chord member in one panel will be severed at a time.

- SP0 – Base Case without Severed Bottom Chord
- SP1 to SP14 – Severed Bottom Chord from Panel 1 to Panel 14

Panel 1 is at the fixed bearing and Panel 14 is at the expansion bearing.

Three basic boundary conditions were considered for the investigation:

- B0 – Fixed Bearing Condition with Span Guide not Engaged
- B1 – Fixed Bearing Condition with Span Guide Engaged
- B2 – Simple Bearing Condition with Span Guide Engaged

The fixed bearing condition is the “propped cantilever” condition with two fixed bearings and two expansion bearings. The simple bearing condition has three expansion bearings and one fixed bearing. The lateral system design is based on the maximum forces in both fixed and simple conditions.

Since both trusses will have significant lateral movement in the event of a severed chord, it is assumed that the top span guides will be engaged. The proposed span guides will restrain the transverse movements of Joints U0 and U0’, but will allow vertical and longitudinal movements, and rotational movement in all three directions.

The following loads are applied at the truss joints:

- Dead Load (DL)
- Wind Load (50W and 70W)
- H20-44 Truck Load (LL)

The wind load case 70W represents hurricane winds of 70 psf. Because of the severed chord conditions, the structure is not symmetrical for the wind. Therefore, the Wind Load was applied in two separate transverse directions:

- 50W1 - Towards Far Truss (NT)
- 50W2 - Towards Near Truss (FT)

3.3.5 Load Redistribution

Dead Load

A. Axial Force Redistribution

Figure 26 shows the axial load distribution for Base Case SP0 without severed chords, while Figure 27 shows the axial load redistribution when the chord at Panel 7 (L6-L7) is severed. Both are for boundary condition B0. It is obvious that the loads on the Near Truss (NT) with the severed chord are redistributed to the Far Truss (FT). The load path from the Near Truss to the Far Truss is through the bottom laterals in the panel of the severed chord, and that results in a large force in the bottom laterals of that panel. The Far Truss bottom chords experience significant increases in member forces while the dead load forces in the bottom chords of the Near Truss are reduced drastically. The force in the floorbeam at the panel of the severed chord also increases significantly.
B. Moment and Shear Redistribution

Figure 28 shows the moment redistributions respectively for Case SP7. For Case SP0, there is no appreciable moment and shear in the truss members. When the chord is severed, the hangers pick up moment and shear due to sway frame distortions. The hanger in the panel of the severed chord experiences the largest increase.
C. Truss Distortions

Figure 29 shows the distortion of the truss for Severed Chord SP7 under the dead load. It is obvious that the truss moves in the direction of the Near Truss in a distorted manner.

The displacements were investigated for three boundary conditions (B0, B1, B2). The largest displacements are in the transverse direction, indicating severe lateral distortions. The near truss experiences more vertical deflections than the far truss. The lateral displacements at the span guide locations (U0, U0’) are in excess of 0.5”, indicating that the span guides will likely be engaged.

D. Effect of the Span Guide Engagement

The effects of the span guide engagement on the axial force and moment redistribution are investigated through the comparison of results based on Boundary Condition B0 (Span Guide Not Engaged) and Boundary Condition B1 (Span Guide Engaged). In general, the engagement of span guides significantly reduces the member forces as compared with those under span guide not engaged condition.
**Live Load**

The trend of load redistributions due to a severed chord under H20-44 Truck Live Load is similar to that under Dead Load but the distributed forces are much smaller, only a fraction of the dead load forces. Figure 30 shows the Influence Line for Bottom Lateral at P7 for SP7 Chord Severed Condition.

![Sample Influence Line](image)

**Wind Load**

The axial force redistributions due to wind load for SP0 and SP7 with different boundary conditions were investigated. Under the propped cantilever conditions in B0 and B1, wind load produces large axial forces in the bottom chords near fixed bearings. Those forces are not present under the simple bearing condition of B2. The load redistributions for the two types of bearing assumptions are significantly different. The moment redistributions due to wind load for SP0 and SP7 with different boundary conditions indicated that the effects of the boundary conditions are primarily in the hangers. The effect of the span guide engagement on the moment redistribution for the Severed Chord Case SP10 under wind load for both B0 and B1 conditions was also investigated. It is apparent that the span guide engagement significantly reduces the moment in the floorbeam hangers, to which the sways are connected. This moment reduction effect due to span guide engagement is a key design factor that will be discussed later. It should be noted that the engagement of the span guides will also reduce the moments in the hangers under the normal conditions where there are no severed chords.

**3.3.6 Stringer Participation**

The participation of stringers in load redistributions in the event of a severed chord is primarily dependent upon the modeling of the stringer end connections in the axial degree of freedom. The following three connection assumptions were used for this investigation:

- **Case 1 – Each Stringer Released** – Each stringer is released at one end for the axial degree of freedom. This represents the condition in which the stringers do not participate in the load redistributions. This condition was used for most of the models described above so that the global effect of the severed chord can be clearly demonstrated.

- **Case 2 - Stringer Released only at Stringer Relief Joints** – Only the stringer ends at the relief joints were released for the axial degree of freedom. Two stringer relief joints were proposed for the lift span, at L5 and L5’, as per the AREMA requirements. This condition represents the partial participation of the stringers in load redistributions.
Case 3 - No Stringer Release – No end release for the axial degree of freedom was assigned to any stringers. This condition represents the full participation of the stringers in load redistributions.

The stringers’ effect on the overall dead load redistribution of the truss members is not very significant, and mostly localized. When compared to the load redistribution for Case 1, where the stringers do not participate in the load sharing, the effects of the load redistribution of Case 2 and Case 3 are generally 5% to 10% with many members being affected by less than 5%. The noted trend is that the stringers will increase the Near Truss load sharing and reduce the Far Truss, and that Case 3 (No Stringer Released Case) will have more effect than Case 2 (Stringers Released at Relief Joints). The largest effect is at the panel of the severed chord.

3.3.7 Analysis Results

Based on the analysis results, we can draw the following conclusions:

- Since the lift span acts as a 3D space truss system in the event of a severed chord, the forces will be redistributed among the two trusses and the various secondary members. The primary load path for the redistributions is through the bottom laterals in the panel of the severed chord.
- The severed chord will cause significant distortion of the truss system. The largest displacements are in transverse direction, which indicates the probable engagement of the span guide at U0 and U0’ under the severed chord conditions.
- In general, the engagement of the span guides helps the system in resisting the distortion due to the severed chord, and hence reduces the loads in most of the members. The most significant impact of the span guide engagement is on the out-of-plane bending of the hangers under wind load.
- The fixity of the bearings also has impact on the analysis results. Both “simple” and “propped cantilever” assumptions for fixed bearing are considered in the analysis.
- The participation of the stringers in truss load redistribution was investigated through different conditions of stringer end releases. The stringer load sharing is primarily localized near the severed chord and the force in the stringer is only a fraction of the maximum bottom chord force. The transfer of load through the stringers involves out-of-plane bending of the floorbeams, which results in a more flexible load path than the transfer of load through the bottom laterals.
- With the 3D analysis, the load redistributions and load paths were accurately calculated, and reasonable design loads for the conditions of severed chords were developed. The design intent of preventing the span collapse and carrying a service vehicle under the condition of a severed chord can be achieved in a structurally effective and cost efficient way, when the inherent internal redundancy of the space truss is taken into consideration.
- Most of members in the lift span truss system as designed for service loads were found to have sufficient capacities to carry the loads (DL, W, and Truck) in the event of a severed bottom chord in any panel for the load combinations and elevated allowable stresses specified. The large railroad live load versus dead load ratio in the regular design contributes to the overall strength of the system to sustain the event of the severed chord. The sizes of bottom laterals, floorbeams and hangers are marginally increased for the severed chord condition.

4.0 CONCLUSION

The Galveston Causeway Railroad Bridge Replacement Project was complicated and challenging in both design and construction. The success of the project was attributed to the creativity and skills of the engineers and contractors, as well as collaboration among all stakeholders.
Design Challenges of Galveston Causeway Railroad Bridge Replacement

Buck Ouyang, PE

Modjeski and Masters, Inc.
Bridge Location

• Galveston Bay, Texas

Bridge Facts

• Owned by Country of Galveston
• Operated and Maintained by BNSF
• Over Intracoastal Waterway
• Only RR Link between Mainland and Galveston Island
• Vital to Port of Galveston
  o Freight Rail
  o Shipping Channel

History

• 1912 – Original Causeway
• 1922 – Additional Arch Span at both Ends
• 1939 – Construction of Highway Bridge
• 1988 – Replacement of the Original Bascule
• 2008 – Replacement of I-45 Highway Bridge
• 2012 – Replacement of RR Bridge

Original Causeway

• Two Railroad Tracks, an Interurban Track, a 16’ Roadway
• 125’ Scherzer Rolling Lift Bascule Span on Concrete Piers
• Concrete Arch Approach Spans

Replacement Bascule Bridge

• Switch Pivot
• Single Track with Grating Deck on Track
• Realigned to Center of Causeway

Replace a 1998 Bridge, Why?

• Hazard to Navigation

• Truman-Hobbs Project
• OTA – 6/2001
  o I-45 Completed 2008
Navigation Clearance Requirements for New Railroad Bridge

- Minimum Horizontal Clearance for Navigation Channel
  - 300 feet
- Minimum Vertical Clearance for Navigation Channel
  - 73 feet over MHW in Open Position
  - 8 feet over MHW in Closed Position
- Early Decision on Proper Bridge Type
  - Vertical Lift
  - Single Track

Rail and Marine Traffic Requirements

- Maintain Continuous Railroad service during the construction except for prescheduled short track outages
- Maintain Continuous service to the Intracoastal marine traffic during the construction except for prescheduled closures

Replacement Alternatives

- Alternative Development
  - 6 Major Alternatives
  - 15 Total Options
- 4 Alignments
  - Alignment 1 - 50’ North of Old Track No.1
  - Alignment 2 - 9’ North of Old Track No.1
  - Alignment 3 - Old Track No.1
  - Alignment 4 - Current Operating Track
- Foundation Types
  - New Pile Foundation with Pier Shaft
  - New Drilled Shaft Foundation
  - Combined Pile Foundation with New Arch Abutment
  - Existing Foundation Reuse

Summary of Alternatives

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Track Alignment and Profile</th>
<th>Lift Span and Tower</th>
<th>Pier and Foundation</th>
<th>All Options</th>
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Important Evaluation Parameters

- Track Alignment and Profile
  - Alignment
  - Profile
- Lift Span and Tower
  - Structural Efficiency
  - Tower Configuration
  - Mechanical Considerations
  - Electrical Considerations
- Pier and Foundation
  - Foundation Type
  - Extension of Existing Arch Pier
  - Modification of Existing Arch Pier
  - New Arch Abutments
  - Cofferdam
  - Additional Vessel Collision Protection
Important Evaluation Parameters

*Construction
  o Track Interference
  o Construction Clearance
  o Removal of Existing Arch Pier and Span
  o Temporary Retaining Walls
  o Temporary Deck Girder Span

*Float In-and-out and Track Outage
  o Operation
  o Track Outage
  o Removal of Existing Arch
  o Removal of Existing Bascule Span, and Rack Frame

*Cost
  o Initial Construction Cost for Lift Bridge and Approaches
  o Life-Cycle Cost

*Environmental Impact

*Aesthetics

*Railroad’s Preference

Evaluation Method

Raw Score (from 1 Inferior to 4 Superior)*

Weights (from 1 Least Important to 10 Most Important)

Weighted Score = Raw Score x Weight

Railroad Preference (from 0 Unacceptable to 1 Acceptable)

Total Score = \[\sum \text{Weighted Scores}\] x [Railroad’s Preference]

* For cost, the raw score is calculated by the reciprocal of the cost index normalized to the maximum of score of 4.

Summary of Alternatives

<table>
<thead>
<tr>
<th>Track Replacement and Footbridge</th>
<th>Lift Bridge and Tower</th>
<th>Net Approach</th>
<th>Transillumination</th>
<th>Track and Track Outage</th>
<th>Cost</th>
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Replacement Scheme

Design Challenges

There are many design challenges, and we will only discuss a few:

• Span and Tower
• Arch and Foundation
• Severed Bottom Chord Condition for Lift Truss Span

Span and Tower

Extremely Tight Clearance During Construction:

• Extremely Tight Clearance During Construction
• Existing Bascule Span Operating on one Pinion
• Unique Span Guide Arrangement
• Design for Wind Loads
Tight Clearance

Bascul Span on one Pinion

Clearance During Construction

Span Guide Arrangement

**Design for Wind Loads**

- Regular Wind for Span in Open Position
- Hurricane Wind for Span in Closed Position

Additional wind load combinations:

- DL + HW (70 psf) @ 140% Basic Allowable
- DL + E @ 125% Basic Allowable
- DL + E + EW @ 140% Basic Allowable

**Design for Wind Loads**

- Narrow Lift Span Requires Robust Lateral Systems
Design for Wind Loads

- Narrow Tower Requires Post-tensioned Anchors
- No Tension between Column Base and Concrete

Arch and Foundation

- Arches Terminated away from their Original Abutments
- New Integrated Foundation Serves as
  - Lift Span Pier
  - New Arch Foundation
- Design Takes Advantage of the Unique Load Path of Arch
  - Cored holes along the arch for pile driving
  - Removal of strips of arch on both side of track

Arch and Foundation

Arch Analysis

- LUSAS 3-D Model

Driving Piles through Arch

Arch Struts
Arch Removal & Track Stabilization

Foundation

Severed Bottom Chord Condition for Lift Truss Span

BNSF Design Criteria for Severed Chord Condition

- Prevent Collapse
- Facilitate Repairs
- Design Load Combinations
  - DL+W under nearly closed position
  - DL+W+H20 under closed position

Original Design Requirements

- Original Design Criteria requires the bottom chord member forces to be taken by Stringers and Floorbeams similar to the 1988 125-ft Bascule Span
- This would result in a rather bizarre, unreasonable and costly structure with huge stringer sections and floorbeam flanges for a 380-ft Truss Span.
Questions

- Is the traditional way of carrying the chord force through stringer and floorbeam realistic?
- What is the actual load path?
- Can we take advantage of internal redundancy of a space truss?
- Can we take advantage of the lift span which has better lateral resistance due to the presence of the span guides at the top.

A Different Approach

- 3-D Analysis of the Space Truss with various severed chord and boundary conditions.
- To cover all possible scenarios, the following conditions were used in calculating the maximum loads, D/C ratios (demand/capacity), displacements and reactions:
  - 4 Boundary Conditions (Simple, Propped Cantilever, Span Guide Engagement)
  - 14 Severed Chord Cases and 1 Base Case for Comparison
  - 3 Load Combinations
    - LC1: DL @ 100% < 0.7F_y
    - LC2: DL+50W+LL @125% < 0.7F_y
    - LC3: DL+70W @140% < 0.7F_y
**Stringer Participation**

- Stringer End Boundary Conditions
  - Case 1 – End Released
  - Case 2 - Released at Relief Joints
  - Case 3 - No Stringer Released

- Stringer End Boundary Conditions The stringers’ effect on the overall dead load redistribution of the truss members is not very significant, about 5 to 10%, and mostly localized.

**Analysis Results**

- Load redistributed among the two trusses and the various secondary members.
- The primary load path for the redistributions is through the bottom laterals in the panel of the severed chord.
- Most of members in the lift span truss system as designed for service loads were found to have sufficient capacities to carry the loads (DL, W, and Truck) in the event of a severed bottom chord.
- Only Bottom Laterals, Hangers, and Floorbeams need to be marginally increased in sizes to meet the severed chord conditions.

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- County of Galveston
- BNSF Railway
- Cianbro/Brasfield&Gorrie
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- Modjeski and Masters Project Team (Design and CSM)

**Thank You**