INCREMENTAL LAUNCH METHOD FOR STEEL TRUSS BRIDGE ERECTION

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BIOGRAPHY

Jerry is a Principal and Regional Bridge Engineer at FINLEY Engineering Group, Inc**.** Jerry has a Masters Degree in Engineering, over 19 years experience and a uniquely diverse background in the field of bridge engineering.

Bridge structure types designed include prestressed I-girder, steel girder, precast segmental balanced cantilever, cast-in-place segmental, precast span-by-span and cable stayed bridges.

Notable design experience includes ship impact analysis, finite element analysis, seismic analysis, strut-and-tie modeling and time dependent analysis.

Jerry has published seven papers and one webinar on subjects including BrIM, innovative methods of concrete bridge design and analysis. He is a member of the Florida Engineering Society, American Segmental Bridge Institute (ASBI) and has been a speaker on segmental bridge design and construction engineering at industry conferences.

SUMMARY

Today there is a focus on Accelerated Bridge Construction (ABC) to significantly reduce traffic delays and closures on highly congested roads. From the contractor's view, there is a demand to deliver the project safely and more rapidly to meet the economic goals of our industry – to maximize the return on infrastructure investment. While there are many ways to apply ABC, this paper will discuss one application of this concept – the incremental launch method.

The incremental launch system for truss erection is an innovative technique that can be used on projects where there is restricted access to the site, tight project schedule, highly congested roads, or where traditional methods would create an obstruction to navigable waters. The Chelsea Street Bridge in Boston and Checkered House Bridge in Vermont projects are two examples where this technique was applied in two very different applications of incremental launching.

This paper discusses the construction engineering design and details using the incremental launch methods on these projects, including lessons learned and recommendations for future applications of this method.

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Introduction

According to the Federal Highway Administration States across the county have been implementing accelerated bridge construction (ABC) at an increasing rate in order to improve safety and mobility impacts during the repair and replacement our nation's aging bridges and heavily travelled roadways. ABC involves the use of various techniques during the planning, design, contract development, and construction to reduce the time to build or repair a bridge as compared to traditional cast-in-place methods. This paper discusses the incremental launch of steel trusses, an innovative technique that can be used on projects where there is restricted access to the site, tight project schedule, highly congested roads, or where traditional methods would create an obstruction to navigable waters. Boston's moveable Chelsea Street Bridge and Vermont's steel truss Checkered House Bridge are two very distinct examples where this technique was applied successfully.

The Chelsea Street Bridge

1. Project Overview

The original Chelsea Street Bridge, built in 1900, was a single-leaf bascule structure, which connects the cities of Boston and Chelsea, Massachusetts. It serves as a major vehicular route to Boston's Logan International Airport and busy Massachusetts Route 1A. The narrow opening of the lift bridge and structural deficiencies of the aging structure posed a hazard to marine navigation below and presented efficiency and operations challenges to vehicular traffic on top, requiring a complete replacement.

The old bridge had an overall length of 446 ft (136m) and was a six span structure with a main span consisting of a 140 ft (43m) heel trunnion Strauss bascule with 66 ft (20m) and 42.5 ft (13m) spans on the east side and three, 66 ft (20m) spans on the west side. The bascule span itself spanned a 96 ft (30m) wide channel that restricted shipping and was a notable hazard to navigation.

The new replacement structure, designed by HNTB Corporation, consists of a massive constant-height warren-type steel truss that spans 450 ft (137m) between two, 216 ft (65m) high towers, and when raised gives 175 ft (53m) of vertical clearance for shipping. It matches the footprint of the previous bridge and its approach structures and provides for four lanes of traffic (two in each direction) along with two pedestrian sidewalks. The new vertical lift bridge, opened to traffic in May 2012, is the largest permanent lift bridge built by Massachusetts Department of Transportation (MassDOT) to date.

Figure 1 Old Chelsea Street Bridge

2. Concept Development

The Chelsea Street Bridge project presented a particularly challenging site precluding the use of traditional methods of steel erection. There was very limited space particularly on the south side of the creek, the risk to worker safety was higher due to working near/above a water body, soil conditions were too soft to support the cranes needed, the urban location and need to keep the barge and vehicular traffic moving presented additional constraints, and on-site subsurface utilities needed to be protected. Because the fuel barges that pass under the bridge supply 60% of the region's residential heating fuel,

Figure 2 Chelsea Street Bridge Replacement

as well as for Logan International Airport, marine traffic could not be interrupted for more than 60 hours during the construction of the replacement structure.

As part of its appointment to provide pre-bid and construction engineering services to J.F White Contracting Co. for the Chelsea Street Bridge Replacement Project, Finley Engineering Group, Inc. (FINLEY) developed an erection scheme for the bridge. A lack of clearance for larger vessels meant that it would have been difficult to float the truss in on barges as is commonly done for bridges of this type. FINLEY's solution was to launch a fully assembled truss 140 ft (43m) out across the navigation channel using launching girders supported on the existing bridge piers. FINLEY used LUSAS Bridge analysis software to assist with this task and also to analyse the stresses on temporary steel bracing as a result of pouring of concrete into the steel plated counterweights.

3. Bridge Towers

The new lift bridge is supported at each end by two steel towers. Each tower is 65 m (212 feet) high from top of foundation to the machine room floor level.

The towers were erected to mid-height by assembling the individual steel elements in the air in a section-by-section manner. The upper half of the tower structures and the machine room floor were pre-assembled on the ground and raised into position. The pre-assembled upper sections weighed 1,960 kN (440 kips with dimensions 34 m (110 feet) long by 2.4 m (8 feet) wide by 7.6 m (25 feet) deep. The fully assembled machine room floor weighed 1,560 kN (350 kips) with dimensions 23 m (76 feet) long by 7.6 m (25 feet) wide by 4.3 m (14 feet) deep.

3.1 Construction Survey Control

Horizontal and vertical positioning of the permanent structure was performed using CAD technology together with a one second digital total station instrument. Vertical position was maintained with a digital level with bar code rod with accuracy to 0.03 mm (one-thousandth of an inch). The project survey control was maintained through a project coordinate system. The project requirements for vertical plumb of the towers was limited to 1.3 cm (one-half inch) from the tower base to the top of the tower transverse to the project roadway baseline and 2.5 cm (one-inch) along the project roadway baseline.

Project survey control was maintained through a three dimensional survey network. Survey observations were analyzed using a least squares computer analysis software program. The use of least squares averaging for error reduction was critical for maintaining tight traverse control throughout the project.

Critical crane lifts were positioned using CAD and digital total station instruments. Crane positions and movements were engineered with CAD programming and then positioned in the field for precise location.

3.2 Geotechnical Site Investigation

The geotechnical site investigation program consisted of seven soil borings, which were drilled through the fill layer to the top of natural soils. Once the soil borings were complete, three plate load-bearing tests were performed in accordance with ASTM D 1194. Two plate load-bearing tests were conducted on the north side and one on the south side. The north side tests served as the design basis for the crane loads during tower construction as well as the loads induced during the truss launch since the represented the lowest design bearing capacity.

The results of the soil borings indicated that the top layer of soil was an eight to fourteen foot thick fill layer consisting of sand, gravel, cinders, ash and brick. A Standard Penetration Test (SPT) 5 cm (2-inch) diameter split sampler was used to obtain continuous soil samples to the top of the natural soil layer. The fill layer was a medium stiff to stiff material. Beneath the fill layer was a natural layer of organic silt with peat fibers. The organic silt layer was very weak. The design objective for the crane pad used during the tower construction and for the truss launching rails was to distribute the ground bearing pressure loads uniformly through the fill layer to the extent practical.

The plate load bearing tests on the north side of the project showed that the fill layer had an ultimate bearing capacity of twelve tons per square foot (tsf) for the short term loading. The allowable bearing capacity used for engineering the crane road and truss launching rails was 766 kPa (8 tsf); i.e. a safety factor of 1.5.

3.3 Soil improvement design

The crane road for the north tower was designed as a sand and gravel fill over the existing soil with a geogrid installed at the interface between the top of the existing fill and the sand and gravel. The geogrid was utilized to improve the interaction between the two soil layers. Timber mats were used above the sand and gravel as the crane travelling surface. The 30 cm (12-inch) thick hardwood mats served to evenly distribute the crane load over the fill area.

The north area crane road was located adjacent to an existing steel sheetpile retaining wall at the shoreline. The existing retaining wall was designed to retain the existing shoreline soils but was not capable of supporting any additional lateral earth pressures from the crawler cranes or the additional earth fill. Due to the limited capacity of the existing retaining wall, the sand and gravel fill was installed beyond the zone of influence of the existing retaining wall.

The south tower crane road location was limited to a 46 m (150 feet) by 46 m (150 feet) work zone on the west side of the south tower. Work area restrictions were the main limitation at the south tower. The west side work zone location was selected based on water access for barge unloading, close proximity to the bridge and operating area which was available between existing overhead utilities. There were several site restrictions at the south side crane area that were not present at the north. In addition to the limited crane location, work area for upper tower leg section pre-assembly was limited. This limitation required staged construction of the upper sections rather than concurrent assembly.

Several existing subsurface utilities were located beneath the crane area at the south. Three pile supported temporary steel grillage systems were designed to distribute the crane loads beyond the existing utilities. The plate load bearing test results on the south side indicated that the ultimate bearing capacity in this area was slightly higher, but comparable to the results of the north area. The ultimate bearing capacity at the south crane road was 1,235 kPa (12.9 tsf).

It was planned to preassemble the upper half of the north and south towers in the horizontal position at existing grade. Once the upper sections were fully assembled, the assembly was raised and rotated ninety degrees to the vertical position and connected at the mid-height tower splice. The installation methods required the use of a 660-ton conventional lattice boom crawler crane and a 600-ton hydraulic crane. The conventional crane served as the main hoisting crane to install the pre-assembled upper sections. The hydraulic crane assisted the crawler crane through the lift by holding the member load while the piece was horizontal. Once the preassembled section was vertical, the loads were completely transferred to the crawler crane and the hydraulic crane was disconnected. The maximum anticipated ground pressure exerted by the 660-ton crane through the lifting procedure was 730 kPa (7.6 tsf).

3.4 Tower Construction

Following the construction of the crane roads on the north and south sides of the project, the upper tower sections were built and erected into position (Figure 4). Traffic along the existing bridge was maintained during tower construction, with steel erection being completed during night and weekend closures of the bridge.

4. Bridge Liftspan

4.1 Launch Design

A lack of clearance for larger vessels meant that it would have been difficult to float the truss in on barges as is commonly done for bridges of this type. Therefore, the truss installation was engineered as a launch proceeding from the north side of the project toward the south. The liftspan installation utilized a launch system of temporary steel girders on top of the existing support piers. Steel rollers and hydraulic jacks were employed during the launch as a horizontal and vertical positioning system for the liftspan as it progressed over the channel. This method facilitated the preassembly of the span adjacent to the project site prior to closing the bridge, and enabled assembly work to proceed on land and reduce the exposure and risk of work over the water.

The planned assembly methods required the distribution of vertical loads induced by large crawler cranes during tower construction and rolling truss loads through the truss launch procedure. The soil pressure induced by both the cranes and the rolling truss was 766 kPa (8 tons per square foot, tsf). A soil boring and bearing load test program plan was implemented to determine the short term soil bearing capacity in the crane road and the truss launch areas.

The bridge liftspan is 137 m (450 feet) long by 23 m (75 feet) wide and has a launching weight of 10,790 kN (2,425 kips). The liftspan provides 53 m (175 feet) of vertical clearance above the mean high water level in the navigable channel. The liftspan was pre-assembled adjacent to the site in two main longitudinal sections due to space restrictions (Figure 5). The construction staging area could only accommodate up to 82 m (270 feet) of the assembled liftspan. Each section of the liftspan was constructed on top of shallow concrete footings. The footings served as the land-side launching rails for East to west and north to south positioning of the liftspan. Large capacity track rollers were used on top of the shallow foundation launching rails for the liftspan launch. Each of the two main span sections was moved from the staging area into position along the roadway alignment. The two sections were connected once they were both aligned within the

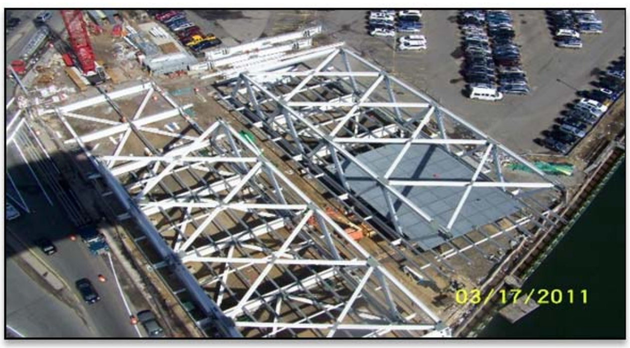

Figure 4 Pre-Assembled Liftspan

roadway. Long stroke hydraulic jacks were used to advance the liftspan. Steel plates lined the top of the shallow foundations as a smooth rolling surface for the track rollers as well as for load distribution.

4.2 Liftspan Launch Support Systems

The existing bridge superstructure was removed to allow the liftspan installation. The existing bridge piers were retained as supports for the steel launching rails . A temporary steel framing system was built on top of the piers to carry the liftspan from the north to the south. The launching rails consisted of double wide flange sections which were stitch welded along the top and bottom flanges. The center core of the wide flange beams were filled with self consolidating concrete. The span of the launching rails ranged from 11 m (35 feet) to 15 m (50 feet) long. The launching rails on top of the existing piers were designed as simple span truss sections. There were five existing piers within the channel. The piers were numbered from north to south in the launch direction. The 27 m (90 feet) span between piers 3 and 4 contained the navigable section of Chelsea Creek. Due to marine traffic restrictions, launching rails could not be installed between piers 3 and 4. During the truss launch, the span was cantilevered through the pier 3 to 4 section.

The design of the temporary supports used for launching is one of the critical aspects of the overall scheme. This project utilized Hilman roller systems to provide a proven, reliable means of moving the heavy supports loads in excess of 650 kips over temporary works designed to support the trusses during the launch.

To support this loading, two 200 ton rollers were combined in a ‘bogey’ that included a 400 ton hydraulic jack. This system provided the flexibility to manipulate the truss during the launch and enabled us to cantilever out over the channel to the correct elevation and allow the bogey system on the other side to take over support of the truss.

Given the heavy support loads required for this launch, considering the friction of the Hilman rollers was a critical component to understand how this impacted the overall launching force. Under vertical loads, the friction coefficient varies with the percentage of load to the roller capacity from near

Figure 3 Launching System

0% when unloaded to approximately 5% when under maximum loading. As we cantilevered further over the channel, the support loads increased and the increase in friction had to be included in the total launching force.

A hydraulic launching system was developed utilizing dual action cylinders to provide maximum control during the launch. Control of the movement was maintained through metered valves and also needle valves at the cylinders. The launch rate was controlled through flow control valves and the cylinder diameter was matched to produce the desired rod speed. Typically, the launch rate was 1 foot every 45 seconds to one minute.

4.3 Liftspan Stress Analysis

A detailed three dimensional beam element model was created with LUSAS software to allow analysis of the truss members during all phases of the launch sequence. The truss members were evaluated for their strength and stability capacities. As a result of this analysis, it was found that a temporary brace was required for two of the truss chords at the supports during the maximum cantilever stage. Additionally the truss gusset plate connections were analyzed for support of the truss on the bottom chord during the launch phases. The analysis program allowed for boundary condition modifications to mirror the changing support locations that will be used during the launch sequence. The program was also able to envelope the member forces and reactions to simplify the determination of critical forces and reactions. The analysis model also provides the displacements of the liftspan truss during each phase of the launch sequence. This information is critical to establish the launch geometry that allowed the determination of the support heights required to maintain clearance of the truss to the launching rails during the launch and also to ensure that the truss will be at a sufficient height at the end of the launch across the channel to reach the support. To finalize the geometry for the launch the launching rails and liftspan were drawn in a three dimensional CAD model. The liftspan truss was then manipulated through each phase of launch sequence iteratively to determine the geometry of the launching rails and supports that could achieve the launch and minimize the amount of site work required to construct the launching rails. In order to eliminate the concerns of settlement of the soil and differential deflections of the truss during the launch, a single support at each end of the truss was used at all times. This lead to large reactions that occurred on the approach pavements well in excess of the original design values. In order to transfer these large reactions of up to 3,290 kN (740 kips) to the soil, a three dimensional analysis model was created to determine the most effective shape of the launching rails that would minimize the bearing pressure with the least of materials cost.

The development of the launching sequence was enabled through the use of LUSAS software to perform three dimensional analysis and CAD software that could be integrated together to fully model each stage of the launch sequence. The three dimensional CAD model was also used during the launch to evaluate the as-build geometry as the liftspan moved through each phase of the launch sequence. The use of these highly developed software programs provided the ability to rapidly evaluate the liftspan during design and launching and were critical to the success of the lift span launching.

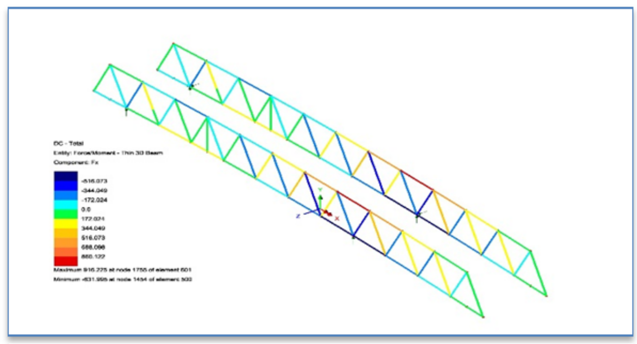

Figure 5 LUSAS Model

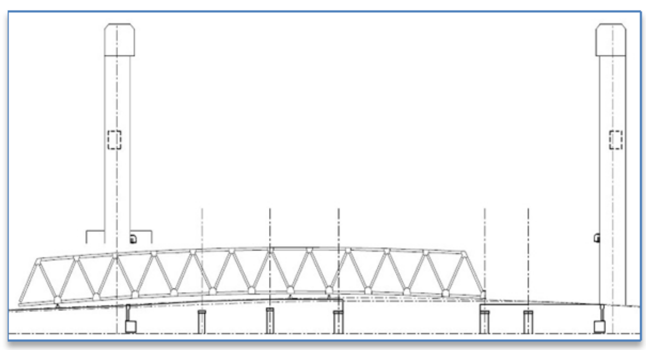

Figure 6 Launch Schematic

Figure 7 Launch

The fully-cantilevered condition was the most demanding, causing the highest jack reactions and forces/stresses in the members. Jack Pecora, Project Manager at J.F. White Contracting Co. said: "FINLEY's use of LUSAS provided us with updated support point reactions (bogey reactions) once the final launch load and sequence was engineered. This was most critical through the launch phases as the truss model was used to confirm observed reactions from the field."

The use of incremental launch method helped to make this project viable in terms of efficiency, continued marine and vehicular traffic operations, and cost effectiveness. This project was particularly challenging because of the required incremental launch, the excessive launch weight of the truss and launch distance, and the high level of marine traffic in the navigable waterway below. Key to the success of the project was the development of a sound conceptual design, which should include an analysis and determination of the best type of launch system, launch sequencing, stability plan during launch, selection of appropriate moving supports, and a mission critical list of conditions that could prevent a successful launch and a plan to mitigate them.

Checkered House Bridge

1. Project Description

Built in 1929, the Checkered House Bridge is a 350 ft.-long steel truss bridge placed on the National Register of Historic Places in 1990. Because of its historical significance, the community and Vermont Agency of Transportation (VTrans) wanted to rehabilitate rather than replace the bridge. This project is the second design-build project undertaken by VTrans since design-build project delivery was authorized by the Vermont legislature. This was the first time a steel truss bridge this size was widened.

The bridge needed to be widened because it was too narrow for two vehicles to travel across the bridge simultaneously. A local road [Johnny Brook Road] that intersects Route 2 at the west end of the bridge had no sight distance and created a dangerous intersection. The bridge had been posted for more than 20 years and it was necessary to rehabilitate the structure to accommodate trucks with greater capacity.

Figure 8 Checkered House Bridge Prior To Renovation

The rehabilitated bridge benefits farmers and other heavy equipment operators. Bordered by farmland, the old bridge was too narrow and the weight capacity too low to accommodate the movement of large and cumbersome equipment. The rehabilitated bridge now allows for efficient and safe access to the local farmers and operators of other large equipment and extends the bridge lifecycle by 75 years.

2. Project Overview

The Design-Build team of Harrison & Burrows and CHA brought in Finley Engineering Group, (FINLEY) early in the bid process to develop the Conceptual Design, Falsework Design, Launching System Design, Construction Manual, Falsework/Launching System Inspection and to provide On-Site Technical Assistance during the launch. Other team members included:

- Boswell Engineering, Inc. - Independent Quality Assurance/Quality Control
- Advance Testing Company, Inc. - Quality Control/Testing
- Vermont Survey & Engineering, Inc. – Surveying
- EIV Technical Services, LLC - Environmental Compliance
- Fitzgerald & Halliday, Inc. - Communications

This rehabilitation widening project had to satisfy Section 106 of the National Historic Preservation Act of 1996 and Section 4(f) of the U.S. Department of Transportation requirements. Approximately 12-ft, 6-in. needed to be added to its width, making it a total of 36-ft wide from truss to truss. This included two 11-ft travel lanes and two 3.5-ft shoulders, increasing the travel surface from 20-ft to 29-ft to accommodate the roadway width.

FINLEY realized that traditional construction methods could not be used on this project so the incremental launch method was selected. FINLEY developed the conceptual design for cutting and moving the entire 350 ft.-long north truss chord in-place by 12-ft., 6-in using AASHTO LRFD and Guide Specification for Temporary Works. The incremental side-launch method allowed the design-build team to meet the project demands and federal laws for construction and rehabilitation of the historic bridge.

Figure 9 Bridge After Renovation

This concept included a falsework and jacking system that allowed the North truss to be moved while still receiving lateral support from the South truss system. The South truss was designed to support the entire existing truss bracing members with the aid of this unique falsework system that stabilized the eccentric self-weight, wind loading and jacking forces through the many phases of the North truss jacking operation.

3. Launch Analysis

RISA software was used for the construction analysis. FINLEY developed an extensive and detailed launch analysis to include all critical load cases and conditions (ice flows, wind loading-45-70mph average gusts daily wind speeds, aesthetics, preservation requirements and environmental protection) and designed a unique jack and roller side-launching system allowing the design-build team to save 80% of the original truss, preserving the bridge as much as possible. Innovative materials were used to reduce dead load while providing a lightweight, strong and durable deck.

Weight was a big challenge because truck loading standards are approximately two thirds heavier than when the original bridge was built in 1929. An Exodermic deck, consisting of steel formwork and light weight concrete was used to reduce dead load while providing a lightweight, strong and durable deck. This efficient system maximizes the use of the compressive strength of concrete and the tensile strength of steel to provide a lightweight, strong and durable bridge deck.

The bridge was designed for a maximum wind speed of 100 MPH. For the top and bottom chord, along with gravity loads, the effect of wind loading and out-of-plane p-delta forces were considered because to prevent uneven jacking geometry during the side launch.

4. Launch Design – Temporary Supports / Falsework

To maintain its historical integrity, the plan was to widen the bridge leaving as many of the original steel members as possible and installing new structural bracing members within the widened portion of the bridge only. The design of a unique jack and roller side-launching system saved 80% of the original truss, preserving the bridge as much as possible. The innovative falsework and jacking system allowed the north truss to be moved with lateral support being provided from the south truss. The south truss was designed to support the entire existing truss bracing members with the aid of this unique falsework system that stabilized the eccentric self-weight, wind loading and jacking forces through the many phases of the north truss jacking operation.

Figure 8 Falsework

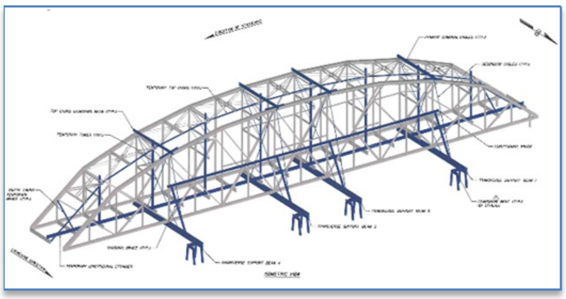

Figure 9 Schematic

The falsework was connected to the truss with a clamping system was used with PT bars to create friction connections that limited damage to the existing members. The falsework had two load paths, the transverse beams offered a significant portion to the vertical capacity and lateral resistance. An interior temporary truss was also used to resist vertical loads. The combined capacity of these two systems provided approximately 50% additional load capacity.

For the hydraulic system, FINLEY used an analysis of the anticipated forces due to friction resistance primarily, along with a comfortable safety factor for additional capacity.

5. Launch System

FINLEY designed the hydraulic side-launching jacking system that assisted with separation of the truss members from the existing connections, moved the North truss and facilitated fit-up of the new bracing members, as well as providing a means to adjust the camber of the North truss.

This hydraulic system had two functions. The first was the initial separation of the North truss chord from the truss. Following separation, the launch cylinders functioned to control the movement of the truss chord. The two cylinders at the end bearings were used as the driving cylinders, the remaining eight cylinders were used to maintain support of the upper and lower chord truss nodes.

The hydraulics consisted of dual action cylinders with a 3,500 psi power pack. Launching speed was controlled by flow control valves and individual needle valves at each cylinder. Ten specially designed 18-in. stroke capacity hydraulic ram systems were placed on the top and bottom chords and at each abutment, and provided carefully monitored constant pressure to nudge the 65-ton north truss on Hilman Rollers to its new location. Only 1/3 of the 3,500 psi capacity was necessary to move the rods.

Figure 10 Launching System

With a launching weight of 135 kips, developing launching force wasn't the critical aspect of this project; in fact, stability of the separated truss chords proved to be the most challenging aspect of the operation. The moving truss chord was not geometrically stable and also did not have the required internal buckling capacity in the top chord. The remaining truss chord had to support all the lateral bracing, which became an eccentric loading.

Each launch point was monitored with measuring rulers to ensure all launch points were moving together. Hydraulic cylinders were sized with two-times the required capacity for redundancy during the launch. Additionally, PT rods were added with locking nuts to limit any movement should a hydraulic cylinder fail.

During the side launch, the truss chord had to be maintained within a tolerance of +/- 1 ½" out-of-plane. The primary purpose of the eight cylinders on the top and bottom chords became maintaining the in plane shape of the truss chord. Each jack was monitored and controlled by workmen during each launch increment of 12 inches. After no more than three increments, the side launch operation was halted and the jacks were used to restore the equal total movement before continuing the launch. Additionally, the top chord was monitored by survey from both ends of the bridge for overall stability.

The most important aspect of the system was to provide stability to both truss systems during the launch. Close monitoring was performed to ensure the truss maintained a stable position at all phases of the launch.

Figure 11 Completed Launch

6. On-Site Launch

FINLEY's Engineer was on-site during the launch to monitor 10 critical connection points that, when cut free, would expose the truss to potential distortion and twisting. The side-launching was completed in 1.5 days, achieving a launching rate of 2 feet per hour.

7. Project Summary

To preserve as much of the bridge as possible, FINLEY developed the incremental side-launch concept for a falsework and jacking system which preserved 80% of the existing bridge. The design of falsework and jacking system allowed the north truss to be moved with lateral support provided from the south truss, along with a hydraulic launching system with dual action cylinders for the launch. New materials were used on the bridge deck to provide a strong, lightweight and durable deck which removed the load capacity restrictions the bridge had prior to the renovation. The design-build team worked closely together to provide an innovative engineering preservation solution to widen the bridge, meet the owner's needs in the most effective and safest manner, and provide the optimum financial and historic value to the traveling public. This project was completed on-time and on-budget. VTrans opened the Checkered House Bridge on May 28, 2013. The rehabilitation project makes the bridge safer, removes load capacity restrictions and provides historic value for the traveling public for the next 75 years.

Carolyn Carlson, P.E., VTrans Structures Project Manager had been involved with this historic project for 22 years. She comments, *"This was the first time that I had worked with FINLEY and I was very impressed with the innovative thinking that went into the bridge widening. During the widening, FINLEY was onsite providing technical support which proved to be critical in keeping the launch on schedule. FINLEY's expertise in both design and construction engineering was invaluable for this "first of its kind" project."*

This project won the following awards: 2014 Florida Institute of Consulting Engineers Engineering Excellence Grand Conceptor Award; 2014 Vermont ACEC Engineering Excellence Awards Grand Award; and 2014 ACEC Honor Award.

Summary

The use of incremental launch method helped to make this project viable in terms of efficiency, continued marine and vehicular traffic operations, and cost effectiveness. This project was particularly challenging because of the required incremental launch, the excessive launch weight of the truss and launch distance, and the high level of marine traffic in the navigable waterway below. Key to the success of the project was the development of a sound conceptual design, which should include an analysis and determination of the best type of launch system, launch sequencing, stability plan during launch, selection of appropriate moving supports, and a mission critical list of conditions that could prevent a successful launch and a plan to mitigate them

Credits

Chelsea Street Bridge photos courtesy of J.F. White Contracting. Figures 1, 2, 3.

Chelsea Street Bridge excerpts from a paper by John Pecora, III, P.E., Franklin M. Grynkewicz, P.E. and Jerry M. Pfuntner, P.E.

Checkered House Bridge photos courtesy of CHA - Figures 8, 9 and Jared Katz - Figure 1