HEAVY MOVABLE STRUCTURES, INC.

FIFTEENTH BIENNIAL MOVABLE BRIDGE SYMPOSIUM

JOINT WITH

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Advanced Control System Design for Bascule Bridges

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New Orleans French Quarter
Marriott Hotel, New Orleans

Abstract

This paper will describe the control system design and implementation for the Spit Bridge Upgrade project in Sydney Australia. The project operated under significant stress through its life-cycle, but the end result was successful completion without undue issues. Many factors contributed to this result, but the controllability, reliability and flexibility provided by the control system design, hardware and software was the main reason why the changeover could be completed in line with customer expectations.

While the controls are "advanced", the concepts are not, and should be available to other practitioners.

The Task

Spit Road, at the entrance to Middle Harbour in Sydney NSW Australia is a very busy arterial feeder to the city's central business district, and is the main transport route for approximately 200,000 people. Along its route, Spit Road crosses Middle Harbour on the Spit Bridge, a relatively low-level Bascule-type bridge.

The project task was to improve the reliability of the Spit Bridge Bascule system.

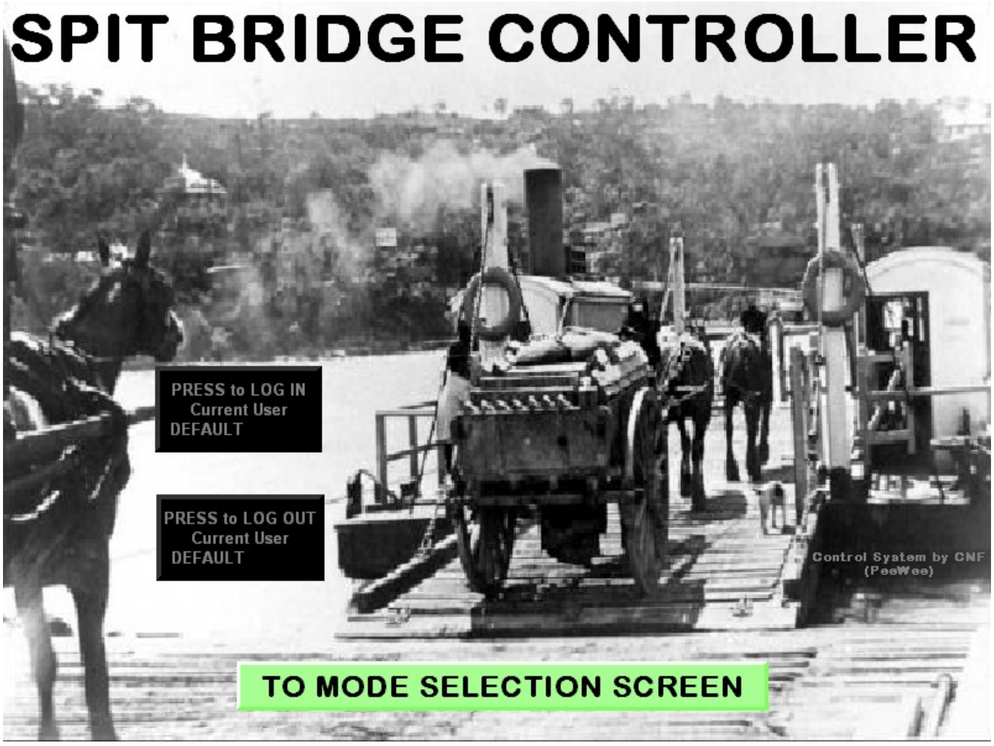
The higher priority traffic is automotive, but the bridge also carries a significant number of pedestrians and cyclists.

The lower priority traffic is marine. Any tall vessel wishing to pass must be at the bridge ready to proceed as soon as the Span is opened. If there are no boats in the vicinity, the operator does not proceed with the opening.

The operation of the Spit Bridge is manual and it is expected that this will never change.

The Background

This harbour crossing has provided the inner city residents of Sydney easy access to some of the most beautiful surf beaches in the Australia. In the very early days, this crossing was done by boat, then a floating ferry powered by horses and then a steam engine and finally a bridge.



The original Steam powered punt.

The original bridge was too low for tall ships to pass up into Middle Harbour. In the mid to late nineteen fifties, a movable bridge was built. This bridge would allow tall masted craft to access this section of Sydney Harbour. The bridge design was a single Bascule with a length of 37 metres (120 ft.), overall span of 227 metres (745 ft.), and it had four lanes for road traffic. The estimated mass of the bridge span and counter-weight is about 750 Tonnes.

The drive system that was installed on this movable bridge allowed the opening and closing times to be about 60 and 70 seconds respectively. A normal complete Raise/Lower cycle was expected to be less than 10 minutes to minimize road traffic disruption. The Spit Bridge has a published schedule of opening times. During normal week days, the bridge can be opened up to five times and on weekends an additional two openings are available on Saturdays and Sundays.

From the late 1950's to the turn of the century, this bascule bridge provided easy road access to the northern beaches of Sydney and a convenient passage for the larger boats which needed to enter and exit the Middle Harbour. During this period, Sydney has like most cities grown rapidly. The northern beaches region of Sydney has become a very sought-after residential and recreational area. As the population has increased, so too has the traffic, both land and marine-based.

Over time, the drive system became less reliable, due mainly to normal wear and tear. Mechanically, components were starting to fail due to fatigue and the electrical drive system was failing regularly, again due to component fatigue. The owners of the asset were under a lot of pressure to address the reliability of the bridge. The primary focus being to ensure that the bridge would close reliably to allow the road traffic to pass without delays.

After many studies had been completed over an extended period that saw several changes of the State government which has jurisdiction over the asset, a project was developed to address the reliability of the bridge movement. It was decided to completely replace the bridge movement drive system. The original electric motor and open bull-gear systems were to be replaced with a dual drive system which would provide redundancy to maximise reliability of operation.

The Issues

Political

Spit Road which services the Northern Beaches is widely recognised as the most difficult in Sydney to find an alternate route. Due to the city sprawl and the size of Middle Harbour, the closest alternate route would require vehicles to travel 20-30 kms (10-20 Miles) further than would normally be done. Extended road closure of this major city artery is virtually forbidden. It is a common scenario where, if the Spit Bridge Opens and stops the road traffic for more than 10 mins, politicians will receive severely negative publicity. It is also politically sensitive if the Spit Bridge does not open to allow tall ships to enter and leave Middle Harbour. Managing this aspect of the project imposed difficult time constraints.

Public Relations

During the change-over, the Spit Bridge drive system would be inoperable. For many months prior to the changeover, the local residents and harbour users were notified of the changes. The plan was to complete the work within a two week period during which time the bridge was not opened. All marine traffic was restricted to vessels which could fit under the bridge. It was planned that on the final night after the two week shutdown, the new drive system was to be site-tested. The road traffic would be stopped for 3 hrs while the new equipment was tested and then put back into service in time for morning peak hour traffic.

Technical

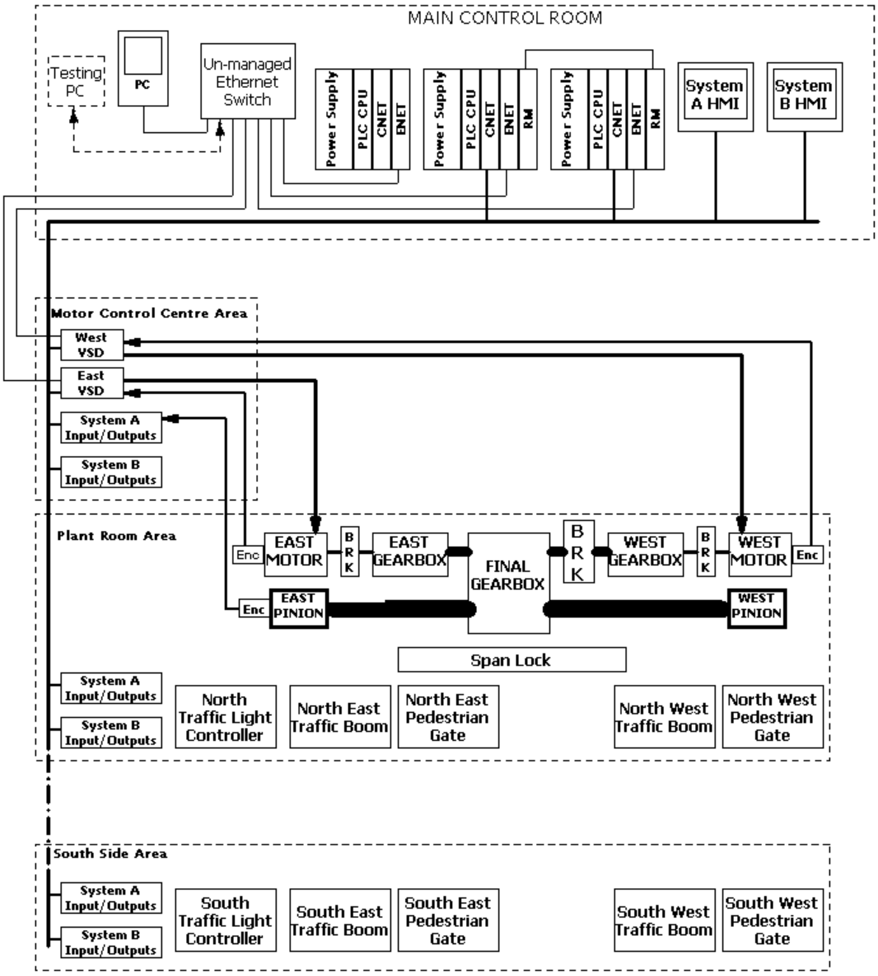
To fulfil the client's requirement of Reliability, the scope of this project was to replace the entire Mechanical and Electrical components of the Span Drive system as well as replacing the electrical wiring of the Pedestrian Gates and Traffic Booms and installing a new Traffic Signal Control System. The project is the subject of another Paper being presented at this symposium and as such, this Paper is focused on the electrical control design and commissioning aspects of the project.

The client's specification was a constant source of confusion due to the convoluted manner in which it had been written. Some of the key factors relating in particular to the issue of Reliability were conflicting. This became a major issue for the project team to manage.

The prime contractor is a specialist infrastructure builder, comprising a team of highly skilled engineers and project managers experienced in building roads, bridges and other civil projects. They were successful in winning this Iconic project due to their bridge building expertise. Within the corporate team, they had electrical engineers who regularly designed power distribution and lighting systems, however they lacked the specialist knowledge to manage the complexities of machine control. Understanding compliance with safety and other relevant industrial control standards was a major concern for the project managers. We were invited to join the team to add some expertise in this area of machine control. Meetings with the client to resolve issues were often unfruitful as much of the specification was contradictory.

Time was becoming a major factor and so we took the approach of over-ruling the contradictory factors in question in favour of the understood intent. We talked to the operators and maintenance personnel about how the bridge should operate and what were the major issues which we needed to address. Based on this research and a general understanding of the intent of the specification document, we formulated an electrical system design.

- Dual mains power from the North side and the South side via an Automation Transfer Switch provided redundant 3 phase power.
- Dual PLC CPUs with duplicated inputs and outputs provided the redundant control architecture.
- Dual VSDs which are connected to the dual Motor/Brake/Gearbox assemblies provide the redundant drive systems.
- Dual high intensity HMIs in the control house provide redundant operator interface.



Additional equipment installed in the control house is the spare PLC system and a PC which is loaded with the various software packages required to support the total system.

Duplication of all critical equipment ensures that if any component in the system fails, it will not prevent the Bridge Span from being returned to the deck, allowing the priority terrestrial traffic to return to normal flow.

Implementation

Ancillary Equipment

The new control system replaced all existing electrical functions. Two new road, pedestrian and marine Traffic Signal Controllers were installed, one on the South side and one on the North. The Pedestrian Gate movement drives and Traffic Boom movement drives were retained, however the position sensing limit switches and cabling were replaced. The existing Span Lock unit was retained; however the Motor/Brake/Gearbox was replaced as well as the sensing limit switches and cabling.

All critical sensing devices were equipped with dual contacts which were wired to their respective system inputs.

With all critical equipment sensing and control devices duplicated, the system is capable of tolerating a single point of failure. The above shows a simplified control network of the major components and identifies the locations of that equipment.

Span Drives

The mechanical design of the drive system has two independent Motor/Brake/Gearbox sets which are direct coupled to the input shaft of the final gearbox. There is also a large holding brake mounted on this input shaft.

The Span can be Raised and Lowered by either motor. It is possible to de-couple each Motor/Brake/Gearbox set for repairs and maintenance without impacting on the bridge serviceability. However, normal span Raise/Lower activities are done with the duplicate systems continually coupled. When the Span is moved, all brakes are released and both motors rotate. One motor is under power and the other is coasting. As well as these two larger main drive systems, the span can be slowly Raised/Lowered with a smaller Pony motor, and if all else fails, a diesel motor can be quickly deployed.

Servo Drives

Each main drive motor has its own dedicated VSD. These AC drives are equipped with Closed Loop Position Control capability. They were selected due to our concerns regarding Over-travel while Raising and then deceleration, parking and holding while Lowering. We were aware of the limited amount of time which we would have to commission this system and so we wanted the most powerful and flexible PLC and Servo Drive system available. Many additional benefits were also gained by this selection, particularly in the event of an Emergency Stop during a routine Raise/Lower of the span. We were able to demonstrate that a fast stop could be executed without excessive stresses being applied to the mechanical equipment. Recovery from an Emergency Stop is also simple, regardless of the position or the direction of travel of the span prior to the Estop activation.

Implementation of Closed Loop Position Control requires position information to be read by the Position Controller within the VSD. Each of the main drive motors was modified and an encoder directly coupled to the fan end. These encoders provide very precise motor shaft position and speed information to the controller. Absolute Multi-turn encoders were used so that in the unlikely event of a power outage, the Span Position would be retained.

Since both main drive motors are always connected, a software test ensures that these two encoders provide a position comparison between the driving motor and the coasting motor. A third encoder is mounted on the final output drive shaft. This completely independent slow turning device monitors the actual movement of the Bascule Drive pinions. A software test ensures that the final drive is moving in harmony with the motors. Any out of tolerance movement of the final drive shaft will stop the span drive system.

Span Drive Brakes

Each main drive motor is equipped with a small stopping drum brake which is intended to assist with deceleration of the span movement, particularly in the event of a fault.

Due to the dual coupling of both main drive Motor/Brake/Gearbox systems, the Stopping Brakes are controlled by the same contactor. This ensures that both are Released and Set with the same control function irrespective of which main drive motor is being used to Raise/Lower the Span. Control of these brakes is done using conditions and logic derived in the PLC CPU and controlled by PLC outputs.

A large holding disk brake is installed on the main gearbox input shaft. This brake is intended to hold the Span once it has stopped moving. Control of this brake is done using conditions and logic derived in the VSD and is controlled by VSD outputs.

Control of both brake systems is very critical. The smaller drum brakes must be regularly Set while the Span drive is moving to ensure the surfaces are in peak braking condition. Conversely, the larger disk brake should only be Set once the Span drive is stationary to ensure that the surfaces are not damaged from excess friction.

Coordination of the Releasing and Setting of these brakes is very critical to the behaviour of the Span. Landing 750 tonnes of steel on the road way without any significant bump and then winding the backlash out of the gear train prior to brake Setting became a key factor in the coordination logic.

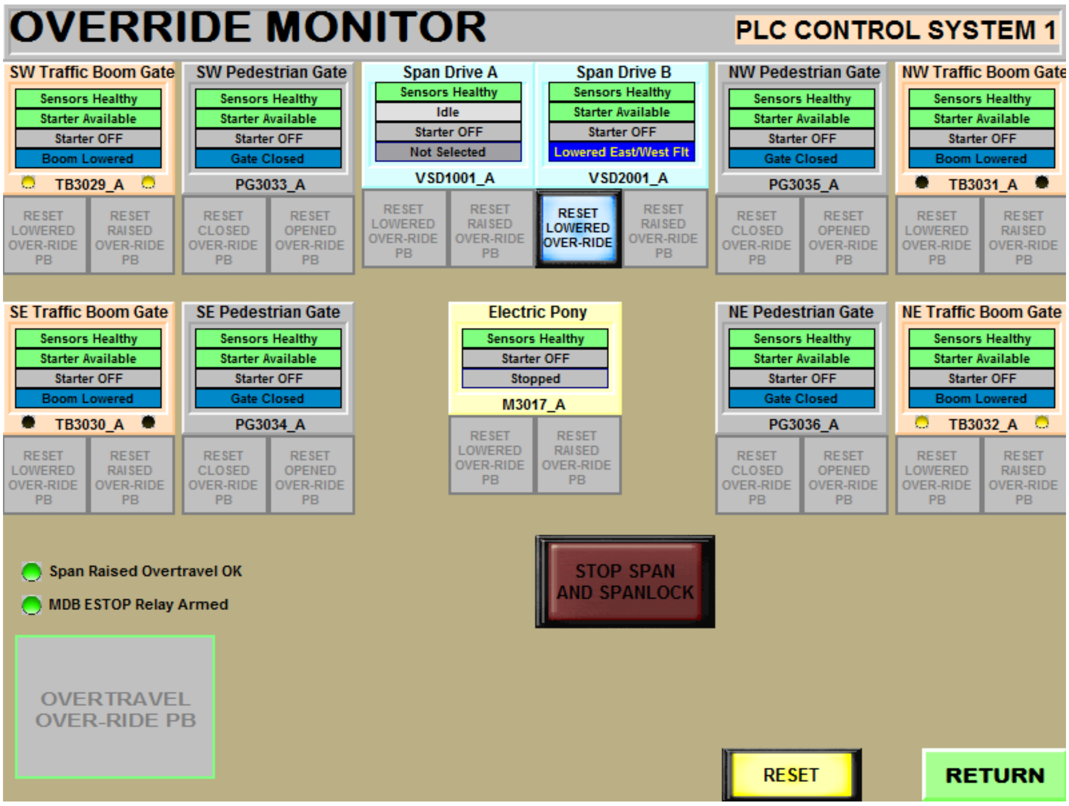
Software Development

The PLC program is written in ladder logic. As an aid to ensuring consistency of the Ancillary Equipment, generic logic functions were developed for each device. These included the Traffic Lights, Pedestrian Gates, Traffic Booms, Span Lock and the Span Drive VSDs.

Once these sections of logic were developed and Lab tested, the code was encapsulated and stored as generic Add On Instructions (AOIs). Each device is then programmed using these AOIs which ensure that the resultant behaviour of the device is identical to the others. This method of coding helps to minimise errors and guarantees consistency in the logic which is a key factor when commissioning with virtually no time.

As an example of the potential commissioning issue we faced, there are four Pedestrian Gates and each gate was programmed into two systems. This resulted in eight pieces of code which were identical except for the input and output addresses. Using this AOI feature virtually eliminated any possibility of programming errors.

All critical sensors are duplicated or have dual contacts. The inputs from these sensors are initially processed with some logic which checks this duplicity and provides an operator override feature in the unlikely event of a total failure of the device. Prior to commencing a normal Span Raise/Lower cycle, the operator must reset all overrides.



The above screen grab shows the override monitor display.

Software Simulation

Development and testing of the software was done entirely off site. It was not at all possible to do any testing of logic on the Bridge as it is in constant heavy use.

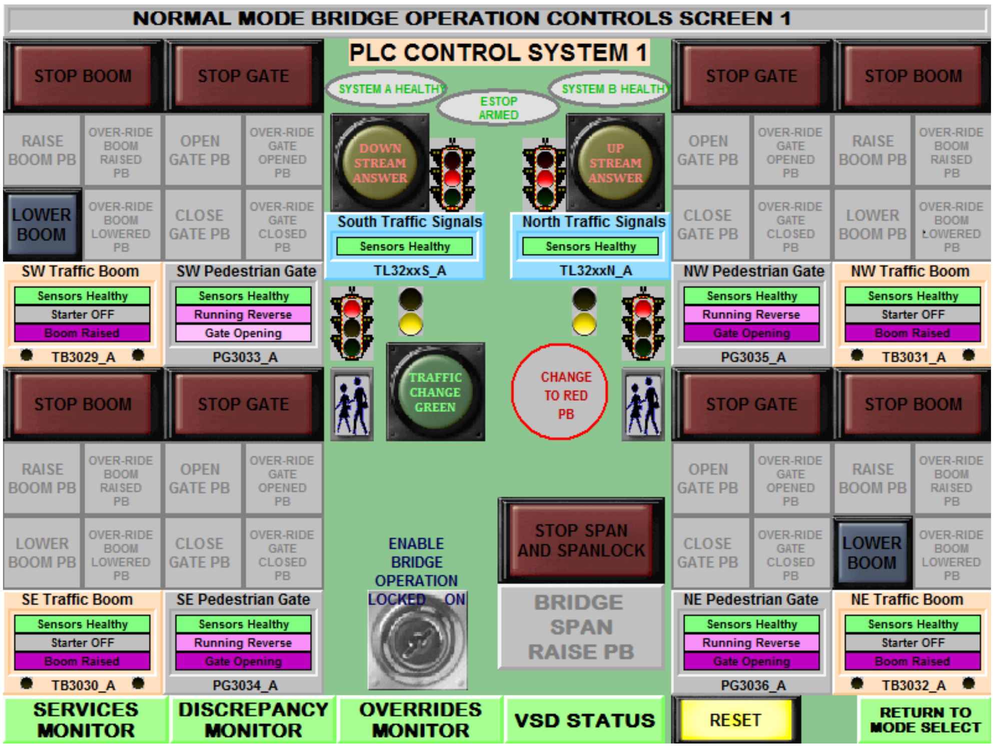
This posed a significant issue which was resolved by developing simulation code for each of the devices. The operating logic for the Pedestrian Gates, Traffic Booms, Traffic Lights, Span Lock and Span Drives were all developed and tested using simulation functions running as background tasks in the PLC. These simulation functions which also evolved as the project progressed became the “Virtual Spit Bridge”.

At various key project Hold Points, we were able to demonstrate our software progress without stopping one Bus, Bike or Boat.

HMI Screens

The original functional specification was very orientated towards the operator using PBs, PLs and gauges on the operator console. There was a screen, but it was only a status, alarm and trip monitor. The specification identified various motor status data and span position indication and the remaining functionality was intended to be services status/alarm and trip indication.

We were concerned about how the operators would cope with the new system which in basic terms is identical in functionality to the old system which was being replaced. The major differences were how the operators interacted with the new system. Even though the operators were initiating the same functions that they had always done, the look and feel was always going to be very different. It was a key challenge for us to make this dramatic change as painless as possible for the operators.



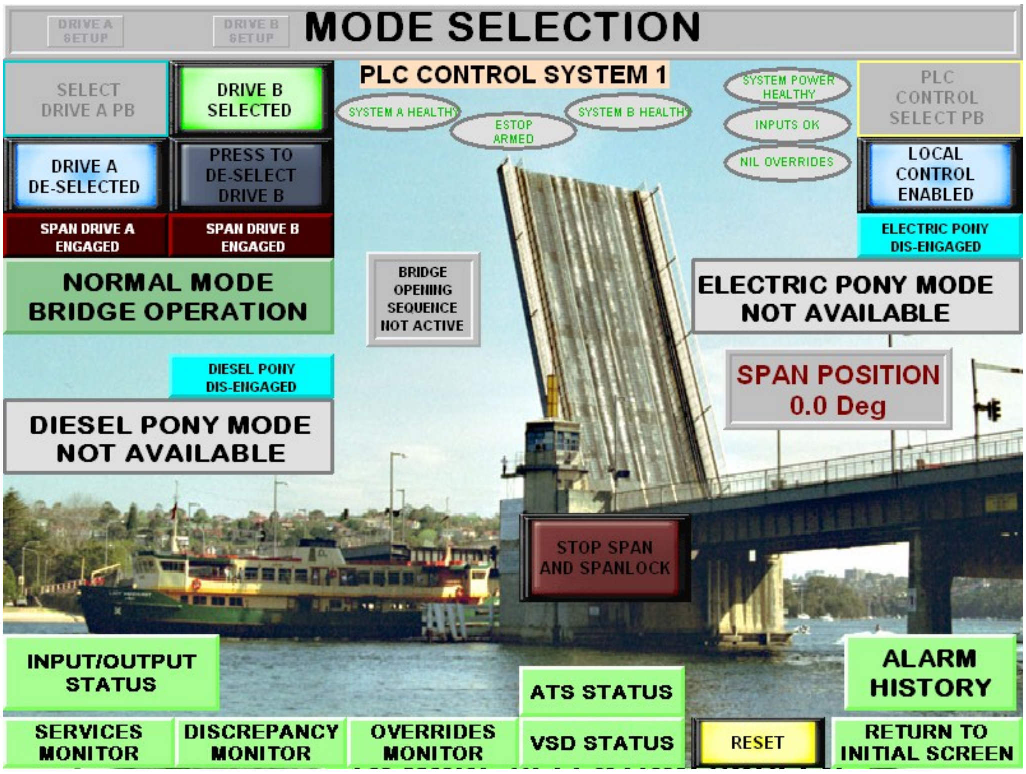
The above screen shows the use of the HMI to reduce confusion of which functions are available to initiate. The Stop Buttons are always active, but the only functions which are available to move at that point are the SW and NE Traffic Booms (We drive on the LHS of the road) which ensures that the approaching road traffic lanes are blocked prior to blocking the exiting traffic lanes.

Real estate in the control house is very scarce and we needed to position the control console close to the window so that the operator could easily see the total bridge deck. Due to the manual operation of all equipment, this is a very important feature which allows the operator to visually check all Gates, Booms and the Span prior to initiating any equipment movement.

We proposed that the HMIs would provide a more user friendly interface for the operators. Only functions which are safe to initiate are highlighted on the screen. This eliminates the possibility of incorrect operation of a device. If the button is greyed out, the device is not ready to operate. Software interlocks do prevent these conflicts; however, operator training can become an issue. Without some visual feedback relating to availability, operator controls which sometimes work and other times don't, are always sources of confusion. The greyed out buttons on the HMI solved this issue.

After demonstrating the functionality of a software based operator terminal, the client accepted this solution. The control desk now has a master ON/OFF switch, an Estop PB , a Reset PB and two high intensity 12inch HMIs.

Another factor which was solved using the HMI,related to the multiple alternate operating modes which are available. This was greatly simplified by using the HMI to select these modes.



1) East Drive (90kW VSD)
2) West Drive (90kW VSD)
3) Pony Drive (15kW DOL)
4) Diesel Engine (used when all else has failed)

In addition to providing a simple operator interface, the HMI monitors can be used to assist in troubleshooting the PLC control system. All Inputs and Outputs are displayed on separate screens which show the state of the inputs and also the state of the duplicated input pairs.

Factory Acceptance Test

The mechanical drives systems contractor was based in Newcastle, approximately 2hrs north of Sydney. Once the equipment was assembled in their test area, we were able to debug and run the control system in a limited manner. Over a period of three weeks, we bench tested all components. As an aid to simulating the span behaviour in a test environment, a dummy inertia disk and a dynamometer were used. This provided a simplified characteristic of the load and was invaluable in setting up the VSDs.

A great deal of care was taken in tuning the VSDs to the induction motors. The initial tune was done with the motors de-coupled from the first gearbox. Once this was completed, the couplings were reconnected and then a full inertia tune was performed. The result of this VSD/Motor tuning process is a very stable drive system. Our subsequent testing demonstrated absolute smooth performance. There is no significantly detectable vibration or audible noise from the mechanical drive equipment.

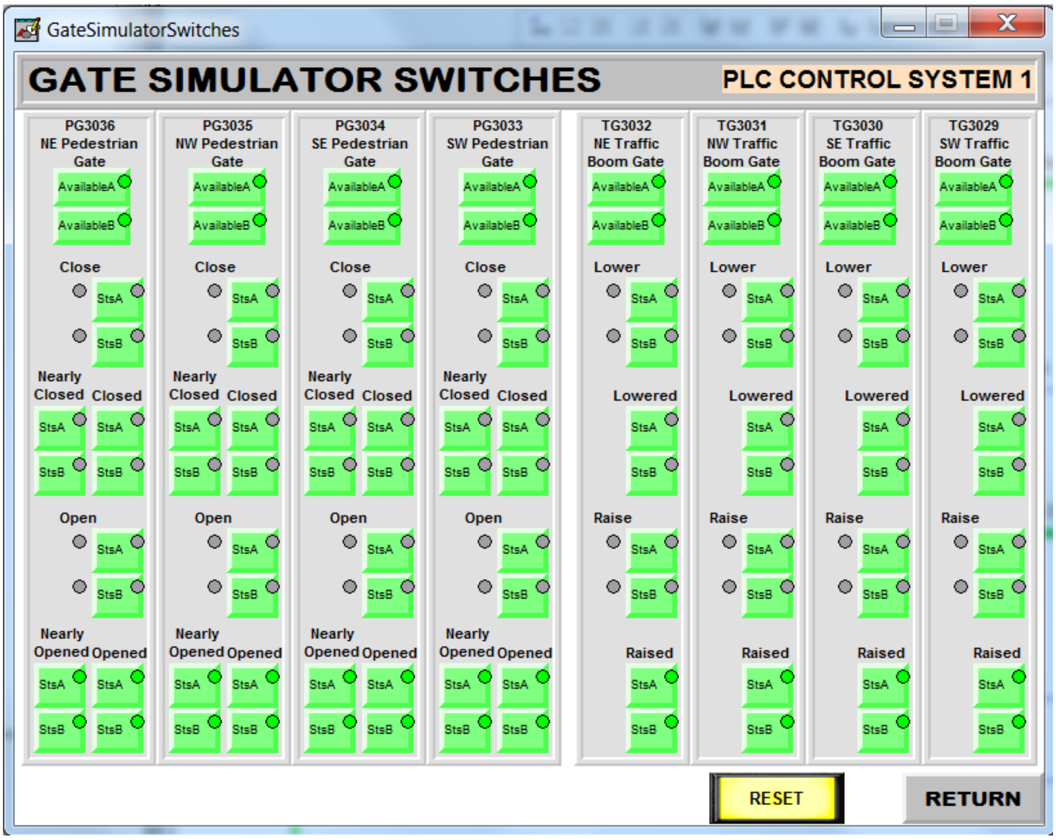
Training System

A critical requirement from an operator's perspective is a system which allows off line training. Again, due to the heavy demand on the bridge, it is impossible to allow any type of operator training on the real system. Any extension of the advertised opening times would almost certainly result in public outcry and inevitable political intervention. It was clear that a computer based training system would be the only practical and economical solution.

During the software development and testing phase, we created a Virtual Spit Bridge. These software modules were used as the core of the training system. Each of the devices (gates, Booms and Main Drives etc.) had a small section of PLC logic devoted to simulating the behaviour of the field sensors and actuators. The training system is actually run in the spare PLC processor in the control desk. This simulation logic is enabled and the Input/Output modules are disabled to prevent logical conflicts in the processor. A version of the HMI screens is installed on the control house PC. This HMI system is virtually identical to the hardware-based 12-inch touch screens mounted on the control desk. The only difference relates to the fact that the screens appear in a window on the PC and all operator actions are initiated using a mouse. All other functions are executed within the spare PLC which is running a modified version of the real system. These modifications are essentially related to the simulation of the field sensors.

Implementation of the training system was simply an extension of the real system. Delivery was done prior to final commissioning which allowed the operators to become familiar with how the final system would "look and feel". After a day or two of using the training system, the operators had no issues when it came to Raising and Lowering the real bridge span.

As an additional feature, the training system has some special screens which can be accessed to generate faults in the off line system. These faults allow the operators to understand what happens when a faulty device is detected by the control system. An essential feature of the control system is its ability to tolerate a single point of failure without compromising the bridge Raise/Lower functionality. The training system is designed to allow operators to experiment with this feature.



This is one of several screens which are accessible on the training system. Artificial faults can be created in the training system from these screens.

Data Logger

The client requested that all operator actions, process alarms and trips be logged to a data base. We installed a data logger Historian software package which will store this data for at least 5 years. We understand the reason for having this historical record was to assist in resolving any ongoing reliability issues. However, we have not had any call-outs, so it would seem that the system is very reliable and the data logger is probably an unnecessary inclusion.

The Change Over

With all of the mechanical and electrical equipment completed and tested, the change-over was scheduled. In order to satisfy public access requirements, the maximum time allowed for the change-over was two weeks. During this time, it would not be possible to raise the Span. For several months prior to the critical time, many tasks had been completed while the Span was still operational. Some items had been removed and many electrical cables had been installed but not terminated. In short, all that could be done prior to the change-over was done, however, it only amounted to a very small part of the total effort required to finalise the project.

Spit Bridge Control Room during the Change-Over

This two week period was typically chaotic. There was demolition, civil construction, mechanical assembly and electrical installation and commissioning happening. In many cases, these disciplines were working concurrently. The Spit Bridge control pier is quite compact and so the available area for installing equipment is very small. Coordination of this work was very complex, and as is always the case, various tasks were delayed and the schedule was always under stress.

The day before the scheduled change-over, and after a mid-night crisis meeting with the client, a five day extension was agreed.

The following Friday morning at 1:00am, the road traffic was stopped for three hours while we commenced our setup and testing procedures. After raising and lowering the Span slowly with the small pony motor (which allowed limit switch positions to be located), we then initiated a Raise and Lower of the Span with one of the main drive systems. As expected, it all worked exactly as planned. The initial return of the Span to the roadway was not ideal and it was clear that several attempts would be required before the lowering, parking and brake setting sequence was perfected. At 3:30am, we were instructed to stop testing and the Spit Bridge was put back in service.

Over the next few days while the Span was being operated normally we made small adjustments to the re-entry sequence. The deceleration settings and the creep speeds were modified so that the Span Lowering phase was as fast as possible without bumping the toe of the Span harshly on the supports. The Setting of the Stopping Brakes was also adjusted to ensure that the gear train backlash was fully reversed before the Main Drive was stopped and the Holding Brake was Set. Once this fine tuning completed, no additional testing or modifications to the Span Drives have been required.

The above trend shows the performance of the servo controlled Span movement. These traces show Span Velocity (Light Green), Span Position (Purple), Motor Velocity (Cyan), Motor Position (White), Motor Current (Orange), Motor Stopping Brake Status (Blue) and Holding Brake Status (Dark Green). It can be seen that as the Span is Raised, the Velocities increase together. The Motor Current increases dramatically and appears to be unstable until the Velocity stops increasing. This apparent instability is generated by the Servo Control of the Motor. As the mass of the Span is accelerated, the motor current is very high. The variations in instantaneous current are the result of the small variations in load as the pinion gear moves from tooth to tooth. The Servo Drive is maintaining its trapezoidal move profile. Any variations in load result in a corresponding variation in required torque which is seen in the current variations. If you look closely at the peaks and troughs of the motor velocity and current, you can see 27 ripples which relate to the number of teeth which the pinion gear presents to Span gear. Similarly when the Span is Lowered, the same behaviour can be observed.

This tight control of the Motor is possible due to the performance of the Servo Drive system and the critical tuning which was done during the FAT.

The timing of the two brake systems is also evident. The Stopping Brake is Released prior to moving and is Set prior to stopping. The Holding Brake is Released as the Motor starts to move and is Set as the Motor stops moving.

The success of the controls aspect of this project can be attributed to the choice of the equipment and the use of Closed Loop Position control, along with the efforts of the electrical team. The time constraints imposed by the late running “other” works were extreme, and in reality amounted to no more than two hours to test and tune the control system. Despite this, the bridge has continued to operate reliably since commissioning. This outcome would be unlikely with conventional-type controls.

Summary

The adoption of advanced controls for the Bascule system at Spit Bridge has resulted in a highly reliable, high performing bridge, which was successfully commissioned and tuned in two hours.

The controls, while advanced, are not overly complex. They provide operators with the tools to operate the bridge in a safe and reliable way, and minimise operator error.

Benefits have flowed to all parties involved; the client, the operators, the contractor, and the travelling public.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15-18, 2014

Enhancing Pontoon Bridges in the Bayou State: Constant Tensioning System

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Supreme Integrated Technology, Inc.

Theme: Many of Louisiana pontoon bridge systems are being converted from a single point of contact device (e.g. electric driven split drum logging winch) that was difficult to operate, required the cable to be submerged on the bottom of the bayou during openings for vessel crossings, caused a safety issue to the operator, required clear openings into the bottom of the operator house, and were difficult and expensive to maintain; to a centralized hydraulic operated winch system that is PLC controlled and separated by pipe runs exterior to the operator house into two opposing winch devices that maintain cable tension at all times to keep the wire rope clear of the water.

New Orleans French Quarter Marriott Hotel

Synopsis:

LA DOTD Maintenance Division sought an inexpensive means to convert at least five Pontoon Bridges to a hydraulic winch system that would fulfill the following needs:

1. Simplify operator control.
2. Reduce safety hazards to the operator.
3. Reduce short and long term maintenance concerns and costs for the State.
4. Maintain wire rope clear of the water during entire period of operation and while open or closed.
5. Reduce corrosion and abrasion effects on wire rope cables from laying on or in the bottom.
6. Ensure that during and after testing of the new system, that the old system could still be connected and operated in an emergency.
7. Provide constant back tension to assist positive control in acceleration and deceleration.

The concept produced by Supreme Integrated Technology, Inc. (SIT) utilized two opposing winches in near proximity to the Operator house, but on opposite sides of the pontoon bridge that controlled the position of the pontoon by imparting a pulling action at one end of the pontoon and a constant tension drag on the opposite end. The wire rope was run through two new positioned sheaves to maximize the pulling forces on the bridge, yet allow the wire rope to run alongside the pontoon clear of the water when the bridge was fully open. This ability removed any need to run wire rope through sheaves located on the far side of the water way and allowed the winches to exert direct forces on the pontoon bridge at all times.

Hydraulic power for the winch system is provided by a centralized hydraulic power unit using redundant electric motor/pump combinations for back-up. Each pump provides power directly to a common hydraulic manifold with porting and valves for winch directional control, brake control, and return fluids. This system successfully removed all operating machinery and associated noise from the operator house.

Remote control of the hydraulic power unit and winch system was provided in a shelf style operator console with joy stick functional control and touch screen system feedback display. This control system provides an automatic function of creep speed, ramping speed, and full speed based on the position of the bridge in the canal. A back up manual and high current control ignores the positioning feedback and places the system in a high pressure state for power and low flow for control condition. The operator merely maneuvers a joy stick in either condition to open or close the bridge. The only condition for movement that is still provided from the previous system is that the end gates have been lifted.

The system at Bayou Sorrel has been operational for nearly four years without operational or maintenance incident, other than minor adjustment. It has proven itself in high wind and high water/current situations, and aids in operator control of the waterway and roadway. This paper defines in detail the conversion process from wire rope routing modifications, equipment operation, isolation and new installation, electrical power upgrades and control system implementation, and hydraulic operation protocols. All of the objectives of the project were met, and additional bridges have been or are in the process of being converted to this system.

Background:

Pontoon style bridges afford a means to install movable bridge sections in remote areas that incur a high degree of water fluctuation and may not be economically or operationally suited for a fixed structure with center swing span or side loaded bascule span. These floating bridges are almost always pivoted about a lever arm, bushing, and pin and moved by wire rope cables that pull the far ends of the bridge about the pivot point to an "open to marine traffic" or "closed to marine traffic" position. Movable ramps on each end both lock the bridge in the closed position and act as a ramp for vehicular traffic. These ramps are lifted prior to bridge movement and remain lifted while the bridge is in the open position. When raised, the ramps serve as a traffic barrier to protect vehicular operators from traveling off the ends of the road opening. When closed, the ramps move with the pontoon bridge based on the crossing's water height and operator controlled ballasting of the pontoon.

Bayou Sorrel Operator House Winch

As described, the theoretical pivot arm of these bridges extends nearly the full length of the pontoon from pivot point to wire rope connection point. The nominal wire rope line pull required to achieve movement in the least favorable conditions of wind and current is no less than 10,000 lbs. A ¾" EIPS IWRC wire is used to provide resistance to chafing and protect the system should the pontoon come in contact with floating debris or vessel traffic. In the case of Bayou Sorrel, the bridge was operated with 15 HP Electric Driven Logging Winch. This winch required the operator to sit in the driver's seat and operate foot brakes and clutches, as well as, hand operated directional speed controls. The operation required pulling on one cable as another cable was slackened. Additionally, since the cables were fairlead out of the bottom of the operator's house and perpendicular to the lie of the bridge, the cables needed to be fairlead again around sheaves to the opposite sides of the bridge. Although the opening cable would be connected directly to the far end of the pontoon, the closing side cable was fairlead again to the far side of the waterway to provide the necessary angle for pulling the barge into the closed position.

When the bridge was in the open position, the closing cable needed to be laid along the bottom of the waterway to allow marine traffic to cross over the cable. Even though it is a sound method for maneuvering the bridge and was utilized for many years, a series of problem areas exist with this configuration:

1. All wire rope cables were routed to the inside of the Operator House leaving an opening to the environment in the floor.
2. The Operator sat on the machine while tensioning cables fed to drums directly adjacent to the operator, placing the operator in a hazardous area should a cable part as it was reeling onto the drum.
3. While operating the bridge, the operator did not have rapid control of marine traffic via marine band radio.
4. Should a collision occur between the pontoon and an obstruction or marine traffic, or the wire rope when lying near the bottom is caught in a vessel's propellers, the risks of damage to equipment and operator are multiplied.
5. The acts of shock loading the cable and laying it into a muddy or sandy bottom shortened the life expectancy of the wire rope do to corrosion and abrasion of the wires.
6. In several cases, vessel propellers actually lifted the cables into the propeller blades causing catastrophic overloading of rigging and parting of the cables.

As events and risks were realized, changes in operation were sought. Both Belle River and Bayou Blue Pontoon Bridges were modified with slightly different takes on the conversion presented in this paper, but none has been as smooth a conversion as Bayou Sorrel Bridge.

The New System:

A new hydraulic winch system was envisioned that placed separate 10,000 lbs SWL winches on accessible platforms in line with the operator house. One winch would be in close proximity of a Central Hydraulic Power Unit (HPU) on a platform directly behind the operator house, and the other winch would be on the opposite side of the roadway and connected to the Central HPU by piping. The Central HPU would utilize two redundant 25 HP electric motor/pump combinations such that either unit could open or close the bridge in under 3 minutes. A common manifold on the Central HPU contained the control valves, pressure reducing valves, and relief valves necessary to control the winch brakes and directional movement. Proportional control would be provided to vary the speed of movement via a PLC output. The position of the bridge would dictate the speed of the bridge, and automatically stop the bridge at certain fully open or fully closed points via a PLC input from an analog rotational feedback device.

When the system was started and brought up to pressure, the brakes would release, and the winches would exert a line pull tension necessary to hold the wire ropes clear of the water. Once the barriers or ramps were raised, a relay allowed permission to operate the winches. To open the bridge under normal conditions, the operator simply moved a joy stick in the open direction and the proportional valve would shift to open the bridge starting in a creep speed. A rotational feedback unit would provide a signal to the PLC equating to angular rotation of the bridge about its pivot arm. Three zones were setup, including nearly open, midrange, and nearly closed. The direction of movement and position of the bridge would dictate the PLC output for valve ramping and flow setting, while the joy stick only controls direction of movement.

While the bridge is in movement, separate outputs are given to valving on the opposite winch to provide a drag pressure, thereby keeping the wire rope taut while allowing the winch to slip as needed under a load. This is accomplished by a set of 2-way solenoid cartridge valves, associated preset relief valves, and a suction line off the reservoir to keep the winch motor flooded. Cross port relief valves protect the winch motors from over pressurization and counter balance valves act to bring the motor to rest under a hydraulic lock when hydraulic pressure is not available or the proportional valve closes.

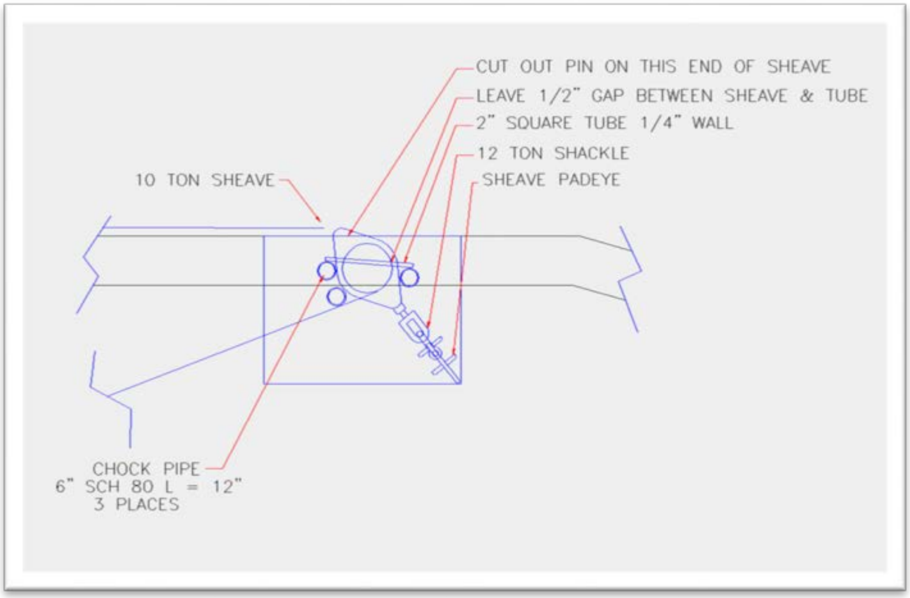
During periods of high wind or current, the system can be shifted into High Pressure mode. In this mode, the proportional valves are bypassed and normal solenoid operated valves are utilized with higher pressure, but at a reduced flow rate. Control of the bridge is firmly in the operator's hands and the bridge electronic position is not affected by bridge speed, making this the back-up mode should the rotatory feedback unit become inoperative.

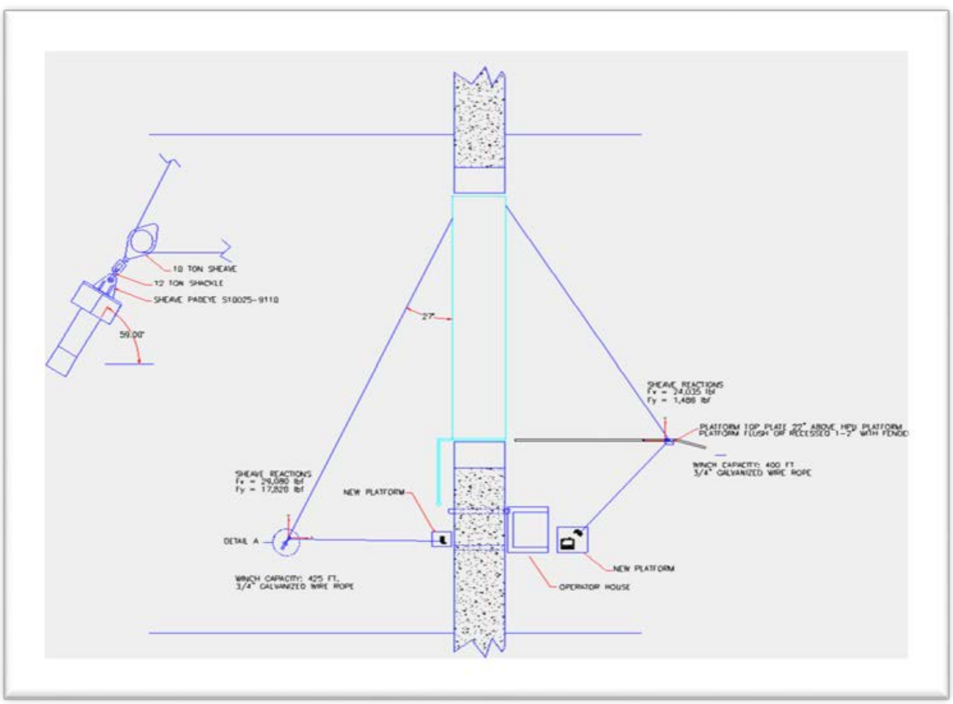
Wire Rope Routing Modifications:

The earlier bridge design routed cable in directions 180 degrees from each other off the double drum of the logging winch directly underneath the operator house. The wire rope needed to be fairlead out to sheaves normally located at the ends of a fender system where the cables were turned to cross the shipping channel towards the head of the bridge. Because the pivot arm is on one side of the bridge, the pontoon bridge only swings in one direction. The cable pulling the bridge open to marine traffic was connected directly to the bridge and gained advantage based on the changing angles from the bridge connection to the fairlead sheave as the bridge drew closed. The wire rope on the opposite side of the bridge needed to be fairlead across the channel and once again from the opposite shore or dolphin prior to being connected to the bridge. This was so the cable could be slacked while opening the bridge, but had a stronger pull angle when closing the bridge. The problem caused by this rigging arrangement was that the wire rope had to be fully slacked to allow it to rest on the bottom of the channel for vessel passage.

Although the State desired to have the system ready to reuse, if necessary, the cable routing was of little concern. In all cases, the fairlead position for the opening side winch would remain unchanged. However, to produce a drag on the closing side winch and keep the cable from interfering with any obstructions and out of the water during operation, this fairlead needed to be positioned in line with the closing side edge of the pontoon when closed, yet at an angle to allow pulling the pontoon bridge fully closed. This meant that a new dolphin needed to be put in place with a steel plate on top in order to weld the sheave padeye. The dolphin also needed to be protected from vessel traffic by a newly designed fender system. Once completed, the new winches could be set on their platforms with the center of the winch perpendicular to their respective fairlead sheave. Because the closing winch cable would remain taut throughout the operation, it was connected directly to the original padeye on the closing side of the pontoon bridge. The only difficulty was in blocking the blocking the sheave on top of the dolphin to restrict its ability to swing out of line with the bridge when closed.

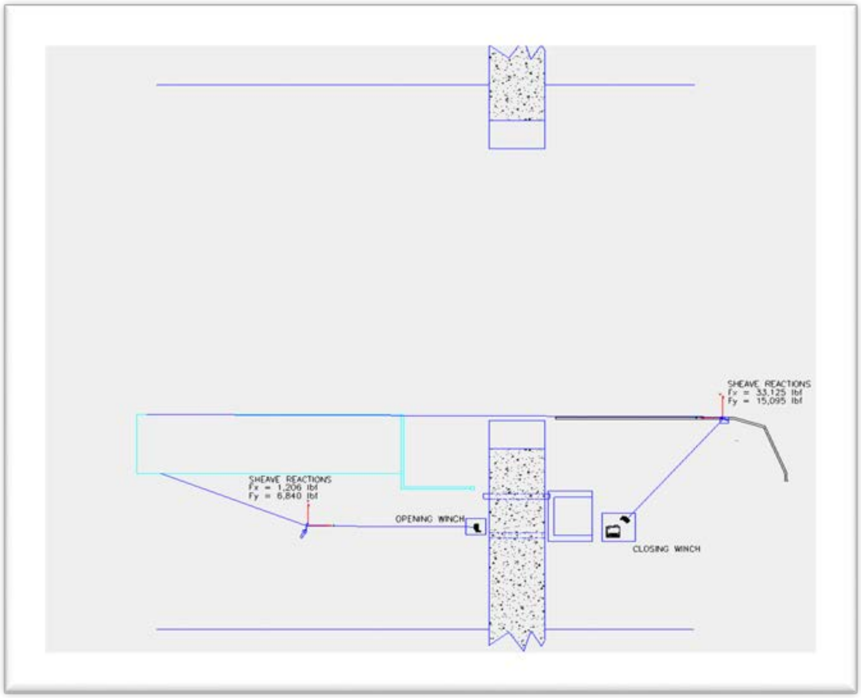
Closing Winch Fairlead Sheave





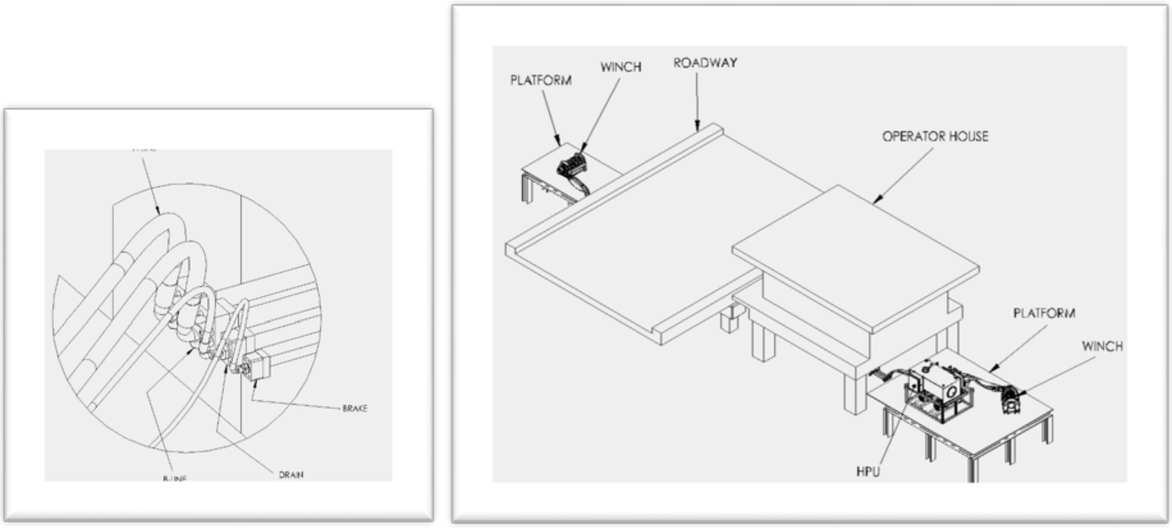
Bridge Closed to Marine Traffic, and Inset of Opening Winch Fairlead Sheave

Bridge in Open to Marine Traffic Position.



Equipment Installation:

Besides the enhancement to the fender system and the new dolphin, upgrades to the electrical wiring was required to shift from a 15 HP logging winch to the 25 HP HPU (discussed later). Installation also includes a new platform for the new HPU, closing winch, and opening winch, installation of the new operator control panel, PLC cabinet, and uninterrupted power supply (UPS) enclosure, and interconnecting cabling and piping as required. In all cases, the State Wide crew was responsible for pile driving and building the platforms as defined by SIT. When these were complete, the equipment could be lifted onto the platforms and aligned. The HPU and winch foundations were then welded to the platforms. The operator panel was mounted on the wall in the best location for the operator to view the opening and closing operation. The PLC cabinet and UPS enclosure were similarly mounted to a clear space on a wall in the operator house. Interface cables were run through conduit between the original control console, new operator control console, PLC cabinet, UPS enclosure, and the HPU dual motor starter and junction box. A rotary feedback unit was mounted on top of the pivot arm pedestal with a swing arm connected to the pivot arm. Armored cable was used to route the signal back to the PLC cabinet. Hydraulic piping and hose assemblies were routed between the HPU manifold assembly and the two winches.



General Arrangement of HPU and Winch and Piping Inset

Operator Console

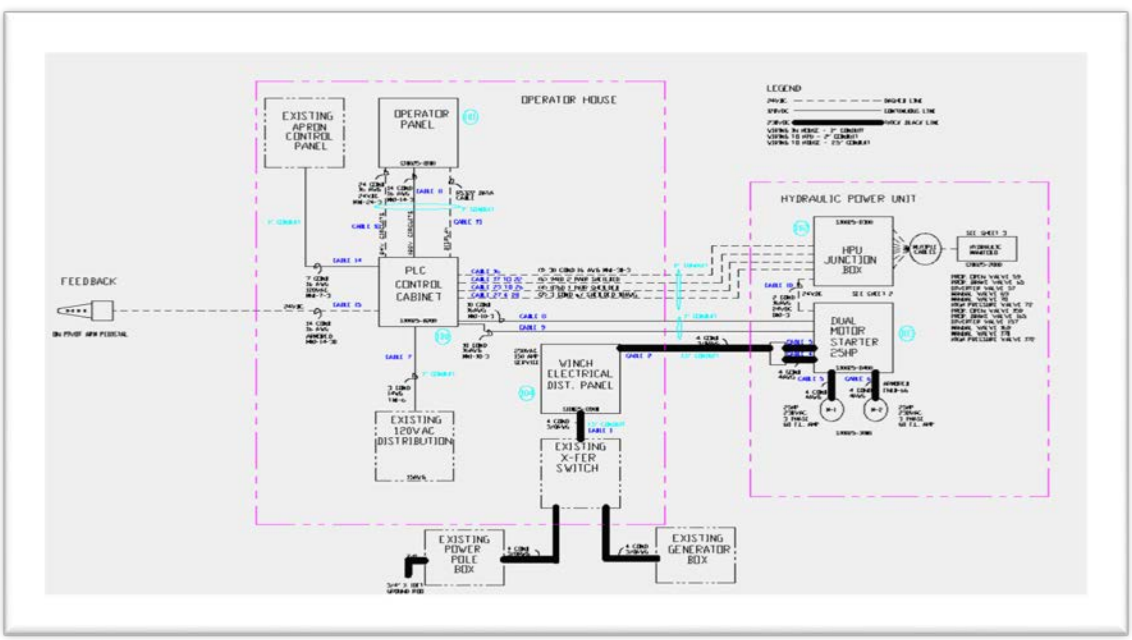
PLC Cabinet

Opening Winch

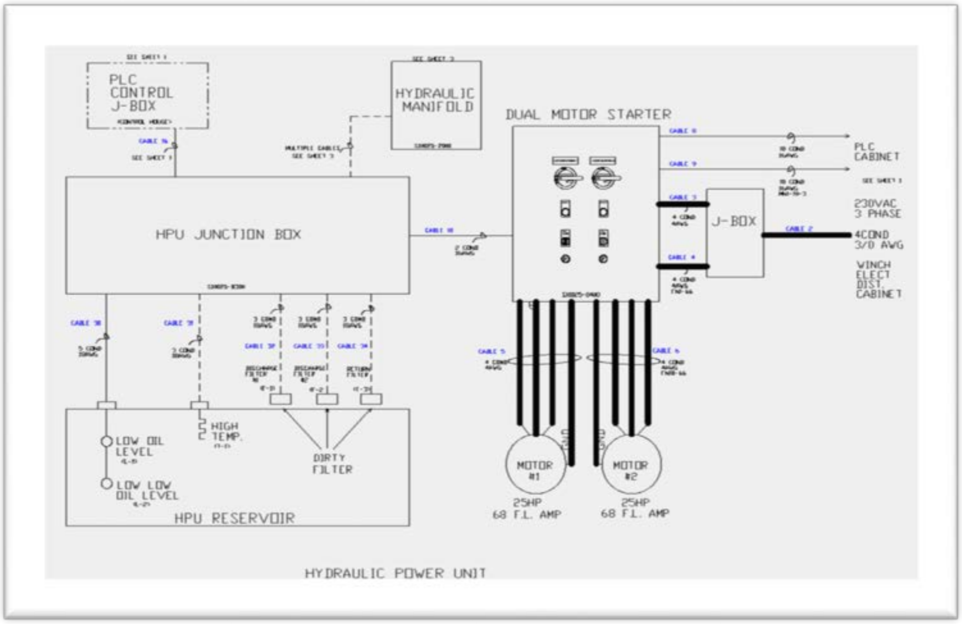
Hydraulic Power Unit

Electrical Power Upgrades and Control System Implementation:

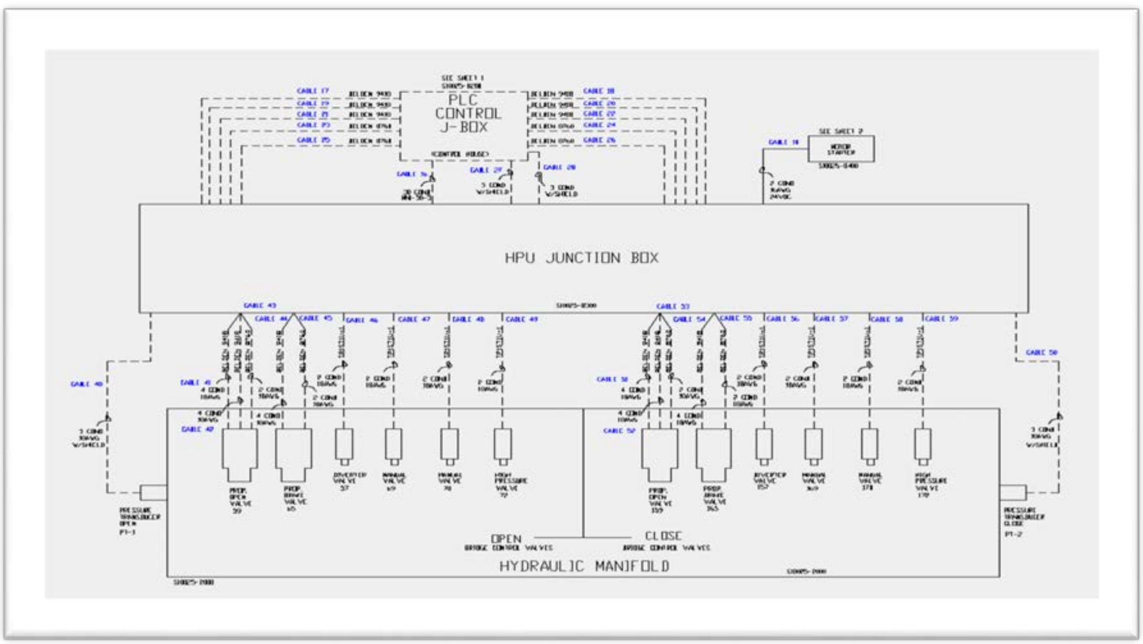
Typically, the bridges where we have conducted these change outs have had flooding issues or the electrical cabling has required replacement from the power supply box to the operator house, due to age or capacity. The generator cabling and transfer switch have not been affected, but a new circuit breaker is always required to handle the HPU power supply. The one line diagrams below show some of the interconnecting cabling.



Electrical Block Diagram



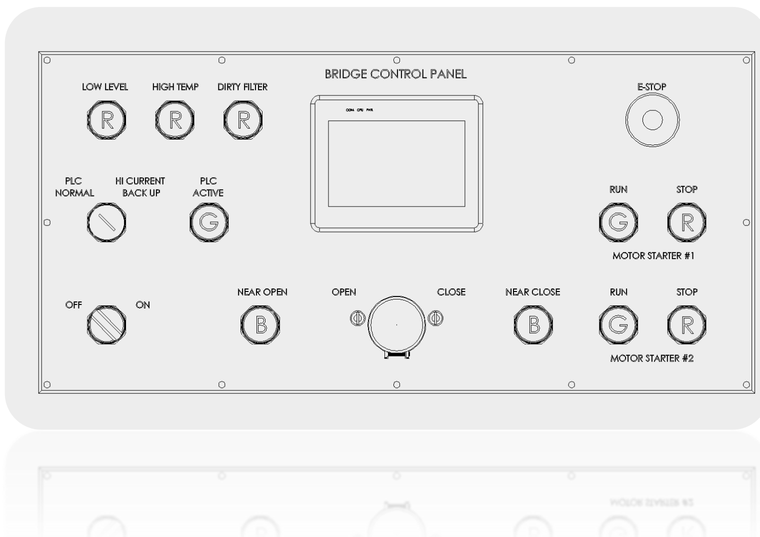
HPU Electrical Connections



HPU Manifold Assembly Electrical Connections

As stated earlier, the operator control console is located in the operator house and is used to control the movement of the bridge. The console also provides information about the system via the touch screen and indicating lamps. The touch screen provides such information as bridge position, bridge speed, braking pressure, system pressure, etc. The indicating lamps provide information such as dirty filter, oil low level, high temperature, and PLC status.

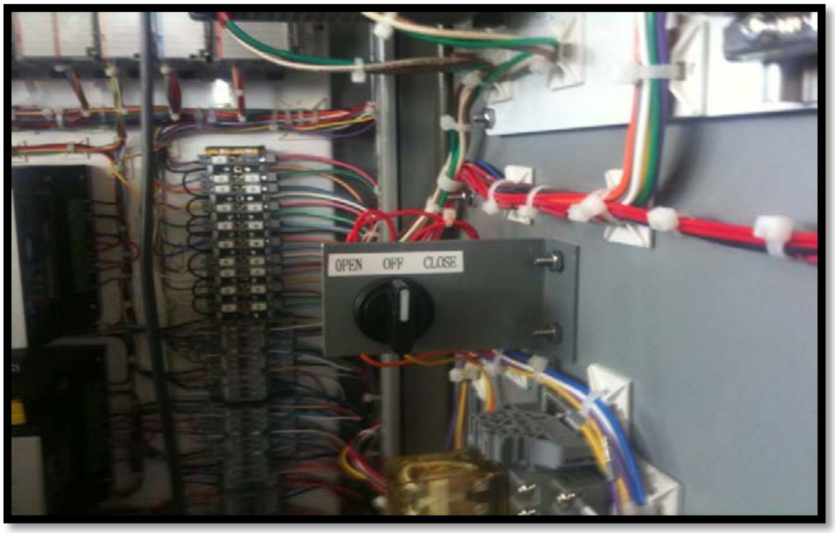
The dirty filter lamp indicates that the filter needs to be changed and the filter is in bypass mode. The low level indicator indicates that the fluid level in the reservoir has dropped below the recommended level and should be refilled and checked for leaks. If the oil level drops to a low enough level, the hydraulic power unit will automatically shut off preventing any damage to the hydraulic pumps. The high temperature lamp indicates that the hydraulic power unit is above 140 degrees. The PLC status lamp indicates if the PLC is active or not. If the key switch is in “HIGH CURRENT BACKUP” mode the lamp should not be illuminated. There are also two motor run and stop lamps. These lamps indicate if the electric motors are running. If the motors are running the green “Run” lamps will be energized and if the electric motors are off then the red “Stop” lamps will be illuminated.



Operator Console

In the rare event that something doesn’t function as planned and the bridge needs to be stopped immediately there is the red E-stop button on the control panel that should be depressed to stop the power unit and engage the brake.

In the event a cable has to be changed, individual winch operation may be required. An override in the PLC junction box can be switched to “OPEN” to operate the winch that pulls the bridge open or “CLOSE” to operate the winch that pulls the bridge closed.

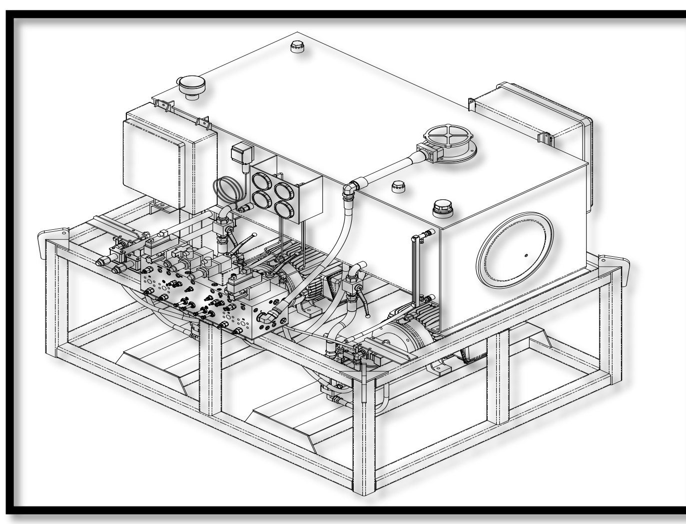


Individual Winch Operation Selector Switch

Once open or close winch is selected, then a winch direction, “In” or “Out”, can be commanded using an override toggle switch to heave in or pay out wire on the selected winch.

Hydraulic Operation

The hydraulic power unit is a 200 hundred gallon dual 25 HP unit with miscellaneous valves, filters, and indicators. Each motor and pump combination can operate either winch at half speed in the event of a failure of one of the units. Each power unit is designed to operate at 3000 PSI and 12 gpm during normal operation. In the event of high current situation where the bridge is not moving, the system can be switched to high current backup mode. In this mode the hydraulic pressure increases to 5000 PSI and flow is reduced accordingly through torque limiting on the hydraulic pumps to get the bridge moving, but at a slower pace.



Hydraulic Power Unit

Once the HPU motor(s) are up to speed, a timer shifts a vent valve to provide pressure to the system and the hydraulic winch brakes are released. At the same time, pressure is sent to each winch to keep the wire rope taut. When the operator signals the HPU to open or close the winch, the pulling winch receives pump pressure and flow, and the drag line winch maintains a certain amount of back pressure as cable is pulled off the drum. Each time the bridge is stopped, the winches return to a constant tension state keeping the wire out of the water.

Conclusion:

Since these modifications have been made, the bridges have operated seamlessly with only minor tuning issues or operator imposed alarms. There have been no power disruptions of the 24 VDC control system, with the exception of difficulties generated from a bad UPS battery during shifting of power. The maintenance crew was previously replacing sheaves or wire rope every six months, and has yet, to our knowledge, needed to replace a length of wire rope or a sheave in any of the modified bridge systems. The operator house has become a safe environment, without cables and moving equipment residing within it. The ease in which the bridge is moved and controlled allows a greater number of personnel to qualify as an operator. SIT is proud that their concepts and systems have resulted in safe, reliable, efficient, and maintainable operations.

SUPREME INTEGRATED TECHNOLOGY, INC.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Technologies for Protecting Critical Electrical and Control Systems on Movable Structures

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Introduction

The technologies behind the control of our movable structures continue to advance and increase in sophistication. While relay logic is a mainstay in the control of these structures, Programmable Logic Controllers (PLCs) and other sensors are increasingly being used to monitor and in some cases control the operation of the bridge. Many of these movable structures are located in the Southeast region of the United States where the power and electronics are exposed to more frequent and higher levels of surges. This paper will examine the different technologies that can be used to protect against surges and transients and how these technologies can be applied to provide a holistic approach in protecting critical electrical and control systems on movable structures.

Need for Surge Protection

Definition of a Surge

The International Electrical Commission (IEC) defines a surge voltage as a "transient voltage wave propagating along a line or a circuit and characterized by rapid increase followed by a slower decrease of the voltage". In simpler terms, a surge event is a rapid rise in current along a circuit that occurs in a very short period of time. Depending on the type of event, the current will typically rise to a maximum value on the order of 8-10 microseconds and dissipate over a period of twenty to several hundreds of microseconds. The two main causes of surge events can be attributed to lightning discharges and switching operations. The third main type is electrostatic discharges but because we are looking at a heavy industrial application, we will focus on lightning and switching transients. The following graph shows the typical discharge waveforms that surge protection devices are tested against. These waveforms are approximate representations of the two main types of surge events:

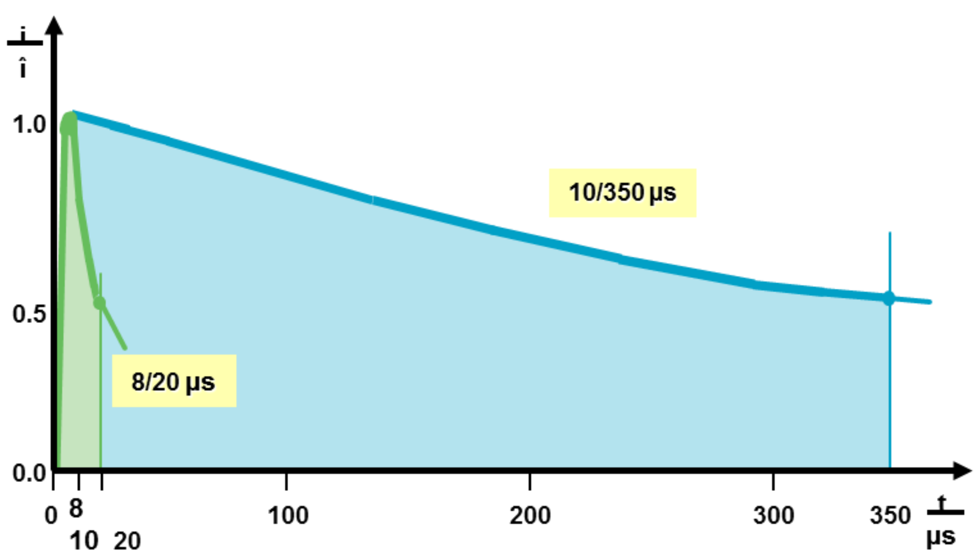
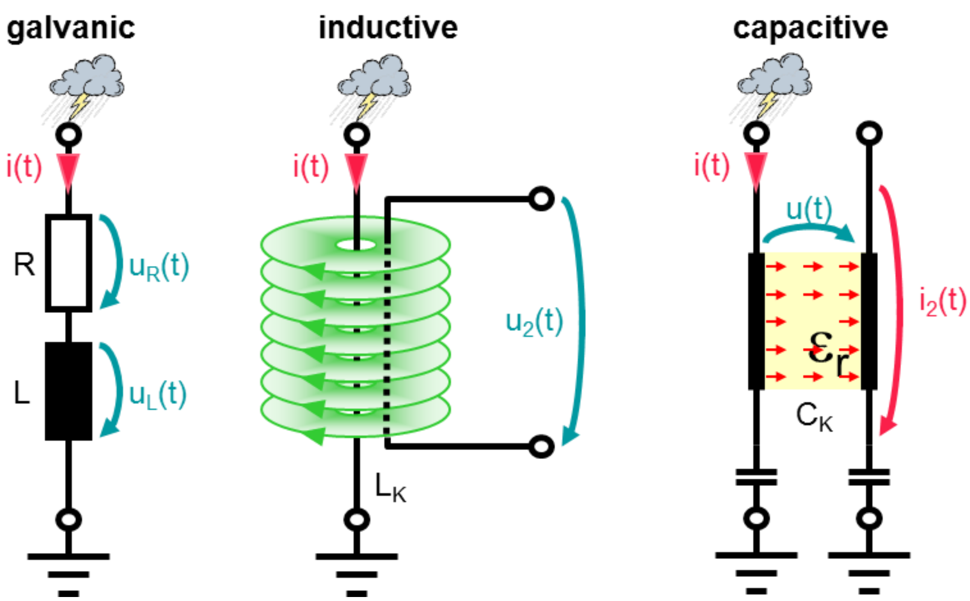


Figure 1: Waveforms used in Testing Surge Protective Devices

The graph represents two different waveforms with the first number representing the time to rise to maximum surge or current (8 or 10 µs) and the decay time to half value of the surge event (20 or 350 µs). The area under the curve represents the amount of energy. The 10/350 µs waveform on the graph attempts to represent the two main currents of lightning strokes, impulses with short duration of less than two milliseconds and long strokes with durations greater than two milliseconds. This waveform is part of the IEC 62305 standard.[1] As can be seen from the graph, lightning results in a tremendous amount of energy that can be very damaging to electrical and control systems. The other type of surge or transient event is a switching event and examples of these include switching in the power grid, starting of large AC/DC motors, and starting of generators. They are more closely approximated on the graph by the 8/20 µs waveform. All of these types of switching events could be possible when one is looking at a movable structure.

The way in which these types of events can affect circuits is multiple. Lightning currents can couple into electrical and electronic systems either directly or galvanically, inductively or through capacitive means. The following diagram provides a pictorial representation of the different types of coupling mechanisms.



Lightning surges seek multiple paths to ground so galvanic coupling is the surge current being directly applied on to the circuit as it seeks to go to ground. Lightning surges create intense magnetic fields and as such can induce voltages across wires that are in the magnetic field. As much as 70 V per meter of cable can be induced on a cable from a lightning strike that is a three-dimensional mile away. These strikes do not have to be cloud-to-ground strikes; induced coupling can occur from cloud-to-cloud lightning. Capacitive coupling is derived from positively and negatively charged ions passing over conductors due to the potential differences between the wires. Capacitive coupling can create significant noise on analog circuits disrupting the signal to a control system.

Switching transients affect circuits through inductive coupling. When equipment is turned on or off or when switching operations are being performed by the local power company, the currents in those lines will increase thereby creating surge voltages that can be induced on nearby cables and wires. These types of transients occur much more frequently and thus need to be considered when developing surge protection scheme.

Need for Surge

There are approximately 845 movable bridges in the United States. Of those 845 movable bridges, close to 300 of them are in Louisiana or Florida, which are areas with high densities of lightning strikes. The following map of the USA shows the average number of lightning ground strikes or flashes per year per square kilometer. From this map, one can see that Florida has certain areas that experience 14 flashes or greater per sq km per year.

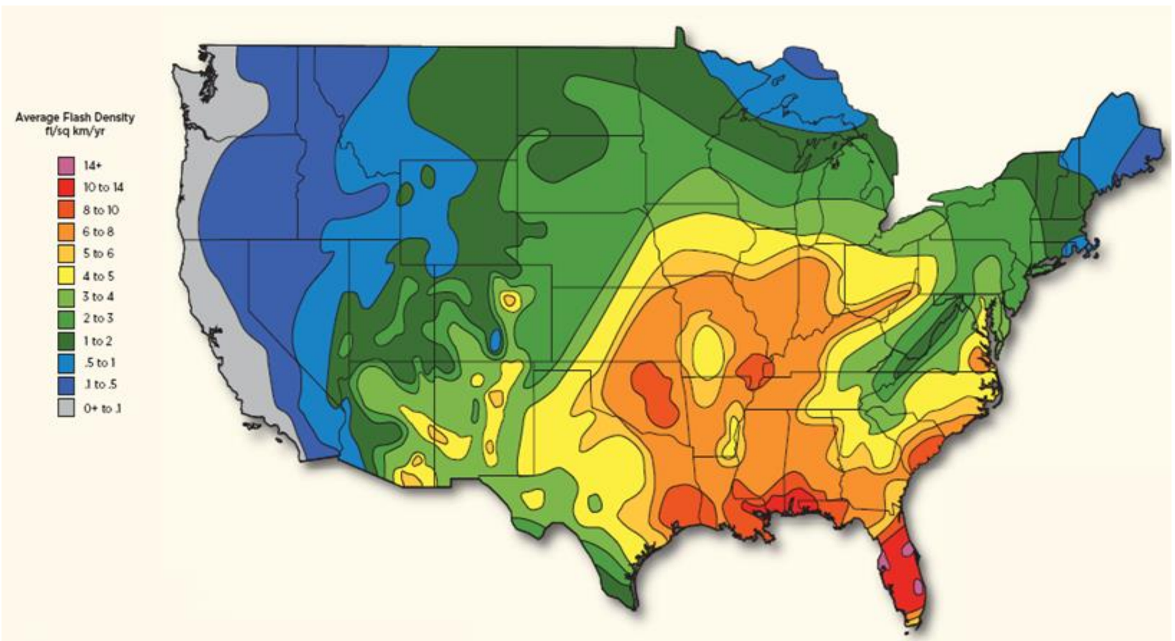


Figure 2: 1997-2007 Average US Lightning Flash Density Map, Insurance Institute for Business and Home Safety, 2012.

However, this graph does not represent cloud to cloud lightning strikes which are the more prevalent types of events. Also, -t is just not lightning that can create problems for electrical and control systems. We also have to be concerned with switching transients and surges that can occur on a daily basis. Eighty percent (80%) of equipment failures can be attributed to switching events although it is harder to correlate because these types of transients are typically not monitored and recorded.

Damage to the electrical and control systems of a movable bridge due to surge events is costly from a replacement and maintenance standpoint but it also render the bridge inoperable until it is repaired which has far more reaching economic impacts, particularly in waterways used for the transport of freight. Thus, it is imperative to consider surge protection as part of a reliable design of any movable bridge.

Surge Protection Technologies

Surge protection devices (SPDs) are designed to divert the higher voltages and currents away from sensitive equipment without interruption of the circuit. There are four main types of surge protection technologies used in SPDs by most manufacturers. These technologies can be split into two categories, voltage switching and voltage limiting. Within voltage switching technologies, there are gas discharge tubes (GDTs) and spark gap technologies. Voltage limiting technologies include metal oxide varistors (MOVs) and suppressor diodes.

Voltage Switching Technologies - General

Voltage switching technologies are more coarse protection elements that are characterized by an ignition voltage at which the device switches on. In the order of nanoseconds following the ignition voltage, the device changes to a low-resistance state and discharges current over a low voltage (10-30 V) known as the burning or arc voltage. One of the things to recognize is that the ignition voltage is not constant and will vary within 20% depending on the rate of rise of the surge voltage. The following graphs depict the characteristic curves of GDTs and spark gap technologies, with U_z representing the ignition voltage and U_B representing the burning voltage.

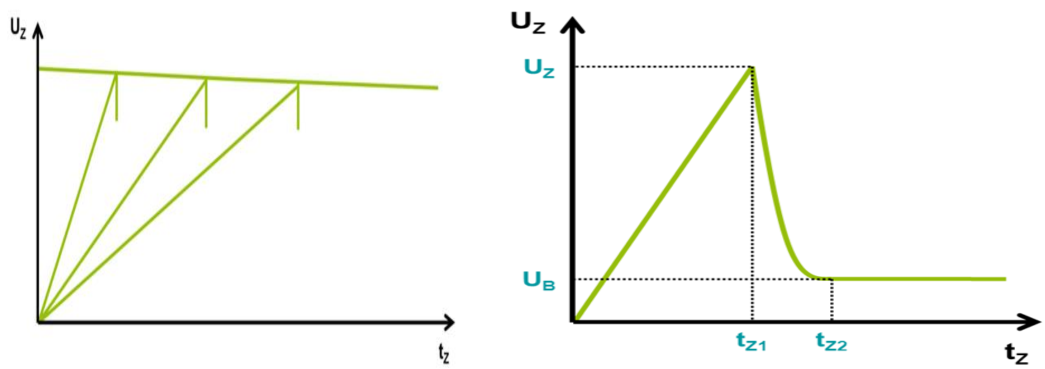
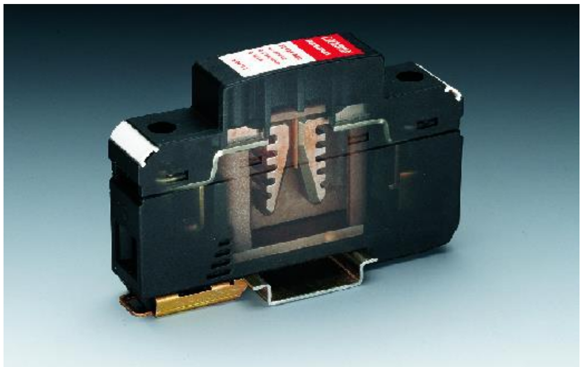


Figure 3: Characteristic Curves of Spark Gaps and Gas Discharge Tubes

Voltage Switching Technologies - Spark Gaps

Traditional spark gap devices have been around for many years and have often been used by electric utilities in high voltage applications. However, some of the older types of devices have not been used on the secondary side of the transformer because of the difficulty in extinguishing the arc and the potential for line follow currents. Thus, the device may “turn on” and actually not “turn off”. The result is a short circuit on the system that will trip upstream circuit protection devices and limit the reliability of the system. To prevent tripping of upstream circuits, properly sized fuses need to be installed upstream of the SPD.

Advances in surge protection have led to the development of arc chopping spark gaps, which have the ability to eliminate line follow currents making them ideal as lightning arresters for power applications. These spark gaps have tremendous energy capabilities of up to 50 kA (10/350 μs) and are used as lightning arresters. The unique feature of these devices is the quenching and baffle plates arranged around the spark horns that help to quench the arc and the associated line follow currents. Because of its capability to quench and dissipate the surge energy, arc chopping spark gaps can take multiple strikes before reaching end of life. The following is a picture of an ARC spark gap with quenching plates and the associated symbol used in circuit diagrams:



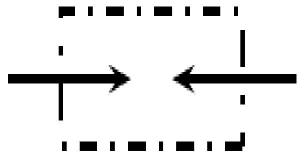
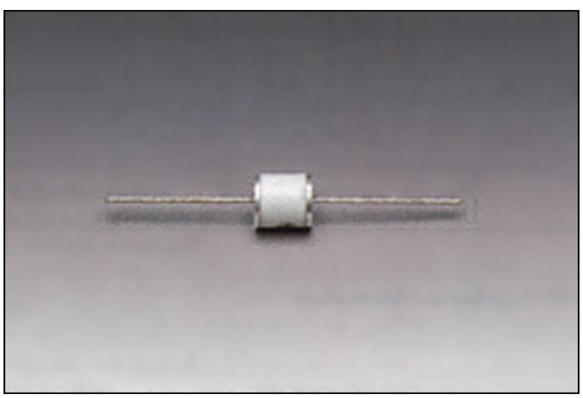


Figure 4: ARC Spark Gap Surge Protection Device and Symbol

Voltage Switching Technologies – Gas Discharge Tubes

The other type of voltage switching technology is gas discharge tubes. GDTs consist of an electrode arrangement in a ceramic or glass tube. Between the electrodes is some type of inert gas such as neon or argon. Once the ignition voltage is reached, an arc voltage between 10 and 30 V typically occurs. . The most commonly used GDTs can discharge transient currents in the range of 10 kA – 100 kA (8/20 µs) and are typically used in conjunction with suppressor diodes to protect low voltage signal circuits. The following is a picture of a typical GDT and the symbol utilized to represent these devices in circuits.



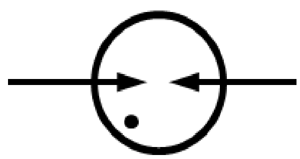


Figure 5: Gas Discharge Tube Surge Protection Device and Symbol

Voltage Limiting Technologies - General

Voltage limiting technologies such as suppressor diodes and metal-oxide varistors (MOVs) are used in both power and signal applications. These devices are voltage dependent and have very specific turn-on voltages with rapid response times in the nano and pico second range. The following shows typical characteristic curves for voltage limiting devices:

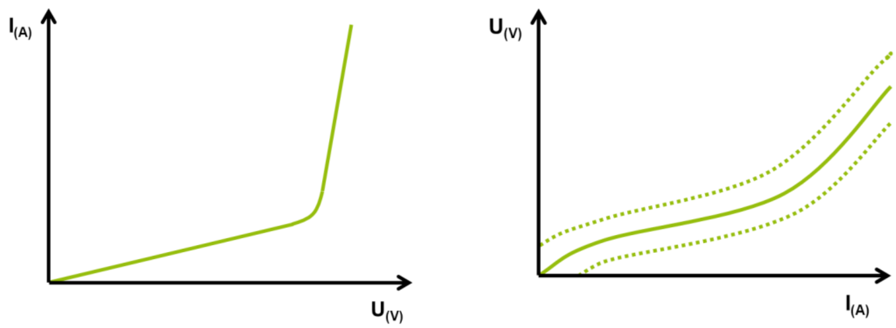


Figure 6: Characteristic Curves of Suppressor Diodes and Metal Oxide Varistors

Voltage Limiting Technologies – MOVs

As their name implies, MOVs consist of a matrix of metal oxides squeezed between two metal plates or electrodes. Depending on the size of the MOVs, they have the capability to handle large surge currents of up to 35 kA for 10/350 waveform. While having the advantage of fast response times and fairly large surge capabilities, MOVs do have some disadvantages and must be properly applied. For one, MOVs degrade over time and will start to draw leakage current over time. The amount of leakage current can be significant as the MOV ages, which can lead to disruptions in analog signal circuits. The high capacitance of the varistors can lead to attenuation of signals in high frequency application so they are not used in data transmission lines with high frequencies. For frequencies up to approximately 30 kHZ, the attenuation is almost insignificant. The following are pictures of small MOVs and the symbol utilized to represent these devices in circuits.

Figure 7: Metal Oxide Varistor Surge Protection Device and Symbol

Voltage Limiting Technologies – Suppressor Diodes

Suppressor diodes or silicon avalanche diodes (SADs) are diodes that are made of silicon, have extremely fast response times and very specific turn-on voltages. SADs are used in conjunction with other surge protection technologies, such as GDTs, in signal circuit protection since they have lower energy handling capabilities. One disadvantage to SADs is similar to varistors in that the capacitance of the device can lead to attenuation of signals in high frequency applications. The following is a picture of a typical SAD and the symbol utilized to represent these devices in circuits.

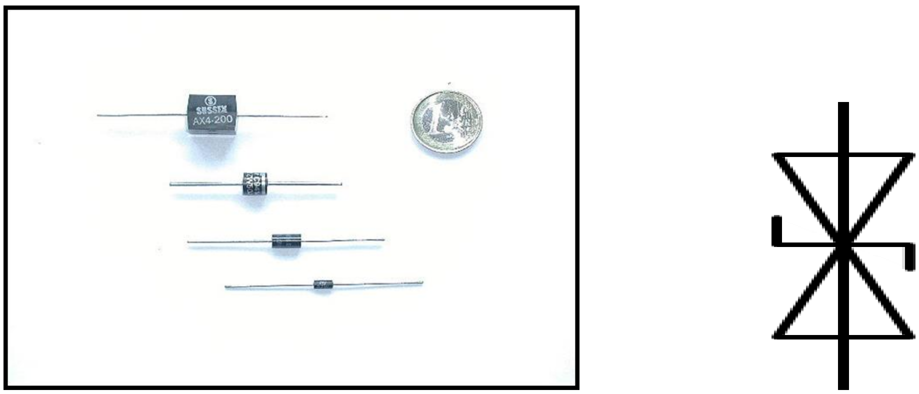


Figure 8: Suppressor Diode Surge Protection Device and Symbol

Classification of Surge Protection Devices

Before we look at how to apply surge protection devices and technologies, it is important to understand how they are classified by two of the most important standards organizations: Underwriter's Laboratory (UL) and the IEC. The UL standards are predominantly followed in North America whereas the IEC standards are more European centric.

In 2009, UL released the 3rd edition of their standard for surge protection titled UL Standard for Safety for Surge Protective Devices, UL 1449[1]. There are a number of significant modifications between UL 1449, 2nd edition to UL 1449, 3rd edition. The nomenclature for referring to surge suppressors was modified from Transient Voltage Surge Suppressor (TVSS) to Surge Protection Devices (SPDs). UL 1449 now applies to devices used to repeatedly limit transient voltages on 50/60 Hz circuits 1000 volts and below. This is an increase in voltage from 2nd Edition, which covered devices 600 volts and below. One of the most important factors that differentiate UL standards from IEC standards is the classification. UL 1449 3rd Edition gives four designations to surge protective devices depending on where in the electrical system the device is connected.

- **Type 1 -** Permanently connected device installed before the service disconnect overcurrent device and intended to be installed with no external overcurrent protective device. This type of SPD most closely relates to devices that were called secondary surge arrestors prior to 3rd Edition.
- **Type 2 -** Permanently connected device installed after the service disconnect overcurrent device. This type of SPD most closely relates to devices that were called transient voltage surge suppressors prior to 3rd Edition.
- **Type 3 -** Point of use SPDs that are installed with a *minimum* of 30 feet of conductor length from the service panel. The 30 feet of conductor length does not include conductors used to attach the

SPD. Some examples of Type 3 SPDs are cord connected, direct plug-in and receptacle type SPDs.

- **Type 4-** Component SPDs and component assemblies.

The IEC product standard is 61643-11[3] and the devices are split into three categories based on their surge capability rather than location within the electrical system. The three categories are as follows:

- **Type 1:** Protection level < 4 kV, Lightning arresters are for the effects caused by direct or close-up strikes designed to protect the installation and equipment at the interfaces for the main incoming power. Type 1 arresters are always recommended if the building has an external lightning protection system.
- **Type 2:** Protection level < 2.5 kV, Surge arresters for the effects caused by remote strikes, inductive or capacitive coupling, and switching surge voltages designed to protect the installation, equipment, and termination devices typically in the sub-distribution level.
- **Type 3:** Protection level < 1.5 kV, Device arresters are designed to protect particularly sensitive termination devices to further reduce the voltage level. These may include devices for permanent installation in distributions or portable protective devices in the socket area directly before the termination device that is to be protected.

Effective Surge Protection Principle

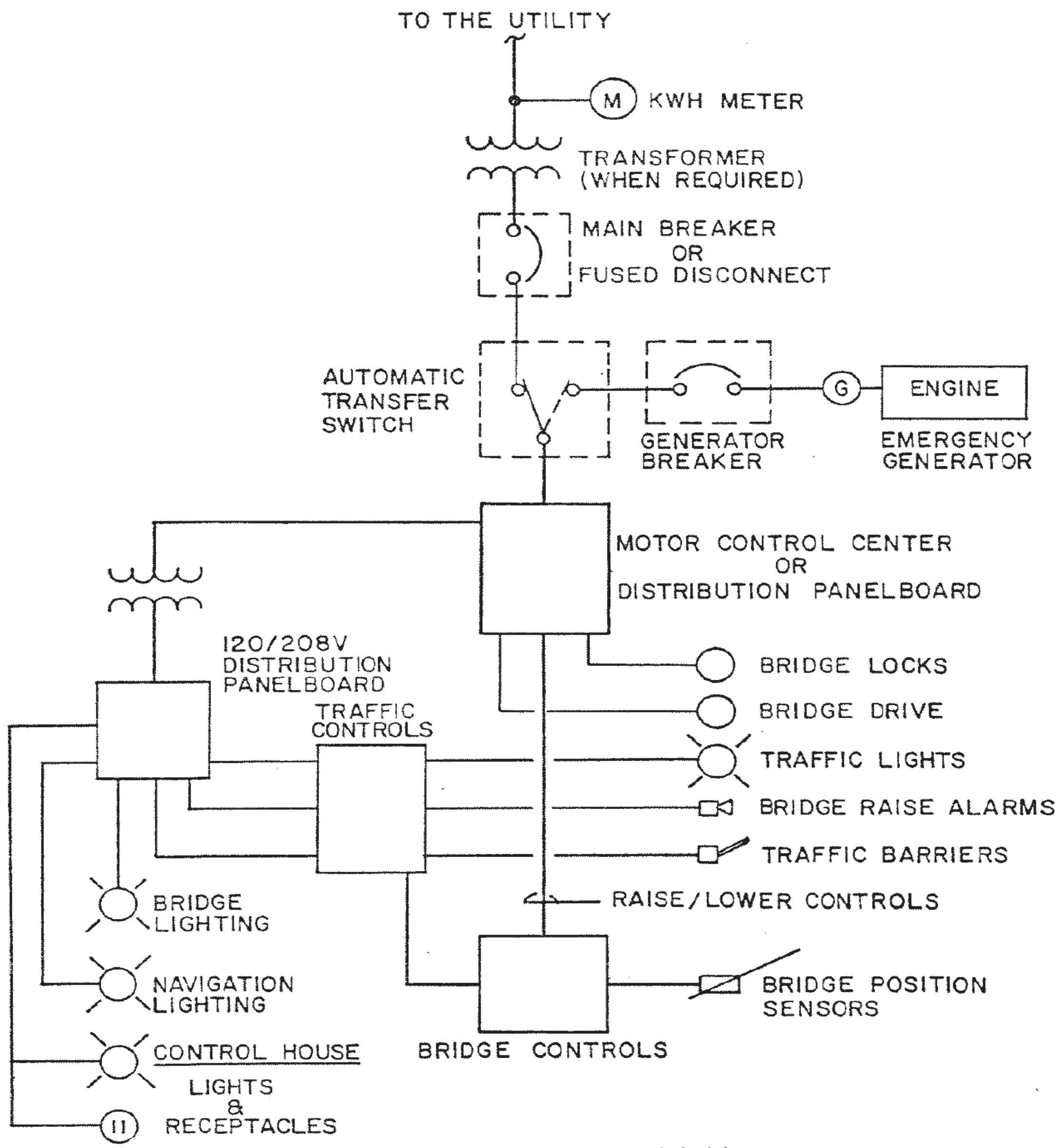
When applying surge protection technologies to any electrical or control system, one should always take a big picture approach and look at the overall facility. If the incoming power is not properly protected, then the downstream surge protection will not be properly coordinated to protect the lower voltage electronics. Similarly, even if the power is properly protected, surges can still enter the system and damage sensitive electronic equipment such as Programmable Logic Controllers (PLCs) and Human-Machine Interface (HMI) devices. The best surge protection scheme encompasses a cascade or step approach that protects all circuits going in and out of the facility including power, control and measurement signals, data/communication lines and transceiver/antenna connections.

In addition to taking a cascade approach in properly protecting a facility, the other key factor is ensuring that the surge protection and equipment within the facility are effectively grounded and bonded. Surge protection works essentially as a switch; when a surge event occurs, the SPDs switch on and divert the additional current to ground. Without effective grounding and bonding, surge protection will not properly protect the facility and the end user will have a false sense of security.

Movable Bridge Example

Movable bridges are an important component of our transportation infrastructure in the United States. Thus, it is important to design our bridges with the ability to withstand many different factors for increased operational availability and uptime. Obviously, availability needs to be balanced against financial constraints but a good surge protection scheme can provide significant increases in uptime at a relatively low cost.

Let’s examine a typical movable bridge electrical schematic. The preferred voltage for powering most movable bridges is 480 VAC, 3 phase 60 Hz power. Most likely a transformer will be needed to step down the voltage from the local power utility. Most movable bridges are also equipped with standby generation system(s) to provide a backup source of power in the event the utility service fails. The following is a typical electrical block diagram for a movable bridge[4]:



BRIDGE ELECTRICAL
BLOCK DIAGRAM

Using the diagram above as a basis, the first point at which surge protection should be installed is after the Automatic Transfer Switch (ATS) on the incoming power to the Motor Control Center (MCC) or Distribution Panelboard. For the purposes of this discussion, we will refer to device by Type # according to IEC standards. The type of device that should be installed downstream of the ATS is a Type 1 Lightning Arrester device. It is recommended that the device be capable of handling a surge of 25 kA for L-N connections and up to 100 kA for N-GND. One important factor to note is that the device should be tested to the typical lightning transient waveform of 10/350 µs to insure a quality product. One option to a Type 1 device is a combination Type 1/Type 2 device that coordinates both lightning and transient protection in a single panel or enclosure. As noted in the discussion about surge technologies for voltage switching devices, it is imperative that properly sized fuses are provided upstream of the SPD and there are manufacturers on the market that build in fusing to the surge protection solutions.

The next point of surge protection is to protect critical components downstream of the motor control center. For critical motors and drives, a Type 2 style surge arrester rated at 40 kA would be appropriate at an 8/20 waveform.

For the 120 VAC distribution panel board, a Type 2 SPD rated at 20 kA for L-N and N-GND and a max discharge current of 40 kA should be installed in front of the panel board to protect the downstream power and components. Depending on the distance between the panel board and the control panels, a Type 2 or Type 3 SPD may be warranted for the incoming power to the control panel. A rough rule of thumb is that for distances less than ten feet, a SPD should not be necessary. Over ten feet, one needs to consider the distance and the potential for induced surges on the wires and design a Type 2 or Type 3 SPD for protecting the power.

For signals entering the control panel, SPDs should be installed to protect the PLC and other electronics within the control enclosure. Depending on the application, there are SPDs designed for 24 VDC analog circuits and 24 VDC and 120 VAC discrete circuits that can handle total surge currents of 5 kA to 20 kA depending on the voltage.

Operation and Maintenance of SPDs

While surge protective devices do protect electrical and control system components from failure resulting in reduced maintenance costs for these components, the SPDs will eventually reach an end of life and require maintenance and replacement particularly in areas prone to high levels of surges. Thus, it is important that ease of maintenance of these devices is a feature set when selecting SPDs to not only reduce replacement time costs but to insure increased system availability. There are a number of manufacturers on the market that have maintenance friendly devices. When considering SPDs, we recommend a number of features be included as part of the specification to insure ease of maintenance:

- Pluggable – many manufacturers have two piece SPDs in which the actual surge components are housed in a pluggable element that inserts in a base element. The wires to the SPD are terminated on the base element. When the device reaches end of life, the pluggable element is hot swappable and can be easily removed with standard tools and replaced with a new plug. Unless there is significant damage to the base element, many hours of labor can be saved from disconnecting and re-terminating wires on the SPD. However, a properly coordinated surge protection strategy should eliminate significant damages to base elements. Pluggable SPDs come in all IEC styles from Type 1 Lightning Arresters to Type 3 Device Protection as well as signal circuit SPDs.
- Remote Indication – to reduce routine maintenance hours, many SPDs come with a remote indication dry contact that can be wired to a PLC or indicating light to note when the device has reached its end of life. Remote indication can be found on both power (Type 1 -3) and signal circuit SPDs.

  There are other options for large banks of signal circuit SPDs protecting multiple analog and discrete input/output points. One option is to jumper the contact to common the alarm to the control system. Another option that is specific to one manufacturer is a head end controller that monitors the status of each SPD through a T-bus connector along the DIN rail. The head end controller has the same form factor as the SPDs and provides two contacts, one for performance level nearly reached and one for end of life. The controller requires 24 VDC power and can be used for analog, 24 VDC discrete and certain communication signals.

- Visual Indication – if a routine spot check of panels and electrical components is a part of the maintenance program, visual indication of end of life either through a LED light or red-flag is useful for quickly determining which SPDs require replacement. It is recommended that the SPD source the power for the LED indications separate from the circuit that it is protecting to avoid any possible interference with the circuit.

- Testable – many manufacturers have the ability to test their SPDs with some type of proprietary testing device. This can be useful as part of a routine maintenance check on an annual or semi-annual basis. One manufacturer utilizes a testing device that allows you to determine if the SPD has certain elements that have been compromised or reached its end of life. The tests can be stored in memory or printed to keep an official record. One key point is to insure that the SPD is hot swappable and can be tested without affecting the circuit or signal.

Summary

In summary, movable bridges are a very important part of our nation's infrastructure. To protect these structures and to expedite operation of the bridges, they are increasingly being monitored and controlled through sensors and analyzers that are connected to PLCs. To maximize availability of movable bridges and to protect the power and sensitive electronic equipment used to monitor and control the bridge, it is crucial to employ an effective and encompassing surge protection scheme.

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“Touchscreen Technologies for Heavy Civil Engineering”
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Electro Hydraulic Machinery Co.

NEW ORLEANS FRENCH QUARTER MARIOT HOTEL
NEW ORLEANS, LOUISIANA

Since the dawn of the industrial revolution, engineers have been seeking innovative ways to control and monitor the lifting and drive of heavy movable structures. The need is compelled by the fact that one person, an operator, can move a huge structure with a push of a finger. It’s important for an operator to monitor the progress of this move. It’s also critical to the structures’ owner that this move be completed safely. Damages from powerful forces which can react on the structure could be catastrophic. No matter what type of heavy moving structure you look at, the basic needs are alike. You must initiate a signal for movement, get some sort of feedback the structure is in the position you want it to be, and provide controlling interlocks in the background to insure safety. This paper examines this control evolution from the early days where pushbuttons and lights accomplished these goals, to a new wave of modern day touchscreens that is about to explode on the field of civil engineering.

The first of these three steps is for the operator to initiate the operation. With a push of his finger an operator turns a very light weight signal into progressively larger forms of energy to eventually move a structure to a new position. For the past 100 years various forms of pushbuttons or “pilot devices” have been used for this purpose.

The second step in the process is feedback. The operator needs a visual cue of the structures’ exact position. Indicator lights provide yes or no position feedback to the operator but oftentimes a variable feedback to recognize exact position is required. In the past this has been done with combinations of mechanical and electrical devices.

The final step is controlling interlocks which run in the background. To insure the safety of operation, a system has to be in place to make it impossible for an operator to initiate a signal until the proper safety sequence has been met. Devices to provide these safety interlocks have also been combinations of mechanical and electrical devices.

A great example of old and new technology for these control systems in heavy movable structures can be seen at the Panama Canal where huge steel doors at navigational locks are opened and closed to allow movement of large ships over land. From the comfort of a climate controlled room, a single operator can safely raise and lower these enormous vessels by opening steel doors to allow ships to enter the lock, control water level accurately to raise or lower each vessel in the lock, and finally to open another set of steel doors for the vessels’ release. While this sounds very simplistic, it’s crucial the operator cannot make a mistake in sequence and open the release gates before the vessel is at a safe level to enter or exit the lock. The results could be catastrophic. Safeguards to protect these investments must run automatically in the background so there is no possibility of operator error.

At the Miraflores Lock in Panama, a museum of turn of the century emerging electrical control technology that operated the locks for many years can be seen can be seen right alongside today’s state of the art computer technology. From the early beginnings of industrial electrical controls in 1913, robust switching levers moved mechanical fingers to a series of electrically charged buss bars to transmit a signal. Interlocks were designed into the lever arms such that power could not be applied out of sequence. Visual feedback came from equally ingenious mechanical/electrical devices that had rarely been seen in the newly emerging industrial world of electrical controls (Figure 1 & 2).

Figure 1. Miraflores Locks and Museum Panama Canal

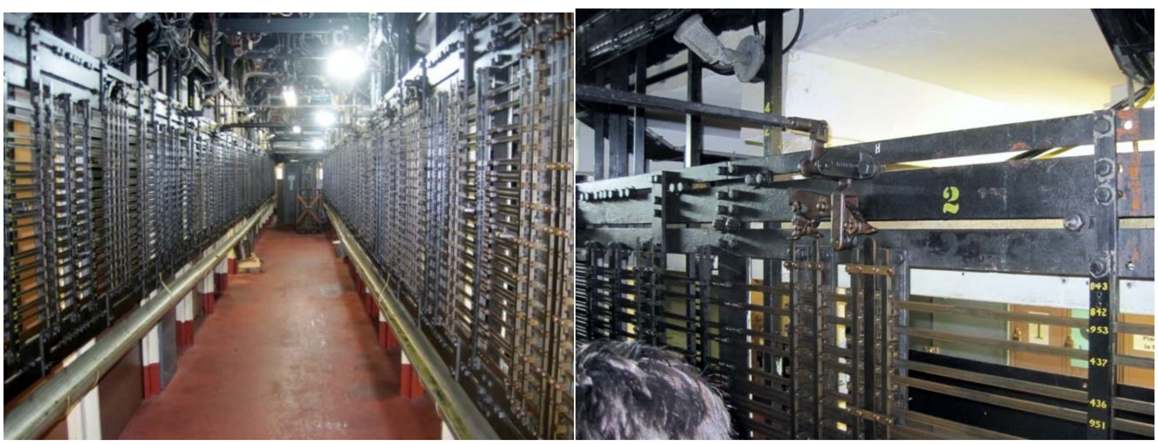

Figure 2. Buss bar gallery underneath operator station

Present day controls (Figure 3) at the Miraflores Lock have a single operator moving many vessels through multiple locks from a comfortable chair surrounded by computer monitors. With a keyboard and the click of a mouse, he can control operations many miles away. He gets feedback from the monitor screens validating all conditions and safety interlocks are provided by Programmable Logic Controllers (PLC’s).

Figure 3. Modern State of the Art Control Technology

From these early developments in electrical controls technology came pushbuttons and lights which have dominated the industry for the last 100 years. While still widely used in today’s movable structure controls, pushbutton and lights are being integrated more and more with various forms of visual information screens (Figure 4). Everything from LED digital displays to complete touchscreen only operation is taking place.

Figure 4. Combinations of lights, analog meters, digital displays and touchscreens

While the stage may have been set for this next wave in the industrial controls evolution, there have been several inherent problems that have only been resolved in the last couple of years. One of the biggest issues was the need to touch two places on the screen at once. It doesn’t sound like much but many operations need to occur independently for safety reasons yet simultaneously for cycle time reasons. Just like clicking with the mouse on your computer, when you select a point on a touch screen with your finger, you are selecting a precise point on an X/Y grid. Up until recently touchscreen technology has only been able to distinguish one grid location at a time.

Touchscreens have used a variety of techniques to detect the placement of a finger on the screen ranging from mechanical, optical, and electrical sensing. Perhaps the simplest form of touchscreen is a resistive screen which includes two layers of electrically conductive material. Layers are separated by transparent insulating dots that keep them apart; you can see these dots as faint traces on some touchscreens. When a person presses on the screen these two layers touch and a circuit is completed. The position of the touch along the sensor changes the voltage being sent through the circuit which the screen's controller uses to measure the coordinates of the touch and feed this information to the running software. Conversion is typically handled using an analogue-to-digital converter that with typical 10-bit resolution can differentiate up to 1024 different locations across each axis.

Because they use a physical process resistive touchscreens work even when users are wearing gloves making them preferable for outdoors or industrial environments. They're also highly durable which makes them common in point-of-sale terminals such as those used continuously by restaurant staff.

Today's capacitive electrical touchscreens have proven to be the most versatile and efficient way to sense human touch. A capacitor is an electrical circuit that, in its simplest form, is composed of two conductive electrodes separated by an insulating gap. A direct current (DC) of electricity can't straddle this gap, but an alternating current (AC) can induce a charge to flow from one side to the other. The surface of a touchscreen is blanketed with a grid of electrodes. Wherever our finger comes to rest, a capacitive contact is formed and the AC current generated within the device induces a corresponding current within our body which helps span the gap and complete the circuit.

Capacitive screens work using a sheet of glass that's coated on the inside with horizontal and vertical electrodes made of a conductive material. Because your body also has an electrostatic field its presence affects the capacitive potential of the conductive material layers when you touch the insulating glass. This change is registered as a touch by the screen's controller which determines the location of the touch along each axis and relays it to the controlling software.

This design makes capacitive screens more sensitive since you don't have to physically push down on the screen to make it work; it also enables novel applications such as the touchscreen property viewers now find on cell phones and small tablet computers. The ability to use multiple fingers, swipe, squeeze, slide, and rotate are all new features that are making their way into industrial touchscreens. One downside of this capacitive design is that you can't use them with gloves on or with any other implement; a stylus can be used as long as it carries an electrostatic charge.

The AC currents in touchscreens are within levels for natural charge conduction in our bodies but the true revolution and utility of modern touchscreens lies in the rapidity of their responses. Behind every electrode on a touchscreen grid lies an embedded microcontroller that has a clock speed of nanoseconds. It is this fast response time that enables modern smartphones to have such smooth interaction with human touch, and it is this progress that has driven the growing appeal of touchscreens in recent years.

Another touchscreen technology worth mentioning but so far not widely used is optical imaging which suits large-screen devices like the Surface Table PC (Figure 5). Optical imaging uses infrared LEDs aimed at the screen from its inside; when you touch the surface the infrared light bounces back and is picked up by cameras positioned around the edges of the screen. This approach supports many fingers at once and also supports the use of image acquisition and barcode reading using the same cameras.

Figure 5. Surface Table PC Technology

The next big obstacle for touchscreens in movable structure applications was the software interface. Most processes use Programmable Logic Controllers with very different operating systems than laptops and PC's. Only until recently it was difficult to interface a PLC with any other manufacturer's touchscreen except the manufacturer of the PLC. To add to the complexity, touchscreens required different software than that of the main PLC even though they were from the same manufacturer. Integration of the two software's to work in harmony was cumbersome at best.

Even if pushbuttons and lights are performing the bulk of actual structure moving operations, one big advantage of having a screen available is the ability to display enormous amounts of information about the health of various systems affecting the structure. Perhaps even more importantly to monitor operations that prevent accidents or failures. The ability of the screen to display this available data in so many different and changeable ways offers flexibility that opens up many new possibilities for interested maintenance departments.

Owners want to integrate this newly available data with Windows based operating systems using their desktop PC's, laptops, tablets, and phones. Older touchscreen technologies need expensive high end software to provide the interface. Especially if there was a need to store and transmit large amounts of data over a network.

Today's operator interface applications range from basic monitor and control to high-end, feature-rich HMI software with Supervisory Control and Data Acquisition (SCADA). To boost communications capability with any network, PLC, Web client, or database you can use a low cost web-enabled Visual Designer operator interface software. Open architecture software can host third party controls such as ActiveX, .NET, ODBC, and Visual Basic programs.

These new software advances make it possible to have what in Windows terminology is called "Extended Desktop" where you run multiple files from different processors on the same screen. You can overlap them or just tile them together just like you would on the desktop of your PC. You could be looking at local weather radar as well as your CCTV screens during an operational cycle.

All this new data capability can be viewed remotely through your desktop without having to install any software on your PC. Access to the system from anywhere using a standard Web browser is all that's required. Which begs the question: "What about security?" If you chose to make a remote connection available to the outside world via the internet you will need the same precautions that you use on any network today. However, it is possible to have a local connection via Bluetooth or controlled Wi-Fi that does not access the outside world. Many maintenance departments are taking advantage of these new capabilities.

At the Flagler Bridge in Palm Beach, Florida they will be monitoring 12 gearboxes with various predictive maintenance sensors looking at vibration, temperature, and oil quality. Data logging this information along with brake torque, changes in motor torque, speed, and weather provides a wealth of interactive data that is continuously monitored. Being able to detect and react to changes in historical data is changing the face of maintenance and operational capability.

COST CONSIDERATIONS

Just like all of today's computer equipment, prices for hardware and software gets more affordable every day. It may be possible to build a new control system with a touchscreen for less than the cost of a conventional pushbutton, light, and relay system but except for a few temporary control systems, most owners are not making that switch to touchscreens. The applications in movable structures will mostly be afforded by system designs that could reduce operating and maintenance costs.

CONCLUSION

Almost every new project is making use of touchscreen technology in some way. With all the new features that let you swipe, zoom, scrunch, and tap like your cell phone, touchscreens will be exploding on the scene with all sorts of new apps to interface with other devices you already own. Maintenance departments and owners will be demanding more information and of course the ability to do more with less people. It's no doubt these new touchscreens will play an increasing role in that effort.

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SKF

HEAVY MOVABLE STRUCTURES

FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

A DFSS (Design For Six Sigma) approach for increasing reliability in the bearing system of the world's largest Observation Wheel.

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Bridges & Large Movable Structures

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Abstract

A robust and reliable bearing system is a critical component for the reliable operation of heavy movable structures. This paper will demonstrate how Design for Six Sigma (DFSS) based simulation tools and methods lead to improved reliability of the bearing system. In particular, this paper will show how DFSS methods can help manage risk when designing complex and expensive traditional movable structures such as movable bridges, and unique structures such as movable roofs and observation wheels.

This methodology allows for the detailed evaluation of different design variations and their effects on bearing fatigue life, bearing loading and structural deflections. This approach also shows how to study combinations of design parameters that can increase or decrease the reliability of the bearing system.

The simulation analysis of an observation wheel bearing system is used to show the influence of the housing flexibility and the non-linear bearing stiffness on the system performance. The load distribution and deformation of housing components and bearings is also analyzed.

The flexibility and accurate stiffness modeling led to great insights on the overall system interactions and performance. The simulation results, combined with DFSS techniques, led to an improved bearing design and higher predicted reliability.

Project Members

American Bridge Company	Construction & Project Management	Coraopolis, PA
ARUP	Observation Wheel Designer	San Francisco, CA
SKF	Bearings, Seals, Lubrication	Lansdale, PA
Caesars Entertainment Corp	Owner & Developer of the Linq Complex	Las Vegas, NV

Project Background

American Bridge requested that SKF participate in the selection, manufacture, and assembly of the spindle bearing system for the proposed world's tallest observation wheel. The design was analyzed utilizing SKF proprietary simulation tools and Design for Six Sigma methodology. The primary objective of the project was to evaluate the complex interaction of all the components in the system and to identify key performance indicators. Particular attention was dedicated to the evaluation of the effects of loadings and deformations to the bearing performance in terms of forces and motion.

ARUP, the designer of the observation wheel, identified the following challenges for the application:

- High loads
- Housing deformations
- Varying alignment of the spindle
- Bearing loading during installation

Project scope

The project scope included analyzing the performance of a large custom spherical roller bearing, SKF BS2-8067, using SKF proprietary simulation tools. The SKF advanced simulation tools have been specifically developed for the investigation of rolling bearing applications. Through the simulation models it was possible to evaluate the many factors that influence the system behavior. Some of these factors are the clearance in the assembly, varying misalignment, flexibility of the supporting structure and different boundary conditions.

The main performance characteristics calculated during this project were summarized to the following points:

- System interaction, including the effect of housing deformations.
- Contact load distribution and contact pressures on the bearing raceways.

SKF Simulator (Orpheus)

The software package, SKF Simulator (Orpheus), was developed for the investigation of rolling bearing applications as a total integrated system. It is capable of analyzing static and modal behavior of the application. A SKF Simulator model is built by connecting all types of machine components, such as bearings, shafts, gears and casings. An arbitrary combination of forces, prescribed displacements, and rotational velocities can be used to define the loads on the components. The components themselves may be special (nonlinear) elements, defined within SKF Simulator (i.e. the rolling bearings) as well as arbitrary elements like shafts and housings. The latter must have a linear behavior and their stiffness and damping properties are obtained by means of the finite element method. Special reduction methods are applied (on the original finite element models) to reduce the number of degrees of freedom and thus to reduce the calculation time for analyses. (The calculation time is proportional to the number of degrees of freedom to the power of three.)

Bearing Fatigue Life

Two bearing life calculations were used for the analysis. The first was the DIN ISO 281. The second was the SKF Advanced L_{10} Fatigue Life (AFC) Calculation method.

The bearing calculated fatigue life values referenced to in this paper are the $L_{2.53}$ and the L_{10} raceway fatigue life. L_{10} bearing life is defined as 90% of the bearings exceeding the calculated value with 10% failure probability. Fatigue life with a lower failure probability can be calculated using a Weibull distribution with a shape factor equal to 1.5 and a form factor chosen appropriately. Alternatively correction factors are available in the ISO standard and in the SKF General Catalog in table 1 on page 53. The $L_{2.53}$ was chosen to increase reliability of the system.

All life calculations for this project were calculated using the SKF advanced simulation program, SKF Simulator.

For modern high quality bearings the nominal or basic rating life can deviate significantly from the actual service life in a given application. Service life in a particular application depends on a variety of influencing factors including lubrication, the degree of contamination, misalignment, proper installation, environmental conditions, structural movements, vibrations while the bearing is stationary, and electrical current discharges.

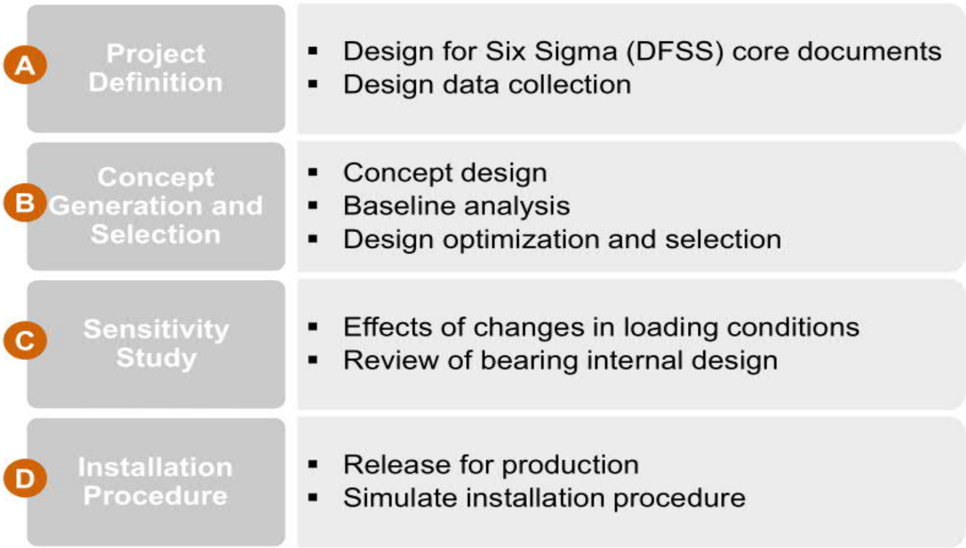
The AFC method is the SKF standard and the conclusions described in this paper will be based on this life calculation method.

SKF advanced fatigue calculation (AFC) life

The SKF Advanced Fatigue Calculation (AFC) life method takes the full integration of rolling element contact stress across the roller length into consideration and evaluates the total number of stress cycles until life in the entire loaded volume is consumed. SKF AFC life also incorporates the condition of the lubricant, taking into account operating film thickness and contamination for each contact individually .

Project Phases

The project was divided into four phases as listed below. The project definition was first defined and DFSS documents clearly identified. The data used for the analysis was provided by American Bridge, ARUP, and SKF. SKF then moved into concept generation and selection. Initial optimization runs identified the possibility of life improvements by varifying roller profile and shaft/hub stiffness. The optimal design was selected and then simulations of the various loading conditions were evaluate. Service life requirements were exceeded under all conditions. Finally, the results of the simulations were utilized to determine the procedures and equipment that were required for the installation.



DFSS Core documents

A project charter was developed to define the goals, requirements, scope, and collect input data. The interactions and all key interfaces were defined in a Boundary Diagram. A P-diagram was used to link system inputs and outputs as well as to design controls and noise parameters. Potential failure modes and causes, and the risk management plan were identified in the Cause and Effect Diagram, and FMEA (Failure Modes and Effects Analysis).

Project Charter	▪ Define goals and design requirements ▪ Define project scope and identify stakeholders ▪ Collect input data
Boundary Diagram	▪ Identify and map all interfaces and interactions
P-diagram	▪ Link system input and outputs to design controls and noise parameters
Cause and Effect Diagram	▪ Identify potential failure modes and failure causes
FMEA	▪ Failure Mode and Effects Analysis (FMEA) ▪ Drive the development of engineering activities ▪ Develop a complete risk management plan

Bearing System P-Diagram

The P-diagram displays the parameters that influence the interaction between the inputs and the outputs of the system. The objective of this diagram is to identify all the design parameters and the noise factors that can be of importance for the desired output. The design parameters are those parameters that can be used to control the desired output. The noise parameters are uncontrolled parameters (parameters with variation) that can disturb the desired output.

P Diagram

Control parameters:

- Cable tensioning
- Clearance
- Number of Passengers
- Lubrication
- Bearing internal design.
- Speed

Inputs:

- Loads (Cable, wind, passengers)
- Contamination (dust, water)
- Heat

Spherical roller bearing + housing

Desired Outputs:
- Fatigue life (1-a)
- Wheel rotation (2)
- Accommodate heat growth (3)

Undesired Outputs:
- Friction (3)
- Vibration (5)

Noise factors:

-Piece-to-piece variation
-Degradation over time
-Customer usage
-Environment
-System interaction

Piece-to-piece variation

- High misalignment at operating conditions.
- High misalignment at mounting.
- Low clearance after mounting.
- High clearance after mounting.
- Bearing preload after mounting.
- Variation of geometrical tolerances
- Out of roundness (brg + seal seat)
- Alignment
- Cable tensioning

Degradation over time

Bearing wear
- Cage wear
- Seal wear
- PTFE degradation/wear
- Roller NoWear coating wear
- Bearing fatigue
- Fretting corrosion

Environment

- Temperature variations.
- Contamination (Dust).
- Wind conditions.
- Vibrations/earthquakes.
- Rain/water.
- Load variation due to weather conditions (unbalance due ice, rain).

Customer usage

- Uneven load distribution
- Higher speed
- Higher passenger load
- Irregular maintenance
- Insufficient lubrication
- Poor lubrication cleanliness
- Installation damage
- Storage damage

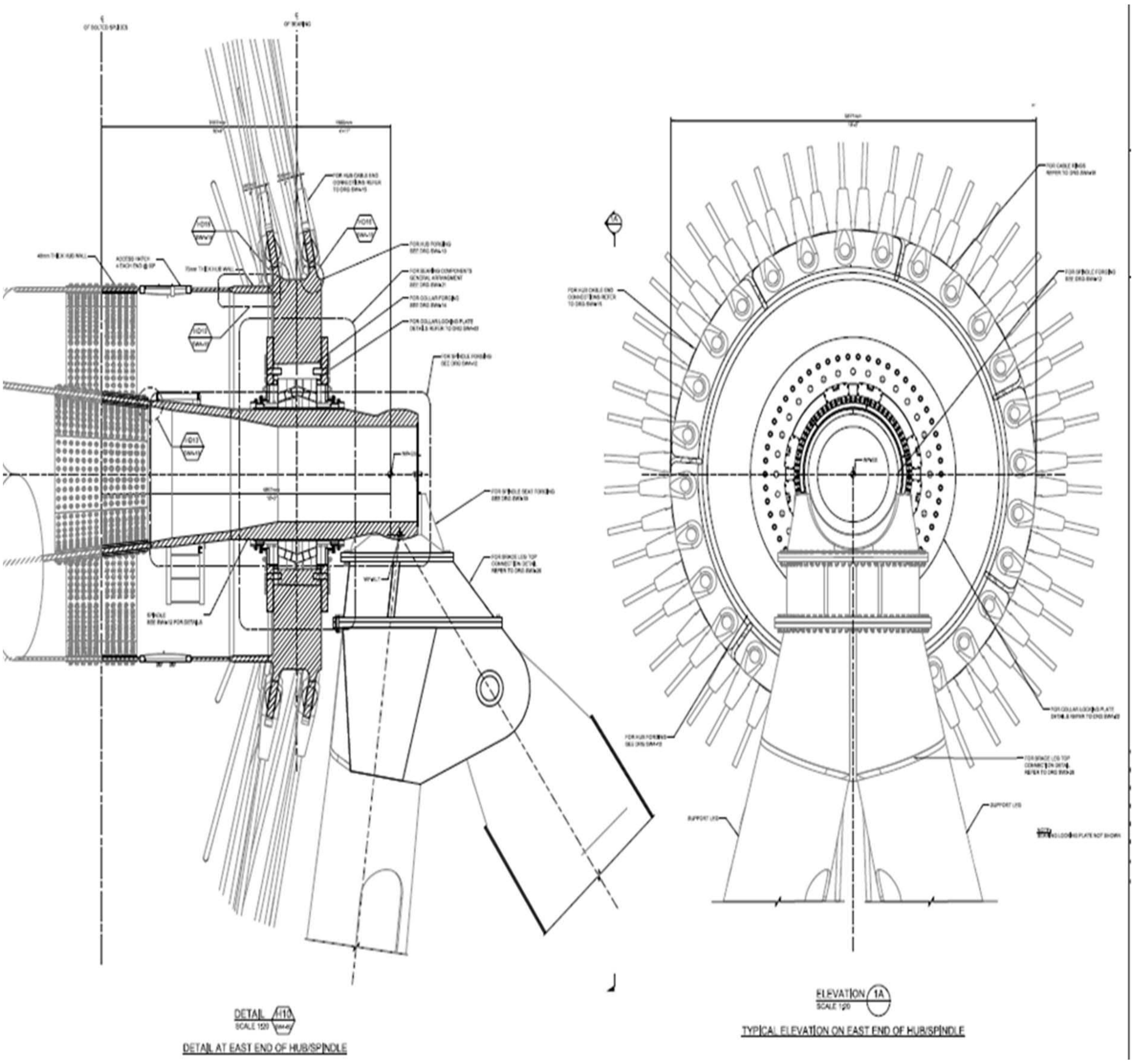
System interaction

- Loading variation due to cable tensioning differences
- Loads and misalignment generated by movements of the supporting legs.
- Electrical damage

Failure modes and effects analysis

The Failure Mode and Effects Analysis (FMEA) was used to help the design team identify the potential causes of failure based on previous related experiences. The FMEA served as a reference in selecting design features, simulation activities, and recommendations. The FMEA was developed by a team consisting of ARUP, American Bridge, and SKF. Due to the confidentiality of the content, the specific FMEA documentation has not been included.

Design



Simulations to test sensitivity

Four primary service cases were analyzed along with eight secondary variables. Service cases SC1 thru SC4 examined various wind loading and seismic events. There were also simulations that reviewed various bearing roller profiles, misalignment, and lubrication factors.

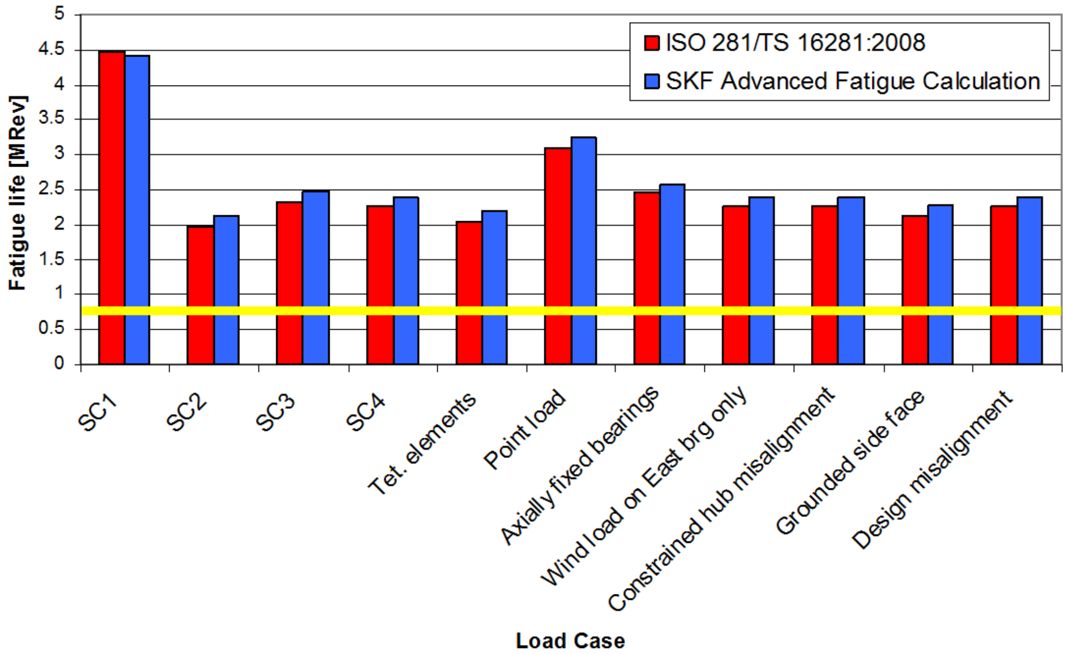


Table 1

The simulation results found that the bearing would exceed the required $L_{2.53}$ Fatigue Life of 657,000 cycles in all cases. Wind loading had a dramatic effect on calculated life of the system. Winds, and the resulting axial loads, reduced calculated life in several simulations by over 50%. Axial loads in a double row spherical roller bearing cause one row to "unload" and carry less of the load. The adjoining row is forced to carry a higher percentage of the load, resulting in higher stresses. The East bearing was the fixed. The West bearing floated.

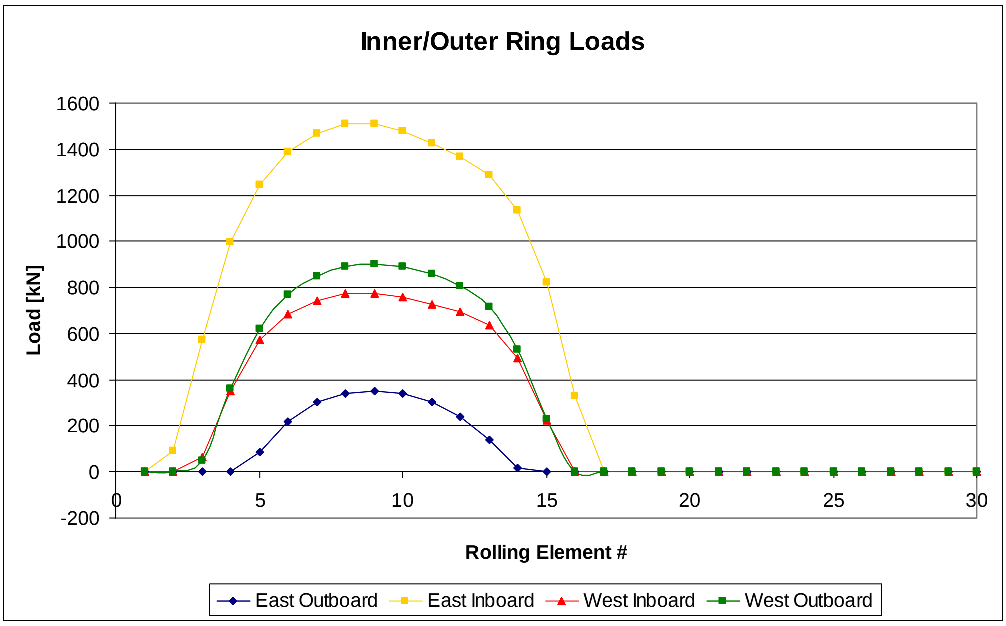


Table 2

Table 2 demonstrates the effect of axial loads on this particular bearing's load distribution. The plot identifies the raceway loads and the numbers of rollers in contact with the inner and outer raceway. The East bearing inboard roller set, supporting the thrust load, has 15 rollers in contact with significantly higher roller loads than the outboard row which only has 10 rollers in contact.

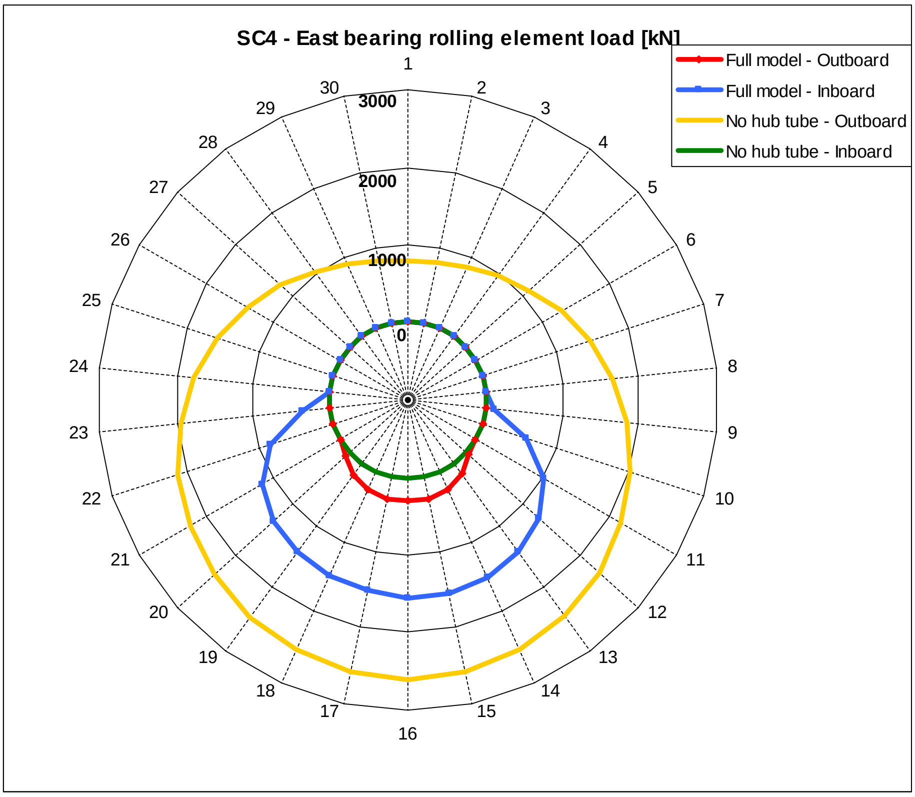


Table 3

Table 3 was used to analyze the impact that the stiffness of the hub tube had on rolling element load, and the resultant calculated fatigue life. The table compares rolling element load with and without the hub tube. Stresses on the rollers significantly increase as the flexibility of the housing increases. This information was utilized to derive the best hub stiffness, given the economic considerations of increasing the hub stiffness (thicker profile, braces) and the resultant increase in $L_{2.53}$ life.

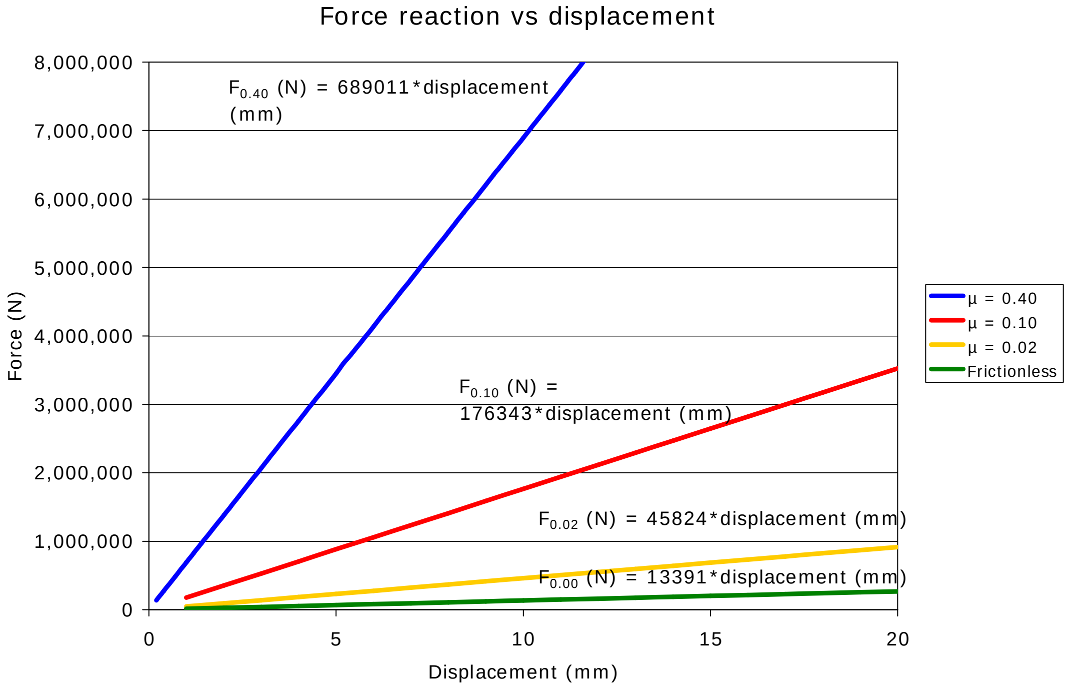


Table 4

Table 4 was used to determine installation requirements. Due to outer ring rotation, a press fit was required between the outer ring and hub. This was accomplished by using a tapered sleeve and tapered hub arrangement. After reviewing the potential pull in force requirements, and the number and size of pull in pistons, it was decided to include hydraulic injection assist into the design specifications. The install confirmed the need for hydralic injection assist.

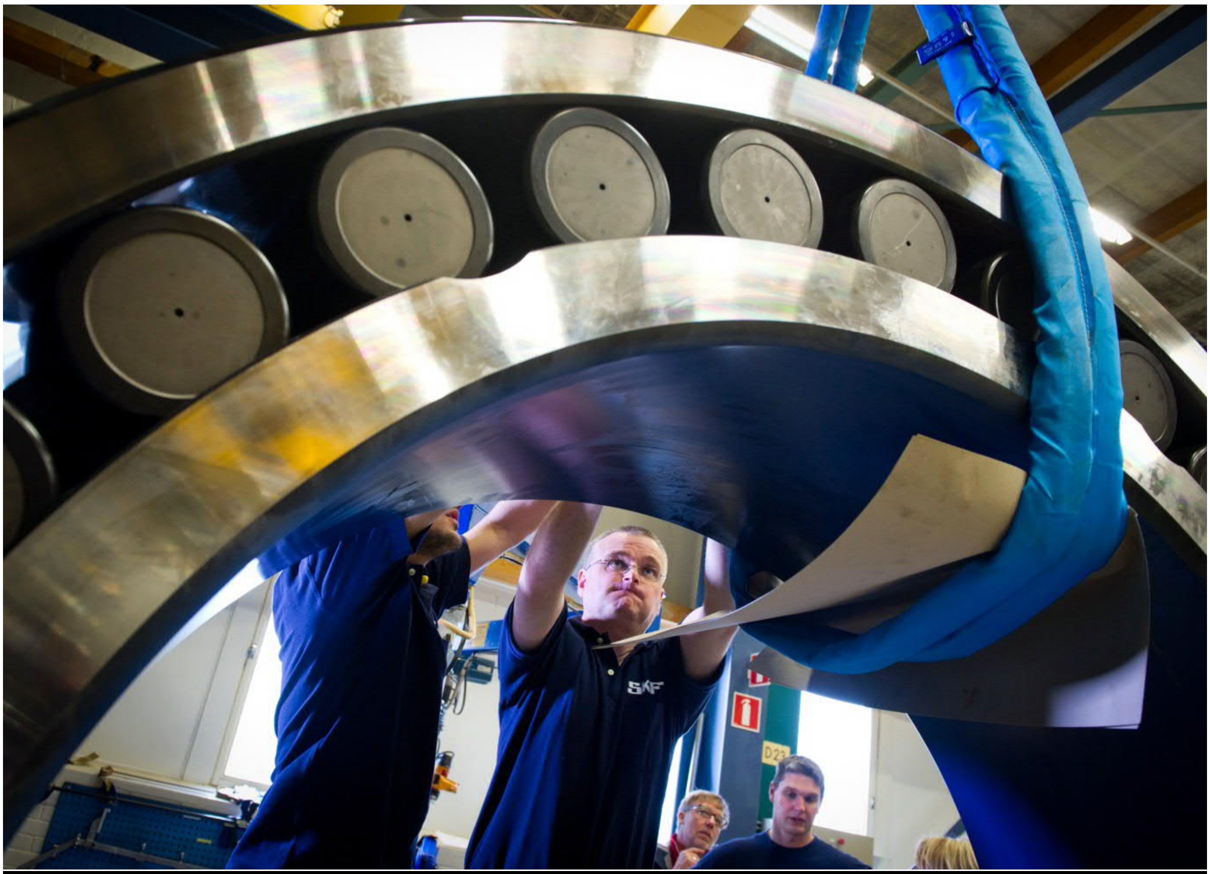
<u>Conclusions</u>

SKF generated significant $L_{2.53}$ increases utilizing the DFSS process and the resultant simulation analysis. Modifying the standard bearing roller profile, improving the stiffness of the hub, adding SKF's NoWear low friction, wear resistant carbon coating to the rollers, and specifying an auto lube system increased the calculated service life by over 10%. The bearing was custom designed for the application as compared to most bearings where the objective is to fullfill a variety of applications. SKF modifed the roller profile to meet the low speed and high load application conditions.

Various simulations provided SKF with the data to recommend the best hub and shaft stiffness, considering technical and cost considerations. Rolling element contact pressures are directly related to stiffness of the hub and shaft. The cost of improving the Hub and Shaft stiffness was correlated to the expected improvement in $L_{2.53}$ life. The decision was made, based on these studies, to improve the stiffness of the hub and shaft components.

The installation procedure simulations were nearly identical to actual field assembly.   The installation, as predicted, required the use of hydralic injection assist to mount the bearings properly.   The axial drive up required to achieve the final mounted internal clearance was within 2% of the calculated value. This was very impressive given the complexity of the hub design.

The Six Sigma approach to the project, along with the simulation tools, provided SKF with the process and tools to design a hub assembly solution that met the technical requirements of the project while considering manufacturing, transporation, construction, and financial constraints.   The spindle assembly solution provided by SKF was delivered on time and within budget.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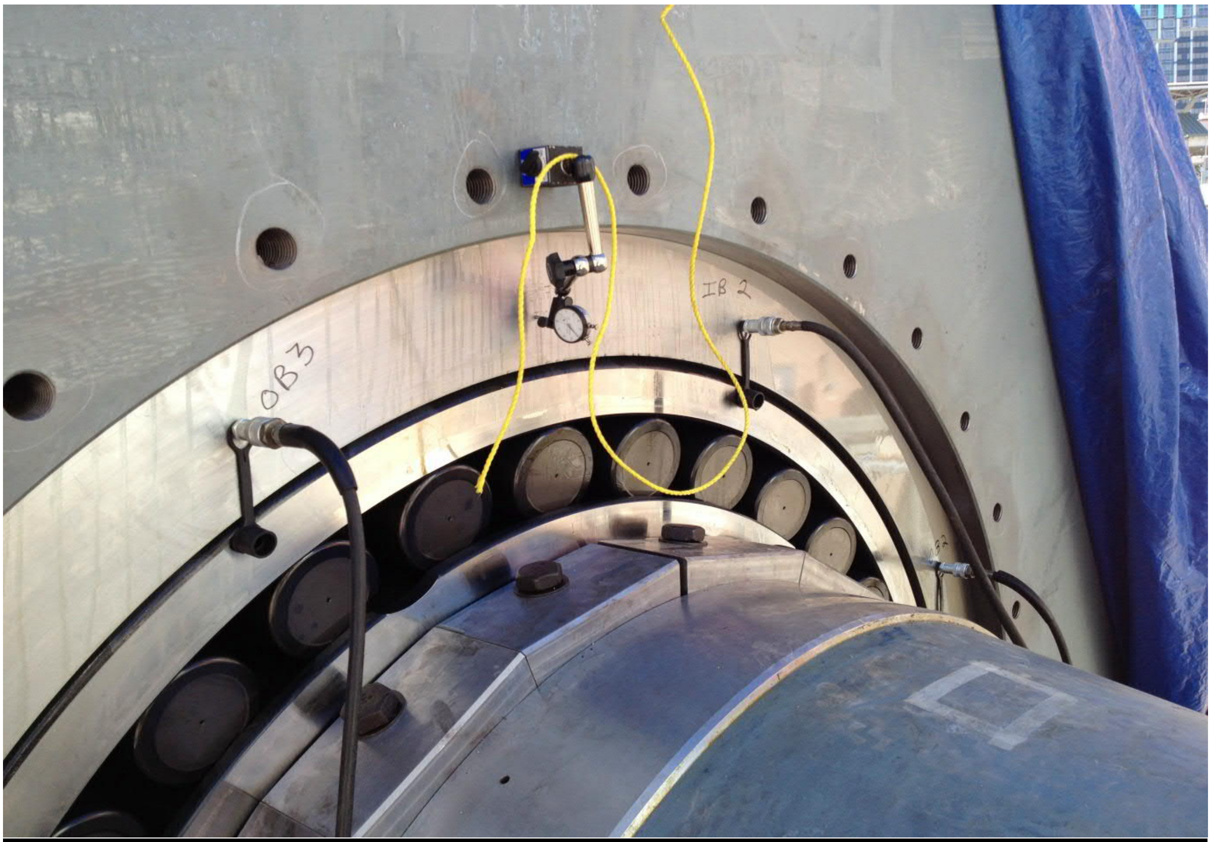
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HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Analyzing Center Pivot Bearing Discs, Was Hovey Right?

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Introduction

In his two-volume book titled "Movable Bridges" [1], Otis E. Hovey gives a method for determining the geometry for a dual-disc type pivot bearing assembly for swing bridges. One disc, the lower disc, is concave and made of steel, and the other disc is convex and made of bronze. The disc set is supported on a steel casting or weldment called a pedestal and is anchored to the pier top. The discs, in turn, support a steel casting attached to the swing span structure. The bearing supports the total weight of the span when turning, and a small amount of the live load as well (see Figure 1). One of the key geometric properties of the disc bearing set is the difference in spherical radii (at the concave/convex interface) between the two mating discs, i.e. the steel disc radius is slightly larger than the bronze disc radius. This difference in radii determines the contact area and resulting moment arm due to friction between the two discs. Hovey has done some of the work for the reader and gives suggested spherical radii for discs of various diameters (see Table 1)[2]. But the entire derivation is also shown so that the reader can calculate their own spherical radii and tweak it to adjust the various operating parameters as needed.

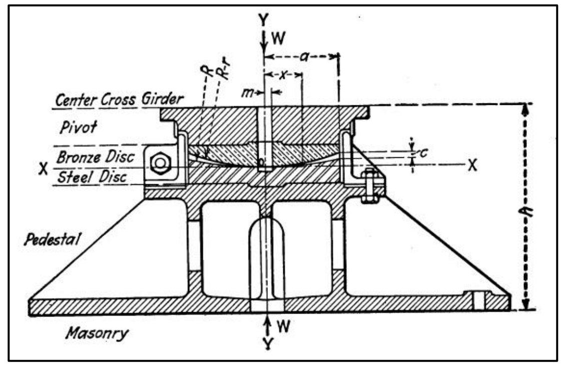


Figure 1 - Typical center bearing arrangement

Diameter 2A	Turning Load W	Fixed Load W_F	Rad. St. R	Rad. Bz. R − r	Pivot P	Disc T	Discs 2T	Pedestal G	Height H	Diameter K
12″	291,000	591,000	2′-6″	2′-5″	4″	1¾″	3½″	1′-0″	1′-7½″	3′-8″
15″	471,000	771,000	3′-0″	2′-11″	4″	2″	4″	1′-1½″	1′-9½″	4′-2″
18″	678,000	978,000	4′-0″	3′-10¾″	4½″	2¼″	4½″	1′-3″	2′-0″	4′-8″
21″	939,000	1,239,000	5′-0″	4′-10″	6″	2½″	5″	1′-5″	2′-4″	5′-3″
24″	1,245,000	1,545,000	6′-0″	5′-9¾″	7½″	2¾″	5½″	1′-6½″	2′-7½″	5′-11″
27″	1,590,000	1,890,000	7′-0″	6′-9½″	7½″	3″	6″	1′-7″	2′-8½″	6′-6″
30″	1,980,000	2,280,000	8′-0″	7′-9″	8½″	3¼″	6½″	1′-9″	3′-0″	7′-2″
33″	2,412,000	2,712,000	9′-0″	8′-8¾″	9″	3½″	7″	1′-11″	3′-3″	7′-9″
36″	2,886,000	3,186,000	10′-0″	9′-8¼″	10½″	3¾″	7½″	2′-2″	3′-8″	8′-5″
40″	3,582,000	3,882,000	11′-0″	10′-8″	11½″	4″	8″	2′-3½″	3′-11″	9′-4″
44″	4,356,000	4,656,000	12′-0″	11′-8″	1′-1″	4½″	9″	2′-5″	4 -3	10′-2″
48″	5,205,000	5,505,000	13′-9″	13′-4¼″	1′-2½″	5″	10″	2′-6½″	4′-7″	11′-1″
52″	6,129,000	6,429,000	15′-0″	14′-7″	1′-4″	5½″	11″	2′-8″	4′-11″	12′-0″

Table 1 - Hovey Suggested Disc Geometry[2]

Hovey provides a list of criteria for good design of center bearings, this list is a direct excerpt from his book[3]:

- Two discs are used.
- The pressure on the projected net area of the discs is 3000 psi.
- The radius of the spherical bearing surface of the upper, or bronze, disc is made smaller than that of the lower, or steel disc, to diminish the radius of friction and the power required for turning the bridge. The difference of the radii is such that the maximum pressure, under full load, is 10,000 psi at the edge of the central oil hole.
- The maximum unit compression in the central part of the lower center casting is 10,000 psi.

[1] "Movable Bridges" by Otis Ellis Hovey, Vols. I and II, New York, John Wiley and Sons, Inc., 1926
[2] Ibid, Volume II, p. 302
[3] Ibid, Volume II, p. 301

- The pressure on the masonry is 400 psi of net area of the base casting.
- The maximum fiber stress, due to bending, in the castings, is 16,000 psi, assuming that the center girders are rigid and that the masonry pressure is uniform.

The third bullet point will be the focus of this paper, which analyzes the balance between stress concentrated near the center of the discs and the contact area and resulting lever arm of friction. When the difference in spherical radii between the two discs is increased, the stress near the disc centers increases, however the contact area and friction arm are decreased. Hovey sets an upper bound of 10,000 psi for the stress at the edge of the center hole of the discs. This paper will compare the results of Hovey's stress calculation method with results from finite element analysis of a 3D model.

Hovey Method of Disc Analysis

The detailed analysis of the stress distribution of the center bearing discs is in Appendix A of Volume I of "Movable Bridges". Hovey begins the analysis by giving the algebraic equations of the curved surfaces of spherical discs and parabolic discs. He then states that calculations show that "the differences between the spherical and parabolic surfaces are no more than the errors that are likely to occur in finishing the discs with ordinary machine tools". The analysis then proceeds assuming a parabolic disc. This statement was verified for the two disc cases discussed below. The largest difference between the spherical and parabolic surfaces for each case was found to be 0.002".

Hovey uses the theory of elasticity and states that the unit stresses in the bronze disc are proportional to the distortions that occur under load. He then proceeds to derive an equation for stress, but ends with two unknowns in the equation. The next step uses a relationship for work and assumes that the total external work of the bridge weight is equal to the internal work of compressing the center discs. He then continues to derive equations for the appropriate difference in spherical radii and the resulting lever arm of friction (with the geometry of the discs and support castings used as inputs). Hovey provides example results for discs of varying diameters and loads, shown above in Table 1.

A spreadsheet was created to perform Hovey's calculations for this investigation. The first example to be investigated will be referred to as "Disc Set A", which has a 15" diameter and a span weight of 497 kips. Example results for varying disc geometry are given in Table 2. Row 1 of Table 2 uses the recommended span weight and disc geometry given by Hovey in Table 1 for a 15" diameter disc. The first result to notice is the stress at the edge of the central hole is about 11,500 psi, higher than the 10,000 psi limit set by Hovey. This number is influenced somewhat by the average modulus of elasticity ("E") of the entire pivot assembly, which can vary from bridge to bridge. Additionally, the average modulus for any given center bearing assembly can vary, and typical assemblies (as per our models) with voids near the pedestal base center (see Figure 2) will have a lower average modulus in the center. However, a significant decrease in the average "E" is required to decrease the stress to 10,000 psi; in this case the "E" would need to be 17,000,000 psi (75% of the actual). Although, a small change in the bronze disc radius decreases the stress at the center significantly, as shown in Row 2. Rows 3 through 13 show several different variations of steel and bronze disc radii combinations, with Row 5 being the disc chosen as the optimal design that will be further investigated. It can be observed that changing the difference in radii has a significant effect on the stress at the center, and a lesser effect on the overall turning friction.

Calculate Stresses in Discs, Radius of Contact, & Lever Arm of Friction using Hovey's equations

Inputs

471,000	Span dead load from Hovey for disc size (lb)
497,000	New span dead load (lb)
22,530,000	E (avg for assembly) (psi)
21	(h) - height of Assembly
7.5	(a) - radius of steel/bronze discs (in)
0.8125	(m) - radius of central hole (in)

0.75	oil groove width
3	number of oil grooves
0.15	friction ceofficient
7.5	Motor HP
870	Motor RPM
4583.33	Total Reduction, Motor to Pivot Bearing

Row	(W) - Dead Load (lbs)	(R) - Steel Disc Radius (in)	(r) - Diff. in Radii (in)	(R-r) - Bronze Disc Radius (in)	(c) - unloaded Dist. Btwn discs at edge (in)	Contact Pressure on Projected Area (psi)	(S) - Stress at Central Hole (psi)	(x) - Radius of contact (in)	Clearance at edge (loaded) (in)	(L) - Lever Arm of Friction (in)	Moment due to Friction (ft-lb)	Req'd FLT of Motor (%)
1	471,000	36	1	35	0.023	3295	11,426	5.19	0.012	2.88	16,954	8.17
2	471,000	36	0.75	35.25	0.017	3295	9,859	5.57	0.008	3.08	18,132	8.74
3	497,000	36	1	35	0.023	3476	11,737	5.26	0.012	2.91	18,109	8.73
4	497,000	36	0.875	35.125	0.020	3476	10,959	5.43	0.009	3.01	18,683	9.00
5	497,000	36	0.75	35.25	0.017	3476	10,127	5.65	0.007	3.12	19,371	9.33
6	497,000	38	1.25	36.75	0.026	3476	12,443	5.11	0.014	2.84	17,637	8.50
7	497,000	38	1	37	0.020	3476	11,091	5.40	0.010	2.99	18,582	8.95
8	497,000	38	0.75	37.25	0.015	3476	9,572	5.81	0.006	3.20	19,880	9.58
9	497,000	40	1.125	38.875	0.021	3476	11,169	5.38	0.010	2.98	18,522	8.93
10	497,000	40	1	39	0.018	3476	10,513	5.55	0.008	3.07	19,041	9.18
11	497,000	42	1.375	40.625	0.023	3476	11,773	5.25	0.012	2.91	18,084	8.71
12	497,000	42	1.25	40.75	0.021	3476	11,208	5.38	0.010	2.98	18,493	8.91
13	497,000	42	1	41	0.017	3476	9,993	5.69	0.007	3.14	19,489	9.39
	Inputs			**Results**								

Table 2 – "Disc Set A" Calculation Results

Finite Element Analysis Method

It was desired to find another method of checking the stress distribution in the pivot bearing discs to verify the values calculated from the Hovey method. To do this, a finite element model was constructed to perform a static stress analysis on the pivot bearing discussed above, calculated in Row 5 of Table 2 (see Figure 2). The same span load was applied and the model was analyzed for stress and deflection. To view stresses through the center of the discs, half of the model was analyzed and a symmetry boundary condition was placed on the cut plane. The stresses calculated by Hovey are in the vertical direction, so the best FEA results to use for comparison are the axial stresses in the vertical direction. A

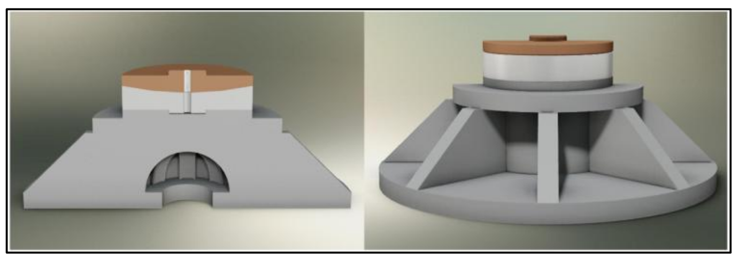


Figure 2 - Model Created for Finite Element Analysis

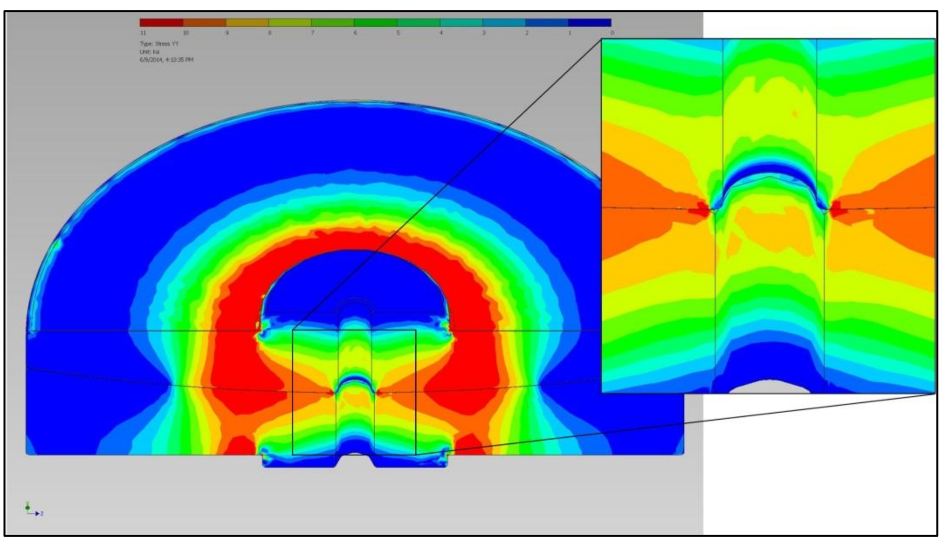


Figure 3 – Vertical Stress Distribution for Bearing "Disc Set A"

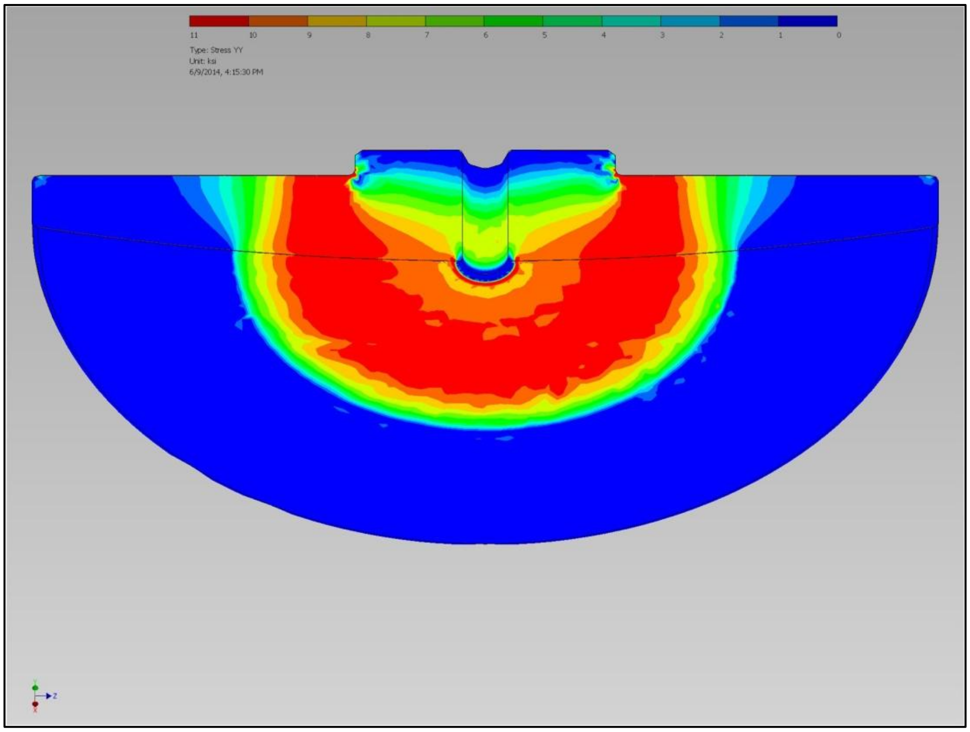


Figure 4 – Vertical Stress Distribution on Bronze "Disc A" Spherical Surface

combined view of the vertical stress distribution for both the bronze and steel discs is shown in Figure 3. In this figure, the color red denotes higher compressive stresses areas, and blue denotes lower compressive stresses. The scale is set from 0 to 11,000 psi, so compressive stresses denoted in red are greater than 10,000 psi. The stress is concentrated right at the edge of the hole, decreases somewhat, and then increases in the area directly between the loaded portions of the discs (the bosses on the bronze and steel discs have a small clearance with their bores are not loaded). The highest stresses in the red region are just over 11,000 psi. Referring to Row 5 of Table 2, the max stress at the edge of the central hole was calculated to be 10,127 psi. Also, the radius of contact was calculated to be 5.6 in, which is about 75% of the disc radius. In Figure 3, the stress decreases dramatically just before about 75% of the radius. These two FEA results seem to match fairly closely

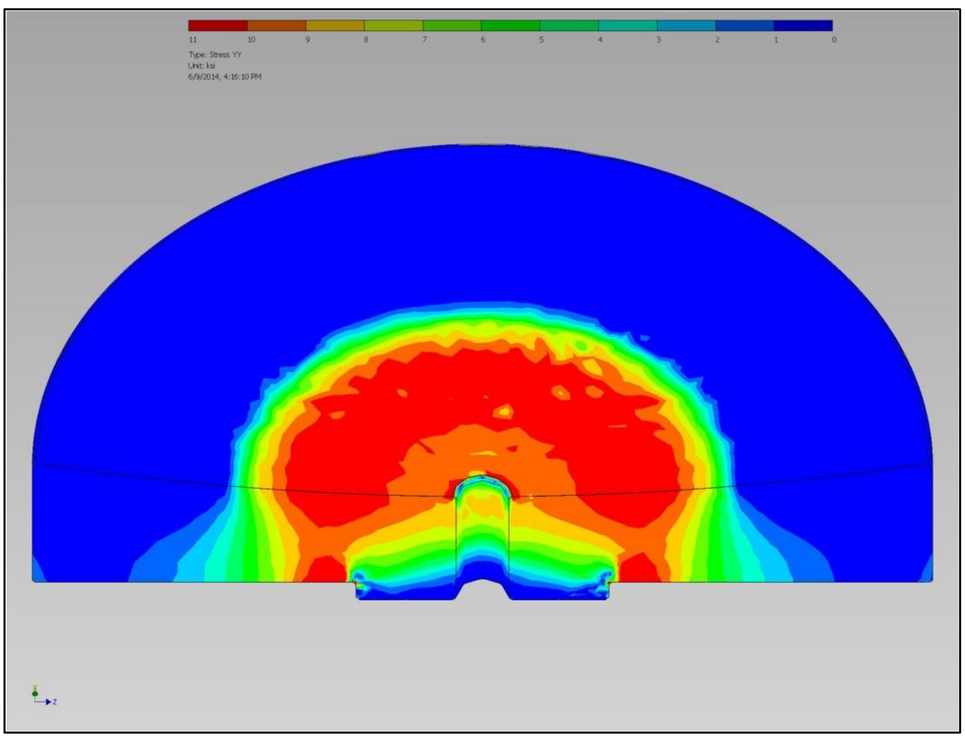


Figure 5 – Vertical Stress Distribution of Steel "Disc A" Spherical Surface

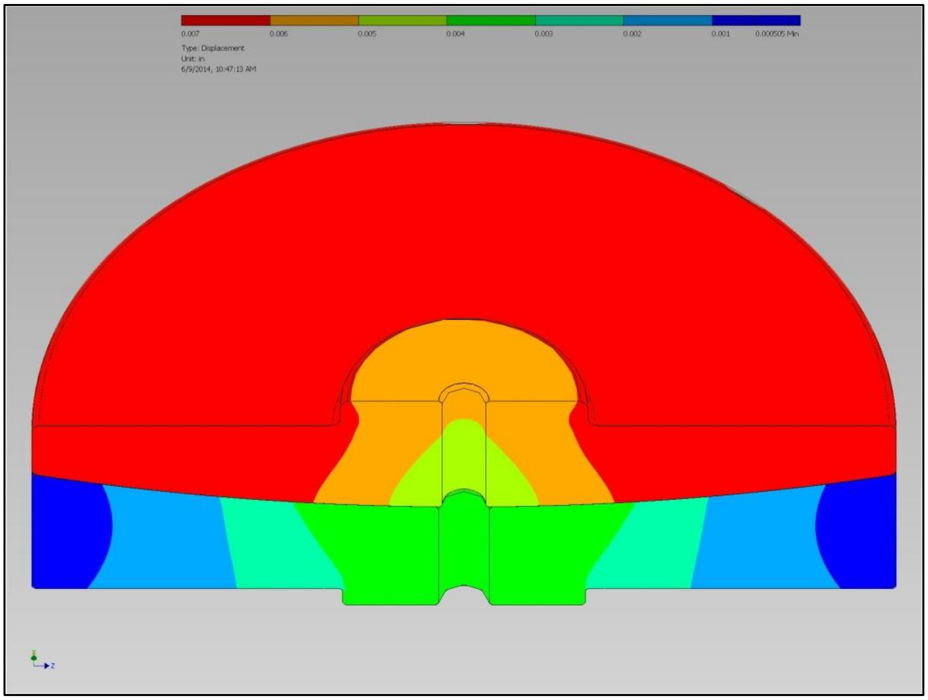


Figure 6 – "Disc Set A" Deflection

with the Hovey calculated results, with the max stress in the main contact area being about 1,100 psi higher than the 10,000 psi theoretical limit. For a view of the spherical contact surfaces, see Figures 4 and 5. Also, the deflection of the steel and bronze discs is presented in Figure 6, where the max deflection of the bronze disc is 0.007. The calculated decrease in the clearance at the edge of the discs (from Row 5 of Table 2) is 0.010".

To verify the FEA method, another disc set of different size and loading, referred to as "Disc Set B", was modeled using the same types of inputs and constraints. This disc set has a 25 ½" diameter and a span weight of 1,257 kips. The Hovey calculations were performed for this disc set and are presented in Table 3, and the results of the FEA model are shown in Figures 7 and 8. The same scale has been used from the first disc set, so the compressive stresses higher than 10,000 psi are depicted in red. At the edge of the central hole of the bronze disc, the stress is approximately 10,000 psi, with a similar distribution to the first disc set analyzed, only the stress does not increase beyond 10,000 psi. The radius of contact calculated in Table 3 is 8.8" (about 70% of disc radius), and the stress decreases significantly at 60-65% of the radius. The deflection at the edge of the bronze disc is shown in Figure 10 to be 0.014". The calculated decrease in clearance at the disc edge in Table 3 is also 0.014". The Hovey calculated and FEA results for this set match very closely.

Calculate Stresses in Discs, Radius of Contact, & Lever Arm of Friction using Hovey's equations

Inputs			
1,417,500	Span dead load from Hovey for disc size (lb)	1.25	(m) - radius of central hole (in)
1,257,000	Span dead load (lb)	0.75	Oil groove width (including side radii)
20,766,411	E (avg for assembly) (psi)	3	Number of oil grooves
27	(h) - height of Assembly	0.15	friction ceofficient
12.75	(a) - radius of steel/bronze discs (in)	25.00	Motor HP

Row	(W) - Dead Load (lbs)	(R) - Steel Disc Radius (in)	(r) - Diff. in Radii (in)	(R-r) - Bronze Disc Radius (in)	(c) - unloaded Dist. Btwn discs at edge (in)	Contact Pressure on Projected Area (psi)	(S) - Stress at Central Hole (psi)	(x) - Radius of contact (in)	Clearance at edge (loaded) (in)	(L) - Lever Arm of Friction (in)	Moment due to Friction (ft-lb)
1	1,417,500	78	2.25	75.75	0.031	3127	**11,561**	8.92	0.016	4.92	87,137
2	1,417,500	78	1.75	76.25	0.024	3127	**10,161**	9.51	0.011	5.22	92,501
3	1,257,000	72	1.75	70.25	0.029	2773	**10,395**	8.86	0.015	4.89	76,777
4	1,257,000	72	0.03	71.97	0.0005	2773	**1,344**	24.43	-0.001	13.09	205,733
	Inputs			Results							

Table 3 – "Disc Set B" Calculation Results

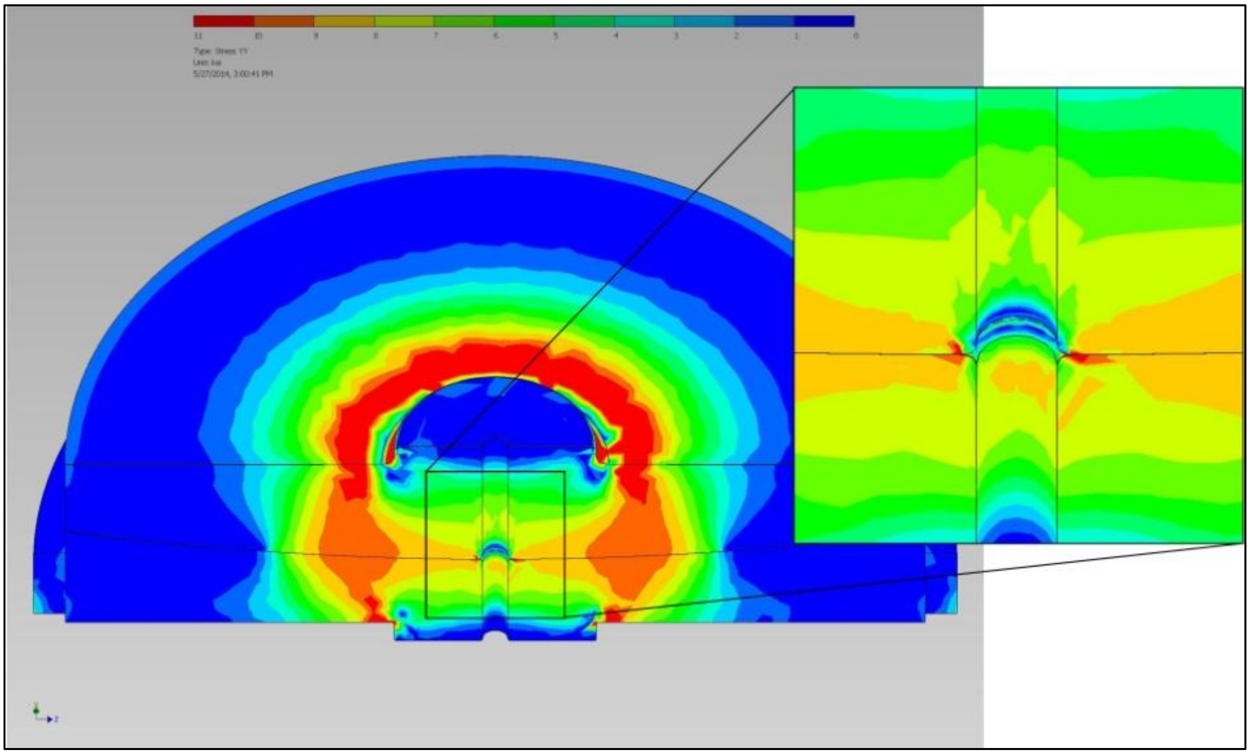


Figure 7 – "Disc Set B" Vertical Stress Distribution

These similarities between the FEA results and the calculated performance using Hovey's equations, performed for two separate disc sets, give confidence that the two methods are in agreement. More importantly, the analysis of two separate cases shows that the Hovey method for choosing the spherical radii, when the concern is stress and contact area, is consistent. For a best comparison to the Hovey equation results, the above FEA results include the vertical axis stress only. However, for a better illustration of the actual stresses expected in the disc set, the von Mises equivalent stress

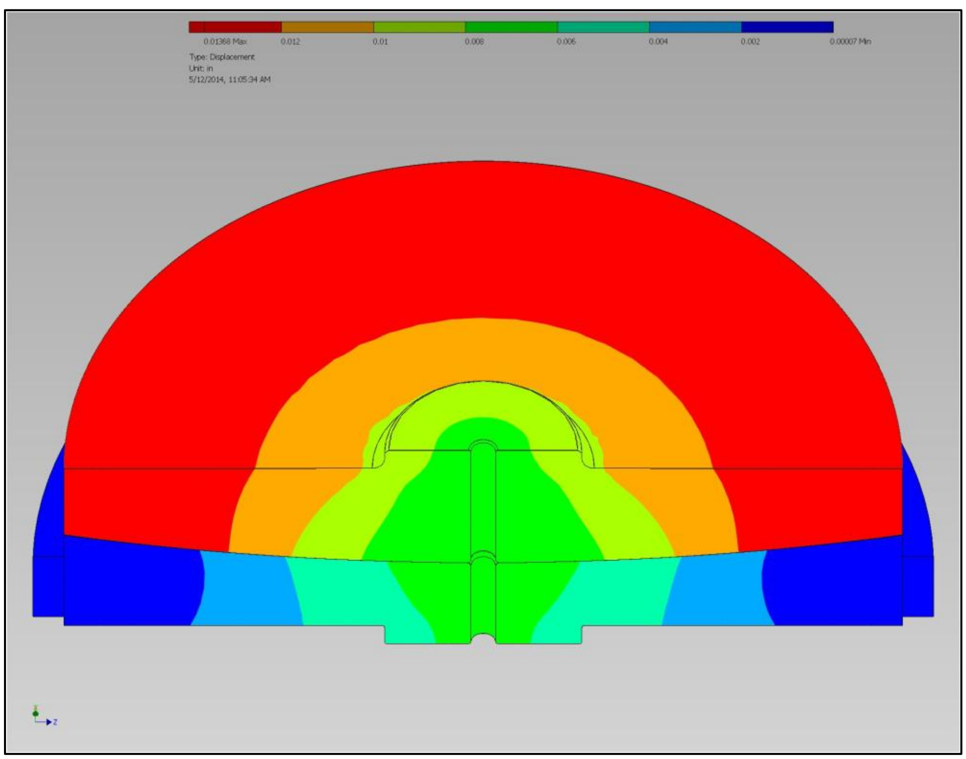


Figure 8 – "Disc Set B" Deflection

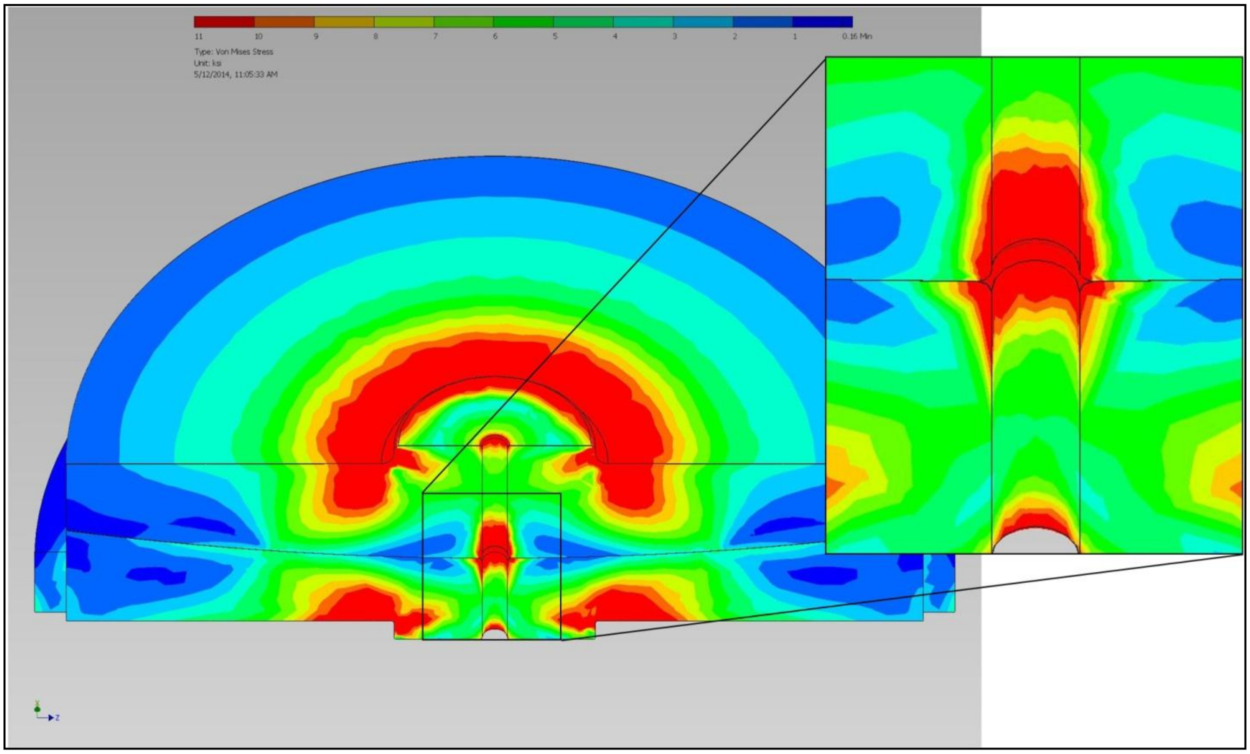


Figure 9 – “Disc Set B” Von Mises Stress Distribution

calculated by the FEA analysis is typically used to compare against the material yield stress. For “Disc Set B”, the von Mises stress distribution is shown in Figure 9. The same scale is used as in the previous analyses, where stress higher than 10,000 psi is shown in red. Compared to Figure 7, there are higher stresses around the central hole due to stress concentrations. But a similar distribution can be seen in the rest of the disc where stress increases in the area directly beneath the loading and decreases further towards the disc edge. So even though the von Mises stress is not the best direct comparison to Hovey’s calculations, it shows that Hovey made a fairly good approximation by considering only the vertical stress.

Analyzing a “Non-Conforming” Disc Set

We can now use both methods to predict the performance of a disc that does not conform to the guidelines set by Hovey. The second disc analyzed above was modified to have very little difference in spherical radii between the two discs, see Table 3, Row 4. The load and overall geometry was left unchanged, but the difference in radii was reduced drastically to 0.030” (from the ideal 1¾”). This left an edge clearance of only 0.0005” with no load on the bearing. With the load added, the clearance at the disc edge is nonexistent with the full surface area of the discs in contact. It should be noted that the radius of contact is calculated to be nearly twice the actual radius of the disc and the lever arm of friction is slightly larger than the disc radius. The contact radius calculation solves for the point of zero distortion, starting at the disc center and moving outward. The stress at the disc center is the starting point, and a ratio along the parabolic curve (approximating the spherical surface) is used to find the point at which stress and distortion are zero. The clearance at the edge of the unloaded discs is used as an input into this calculation to find the ratio. In the contact radius formula, the edge clearance variable is in the denominator, and since the clearance is very small in the present case, the contact radius seems to have become erroneously

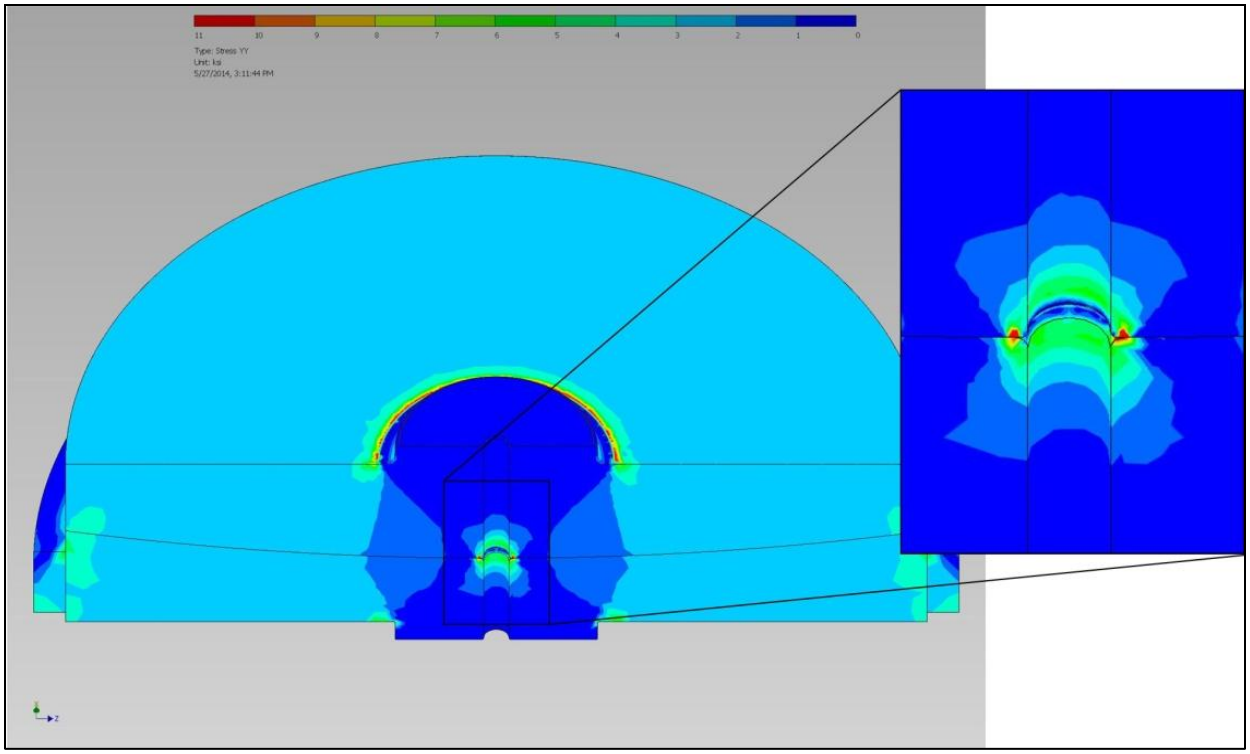


Figure 10 - Non-Conforming Disc Set Vertical Stress Distribution

large. It is apparent that the method outlined by Hovey breaks down at some point and the outputs may not be reliable. The scope of this paper did not include finding the point at which the Hovey method may become unreliable.

To further investigate the case of very small difference in spherical radii, a finite element model was produced and analyzed, and the stress distribution is shown in Figure 10. The scale is the same as the previous analysis of Disc Set B for direct comparison. The scale is 0 – 11,000 psi, with any vertical compressive stresses over 10,000 psi being shown as red. With the exception of some localized high stresses at the central hole, the stress is fairly

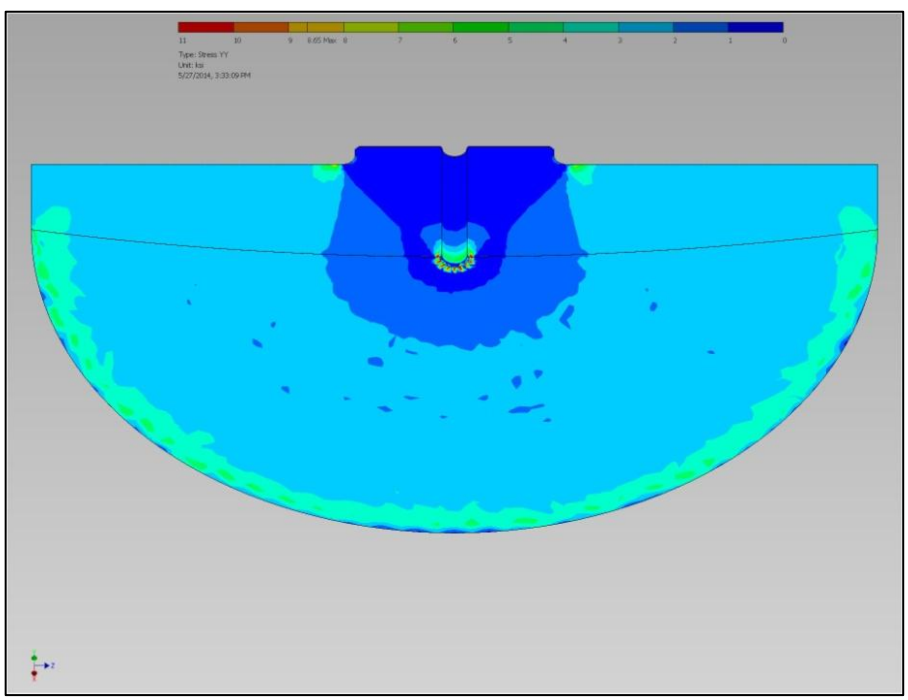


Figure 11 - Non-Conforming Bronze Disc Stress

evenly distributed across the entire spherical surface, with slightly higher stress closer to the outer edge of the disc (see Figure 11 for stresses in the bronze disc only). This is apparently due to slight deflection of the pivot base, shown in Figure 12, allowing the center of the discs to deflect downward slightly, loading the edges of the discs. The base deflects in a similar manner in Disc Sets A & B, however the outer edge of Discs A & B have some clearance so this base deflection does not affect the loading on the outer edge of the discs.

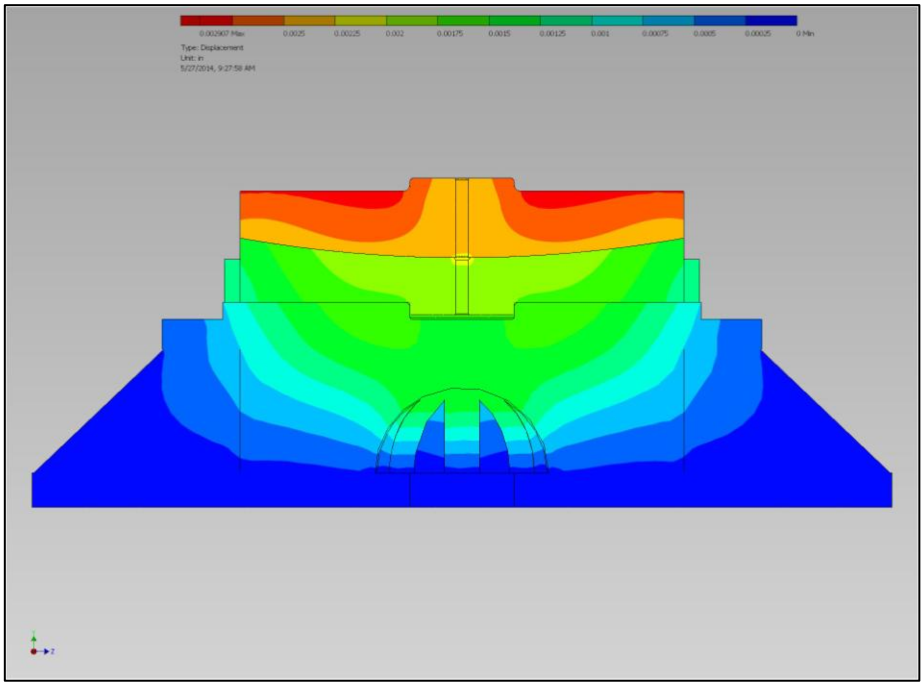


Figure 12 - Non-Conforming Disc Set Deflection

Since the stress is somewhat higher near the outer edge of the discs, this will bias the lever arm of friction more towards the disc edge, likely beyond the length of 2/3 of the disc radius typically used for pivot bearings. As such, this disc geometry would produce significantly higher turning friction than a disc set designed using Hovey's method.

Conclusion

Given all of the above information, an obvious conclusion is the Hovey method for designing pivot bearing discs is well thought out and easy to implement. It provides a good compromise of stress and friction so that prime movers can be efficiently sized while not overloading the pivot bearing elements. The vast majority of center bearing pivot swing bridges encountered by the author and by Modjeski and Masters, Inc, follow the theory of incorporating a small but important difference in spherical radii in the bearing discs. This arrangement has proven to be robust requiring little maintenance beyond regular lubrication. A drawback would be some sensitivity to the intrusion of water and debris during flood stages of waterways at lower level bridges. Efforts have been made to include seals in these installations with some success. However, the overall assembly design facilitates relatively easy cleanout of the bearing disc area and the oil grooves, and even removal and replacement of the bearing discs with the obvious requirement that the span load be relieved prior to bearing removal.

Further research could be performed to find the actual application limits of Hovey's derived equations, to find how small the difference in radii can actually be made and still be predicted by Hovey's method, but there is little benefit in doing so. The method provides a sound design with little compromise and should be considered for all center bearing swing span designs.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Gearbox Maintenance and Lubrication

John G. Proven, P.E.
Nuttall Gear

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

HMS Symposium
Presentation
Gearbox Maintenance and Lubrication

Presented by John Proven-Nuttall Gear
National Sales Manager

Gearbox Maintenance and Lubrication

Reliability and durability of gearboxes depends on the following

- The design parameters are properly specified
- The unit is properly maintained
- The unit is receives proper lubrication of gearing and bearings

Design Specification:

Gearboxes must be engineered and designed, or properly selected for the specified operating conditions which include the following:

1. Input speed and power
2. Required output speed and torque
3. Service factor based on application-AGMA standard recommendations are good but the user must identify any unique factors.
4. Environment:
5. Configuration requirements
6. Duty Cycle
7. External loading requirement
8. Desired design life- not necessarily infinite

Most gearbox manufacturers are experts in engineering, designing, and manufacturing gearboxes. They are not necessarily experts in all of the processes and industries that they support. In many cases, the process is confidential and proprietary, and must not be disclosed to anyone outside the company. Very simply a gearbox is required when the process requires normal speeds of equipment to be different than the electric motor, diesel engine, turbine, or other device that is driving the equipment

The size of the unit is dictated by the amount of power and torque that needs to be transmitted. How much power is required? Again we, the gearbox designer/manufacturer require the expertise of the bridge engineer/designer, the steel mill OEM/user, the mass transit designer/operator, the ethanol producer, and engineers and designers of a multitude of other industries to define the operating conditions and performance requirements of the driven equipment.

Gearbox Maintenance:

All gearboxes must receive periodic maintenance including an oil change. Oil should be checked regularly for contamination from dirt, debris, and other fluids such as water. The oil should also be changed periodically based on hours of operation and based on oil temperature. Oil that operates and elevated temperatures, above 150 degrees F needs to be changed more often than oil that operates at 120. As the temperature increases up to 180 degrees F the oil change frequency increases significantly. Between 180 and 200 degrees F the recommended time between changes goes is reduced by 75%. The elevated temperatures accelerate the breakdown of the molecular structure of the oil thereby inhibiting its ability to form a protective film. If oil continuously operates above 200 degrees F, a circulating lube oil system should be considered to cool the oil.

AGMA recommends that the oil be changed after the first 500 hours or 4 weeks of operation, whichever comes first. After the initial operation of the unit, AGMA recommends that the oil be changed every 2500 hours of operation or every 6 months, whichever comes first. AGMA further suggests that these intervals can be adjusted based on the system configuration as recommended by the manufacturer and furthermore that a condition monitoring program that identifies changes in the lubricant such as color, viscosity, oxidation, water concentration, contaminant concentration, percentage of sludge, and change in oil chemistry, primarily the additives, can be implemented to extend the change intervals. Basically check out the system and make a change if it makes sense.

In addition to oil, the physical condition of the unit including the foundation, protective coating, seals, breathers, circulating oil system, couplings, and bearings should be inspected periodically. A problem with any of these items identified in the early stage by plant personnel can help avoid a catastrophic premature failure of the gearbox.

A worn bearing may cause uneven wear on gear teeth, but prolonged operation in this condition can lead to more severe conditions resulting in broken gear teeth which can feed to other gears in the train and cause damage to more components that might not have otherwise required replacement. An adverse condition may not be obvious to the operator but a periodic inspection of the gearing and any changes or acceleration in wear patterns indicate that something has changed and it should be investigated.

Condition monitoring programs evaluate changes in operating parameters and provide valuable quantitative data that can help forecast when failures might occur. These services can be performed by in house personnel or contracted. Oil temperature, level, and condition, vibration, noise and physical condition of seals and breathers are some of the parameters that should be monitored. After an initial baseline evaluation of the system, periodic inspections, photographs, and data analysis are used to identify and evaluate any changes or trends that might signal a problem.

Lubrication:

Proper lubrication is the single most important factor in ensuring the continued performance of a gearbox. Gears and bearings require properly specified and maintained lubrication. The oil must be selected with the proper viscosity, pour point, and chemical make-up based on each application. All of the design factors listed above contribute to the selection. Most of the applications that will be addressed by this audience will involve relatively slow speed applications; 1800 RPM motors being slowed down to single digit speeds generally working with high torque requirements.

Relatively slow speed gears generally operate at pitch line velocities that are less than 2000 feet per minute. Oil operating at these speeds will not generally be subjected to overheating as a result of churning or internal heat build up from friction. Oil shear, the breakdown of the molecular structure of the oil, and air entrainment, both conditions that reduce the effectiveness of the oil, will normally not occur at the lower speeds. Therefore the performance of the oil will very predictable.

Lubrication effectiveness is a function of the oil film thickness and the ability of the oil to flow on the gear tooth surface. The viscosity of the oil varies with operating temperature and is the primary means of determining the effectiveness of the selected oil. If the viscosity is low, to the point that the oil does not have time to flow adequately to cover the tooth surface, insufficient lubrication will ultimately cause metal-to-metal contact between the mating gear teeth.

Oil film thickness is not only a function of the oil viscosity, but also the pressure on the gear teeth. Many factors including the gear tooth design, pressure angle, diametral pitch, crowning and others determine the forces where two gear teeth mesh. Oil is essentially a non-compressible fluid that will be squeezed out from between the teeth when the force is applied. The film can never be completely eliminated but it will become very thin under extreme operating conditions.

The oil film that remains is important because relative motion exists between the gear teeth and this motion is primarily sliding not rolling. The pitch line of the gear teeth is the only point on the involute profile that the mating teeth experience relative rolling motion. The balance of the contact is sliding. Therefore, with two sliding metal surfaces mating, adequate lubrication film thickness is imperative for long gear life.

The surface finish, or roughness of the face of the gear tooth defines the recommended oil film thickness. A microscopic cross-sectional view of every surface reveals surface peaks and valleys defined by the surface finish. The roughness or surface finish of the face of the gear tooth can be measured in microns. As a general practice, the calculated oil film thickness should be approximately 2.5 to 3 times the surface asperities, the magnitude of the peak and valleys. This amount of oil film ensures that under the specified operating conditions, providing that the oil has been properly maintained, that the oil film will be adequate to prevent metal-to-metal contact between gear teeth. Should metal-to-metal contact occur, scuffing, scoring, pitting and premature wear will be observed. These conditions represent several failure modes of the gearbox related to lubrication.

Oil must also flow properly in order to achieve proper lubrication. As the gear teeth mesh, the viscosity must be such to allow the oil to flow into the mesh. If the oil is too thick, yes too thick!, the oil will not have time to form a proper film between the mating teeth. If the oil is too thin, the film thickness will not be adequate. Both conditions will result in metal-to-metal contact between gear teeth, initiation of premature failure of the gearing.

In gear units that require splash lubrication, the gear teeth pick up oil from the bottom of the gear case and deposit the oil on the mating teeth. The gearbox will experience lubrication issues if the oil selected is not viscous enough or if the gear speed, the pitch line velocity, is too high. In this case, centrifugal force will not allow enough oil to remain on the gear to produce an adequate film thickness. Again, the result will be pre-mature pitting and scuffing leading to failure.

Applications such as bridges where gearboxes may be exposed to a variety of temperature and weather conditions, a synthetic grade of oil might be considered, as the viscosity is constant over a larger temperature range. The use of synthetic oil selection may not require that the oil be changed out with the changing weather. If more standard oils are selected, oil heaters, oil coolers, or perhaps replacement of oil during different seasons may be required. A factor for operators that must be addressed is that the synthetic oils have significantly higher costs than the standard grades. The reduced frequency of replacement could justify the additional cost.

Summary:

The emphasis of this discussion has been on lubrication for gearbox maintenance. I cannot over emphasize the importance of properly specified and maintained lubrication and would encourage you to review your systems and upgrade your condition monitoring programs as necessary,

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Hydraulic Control of Florida Polytechnic University Louvered Roof, Lakeland, FL

Robert Ferrara, Thomas Behling
Atlantic Industrial Technologies

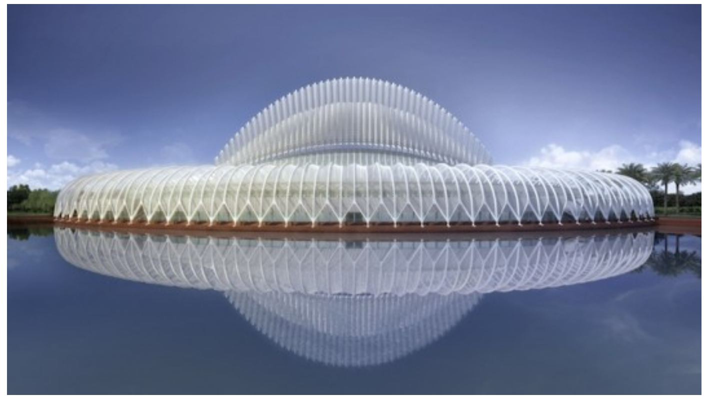
NEW ORLEANS FRENCH QUARTER MARIOT HOTEL

NEW ORLEANS, LOUISIANA

Abstract

The 160,000-square-foot innovation, science and technology building at Florida Polytechnic University will be the cornerstone of the new campus in Lakeland FL. Less than 20 minutes from Orlando, the building was designed by Santiago Calatrava and is being constructed by Skanska. The architecture combines elements of light, air, open views, reflecting water and innovative solutions.

Atlantic Industrial Technologies (Atlantic) is responsible for the design, manufacture, installation and commissioning of the 94 louver system that will act as a moving architectural sun shade for a large central skylight in this building. The louvers range in length from 61' to 20'. Safely moving and controlling a louver can take up to 50 tons of hydraulic force.


Structure conceptual drawing

Atlantic chose to control the louvers with proportional hydraulic valves that communicate with a sophisticated centralized automation controller. Moving against the trend toward greater local intelligence was chosen, because the operating components will be exposed to harsh weather on top of the building. Given this environment, Atlantic is keeping the valves simple and putting the intelligence in a remote, watertight and corrosion resistant NEMA 4X enclosure. This approach allows the necessary intelligent control to be unaffected by many seasons of Florida weather, and perhaps the occasional hurricane.

Introduction

When officials at Florida Polytechnic University realized that their existing campus could no longer support a burgeoning population, it made sense to commission world-renowned alumnus Santiago Calatrava to design a new one. Construction is almost complete on the egg-shaped Innovation, Science, and Technology building, which will stand as the sole new campus until the entire plan is finalized. The two-storey Innovation, Science, and Technology building clad in white grating promotes natural ventilation and daylighting and lies on the northern edge of campus. When complete, it will feature classrooms, laboratories, administrative offices, community space, and a large amphitheater that will be used for holding various public functions. . In the Calatrava style, lighting and how the sunlight is manipulated is such an important part of the design.

Ninety four louvers will cover the glass skylights that cover the spine of the building. These louvers can be individually hydraulically controlled to form a number of different visuals from a straight line of them to a potato chip shape.

Hydraulic and Control System

A total of (2) hydraulic power units will power the hydraulic louvers. Each power unit contains (2) 60HP electric motors driving Parker PDXXX variable displacement piston pumps for high speed movements and (2) 10HP electric motors driving Parker PDXXX for slower "sun tracking" movements of the louvers. Each of the power units contain a 300 gallon oil reservoir with atmospheric containment utilizing a Parker-Hannifin bladder type system that will expand and contract as the oil level inside the reservoirs changes from cylinder movement.

Rooftop prior to louver installation

The hydraulic power units are located rooftop at each end of the building. Each power unit powers 47 the 47 louvers closest to it with backup capabilities to drive all 96 louvers.

Two roof mounted hydraulic power units

Each louver is powered by its own hydraulic cylinder. The cylinders range in bore size from 3.25" to 8" and all have a stroke of 59.6". The cylinders "pull down" one end of the louver on a bracket that acts as a fulcrum to lift the opposite end of the louver into the sky.

The design challenge in the cylinder control circuit manifold was to provide stable, consistent operation of each louver axis, while also maintaining the safety and functional needs of the system. These cylinder-mounted manifolds provide bi-directional control and load-holding of the louver cylinders. A pair of proportional flow control valves, each sized uniquely for the desired speed range, regulate flow to each chamber of the cylinder. The valves are self-compensated and function as throttling valves to provide a pressure drop a act as a load-sensing external compensator.

This flow control is fairly resilient to both supply and load pressure variation, which is key in this application, given the large range of gravity- and wind-induced load forces and speeds.

For controlling flow out of the cylinder (louvers moving down), two counterbalance valves are in a "load hold and purge" or "cushion lock" configuration typical of boom and crane circuits. Ultra-restrictive counterbalance valves were selected to provide the proper flow-induced backpressures for this low-flow application, while also functioning as reliefs to limit the maximum pressure applied to or by the actuator, preventing buckling of the rod or overloading wind forces on the louver. Two anti-cavitation check valves provide make-up oil in case of overrunning loads.

Central Florida sees great weather fluctuation and the changes come fast. The louvers must retract fast even if power is lost. For this emergency operation, two "emergency-close" pilot-operated check valves in each manifold can be externally operated by an accumulator-power normally open valve circuit, which can be pressurized by the controller, by loss of power, or by a hand pump. One P.O. check valve, which has a manual override, opens to allow flow from the rod (loaded) end to the cap end through a fixed-setting flow control valve. The other P.O. check valve allows make-up oil to be sourced from the pressure line, through a fixed orifice, to the cap end of the cylinder. Together, these two can function to rapid-extend and actively power-down the louver.

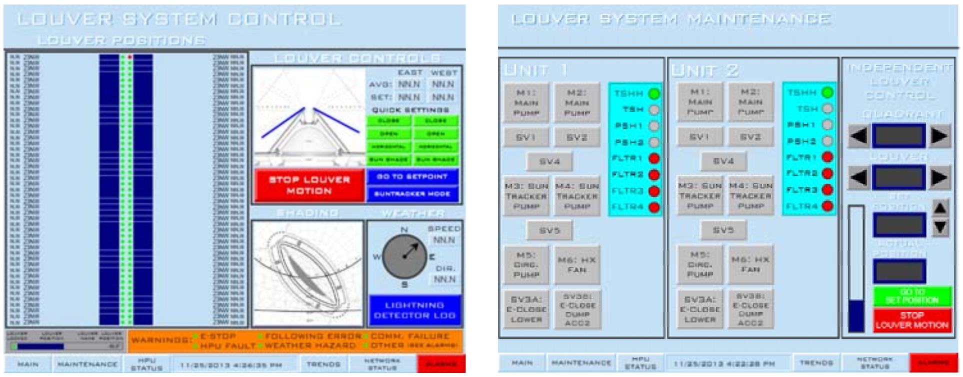
Hydraulic cylinders “pull” louver end over fulcrum to raise opposite end

The heart of the controls system is the Allen Bradley ControlLogix processor, which manages all control, feedback, and operator interface centrally. The controller closes the loop on all 94 axes with individual PID instructions to follow the a master virtual axis, which is set by user input on the HMI or run in an automatic "sun-tracking" mode. The controller also controls and monitors feedback from the two HPU's and the locking mechanisms on each axis.

Louvers can be programmed to any altitude configuration

The PLC is connected to an Ethernet device-level ring of I/O enclosure on the roof, each of which contain a remote I/O rack for command signals, valve driver cards, and an Ethernet-to-serial gateway for feedback. The analog command signal is sent from the remote I/O rack to off-board valve driver electronics, which are individually configured for each direction of each axis, to drive the proportional flow control valves. Louver position feedback comes from directly-mounted inclinometers. These inclinometers, which were designed for the solar power market, report absolute louver angles digitally over several serial busses with high accuracy and a sufficient update rate.

The control software allows for each louver to have its own motion profile which also allows for louvers to be staggered for various visual effects. The system can also be configured where the louvers will follow the sun to allow for optimal shade characteristics throughout the day.



Sample of diagnostic and operating screens on HMI

Operators are able to view diagnostics and louver positioning in real time and even "playback" events that have taken place to review for possible danger causing events.

The control system is also tied in with an anemometer and lighting detector. When set wind speed limits are detected the louvers set will automatically retract to a safe position. Even while retracted, wind forces can cause the louvers to want to "lift". Locking mechanisms are set below each of the louvers to capture and secure each louver's tip even during power loss emergency shut-down mode. Feedback from every one of the 94 locking mechanisms will confirm security of the louvers.

Locking mechanism

The university is set to open and will be welcoming students for fall semester 2014. All install and system commissioning will take place during the spring of 2014. Final testing to be completed by June 30^{th}.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Improving Spur Gear Designs with Long and Short Addendum Modifications

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NEW ORLEANS, LOUISIANA

NOMENCLATURE

Illustrations of gear nomenclature are provided in Figures 1 and 2.
Subscripts 1 and 2 are used to represent the pinion and gear, respectively

C_f - Surface condition factor
C_p - Elastic coefficient, $(lb/in^2)^{0.5}$
D - Pitch diameter, in
D_a - Outside diameter, in
D_b - Base diameter, in
F - Tooth face width, in
h_{ao} - Nominal tool addendum, in
I - Pitting resistance geometry factor
J - Bending strength geometry factor
K_B - Rim thickness factor
K_f - Stress correction factor
K_m - Load distribution factor
K_o - Overload factor
K_s - Size factor
K_v - Dynamic factor
m_p - Contact ratio
n - Number of teeth
P_{nd} - Diametral pitch, in^{-1}

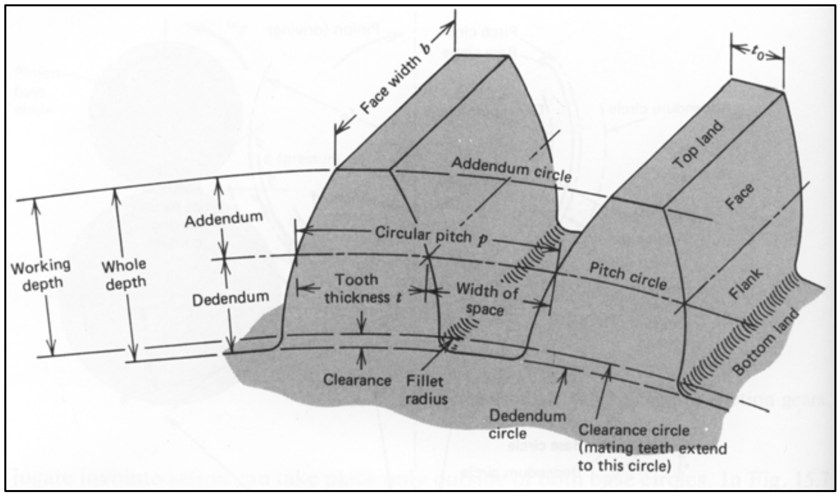


Figure 1. Gear tooth nomenclature. Reprinted from Spur Gear Modeling in Pro/E Wildfire 2.0/3.0. Retrieved from http://coewww.rutgers.edu/classes/mae/ mae488/hw/lectures/gear/gear.htm.

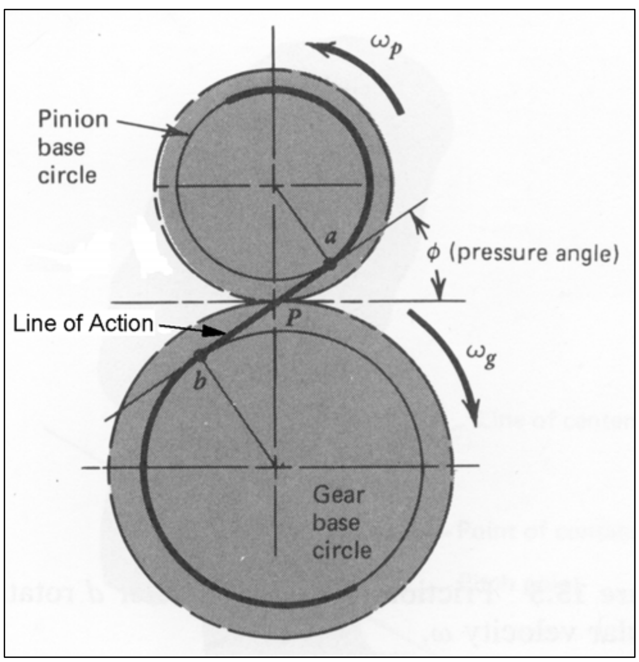


Figure 2. Base circle, line of action, and pressure angle. Adapted from Spur Gear Modeling in Pro/E Wildfire 2.0/3.0. Retrieved from http://coewww.rutgers.edu/ classes/mae/mae488/hw/lectures/gear/gear.htm.

p_b - Base pitch, in
R_b - Base radius, in
R_o – Outside radius, in
R_{ti} - Radius at which the involute profile starts at base of tooth, in
s_c - Contact stress number, lb/in^2
s_{na} – Tooth top land thickness, in
s_t - Bending stress number, lb/in^2
W_t - Transmitted tangential gear load, lb
x - Addendum modification coefficient
Y - Tooth form factor
Z - Active length of action, in
$Z_{approach}$ - Length of approach action, in
Z_{recess} - Length of recess action, in
Δs_{nl} - Tooth thinning for backlash, in
θ - Load pressure angle at the analyzed point of contact, radians
γ - Specific sliding
ρ - Radius of curvature at the contact stress location for the pinion, in
ρ_{ao} - Tool tip radius, in
Φ_n - Pressure angle, radians
Φ_r - Operating pressure angle, radians

Introduction

Figure 3. Swing bridge main pinion and gear.

Open spur gear pinions with lower numbers of teeth are commonly utilized in applications where bending strength is more important than surface durability/wear such as main pinions on movable bridges. In a speed reducing gear mesh, the driving gear with a lower tooth count is often referred to as the "pinion" while the driven gear is labeled the "gear." In general, a standard 20° full-depth involute spur gear with fewer than 18 teeth will exhibit undercut teeth. Undercut pinions are not ideal but are functional and can be found in older movable bridge machinery, especially on swing bridges. Figure 3 is a photograph of a swing bridge drive train with an undercut, 12 tooth, 20° full-depth involute main pinion that engages a pier mounted gear to rotate the span. Rehabilitation designs for this drive train included replacing the main pinion and gear along with a secondary pinion and gear set. One of the 13 undercut teeth on the secondary pinion, shown in Figure 4, fractured during an overload event. One main design challenge for the rehabilitation was to avoiding undercutting in the new gears while maintaining a similar center distance and gear ratio.

Figure 4. Fractured tooth on a 13 tooth, 20° involute, full-depth, undercut pinion.

Designs for new movable bridges often incorporate pinions with enough teeth to avoid undercut tooth profiles. Equation 1 determines the number of teeth on a spur gear based on the diametral pitch, P_{nd}, and the pitch diameter, D. The diametral pitch is inversely related to the tooth size. For rehabilitations, increasing the number of teeth requires decreasing the tooth size, enlarging the pitch diameter, or a combination of both. Space restrictions and gear ratio requirements often limit changes to the pitch diameters. If stronger materials are not effective to rate smaller teeth for the design loads, other methods must be investigated to avoid undercutting as was the case in the rehabilitation design of the gears in Figure 3.

$$\boxed{n = P_{nd} \times D}$$

Equation 1

Another method to avoid undercutting is to increase the pressure angle. Spur gears with a 25° pressure angle do not exhibit undercutting until the number of teeth is less than 12. Gears with higher pressure angles create higher bearing loads and operate with increased vibration and noise [1]. Gears with 20° pressure angles provide a good balance between tooth strength and load transfer and are specified/recommended by the American Association of State Highway and Transportation Officials [2] and the American Railway Engineering and Maintenance-of-Way Association [3]. Analysis in this paper is limited to spur gears with 20° pressure angles.

Lengthening the pinion addendum is a proven method of avoiding undercutting [1]. A long and short addendum gear pair is created by lengthening the pinion tooth addendum (decreasing the pinion dedendum) and shortening the gear tooth addendum (increasing the gear dedendum) by equal but opposite magnitude shifts. Long and short addendum gear pairs operate at the same center distance, pressure angle, and gear ratio as a standard gear mesh with the same diametral pitch and number of teeth, which is very beneficial for rehabilitation designs.

Many tooth properties and gear mesh operating parameters are affected by addendum modification changes to the tooth profile. Favorable strength or operating improvements can be achieved with addendum shifts beyond simply avoiding undercutting; however, adjustments also have the potential to negatively impact a design. Prior to implementing addendum modifications, a designer should have a good understanding of the ramifications.

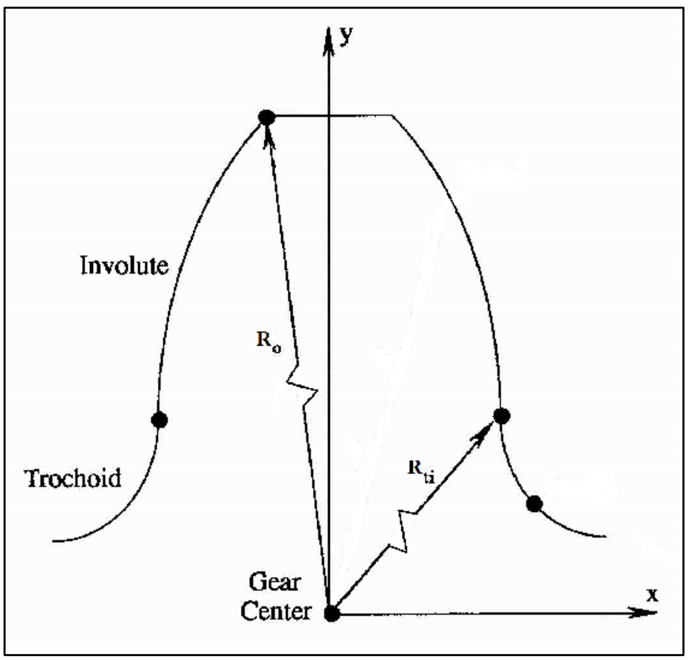


Figure 5. General pinion tooth profile. Adapted from Determination of Addendum Modification Coefficients for Spur Gears Operating at Non-Standard Center Distances, by M. A. Arikan. *Proceedings of ASME 2003 Design Engineering Technical Conferences and Computers and Information in Engineering Conference*, p. 4. Copyright 2003 by ASME.

The objective of this paper is to investigate the use of long and short addendum modifications to improve spur gear designs with standard center distances. This work includes providing an overview of long and short addendum modifications, highlighting important gear tooth properties and operating parameters, and demonstrating how their values are affected by addendum modifications. Discussion on gear design optimization using addendum modifications is also included.

Undercutting Phenomenon in Pinions with Low Number of Teeth

Undercutting is an unfavorable phenomenon that occurs in standard spur gear pinions with lower number of teeth based on the tooth geometry. Two curves make up the gear tooth profile. The involute curve comprises the working portion of the tooth while the trochoid curve forms the fillet portion at the base of the tooth, illustrated in Figure 5. Involute interference will occur, if the radius to the starting point of the involute curve is larger than the theoretical limit radius, which is the radius where the involute profile must start in order to make use of the tooth's full involute surface [4]. A pinion with involute interference would have extra material at the base of each tooth that jam against the gear tooth tips preventing proper meshing. Undercut teeth are formed when a pinion with involute interference geometry is generated by a hob or gear cutter. During fabrication, teeth on the generating cutter extend beyond the pinion's base circle removing the extra material at the root of each tooth. An undercut tooth profile is presented in Figure 6. While undercut teeth will mesh with a mating gear without binding, the tooth form is significantly weakened.

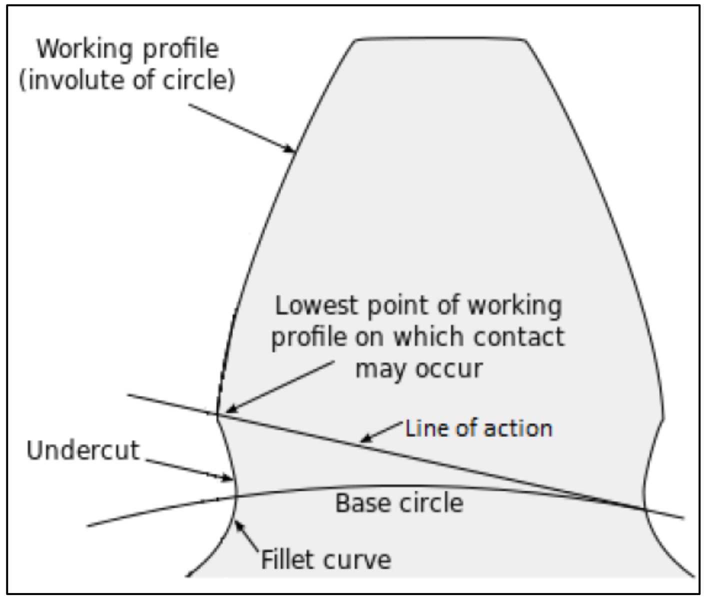


Figure 6. Undercut pinion tooth profile. Reprinted from File:Undercuts.svg. In *Wikimedia Commons* 2008. Retrieved from http://commons.wikimedia.org/wiki/File:Undercuts.svg.

The severity of undercutting depends on the generating cutter and gets worse as the number of teeth on the cutting tool increases [1, 5]. Rack type cutters (i.e. hobs, rack cutters, and generating grinding wheels) correspond to a cutter with infinite number of teeth and therefore generate pinions with the worst undercutting. In contrast, pinion cutters are gear generating tools with a lower finite number of teeth. Teeth cut by pinion cutters will not exhibit as much undercutting; however, there is potential for involute interference to remain, causing meshing problems. Gear analysis assuming results from rack type cutters is the most conservative.

Undercutting can significantly weaken gear teeth in several ways. Tooth stresses are highest at the root and increase as extra material is removed. Additionally, since the undercut profile is not involute, there is a loss of mating surface between the pinion root and gear tip resulting in a decrease to the length of tooth contact, thereby reducing the contact ratio corresponding to a decrease in the amount of tooth load sharing. Finally, the highest point of single tooth contact (HPSTC) is shifted up along the tooth profile contributing to further increases in the tooth root stress.

Long & Short Addendum Modifications

Standard generating tools can be used to produce the non-standard tooth profile of addendum modified gears [4]. Figure 7 depicts using a rack cutter to generate a gear. Non-standard tooth dimensions are created any time a gear is cut with the pitch line of the generating tool at a diameter other than the gear's standard pitch circle. Undercutting can be eliminated in a pinion with lower number of teeth by withdrawing the cutting tool from the gear blank until the full involute profile can be generated on the tooth. Withdrawing the tool from the gear blank, defined by a positive addendum modification coefficient ($x > 0$), creates a long addendum tooth. Alternatively, advancing the cutting tool into the gear blank, defined by a negative addendum modification coefficient ($x < 0$), generates a short addendum tooth. Since standard cutters are used, the gear tooth's whole depth remains constant. Therefore, increasing a tooth's addendum will proportionally decrease the dedendum, or vice versa. Varying amounts of addendum shifts are shown on the teeth of a 10 tooth pinion in Figure 8.

A long and short addendum gear pair consists of a long addendum pinion ($x_1 > 0$) and a short addendum gear ($x_2 < 0$) with equal but opposite addendum shifts. Since the sum of the addendum modification coefficients for this type of gear pair equals zero ($\Sigma x = x_1 + x_2 = 0$), the standard center distance, pitch diameters, pressure angle, and gear ratio will remain unchanged [4]. Many other tooth modifications are possible with the involute profile system. Gear pairs may be designed with their sum of addendum modification coefficients not equal to zero ($\Sigma x \neq 0$). One drawback to this type of design is that the gears will operate at a non-standard center distance, non-standard operating pitch diameters, and a non-standard operating pressure angle, which does not work well for rehabilitation designs on existing machinery. This paper focuses only on long and short addendum gear pairs with equal and opposite addendum modification coefficients ($\Sigma x = 0$).

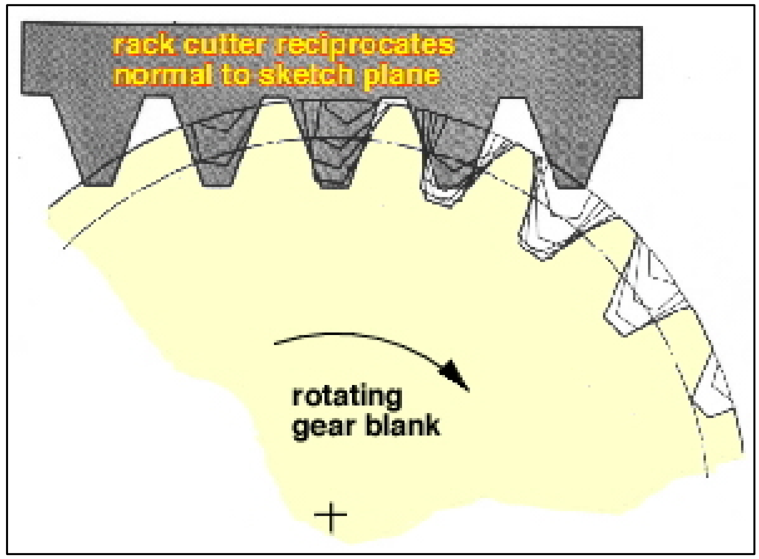


Figure 7. Gear generation with a rack type cutter. Reprinted from Gear Tooth Generation, by Douglas Wright, 2005. Retrieved from http://www-mdp.eng.cam.ac.uk/web/library/enginfo/textbooks_dvd_only/DAN/gears/generation/generation.html.

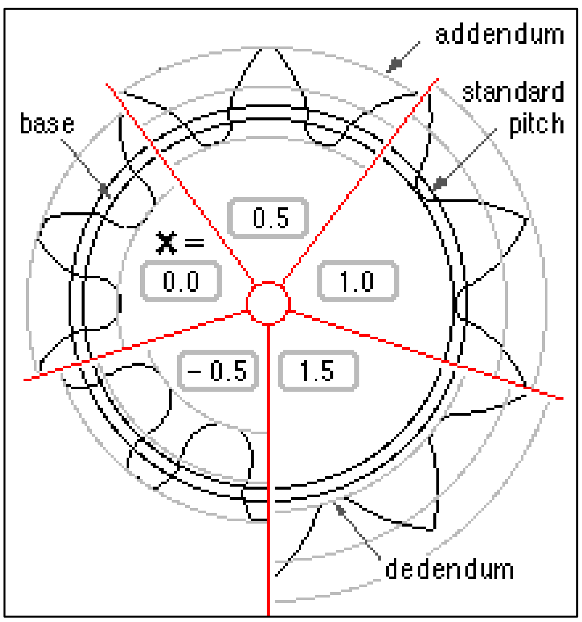


Figure 8. Pinion tooth profiles with increasing addendum modification. Adapted from Gear Tooth Generation, by Douglas Wright, 2005. Retrieved from http://www-mdp.eng.cam.ac.uk/web/library/enginfo/textbooks_dvd_only/DAN/ gears/generation/generation.html.

Required Addendum Modification to Prevent Undercutting

As discussed above, avoiding undercutting in a low tooth count pinion can be accomplished by lengthening the addendum. Equation 2 is used to determine the minimum addendum modification coefficient required to avoid undercutting in a pinion cut by a generating rack [6]. It is important to note that values for the nominal tool addendum, h_{ao}, the tool tip radius, ρ_{ao}, and the tooth thinning for backlash, Δs_{nl}, must be made dimensionless in this equation (multiplying by the diametral pitch) since all equations in AGMA's

Information Sheet 908-B89 are derived for a unity diametral pitch [6]. The actual dimension of generating rack cutter shift, in units of inches, can be determined by dividing the addendum modification coefficient by the diametral pitch.

$$x_{min} = h_{ao} - \rho_{ao}(1 - \sin\phi_n) - \frac{n}{2}\sin^2\phi_n + \frac{\Delta s_{nl}}{2\tan\phi_n}$$

Equation 2 – Addendum Modification Coefficient (Min) to Avoid Undercutting

Recommended Window of Allowable Addendum Modifications

Avoiding pinion tooth undercutting forms the lower bound of the recommended window of allowable addendum modifications. The allowable modification window has two potential upper bounds, avoiding inducing undercutting in the gear teeth and maintaining adequate pinion tooth top land thickness. A short addendum gear ($x_2 < 0$) that would otherwise not exhibit undercutting can become undercut if the generating cutter is advanced too far into the gear blank. Also, the thickness of a pinion's top land is decreased as the cutter is removed further from the blank (increasing x_1), until the pinion tooth tips become pointed. A pointed tooth tip has little capacity to resist loads and is easily over-hardened during heat treating processes. To avoid this, AGMA recommends a minimum top land thickness of 0.3 divided by the diametral pitch [6]. Undercut and pointed tooth profiles are illustrated in Figure 9.

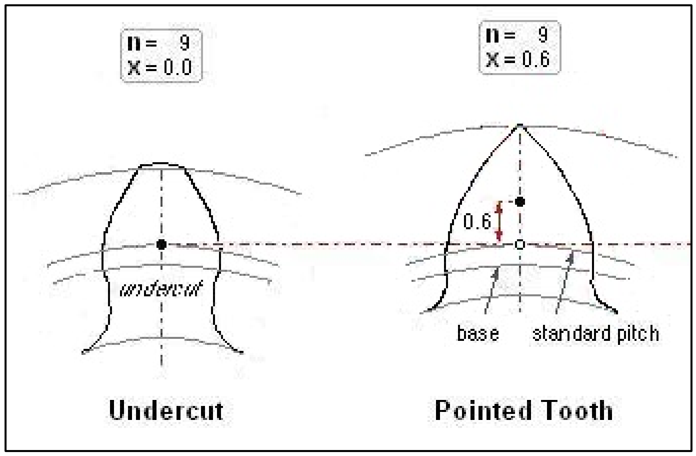


Figure 9. Undercut and pointed tooth profiles. Adapted from Reprinted from Gear Tooth Generation, by Douglas Wright, 2005. Retrieved from http://www-mdp.eng.cam.ac.uk/web/library/ enginfo/ textbooks_ dvd_only/DAN/gears/generation/generation.html.

Gear Tooth Properties and Operating Parameters

Determining the minimum addendum shift necessary to avoid undercutting is a good starting point for a design utilizing a pinion with a low tooth count; however, this modification often does not provide designs with the most desirable characteristics. Further improvements are possible with additional increases to the pinion addendum. It is important to note that addendum modifications can also achieve improvements to designs with pinions whose standard tooth profile is not undercut (i.e. 20° full-depth pinions with greater than 18 teeth).

Plots of sample calculations have been included, for each property or operating parameter discussed below, to demonstrate the effects of addendum modifications. Results are given for the analyzed gear pairs within the recommended allowable window between the lower bound of avoiding undercutting and the upper bound of maintaining a top land thicknesses greater than $0.3/P_{nd}$. Lines in each graph represent the general trends from calculated data, not actual curve fit trend lines. Plotted results for each property described in this section have been calculated for a 24 tooth pinion meshing with a 125 tooth gear with a unity diametral pitch, 20° pressure angle, and no thinning for backlash. Cutter geometry was assumed as a standard full-depth rack type cutter with a tool tip radius of 0.3". Additional plots are provided at the end of this document with similar results combined for 12 and 16 tooth pinions, each meshing with a 125 tooth gear.

AGMA Gear Sizing

According to the AGMA gear sizing methodology, criteria for surface durability (pitting resistance), bending fatigue strength, and overload yield strength dictate gear design requirements [7]. All three categories represent different failure modes and must be investigated individually.

AGMA makes use of geometry factors to consolidate aspects of the meshing gear teeth geometry affecting the rating criteria [7]. Evaluation of contact stresses for surface durability incorporates the pitting resistance geometry factor, I, and both the bending fatigue strength and overload yield strength equations utilize the bending strength geometry factor, J. This paper focuses on the impact of addendum modifications on AGMA rating equations through changes to the geometry factors.

Pitting Resistance Geometry Factor, I

$$\boxed{I = \frac{\cos(\phi_r)}{\left(\frac{1}{\rho_1} + \frac{1}{\rho_2}\right) \times D_1 \times m_N}}$$

Equation 3 – Pitting Resistance Geometry Factor, I

Compressive contact stresses between gear teeth are understood to cause the fatigue phenomenon of surface pitting [7]. The pitting resistance geometry factor, I, is a dimensionless number that accounts for aspects of gear tooth mesh geometry affecting contact stress, including the radius of curvature of the tooth surface at the contact point, gear tooth load sharing, and the normal component of the transmitted load [6]. A mathematical procedure to calculate the I factor was first introduced in AGMA's 229.06 Standard, based on the analysis of two cylinders in contact according to the Hertzian theory [6]. AGMA provides Equation 3 to calculate the I factor for external spur gears, which was simplified from the original version yet numerically equivalent [6]. The operating pressure angle, Φ_r, is the same as the standard pressure angle, Φ_n, for long and short addendum gear pairs with equal and opposite shifts. The I factor is evaluated at the pinion's lowest point of single tooth contact (LPSTC), which is considered to be the most critical contact stress location.

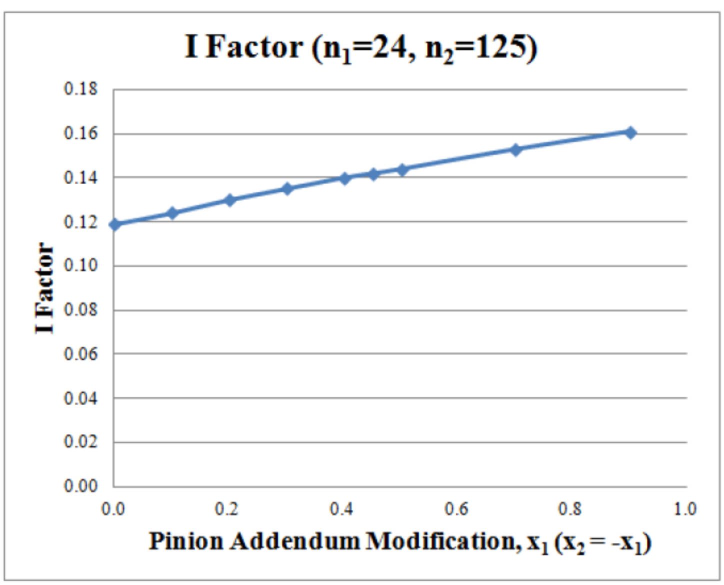


Figure 10

Contact stress and the likelihood of pitting are decreased with positive addendum shifts to the pinion in a long and short addendum gear mesh. This trend is shown in Figure 10. The pinion tooth radius of curvature at every contact point is lengthened with larger pinion addendums, thereby increasing the pitting resistance geometry factor. The contact stress number, calculated by Equation 4, is used in AGMA's surface durability rating equation where the contact stress is inversely proportional to the square root of the pitting resistance geometry factor [7].

$$s_c = C_p \sqrt{W_t K_o K_v K_s \frac{K_m}{D_1 F} \frac{C_f}{I}}$$

Equation 4 – Contact Stress Number

Bending Strength Geometry Factor, J

The bending strength geometry factor, J, is a dimensionless number used to calculate tooth root stress [6]. The J factor is affected by the shape of the tooth, the worst loading position, stress concentrations, and bending and compressive tooth loads. If load sharing exists between adjacent teeth, the J factor is evaluated at the HPSTC. If load sharing is not present due to variations in the tooth form or center distance, the J factor is calculated for a tooth tip loading condition.

$$J = \frac{Y}{K_f}$$

Equation 5 – Bending Strength Geometry Factor , J

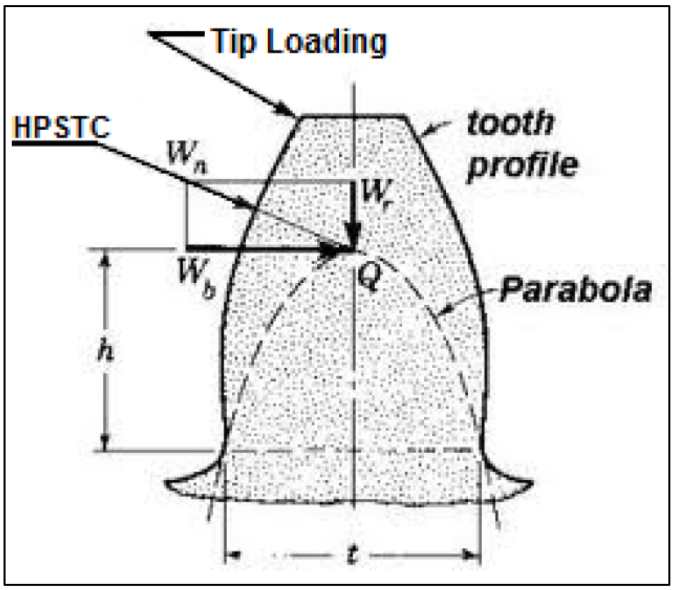


Figure 11. Lewis parabola used in determining bending strength of a gear tooth. Retrieved from http://web.itu.edu.tr/~fetvacic/femgear/modeles1.htm.

AGMA provides a semi-analytical method to determine the J factor, based on Equation 5 for spur gears [6]. This method incorporates addendum modifications and involves numerical algorithms to determine the point of tangency between the Lewis parabola and the tooth root profile, which is the location of highest tooth bending stresses [1]. An illustration of the Lewis parabola is provided in Figure 11. AGMA's Information Sheet includes tables of calculated I and J factors for select gear pairs with 20° or 25° pressure angles, helix angles ranging from 0° (spur gears) to 30°, and equal but opposite addendum modification coefficients of $x_1 = -x_2 = 0.0$, $x_1 = -x_2 = 0.25$ and $x_1 = -x_2 = 0.50$ [6]. Geometry factors given in these tables can be used to simplify the strength analysis of long and short addendum teeth. AGMA specifically warns against the use of interpolation with results in these tables [6].

$$s_t = W_t K_o K_v K_s \frac{P_{nd}}{F} \frac{K_m K_B}{J}$$

Equation 6 – Bending Stress Number

Bending stresses can cause fatigue failure in the form of cracks at the tooth root fillet. AGMA rating criteria for bending strength seeks to avoid root fillet cracking for the design life of the gear using Equation 6 to calculate the bending stress number [7]. The bending stress number is inversely proportional to the bending strength geometry factor.

In most gear sets with standard tooth profiles, higher bending stresses are experienced by the pinion compared to the gear, due to differences in tooth geometries and evident by lower pinion J factor values. As the pinion addendum is lengthened, the bending strength geometry factor is increased for the pinion and decreased for the gear, as shown in Figure 12. Addendum modifications are effective for improving the pinion's geometry to obtain greater resistance to fatigue cracking resulting from bending loads, which is also beneficial since the pinion teeth experience more load cycles over the application's lifetime.

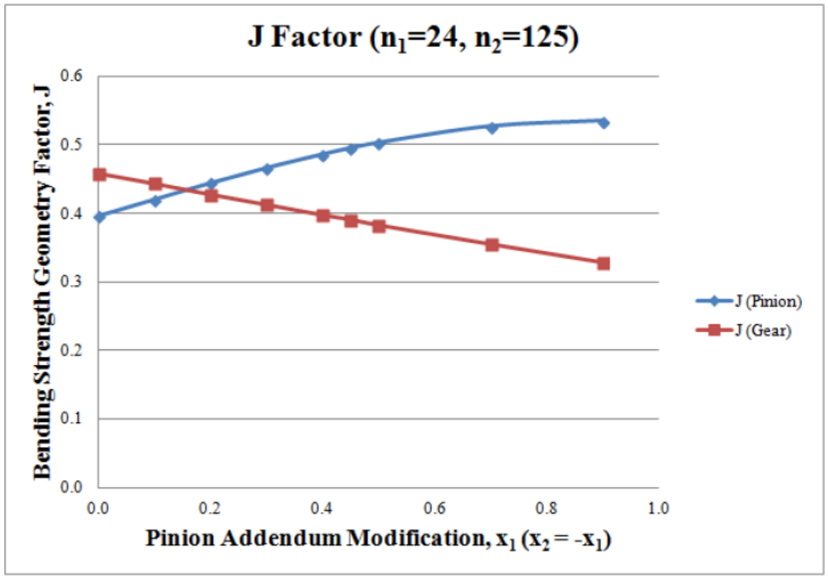


Figure 12

$$\boxed{m_p = \frac{Z}{p_b}}$$

Equation 7 – Contact Ratio

Contact Ratio

In a typical gear tooth operation sequence, contact will initiate between the pinion tooth base and mating gear tooth tip while the adjacent pair of teeth remains in contact. Load is shared between adjacent teeth until contact on the first tooth reaches the end of the line of action and separates from the mating gear. At this point in the operation, contact on the second pinion tooth is located at the lowest point of single tooth contact (LPSTC). The full driving force is carried by this one tooth as contact continues up the pinion involute profile. Once the contact point reaches the highest point of single tooth contact (HPSTC), load sharing begins as the next pair of teeth initiates contact. The combined force distribution applied to a tooth during contact is depicted in Figure 13. Contact ratio is a value representing the average number of pinion teeth in contact with the mating gear during operation. The higher the contact ratio, the more load sharing exists and the less one tooth must transfer the entire driving force. The contact ratio is determined by Equation 7 [6].

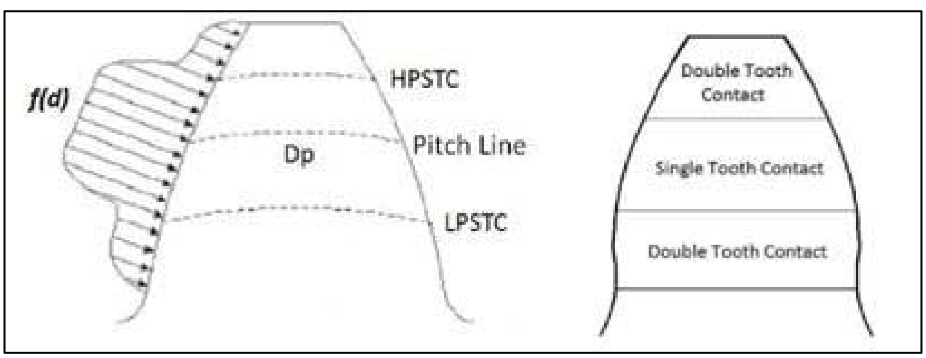


Figure 13. Distribution of force experienced by tooth due to load sharing. Adapted from The Influence of Contact Stress Distribution and Specific Film Thickness on the Wear of Spur Gears during Pitting Tests by M. A. Muraro, F. Kado, U. Reisdorfer, C. H. Silva, *Journal of the Brazilian Society of Mechanical Sciences and Engineering*, 2012. Retrieved from http://www.scielo.br/scielo.php?script=sci_arttext&pid=S1678-58782012000200005.

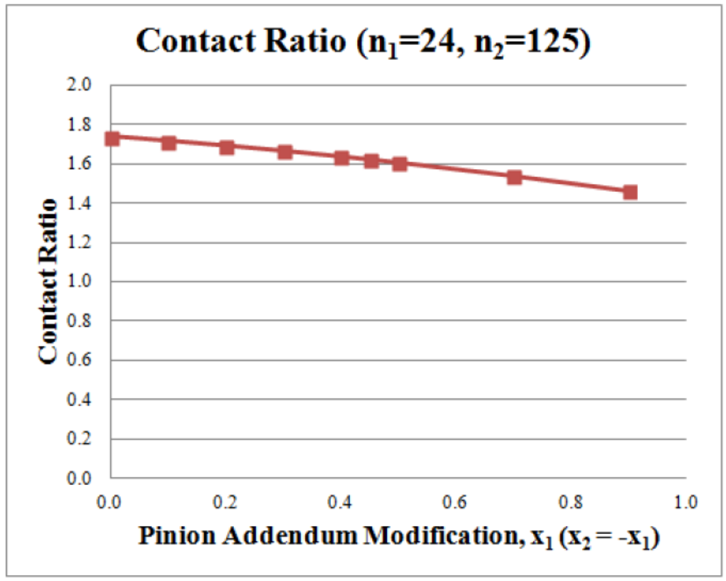


Figure 14

As seen in Figure 14, contact ratio is highest with zero addendum modification (or the minimum required to avoid undercutting in other gear sets) and declines with increased pinion addendum modifications. This creates a trade-off that a designer must weigh against other benefits obtained by lengthening the pinion addendum. Usually, the advantages to other parameters resulting from pinion addendum increases outweigh the disadvantages of contact ratio loss [5].

Approach and Recess Gearing Action

The length of action is split into two portions, approach action and recess action. Contact with a mating gear initiates on the involute profile at the base of the driving pinion tooth. Approach action occurs from this point until the point of contact reaches the tooth pitch line. Recess action takes place as the point of contact travels from the pitch line to the pinion tooth tip. In other words, approach action takes place during contact along the driving pinion dedendum (driven gear addendum) and recess action occurs with contact in the driving pinion addendum (driven gear dedendum).

A combination of rolling and sliding comprise the surface contact between two mating gear teeth. Rolling velocity remains nearly constant along the tooth profile, directed towards the pinion tooth tip. Contact at the pitch line consists of pure rolling with no relative sliding. During approach action, the rolling and sliding velocities are opposed with the sliding velocity directed toward the pinion tooth root, increasing operational roughness and the likelihood of pitting and scuffing [1]. During recess action, rolling and sliding velocities are oriented in the same direction, toward the pinion tooth tip, providing smoother operation. Increasing the smooth recess portion of the line of contact, thereby decreasing the harsh approach action, is beneficial in gear design. Equations 8 and 9 provide the calculation for the length of approach and recess action, respectively [5]. Dividing these values by the base pitch provides a dimensionless number representing the approach and recess portions of the contact ratio.

$$\boxed{Z_{approach} = \frac{\sqrt{D_{a2}^2 - D_{b2}^2} - D_2 \sin(\phi_n)}{2}}$$

Equation 8 – Length of Approach

$$\boxed{Z_{recess} = \frac{\sqrt{D_{a1}^2 - D_{b1}^2} - D_1 \sin(\phi_n)}{2}}$$

Equation 9 – Length of Recess

The favorable recess portion of the line of action is lengthened with greater pinion addendum shifts, while simultaneously decreasing the harsh approach action, see Figure 15. Some designers have investigated designs with gears that operate entirely in recess action [1]. This is accomplished by lengthening the pinion addendum 200 percent and eliminating the gear addendum, which would likely cause pointed pinion teeth or induce gear undercutting. As such, this configuration is not used in power transferring applications. Also, increasing the recess action is also often limited in application where coasting or overhauling/braking conditions may occur. In these situations, the gear drives the pinion reversing the recess and approach actions.

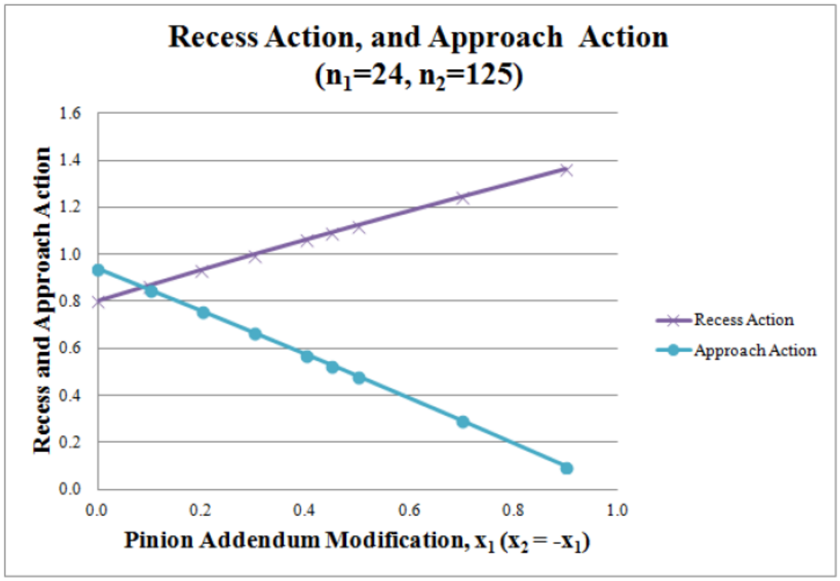


Figure 15

Specific Sliding

Pure rolling between tooth surfaces takes place when contact occurs at the pitch diameters of two mating gears. A combination of rolling and sliding action comprise the surface contact at all other areas above and below the pitch line. Contact locations furthest from the pitch line encounter the highest relative sliding velocities [1]. These maximum velocities occur where the pinion root first comes in contact with the gear tooth tip and where the pinion tip ends contacts with the gear root [1]. Specific sliding, γ, is the term used to quantify relative sliding velocities and is directly impacted by pinion and gear tooth geometries [8]. The specific sliding value can be determined from Equation 10 for contact locations below the pinion pitch line and by Equation 11 for contact locations above the pinion pitch line [8].

$$\gamma = \frac{R_{b1}}{R_{b2}} \frac{R_{b2}\tan(\theta_2)}{(R_{b1}+R_{b2})\tan(\phi_n) - R_{b2}\tan(\theta_2)} - 1$$

Equation 10 – Specific Sliding "Velocity"
(for contact below pitch radius)

$$\gamma = \frac{R_{b2}}{R_{b1}} \frac{R_{b1}\tan(\theta_1)}{(R_{b1}+R_{b2})\tan(\phi_n) - R_{b1}\tan(\theta_1)} - 1$$

Equation 11 – Specific Sliding "Velocity"
(for contact above pitch radius)

Relative sliding during gear mesh contact contributes to tooth scuffing damage [9]. AGMA uses the term "scuffing" to describe what is often referred to as "scoring" in other resources, which is damage caused by the welding and tearing of small areas on the tooth surface resulting in metal transfer between mating gear teeth, indicated by radial scratches on the tooth surface [9, 10]. An example of scuffing is shown in Figure 16. Once scuffing has advanced, it often progressively increases in severity. As scuffing increases, so does gear mesh operating

Figure 16. Gear tooth scuffing.

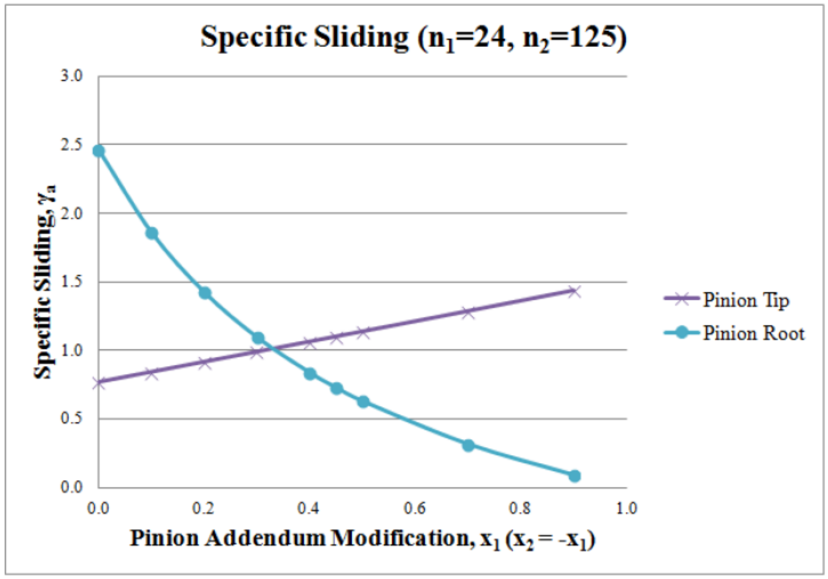


Figure 17

noise, vibration, and dynamic loads, which may induce other forms of tooth failure.

Scuffing is not considered to be a fatigue phenomenon and may occur instantaneously with little or no prior indications [7]. Many factors are understood to affect scuffing, including sliding velocity, lubrication viscosity and additives, operating temperatures, surface finish, metal properties, surface pressures, geometric errors, and dynamic loads [7]. Minimizing sliding velocities is one way to minimize the risk of scuffing. Continuing research is seeking to more accurately reduce the risk of scuffing [9].

In gear pairs with zero addendum modification or the minimum shift to avoid undercutting, specific sliding at the pinion tooth root is generally much higher than at the pinion tooth tip. Compressive stresses are highest at the pinion root causing sliding in this region to be more harmful than at the pinion tip [1]. Figure 17 reveals how specific sliding values at the pinion decrease rapidly as the pinion addendum shift is increased. Greater specific sliding develops simultaneously at the pinion tip; however, much less dramatically. These modifications can aid in the avoidance of tooth scuffing and pitting.

Design Optimization with Addendum Modifications

Addendum modifications impact gear tooth properties and operating parameters in different ways. No addendum modification allows for maximizing all beneficial parameters. Therefore, design optimization is not clearly defined and often varies depending on the application. This section presents several methods used to establish optimum designs using addendum modifications, including balancing static tooth strength, balancing dynamic tooth strength, and equalizing specific sliding. Instead of optimizing one parameter, a designer may also select to use an addendum modification that provides a favorable compromise for several criteria. It should be noted that there are many other factors besides addendum modifications that affect a given gear design and can be adjusted for optimization purposes, e.g., material properties, hardening methods, generating cutter details, etc. Investigation into these other factors is outside the scope of this paper. Ultimately, it is up to the gear designer to select the addendum modification which will produce the best results for each application.

Balanced Static Pinion and Gear Tooth Strength

In a standard tooth profile gear mesh, the pinion tooth geometry is generally weaker in bending than the mating gear, assuming materials properties are the same [11]. To correct this, researchers have studied ways to equalize tooth strength. By analyzing gear teeth as a cantilevered beam using the Lewis equations, Mabie Walsh, and Bateman [12] proposed a method to determine the required addendum shift to equalize static stress experienced by the pinion and gear teeth. Equalizing the pinion and gear bending strength geometry factors accomplishes a similar goal.

Balanced Dynamic Pinion and Gear Tooth Strength

During actual gear operations, dynamic loads have been found to be much higher than static loads, especially at high speeds [13]. Accounting for this, Liou, Lin, Oswald, and Townsend [13] conducted dynamic load studies to determine the best addendum shifts to balance dynamic strength between pinion and gear teeth. Their research focused on long and short addendum modifications with equal and opposite shifts. For gears operating at high speeds, they recommended maximizing the recess action of the gear mesh, thereby decreasing dynamic loads by providing the smoothest possible operation. This shift is often limited by the upper bound of avoiding pointed pinion teeth. Dynamic loads were not as significant for gears operating at lower speeds and their results at these speeds matched similar findings for addendum modifications to balancing static tooth strength.

Balanced Specific Sliding

Another optimization method is to shift the pinion addendum such that specific sliding is equal at the pinion tooth root and tip. Pedrero and Artés [8] have developed an analytical method to estimate the required addendum modifications for balance specific sliding. Their method determines the relationship required between the pinion and gear addendum shifts to balance specific sliding, which may then be applied for long and short addendum gear pairs ($\Sigma x = 0$).

Optimization Compromises

Instead of focusing on one parameter, another optimization approach is to find an addendum modification that offers a favorable compromise. Two such compromises are proposed in Dudley’s Handbook of Practical Gear Design and Manufacture [1]. Each compromise assumes long and short addendum shifts of equal and opposite magnitudes.

The first suggested compromise (Compromise A) recommends small addendum modifications to help balance pinion and gear tooth strength while also decreasing potential for pitting and scoring [1]. This recommendation is intended for power transmitting gears with a driving pinion and since only a modest amount of addendum shift is proposed, the generated gears operate acceptably in reverse, which is beneficial for applications where coasting situations can occur [1]. The second compromise (Compromise B) is based on experimental findings and attempts to provide a favorable balance between strength, sliding and scuffing resistance [1].

Addendum modification values suggested for Compromise A have been developed for gear sets with 12, 16, and 24 tooth pinions, whereas Compromise B suggested values are given for all gear sets and is determined by the gear ratio [1]. The amount of shift to achieve each compromise is plotted in the combined sample calculation graphs as vertical lines.

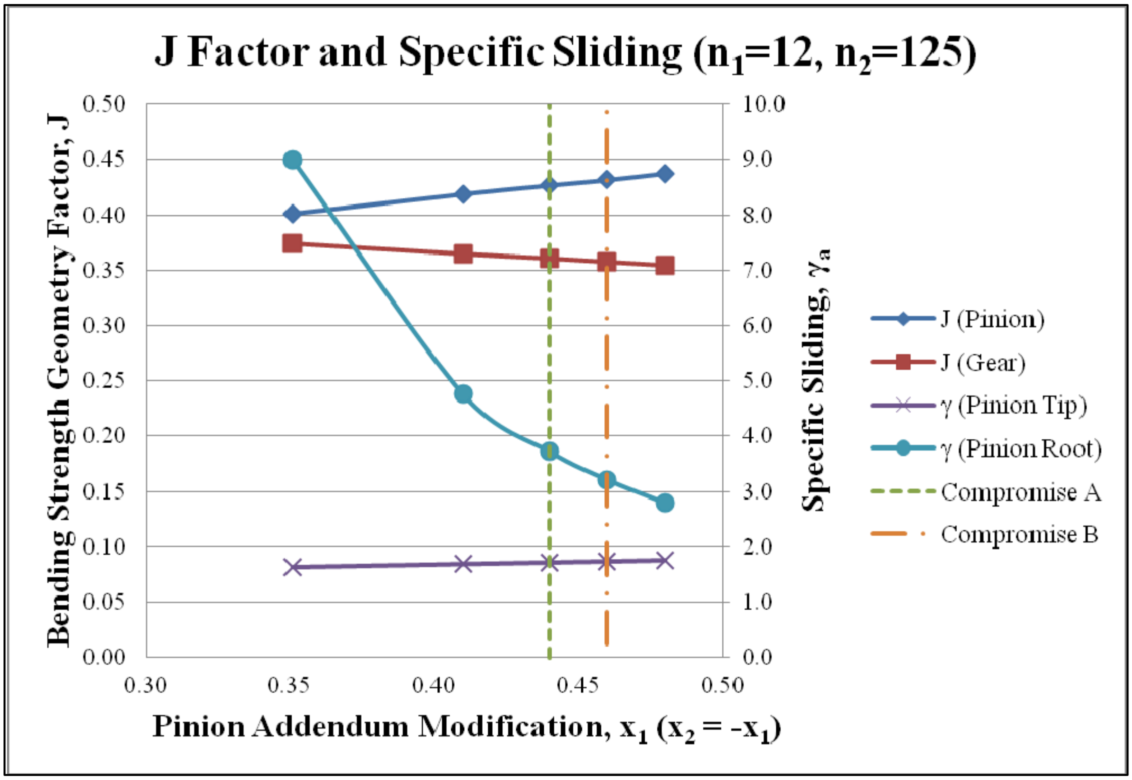
Conclusion

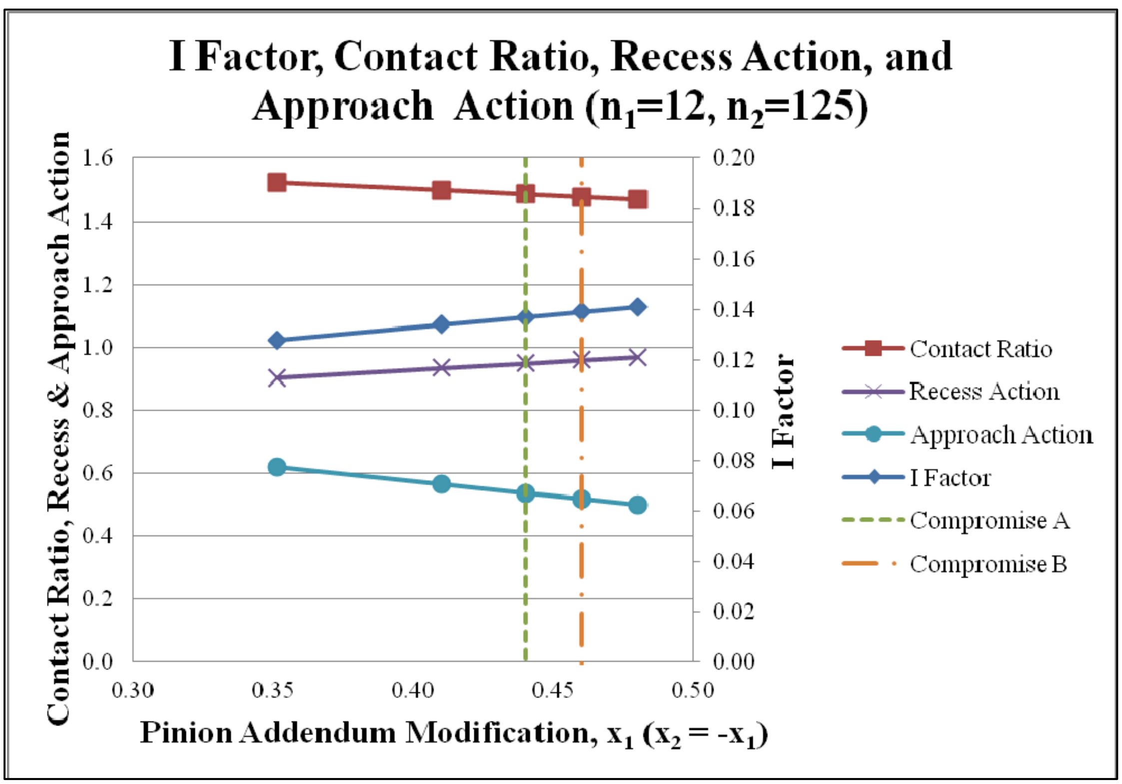
Modifying gear teeth addenda is a powerful tool for gear designs, providing means to improve standard tooth configurations and avoid undercutting in pinions with low number of teeth. This paper highlights the limitations of addendum shifts governed by avoiding undercutting and pointed teeth as well as the gear characteristics affected by such modifications, including AGMA's pitting resistance and bending strength geometry factors, contact ratio, approach and recess action, and specific sliding.

There are many potential methods to optimize designs. Ultimately, the designer must choose the amount of addendum modification that is best for each application. Further research must be conducted to finalize the optimum shift for movable bridge machinery applications; however, a modification similar to Compromise A may be best suited due to the improved wear characteristics and better balanced pinion and gear tooth strengths for power transmitting gear sets that must maintain satisfactory coasting and overhaul/braking capabilities. The analysis in this paper is focused on geometrical changes resulting from addendum modifications and does not account for material differences between the pinion and gear. Since the pinion teeth experience more operating cycles than the gear, it is common to fabricate the pinion from a harder, stronger material. Future analysis to determine the optimum long and short addendum shift for movable bridge applications must incorporate unequal material strengths.

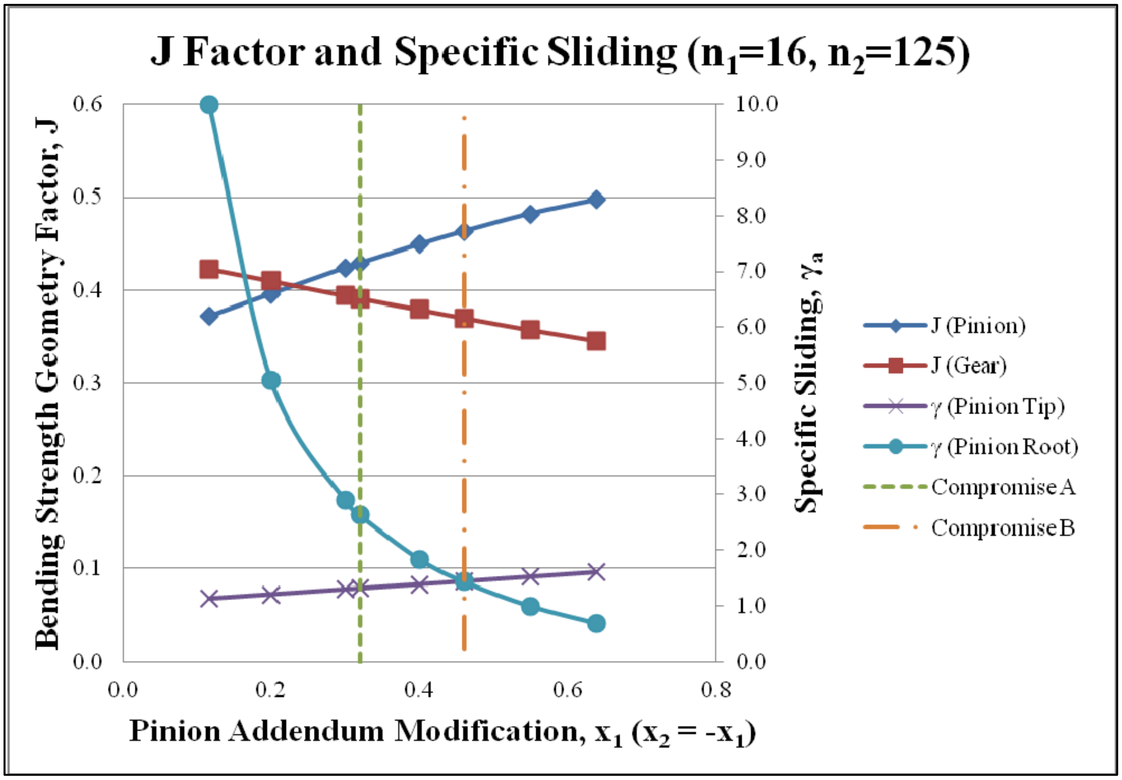
Combined Sample Calculations

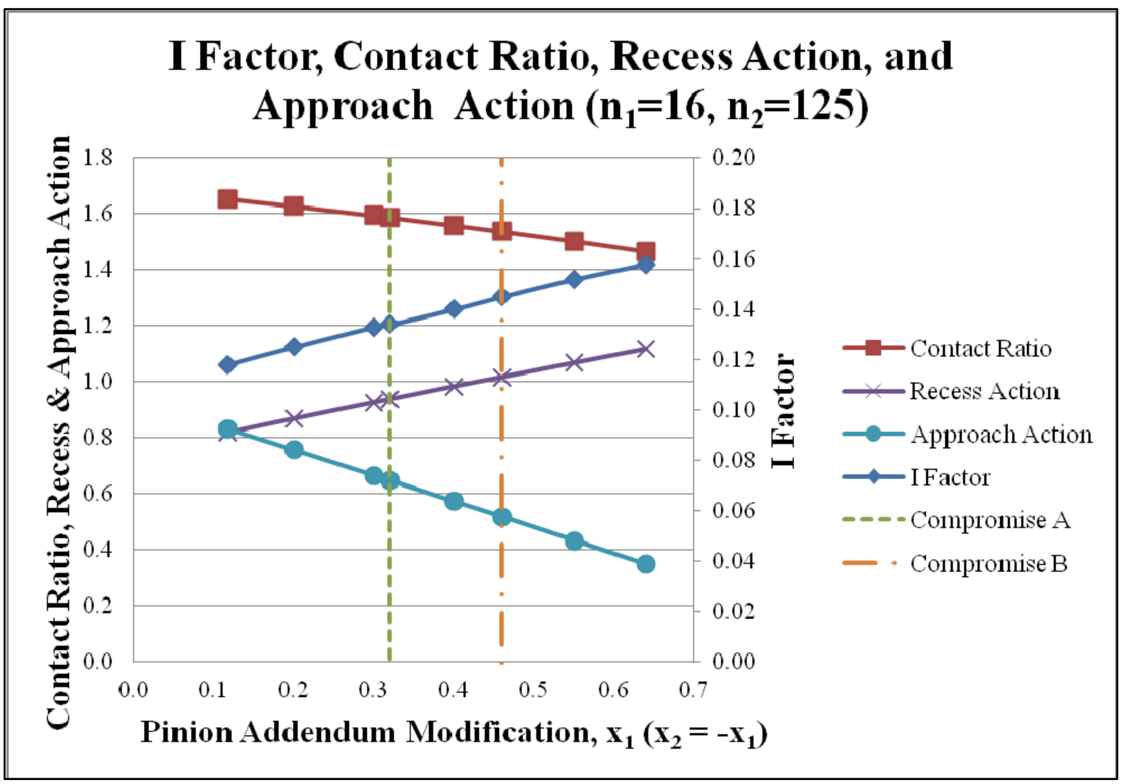
12 Tooth Pinion and 125 Tooth Gear



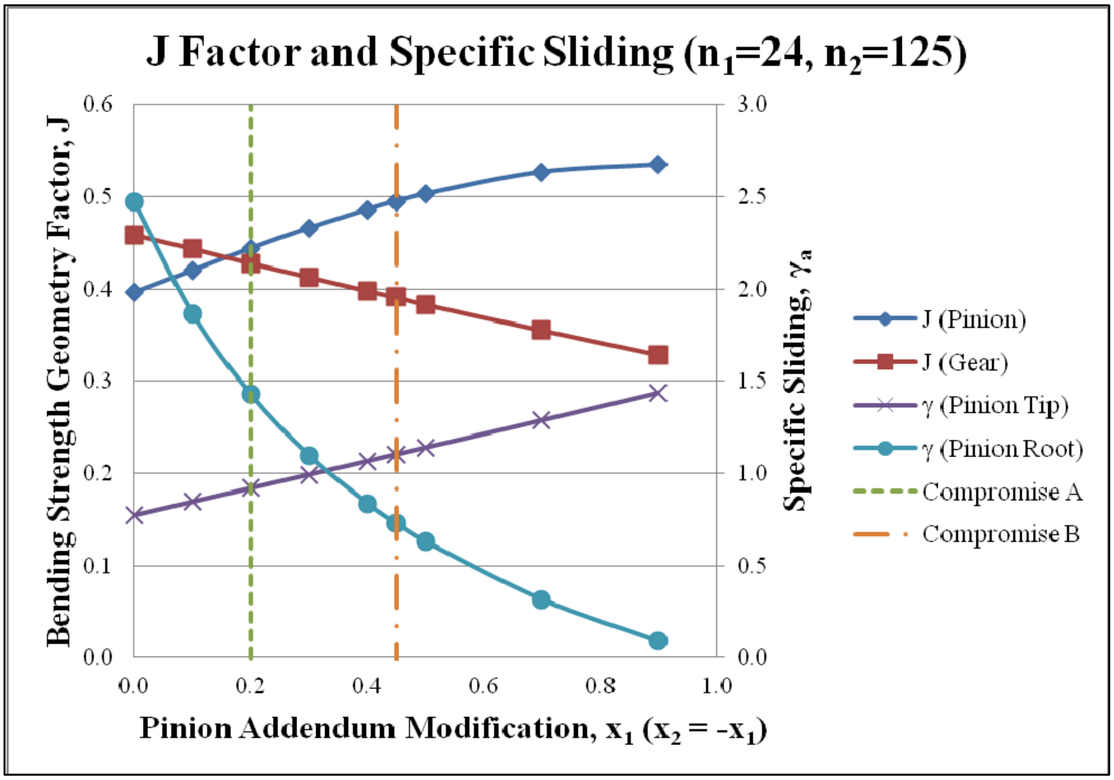


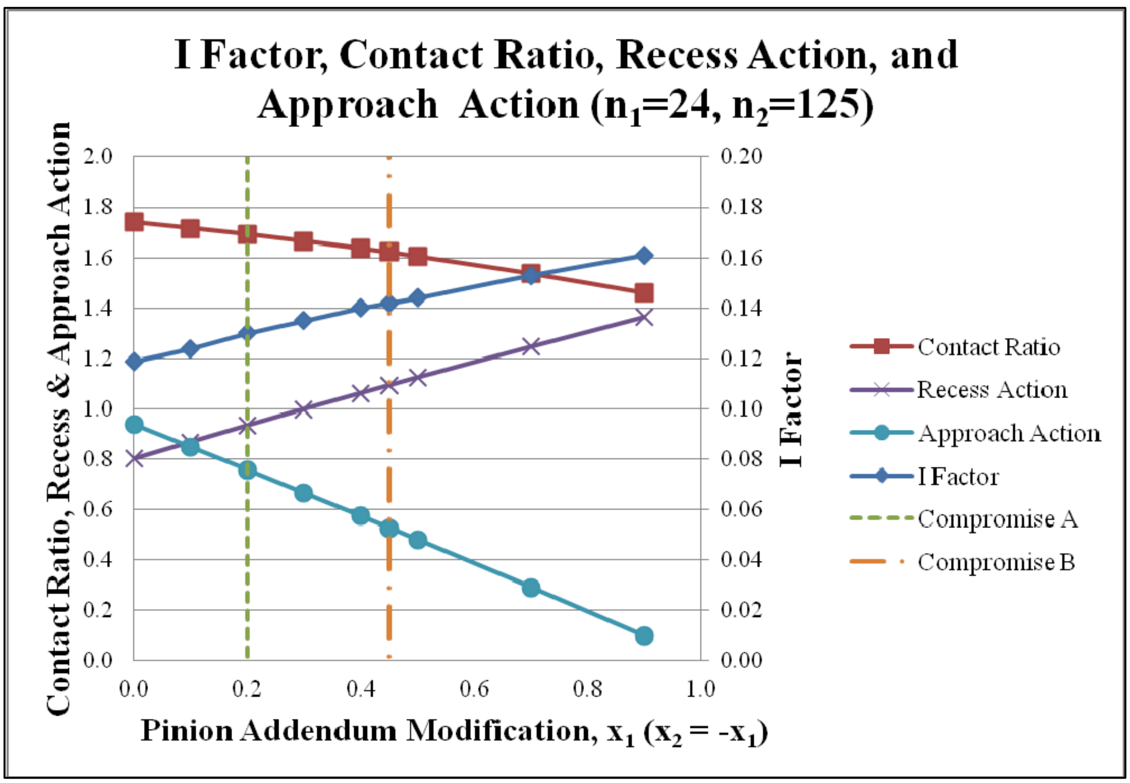
16 Tooth Pinion and 125 Tooth Gear





24 Tooth Pinion and 125 Tooth Gear





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HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Mechanical Details in Detail

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HMS 2014

Mechanical Details in Detail

This paper explores examples of typical details found in most movable bridge machinery installations. The current standard specifications, and shop pactices are discussed for example keys, keyways, turned bolts, force fits, and other mechanical features that are encountered in the drafting and construction of movable bridges. Often these seemly minor details create major problems for the detailer, machine shop and, if not done properly, the owner. There are numerous interpretations of the industry, movable bridge and railroad standards that do not become evident until a project is well underway. The result is usually an unnecessary delay to sort out what the owner and engineer want vs. the common practice and capabilities of the shop doing the work. The objective of this paper is to highlight some of the issues and conflicts and illustrate some examples that should provide at least a template for avoiding delays in drawing approvals and shop work.

Turned Bolts:

The current AASHTO standard gives the requirements for fasteners, turned bolts and nuts in the same paragraph, 6.7.15. It basically says that bolts should conform to ASTM A325 and that turned bolts have a shank 1/16 larger than the tread major diameter.

Turned bolts are not standard items. They are made to order for the job. The dimensions and finishes are shown on the shop drawings. A turned bolt can be made in several different ways. A standard bolt can be used and the treads cut off, A “blank” bolt can be made, or hex bar can be used. The most common approach is to use a blank bolt. This is a forged bolt with a head but no threads. The most common have a head sized in proportion to the blank and are fairly commonly available. Special blanks with non standard head sizes can be obtained but are usually made to order and have longer delivery times. Occasionally a requirement for fastener head sizes to be the same as the nut ends up in a job that uses turned bolts. This requirement is impractical when turned bolts are involved. Special blanks would have to be made with heads smaller than the standard for the body size. As mentioned above, this increases delivery times and cost. Furthermore it creates a bolt with less than normal head bearing area. While the lower area may not be a problem, the requirement should not be applied to turned bolts.

For particularly large diameter or long shanks, the blank size may need to be substantially larger than the final thread size to assure that the full length of the body cleans up in the machining process. AASHTO 6.7.15 states that the blank size is “usually” 1/8 inch larger than the thread size. As an example, for a one inch treaded sized turned bolt, the shank would finish to 1-1/6 in diameter per the specification. Therefore if the blank is 1-1/8 inch diameter, the machine stock on the shank would be only 1/32 inch per side. For a cap bolt in a pillow block or a bolt holding a rack to a rack support, the shank might be over 12 inches long. There would be a high probability of the blank being slightly bent or bowed or have a local scratch or blemish that would not machine completely out if only 0.03125 inches of stock is allowed. The designer must keep this in mind when reviewing drawings. A shop may opt to allow more stock on the blanks to avoid scraping blanks or sending them back if they are not perfect.

The other consideration is that larger blanks means larger heads. Both the designer and the fabricator/detailer need to check for wrench and head clearance and account for heads that in some cases will be substantially larger than the nut size. Few manufactured components are made with oversize heads in mind. Electric motors and standard manufactured gearboxes and brakes rarely have sub-drilled or under sized holes in them. Therefore to mount them with turned bolts, the shop must ream the production holes to a larger size. The result is the heads of the turned bolts will be much larger than what the manufacturer of the motor or brake intended. Sometimes the head is too big for a spot face or weld clearance or wrench access is limited. This issue is one that can cause delays late in the job and at the worst possible time.

As stated above, few manufactured items, electric motors in particular, are made with in place reaming of mounting holes and installation of turned bolts in mind. Motors all have standard frame sizes including mounting feet and hole sizes. Few, if any motors have feet that extend beyond the frame for reaming from above. Even if the support is fabricated without holes, the motor must be placed, holes transferred and then removed to drill and ream the holes separately. It is possible in some situations to ream from below. The process is time consuming and error prone.

Many industrial applications simply drill dowel pin holes through the base of the motor and support after alignment. The dowel holes can be drilled on an angle if necessary to prevent having to move the motor off the support after alignment.

AASHTO Table 6.7.8-1 gives the fits and finishes required for various mechanical elements. The fit for a turned bolt in a finished hole is LC6. AREMA Table 15-6-5 calls for the fit of turned bolt in a hole to be LT1. Assuming that most turned bolts will be between ½ and 3 inch in diameter, the LC6 total fit is as little as 0.0006 in clearance for the ½ inch bolt to as much as 0.006 inches clearance for the big 3 inch bolt. While the ANSI standard is scaled somewhat by diameter, there is no mention in any of the standards of any variations in the fit due to length. The considering the tolerances of the bolt, the hole, the depth of the hole, the possible limitations and constraints on the access for assembly, these are very difficult to install. Now look at the AREMA LT1 fit for the same sizes, ½ inch is 0.0002 INTERFERENCE to a maximum of 0.0015 inch clearance for a 3 inch diameter turned bolt! These types of fits are extremely costly and time consuming to make. The effort and time and cost must be weighed against the benefits.

Neither of the specifications take into account the length of engagement of the turned bolts or the number of turned bolts involved in a particular connection. A bascule rack segment mounted in a rack support has historically been mounted with many turned bolts. The bolts often extend through the support plates and the entire width of the rack. The bolt lengths are quite long and on the order of 12-16 inches. There are usually bolts the entire length of the segment sometimes in a staggered pattern totaling 50 – 100 bolts in the connection. The active body fit of the turned bolt is only the ends of each bolt where it goes through the support plates and the first inch or so of rack material. Is it necessary to hold the LC6 or LT1 fit the entire length of the bolt? No. Is the bolt drawing going to be approved if it has different tolerances in the middle? Maybe. With 50 bolts in one connection with about 0.001 inch of “play” in a 12 inch long hole the statistical likelihood of the rack being able to move is nearly impossible. The likelihood of getting three or four of the bolts stuck is highly probable. Is the cost and time required to meet the spec in this application really necessary? No.

Given the hole clearance discussion above, hopefully it is obvious that turned bolts should never, ever be used as cap screws where the bolt has to be rotated from the head to go into a threaded hole with a LC6 or LT1 tolerance.

Often turned bolts get treated as structural bolts by misapplication of general standard specifications. Typical specs that are applied are; thread stick through maximums, torque values, thread length, threads in the grip, coatings, head stamp requirements, washer requirements, and ro-cap tests. None of these specifications should be applied to turned bolts. Turned bolts are typically used in shear connections for mounting machinery elements. Shims are almost always used in the grip. It is impossible to make one turned bolt fit every combination of shims if there are certain maximum thread stick through requirements or conversely a prohibition of threads in at least a portion of the grip. It is impossible to make a turned bolt meet the thread length dimensions of an A325 bolt and still have enough thread to account for most shim combinations and double nuts. If turned bolts are required to be tensioned or torqued, then the design engineer needs to provide the values in the contract documents and the method to be used to accomplish the desired tension. Typical turned bolts with larger bodies and heads don’t fit in a standard Skidmore testing machine. Most shops don’t account for making additional bolts for testing and ro-caps. Tension values for 7/8 inch diameter structural bolts connecting two relatively thin, blasted and primed plates do not correlate to a 1-1/2 diameter, 10 inch long turned bolt going through the 3 inch thick machined base of a reducer, 1-1/2 inch of stainless shims, and a 2 inch thick machined base plate. These are not friction connections. How can you tension any bolt with double nuts? What torque goes on the second nut? Either spell it out or allow all turned bolts to be “snug tight”.

Keys and Keyways

Keys and keyways have been in use for securing hubs on shafts for thousands of years. You would think that by now we would have this perfected. It’s not. First of all the key must be drawn correctly on the detail drawings. For a square or rectangular key, the depth of the keyway in the shaft and hub is not measured at the center of the key, it is measured at the edge. This can be somewhat confusing since we normally think of half of the key in the shaft and half in the hub. With the curve of the shaft, it is natural to think of there being ½ inch of a 1 inch high key in the hub at the center of the key and ½ inch in the shaft at the center. In fact this is not the case due to the curve of the shaft. Looking at the example detail, you can see that the key should be half in the shaft and half in the hub at the sides of the key not the center. This is logical because the sides are where the load is and therefore an equal distubution of the forces from the shaft to hub is achieved with this arrangement. The generally accepted standard for sizing and tolerancing keys and keyways is ANSI B17.1. Unfortunately, AASHTO does not directly reference this standard for fits and sizes, only for corner radiuses. Actually, ANSI B17.1 does not require use of corner radiuses and chamfers, it simply provides a suggested table for them “when used…..as a guide”. AREMA states that “Details of keys and keyways shall conform to ANSI B17.1 except for fit….” Of course the whole point of ANSI B17.1 is to establish tolerances and fits. As mentioned above, it does not require filleted keyways or chamfered keys so it these details are required, please make it clear. For simplicity in machining and measuring the depth of keyseats and keyways, the dimension from the bottom of the keyway to the back of the shaft and from the opposite side of the bore to the bottom of the keyway is used on detail drawings to establish the depth. These dimensions are given in ANSI B17.1 and the formulas for calculating them. This is the simplest and most direct way of measuring the depth of a keyway. The curvature of the bore prevents using a depth mic.

Now that we have the dimensional details, we must address the tolerances and the fits. Niether AASHTO nor AREMA use the ANSI standard for the fit of the keys. AREMA references the fit and finish table 15-6-5 which simply states FN2. It does not differentiate between the height and width of the key. AASHTO uses table 6.7.8-1 that specifies FN2 fit on the sides and LC4 fit top and bottom. Niether of these specs is consistent with ANSI B17.1 or practical. In fact, if the AREMA specification is taken to the extreme and keys are set with FN2 fits all the way around, the hubs on some products may be over stressed simply from the keys. No coupling manufactures that I'm aware of require any interference on the height of the key. Even with the AASHTO standard of LC4 the The bottom line is that use of the ANSI table is much more complete and practical.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Replacement of the Counterweight Trunnions on the Lea Joyner Bridge

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Introduction

Spanning over the Boeuf River (tributary of the Ouachita River) the Lea Joyner Bridge carries east-west vehicular traffic on Louisville Avenue (LA 15) from Monroe to West Monroe Louisiana. Over the navigable channel the sixteen span bridge includes a bascule span with an under deck articulated counterweight. This type of bascule span is often referred to as a "Strauss bascule" as most, including the Lea Joyner Bridge (1931), were designed by the Joseph Strauss Engineering Corporation. The Lea Joyner Strauss bascule span consists of two 80'-0" leaves from centerline of trunnion to the toe. In its full open position, each leaf can rotate 79.5 degrees providing 130'-0" of clear navigable channel width. The bascule girders are built up I-sections and support a typical floorbeam-stringer deck system as well as the articulated counterweight. The original bridge, constructed in 1936 by the Nashville Bridge Company, had a 40'-0" wide timber deck with asphalt planks and two 6'-6" sidewalks. In 1950, the timber decking was replaced with an open grid deck. To account for the reduction in span weight, counterweight balance block pockets were installed in the deck to allow for adjustment.

Figure 1: Lea Joyner Bridge Elevation in Open Position

Figure 2: East Leaf in Open Position

Strauss trunnion type bascule spans with underdeck counterweights have two points of rotation per girder; the main trunnion and the counterweight trunnion. Rotation of the counterweight during span operation minimizes the sweep radius allowing the size of the bascule pier to be reduced longitudinally compared to trunnion bascule spans with fixed counterweights. However, the additional moving parts required for articulating the counterweight of the Strauss trunnion bascule span increases the potential for operational issues. Most, if not all, of the Strauss bascule spans were designed and constructed during the first half of the twentieth century. Many of the Strauss bascule spans still in service are requiring retrofits of the counterweight trunnions and hanger assemblies.

Background

In 2007, a rehabilitation project began on the Lea Joyner Bridge, including the bascule span. This work included rivet replacement, structural steel rehabilitation, spalled concrete repairs, grid deck replacement, electrical system rehabilitation and other miscellaneous repairs. To reset the proper imbalance condition of the bascule span the work required a pre-construction bascule span balance test. Strain gauges affixed to the operating machinery shafts were used to determine the pre-construction imbalance. The results from the strain gage testing revealed a significant operational resistance in the form of friction during span operation. This data was confirmed when a temporary winch system designed to operate the span could not overcome the frictional resistances. Through the data provided by the strain gage test and the winch system in addition to field observations, it was determined that the counterweight trunnions require replacement. This work included in-kind replacement of the trunnion pin, sleeve, and bushing. The trunnion bearing housing was to be inspected to determine its condition. If not satisfactory the bearing housing was to be replaced. In addition to the trunnion bearing assembly replacement, the top portion of each counterweight hanger was replaced.

The Lea Joyner Bridge is a main passageway for marine traffic and in particular, barges, requiring it to open on average 6 times per week. At the same time, Louisville Avenue is a heavily traveled roadway with an average daily traffic count (ADT) of roughly 40,000. Considering both of these factors, it was ultimately decided keeping the bridge open to marine traffic to allow for the delivery of cargo was the higher priority. This decision significantly complicated the construction erection and sequencing of the trunnion replacement.

In the open position, jacking and supporting the counterweight proved to be an intricate task. Numerous methods were examined to accomplish this work with a priority placed on limiting the duration of the road closure as much as possible. The retrofits were to be completed one leaf at a time, starting with the West leaf, and within a 4 week time frame. To expedite the process, it was imperative that the grillage and jacking system were designed to allow parts to be assembled and installed prior to the bridge closure.

Jacking Plan Development

Figure 3: Notched Portion of Counterweight

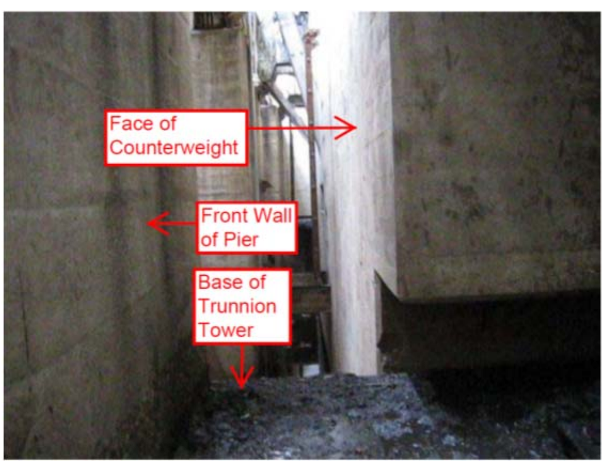

Figure 4: Counterweight in Open Position

Being a Strauss bascule bridge, the Lea Joyner Bridge has a counterweight that rotates about its trunnion as the span opens; therefore it remains vertical throughout the entire opening. This allows the designer to decrease the size of the bascule pier while maximizing the potential of the counterweight. In the case of the Lea Joyner Bridge, the pier size was optimized by cutting out the bottom of the counterweight to fit around the concrete base of the trunnion tower (Figure 3). This notched area reduced the amount of space required for the counterweight. As shown in Figure 4, when the span is in the fully open position, there is a clear distance of less than 1 ft from the front face of the counterweight to the front face of the pier wall. Additionally, only a few inches remain between the bottom of the counterweight and the top of the trunnion tower platform. While this is an efficient design, it does not lend itself to quick and easy retrofits that require access and equipment in the bascule pier and counterweight pit.

In order to work within the confined space in the pier, the angle of the span where the work was being completed was limited to 35 degrees, while the other span was fully opened (Figure 5). This provided a sufficient clear channel for marine traffic as well as enough workable space in the pit to develop a jacking and support system. Numerous methods were examined in depth for the jacking scheme and trunnion replacement. After discussing these options with the contractor and understanding the constructability constraints, a final system was developed.

Jacking Scheme

Figure 5: Bridge Open to 35º Angle

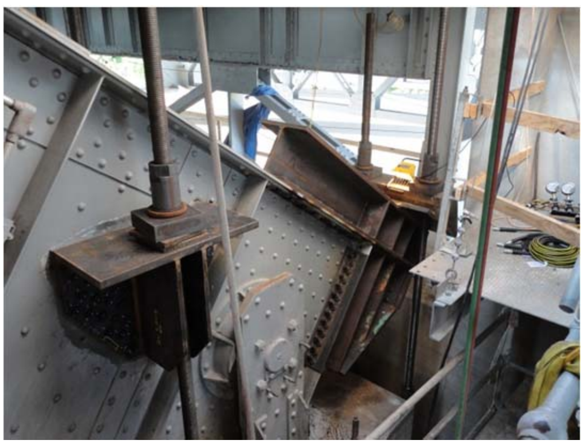

Figure 6: Brackets off back of girder.

The system employed to support the counterweight maintained the center of gravity of the counterweight by transferring its weight through the heel of the girder. This was done using tension rods which were hung from support brackets attached to the web as well as cantilevered off of the top flange of the girder as seen in Figure 6. The brackets were positioned on either side of the trunnion to transfer the dead load of the counterweight through its original load path. This method allowed utilization of the counterweight as a "dead man" while retaining the balance in the span and relieving the load in the trunnion pins. The only additional restraint necessary to provide was a brace against wind loads. An hollow structural section (HSS) was connected to the bottom flange of the girder and the front corner of the tower platform to address this issue.

Both brackets were unique weldments that were designed to utilize existing rivet holes to connect to the bascule girder. These rivets were removed and replaced with high strength bolts. The shape of the connection plate of the web brackets (see Figure 7) was derived from the spacing of the rivets, minimum bolt spacing requirements, and the number of bolts necessary to withstand the jacking load. Being closer to the centerline of the trunnion, the brackets were designed to take more than double the load compared to the top flange brackets. The top flange bracket had to be built up in order to clear the back of the

Figure 7: Web Bracket

Figure 8: Top Flange Bracket

counterweight. It was vital the tension rods cleared the counterweight to prevent the need to core through the concrete. The bracket consisted of a W36x256 connected to the back end of the girder, a W14x159 attached to the top flange, and a stiffened welded box attached to the top flange of the W14x159 (see Figure 8).

Figure 9: Bottom Grillage System

Figure 10: Spherical washers are bearing plates.

Figure 11: Short column weldment on top of bottom grillage.

The tension rods that were hung from the support brackets were pinned at the bottom of the counterweight by a grillage system, which was comprised of three W14x233 members connected with 1½" top and bottom cover plates. Two sets of this cribbing were provided per trunnion and were used to sandwich 4 hydraulic pancake jacks (see Figure 9). The tension rods passed through both sets and were tightened against the bottom-most plate. This created two support points for each tension rod with the jack and trunnion encapsulated allowing the jacks to tension the rods while also compressing the hangers to relieve the dead load stress. At each connection point, spherical washers and bearings, as seen in Figure 10, were employed to ensure the rods would remain plumb.

Due to the unconventional shape of the bottom of the counterweight, a flat grillage would have been unable to transfer the dead load through the hanger. As a result, short column weldments built up from HSS20x12x⅝ and W27x281 members were used to fill the gap between the top flange of the top of the W14x233 grillage set and the bottom of the counterweight at the hanger location (see Figure 11). These columns were connected with a common top flange so the load was evenly distributed to the bottom of the counterweight. The main purpose of these weldments was to create a direct load path from the top grillage set to the hanger.

Jacking the Counterweight

The innovative jacking procedure for this work involved tension rods pinned between the top brackets and the bottom cribbing with jacks sandwiched between the bottom sets of cribbing. By placing the jacks between the pin points of the rods, it allowed the opportunity to jack directly against the bottom of the counterweight while also maintaining the dead load in the bascule girder. This placed the rods in tension while compressing the hangers. However, the actual weights of the counterweight and span were unknown; therefore, determining when the load had been successfully relieved in the hangers became an issue since the system was designed to jack against itself.

As the hangers were being unloaded, it became possible for them to be compressed against the trunnion pin and experience a stress reversal. Even though this would not affect the overall stability of the bridge, it would be a major problem when they were cut.

Figure 12: Dial indicators on hanger plate.

The solution that was employed involved dial indicators, which were welded to the web of the girder, as seen in Figure 12, to measure the vertical and horizontal displacement of the hanger. The existing trunnion pins were designed to have a nominal clear spacing of $^{1}/_{16}$" to the counterweight sleeves. However, because these members were installed in 1936, it was unknown how much deterioration, if any, had occurred. In an effort to determine when the full weight of the counterweight had been captured, the dial indicators were monitored throughout the process until a satisfactory vertical displacement was achieved. As a supplementary check, an estimated weight for the counterweight was calculated and used as a starting point for expected jack pressures. Due to the uneven spacing and thus varied loading in the tension rods, the jacks could not be uniformly pressurized. At each hanger location, there were two front jacks (channel side) and two rear jacks (approach side) provided. All of the front jacks were loaded equally using one manifold, while the rear jacks were loaded via a second manifold. This provided the ability to set the jacks to the desired pressure. As the counterweight was jacked, shims were driven in at designated locations to shift the load from the jacks to the shim stacks.

Hanger and Trunnion Replacements

Figure 13: Existing hangers showing signs of pitting and corrosion.

Once the counterweight was successfully jacked, the top portions of the hanger plates were flame cut and ground down to a smooth surface. This section of the existing hanger was removed for two main reasons. First, the hanger plates and connecting angles showed signs of severe corrosion and pitting where the hangers protruded from the counterweight. In some cases, the hanger web plates suffered 20%-30% section loss (see Figure 13). While this was not an immediate threat and did not compromise the integrity of the structure, it was determined that a repair was necessary to prevent any further damage. The second reason was to facilitate installation of the new trunnion. Installing new hangers with new trunnion pins, sleeves, and bushing limited the field machining solely to the existing housing. This expedited the work done in the field and ensured a proper fit up.

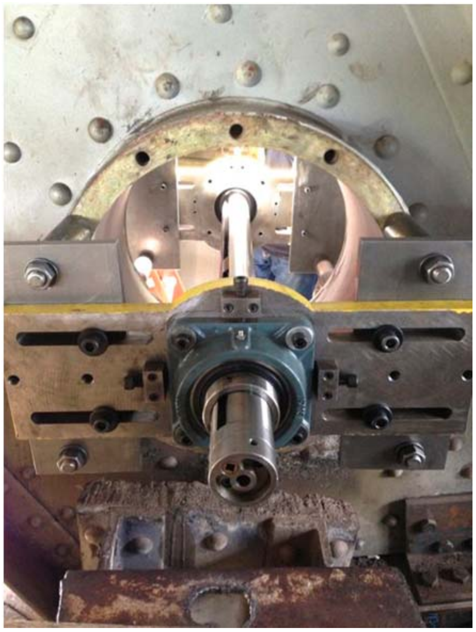

Figure 14: Cleaning of existing housing.

Figure 15: New bushings packed in dry ice.

After removing the hanger plates, the existing trunnion pin, sleeve, and bushing were removed as well. The existing housing was machined, cleaned and polished, and inspected to determine whether it needed to be replaced (see Figure 14). None of the four housings showed signs of damage that would require replacement. After the visual inspection, the new bushings were packed in dry ice so they could shrink facilitating its fitting into the existing housing (see Figure 15). Once the new bushings were in place, the pin and sleeve were installed (see Figure 16). After they were in place, the new hangers were spliced to the existing bottom portion (see Figure 17). With the new hangers and trunnions in position, the counterweight was jacked until the shims could be removed. As the shims were knocked out, the full weight of the counterweight was transferred back into the trunnion and hangers thus completing the replacement.

Figure 16: New bushing, pin, and sleeve installed in the existing housing.

Figure 17: New hanger plates spliced to existing member.

Conclusion

Figure 18: A well-lubricated trunnion is a happy trunnion.

The Lea Joyner Bridge closed to traffic on June 1st 2013 to begin work on the trunnion replacement. Work was completed on the West Leaf first. After the new counterweight trunnions were installed, the bridge was reopened to traffic for July 4th weekend. The bridge closed once again on July 9th, to begin work on the East Leaf. All work was carried through to completed construction prior to the scheduled August 9th 2013 reopening date.

As the current Strauss bascule spans continue to age, retrofits and replacements are becoming more prevalent. These structures employ similar design philosophies but each have their own unique execution of them. In the case of the Lea Joyner Bridge, the shape of the counterweight was defined by the geometry of the pit. By eliminating some of the corners of the counterweight, the pit was optimized to be the smallest shape as possible. This drastically increased the difficulty to complete the necessary repairs. These types of repairs require extensive amounts of planning to ensure a proper support grillage for the counterweight while allowing adjustability in the entire system. In addition to the challenges faced when completing the actual trunnion replacement, issues such as constructability, installation options, and vehicular/marine traffic patterns must be taken into account. These types of bridges will necessitate massive repairs in the near future and it is important to fully understand the requirements for a successful project.

Acknowledgements:

Lea Joyner Bridge Rehabilitation - Counterweight Trunnion Replacement

Bridge Owner and Operator:
Louisiana Department of Transportation and Development, Baton Rouge, LA

General Contractor:
PCL Civil Constructors, Tampa, FL

Steel Fabricator:
Florida Structural Steel Fabrication and Erection, Tampa, FL

Machinery Fabricator:
JC Machine Works, Corp., Miami, FL

Design Engineers:
Hardesty & Hanover, LLC, New York, NY

HEAVY MOVABLE STRUCTURES, INC.
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September 15 – 18, 2014

Transverse Balancing Considerations For Tower Drive Vertical Lift Bridges

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Abstract

Balancing of movable bridges requires the judicious interpretation of the results of strain gage testing from the instrumented shaft and the subsequent manipulation of weight to achieve the desired reactions at the live load bearings, as defined by AASHTO/AREMA. For tower drive vertical lift bridges in particular, the transverse balance is an important consideration when evaluating balance corrections and weight changes. Several factors must be kept in mind when proceeding with balancing of a tower drive vertical lift bridge.

First, it is important that any sort of shaft locking (indexing) device is active (locked) while recording strain, since any indexing devices that slip (torque couplings, clutches, etc.) can result in inaccurate recommendations for the final seating imbalance. This is primarily due to the torque sharing behavior of a torque coupling or clutch while slipping or exceeding its design holding torque. Additionally, the relationship between friction and imbalance loads must also be carefully considered to keep operating loads to a minimum (preserving AASHTO/AREMA reserve capacity), maximize service life of the machinery components, and ultimately resulting in reduced overall life cycle costs. Lastly, consideration must be given to the fact that mitigation of any transverse imbalance using existing counterweight pockets is limited.

Case studies are presented, demonstrating and expounding on unique concerns. The Marine Parkway Bridge in Brooklyn, New York provided various challenges with balancing due to excessive friction in one tower, tower “plumbness” issues, as well as an asymmetric load distribution across the lift span. The friction issues are discussed in detail. The James River Bridge in Newport News, Virginia, incorporated a torque coupling for indexing purposes and presented challenges for balancing/seating due to the coupling slipping with each operation. For these two bridges, empirical data was obtained from the field through testing and observation, as well as theoretical results calculated to validate the findings. Recommended courses of action, including justifications and implications of the transverse imbalance, are discussed for each case.

Introduction

The proper balance of movable bridges is essential for reliability and maximum design service life. If the imbalance is too great, the result is excessive loading and premature wear to the mechanical drive train and electrical system components. Additionally, the reserve capacity to operate under adverse weather conditions is reduced. For vertical lift bridges, there is an overall end to end imbalance (average imbalance condition at each end of the lift span). The end imbalance condition, since it is an average, includes the effect of any transverse imbalance between the corners at the end of the lift bridge.

To obtain usable data for bridge evaluation from the field, a practical measurement and data collection means must be employed. The strain gage method of testing is generally regarded as the industry standard for assessing a bridge overall balance condition, though it is not without its limitations. Prior to testing, the instrumented shafts need to be zeroed, to eliminate any residual torque that would affect the accuracy of results. Additionally, abnormal friction loads can potentially reduce the accuracy of the measured imbalance loads. The friction must be properly accounted for and isolated.

For vertical lift bridges, particularly those having tower drive machinery arrangements, the cross shafting must rotate synchronously for repeatable bridge seating. This is normally not an issue provided that any indexing device is locked. The purpose of the indexing device is to fine tune the seating of the lift span at each end such that the live load shoes contact their respective sole plates simultaneously. This assures the best seating condition without having to force the lift span down with the drive train or rely on excessive span imbalance to maintain proper seating (firm contact of the live load bearings with the pier).

Following is a brief overview of strain gage preparation, specifically on a tower drive vertical lift bridge with key considerations emphasized. The importance of proper zeroing is discussed as well as ensuring that cross shafts are properly indexed. Two case studies are reviewed, the James River Bridge in Newport News, VA and the Marine Parkway Bridge in Brooklyn, NY. Each bridge has different shaft indexing features and encountered unique challenges during operational testing.

Background - Strain Gage "Zeroing" and "Indexing"

The point of "zeroing" prior to strain gage testing is to establish an unloaded, zero torque condition in the drive train shafting. Prior to testing, all residual torque in the instrumented shafting must be removed to establish a zero torque baseline. This is typically achieved by checking for backlash on both sides of the engaging main drive pinion teeth, such that the gear teeth are not in contact. Ascertaining zero often requires that the brakes be released, since they typically hold residual torque in the drive train from previous bridge seating operations. The failure to provide a zero torque condition can impact the results such that the testing cannot provide the intended results.

A means employed for span leveling or proper seating is known as "indexing". Over time, and often as a result of severe imbalance, counterweight rope slippage/stretch may occur around the rope sheaves, affecting proper seating. Additionally, if the planes of the span, live load shoes, and live load seats are not parallel, the indexing for level operation will not match the indexing for seating.

The cross shafting on tower drive vertical lift bridges typically includes specialized machinery to level the span with respect to the seats. The specialized machinery can include a clutch, adjustable coupling, or torque limiting coupling. The function of the indexing machinery is to allow for independent movement of both transverse corners of the lift span. Occasionally, one corner live load shoe may seat first before the opposite corner seats. While the indexing machinery is disengaged, each corner can be driven into its respective seat for full contact at the live load bearings, provided the live load bearings are reasonably close to their correct elevations. Once in contact with the live load bearings, the cross shafting can then be locked together by re-engaging the indexing machinery.

Following are two case studies where indexing as well as other operational challenges needed to be considered in order to arrive at sound recommendations for improving the movable bridge operation.

James River Bridge, Newport News, VA (*Virginia Department of Transportation*)

Slippage at Torque-Limiting Coupling

As part of the testing that was conducted at the James River Bridge in late 2012 in support of the grid deck replacement, slippage was documented at the cross shaft controlled torque couplings. The resulting impact on the transverse imbalance was also recorded. As a result of this condition, only the end to end imbalance was evaluated at the bridge for the final balance. Since the coupling slippage is likely to have an impact on the reliability of span operation in general, and span seating in particular, it was important to have this documented. Additional thoughts are provided on things that could be evaluated to troubleshoot the issue. Over the course of the testing, the span end to end test results have been consistent, with the results being reasonably close to the theoretical calculations. Corner to corner test results have been *inconsistent* throughout the course of testing and have also yielded negative friction values for some test runs. To address the negative friction results, the quantity of test runs has been increased to five test runs and only runs with positive friction were used to determine the balance condition.

The corner to corner inconsistencies could be attributable to the following:

1) Machinery Indexing
2) Gaps at the Live Load Supports
3) Span Transverse Balance

Results of testing follow below in Figure 1:

Lift Span, North End Test Date: December 3, 2012						
Test 1	Imbalance (LB)			Friction (LB)		
	NW Corner	NE Corner	North End	NW Corner	NE Corner	North End
Run 2	+6,862	+8,906	+15,769	861	2,188	3,049
Run 4	+5,283	+10,400	+15,683	92	2,970	3,062
Run 5	+6,095	+9,797	+15,893	51	2,972	3,023
Average	**+6,080**	**+9,701**	**+15,782**	**335**	**2,710**	**3,045**

Lift Span, South End Test Date: December 3, 2012						
Test 1	Imbalance (LB)			Friction (LB)		
	SW Corner	SE Corner	South End	SW Corner	SE Corner	South End
Run 2	+7,246	+6,433	+13,680	2,112	1,477	+3,590
Run 4	+8,549	+5,013	+13,561	3,106	495	+3,600
Run 5	+7,447	+5,944	+13,391	3,206	468	+3,674
Average	**+7,747**	**+5,797**	**+13,544**	**2,808**	**813**	**3,621**

Figure 1 – Table of Imbalance & Friction Results

At the beginning of each testing, during the zeroing process, machinery indexing was confirmed to be inconsistent from corner to corner. At the completion of testing it was confirmed that the NW live load shoe exhibited a small gap. Additionally, the transverse imbalance at the north end was 3,621 lbs and 1,950 lbs at the south end.

There were two options going forward:

1) Attempt to make a transverse weight change in an effort to eliminate the gap at the NW live load support.
2) Continue to make weight changes based on deck panel weight changes and accept the variation in the corner to corner results.

Option 1 above is a relatively small weight change which would not be enough to eliminate the gap at the NW live load shoe. Option 2 was the better choice moving forward.

During the next balance test, the secondary reduction input shaft coupling was marked and monitored for any slippage. This was to confirm the contention that variations in machinery indexing were affecting the ability to obtain repeatable test results corner to corner with the goal of validating the proposed approach to allow the total imbalance at each end of the lift span govern the final balance test. Gaps at the live load supports were also checked at the completion of testing

As part of the testing conducted on December 10, 2012, the cross shaft controlled-torque couplings were provided with a match mark at the outset of the work, and the match marks were monitored throughout the testing. The match marks demonstrate that the cross shafts were slipping as a result of span operation. The coupling slippage was also evident in the machinery loads as the torque signatures of the instrumented drive shafts were not repeatable and varied from run to run. This was evident through reviewing a run to run comparison of the strip charts.

Transverse Imbalance

Strain gage testing is predicated on obtaining uniform and repeatable span behavior. Due to the demonstrated inability to obtain repeatable loading from run to run which is wholly attributable to the slipping torque-controlled coupling, it is not possible to accurately determine the transverse balance of the bridge. Further, the changing loads resultant from torque coupling slippage during operation provided a basis for the negative friction test results, which were obtained on prior tests throughout the project; however, prior test results on this project have also found that the end to end imbalance has correlated well with the theoretical calculations. Therefore, the basis for the final balance test was to ensure that the ends of the lift span met the project specifications. No consideration was given to evaluation of the transverse imbalance.

In light of the controlled torque coupling slippage, the recorded strain relative to full load motor torque was evaluated. Full load motor torque equates to approximately 150 microstrain in the instrumented shafts. At all locations, the shafting experiences in excess of 150 microstrain while accelerating and decelerating during the seating sequence. Whereas the loading at the north tower just marginally exceeds 150 microstrain, the loading at the south tower peaked over 200 microstrain. These peak loads are substantially higher than the nominal load required to raise or lower the bridge and provide the likely basis for the coupling slippage.

Because the James River Bridge is under contract for a mechanical and electrical rehabilitation design, these issues will be revisited in greater detail then. The transverse imbalance, coupling indexing, and span seating will be addressed during the rehabilitation. At the time of this writing, the lift span is appropriately balanced end to end, with the grid deck replacement completed.

Marine Parkway Bridge, Brooklyn, NY (*MTA Bridges &Tunnels*)

The Marine Parkway Bridge, originally constructed in 1937, is a vertical lift bridge which spans between Brooklyn and Queens. It has undergone several rehabilitations, including counterweight rope replacements, elevator upgrades, and steel repairs. In 2003 a contract was executed for miscellaneous steel repairs and the addition of a sidewalk along the west side of the bridge, including the lift span. This was an enhancement in that it relocated the walkway from inboard to outboard of the west truss, increasing the roadway width. To counteract the resulting transverse imbalance of the new sidewalk, the east roadway barrier remained steel while the west barrier was lightened by using of aluminum. Additionally, a droop cable system was located along the east side of the lift span, which also served to offset a portion of the transverse imbalance moment. Lastly, concrete blocks and steel plates were added to the west counterweight pockets of each counterweight. Strain gage testing at the time showed the imbalance to be evenly distributed at all four corners of the lift span. As part of a current electrical and mechanical rehabilitation contract for the Marine Parkway Bridge, the lift span imbalance and machinery loading were revisited.

Transverse Imbalance (Investigations)

Since previous inspection reports noted hard contact at the transverse guide rollers, initial field investigations included their observation during bridge operations as well as through a partial disassembly of the east upper transverse guide rollers. During operations, it was observed that certain transverse guide rollers consistently came into contact with their guide rails (the east upper and west lower). The partial disassembly of the east rollers, located within the upper chords of the east truss, revealed wear at the bushings and heavy corrosion of the fasteners and steel. Though AASHTO does not explicitly address transverse imbalance, a reference from the 1898 text *De Pontibus*, by Dr. J.A.L. Waddell (P.110) states that the transverse guide rollers do not contact the vertical guides "unless there is a sufficient wind force to bring them to a bearing". In order to best ascertain the magnitude of the transverse imbalance and its implications, the following investigations were performed:

Investigation	Purpose	Findings
Previous Reports & As-builts	Historical Background Information, Timeline of Events	Previous construction contract weight changes, north tower friction issues, uneven gear wear, north tower "plumbness" issues
Calculations	Estimate Magnitude of Transverse Imbalance	Transverse imbalance allegedly still exists, approximately 75 kips
TBTA Interviews	Corroborate Report Findings	Problems seating bridge transversely at live load shoes, particularly SE location, during a previous construction contract
Working Bridge Model (Figure 2)	Demonstrate Behavior of Counterweight and Lift Span Due to Imbalance	Differential clutch slippage resulted in lift span tilting into span guides and cwt tilting into cwt guides to compensate (Figure 3)
Initial Field Inspections	Observe Span Guide Roller Clearances During Operation	Transverse guide rollers repeatedly contact guides at east upper and west lower locations; plastic flow present; clearance at other rollers

Figure 2
Working Model of Marine Parkway Bridge: Model includes locking differential to duplicate function on actual bridge.

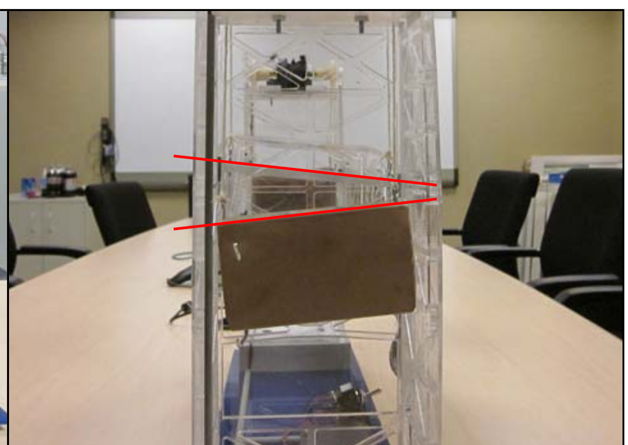

Figure 3
Bridge Model with Weight Added to One Side of Lift Span: Note exaggerated misalignment of counterweight with respect to lift span.

A simplified representation of the forces on one half of the lift span is shown in Figure 4. When the bridge is seated, the live load and imbalance load of the lift span is transmitted to the substructure through the bearing seats. When the bridge is raised the entire dead load of the lift span is carried by the counterweight ropes.

Upon leaving the bridge seats during a bridge lift, the differential remains active (differential clutch disengaged) and the lift span tilts downwards from east to west, until the east upper and west lower span guide rollers contact the tower rails. This is the case at both the north and south tower locations and is repeatable for every bridge opening.

The estimated contact load on the transverse guide rollers is about 16 Kips. It is stressed that the figure below is for illustration only, and that rope tension measurements, used to resolve the forces in the free body diagram are inherently subject to appreciable error.

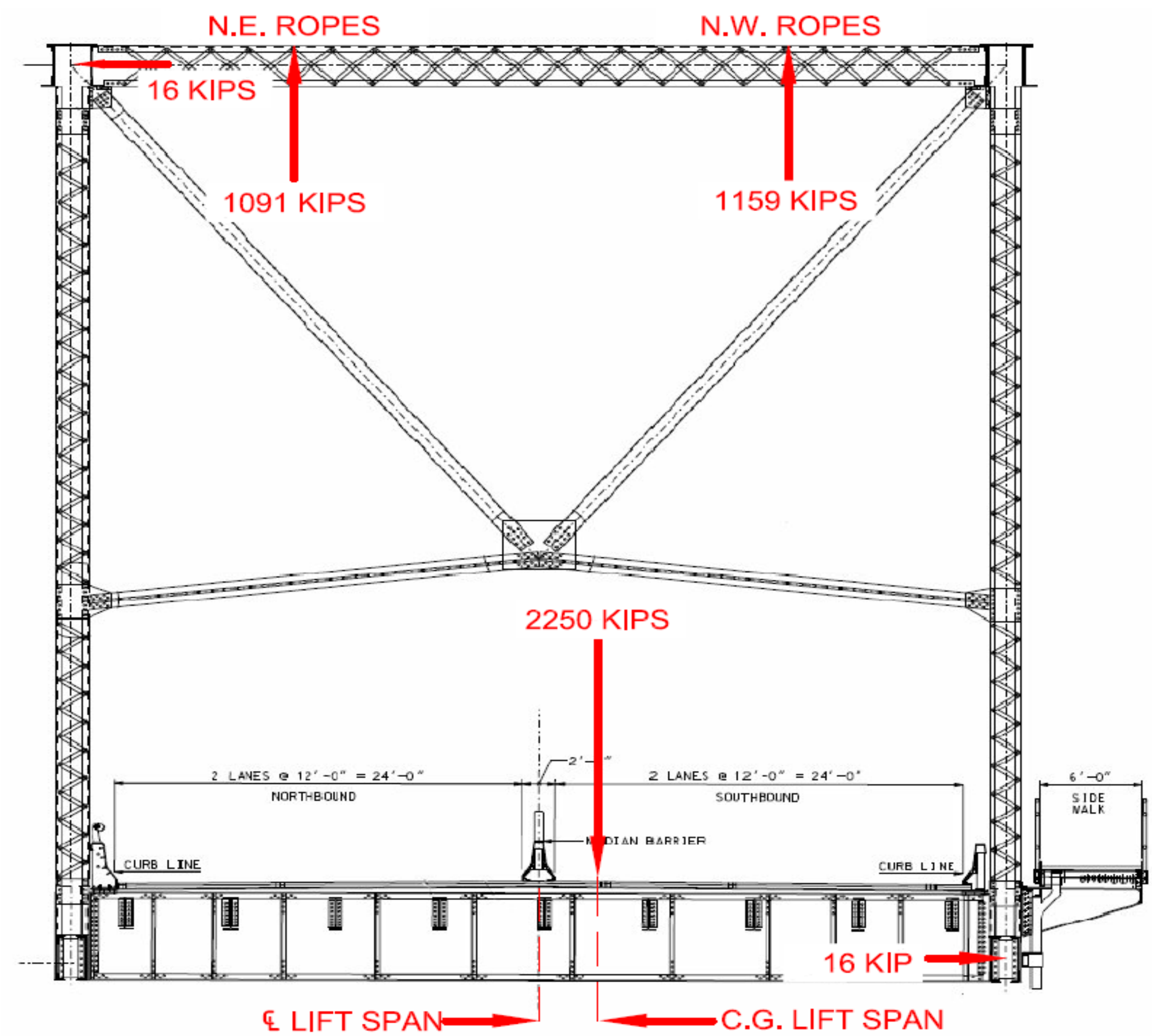

Figure 4 – Lift Span Half-Loads with Span Off Seats (Looking South)

With regards to why corrective action is needed, transverse imbalance becomes an issue for the lift span during bridge operations for the following reasons:

1) Counterweight ropes may stretch unevenly due to uneven loading of the counterweight. This can result in counterweight tilting if the ropes do not have adequate provision for adjustment.
2) Tilting of the lift span due to transverse imbalance causes continuous guide roller loading resulting in plastic flow at guide wheels as well as tower guide (see Figure 5 below).
3) Tilting of the lift span due to transverse imbalance increases friction at counterweight guides potentially resulting in a reduction in normal service life (see Figure 6 below).
4) Guide roller contact affects the accuracy of resultant balance recommendations between transverse corners and limits the useful balance data to the ends of the lift span only.

Figure 5
NE Upper Transverse Guide Roller and Guide Rail: Note wear (arrow) at guide roller and guide rail.

Figure 6
NW Lower Counterweight Guide: Note evidence of hard contact between guide shoe and structure.

Transverse Imbalance (Field Testing)

Field testing involved three independent tasks that included machinery strain gage testing, simultaneous measurement of span and counterweight tilt, and pressure film analysis at the transverse guide rails. The intent was to measure the affects of the incremental addition of calibrated loads to the east side of the lift span. It was anticipated that the strain gage tests would record the incremental changes in transverse imbalance and ultimately lead to valid balancing recommendations. The measurement of span and counterweight tilt was performed to document if the lift span were “leveling” with the addition of weight to the east side of the lift span. Lastly, the pressure films were located on the transverse guide rails for the purpose of establishing a trending load reduction, if any, with the incremental addition of weight to the lift span. Electric power readings were monitored at the control desk during all span operations. A summary of the various tests performed with their findings follows below:

Test	Purpose	Findings
Strain Gage Test	Imbalance and Friction Data	Inconclusive corner imbalances obtained due to "active" differential resulting in guide roller contact, which caused load sharing and unreliable transverse imbalance data
Inclinometer	Tilt of Counterweight Relative to Tilt of Lift Span	Lift span and cwt tilt behavior was equal and opposite, consistent with a severe transverse imbalance, similar to behavior of bridge model
Water Level	Tilt of Lift Span at Roadway Level	Corroborated tilting of lift span
Pressure Film	Magnitude of Guide Roller Loads with Known Weight Changes to Lift Span	Showed a reduction of guide roller load with the addition of calibrated test loads on lift span; test loads corroborated closely with strain data

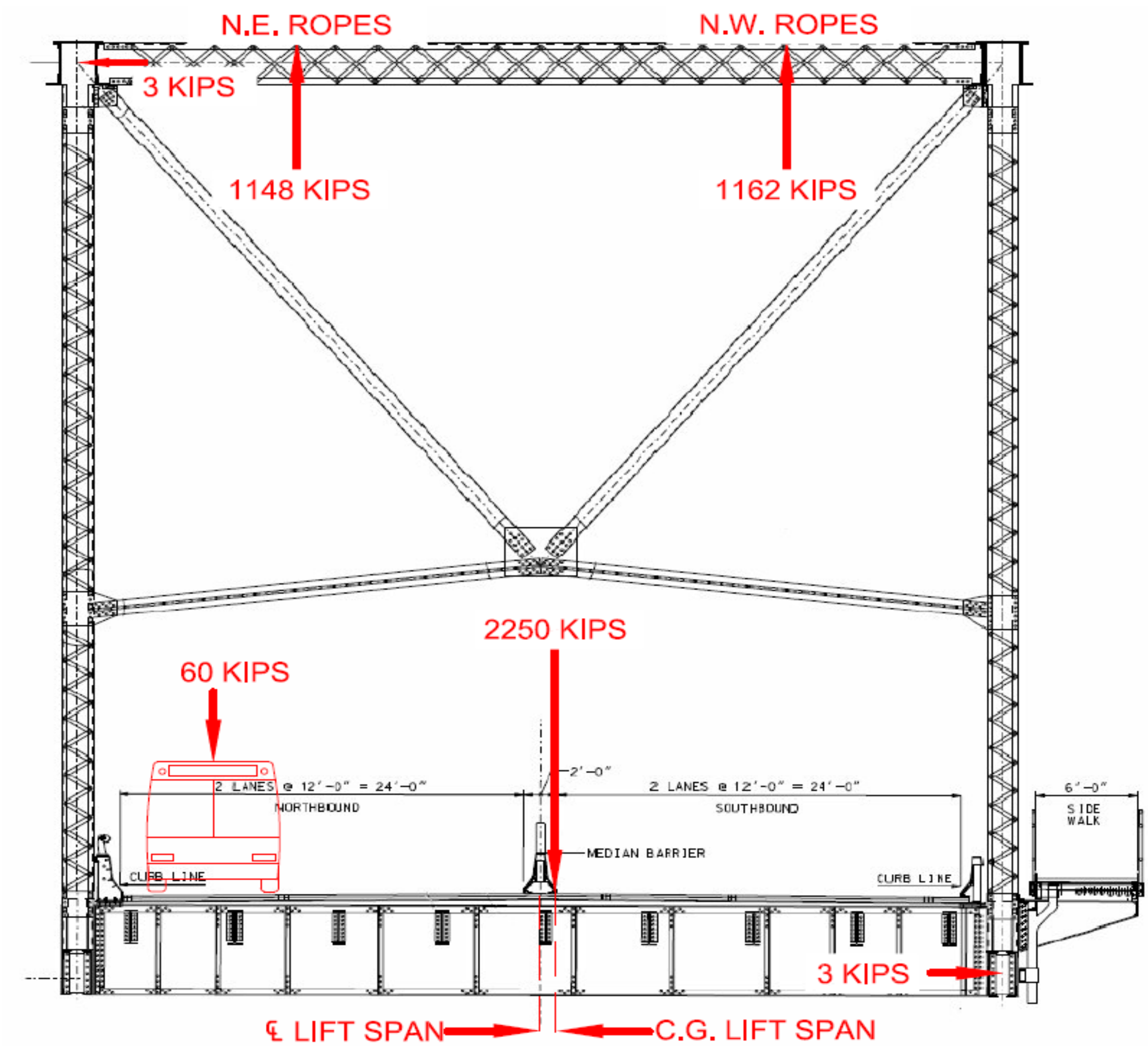

Figure 7 – Lift Span Half-Loads with Span Off Seats and Trucks Reducing Guide Roller Reactions (Looking South)

Figure 7 above theoretically illustrates the effects of placing known loads on the lift span to offset the transverse imbalance moment. The 60 Kip load causes the C.G. to shift approximately 6 inches eastward, reducing the transverse imbalance. Note also the loads at the guide rollers have dropped significantly.

Survey Work (Lift Span & Counterweight Tilt)

This work involved surveying the transverse tilt of the lift span at the north end during bridge lifting operations. For the survey work, a water level was monitored against fixed scales located on the east and west inboard sides of the roadway barriers. The water level was placed across the roadway once traffic was stopped. Additionally, electronic inclinometers were located at the top of the north lift span portal truss as well as on the north counterweight.

The water level testing demonstrated that the net tilt between the east and west barrier was approximately one-inch (the east side was one inch higher than the west side during bridge lifting). The inclinometer results measured the angular movement of the lift span at the north end. Regardless of test lift, the inclinometers recorded a counterclockwise transverse lift span rotation (facing north) of 0.150 degrees, which would approximate a one-inch elevation change across the roadway. This was consistent with the water level values, representing an uplift of 1 inch on the east side of the lift span. Additionally, the counterweight rotated in the opposite (clockwise) direction, corroborating the results from the lift span movement.

Strain Gauge Testing

A previous strain gage report dated April 9, 2009 showed approximately equal seating reactions at all four corners of the lift span (seating reactions were about 11 kips per corner for the north end of the lift span an just under 10 kips per corner at the south end of the lift span). The opening torque for both towers was higher than the closing torques, as would be expected for a typical span heavy balance condition. It should be noted that strain gage tests in this instance showed equal reactions per corner and were indicative of the differential in the central reducer splitting the load, and as such did not prove balance in the transverse direction.

A total of 7 lifts were performed as a part of the testing. The initial three lifts established a baseline to ensure consistent results for the strain gage data collection. The fourth lift included the weight of one 40-kip truck, parked at the center of the lift span (north bound, right lane). For the fifth lift, two trucks were located at the center of the lift span. For the sixth test lift, three trucks were placed on the lift span, two at opposite ends and one at the center of the span. The sixth test lift was initially aborted as a precaution due to excessive motor power readings at the control desk. The lift was subsequently completed without the center truck on the lift span (two trucks remained). Lastly, a seventh lift without trucks was conducted as a final confirmation of the baseline lifts.

The strain gage results for the testing completed recently showed consistent results as compared with previous testing on an end to end basis. The south end showed just over 20 kips of seating imbalance and the north end just under 21 kips of imbalance (10 kips per corner of imbalance is conventional for lift bridges of this size/magnitude). Friction in the north tower was about 5 times the friction in the south tower. This high friction value contributed to the inability to raise the bridge with the 3 trucks on the lift span. Since the 3 trucks can be regarded as simulating the equivalent of an AASHTO snow & ice design load, the friction diminishes the bridge machinery's reserve capability to overcome these loads. This discovery prompted the owner to fast track a friction mitigation task (discussed later in this paper).

Of particular note is the representation of approximately equal transverse seating imbalance (both north and south locations). The differential clutch was observed to slip upon every test lift, occurring as the lift span just left its seats as well as during final seating. The clutch slips just until the east upper and west lower span guide rollers come into contact with their respective guides. Once the span guide rollers made contact the loading between east and west sides of the lift span became more evenly distributed. It may be stated that the strain gage testing recently performed has accurately assessed the magnitude of the imbalance for the lift span as a whole and for each end of the lift span, but that the test method has been unable to assess transverse load changes. The problem with the inability to assess the transverse loading is due to the differential lock-out clutches allegedly slipping in the primary reducers.

Through a collaborative effort between the owner and several consultants, it was established that the clutches were in fact disengaged the entire time due to reversed wiring in the clutch limit switches, allowing the differentials to remain active. It was therefore recommended by the team that the limit switches be rewired and the differential clutches engaged or "locked" as necessary to carry the transverse torsional load. The differential clutch was subsequently locked successfully and the strain gage testing repeated; however, with the differential clutch locked, the seating of the bridge became unreliable. It was concluded by the team that the clutches should not be engaged (differentials locked) without first correcting the transverse imbalance. The Marine Parkway Bridge differential clutches remained deliberately disengaged for this reason.

Pressure Film Analysis

In an attempt to obtain a direct measurement from the truck balance testing, a pressure sensitive film was used at the NE and SE transverse upper span guides (see Figures 8 and 9). The film was intended to detect the line loading stress of the steel span guide rollers as they roll over the guide rails during the bridge test lifts. In order to capture all variations in loading, the film capacities ranged from a low of 1.4 ksi to a high of over 43 ksi. Each of the films was secured to the roller guide track about 4 feet above the top of the lifting truss at both the NE and SE guide locations. The films were all replaced with new films for the test lifts that included the baseline lift, lift with 1 truck, and lastly 2 truck lift. The films were then sent out for analysis to obtain the stress values (see Figures 10 and 11).

Figure 8
NE Upper Transverse Guide Rail: Note location of pressure film.

Figure 9
Note pressure film pattern after contact with guide roller for first test run (NE upper location).

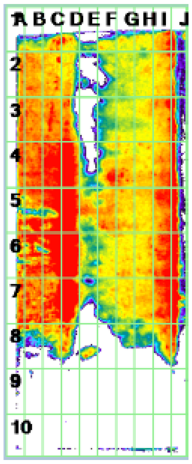

Figure 10
NE Upper Transverse Guide Rail, 2nd Run: Note load distribution as submitted for analysis.

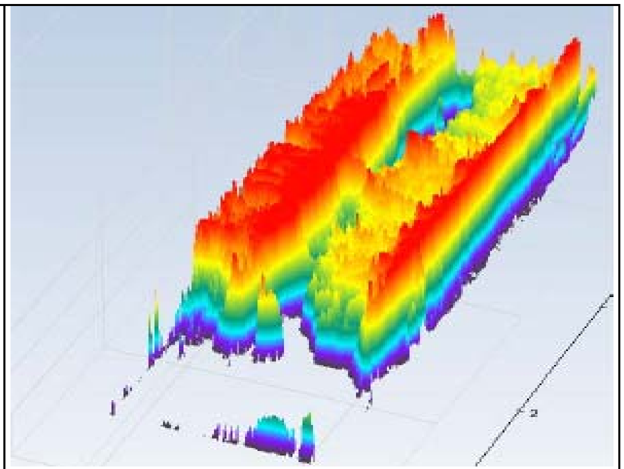

Figure 11
NE Upper Transverse Guide Rail, 2nd Run: Note 3-Dimensional load profile.

Results of the stress at the guides showed a reduction in load with the addition of weight along the east side of the lift span (see Figure 12 below). In fact, the SE roller actually was clear of the span guide (no contact) during the final truck test lift. The NE roller remained in contact, presumably due to the magnitude of friction previously noted for the north tower. In other words, the friction load was greater than the corrective imbalance load to notice any appreciable difference.

	NE Upper Guide Roller (average pressure, ksi)	SE Upper Guide Roller (average pressure ksi)
Baseline Lift	**18**	**13**
Lift with 2 Trucks (80 Kips)	**16**	**0**

Figure 12 – Pressure Film Results at Transverse Guide Rollers

Counterweight Pocket Survey

A counterweight pocket survey was performed to better understand the existing weight distribution within the counterweights as well as the remaining space available for additional corrective weight to transversely balance the lift span. In general, the west pockets of the counterweight were substantially filled with concrete blocks and steel plates in comparison to the east pockets. Should additional weight be necessary, lead blocks may be required, since the remaining space in the west counterweight pockets is limited.

An additional limitation for counteracting any transverse imbalance that needs to be considered is the actual locations of the corrective weights. With respect to moment arms, the effect of any weight adjustment within the counterweight pockets is limited to acting at the centerline of the counterweight ropes. Therefore, the effectiveness of correcting for any transverse imbalance moment of the lift span through counterweight pocket adjustments alone is dependent on the spacing, or moment arm, between the rope groups.

Existing High Friction (Background)

A notable deficiency that was identified as part of earlier strain gage balance tests performed on May 2, 2007 follows below. The objective of the testing was to establish the balance condition at the outset of a construction contract. As part of this testing, the operational behavior of the bridge was documented, as well as the loading to which the machinery was subjected. This additional information was reviewed through the course of the balance analysis and a notable deficiency was identified with regard to the data acquired from the north tower drive machinery. The deficiency regarded the excessive magnitude of system friction determined through the analysis. The friction for the north tower machinery was ***nine*** times greater than that for the south tower. The actual magnitude was equivalent to a force of approximately 59,000 lbs. at the main counterweight ropes. This deficiency was significant for the following reasons:

1. Strain gage balance testing was conducted as part of a previous contract on July 24, 2003. The friction in the north tower machinery determined as part of the analysis for that testing was equivalent to approximately 12,900 lbs. at the main counterweight ropes. Therefore, the 2007 test indicates that the system friction in the north tower has increased by 46,100 lbs. This increase may be an indication of a significant problem with structural interferences, mechanical binding, or a combination thereof.

2. The Special Provisions for the then current contract limited the increase in span balance due to construction activities to 2,000 lbs. per corner of the lift span (based on a nominal 10,000 lbs. per corner starting condition, which is reasonably close to the actual imbalance as documented in the balance report). The increased loading of the north tower machinery due to the excessive friction has completely marginalized the effect of any increase due to the allowed imbalance. Additionally, if the source of the friction is a deteriorating condition, a further increase in friction can be expected which may be far in excess of the allowable increase in imbalance.

As previously discussed, the recent testing from 2013 established that the bridge electrical system was unable to overcome a simulated snow and ice load on the lift span, as defined by AASHTO Condition C, without overloading. Additionally, and as a direct result of this high friction, the north tower auxiliary drive clutch was also unable to hold. Over time the clutch slipped more frequently, and had reached the limits of its maximum adjustment. This was an immediate reliability concern in that bridge operations on hot summer days would sometimes be required during ”brownouts”, and the normal PLC system would fault due to under voltages, leaving the auxiliary system as the only means of operation..

Friction Mitigation Task

Through additional collaborative efforts with the owner as well as input from the consultant teams, an accelerated friction mitigation task was recommended to address the north tower high friction. As an initial step it was recommended to thoroughly lubricate the north tower machinery, and then repeat the strain gage testing. If this step did not result in decrease in friction, more extensive investigation would be required, including inspection of trunnion bearings, removal of trunnion bearing caps, and inspection of the span and counterweight guides.

Since a thorough lubrication did not markedly improve the friction, the additional inspections were all performed. It was revealed that the trunnion journals in the north tower were severely scored, which was consistent with prior biennial inspection report observations. A solution is currently under design that includes improving the surface finish of the trunnion journals and clearing grease passages within the bearings. Additionally, since evidence of hard contact and resultant wear was observed at the northwest counterweight guide shoes, they will be replaced.

As discussed earlier, certain transverse span guide rollers are in hard contact and exhibit excessive bushing wear. These will be replaced in kind. This construction opportunity will also include a stepped approach for correcting the high transverse imbalance, initially through incremental counterweight adjustments followed by strain gage verifications. Correcting for the transverse imbalance should restore the intended function of the transverse guide rollers as well as the counterweight guide shoes. These rehabilitation items are all anticipated to be addressed in early 2015.

Conclusion

At the time of this writing, both the James River Bridge and Marine Parkway Bridge are undergoing a mechanical and electrical rehabilitation design. The James River Bridge rehabilitation is in the early stages of investigation and preliminary design. As mentioned earlier, the extenuating causes of operational difficulties need to be addressed in a logical sequence, correcting those items that can affect the proper balance determinations and follow up recommendations. The primary factors that can affect the results of accurate balance determination through strain gaging are excessive friction and improper machinery indexing.

It is not sufficient to simply lock up the indexing device, as this would only be treating an operational symptom of excessive transverse imbalance. Though locking the indexing device is required in order to obtain accurate transverse imbalance loads, the indexing device should be permitted to slip (remain disengaged) until the imbalance is corrected. The underlying reason for remaining disengaged is to facilitate reliable span seating and prevent excessive, asymmetric loading of the machinery drive train at the imbalanced corners.

Corrective measures at the Marine Parkway Bridge that are currently in design include mitigating the high friction in the north tower machinery, correcting excessive transverse imbalance, and replacing worn components affected by the imbalance and high friction. Friction mitigation efforts include improving the journal surface finishes and cleaning out all trunnion bearing grease passages. New counterweight guide shoes that will reduce sliding friction are also under development. Since specific span guide rollers were always in contact during bridge lifts, the roller bushings are worn and plastic flow has become evident along the tower guide rails. The guide rollers are intended for resisting wind loads only and ideally maintain clearance with the guide rails throughout operation in the absence of wind forces.

Adding corrective weight to the west counterweight pockets is the most practical solution at this time for correcting the transverse imbalance, allowing the differential clutches to be restored to their original purported service. Correcting the transverse imbalance will also reestablish the normal functioning of the transverse guide rollers and counterweight guide shoes. These rehabilitative measures, in addition to others under the mechanical and electrical rehabilitation design contract, are intended to allow the Marine Parkway Bridge to meet or exceed the required 20-year service life while meeting current AASHTO requirements for movable bridge operation.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

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ALTERATION OF GALVESTON CAUSEWAY RAILROAD BRIDGE AND EJ&E RAILROAD BRIDGE UNDER THE COAST GUARD BRIDGE ALTERATION PROGRAM

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Abstract

The U.S. Coast Guard Office of Bridge Programs administers the alteration of bridges that are unreasonably obstructive to navigation under the Truman-Hobbs Act of 1940 (33 U.S.C §§ 511-524). Under the authority of the Act, the Coast Guard has the duty and responsibility to preserve the public right of navigation and can order the alteration of obstructive bridges. Recently two bridges were replaced under the Act, the Galveston Causeway Railroad (GCRR) Bridge over the Gulf Intracoastal Waterway, Galveston, TX: owned by Galveston County, and the Elgin, Joliet, and Eastern (EJ&E) Railroad Bridge over the Illinois Waterway, Divine, IL: owned by Canadian National Railway Company. The old movable spans of these bridges were replaced with new long lift spans that provide sufficient navigation clearance to meet current and future navigation needs. This paper will address the history of these two bridges, the Coast Guard's bridge alteration program, and two different methods of span change-outs as part of Accelerated Bridge Construction (ABC). Also, the paper will assess two different operating machinery systems and specific mechanical design features, and explore the various challenges and unusual construction techniques utilized to solve mechanical operating system issues.

Coast Guard Program of Alteration of Bridges

Under the Coast Guard program of Alteration of Bridges, the Coast Guard investigates mariners' complaints regarding navigation through bridges that deem to be safety hazards and unreasonable obstructions to navigation. If it is determined through this investigation that a bridge is an unreasonable obstruction to navigation and the navigation benefits gained from its alteration equal to or exceed the cost of its alteration, the Coast Guard issues "Order to Alter" to the bridge owner requiring the bridge owner to alter their bridge by replacing the narrow span over the navigation channel with a longer span that provides sufficient navigation clearances which meets today's as well as future navigation needs. The law and provisions covering the Coast Guard program of Alteration of Bridges are contained in the Truman-Hobbs Act (T-H) (54 Stat. 497; 33 U.S.C. §§ 511-524). Once a bridge receives an Order to Alter from the Coast Guard, it becomes eligible to receive federal funds that cover a large portion of its alteration cost.

Only the bridge location and the vertical and horizontal navigation clearances of a bridge's navigational opening affect its eligibility for alteration under T-H. The structural integrity of a bridge or its adequacy for land transportation has no bearing on the Coast Guard determination that a bridge unreasonably obstructs navigation. Over the last several years, various bridges were replaced under the Coast Guard Bridge Alteration Program, including the Burlington Northern Santa Fe (BNSF) Railway Company Bridge over the Upper Mississippi River, Burlington, IA, the CSX Transportation Bridge over the Mobile River, Hurricane, AL, and the aforementioned Galveston Causeway Railroad Bridge over the Gulf Intracoastal Waterway, and the Elgin, Joliet, and Eastern (EJ&E) Bridge over the Illinois Waterway; both of which will be addressed in this article.

Galveston Causeway Railroad Bridge:

Description of the Old Bridge

The Galveston Causeway Railroad (GCRR) Bridge is a single-track structure, about 10,675 feet in length, which consists of 6,825 feet of reinforced concrete arch bridge spans and 3,850 feet of protected earth fill that crosses the Gulf Intracoastal Waterway (GIWW), Mile 357.2 , near Galveston, TX (Figure 1). The

Bridge is owned by Galveston County and operated by Burlington Northern Santa Fe Railroad Company under a 999-year lease agreement. Most of the original bridge was constructed in 1912. The old movable span of the bridge over the navigation channel was a single-leaf bascule span, built circa 1988, that provided a horizontal navigation clearance of 109 feet. The old bridge provided 8 feet of vertical clearance above mean high water in the closed position and unlimited vertical clearance in the open position. The bridge was located approximately 760 feet east of the fixed I-45 Dual Highway Bridge that provides 300 feet of horizontal navigation clearance. The GCRR Bridge carries approximately 15 trains daily and provides the only rail service into the port of Galveston, TX. Additionally, an average 50 vessels pass through the bridge per day, most transporting petrochemical materials.

Figure 1 – View of the old Galveston Causeway Railroad Bridge

Description of the Waterway

The Gulf Intracoastal Waterway (GIWW) serves ports for more than 1,100 miles between Brownsville, TX, and Apalachicola Bay, FL. The GIWW lies mainly behind barrier islands and provides a channel 125 feet wide and 12 feet deep. In the vicinity of the GCRR Bridge, the channel is 200 feet wide and 12-14 feet deep. With the exception of the Port of New Orleans, the Coast Guard has established minimum horizontal and vertical bridge guide clearances of125 feet and 73 feet, respectively, above mean high water from Pensacola Bay, FL, to Brownsville, TX.

The GCRR Bridge is in a section of the GIWW that consists of two reverse curves and two bridges (GCRR and I-45 Dual Highway Bridges) spaced 760 feet apart. The channel is approximately 200 feet wide on the west side of the GCRR Bridge, narrowed to 104 feet between two timber protection fences, each about 800 feet long, and the old GCRR Bridge, and then transitioned back to a channel that is 200 feet wide east of the bridge. The old bridge was designed for a single barge tow with a maximum beam of 35 feet wide. Modern tow sizes now range up to 1,180 feet long and 108 feet wide.

Navigation Problems

The Coast Guard received many complaints from the marine industry concerning the GCRR Bridge. The majority of these complaints concerned the restrictive 109-foot horizontal clearance provided by the old bridge's navigation span. Over the years, vessel allisions with the GCRR Bridge caused significant damage to both the GCRR Bridge and the involved vessels. The restrictive horizontal opening also caused unreasonable delays to vessel traffic, because significant reductions in speed were required to safely transit the bridge.

The restrictive old bridge navigation span width and the angle of the bridge crossing with respect to the approach channels, prevailing winds, and tidal currents all combined to pose serious problems for tows trying to properly align to transit the old navigation span of the bridge. The bridge is located in an open bay, and both approach channels had sharp bends located 3,000 feet to the west and 2,600 feet to the east of the navigation span. The safe transit through the restrictive old navigation span was further exacerbated by inclement weather conditions.

Elgin, Joliet & Eastern (EJ&E) Railroad Bridge:

Description of the Old Bridge

The Elgin, Joliet & Eastern (EJ&E) Railroad Bridge crosses the Illinois Waterway at Mile 270.6 at Divine, Illinois. The EJ&E Railroad Bridge is a single-track structure, about 780 feet in length, which consists of 4 -150 foot of riveted steel through –truss spans and 180 feet of timber trestle on the north approach. The second truss span from the north was a vertical lift span (Figure 2). The four truss spans were originally constructed in 1895; all as simple fixed spans. Since that time, there have been significant changes to both the navigability of the Illinois Waterway and the size, type and character of the vessels that transit the bridge. In 1933, the second truss span on the north end of the EJ&E Bridge was altered from a fixed to a vertical lift span. This was due to the opening of the Illinois Waterway for barge traffic. The movable span of the bridge over the navigation channel provided a horizontal navigation clearance of 120 feet. The bridge provided 26.3 feet vertical clearance above normal pool elevation in the closed position, and 56.3 feet vertical clearance above normal pool elevation in the open position.

Figure 2 – View of the former Elgin, Joliet & Eastern (EJ&E) Railroad Bridge

Description of the Waterway

The Illinois Waterway is about 327 miles in length and flows in a southwesterly direction connecting Lake Michigan to the Mississippi River. The waterway is navigable for its entire length. Vessels ascending from the Mississippi River enter the Illinois River near Grafton, Illinois; continue to the junction of the Des Plaines and Kankakee Rivers at Mile 270.1, then transit via the Des Plaines River to Lockport Lock, Mile 291.1, where the waterway connects with the Chicago Sanitary and Ship Canal, completing the route to Chicago. The channel from Grafton, Illinois to just below Lockport Lock is generally 300 feet wide and nine feet deep. In the 36 miles from Lockport Lock to the Chicago Harbor, the channel is generally 160 feet wide and 17 feet deep. The Corps of Engineers has constructed eight locks and dams on the main stem of the Illinois Waterway. The Dresden Island Lock and Dam is located approximately one mile upstream of the EJ&E Bridge, with a lock chamber that measures 110 feet wide by 600 feet long.

Navigation Problems

Given the federally authorized and maintained navigation channel both upstream and downstream of the bridge, and throughout the Illinois Waterway is approximately 300 feet wide, the Coast Guard received many complaints concerning the extremely restrictive 120 feet horizontal clearance provided by the EJ&E Bridge. In addition, the position of the bridge near the spillway of the Dresden Dam, shallow water depths near the bridge, and severe cross currents, all combined to pose serious problems, even for the most experienced pilots. Safe transit through the restrictive navigation span often required extensive tow maneuvering. During periods of high water, the narrow horizontal opening made it virtually impossible for tows to transit the bridge without contacting the bridge piers or protection cells. The downbound approach was further complicated by strong currents that move from the right descending bank toward the left descending bank as the result of the overflow from the Dresden Dam entering the main channel above the bridge. Further, a mixed tow of loaded and empty barges was not protected from the wind, thus compounding the difficulty in passing the bridge site. During periods of high flows, use of helper boats and double tripping was often necessary to safely transit the bridge. These practices are not only considered dangerous, but posed severe time and financial restrictions.

Coast Guard Investigation of the Obstructive Character of Galveston Causeway Railroad and EJ&E Railroad Bridges

Mariners' complaints and the frequent collisions that used to occur between commercial vessels and the Galveston Bridge and EJ&E Bridge prompted the Coast Guard to investigate the alleged obstructive character of these bridges. Based on the Coast Guard investigation and the positive ratio between the navigation benefits that will be gained by replacing the draw span of the bridge to the cost of this replacement, it was determined that both bridges are unreasonable obstructions to navigation and their alteration under the Truman-Hobbs Act was necessary to allow vessels to pass through the bridges reasonably free, easy, and unobstructed. The Coast Guard concluded that the alteration of these bridges qualifies for federal funding under Section 3 of the Truman-Hobbs Act (33 U.S.C.A. Sec. 513). On June 18, 2001, the Coast Guard issued to the Galveston County an "Order to Alter" requiring the Galveston County to replace the bascule span of the Galveston Bridge with a vertical lift span that provides at least 300 feet horizontal clearance and 73 feet minimum vertical clearance above ordinary high water when the bridge is in the open position. On February 14, 1995, the Coast Guard issued to the CN an "Order to

Alter" requiring the CN to replace the old vertical lift span of the EJ&E Bridge with a new vertical lift span that provides at least 300 feet horizontal clearance and 60 feet minimum vertical clearance above ordinary high water when the bridge is in the open position.

Design of Galveston Causeway and EJ&E Railroad Bridges

The Bridge Program Office of the Coast Guard has overseen the alteration of the Galveston Bridge and EJ&E Bridge and worked closely with the bridge owners to develop the plans, specification, and bid documents of the new bridges. Modjeski and Masters, Inc., New Orleans, LA was selected for the design and the construction engineering services of these projects. The goal of the design was to find the least costly scheme that would serve the present and future navigation needs. A construction sequence was developed to remove the existing draw spans and install the new lift spans of both bridges while minimizing the impacts on rail and marine traffic. The existing bridges were to remain in service during the construction to carry the trains that cross the bridge daily. A balanced construction plan was developed that would allow the contractor a reasonable amount of free track time and at the same time would not severely impact the rail and marine traffic. The new structural steel lift spans were designed to be assembled off line near the bridge. Span change-out methods were recommended by the consultant to replace the old draw spans of the bridges with new lift spans.

The Coast Guard set several safety rules for the contractor in order not to endanger or interfere with the movement of trains or vessels. Also, the Coast Guard set the conditions that the contractor must follow to keep the navigation channel open throughout the duration of the construction, except for very few defined periods that were allowed to the contractor to complete critical construction activities.

Construction of Galveston Causeway Railroad Bridge

The project was advertised for construction in December 2009. Bids were opened on February 25, 2010. The construction contract was awarded to Cianbro/Brasfield & Gorrie Joint Venture (C/B&G JV). Galveston County issued Notice to Proceed to the contractor on June 01, 2010. The project was substantially completed on January 17, 2013 including the relocation of the two water pipelines carried by the old bridge and the removal of the old bascule span.

The lift span was erected on cast-in-place concrete piers and footings located under every other panel points at the C/B&G JV span yard located on the shore approximately two miles from the bridge, see Figure 3. In the days leading up to the float-in and after the vast majority of the permanent material had been installed on the lift span, C/B&G JV jacked the lift span at the endfloor beams using 565 TN jacks and 500 TN load cells in order to make a final weight calculation and be able to make any necessary balance block adjustments prior to float-in.

Figure 3 – Lift span erection at span yard

Figure 4 - Lift span loading onto barge at bulkhead

Eighty two hours of marine closure and 16 hours of train traffic closures were provided for the contractor to float out the existing bascule span and float in the new lift span and placed it in motion at its location over the piers. The lift span was moved approximately 1300 ft from its erection stand to the load-out bulkhead with self propelled modular transporters. When the lift span arrived at the bulkhead, the float-in barge was docked, waiting to be loaded, see Figure 4. Once the span was loaded on the barge, it was floated to the bridge site, see Figure 5.

Figure 5 – Lift span floating towards its final position

Prior to placing the new lift span over its bearings, it was necessary to remove the bascule span's operating rack and pinion facing the approaching float-in barge to provide clearance for the lift span to approach its final location. To allow room for the barge to be positioned under the bascule span while unloading the lift span, the bascule span was raised 37 degrees, see Figure 6. Once the lift span had been transferred from the float-in barge and placed on its bearings, the float-in barge was removed and the bascule span was lowered on only one pinion to allow the movement of train traffic until C/B&G JV completes all work required to place the lift span in motion. Between hours 16 and 76 of the 82 hour marine outage, the C/B&G JV removed the counterweight of the bascule span in preparation for floating the span out. Prior to float in of the lift span, its counterweight ropes were hung on the sheaves and marked according to their proper location on the lifting girders. All 64 Rope Sockets were installed during the closure period, followed by the keeper angles. By the time the 82-hour marine closure expired, the lift span was raised to allow the old bascule to be floated out and the channel was reopened to marine traffic, see Figure 7.

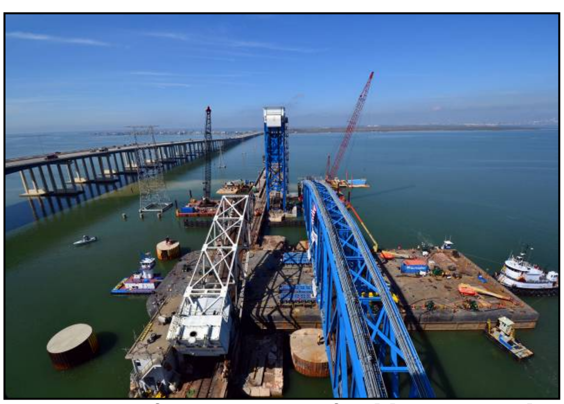

Figure 6 – Lift span near its final location and float-in barge under existing bascule span

Figure 7 – Lift span in motion and old bascule span floated-out

Construction of EJ&E Railroad Bridge

The project was advertised for construction in March 2009. Bids were opened on July 9, 2009. The construction contract was awarded to James McHugh Construction (JMC) Company. Canadian National issued Notice to Proceed to the contractor on October 19, 2009. The project was substantially completed on December 8, 2011 including the removal of the old vertical lift span.

A launch method was utilized to install the new lift span. The new 348-foot through-truss vertical lift span was erected on the north shore of the river, atop of a temporary steel trestle set parallel to and at a distance of 42 west of the center of the existing railroad. The temporary trestle was over 600 feet long and served also as the truss launch and receiving platform. Also, the existing dolphin cells on each side of the navigation channel served as points of launch and receive of the truss. See Figure 8 of the construction site prior of the launch operation.

Figure 8 – Construction Site prior to the launch operation of EJ&E Railroad Bridge

The span was launched across the river channel from north to south to the receiving point over the dolphin and then to a temporary receiving structure using hydraulic skid shoes. The span was then jacked up and rolled from west to east into its final alignment. A temporary 29 feet launch nose was designed to extend the reach of the truss across the navigation channel to bear at the receiving point on the dolphin, see Figures 9 and 10.

Figure 9 – Launch of the new lift span of EJ&E Railroad Bridge

Figure 10 – Lift span ready to be launched into its final alignment

JMC was allowed a single 84 hours river and train closure period to complete the change-out operation and place the new lift span in motion. The removal of the old lift span and its adjacent fixed truss spans was included in the 84 hour closure period. The removal of these spans needed to occur in less than 24 hours in order to allow sufficient time for the entire span change to be completed within the 84 hour schedule. Explosive demolition was not an option as the new adjacent foundations, lift towers, and temporary trestle could not be subjected to risk of any damage. Advanced planning was developed in order to expedite the removal of the old lift spans and its adjacent truss spans. All torch cutting locations were pre-marked and hoisting riggings were pre-installed to catch the span sections being cut and removed. See Figure 12. The change-out operation was successfully completed and JMC continued the post change out work to complete the project and deliver it to CN.

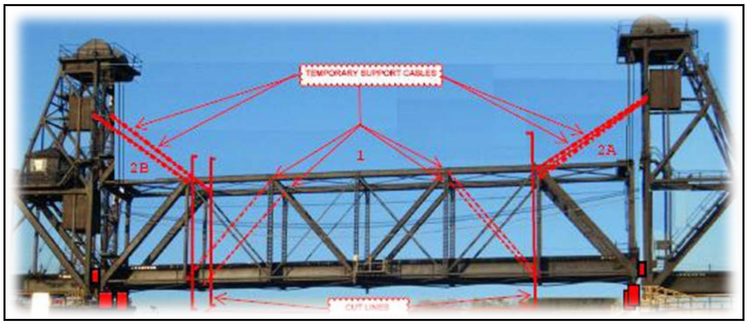

Figure 11 – Removal of the old lift span

Movable Bridge Selection

Movable bridge selection must consider a wide variety of factors. The two subject bridges were replacing existing shorter spans. The length of span and the staging required to work around the existing movable span made the vertical lift most desirable. Rest piers and towers could be erected around the existing structure. The counterweights could be hung and poured in their upper position in the tower (see Figure 12). The lift spans could be brought into alignment, connected to the counterweights, and made operable during relatively short outages.

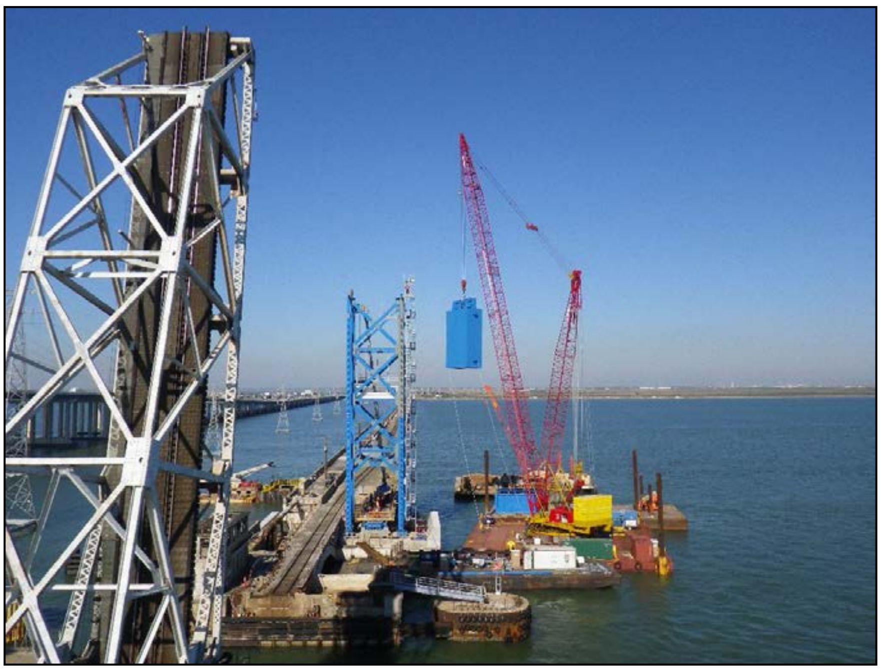

Figure 12 – Galveston counterweight box being installed in tower.

Vertical Lift Bridges - Tower Drive vs Span Drive

There are two predominant styles of vertical lift bridges, tower drive, and span drive. The driving factors for the different styles chosen are many and must be evaluated on a case-by-case basis. The evaluation is not typically cut and dry. Site conditions impact bridge style. Owner preference is a major component. A list of considerations during design and discussion with the bridge owners is shown in Table 1.

Table 1 – Comparison of Tower Drive and Span Drive Vertical Lift Bridges

Vertical Lift Type	Pros	Cons
Tower Drive	• Smaller Machinery • No operating ropes • Machinery away from operator house • More space for NEC • Machinery Enclosed • Cost??	• Larger Distance to access machinery • Electrical controls between towers needed • Electrical power to motors in two separate locations • Align before and after dead load • 2 sets of machinery • Large ring gears • Need more personnel to operate on emergency
Span Drive	• 1 set of machinery • Less likely to align machinery twice • Less electrical control required – reliability, skew • Electrical power to drive and motor at single location • Little to no open gearing • Single man emergency operation • Cost??	• Larger Machinery • Operating ropes maintenance • Machinery near operator house • Less space for NEC

Mechanical Design Summary

The Galveston and EJE bridges were required to have the same navigational channel width. Construction staging around existing elements made the span lengths slightly different. Similar operating times for lift heights of 65 feet and 30 feet resulted in a significant difference in horsepower requirements. See Table 2 for a comparison of key mechanical design characteristics.

Table 2 - Comparison of Mechanical Design Features for Galveston and EJE Bridges

	Galveston	EJE
Channel Width	300 ft	300 ft
Lift Span Length	383 ft	348 ft
Width Between Trusses	20 ft 6 in	18 ft
Weight	3,150 kips	2,300 kips
Lift Height	65 ft	30 ft
Time of Operation	1.7 minutes	1.5 minutes
Main Motor Total Horsepower	120	50
Auxiliary Motor	7.5 HP (1/10 speed)	15 HP (1/4 speed)
# Main Counterweight Ropes	64	48
Size Main Counterweight Ropes	2 1/4" Dia. EIPS	2 1/4" Dia. EIPS
Main Counterweight Sheaves	15 ft PD	15 ft PD

Galveston Tower Drive Summary

Galveston is a tower drive vertical lift bridge. A similar set of operating machinery is housed in each tower to raise and lower the lift span (see Figure 13). Main counterweight ropes are connected to the lift span, pass over the main counterweight sheaves in the towers, and connect to a concrete filled steel box counterweight hanging in each tower. Lift span movement originates from an open gear mesh between the main pinions and ring gears bolted to the main counterweight sheaves (see Figures 14 and 15).

Figure 13 – General elevation of the Galveston tower drive vertical lift bridge. Span drive machinery is housed in each tower (red ovals).

The span operating machinery is placed during tower construction. The top level of the tower receives the span operating machinery. The machinery is aligned temporarily before the application of the lift span and counterweight dead load. Once the lift span load is transferred to the counterweight ropes, the final machinery alignment takes place.

Load sharing and span skew from tower to tower is controlled electronically with the motor drives. Load sharing within one tower is accomplished by a custom differential gear reducer.

Figure 14 – View of main counterweight sheaves being installed in the Galveston Towers. Note large ring gears bolted to sheaves (red arrow) and machinery support beam (white arrows).

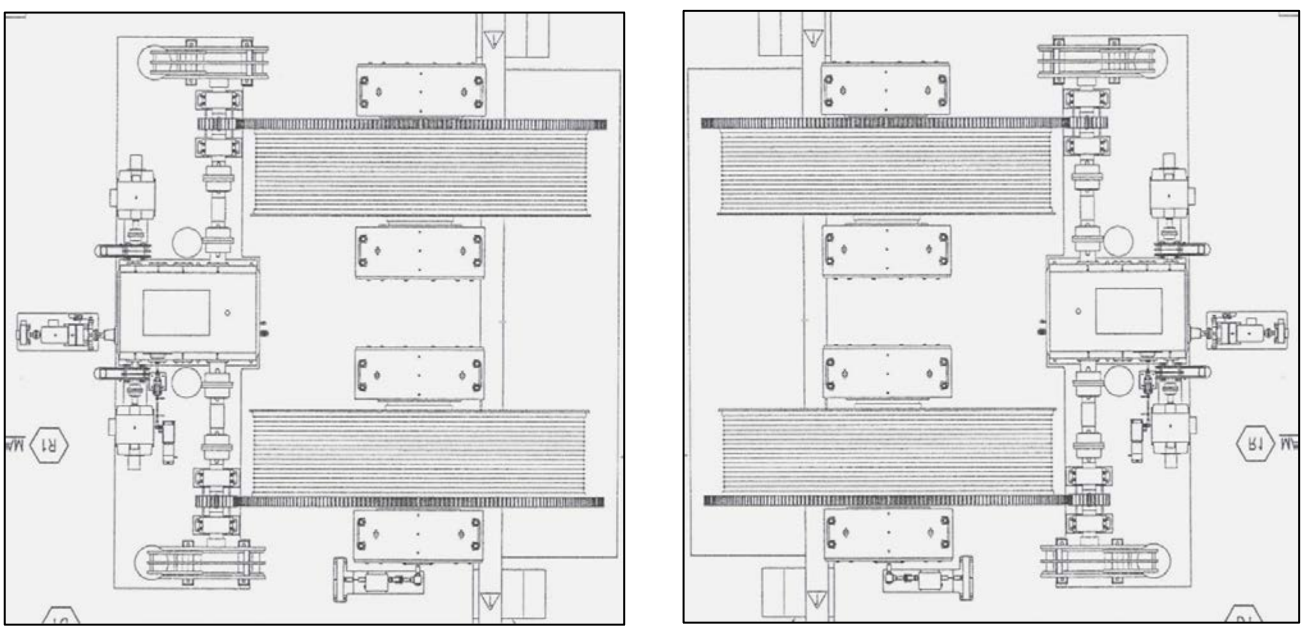

Figure 15 – General plan view of Galveston span drive machinery – West tower on left and East tower on right.

EJE Span Drive Summary

EJE is a span drive vertical lift bridge. This span drive example locates a majority of the machinery and electrical at the center of the lift span above the truss top chords (see Figures 16-20). A single motor drives 4 operating drums using a primary parallel shaft reducer and two secondary right angle gear reducers. The operating drums are double-reeved to pay out a pair of downhaul ropes while they take on a pair of uphaul ropes when raising the span and vice versa when lowering the span. The wire ropes run along the top chords of the lift span and wrap 90 degrees around deflector sheaves at the end of the lift span. The terminations of the uphaul ropes are near the top of the the tower and the downhauls are attached near the bottom of the tower. The span is pulled up or down. The machinery can be installed and aligned prior to float in. If the lift span can be supported similar to the final position when machinery is installed, very little change in alignment is expected.

Figure 16 – General view of EJE span operating machinery located on the lift span.

Figure 17 – General view of deflector sheave on the EJE span drive bridge.

The haul ropes act as the load sharing and skew control for this type of bridge. The tension of the haul ropes dictate how load is shared and the operating behavior of the lift span. Electrical controls are not necessary for skew control and tower to tower load sharing. In addition, a custom differential gear reducer is not required.

Figure 18 – EJE main counterweight sheave. Tower is open and sheave covers will be installed. No span drive operating machinery is located in the tower.

Figure 19 – General elevation view of the EJE span drive bridge. Note large machinery house on lift span (red arrow).

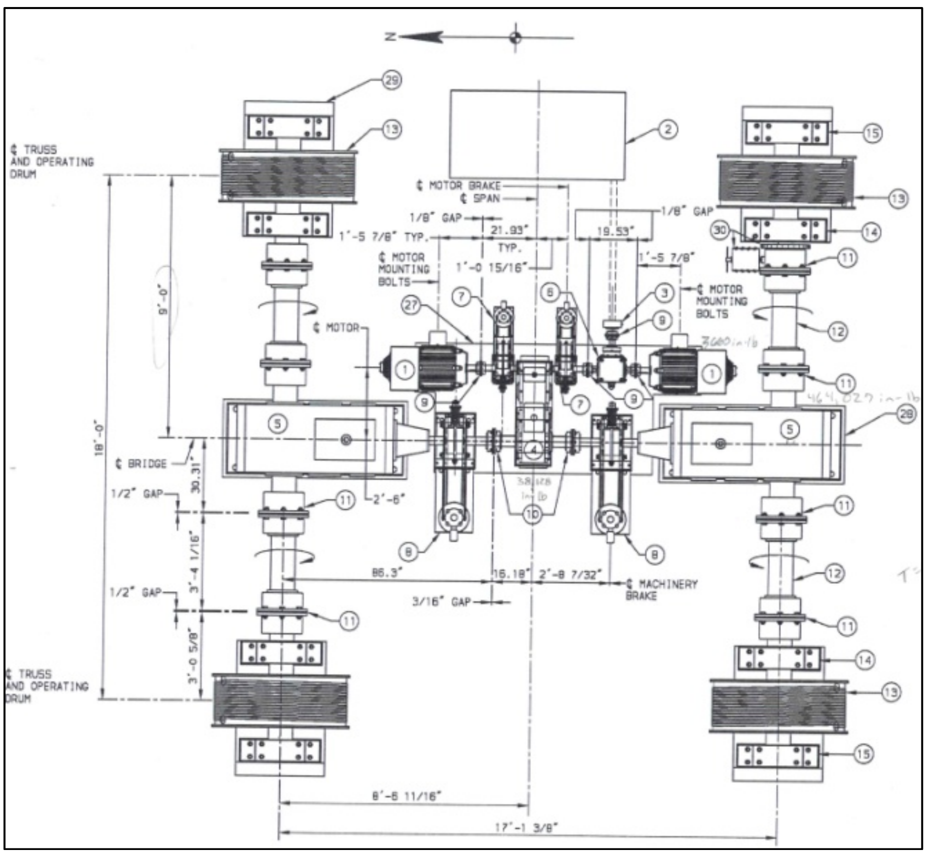

Figure 20 – General plan view of EJE span drive machinery

Construction Challenges and Design Features

Maintaining Existing Bascule Bridge Operation

The new Galveston bridge replaced a rolling bascule, and the new bridge is less than 25 feet off the old bridge alignment. A float-in was performed to install the new lift span. Existing bascule span operation had to be maintained during and after float in to provide adequate barge access and meet the railroad operating outage (see Figure 21).

The close proximity (only inches of clearance between the two) of the new alignment required that the near side rack frame and rack gear be removed prior to lift span float in. As a result, the existing bascule span would have to operate using only 1 of 2 drive pinions. Since the bascule span was designed to be operated with two pinions, there were concerns about operating on a single pinion.

Figure 1 – General view of close proximity between new and old Galveston bridge alignments. Near side rack frame is in the process of being removed (see red oval).

The elimination of the near side rack and pinion significantly reduced the load capacity of the original operating mechanism of the bascule span, which was designed to be operated in a symmetrical, load sharing fashion using a differential type gear reducer (see Figure 22). The change-out sequence required that the bascule span be operated one time in this crippled fashion. The bascule span needed to be raised before float-in of the lift span to provide vertical clearance for the barge carrying the lift span into position, and then it had to be

lowered one more time after setting the lift span and removing the barge, so that the bascule span could carry rail traffic for two more days until the bascule span was removed entirely and all rail traffic was transferred onto the new lift span.

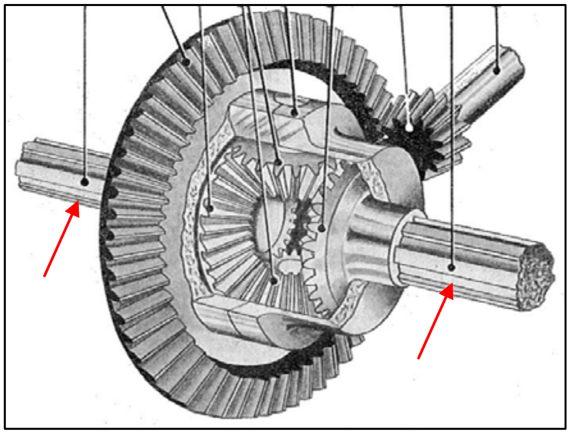

Figure 22 – Section view of a differential gearing assembly. Relative rotation between output shafts (red arrows) is possible when loads are unequal. Holding one output shaft stationary would double the speed of the moving output shaft and reduce the amount of output torque 50%.

The concerns among the project team, including the Railroad, the contractors and the designers, were the need to address the fact that an active differential in the gear reducer remains after one side of the gear train is demolished, span balance needs to be measured and adjusted as necessary, and will the bascule span "track" appropriately while operating on 1 of 2 rack and pinions. A simple device was designed and installed to lock the output shaft of the primary reducer on the side of the removed rack and pinion (see Figures 23 and 24). After having found that the span was significantly counterweight-heavy, weight was added to the toe of the bascule span to adjust the span balance. A satisfactory balance condition of the bascule span was confirmed by determining the torque of the drive machinery with strain gages. The single pinion operation was conducted at a slow operating speed. The stiffness of the bascule span and counterweight structure helped to ensure good tracking of the span. As a result of good planning, a relatively simple approach provided an efficient and cost effective alternative for this phase of the float in.

Figure 23 – Output shaft lock installed at coupling.

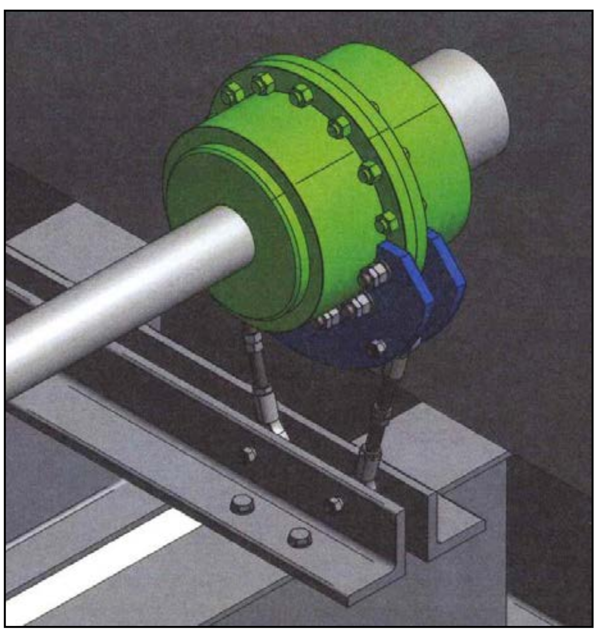

Figure 24 – 3-D model of output shaft lock during design.

Rigging Main Counterweight Sheaves

The main counterweight sheaves are the largest piece of machinery on vertical lift bridges. The Galveston sheave assemblies topped out at 163 kips (with trunnion shaft and bearings) each. Not only is this a heavy lift, but the lift is to the highest point on the project site and the assembly geometry requires some custom rigging. This assembly is one of the controlling cases for crane sizing for the entire project.

Figure 25 – Galveston main counterweight sheave utilized a custom jig to install the final rigging.

The asymmetrical feature created by the ring gear typically creates additional challenges. Some projects have required a ring gear segment be removed to accomplish the lift. At Galveston a custom jig was required just to install the final rigging bar to perform the lift (see Figures 25 and 26).

Figure 26 – Final rigging in place for main counterweight sheave at Galveston.

Counterweight Wire Ropes

Wire Rope Length

Part of typical vertical lift bridge installation is working with wire ropes (see Figure 27). Typically the ropes are installed at the counterweight connection ahead of time and tied in place on the span side of the tower in preparation of span float in. Wire rope has a lot of unique properties not found in any other movable bridge machinery. These properties have to be appreciated and accounted for in design and erection.

Wire rope has elastic stretching behavior which results in lengthening due to an applied tension. This amount is straight forward and can be calculated based upon the tension and the modulus of elasticity of the wire rope. A 150 foot wire rope loaded to 1/8 of its ultimate strength will stretch approximately 4 1/2 inches.

Figure 27 – Barge filled with wire rope at the Galveston project.

Wire rope must be pre-stretched after fabrication in the shop to attempt to remove construction stretch. Construction stretch basically has to do with the strands settling into place on the inner core. This amount of lengthening is harder to predict and is typically assumed to be 0.25% to 0.75% of the rope length. A 150 foot rope may experience 13 ½" of construction stretch prior to socketing in the shop. After pre-stretching in the shop the ropes are wrapped on spools and shipped to the site. Ropes are hung under no load prior to float in. It is expected that some construction stretch relaxation will occur prior to float in.

With the two forms of stretch occurring in the vertical lift bridge application, our example could result in 18 inches of rope length change during erection. The jacking and float in plan must accommodate these changes in length while supporting and transferring large lift span weights.

Wire Rope Twist

Wire rope manufacturers are required to stripe the full length of the wire rope under tension at the proper twist. This stripe is a critical reference to the installer in the field. The wire rope has a tendency to relax and during relaxation the free socket will "untwist" relative to the connected socket at the other end of the rope (see Figure 28). When the span connection is made, the installer must use the stripe to ensure the rope twist, and therefore the length, is correct. The stripe is most visible when oriented outward toward the approach and channel when installing. Once the field dressing is applied the stripe is very difficult to see.

Figure 28 – EJE main counterweight rope sockets at the lift span connection during float in and load transfer. The Contractor used wood wedges to maintain socket alignment with the bearing surface of the castings.

Span Guides

Span guide machinery is typically part of the float in activities since their location has them tightly aligned to the lift span and tower. At least one truss will have the guides removed for float in. What makes this assembly even more critical to focus on is the cross-discipline coordination of details. Span guide clearance once installed is typically about 3/8” in the longitudinal and transverse directions. Adjustability is required to provide the proper clearance at the installation temperature and considering the erection and fabrication tolerances of the lift span and tower legs. The EJE span guides were made up of steel roller and plain spherical bushings to provide for some degree of angular misalignment between the roller and guide rail. The roller assemblies were supported by eccentric shafts to allow for +/-1/2” radial adjustment of roller position with respect to the guide rail on the tower (see Figures 29 and 30).

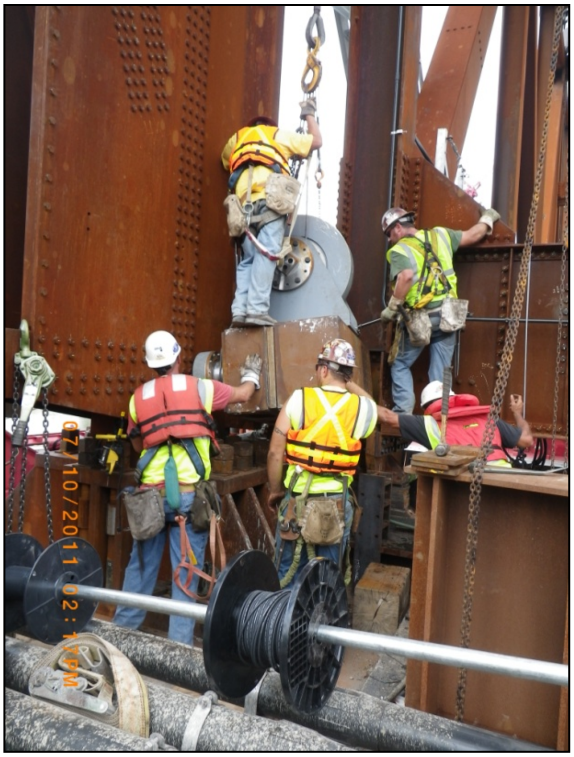

Figure 29 – Contractor installing EJE span guide assemblies after float in.

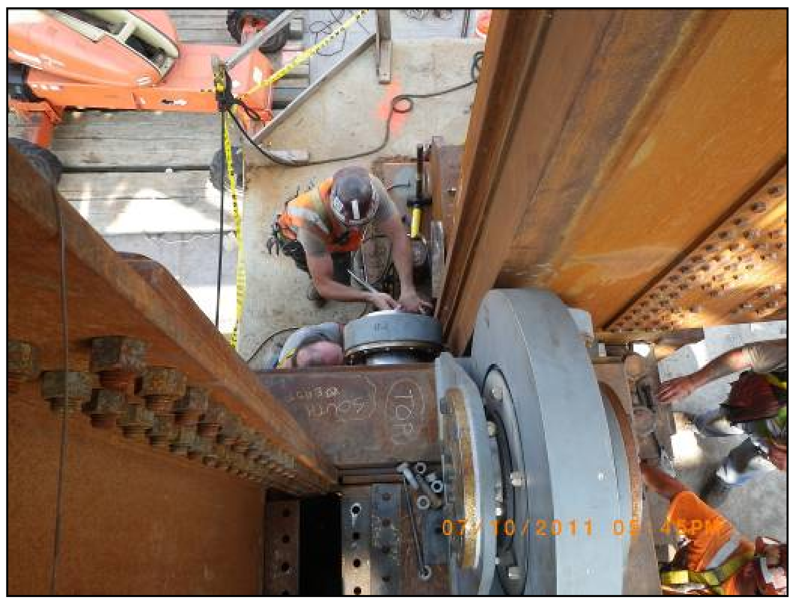

Figure 30 – Contractor adjusting EJE span guides for proper gap with the tower leg mounted span guide rail.

Emergency Drives

A non-electric or even a low horsepower emergency drive which can be powered by a small portable generator is a necessity when commissioning movable bridges in short outage windows. A hydraulic version was provided on the EJE project (see Figure 31). A small 3 cylinder air cooled diesel engine powers the unit which supplies pressure and flow to a hydraulic motor coupled to the auxiliary input of the primary gear reducer. Hydraulic piping or hose is installed between the power unit and the hydraulic motor. This provides flexibility to locate the power unit somewhat remotely with respect to the main drive machinery.

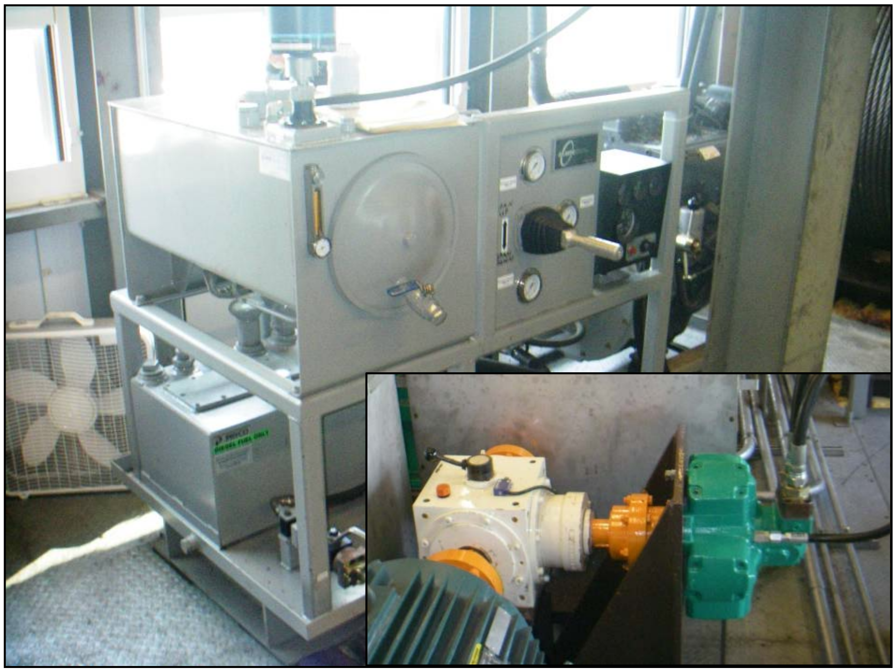

Figure 31 – EJE emergency drive hydraulic power unit and auxiliary input of the main operating machinery for the hydraulic motor (inset photo).

Manual joystick controls allow for very simple operation. Hydraulic circuitry includes relief valves to prevent overloading and to allow the bridge to be seated positively on the live load bearings. This unit was used for spin testing the equipment prior to float-in, indexing the drive machinery during operating rope installation, and for the first few days of operation until the electric drive could be brought on line. Emergency drives are typically anywhere from 1/5 to 1/10 the speed of the main drive. All other machinery was operated by hand such as the engagement clutch, brakes, and span locks.

Span Locks

Span locks are typically provided for locking the lift span in the closed position. Vertical lift bridge span locks are not required to be a tight clearance fit with their receiver on the lift span. The function of the lock is to prevent lift span up lift and excessive vertical misalignment of the lift span and approach rails/decks. There is no live load transfer at the span locks.

The new EJE bridge employs span locks which operate using an electromechanical linkage (see Figure 32). The linkage is designed to provide 360 degree failsafe rotation without binding in the event of a limit switch failure. Prior to raising the lift span the locks pull. When the lift span is lowered, and at nearly seated position, the locks are driven. As the lift span reaches the fully seated position, a tapered receiver slides over the spring-loaded lock jaw forcing it back temporarily until it springs back into locked position as the lift span becomes fully seated. This passive span lock simplifies seating of the lift span in terms of control logic.

Figure 32 – EJE span lock assembly prior to installation on the rest pier. Spring loaded link (red arrow) and 360 degree eccentric crank (red oval).

Span and Counterweight Weight/Balance

Span and counterweight weighing is part of preliminary balancing and is usually part of the necessary work as the Contractor erects the span and pours the counterweights. This occurs prior to float in and before the wire ropes are connected. Calibrated jacks are typically used for this task (see Figure 33). It should be noted that jacks of this size will have a 2% error and this can be a significant amount when dealing with 3,000,000 pounds (60 kips error). Load cells can be used as well and can provide tighter tolerances in the weight.

Figure 33 – Large jacking assembly required for load and stroke when transferring lift span and counterweight loads to the wire ropes.

Often times, desired balance conditions are much less than the error that can be assumed in loads read from calibrated jack pressures. Contingency plans become necessary. Adding pockets in addition to the specified adjustment pockets to provide adjustability after float in is an option that is sometimes employed. Actual cured density of counterweight concrete is a variable in the overall balances too. Lead or steel shot have been used in the final pour of concrete to increase the density of these portions. Care must be taken to provide symmetry about transverse and longitudinal centerlines of the counterweight.

After the wire ropes are connected the lift span and counterweights are all part of a dynamic system. This dynamic system includes friction effects and rope weight transfer. As soon as practical after the connection of the ropes it is advisable to perform strain gage balance testing. Friction can be factored out of the data, and if performed under calm weather conditions this method will determine the actual imbalance in all positions of the lift span. This will be the first opportunity to adjust the balance condition with certainty.

Machinery Modules

Whenever possible, machinery assemblies should be palletized or mounted on a common support as a module. This creates an opportunity to perform an alignment in the shop. Many times this alignment can be finalized if the interface between the common support and the bridge structural steel has an appropriate balance of rigidity and adjustability. In the field at final installation the Contractor can pick the entire assembly as one. Rigging does get more complicated and a detailed plan needs to be in place, but the time savings realized by avoiding a majority of fine tune machinery alignment makes this approach very worthwhile.

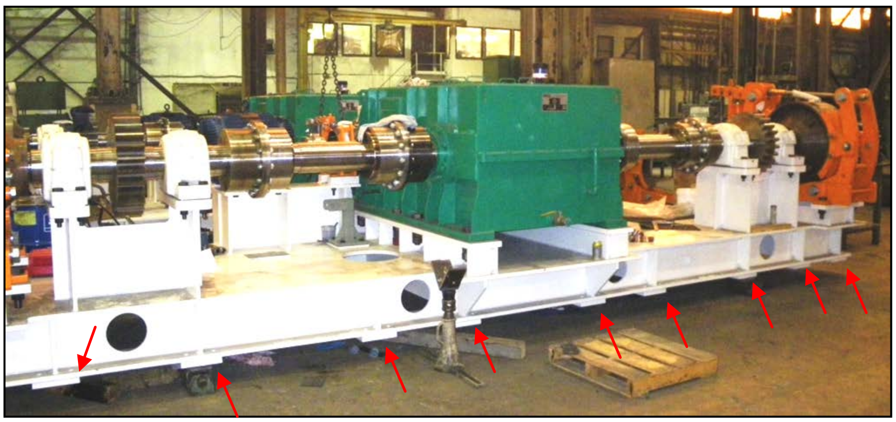

Figure 34 – Galveston span drive machinery module. Turned bolts were installed after alignment in the shop. The machinery pads (red arrows) under the module provide shimmed and bolted connection to the tower top steel.

In the case of Galveston, a common support was used for each span drive assembly (see Figure 34). 1 ¾" thick steel pads were welded to the bottom of the support at locations where the structural steel would be on the bridge.

All pads were machined flat with one another after stress relief. The common support was 24 ft long and 4 ft wide weighing approximately 15,000 pounds.

Miter Rail Joints

There were some minor issues on both jobs related to coordinating the actual details of the manufacturer's joints, the required steel tie details, and the existing timber ties and track details/elevations to make everything match up. All parties need to be sure to coordinate all of these details in advance to avoid reworking any pieces to make everything fit-up correctly. These issues were not the result of the particular joint type, but are due more to the fact that different joint types and manufacturers and owners have and prefer different details with different dimensions. This includes features for the joints, bed plates, shock pads, steel ties, shims, and the existing track component details. It is crucial to solve these problems before float-in to avoid delay. A major hurdle is determining who should be responsible for corrections needed to make all components fit-up properly.

Figure 35 – General elevation view of miter rail joint during installation at the EJE bridge.

The sooner the type of joints and associated component details are known, the sooner the coordination can take place and the more likely that nothing will have already been fabricated or ordered to the wrong dimensions (see Figures 35 and 36).

Figure 36 – General plan view of miter rail joint during installation at the EJE bridge.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Development of 3D Contract Documents for a Spillway Taintor Gate

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Introduction

3D Drafting is a tool that is already utilized in many design oriented industries. The following is intended to discuss some of the logistics and benefits that go along with implementing 3D Drafting into projects specifically for Heavy Movable Structures. A few different software platforms will be discussed along with the added capabilities and challenges. Another aspect that will be addressed is the challenges that will need to be faced by the clients, consultants, and contractors in implementing this new tool.

For the purpose of this paper, the rehabilitation of the Green Peter Dam taintor gate wire rope hoist replacement for the Army Corp of Engineers will be discussed . The contract documents were generated using 3D Microstation and AutoCAD Inventor. The example will be used to discuss the challenges and lessons learned as it pertains to 3D drafting.

Project Description

Green Peter Dam is a hydropower facility owned and operated by the Portland district of the Army Corp of Engineers. It is located along the Santiam River at the base of Green Peter Reservoir in Linn County, Oregon. The function of the structure is to generate hydroelectricity, prevent flood damage, provide water for irrigation and improve water quality.

As part of the flood prevention function, the dam structure has a spillway with two radial taintor gates. Each gate is flanked by an outer pier and a shared center pier. The outer piers support the outboard drum assembly and outboard trunnion assembly for each respective gate. The center pier supports the operating machinery, inboard drum assembly, and inboard trunnion assembly for both gates (Figure 1). The operating machinery for each gate consists of a single 10 HP motor with primary and secondary single stage worm gearboxes and one set of open spur gearing.

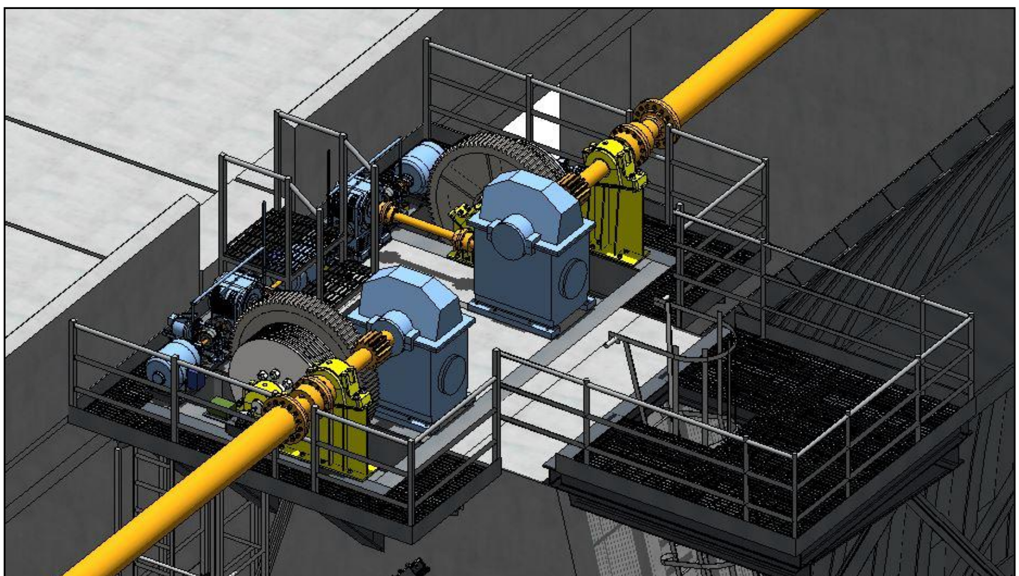

Figure 1: Operating machinery on the center pier

The rehabilitation of two taintor gates included structural repairs, trunnion rehabilitation, and a complete mechanical and electrical operating system replacement. The design was a modification to an earlier design contract. The earlier contract included the structural repairs and trunnion rehabilitation. As part of the contract modification, the mechanical systems were to be modeled in 3D using Bentley Microstation.

3D Modeling of a Taintor Gate Hoist

The first step in beginning a project is to assign the personnel with the appropriate technical training. Most of the staff had experience with other 3D software like Solidworks and Inventor. These software packages are more geared towards developing 3D assemblies than Microstation. To better aid in the production of the contract drawings, methods where researched using Microstation online forums and internal procedures were developed.

In order to meet the accelerated project schedule, it was imperative that the drafting work was planned so that the appropriate personnel were utilized as effectively as possible. In this vein, the work was split up so that a portion was done in Inventor and then converted to Microstation. The complexity of the drum assemblies made this portion of the work suited to Inventor The remaining of the structural and mechanical modeling was done in Microstation 3D.

Microstation 3D

Creating 3D models within Microstation is not very much different from working with 2D line work. Operations such as extrusions, creation of simple 3D geometric objects, and solid element additions and subtractions, known as Boolean operations, are additional tools needed for 3D drafting and can be learned with very little training.

The existing dam structure model was created first in 3D. A couple of things needed to be kept in mind during the creation of this model. The first is that existing components that were to be replaced would have to have the capability of being "turned on/off". This was so that the existing and proposed components could be shown on the same model and then just turned on and off as needed for additional views. The mechanism chosen to do this was to put each component into its own model and then reference it into an Assembly model. Another consideration was the appropriate level of detail to show in the existing structure. Showing too much detail becomes time consuming and increases file sizes. Too little detail may make the 3D model inaccurate which defeats one of the major advantages of working in 3D, identifying potential interferences.

After the existing structure was modeled 3D models for the new components were created. A lot of manufacturers have made 3D models of their products available. They were generally not in Microstation format, but are easily converted. All designed and fabricated components, such as shafts, were created from scratch. Each component was detailed within its own model and then referenced into a sub-assembly model. These were then referenced into the main assembly model with the existing structure.

This is a very simplified explanation of how this 3D model was created. When working with a large structure, such as a dam, there are a lot of individual models that need to be generated. The client standards and our own internal standards did not provide direction on how to organize these models. The designer should consider when beginning a project like this:

- What should the physical boundaries of the model be?
- How complex do the models have to be? File size can quickly become too large for standard computer processors to effectively handle model changes or manipulation.
- Organization of how models are referenced into assemblies is key.

- Can a level convention be adopted that will help with organization and still comply with client standards?

Autodesk Inventor

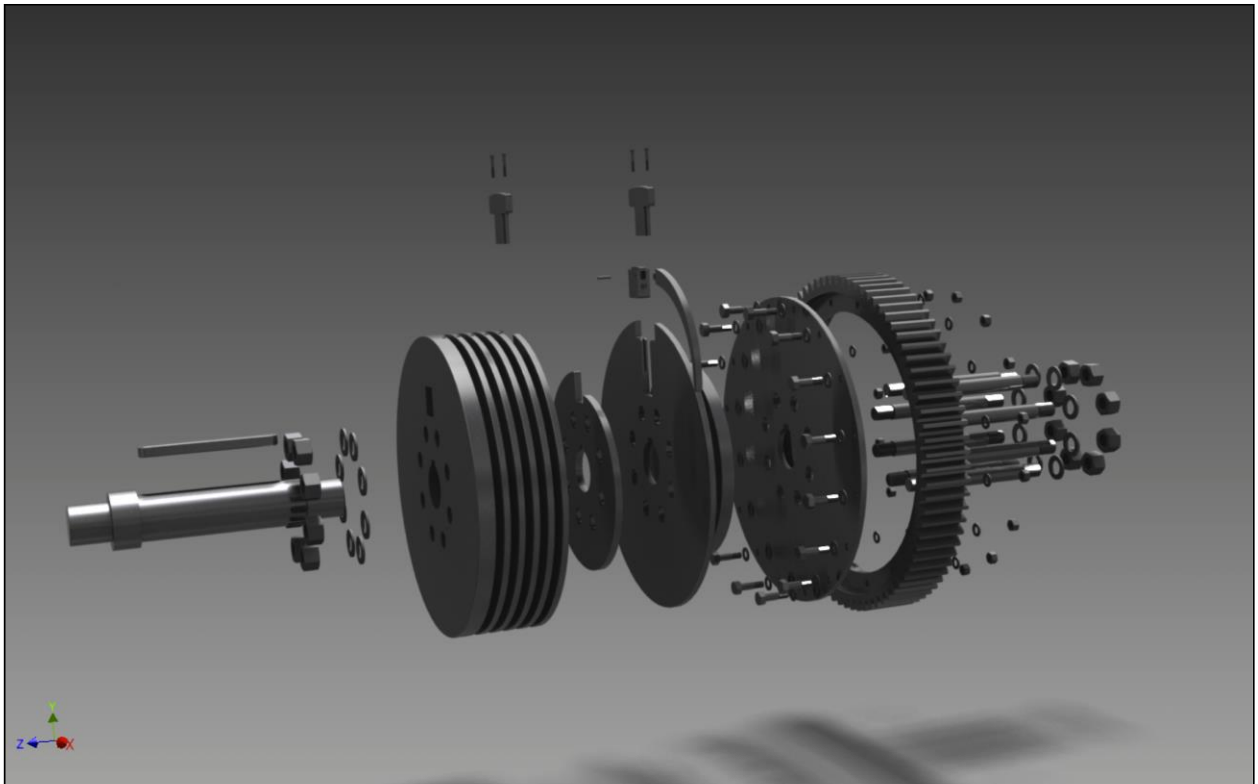

Figure 2: Exploded view of the drum

The drum model was created in Inventor to allow fluidity of design changes and simplicity when assembling many parts. Like any CADD software the drawing starts with 2D sketches. The sketches then are extruded into the individual components and modified by adding and subtracting material. The individual parts are arranged and linked together to form assemblies. Inventor has the ability to link dimensions between parts while maintaining the assembly automatically. This allows rapid changes to the any part and all associated components. The assemblies can be modified using a bottom up approach where each individual component is dimensionally modified, or using a top down approach where the assembly itself is dimensionally modified and added to while the individual components adjust to accommodate these changes. For example, the ribs on the weldment drum gear were generated by tracing the void between the rim and the hub as a 2D plane (Figure 3). A thickness was then defined and the number needed is specified filling in all of the stiffeners (Figure 4).

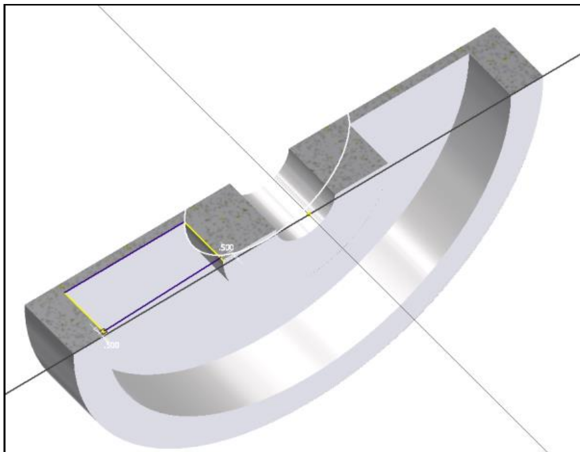

Figure 3: Sketch of the gear stiffener.

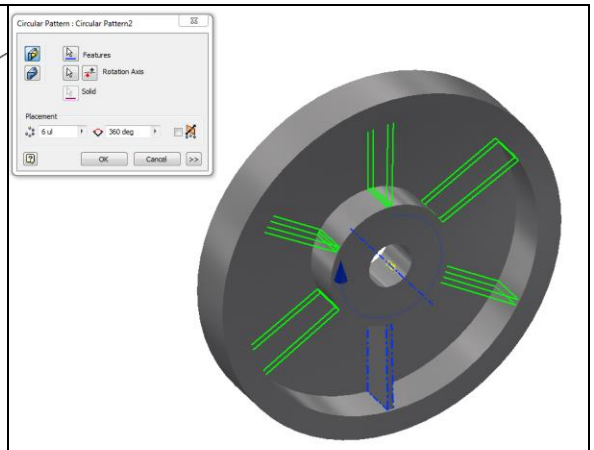

Figure 4: Setting up the stiffener quantity for the drum gear.

Since multiple drawing platforms were used and not everyone working on the project was an expert in each frequent communication was required to maintain organization. To interface with other designers on the project who were not using Inventor, 2D sketches and 3D models were exported from or imported to Inventor. Doing so helped ensure that everything would fit together seamlessly on the first try. Below is an example (Figure 5) of a cross section used to discuss developing the capacity of the keyway within the plates and an interference between the drum bolts and bearing. A drawing like this can be generated in minutes and useful to convey design issues and help make corrections.

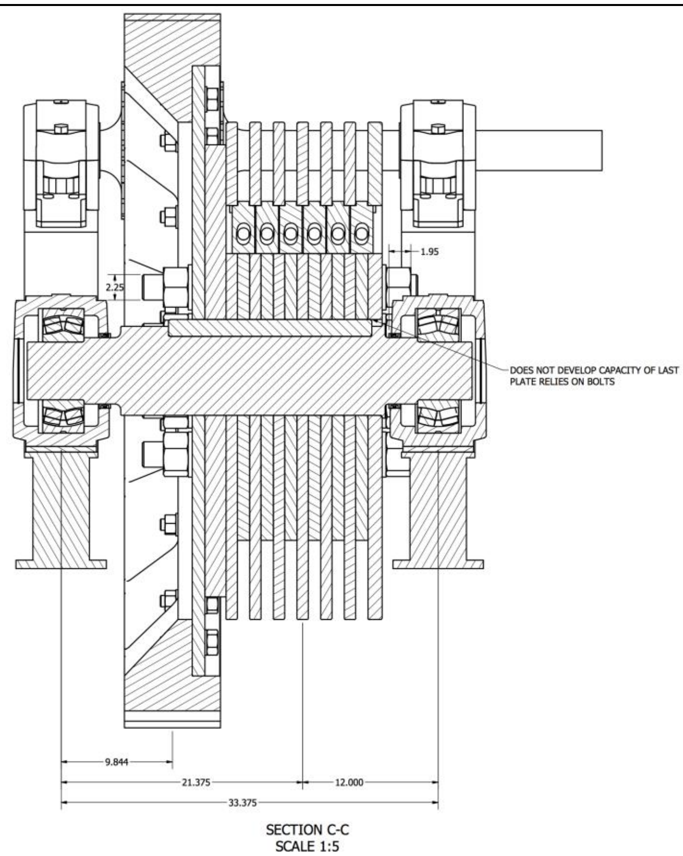

Figure 5: Section of the assembled drum used for discussion showing an interference between the drum bolts and bearing as well as a lack of key engagement.

Creating Contract Documents in Microstation

Creating a 2D set of contract documents from the 3D model was one of the more complex processes involved in the project. This process requires a meticulous organization of the models involved as discussed previously. Once the models are generated and organized, 2D views and section can be created from these 3D models to be referenced into contract documents.

Saved views are the way in which Microstation creates 2D views from 3D Models which generate 'flat' line work for annotation. These views let you set a number of parameters. The view direction can be selected such as top, front, side, isometric, or a user defined view. A clip volume can be assigned to the saved view so that the depth of the view shows the appropriate line work. The display settings can be adjusted. These settings can show line work with hidden lines, without hidden lines, or with section cuts. Once these are set, a saved view can be generated and referenced into sheet files for line work. Now the line work is ready for annotation.

Benefits of 3D Contract Documents

One key benefit of working with 3D models is the ability to communicate the complex assembly of multiple components. The drum assembly is a good example of a complex assembly. A supplemental detail that was supplied with the contract documents was an exploded isometric view of the drum assembly (Figure 6). This clearly shows how the assembly fits together and would be very difficult to recreate to scale with traditional 2D drafting.

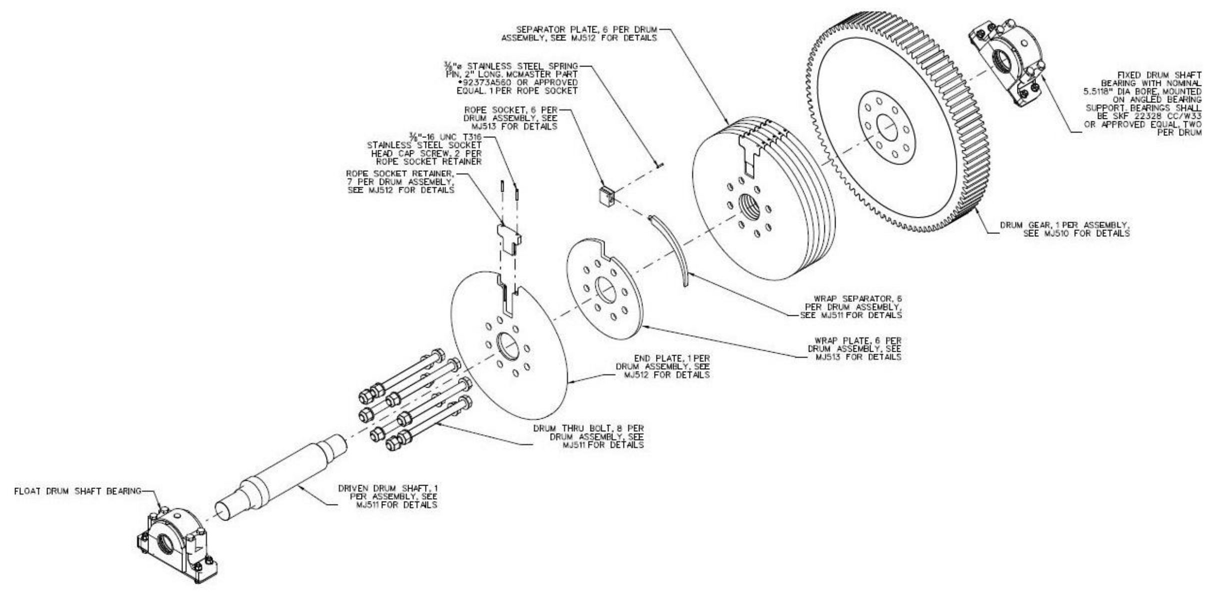

Figure 6: Exploded Isometric view of drum assembly.

Another benefit of using 3D drafting to generate 2D drawings is that all the sheet files are referencing off of a single model. This means that when a change needs to be made to the design, it only needs to be done once and can filter throughout all the referenced sheets. A single model can also be useful for inter-discipline coordination where all disciplines details are captured in the single model.

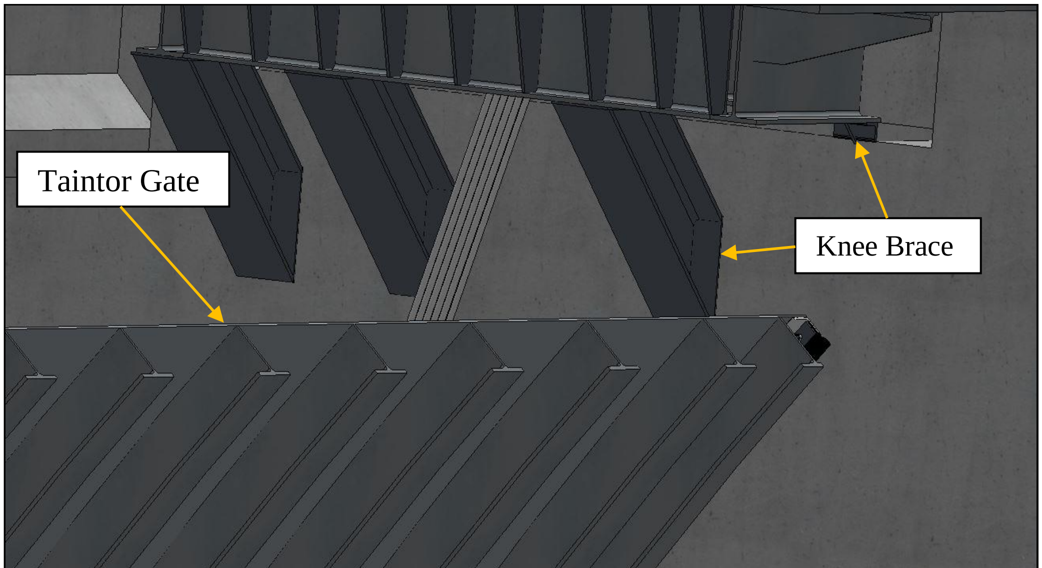

Figure 7: View of 3D model showing existing taintor gate and proposed knee braces.

3D models can also be useful in identifying interferences. An example of checking interferences is when the center pier platform for the example project had to be modified. In order to take the additional loads per the rehabilitation design, knee braces had to be installed on the existing platform, but were close to interfering with the existing gate when it operated. A traditional 2D analysis could have been done to determine if there was a potential problem, but a 3D model made it clear immediately. Since traditional drawings are intended to convey the design, the views required to verify interferences may not be available. In this case, the designer would be required to create additional sketches for analysis of interferences. With a 3D Model, all views are available as long as the model is drawn accurately and with the level of detail that is required (Figure 8).

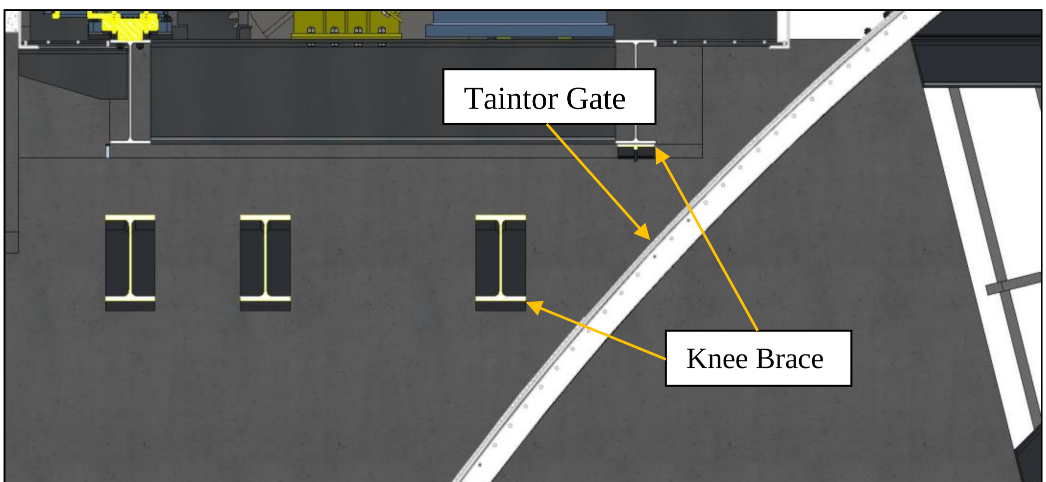

Figure 8: Section through 3D Model showing no interference (gate shown 30° open).

Additional Benefits of 3D Drafting

Speed

Software such as Inventor, ProEngineer, and Solidworks offer the ability to generate 2D drawings in an extremely short time frame. Models can be generated and modified very quickly. Views and section cuts can be taken from models in seconds while dimension can be automatically inserted from the model. Any changes to the model will automatically update the drawings and associated dimensioned. For the Green Peter project there were two versions of the drum assembly that were needed. Although similar the two drums on the center pier (Figure 9) were different than those on the outer piers (Figure 10). The differences were handled using the *level of detail* and *ipart* features allowing both configurations to be generated from one model and eliminating the need to reconcile differences in the drum assemblies for all of the common parts when changes are made.

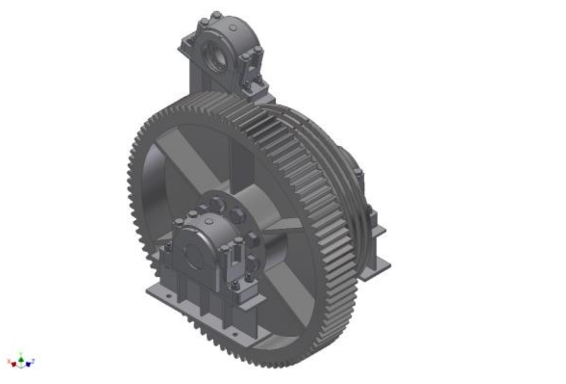

Figure 9: Driving drum model

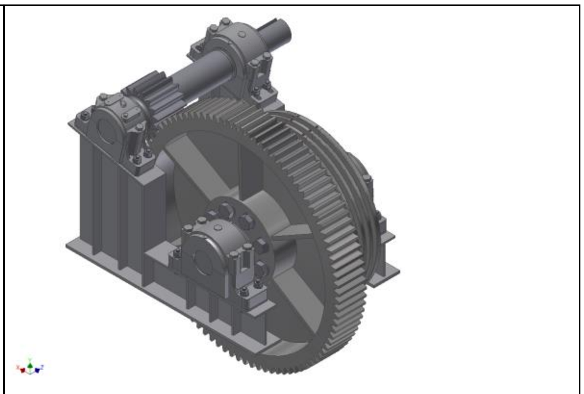

Figure 10: Driven drum model

The biggest drawback to software such as Inventor, ProEngineer, and Solidworks is that the drawing formats and styles have not been adopted by many agencies. This inability to create drawings directly in a given CADD standard usually requires an extra stage of work in that the models or 2D drawings have to be exported to either AutoCAD or Microstation. The line weights and styles must be setup and the dimensions and annotation have to be handled in the clients preferred software. The process can be somewhat automated however must be performed each time a significant change is made to the model.

Finite Element Analysis (FEA)

FEA is often used for projects where hand calculations would not provide adequate accuracy due to unusual geometry or other factors, which may cause uncertainty. One example of FEA performed in Inventor is that of a clevis connection (Figure 11). The actual clevis and bolted connection can easily be solved with hand calculations however the stiffening and fillets are not easily solved by hand. The base and stiffeners were initially estimated as cantilever sections or plate elements spanning between stiffeners. The fillets and thicknesses were then fine tuned using finite element analysis to ensure that no local areas were overstressed. Inventor is suitable for detailed stress analysis in single parts and simple assemblies. It automates many of the mesh controls and will help to automate the convergence of the results. Inventor is not suitable for large scale models of structures, detailed thermal analysis (without addons to the program) or some dynamic simulations between multiple surfaces.

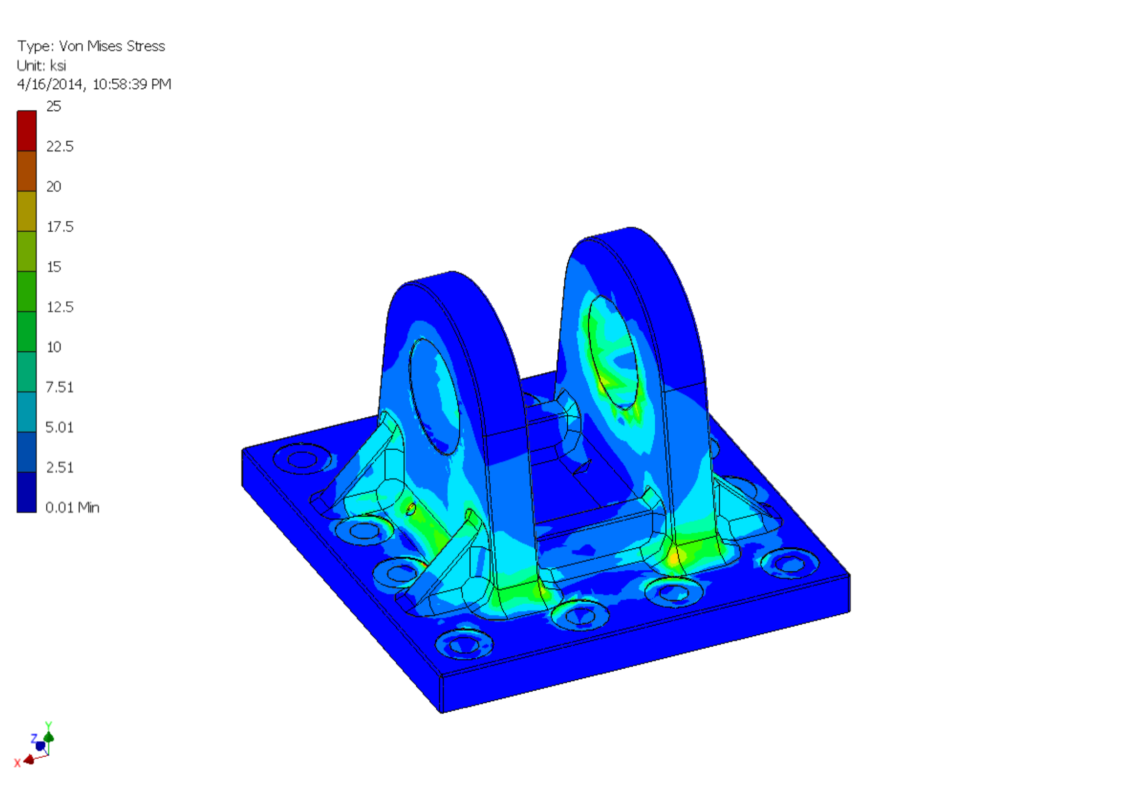

Figure 11: FEA of a clevis

Photo Renderings

In many cases a picture can convey a concept clearly than any 2D drawings. Most of the 3D drawing software including Inventor and Microstation offer a form of photo rendering abilities. These are useful for marketing, and to convey design concepts clearly for complex work. Control over lighting, camera angles, and even video of an assembly can aid in transmitting design concepts to the project team in a way which is easy to understand.

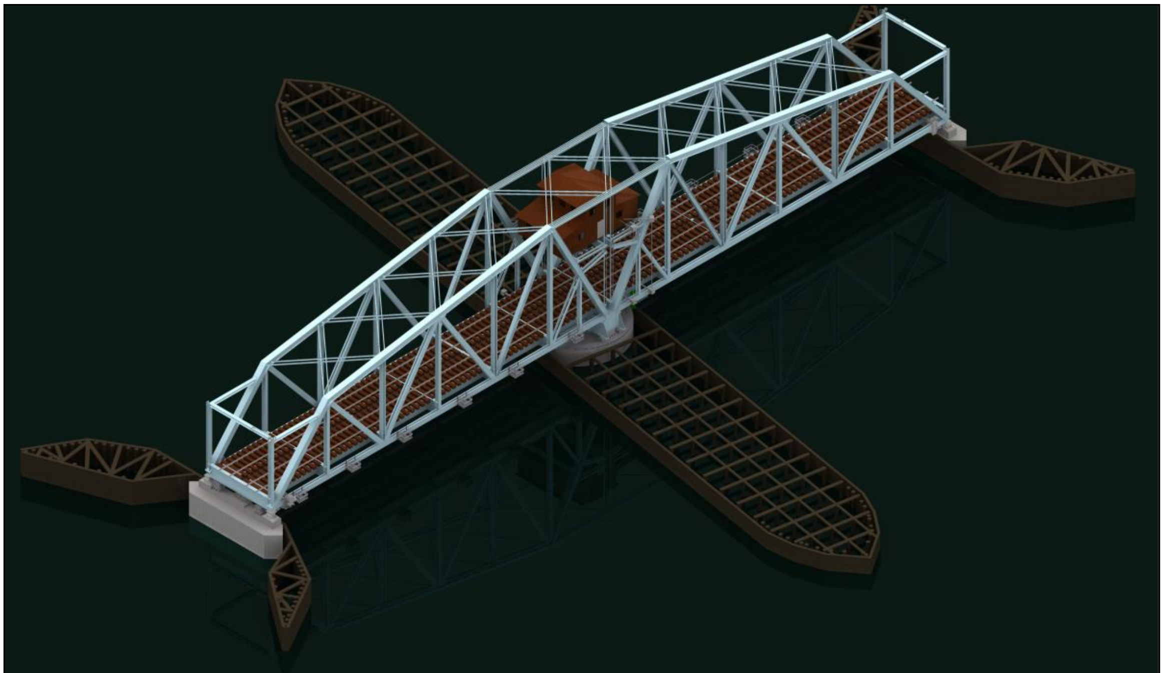

Figure 12: Rending of a swing bridge

3D PDF Printing

3D pdfs are a useful way to transmit 3D models in a format which can be viewed by any pdf reader. They are useful for transmitting design concepts to the project team and for presentation purposes. The models can be rotated and section cuts may be taken at any location without any requirements for specialized software.

Conclusion

The Green Peter Dam Hoist Rehabilitation design benefited from the use of 3D modeling. The tight schedule required multiple design teams to be working simultaneously on mechanical systems that required coordination for a seamless assembly and integration. The modeling aspect allowed for increased communication between team members and reduced the amount of rework required for coordination purposes. This was a savings of both time and hours. 3D drafting also saved time since the 3D model was drawn once and the 2D drawings referencing the 3D model update automatically. The 3D modeling also added value to the design by helping to identify interferences.

3D modeling can have additional benefits by providing a base model for performing finite element analysis if setup properly. 3D models can also be used as an effective communication tools through photo renderings and 3D PDF printing.

In addition to the benefits, it is also important to understand the issues involved with 3D drafting and have an approach to address these issues. “What is the appropriate software to use to complete the task?” and “What software format will the client accept?” are important questions to be answered. Budget is always an issue, and it has to be determined if 3D modeling will require more time than traditional 2D drafting for contract documents or less based project objectives

As with all design tools it is important to compare the advantages and disadvantages of each one. A 3D modeling approach has the functionality to produce a drawing set with additional design features. It is up to the design team to evaluate if 3D modeling is appropriate for each project on a case by case basis.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Feasibility and Design Considerations for the Columbus Road Lift Bridge Rehabilitation Cleveland, Ohio

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Nicholas Fisco, PE

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

INTRODUCTION

Photo 1: East elevation of the Columbus Road Lift Bridge.

The Cuyahoga County Department of Public Works (CCDPW), the City of Cleveland and the Ohio Department of Transportation (ODOT) identified the need to perform major rehabilitation or replacement of the existing Columbus Road Lift Bridge (SFN 1833758) located in the City of Cleveland, Cuyahoga County, Ohio. The bridge was constructed in 1940 by the Wisconsin Bridge and Iron Company of Milwaukee, Wisconsin, for the City of Cleveland and the Federal Works Agency – Public Works Administration. The current structure is a five-span, vertical lift bridge with a 242'-0" long main lift span flanked by approach spans (see Photo 1 and Figures 1 and 2). The bridge is a 359'-0" long Waddell-design, with built-up steel towers, concrete counterweights, and an operator's house perched in the center of the span. The lift truss span is a Pratt configuration truss with an out-to-out width of the deck of 58'-7". The Columbus Road Lift Bridge was identified in the ODOT 1983 Ohio Historic Bridge Inventory, Evaluation, and Preservation Plan as a "Selected Bridge" and as such, it is eligible for listing in the National Registry of Historic Places (NRHP).

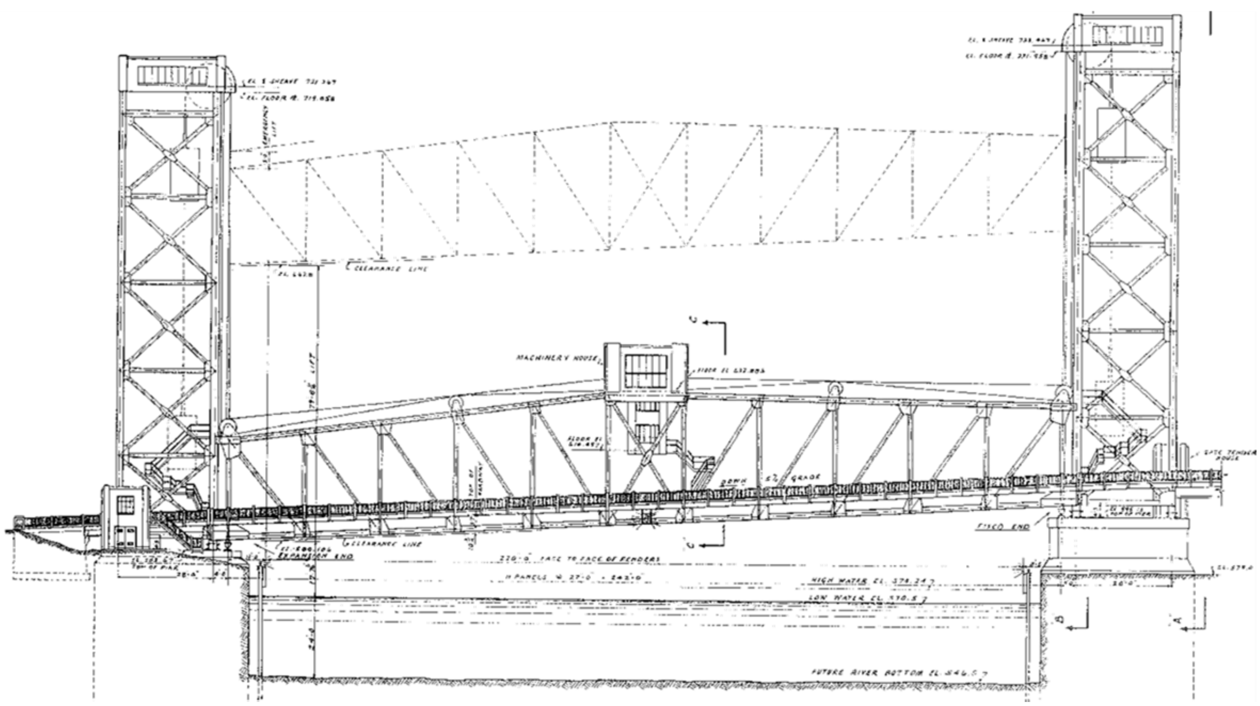
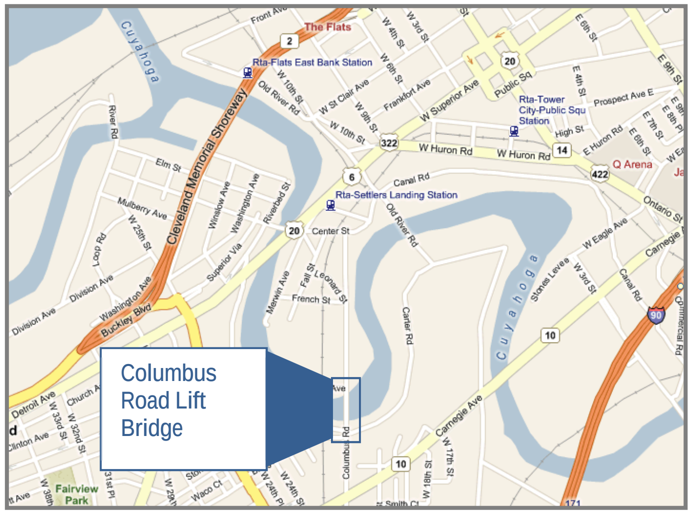


Figure 1: West elevation of the Columbus Road Lift Bridge and typical section of the lift span.

The Columbus Road Lift Bridge is located in Cleveland, Ohio over the Cuyahoga River at waterway milepost 1.9 (see Location Map).

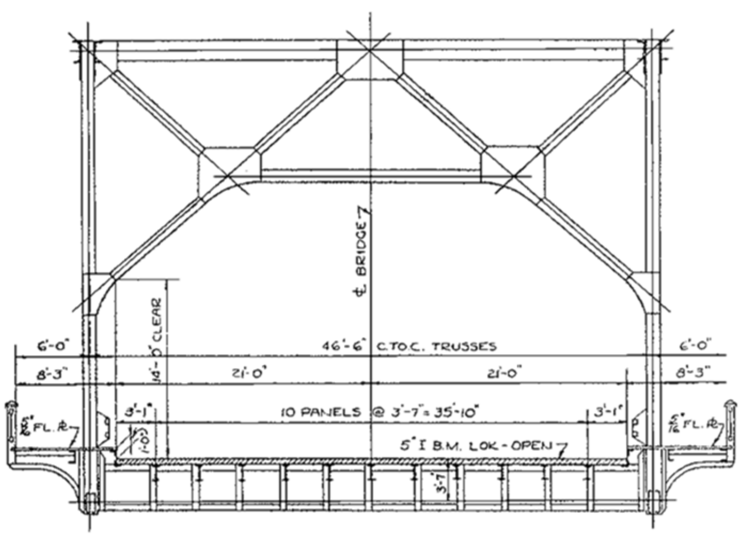


LOCATION MAP

The lift span of the Columbus Road Lift Bridge is a single span through Pratt truss, and carries two lanes of traffic. The roadway deck is 42' feet wide (toe to toe of curbs) and has two 6' wide sidewalks. The roadway surface consists of open steel grating on a rolled stringer and built-up floorbeam system (see Photo 2). The floorbeams are supported by two trusses. The lift span is connected to counterweights at each end by 24 wire ropes (a total of 48 wire ropes). These wire ropes are tensioned by the counterweight and lift span and wrap over the sheaves supported at the tops of the towers.

The lift span opens and closes along two 137'-2" tall truss towers that support the dead loads of the lift span and the counterweights (see Photo 3). The towers also support vehicular traffic on a roadway span that meets the lift span roadway when the bridge is closed. Additionally, there are short approach spans at the north and south ends of the bridge, approximately 9' and 35' long respectively. The roadway profile has a northerly downward slope of 5%.

Photo 2: View of the underside of the lift span.



TYPICAL SECTION THRU LIFT SPAN

Figure 2: Typical section of the lift span.

Photo 3: West elevation of the South Tower.

The bridge was designed to accommodate the passage of large marine traffic on the Cuyahoga River with a clear channel width between the fenders of 220', with an approximate 97.3 foot vertical clearance to the low waterline. The bridge is capable of an additional 5' of lift in emergencies based upon the original design plans.

The machinery for the movable span utilizes a conventional “span drive” layout, where the machinery is mounted on, and moves with, the movable span during operation (see Figure 3). The main drive train is located in a machinery house at mid-span above roadway level. The machinery utilizes an electro-mechanical drive train to transmit power from the prime mover, which is an electric motor, out to operating rope drums which are all driven by the same central gear train. There are four operating rope drums; each drum containing two uphaul and two downhaul ropes that serves one corner of the movable span. The uphaul ropes are terminated at the top of the tower and the downhaul ropes are terminated at the foot of the tower at pier level. All ropes run along the tower legs and pass around deflector sheaves at the top chord of the truss, then run through a series of rollers and deflector sheaves back to the operating rope drums.

When energized, the electric motor rotates the operating rope drums, which pay in or pay out the operating ropes and thereby result in the lift span raising or lowering along the operating ropes. The advantage of a span drive vertical lift bridge is that it provides inherent mechanical skew control and does not, as with a tower drive vertical lift bridge, require complex electrical devices to maintain the equal operation of each end of the lift span.

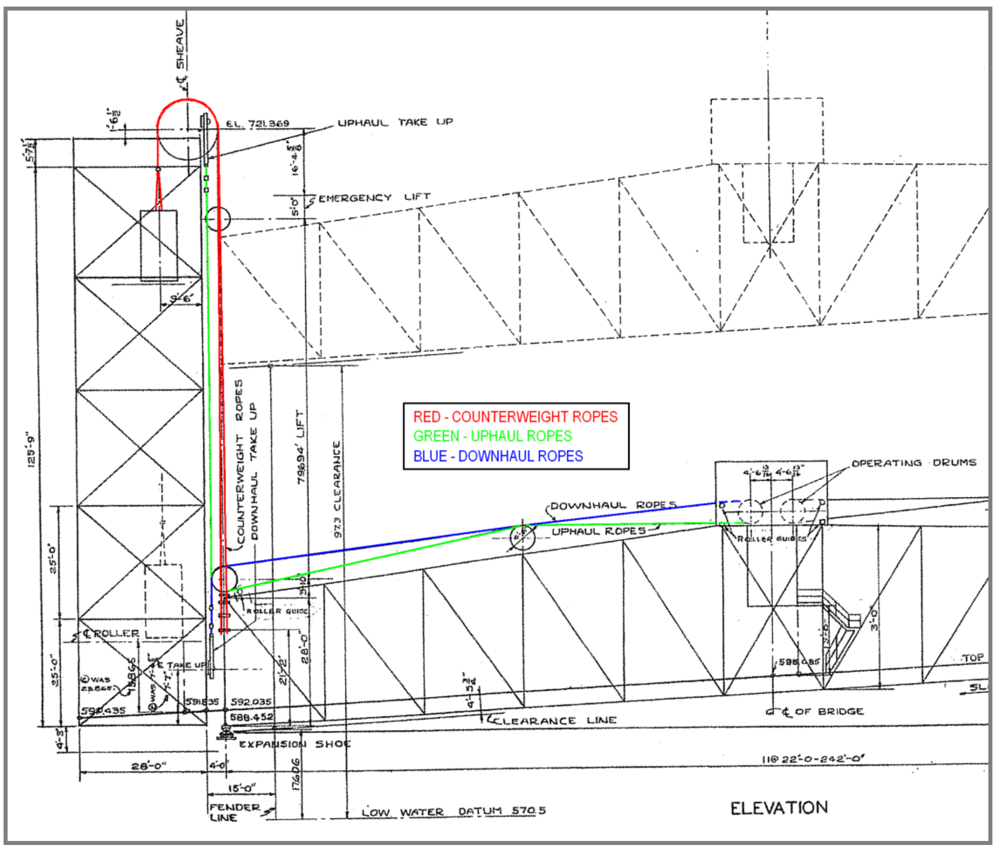


Figure 3: Detail of the span drive cable system of the Columbus Road Lift Bridge along one side of the bridge.

The span is opened and closed by traditional wound rotor motors operating in tandem. The motors are controlled using stepped resistance through the use of a full-voltage drum controller located within the operator's control house. Other controls necessary for the bridge operation are also located in the operator's control house. Electrical service is a 480 VAC, three-phase system. There is no local source of back-up power in case of loss of main service.

COLUMBUS ROAD BRIDGE HISTORY

Since 1835, four bridge types have carried Columbus Road (or Street, as it has been called in the past) over the Cuyahoga River in Cleveland, Ohio. The first was a timber covered bridge with a center draw span. This was replaced by an iron bridge in 1870, which in turn was soon replaced by the world's first double swing bridge in 1895 (see Figure 4, Photos 4 and 5). This remarkable double swing bridge consisted of two trusses, each mounted on a separate drum girder and pier, with drum girders at 151' center to center. This gave the bridge a 115' clear span between the fenders that protected the piers.

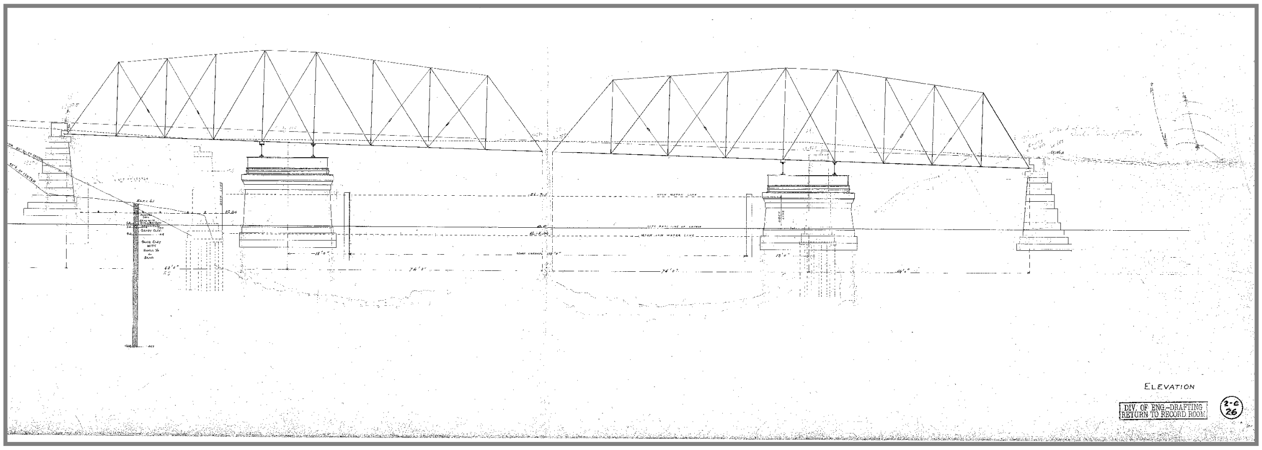


Figure 4: 1895 double swing bridge elevation.

Photo 4: 1895 double swing as viewed from the southeast. Image from Cleveland State University Library.

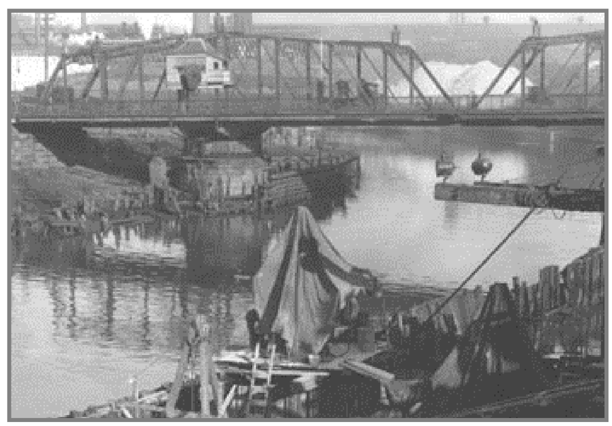


Photo 5: Detail of the north swing span. Image from Cleveland State University Library.

The double swing was replaced in 1940 by the current, steel vertical lift bridge with a 242' long lift span, designed by the Cleveland firm of Wilbur Watson and Associates.

The current bridge has been exhibiting corrosion and section losses for years. During the 1980's, some concerns for the condition of the bridge were expressed and possible replacement was discussed in the Historic American Engineering Record for the National Park Service, HAER No. OH-55, dated 1986. In 1991, selected lower lateral bracing members at the end bays of the lift span were replaced due to extensive deterioration and section losses. In 1999, a more extensive rehabilitation was performed. Work included replacement of the steel grid deck, selected lift span stringers and lateral bracing members, replacement of the north approach superstructure, partial painting, abutment work, drainage work, and new traffic gates.

PROJECT TEAM

TranSystems contracted with the following engineering firms to assist in completing the inspection and rehabilitation design:

- **Stafford Bandlow Engineering, Inc.** Provided Mechanical engineering and construction oversight.
- **Flanders Engineering Group** Provided Electrical engineering and construction oversight.
- **Michael Baker, Jr., Inc. (Baker)** Provided roadway engineering, developed the public involvement plan and performed miscellaneous environmental field studies.
- **BBC&M Engineering, Inc. (BBCM)** Performed geotechnical services and foundation analysis.
- **Northwest Consultants, Inc. (NCI)** Performed a topographic and structural survey and completed the project basemap.
- **Collins Engineers, Inc. (Collins)** Performed underwater inspection.
- **TBE Group** Performed utility identification and coordination.

PURPOSE AND NEED

The primary purpose of this project was to provide a safe and adequate bridge crossing over the Cuyahoga River on the existing Columbus Road alignment that serves the existing roadway network in the Cuyahoga River Valley. Secondary purposes were as follows:

- to maintain the historic character of the bridge and the adjacent historic districts
- to minimize or avoid adverse effects to the bridge, the roadway network, roadway and river traffic, and the surrounding neighborhoods and businesses.

Additionally, the project would provide a cost-effective solution that was within the fiscal constraints of the owner both during construction and during the long-term maintenance period of the bridge. The Purpose and Need Statement was used as a framework to develop and evaluate the alternatives, and ultimately determine the recommended alternative. The results of the evaluation were presented to the Technical Advisory Committee (TAC) for their consideration.

FIELD INVESTIGATION

The project began with a field evaluation of the structural, mechanical, electrical, geotechnical, environmental and geometric conditions. An in-depth structural inspection was performed on the Columbus Road Lift Bridge between June 9 and June 20, 2008, in accordance with National Bridge Inspection Standards (NBIS). The results of the field evaluation, combined with the results of the data search, provided a comprehensive evaluation of the system. As part of the evaluation, TranSystems developed a 3D model of the structure to analyze the lift span and towers for a more accurate structural response under static and dynamic forces, and various loading conditions. The model was subsequently used to evaluate various schemes within each investigated design alternative.

INSPECTION FINDINGS

The existing bridge had many structural, mechanical and electrical defects that caused several weeks of closure for repairs and retrofits each year. The majority of the repairs and retrofits that were performed in the past were temporary in nature and subject to budgetary constraints. The accumulated effect of the repairs caused changes in the integrity of the structural, mechanical and electrical components.

<u>Summary of Structural Inspection Findings:</u>

The bridge was found to be in Poor Condition [4-NBIS] overall, with the following inspection findings:

Deck: The deck was in Poor Condition [4-NBIS] overall with spalling and advanced section loss to the concrete filled deck grating system, vertical misalignment of the north finger joint, heavy leakage at the south abutment joint, advanced section loss to the traffic rail posts (see Photo 6) and impact damage to the traffic rails.

Superstructure: The lift span was found to be in Poor Condition [4-NBIS] overall, with advanced section losses and holed through sections of the stringers, floorbeams, vertical and diagonal truss members (see Photos 7 & 8). The lower chord members and the lower gusset plates exhibited up to 3/16" losses. The protective

Photo 6: Traffic rail post with laminating rust at the flanges and holed through web.

Photo 7: L6-U5 has a 1" diameter hole in the west flange above the sidewalk plate.

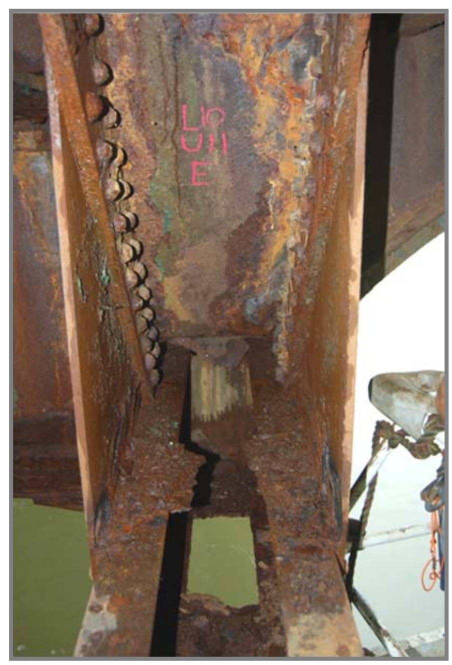


Photo 8: Lift Span lower chord with advanced deterioration to lower chord, batten plate, gusset plates and diagonal. Panel Point L10 East is shown.

paint coating had typically failed throughout the lift span's steel members.

The towers were in Fair Condition [5-NBIS] overall with advanced section losses localized in the tower floor system. Excessive wear was noted at the forward tower columns due to the misalignment between the lift span and the towers that caused the upper operating sheaves to gouge the North Tower column and the guide wheel to fail at the South Tower. The protective paint coating had typically failed throughout the towers' steel members.

The approach spans were noted to be in Poor Condition [4-NBIS] overall due to areas of advanced section losses of up to 3/16" in the steel members underneath the South Abutment deck joint. Additionally, areas of the girders and floorbeams on the South Approach Span have been rehabilitated with additional steel plates, angles, and beams due to advanced deterioration of the superstructure. The protective paint coating has typically failed throughout the south approach span's steel members.

Substructure: The substructure units were in Poor Condition [4-NBIS] overall due to the South Abutment and the tower piers exhibiting large areas of spalled concrete with exposed reinforcing steel, areas of delaminations and full height cracks with efflorescence.

Tender Houses:

The South Tender house was in Poor Condition with the concrete portions exhibiting large areas of spalled concrete with exposed reinforcement, and the stone masonry sections exhibiting isolated protruding stones. The concrete section's south wall was noted to have approximately 125 square feet of spalled concrete with exposed reinforcement, while the east wall exhibited spalled concrete with exposed reinforcement for approximately 90% of its area. Along the east face, the diameter of the reinforcement was noted as small as 3/8". The interior of the concrete portions of the tender house were in good condition with a shallow 2 square foot spall and a 4' crack with efflorescence noted in the ceiling.

The stone masonry exhibited dislodged stones in two locations: at the brick/stone masonry interface in the north east corner and the upper two courses in the south east corner. At the brick/stone interface, the top 3 corner stones were noted to be dislodged by 1" at the top stone, and are leaning in the eastward direction (see Photo 9). In the southeast corner, the top two stones in the south wall are dislodged by 1" in the southerly direction (see Photo 10).

Photo 9: Dislodged stone in the north east corner at the brick stone interface.

Photo 10: Dislodged top stones in the south east corner of the south wall (Note the broken window panes).

Mechanical Inspection Findings:

The mechanical inspection of the Columbus Road Lift Bridge revealed the following:

- The primary mechanical drive train components were in fair condition, though inefficient and/or obsolete.
- The drive machinery located outboard of the machinery house was in poor condition due to deterioration from wear and environmental exposure.
- The span locks were not in service and had been abandoned.
- The counterweight sheaves, counterweight ropes and sheave trunnions are in good to fair condition with light surface rust.
- The trunnion bearing journals and bronze bushings were in poor condition due to typical abrasive wear and bands of bronze embedment across the full width of the bearing journals. One journal exhibited severe abrasive wear in excess of a 500 microinch surface finish.
- During the inspection, it was noted that the moveable span was binding and/or twisting between the towers in and near the seated position, causing direct contact and excessive wear between the lift span and towers (see Figure 5 and Photo 11). The north deflector sheaves and north uphaul ropes exhibited excessive wear, and one of the southeast span guide roller axles had failed.
- The load supports were in fair condition, with the fixed supports exhibiting accelerated wear at the pin due to the binding issue.

Photo 11: 1/2" deep gouge from the roller guide in South Tower, East Forward Column.

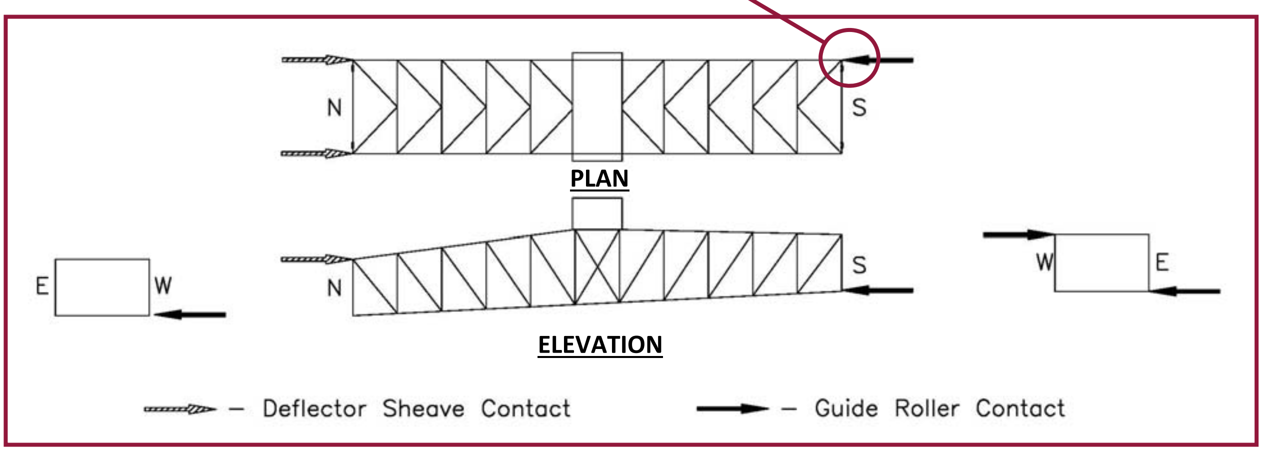


Figure 5: Schematic View of Lift Span depicting contact points from deflector sheaves and upper and lower guide rollers.

Electrical Inspection Findings:

The majority of the electrical equipment in use on the Columbus Road Lift Bridge was noted to be obsolete and/or in poor condition, and rapidly reaching the end of its useful life. Most of the electrical equipment was installed in 1940 as part of the original bridge. The bridge control system was typical of an older installation, where a skilled tender is necessary to operate the machinery. Position indicators for the span and other related machinery was minimal, and in some areas, non-functioning. Span locks installed on the bridge had been disabled and maintenance for the electric lock drive motor and limit switch assembly had been discontinued. There was no stand-by electrical power source noted during the time of inspection; therefore, the loss of utility service would place the bridge out of service until power was restored.

The electrical inspection uncovered several deficiencies, ranging from minor issues related to the degradation of the installed equipment to major concerns regarding safety, such as the disabled barrier gate. In general, the aging equipment installed on the bridge was in need of major rehabilitation and/or replacement to achieve an acceptable level of safety and operational reliability (see Photos 12 and 13).

Photo 12: Flexible cables. Note the flexible cables are twisting.

Photo 13: Flexible Cables. Note the splices necessary to repair the cables.

GEOTECHNICAL INVESTIGATION

The foundation of each reinforced concrete tower pier consists of 6 caissons and 12 battered piles. The caissons are 30" diameter, 5/8" thick steel pipe filled with concrete, with a 14" deep wide flange core (see Figure 6). One caisson is located below each rear tower column, and two are located below each forward tower column. The bottoms of the caissons are reinforced with an additional rebar cage detail, and are socketed 9'-9" into bedrock. The battered piles are 16" diameter, 3/8" thick steel pipe piles with a 30º batter. Ten (10) piles are doweled into the rear side of the piers, battered in the longitudinal direction of the bridge. Two piles are doweled into the forward side of the piers, battered transversely to the bridge. Based on the 1939 bridge plans and construction print drawings, the battered piles at the piers were driven to a 100-ton capacity. Assuming that this is an

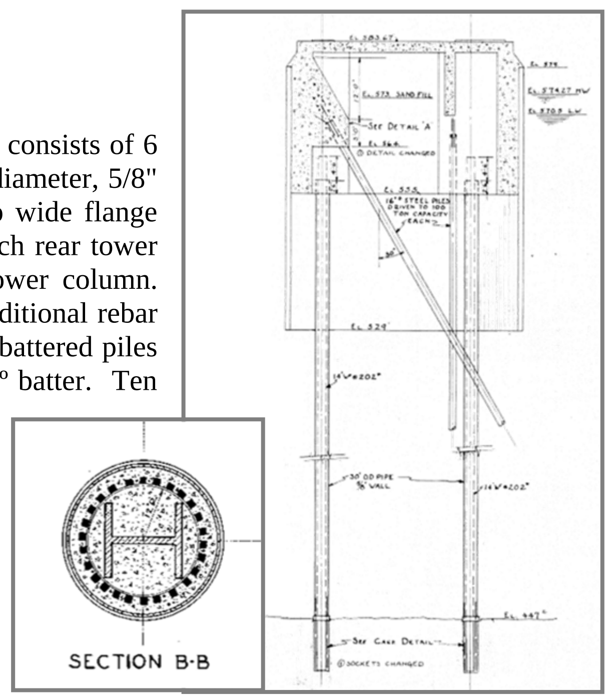


Figure 6: Tower pier foundation elevation and caisson cross section.

allowable capacity with a factor of safety of 2.0, a batter of 30-degrees, and a total of 12 piles per pier, it was estimated that the piles have an ultimate vertical capacity of 4,156 kips per pier. If the full vertical geotechnical capacity of the battered piles is added to the axial geotechnical capacity of the shafts, the factor of safety of the pier foundations with respect to axial capacity is estimated to be 1.27 and 1.49 for the south and north piers, respectively.

In FHWA NHI 05-042 “Design and Construction of Driven Pile Foundations”, the factor of safety recommended for new construction ranges from 2.0 to 4.0, depending upon the reliability of the particular static analysis method; most use a factor of safety of 3.0. FHWA NHI 05-042 recommends that the factor of safety used in a static analysis calculation be based upon the construction control method specified, with recommended factors of safety ranging between 2.0 and 3.5, depending upon the control method that will be used in the field during foundation installation. Similar factors of safety are recommended in FHWA-IF-99-025 “Drilled Shafts: Construction Procedures and Design Methods” for Allowable Stress Design.

Based upon the existing factors of safety determined from the preliminary analyses, it was determined that underpinning of the pier foundations would not be necessary as long as the overall weight of the structure was minimized.

STRUCTURAL ANALYSIS

The Columbus Road Lift Bridge was originally designed for an H-20-33 truck load (20 ton tandem truck), a lane load of 640 pounds per liner foot with a 32,000 pound concentrated load, and a sidewalk load of 60 pounds per square foot. A 3D model (see Figure 7) representing the existing lift span, towers and their support systems was developed to perform a finite element analysis for As-Built and As-Inspected conditions, using the original shop drawings, and the 2008 field inspection records for section properties. The analyses and ratings are prepared with the HS20-44 truck (36 ton truck) and lane loading, with a sidewalk load of 60 pounds per square foot for an inventory level rating. Four (4) Ohio legal truck loads, in addition to the HS20-44 loads and sidewalk load were used for an operating level rating.

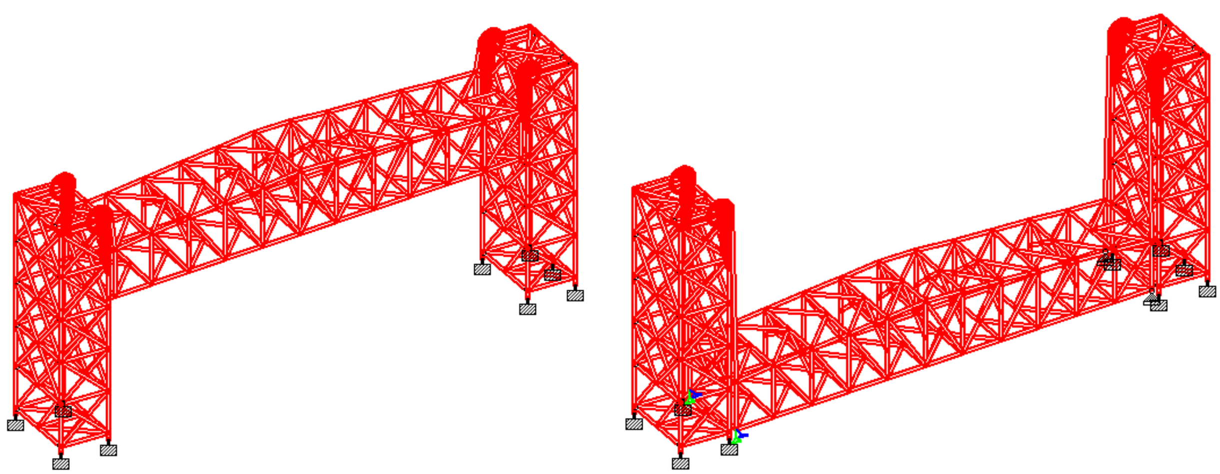


Figure 7: STAAD.Pro 2007 model of Lift span in “Open and Closed Position”

Table 1 lists the structural elements in red that are inadequate to carry the design loads.

LIFT SPAN TRUSS AS-INSPECTED LOAD RATING SUMMARY						
Location	HS20-44 (36 TONS)		2F1 (15 TONS)	3F1 (23 TONS)	4F1 (27 TONS)	5C1 (40 TONS)
	Inventory (T)	Operating (T)	Operating (T)	Operating (T)	Operating (T)	Operating (T)
Upper Chord	40	73	87	87	82	100
Lower Chord	30	80	104	104	104	112
Verticals	8	55	38	40	41	48
Diagonals	15	57	56	56	57	62
Floorbeams	21	31	22	25	28	45
TOWER AND APPROACH SPAN AS-INSPECTED LOAD RATING SUMMARY						
Location	HS20-44 (36 TONS)		2F1 (15 TONS)	3F1 (23 TONS)	4F1 (27 TONS)	5C1 (40 TONS)
	Inventory (T)	Operating (T)	Operating (T)	Operating (T)	Operating (T)	Operating (T)
Girders	38	57	40	42	46	73
Floorbeams	22	33	25	27	29	48
Stringers	39	55	36	42	44	73

Table 1: The As-Built and As-Inspected Capacity Summary tables for the controlling Tower members. **BOLD RED** numbers indicates a capacity that is below the loading/stress demand.

The towers members above the roadway surface do not carry vehicular live loads, and as such, a capacity analysis rather than a load rating analysis was performed for these members. Capacity to Demand (C/D) ratios were calculated for axially loaded members and flexural members (see Table 2).

TOWER CAPACITY TO DEMAND (C/D) RATIO SUMMARY FOR AXIALLY LOADED MEMBERS (>1)				
Location	NORTH TOWER	SOUTH TOWER	NORTH TOWER	SOUTH TOWER
	AS-BUILT C/D	AS-BUILT C/D	AS-INSPECTED C/D	AS-INSPECTED C/D
Tower Columns	1.36	1.36	1.16	1.13
Struts (South Face)	1.80	1.80	1.74	1.74
Diagonals (South Face)	1.27	1.27	1.24	1.22
Sheave Girders	278.17	278.17	270.53	254.89
TOWER CAPACITY TO DEMAND (C/D) RATIO SUMMARY FOR FLEXURAL MEMBERS (>1)				
Location	NORTH TOWER	SOUTH TOWER	NORTH TOWER	SOUTH TOWER
	AS-BUILT C/D	AS-BUILT C/D	AS-INSPECTED C/D	AS-INSPECTED C/D
Top Forward Struts	1.88	1.88	1.86	1.85
Sheave/Jack Girders	1.45	1.45	1.42	1.42
TOWER COMBINED STRESS RATIO SUMMARY - COMBINED AXIAL AND FLEXURE (<1)				
Location	NORTH TOWER	SOUTH TOWER	NORTH TOWER	SOUTH TOWER
	AS-BUILT C/D	AS-BUILT C/D	AS-INSPECTED C/D	AS-INSPECTED C/D
Top Forward Struts	0.77	0.77	0.77	0.80
Top Rear Struts	0.01	0.01	0.02	0.01
Sheave/Jack Girders	0.69	0.69	0.71	0.71

Table 2: The As-Built and As-Inspected Capacity Summary tables for the controlling Tower members.

Additionally, the 3D model was used to evaluate the structure under a number of loading conditions for the investigated alternatives, with three schemes within each alternative:

- Scheme 1 - Conventional Concrete Deck
- Scheme 2 - Half Filled Concrete Steel Grid Deck
- Scheme 3 - Open Grid Deck with Concrete Fill Along the Curb Line and at Ends of the Deck

Analysis of the total rehabilitation alternative generated a list of the structural elements with low capacity for replacement. This list included stringers, deck, floorbeams, several truss members and several secondary members of the tower structures (see Table 3 for the lift span).

LIFT SPAN TRUSS - SUMMARY OF OVERSTRESSED MEMBERS (ALTERNATIVES 2&3) HS20-44 DESIGN LOADING						
Location	SCHEME 1		SCHEME 2		SCHEME 3	
	No. of Overstressed Members	Inventory Rating Range (TONS)	No. of Overstressed Members	Inventory Rating Range (TONS)	No. of Overstressed Members	Inventory Rating Range (TONS)
Upper Chord	9 of 11	**0** to **2**	9 of 11	**5** to **17**	9 of 11	**21** to **34**
Lower Chord	4 of 11	**28** to **33**	-----	-----	-----	-----
Verticals	3 of 12	**31** to **35**	-----	-----	-----	-----
Diagonals	6 of 12	**11** to **33**	3 of 12	**26** to **35**	-----	-----

Table 3: Analysis Summary of the deficient members for the Lift Span. BOLD RED numbers indicate ratings below the design load limits.

If we determined the Scheme 1 loading be implemented, a substantial amount of new truss member design must have performed, including replacement of the entire top chord. Similarly, Scheme 2 required new truss member design, including replacement or major rehabilitation of the top chord. The controlling rating for Scheme 3 was 21 tons (shown in Table 3), 42% below the design limit. The dead loads for the three schemes were then incorporated into the tower analyses. This resulted in the need to retrofit the tower columns should Scheme 1 be used for the rehabilitation or partial replacement options (Alternatives 2 and 3).

EVALUATION OF ALTERNATIVES

Six (6) conceptual alternatives were originally identified by the TAC and project team and the four alternatives that were determined to be feasible were further evaluated to rehabilitate and/or replace the bridge to provide a safe crossing for pedestrian and vehicular traffic with no legal weight limit restrictions:

Alternative 1 - <u>No-Build</u>: This alternative included the cost of repairs and maintenance required to keep the bridge operational for next 50 years with restricted weight capacity.

Alternative 2 - <u>Total Rehabilitation</u>: This alternative included a rehabilitation program to replace, strengthen, retrofit and repair structural elements while replacing most of the mechanical and all of the electrical equipment. This alternative required closing the bridge to all roadway traffic and utilizing a detour route.

Alternative 3 - Rehabilitation with Partial Replacement: This alternative included a rehabilitation program to replace the lift span, retrofit and repair any deteriorated tower elements, replace most of the mechanical and all of the electrical equipment. This alternative required closing the bridge to all roadway traffic and utilizing a detour route.

Alternative 4 - Total Replacement on Existing Alignment: This alternative would have replaced the structure in its entirety, closing the bridge to all roadway traffic and utilizing a detour route.

Alternative 5 - Total Replacement on New Alignment: This alternative would have replaced the structure in its entirety on a new upstream alignment. This alternative was eliminated from further study at the conceptual alternative evaluation stage.

Alternative 6 - Remove the Bridge and Eliminate the River Crossing: This alternative would have removed the bridge in its entirety, eliminating the river crossing at this location. This alternative was eliminated from further study at the conceptual alternative evaluation stage.

Based upon the 6 alternatives listed above, the following table outlines the associated costs and residual values for each alternative. Note that the costs presented for Alternative 1 included weight restrictions and partial closures during the investigated period, and therefore it was not an acceptable alternative.

	ALTERNATIVE					
	1	2	3	4	5	6
	NO BUILD	TOTAL REHABILITATION	REHABILITATION WITH PARTIAL REPLACEMENT	TOTAL REPLACEMENT ON EXISTING ALIGNMENT	TOTAL REPLACEMENT ON NEW ALIGNMENT	REMOVE RIVER CROSSING
INITIAL COST IN 2011	$0	$34,700,000	$37,700,000	$54,300,000	$60,900,000	$3,600,000
PRESENT WORTH IN 2011	$34,500,000	$49,200,000	$49,100,000	$63,900,000	$70,500,000	$3,600,000
EQUIVALENT ANNUAL COST	$1,400,000	$1,900,000	$1,900,000	$2,500,000	$2,800,000	$140,000
RESIDUAL VALUE IN YEAR 2061	$0	($9,900,000)	($13,600,000)	($29,100,000)	($29,100,000)	$0

Roadway - The roadway impacts for Alternatives 2, 3 and 4 were the same. They would result in minimal impact to the roadway, from north of the Franklin-Riverbed-Carter intersection to south of the Merwin Avenue intersection. These impacts included a minimal amount of pavement work to connect the bridge with the existing roadway network, and utility coordination with minimal anticipated impacts to the electrical, water, gas utilities. There would be no impacts to driveway connections and there are no anticipated right-of-way takes. Alternative 1 would not impact the roadway. Alternatives 5 and 6 were not evaluated for roadway impacts because they are not feasible alternatives.

PUBLIC INVOLVEMENT

The purpose of the public involvement program was developed to engage the Technical Advisory Committee (TAC), stakeholders and members of the general public in the Project Development Process. These agencies and individuals had become informed and involved, enabling them to provide critical input into the project. The goal of the public involvement program was to optimize the participation of affected parties, with a focus on meetings between the team and agency and community representatives. Public involvement occurred throughout the project, beginning in the conceptual design phase and continuing through final design.

The TAC and project team determine which alternatives were feasible and should be further developed and which alternatives should be eliminated. The Purpose and Need Statement's primary and secondary purposes were used to assess the feasibility of each conceptual alternative, to determine how well the alternatives satisfied the Purpose and Need. The TAC discussed each of the alternatives. They determined that Alternative 5 (a new bridge on a new alignment) and Alternative 6 (eliminate the bridge and remove the river crossing) did not meet the project Purpose and Need. The decision to eliminate Alternatives 5 and 6 was consistent with the recommendations made by most of the stakeholders, including the City's Landmarks Commission staff, as well as the general public. Alternatives 5 and 6 were described and evaluated below and were not further investigated as feasible alternatives. The remaining alternatives were advanced as feasible alternatives and as such, were subject to further study and evaluation. Although Alternative 1 (No-Build) did not meet the Purpose and Need, it was advanced for analysis and comparison as required by ODOT's Project Development Process.

Purpose and Need	Alternative 1 No Build	Alternative 2 Total Rehabilitation	Alternative 3 Rehab with Partial Replacement	Alternative 4 Replace on Existing Alignment	Alternative 5 Replacement on New Alignment	Alternative 6 Remove / Eliminate Bridge
PRIMARY PURPOSE						
Provide safe, adequate bridge crossing over the Cuyahoga River on the existing alignment	RED	GREEN *	GREEN *	GREEN *	RED	RED
Provide bridge that serves existing roadway network in Cuyahoga River Valley	RED	GREEN *	GREEN *	GREEN *	GREEN *	RED
SECONDARY PURPOSE						
Minimize / avoid adverse effects to the Columbus Road Lift Bridge	RED	GREEN	GREEN *	RED	RED	RED
Maintain historic characteristics of the Cuyahoga River Valley vertical lift bridges	YELLOW	GREEN	GREEN *	RED	RED	RED
Maintain characteristics of Irishtown Bend and Ohio City Historic Districts	GREEN	GREEN	GREEN *	YELLOW *	YELLOW	GREEN
Minimize impacts to vehicular, pedestrian and bicycle traffic (motorized and non-motorized traffic on bridge); river traffic (during construction)	RED	YELLOW	YELLOW	YELLOW	YELLOW	RED
Minimize impacts to vehicular, pedestrian and bicycle traffic (motorized and non-motorized traffic on bridge); river traffic (permanent conditions)	RED	GREEN	GREEN	GREEN	GREEN	RED
Minimize impacts to surrounding neighborhood (facilities, amenities, businesses and residences); and planned projects in area	RED	GREEN	GREEN	GREEN	RED	RED
Provide cost-effective solution within fiscal constraints of project both during construction and for long-term maintenance of bridge	YELLOW	GREEN *	GREEN *	YELLOW *	RED	GREEN
Public input (acceptable alternative)	YELLOW: 6 yes 11 no	GREEN: 16 yes 1 no	GREEN: 19 yes 0 no	YELLOW: 11 yes 5 no	YELLOW: 5 yes 9 no	RED: 0 yes 18 no
Stakeholder input (acceptable alternative)	RED: no (100%)	GREEN: As Needed	GREEN: As Needed	GREEN: As Needed	YELLOW: Mixed (no > yes)	RED: no (100%)

Table 4: Alternatives Analysis – Determination of Feasible Alternatives Matrix

RED	RED boxes indicate negative result
YELLOW	YELLOW boxes indicate medium result
GREEN	GREEN boxes indicate positive result
*	Assessment Depends on Analysis Results; to be Finalized by the TAC

Input received from the stakeholders and the general public was incorporated into the alternatives evaluation process, as indicated in the conceptual alternatives evaluation matrix. The feedback received from the stakeholders during the individual stakeholder meetings was generally consistent with the comments received on the questionnaires from the stakeholders and the public. The stakeholders unanimously and strongly reported that not fixing the bridge (Alternative 1) and eliminating the river crossing (Alternative 6) were not viable alternatives. There was near universal agreement that the decision to repair or replace the bridge should be dictated by need and cost. Specifically, the bridge should not be replaced unless the findings from the analysis indicate that replacement were necessary. If a new bridge was required, replacement on the existing alignment was preferred to replacement on a new alignment.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the results of the Preliminary Engineering Study for the Columbus Road Lift Bridge, TranSystems developed six (6) alternatives to meet the requirements of the Purpose and Need to provide a safe and adequate bridge crossing over the Cuyahoga River on the existing Columbus Road alignment that serves the existing roadway network in the Cuyahoga River Valley. The conclusions of each of our studies results are outlined as follows:

<u>Inspection Findings and Load Ratings Conclusions</u>

Based upon the in-depth inspection, the bridge was in Poor Condition [4-NBIS] overall due to advanced section losses and holed through sections of the deck and floor system, lift span truss members, misalignment of the finger joints, and areas of spalls at the substructure units. Based upon these findings the floor system, several truss members, approach span superstructures should be replaced. Additionally, the drive and support machinery and electrical systems should be replaced. The machinery was obsolete, and exhibited deterioration due to wear and environmental exposure. The machinery exhibited damage due to the binding/twisting of the lift span and towers. The electrical system was obsolete and in poor condition and was rapidly reaching the end of its useful life. Position indicators for the span and other related machinery was minimal, and in some areas, non-functioning. Span locks installed on the bridge were disabled and maintenance for the electric lock drive motor and limit switch assembly were discontinued. There was no stand-by electrical power source; therefore the loss of utility service would place the bridge out of service until power was restored.

A 3D model representing the existing lift span, towers and their support systems was developed to perform a refined analysis for As-Built and As-Inspected conditions, using the original shop drawings, and the 2008 field inspection records for section properties. The analyses and ratings were prepared with the HS20-44 truck (36 ton truck) and lane loading, with a sidewalk load of 60 pounds per square foot for an inventory level rating. Four (4) Ohio legal truck loads, in addition to the HS20-44 loads and sidewalk loads, were used for an operating level rating. Based upon the conditions and results of the analysis, several secondary members of the tower structures should be replaced or rehabilitated in addition to members to be replaced due to the inspection findings.

Purpose and Need Conclusions

TranSystems had concluded that Alternatives 2, 3 and 4 met the primary objective of the Purpose and Need Statement; however, only Alternatives 2 and 3 had met the secondary purpose to maintain the historic character of the bridge and to minimize the adverse effects to the bridge, roadway and river traffic, and the surrounding neighborhoods and businesses.

Stakeholders and Public Involvement

The Stakeholders agreed that the decision for repair or replacement should be dictated by need and cost. Alternative 4 required a longer temporary road closure at the river crossing and was approximately $15 million more expensive than Alternatives 2 and 3 in a fifty year life cycle. Additionally, input from the Public Involvement Process established that a new bridge would affect the historical characteristics of Irish Town Bend, Ohio City, and the surrounding vertical lift bridges. Based on this information, Alternative 4 was eliminated from further consideration.

Alternative Studies Conclusions

Six (6) conceptual alternatives were originally developed for the Preliminary Engineering Study:

Alternative 1 – No Build
Alternative 2 – Total Rehabilitation
Alternative 3 – Rehabilitation with Partial Replacement
Alternative 4 – Total Replacement on Existing Alignment
Alternative 5 – Total Replacement on New Alignment
Alternative 6 – Remove the Bridge and Eliminate the River Crossing

Alternatives 5 and 6 was eliminated at the conceptual stage due to their impacts on the roadway network, the area residences and businesses, cultural resources impacts, and their associated costs. Furthermore, these two alternatives did not meet the Purpose and Need Statement. Because Alternative 1 did not meet the Purpose and Need, TranSystems eliminated this alternative too. Due to costs, and the adverse effects to the historical characteristics of the surrounding areas, Alternative 4 was eliminated.

Both Alternatives 2 and 3 required the retrofitting, strengthening and replacing of select structural members of the towers, repairing the substructure units, and removing the approach spans. Both alternatives also included replacing most of the mechanical and all of the electrical systems of the bridge. The difference was the planned work for the lift span. While both alternatives required the removal of the lift span from the bridge site, Alternative 2 work included a rehabilitation of the existing lift span at a fabrication shop, and Alternative 3 would have completely replace the lift span with a new span of similar appearance.

For Alternative 2, the rehabilitation of the existing span would have required careful removal and transportation to avoid damage to its components. It would have required continuous bracing of the structure to begin removals of the portions that need to be replaced. This would have been done without damaging the portions that were to remain. These remaining members would then have to be carefully cleaned to remove all possible rust. The areas that were to be connected to strengthening plates or new steel members must have been carefully measured for proper fit up, taking into consideration that the original rolled sections are no longer produced. All top end gussets, lifting girders, and top chord members would have to been retrofitted to allow for the continuing creep movement of the south side slope. The approximate percentage of truss members (diagonals, verticals, upper and lower chord members) that were to be replaced was approximately 47%. The entire floor system, deck, machinery, electrical components and operator's house were recommended to be replaced.

Under Alternative 3, the Columbus Road Lift Bridge would be partially replaced with a new lift span that could potentially be fabricated in the shop without interruption to traffic. This would shorten the bridge closure and detour time to the amount of time required for tower rehabilitation, removal and installation of new lift span, and final painting. With a new lift span, there would be less maintenance and the design of the structure would better accommodate future movements due to creep.

Cost Analysis

The remaining alternatives and their associated estimated costs were as follows:

Alternative Description	Initial 2011 Cost	Present Worth Cost
Alternative 2 – Total Rehabilitation	$34.7 million	$49.2 million
Alternative 3 – Rehabilitation with Partial Replacement	$37.7 million	$49.1 million

The present worth costs include the initial 2011 construction costs and fifty year life cycle of rehabilitation and maintenance costs, and were based on year 2011 dollars.

Recommendation

TranSystems recommended Alternative 3 - Rehabilitation with Partial Replacement. This alternative accomplished the following:

- Met the primary and secondary purposes of the Purpose and Need Statement
- Repairs/replaces the deficient conditions of the existing structure
- Did not affect the historical characteristics of Irish Town Bend, Ohio City and surrounding vertical lift bridges
- Required a shorter detour length and less annual maintenance
- Was the most cost efficient in a 50 year life cycle analysis

STRUCTURAL DESIGN CONSIDERATIONS

A new 241'-9 1/2" Pratt-like through truss lift span consisting of rolled I-section lower chord and web members along with built-up upper chord members was designed to mimic the design of the original lift span. Like the original truss, the new truss lift span will be fracture critical and was designed at a 5% grade. The new lift span was designed to be 2 1/2" shorter than the original in order to account for the movement of the South Pier since the original construction of the bridge (see Figure 9). Also, to account for possible future movement of the South Pier, the deflector sheave at the north end of the truss was designed to be adjustable, which required an enlargement of the gusset plates at those locations (see Figure 10).

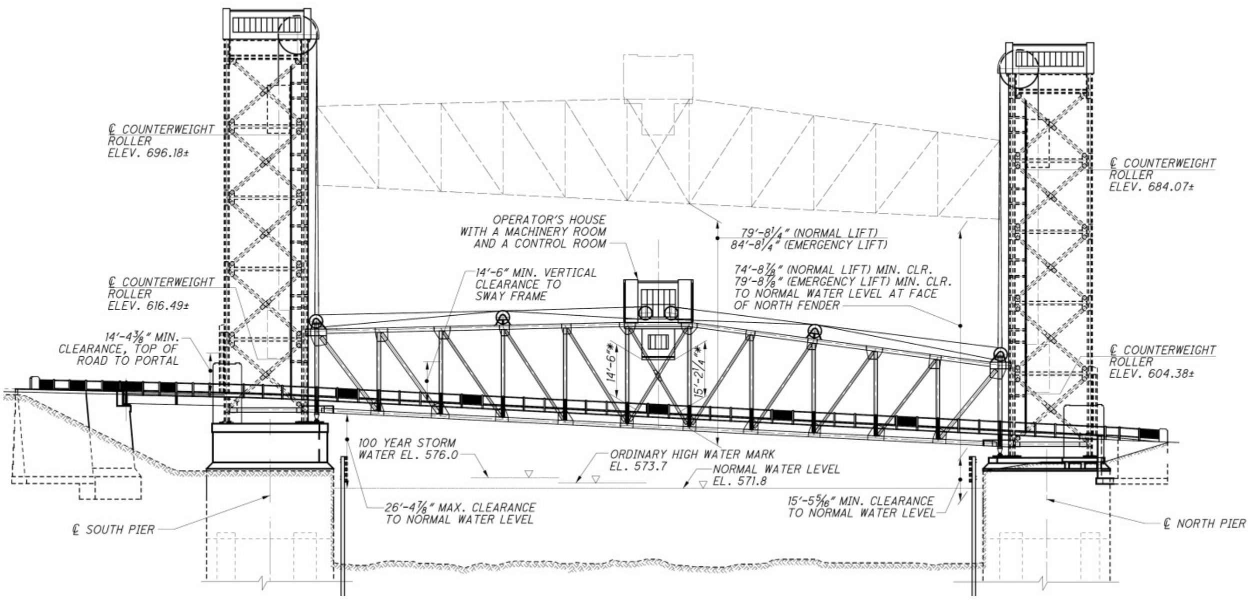


Figure 9: East elevation of new lift span and rehabilitated towers.

The results of the geotechnical investigation revealed that the factors of safety for the axial geotechnical capacity of the caissons were below 2.0 for both piers. Therefore, it was imperative to minimize not only the lift span weight during design but also the weight of the towers. To save weight on the truss, 2" fiberglass grid deck was used for the sidewalks in place of the original concrete fill grid sidewalk. Also, to save weight on the structural steel members, castellated beams were designed for the upper and lower lateral bracing and the sway bracing. For the roadway decking on the South Approach Span and Tower Spans, grid deck half-filled with lightweight concrete was used. On the lift span, open grid steel deck was used predominantly; however, lightweight concrete was for a four foot width along the curb lines and for a two foot width over the floorbeams to protect the structural steel framing.

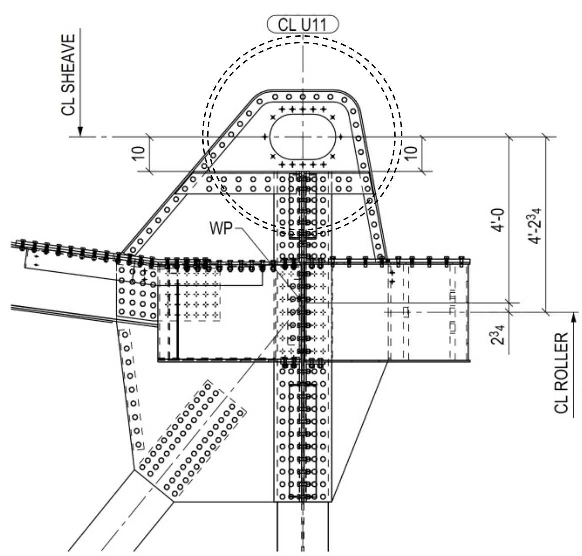


Figure 10: Enlarged gusset plate at U11 for deflector sheave.

The bases of both towers were noted to exhibit advanced section loss during the structural inspection. As a result of this section loss, it was necessary to remove and replace the bottom longitudinal struts between the forward and rear tower columns on both towers. Additionally, as part of this repair, the lower gusset plates at the base of the forward tower columns had to be removed and replaced due to deterioration. Several of the bearing angles at the bases of the forward columns also had to be replaced because of advanced section loss.

During the mechanical and structural inspection of the structure, excessive wear and gouging was noted on the South Tower running plates and the lift span was observed binding during operation. These issues can be attributed to the movement of the South Tower, which was surveyed and found to be leaning to the north. As part of the rehabilitation, TranSystems designed a jacking scheme and jacking lugs to be attached to the structure to allow for the towers to be jacked (see Figure 11). During these jacking operations, the heavily deteriorated pedestals at the bases of the towers will be replaced with grout and the towers plumbed.

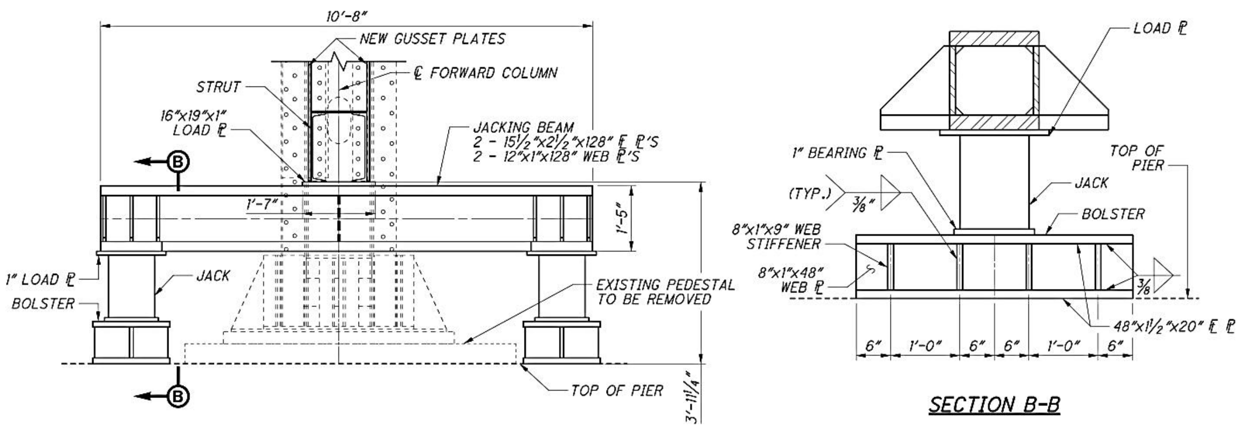


Figure 11: Details of tower jacking beam.

Replacement of the damaged portions of the counterweight running plates on the South Tower presented a unique problem, as the original running plates were connected to the tower legs with double countersunk rivets in order to eliminate any interference with the connector heads and the lift span rollers. Plug welding the running plates to the tower legs was not a feasible option due to cost concerns and due to concerns about replacing the running plate in the future. Therefore, the final design called for replacing the original double countersunk rivets with new ones even though there were concerns about finding anyone with the skill level and knowledge required to install them. Upon award of the construction contract and during the shop drawing review phase, the contractor, American Bridge, informed us that they were unable to find anyone who could install the double countersunk rivets as specified in the plans. Therefore, they proposed the use of double countersunk bolts which they would manufacture and test independently. These bolts consisted of a normal countersunk bolt that would mate with a machined countersunk barrel nut. Independent testing showed that these bolts provided sufficient clamping force and shear strength for use on the tower running plates, thus eliminating the need to use the double countersunk rivets.

As part of the total rehabilitation, the Approach and Tower Span roadway framing were to be replaced in their entirety. A new 35'-6" long multi-girder floor system was designed for the South Approach Span to match the existing framing configuration and make use of existing connections to the towers where possible. Similarly, the North and South Tower Spans were replaced with multi-girder framing configurations that made use of existing tower connections. The rear tower legs on both of the towers were filled with concrete during the original construction of the bridge, making connecting the girders and floorbeams to the rear columns using bolts not a viable option. Therefore, connection assemblies were designed that would bolt to the web of the floorbeam or girder and would be welded to the tower leg. The 9' long North Approach Span was eliminated completely by extending the North Abutment to the North Tower.

While designing the Tower Span Framing, it was determined that the minimum vertical clearance required by ODOT (14'-6") could not be met due to the geometry of the tower portal. Therefore, a design exception was granted in order to maintain the historical nature of the structure (see Figure 12).

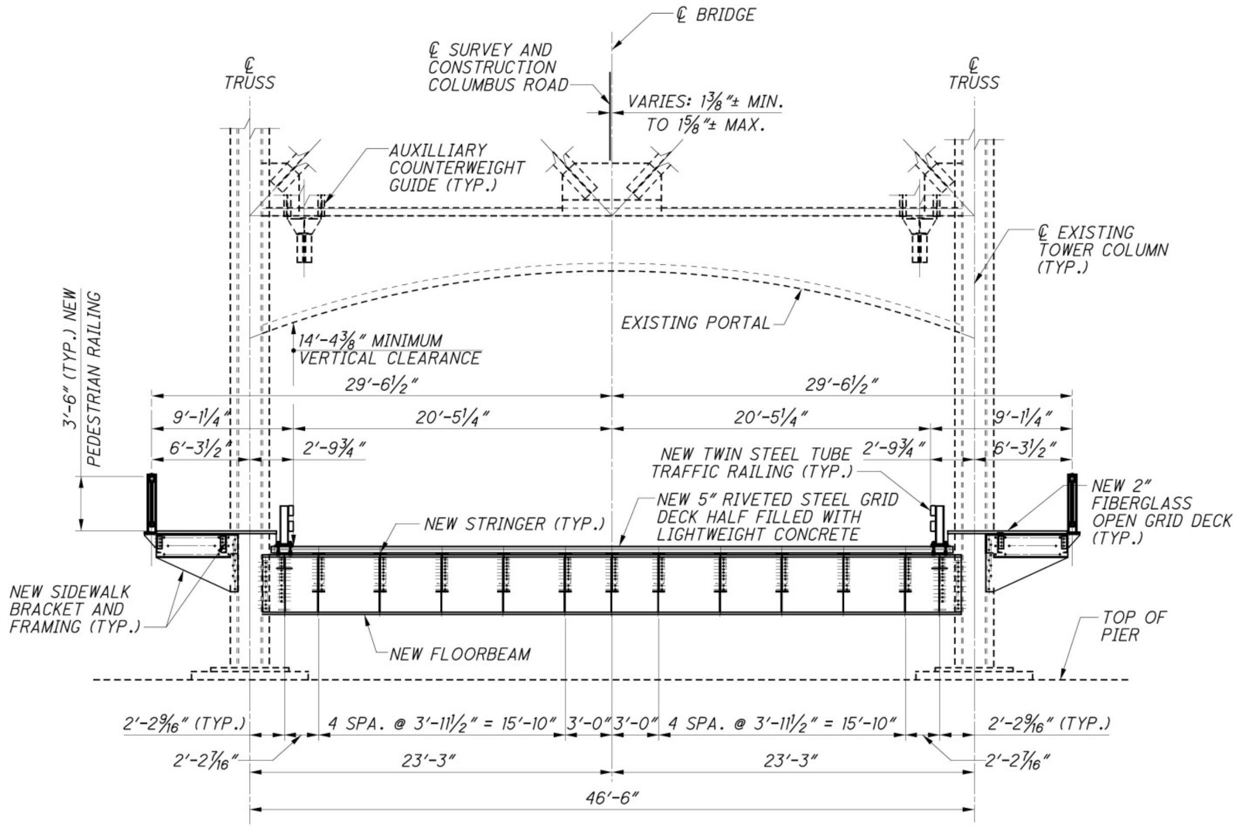


Figure 12: Typical section of rehabilitated tower span roadway framing.

Another unique design aspect of the rehabilitation was the replacement of the access platforms and ladders on the towers. At the request of the owner, TranSystems designed new access for the towers consisting of stairs and larger platforms to facilitate easier maintenance on the structure. This design was a challenge due to the need to meet OSHA requirements for platforms and stairs while staying within the limited space constraints of the existing towers and conforming to SHPO standards. Additionally, due to the concerns about the overall weight of the structure, these platforms were designed to be as light as possible. The final access system consists of eight platforms in each tower constructed of HSS tubing with fiberglass grating. These platforms are supported by I-sections that are cantilevered off of the tower forward struts and diagonals.

CONSTRUCTION SERVICES

On November 9, 2011, Cuyahoga County Department of Public Works awarded the rehabilitation of the Columbus Road Lift Bridge to American Bridge Company of Coraopolis, Pennsylvania for $30.3M. The funding source of the project is FHWA (80%), City of Cleveland (10%) and Cuyahoga County (10%). Notice to proceed was given on January 17, 2012 with a completion date of August 14, 2014. Throughout the duration of the construction, the project team has been providing construction services for the structural, mechanical, electrical and architectural elements of the bridge as well as the reviews of the float-out / float-in construction methods for the new truss and stowing the bridge in the open position while maintaining river traffic (see Photos 14 and 15).

Photo 14: On-going construction of the new Columbus Road Lift Span.

Photo 15: Float out of the existing Columbus Road Lift Span.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15-18, 2014

FULL STAKEHOLDER PROJECT RISK MANAGEMENT SOLUTIONS TO CONSTRUCTION CHALLENGES ON THE GILMERTON BRIDGE PROJECT

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
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ABSTRACT

The Gilmerton Bridge Replacement Project replaces the existing bascule bridge (circa 1938) with a state-of-the-art vertical-lift bridge spanning the southern branch of the Elizabeth River on Military Highway (Route 13) in the City of Chesapeake, Virginia. The innovative phasing of construction allowed for the 35,000 vehicles per day and maritime traffic to be maintained while the new bridge was built in parallel alignment underneath and over the existing bridge.

The project included several significant construction challenges which were compounded by an access restrictive site. The full replacement of the existing bridge was completed over top of the existing, active bridge and was constrained on either side by an existing rail bridge and existing businesses. Some notable challenges which required extensive coordination between stakeholders included the following:

- Phased Construction Allows for Minimum Impact to the Public: The phased construction allowed the project to be constructed with minimal impact to vehicular and maritime traffic.

- 12-Foot Diameter Drilled Shafts: 12-ft. diameter drilled shafts are among the largest ever constructed in the United States using the oscillator method with temporary casing.

- Lift Span Float-In: The 5.2-million pound lift span was constructed off-site and transported 7 miles through the Port of Hampton Roads. A full channel closure was obtained through the US Coast Guard with a fourteen day roadway closure. Through close collaboration between all stakeholders, the roadway was opened to the traveling public in half the allotted time.

- Challenging Machinery Installation & Alignment: Large operating components were required to be installed at heights of 125-ft. This required intense engineering and erection planning which then led into a detailed final alignment procedure in order to maintain the tight machinery tolerances specified on the project.

In order to address these challenges, the owner, contractor, engineer and construction manager participated in a full stakeholder/joint risk management program to provide a collaborative, project-focused process to resolve conflicts. This program resulted in the successful completion of the project and the development of effective solutions to our construction challenges.

PROJECT HISTORY

The Gilmerton Bridge Replacement Project replaces an existing twin-leaf bascule bridge completed in 1938 with a new $140M vertical lift bridge spanning the southern branch of the Elizabeth River. The bridge connects approximately 35,000 vehicles per day between the City of Chesapeake with the Cities of Portsmouth and Suffolk along US Route 13.

On February 12, 1937 the Carpenter Construction Company submitted a low bid of $448,504 for the construction of a drawbridge over the Southern Branch of the Elizabeth River[1]. The bridge provided a much discussed toll-free entrance into the City of Norfolk and was known as the “Toll-Free Bridge” near Gilmerton[2]. By the time construction was completed on March 25, 1938 the bridge had inherited the name of nearby Gilmerton and had become the *Gilmerton Bridge*.

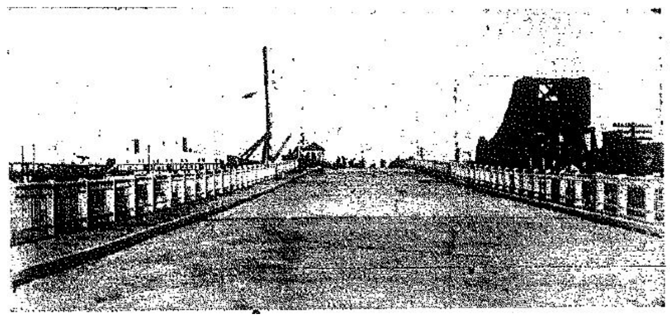

Figure 1. Norfolk's new toll-free route to the west in 1938.

The Gilmerton Bridge was uniquely situated in close parallel alignment with the Norfolk and Western Railroad at the Southern Branch of the Elizabeth River which was originally completed in 1908. It was the first case in the state of Virginia that two bridges were so closely situated that lifts would need to be performed simultaneously. At the time of construction, this required close coordination between the officials from the Highway Department, Norfolk and Western Railway and the District Engineer of the War Department, who controlled the regulations[3].

The bridge served as a vital link for people, goods and the military. Norfolk County and Gilmerton grew, prospered and then evolved into the City of Chesapeake. In the early 1990s permitting and design of a new bridge began. The risks to construction that existed in 1937 continued to exist in 1990: coordination of stakeholders, proximity of independent moveable bridges and preserving the interests of the travelling public.

Figure 2. M/V Sea Pearl passing through the Gilmerton Bridge.

The project was advertized for construction in 2009. The Virginia Department of Transportation recognized the significant risk spectrum of the project. Not only did the Department incorporate controls into the project, but also elected to sponsor a full stakeholder/joint project risk management program. When the project was awarded in late 2009, the contractor was invited to participate in this program to provide a collaborative project-focused process to ensure successful project delivery.

From the onset of the Gilmerton Bridge’s history, coordination between stakeholders in order to develop solutions was vital to success. A clear understanding of the projects challenges and vision of the future was necessary in order to meet the demands of the project and to serve the interests of the public.

FULL STAKEHOLDER PROJECT RISK MANAGEMENT

In collaboration with VDOT's State Construction Engineer, a Full Stakeholder/Joint Project Risk Management Plan was developed by Parsons Brinkerhoff's Marc E. Papini. Once the framework was developed, the contractor, PCL, accepted the invitation to participate in this program to much success. This plan was developed based on established framework used in other industries. The program affords the Owner, Contractor, Engineer, and Construction Manager the opportunity to take advantage of all the collective knowledge of each. Through this process, a risk-aware culture was fostered among all stakeholders, creating the foundation for proactive management of the project.

Project risks were identified by the team members in workshops which included all the stakeholders on the project. Large quantities of risks were systematically listed, categorized and assessed based on probability of occurrence and severity. This was used to assign a risk score to each item. This data was consolidated into a project risk register which was then tied to activities in the CPM schedule.

Using the risk register as a guide, the project team developed detailed preventative action plans to reduce the probability of occurrence of each risk. These plans were developed onsite and submitted for review and approval. Implementation of the action plans was tracked on the project. Pre-operational meetings were conducted before beginning project activities to review the action plans with project supervision and craft workers directly involved with the work.

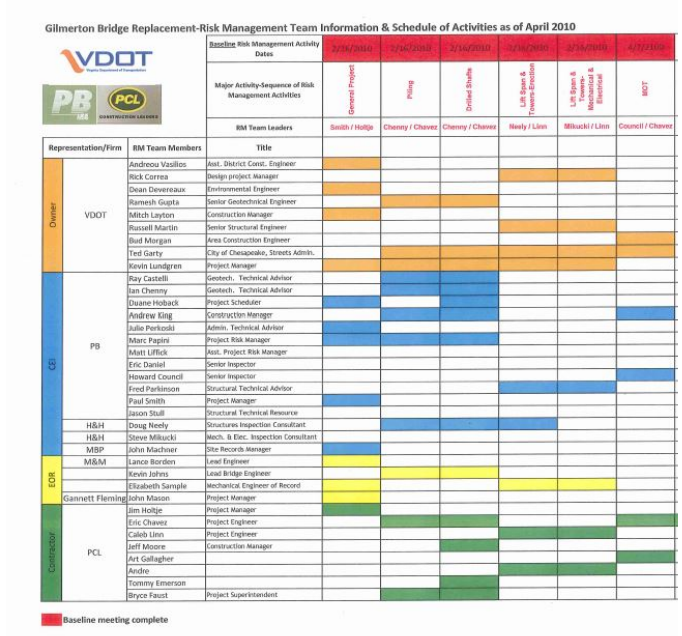

Figure 3. Risk management workshop schedule.

Risk items which reached a significant threshold of probability and severity were flagged as high risk items. These items were assigned a detailed mitigation plan and resolution flowchart. In many cases these resolution plans were tied into provisions of the Standard Specifications. However since the resolution plans were project specific, they were developed in much greater detail and provided for a more comprehensive resolution process which was also consistent with all provisions of the contract.

As the work progressed, the management plan developed further as lessons learned on the project were incorporated into previously identified risks. Lessons learned were discussed, documented and put into action. Preventative action plans were revised and mitigation plans were adjusted accordingly.

Further to this strategy of risk management, a dedicated public affairs officer and support staff worked in collaboration with VDOT and the project team. The public affairs team keeps all stakeholders informed and manages all communications with the media, surrounding residents, businesses and the traveling public. The successful communications plan created favorable press coverage and kept the vehicular and maritime traffic informed.

The implementation of the risk management program kept the team focused on project specific risks which could potentially derail the project provided a basis for resolution of both minor and major issues. The program ultimately resulted in several major issues being resolved early, before negatively impacting the project. This saved money, time and resources for all parties involved.

In 2012, the Gilmerton Bridge project was recognized for this by receiving VDOT's highest honor, the VDOT Commissioner Award for Outstanding Achievement in the Innovation and Quality Improvement category for our Project Risk Management Program.

Risk Register - RMS

Risk Management Process					Step No. 1: Risk Identification	Step No. 2: Risk Assessment					Step No. 3: Response Planning					Step No. 4: Monitor and Control			
Risk ID	Major Activity-Activities taken from the Project Schedule	Level 1: RBS Pursuant to the VDOT Risk Breakdown Structure	Level 2: RBS - Pursuant to he VDOT Risk Breakdown Structure	Level 2: Description	ID Description	Probability	Cost Impact	Schedule Impact	Risk Score	Impact ID	Mitigation Efforts (General)	Mitigation Team Leaders	Risk Mitigation Tools	Preventative Action	Response Action	Project Risk Owner	Trigger Event	Baseline (2/16/10) /Added Date	Status
DS-18	Drilled Shafts	CM	Construction Means & Methods		Failure to maintain continuous water head during excavation.	3	2	3	9	SIP-02202,SIP-02230		Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	Y	Baseline	OPEN
DS-12	Drilled Shafts	CM	Construction Means & Methods		Failure to advance casing full length to design tip elevation.	3	3	3	9	SIP-02202,SIP-02230		Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	Y	Baseline	OPEN
DS-20	Drilled Shafts	CM	Construction Means & Methods		Improper bottom cleaning prior to pouring	3	3	3	9			Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	Y	Baseline	OPEN
DS12-10	Drilled Shafts	CM	Construction Means & Methods	Adequacy of the contractors & subcontractors work plan, value engineering proposals & adequacy of labor, equipment, material, & staffing to perform the work	Inexperience in construction of 12' diameter drilled shaft installation	3	3	3	9	SIP-02230		Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	N	Baseline	OPEN
DS-14	Drilled Shafts	CM	Construction Means & Methods		Improper mix design - Design Issues: workability, etc.	2	3	3	6	SIP-02202		Chaney/Chavez	SP - Hydraulic Cement Concrete Operations for Massive Construction, Pg. 513	Y	Y	PCL	Y	Baseline	OPEN
DS12-2	Drilled Shafts	CM	Schedule	Adequacy of the time allowed for the work & accuracy of the schedule	Delayed approval of drilled shaft submission	2	1	2	4	SIP-022304,SIP02202		Chaney/Chavez	SP - Working Drawings & Submittals, Pg. 256	Y	Y	CEI	Y	Baseline	OPEN
DS12-1	Drilled Shafts	CM	Schedule		Incomplete Submittal from Contractor	2	1	2	4	SIP-022304,SIP02202		Chaney/Chavez	SP - Working Drawings & Submittals, Pg. 256; SP - Drill Shafts, Pg. 473	Y	N	PCL	Y	Baseline	OPEN
DS12-6	Drilled Shafts	CM	Quality		Mass concrete temperature exceeds 85 degrees f-non allowance of cooling tubes	2	2	1	4	SIP-02230		Chaney/Chavez	SP - Hydraulic Cement Concrete Operations for Massive Construction, Pg. 513	Y	Y	PCL	Y	Baseline	OPEN
DS-3	Drilled Shafts	CM	Schedule		Inability to procure casing material to meet the schedule requirements	2	1	2	4			Chaney/Chavez	SP - Progress Schedule, Sect 106.01, Pg. 371 - Source of Supply & Quality Requirement	Y	Y	PCL	Y	Baseline	CLOSED
DS-29	Drilled Shafts	CM	Construction Means & Methods		Cold joint(s) due to delayed concrete placement or loss of tremie pipe embedment	2	2	2	4			Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	Y	Baseline	OPEN
DS-30	Drilled Shafts	CM	Construction Means & Methods		Concrete Defects.	2	2	2	4	SIP-02232		Chaney/Chavez	SP - Drilled Shafts, Pg. 473	Y	Y	PCL	Y	Baseline	OPEN

Thursday, April 29, 2010

Page 1 of 3

Figure 4. Risk register for drilled shaft construction.

PHASED CONSTRUCTION

Due to the limitation of available right of way, the new lift bridge had to be constructed in the same alignment as the existing bascule bridge. Foundations needed to be constructed underneath the active bridge and the lift bridge towers needed to be constructed overtop of the existing bridge. Considering the proximity of construction activities to the travelling public and unique phasing of the work, the project team immediately recognized that ensuring safe passage of vehicular and marine traffic was of paramount concern.

By analyzing traffic counts performed by the Department, the team developed a nightly closure schedule which minimized impacts to the motoring public. From here the team jointly developed modifications to the traffic control plan which led to fewer impacts to motorists and allowed for safe prosecution of the foundation work and overhead work.

Although work activities were restricted to night, debris netting, fabric and platforms were used to enclose storage areas on the structure. The towers were then jointly inspected at the end of every shift by a team of both contractor and engineer members. Every loose piece of debris was removed, tools were secured in toolboxes and materials were lashed down to prevent any possibility of movement during the day.

Tower sweeps continued for over three years on the project through various phases of the work. This was done with the goal of reducing exposure to the travelling public and minimizing the risk of having a significant traffic incident on the project. Through dedication of a risk-aware culture on the project, this goal was achieved and it was done so without missing a single rush-hour opening.

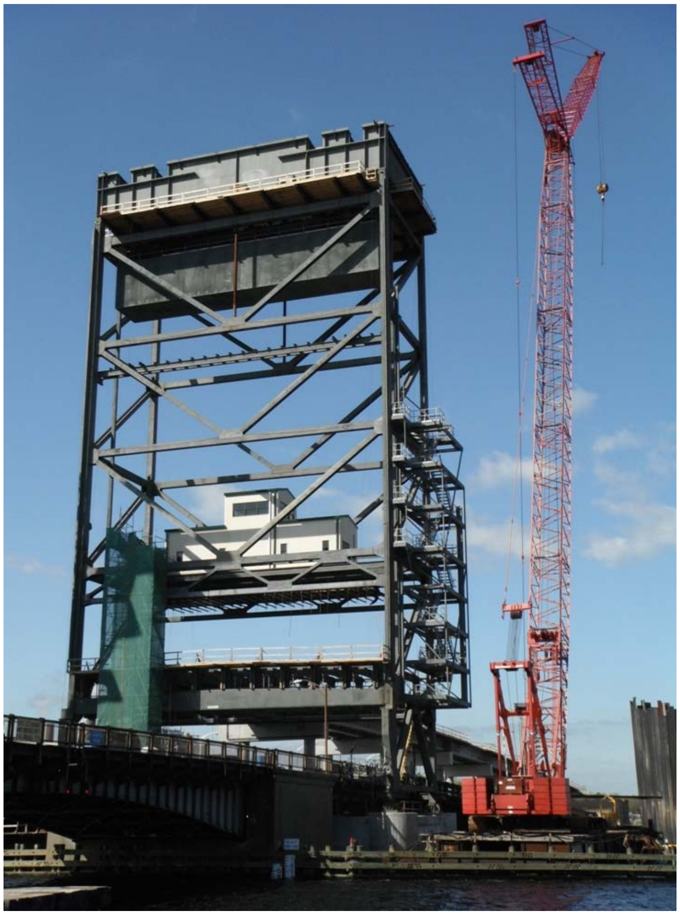

Figure 5. Nightly sweeps were conducted throughout the entire tower to ensure no risk to the public.

DRILLED SHAFT FOUNDATIONS

The plans called for the installation of eight each, twelve foot diameter fully-cased drilled shafts to support the new lift towers. Each tower was founded on four shafts which were tied together with a cast in place concrete cap referred to as the diaphragm. Each drilled shaft was to be 184 feet long with approximately 105 feet of embedment into the Yorktown Formation. The approach spans were founded on five foot diameter drilled shafts and twenty-four inch concrete pile. The Yorktown Formation contains greenish-gray, medium dense, preconsolidated, fossiliferous fine sand.

In order to validate the design criteria, the project called for the performance of three five foot diameter technique shafts. Two of these shafts required a static load test and one called for an Osterberg Load Test. This data was used to validate the unit skin friction and bearing capacities of the in-situ soils. This information was used to indirectly validate the capacity of the twelve foot shafts.

Due to the proximity to the Norfolk Southern Railroad Bridge, the project included several provisions for mitigating risk of damage to this existing facility. Provisions for preconstruction surveys and vibration monitoring were included. The project requirements also precluded the use of vibratory hammers due to the risk of settlement of the existing railroad bridge. Isolation cells were required if impact hammers were selected. Telescoping casing was required if drilling was selected. The team clearly recognized the Owner’s concern of damage to the existing railroad bridge and developed a plan in collaboration with nationally recognized drilled shaft experts to successfully complete the foundations.

After several meetings and reviews, the team elected to proceed with the oscillator method – a method rarely attempted before at this size. This method included the use of a large hydraulic oscillator which twists the casing while producing zero vibration into the surrounding soils. The weight of the casing and the ‘crowding’ ability of the oscillator pushed the shaft casing into the ground. As the casing was advanced soils within the casing were removed with a hammer grab. Once the casing reached the target elevation the bottom was cleaned and prepared for concrete. A full length reinforcing cage was placed into the excavation. Concrete was then deposited using the wet method while the casing was simultaneously extracted by the oscillator.

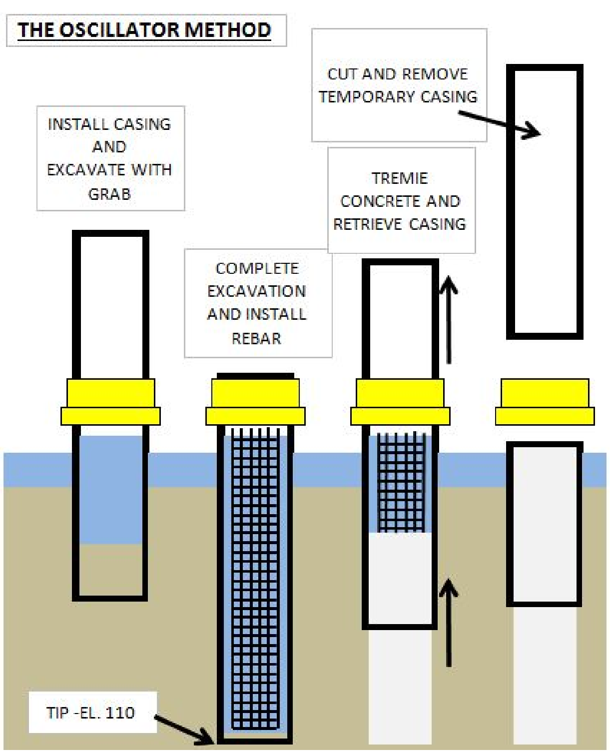

Figure 6. Diagram of the oscillator method with temporary casing.

Figure 7. Placing the rebar cage after excavation is complete.

This alternative plan allowed for the incorporation of a bi-directional Osterberg test. An Osterberg test provides direct measurement of the side resistance and base resistance of the shaft by sandwiching an expendable jack between an upper and lower load plate within the test shaft[4]. By performing a direct load test on a production shaft designers were able to further refine their design parameters for the foundations with a higher degree of certainty. The cumulative effect of this alternative was to provide direct tested drilled shafts, using a zero vibration method with an increased unit skin friction capacity. The financial effect was a $420,000 cost savings to the project.

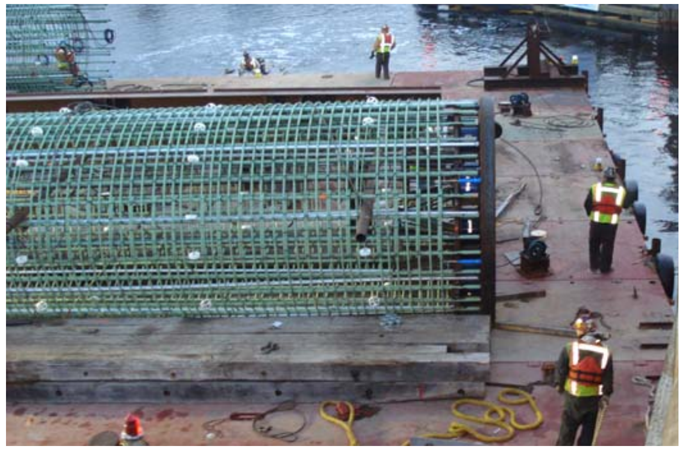

Figure 8. Reinforcing steel cage with Osterberg Load Cells installed.

The foundations were successfully completed using the methods developed by the project team. Throughout the foundation work the team continued to apply the principles developed in the risk management plan. Lessons learned were developed and incorporated into the remainder of the work. The team was able to jointly manage, mitigate and successfully complete one of the riskiest components of the project by applying the collective expertise of all the affiliated stakeholders.

LIFT SPAN FLOAT-IN

In order to minimize disruption to the travelling public, the project included provisions for a float-in of the new lift span. While the lift towers were being constructed onsite the new span was simultaneously constructed at an offsite location approximately seven miles away within the limits of the City of Norfolk. Once completed the new span would be 250 feet long by 90 feet wide and weigh approximately 2,700 tons. The course the span would take was complicated by the need to pass through five existing bridges. The channel configurations of these bridges limited the barge size to a maximum of 125'x125'. The span was constructed at grade and would need to transit the Elizabeth River at an elevation of thirty feet above the waters surface.

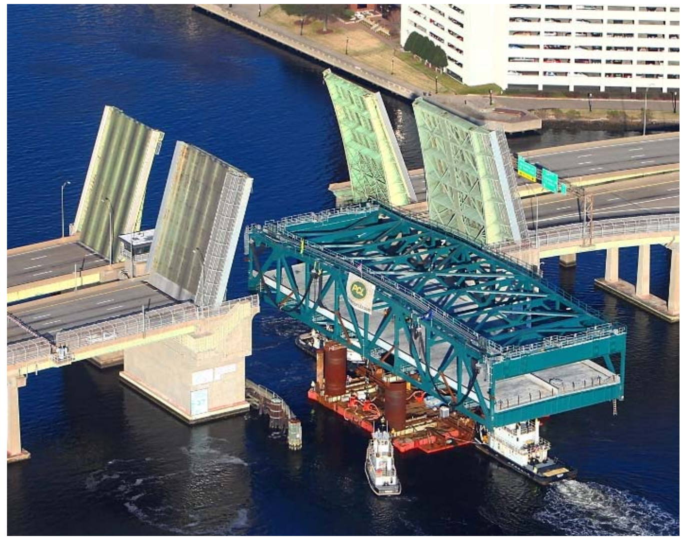

Figure 9. The Gilmerton Bridge lift span passing through the Berkley Bridge in Norfolk, VA.

Based on the selected preassembly location, the span would need to move from the Eastern Branch of the Elizabeth River to the Southern Branch. This route required the span to travel through the Port of Hampton Roads. This would affect the operations of the US Navy, US Coast Guard, major industrial river traffic and recreational boaters. Through the use of dedicated points of contact, the project developed a comprehensive outreach plan to initiate and maintain contact with these groups. In close coordination with the US Coast Guard and city officials, a robust communication plan was developed and successfully implemented.

Of paramount significance to the project was the safe transport of the span through the port. In order to ensure success, several mitigation strategies were employed by the project. The float-in plan was developed by the team and then reviewed by independent structural engineers. Due to the unique stability characteristics of the barge and the span the team also employed a specialty naval architect/marine engineer to review the plan. Plans were finalized, sealed and submitted for regulatory review by the US Coast Guard and the Department of Transportation.

Figure 10. A full scale mock up on the float in barge being conducted at the Gilmerton Bridge site.

Additionally, the team conducted several, full scale mock up trials to ensure success and eliminate problems in a controlled manner. This included transporting the float-in barge in an unloaded but fully ballasted condition from the site to the span assembly yard. This was done by ballasting the barge with water. This provided the team accurate schedule information and allowed tug boat captains the opportunity to handle the barge in a low risk scenario. Once at the assembly yard, a test lift was conducted to verify the structural integrity of the barge to support the span.

In the days leading up to the float-in a series of meetings were conducted between the critical stakeholders. These meetings included the owner, contractor and the US Coast Guard. A daily assessment of preparations and weather conditions was made and formally documented. Then in the early morning hours of January 7, 2013 the new lift span was moved into position on the Eastern Branch of the Elizabeth River.

The span was transported using a temporary safety zone established by the US Coast Guard and the City of Chesapeake Marine Police. At the confluence of the Eastern Branch and the Southern Branch an escort was provided by the CGC Shearwater. As the span travelled the river, the safety zone was lifted allowing port commerce to continue.

Once the span was safely underway removal of the existing bascule span commenced. The existing bascule span needed to be removed since it conflicted with the new span and barge. The removal was accomplished with the span in the down position. This allowed a rapid float-out of the existing span.

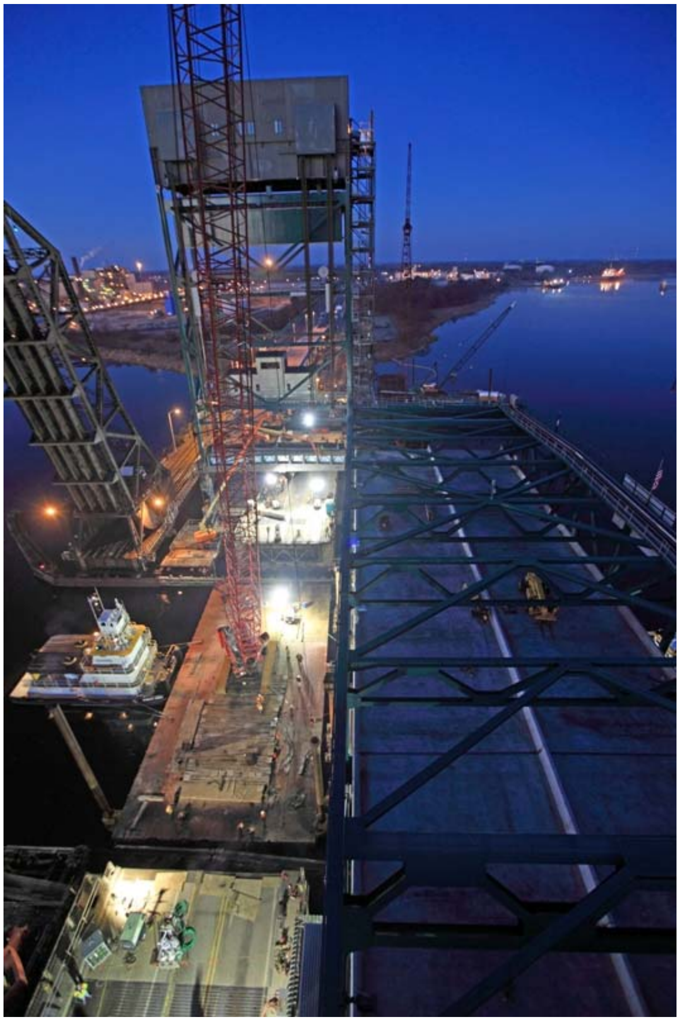

Figure 11. The new lift span moves into position between the lift towers.

With the existing span removed, the team was able to position the new span in between the lift towers. Installation of the wire ropes, final alignment of the machinery and placement of the closure joints immediately started and continued around the clock. Ultimately the Gilmerton Bridge was re-opened to traffic on July 14, 2013 in half the allotted time to a warm reception of motorists and media[5].

In an operation described by the US Coast Guard as “flawless”, the project team was able to successfully accomplish the most significant task associated with the project. This was achieved through the dedication and perseverance of all team members to maintain focus on the objective. The projects joint risk management program provided the framework and tools needed to meet this goal.

MACHINERY INSTALLATION & ALIGNMENT

The Gilmerton Bridge is a tower drive vertical lift bridge which operates by mechanically turning the tower top sheaves. The sheaves move the counterweight ropes attached to the span and counterweight. Drive pinions engage the circular rack sections attached to each sheave. Both towers have identical machinery which is controlled electronically to keep the span level.

Figure 12. Right hand secondary reducer and sheave assembly.

Each tower houses four fifteen-foot diameter, welded steel sheaves which support twelve 2-1/4" counterweight ropes each. Ring gears on each sheave mesh with the pinion gears which turn the sheaves. The sheaves are driven by a pinion shaft which is coupled to a 10:1 secondary reducer. Two secondary reducer assemblies in each tower are driven by a floating shaft which is coupled to the output shaft of the single 8.24:1 primary reducer. The main drive motors are 200 horsepower, six pole, totally enclosed three phase motors operating at 1170 RPM synchronous speed.

Figure 13. Setting an 80 ton sheave assembly.

Installation and pre-alignment of the machinery was completed before beginning the lift span float-in. The sheer size of the components presented challenges to the project team. The sheaves and trunnion assemblies were approximately 80 tons each and were hoisted 200 feet from a floating crane. Logistics needed to be coordinated between many stakeholders to ensure success.

During the construction of a tower drive vertical lift bridge, significant deflection of the structure can be expected due to the dead load of the lift span and counterweights[6]. Due to the width and height of the new span and towers, provisions were made for the deflections during pre-alignment of the machinery. The theoretical forward deflection of the towers was 7/16 inch which was compounded by a theoretical deflection of the machinery house floor of 2.86 inches. The machinery was initially installed with intentional misalignment so that when the structure deflected, the shafts, pinions and sheaves would be as close as possible to proper alignment during the float-in closure.

Just prior to starting the closure an unexpected complication significantly increased the stakes of a successful float-in. Dominion Power took a coal delivery from the M/V Caribe Pearl at their pier facility immediately down river from the construction site. This vessel is a 600 foot long by 95 foot wide, 38,760DWT cargo ship. Although the new bridge wouldn't open to traffic until the seventh day of the closure, it was imperative that the bridge open to allow passage of the Caribe Pearl on the fifth day of the closure to avoid demurrage costs for the vessel. This late developing wrinkle very rapidly became one of the projects greatest risks.

The team fell back on a pre-established mitigation strategy to ensure success of the interim machinery alignment and startup of the new lift system. Minimum construction alignment tolerances were pre-established with the EOR and a plan was developed to use the auxiliary drive system to run the bridge. Engineers, millwrights and support staff were split into day shift and night shift teams. With the full application of the dead load, iterations of machinery alignment were performed until the construction tolerances were met. On the fifth day of the closure the new lift span was successfully lifted to its full open position with the use of the auxiliary drive system.

Figure 14. Trunnion piano wire illuminated by a head lamp. Four sheaves were aligned to a 1/16" concentric tolerance of each other.

CONCLUSION

Today Gilmerton is part of Virginia's third largest city and continues to serve as a vital transportation link for the community. When the project began in 2009, the Virginia Department of Transportation recognized the risk spectrum of the project. In partnership with the construction team, the Department developed a robust, site-specific full stakeholder project risk management plan.

Figure 15. Barge traffic can move easier under the new 35-ft minimum clearance without the need for an opening.

A full stakeholder project risk management program aligns team members around solutions to problems. A pool of solutions are created early in the project which can then be applied later should an event occur[7]. The success of the project was predicated on the interaction of technical experts to quickly develop solutions to unavoidable construction challenges.

By having a pre-established risk mitigation strategy in place long before the work was scheduled to begin, the team had built an effective tool to manage the dynamic changes associated with complex bridge construction. A unique aspect of the Gilmerton Bridge project was the formal application of this strategy between all stakeholders. By making a significant investment early in the project in the people who would execute the work, the project was able to capitalize on the technical expertise of all participants in order to successfully deliver the project.

Figure 16. The nearly complete Gilmerton Lift Bridge.

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[3] Willson, G.B. (1938). "The Gilmerton Bridge". Virginia Highway Bulletin (April 1938).
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[5] Forster, D. (2013) "New Gilmerton Bridge Opens Early." The Virginian-Pilot (January 15, 2013), pp. 3.
[6] Koglin, T. L. (2003) "Moveable Bridge Engineering, Edition 1". Wiley, John & Sons, Inc. Hoboken, NJ.
[7] Cacamis, M. E., El Asmar, M. (2014) "Improving Project Performance through Partnering and Emotional Intelligence". *Pract. Period. Struct. Des. Constr.*, 10.1061/(ASCE)SC.1943-5576.0000180, 50-56.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Heavy Duty Riveted Bridge Deck Fatigue Testing under AASHTO H20 Loading

Dr. Craig C. Menzemer, PhD, University of Akron
Kenneth P. Apperson, PE - Ohio Gratings Inc.
Aristotle Zournas – Ohio Gratings Inc.


Laboratory Fatigue Testing Confirms Long Service Life Expected

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

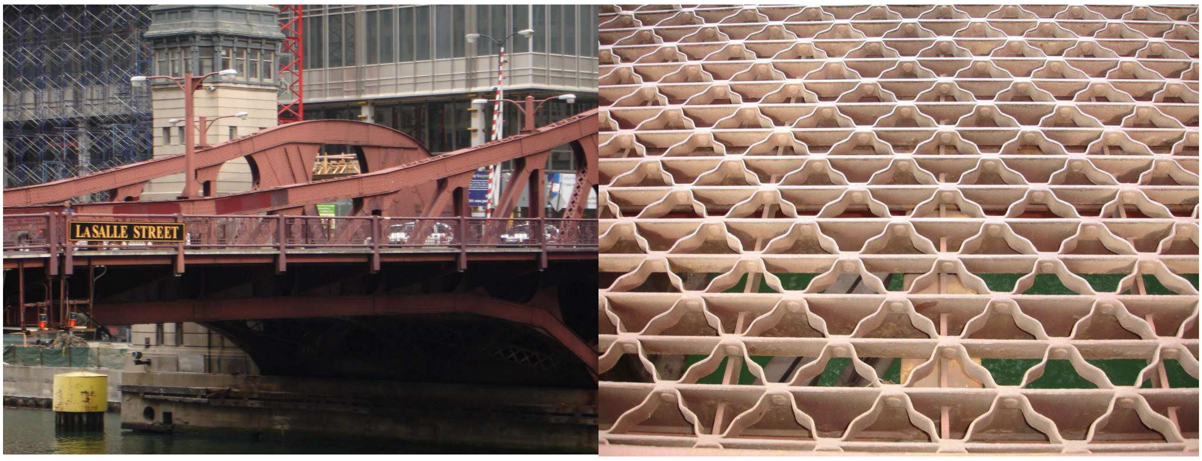
Introduction

This paper presents results of on-going fatigue testing conducted at the University of Akron Civil Engineering Labs from 2011 thru 2014. This paper is a continuation of a previously presented paper during the 2010 HMS symposium. It presents results from fatigue cycle testing of Heavy Duty Riveted Bridge Deck Grating under AASHTO H20 loading with a 30% impact factor, while highlighting the significance of the attachment methods and how they are related to fatigue. Fatigue test results are presented for full-scale decks cycled to over 2 million cycles. Deck samples were also tested to higher stresses to establish the fatigue detail of the riveted design. Loading with an actual truck axle and heavy duty tires is presented.

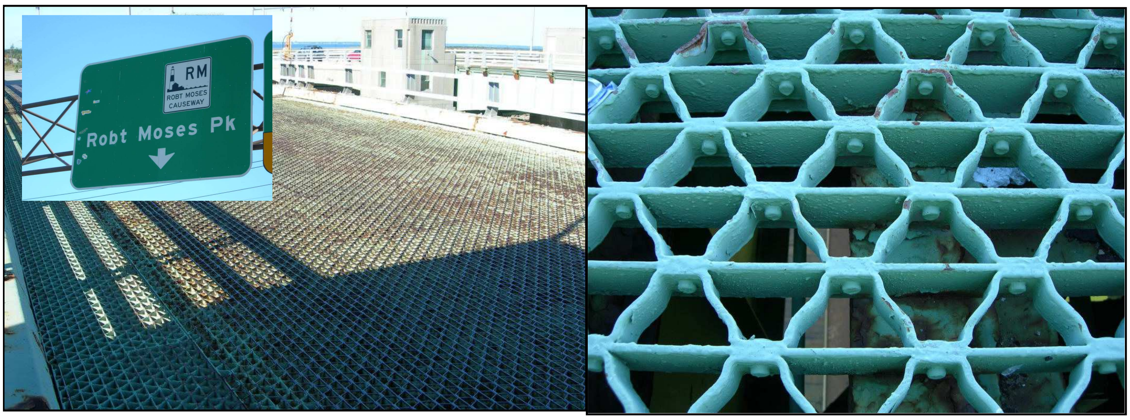
Background and History of Steel Bridge Decks

As discussed in the previous paper presented during the HMS 2010 symposium, history shows the durability of the heavy duty riveted bridge deck. Its proven long life has been observed in many moveable bridges around the nation. Some examples include the Veterans Memorial Bridge in Bay City, Michigan (shown directly below), the historic LaSalle Street Bridge in Chicago, IL, and the Robert Moses Causeway Southbound Bridge at Captree State Park in Long Island, New York. Heavy Duty Riveted Grating is a proven open steel bridge deck solution.

Veteran's Memorial Bridge Bay City, Michigan Riveted Grating installed in 1994. Like new after over 15 years in service.



The LaSalle St Bridge in Chicago, Illinois with
Riveted Bridge Deck installed in 1971 is still in good condition after over 37 years in service.



The Robert Moses Parkway Bridge in Long Island, New York with riveted steel deck installed in 1951 was still in service after 56 years.

Laboratory Testing of Riveted Deck

During the time period between 2011 and 2014, the University of Akron continued the research and testing program sponsored by Ohio Gratings Inc. that started in 2009. One purpose of the project was to successfully reach 2 million cycles under the AASHTO H20 wheel load with 30% impact factor, while using a bolted method to secure the deck to the supports. In addition, the project investigated the fatigue resistance of the heavy duty riveted bridge deck in order to establish the fatigue behavior, especially in the negative bending moment areas over stringer supports. It was assumed that one reason the riveted decks have performed so well in the field was due to the fact that there are no welds at the top surface where the negative bending puts the top surface in tension. The fact that the rivets are centered below the top surface in a lower stress area is believed to be a major reason for the outstanding performance in the field.

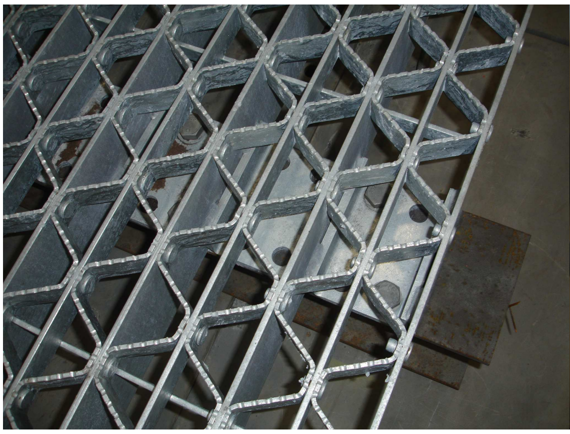
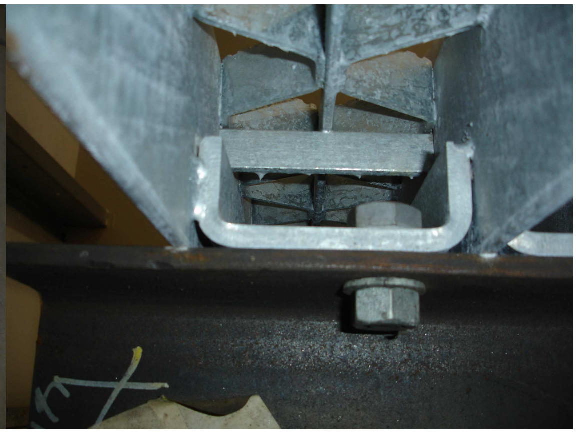
Fatigue Tests:

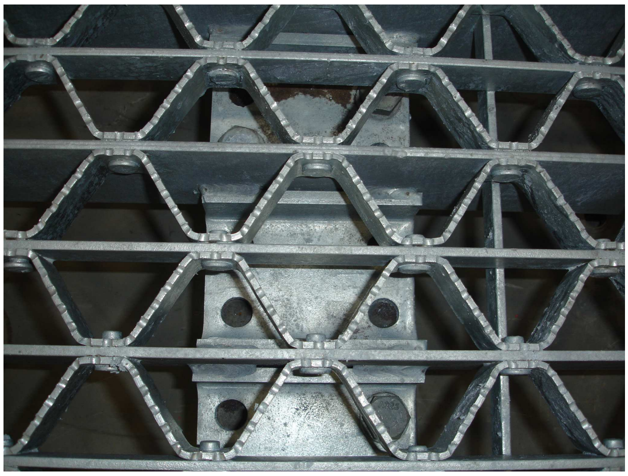
The fatigue test data presented is based on the same Ohio Gratings type 37R5 Lite 5” x ¼” with bearing bars of type ASTM A-36 steel that was introduced in the previous paper presented at the HMS 2010 symposium. The stringer spacing was increased from 49” on center to 50” on center. Another difference in relation to the previous test is that this time a bolted attachment method is used. Previously, the deck was attached directly to the supports by using the recommended by AASHTO fillet welding (1-1/2” long, 3/16” staggered) at every support / bearing bar intersection.

In an attempt to perform the testing with loads as close as possible to actual loading conditions, the lab attempted to load the bridge deck grating with an actual truck axle adapted to laboratory equipment for loading. Typical truck tires available are rated for 6,500 lbs. /each thus 13,000 lbs. per set. The axle was equipped with tires of the highest rated tire available which was 9,900 lbs. / tire. This would allow a maximum axle load of 39,600 lbs. which is less than the 41,600 lbs. per H20 with 30% impact. The idea of continuing the fatigue testing with this load configuration was short lived. Once the axle was loaded in order to produce the 41,600 lbs., the tires were compressed to such an extent that the safety of the lab technicians was a concern. Therefore, the idea of performing the fatigue test with an actual axle and tires had to be rejected for practical reasons. This does point out the fact that the axle load levels used in this testing far exceed those from actual trucks in service.

A spreader beam was then utilized to simulate the axle. Two 10” x 20” steel plates were welded under the spreader beam. These plates were used to provide the AASHTO tire patch for an H20 loading. The plates were placed 72” on center to simulate a design truck axle. High durometer rubber pads were placed under the plates in order for the load to act like a “tire”. Loading was arranged to produce the maximum negative moment over the center support, and was representative of 16 kip wheel load plus 30% impact. A small positive loading ratio, R, was maintained: axle loads varied from 1000 lbs. to 42,600 lbs., producing an effective load range of 41,600 lbs. and thus producing the 20,800 lbs. wheel loads. With the maximum negative moment occurring over the support, the details of interest were the rivet connections between the main bearing bar and adjacent reticuline bars, as well as the attachment of the deck to the supporting structure. In this case, the supporting structure was represented by a series of W 8 x 25 I-beams intended to act like stringers. Three such supports were spaced at 4’ 2” (50 inches) on center.

As previously mentioned, the attachment to the supports was a main focus for this testing. During the previous testing, the welded connection of the deck directly to the supports was shown to be a limiting factor for the fatigue life of the deck. Therefore, for this testing C-shaped steel attachment brackets were shop welded in the grates. The bracket thickness is 3/8”. Each bracket has pre-drilled holes for up to four bolts for attachment to the supports. The brackets are welded in between the 5” x 1/4" bearing bars, at the location of the supports.



Bracket attachment to the grates:
Tensile residual stress fields exist adjacent to a vast majority of weldments, due to the uneven heating and cooling that occurs during the joining process. Often these local residual stresses may be on the order of the yield point of the base metal. Therefore, and as confirmed on the previous test results, welding is not desired at a higher stress area. For this reason, the C-shaped brackets were attached to the bearing bars by applying 1-1/2” long 1/4” fillet welds at each corner of the bracket. The welds were therefore closer to the neutral axis of the bearing bars, and thus subjected to lower stresses during load cycling.

Two individual panels each having a 11’ 5” span x 3’ 0-1/8” width were tested. The panels were side spliced at the panel separation with 5/8” dia bolts and spacers at 15” on center. Initially, the “wheel” loads were placed at the center of the panel width, on one of two panels. The loads were located in order for their centers to be placed 3’ from the center of the middle support. A total of 2 million loading cycles were applied with a frequency of 1 Hz. The maximum recorded strains at the negative bending moment region at the center support were about 550 micro strains measured by a stain gage ¼” from the top of the main bar, translating to approximately 16,000 PSI of bending stress. After the completion of the 2 million cycles, no evidence of any fatigue cracks was observed in the panels. The loads were then moved to the other panel and located adjacent to the panel's edge, at the side splice separation. The same level of cyclic loading was repeated for 2 million cycles with no evidence of any fatigue cracks.

Following the successful completion of the above two tests, which used the AASHTO H20 16 kip wheel load with a 30% impact factor, the same exact two test configurations were repeated using the same sample panels, but this time having an impact factor of approximately 61%. This overloaded condition translates to an axle load of 51,600 lbs. or wheels loads of 25,800 lbs. Under this excess load, the deck samples were cycled for an additional 630,000 cycles with no sign of fatigue cracks.

Conclusion:
The laboratory testing confirms that the heavy duty riveted bridge deck is very resistant to fatigue cracking under heavy truck loading. This type deck can be relied upon to provide decades of service even when exposed to heavy truck loading.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Moving the Sellwood Bridge

Jon P. Henrichsen, PE
Carly Clark, PE
Multnomah County, Oregon

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
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Contents

NEPA

Background

By 2005 the Sellwood Bridge was in very bad shape. The bridge had been load limited to 10 tons, resulting in the removal two bus routes from the bridge and banning all large truck traffic. Girders in the west approach structure had cracked through and had to be splinted with steel plates and all thread rods to be left in service. The two 12-foot lanes and west end interchange were inadequate to meet the requirements of the 30,000 vehicles that crossed the bridge each day. Evening commutes were particularly awful, backing up traffic on Oregon Route 43 for miles. Although the bridge crosses a park on the west end, is adjacent to a primary bike route through River View Cemetery, and crosses the Springwater Trail (a major bike/pedestrian corridor on the east side), the original structure was a major gap in Portland's cycling infrastructure due to poor connections on and off the bridge and a narrow 4-foot sidewalk, made even narrower by light poles every 100 feet. The cumulative impact of all of these factors led the County to begin a major project to either rehabilitate or replace the Sellwood Bridge and correct as many deficiencies with the existing bridge as was feasible given the limitations of the project site and public opinion about traffic and lanes on the bridge. The first step to get the project moving toward construction was the development of an Environmental Impact Statement (EIS) in accordance with the National Environmental Policy Act (NEPA), culminating in a record of decision that would be a road map for design, right of way acquisition, and construction. In 2006, the County began the NEPA process for the Sellwood Bridge.

Development of the Public Process

The purpose of developing an EIS is to delineate the impacts of the proposed project to the built and natural environments. With that, a community can weigh the proposed alternatives against factors including the number of businesses or residences that would be displaced, traffic impacts in the immediate future and in 20 years, and the potential number of animals taken. To answer these questions and develop a preferred alternative for the EIS, the County formed a Project Management Team (PMT) made up of Consultants, the County, the City of Portland, Oregon DOT, and Metro (the Portland area regional planning organization), which then formed various advisory committees. These committees included one populated by regional political leaders (Policy Advisory Group or PAG), one by heads of impacted government agencies (Senior Agency Staff or SAS), and one by interested citizens, business leaders, and various advocacy groups (Community Task Force or CTF). Figure 1 is a diagram of how the public process was set up to take public input, distill it into project alternatives, and develop a recommendation on how to move forward. The PMT and PAG developed a six-step decision making

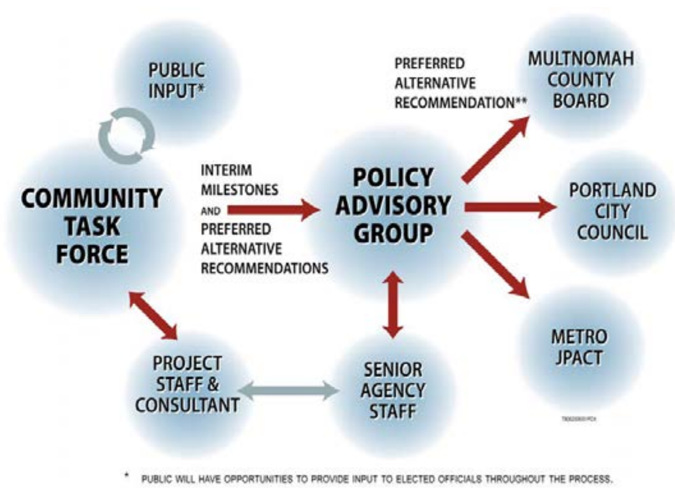

Figure 1: Sellwood Bridge Project Public Involvement Process (Jeanne Lawson Associates, October 2008)

process. Each step included a round of community outreach (including stakeholder briefings, newsletters, public open house, featured sections on the projects interactive website, and online surveys) that resulted in a recommendation that was then vetted with the SAS and County board (Jeanne Lawson Associates, October 2008).

Purpose and Need

The first step of the decision making process was to identify the project's goals in order to measure success against them. After the first round of public outreach, the following goals were determined and used to develop the statement of Purpose and Need for the project (Jeanne Lawson Associates, October 2008):

- Making the bridge and approaches safer for all users
- Providing better bicycle/pedestrian access and connections to area trails
- Maintaining neighborhood livability (two-lane bridge, follow the *Tacoma Main Street Plan* (City of Portland Offcice of Transportation, 2001) and reducing cut-through traffic impacts
- Restoring bus transit on the bridge and/or accommodating future light rail/streetcar
- Building with the future in mind; ensuring adequate bridge capacity for all users

Purpose:
The purpose of the proposed action is to rehabilitate or replace the Sellwood Bridge within its existing east-west corridor to provide a structurally safe bridge and connections that accommodate multimodal mobility needs.

Project Needs:

- Provide structural capacity to accommodate safely various vehicle types, including transit vehicles, trucks, and emergency vehicles; and to withstand moderate seismic events;
- Provide a geometrically functional and safe roadway design;
- Provide for existing and future travel demands between origins and destinations served by the Sellwood Bridge;
- Provide for connectivity, reliability, and operations of existing and future public transit;
- Provide for improved freight mobility to and across the bridge; and
- Provide for improved pedestrian and bicycle connectivity, mobility and safety to and across the river in the corridor.

Alternative Selection

Based on the project Purpose and Need statement, five alternatives were eventually chosen for consideration; A – Rehab existing bridge with new bike/pedestrian bridge to the north, B – Rehab existing bridge with temporary detour bridge to the north, C – Replace bridge in existing alignment, D – Replace bridge widened to the south, E – Replace bridge relocated to the north with transit lanes. In addition, five west side interchange configurations were considered, including two signalized interchanges, two roundabout interchanges, and one trumpet interchange. Four east side connections (two signalized and two un-signalized) and four bridge cross-sections were also considered (Appendix 1). The matrix used to make the comparison between the Purpose and Need statement and the various alternatives included the following considerations; impact to traffic, impact to bikes and pedestrians, business relocations, residential relocations, total closure time, and cost.

After public and agency comments were collected and evaluated, the project's CTF identified Alternative D as the best choice to advance to the PAG for recommendation as the preferred alternative. The PAG made that recommendation and Multnomah County, Clackamas County, Metro, City of Portland, and ODOT adopted Alternative D as the preferred alternative in February and March of 2009 (FHWA and ODOT, 2010).

Alternative D Refined

Once an alternative had been selected, it was refined to conform to public and agency comments received on the DEIS and to minimize environmental impacts. Those changes included:

- Modifying the OR 43 footprint to reduce park impacts
- Adjusting pedestrian and bicyclist facilities to improve access, improve safety, and reduce park and natural resource impacts
- Relocating a private driveway to improve safety and reduce park impacts
- Reducing the width of the bridge deck by one lane on the west end
- Refining an access roadway footprint to accommodate a future streetcar line, as requested by the City of Portland

With those refinements completed, Alternative D Refined consisted of a replacement bridge on the existing alignment, widened to the south. The basic bridge cross-section was set at 64 feet wide and consisted of:

- Two 12-foot-wide travel lanes
- Two 6.5-foot-wide shoulders/bicycle lanes
- Two 12-foot-wide shared-use sidewalks
- 1.5-foot-wide railings on each side

In addition, the ends of the bridge were altered to improve the connections to Tacoma Street and Oregon Hwy 43.

- West end – the bridge cross section was changed to include two 12-foot wide travel lanes eastbound to facilitate movements from the west-side interchange, which merges into one travel lane eastbound. Likewise, one travel lane westbound on the bridge widens to two 12 foot wide travel lanes approaching the west-side interchange to separate northbound from through and southbound movements and to provide for queuing.
- East end – on the east end, the bridge cross section was changed to one travel lane in each direction, with an eastbound left-turn lane at the intersection of SE 6th Avenue with SE Tacoma Street. East of SE 6th Avenue, SE Tacoma Street remained one travel lane in each direction with a center-turn lane.

In addition, Alternate D Refined was envisioned to be constructed in stages to maintain traffic on the bridge during as much of the project as was feasible. The stages would consist of the south half of the bridge being constructed to the south of the existing bridge, moving traffic over, and the old bridge being removed to make way for construction of the north half of the new bridge.

The west-side configuration became a signalized intersection on the upper level of the interchange to control traffic entering and exiting the Sellwood Bridge and River View Cemetery, with OR 43 passing under this intersection on the lower level. Ramps from the signalized intersection provide access to and from OR 43. A new roadway originating on the west side of the signalized intersection provides access to River View Cemetery and its Superintendent's House. It was also planned that a new roadway would pass under OR 43 (referred to by the project team as the "horseshoe ramp") south of the signalized intersection to provide access to Powers Marine Park and the Staff Jennings property.

On the east side of the bridge, the SE Tacoma Street intersection with SE 6th Avenue is to have a bicyclist/pedestrian-activated signal. The signal allows bicyclists and pedestrians to safely cross SE Tacoma Street to access the Springwater Corridor Trail (via SE Spokane Street) and the City of Portland-designated bicycle boulevards on SE Spokane and SE Umatilla Streets (FHWA and ODOT, 2010).

Adding a Detour

The Sellwood Project was planned to be delivered as a Construction Manager/General Contractor (CM/GC) project in order to gain efficiency of design and construction by having a collaborative team effort between the County, Construction Contractor, and Design Consultant during the final design phase of the project. After the FEIS was completed and the project had secured a Record of Decision (ROD), the County began preparing for the final design and construction phase of the project by hiring a design team and construction contractor (CM/GC) to develop the final plans. Once the CM/GC and design consultant were on board and design was under way, the County and its team began discussions of how to approach the design and construction of the new bridge.

During the EIS process, a detour bridge had been considered as one of the options, but was rejected for its cost and impact to business, residents, and parklands. However, in discussions with the design consultant the idea was put forth to make a detour from the existing bridge by constructing temporary piers and pushing it north, out of the way of the new bridge. The CM/GC was consulted and agreed that plan was constructible and would save project budget and schedule. The County then approached local residents, Parks, ODOT, and FHWA to discuss the possibility of a detour bridge and what impact it would have on each stakeholder group. The County completed several studies to determine potential effects of the temporary detour on air quality, noise, and increase in vibrations to neighboring residents. As a result of those studies, the proposed location of the tie-in of the detour structure to the existing east approach was moved west towards the river, avoiding the potential impacts described in the reports (Jeff Buckland, 2011).

The changes to the project ROD that were proposed to accommodate the detour structure included:

1) Bridge replacement using single-stage rather than two-stage construction.
2) Temporary displacement of residence R2 at Riverpark Condominiums.
3) Temporary closure to pedestrian traffic of the private connection to the Willamette Greenway Trail (East Bank), south of SE Spokane Street

Bridge Replacement Using Single Stage Construction

One of the biggest benefits of the newly proposed detour was the opportunity to replace the bridge by single-stage rather than two-stage construction. Single-stage construction required a temporary traffic detour around the bridge replacement site, similar to the detour described in FEIS Alternative B, but following an alignment close against the existing bridge alignment. The detour was proposed to be accomplished by constructing the Phase 1 temporary detour bridge (shoofly) where a temporary work platform was originally proposed, immediately downstream (north) of the existing bridge. The revised construction approach required pushing existing four span continuous truss bridge approximately 40 feet north onto temporary bents.

Moving the Sellwood Bridge

The CM/GC model of contracting facilitates fluidity in the development of the project plans, allowing for continuous refining of specific elements of the project to take advantage of a Contractor's skill set, equipment, staging needs, and opportunities for value engineering. As discussed above, the choice to pursue a detour bridge and single-stage construction originated as a value engineering idea from discussions with the project design team and the CM/GC. The specific reasons for selecting the detour bridge option as proposed by the project team were:

1) Save project up to 12 months of schedule.
2) Reduce cost of project by $5M to $10M.
3) Increase separation between workers and driving public.
4) Eliminate redundant arch ribs (reduces from 4 to 2), improve appearance (Figure 2).
5) Lessen environmental impact by reducing number of work bridges required, shorten construction time, and reduce long-term in-water and riparian impacts.

Detour Bridge Design

Figure 2: 1-stage vs. 2-stage Construction

With the decision made to move forward with reusing the existing truss as a detour bridge, T.Y. Lin International set about developing plans for the detour temporary piers with steel "translation beams" (Figure 3) spanning from the existing to the new piers. Their plan set assumed high-strength rollers would be used atop the beams to move the bridge north, but left the responsibility of designing the translation system up to the general contractor. Permission was secured to install pile supports for the detour structure bents December 2011 to January 2012. Winter in-water work was required because there was insufficient time to build temporary and work bridge foundations during the July-October 2012 window due to a limit of five piles driven per day or 1000 blows from a pile driving hammer that was a condition of the projects environmental permits. With temporary piles installed during thewinter, the temporary bents were completed by working above the water from February to May 2012 (Jeff Buckland, 2011). During this time, and for the next 11 months, the County, the designers, and the CM/GC and their specialty subcontractors engaged in an extensive and detailed planning process to ensure that the existing 80-year-old four-span continuous truss could be moved safely, such that at the end of the process the County would be able to state with defensible confidence that the detour structure was safe for use by the general public.

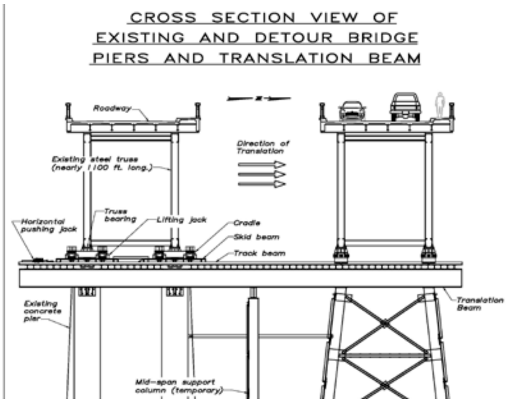

Figure 3: Detour Translation Plan Cross Section

Development of Translation Design Criteria

The County's consultant, T.Y. Lin had designed new piers, translation beams to connect the old piers to the new piers, and an alignment for the detour structure. The CM/GC was then responsible for designing the east approach (partially new, partially existing east approach) and west approach (all new), their connections to the detour structure, and for developing the method and procedures to move the Sellwood Bridge truss from its original location to the new piers in the new alignment. In order to give the CM/GC clear guidelines for the translation of the Sellwood truss that would give the County confidence that the truss had not been damaged during the move, the County commissioned the Sellwood Bridge Translation Tolerance Study by David Evans and Associates. The purpose of the study was to determine the allowable stress that could be imparted to the various truss members without causing damage, and to relate those stresses to relative deflections, pier to pier, that could be easily measured during the move.

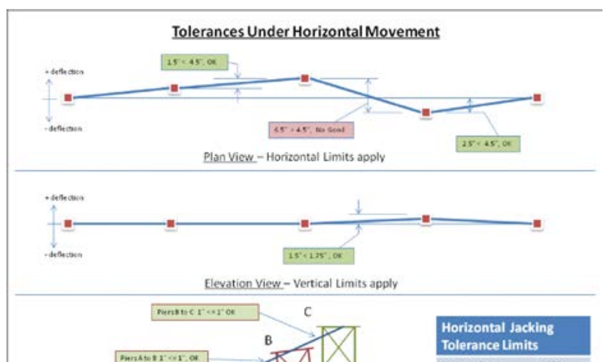

Figure 5: Translation Tolerances - Horizontal Movement (David Evans and Associates, 2012)

The study used a 3-dimensional model and a variety of assumptions that were made to add conservatism to the allowable stress. The basic premise was to set "a primary tolerance limit of approximately 66% of the calculated limit to be used in the primary direction of movement. Any concurrent distortions in the two directions other than the primary direction have a tolerance of 33% of the calculated limit." (David Evans and Associates, 2012) This resulted in "different concurrent tolerances sets for when the bridge is being jacked horizontally or vertically. In all cases the tolerances are for the relative displacements between the piers from their undisplaced position." (David Evans and Associates, 2012). The study was completed and delivered to the County in April 2012. The results of the study are shown graphically in Figure 4 and Figure 5.

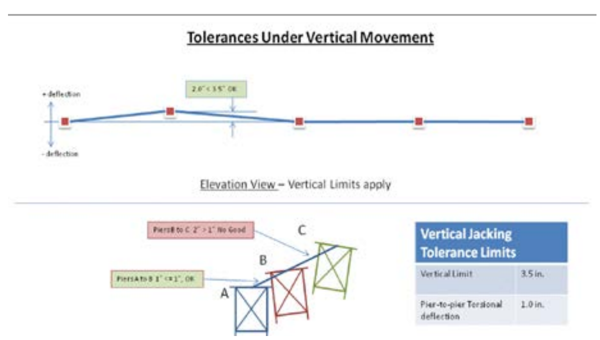

Figure 4: Translation Tolerances - Vertical Movement (David Evans and Associates, 2012)

Translation

Once the constraints had been developed for the bridge translation, the Contractor and County teams began the process of developing a final plan to move the bridge. The plan included the following elements:

1) Translation Path development
2) Cradle Design
3) Translation Equipment
4) Translation Procedure
5) Translation Monitoring Plan

Translation Path Development

One of the most challenging aspects of moving the Sellwood Bridge was that the new alignment was not parallel to the existing alignment. The bridge needed to be moved further at one end than the other because the west end of the existing bridge had to be moved far enough north to be out of the way of the west end of the new bridge. However, condominium buildings on the east end of the existing bridge limited how far to the north the east approach to the detour structure could be constructed, so the detour was only moved far enough north to construct half of the east approach of the new bridge while the detour would be in use. This resulted in the existing bridge being moved to a detour bridge alignment that was at an angle to the existing bridge centerline, with the west end of the bridge needing to move 66 feet, and the east end of the bridge needing to move 33 feet. To move the Sellwood bridge to this new alignment, it had to be rotated along an arc with a 2196-foot radius at the western-most bent, whose center of rotation was 20 feet to the east of the centerline of the existing bridge at the east end. This meant that the tangent line to the arc was not parallel to the line between the existing bridge bearings, and that the path of the bridge would be started at an angle relative to a line drawn between the bearings at each pier, which presented a number of engineering challenges to the team developing the moving plan.

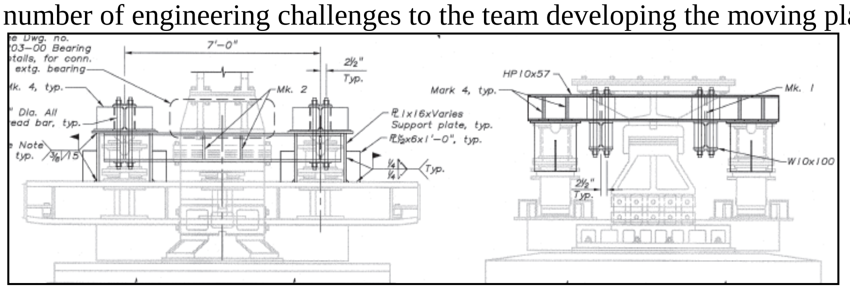

Figure 7: Cradle Elevations

Cradle Design

One of the key consequences of the translation needing to start at an angle to the centerline of the north and south bearings on each of the pier caps was that the cradles that would transfer the force of the vertical jacks into the truss needed to be built at an angle to the bearing seat. However, the cradles also needed to support the truss as close to the bearing seat as possible so that the truss did not need to be reinforced to take the bearing load in the individual lower chords (Figure 6). This was complicated by the fact that the bearing castings that were bolted to the truss were triangular shaped with stiffeners, so the cradles also needed a bearing interface to fit between the flat surface of the beams that the jacks pushed up against and the sloped surface of the bearing castings (Figure 7). Since each casting was different, the cradle bearings had to be cut specifically for the casting it was interfacing with and then had to be filled with grout to insure full contact on the bearing surface so that point loads were eliminated.

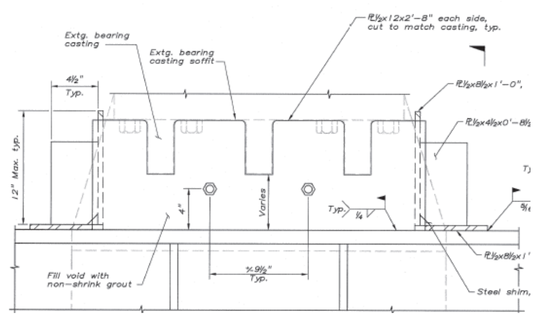

Figure 6: Cradle Bearing

Translation Equipment

The translation system consisted of tracks, skid beams, vertical jacks, horizontal jacks, vertical jack hydraulic power pack, and a custom-made, proportionally controlled, hydraulic system for the horizontal jacks. The track system is shown in figure 8. The protrusions on the outside of the skid beams are “dogs” for the horizontal jacks to lock into during forward thrusting, and the small pads sitting inside the tracks are Teflon pads whose purpose was to decrease skidding resistance. The skid beams were constructed from large wide-flange beams with metal cylinders welded to the top to hold the vertical jacks, and stainless steel cladding on the bottoms to decrease sliding friction (SSJV, 2012). At each pier, four skid beams were required, one set of two under each bearing. Because horizontal jacking only took place at the south skid beams, the north and south skid beams were tied together by heavy steel angles welded between the SE and NE skid beams, and the SW and NW skid beams (SSJV, 2012).

Figure 8: Track System

The vertical jacking system was designed to lift different loads depending on the pier that it was applied to. The weight of the truss at each of the three central piers was about 1.8M pounds (SSJV, 2012). For each central pier two 150 metric ton (MT) were used per skid beam (8 per central pier) for a total available jacking capacity of 2.645M pounds (SSJV, 2012). The weight of the truss at the end piers was 0.672M pounds (SSJV, 2012). For each end pier one 150 MT jack was used per skid beam (4 per end pier) for a total available jacking capacity of 1.323M pounds (SSJV, 2012). The jacks at the central pier had a 6-inch stroke, while the jacks for all of the other piers had a 14-inch stroke (SSJV, 2012). In addition, provisions were made at each jack location to insert two 55 MT jacks if one of the 150 MT jacks failed during jacking. The 55 MT jacks would allow the bridge to be lowered back onto its bearing so that the failed jack could be removed and replaced in kind (SSJV, 2012). The horizontal jacking system used the same push/pull jacks at every pier, which were attached to the southern skid beams. Each horizontal jack had a capacity of 75 MT and was set in a frame with a set of tabs protruding from the back that latched into the track “dogs” (SSJV, 2012).

Translation Procedure

Before the bridge could be translated, the first thing that needed to be done was the bridge had to be lifted off of its bearing seats and supported on the skid beams. Each pier was jacked separately by moving a hydraulic power pack along the roadway deck until it was situated above the pier to be jacked. Hydraulic lines were then run from the hydraulic power back down to the four or eight vertical jacks, depending on which pier was being worked on. The hydraulic jacking power pack had four separate pumps feeding four separate outlets, each set to the same flow rate so that all of the jacks on each skid beam and on each pier would lift together (SSJV, 2012). Once jacking was completed on a given pier, load carrying plates were inserted beside each vertical jack and welded in place. The vertical jacks were then backed off to 25% of full load. Next, the horizontal jacking system was used to move the bridge. The horizontal jacks were controlled by a single hydraulic power pack located on the central river pier, which was connected through a proportional control system to all ten horizontal push/pull jacks by hydraulic lines of equal length. The proportional control system was set up to move the bridge a distance at each pier relative to the east pier (17), such that each time the bridge at pier 17 was moved 1 foot, the bridge at pier 18 moved

1.22 feet, the bridge at pier 19 moved 1.49 feet, the bridge at pier 20 moved 1.765 feet, and the bridge at pier 21 moved 1.99 feet (SSJV, 2012). This meant all bridge sliding movement could be controlled from a central location at pier 19 for larger movements, while still allowing for adjustments at individual piers if needed. The contractor had developed an offset table that showed how tenths of a foot movement at pier 17 translated to movement at each of the other piers based on the proportional control settings. Each time the bridge was moved, the system was activated for approximately 10 seconds. Workers at each pier would then measure how far the bridge moved at their location and report back to the operator who would compare the movement to the offset table. If the movement was behind at any pier the horizontal jacks at that pier were operated independently to move it to the proper position before another 10 second movement of the entire bridge was performed. The horizontal jacks had a stroke of 48 inches. Each time the jack at one of the piers stroked out, its tabs were disengaged from the track "dogs," the jack was retracted, and the tabs were engaged with the next set of track "dogs." Once the bridge was in its final position, the vertical jacking system was used to lower the bridge onto its new bearing seats.

Monitoring Plan

The purpose of the monitoring plan was to ensure that the Contractor stayed within the tolerances developed in the Sellwood Bridge Translation Tolerance Study. The Monitoring plan had two components, monitoring to be done by the Contractor during the move and monitoring to be done by the County during the move as a quality assurance check on the Contractors work. The contractor used GPS units mounted on the north and south roadway crash barriers over each pier to measure vertical movement, laser beams projected through holes in targets ate each pier from the center span of the bridge east and west along the bridge catwalk, and ten prisms mounted on the north face of the truss at each pier line, top and bottom of truss (SSJV, 2012). The County supplemented these measuring techniques with strain gauges mounted on key vertical and horizontal truss elements at each pier that were remotely monitored and recorded in real time during the bridge translation, and smart levels used to measure the angle of inclination of the vertical truss members.

Of all of the measurements taken during the bridge slide, the most useful were the smart level measurements and the strain measurements. Using the strain measurements, the County was able to create a record of the movement in strain directly measured in the truss as the bridge was moved. After each 10 second push of the bridge the strain gage readings were quality checked by County staff that entered actual measured bridge displacements into a spreadsheet that then calculated an estimate of strain at the points the strain gages were taking readings. The smart levels, while being perhaps the lowest tech measurement taken during the bridge slide, provided very actuate measurements of the tilt of the bridge in the north-south direction as the bridge moved. The Contractor's GPS units were not effective because they did not have the required sensitivity for the movements of the bridge in the vertical plane. The Contractors survey was only effective part of the day due to heavy fog in the am and the onset of darkness before the move was completed. Finally, the Contractors laser system was too sensitive to movement of the bridge catwalk, which was traversed hundreds of times during the day by workers and dignitaries moving back and forth over the bridge, to provide an accurate and reliable measurement of pier to pier displacement.

Short-Term Planning

Months leading up to the translation, the project team began a series of weekly meetings to discuss what would be necessary leading up to the move as well as all monitoring that would be required during the move. Because of the project location (the bridge connects city streets to a state highway), the planning involved the City of Portland, ODOT, USCG, and Multnomah County, including the Sheriff's Office. To ensure that all planning was properly enacted, a QA Checklist was created, listing all pre-, during, and post-move tasks and their respective QC and QA leads. These tasks included verifying all required submittals had been reviewed, plan changes were made and distributed, monitoring systems were installed and ready to record, and all inspections were complete. The purpose of the checklist was to provide the County traceable documentation that the detour bridge had gone through all QC/QA checks and was safe to put traffic on.

Today

Moving the existing truss out of the new bridge alignment allowed Multnomah County to keep traffic moving across the river during construction, provided a more structurally sound crossing, and resulted in the ability to build the new span in one phase, saving the project up to $10 million and a year in construction time. Since the move, landslide mitigation work has been completed, foundations for the new spans have been built, arch steel has been fabricated, and various other portions of the project have marched closer to completion.

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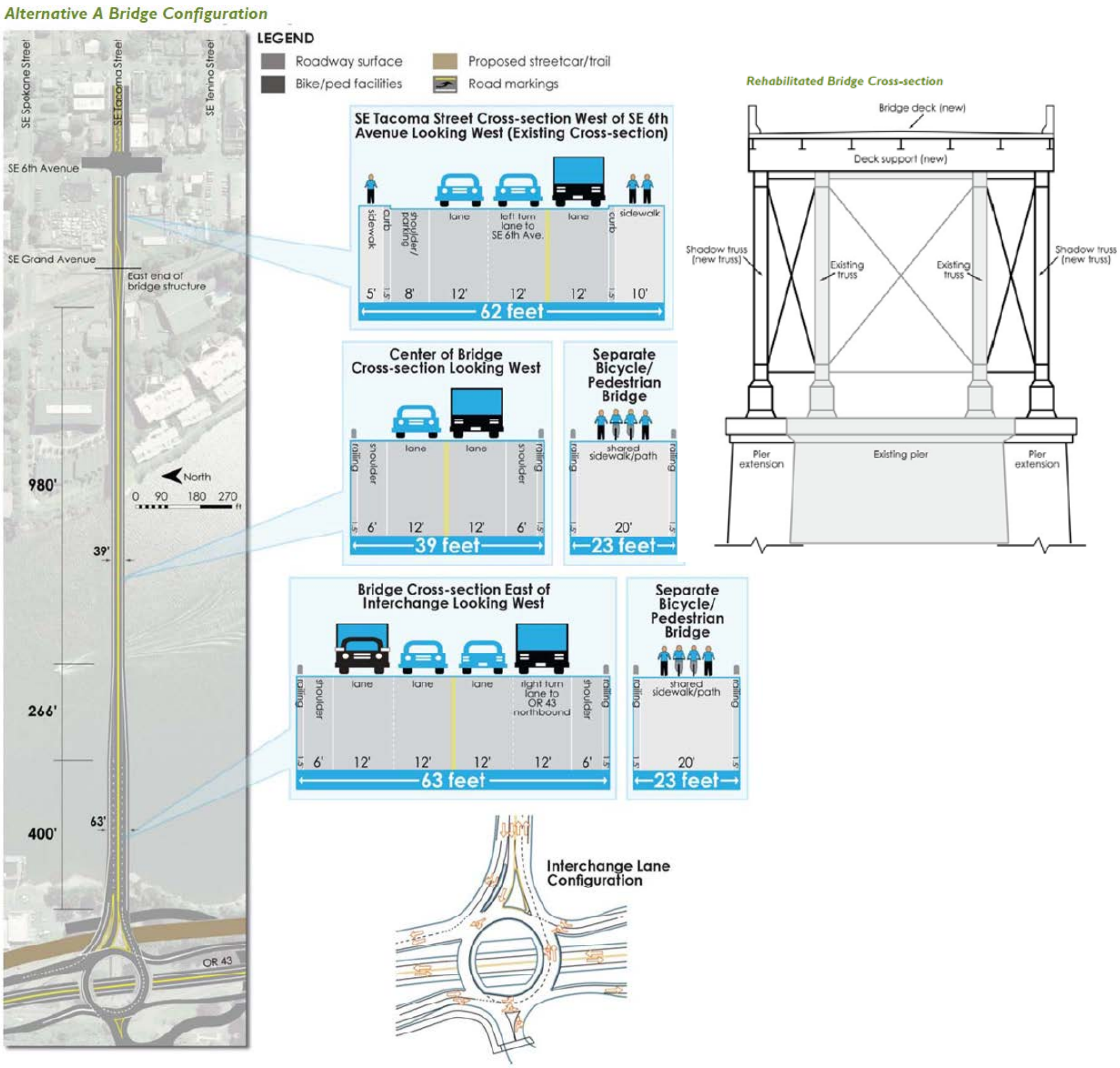
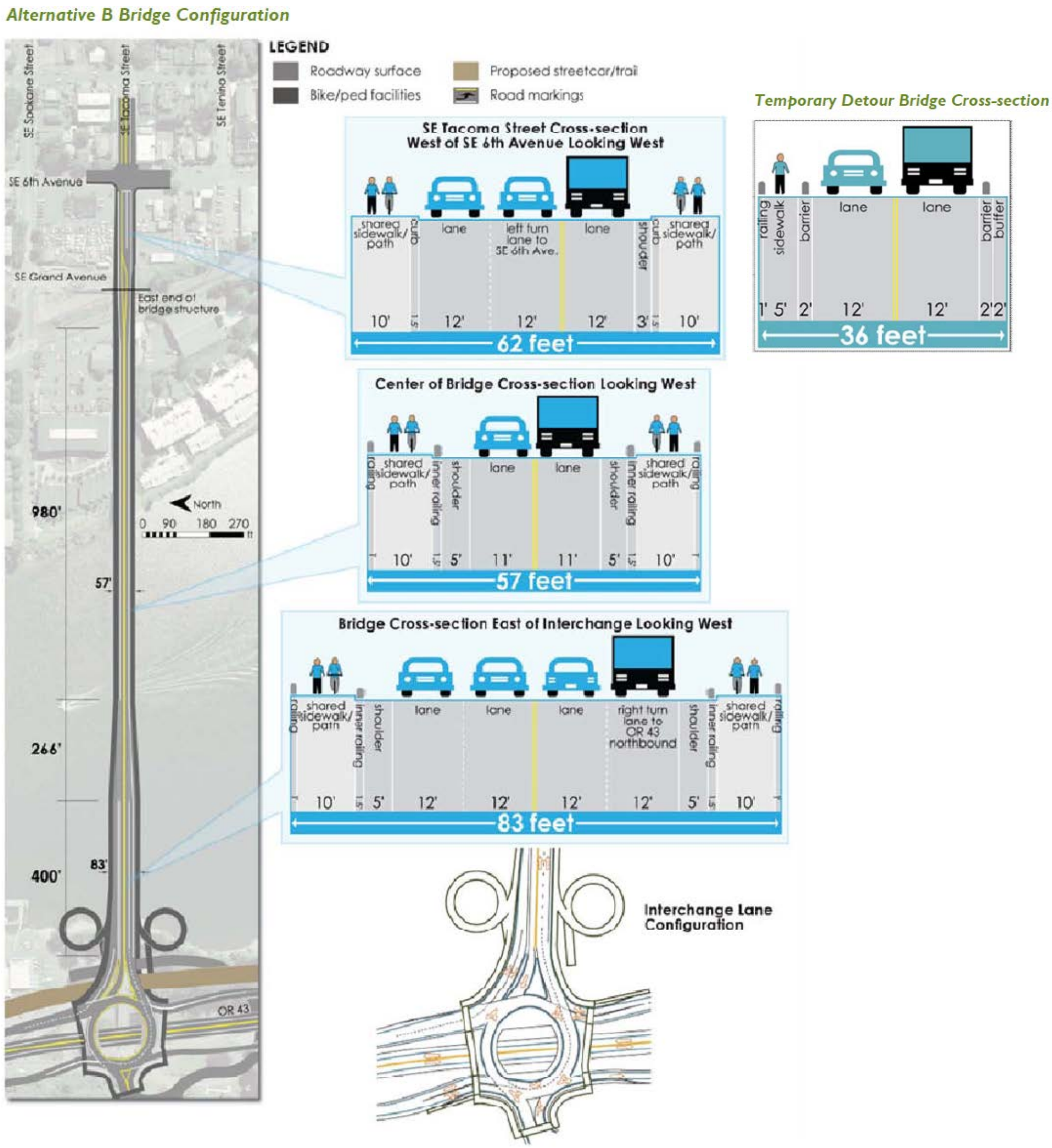
APPENDIX 1

Alternate cross sections for Public Considerations during the FEIS
(FHWA, ODOT, and Multnomah County, 2010)

Alternative A

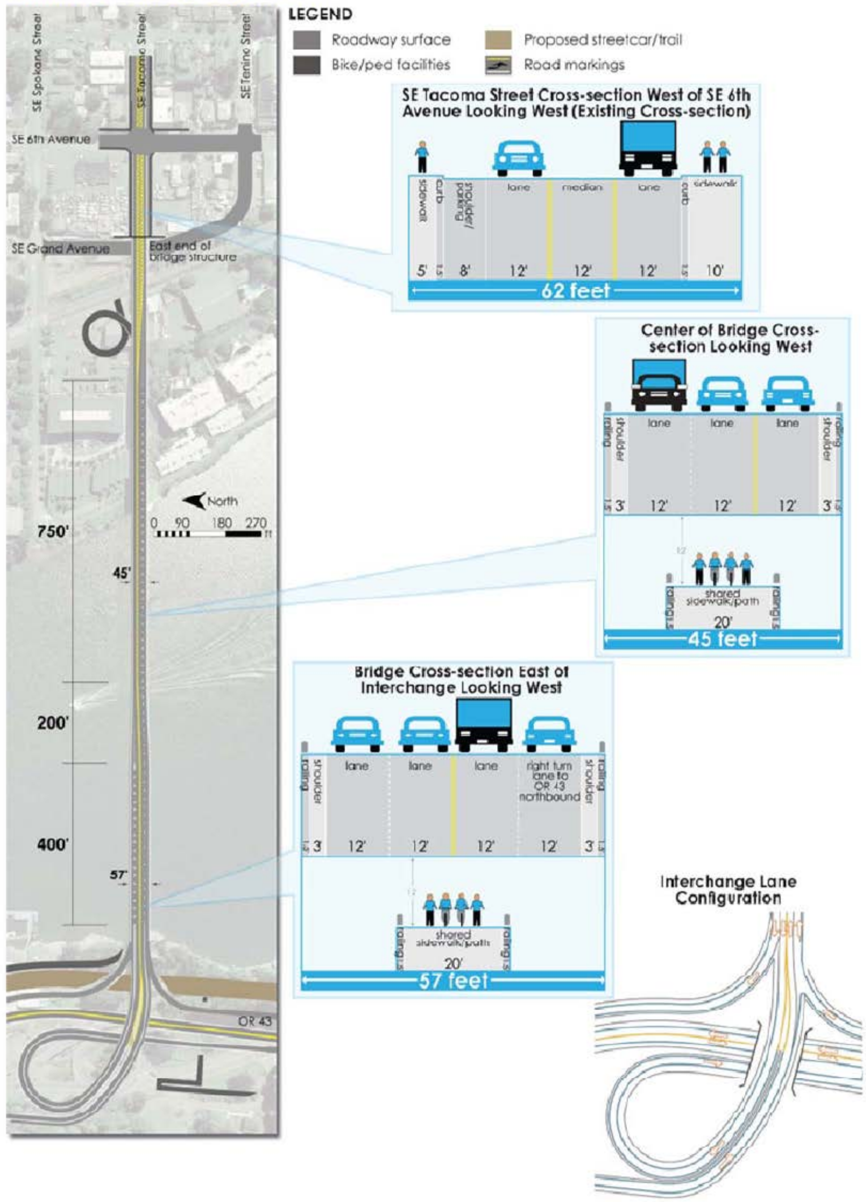


Alternative B



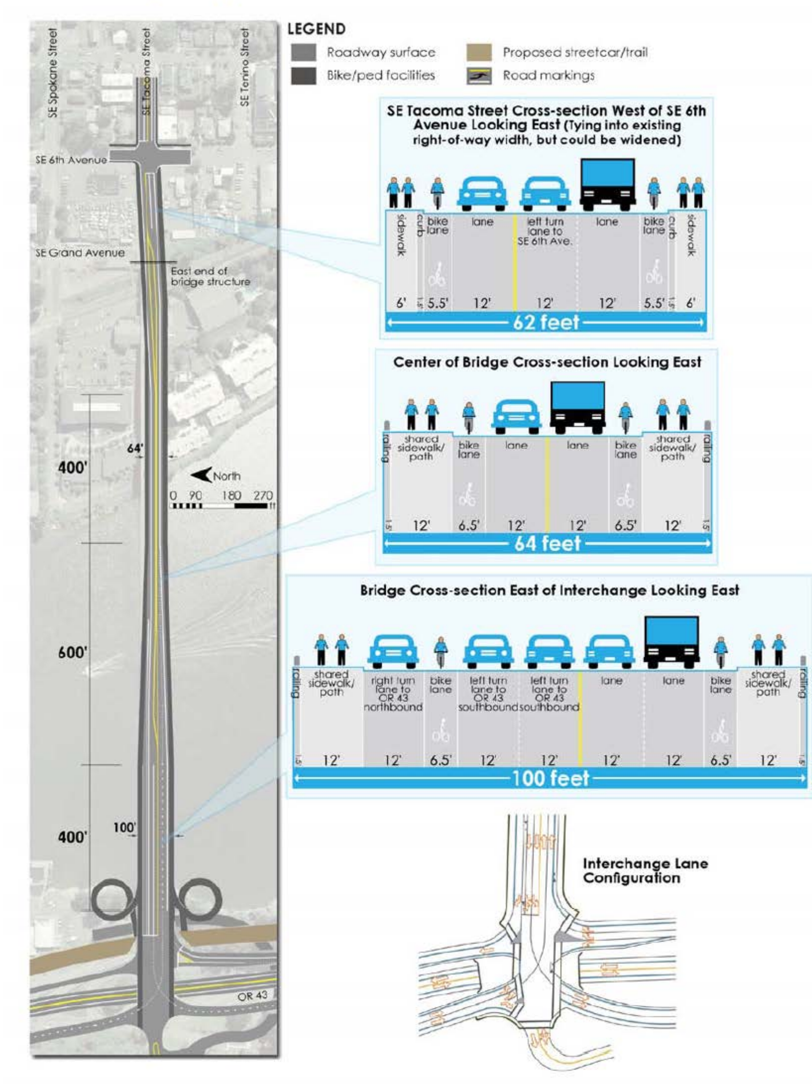
Alternative C

Alternative C Bridge Configuration



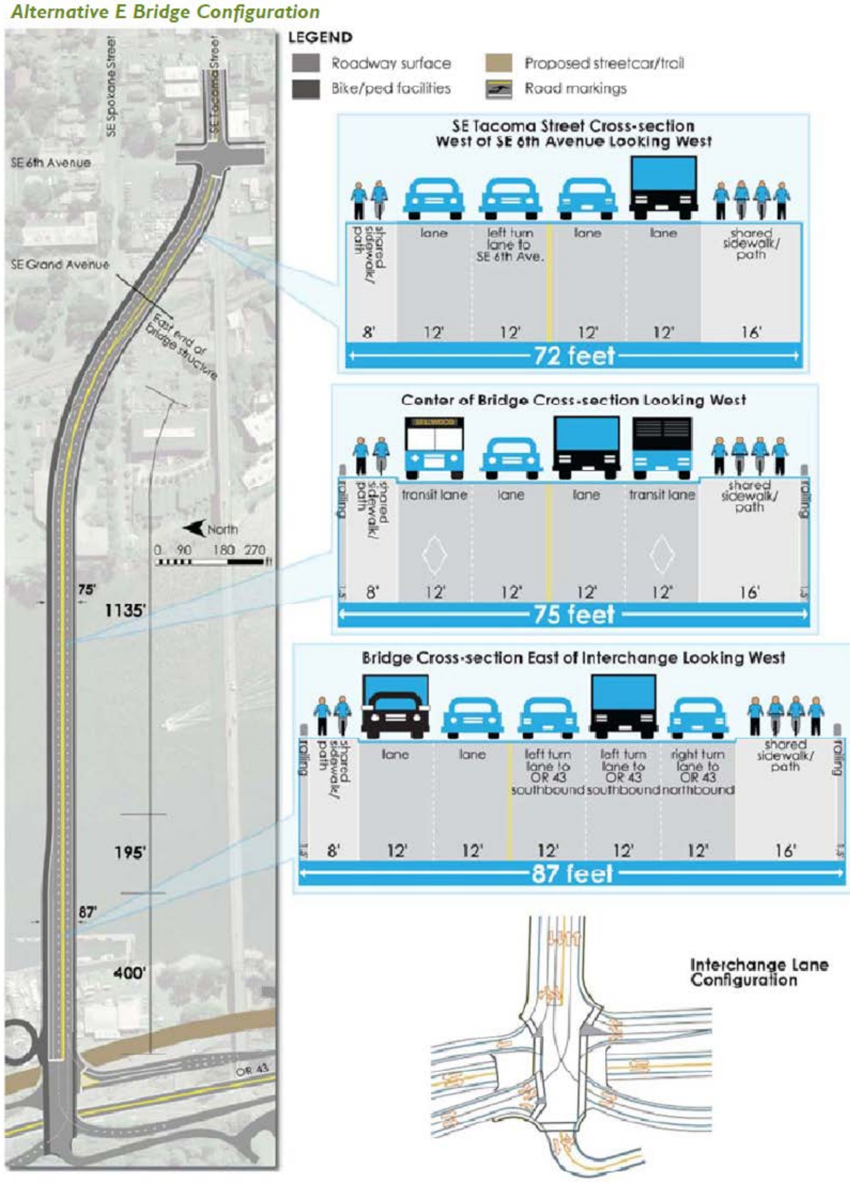
Alternative D

Alternative D Bridge Configuration



Alternative E

Alternative E Bridge Configuration



HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15-18, 2014

Transfer Bridges of New York Harbor
Past, Present, and Future

Giancarlo Schiano P.E.
Kevin Ciampi P.E.
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HDR Engineering Inc.

NEW ORLEANS FRENCH QUARTER MARRIOT HOTEL
NEW ORLEANS, LOUISIANA

Abstract

New York Harbor is one of the largest natural harbors in the world and the Port of New York and New Jersey currently is the busiest port on the east coast of the United States handling freight in excess of $200 billion annually. The transferring of goods in the region is aided by a well-developed system of marine, rail, and air cargo within the Port. One of the most intriguing, yet largely unknown, forms of goods transfer across the harbor was developed in the 1860's and thrived until the 1950's. In the late 1800's railroads began loading railcars onto barges fitted with track via moving bridges. These barges became known as carfloats and the moving bridges became know as transfer bridges. The carfloats were transported across the harbor with an army of tugboats operated by the railroads. The waterfront of New York and New Jersey during this period was congested with at least 80 transfer bridges, hundreds of piers and hundreds of car floats joining passenger ferries and cargo ships in a cluttered intricate dance across the harbor. At its peak, freight traffic across the harbor was controlled by dozens of railroads however has declined to a single active transfer bridge operation in the harbor today.

This paper will explore the rise and decline of car floating in New York Harbor from the 1880's to the present day and touch on the various aspects of the different types of rail transfer bridges utilized to load and unload car floats.

The Origins & Purpose of Transfer Bridges

Origins of Transfer Bridges

The origins of rail transfer bridges date back to approximately 1838 in the Mid-Atlantic States prior to the American Civil War. The first rail transfer bridge operation was begun via a joint venture between the Camden & Amboy Railroad and the Philadelphia, Wilmington, & Baltimore Railroad in 1837 or 1838 for car ferries. The ferry route was approximately 1 mile long over the Susquehanna River between Havre de Grace, MD and Perryville, MD. In 1862, the first car floats were being used by the Union Army to transport railcars 60 miles from Alexandria, VA to Aquia Landing, VA on the Potomac River. These transfer bridges were built under Brigadier General Herman Haupt, former Chief Engineer of the Pennsylvania Railroad (PRR). Photo #1 shows the transfer bridges at Alexandria. Photo #2 shows a carfloat at Aquia Landing.

Photo 1

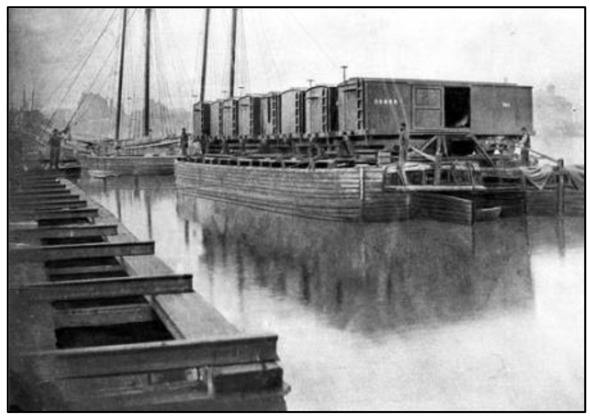

Photo 2

Transfer Bridge Introduction

A transfer bridge is an articulating bridge used to move rail cars on and off a barge or ferry, which are fitted with rail, directly to or from a rail yard on land. Transfer bridges are also referred to as “float bridges” and the terms are synonymous. The bridges are hinged on the land end while tidal variations and varying submergence of the float or ferry are accommodated through the raising or lowering of the free end of the bridge. The bridges have varied from single articulating bridges to double articulating bridges. The bridges have ranged in length from approximately 80 feet to 150 feet to maintain a rail grade suitable to load and unload rail cars safely without binding or decoupling. The grade for the bridges was usually kept to maximum 8% grade, although steeper grades were accommodated. The daily tidal variation in New York Harbor is approximately 6 feet but the variation between extreme tides is about 11 feet.

A single articulating transfer bridge is hinged at the land end, to allow rotation at an abutment or pier, and supported from either above or below at the free end. Bridges supported from below typically rest on a floating pontoon and were the first type introduced. Bridges supported from above are typically hung by wire rope, chains, or vertical screws from an overhead gantry. These bridges typically range in length from 80 feet to 100 feet to keep the maximum grade of the rail for operations around 8%.

A double articulating transfer bridge is typically hung at the free end and consists of two spans. The main span is called the bridge span and spans between the hinges on the land end to an overhead gantry at the free end. An apron span is a shorter, more flexible span that is hinged to the bridge span and partially supported for dead load at typically a second gantry. The bridge span is about three times longer than the apron span in this configuration. These bridges typically range in length from 100 feet to 150 feet to keep the maximum grade of the rail for operations around 8%.

Float Introduction

The two primary modes of transporting rail cars over water (other than a fixed or movable bridge) are car ferries and carfloats. A Car Ferry is self-propelled marine vessel fitted with track typically for longer voyages than the engineless carfloat. A Carfloat is a scow barge fitted with track that is transported by a tugboat. Carfloats have ranged from 100 feet to just under 400 feet in length. Tugboats can handle up to two carfloats at a time. Photo #3 shows the two types of carfloats used in New York Harbor.

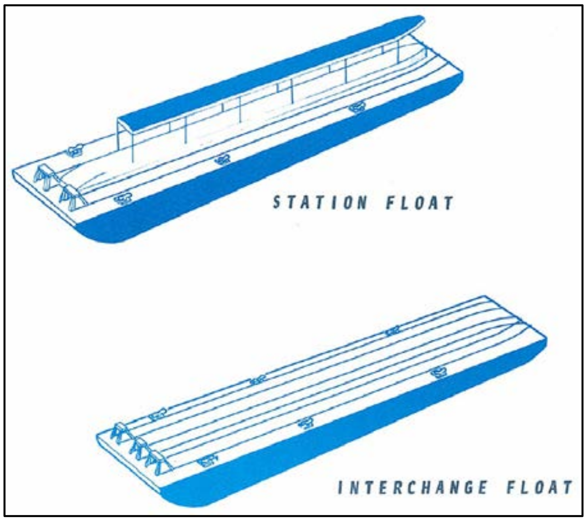

Photo 3

There are two types of carfloats, interchange floats and station floats. Interchange floats carry heavier and bulkier freight than station floats. This was primarily a function of the lack of efficiency of moving the heavier and bulkier cargo off station floats with hand-trucks and small machinery. Both methods of carfloating require a transfer bridge to load railcars to the carfloats. Nearly all carfloats used in New York Harbor were interchangeable between railroad facilities. The rail alignment and location of turnouts at the ends of the carfloats that met up with the transfer bridges and the size and location of the toggle pin receiver pockets was standardized for all carfloats regardless of railroad. Likewise, the location of the track, turnouts, and toggle pins on the transfer bridges were the same throughout the harbor. Photo #4 shows a tugboat with two interchange floats for the Harlem Transfer in 1959. Photo #5 shows three station floats for the Baltimore & Ohio Railroad at the Duane Street Freight Station in NY in the 1950's.

Photo 4

Photo 5

Interchange carfloating operations exchange loaded railcars from one terminal to another terminal. These floats typically are 300 feet long and have 3 to 4 tracks utilizing the entire deck with a capacity of 12-20 cars. A turnout was located at the bridge end of the float to allow the railcars to utilize the inner tracks. Interchange floats are also commonly referred to as transfer floats and are suitable for all types of railcars.

The only active interchange float operation in New York Harbor is operated by New York New Jersey Rail LLC (NYNJR), which was acquired by the Port Authority of New York & New Jersey (PANYNJ) in 2008. NYNJR floats approximately one mile between Greenville Yard in Jersey City to Bush Terminal in Brooklyn.

Station floating operations are performed such that a railcar is loaded onto a carfloat via a float bridge at a rail yard, floated to a pier typically not serviced by a rail system, where the cars are unloaded and/or loaded with the railcar never moving from the float. These floats have a center platform between the tracks at the same elevation of the railcar floors, allowing small machinery or hand-trucks on and off the float at the pier to unload/load the railcars, and handle up to 12 railcars per float. Station floats may also be unloaded/loaded directly from pier side allowing for simultaneous loading and unloading via the center platform and the pier. Station floats are also commonly referred to as pier floats and is not suited for all types of railcars. There are no active station float operations in New York Harbor currently although interchange floats have the ability to be unloaded similarly at a pier. Photo #6 shows a station float for the Baltimore & Ohio Railroad (B&O) being loaded via the central platform in the 1920's.

Photo 6

The Rise of Transfer Bridges in New York Harbor

Early Transfer Bridge in NY Harbor – The Purpose & Business Involved

In the mid 1800's there were dozens of railroads operating in New York Harbor. The first transfer bridges in New York harbor appeared in 1866. The harbor is encompassed by New Jersey to the west, and Manhattan, Brooklyn, Queens, the Bronx, and Staten Island. The railroad companies in the area ranged from the goliaths of the day with the New York Central (NYC) and the PRR to many smaller railroads operating only as exchange terminals. Railroad marine operations were born out of economic and geographic necessity in New York Harbor. Only one railroad, the NYC had access to Manhattan at the time and all other railroads imported and exported freight via lighters and carfloats to Manhattan.

The eastern shore of New Jersey from Weehawken to Bayonne was filled with railroad terminals, transfer facilities, freight piers, and passenger ferry facilities to transport both goods and passengers to New York City. Likewise, the shores of Manhattan, Brooklyn, and Queens were littered with similar facilities to receive the freight and passengers. The waterfront was packed with facilities and land was at a premium in the harbor. To accommodate the required land, wharf room extended into the harbor.

Figure #1 shows the actual waterfront mileage throughout the harbor and the wharf room that was built to accommodate the freight by 1906. Photo #7 shows the New Jersey Hudson River shoreline from the PRR's Exchange Place in Jersey City (to the left) to the Erie and Lackawanna Terminal in Hoboken (to the right) in 1962. Photo #8 shows an Erie and Lackawanna railroad terminal facility at Newport in Jersey City, NJ covered in snow in the 1950's. Photo #9 shows New York Harbor and lower Manhattan with a tremendous amount of marine traffic, making it difficult to navigate for carfloats for the typical 1-2 mile voyage in 1928. Photo #10 shows Manhattan with piers extending into the Hudson River nearly doubling the waterfront length. Photo #11 shows a tugboat transporting carfloats from New Jersey to Brooklyn.

Location	Water Front (miles)	Wharf Room (miles)	Increase
Manhattan	44	93	111%
Brooklyn	132	197	49%
Queens	116	132	14%
Richmond (Staten Island)	51	69	35%
Bronx	105	113	8%
NJ Shore - Amboy to Fort Lee	30	96	220%
Totals	478	700	46%

Figure 1

Photo 7

Photo 8

Photo 9

Photo 10

Photo 11

Before 1924, the only rail bridge to cross the Hudson was in Albany, roughly 150 miles from Manhattan. In 1924 the Castleton Bridge was built in Selkirk, NY shortening the trip by 20 miles. This became known as the Selkirk Hurdle. The NYC owned the only direct rail access into Manhattan at the Spuyten Duyvil swing bridge and even then the freight would need to be unloaded for transit to Queens, Brooklyn, and Long Island via lighters, floats, or trucks. This trip on land via the Selkirk Hurdle could take anywhere from 1-3 days, with a roundtrip about 300 miles. The likelihood of the NYC granting competing railroads access to their lines was slim. It wasn't until 1916, when the PRR completed construction of the Hell Gate Bridge that Queens, Brooklyn, and Long Island were hard linked to the freight rail infrastructure of America.

In addition to the long trip around the Selkirk Hurdle, freight rail was second priority to passenger rail in New York City. The scale of passenger transport was massive across the harbor in the early 1900's. The Holland Tunnel opened in 1927 which was the first vehicular crossing across the Hudson River from New Jersey. The PRR Hudson River rail tunnels first opened in 1908 but only supported commuter rail. In 1906, approximately 1,080 passenger trains arrived and departed daily from Jersey City alone and over 100,000,000 people crossed the Hudson River yearly via passenger ferry services operated by the railroads. In 1880 the New York surface and elevated rail lines carried 287,000,000 passengers. In 1890, that figure grew to over 603,000,000 and by 1902 the number of passengers increased to 1,200,000,000 yearly. The growth of freight transport over rail in the U.S. at the time was growing at a similar rate to the commuter traffic in New York. In 1890 there were 65 billion ton-miles of freight transported across the country. In 1897 the number of ton-miles increased to 93 billion and by 1904 it had ballooned to 173 billion ton-miles, an increase of 166% in 14 years. Figure #2 shows the number of passengers on New York City mass transit from 1880-1902. Figure #3 shows the rail freight ton-miles from 1890-1904.

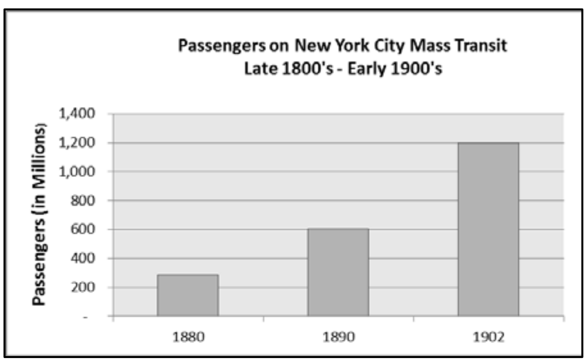

Figure 2

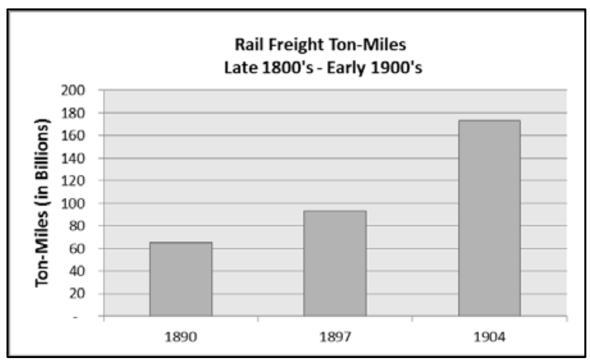

Figure 3

With the surface rail system essentially locked up to commuter trains and the increase of freight entering the harbor it became evident that freight must circumvent the commuter rail system to transport freight across the harbor effectively and the simplest solution was to float freight between New York City and New Jersey. The average cost in 1911 per revenue ton-mile across the country was 0.754 cents while the handling per car at a terminal was 25 cents. The foreign goods entering New York Harbor, at the time, accounted for nearly half of all foreign goods entering the entire United States. In 1911 terms, the railroads brought approximately 13,000,000 tons of freight to New York City annually. Of the 13,000,000 tons, 2,000,000 tons was lightered across the harbor while the remaining 11,000,000 tons was floated via interchange and station floats. At the same time approximately 4,500,000 tons were exported annually from New York City with 75% of that freight transported via station and interchange floats and the rest lightered across the harbor.

Photo #12 shows the eastbound station floating in the harbor. Photo #13 shows the eastbound interchange floating in the harbor. Photo #14 shows eastbound lighterage in the harbor. Photo #15 shows New York Harbor's rail terminals with the terminals in red along the waterfront.

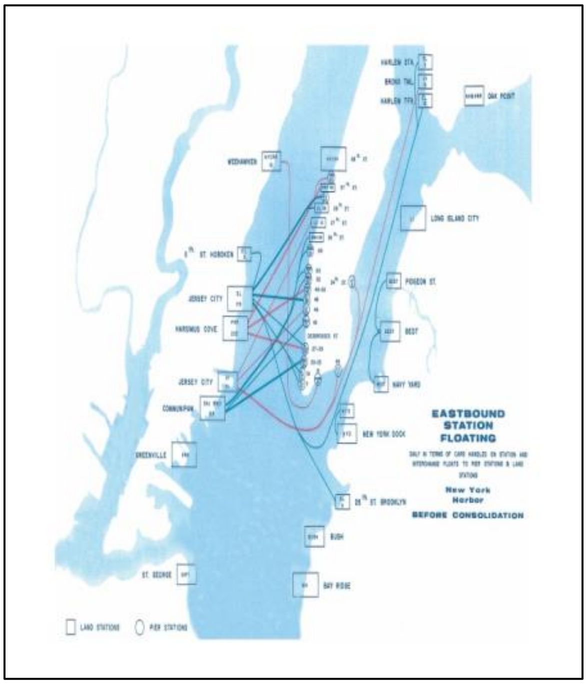

Photo 12

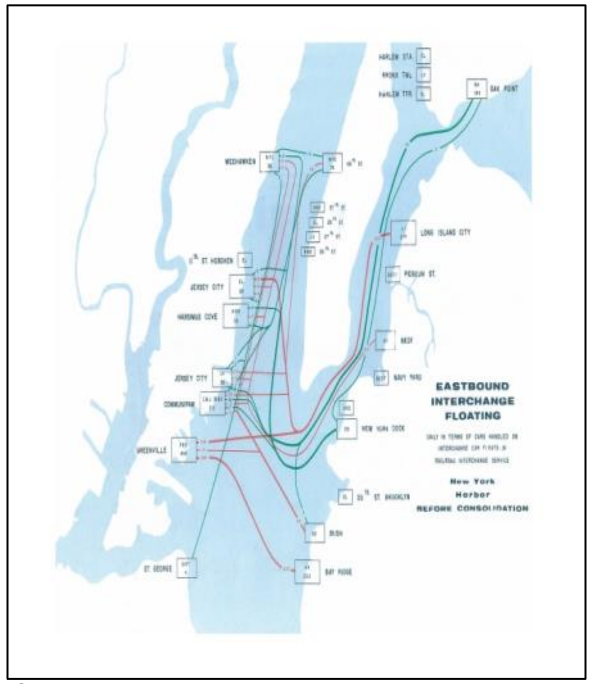

Photo 13

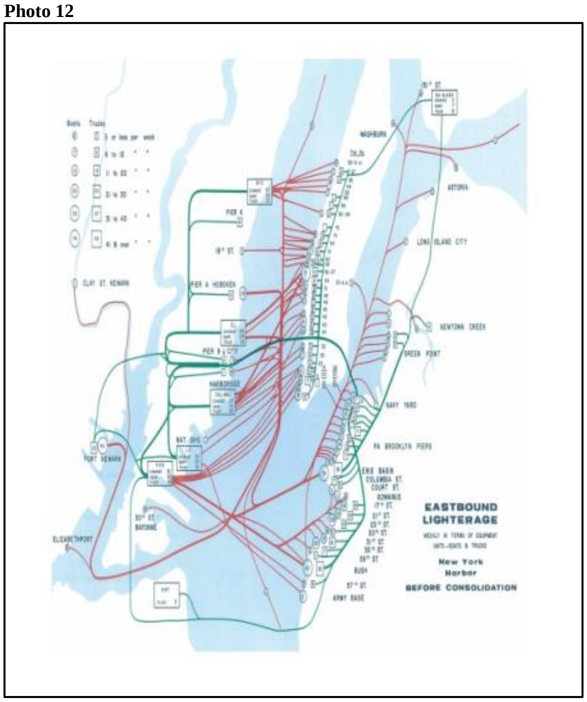

Photo 14

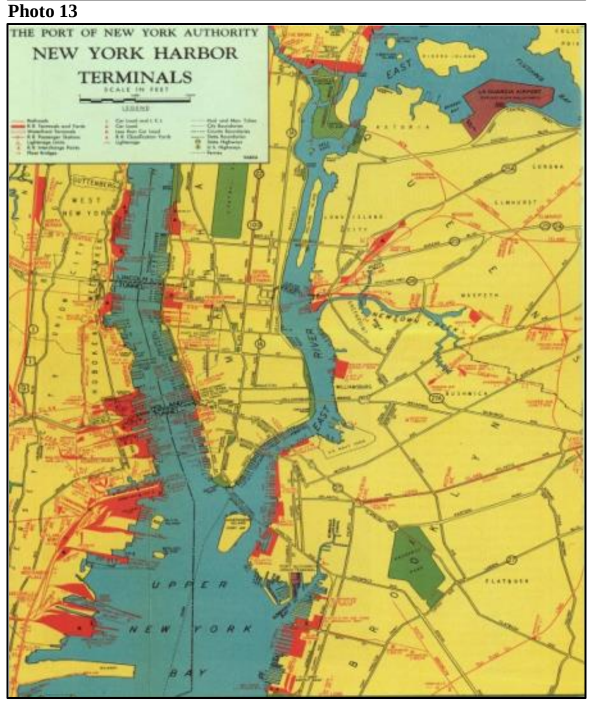

Photo 15

Even though there we only two rail lines entering New York directly in 1911, fourteen other railroads had terminals in New York City. Thus the remaining fourteen railroads found it necessary to move freight via water transportation. Many terminals were jointly owned and operated by various railroads. There were nearly one hundred rail and water transportation companies having freight stations in New York City with some having upwards of seven different stations in the City. With the combination of massive commuter transit demand, a large freight demand, quicker freight movement demands, limited waterfront and land for terminal facilities, rail and roadway congestion, the handling of freight through New York was described as the most complicated terminal problem in the world.

All of the early versions of the transfer bridge were single articulated bridges supported by pontoons until 1888 when the PRR built the first transfer bridge suspended by an overhead gantry system at Harsimus Cove in Jersey City. The railroads operating these facilities had massive fleets of equipment to support the commerce across the harbor. Amongst the hundreds of switcher locomotives to move the railcars, there were also upwards of 10,000 marine crafts used in the harbor. Of the 10,000 crafts, only about 1,000 had motive power, which included almost exclusively the tugboat fleets of the railroads. The remaining 9,000 crafts were station floats, interchange floats, barges, scows, and stick boats. The Bronx Terminal facility even required modified tugboats with shorter stacks and a lower wheelhouse to allow for passage under the Harlem River's movable bridges without requiring openings. The lack of available land at the terminal also led to some creative arrangement of facilities. One of the most unique may have been the Central Railroad of New Jersey's (CRRNJ) Bronx Terminal on the Harlem River near 3rd Avenue. This terminal was put into service in 1906 with the CRRNJ sharing the facility with the B&O, the Erie and the DLW. The CRRNJ took sole ownership of the facility in 1907 and finally ceased operation at the terminal in 1961. Photo #16 shows the Bronx Terminal in 1944 with a circular freight house. Photo #17 shows the track plan for the Bronx Terminal with the transfer bridge to the bottom left of center.

Photo 16

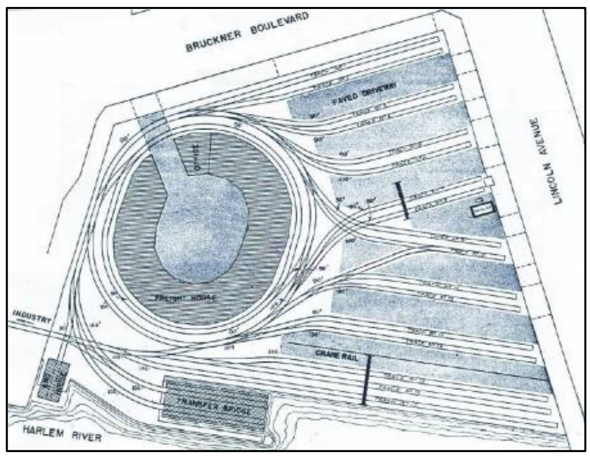

Photo 17

Photo #18 shows the East River with tugs transporting carfloats. Photo #19 shows a carfloat approaching the Brooklyn Bridge in addition to the station floats docked on the Manhattan side of the Bridge. In the 1920's, there were approximately 6,000-7,000 railcars transferred daily via floating operations. Brooklyn, Queens, and Long Island imported and exported all its freight via lighters and carfloats. Cargo transported via carfloats helped New York City expand in the early 1900's at an exponential rate.

Photo 18

Photo 19

Types of Transfer Bridges

The types of transfer bridges that will be discussed are the Pontoon Type, the Separate Apron type, the Mallery Type (swivel head block), and the French Type (contained apron). The Pontoon Type is typically single articulating transfer bridge supported from below. The Separate Apron Type, the Mallery Type, and the French Type are supported from above from gantry systems. Photo #20 shows the basic difference between the types of transfer bridges.

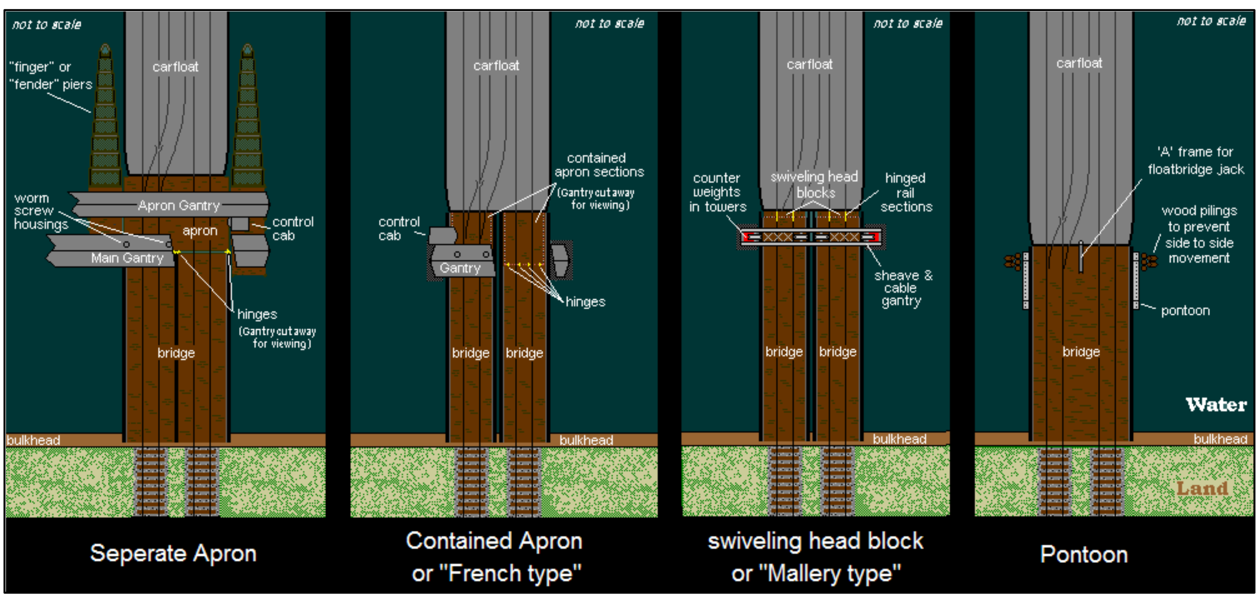

Photo 20

Pontoon Transfer Bridges

The pontoon type of transfer bridges were the first to be introduced in New York Harbor and are a passive transfer bridge system. This type of transfer bridge system commonly consisted of a single articulating bridge span, which on the land end was supported by a rocker bearing and at the free end supported by a pontoon. The rocker/roller would be made of a simple round log or cut timbers to form a cylinder. In either case, the timber rocker/roller would have been reinforced with iron or steel banding around the circumference. Photo #21 shows a typical rocker arrangement. Photo #22 shows a pontoon bridge under considerable torsion from only loading one half of the bridge or float.

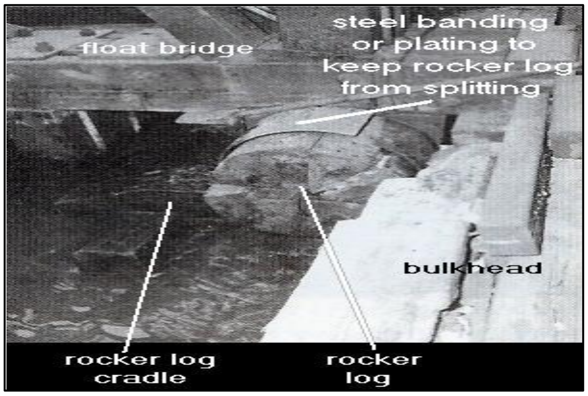

Photo 21

Photo 22

The early pontoon type transfer bridges were two-track transfer bridge with three-timber pony Howe trusses with a center truss between the two tracks. This type of bridge would rise and fall with the tide without the need for power. The timber Howe trusses were approximately 100 feet long with truss centers spaced at 16 feet. The distance between chord centerlines was about 10 feet with panel spacing of about 8 feet. Timber floorbeams spanned from truss to truss, hung from the bottom chord by iron rods. The floor beams were typically 10”x16” yellow pine members. The pontoon transfer bridge deck was typically designed to be above the elevation of an empty carfloat deck and consisted of timber planking laid across the floorbeams, parallel to the bridge. The rail was inset on the deck. The track centers for these double track bridges varied throughout the harbor but typically would be at 16’-8” centers. Timber trusses and timber pontoons were replaced with steel plate girders and steel pontoons although some railroads, such as the B&O preferred the timber trusses and when some of their bridges were due for replacement in 1954, replaced them with the timber variety. There were also a hybrid type in the harbor at the Jay St. Terminal, which had a center truss and outer plate girders. Photo #23 shows an elevation view of a typical pontoon transfer bridge with Howe trusses and a hinge at the pontoon-bridge interface.

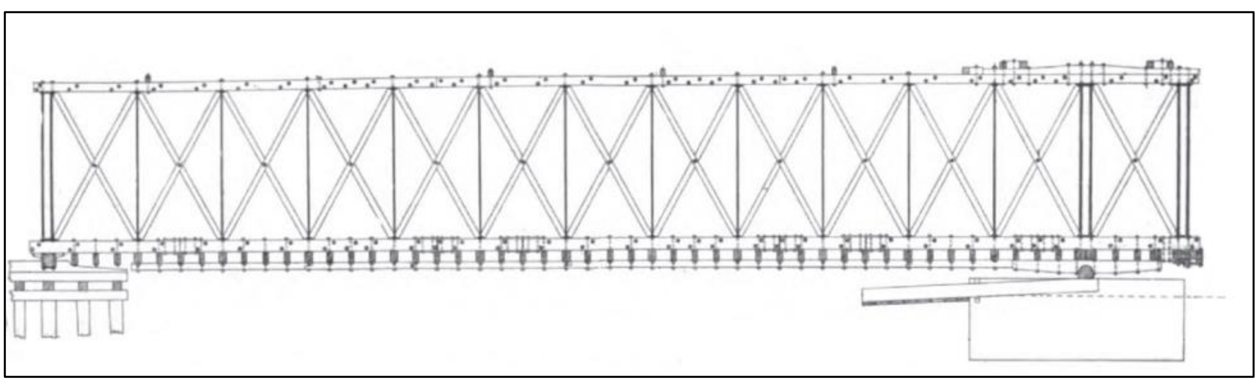

Photo 23

There is currently one operational pontoon type transfer bridge in New York Harbor at Greenville Yard in Jersey City, NJ. The bridge is a 100 feet long by 33 feet wide steel through-girder bridge with three steel pontoons supporting the free end. Photo #24 shows the active pontoon transfer bridge (formerly know as

"Bush 2") at approximate high tide, operated by NYNJR, LLC. Notice how the bridge becomes partially submerged daily due to normal tidal fluctuation in the harbor.

Photo 24

The ends of these bridges took a beating from the impact of carfloats and railcars. To help minimize the carfloat impact the ends of the bridges were arranged to help provide a means of dampening. Early on spiral springs, rubber cushions, and other devices were used to provide dampening. These devices failed easily and were destroyed rather quickly by the carfloats. More commonly, timber blocking was installed at the interface to act as a buffer most times rather than other devices. Two advantages of timber blocking were it was readily available, and replaceable. If no buffering is provided the heel bearing on the land end is required to take the additional impact.

The original timber pontoons were closed rectangular boxes, which were planked, caulked and coppered. As steel pontoons were introduced, the pontoons were simple steel boxes with internal framing. The pontoons were designed and constructed so that the size was enough to support the dead load from the bridge and to maintain a freeboard to easily match the carfloats. The bridge and pontoon connections varied depending on the particular bridge and owner. Some of the pontoons were rigidly connected to the bridge at multiple locations along the pontoon length while the more widely used connection to the pontoon was a roller or sliding surface at the bridge-pontoon interface. The pontoon would have timber cribbing of some sort, which would bear against the bridge. Later, Teflon plates were introduced between the bridge and timber cribbing to allow for easier sliding of the interface as the angle and submergence of the pontoon changed during tidal variation and interchange operations. The pontoons were typically 9 feet deep by 40 feet wide. Most pontoons had a total length between 30-45 feet and could be made of a single pontoon of the length or multiple pontoons to make up the total required length. Photo #25 shows a typical steel pontoon without bridge. Photo #26 shows the top of the pontoon at the pontoon-bridge interface. This particular bridge utilizes a Teflon plate between the bridge girders and timber bearing on the pontoon to allow sliding.

Photo 25

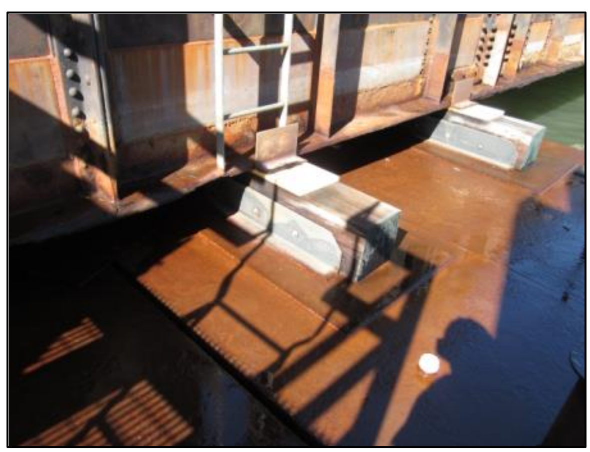

Photo 26

In the case where the transfer bridge was above the carfloat, locomotives and/or "reacher" cars would then be pushed onto the bridge to sink the pontoon until the deck of the bridge and deck of the car float were approximately level to drive the toggle pins. These transfer bridges would be simply supported for live load from the rocker to the toggle pins at the carfloat with a slight upward reaction from the buoyancy of the pontoon. Photo #27 shows a locomotive driven onto the bridge to submerge the bridge to allow driving of the toggle pins. Photo #28 shows the interface between the carfloat (left) and bridge (right) as the far side toggle pins are being driven. Photo #27 and Photo #28 are occurring at nearly the same time.

Photo 27

Photo 28

The proceeding photos are an example of a simple pontoon type transfer bridge. The bridge in the photos is from the former B&O Railroad transfer facility in Manhattan at West 26th Street. The bridge is still in existence on the bank of the Hudson as a pedestrian park. Photo #29 shows the bridge from the river end in the 1970's. Photo #30 shows the trusses from the land end in the 2000's as a pedestrian park.

Photo 29

Photo 30

In cases where the bridge deck was below the carfloat deck, means of lifting the pontoon bridge would be required and were provided, with varying degrees of success. There were three primary methods of lifting a pontoon bridge so that the bridge deck and float deck were level.

One method utilized hydraulic jacks to lift the bridge, jacking against the carfloat to raise the bridge to the level of the carfloat. In this method, a high carfloat would approach the bridge and be moored to the bridge for horizontal alignment. Once basic horizontal alignment was achieved, deck hands would place timber blocking on the deck of the carfloat at the location of the hydraulic jack. The piston would then be extended until the bridge deck and carfloat deck were level enough to drive the toggle pins. Photo #31 to the right is an example of the hydraulic jack used in this method. This photo is from the transfer bridge at Jay Street Terminal in Brooklyn.

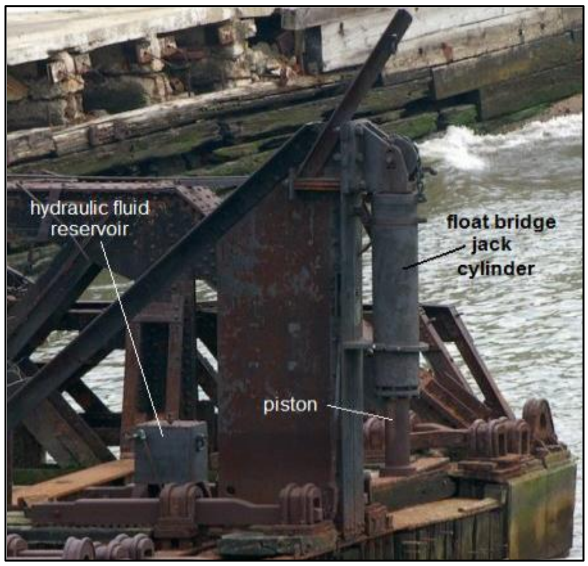

Photo 31

A second method utilized a light gallows frame supported on an independent foundation. A drum would be outboard of the bridge with lifting chains going up and around a sheave above on the frame and connecting back down to the bridge. The drums could be powered through a motor or could be manually operated by a hand wheel through a train of gearing. This method was rarely used due to operation time constraints. The gallows frame principal use became as a support for the bridge incase repairs were needed on the bridge or pontoon. The gallows frame and hoisting chains were designed to take the dead load reaction of the bridge at the free end. To operate the bridge while loading and unloading railcars, the chains would be slacked or unhooked from the bridge since the chains were not intended to support live load. The proceeding photos show a pontoon type transfer bridge with a light frame to lift the bridge. Photo #32 and Photo #34 are two sketches are from the New York, New Haven, and Hartford Railroad Transfer Bridge at Harlem River. Photo #33 is a picture of a similar type arrangement at Fulton Terminal in Brooklyn in 1911.

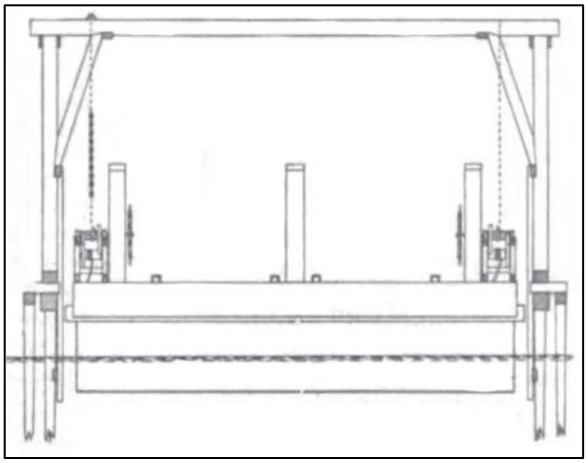

Photo 32

Photo 33

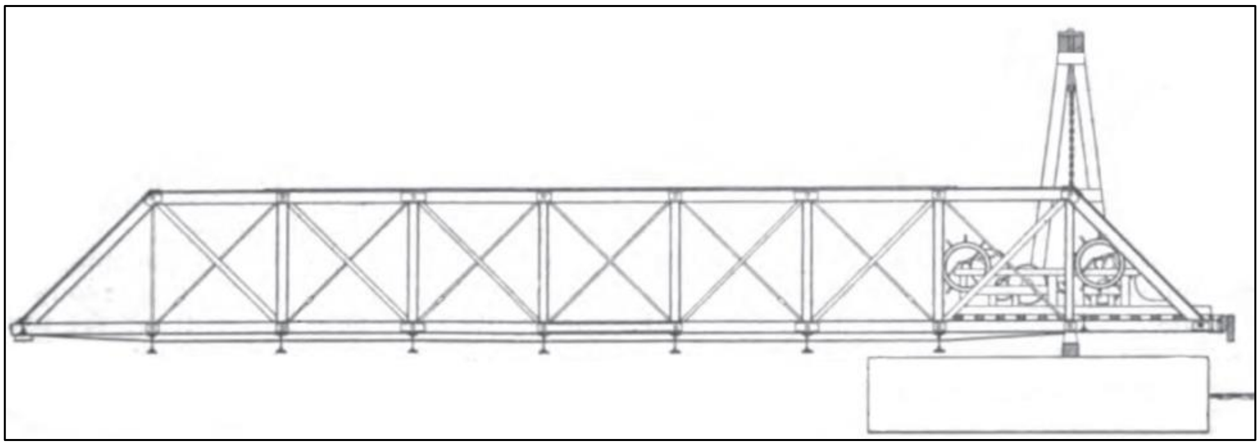

Photo 34

A third method utilized hydraulic pumps and flood valves to flood/empty ballast chambers in the pontoons to raise and lower the bridge. This method was the most time consuming and was the least successful method to raise the bridge reliably.

The hydraulic jack, light duty gantry, and flooding/emptying the pontoon methods of raising the bridge to meet with the carfloat were all performed under no live load. The toggle pins locked the bridge and carfloat vertically and transversely and ensured the rail alignment on the bridge and carfloat were suitable enough to transfer railcars between the bridge and carfloats.

The methods utilized to raise and lower the pontoon bridges were also utilized at times to decouple the toggle pins from the floats. The toggle pins were 5-inch square and 7 feet long. There were two toggle pins per track therefore for the double tracked transfer bridges there were four toggle pins. The toggle pins were situated outboard of the rails at each track. The pins had guides at the end of the transfer bridges and corresponding receivers on the carfloats. The bridge and carfloat would need to be approximately level to allow the toggle pins to be retracted easily. As mentioned earlier, all carfloats in the harbor were standardized so carfloats could be interchanged between railroads easily. Effectively the standardization on the carfloats dictated the size and location of the toggle pins and the location of the tracks on the bridges.

Photo #35 and Photo #36 show the toggle pins, guides and receiver for the pontoon bridge at Greenville Yard in Jersey City, NJ.

Photo 35

Photo 36

The pins, guides, and receivers were often damaged during decoupling of the carfloat and bridge. One issue is with the standardization of the carfloats in the harbor. As railcars evolved into heavier and longer cars the load inevitably increased although the size of the toggle pin did not change over the years. The other issue, which causes extreme damage to the pins, guides, and receivers, is how the carfloats are released from the bridge. Some of the early practices simply involved releasing the mooring lines and pulling the carfloat off the bridge with the pins engaged. This method often bends the pins severely. Other methods prolonged the life of the pins. Running a locomotive or reacher car onto the bridge to depress the bridge enough to easily retract the pins was the most common way to decouple the bridge and float. This method is somewhat time consuming as the railcar would need to be on one track to depress the bridge and remove the pins, and then go back on land to the switch and go on the other track to depress the other side of the bridge and retract the pin. This method also puts an extreme amount of twisting on the bridge and toggle pin since it is pinned on only one side during this operation. The Long Island and Lehigh Valley perhaps had the best and supposedly quickest method of decoupling the bridge and float. Each truss end post was equipped with a jack to bring the decks of the float and bridge level. They were then able to retract all four toggle pins at once without adding additional strain to the pins as the other methods.

The main drawbacks of pontoon style transfer bridges are icing of the river, which limited the effectiveness of the pontoon transfer bridge in the winter months, silting under the pontoon, which could bottom out the pontoon in the mud and required regular maintenance to dredge in the pontoon area, and failure of a pontoon due to leaking. In the later years of the pontoon transfer bridges; owners would fill the void in the pontoon with athletic balls to prevent the bridge from sinking in the river suddenly. The pontoon type of transfer bridge also has a short useable life. The closeness to the waterline, brackish environment, and partial submergence into the river during operation accelerated the deterioration of the truss and plate girders bridge spans. The wooden trusses could last approximately 50 years while the steel trusses and plate girders had significant rusting issues and would last about 25 years before being taken out of service.

Pontoon supported transfer bridges were a simple technology that suited the needs of the railroads in New York Harbor quite well at the time. As longer, heavier railcars emerged and increasing operational demand, the pontoon type transfer bridges were slowly phased out. First with steel replacing timber trusses, frames, and pontoons to what would become the dominant type of rail transfer bridges, the advanced overhead gantry hung transfer bridges. The proceeding two photos show pontoon transfer bridges rebuilt with steel plate girders in Photo #37 and steel trusses in Photo #38

Photo 37

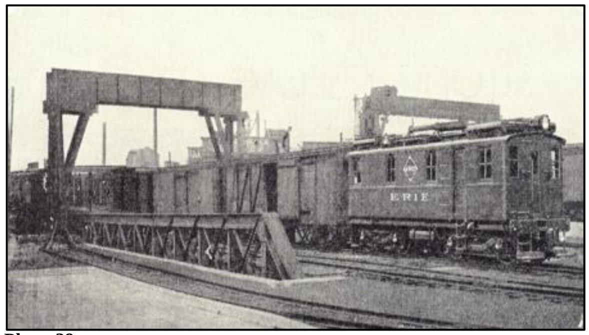

Photo 38

Gantry Supported Transfer Bridges

Separate Apron Type at Harsimus Cove

The larger load demands, increased operational efficiency demand, and required reliability of operation in nearly any condition led the PRR to develop a more advanced transfer bridge system. The first of these new types of transfer bridges were built at Harsimus Cove in Jersey City. The new transfer bridges were hinged at the land end and hung from machinery on an overhead gantry at the river end. The overhead gantry type could vary from a single articulating span to a double articulating span. For the double articulating form, the main span was typically referred to as the bridge span and spans between the hinges on the land end to an overhead gantry at the free end while the apron span is a shorter, more flexible span that is hinged to the bridge and interfaces with the carfloat. The bridge span was typically three times longer than the apron span. Photo #39 shows a sketch of a section at the gantry for the transfer bridge at Harsimus Cove. Photo #40 shows a sketch of an elevation of the transfer bridge at Harsimus Cove. The counterweight is in both bays of the gantry but the lifting screws are only in the far bay (to the right in the photo).

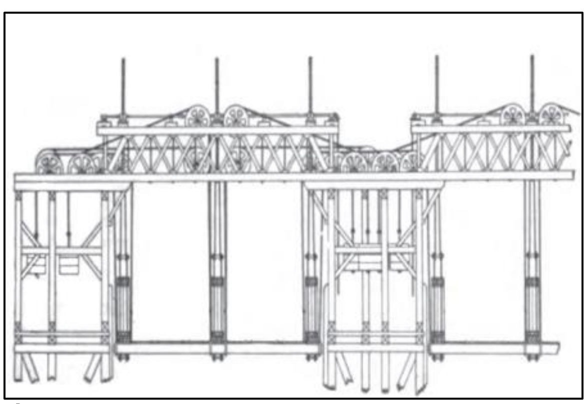

Photo 39

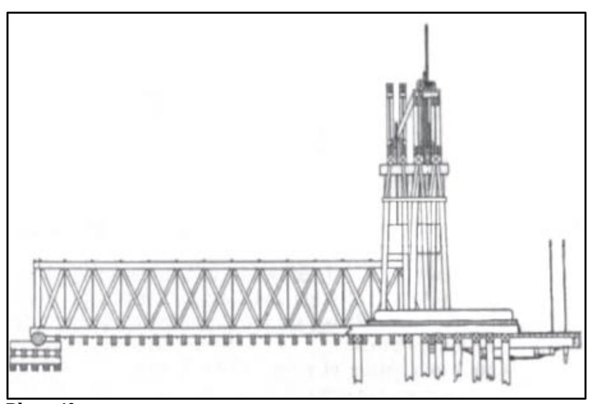

Photo 40

The first gantry supported rail transfer bridge was developed by the PRR in 1888 at Harsimus Cove in Jersey City, just north of the passenger ferry terminal at Exchange Place in Jersey City. The PRR built two of these bridges at this location at the time. The double tracked bridge span consisted of three 71’-0” long Howe trusses spaced out approximately on 16 foot centers. The trusses were constructed of primarily timber (yellow pine) members with iron rod hangers at each panel point. 10”x16” timber floorbeams were hung by 1¼” diameter iron rods from the bottom of the truss at 2’-6½” spacing and spanned from truss to truss. The floor system was constructed of timber planking running parallel to the trusses with the rail inset in the planking. The bridge was partially counterweighted (90% of the dead load) at the end of the bridge span through an iron counterweight supported by wire rope over sheaves on the gantry system. The counterweighting only supported the dead load in this version. The bridge span was originally powered by steam and later by electric motors.

The apron span was 25’-6” long, spanning from the hinges on the bridge to the carfloat. The apron hinge at the bridge consists of friction gearing which was forceful enough to hold the apron in place when no carfloat is at the free end yet allowed for rotation when a carfloat was loaded or unloaded. The apron span was also partially counterweighted (90%) at the end of the apron. The apron was operated originally by hand and later by electric motors. Similarly to the pontoon type of transfer bridge, the apron span had two tracks with one track additionally having a switch for loading the center track of a carfloat in addition to toggle pins. One of the advantages over the pontoon type that the original overhead gantry type at Harsimus Cove demonstrated was the time saving in coupling and decoupling the toggle pins and the carfloat. The bridge could be simply operated up or down slightly to meet up with the carfloat or to free the pin from the carfloat and the pins retracted without causing damage to the pins. This operation could be done in as little as 30 seconds.

The gantry was an open timber frame approximately 30 feet high and 60 feet wide per bridge. The gantry towers were outboard of the transfer bridge with a timber Howe truss spanning between towers supporting counterweight sheaves and operating machinery. The counterweights were large iron blocks hung from wire rope. The unbalanced dead load and the train load were carried by three 4¼” diameter lifting screws, one at each truss on the bridge. Worm gears turned a large bronze nut, lowering or raising the bridge typically 12” at a time. The operating machinery was designed only to raise and lower the unbalanced dead load of the bridge and apron and not used to lift the live load. Future bridges were able to raise and lower the live load as well.

Photo #41 shows PRR Bridge #4 and Photo #42 shows the PRR Bridge #3 at Harsimus Cove.

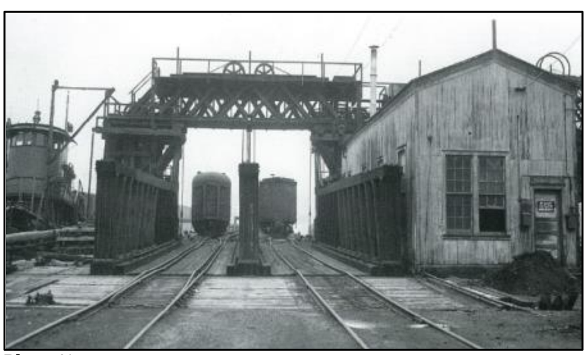

Photo 41

Photo 42

Just north of the original bridges in the Harsimus Cove facility, the PRR had three additional bridges. Photo #43 shows PRR Bridge #6 from the river end. Photo #44 and #46 shows PRR Bridges #5, #6, and #7 from the land end. Photo #45 PRR Bridges #5, #6, and #7 from the river end.

Photo 43

Photo 44

Photo 45

Photo 46

Separate Apron Type Advancements at Greenville Yard

The PRR had a lot of success when they introduced this type of bridge to New York Harbor that they built many more in the harbor. Due to land constraints, demand, and heavier loads the PRR built a much larger facility 3.5 miles south at Greenville in Jersey City, NJ which became known as Greenville Yard and planned for the construction of three new transfer bridges. The plan for this expansion was first introduced only 2 years after the bridges at Harsimus Cove were built in 1890 and was expected to be the largest waterfront terminal in America. In 1901 work began on filling the mudflats at Greenville for a terminal at the site. Most of the site was 5-6 feet underwater and would be filled to 8 feet above Mean Sea Level at the bulkhead and around 47 feet at the hump at the back end of the yard. 22,000,000 cubic yards of fill was estimated including fill from Manhattan from the excavation for the new PRR station in Manhattan. The new yard was to have a capacity of at least 8,000 railcars. Photo #47 shows the layout of Greenville Yard in 1905, including at the time three transfer bridges on the north end of the yard, a 1,000 foot long lighterage pier to the south and a 275 foot long coal pier further south. The number of transfer bridges would increase to a total of six over time.

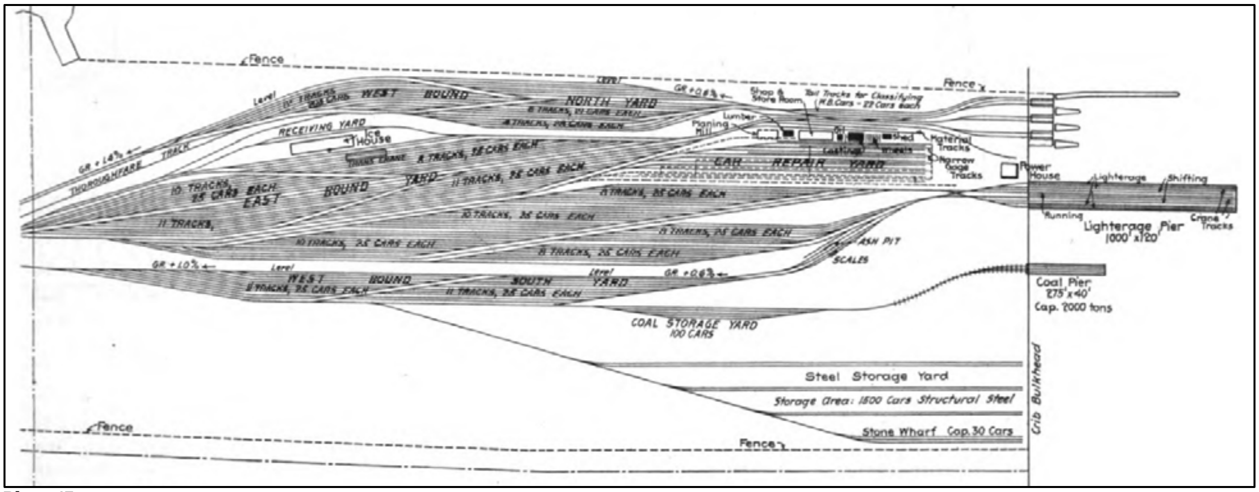

Photo 47

The three transfer bridges at Greenville Yard would be built more robustly than the Harsimus Cove counterparts. The new transfer bridges at Greenville Yard would be related to the original bridge type at Harsimus but included new major advancements. The bridges at Greenville Yard introduced an apron live load counterweighting and operating system and also improvements in the machinery system which included an operating system and lifting screws capable of moving the unbalanced dead load and the train live load. Photo #48 shows a sketch of an elevation of the bridges at Greenville Yard. The gantry on the right is the bridge gantry. The lifting screws connecting down to the bridge are to the left of the counterweighting to the bridge. The gantry to the left is the apron gantry showing the live load counterweight sheaves. Both gantries were enclosed under a single wooden housing until 1931. Photo #49 shows the transfer bridges from the river end. Photo #50 shows the transfer bridges from the land end. Notice the wooden enclosures over the trusses. Photo #51 shows the transfer bridge being unloaded. Photo #52 shows the transfer bridge without the enclosure over the gantries.

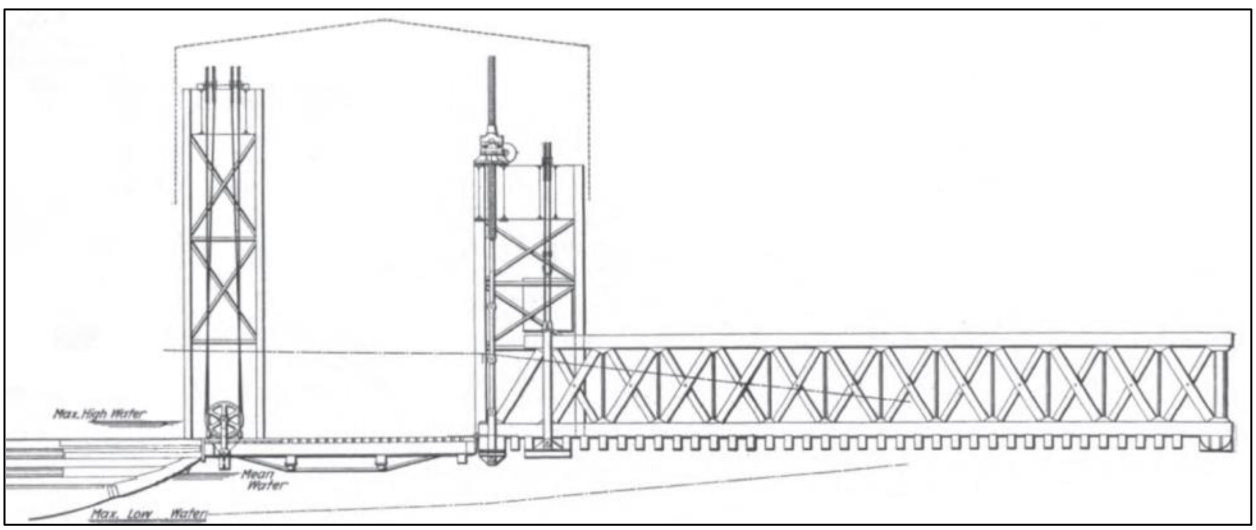

Photo 48

Photo 49

Photo 50

Photo 51

Photo 52

The original three transfer bridges at Greenville Yard were built in 1904 spaced 75 feet on center at the bulkhead. These new bridges were Bridge #11, Bridge #12, and Bridge #13. As the demand grew at the yard Bridge #14 was added in 1910 and Bridge #10 was added in 1925. In 1931 a massive fire at the yard destroyed the wooden structure. After the fire the bridges were constructed of steel plate girders. In 1943 Bridge #9 was added to the north and two years later Bridge #14 was rebuilt. The bridge spans had three 8 foot tall plate girders arranged similarly to the truss bridge spans. The bridge and apron gantry designed remained nearly the same except the two gantry were housed separately. Photo #53 is a spliced photo showing the Howe truss bridges and the plate girder bridges for the PRR Annual Report in 1945.

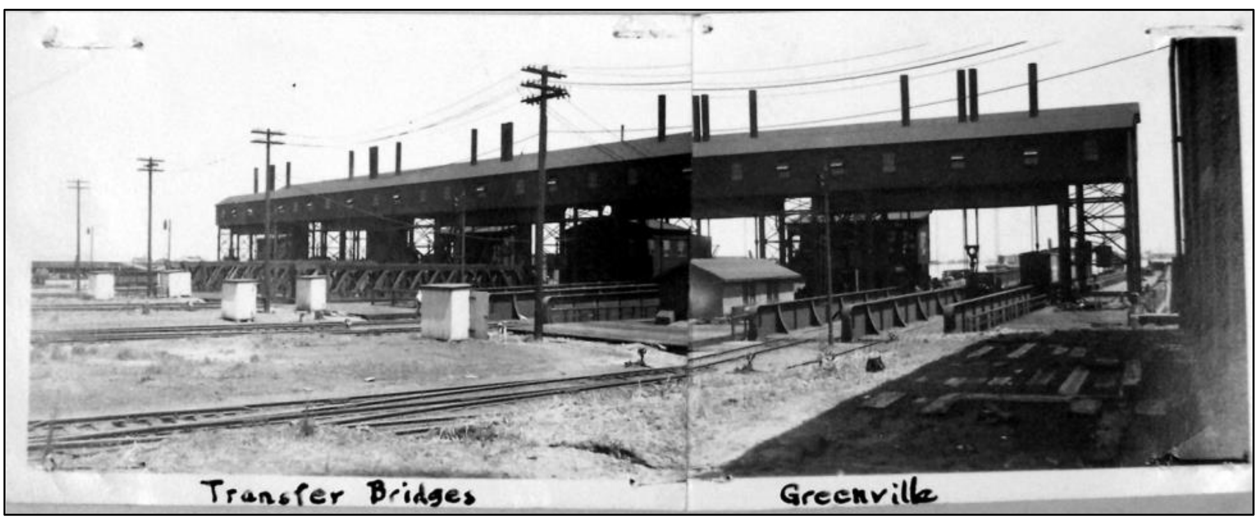

Photo 53

Photo #54 shows a view from the land end facing east through the bridge at Bridge #11. Photo #55 shows a view from the river end on the apron facing west through the bridge at Bridge #9.

Photo 54

Photo 55

The original bridge span consisted of three 80 foot long timber Howe trusses, with the center truss being of heavier construction. The truss chords were spaced at 9’-6” centers with 13 evenly spaced panels. The floorbeams and deck were of similar construction to that of the Harsimus Cove bridges. The trusses were enclosed by a wooden casing. Later, the bridges were constructed with built up steel plate girders and steel floorbeams spanning under the girders. The Howe trusses used a semicircular piece of timber to allow rotation. The steel bridge versions utilized two different assemblies. One assembly consisted of a convex steel bearing mounted on the bridge girder sitting in a matching concave steel saddle. The second assembly consisted of pin through the plate girder resting on a contained bearing. The apron was 32 feet long and was hinged at the end floorbeam of the bridge span. The apron construction consisted of longitudinal timber stringers with a solid deck made of timber cross ties running perpendicular to the stringers across the entire width of the apron. Photo #56 shows a convex/concave heel bearing at Bridge #9 out of the saddle. Photo #57 shows the pin heel bearing at Bridge #10. The bearings experienced extensive rusting being in the splash zone and submerged twice a day due to the tide.

Photo 56

Photo 57

Near end of the bridge span a steel overhead gantry was constructed to support the bridge counterweight system and lifting screws. The bridge gantry is approximately 30 foot tall to the top of the gantry beam. The gantry column legs are typical laced steel columns. A 6 foot deep gantry beam (plate girder) spans across the bridge and supported the lifting screws and sheaves for the bridge counterweight system. Photo #58 shows a sketch through the bridge gantry at the lifting screws. Photo #59 shows a sketch through the bridge gantry at the counterweight. Photo #60 is a construction photo of the gantries.

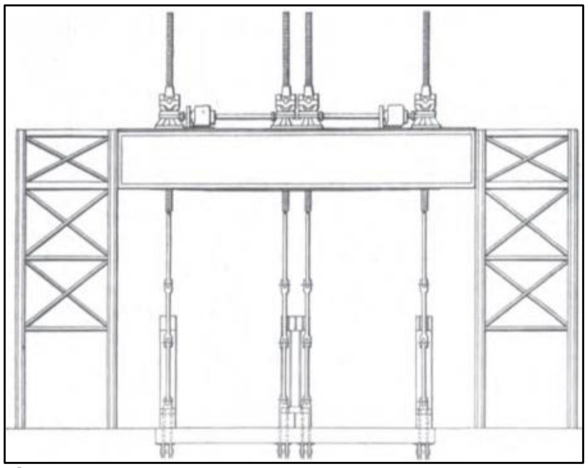
Photo 58

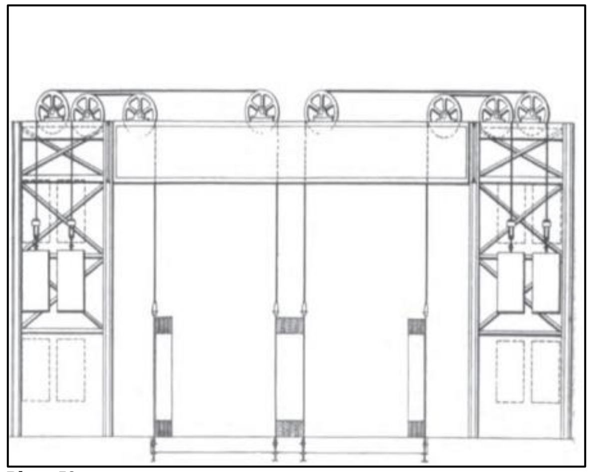
Photo 59

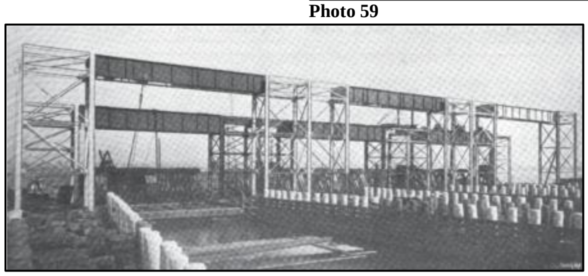
Photo 60

The end of the bridge span was counterweighted for 90% of the dead load in addition to being supported by four 7” diameter lifting screws. The lifting screws for these bridges were arranged to have one screw over each of the outer trusses and two screws over the center truss. The screws are connected to a linkage system connected down to the bridge. Two styles of lifting screws were employed over the years. The early gimbal and the later French type lifting screws both allowed the bridge to swing forward, backwards and side to side while being driven by a worm gear, reduction gearing, and two fifty horsepower motors. The machinery was powerful enough to operate the bridge loaded at 300 tons at a speed of 4 feet per minute. Both types of lifting screws were buttress threaded and the nuts allowed the bridge to “float” upwards and disengage to prevent bucking of the threaded rods if the screws were overdriven. Photo #61 is a view of the French type lifting screw. Photo #62 is a sketch from French’s Patent of the French type lifting screw. Photo #63 is a view of a gimbal type of lifting screw.

Photo 61

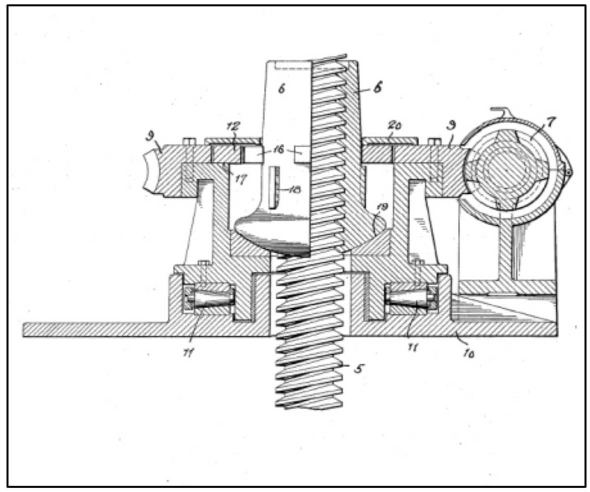

Photo 62

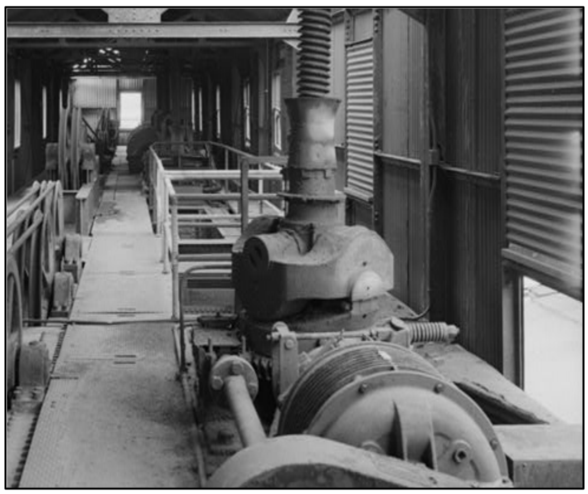

Photo 63

Near the end of the apron span a similar steel gantry supported a 90% dead load counterweight system in addition to a live load counterweighting system and apron drive system, which was employed on some of the earlier Greenville bridges. The apron gantry was 9 feet taller than the bridge gantry. Within the apron gantry, between the columns, the operator's house overlooked the end of the apron. Photo #64 is a view inside the common timber enclosure of the gantries prior to the fire. Photo #65 is a sketch through the apron gantry at the dead load counterweight and live load counterweight.

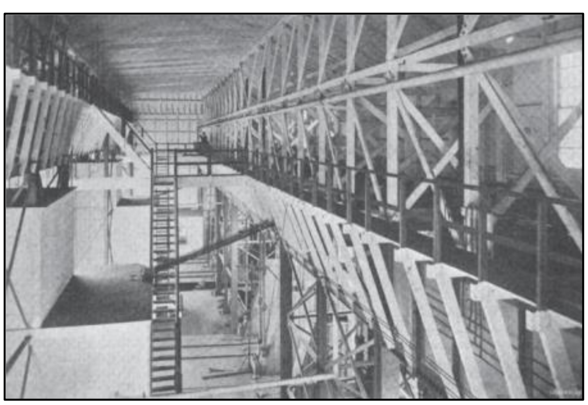

Photo 64

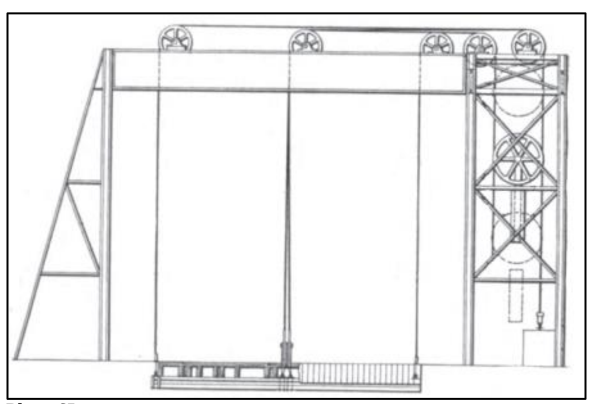

Photo 65

Francis Du Bosque patented the apron live load counterweight and drive system, which was utilized originally at Greenville Yard, in 1914. The system utilized the weight of the live load counterweight, which normally rested on the foundation at the base of the gantry tower to provide a reaction force to hoist the apron via rope K in Photo #66 up above the level of the carfloat deck. Since the apron was 90% counterweighted, and had a 2 to 1 mechanical advantage, the live load counterweight and the thirty-five horsepower winch had to provide only the force of 5% of the apron weight to position the apron. To dock a carfloat the apron was lowered until the toggle pins rested on the deck of the float shown in blue in Photo #68. One of the two ropes in Photo #68 was hooked directly to the front of the float, and the other was hooked under a sheave on the apron and to the same connection point on the front of the float. With the additional mass of the carfloat attached to the apron the live load counterweight was then raised off its pedestal (Photo #67) to a point where it would not crash into the structure from bridge movement due to live load and tides. The live load counterweight would then float up and down providing an upward reaction of 120 tons to the apron and end of the carfloat (Photo #68). Additionally the use of the sheave at the end of the apron served to pull the carfloat tight to the structure. The carfloat was prevented from moving away from apron by a collar fixed to the rope shown in green and at the sleeve stop shown in red in Photo #68.

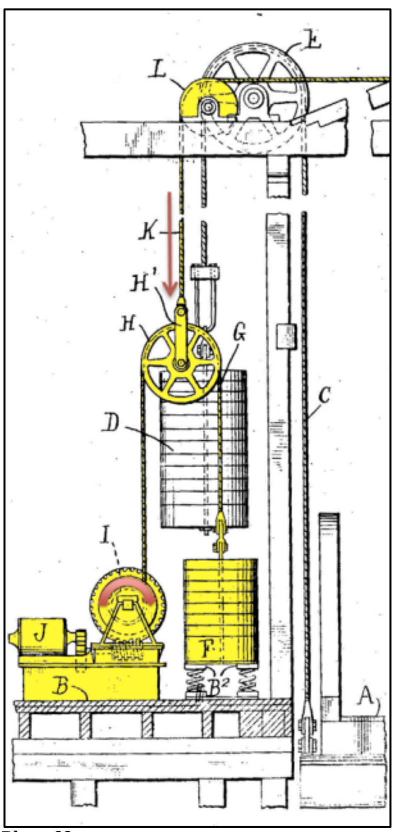

Photo 66

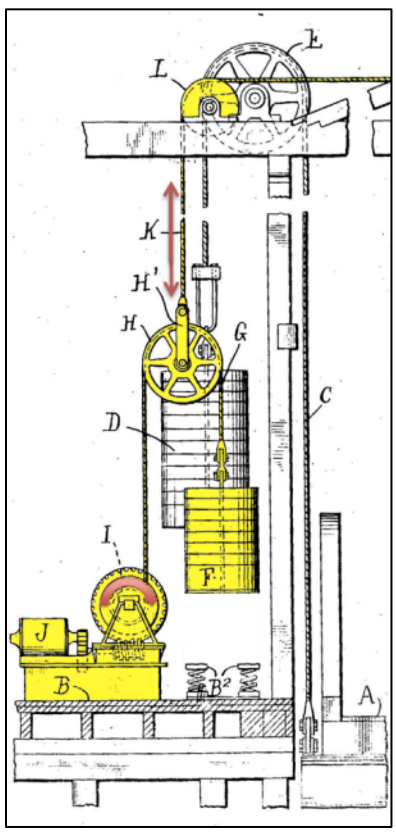

Photo 67

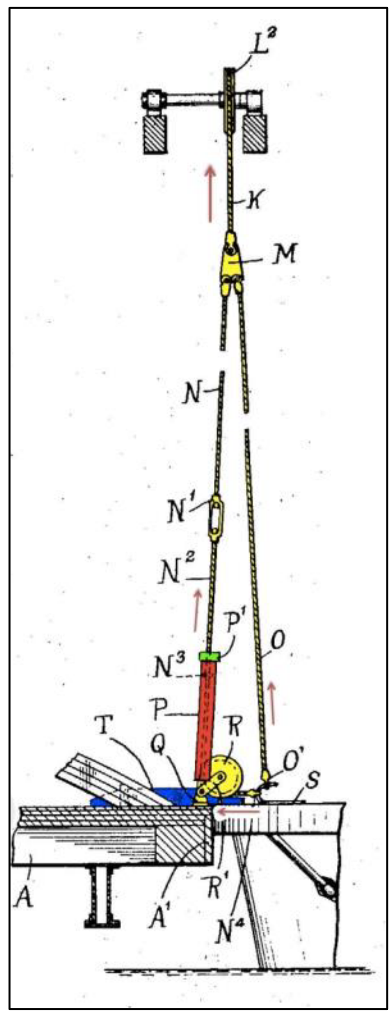

Photo 68

Later apron systems omitted the live load counterweighting and apron drive system in lieu of the friction cylinder. To move the apron, the bridge lifting screws were operated, and the friction cylinders were used to overcome the imbalance and friction at the sheaves in the gantry so the apron would move with the bridge. Once a carfloat was connected, the carfloat and rail live load would easily overcome the friction in the cylinder and the apron could float and twist as needed. The cylinder itself is relatively simple. It is a split hollow tube over a cylinder with bronze wear plates between the two. Bolts and plates clamp down on the split hollow tube squeezing the bronze plates until the desired amount of friction is achieved. Photo #69 and Photo #70 show the friction cylinder connecting the bridge to the apron.


Photo 69


Photo 70

In 2012 the storm surge from Super Storm Sandy damaged the Greenville Yard Transfer Bridge facility beyond repair. A construction barge's spuds were sheared off in the storm, sending the barge into the apron gantry columns. The gantry columns were damaged and and entire bridge and apron gantry system was deemed unstable and unsafe for continued use. The gantry and bridges were removed from Greenville Yard and a "new" emergency temporary bridge was installed. The removal of the old Greenville Yard structure to installation of the "new" pontoon transfer bridge was accomplished in 52 days. "Bush 2" was an old pontoon bridge just recently removed from service at 51st street in Brooklyn. The pontoon bridge was taken by barge to Greenville Yard, repaired, and put into service in January of 2013. Photo #71 shows the six transfer bridges at Greenville Yard in the 1940's or 1950's. Photo #72 shows the transfer bridges in at Greenville Yard in the 1970's. Photo #73 shows five transfer bridges in 2011 although only Bridge #11 was in service. Photo #74 shows the transfer bridges in 2012 a few day after Super Storm Sandy. Photo #75 show the tranfer bridge structures being demolished due to instabilities from the storm. The storm surge submerged the entire yard in over 8 feet of water casuing widespread damage in the yard. Thankfully nobody was harmed during the event.

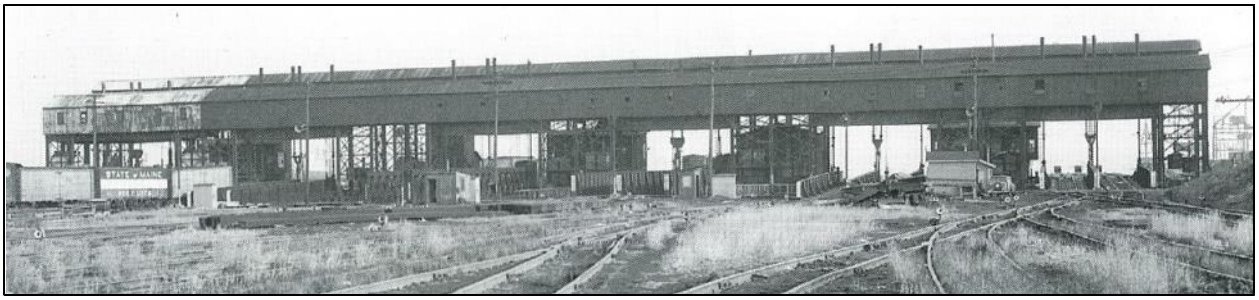

Photo 71

Photo 72

Photo 73

Photo 74

Photo 75

Mallery Type (Swivel Head Block) at Long Island City

The Mallery type of transfer bridge was developed first for the Long Island Railroad (LIRR) at Long Island City in Queens in 1904. This type of transfer bridge was supported from a single overhead gantry by wire rope. The transfer bridge unit contained two independent single tracked, two girder bridges with a short rotating block which was allowed to swivel as the carfloat listed during loading and unloading. The swivel block was a short span 2 feet deep by the width of the single track bridge.

The bridge unit was supported by a single gantry unit, which supported both single track bridges within the transfer bridge unit. The bridge girders were typical steel plate girder which tapered down at the swivel block. The bridge was counterweight for 90% of the dead load. The remaining dead load was handled by a differential pulley chain block which also allowed the bridge to be raised or lower by hand without any live load on the bridge. The gantry was designed to only support the counterweights, sheaves, and dead load of the span. There were no provisions to support live load on the gantry. The short span would span between the counterweight ropes and the float, utilizing the carfloat essentially as a pontoon. The live load reaction was taken at the toggle pin and carfloat interface putting a large load at the pins. Photo #76 is a plan view of the bridge span of the Mallery type from A.H. Mallery's patent. Photo #77 is a sketch through a typical gantry of the Mallery type from A.H. Mallery's patent. Photo #78 is a view of the Mallery type, with the French type modifications, for the Long Island Railroad at Long Island City in Queens. It is currently a pedestrian park, Gantry Plaza State Park.

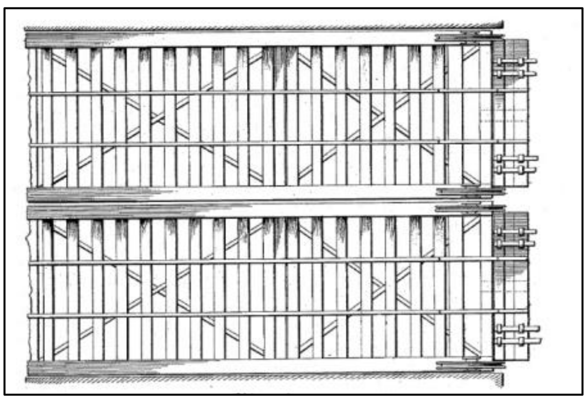

Photo 76

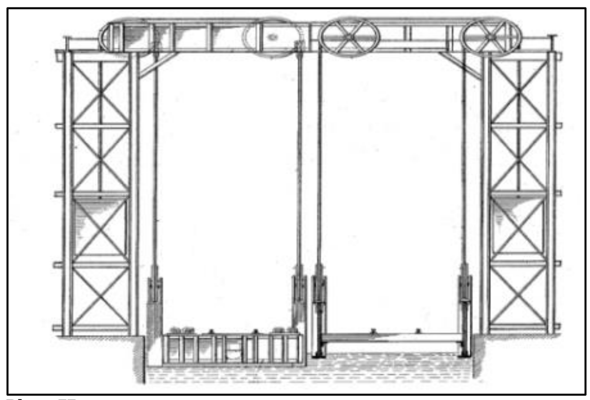

Photo 77

Photo 78

Each single tracked bridge had its own swivel block at the end. The swivel block rotated with the listing carfloat between the toggle pins on either side of the rail for each track. The swivel block was connected to the bridge with a 10" diameter cantilevered pin which rotated about the bridge and fixed to the swivel block. The pin was centered between the rails. The toggle pins were mounted to the swivel block and as the carfloat would twist from unbalanced loading the swivel block would follow. The rails of the track were hinged to accommodate the twisting movement while maintaining top of rail continuity allowing railcars to be loaded and unloaded. Photo #79 shows a section through the swivel block and cantilevered pin. The bridge span is to the left in the photo and the carfloat to the right in the sketch. Photo #80 is a view of the bridge and swivel block facing the bridge. The cantilever pin is located at the center of the block.

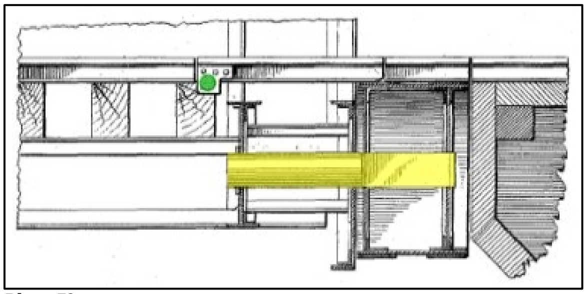

Photo 79

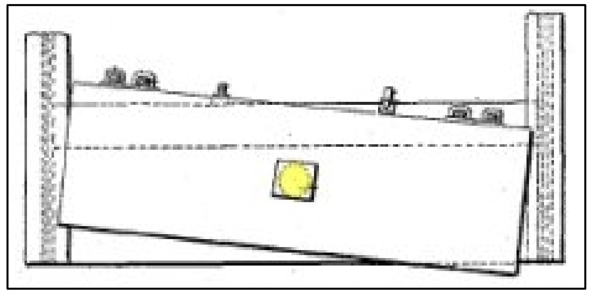

Photo 80

The cost benefits of the Mallery Type transfer bridge could be substantial, saving costs on both the structural and mechanical aspects of the structure. Since only one gantry structure was required, the cost for materials and construction of a foundation was omitted and considering the bridge mechanical system consisted primarily of sheaves and counterweights the cost for a lifting screw assembly and motors was also omitted.

The Mallery transfer bridge had certain design flaws which cause some spectacular failures. One instance of failure occurred during a combination of low tide and a heavy carfloat operation. The combination of both the tide and heavy float forced the free end of the bridge down which in-turn meant the counterweight would be raised. The counterweight was lifted until it hit an obstruction in the gantry, likely the sheave support beam or bumper block, which caused the dead and live load reaction at the end of the bridge to transfer to the connection between the bridge girder and counterweight ropes. The connection, being only designed for dead load, broke sending the bridge into the river with railcars and the counterweight dropped 30 feet from the top of the gantry through the gantry's timber foundation.

After the reconstruction problems were also encountered with the 10" cantilever pin that the swivel head blocks were connected and the bridge was modified and evolved into the French Type transfer bridge.

French Type (Contained Apron) at West 69th Street Terminal

In light of the problems experienced with the Mallery type transfer bridge and to cost saving associated with fewer gantries and operating machinery French helped develop a transfer bridge to address the problems and maintain the cost benefits of less structure. The French type would be more suitable to heavy loads than its predecessor, the Mallery type.

In lieu of adding a second gantry to support the apron and operating machinery to support an apron modifications were made to the river end of the bridge to address the issue with the cantilever pin. The main girders were essentially cantilevered from the eye bar support and a crossbeam was added at the eye bar location and at the end of the cantilevered girders of the bridge. On the crossbeam a rocker support was added in the center of the track at both the eye bar crossbeam and the crossbeam at the end of the bridge at the pin to allow for the same rotation as the Mallery type but without a cantilever pin. The apron was thus contained within the bridge and where the "contained apron" name derives. Photo #81 shows a section through the bridge and gantry of the French type at West 69th Street. Photo #82 shows an elevation of the bridge at the gantry at West 69th Street. Photo #83 shows a plan view of the bridge at West 69th Street.

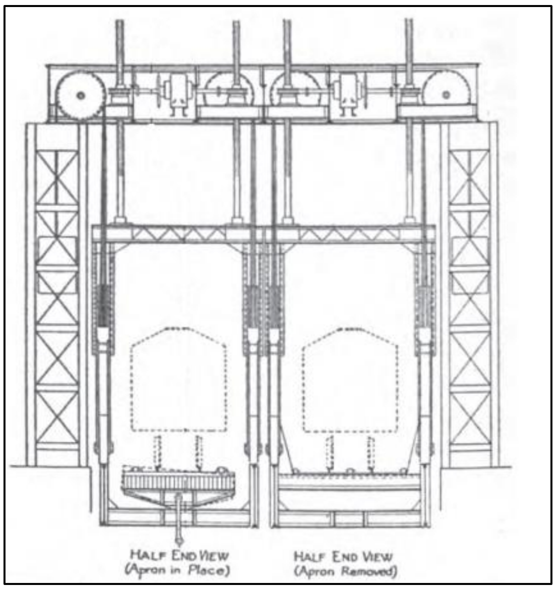

Photo 81

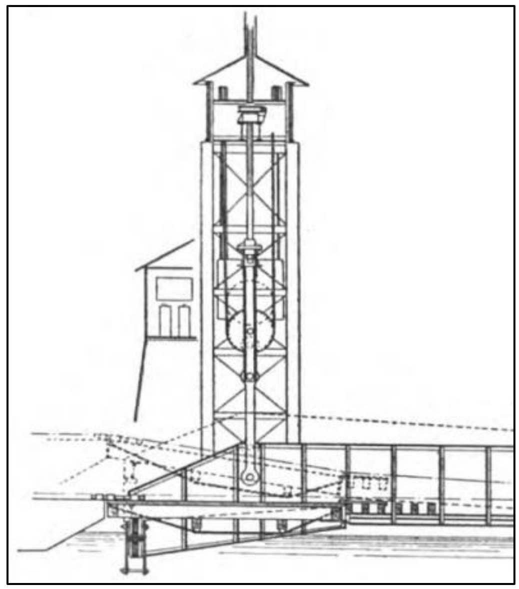

Photo 82

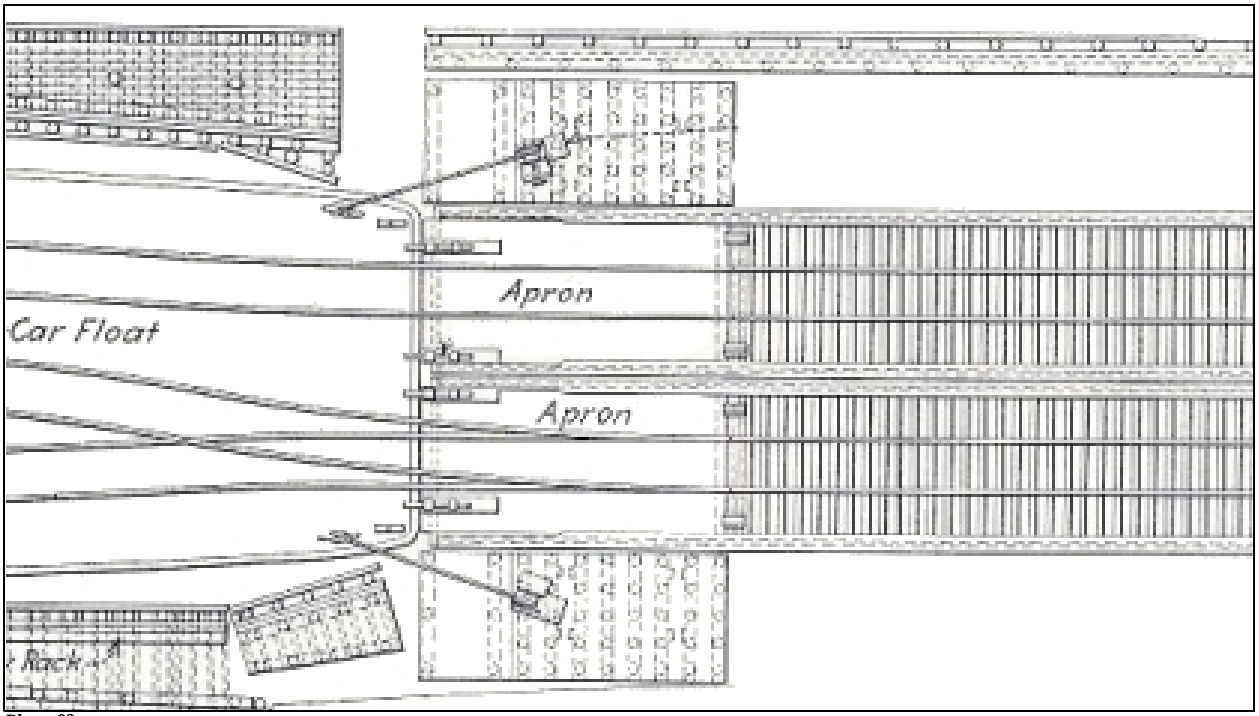

Photo 83

The New York Central Railroad built the French type bridge at its West 69th Street Terminal on the Hudson River in 1909. The bridge incorporated the modifications of the Mallery type transfer bridge. The bridge was comprised of two 8 foot deep steel plate girder, 110 foot long from the heel bearing to the end of the apron. There were two of these single track bridges in a single bridge unit. The portion of the bridge that was cantilevered from the eye bar to the end of the apron was 20 feet long. The apron was 30 feet long and hinged to the bridge 10 feet landward from the eye bar hanger. At the far end of the bridge there is a single gantry. Photo #84 shows the NYC West 69th Street transfer bridge from the river end. Photo #85 shows the same bridge from a slight side view.

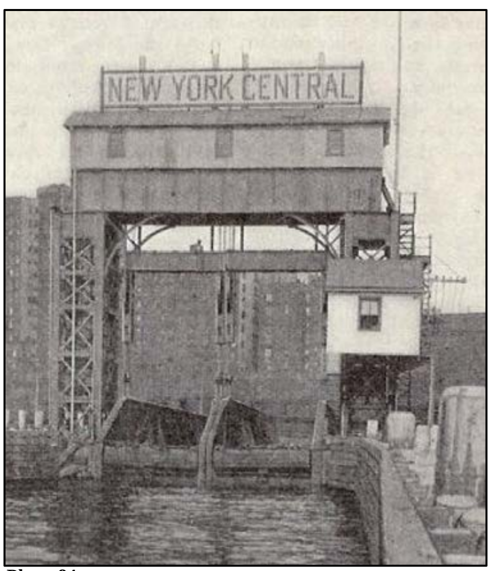

Photo 84

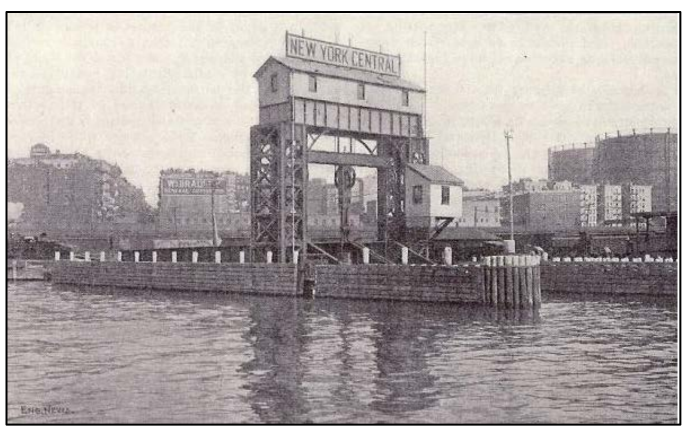

Photo 85

There were four total lifting screws supported on the gantry above, two per single track bridge, which were 7 5/8” in diameter powered by two 50 horsepower DC motors. The lifting capacity of the bridge allowed the bridge to be raised at the far end, with fully loaded 50 ton capacity railcars, at a rate of 4.5 feet per minute. The lifting screws and counterweight ropes are connects to a semi-rigid steel yoke from which the bridge is suspended from connecting eye bars. The eye bars connect the yoke to the bridge through a 7 ½” pin set in a bronze bushings. The lifting screws sit in a gimbal type mount. Photo #86 shows the gimbal mounted lifting screw at West 69th Street. Photo #86 shows the yokes at the LIRR Long Island City Mallery type bridge which was basically modified to be a French type bridge with lifting screws

Photo 86

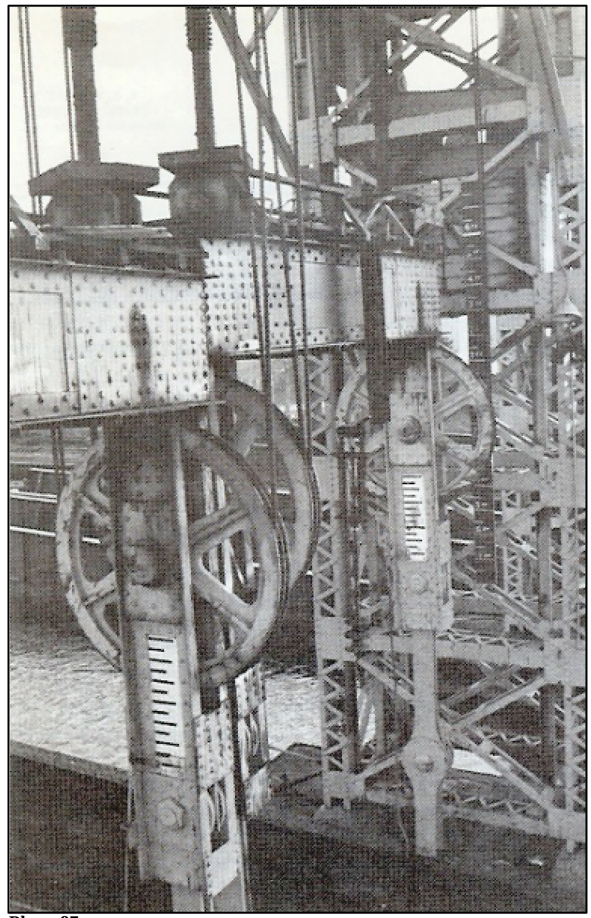

Photo 87

The Decline of Transfer Bridges in New York Harbor

Demographics & Economics

The combined population of Manhattan, Brooklyn, Queens, and Long Island is approximately 9,200,000 as of the 2010 U.S. Census. This sub-region, if a state, would rank 11th in population out of all U.S. states and territories. In the 1860's, at the beginning of carfloating operations in New York Harbor, this sub-region had a population of 1,200,000 people. By the time of major consolidation in the harbor rail activities the population was approximately 8,000,000 in the 1960's. The regions population increased the demand for good in the harbor.

In the early 1900's nearly half of the foreign freight coming into the United States was going through New York Harbor. By 1917 rail-mileage had peaked in the United States and a decade later the Great Depression hit the railroads hard. Rail revenue from 1928-1933 for freight and passenger service fell 50% and by 1937 30% of all rail miles were in receivership. Most major railroads were on the brink of bankruptcy by the time WWII began. WWII increased rail traffic nationwide and gave a temporary bump to declining rail volumes. By 1949, rail volumes sunk below pre-war levels and many railroads were near bankruptcy. There are no exact figures of rail traffic in New York Harbor indicating the moment when rail volumes began to drop but the major railroads were expanding the float facilities into the late 1940's. The freight traffic in the harbor likely peaked during the WWII era. By 1970 the share of intercity freight dropped to 35% whereas it was as high as 75% in the 1920's. This drop in rail freight traffic in the harbor coincided with the drop in rail freight traffic across the United States but also was related to regional events in the harbor as well. The causes for the decline in float volumes in New York Harbor cannot be traced to a single event but was a perfect storm of events that are interrelated and amplified each others effect. Figure #4 shows the U.S. Railroad Mileage until 1920, which peaked in 1917 (Line mileage peaked in 1917, although track mileage did not peak until 1930). Figure #5 shows the Railroad Share of Intercity Ton-Miles from 1930-1970 illustrating the competition of freight transport the rail industry experienced.

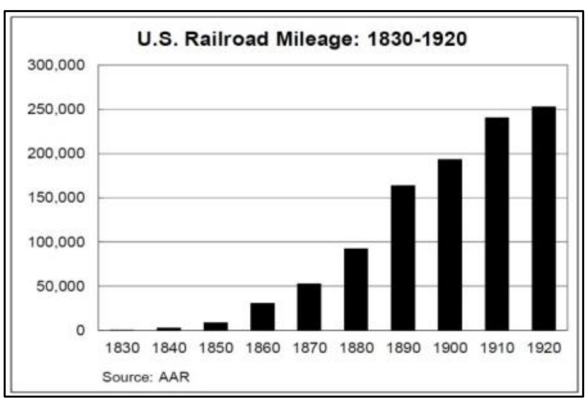

Figure 4

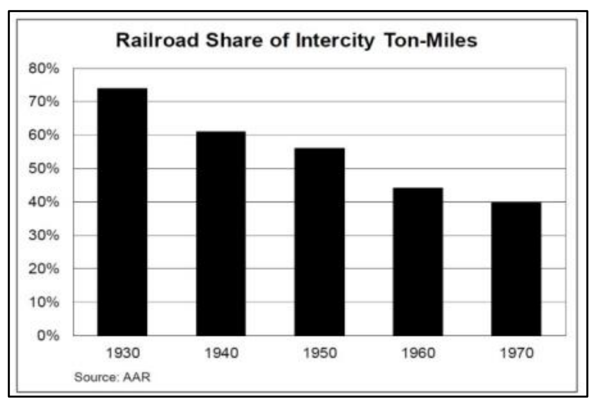

Figure 5

New York Harbor Float Traffic Decline

One of the first causes to the downfall of freight rail activity across the harbor was the construction of the Hell Gate Bridge by the PRR. The bridge was opened in 1916 and gave direct access to the PRR into Queens. It was constructed to link the New York region with the New England region by connecting the PRR with the New Haven Railroad. Both of these railroads had float operations in the harbor at the time of construction and continued float operations after its construction but allowed certain traffic to be diverted. Photo #88 shows the Hell Gate Bridge in 1917.

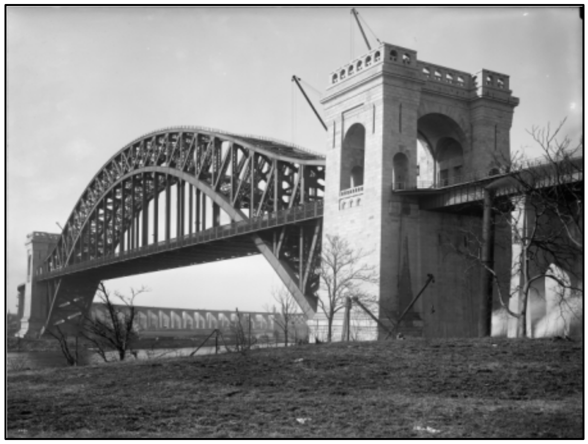

Photo 88

The second cause of the decline in float volumes in the harbor was due to capped rates of freight over rail. The ICC set minimum and maximum rates on rail freight regardless of demand and cost to transport the goods. The rates were set in an effort to keep transportation cost for commodities, such as grain and manufactured goods, low for the public but had unintended consequences to both rail traffic nationwide and float traffic in the harbor. Many manufactures found it much more economical to ship their goods via trucks on the highways forgoing floating through the harbor thus reducing total demand for float operations. The Staggers Act of 1980 ended the ICC control of railroad rates but by this time the damage was done.

The third cause was the Interstate Highway System. The Federal Aid Highway Act of 1956 set for the construction of the U.S. Interstate System. As of 2012, it is the 2nd most extensive network of highways in the world behind China. The beginning of the Interstate Highway System coincides with the reduction of rail freight nationwide and rail freight in the harbor. Along with the capped rates set by the ICC, manufactures had a new means of transporting goods over the Highway System at a lower price than by rail.

The fourth cause is the construction of the bridges and tunnels across the harbor. Aside from the PRR Hudson River tunnels and passenger ferry service in the harbor, there was no direct access to Manhattan from New Jersey until 1927 for commuters. The PRR tunnels could not handle freight rail, thus the reliance on float operations to transport freight. Unless commuters were getting to New York City via the PRR tunnels, commuters were forced to ferry across the river at the railroad ferry terminals in Hoboken and Jersey City. In 1927 the Holland Tunnel was opened. In 1931 the George Washington Bridge was opened (upper deck). In 1937 the Lincoln Tunnel was opened. In 1955 the Tappan Zee Bridge was opened. In 1962 the lower level of the George Washington Bridge was opened. And finally in 1964 the Verrazano Bridge was opened. All these new harbor crossings could handle both commuter traffic and, more importantly to the float operations, freight traffic. As rail transport became too expensive or less efficient, goods were taken off the rails and simply put on trucks traversing the regions' vehicular river crossing. Photo #89 shows the New York City area harbor crossings.

Photo 89

The fifth cause of the decline in the usage of carfloating in New York Harbor is globalization and related containerization of freight across the world. The containerization system was first developed after WWII. It reduced transportation cost and removed the need for warehousing, which was a major function of the railroad terminals in New York Harbor. In the 1950's the current intermodal container form was developed and ships began to be constructed to ship the intermodal containers exclusively. The intermodal container allowed for the container to be transported by either rail or truck after it is unloaded at the port. Intermodal competition from truck lines played a large part in the decline of float operations. Port Newark-Elizabeth Marine terminal is the principal container ship facility in the northeast. It is the 22nd busiest container port in the world, 3rd busiest in the United States, and the busiest container port on the east coast. The terminal has nearly direct access to the rail system and interstate highways. Photo #90 shows standardized shipping containers at Port Newark-Elizabeth Marine terminal.

Photo 90

Consolidation of Railroad Marine & Lighterage in New York Harbor

In the face of lower demand and revenue on the rails as a whole throughout the country and within the harbor, the Tri-State Transportation Committee performed a study for consolidation of marine rail operation in New York Harbor published in 1964. The study determined that consolidation in the harbor was both feasible and would be beneficial on an economic and service level in the harbor. Until the economic reality of lower freight demand negatively impacted the individual railroads' bottom line, the railroads were content with operating their respective facilities independently. In 1964 the amount of railcars floated was 653,510. By 1973 it totaled between 50,000 – 55,000 railcars. The large decline from the 1960's to the 1970's is strongly correlated to the opening of the Verrazano Bridge in 1964 and the lower level addition to the George Washington Bridge in 1962. The main recommendations were the following (taken directly from the study).

1. All railroad marine operations in NY Harbor would be consolidated into a single integrated service. This included the operation and maintenance of tugboats, carfloats, scows, barges, and other marine equipment. It excluded passenger ferry service provided by the individual railroads.
2. Interchange carfloating for all harbor railroads would be consolidated on the west side and east side of the harbor. On the east side of the harbor all interchange carfloating would be consolidated to PRR facility at Greenville Yard in Jersey City. On the west side the majority of carfloating would be consolidated to the LIRR facility in Bay Ridge, Brooklyn.
3. Lighterage would be consolidated to the Jersey City waterfront instead of seven different handling facilities between Weehawken and Greenville (9 miles of the Hudson River waterfront).
4. The creation of a Railroad Marine Agency jointly owned by the harbor railroads and operated on a non-profit basis. This included leasing, maintaining, and operating a marine fleet and to lease and operate facilities for the handling of all Lighterage freight.

The twelve remaining operating railroads in the harbor at the time of the study were the Baltimore & Ohio Railroad (B&O), Brooklyn Eastern District Terminal Railroad (BEDT), Bush Terminal Railroad, Central Railroad of New Jersey (CRRNJ), Erie-Lackawanna Railroad (EL), Lehigh Valley Railroad (LV), Long Island Railroad (LIRR), New York Central Railroad (NYC), New York Dock Railway (NYD), New York, New Haven & Harford Railroad (NYNHH), Pennsylvania Railroad (PRR), and the Reading Company. At the time of the study the harbor railroads owned 73 interchange floats, 160 station floats, and 73 tugboats in total.

The proposed consolidation of marine operations is shown in the series of the following four maps showing before and after consolidation for interchange floating and station floating. Photo #91 and Photo #92 show the interchange floating prior to consolidation and after proposed consolidation respectively. Photo #93 and Photo #94 show the station carfloating prior to consolidation and after proposed consolidation respectively.

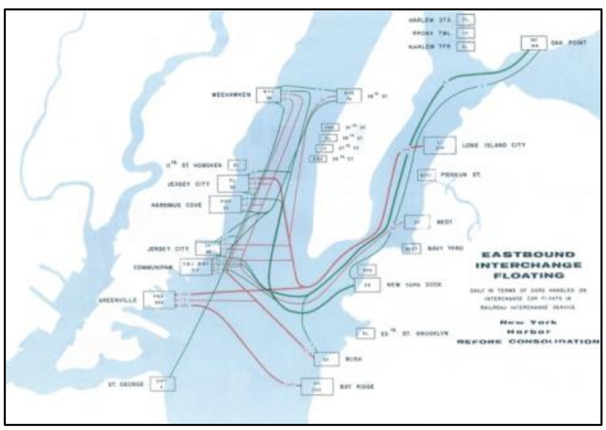

Photo 91

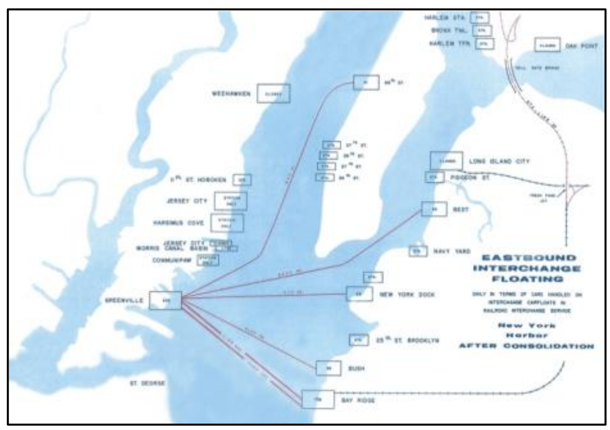

Photo 92

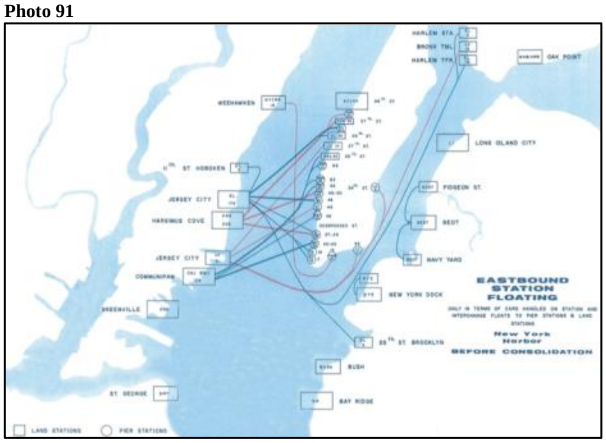

Photo 93

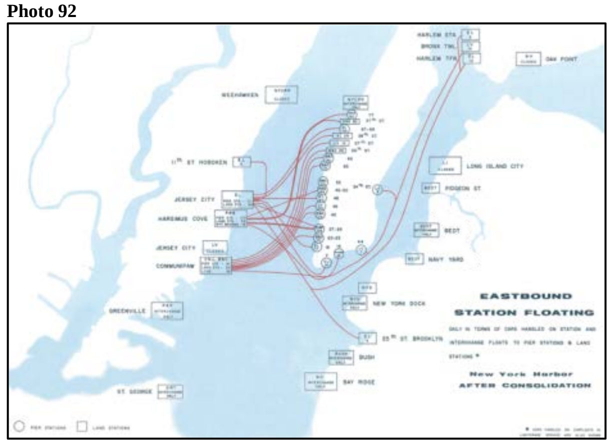

Photo 94

Consolidation of Railroads

Consolidation of the railroad industry began as revenues declines and the railroads sought means stay in the black. By 1957 the PRR and NYC began merger talks. The merger of the two biggest railroads servicing New York Harbor was finally completed in 1968. The merged company, The Penn Central Railroad, entered the merger profitable. In 1969 the NYNHH joined Penn Central. Then in 1971 Penn Central filed for bankruptcy protection and was kept afloat until 1974. The Regional Rail Reorganization Act of 1973 helped form the Consolidated Rail Corporation, known commonly as Conrail, to take over the regions freight lines. The freight lines of the Penn Central, Central Railroad of New Jersey, Lehigh Valley Railroad, Lehigh & Hudson Railroad, Erie Lackawanna Railroad, and Reading Railroad were consolidated to form a Conrail in 1976.

The Decline to Present Day

The demand for carfloating by the 1970's was waning. As new modes of transporting freight over the highway system drained the balance sheet of the harbor railroads, the use and cost effectiveness of carfloating declined. By 1973 there were three primary carfloat carriers in the harbor operated by the Penn Central, Lehigh Valley Railroad, and the Erie-Lackawanna Railroad. The Penn Central owned 2 float bridges, the Lehigh Valley owned 1 float bridge, and the Erie-Lackawanna owned 1 float bridge at the time, although the Lehigh Valley and Erie-Lackawanna utilized LIRR float bridges.

As the Lehigh Valley and Erie-Lackawanna carfloat operations stalled in the harbor and the Penn Central reorganized into Conrail, the last active carfloat operation was operating out of Greenville Yard. Conrail contracted the float operations to BEDT, which maintained float operations in the harbor until 1983. In 1983 the New York Cross Harbor Railroad (NYCH) was formed from the assets of BEDT and NYD. NYCH operated from Bush Terminal and Atlantic Terminal in Brooklyn to Greenville Yard. A further detailed explanation of these transactions can be found at *members.trainweb.com/bedt/indloco/nych.html*.

In 2006 the New York Cross Harbor ceased operations and changed ownership to New York New Jersey Rail, LLC (NYNJR). In 2008 the Port Authority of New York and New Jersey acquired NYNJR and is the current owner. NYNJR float operations now operated between Greenville Yard in Jersey City and Bush Terminal in Brooklyn.

The Abandoned Facilities & Land Reclamation

As the railroads left the waterfront in dire financial condition, most of the transfer bridges were abandoned. Most of the land was taken back by the local municipalities and grew into some of the most valuable real estate in America. In New Jersey, Weehawken, Hoboken, Jersey City, and Bayonne removed abandoned track and terminals and rezoned the land for commercial and residential use. A large portion of Liberty State Park is land acquired from the Central Railroad of New Jersey Terminal at Jersey City. Land from the Pennsylvania Railroad and Erie-Lackawanna Railroad at Exchange Place and Harsimus Cove turned into Harborside Financial Center. In Manhattan, most of the float bridges and piers were removed as the City filled the land to expand the island out into the river. Lower Manhattan, once a major center for receiving goods became the Financial District and the Battery. The former BEDT facility in Brooklyn is in the heart of a revitalized Williamsburg. Photo #95 shows the expansion of Manhattan Island from 1650-1980. The growth from 1965-1980 is the period of decline of freight traffic via carfloating in the harbor and you can see the growth went past piers. Photo #96 shows a view of the abandoned CRRNJ terminal in Jersey City in the 1970's, which would later become apart of Liberty State Park. Photo #97 shows the Pennsylvania Railroad elevated tracks to Exchange Place being removed at Grove Street in Jersey City. This was formerly known as Railroad Avenue but now known as Christopher Columbus Drive. Photo #98 shows the CRRNJ abandoned terminal facing east.

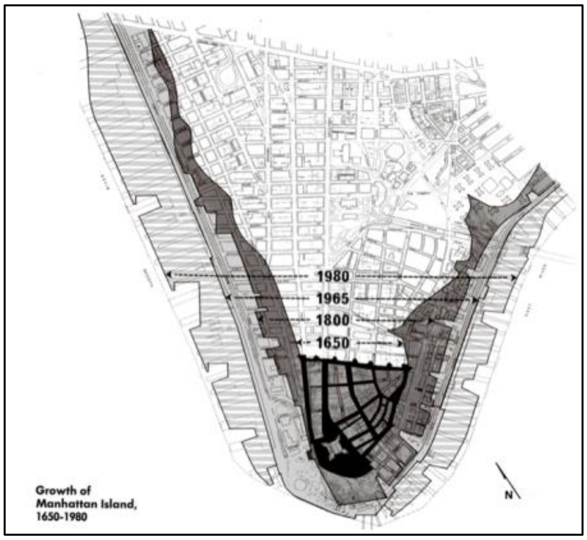

Photo 95

Photo 96

Photo 97

Photo 98

Pedestrian Park Conversions

Some of more inventive way to conserve the history transfer bridges was to convert them into pedestrian parks. Turning the bridges into parks allows the public access to the unique structures that were not possible during the operational heyday and unsafe during the abandonment of the structures. The best examples of pedestrian park conversion are in Manhattan and in Queens. The B&O pontoon float bridges at West 26th Street are preserved on the Hudson Waterfront downtown. The NYC French type is preserved at the former 69th Street float bridge facility on the Hudson Water Front uptown with plans for a pedestrian park. In Long Island City, Queens the LIRR Mallery type was turned into Gantry Plaza State Park. The preservation of the various types of transfer bridges in New York City allows for up close admiration of the technology used in New York Harbor to transport freight on carfloats. Photo #99 shows the old pontoon type transfer bridge Howe trusses at the pedestrian park at West 26th Street in Manhattan. Photo #100 and Photo #101 show the Mallery type transfer bridges at Gantry Plaza State Park in Long Island City, Queens. Photo #102 shows the planned pedestrian park around the French type transfer bridges at West 69th Street in Manhattan.

Photo 99

Photo 100

Photo 101

Photo 102

Abandoned & Deteriorating Facilities

At some locations in Brooklyn and Queens the abandoned float bridges remain in disrepair along the waterfront. Photo #103 shows the abandoned transfer bridges at Oak Point, which are no longer there. The Oak Point bridges were planned to upwards of twelve transfer bridges and were similar in type to the bridges at Greenville Yard. Photo #104 shows a sunken pontoon bridge in Brooklyn, formerly the Erie West 28th Street Terminal Float Bridge in July 2006. Photo #105 shows the abandoned Brooklyn Navy Yard float bridge in April 2012. Photo #106 shows the Fulton Terminal Pier Yard and south float bridge in August 2006. Photo #107 shows 50th Street Float Bridge "Bush 1" in June 2008. "Bush 2" at 51st Street is in the background and still in-service at that location then although now "Bush 2" resides at Greenville Yard in Jersey City. Photo #108 is a view of Bridge #10 at Greenville Yard in 2011. Photo #109 is a view at Greenville Yard in the bridge gantry in 2011. Photo #110 is a view at Greenville Yard looking between the bridge gantry and apron gantry after Super Storm Sandy in 2012 showing the apron gantry substantially leaning east.

Photo 103

Photo 104

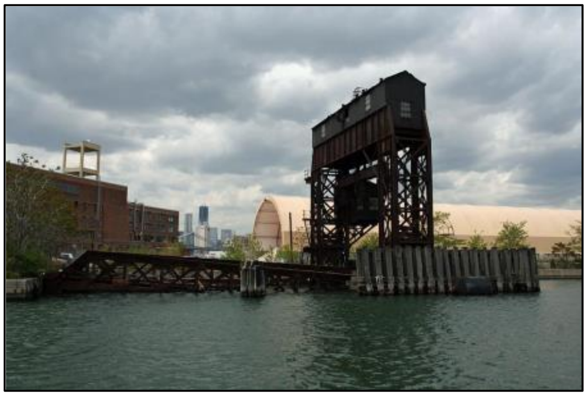

Photo 105

Photo 106

Photo 107

Photo 108

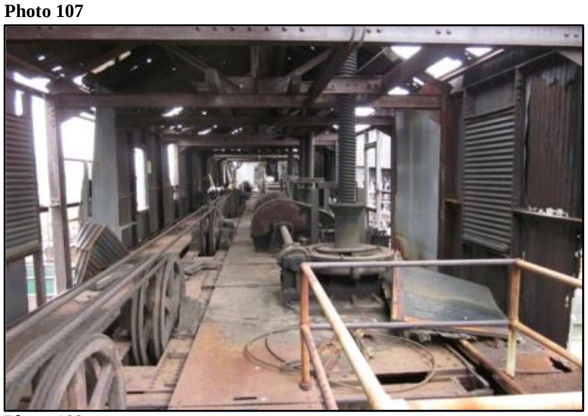

Photo 109

Photo 110

The Future

The Case to Increase Float Operations

In the beginnings of carfloating in New York Harbor the volume of freight transported across the harbor was enormous. Since the decline, the volume of freight floated across the harbor has declined over 99%. Within the decline in volume, the capacity of float operations across the harbor has also been reduced to one active float operation, floating freight from Greenville Yard in New Jersey to Bush Terminal in Brooklyn. The active operation has capacity to increase service with two transfer bridges at Bush Terminal and currently one transfer bridge at Greenville Yard, with a second forthcoming. The estimated current capacity is approximately 2,000 railcars per month equating to 24,000 per year. According to the Rail Freight Yard Requirements/Land Assessment study in 2003, the Greenville Float Yard had approximately 4,800 annual carloads utilizing only 20% of capacity. In addition to the capacity to handle the increase in rail, the opposite is true for expanding highway capacity. The region is the most densely populated areas in the country with almost no room to expand capacity of the highway system to the scale that is required now and in the future, with gains in capacity only realized by increasing the efficiency of the system.

Congestion & Future Freight Growth

In the New York Region the modal share is heavily skewed towards truck traffic. The breakdown of freight share is 80.7% by truck, 18.3% by water, 0.8% by rail, and 0.2% by air in the New York Region. Most of the region's freight is exchanged from commerce centers on the East Coast and the Midwest being handled at the port terminals in New Jersey. The majority of freight entering the region are unloaded onto trucks eventually crossing the George Washington Bridge and Verrazano Bridge with the most trafficked highways being I-95, I-80, and I-278. Figure #6 shows the modal freight split in the New York Metro Region.

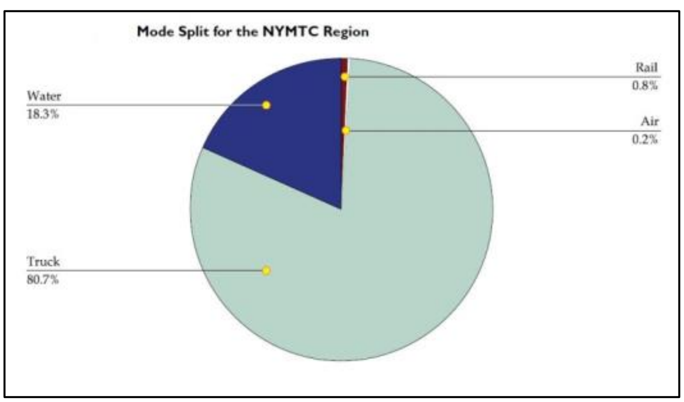

Figure 6

The large share of trucking in the region is due partly to the lack of rail access in the region. The freight ton-miles share on rail in the nine Northeast states is only 19%. In the American West and Mountain region the freight ton-mile share for rail is 64% and in the Mid-American region the freight ton-mile share for rail is 34%. In the New York region the share is less than 1%, forcing freight to be transported on trucks.

In the 15 year period from 1987-2002 truck traffic on the Interstate Highway System and local Highways increased 62%. Inbound freight to the region is expected to grow at an accelerated pace in the coming decades. In Nassau and Suffolk counties of Long Island alone received 56 million tons of freight in 2004, nearly all by truck. It is expected to rise to 98 million tons by 2030, an increase of 75%. The total amount of freight entering and leaving the entire New York region is expected to increase by 47% in the next 25 years. The US DOT estimates freight nationwide will increase from 19.3 billion tons in 2007 to 37.2 billion tons in 2035 an increase of 93%. Figure #7 shows the estimated freight growth in the New York Metro Region.

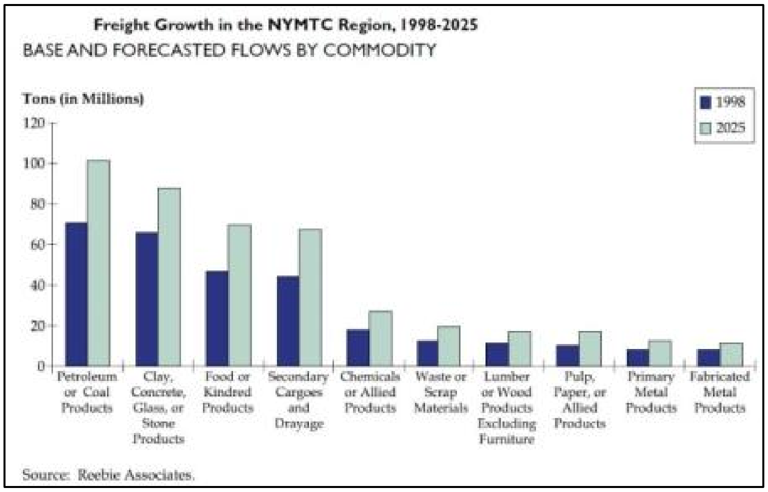

Figure 7

Benefits

The case to expand rail service in the region and across New York Harbor by float operations can be made on a societal and economical level. Most freight coming into the region comes via intermodal containers. The intermodal containers can be transported by truck as well as rail, which lend itself the ability to be transported across the harbor on carfloats. While Manhattan has no remaining active transfer bridge facilities, the remaining transfer facility at Bush Terminal in Brooklyn allows for the possibility of freight to be transported to service Brooklyn, Queens, and Long Island.

The socio-economic benefits of removing freight off the roads and onto carfloats are far reaching. Removing freight from the roads will reduce congestion and time spent in traffic by commuters, which in turn increase productivity of the workforce. In the decade from 1993-2003, the cost of highway congestion nationwide increased from $39.4 billion per year to $63.1 billion per year, an increase of 60% over that ten-year period. It is estimated that cost associated with congestion could be as high as $200 billion per year if productivity losses, cargo delays, and other associated impacts are taken into account. Reducing truck traffic also reduces vehicle emissions and energy usage. This will improve the air quality and better utilize fuel since rail transport and water transport is generally more efficient than trucking freight. The largest impact of removing freight from the roads and onto rail may be on DOT and agency budgets. The disproportionate amounts of heavy trucks on the highways, through the tunnels, and over the bridges, in the region, take a heavier toll on the infrastructure than typical lighter vehicles. Removing freight from the highway system and onto rail will extend the life of the highway infrastructure; reduce the maintenance costs associated with repair due to the heavy loads, and open up the roads for other commerce.

Figure #8 shows the relationship of growth of the lane miles (capacity) and vehicle miles traveled (demand/usage) over 25 years from 1980-2005. The figure shows how the capacity on the roadways has been relatively stagnant, increasing 5.7%, while the usage has increased by 96%. Without significant investments to expand capacity of the highway system in America and the expected growth in freight traffic, transporters of goods will be forced to seek alternative methods of shipping. Figure #9 show a similar relationship of track miles (capacity), ton-miles (demand), and ton-miles per mile of track (density). The figure shows how from 1980-2005 the amount of ton-miles on the system has increased 100% and the ton-miles per mile of track increased 200% while the mileage of tracks has been reduced by 40%. Combined, the two figures demonstrate the need to expanded capacity to keep up with demand.

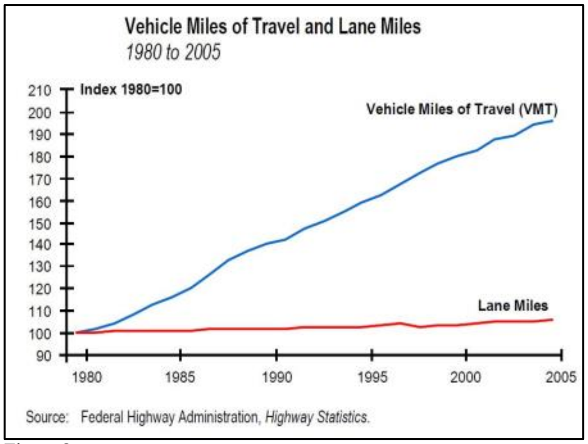

Figure 8

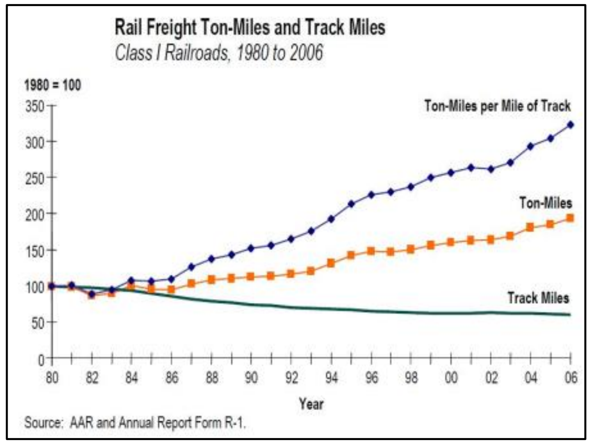

Figure 9

Along with freight traffic adding to congestion in the region, waste traffic from garbage also clogs the region's roadways. New York City sends thousands or garbage trucks per day to rail facilities in New Jersey to transport garbage to landfills across the country. The planned expansion at Greenville Yard includes plans to handle the waste with a new barge-to-rail operation with the capacity to handle 60,000-90,000 containers of solid waste from New York City and eliminating and estimated 360,000 garbage truck trips per year.

Capacity for the Future

Float operations in New York Harbor have the capacity, at a minimum, to reduce the amount of congestion at the historic bottlenecks of the bridges and tunnels in the region and help reduce the need to immediate capacity increases on the highway system to meet demand. While float operations may not solve the current and future congestion issues within the region alone, the carfloating of freight in the harbor can play a significant role in reducing congestion. With further investment of the rail network in Brooklyn, Queens, and Long Island, the rail system could further alleviate congestion on the Island by shedding truck traffic to the rails and open up the bridges and tunnels for the commuting public.

Photo #111 and Photo #112 show the former "Bush 2" pontoon bridge installed at Greenville Yard in December 2012. Photo #113 shows the pontoon bridge at Greenville Yard with a carfloat approaching in January 2013, less than 3 months after Super Storm Sandy damaged the facility. Photo #114 shows the transfer bridges at Bush Terminal at 65th Street in Brooklyn.

Photo 111

Photo 112

Photo 113

Photo 114

Thank you to the staff at New York New Jersey Rail and Port Authority of New York and New Jersey for sustaining float operations in New York Harbor and performing the monumental achievement of restoring operations after Super Storm Sandy. In addition to the owner operators, a thank you must go out to Philip Goldstein and his associates for maintaining and sharing to the world the history of carfloating in the harbor via their website, which I strongly recommend everyone visit at,

http://members.trainweb.com/bedt/indloco/developmenttransferbridge.html.

Railroads of the Harbor

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HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Unionport Bridge

Tight Constraints Create Opportunity for Innovative Replacement of a Double Leaf Bascule with Twin Single Leaf Bascules

Rahul P. Shah, PE
NYCDOT – Division of Bridges

William E. Nyman, PE
Hardesty & Hanover, LLC

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Introduction

The existing Unionport bridge is a double leaf bascule built in the 1950's over Westchester Creek, an industrial waterway in New York City. Originally there were a pair of double leaf bascules at the site, but the northerly span was removed in the late 1960's when elevated highways were built adjacent to both sides of the southerly span. At the time, the elevated highways diverted much of the traffic from the lower level bascule, but it continued to serve local city streets and traffic grew over time. Because of this, the remaining bascule proved too vital to traffic to take it out of service for major rehabilitation and the bridge continued to deteriorate over time. Alternate replacement schemes involving temporary movable bridges and complex work phasing also were eliminated from consideration due to cost and schedule implications, constructability concerns and because the final configuration would involve numerous machinery sets confined to tight work spaces making maintenance challenging.

An innovative phasing plan was developed which allowed twin single leaf bascules to be built in two major construction stages, while maintaining traffic on the existing bridge. The single leaf bascule built in the open position, offset from and behind the existing bascule will allow traffic to continue on a skewed alignment across the existing bridge as the new span is built. The resulting twin bascules will allow one span to be taken out of service if required for future maintenance or reconstruction while providing ample room for traffic on the remaining span.

This paper discusses the specific design challenges encountered and how the project moved past the conceptual phase into final design.

Existing Unionport Bridge

Background

Westchester Creek has been in use as an industrial waterway since the 1800's. However, there were limitations due to shallow water and shoaling prior to adopting a federal project at the waterway in the early 1900's. The Creek was originally part of Westchester County prior to the annexation of the area surrounding the creek into New York City in 1895. Westchester County built the first bridge at the site of the current Unionport Bridge. The original bridge was a swing bridge connecting local streets. This bridge has become so important to traffic in the area that by the time the area was annexed by the City a new bridge was needed. The second generation replacement bridge was a double leaf bascule completed in 1915 in coordination with channel improvements being made by the Army Corps. This bridge was built on the prior bridge alignment while maintaining traffic on a temporary swing span built to the north. The area to the east of Westchester Creek was less developed until the completion of the Bronx Whitestone Bridge in 1939 and the associated extension of the Hutchinson River Parkway. By this time the street passing over the Unionport Bridge had been renamed Bruckner Boulevard and had become a major thoroughfare in need of expansion.

The existing, third generation Unionport Bridge was constructed in the early 1950's following design plans created in the late 1940's. It consisted of two parallel, independently operable, double leaf trunnion bascules carrying the Bruckner Boulevard over the Westchester Creek in the Bronx. The longitudinal centerline of the north bascule leaves was 82 ft. to the north of the center line of the south bascule leaves. A portion of the bascule piers in the creek were constructed on foundations of an earlier bridge that had been built circa 1915.


,
Twin Double Leaf Bascules Prior to Construction of the Expressway

In 1971, the Cross Bronx and the Bruckner Expressway structures were built above the existing Bruckner Boulevard and that roadway was renamed the Bruckner Expressway Service Road. At that time the north bascule bridge superstructure and abutments were demolished to accommodate the interstate highway alignments. The approach roadways, ramp structures, and south bascule span were altered accordingly to accommodate two way traffic. The resulting condition placed a maze of elevated highways around one of the legacy double leaf bascules.

Project Location

In addition to maintenance over the years, several more significant repairs were made in the 1990’s. The mechanical and electrical systems and traffic control devices were rehabilitated following plans dated 1991. The bascule span open deck grating and grating support channels were all replaced by NYCDOT Bridge Maintenance in 1998. Initially, the plan was to do a major rehabilitation of the bridge. During the development of the rehabilitation plans, it became apparent that there was no practical way to maintain traffic during the construction. Temporary movable bridges were considered to allow the rehabilitation work but there were no good locations for these temporary movable bridges due to the columns supporting the adjoining state owned interchange structures. Further, the Coast Guard was not in favor of the two temporary movable bridges in close proximity along the waterway. In spite of the efforts to keep the existing bridge in service, it became apparent that replacement would be needed and traffic would have to be maintained during replacement.

Project Development

The main span of the existing Unionport Bridge is a double leaf bascule that carries three eastbound lanes and two westbound lanes of the Bruckner Expressway Service Road vehicular traffic over the Westchester Creek. The bascule is tightly constrained and is interconnected with numerous roadways. The west approach structure connects a westbound off-ramp designated Ramp A (BIN No. 1-06651-A) down to grade at Zerega Avenue which connects to the Cross Bronx westbound service road directly across Zerega Avenue. The westbound service road also connects to Ramp C which consists of two westbound Bruckner Expressway Service Road lanes that subsequently cross over Zerega Avenue (on a separate arterial bridge BIN No. 1-07613-0). Two eastbound lanes of the Bruckner Expressway Service Road originate from the eastbound Bruckner Expressway Service Road merging with the Cross Bronx service road (at Havermeyer Street) as they span over Zerega Avenue (on a separate arterial bridge BIN No. 1-07615-0) to form Ramp D and connect to the Unionport Bridge. An eastbound on-ramp from Zerega Avenue designated Ramp B (BIN No. 1-06651-B) also connects to the Unionport Bridge. The east approach carries two westbound lanes from the Bruckner Expressway Service Road, which receives traffic from Hutchinson River Parkway and two eastbound lanes of the Bruckner Expressway Service Road which feed traffic to the Hutchinson River Parkway and a right turn lane for Brush Avenue. Traffic on Brush Avenue has seen significant growth over recent years.

The bridge is flanked on the north and south sides by the high-level mainline structures of the Bruckner Interchange, which serves both the Bruckner and Cross Bronx Expressways, providing high speed connections to the Whitestone Bridge. The service roads crossing over the bascule span are relied upon as the means of direct access between the Hutchinson River Parkway and the two expressways, and also provide access to and from the local community. The bridge is the only crossing of Westchester Creek readily accessible to the local residential, industrial, and commercial areas on either side of the creek.

In 2006 and earlier, NYCDOT was considering the rehabilitation of the Unionport Bridge; however ongoing deterioration caused NYCDOT to re-evaluate rehabilitation measures. The main structural members of the bascule span framing were found to be severely deteriorated. NYCDOT then considered the replacement of the bascule span starting in 2007; however, it was determined that such a replacement would require the construction of at least two temporary movable bridges to accommodate traffic - this option was determined infeasible for a variety of reasons including right of way issues, utility conflicts, navigation needs, and cost. Finally, based on a 2010 feasibility study, NYCDOT determined that the entire existing bridge, including approaches and ramps, would be replaced with a new widened bridge that would eliminate or improve nonstandard geometric features, provide a longer life-span for the bridge, and benefit the community more than a rehabilitated bridge. The widened bridge would need to be built while maintaining two lanes of traffic in each direction on the bridge and one lane of traffic on Ramps A through D, throughout the construction duration.

Once the rehabilitation alternate was eliminated from consideration, NYCDOT reassessed the project goals to better define the direction of the project. In the course of this reassessment, Hardesty & Hanover was selected to develop plans for a replacement bridge. The information collected during the project’s long history was reviewed and validated. Care was taken to avoid sticking with prior decisions if they were not still valid based on current conditions. Based on this updated assessment, the project purpose and need was refined.

The primary purpose of this project is to improve the safety and serviceability of the bridge while maintaining traffic and improving traffic level of service. To this end the following project objectives have been identified:

1. Restore the structural integrity for at least a 75-year useful life in accordance with AASHTO LRFD including providing a bridge live load capacity of HL93;

2. Improve overall traffic conditions without adding roadway capacity by using cost effective methods to reduce delay and to provide an acceptable level of service, for a minimum design period of 30 years;

3. Address non-standard geometric features to improve traffic flow and facilitate traffic operations including providing three standard lanes and shoulders in each direction on the mainline bridge and a solid roadway deck in lieu of the current open grid deck;

4. Improve non-motorized access by implementation of a standard width sidewalk and a twelve foot wide combined pedestrian / bicycle facility;

5. Minimize life cycle costs by utilization of fill at the approaches in lieu of structure whenever possible;

6. Facilitate future rehabilitation work by use of twin movable spans, allowing one to be taken out of service for rehabilitation, while allowing for maintaining two-way traffic across the bridge on the other span during construction;

7. Maintain traffic across the bridge during construction including a minimum of two lanes in each direction on the mainline movable span and one lane on each of the ramps;

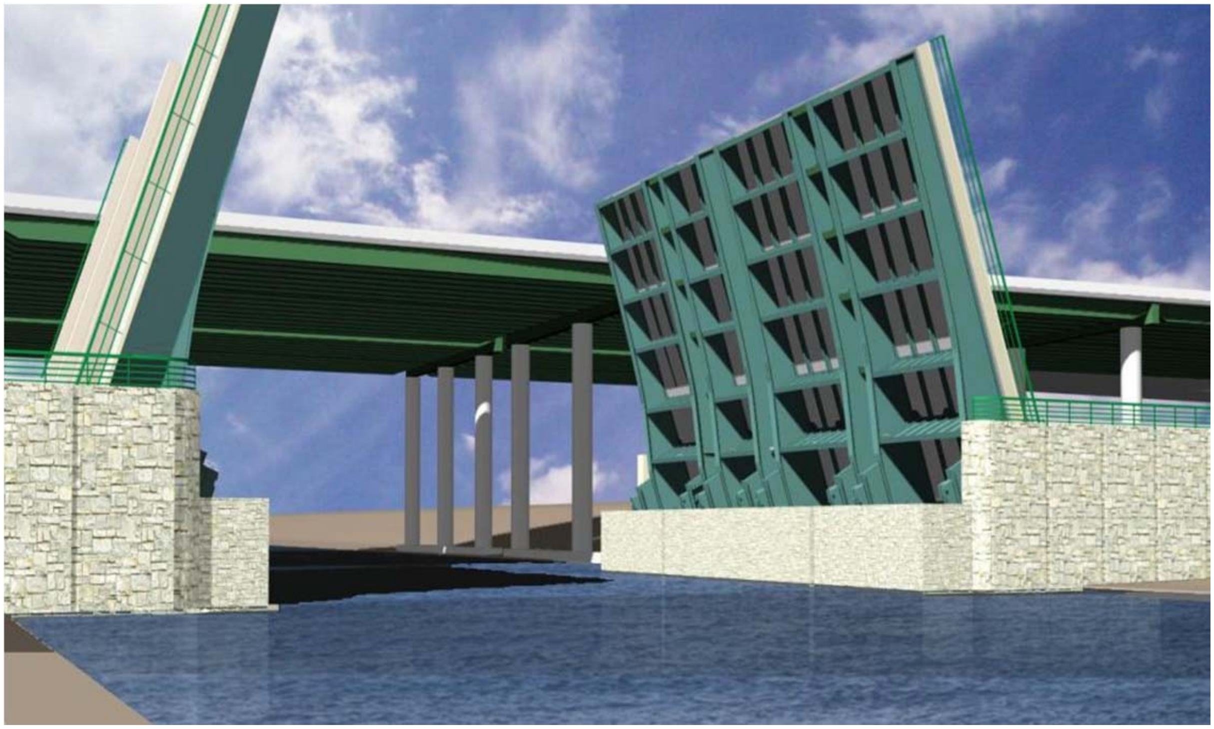
8. Maintain navigation in Westchester Creek by use of a new movable span with clearances matching the existing;

9. Provide seismic resistance to the bridge structure in accordance with the current AASHTO, State and City seismic requirements;

10. Provide the following ancillary elements: control house; maintenance storage area; utilities; machinery and electrical systems; traffic control equipment; fender system and dolphins; street lighting; sign structures; and traffic signals.

The proposed new Unionport Bridge project will include new widened bascule bridges, the addition of a bikeway, and the replacement of the associated on and off ramps. Due to overhead and lateral site constraints, other movable bridge types such as swing spans or vertical lift bridges were not deemed feasible.

A concept that was developed prior to Hardesty & Hanover’s involvement in the project included the staged construction of eight bascule leaves and later joining them to form a total of four bascule leaves in a twin double leaf arrangement. The bascules were proposed to be short rolling lift spans rather than trunnion type bascules. The thought was that the new bascule piers could be built between the existing bascule piers and the adjacent shoreline and be elevated above the channel bottom. This baseline concept needed to be reviewed and compared to other potential design solutions. The comparison of the baseline solution to an alternate twin single leaf bascule solution is described below:



Eight Leaf Bascule

Baseline Eight Leaf Bascule Concept:

The baseline concept consisted of multi-stage construction of short rolling lift spans around the existing bridge. Initially, narrow double leaf (rolling lift) bascules would be built north and south of the existing bridge utilizing the tight space between the existing bridge and the adjoining elevated state structure. This solution, although plausible, created a series of challenges including a tight work zone, complex maintenance of traffic, installation and span alignment concerns, and long term maintenance issues due to the tight maintenance workspaces available. These project challenges are further defined as follows:

1. Staged Construction

Staged construction is feasible for the double leaf bascule bridge alternative. The first stages involve building north and south of the existing bascule bridge while maintaining traffic on the existing structure. Traffic would then be moved to the new outboard structures while the existing bridge is demolished. The final portions of the bridge would then be built in a confined work zone between the new structures after the existing Unionport Bridge is demolished. This method allows for a bridge replacement while maintaining two lanes of traffic in each direction. The movable bridge cross section will have to be modified multiple times to adjust cantilevers, diaphragms and barrier locations. The building envelope in the vicinity of the Unionport Bridge is limited by the nearby overhead structures. A clear width of roughly 134 feet exists between the Bruckner Expressway to the north of the Unionport Bridge and the Cross Bronx Expressway to the south of the Unionport Bridge. Construction is not permitted close to the overhead structures' fascia, further limiting the available width. With the proposed staging and the overall cross sectional width of the proposed bridge, limited clearance exists in the proposed final condition. Tight construction areas, including construction areas in the middle of live traffic, and a long staging process will increase the cost of the bridge.

2. Bascule Span Alignment (Structural and Mechanical Components)

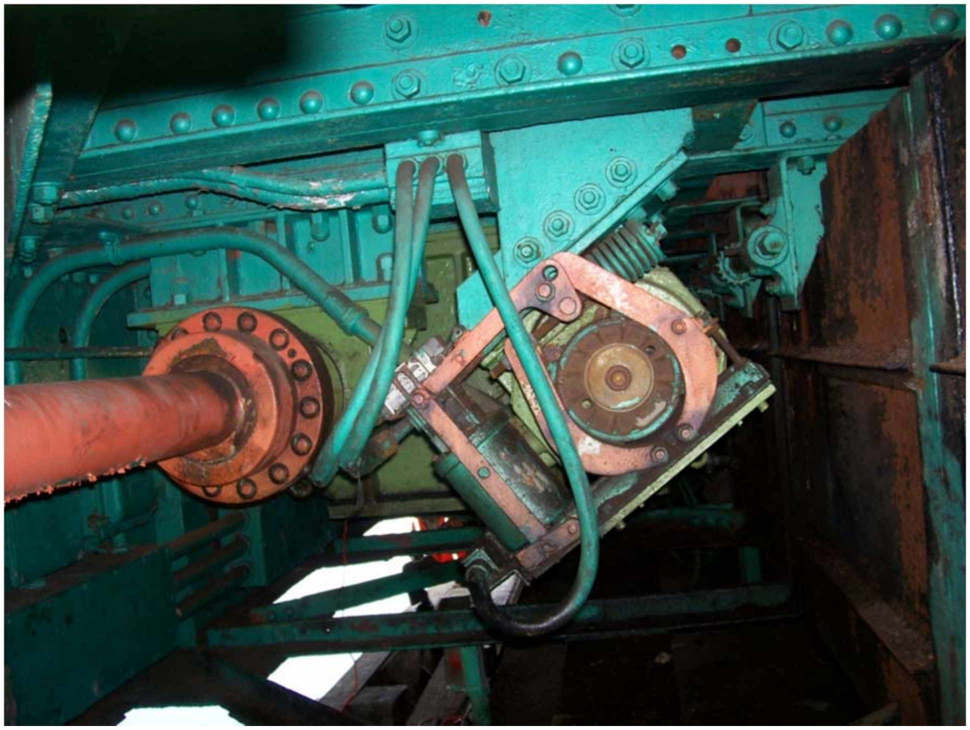
While the double leaf rolling lift type of bridge provides opportunity for shorter bascule leaves and a shallower substructure, this type of system inherently creates several disadvantages. A Scherzer Rolling Lift Bridge operates by simultaneously rotating about one axis while translating in the horizontal direction. The movable span translates on a curved, radial segmental steel plate with teeth rolling along a fixed, linear segmental steel track. Several structural and mechanical components must be aligned and multiple alignment iterations may be required over the life of the bridge. One inherent problem with a rolling lift bridge is its tendency to become misaligned over time. Unanticipated stresses may occur in both structural and mechanical components of each leaf if they are not aligned properly. This process is further complicated with multiple parallel moving leaves. During construction, the bridge will operate as eight separate leaves that will be joined together in the final condition to form four movable leaves. The leaves must be precisely aligned and tied together and the machinery must operate simultaneously creating additional alignment challenges.

Transverse restraints on a Scherzer Rolling Lift span are required for seismic detailing. The segmental track must be designed for these transverse loads. The pintles, or steel guides within the track system must be designed to allow for the required tolerance for span rotation and translation, while still providing restraint for the seismic loading. This structural detail can become large depending on the seismic load, adding additional cost to the segmental system.

3. Machinery Access and Maintenance

Additional machinery components are required to operate a double leaf bascule bridge as compared to a single leaf bridge. Operating machinery is located on each side of the navigable channel, used to rotate each leaf for opening and closing. With the proposed double leaf bascule bridge eight sets of operating machinery will be required to operate eight individual movable leaves during staged construction. During the final condition, when the bridge will act as four movable leaves the structural components and the machinery must be aligned in order for the machinery to function properly.

Regular maintenance is required for all of the operating machinery. On a double leaf bascule bridge maintenance involves having a crew work in two locations or for the Owner to provide two maintenance crews to perform the work simultaneously. Additionally, a double leaf bridge crossing a short channel such as the Unionport Bridge will have small bascule piers, only allowing for very small work areas and limited access to machinery. This access can be very cumbersome for inspection crews, with limited open areas between the gearing and operating shafts. The existing short span double leaf bascule also faces these challenging maintenance access issues. A Rolling Lift Bridge rotates and translates simultaneously requiring the operating machinery to be structurally connected to the moving span. Sometimes maintenance crews need to perform this work while the span is in motion. Since the bridge and machinery are both rotating and translating during span operation, special consideration to access should be addressed to allow for maintenance crews to stand on the moving span at all angles of opening.



Existing Unionport Bridge confined Operating Machinery spaces

A double leaf bascule bridge such as the Unionport Bridge is modeled as two cantilevered spans with a shear lock at the center of the two leaves. In order to facilitate the live load shear transfer between movable leaves at the pinned connection, a lock bar is driven from one leaf to the opposing leaf in the vicinity of each bascule girder when the spans are in the closed position. The lock machinery requires routine maintenance and inspection to ensure a tight clearance fit between the lock bar and the guide machinery. The lock machinery can loosen up over time. This results in overstressing of bridge components (main members and secondary members) along with additional impact forces and excess bouncing of the movable leaves. The bascule leaves will function as pure cantilevers if the lock machinery does not function as designed.

On double leaf bascule bridges, the lock machinery is located on the movable structure. For small double leaf bascules such as this, headroom below deck is very limited at the toe of the leaf making lock machinery maintenance at that location challenging.

4. Other issues

Channel Hydraulic Conditions: The existing Westchester Creek is shallow and subject to accumulation of contaminants and sediment. Keeping the channel open is important to allow flushing of the waterway and reduce accumulation of sediment. The piers for the double leaf bascule would be suspended above the channel bottom but would reduce the hydraulic cross section of the channel. This could pose permitting issues.

Sewer Outfall: A major sewer outfall lies below the existing Unionport Bridge. The underground sewer needs to be relocated and the outfall ideally will be placed north of the proposed Unionport Bridge and south of the footing of the overhead Bruckner Expressway. The sewer will not fit between the Bruckner Expressway footing and the proposed north bascule pier for the eight leaf bascule configuration. The sewer will need to be routed significantly farther away, significantly increasing project costs and agency coordination issues.

The staging, machinery alignment and maintenance concerns prompted investigation of alternate design solutions while the channel hydraulic concerns and sewer outfall posed major challenges which pushed for an alternate solution.

Alternative Twin Single Leaf Bascule

A twin single leaf bascule bridge can be used to replace the Unionport Bridge and meet the project objectives. Two parallel single leaf bascule spans can be used to facilitate staged construction while maintaining the use of the existing double leaf bridge. Each single leaf bascule will be a simple trunnion bascule span with a closed pit bascule pier. The southerly bascule pier is positioned behind and offset from the existing bridge to facilitate staged construction in the open position while maintaining traffic. The advantages & project challenges of this alternative are further described as below:

Twin Single Leaf Bascule

Advantages

1. Vertical and Horizontal Clearances

As with a double leaf bascule bridge, a single leaf bascule bridge could be designed to provide for unlimited vertical clearance within the navigable channel if this were required by USCG. The existing Unionport Bridge is located between two parallel high level fixed bridge structures which define the vertical clearance requirements of the channel at this location. A single leaf bascule bridge can be designed with approximately 45 degrees angle of opening to provide the required 52 foot minimum vertical clearance and match the clearance afforded by the adjoining NYSDOT fixed interstate structures. Limiting the angle of opening, allows for a single leaf bascule bridge with a smaller bascule pier. The size and depth of the bascule pier for the single leaf bridge will be comparable to that of a double leaf bascule bridge designed for the same location.

2. Bascule Span Alignment (Structural and Mechanical Components)

A single leaf bascule bridge requires fewer machinery alignment concerns as there are only half the number of moving components as compared to a double leaf bascule bridge. Two parallel single leaf bridges will require structural alignment of two moving leaves, as opposed to eight/four leaves for a double leaf bridge. The toe of the single leaf rests on a solid pier which reduces deflections and makes maintaining alignment easier.

For a simple trunnion arrangement span movement is limited to rotation about one axis using the trunnion shafts, as opposed to combined rotation and translation seen in a rolling lift span. Span operation is more reliable over the life of the bridge with respect to alignment concerns.

3. Machinery Access and Maintenance

A single leaf bascule bridge allows for all of the bridge operating machinery to be located in one centralized location on one side of the navigable channel, as compared to a double leaf bascule bridge which has machinery on both sides of the channel to operate multiple opposing leaves. Having one centralized location allows for simplified installation, only requiring half the amount of field alignment of gear reducers and shafts as compared to a double leaf. While the machinery is larger in size as compared to a double leaf spanning an equivalent channel, there are far less components to manage in the field during installation as well as throughout the design life of the structure and machinery, thereby reducing life costs.

Maintenance crews will only be required to work in one primary location. A single leaf bascule bridge will have a much larger machinery room as compared to double leaf span over a similar channel size, providing a larger work area for maintenance crews. With a simple trunnion bascule bridge the machinery does not rotate with the moving span, as it would with a rolling lift bridge, providing easier working conditions for maintenance crews. This also simplifies electrical connections and conduit routings and simplifies access for the Bridge Operator, since the Control/Operator’s House will be located on the same side of the channel as the machinery room.

A single leaf bascule bridge is modeled and performs as a simple span under live load; lock machinery will not be required to provide live load shear transfer between the moving leaf and the fixed pier. However, lock bars will be used to function as a safety “hold down” mechanism, providing for a redundant system to ensure safety in the event of operating machinery limit switch failure.

Maintenance access will also be improved in the vicinity of the lock bar machinery for a single leaf bridge. The lock machinery will be placed on the fixed rest pier with the lock bar driven to a receiving socket on the single leaf toe floorbeam. This will allow maintenance crews to access the lock machinery from the fixed span with the movable span in the opened or closed position. Access platforms will be situated outside the navigation channel with sufficient head room clearance to allow for simplified access for routine maintenance including inspection and lubrication of the lock actuator.

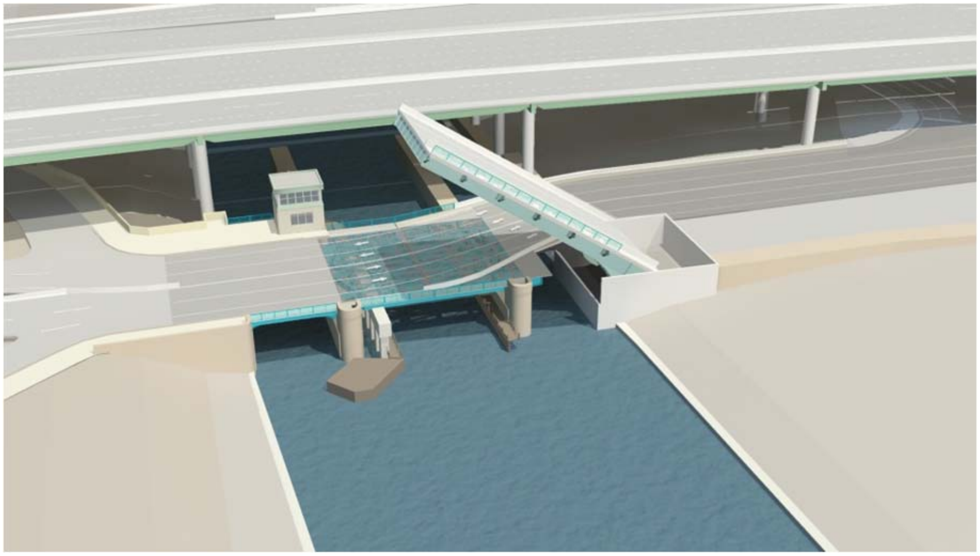
Project Challenges

1. Structural Geometry

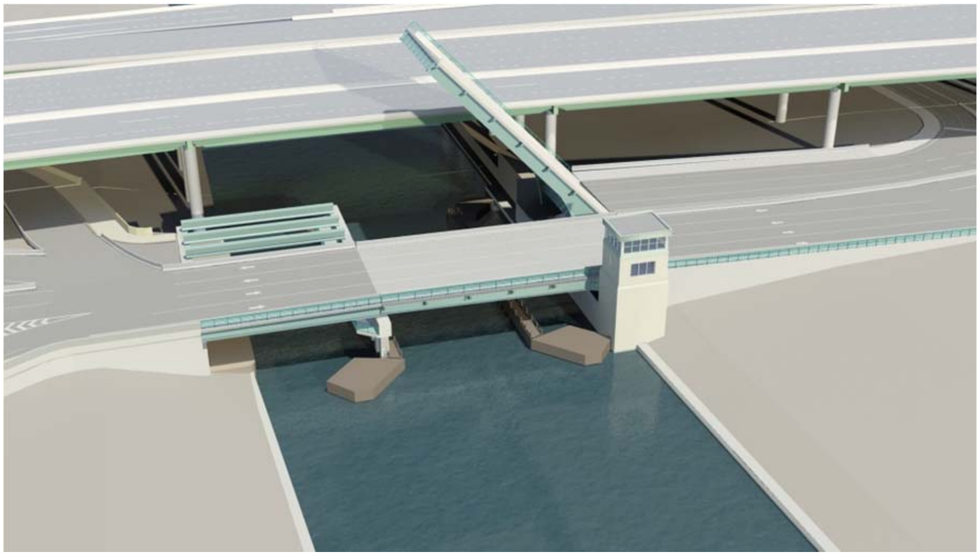
A single leaf bascule bridge requires a longer girder rear of the trunnion to balance the forward moment arm as compared to a double leaf bascule bridge crossing the same navigable channel width. Since the vertical profile of the bridge is not high enough to accommodate the longer girder while remaining above the water level, a closed pit bascule pier is required to allow for the longer girder to rotate in an enclosed area below the water level. The closed pit pier for this bridge would be located mostly on land and its size will be controlled by limiting the span opening angle and use of a smaller, higher-density counterweight.

2. Staged Construction

Staged construction is feasible for the twin single leaf bascule bridge alternative. Two main construction stages will occur. During the first stage traffic will remain on the existing structure and a new bascule bridge will be constructed on the south side of the existing bridge. The new south bascule leaf will be built in the open position such that marine traffic can continue to navigate along the Westchester Creek. As insufficient space exists to build a bridge capable of carrying four lanes of traffic without interfering with either the overhead state structure or the existing bridge, the new south leaf will be diagonally offset from the existing span to the greatest extent possible. Traffic will run across the existing bridge in a diagonally opposite direction. Some modification of the North East corner of the existing bridge is needed in order to make this happen. The new rest pier will be built in front of the existing rest pier to provide a place for the toe of the new leaf to land. This results in a temporary narrowing of the navigation channel. A critical step in construction is the shifting of traffic from the existing to the new bridge. During a short term roadway closure, the old movable span will be removed to make space for the new bascule to be lowered onto the rest pier and traffic to be shifted. The second stage would involve moving traffic to the newly built south bascule leaf, demolishing the remaining portions of the existing Unionport Bridge and constructing the new north bascule leaf.



Stage 1



Stage 2

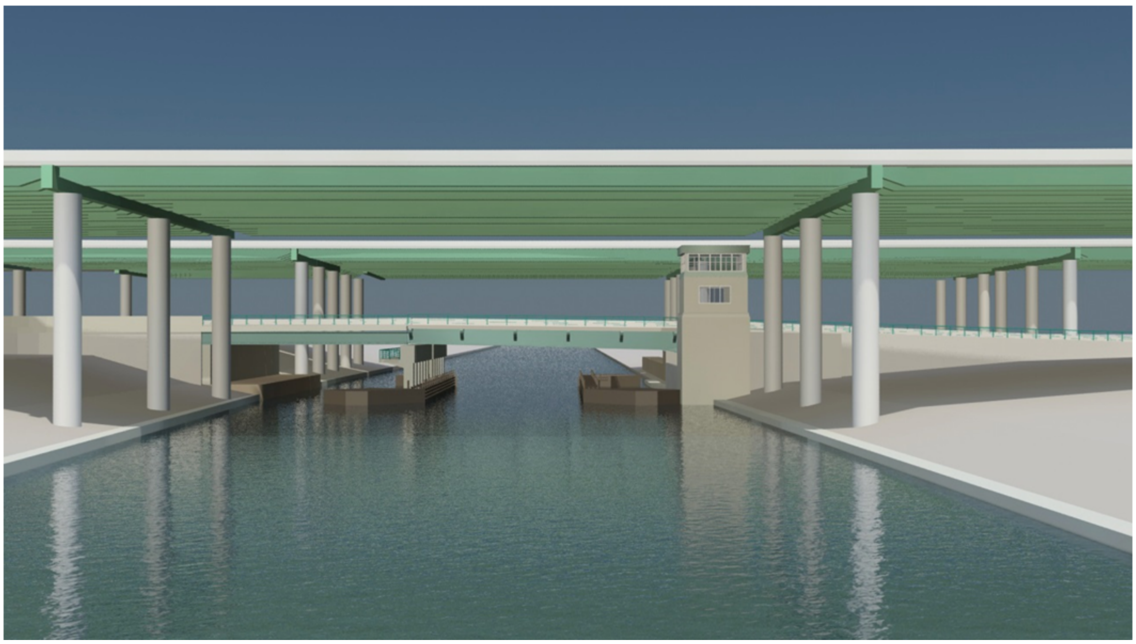
This option allows for marine traffic to be undisturbed during the majority of construction. However, special care will need to be taken while constructing the new bascule leaves in the open position. The bridge will need to be tied back and supported in the open position. Care will also need to be taken when installing the prefabricated deck system as concrete cannot be placed while the bascule span is in the open position. Such challenges add to the cost of this alternative.

3. Realigning Navigable Channel

Due to staged construction and necessary traffic maintenance in the area, Westchester Creek will require localized realignment at the bridge. The same 60 foot channel width will be maintained as it is currently, but the channel will be shifted roughly 10 feet to the east. A temporary channel width reduction is needed during specific construction stages.

4. Relocation of Sewer and Outfall

The storm sewer outfall at the pier at the east bulkhead line needs to be rerouted in order for the proposed bridge to be constructed. The storm sewer is a 10 foot wide by 8.5 foot deep reinforced concrete box sewer supported on piles. The new sewer outfall will be moved such that it discharges into the Westchester Creek just north of the proposed Bascule Pier and south of the existing state footings. The staggered pier locations inherent in the single leaf bascule staging conveniently create a space for the new outfall.



Proposed Twin Single Leaf Bascule

Conclusion

The existing Unionport Bridge is in need of replacement but is confined by tight site constraints. By thinking outside the box, innovative staging can be used to replace the existing double leaf bascule with a new twin single leaf bascule and achieve the project goals including maintaining both marine and vehicular traffic and constructing a new movable span which is easy to construct and maintain. This project is currently in Final Design and is expected to be bid for construction in 2016.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Construction Challenges During Heel Trunnion Replacement for an Historic Strauss Bascule

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Introduction

Purpose

This paper presents the challenges encountered and lessons learned during the process of replacing the heel trunnions for an historic Strauss bascule span located in Toronto, Ontario, Canada. The project was originally scheduled to be completed during a bridge closure from December, 2012 to June, 2013. There were numerous challenges related to unloading and removing the existing heel trunnions, completing structural repairs to the bascule truss, refurbishing the existing trunnion bearings, installing the new heel trunnions to maintain proper alignment and repairing the structural steel that supports the heel trunnions. As a result of these issues, the bridge was not re-opened until September, 2013.

The Ship Channel Bridge on Opening Day, 1930

History

The Cherry Street Bridge is a single leaf Strauss heel trunnion bascule bridge and Warren truss in Toronto, Ontario, Canada. Located in the industrial Port Lands area, it carries Cherry Street over the Toronto Harbour Ship Channel and opens to allow ships to access the channel and the turning basin beyond. The bridge was built in 1930 by the company of Joseph Strauss and the Dominion Bridge Company. The north side of the bridge has 750-ton concrete counterweights that allow the bridge to pivot to open.

Existing Conditions Prior to Repairs

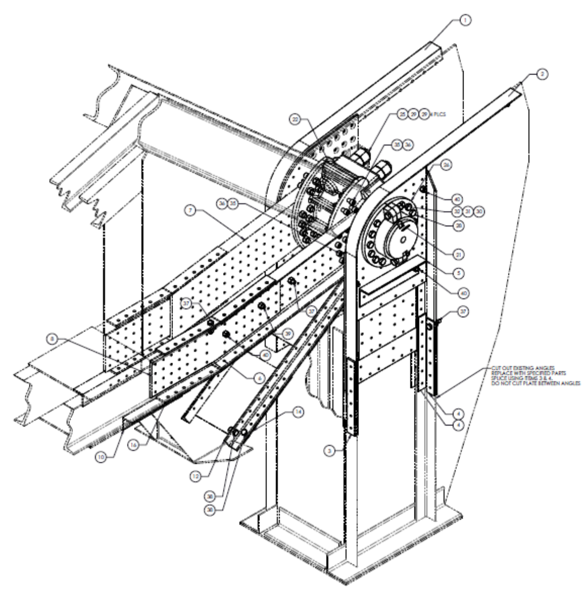
The bridge is designed to carry two lanes of traffic, and was designated under the Ontario Heritage Act in 1992 as architecturally historical.

In addition to the City using salt to de-ice the roads, there has been a salt depot on the south shore of the ship channel for 82 years. After trucks are filled with salt they drive over the bridge dropping some of their load and accelerating corrosion of the structure. In 2011, an inspection identified that the structural steel in the vicinity of the heel trunnions was heavily deteriorated to the extent that repairs were needed on an expedited basis. Plans were developed for repairs to the “shear plates” which support the heel trunnions and included a means of supporting the counterweights during the repairs. The project was tendered on May 15, 2012 and the project was awarded to Facca Inc. on September 27, 2012.

Scope of Work

The following summarizes the scope of the repair work described in the Plans:

- Supply and installation of temporary traffic controls and barriers to provide Single Lane traffic.
- North approach span structure and deck replacement. This included;
    - removal and storage of the existing railings
    - removal of asphalt paving
    - removal of existing concrete deck and curbs
    - removal of existing structural steel framing
    - partial demolition of the existing concrete piers as noted
    - driving of new steel H piles, new pile caps and piers
    - new structural steel framing
    - new reinforced concrete deck slab and curbs
    - new waterproofing
    - new asphalt paving
    - reinstallation of existing railings with new fastening system.
- Supply and installation of a Counterweight Support Structure, as per the Project Drawings to be used to safely support the weight of the counterweight (750 Tons). Installation to include penetration and excavation of the roadway above the pile caps, as shown on the drawings and supply and installation of additional materials to reinforce the pile caps.
- Supply and installation of temporary support members, as per the project drawings, to support and resist vertical loads at the North end of the bridge close to the trunnion.
- Supply and installation of temporary members, as per the drawings, to prevent all lateral movement of the bridge.
- Supply and installation of temporary members at the nose to prevent longitudinal movement of the bridge.
- Reinforcement of the leaf truss situated close to the trunnion.
- Excavation of concrete and asphalt, as per the drawings, on either side of the East Trunnion to provide access to the items to be replaced.
- Survey to establish the existing relationship of the Trunnion Bearing, Trunnions and Lower Cord Members.
    - Survey to include records of datum points noted indicating the existing location of structures and the installation of "Monuments" to be used as reference points once the work is completed to ensure the structure is in the correct location.
- Removal of the two (2) cast steel collars, including all bolts, from either side of the East Trunnion.
- Removal of the three ½" plates and bent angle iron from each side of the East



Repairs in the vicinity of the heel trunnion.

Trunnion including the careful removal of all associated 7/8” rivets, as per the drawings.
- Clean the remaining Shear Plates using powered wire brushes and grinders to achieve an SSPC-SP-3 surface finish.
- Removal of the top half of the Trunnion Bearing. Remove the top half of the bronze East Trunnion Bearing from this housing.
- Removal of the existing East Trunnion Pin.
- Complete another survey to confirm and record the existing layout of the trunnion bearing in relation to the span.
- Removal of the bronze, bottom half, of the East Trunnion Bearing.
- Removal and replacement of various elements of the truss in the vicinity of the trunnion bearing housing.
- In the shop, line bore the large Trunnion Pin Holes (2) in the new Trunnion Plates to fit the new Trunnion Pins and to match the hole dimensions and locations in the existing Gusset Plates.
- Supply and install the new bottom half, bronze, Trunnion Bearing into the Trunnion Housing.
- Supply and install the new Trunnion Pin ensuring adequate lubrication during fitting operation.
- Supply and install a new, bronze, top half Trunnion Bearing into the Top half of the Trunnion Housing.
- Install the top half of the Trunnion Bearing onto the Trunnion Bearing Housing. Supply and install shims as required to achieve the desired fit.
- Survey to ensure the new assembly matches the original location of the Trunnion Bearing and Trunnion Pin relative to the Lower Cord Members and the moveable span. Survey to be conducted using the previously install “Monuments”.
- If the bridge is out of alignment proceed to a temporary shut down and realign as per original survey.
- Apply rust resistant coatings to all exposed steelwork as per specifications.
- Complete the Replacement of the West Trunnion, West Trunnion Pin and West Trunnion Bearing and structural repairs same as above directly after the work on the East Side is completed.
- Remove all temporary bracing from the bridge.
- Remove the Counterweight Support Structure from the North Side of the bridge. Store as directed by the TPA.
- Repair approach span deck where removed to provide access to the heel trunnion work.
- Remove temporary traffic controls and barriers and Commission the bridge.

Planning Phase

Constructability Review

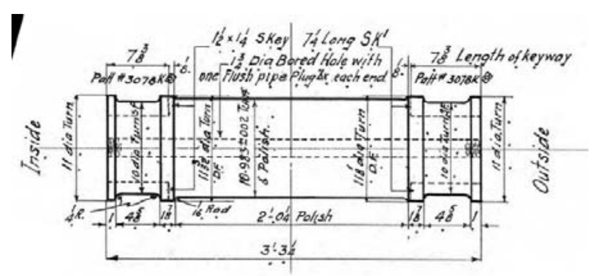
In October, 2012 Facca contracted with Stafford Bandlow Engineering (SBE) to provide limited engineering assistance for the project. The initial task for SBE was to review the Plans and specifically the requirements therein to perform the trunnion replacement one at a time. Facca was concerned about the construction schedule and believed that there were significant benefits to performing the work at both trunnions simultaneously. As part of the effort to understand why the work was being limited to one location at a time, it was necessary to understand the full sequence of the repair work. As such SBE suggested performing a cursory constructability review with the goal of preparing RFI's that would address Facca's concerns about the sequencing. Subsequent to that review the following comments were provided:

1. The scheme for supporting the counterweight relied on deflecting the jacking frame to introduce the necessary load.This did not account for the need to raise the counterweight a significant amount in order to unload the balance links.

2. There was a passive blocking system below the heel trunnions intended to accept the load of the truss as the counterweight was supported. There was no means of measuring the load at the support to confirm load transfer and no provision to jack the truss if the trunnion was still loaded.

3. The toe end of the span was designated to be held in all directions so that while the main trunnion assembly was being replaced the heel end of the truss would become the expansion end. There was a concern that alignment would be altered as a consequence.

4. Most significantly, the sequence of work related to the heel trunnion replacement was not practical due to the fact that the trunnion pin was stepped and installed with driving fits into the fixed structure. As such, it was not possible to remove or reinstall the pin axially with the bottom bushing in place. Even if the bushing was removed, it was likely that the forces required to drive the trunnion out of the shear plates would result in damage to the heavily corroded steel that supported the trunnions.

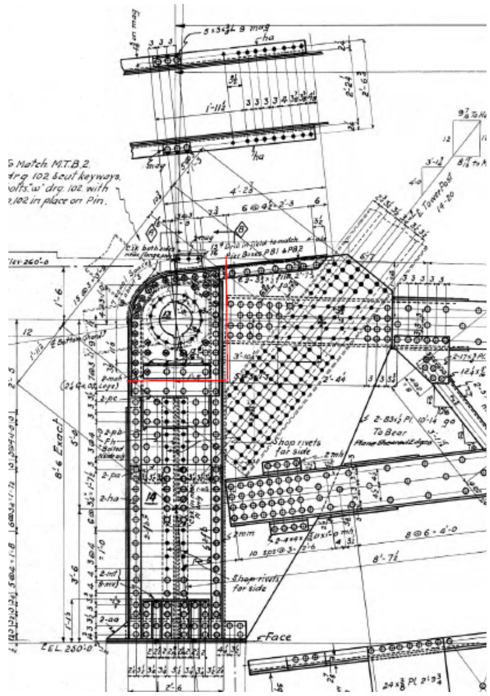


Heel trunnion details.

The Engineer found the constructability review comments to be valid and a series of conference calls were held to collaborate on solutions to each point.

Revised Sequence of Work and Redesign of Shear Plate Repair

Several alternatives were explored to overcome the issue of removing and replacing the stepped trunnion pin. Ultimately, it was determined that the preferred course of action for all sides was to cut out a portion of the shear plate and weld a new plate in to restore this area. This solved two key problems: The risk of damage to the existing shear plate when the existing trunnion was driven out and the challenge of removing the bottom bushing half prior to trunnion removal and installation of the bottom bushing half after reinstallation of the new trunnion. It was agreed that the work could proceed at both trunnions simultaneously. The new sequence of work was as follows:



Proposed cut line for shear plate repair.

1. Jack counterweight and unload heel trunnions.
2. Remove angles and plates from shear plate.
3. Remove trunnion bearing caps.
4. Cut shear plates and remove trunnions.
5. Implement repairs to the truss.
6. Refurbish trunnion bearings, install new bushings.
7. Install new trunnions. Install bearing caps with temporary shims to reduce bearing clearance to maintain alignment of the trunnions.
8. Heat new shear plates and install onto trunnions with FN2 interference fit.
9. Weld new shear plate to existing.
10. Install new angles and plates reinforcing the shear plates.
11. Install new trunnion collars.
12. De-jack counterweight and commission leaf.

One alternative scheme for implementing the repairs that was rejected was to incorporate a field line boring operation into the sequence of work. Although there was a general agreement regarding the benefits of line boring the bushings and shear plates, it was felt that the cost and schedule impacts were too great.

Revised Counterweight Jacking and Span Support Scheme

SBE prepared calculations and determined that in order to unload the balance links and thereby eliminate any horizontal forces on the leaf it would be necessary to jack the counterweight several inches. Facca developed a new design for the counterweight jacking frame that included provisions for installation of jacks with locking collars to raise the counterweight and secure it in position.

A scheme was also developed to modify the pier allowing for installation of low profile jacks with locking collars below the truss at the connection to the floorbeam adjacent to the heel trunnions. This provided the necessary means to raise and lower the truss to ensure that the heel trunnion was unloaded prior to removal of the trunnion bearing caps.

Expansion bearings were designed to be installed at the toe end of the bridge to support the dead load of the truss without generating forces at the heel end of the bridge due to thermal expansion.

Construction Phase

Counterweight Jacking & Unloading Heel Trunnions

Counterweight jacking frame.

The Contract stipulated that during counterweight jacking the Engineer would monitor loads in the jacking frame and the bridge structure with strain gages and provide information to the Contractor as to when the balance links were unloaded. During the initial two attempts to jack the counterweight, it was found that these measurements were inconclusive. Facca requested SBE to provide a procedure and field assistance with determining when the counterweights were fully supported.

The method recommended by SBE involved monitoring the balance link pin connections for movement. With the counterweight unsupported, the balance links have a tensile load producing an uplift at the lower balance link pin mounted on the truss. Since the balance link pin bearings are plain bearings, it should be possible to observe the movement of the link down within the limits of the clearance in the lower link pin bearings.

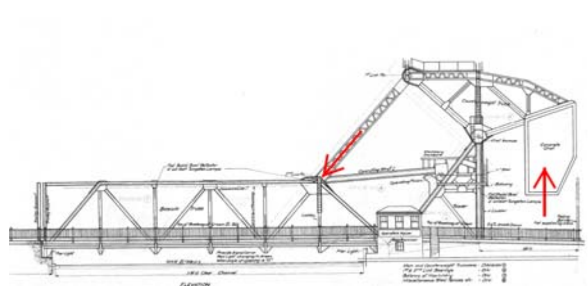


Illustration of counterweight and balance link movement during jacking operation.

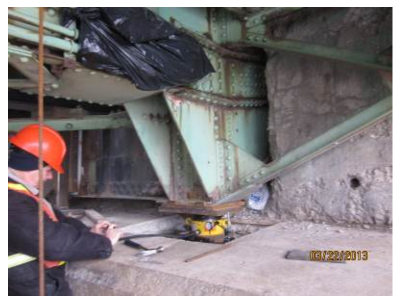
Since it was critical to eliminate all horizontal loads from the balance link to the truss it was crucial that it be confirmed that the balance link was not in tension or compression. Once the balance link was unloaded, if the counterweight was jacked further movement at the upper pin should become apparent before the balance link picked up a compressive load.

Dial indicators were secured in place with magnetic bases and the movement of the link pins was observed as the jacking pressures were increased and the jacks extended to raise the counterweight. Facca recorded jack pressures, jack travel and monitored the elevation of the counterweight. With the manifolds and valving provided it was possible to raise each counterweight independenty in the event that loads were not equal. It was confirmed that the jacking pressures increased in a fairly linear manner as the counterweight was raised with no movement at the dial indicators, indicating that the tension was being released from the balance link. Once movement was confirmed at one lower balance link bearing, that side was locked off until the other side "caught up" which ocurred with only a small amount of additional travel. Both counterweights were then raised in small increments. It was confirmed that there was a negligible change in the jacking pressures indicating that the load in the balance link was no longer changing. The


Monitoring balance link movement.

counterweights were raised until movement was observed at the upper link bearing. Each counterweight was then lowered to a mid point between where movement occurred at the lower and upper link pins. The jack stroke, counterweight elevation, jack pressure and link pin movement data was reviewed with the Engineer on site to gain concurrence that the balance link was unloaded.

During the counterweight jacking procedure the truss was not supported allowing the heel end to settle and be supported by the trunnion bearing with the trunnion at top dead center in the bearing. Once the balance link was unloaded, jacks under each truss at the intersection with the last floorbeam were used to raise the truss. A similar procedure was used: Facca recorded jack pressures, jack travel and monitored the movement of the truss with dial indicators. With the manifolds and valving provided it was possible to raise each jack independenty in the event that loads were not equal. It was confirmed that the jacking pressures increased in a fairly linear manner with no movement at the dial indicators, indicating that the load was being released from the trunnion. Once movement was confirmed at one trunnion bearing, that side was locked off until the other side "caught up" which ocurred with only a small amount of additional travel. Both trusses were then raised in small increments. It was confirmed that there was a negligible change in the jacking pressures indicating that the truss was moving within the limits of clearance in the trunnion bearing. The jacks were raised in small increments until movement stopped and load began to pick up on the jacks indicating that the limit of bearing clearance was reached. Each jack was then lowered to a mid point between the limits of movement. The jack pressure and movement data was reviewed with the Engineer on site to gain concurrence that the trunnions were unloaded. .


Monitoring truss movement during jacking.

Baseline Surveys

Once the heel trunnion pins were successfully unloaded and the asphalt and concrete adjacent to the heel trunnions were removed, an initial baseline survey was conducted by Facca. The baseline survey established the existing relationship of the trunnion pin to fixed reference monuments located on the adjacent concrete structure to an accuracy of 0.008" in three dimensions. The survey technique used to establish this relationship was a traditional piano wire survey method.

The existing trunnion pins were severely corroded adding a degree of complexity in maintaining the survey accuracy desired. To overcome this issue, the bores through the existing trunnion pins were honed and a precision-machined alignment shaft was fabricated and inserted in the bore. Piano wire was strung and centered in the bore of the alignment shaft, and thus the trunnion pin, and measurements were taken and recorded from the piano wire to fixed reference monuments located on the adjacent concrete structure establishing the baseline data for the location of the trunnion pins. The alignment shaft provided a machined surface to take measurements from


Piano wire alignment measurements, existing trunnion.

mitigating the risk of an inaccurate survey due to severe corrosion. A separate piano wire survey was completed for each trunnion as there was no direct line of sight between the east and west heel trunnions. This initially posed a concern since the orientation of the trunnion pins relative to one another and to the existing structure could not be established based on the piano wire survey method.

In order to develop a relationship between the east and west heel trunnion pins, a 3D scan was carried out using a laser tracking survey instrument. The laser tracker survey would establish the location and orientation of the existing trunnion pins relative to one another and relative to the existing structure. Six benchmarks were mounted at various points on the steel structure, 4 on the fixed span and 2 on the movable span which were used as control points. This was not the preferred location to mount control points, as thermal expansion of the steel structure would alter the location of the control points reducing the accuracy and repeatability of the survey. Ideally, the control points should have been mounted to the concrete structure reducing this error. The data provided from the laser tracker survey provided valuable information on the orientation of the trunnion pins relative to one another and to the existing structure. The trunnion pins were found to be considerably out of coaxiality from one another. It was hypothesized that this was partly due to a previously known ship impact with the structure.

Laser Tracker.

Existing Condition of Structural Steel

The exposed structural steel in the vicinity of the heel trunnions was overall in very poor condition due to areas of severe corrosion and section loss. The east side of the span exhibited an increased amount of corrosion compared to the west side of the span. This was mainly due to road salt being continually spilled while transported over the bridge on the east side of the span resulting in an increased amount of corrosion.

Existing conditions at inboard end of trunnions.

The exposed bearing components, i.e. the ends of the trunnion pins and collar assemblies, were also severely deteriorated. Since the heel trunnions are located below the bridge deck elevation, road salt and other debris repeatedly covered the exposed surfaces of the heel trunnions and structural steel advancing the corrosion in that vicinity. As a result, the corrosion was much worse on the inboard sides of the heel trunnions compared to the outboard sides.

The trusses on the movable span in the vicinity of the trunnion bearing housing were also in very poor condition due to corrosion. Approximately 35-40% of each gusset plate at both the east and west heel trunnions had severe areas of deterioration and section loss that needed to be addressed prior to the truss being reinforced. Also, several large cracks in the main gusset plate at the east trunnion bearing connection location were discovered that required repairs.


Gusset plate corrosion.


Gusset plate crack.

Bearing Disassembly & Structural Steel Removals

The methodology for disassembly of the trunnion bearings changed significantly from the original scope of work due to the stepped trunnion pin. In a collaborative effort between all parties a new removal sequence was generated to overcome this issue.

The revised removal sequence included the following tasks:

- Removal of the two cast steel collars from the inboard and outboard ends of the trunnion pin
- Removal of the trunnion bearing cap, upper half bronze bushing and trunnion shims
- Cut a portion of the main shear plates at the trunnion pin connection
- Removal of the trunnion pin
- Removals of the remaining shear plates and angles by removing all associated rivets
- Removal of the lower half bronze bushing


Cutting shear plates to remove existing trunnions.

During the trunnion pin removal process, dial indicators were positioned to monitor the north-south (longitudinal) movement of the movable span relative to the fixed span while the shear plates were cut. Upon completion of this task, the movable span shifted towards the toe of the bridge 0.001” at the west trunnion and 0.008” at the east trunnion. The results of this survey were immediately provided to the Engineer. In hindsight, dial indicators should have also been positioned to monitor the east-west (transverse) and vertical movement, but this was not implemented prior to cutting the shear plates.

As disassembly progressed, numerous additional structural steel components that formed the trunnion tower assembly that were not included under the original scope of work were required to be removed since they interfered with the removal and installation of the bearing caps and shear plate components that were scheduled to be replaced. These additional removals resulted in multiple RFIs and new drawings being issued by the Engineer to account for these unanticipated repairs.

Truss Repairs

Worked continued with removals of various structural steel elements of the truss in the vicinity of the trunnion bearing housing. Again, numerous additional structural steel components not accounted for the original scope of work were required to be removed due to interferences with removals of the truss components.

Once all removals were complete, a more thorough inspection of the condition of each gusset plate on the movable span was conducted by the Engineer and a repair procedure was developed to repair the areas of severe deterioration and section loss. All section loss repairs to the gusset plates were an addition to the overall scope of work. The repair procedure included removing and replacing approximately 35-40% of each gusset plate by means of welding new sections of plate to the existing gusset plate. All welds were prequalified full penetration groove welds and were subject to 100% visual and magnetic particle examination.

The existing cracks through the shear plates at the bearing connection location were were found to have been previously repaired by means of welding. Due to the location of these cracks and welds, all parties were concerned that the bearing housing was potentially compromised. A magnetic particle examination of the bearing housing was completed revealing that the cracks did not propagate into the bearing housing thereby alleviating this concern.


Gusset plate corrosion before repairs.


Gusset plate after repairs.

The weld repairs proved to be a very challenging and time consuming task to complete due to the confined working environments and poor condition of the existing structural steel at the welded joints. Facca was also significantly concerned with the possibility of severely distorting the existing gusset plate due to the large quantity of weld, which would ultimately effect the overall alignment of the bearings. Facca’s welding engineer put strict precautions in place to minimize heat distortion of the gusset plates. These precautions included controlling the heat input, the use of small stingers, and maintaining the interpass temperature at the minimum allowed per the WPS.

These unanticipated repairs had a significant effect on the overall progress of the project and were an addition to the overall scope of work. The additional truss repairs took in excess of 4 weeks to complete.

Intermediate Survey

In late June, A post-disassembly piano wire survey was completed with new bearing components installed once all removals and truss repairs were complete. The piano wire was re-aligned to the previously established monuments based on measurements taken from the baseline piano wire survey and measurements were taken and recorded from the piano wire to the bore of the new trunnion pin.

The intention of this survey was to establish the current location of the new trunnion pins in relation to the original position with all removals and truss weld repairs complete. The results of this survey indicated that the east and west bearing housings were misaligned on the order of 0.237” at the worst location indicating significant movement of the bearing housing. This information provided the Engineer an indication on the magnitude of misalignment of the east and west bearing housings in relation to the original position.

The initial response from the Engineer was that there must be some error in the survey data as it did not seem plausible that the alignment of the bearing housings could have changed to such a degree. This was largely due to the fact that without a line of sight for a conventional piano wire survey it was necessary to rely upon the 3D survey data and “trust” the accuracy of the instrument.

Survey Interpretation & Validation

The skepticism toward the survey results led to another round of re-surveying using the same method. The Contractor and Engineer exchanged a series of letters back and forth and held several meetings attempting to reach concurrence as to the validity and significance of the survey results with little progress over the course of several weeks. Facca engaged SBE to assist in the interpretation of the survey data and validate the data using independent means where possible. The following work was performed:

1. A complete review of all survey data was performed with the goal of identifying the repeatability of the benchmark measurements and the accuracy of the measurements of “known” features such as the length of the new trunnions. Some errors were identified in the compilation of the data by the surveyor. Once these errors were corrected it was confirmed that the instrument appeared to be highly accurate and repeatable at measuring benchmarks and known features.
2. The laser tracker survey was repeated with an independent firm. This survey further corroborated the accuracy of the instrument at measuring benchmarks and known features but cast further doubt on the accuracy of the trunnion location due to significant variations from the prior survey, particularly in terms of the vertical location data.
3. A series of traditional optical survey measurements were performed using a theodolite and a K&E level. There was a high degree of confidence between SBE and Facca in the vertical location data obtained with the K&E level. Correlating the K&E level data to the 3D tracker data yielded further inconsistencies.
4. The piano wires were re-installed, set to their original locations relative to permanent benchmarks and alignment measurements were performed. To test the theory that there was ongoing movement at the trunnion bearings, several measurements were repeated over the course of a day. It was confirmed that for a 6°C change in temperature there was movement on the order of 1/32”.

With the discovery that there was significant thermal movement over the course of a day it was now possible to gain concurrence from the Engineer that all of the various survey efforts were in fact valid and accurate but that the measurements were taken of a moving target. The Engineer now accepted that through the course of the work the alignment of the trunnion bearings had become altered to an unacceptable degree, either as the result of pre-existing conditions from a ship impact or as a result of the weld repairs to the truss. Critically, it was also determined that there was no evidence that the piano wire location had changed through the course of the alignment work. Since this was the most expeditious means of checking for changes in alignment it was agreed to heavily rely on the piano wire for the remaining re-alignment activities. Unfortunately, nearly a full month elapsed between reporting the trunnion bearing alignment and accepting the measurements as fact and beginning to move forward on solutions to completing the project. It was now mid-July.

Trunnion Bearing Housing Alignment Corrections

As the schedule pressure was increasing for all parties, Facca and SBE agreed that the best path forward was for SBE to provide field assistance until the trunnion alignment was accepted and the reassembly process could proceed. The primary objective regarding trunnion bearing housing alignment was to achieve co-axial alignment of the shafts so that as the leaf opened the shaft to bushing alignment would not change and result in binding as this would create excessive friction and additional loading of the structure. The initial suggestion put forward by the Engineer was to leave the alignment of the bearing housings as-is and increase the bore diameter to provide additional clearance thereby mitigating the binding concerns. SBE and Facca discussed and agreed that there was too much risk resulting from this approach due to the amount of misalignment present, the amount of clearance required and a lack of understanding regarding how the bearing would perform with that degree of clearance and changing misalignment. In response a plan was developed to make improvements to the alignment as follows:

1. The vertical misalignment (shaft out of level) would be improved by jacking one end of the trunnion to “roll” the bearing housing and reduce the cross slope.
2. The horizontal misalignment would be improved by removing material from the bearing liner thus producing an “egg shaped” bore that was undersized in the horizontal direction. It would then be possible to set up a boring bar with the correct horizontal orientation and field machine the bore to “true it up.”
3. Through the course of reviewing the alignment data it was also determined that the axial spacing of the bearing housings was altered and it would be necessary to remove material from the thrust face of one bushing to eliminate interference.
4. It was agreed to increase the bushing bore diameter to take the bearing clearance into the range of an ANSI RC9 fit as it was expected that some misalignment would remain.

Vertical/Cross Slope Alignment Corrections:

Approval was granted to perform a series of tests to determine if it was possible to alter the trunnion vertical alignment. Although the initial results were not promising, it was found that by disassembling the bolted diaphragms and stiffeners restraining the bearing, it was possible to make significant improvements to the cross slope of the shaft. More importantly, it was confirmed that when the bolted diaphragms and stiffeners were

Jacking the trunnion to “roll” the bearing housing and correct cross slope.

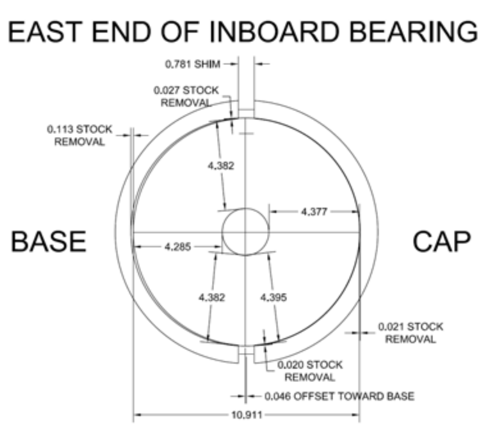
retightened with the jacks in place the alignment corrections could be "locked in" to a degree. An understanding of the magnitude of the forces involved to maintain the position of the shaft was also gained and this information was related to the Engineer.

Horizontal Corrections:

After the cross slope corrections were completed the following steps were performed to set up the boring bar for machining the bores:

1. Survey data was reviewed and calculations were performed to determine the amount of horizontal misalignment and thus the correction needed.
2. Trunnion pin was removed.
3. The bearing liner was machined to remove stock and create an egg shaped/undersized bore.
4. Calculations were prepared determining the required boring bar offsets and resultant material removal taking into account
    a. Misalignment
    b. Current bushing bore diameter
    c. Stock removal from bearing liner
5. A sketch was prepared for each end of each bearing showing the location of the boring bar in relation to the bushing with reference dimensions.
6. The boring bar was set up and an inside micrometer was used to adjust the bar position until the location depicted in the sketch was achieved. All of the measurements depicted in the sketch were documented and reported to the Engineer.
7. As an independent check the boring bar location was measured with a theodolite relative to piano wire benchmarks (piano wire could not be used as the boring bar was solid).
8. All of the survey data, calculations and measurements were reviewed with the Engineer in the field to gain concurrence with proceeding with the field machining.
9. Field machining proceeded to:
    a. Increase the size of the inside diameter to produce an ANSI RC9 fit.
    b. Correct the horizontal coaxial misalignment.
    c. Remove material from one thrust face for each bushing.
10. After machining, hand work was needed to bring up the finish on the bushings and make corrections to the grease grooves .

Field machining was completed on August 9, 2013. Achievement of this milestone concluded SBE's involvement in the project.



Boring bar setup sketch provided to machinists.

Boring bar setup in the field.

Reassembly of the Heel Trunnion and Final Alignment

Reassembly of the trunnion bearing components commenced once the field machining of the bronze journal bushing was complete. The reassembly sequence was as follows for each heel trunnion:

1. Assemble the trunnion in the bearing housing:
    a. Clean the bearing and trunnion.
    b. Install permanent bearing liners.
    c. Install the properly lubricated trunnion pin into the bottom half of the bearing housing.
    d. Install the bearing cap with new studs and torque the bearing cap nuts.
2. Shrink fit the inboard and outboard shear plate assemblies onto the stepped portion of the trunnion pin.
3. Conduct piano wire survey with trunnion pins at bottom dead center in journal bushing
4. Align the trunnion pin within the clearances of the bronze journal bushing as directed by the Engineer
5. Field drill and install all remaining bolts
6. Weld inboard and outboard shear plate assembly to the existing gusset plate.
7. Install remaining structural steel components.
8. Install new bearing grease lines

Upon completion of the assembly of the trunnion bearing housing, the next task was to install the inboard and outboard shear plate assemblies onto the stepped portion of the trunnion pin. The fit between the bore of the shear plate assemblies and stepped portion of the trunnion pin was an FN2 interference fit. To overcome the FN2 interference fit and gain the required clearance, the shear plate assemblies were heated to approximately 400 degrees Fahrenheit by the use of multiple oxy-fuel rosebud heating torches. The rosebud heating torches were applied evenly around the shear plate bore until a minimum clearance of 0.015" was achieved. The temperature of the shear plates were strictly monitored by the use of an infrared thermometer and the diameter of the shear plate bores were measured using an inside micrometer. Once the required clearance was achieved, the shear plates were maneuvered onto the stepped portion of the trunnion pin and aligned with the keyways in the bore and previously cut out section of the main gusset plate. A puller mechanism designed by Facca was used to aid in the installation process.

Heating shear plates for installation.

Prior to welding the shear plate assemblies to the existing gusset plate, Facca conducted a final piano wire survey with the trunnion pins at bottom dead center in the journal bushing to establish the current location of the trunnion pin relative to the original position. Facca also completed a Theodolite survey at the request of the Engineer to determine the relative elevation of the ends of the piano wires i.e. the original elevation of the trunnion pins relative to one another. The results of these two surveys were immediately provided to the Engineer to establish the final alignment of the trunnion pins and bearing housing prior to welding.

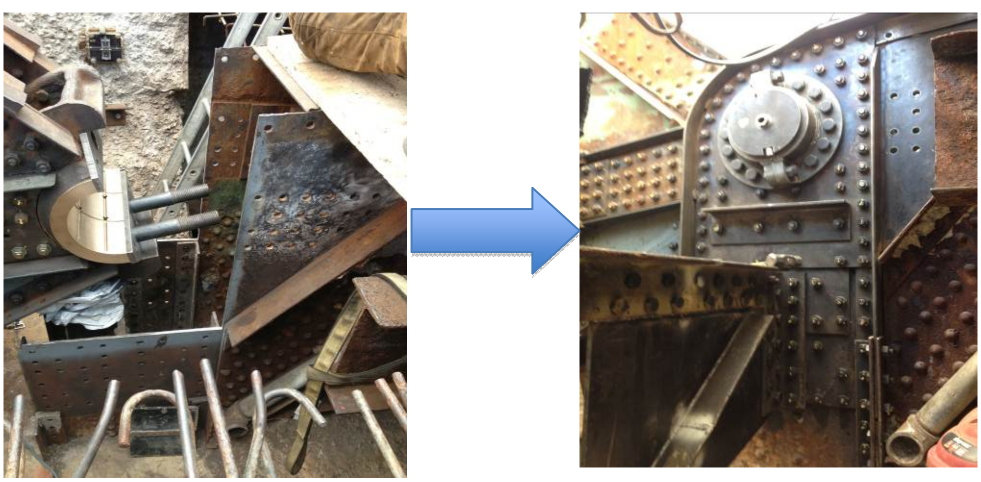
The goal of the Engineer was to adjust the vertical location of both the east and west bearing housings and location of the trunnion pins within the clearances of the journal bushing to set the trunnion pins as close to co-axial as possible and to the original elevation of the east trunnion pin. The Engineer determined that in order to achieve this goal, the east bearing housing was to be raised 0.249" and west the bearing housing was to be raised 0.197". Also, if possible the cross slope of each bearing should be improved and the east trunnion pin should be moved 0.036" to the north to correct the north/south alignment. Re-alignment was to be achieved by jacking the underside of the bearing housing.

Facca completed and achieved the final alignment of the bearing housing and trunnion pins as follows:

1. Dial indicators were positioned to monitor the vertical movement at the east and west heel.
2. Dial indicators were positioned to monitor the horizontal and vertical movement of the east and west bearing caps.
3. A machinist level was positioned to monitor the cross slope of the trunnion pin.
4. Pressure was applied to the hydraulic jacks located at the underside of the east bearing housing to a maximum load of 40 ton. Minimal movement was recorded. Pressure was then released.
5. Pressure was applied to the hydraulic jacks located at the east and west heels. At 4500psi the weight of the movable span was entirely supported and the required elevation change of the east and west bearing housing was achieved.
6. Pressure was then applied at the jacks at the underside of the east and west bearing housing and reduced at the heel jacks to achieve the best cross slopes at the bearings and also maintain the target elevations of the bearing caps.
7. Jacks were locked in position and pressure was released.

Final alignment achieved was within 0.011" of the target elevations outlined by the Engineer and the cross slope at each bearing housing was also improved. The north/south alignment of the east trunnion pin could not be achieved.

Work continued with field drilling and installing the remaining shear plate bolts and welding the new filler plate to the existing gusset plate. All welds were prequalified full penetration groove welds per CSA W59 and were strictly supervised by a CWB certified inspector. The welds were subject to 100% visual and magnetic particle examination and also 20% ultrasonic examination. All welding work was an addition to the overall scope of work.



Facca conducted a final piano wire survey once all welding and installation of the remaining structural steel components were complete to document to final location of the trunnion pins.

Work continued with the installation of new grease lines, comprehensive lubrication of all span support machinery and operating machinery, removal of all temporary bracing from the bridge, replacement of the concrete and asphalt that was previously removed to access the heel trunnions, de-jack and removal of the counterweight support structure and ultimately commissioning of the bridge.

Commissioning

The Engineer closely observed the rehabilitated trunnion assemblies for any indication of binding, movement or abnormal noises. No indications of any problems were observed. In addition, the Engineer monitored the operating loads and loads in the structure as the bridge opened using strain gages that had been present for the duration of the project. Feedback from the Owner indicated a 40% reduction in strains to the original benchmarks.

The reduction in loads confirmed that It was now apparent that the pre-existing condition It is unknown how much of prior high operating loads were due to a pre-existing binding condition due to the ship impact versus the degradation and wear of the original trunnions and bushings.

At a fairly late stage of the construction project it appeared that this repair would only be able to serve as an interim solution followed perhaps by another rehabilitation project within 10 years. The reduction in operating loads and a lack of noise and vibration emanating from the trunnion assemblies during operation were a hopeful sign that that the rehabilitated trunnion assemblies will provide reliable service for the long term and further repairs to this element of the structure will not be needed.

Lessons Learned

Throughout the course of the project, lessons were learned and opportunities for improvement were discovered. Some of the lessons learned from this project are as follows:

- This project was extremely challenging from a technical standpoint. There was a high level of detail involved in developing the means and methods that were ultimately successfully implemented. A major factor which led to the number and nature of the change orders with this project was that the Engineer took a highly prescriptive approach toward the work. The Contract read as a list of instructions on how to perform the work with very little requirement for the Contractor to perform independent engineering analysis and development of means and methods. This meant that virtually every change to the means and methods resulted in a change order. The change order process can create an adversarial environment and inhibit the collaboration needed to complete the project.

  While it is clear that a lot of detail was required in the Plans in order to bid the work it seems in retrospect that giving the Contractor both more responsibility and more freedom would have been advantageous in this case. The following are several suggestions for alternatives to narrowly defining how similar projects with highly technical work be carried out:
    - The detailed means and methods should be presented with the intent of quantifying the work for bidders rather than as requirements that may not be deviated from.
    - The Contractor should be required to perform independent planning and engineering to validate and adopt or revise the means and methods. This allows the Contractor to modify their methods to suit available equipment and materials and forces the Contractor to “buy in” to the work depicted in the Contract.
    - Consideration should be given to including experience requirements for the engineering staff on the project for certain roles. Many recent projects with complex structural-mechanical elements have included designating a Professional Engineer with specific movable bridge project experience as a “Movable Bridge Project Coordinator.” This individual acts as the conduit through which all technical matters are conveyed to the Engineer. This is the role played by SBE on this project however their involvement was generally reactive, not proactive.

- The Engineer produced a detailed 3D model of the structure and components as part of preparing the plans. This greatly helped with understanding the scope of work in a much more detailed fashion. The detailed 3D model of the heel trunnion greatly helped in demonstrating the disassembly and reassembly sequence to the millwrights performing the work. Also, the 3D model was a strong tool used during meetings to help explain issues or potential concerns over the course of the project.

- The project requirements for surveying were limited to documenting the original location of the heel trunnions. As the project progressed, a lack of understanding of the pre-existing geometry of the structure was limiting in some cases. In retrospect, conducting a complete baseline survey of the structure long in advance of disassembly would have helped to further understand the existing conditions and geometry of the structure in significant detail. Perhaps anomalies would have been identified which would have hinted at the extent of the ship impact damage and associated risks.

- For any project involving replacement of elements like trunnions which require precision alignment and rely on dimensional stability of the structure, continual monitoring of alignment is warranted. The failure to recognize the magnitude and significance of the alignment changes through the course of the structural work until a very late stage of the project was perhaps the most significant factor in the length of the schedule delays. Furthermore, continual monitoring would have likely identified the movement due to thermal affects in due course of the work which would likely have averted a month of delays.

- The level of communication and coordination needed with the 3D surveyor was underestimated. The firm selected to perform the work was technically capable of collecting data with the laser tracker but their ability to analyze and present the data was lacking. If a 3rd party surveyor is used it is important for the Contractor to be highly involved in the survey preparation and survey methodology. The following steps are recommended:

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  - Prior to executing the survey, ensure all parties involved (including the Engineer) are in agreement with the survey methodology. This will ensure confidence in the survey data and expedite understanding of the results.

  - Develop procedures to serve as quality controls for 3D surveys so that if there is an unexpected variation in the survey data the theory that survey data is corrupt can be tested.

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  - Fully understand the means and methods used in the survey and what external forces could have potentially affected the components being surveyed i.e. thermal expansion of a steel structure.

- This project was an excellent illustration that when you are planning to take something that is nearly 85 years old apart you will generally need to go a step or two further than expected to put it back together again. In the planning stages of similar projects it is important to always ask the question: What happens if your assumptions are wrong? What can I do differently to avoid making that assumption? Are the risks worth the cost savings? For example, during the first round of changes to the means and methods there was a resistance to utilizing field machining. In hindsight, the flexibility offered by field machining was crucial to the ultimate resolution of the alignment issues, enabling the team to “go one step further” than originally expected.

- With all of the challenges encountered along the way, it was very important to have an open and transparent relationship between the Owner, Contractor and Engineer to facilitate changes when needed. With the pace of changes occurring in the field, the only way to keep the work moving was based on handshake agreements between parties. Once all of the dust settled all change orders were settled to the satisfaction of Facca and the Owner which is a testament to the integrity of all those involved.

HEAVY MOVABLE STRUCTURES, INC.
FOURTENTH BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Design and Construction Engineering for the New ArcelorMittal Rail Bridge over the Indiana Harbor Canal

Dan Machamer, PE
URS
&
James M. Phillips III, PE
E.C. Driver & Associates, Inc.

NEW ORLEANS FRENCH QUARTER MARRIOTT
HOTELNEW ORLEANS, LOUISIANA

Introduction

ArcelorMittal is the world's largest steel producer and the number one steel producer in the United States. ArcelorMittal's Indiana Harbor facility is the largest steelmaking complex in North America. This facility is located on the southern shore of Lake Michigan in East Chicago, Indiana, approximately 20 miles southeast of Chicago. A unique aspect of the plant is that it has facilities on both sides of the Indiana Harbor Canal. To improve plant operations, ArcelorMittal embarked on a project to construct a new rail bridge across the Canal to link facilities on both sides. The connection was needed to allow movement of materials, including hot metal, between adjacent plants to vastly improve efficiency of blast furnace operations.

To engineer the project, ArcelorMittal contracted with URS to perform preliminary engineering, final design, and construction oversight for a new single leaf, rolling-lift bascule railroad bridge. Structural and electrical control systems design was performed by URS's Chicago staff. Bridge machinery was designed by EC Driver and Associates, Inc., a URS subsidiary located in Tampa, Florida.

This paper discusses specific, and in some cases unique, measures taken in design, planning and construction engineering to produce a movable bridge that could be rapidly constructed to exacting tolerances. Topics include provisions for precision alignment of the tracks and treads, float-in erection of the bascule toe section, steel erection to exacting tolerances and bridge commissioning. The unique role of the design engineer as an integral member of the construction team is also presented.

Design

At the onset of the project the following criteria were established for the new bridge.

Design Standards:	AREMA ANSI/AASHTO/AWS Structural Welding Code D1.5
Design Loading:	Cooper E-80 Reichard 300 Ton Capacity Hot Metal Car
Channel Clearances:	90-foot Horizontal Unlimited Vertical Clearance – Span Open

Indiana Harbor Canal

In addition to the requirement to carry the heavy axle loads of the steel mill's hot metal rail cars and freight loading, the bridge was also required to carry the plant's truck traffic. To meet this requirement, a 5-inch galvanized open steel grid deck was incorporated into the design of the timber tie rail deck.

Preliminary engineering included concept plans, cost estimates, construction permitting documents, and coordination with the USACOE and the US Coast Guard during an Environmental Assessment. This early phase was a crucial step in the owner's investment planning and eventually led to project approval by upper management.

The Indiana Harbor Canal is an active navigable waterway that services heavy commercial traffic, specifically frequent barge/tug combinations. Discussions with the USCG established that the channel was to remain open during construction except for an allowed 72 hour closure window scheduled for erection of the bascule span. To facilitate this requirement, it was decided to design the movable span so that it could be prefabricated as much as practical and so that it could be field erected in two major sections to optimize the amount of erection that could be done while avoiding interruptions to the navigation channel.

The bascule span was designed so that the section on the back side of the center of roll (away from the channel), also referred to as the heel section, could be constructed over the bascule pier while the channel remained open to navigation. This allowed for the most time consuming construction activities, such as assembly and alignment of the segmental girders, machinery installation, and counterweight construction, to be performed outside the closure window. Similarly, the toe section, comprised of the portion of the structure, located channel side of the center of roll was designed so that it could be prefabricated off-site and erected using float-in methods. The key element of this approach to construction was the use of a main girder splice, located approximately 4 feet channel side of first position of roll.

Figure 1 shows the basic configuration and geometry of the new bridge in elevation. The movable span is 110 feet long from center of roll to toe. The counterweight is configured overhead and provides 22 feet of vertical clearance above the top of rail. Figures 2 and 3 portray the bridge in cross section on the span and at the bascule pier respectively. The main longitudinal load carrying elements of the bridge are plate girders connected to the counterweight and machinery frame with a tension tie and bracing. The heel section of each main longitudinal member is attached to a curved tread plate with a 25 foot radius. Based on traditional terminology developed by the Scherzer Rolling Lift Bridge Company, the heel section with a curved bottom flange is referred to as the segmental girder, even though in this case it is a single weldment rather than a series of segmented castings. The center to center spacing of the main members is 20 feet.

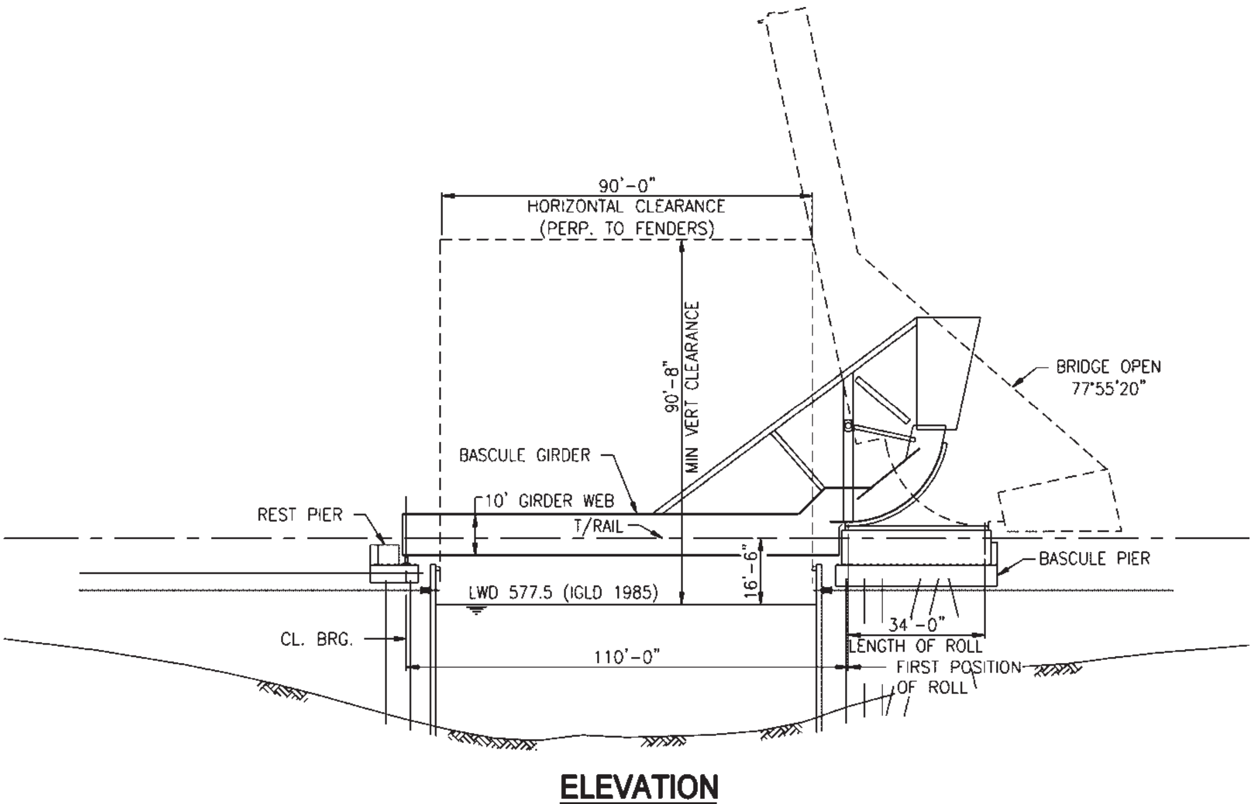


Figure 1

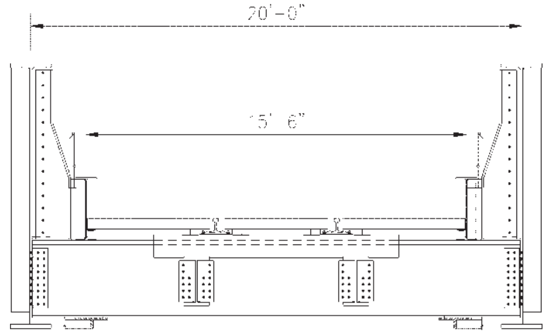
The bascule span design features an open deck plate girder with rolling segmental girders and an overhead counterweight. The counterweight consists of a steel plate shell supported on steel framing that is filled with concrete. The bascule span is supported on a large reinforced concrete bascule pier and rest pier that utilize 120 foot long steel H-piles driven to refusal in bedrock underlying very soft clay.

The bridge piers are constructed inside of new tied-back steel sheet pile enclosed peninsulas that extend roughly 100 feet out into the 300 foot wide canal from both dock walls. These walls were designed for the heavy rail surcharge as well as a 45 foot height of retained backfill. The excessive height of these walls coupled with the poor underlying soil conditions led to the use of lightweight geotechnical fill produced from expanded shale. Cold formed heavy wall sheet piles were used with lengths of 85 foot.

The movable span and associated machinery are controlled by a modern programmable logic controller based control system, operated from a control house on the east shore of the canal. The mechanical system utilizes two redundant 60 HP AC induction motors (used alternately), a four stage central speed reducer, two thruster brakes, and rack and pinion gear sets. Motion of the leaf is controlled by a modern flux-vector drive coupled with each motor.

For a movable bridge to function properly, key elements must be constructed to exacting tolerances. The following are some of the critical dimensional tolerances specified for the ArcelorMittal IHC Bridge:

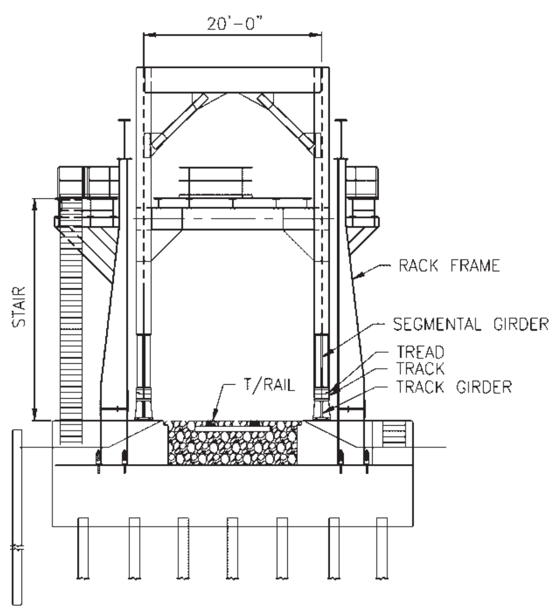
Dimension	Tolerance
Roll Radius	300" ± 1/16"
Track Profile	Level within 1/64"
Track or Tread Centerline	Parallel ± 0.003'
Centers of Roll	Concentric ± 0.020"
Pinion Column Webs	Perpendicular to Axis of Roll ± 0.005°
Rack Girder Position	Plan Location ± 0.003'
Rack Girder Bottom Flange	Level ± .001'



SECTION THRU BRIDGE

Figure 2

A key innovative element of the project was the design of a temporary shoring tower to support the counterweight during shop assembly and field erection. The contract with the structural steel supplier required full shop assembly and alignment of the movable span, including all structural steel, segmental girders with tread plates, counterweight box and machinery support framing. The bottom of the main girders is located 7.3 feet below the rolling track level. The bottom of the counterweight box is positioned 24 feet above the rolling track level. Therefore, the counterweight bottom is 31.3 feet above the low member of the toe section. The height of the counterweight bottom above the top of the bascule pier footing is a similar 33.75 feet. Rather than using different shoring towers in the shop and field a single tower was designed that could be used for both purposes.



SECTION THRU BASCULE PIER

Figure 3

The counterweight shoring tower was designed to fit the bascule pier and track girder geometry, and to support the full load of the counterweight and the steel framing of the heel section. It was also designed so that it could be readily jacked from its base for vertical adjustment on shims, or set on rollers for horizontal position fine-tuning. This feature, that enabled small adjustments, proved invaluable in optimizing the alignment of the bascule leaf in the shop prior to final bolt hole drilling during assembly, and again in the field where conditions for re-assembly were more challenging.

With the four column shoring tower required to carry a dead load of approximately 500 tons, its design utilized robust rolled sections with heavy welded stiffeners at load points and jacking locations. Its self-weight was 15 tons. The constraints of the bascule pier required that it be positioned eccentrically several feet forward from the center of gravity of the applied load, resulting in the two rear columns carrying 80% of the weight. Given the criticality of this temporary structure, the design team decided it should be monitored carefully during counterweight pours to ensure it was behaving as desired. In the field a strain gauge monitoring system was installed with one gauge attached to each flange of each of the four columns near the base. With the weather changing quickly, an additional gauge was attached to a separate unloaded plate for the purpose of temperature corrections to the strain readings. Prior to each concrete pour the estimated column loads were calculated and then compared to actual loads derived from the strains during the pour. This procedure worked well and kept confidence high that the design loads were accurate and the counterweight sized appropriately.

Construction

Engineer's Role

Design and Construction was fast-tracked on an accelerated schedule due to the value the project would bring to plant operations when completed. ArcelorMittal served as their own general contractor and constructed the project with the assistance of a labor and equipment contractor and a number of specialty subcontractors and suppliers.

To ensure the fast-tracked project delivery schedule, the design team provided 8 separate bid packages released in a critical path order in a design-build fashion. After concept phase, the process took 20 months from the time of management's project approval until the crossing of the first train on the completed bridge. The key phases and schedules were as follows.

PHASE	CONTRACT AWARD	FIELD/SHOP START UP	WORK COMPLETED	WORK SCOPE
Marine Work	Jan 2011	March 2011	Nov 2011	Sheet Piling, Bearing Piles, Lightweight Fill, and Protection Cells
Foundations	May 2011	Sept 2011	Nov 2011	Bascule Pier and Rest Pier Concrete Work
Superstructure Fabrication	Jan 2011	April 2011	Nov 2011	Bascule Span Steel Fabrication and Shop Assembly
Machinery Fabrication	Feb 2011	April 2011	Aug 2011	Shafts, Gears, Gear Boxes and Bearings
Track & Rail Work	Feb 2011	Sept 2011	March 2012	Track beds, Ties, Rails, Miter Joints, and Ballast
Electrical Equipment Supply	April 2011	May 2011	Sept 2011	Control System, Motors, Drives, and PLC
Electrical Installation	June 2011	Sept 2011	March 2012	Equipment installation, Wiring, and Conduit
Bridge Erection	July 2011	Dec 2011	March 2012	Steel Superstructure, Counterweight and Machinery Installation
Float - In	July 2011	Jan 2012	March 2012	Transport Shop Erected Toe Section from Shop to Site to Barge. Float span into position and assist erector during final connection.

Throughout construction the engineering team provided typical designer functions such as shop drawing reviews and responses to requests for information. However, the distinctive nature of this project, including the fast-track schedule and method of contracting, required the engineering team to take on a unique role. To best meet the needs of the owner, the engineering team functioned much more as an extension of the owner's staff than is typical. The engineering team became fully involved in steel fabrication inspection, shop test witnessing, assembly sequencing and field oversight of structural, mechanical and electrical installations. Engineers from the design team were embedded with the contracting team during several months of critical field erection and shop assembly. As the project evolved, the engineering team performed a number of construction engineering functions including preparing detailed balance calculations, performing strain gauge balance instrumentation, designing fabrication and erection falsework, and preparing detailed erection procedures for a number of key activities unique to movable bridge construction, including those related to toe assembly float-in.

Construction Team

The following firms contributed to the construction of this project:

ArcelorMittal USA, East Chicago, IL – Owner and General Contractor
URS, Chicago, IL & Tampa, FL – Construction Engineering
Superior Construction Co., Inc., Gary, IN – Labor and Equipment Contractor
Industrial Steel Construction, Gary, IN – Steel Fabrication
Production Tool Co. Chicago, IL – Shop Machining

DLZ Engineers, Drillers, Surveyors – Precision Surveying
American Marine Constructors, Inc., St. Joseph, MI – Pile Driving, Sheeting & Filling
Tranco Industrial Services, Inc., Burns Harbor, IN – Track and Rail
EMCOR Hyre Electric Co. Highland, IN – Electrical Installation
LML Automated Systems, Inc., Burns Harbor, IN – Electrical Equipment
Mammoet USA, Rosharon, TX – Float-In Equipment and Operation
GPL Industries, Inc., Thornton, IL – Shop Machining
In-Place Machining Company, Milwaukee, WI – Field Machining

Construction Plan

The engineering team worked with ArcelorMittal, Superior Construction and Mammoet to develop a detailed erection plan for the bridge aimed at meeting the allowed channel closure window while also achieving accurate dimensional control of the movable span. The resulting plan included general procedures and specifications for shop fabrication and assembly of the following items:

- Forging and machining treads and tracks
- Fabricating track girders
- Fabricating segmental girders
- Numeric roll testing of the mating tread and track pairs
- Fabricating machinery
- Fabricating structural steel of the toe and heel sections of the movable span
- Field installing and aligning the track girders
- Erecting and aligning the heel portion of the bascule span
- Counterweight construction and concrete unit weight testing
- Machinery installation, alignment and testing
- Erecting bascule span toe section
- Initial bridge operation
- Final testing and alignment
- Final bridge balancing

Shop Fabrication

The curved tread plates, segmental girders, track forgings and track girders were machined using a horizontal milling machine. The radius on the tread plates and flanges of the segmental girders was achieved using Computer Numerically Controlled (CNC) circular interpolation. Verification of dimensional control of these components was conducted while the element was still set up on the milling machine. Each pair of matching tread plates and track forgings was measured and a numeric roll test performed to confirm the relative positioning of the track lugs with the tread lug pockets.


Shop Assembly on Shoring Tower

Shop fabrication and complete bascule span preassembly was performed by Industrial Steel Construction, Gary, Indiana. The bascule leaf steel was fabricated with camber to offset dead load deflection. In the shop, the leaf was fully assembled in the cambered position. In this condition the tip of the toe section at the farthest point from the center of roll was cambered 2.5 inches upward and the girder incrementally blocked along its length to follow the design camber shape. At the main girder splice there is no bending stress present in the shop with the splice plates aligned. Similarly, the tension strut is under no load with the connection plates aligned at the connections to the girder and the portal framing.

At shop assembly critical fabrication dimensions were checked progressively throughout assembly using total station surveying equipment, standard steel tape measure, and/or a FARO Laser Tracker. The FARO Laser Tracker is a state-of-the-art interferometer (IFM)-based measurement system that provides three dimensional linear and positioning measurements with a tolerance in the range of ±0.001 to ±0.004 for elements the size of the bridge heel section.

Using the FARO enabled detailed examination of alignment not previously possible with traditional surveying and measuring equipment. When initial FARO surveys indicated deviations from the specified tolerances for the radius of the curved treads and positioning of pinion bearing bore hole at the center of roll of both girder assemblies (segmental girder, pinion column and associated bracing), the engineering team was at first skeptical of the accuracy of the measurements. After all, the radius of the tread plates and bore hole had been accurately machined and the tolerances verified in the shop. To confirm the accuracy of the measurements, two independent sets of measurements were made using the FARO with the shop temperature at a relatively constant value. Comparison of these measurements indicated very good repeatability with values consistently matching within a few thousandths of an inch. In addition, measurements were confirmed with a calibrated steel tape.

Once it was established that the method of measurement was reliable, the dimensional discrepancies became a major focus throughout shop assembly. Challenges were created by temperature variations as the work progressed through the fall and the shop temperature varied from 65 to 25 degrees Fahrenheit.

Tracking the radius of the curved treads measured with the FARO and calculating a best fit center, it was determined that the pinion bore holes were not located at the center of rotation within the specified tolerance. Through a series of checks it was determined that the bore holes were out of position by just under 9/64". At that point in fabrication, with the girder assembly, pinion column and associated bracing fully assembled and all bolt holes drilled, it was decided to relocate the bearing bore holes rather than try to reposition the columns. The specified fit of the bearing housing in the bore would be sacrificed in favor of more concentric and accurate center of roll locations.

From the outset the plan had been to wait and machine the webs of the pinion columns after shop assembly measurements, such that a truly perpendicular (to the axis of roll) mounting surface could be provided for the pinion bearings. Therefore, after the shop assembly was completed the pinion columns were shipped to GPL's machine shop and the bores were repositioned in the same set up that the webs were machined.

Originally the bearing was to have an LC6 fit in the column web for the purpose of locating the bearing. In repositioning the bore hole, it was enlarged and offset such that the hole became slightly irregular. To compensate, scribe lines were established to define the bearing centerline. Per the original design, turned bolts were used to fix the position of the bearing.


Pinion Column Web Machining

The web machining was done using a horizontal milling machine. Prior to disassembly and shipment to GPL, the webs of the pinion columns were surveyed with the FARO and a series of fixed points on the webs were marked for reference. These points were used in set up on the milling machine to position the column so that the web would be accurately cut perpendicular to the precise axis of roll determined during shop assembly of the bascule leaf. Given the importance of the desired results of this corrective procedure and the aggressive schedule, members of the design team directed this machine work face to face with the machine operator.

Field Erection

Field erection proceeded in the following general sequence:

1. Construct sheet pile peninsulas, backfill, and drive bearing piling
2. Construct bascule pier and rest pier concrete substructures
3. Establish survey control points on the concrete piers
4. Mount the 34 foot long track girder assemblies, including track forgings
5. Install the counterweight shoring tower
6. Erect the heel section of the bascule leaf on the tracks and counterweight shoring tower
7. Erect the rack frames
8. Install the shop assembled counterweight steel box
9. Install the machinery
10. Pour the concrete counterweight (four separate lifts)
11. Float in the toe section on barges and connect to the heel section
12. Complete the assembly and track work and test

When the track girder assemblies were being positioned on the bascule pier and survey checks performed, it was discovered that both tracks had a sweep of roughly 0.10 inches in them that had not been evident in the shop. Several attempts to remove the sweep by adjusting the anchor bolts and leveling bolts prior to grouting were not successful. Even heavy steel angles bolted to the pier, and used as jacking rails, could not secure the large forces needed to hold the track girders straight. In the end, the track assemblies were returned to the machine shop to be checked and corrected.

The sweep in the tracks appears to have resulted from the shop's process for mounting the 6-inch thick track forgings to the 2-inch top flange of the track girders. Prior to assembly, the forgings were machined and verified to be straight and true. They were then assembled to the track girders in a horizontal milling machine, where final holes were drilled through both parts and turned bolts installed while the assembly laid sideways in the machine. The 34 foot long track girders were not adequately supported in the center during this process. Despite the large stiffness of the girder about its weak axis, it was in a deflected shape under its own weight when the track plates were drilled and mounted. In the same set-up, final dimensions were checked and showed the tracks to be straight. Scribe lines were then cut, while the assembly was still in its sideways position in the mill. Once removed and stood upright the girders un-deflected, causing the track plates to sweep horizontally, undetected until field installation.

Track / Track Girder Dowel Pin

Shop correction of the track/track girder assembly sweep involved replacing the turned bolt connection between the track forging and flange plate. With the girder in the upright position, the 1.25-inch diameter turned bolts were removed and the girder sprung to its un-deflected shape, validating the assumed cause of the problem. Removing the deflection from the track girder and track left the original turned bolt holes misaligned in many cases. The hole overlap varied from 0.10 inches at the center of the girder to zero inches at the ends. To correct this, the turned bolts that did not fit their original holes were replaced with either dowel pins or high strength bolts. Custom made 1-5/8" diameter dowel pins were distributed over the length of the track to permanently position and secure the alignment horizontally. In between the dowel pins, high strength bolts were installed to clamp the track to the girder. The dowel pins, made of quenched and tempered 4140 steel (153 ksi tensile strength), were installed with an FN2 fit by shrinking in liquid oxygen. The work was performed on a horizontal milling machine using a right angle head so that the track girder was positioned vertically, rather than horizontally as had been the case initially. Care was taken to support the track girder at several points along its length to limit deflection.

Once the track/track girder assemblies were straightened, they were reinstalled and aligned on the bascule pier. The assemblies were positioned to align the tracks parallel to each other, level, and to position the first position of roll on each track at the proper longitudinal station.

Erection of the heel section was performed in a specific sequence aimed at achieving the desired alignment. Steel erection progressed in the following general manner:

a) Position steel members(s) using a crane,
b) Secure connection(s) with drift pins,
c) Stuff all bolts in the connection,
d) Snug tight a few bolts if needed to achieve tight steel,
e) Check alignment via surveying,
f) Make adjustments if needed to align,
g) Tension bolts with alignment confirmed.

Alignment surveys were performed to verify that the treads were aligned with each other and with their mating tracks. The position of the center of roll relative to the established first position of roll on the tracks was also carefully monitored. Adjustments were made by positioning the counterweight shoring tower and by use of chain hoists to pull the pinion columns into a plumb position and hold them in alignment while connection bolts were tensioned.

To facilitate erection and alignment of the heel section, the structure was designed with a pair of W40x149 cross beams connecting the segmental girders together and supporting the counterweight box. The shoring tower was designed to fit under these beams and be connected temporarily to the bottom flange of the W40s. Therefore, in both shop and field erection, the W40s were set on top of the shoring tower, positioned and locked in place with temporary bolts. The rear of the segmental girders was then connected to the ends of the W40s and secured at the design elevation and position (vertical and horizontal). The W40s, in conjunction with the shoring tower, provided a convenient support system for the back end of the segmental girders. The front end of the segmental girders was set on the track forgings with the first position of roll aligned. The jacking and positioning provisions designed into the shoring tower enabled precision alignment of the heel assembly until the connections were fully tensioned.

After the heel section of the bascule span and rack frames were erected and their alignment was verified, the counterweight was constructed. The machinery and bridge electrical power and control was installed on the heel section. Machinery was spin tested and the brakes verified before the racks were installed.

While the heel section was being erected and prior to the channel closure, the toe section was shipped from the shop as an assembly and moved onto a barge. The toe assembly, weighing approximately 225 tons, was placed on a pair of Mammoet's self-propelled transporter trailers. The toe assembly was aligned on the longitudinal axis of the barge with the transport trailers spaced at approximately 36 feet apart and oriented perpendicular to the longitudinal axis of the bridge. Each transport trailer has six pairs of independently controlled wheel bogies that allow the load to be maneuvered with precision. The transporters are also capable of controlled lifting and tilting to position and align the load.

Prior to moving the toe assembly into position, jacks and adjustable hard wood blocking were positioned on the rest pier and on the steel sheet pile enclosed peninsula just channel side of the bascule pier. Rest pier jacks were set on hardwood blocking on the bridge seat. Bascule pier jacks were set on steel mats to distribute the load between the peninsula sheet pile wall and bascule pier footing.

Once the channel closure was implemented, the barge and toe assembly were moved into position in the channel just north of the bridge location and secured in place. Using the transport trailers, the toe assembly was then rotated 90 degrees, lifted vertically and walked forward to align with the heel section. Once in position, with the main girder splice aligned, the leaf was adjusted with the transport trailers so that the splice plates could be installed and a number of drift pins driven in the bottom flange splice plate. The toe assembly was then set on the jacks at both ends. Using the jacks, the toe assembly was positioned so that the remainder of the splice connection could be pinned and stuffed with bolts.

A key element of the erection procedure was to properly address camber and deflection so that the finished structure conformed to the design geometry and unanticipated deformations were avoided. Throughout heel section erection and counterweight construction, the position of the center of roll was monitored relative to the first position of roll on the track forgings. Elastic deformation in the counterweight shoring tower and resulting movement of the center of roll was anticipated and adjusted for by initially setting the counterweight a little high and by jacking and shimming the tower as needed.

Pivoting Toe Assembly on Barge

The erection procedure was developed to recreate the shop alignment at the time the field splices were connected. However, in the field there is no shop floor to incrementally block the girder into its cambered position. To compensate for this, the tip of the leaf was positioned at a height of 4.5 inches above its final position when floated in and set down on jacks. This accounted for the vertical camber at the tip and the rotation of the girder at the splice which was created in the shop by the incremental blocking. During float-in, this position was held until the main girder splice was secured with enough drift pins and bolts to prevent movement. Once secured, the girder tip was lowered by jacks until the tension strut could be connected.

Aligning Girder Splice

After the tension strut was connected, but before the counterweight shoring tower could be removed, a series of tests were made to confirm alignment and the balance condition. Alignment of the bridge centerline, first position of roll and treads was performed using total station equipment. Although detailed balance calculations had been performed, a step by step procedure was implemented to confirm that the span was toe heavy and that the machinery was capable of holding the imbalance with adequate reserve for wind loads. The span balance field tests included the following steps.

a) Chain hoists were installed at the toe of the bascule span to hold the toe down to the rest pier.
b) The bolts connecting the framing under the counterweight to the shoring tower were removed.
c) The toe of the span was jacked down to rest in its final position (bridge lowered) on the load shoes. This effectively removed the dead load camber and tensioned the tension tie.
d) The strain in the counterweight shoring tower columns was monitored, confirming that the counterweight had been lifted some amount.
e) The brakes were applied.
f) The shoring towers were removed.
g) The chain hoists at the end were released slowly to create slack – the span tip lifted but only slightly – these results were not conclusive.

h) The chain hoists were further released and the span tip lifted a little more then stopped without applying the brakes indicating the span was near balanced at this position.

i) The span was lifted to approximately 7 degrees from fully closed using the drive machinery operated at creep speed. With one brake released the span did not move. With both motor and machinery brakes released the span drifted down slowly. This indicated that the span was in a somewhat toe-heavy balance condition, but well within the capacity of the drive system and brakes.

The span was raised slowly at creep speed through a series of positions and stopped at increments of approximately 5 degrees. At each position, the brakes were released one at a time to confirm that one brake (motor brake) was capable of controlling the unbalanced load. This allowed the capacity of the second brake (machinery brake) to be available in the event of wind gusts. Clearances between moving parts including the rack and pinion backlash and the track lug to tread pocket clearance were also checked at each position. Dykem Steel Blue layout fluid had been applied to the rack teeth and track lugs to allow contact patterns to be clearly observed. Initial rack/pinion measurements indicated that adjustment by way of shimming the racks would need to be performed to achieve the desired backlash. The track/tread alignment was confirmed as acceptable.

Following the initial operation of the bridge, the bridge was put through a number of test operations and the racks were shimmed to set the rack/pinion backlash. Electrical installation was completed including setting of limit switches. The span locks, centering device and connecting track were completed. Once all equipment was set and adjusted, full functional testing of the bridge was conducted. After functional testing the bridge was load tested with one of ArcelorMittal's engines and subsequently with various capacity hot metal cars.

First Engine across Bridge

Summary

ArcelorMittal, the world largest steel producer, embarked on a project to construct a new rail bridge across the Indiana Harbor Canal to link facilities it owns and operates on both the east and west sides. The new rail connection allows movement of materials, including hot metal, between adjacent plants, thereby greatly improving efficiency of operations.

To accomplish ArcelorMittal's objectives, URS designed a new rolling-lift bascule bridge with overhead counterweight. Specific and in some cases unique measures were taken by the owner and design team to produce a movable bridge that was rapidly constructed to exacting tolerances. Innovative design details, such as a main girder field splice, were combined with innovative construction methods, such as float-in erection of the toe assembly and design of a special counterweight shoring tower, to meet the demanding construction schedule and satisfy the USCG's requirements for maintaining marine traffic.

Working with the owner as an extension of their staff throughout the construction process, the engineer developed detailed construction procedures that were implemented to achieve the exacting construction tolerances necessary to produce a well-constructed movable bridge. These processes evolved with the project and were adapted to various challenges which occurred during fabrication and erection. This approach proved valuable as it allowed early identification of construction issues and provided a flexible format capable of adjusting to challenges. In general, the detailed procedures that were followed led to excellent results. In cases where construction tolerances were not initially met, processes were adapted or supplemented to produce the desired results.

HEAVY MOVABLE STRUCTURES, INC.

FIFTEENTH BIENNIAL SYMPOSIUM

September 2014

FAIL-SAFE CONTROL SYSTEMS FOR HEAVY MOVABLE STRUCTURES

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Fail–Safe Control Systems for Heavy Movable Structures

INTRODUCTION

Heavy movable structures can use many different control system architectures. These include hardwired electromechanical relays and various programmable electronic control systems such as the programmable logic controllers (PLCs), direct digital controllers, distributed controllers, or hybrids of each. The overall bridge control system architectural configuration may include control sub-systems dedicated to motor drives, hydraulic power units, navigation and signal lights, and other equipment. The control sub-systems may be separate, stand alone hardwired relays, PLCs, proprietary electronics, or hybrids.

It is necessary to design the overall bridge control system to include fundamental fail-safe characteristics regardless of the architecture used. A fail-safe system is one in which the failure of any component in the system will not prevent unsafe operation of the controlled equipment.[1] Typically, this means a fault will still allow equipment to be shutdown. More important, a fault should not cause the unintended operation of equipment. When analyzing the specific fail-safe requirements of the application, it may be necessary to exclude some control system architectures from consideration.

Circuits and programs used for starting and stopping equipment, machinery shutdowns, emergency stops, interlocks, permissives, and feedback control must be analyzed with regard to cause and effect for an overall fail-safe control system.

The objective of this paper is to explore different fault scenarios common to control circuits and systems. The focus is fail-safe control system design for heavy movable structures, and more specifically for movable bridges. Emergency stop control circuits that are both fail-safe and fault tolerant are presented. The techniques discussed can be extended to many other control system applications with success.

CONTROL SYSTEM ARCHITECTURES

Hardwired Electromechanical Relays

One of the most widely used control systems for all types of applications is the electromechanical relay control system. Relay control systems date back to the 1800s and remain popular today. An example of relay control architecture on a movable bridge is shown in Figure 1. Relays use an electromagnet to switch contacts from open to closed or closed to open (Figure 2). Springs are used to return the contacts to their de-energized position. Latching relays use dual electromagnetic coils to drive the contacts to either open or closed states. Latching relay contacts stay in the last position until the coil of the opposite state is energized.

Future reference to relays in this paper will imply the electromagnetic control relay unless stated otherwise. The energizing or de-energizing of relays and the switching of the associated electrical contacts provide a way to implement control logic by controlling and directing the flow of electrical energy. Fail-safe considerations for relays are primarily an analysis of what happens when a relay does energize and what happens when it de-energizes.

Electronic Control Systems

Programmable electronic systems include the PLCs (programmable logic controllers), DCSs (distributed control systems), network field bus types (Profibus, Fieldbus, Hart, etc.), direct digital or distributed control using mainframe computers, microcomputers, and personal computers. These digital electronic devices use microprocessor-based hardware to execute software and firmware application control programs developed by the control system engineer.

The PLCs and DCSs have gained the widest acceptance and use among the electronic control systems. DCSs are rarely used on movable bridges because their high costs outweigh their benefit for this type of application. PLCs are widely used on movable bridges (Figures 3 and 4).

The PLC is defined as a digitally operating electronic system designed for use in an industrial environment that uses a programmable memory for the storage of user-oriented instructions for implementing specific functions such as logic, sequencing, timing, counting, and arithmetic to control various types of machines or processes, through digital or analog inputs and outputs.[2]

The PLC was developed in the 1970s to be a relay replacement device for discrete control. That is, control that can be implemented with logic states of "ones" and "zeros," "on" and "off," "high" and "low", and so forth. In the 1990s, PLC capability expanded to include the more sophisticated analog control that was previously available only in single loop controllers and distributed control systems.

Personal computers and microcomputers are advancing steadily in control system usage, but their reliability and fail-safe diagnostics lag behind the PLCs.

PLCs are developed with rugged hardware, and strict internal microprocessor diagnostics for software and firmware. Electronic devices do not necessarily fail to a logic state of "zero." PLCs are manufactured with built in fail-safe features. The operating system and application software used in PLCs are rigorously tested for efficiency and the "bloat-ware" commonly found in personal computers is typically not allowed by the PLC manufacturers.

Today, most movable bridges use PLC control, hardwired relay control, or a hybrid of the two. Drive systems are being manufactured more commonly with integral microprocessor based controls. This leads to the possibility that the embedded logic may not necessarily be fail-safe.

The engineer must develop fail-safe features in the relay control schematics, the PLC application programs and input/output configurations, and the drive system parameters.

Case History

The Hood Canal Bridge is a floating concrete pontoon bridge which spans the 330 foot deep Hood Canal connecting the Olympic Peninsula to the Kitsap Peninsula in western Washington. The floating portion of the bridge is 7,450 feet long and has two retractable draw spans in the center which can be opened to form a 600 foot channel for marine traffic. It carries one lane of traffic in each direction. On the evening Thursday, August 18, 2005 at approximately 11:30 PM the bridge was undergoing construction work to facilitate the widening of the west half of the bridge and the east and west approach structures. The WSDOT construction inspector noticed that the traffic had increased and saw that the east structures stop signals and traffic gate warning lights flashing. The bridge tender went to the control tower on the west half of the bridge and found that all of the indicating lights on the control desk for the east
half of the bridge were lit. When he was unable

to gain control from the control desk in the west control tower, he went to the east control tower and found that all of the indicating lights on the east half of the control board were lit as well. Since he was still unable to control the bridge, he went to the PLC cabinet on the floor below and turned the primary and back up PLCs to “HALT” using the keyed switches which turned everything off at approximately 1:30 AM. He observed that the west end locks had rotated into the OPEN position, that the machinery rooms smelled like overheated motors and that the drive motors were hot to the touch.

A control system fault caused PLC outputs to energize without operator commands and without operators present. Electrical equipment, including motors, was energized and operating as a result. The drive motors were in a locked rotor condition for an extended period, destroying them. There were no PLC module failures identified. The PLC control system was reset by clearing all forced points. Later, the program was reloaded and system power was cycled off and then on. The PLC operations were tested and appeared to be functional.

The cause of the fault could not be repeated, isolated, or conclusively identified. It was more likely caused by equipment failure (hardware or firmware) than by a software failure. Direct human intervention was not the cause. It is possible that events leading up to the fault, such as testing the auxiliary generator or power surges and outages, may have caused, or contributed to, the fault. It may be possible for a power surge to have this effect even with good power filtration and a UPS for surge protection. A power surge could enter the system components through the input power feed, or back feed through the non-isolated output modules. There is one documented case from the manufacturer of a somewhat similar fault being caused by a power surge. Although, there were not enough details of the case to be conclusive. Poor quality control system grounding may also be a contribution factor.

Operator Control Stations

The operator control stations found on movable bridges are mostly hardwired hand switches, pushbuttons, pilot and indicating lights, and alarm displays. Engineering considerations must be given to the hand switch contact developments, spring returned hand switch contacts, captive key-lock hand switches, and dual pilot lights when designing the control console for fail-safe features. Push-to-test indicator lights are good to use when the lights are being used for alarms or to verify the position of machinery. Knowing that lamps are working is not only a helpful maintenance feature, but it allows the operator to know if the dark lamp indications are true. Fail-safe consideration for indicator lights includes using the actual machinery being monitored to give positive feedback directly to the indicating light. Using electrical control signals that command the machinery to also control the indicating lights is an unreliable method of providing feedback to the operator. Additionally, where a device or piece of machinery travels to opposite positions; such as a valve (open/closed), a lock (driven/pulled), a leaf (open/closed), a brake (set/released); it is a good practice to sense both states independently. This provides the operator with an indication that the machinery is in travel or if it has failed during travel.

LED, LCD, or plasma flat screen based graphical operator interfaces have not been widely used on movable bridges for several reasons. Flat screen displays require redundancy because they are fragile when compared to a hardwired control console. Graphical displays on screen are easily washed out by sunlight that usually floods a control tower through the large windows needed for operator visibility. If the displays are left on continually without rebooting, screen burn-in will require their replacement every 2 years. If the displays are turned off between openings, there is a time delay required to warm up the monitor or to reboot the computer that is driving the graphics before operating the bridge.

Eventually these obstacles will be overcome and these operator stations will be used on more movable bridges. For example, high intensity enhanced LCD displays (liquid crystal display) or gas plasma displays could make graphical control stations more practical.

Fail-safe design considerations in graphical operator interface stations would include the performance of all actual control functions in a separate and dedicated PLC. The graphics station should remain strictly supervisory. With this architecture, fail-safe requirements are reduced to only needing the proper techniques to communicate between the graphics stations and the PLCs. The programming techniques of the PLC logic also become critical with this architecture.

FAIL-SAFE CONTROL SYSTEM DESIGN APPROACH

Control systems and devices have changed dramatically over the past 100 years. However, the fundamentals associated with the control systems required for safe operation and shutdown have not changed. It does not really matter if the machines, equipment, or processes being controlled are chemical plants, power plants, or heavy movable structures. It does not matter if the control system architecture uses only mechanical devices, only electrical devices, only electronic devices, or if it is some type of hybrid. What does matter is that the control system is properly engineered to provide for the safe shut down of the machines and equipment in the event that one or more control system component fails. A control system must be engineered to achieve shutdown conditions in an orderly manner with minimum risk of injury or damage to the machines and equipment being controlled.

There should be no compromise between safety and cost when developing control system designs. Using fail-safe techniques does not usually require any significant amount of extra labor or materials. What should be considered instead is the cost of not being fail-safe if there is a failure.

Control system engineers freely adopt proven techniques from similar applications as being a prudent approach to design. Good engineering practice includes assessing whatever works for a specific application elsewhere and considering mirroring it in a similar application.[3] Some standards require a control system that is both fail-safe and fault tolerant. Generally, control system standards used for movable bridges do not accept designs where one or two faults of any kind can cause unintended operations or where a single fault will prevent equipment shutdowns.

Standards and Specifications

There are a few standards available for engineering movable bridge control systems. Guidance is taken from AASHTO publications (American Association of State Highway and Transportation Officials).[4, 5] The AASHTO, *Standard Specifications for Movable Highway Bridges* is the foundation on which the movable bridge is designed. However, AASHTO specifications and recommendations are somewhat limited in regards to control systems and need to be supplemented with additional standards. There are many industry standards, definitions, and symbols specifically dealing with control systems. Those most pertinent are listed as follows:

- AASHTO, *Standard Specifications for Movable Highway Bridges*, 5th Edition, American Association of State and Highway Transportation Officials, Inc., Washington, DC, 1988.

- AASHTO, *Movable Bridge Inspection, Evaluation, and Maintenance Manual,* 1st Edition, American Association of State and Highway Transportation Officials, Inc., Washington, DC, 1998.

- Code of Federal Regulations-CFR Title 33, Parts 118- Navigation and Navigable Waters.

- FHWA, *Manual on Uniform Traffic Control Devices (MUTCD)*, Federal Highway Administrator, 1988.

- ISA, *Instrumentation Symbols and Identification (ANSI/ISA-5.1, 1984)*, International Society for Measurement and Control, Research Triangle Park, NC, revised 1992.

- ISA, *Application of Safety Instrumented Systems for the Process Industries (ANSI/ISA-84.01, 1996)*, International Society for Measurement and Control, Research Triangle Park, NC, 1996.

- ISA, *Identification of Emergency Shutdown Systems and Controls That Are Critical to Maintaining Safety in Process Industries (ANSI/ISA-91.01, 1995)*, International Society for Measurement and Control, Research Triangle Park, NC, 1995.

- JIC, *Electrical Standards for General Purpose Machine Tools and Mass Production Equipment, (EGP-1-67 and EMP-1-67),* Joint Industrial Council, McLean, VA, 1967.

- NEMA, *Industrial Control and Systems: Control Circuit and Pilot Devices, (NEMA ICS 7)*, National Electrical Manufacturers Association, Roslyn, VA, 1993.

- NEMA, *Programmable Controller Standard ,(NEMA ICS 3)*, National Electrical Manufacturers Association, Roslyn, VA, 1993.

- NFPA, *Electrical Standard for Industrial Machinery (NFPA-79, 1991)*, National Fire Protection Association, Inc. Quincy, MA, 1991.

- NFPA, *Hydraulic Fluid Power- System Standard for Stationary Industrial Machinery (ANSI/NFPA/JIC- T2.24.1, 1991),* National Fire Protection Association, Inc. Quincy, MA, 1991.

The National Fire Protection Agency standards relating to electrical control systems include NFPA-70, *National Electric Code*, and NFPA-79, *Electrical Standard for Industrial Machinery.* NFPA standards are primarily concerned with protection against electrical shock and fire hazards. When NFPA-79 incorporated the Joint Industrial Council Standards in 1985, it only included those areas related to electrical shock and fire hazards. [2] NFPA-79 does provide definitions for the terms "Fault," Failure," and "Machinery Control Circuit," but does not define "fail-safe" as related to controls systems.

Additionally, the following standards address programmable control system safety:

- ANSI/ISA-84 Standard for Safety Instrumented Systems-(Instrumentation Systems and Automation Society).
- IEC-61508 Standard for Functional Safety- (International Electrotechnical Commission).

AASHTO Fail-Safe Requirements

AASHTO specifies that motor brakes for movable bridges must be fail-safe mechanically and electrically.[4] Motor brakes are to be held in the set position by springs and released when electrically energized. They are to set automatically whenever the electrical current is turned off. AASHTO also specifies that hydraulic pumps must fail to the zero pumping position and bypass valves must fail open.[4]

AASHTO also requires a level of redundancy in safety systems as for brakes and for safety related instrumentation. Some AASHTO requirements for equipment and associated control circuits are as follows:

- Auxiliary Power (recommendation).
- Two sets of Brakes; motor brakes and machine brakes.
- Two electric compressor type air trumpets and two smaller electric trumpets (requirement on bridges with electricity).
- Two drive motors with provisions for bridge operation by one motor (recommendation).
- Normal stopping controls, and emergency stopping controls.
- Reversing motors shall have mechanically interlocked reversing contactors.

- Span overspeed switches at nearly closed and nearly opened positions interlocked to set the brakes by removing power.
- Hand released brakes shall render the bridge inoperable.
- Disconnect switch to PLC input/output power.
- Master Control Relay (MCR) circuits to remove PLC input/output power.
- Position limit switches (and skew switches on lift bridges) to stop drive motors and set brakes at each end of span travel.
- Operational sequence interlocks: set traffic signals, lower gates, close barriers to block traffic, pull locks, release brakes, open span, etc.

Unfortunately, AASHTO specifications come short of mentioning how fail-safe or fault tolerant control systems are to be achieved.

The engineer should develop the control system plans and specifications in accordance with the applicable industrial standards. Even if not familiar with AASHTO, most reputable control system contractors are familiar with ISA, NEMA, and JIC. NFPA No. 79 and JIC No. EGP-1 standards and symbols for relay control systems are shown in Figures 5, 6, and 7.[1,2] These symbols include the familiar relay coil, timers, pushbuttons, pilot and indicator lights, and switches used in control circuits. JIC standards include symbols for control switches, sensors, and indicators with associated definitions. JIC standards and schematic ladder diagrams are used by most control system engineers, technicians, and electricians. PLC programming formats also include the schematic ladder diagram type of graphical programming adopted from the JIC standards. All symbols and standards rely upon the engineer for the proper application in developing fail-safe controls.

Instruments to detect flows, pressures, temperatures, and levels are used extensively on hydraulically operated bridges. Typically, the symbols used on hydraulic schematics are from NFPA/ANSI standards.[6] NFPA/ANSI nomenclature includes abbreviations for instrumentation and sensors like Float Switches (FS), Pressure Switches (PS), Temperature Switches (TS), and Limit (LS). The abbreviations from the ISA standards (International Society for Measurement and Control) are helpful when differentiating between a Flow Switch (FS), a Level Switch (LS), a Position Switch (ZS), and a Pressure Switch (PS).[7] Control engineers must be careful not to mix symbols from conflicting standards identifying a Level Switch as “FS” for float switch, or a

Position Switch as “LS” for limit switch or as “PS” for position switch. ISA, JIC, and NFPA instrument abbreviations conflict. The ISA standard provides the most comprehensive method for unique identification.

FAIL-SAFE CONTROL TECHNIQUES

There are many ways for control devices to fail to operate properly. While it is not impossible to design control systems that account for every possible combination of faults, it would be very expensive to do so. A more practical approach to designing fail-safe control systems is to account for the most probable modes of failures and provide control devices and techniques necessary for safety.

A fail-safe control device is one that will cause no unintended operations or unsafe functions if the device itself should fail. Figure 8 provides a generalization of some good and poor design practices. A common example is when using normally closed contacts on a control relay that is used in a motor starter circuit (Figures 9 and 10). An incorrectly engineered circuit, one that is not fail-safe, could result in the motor not being tripped if a control relay coil burns up or a fuse blows. A broken wire, or a bad relay coil should not cause a motor to start or prevent it from being stopped.

Another common example is shown in Figure 11 where a ground fault can start a motor unexpectedly if the controls are on the neutral side of the coil. This type of control is a National Electrical Code violation. [8]

Movable bridges use equipment that may become hazardous to the public if the controls should fail. A listing of some equipment and potential hazards follows:

- Traffic Gates and Barriers- A fault could cause the gate to unexpectedly operate with the bridge open to traffic or prevent the operator from stopping a gate operation.
- Center Locks- A fault could cause the lock to unlock with the bridge open to traffic.
- Drives and Brakes- A fault could cause the span to raise with the bridge open to traffic, or could prevent the operator from stopping the span from lowering with a vessel underway.

- Navigation Lights- A photoelectric relay circuit fault could turn off all of the navigation lights putting a vessel at risk of collision with the structure.

It is important to know how a sensor will be actuated and what that means for the associated machinery or equipment (Figure 12). For example: With a limit switch that is sensing the "released" position of a motor brake as required by AASHTO,[4] it is necessary to sense the "set" position independently from the "released" position. It is not the same to have a single switch make contact when the brake is in the "released" position and to assume the absence of a made contact indicates the brakes are "set." A loose switch or a loose wire would also appear to be an open contact to the control circuit and could result in unsafe control. When individual switches and circuits are used to positively sense when the brakes are "set" and "released," a circuit or switch failure can be detected more readily. Fail-safe interlocks and indications require a closed circuit to verify field condition. The absence of a signal should be interpreted as the absence of a control permissive and that conditions are not ready for operation. It is often just as important for the operator to know that a brake is not completely "set" as to know when a brake is fully "released."

In Figure 5, note the position limit switches and the temperature switches. They are available with normally open contacts "held" closed or normally closed contacts "held" open. It is the engineer's responsibility to define the contact development that is essential in designing a fail-safe system. The contact development must fit the application in the circuit for the desired operation during normal conditions and after a sensor or circuit failure. Position limit switch contact developments used on movable bridges are shown in Figure 13.

Figure 14 shows fail-safe and non-fail-safe methods for using position limit switches in a circuit for bridge leaf speed control. Design the circuit so the closure of the nearly open or nearly closed limit switch contact is a permissive signal to go to normal speed. The loss of the signal, whether caused by the limit switch contact opening or a broken wire, should cause the leaf to go to creep speed. The same is true for stopping the leaf using the full open and bridge seated limit switches. The absence of a signal should result in the leaf drive stopping.

In certain situations, consideration should be given to providing redundant switches for the full open or bridge seated limits. These would include situations where the limit switch arm may be prone to a mechanical failure or interference due to icing and other obstructions. The normally closed limit switch contacts are wired in series and are held in the open position when the bridge leaf is fully open or seated.

The temperature switch that is sensing a high temperature should be normally closed and should open upon high temperature conditions. This way a failed contact or broken wire, blown fuse, or loose terminal will result in a de-energized circuit (usually the safe case) and the high temperature interlock will close a valve, stop a pump, or allow a predetermined conditions to exist. The hydraulic pump motor control shown in Figures 9 and 10 illustrates this.

It is important to differentiate between controls used for alarms and indications only, and those needed for equipment shutdown safety. It is not always possible for an alarm to be generated by an open contact or by a de-energized circuit. Alarm conditions are usually annunciated by a light and a horn or buzzer, or by some other energized device. "Off the shelf" annunciators with built-in lights and audible devices are available which can be set to alarm upon sensing an opened contact. PLCs can be programmed to function the same way. Some owners who prefer to use relay control systems still use PLCs or microprocessor based annunciators for alarm handling because they are flexible and provide good historical data collection.

Figures 15 and 16 illustrate PLC control schemes for the hydraulic pump previously reviewed in Figure 10 using relay control. PLC triac outputs are acceptable for indication only. Triacs tend to fail in the short circuit mode. Such a failure would operate any device connected to the output if the control circuit has power up to the triac. For this reason, triac outputs are not recommended for motor control applications. Normally open PLC relay outputs are preferred.

Most PLCs are supplied with a watchdog timer that monitors logic circuits controlling the processor. If this timer is not reset in its programmed period of time (which is equal to one scan period), it will cause the processor to fault. Where a failure of the central processor can result in a significant hazard, an independent (external to the PLC) watchdog timer should be provided (Figure 17).

By programming an internal PLC cycle timer to start and stop external timers, the on and off cycling can be monitored as a "heart beat." The external timers provide a shutdown upon a PLC failure in either a high logic state (logic=1) or a low logic state (logic=0). Detection of unsatisfactory PLC operation should initiate an emergency shutdown. The external watchdog timers with a discrete input fed back to the PLC can be used to verify the operation of the input module, the central processor, and the output module.

There are some exceptions to using standard fail-safe controls. A different control system solution is needed when the equipment must remain energized during any fault condition. These applications are those that must be completely fault tolerant as opposed to fail-safe. This would be true for safety systems. A fire water pump is an example of one such system. The reason for this exception is that during a fire, it is likely for a control system to fault, but the fire is the greater risk. The design of such a circuit may need to consider special techniques including supervisory current to monitor circuit continuity and triple redundancy where two out of three devices can fail without consequence. Failure analysis for this type of system includes verifying the circuit can be turned <u>on</u> if one of the devices has failed, and that it can also be turned <u>off</u>. Sometimes when designing for one condition, the other is overlooked.

There will always be "exceptions to the rules" for the proper application of fail-safe control system techniques. It is therefore necessary for the engineer to assess each installation and application uniquely when developing the control system architecture.

FAULT TOLERANT CONTROL TECHNIQUES

A fault tolerant control system is one that has sufficient levels of redundancy to allow a single control device or group of devices to fail without affecting operations and the ability to control. A fault tolerant control system must be designed such that safety is not compromised in any way. The control interlocks must remain functional during the faults. Fault tolerant systems are often mandatory for the control of many chemical processes, burner management systems, and manufacturing systems where lost time of production or the safety risks outweigh the extra costs associated with fault tolerant control systems. With these types of facilities, even if the controls are designed to fail safely and de-energize all machinery and equipment, sudden or frequent shutdowns may compromise the process equipment or the safety of the facility.

It is not usually necessary to provide fault tolerant control systems on movable bridges except for fire protection systems or similar safety systems. Some level of fault tolerance may be considered for bridges where the volume of roadway and marine traffic are high and a non-operational bridge could cause financial harm or impede emergency vehicles.

Until the 1980s, movable bridge controls were typically ungrounded. This made them fault tolerant for ground faults because if a circuit went to ground, it normally could not complete the short circuit to trip a breaker or blow a fuse, and the system would continue to operate. Operational safety is compromised with this concept since a second ground fault can result in unexpected operations. The NEC allows for ungrounded systems providing there is a ground fault indicator on the control circuit.[9] Some designs may have ground fault indication, but it is on the main service entrance, not on the control circuit. Because this type of system can "appear" to be operating normally, a ground fault can go unnoticed until there is a second fault. Generally, ungrounded control systems do not fail safely. There has been at least one incident resulting in a fatality caused by a second ground fault raising a bridge against moving traffic.[10]

Fault tolerance can also be achieved procedurally. Marine traffic is required to confirm that a bridge is fully open before proceeding underway through the structure. This is not always practical depending on the strength of the local tides, the navigational channel characteristics, and the size of the vessel. For large vessels, the bridge may need to be opened while the vessel is still a mile or more away so that if there is a fault in the control system, the bridge operator has time to employ emergency procedures.

It is a good engineering practice to include redundancy in the design for electrical power service, leaf drive systems, navigation lights, and traffic lights. There should always be an alternative means of operating the bridge. The AASHTO requirements and recommendations for redundancy, previously discussed, should be included in the control system design.

Emergency Stops

Emergency stops are required for all control systems. They are configured to remove power from machinery, equipment, and control circuits by opening hand switch contacts and contacts on master control relays (MCRs). This includes power to PLC outputs and other electronic output devices, and motor drives. Emergency stop circuits should be not be part of the normal operation. All of the emergency stop control devices should be dedicated to stopping all motors and removing control power from the motor controllers. In use, the emergency stop should cause all motors to de-energize and all brakes to set.

Some installations require the emergency stop circuits to be fault tolerant and fail-safe. In Figure 18, if a single MCR should fail, the shutdown circuit is not affected. It will require two MCR failures to affect a shutdown. Conversely, if a single set of MCR contacts become welded or seized together, or a spring fails; then the circuit will still provide a shutdown through the remaining two MCRs.

FAIL-SAFE AS AN ENGINEERING PHILOSOPHY

A lack of continuity in engineering safety techniques has been seen when comparing PLC control systems with hardwired relay control systems. Movable bridge control in some states has evolved from relays, to PLCs, and back again to relays. The knowledge of the engineers who once designed relay systems is not being passed along to the new engineers.

Certain design techniques that are fail-safe when using a PLC as a “relay replacer” are not fail-safe when using the same techniques with hardwired control relays. For example: In PLC logic, the software equivalent to a “normally closed” contact is often used in the control programs, and when the associated logical statement becomes “true,” the software contact is “opened.” This is acceptable in the PLC because internal self-diagnostics and “watchdog timers” constantly verify the PLC system is functional. A failure of any diagnostic test will result in the safe shutdown of the PLC system and all outputs are turned off. This is not the case for the equivalent hardwired control system (Figures 10 and 16). If there is a failure of the relay coil or a broken wire, the normally closed contact stays closed regardless of conditions that are supposed to open it.

It is the joint responsibility of the engineers and the owners to require fail-safe control systems. It is the responsibility of the engineers and their companies to be sure engineering techniques are defined, documented, and disseminated. The experience of the senior engineers must be passed to the engineering interns. At the same time, continuing education in control system safety for the senior engineers is necessary because control system devices are continually changing and being upgraded. Project schedules should provide adequate time for the control engineer to be thorough and complete in the application of fail-safe techniques. The control system symbols used in design look very similar (Figure 8). An improperly selected symbol, or a typographical error in a schematic or PLC program can result in catastrophe.

CONCLUSION

Engineering a control system to include fail-safe features requires knowledge of both the instrumentation and control devices, and the machinery to be controlled. It is necessary to design the control system architectures and circuits such that the machinery and equipment will de-energize upon a control device failure. It is also necessary to provide the engineering needed to ensure that electronic control system application programs and input/output configurations allow the same. While national and international standards address specific techniques, recommendations, and requirements to this end, it is still necessary for the controls engineer to make the final determination of the detailed design for each particular movable bridge facility.

Implementing engineering safety standards, alone, cannot assure absolute safety of operation. The ultimate safe operation and control of the system is in the hands of the contractor during construction, and the bridge tender and the maintenance personnel once completed. Fail-safe designs are not fail-safe if critical limit switches are defeated with jumpers or are bypassed. There is no substitute for diligent, capable, well-trained electricians, operators, and maintenance technicians.[11]

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LIST OF FIGURES

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Governors Island Slip 6&7
Mechanical and Electrical Rehabilitation

Ryan Kanagy, Stafford Bandlow Engineering
and
Dan Weida, Kiewit Infrastructure Co.

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

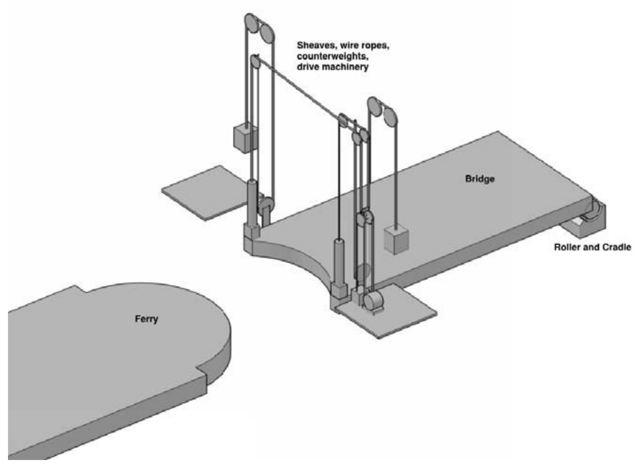
Introduction

Project Overview

Governors Island is located in the New York Harbor between the Boroughs of Manhattan and Brooklyn. The island has been a military hub dating back to the late 1700s and is currently being redeveloped to host a variety of attractions open to the public. The only means of transportation to Governors Island is by ferry via the terminal (slip) located at the Battery Maritime Building adjacent to Battery Park in lower Manhattan. Transfer bridges are in place for traffic traveling on and off the ferry at varying tides. Since the ferry boat is the only means to the island for the nearly 700,000 passengers and 43,000 vehicles that travel on the ferry annually, the reliability of the mechanical and electrical systems for the transfer bridges are crucial.

View of the transfer bridge from the harbor

The Governors Island Ferry Bridges have elements of both the bascule style and vertical lift style bridges. The components to move the bridges consist of an arrangement of wire ropes, sheaves, counterweights, motors, and electrical operational equipment. The land end of the bridge has rollers that sit in a cradle while the other end of the bridge extends out over the water and is supported by wire ropes and balanced with counterweights. The operators use a control station to position the bridge on top of the decks of vessels and then physically join the bridge system with the ferry by utilizing a system of mooring ropes. The mooring system utilizes "live load" counterweights which passively maintain a constant tension on the mooring ropes while allowing vertical movement of the slip due to wave action, freeboard as the vessel is loaded or unloaded and live loading of the slip without placing any load on the operating machinery.



Layout of the Ferry Transfer system

The ferry slip machinery systems were determined to be at the end of their useful life based on a detailed inspection of the components and a review of the recent operation history. A complete mechanical and electrical system rehabilitation design was provided to ensure long term reliable operation of the systems.

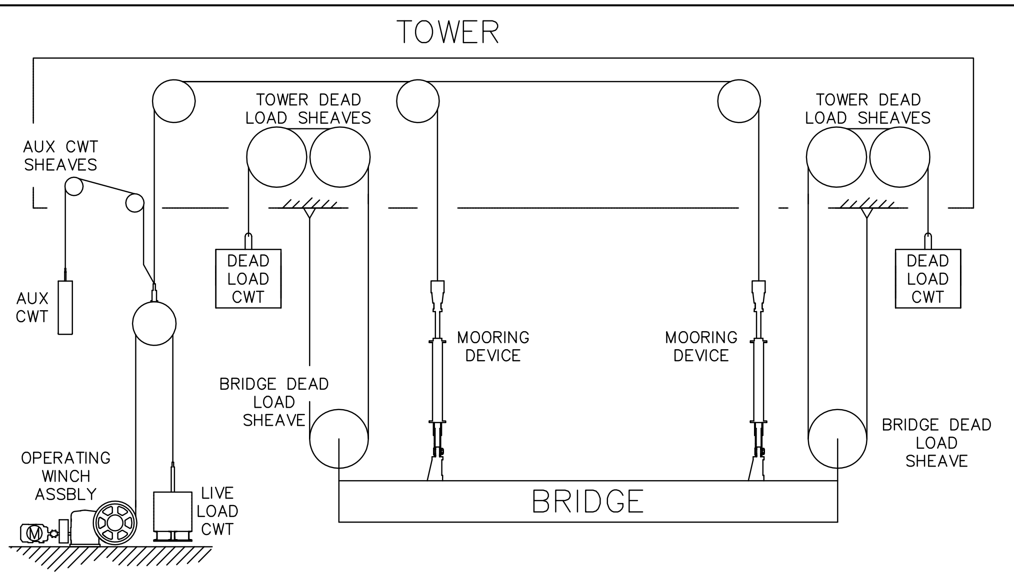
This paper provides a complete review of the entire process leading to the rehabilitation of the mechanical and electrical components that comprise the two Ferry Transfer Systems at the Battery Maritime Building (called Slip 6 & 7, BMB). This review includes inspection findings and important mechanical design details. In addition, the discussion also summarizes important construction details including equipment procurement, construction challenges, sequence of work, temporary bridge support and load transfer, the removal and installation of the slip drive machinery, dead load and live load counterweights, sheaves, mooring posts, and finally the bridge balancing and testing of the BMB Ferry Transfer Systems.

Machinery System Concepts

Machinery Arrangement

The mechanical machinery for each slip at the BMB can be separated into two main systems as described below and shown in the provided schematic.

- Dead Load Counterweight System: A dead load counterweight is connected to the transfer bridge via wire rope which passes over sheaves in the towers and also a sheave attached to the bridge.
- Operating Winch and Live Load Counterweight System: An operating winch is connected to the live load counterweight by wire ropes which pass over a floating sheave assembly. Wire ropes run from a termination at the floating sheave assembly, over sheaves in the towers and terminate at mooring devices mounted to the transfer bridge. In addition to these two main systems, auxiliary counterweights, sheaves and ropes are provided to maintain a minimum tension in the system and also as a means of indicating limit switches.



Ferry Slip Transfer Bridge Machinery Schematic

The dead load counterweight system consists of two separate assemblies located on either side of the transfer bridge. Each assembly is made up of a counterweight, a wire rope, two tower sheaves, and a bridge sheave mounted at the toe end of the transfer bridge at deck level. The counterweight assembly is located within the framework of the tower outboard of the bridge. The wire rope is attached to the top of the counterweight assembly via a socket connection, runs vertically to the top of the tower, wraps around the tower sheaves and runs back down toward the transfer bridge. The wire rope then wraps 180° around the span sheave and runs vertically back towards the top of the tower, where it is anchored to the tower framework. The two part line connection to the bridge reduces the size of the required counterweights by half. The counterweights slide up and down along guide rails as the bridge is raised and lowered. The weight of the suspended counterweight assembly is intended to counterbalance all but a small portion of the transfer bridge weight.

The operating winch and live load counterweight system consists of operating machinery (motor, brake, reducer, open gearing), a winch drum, five wire ropes, a floating sheave assembly, live load and auxiliary counterweights, four tower sheaves, two auxiliary sheaves and two mooring devices. The operating machinery rotates the winch drum, which takes in or lets out two separate wire ropes. These ropes run vertically upward, wrap 180° around the floating sheave assembly, and run back down where they are connected to the live load counterweight assembly via another equalizer bar. The floating sheave assembly is suspended by two operating wire ropes and the auxiliary counterweight rope, all of which are attached to the floating sheaves via an equalizer bar. Both operating ropes run vertically to the top of the tower. One operating rope wraps 90° around each of two adjacent tower sheaves, and runs vertically down toward the transfer bridge. The other operating rope wraps 90° around one tower sheave, runs horizontally through the roof level of the tower to the opposite side of the transfer bridge where it wraps 90° around another tower sheave, and runs down toward the transfer bridge. At the bridge level, each operating rope is terminated at the mooring device plunger. The auxiliary counterweight rope is connected to the floating sheave equalizer bar and travels vertically towards the top of the tower, where it wraps 90° around each of two sheaves, and runs vertically down, and is connected to the suspended auxiliary counterweight assembly.

The live load counterweight assembly is located in the same tower as the operating machinery. As the winch drum rotates, the floating sheave assembly is raised or lowered. The auxiliary counterweight is present to counterbalance the weight of the floating sheave assembly and prevent the live load counterweight ropes from going slack. If no vessel is present, the live load counterweight rests on spring supports and the transfer bridge raises or lowers via the operating ropes. When a vessel is present, the transfer bridge is lowered until it rests on the deck of the vessel, at which point the plungers on the mooring devices lower and pay out the mooring hooks. When the mooring hooks are connected to the vessel, the winch and therefore the direction of travel at the mooring device is reversed. When the slack is removed from the mooring hook cable, tension builds in the operating ropes until the force acting on the floating sheaves is sufficient to lift the live load counterweight. By suspending the live load counterweight, the system ensures that a constant tension is maintained in the mooring devices when the transfer bridge raises and lowers with the vessel under the influence of live load, waves or tidal changes.

In some ways the ferry slip transfer bridge is similar to a winch operated bascule bridge however there are significant differences that impact the design of the machinery systems:

1. Single Direction Loading. Movable bridges are typically provided with provisions to drive the bridge in both the opening and closing directions. The Governors Island ferry slip transfer bridges are only provided with machinery to provide an upward force on the bridge. To lower the bridge it is imperative that the bridge imbalance exceed the system friction; gravity must pull the slip in the lowering direction. A major impetus for the system replacement was the fact that system friction had increased to the point that operation was no longer reliable in cold weather.
2. Load Limitation for Raising and Lowering Bridge. An advantage of the live load counterweight limited system is that it provides an upper limit of loading during when raising and lowering the bridge while allowing for relatively higher loads when the a vessel is moored to the structure. When a vessel is not moored, allowable operation loads due to imbalance, friction, and other loads external

loads are limited to twice the weight of the live load counterweight assembly acting at the operating ropes. If the combination of imbalance, friction, and/or external loads result in operating loads that exceed twice the weight of the live load counterweight assembly, the bridge will be inoperable as the live load counterweight would simply be lifted with any attempt to operate the drive winch. This system can be useful as it provides a physical indication to maintenance and operation personnel of significant changes in operation loads either due degradation of equipment, excessive external loading, or other issues.

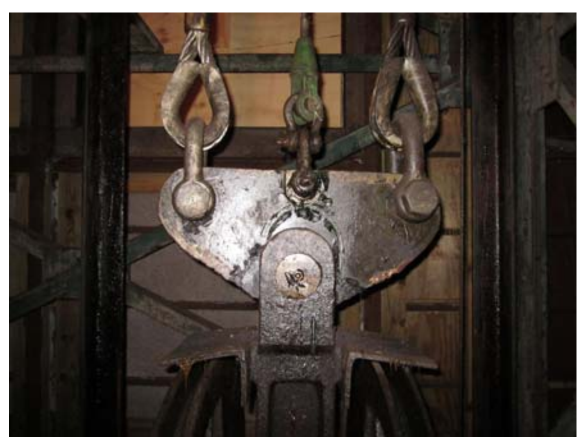
3. Automatic Bridge Movement to Match Vessel While Moored. When a vessel is moored the live load counterweight serves the very important role of allowing the bridge to move with the vessel when it is subject to wave action, live loads, and to other movements due vehicle and passenger movements.

State of Existing Mechanical Machinery

A detailed inspection of the mechanical and electrical systems was performed to assess the condition of the machinery systems. In addition, operation and maintenance personnel were interviewed to understand their perceptions of the issues at the machinery and their general satisfaction with the system in general. As a whole, operation and maintenance personnel found the machinery systems to be simple and reliable but significant recent issues indicated that the machinery was approaching its useful life. Significant noted issues included:

Additional Ballast Placed on Top of Live Load Counterweight to Increase Capacity for Raising and Lowering the Transfer Bridge

- High friction issues at some of the machinery.
  As noted above, the design of the live load counterweight system limits the operation loads when a vessel is not moored. At one slip, additional ballast was added to the live load counterweight in an attempt to increase the capacity of the machinery to raise and lower the span (see photo). Maintenance and operation personnel reported significant friction variation depending on the weather. At times, even with additional ballast on the live load counterweight, the slip was inoperable. Even if the operability could be maintained, the high friction resulted in increased loads throughout the drive machinery.



The distance between the grease witness mark on the live load equalizer plate and the floating shaft assembly indicates excessive wear.

- Although the extent of wear of many of the components could not be verified without disassembly, there were visual external indications of substantial wear at many of the components (see photo). In addition, maintenance personnel reported heavy wear of some components when previously disassembled for inspection.

- The existing dead load tower sheave/shaft assemblies were simply supported in two plain bearings. Although operation records were not available and it was therefore impossible to determine the remaining fatigue life of the shafts, the fatigue life of the existing shafts was clearly finite. A significant contributor to the limited fatigue life was that minimal fillet radii were used at shaft diameter transitions. This is a similar issue that is often found at vertical lift bridge main counterweight sheaves of a similar vintage.


Damaged rope at Slip 7 auxiliary counterweight

- Maintenance indicated varying service life of the wire ropes used in the machinery. An analysis showed rope stresses in excess of AASHTO specifications and sheave/rope diameter ratios that were not ideal. Wear and other damage was noted at some ropes including a heavily damaged Slip 7 auxiliary counterweight rope (see photo)

In addition to the significant issues note above, there were issues of a more minor nature noted at multiple components including the following:

- Live Load Counterweight Buffers: damaged plates and springs at the live load counterweight buffer assemblies (see photo)
- Counterweight Guides: the top counterweight guides for the dead load and live load counterweight assemblies were damaged and, in places, not engaged with the guide rails (see photo)
- Wire Ropes: There was poor documentation of important rope and sheave details leading to a risk of shortening of rope life due to poor matching of the two.
- Span Drive Brakes: The brakes were of an obsolete design.
- Span Drive Gears: Gear tooth damage at Slip 7 pinions indicate that the gear set may have been overloaded.


Damaged live load counterweight buffer assembly


Damaged live load counterweight guide components

Design Philosophy for Rehabilitation

Replacement of all machinery components was recommended to provide long term reliable service of the slips. Because the existing system had generally functioned well a significant modification to the mechanical design scheme was not warranted. To the extent possible, given existing space constraints, the design of select components was upgraded to provide significant improvement of to the reliability and durability of the system as a whole. Significant modifications to the mechanical design are detailed below.

Modification 1: Utilize Rolling Element Bearings Instead of Plain Bearings at the Tower Dead Load Counterweight Sheaves

The most significant issue with the existing system was the reported operational issues due to high friction at some slips. The capacity of the existing drive system and the weight of the existing live load counterweight were compared to expected loads due to imbalance, system friction, inertial loads, and external loads from 2.5 pounds per square foot (psf) wind and 2.5 psf ice loading.

Calculations demonstrated that the weight of the live load counterweight assembly was marginal compared to the calculated loads at the existing system. Although these calculations include theoretical elements such as external loads and friction coefficients, the concern about the capacity of the existing systems was substantiated by operational issues, specifically at the one slip where increased friction loads under certain conditions has resulted in added weights to the live load counterweights and an inability to operate the slip.

Additional margin could have been gained by increasing the weight of the live load counterweight assembly though this has the negative effect of increasing the machinery loads whenever a vessel is moored to the slip. For this reason, the total loads on the system were reduced by using rolling element bearings at the dead load tower sheaves in place of the existing plain bearings.

Modification 2: Redesign the Tower Dead Load Sheave Shafts for Infinite Fatigue Life

The existing dead load tower sheave/shaft assemblies were simply supported in two plain bearings. Although operation records were not available and it was therefore impossible to determine the remaining fatigue life of the shafts, the fatigue life of the existing shafts was finite. A significant contributor to the limited fatigue life was that minimal fillet radii were used at shaft diameter transitions. This is a similar issue that is often found at vertical lift bridge main counterweight sheaves of a similar vintage.

The new dead load tower sheave shafts were designed with infinite fatigue life. The shafts were changed from rotating shafts supported by two plain bearings to stationary shafts with rolling element bearings at the sheave.

Modification 3: Select Ring Gear and Pinion Materials to meet AASHTO

Design of the operating winch ring gear and the driving pinion matched the existing components. The gear materials, however, were selected to meet AASHTO design specifications to ensure long term service.

Modification 4: Upgrade Sheave Diameter / Wire Rope Ratio to the Extent Possible

Selection of the wire ropes and the rope sheaves dimensions involved careful consideration of a number of details with the constraint that it was not possible to make structural modifications that would affect the historically important Battery Maritime Building. The optimum combination of rope diameter, sheave diameter, and rope materials was determined by careful consideration of sheave diameter restraints due to the existing structure, and rope stresses due to the combined effect of direct load and bending load.

Modification 5: Upgrades to the Live Load and Dead Load Counterweight Guides

The existing upper guides at the dead load and live load counterweight assemblies were ineffective due to damaged components. Based on the mass of the assemblies, these upper guides appeared to simply be under-designed. The new counterweight assemblies were provided with stronger upper guides incorporating bronze wear shoes that provided adjustability at installation. An oil impregnated bronze material was selected to minimize maintenance requirements.

Construction Details

The scope of the construction phase of the project was to procure new equipment and to remove/install all of the electrical and mechanical equipment for the two ferry systems with an original contract amount of $6.8M. The procurement phase of the job started in January 2012 with the components being delivered a year later. All of the onsite removals and installations took place during the public access off-season starting October 1, 2012 and finishing May 31, 2013.

Equipment Procurement

Mechanical

The mechanical equipment consisted of the following:

- Slip drive machinery (motor, brake, reducer, pinion shaft, ring gear/winch drum)
- Sheaves (14 ea. 17-57" OD)
- Wire ropes (1/2"to 1-1/2")
- Mooring devices (components that mate the ferry to the slip and live load system)
- Counterweights (about 81,000 lbs./slip)

Steward Machine Co., Inc. and Hardie-Tynes Co., Inc. completed the shop drawings, fabrication, machining, assembly, and testing for all the equipment at their shops in Alabama. Due to the tight project schedule, the entire process, from shop drawings to delivery of the machinery equipment, had to be finished in about a year. Most material consisted of standard A36/A709 steel with a few forgings on critical components. The slip drive machinery was assembled on a single skid and shop tested for functionality before it was delivered to the job site.

Electrical

The electrical equipment supplied by Benfield Control Systems Inc. included:

- Motor control cabinet
- Operators control stations
- Disconnect switches
- Limit switches
- Variable frequency drive
- Programmable logic controller

Major upgrades in the new system included a touch screen, operator safety features at the control station such as an E-stop button, dead-man joy-stick, and alarms and strobes. A smooth transition to this new system, which integrated more modern technologies, was important for all parties involved.

Removal and Installation

Construction Challenges

From a construction point-of-view, this project was technically difficult and required detailed planning and coordination. Major concerns during any construction project are the safety of the craft and public, producing a quality product for owners, and making sure the project finishes on time. However, at Slip 6&7, there were other problems to solve, including accessing the work in difficult areas, moving heavy/bulky objects with limited mechanical equipment, and major engineering involvement in almost every operation.

Some areas of the building are over 100 years old and required major access improvements to work safely and productively. Existing walkways were widened, handrails were installed to eliminate falls and water hazards, 24’ and 36’stair towers for each slip were erected, existing steel members in the overhead tower were removed to create a more ergonomic work area, and five sets of staircases were built throughout the project.

Accessing the work areas with a piece of equipment was nearly impossible and every component being replaced was too heavy to safely move by hand, consequently the team devised several different methods for moving these objects that required designs and in-house engineering approvals. The engineering that was needed to ensure safe operations included supporting the bridges, access, temporary beams to lift from, slab and foundation analysis, lifting plans, and shoring and jacking towers. Planning was started early with involvement from the foreman, company Professional Engineers, and project supervision. The success of the project was dependent on all the planning and communication early on from the engineering department to the craft building the work. Unique tools, such as a magnet (max cap. 4400 lbs.), conveyor rollers, beam trolleys, Teflon, winches, electric and manual chain-falls, and come-a-longs had to be used during construction due to the tough access and existing conditions.

The specific challenges encountered will be detailed in each of the following sections.

36 ft. stair tower erected to access the top towers where the sheaves are located (left photo). 12 ft. staircase built and existing platform decked over with OSHA planks with handrails installed (right photo – looking down).

General Sequence of Work

Space was very limited in the work areas so the sequence of work was very systematic. The general idea was to remove all the items that would be in the way of other removals starting at the bottom and working up vertically. The removals were completed as shown below:

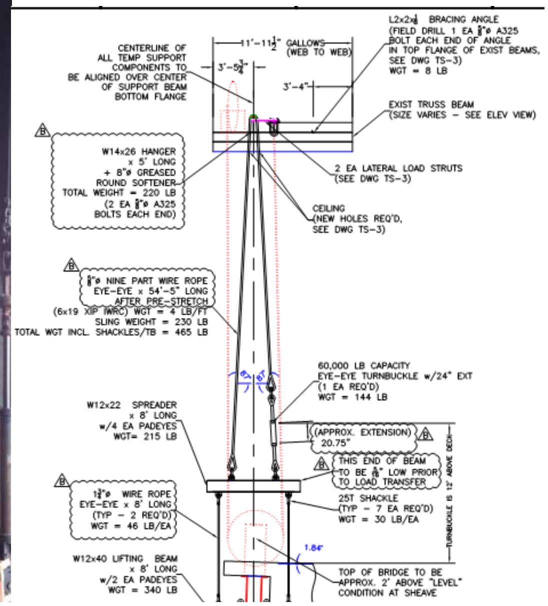
1. Build access to the work areas
2. Temporarily support the bridge, transfer the load of the bridge to the support system and support the counterweights on the shoring towers
3. Power down the ferry slip to remove electrical components
4. Remove wire ropes
5. Remove the Slip Drive Machinery to create room for other removals
6. Remove the Dead Load Counterweights to clear the "shaft" for the sheaves to be removed
7. Remove the Dead Load Counterweights because all of the sheaves needed to go down the same opening
8. Remove the operating rope sheaves and transfer to the Dead Load opening/shaft
9. Remove the Live Load Counterweights
10. Remove other miscellaneous items floating sheave, auxiliary sheaves, auxiliary counterweights, bridge sheaves, and mooring posts
11. Repeat the removals of the dead load counterweight, dead load sheaves, and operating rope sheave for the opposite side of the slip

The installation was completed in a reverse fashion starting at the top and working down. The main difference was once all the components were installed the bridge was tested and balanced.

The main removal/installation operations covered are as follows:
- Temporary Bridge Support & Load Transfer
- Slip Drive Machinery
- Dead Load & Live Load Counterweights
- Sheaves
- Mooring Posts
- Bridge Testing and Balancing

Temporary Bridge Support and Load Transfer

In order to replace the dead load counterweights and wire ropes, the cantilevered end of the bridge needed to be temporarily supported to safely remove these items and to ensure the bridge did not fall into the water during removals. Utilizing the existing holes in the ceiling and overhead structural steel, a designed wire rope and beam support system supported the bridges (weighing between 225 kips and 250 kips). The design consisted of a support beam, wire ropes, a spreader beam, the main wire rope cable, and a beam overhead in the towers for each side of the bridge.



Installing the bridge supports (left) and design drawing (right)

A combination, shoring tower/jacking system was used to transfer the load off the Dead Load Rope cables and counterweights to the temporary bridge supports. This system consisted of a shoring tower with screw legs and an independent jacking system, using 4 25T hydraulic jacks with 14" stroke. The jacking tower was capable of being raised with a pinning system if the 14" stroke on the jack was not enough to transfer the load. The jacks were hooked up to a single pump allowing for an even, simultaneous load transfer between the east and west counterweights. The jacks were raised 1" at a time then supported by the beams on screw legs. This process was repeated until the dead load ropes became slack and the load of the bridge was transferred onto the temporary supports.

Jacking and shoring towers with jacks in place.

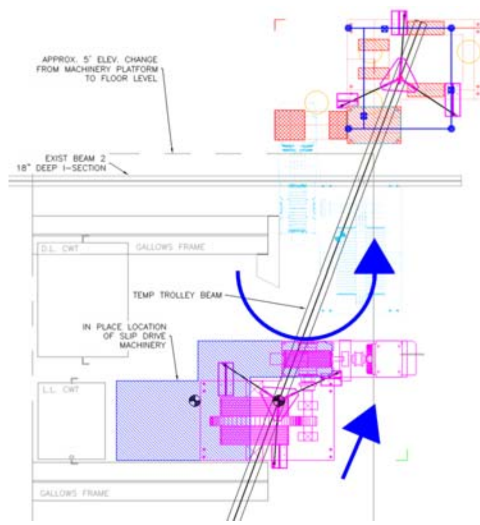
Slip Drive Machinery

There were several obstacles involved with the slip drive installation and removal operations, including a temporary door that was too small for the slip drive to fit through, little to no details on the existing concrete floor slab and foundation pile locations, limited space to move a large object and trying to maintain machinery tolerances throughout the installation.

The Slip Drive Machinery consists of the motor, brake, reducer, pinion gear, and the ring gear/winch drum. The Slip Drive Machinery was shop mounted on a 10'x 6' bed plate, 4.5' high, and weighing 13kips. Extensive planning helped recognize the benefits to remove this first and install the new skid last to create more room in the tight work area. The existing structural steel and building, as well as the weight of the skid, created a situation preventing equipment use during these operations. The foreman brainstormed the idea to install an overhead trolley beam and chain-falls to lift and slide the drive machinery in and out of position. The engineered, 3-point rigging system was utilized to lift the machinery over its center of gravity and eliminated overhead lifting frames. In order to move the existing machinery it had to be raised with the chain-falls, then simultaneously rotated 90̊ and manually pushed towards the five foot drop to the custom fabricated cart on the ground level (same process in reverse for the new machinery).

The engineered cart was used to wheel the drive machinery skids around and designed to distribute the load evenly over the existing concrete slab and capable of being manually moved. Existing drawings of the building could not confirm the reinforcement in the floor but there were known piles below which could support the weight of the skid with a distributed load.

Installation of the new slip drive.



Drawing detailing the angle of the trolley beam and travel path of the slip drive

Dead Load and Live Load Counterweights

Both counterweight systems had to be removed and replaced in-kind. The Dead Load Counterweights are used to balance the weight of the cantilevered-end of bridge, 68 kips/bridge were installed with the similar weight being removed.

The Live Load Counterweights are engaged during mooring mode to balance the loads traveling on/off the ferry, 13 kips/bridge. The weights were removed/installed one at a time using magnets, chain-falls, and roller conveyors to set the weights into position, with the aid of a small forklift to load out/stage the weights on the bridge. Each main block weighed about 2125 lbs. for the Dead Load system and 512 lbs. for the Live Load.

At Slip 7, portions of the exterior wall and internal bracing angles needed to be removed to move the weights in and out. The bridge handrail was removed and the area between the bridge and the structure was decked over to eliminate the water hazard.

Forklift setting a counterweight onto conveyor with a magnet.

Sheaves


Removing/lowering existing sheaves

The sheaves are key in the overall function of the bridge operation for guiding the ropes between the different systems and the bridge. There were a total of 14 sheaves installed that ranged from 17”-57” in diameter with some assemblies weighing about 2000 lbs. The existing steel and low overhead clearance in the towers made material handling very difficult. Special pants with knee pads built in were bought for the entire job team for an overall safety benefit but specifically for crawling from one end of the towers to the other underneath all of the bracing angles (see photo below). The removal/installation had to be sequenced in a particular order since all the sheaves had to go down the same opening. A variety of tools were used during this phase of work, including an electric winch, come-a-longs, trolleys, and Teflon.

The overall alignment of the sheaves is vital for the operation and longevity of the system to reduce wear on the wire ropes and sheaves. The alignment was checked by using monofilament line and clamps while also making sure that the supports were still centered over the webs of the existing steel beams. Once all the components had been aligned, the sheave shaft support bases had to be drilled and reamed in the field for a complete connection between the sheave supports and the existing steel beams.


New operating rope sheaves installed and the difficulties of working around the existing steel members

Mooring Devices

The Mooring Devices are the connection point between the ferry operating system and the vessel. The two are mated with a hook and a wire rope that runs inside the mooring post to a piston-like piece, connected to the operating ropes that terminate at the floating sheave. The floating sheave is the pivot for the slip drive winch and live load counterweight. The mooring devices were relatively simple to remove/install by unbolting the bases and lifting with a winch overhead. The bases needed to be field drilled to match the existing bridge hole patterns. Accessing the underside of the bridge was hard. Since that part of the bridge is suspended over the water, a platform to get under the bridge was built. Also, the work to unbolt and bolt the mooring posts needed to be completed at low tide.

Bridge Balancing and Testing

After all the components were installed, the bridge balancing and operational tests were performed. Movable bridge balance measurements are typically done using the strain gage method. Because there are no live load supports to provide an easy method to free up the machinery and calibrate for zero load, load cells were temporarily installed between the mooring posts and the operating ropes. Load cell recordings were taken as the bridge was raised and lowered to its operating extents. The steady state portion of the load cell recordings were used to determine the system imbalance in a manner similar to strain gage balance analysis. Once the numbers had been analyzed, a calculated amount of counterweight plates were removed to achieve the final imbalance target. After the final balance tests, operational tests, including vessel mooring, were performed to prove out the full functionality of the new systems.

Summary

The rehabilitation of the mechanical and electrical systems for the Battery Maritime Building ferry slips, which provide access for transportation to Governors Island, was completed in 2013. The intent of the rehabilitation was to restore the functionality of the existing systems as a detailed inspection demonstrated that the components were at the end of their useful life. The design of the mechanical machinery systems matched the existing with the exception of significant upgrades to address several issues, including high system friction that had been noted.

The machinery installation involved extensive planning and coordination to address construction limitations due to the location of the ferry within the historic Battery Maritime Building. As a result of the detailed planning, and with a collaborative effort on behalf of all parties, the installation was a success. Both ferry slips are currently being used on a daily basis and are scheduled to service the island seven days a week starting Memorial Day weekend.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Lea Joyner Bridge Rehabilitation

Kelly M. Kemp, P.E.
Louisiana Department of Transportation and Development

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Prior to Rehabilitation

Background

The Lea Joyner Bridge was built in 1936 over the Ouachita River, and currently carries route US 80. The bridge connects the cities of Monroe and West Monroe, which are located in northeastern Louisiana. The total bridge length is 1170 ft. The cross section provides two (2) 6-ft sidewalks and a 40-ft clear roadway consisting of four (4) 10-ft vehicle lanes. The current ADT is approximately 38,000.

The structure consists of the following spans:

- One (1) 160-ft Double Leaf Bascule Span
- Six (6) 100-ft Steel Deck Truss Spans
- Nine (9) 40-ft Concrete Girder Spans
- Two (2) 20-ft Steel Girder Spans Over the Counterweights

South Elevation Before Project

Deficiencies

The project initiated with the determination that the electrical system needed rehabilitation. As the Department began assessing the structure, it became obvious that a larger scale project was needed.

The bridge was load posted and needed structural repairs, cleaning and painting. Section loss was observed on many steel members, including lower chords of main trusses and floor trusses. In some cases, section loss was caused by debris pockets, especially in lower chord webs at splices. The machinery support beams and supporting trusses, one of which braces the bascule main trunnion support towers, were found to have many deteriorated members that needed either replacement or repair. Most of the deck truss spans rocker and fixed bearing anchorages had bolts that had completed corroded away, making it unlikely that the spans had sufficient lateral restraint. Of all the locations in Louisiana, the bridge is located in one of the more likely earthquake areas according to the AASHTO seismic design section.

On the concrete girder approach spans, most of the girders exhibited cracks in the expansion bearing areas. A few of the concrete girder spans and deck truss spans were pushing together. Most of the roadway and sidewalk joints needed replacement, sealing or both. Leaking joints contributed to section loss in steel stringers and deck underside spalls with exposed and corroding reinforcing steel.

The concrete post and rail bridge railing was not up to current standards, and was usually sustaining significant damage during vehicle strikes. Both the concrete deck and the bascule span steel grid floor were worn, which reduces traction.

Even though the bascule span drive machinery appeared to be in good working condition, it was determined that the span was not balanced properly, and was in the "toe light" condition. It was decided that the system would need adjustments and that the gears would be removed, shop cleaned, inspected, and painted, which would coordinate well with the need to repair the machinery support structure.

The operator houses were in need of architectural improvements, including painting and new doors and windows. The clay tile roof and gutters on each house were in poor condition as well as the interiors of the houses, which included some non-functioning sinks and toilets. The walkway/roadway lighting was in need of work, was unsightly, and didn't even consist of uniform fixture types. The navigational lighting system needed repair, and much of the timber fender system had deteriorated.

With the age of the bridge, and the high traffic demand, bridge replacement was considered. The Lea Joyner Bridge is one of only three vehicle crossings between the two cities, with the other two being Interstate 20 and the nearby DeSiard Street bridge. The Lea Joyner Bridge carries the highly developed 4-lane streets of Louisville Avenue in Monroe and Bridge Street in West Monroe. Due to estimated bridge replacement cost of $80 million, right-of-way and environmental impacts, traffic demand on the two adjacent structures, historic value of the existing bridge, and the time required to develop a replacement project in light of the immediate needs at the site, it was decided that rehabilitation was the best solution.

Project Team and Work Scope

The rehabilitation was performed as two separate projects, referred to as Phase 1 and Phase 2. Both projects utilized the conventional design-bid-build process. The phases, designers and contractors were as follows:

- **Phase 1 -** State Project 001-09-0074

  Designers: Huval & Associates, Inc. (Structural)
  Modjeski & Masters, Inc. (Electrical)
  LA DOTD Bridge Design Section (Architectural)

  Contractor: Kiewit Louisiana Co.

- **Phase 2 -** State Project 001-09-0075

  Designer: LA DOTD Bridge Design Section

  Contractor: PCL Civil Constructors, Inc.

The project consisted of structural, electrical, mechanical, and architectural work as follows:

Electrical work consisted of the following:

- Electrical System Replacement (230 V changed to 480 V)
- Control System and Switchboard
- Drive Motors and Brakes
- Conduit and Wiring
- House Lighting
- Traffic Signals and Gates
- Navigational Lighting
- Generator
- Submarine Cables
- Walkway Lighting

Structural work consisted of the following:

- Concrete Repairs
  - Cracked Caps and Girder Ends
  - End Wall Repairs and Added Girder Supports
  - Major Spalls on Caps and Deck Undersides
  - Jammed Fascia Joints
- Timber Fender System (Partial Replacement)
- Deck and Sidewalk Joint Seals
- Added Seismic Lateral Restraints to Deck Truss Spans
- Steel Members (Replace, Strengthen, or Repair)
- Steel Connections, Splices and Stiffeners (Replace, Strengthen or Repair)
- Replaced Deficient Rivets with High Strength Bolts
- Clean and Paint Structural Steel
- New Steel Grid Floor with Partial Concrete Fill
- Deck Epoxy Overlay
- Added Steel Curb Rail and New Concrete Curb
- Replaced Approach Slabs and Back Walls

Mechanical work consisted of the following:

- New Gears
- New Machinery Houses
- Live Load Shoes - Clean and Adjust
- Center Span Locks - Clean and Adjust
- Clean and Paint Machinery and Machinery Houses
- Temporary Operating System (for Gear Replacement)
- Bascule Span Balancing

Architectural work consisted of the following:

- Four (4) Operator Houses Refurbished
  - New Doors and Windows
  - Roof and Gutter Repair
  - New Stairway Hatches
  - New Louvers and Exhaust Piping
  - Removed Sinks, Toilets, Piping, Valves
- Special Surface Finish Applied to Houses and Bridge Structural Concrete
- Added "Lea Joyner" Plaques at Each Bridge End

Since the project involved a large amount of repair work of various details, the plans included photographs showing the existing conditions on the same sheets as the proposed repair details where helpful.

Construction

Electrical Work

Existing submarine cables, located on the south side of the bridge, were disconnected and removed to just above the waterline. New submarine cables were added on the north side of the bridge. The new cables were required to be located fifteen feet below the mud line. The plans provided options of directional drilling and trenching and/or jetting for submarine cable installation. The plans showed the approximate location of a timber mattress located in the vicinity of the cable installation and specified that costs related to cutting through the mattress would be handled during construction by force account, rather than bid as part of the electrical work.

The entire electrical system and components were removed and replaced.

Old Generator New Generator New Disconnect Boxes

According to the original bridge plans, the existing walkway lighting at the beginning of the project was not original. The new poles and fixtures where designed to bring back a similar look as the original bridge fixtures, while also meeting current lighting requirements for roadways and walkways.

Existing Walkway Lighting

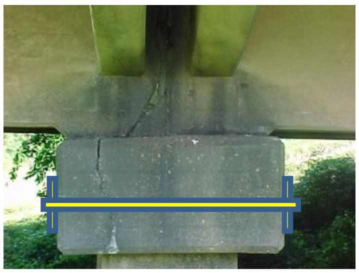
New Walkway Lighting

Structural Work

The concrete approach spans were constructed with simple span cast-in-place decks and girders. Over time, creep contributes to stress concentrations at the bearings that when aggravated by expansion, contraction, and live load rotations, can easily cause structural cracks in the girder expansion ends. Nearly all expansion ends of girders required crack injection repair on this project.

Cracked Girder Bearing

Occasionally, deterioration can extend to the substructure cap. At least one cap required epoxy injection and clamping with plates and bolts.



Annotation Showing
Cap Repair Method

Repaired and Finished Cap

The bridge was the home for many Cliff Swallows. The plans required that required nest removal be performed during times of year where the birds vacate their nests and the nests were verified to be empty.

Cliff Swallow Nests

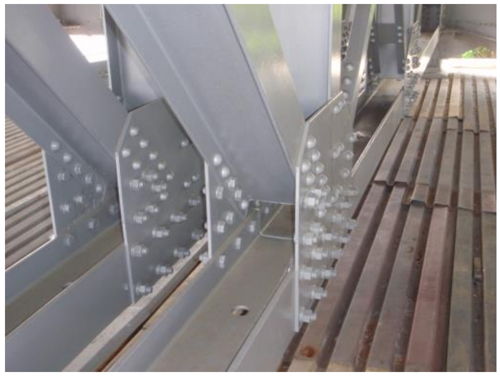
As expected with most rehabilitation projects, the amount of work anticipated from routine inspection reports and inspections before and during plan development increased during construction.

Stringer Web Section Loss Discovered Under Construction

One reason for increased structural steel work is that access to certain elements is limited prior to construction. This leads to basic assessments and findings with planned repair solutions and truly estimated quantities. One example of this is replacing deficient rivets. Another reason is that the operation of abrasive blasting prior to painting often uncovers section loss. Prior to construction, a member believed to only need cleaning and painting, may need to be strengthened. And members believed to only need strengthening, may need to be replaced.

Many truss members were replaced or strengthened, requiring temporary devices to remove load from the replacement member, and sometimes the removal of traffic. All such work requires analysis to determine existing dead loads, loads induced by the repair method, and live loads. Consideration must be given for the redistribution of stresses within a cross section that may have occurred due to the deterioration process. In many cases, strengthening members would only increase their live load capacity, and in some of those cases, dead load represents the majority of stresses, resulting in the need to replace the member.

Replacing members, depending on the repair procedure, can sometimes result in the permanent increase loads in adjacent members.

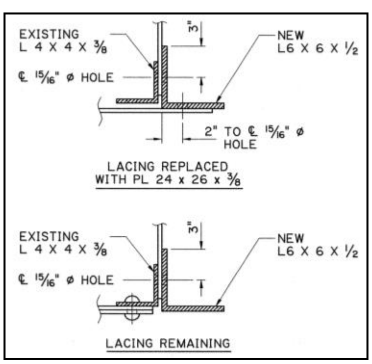


Floor Truss Bottom Chord Replacement

One area receiving work was the machinery support structures. While the gears were removed for shop inspection, the machinery support structures were strengthened. One method used for the support truss top and bottom chords, was to support the truss, remove rivets and replace them with pins, add a new member alongside the deteriorated member with holes drilled to receive the pins, then remove the pins and replace them with high strength bolts. This method was used since completely replacing the structure wasn't necessary and the structure is out of public sight. This method usually requires minimal shoring and can be done with minimal load transfer to members.

Machinery Support Truss

Lower Chord Deterioration



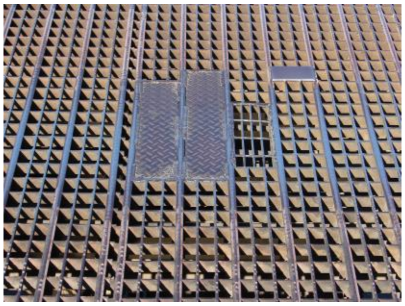
Lower Chord Repair Details

Vehicles occasionally jump the 10-inch concrete curb on this structure, mount the sidewalk, and crash into the concrete post and rail barrier. On one occasion during Phase 1, a passenger truck penetrated the concrete barrier, but was restrained enough to prevent falling into the river. Generally, the concrete posts and rails are repaired several times a year, adding to the bridge's maintenance costs. It was decided that Phase 2 would add a steel curb rail to prevent such occurrences. The rail was designed for appropriate AASHTO railing loads, and a new concrete curb was added to provide the required foundation and anchorage.

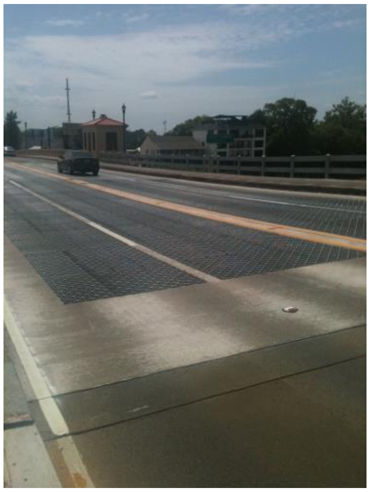
Curb Rail Being Installed on Phase 2

Installed Rail Elevation

The existing steel grid floor had been installed in 1950, and therefore had seen over 50 years of traffic at the onset of the project. It had lost most of its traction and was also falling apart. It was replaced with a welded grid floor having an improved serration design and better section properties. During construction, the Contractor proposed that we distribute the plan required full depth concrete fill and create lane and center lines using half-depth fill. This fill concrete layout would not only help reduce traffic noise, but would provide areas to apply traffic markings. The Department considered this to be a great idea, and implemented their proposal.



Old Grid Floor



New Grid Floor with Concrete Lane Lines

Bascule Span Balancing

On any bascule span rehabilitation, balance will have to be maintained. In the case of the Lea Joyner Bridge, balance was going to be maintained and also modified.

The Lea Joyner Bridge original construction, history, and mechanisms for balancing provided some unique challenges to the plan development tasks of determining existing weight and balance, determining a target weight and balance, and estimating and measuring the final weight and balance.

The Lea Joyner Bridge bascule span is of the Strauss Trunnion type, having a counterweight that pivots on trunnions attached to the two (2) main girders. Review of the original bridge plans provided weight values for the counterweights in terms of reactions at each main girder. The values were shown to be based on the counterweights being two-thirds full of counterweight balance blocks.

The original bridge bascule span was built in 1936 with timber decking and asphalt planks with timber riser beams on top of the steel stringers. In 1950, the decking system and timber risers were removed and replaced with a grid floor, with full concrete fill over the machinery houses and over the exterior stringers, with castellated steel risers on tops of the steel stringers. At some time in the bridge's history, balance blocks were added to the span side of the main trunnions by installing metal trays between the stringers.

Just prior to Phase 2 plan development, it was observed that one counterweight contained no balance blocks, and the other counterweight contained only two (2) blocks. This made sense when observing the blocks that had been added to the spans. Also just prior to plan development, evaluations of span balance using electrical motor resistance revealed that the spans were "tip light," and it was also determined that they had operated that way for quite some time. Strain gauge balancing was performed later to verify this assessment. Therefore, the original balance condition of the counterweights being two-thirds full of balance weight had changed to basically empty counterweights, with weight added to the span in an insufficient amount.

Plans were developed with the goal of the removing the span balance blocks and holders, adding steel curb rail and heavier grid floor, and targeting a 3000-lb toe heavy condition with blocks added back to the counterweight as required. The plans required the contractor to maintain balance throughout the project. There were times where large steel plates had to be temporarily attached to the bascule span hand railing near the toe end of the span.

Mechanical Work

Each leaf of the Lea Joyner bascule span is driven by a main gear box and four (4) open intermediate gear sets. The plans required bids to be based on the gears being removed, taken to a shop, cleaned, inspected, painted, and reinstalled, and that the owner would decide after inspection how to address any needed repairs, refurbishment, or replacement. During construction, due to inspection results, it was determined the gears needed replacement.

New Intermediate Gear Set

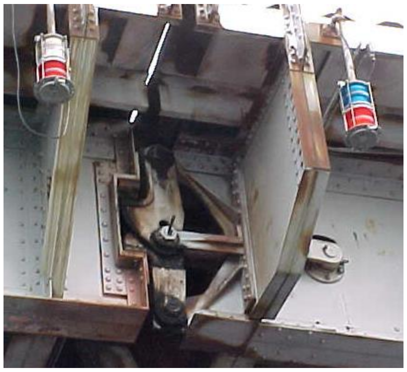
Accompanying the gear removal was the plan requirement for a temporary operating system. The plans required the contractor to design and submit the system for approval prior to installation, testing and operation. The plans recommended a system that pulled on the counterweights from both directions maintaining positive control using cables and motors or pneumatic devices. The plans required the system exhibit both safety and redundancy.

The plans prohibited hydraulic or pneumatic devices from being used to secure the closed or open span. The plans required securing the span to be accomplished by structural static devices capable of restraining the span and counterweight against loads encountered in the closed and open positions. The plans recommended utilizing an external normally closed brake.

System Components Pulling Down on Counterweight

During the gear removal, the machinery support beams were replaced, and the underlying support truss was repaired and strengthened. The support structure was anticipated to receive larger torques due to the increased weight added to the span.

Prior to Phase 2 plan development, the Department had witnessed span free vertical deflections 0.25 inch to 0.50 inches at the center span lock. Therefore, the plans required the live load shoes to be cleaned, adjusted, and shimmed as necessary, followed by the same procedure for the center span lock in order to isolate the source for the movement. If after shimming the center span lock movements still occurred, the center span lock mechanism would be inspected, adjusted, repaired, or replaced as deemed necessary. Under construction, cleaning and shimming was all that was required to achieve proper fit and span support.



Bascule Center Span Lock

Perhaps the most challenging undertaking on Phase 2 was the unplanned replacement of the counterweight trunnion bearings. During Phase 2 construction, increasing bearing noise, followed by discovery that lubrication ports were blocked, led to the decision to replace the bearings. The operation required to remove and inspect these bearings is essentially the same as if following through with replacement. The Contractor's subcontractor, Hardesty and Hanover, designed the temporary support structure for the counterweight and also a replacement of the spliced hanger plates.

Span Bracing System

Removed Bearing Housing

New Hanger Segments

New Bearings

Span Open for Bearing Replacement

Architectural Work

The four bridge houses were completely refurbished. On bridges with historic eligibility, one challenge is determining color, and the local historic preservation office requested the original bridge color. The photo below shows the existing house color to be pink. Sources within the Department confirmed this was not the original color. It was decided to color the houses to match the concrete. It was also decided to have the steel curb rail match the walkway lighting color of black.

Existing House and Lighting

Refurbished House with New Lighting, and Curb Rail

House Rear Elevation

The machinery house in each bascule pier was replaced with an improved design. This included replacing most of the floor plate.

New Machinery House

Project Completion and Results

Timeline

Phase 1:

- Bid Letting		May 2007
- Completion		September 2009

Phase 2:

- Bid Letting		March 2010
- Completion		October 2013

Cost and Breakdown

The total project cost was approximately $26.9 million. The bid amounts totaled to $17 million, with 52% in Phase 1 and 48% in Phase 2. During construction, a total of $9.6 million in additional work was taken on in the form of Change Orders, of which $4 million was to replace the counterweight trunnion bearings in Phase 2. The remaining $5.6 million of Change Orders was split almost exactly 50/50 among Phase 1 and Phase 2 and represents just over 30% of the bid amounts, which is not too surprising on such a rehabilitation project.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

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Lynn River Lift Bridge Rehabilitation

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

LYNN RIVER LIFT BRIDGE REHABILITATION

ABSTRACT

The Lynn River Bridge rehabilitation project involved mechanical and structural upgrades to a 44 year old double leaf bascule bridge located in Port Dover, Ontario, Canada. The already complex project became increasingly challenging when on-site conditions varied significantly for some items of work. All work was completed while maintaining roadway traffic during winter, the period when operation of the lift bridge for marine traffic could be suspended. Jacking of the bridge required special consideration as each bascule leaf had to be restrained in all three axes. All new mechanical components were machined to tight tolerances for proper fit with existing components. The trunnion shafts on which the main bearings are mounted were re-installed horizontally in the field, to achieve an FN1 interference fit connection. An unconventional method using liquid nitrogen to cool the trunnion shafts to minus 195 degrees Celsius was used for their installation. The loaded shafts were restored to their original alignment to an accuracy of 0.25 millimeters. Replacement of the level locks located at mid-span involved the development of a temporary leaf lock assembly to ensure safe passage of roadway traffic. Finally, the rehabilitated bascule leaves were re-balanced for proper operation under live load.

1. INTRODUCTION

Lift bridges are generally the most complex short and medium span bridges. Their design and construction integrates multiple disciplines including Civil, Mechanical and Electrical engineering. Rehabilitation of these bridges is equally complex and Lynn River Bridge was no exception. The unique design features of Lynn River Bridge and some unexpected on-site conditions made this rehabilitation project even more challenging.

Lynn River Bridge is a double leaf bascule lift bridge carrying Highway 6 over Lynn River into the town of Port Dover, Ontario, Canada. The bridge was constructed in 1969 parallel to an older single leaf bascule lift bridge which had reached the end of its service life and was later demolished. The current rehabilitation project was undertaken to primarily address the poor operational reliability of the trunnion bearings of the bridge. This also afforded an opportunity to incorporate additional mechanical and structural improvements as well as to address other less severe issues related to the operation and maintenance of the bridge.

The bridge is operated approximately 2000 times annually with the bulk of the operational cycles between the months of April and October. Hence all construction work was scheduled during the 2012-2013 winter season when the operation of the bridge for marine traffic could be suspended.

2. BRIDGE DESCRIPTION

2.1 General

The bridge has a span of 25.5 metres and an overall deck width of 12.8 metres that includes a 9.14 metres wide steel grating riding surface accommodating two lanes of traffic as well as two 1.83 metre wide raised steel grating sidewalks (Figure 1 and Figure 2). Passage of marine traffic is facilitated by opening the bridge (raising the leaves) providing a 24.4 metres wide channel under the bridge. The bridge is staffed 24 hours a day, seven days a week from April to October. Outside of this period, the bridge is staffed from 5:00 am to 5:00 pm daily.

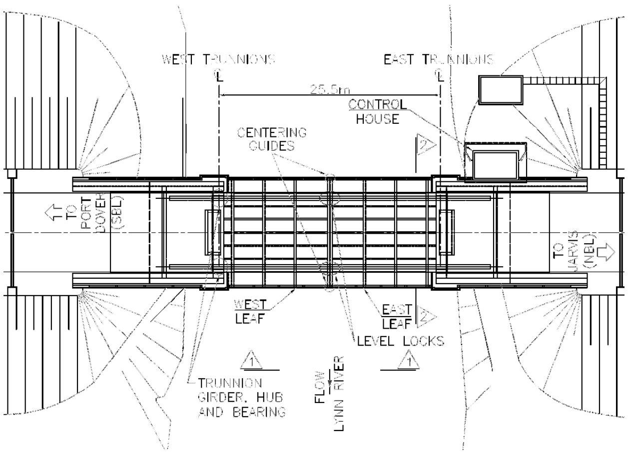


Figure 1. Plan view

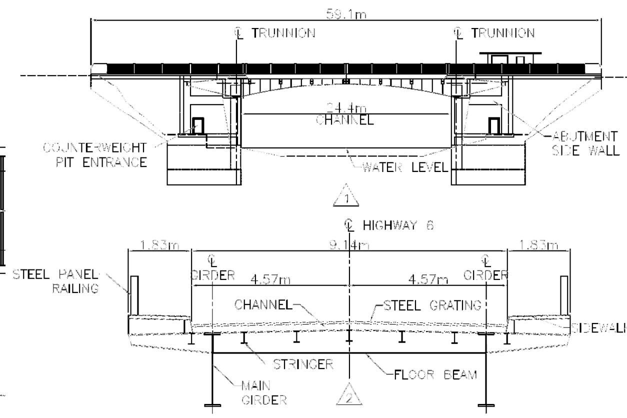


Figure 2. Elevation and section view

2.2 Primary Components of the Bridge

Each leaf is comprised of two longitudinal parabolic steel plate girders (main girders), 0.6 metres deep at mid-span and 2.1 metres deep at the trunnion bearings. Transverse steel floor beams (I-section, 0.69 metres deep) spanning between the main girders are spaced 3.68 metres apart and support longitudinal stringers (W310x39) and transverse channels (C180x15). The channels secure the reticuline steel grating riding surface with “J” hooks. Cantilevered tapered floor beams support the sidewalk which has a steel panel railing barrier system (Figure 2 and Figure 3).

At each abutment (counterweight pit) a transverse trunnion box girder (1.66 metres deep x 0.76 metres wide) spans between the two main girders. There are four steel-on-steel radial spherical plain bearings, two at each abutment. Each trunnion bearing supports one end of a tapered trunnion shaft (1.8 metres long, 330 millimetres maximum diameter, 900 kilograms weight) that is connected to the main girder through a hub assembly and is supported at the other end by the trunnion girder diaphragm. The trunnion girder diaphragm also houses two eccentric sleeves which facilitate minor adjustment in the alignment of the trunnion shaft (Figure 4).

Figure 3. Lynn river bridge – open position

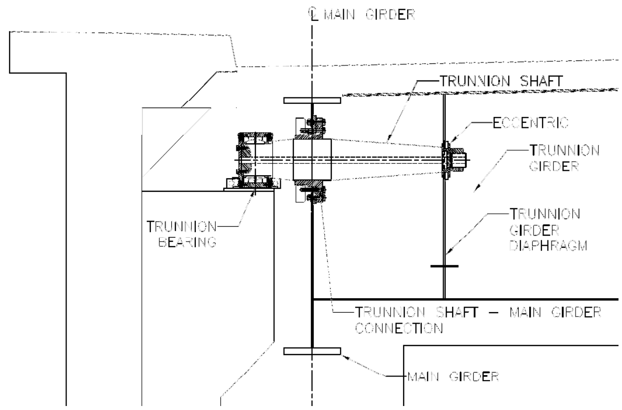


Figure 4. Trunnion shaft layout

Live load and dead load of the superstructure is transferred from the main girder hub assembly through the trunnion shaft to the trunnion bearing. Due to the offset between the hub assembly and the bearing, the trunnion shaft is subjected to bending loads in addition to axial loads.

2.3 Bridge Operation

Each leaf can be operated independently by a set of two hydraulic leaf cylinders located at the abutment. These act almost horizontally near the bottom of the trunnion box girder and are thus offset from the point of rotation. The operation of the bridge is controlled by a Programmable Logic Controller (PLC) and coordinated through radio communication between the two leaves. Counterweights with pockets are located at the abutment-end of each main girder. The weight in the pockets can be adjusted to balance the leaves for optimal operation of the bridge.

In the closed position, the bridge leaves are locked together by two rectangular lock bars which form part of the level lock assembly located at mid-span. In addition, the counterweights are provided with elevation locking pins which can be extended to lock the leaf into either the closed or open position.

Typically, bascule lift bridges are balanced to be span-heavy in the closed position so that they have a tendency to remain closed should the leaf operating machinery and stabilizing components (locks) malfunction or fail. However, Lynn River Bridge is intentionally designed as counterweight-heavy. Though preferred, it was not feasible within the scope of the rehabilitation project to alter the span balance to a neutral or span-heavy condition as this would require fundamental changes to the structural and hydraulic systems of the bridge.

3. DESIGN CONSIDERATIONS

3.1 General

The bridge rehabilitation work was part of the Highway 6 improvement project at Port Dover. Principal design considerations for rehabilitation of the bridge are listed in the following paragraphs. The emphasis of this paper is on the mechanical rehabilitation work, i.e. improvements to the trunnion shaft and main girder connection, replacement of the trunnion bearings, and the installation of new level locks and centering guides.

3.2 Trunnion Shaft and Main Girder Connection

The existing hub assembly utilized RFN 4071 Half Shrink Disk locking mechanism and a field welded hub connection for load transfer (Figure 5). The wedge offset of the locking mechanism resulted in eccentric loading of the main girder and the welded connection was susceptible to cracking due to fatigue. Although the need to improve or replace the trunnion bearings was evident during the preliminary site investigation, the condition that increased concern was the use of the friction collar and welds at the connection of the hub to the web of the main girder. This arrangement also inhibited access for inspection and maintenance of this critical connection.

The original design utilized a lightweight and relatively flexible girder (and bridge) and used the typical Hopkins trunnion arrangement which is very common along the east coast of the United States. The advantage of this arrangement is the elimination of the inboard trunnion bearing thus resulting in a larger angle of opening as compared to a traditional trunnion bascule. The disadvantage is the transverse flexibility of the span and the changing section modulus of the transverse trunnion box girder. In the case of Lynn River Bridge, the box depth in the seated position (1.66 metres) is more than twice its depth in the open position (0.76 metres). This results in differential deflections in the transverse direction and alignment changes in the trunnion bearings during operation of the bridge. Initially, the excessive deflection was controlled by installing a trunnion shaft strengthening device at each trunnion location. During a subsequent rehabilitation in 2006, the original trunnion shafts were replaced with new larger diameter trunnion shafts, except for one newer shaft that was provided with a shrink-fit sleeve to increase its diameter.

As mentioned, the bridge and in particular the bridge girder and trunnion hub connection is a very flexible design. During the initial investigation, it was noted that the already thin (15 millimetres) girder web was modified with welded connections at the hub assembly. Apart from being fatigue prone, this detail was prone to collection of roadway debris at the interface and made the critical connection susceptible to corrosion. It was decided during the design process to improve this detail by stiffening the girder web at this location for the installation of the rehabilitated trunnion shafts (Figure 6).

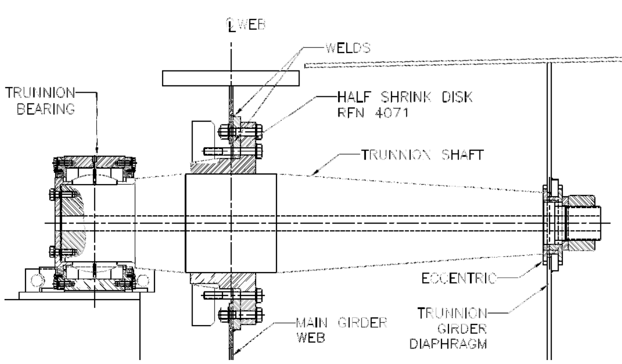


Figure 5. Existing trunnion shaft connection

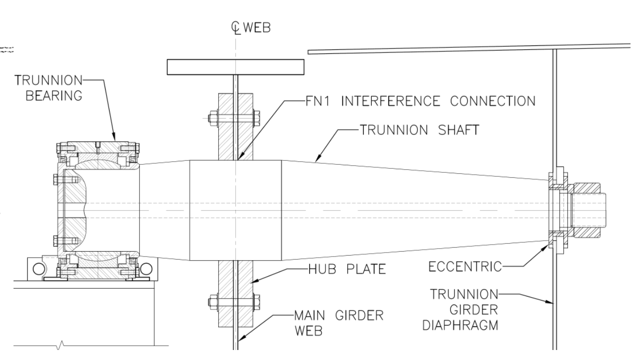


Figure 6. Rehabilitated trunnion shaft connection

The concept of field installed hub plates was unique to this project. Typically, a solid cast hub ring is shrunk into the girder bore in the shop. Further, typically this connection is installed with the girder web lying on the horizontal plane, and the shaft inserted vertically during shop fabrication. In contrast, the Lynn River Bridge rehabilitation involved field installation in a confined work area with the abutment side walls restricting the size of materials that can be installed. The hub plates were hence detailed to facilitate field installation. A bolted connection was designed to transfer the shear load from the hub plates to the girder web thus eliminating the need for shrink fitting of the hub. Locating the hub bore centre was planned by utilizing a temporary alignment shaft and aligning the plates with respect to the inner eccentric centre and previously surveyed monuments. The goal was to achieve the same or better transverse alignment of the shaft than measured at the start of construction.

The trunnion shafts would still need to be removed, machined and shrink-fitted within the hub plates horizontally to achieve an FN1 interference fit connection. Although an FN2 interference fit connection is preferred for trunnion shafts, the design verified that an FN1 connection would provide adequate capacity for the loads at this bridge. This was also more practical from a field installation standpoint when considering the constraints for horizontal installation of the trunnion shafts in a confined work area.

The concept developed for installation of the trunnion shaft utilized dry ice and alcohol for cooling the shaft in a bath cantilevered off the abutment side wall combined with heating of the hub plates using heating elements. The shaft would be inserted through the opening in the abutment side walls constructed during the previous rehabilitation. When the shaft and hub plates normalize, the shrink-fit would provide adequate capacity to transmit the operating torque.

3.3 Trunnion Bearings

The original construction bearings were bronze cylindrical bearings with split, cast steel pillow blocks. During the rehabilitation in 2006, the bearings were replaced with steel-on-steel radial SKF GE 260 ES-2RS spherical plain bearings, which allow only minor shaft deflections and 90 degree rotation of the leaf. The problem with the replacement bearings was the lubrication detail and the steel-on-steel rotating elements. These bearings were prone to lubricant starvation as well as interference of the narrow outer element during extreme thermal expansion conditions of the bridge.

Bridge design codes CAN/CSA-S6-06(1) and AASHTO(2) call for steel inner race (inner "rotating" element) on bronze outer race (outer "fixed" element) for trunnion bearings. In the event of inadequate lubrication, the dissimilar metals perform for temporary periods of time before bearing seizing will occur. Steel on steel bearings have very little tolerance for inadequate lubrication and are therefore not recommended by the current code for bascule bridges. New custom steel-on-bronze radial spherical bearings were designed to replace the existing bearings. The design also included modification of the existing bearing housings to suit the new bearings.

3.4 Level Locks and Centering Guides

Each level lock assembly is comprised of a lock bar held in place by a front and rear guide, all located on one leaf, and a receiving socket located on the opposite leaf. Actuated by a hydraulic cylinder, the lock bar is extended into the receiving socket to lock the leaves together and retracted to unlock the leaves.

The existing level locks exhibited visible misalignment and relative movement between the two leaves of 6 to 8 millimeters under live load. There were no adjustable shoes and no provision for adjustment of the clearance between the lock bar and the mating surfaces of the guides and sockets. The design team detailed new level locks (Figure 7) with adjustable shoes in the housings. The lightweight design of the bridge provided very limited area at the front floor beams to accommodate the new housings (Figure 8). The sockets were also modified to reduce the relative movement between spans under live load and to improve the ride-ability over the bridge. In addition, centering guides were added to the toe of each leaf to ensure transverse alignment of both leaves at the end of each operational cycle. Due to the flexibility of the span, there was a likelihood of misalignment under windy conditions and the centering guides eliminated this problem.

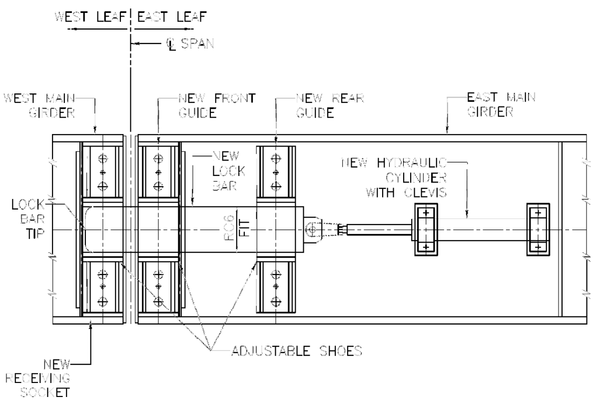


Figure 7. New level lock assembly

Figure 8. Level lock receiving socket

3.5 Miscellaneous Work

Other work which was part of the rehabilitation project but is not covered in this paper included the following:

1) Concrete and structural steel repairs at areas of deterioration.
2) New concrete in-filled steel grating at deck ends for protection of bridge machinery at both abutments.
3) Remediation of hard emergency stop (E-Stop) of the bridge triggered due to unexpected communication failure between the two leaves.
4) Provision of access hatches at the sidewalk level to facilitate maintenance of the new bearings.
5) Installation of grease manifolds at barrier railing level to facilitate lubrication of the level lock components.
6) Troubleshooting issue of cyclic surging of hydraulic pressure in the system during certain stages of the operational cycle.

4. CONSTRUCTION

4.1 Schedule and Weather Constraints

Apart from the need to accommodate pedestrian and vehicular traffic throughout the duration of construction, the bridge provided the only access route for Fire and Emergency Medical Services (EMS). As stated earlier, all work was scheduled during the winter season when the bridge could be closed to marine traffic. These unique requirements necessitated development of a construction schedule involving a total of nine stages for completion of the rehabilitation work.

During design, adequate float time had been included for unforeseen conditions and despite delays during construction, the bridge was re-opened in a timely manner. Major unexpected site conditions which had an impact on the schedule included the following:

1) The trunnion shafts were seized with the hub assembly and extensive preparatory work was required for removal of the shafts.
2) The main girder web faying surfaces had a poor surface profile unsuited for the essential slip-critical connection with the hub plates. This area of the web could be inspected only after removal of the hub assembly.
3) Access and confined space constraints led to a change in the methodology for installation of the trunnion shafts.
4) Poor interface conditions and dimensional variations at the level locks necessitated the development of a temporary locking arrangement and some field adjustments. Due to restricted access, these areas could be inspected only after removal of the level locks.

4.2 Trunnion Survey and Jacking

The survey of the trunnion shaft alignment had to be based on its position with respect to the trunnion box girder and not a typical monument located on the fixed abutment wall. Since the bridge was to be jacked and could move laterally during the jacking process, the shaft position had to be related to the trunnion box girder. Further, the existing hub assembly at the main girder bore would be removed for installation of the new hub plate detail. This removed a critical reference point during construction and added to the complexity in obtaining reliable and accurate survey data. The survey involved measurements with reference to a piano wire strung through the alignment holes of both trunnion shafts and anchored at the abutment side walls. The centreline of the new bore was required to be within 0.25 millimetres of the existing bore as measured during the survey. This survey was in addition to the traditional pre-construction and post-construction top-of-deck and underside-of-girder surveys.

The jacking and span support system used during the previous rehabilitation formed the basis for this project as well. The system consists of a series of steel fabricated elements that support the bottom flange of the bascule main girder directly to the abutment floor at the trunnion shaft location. Since the bearing and shaft were to be removed and traffic maintained for the duration of construction, temporary longitudinal and transverse bracing was detailed to secure the bridge in position. In summary, the jacking system ensured that the bridge was restrained from translation and rotation in all three axes.

4.3 Removal of Trunnion Shaft and Existing Hub Assembly

Removal of the trunnion shafts became challenging when conventional removal methods were found to be ineffective as the shafts were seized with the hub assembly. They were eventually removed by localized heat application of up to 650 degrees Celsius along with an axial pull of over 60 tonnes. The trunnion shafts were subsequently tested to confirm that there was no change in the hardness of the steel due to the application of heat.

Removal of the bolted wedges and the welded hub reinforcement rings revealed some oversized bolt holes, 2 to 4 millimetres deep gouges (partially due to improper weld fusion) and global distortion of the thin web plate of the main girders. This further validated the decision to replace the hub connection as part of this project.

4.4 Fabrication of Components

The trunnion shafts and bearing housings had to be removed from the bridge and transported to a local machine shop for machining and fit-up. These components were carefully inspected in the shop and the contact areas were machined to achieve the required fit-up as well as to remove abrasion marks and other defects.

Each hub plate assembly and bearing assembly was custom fabricated based on actual measurements of each trunnion shaft. This was done to ensure that the required interference fit is achieved between the shaft and the hub plates as well as between the shaft and the inner race of the bearing (Table 1). The tight fabrication tolerances required a high accuracy level in machining of the components.

Table 1. Connection Fit Details

Components Connected	Type of Fit
Hub Plates – Trunnion Shaft	FN 1 interference fit
Bearing Inner Race – Trunnion Shaft	FN 2 interference fit
Bearing Outer Race – Bearing Inner Race	RC6 clearance fit
Bearing Housing – Bearing Outer Race	RC4 clearance fit
Level Lock Bar – Level Lock Guides and Sockets	RC6 clearance fit

4.5 Hub Plate and Trunnion Shaft Installation

The poor surface profile of the girder web faying surfaces significantly reduced the contact area available to provide a slip-critical connection between the hub plates and the girder web. Initial attempts to improve the profile by using titanium putty were unsuccessful. The global distortion of the web plate added another level of complexity as the measured distortion profile was subject to change on application of the clamping force of the bolts. To remedy this problem, a bolted arrangement with bolt hole clearances ranging from minimum 0.08 millimetres to maximum 0.25 millimetres was designed to provide a connection that relied on a combination of friction and bearing.

The new hub plates had to be installed to an accuracy of 0.25 millimetres with respect to the existing bore. An Alignment Shaft (Figure 9(a)) with diameter of its alignment hole matching that of the trunnion shaft was fabricated and inserted in the bore of the hub plates to align them prior to field drilling bolt holes for the connection.

The design envisaged trunnion shaft installation using dry-ice cooling to shrink the shaft and heat application to grow the bore of the hub plates to achieve an FN1 interference fit connection. Due to concerns related to the confined work area and difficulty in uniformly heating the hub plates, the project team agreed to use a two stage method involving cooling of the shaft with liquid nitrogen to achieve the required shrinkage for shaft insertion. This application using liquid nitrogen for horizontal field installation of trunnion shafts in a confined work space was previously untried and posed a major risk that the shaft could get wedged within the hub plates while partially inserted or in an incorrect alignment. This risk was carefully mitigated in the following manner:

1) A split ring (Trunnion Alignment Ring) was fabricated such that its bore was slightly smaller than the bore of the hub plates (Figure 9(b)). The alignment ring would be mounted in front of the hub plate assembly and installation of the trunnion shaft would be abandoned if the shaft got stuck in the alignment ring.
2) A liquid nitrogen cooling trial was conducted to document the amount of shrinkage of the shaft and the working time available (Figure 9(c)).
3) The shaft was cooled in two stages (Figure 10). In Stage 1, the entire shaft was cooled to (-) 84 degrees Celsius. Stage 2 involved the use of a cryogenic jacket to further cool only the hub interface of the shaft to (-) 195 degrees Celsius with the shaft partially inserted in the bore of the hub plates.
4) Mock insertions using a dummy shaft were carried out to rehearse and improve the shaft installation procedure.

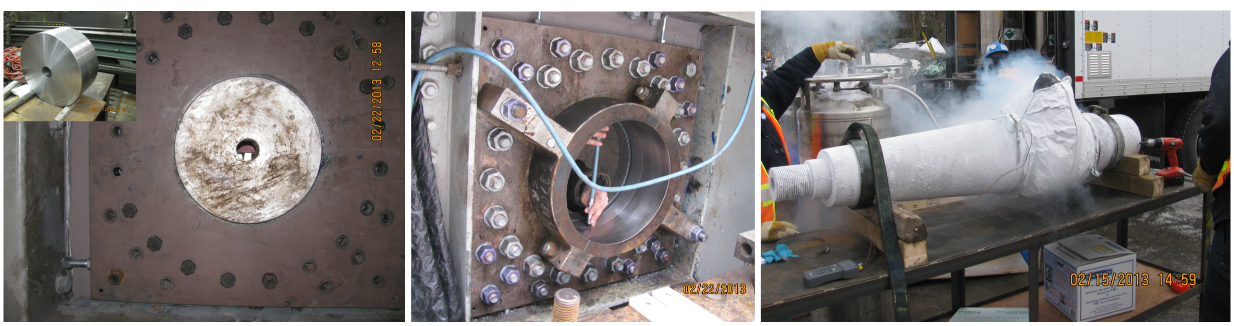


Figure 9. (a) Alignment shaft (b) Alignment ring mounted on hub plate (c) Liquid nitrogen trial

As stated earlier, an FN1 interference fit connection was designed for the connection between the trunnion shaft and the hub plates. To achieve this fit, the trunnion shaft had to be shrunk by at least 0.33 millimetres to enable installation of the shaft in the bore of the hub plates. The two stage method of cooling of the trunnion shaft using liquid nitrogen provided a total shrinkage of approximately 0.51 millimetres at the hub interface and afforded a working time of 20 to 25 minutes for the insertion of the shaft.

The first trunnion shaft installation attempt was unsuccessful and led to further refinement of the installation methodology including better coordination between work groups, procedure for removal of frost condensed on the trunnion shaft, and a more precise axial pulling arrangement.

The second trunnion shaft installation attempt was successful and included flame-heating of the hub plates to 100 degrees Celsius as a frost control measure in addition to the two stage cooling of the shaft. A threaded rod with a bolted arrangement coupled to a hydraulic wrench was used to axially pull the shaft into its final position. The rod was inserted through the alignment hole in the trunnion shaft and on actuation of the hydraulic wrench the trunnion shaft travelled forward on the threaded rod (Figure 10).

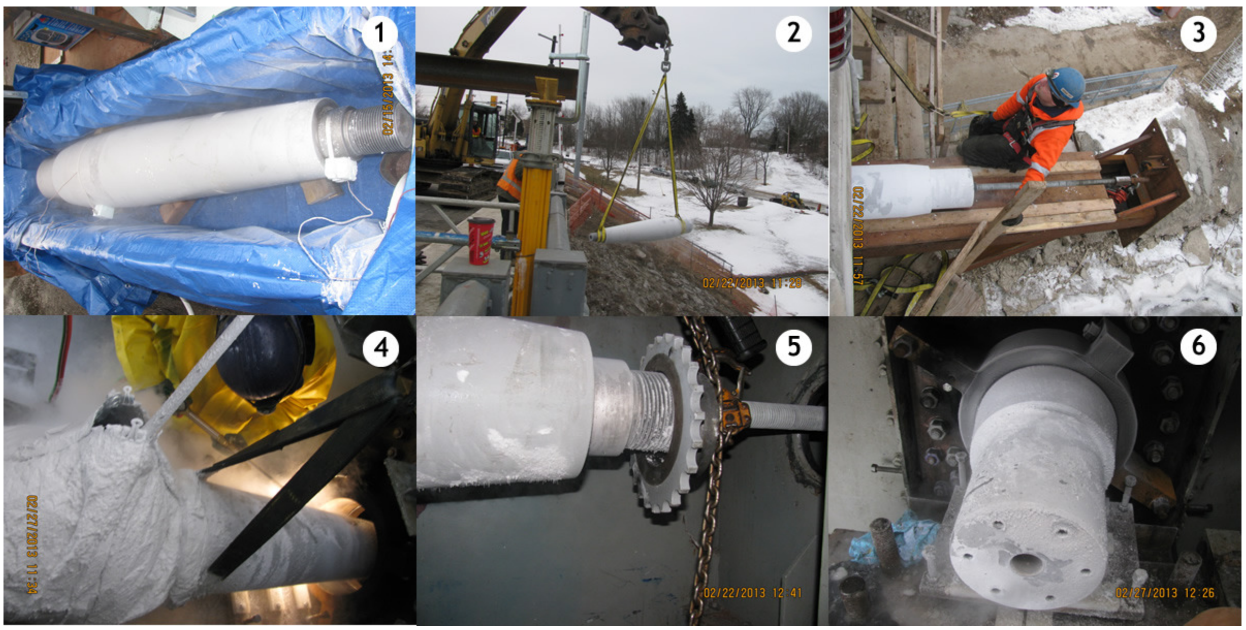


Figure 10. (1) Stage 1 cooling to (-) 84 Celsius; (2) Shaft lowering; (3) Shaft with threaded rod on cantilevered bath; (4) Stage 2 cooling to (-) 195 Celsius; (5) Shaft insertion under progress; (6) Installed shaft

4.6 Installation of Trunnion Bearings

The two-piece trunnion bearings were installed in the field due to the restricted size of the opening at the abutment side wall. The inner race of the new custom steel-on-bronze radial spherical bearings was heated using a high frequency induction heater to 200 degrees Celsius and installed on the trunnion shaft to obtain an FN2 interference fit connection. No unanticipated issues were encountered during the installation of the bearings (Figure 11).

The newly detailed bearing housing provides adequate grease to all surfaces and the grease manifold is conveniently located below the sidewalk access hatch. The access hatches are a new addition which also facilitate inspection and maintenance of the bearings. The new bearings have been operating satisfactorily through extreme heat and cold weather over the past year.

Figure 11. Heating of bearing inner race with induction heater (left); Installed bearing assembly (right)

4.7 Removal and Installation of Level Locks and Installation of Centering Guides

After removing the level locks, it was discovered that the connection interface was in poor condition with oversized bolt holes and improvised field welded connections at some locations. Further, due to the excessive gap between the lock bars and mating surfaces, the impact loading had resulted in perceptible deformation of the contact areas. Some components also had significant dimensional variation vis-à-vis information from the record drawings.

Since the work at the level locks required full closure of the bridge, it was scheduled during night time to minimize disruption to the public. During design, it was envisaged that after each night's work the existing lock bars would be restored to lock the leaves in position prior to opening the bridge to vehicular traffic. As a result of the improvised field welds and poor condition of the contact areas, it was not feasible to restore the lock bars back in position as designed. To overcome this problem, an innovative "Temporary Leaf Lock" assembly was devised which could be quickly set up to lock the two leaves together (Figure 12).

Due to dimensional variations observed after removal of the level locks, some of the new guides and sockets had to be machined or shimmed to achieve the required fit. The oversized bolt holes issue was resolved by increasing the bolt size, reaming bolt holes to suit, and providing a backer plate. The new level locks have housings detailed with adjustable shoes, thus minimizing impact loading and wear. The hydraulic cylinders at the level locks as well as the elevation locks are now installed with counterbalance valves at both ends to prevent drifting of the lock bars.

The centering guides (Figure 13) were installed after verifying that the alignment of the bridge was within tolerance. The addition of the centering guides has resulted in less misalignment indications at the end of the operation cycle.

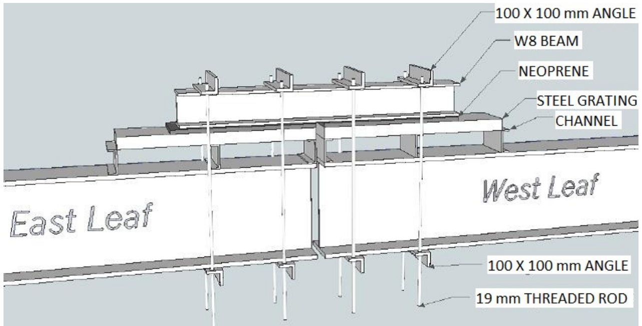


Figure 12. Temporary leaf lock assembly


Figure 13. Centering guide

4.8 Alignment and Balancing

The pre-jacking survey served as the benchmark for the alignment of the bridge. The bridge had to be restored to a post-rehabilitation alignment within 3 (three) millimetres of the pre-jacking alignment. Horizontally braced jacks were used to adjust the alignment as the bridge had racked slightly while on temporary supports during the rehabilitation work.

An initial balance test of the spans was conducted prior to commencement of construction to set reference data for the imbalance moment and the centre of gravity of each leaf. Subsequently, an intermediate balance test was performed after replacement of the trunnion bearings and installation of the concrete in-filled steel deck grating since these changes would have altered the balance of the leaves. The final balance test was performed after installation of the new level locks and the centering guides to verify that the imbalance moment is within 2% of the reference imbalance measured during the initial balance test.

4.9 Commissioning of the Rehabilitated Bridge

After installation of the new trunnion bearings, initial testing was performed over 20 operational cycles during which the hydraulic pump settings and the PLC program settings were adjusted and optimized. The final commissioning of the rehabilitated bridge involved 20 successful operational cycles after completion of all work.

Since completion of the rehabilitation, the bridge has been operating satisfactorily barring minor post-construction work and periodic maintenance work.

5. CONCLUSION

The original design of Lynn River Bridge was lightweight with the intent to be one of the fastest operating lift bridges in North America. However, this was achieved at a cost as the resulting flexibility adversely affects key machinery components and makes the control system more sensitive to changes. In addition, a lightweight bridge is more susceptible to wear due to heavy traffic cycles. Also, corrosion or defects in the components reduce the capacity of the bridge at an accelerated rate.

The unique design features of this bridge necessitated innovative methods and non-traditional solutions both during design and construction. The project team capitalized on the knowledge and experience of the team members to develop practical field solutions for resolving numerous time-sensitive problems. Trials and rehearsals prior to implementation of non-traditional solutions combined with a good risk mitigation strategy ensured successful execution of this complex project. The cost of rehabilitation of the bridge was approximately CAD $1.21 million. The successful rehabilitation of Lynn River Bridge is testament to the synergy in teamwork and close cooperation between all members of the project team.

6. ACKNOWLEDGEMENTS

Lynn River Bridge is owned by the Ministry of Transportation Ontario, Canada. The rehabilitation was designed by Hardesty and Hanover LLP, New York, USA as sub-consultant to Stantec Consulting Ltd, Canada. The contractor for construction was Facca Inc., Ontario, Canada as sub-contractor to Holcim (Canada) Inc.

7. REFERENCES

1. Canadian Standards Association, CAN/CSA-S6-06 Canadian Highway Bride Design Code, November 2006, Canadian Standards Association.
2. LRFD Movable Highway Bridge Design Specifications-2007

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Machinery Rehabilitation at the Columbus Road Bridge over the Cuyahoga River

Cleveland, Ohio

Robert J. Tosolt, P.E.
Stafford Bandlow Engineering, Inc.

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

INTRODUCTION

Due to the active marine commerce on the Cuyahoga River and due to the circuitous path of the river winding back on itself from which the Iroquois Indians begot its name ("crooked river"), movable bridges have been integral to the growth of the City of Cleveland through its history. The Columbus Road Bridge carries Columbus Road over the Cuyahoga River at waterway milepost 1.9 from the mouth to Lake Erie as illustrated in Figure 1. A total of twelve movable bridges exist in the focus area of the map: five active movable bridges (including Columbus Road) are presently owned and operated by the City of Cleveland, one inactive movable bridge is owned by the City but has future plans for a return to service, three bridges are rail owned bridges that remain active, and the three bridges have been removed from service. However, the Columbus Road Bridge holds particular distinction among Cleveland's bridges, as the first permanent crossing of the Cuyahoga River dating to 1835.

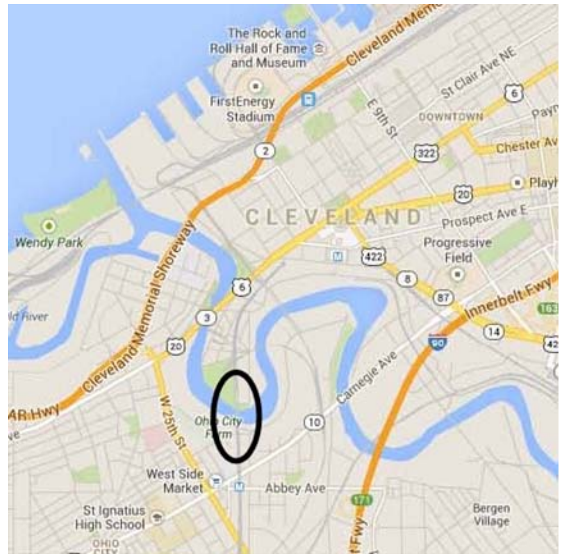


Figure 1. Area map locating Columbus Road Bridge in Cleveland, Ohio.

The present day span drive vertical lift bridge is a single span through Pratt truss vertical lift span that was constructed in 1940 and spans 242'0" from end to end of truss which provides a 220' clear channel between fenders and maximizes the navigable channel to the full width of the river at this location. The bridge can accommodate large freight river traffic with a normal vertical lift of 79' that provides a little more than 90' vertical clearance to the waterline when fully raised

Figure 2. The present day Columbus Road span drive vertical lift bridge dating to 1940 with the City skyline in the background.

When new, the Columbus Road Lift Bridge epitomized the pinnacle of the trend to become bigger, with greater lift and accommodate larger traffic loads. However, as of 2008, the Columbus Road Bridge had shown signs of aging, with local media terming it a "cranky rusting relic from the Roosevelt era". The Columbus Road Bridge was in need of extensive repairs to continue to reliably serve its role for both marine and vehicular traffic.

In 2008, Cuyahoga County engaged an engineering team headed by TranSystems to determine the preferred option with regard to rehabilitating or replacing the bridge to address deterioration, load capacity, environmental and geotechnical issues. The TranSystems team performed a detailed bridge inspection and technical study to corroborate the scope concerns and develop options for rehabilitation and replacement. Following selection of the preferred option by the County, the TranSystems team was retained to prepare the contract documents. The design plans were finalized in August 2011 and construction is currently ongoing with anticipated completion late summer/early fall 2014.

This paper shall describe the significant mechanical aspects of this project including scoping inspection of existing equipment, evaluation of the power requirements, selection of the span drive and trunnion bearing arrangements and the considerations provided to address future settlement of the foundations. Notable construction stage coordination and changes from the contract plans are also addressed.

PROJECT MEMBERS

The primary project members involved in the mechanical portion of the project which is discussed in this paper is as follows. Cuyahoga County administered the project for the Ohio Department of Transportation. The City of Cleveland owns the bridge. Prime consultant for the work was TranSystems with the work being managed out of the Cleveland, Ohio office. Personnel from Stafford Bandlow Engineering, Inc of Doylestown, Pennsylvania provided the engineering for the mechanical machinery systems. The Primary Contractor for the work was the American Bridge Company of Coraopolis, Pennsylvania. American Bridge utilized G&G Machinery based in Russellville, Alabama as the primary machinery fabricator. American Bridge provided in-house personnel to perform the machinery installation.

DESCRIPTION OF MECHANICAL SYSTEMS

The drive machinery for the movable span utilizes a conventional 'span drive' layout, that is, the machinery is mounted on, and moves with, the movable span during operation. The main drivetrain is located in a machinery house at mid-span above roadway level. The machinery utilizes an electro-mechanical drivetrain to transmit power from the prime mover, which is an electric motor, out to operating rope drums which are all driven by the same central gear train. There are four operating rope drums; each drum contains two uphaul and two downhaul ropes and serves one corner of the movable span. The uphaul ropes are terminated at the top of the tower and the downhaul ropes are terminated at the foot of the tower at pier level. All ropes run along the tower legs and pass around deflector sheaves at the top chord of the truss, then run through a series of rollers and deflector sheaves back to the operating rope drums. When energized, the electric motor rotates the operating rope drums, which pay in or pay out the operating ropes and thereby result in the lift span raising or lowering along the operating ropes.

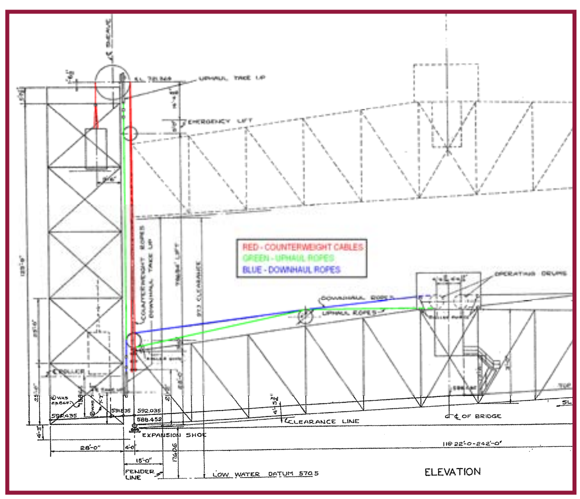


Figure 3. Illustration of Drive System.

The primary span support machinery comprises the main counterweight ropes, main counterweight sheaves, sheave trunnions, and sheave trunnion bearings. A total of forty-eight main counterweight ropes connect the lift span to the two main counterweights which counterbalance it. The ropes are divided into groups of twelve; each rope group connects to one corner of the lift span, passes up and over the main counterweight sheave mounted at the top of the tower, and connects to the main counterweight. The sheaves, which support the entire weight of the movable span and counterweights, are mounted on trunnion shafts that are simply supported in grease lubricated bronze-bushed tower mounted sleeve bearings. An auxiliary counterweight system is provided to compensate for the transfer of weight of the main counterweight ropes from the span side to the counterweight side of the system during span operation.

Figure 4. General View. Counterweight ropes, sheave, sheave trunnion and trunnion bearings.

Figure 5. Expansion End Live Load Support.

The movable span is provided with four live load supports to transmit the imbalance load and the live load due to vehicular traffic from the movable span to the rest piers. The two live load supports at the south end are considered the fixed supports and are designed to maintain the position of the span in the longitudinal direction. The two live loads at the north end of the span are the floating supports and are design to accommodate thermal expansion of the span.

The movable span is equipped with span guides, counterweight guides, and centering devices for positioning the span during operation and seating.

The lift span is equipped with four air buffers. One air buffer is mounted at each corner of the lift span. The air buffers provide cushioning for the span during seating and reduce shock loading in the event that the bridge approaches the live load supports at excessive speed.

The span is equipped with span locks to secure the lift span in the seated position.

Figure 6. Span Lock Machinery. Abandoned in place.

PRELIMINARY SCOPING INSPECTION

The intended scope of rehabilitation as set forth at the outset of the project was to replace all drive and support machinery outright with new, more efficient components with the exception of the main counterweight sheave and sheave trunnions, which would be re-used condition permitting. In accordance with this proposed scope of work, a preliminary inspection was performed in early summer 2008 to verify the scope. The mechanical inspection entailed a cursory inspection of all machinery except for the sheave assemblies to evaluate existing space constraints for the new machinery alternatives. An in-depth inspection of the sheave assemblies was performed to assess the integrity of these components for continued service; this in-depth inspection comprised visual and hands-on inspection of the sheave assemblies as well as non destructive testing (NDT) of the sheave trunnions, including ultrasonic testing (UT)of the trunnions along their longitudinal axes and wet fluorescent magnetic particle testing (WF-MPT) of each trunnion fillet around the full circumference. The City of Cleveland provided the most able assistance with the disassembly of the bearings to facilitate this inspection. The following is a description of the significant findings of the scoping inspection.

SPAN SUPPORT SUMMARY

Inspection of the span support components focused on the main load bearing components: the main counterweight sheave trunnions and the main counterweight sheave trunnion bearings.

<u>Main Counterweight Sheave Trunnions</u>

There are a total of 4 main counterweight sheave trunnions. One trunnion supports each sheave. Each trunnion has a central hub and steps down at each end to the trunnion journals. See Figure 7. Experience has demonstrated that trunnions are susceptible to fatigue cracks at the fillets which form the hub/journal interface. Therefore, the integrity of each trunnion was evaluated via close examination of the trunnion fillets using both visual and NDT (wet fluorescent magnetic particle testing), and the overall integrity of the trunnion was evaluated via NDT (ultrasonic testing). Stafford Bandlow Engineering performed the visual inspection. Stafford Bandlow Engineering retained Team Industrial Services of Cleveland, Ohio to perform the NDT tests.

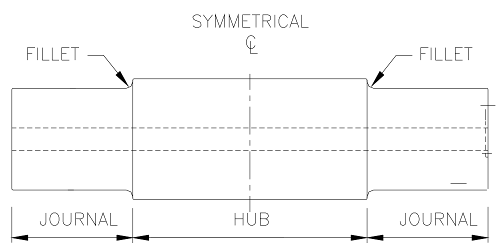


Figure 7. Schematic diagram of trunnion, identifying key geometry features.

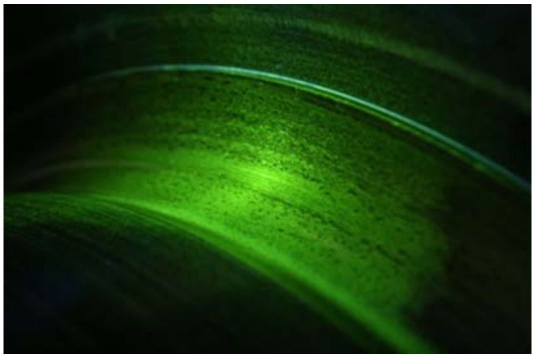


Figure 8. Wet magnetic particle testing of fillet region. No indications.

Visual inspection of the fillets did not reveal any defects, aside from light scoring of the fillet region at several locations.

Magnetic particle testing did not identify defects in any of the fillets. See Figure 8.

Ultrasonic testing identified one indication at the SE trunnion at approximately 19” from the inboard end of the journal. [Note that the journal is 21.5” in length] The indication was at approximately 2” depth and was about ½” in size. This findings was attributed to an original casting defect. No other indications were noted at the other trunnions.

The trunnions were found to be of sound integrity with no physical findings that would preclude continued usage.

Main Counterweight Sheave Trunnion Bearings

There are a total of 8 main counterweight sheave trunnion bearings. Two bearings support each sheave. Each bearing assembly consists of a steel pillow block casting that houses a bronze bushing which supports the trunnion journal and transfers the sheave load to the tower. See Figure 4.

The bearing housings are coated with grit and exhibit paint deterioration and surface corrosion. However, the housings are of stout construction and are in good condition.

The caps for the bearing housings were removed to allow inspection of the internal wearing surfaces. The full width of each trunnion journal was visually inspected around its full circumference. The journals typically exhibited abrasive wear across their full width that varied from light to severe, bands of bronze embedment, and regions of surface breakdown. The worst location was the inboard southwest trunnion bearing (TB-SW-IB) which exhibited severe abrasive wear in excess of a 500 microinch surface finish. Note that AASHTO requires an 8 microinch finish for a new installation. See Figure 9 through 12.

Figure 9. TB-SW-IB. General view of journal. Note bands of discoloration due to bronze embedment.

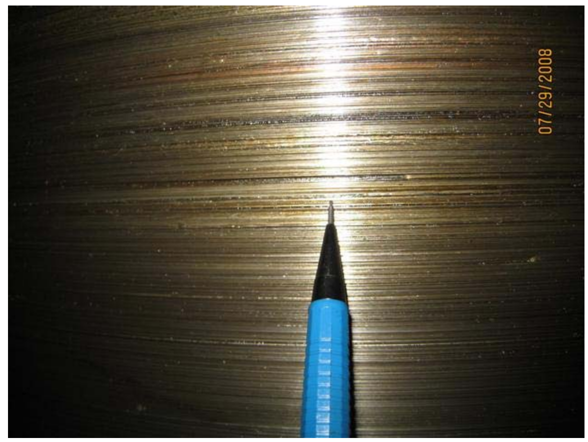


Figure 10. TB-SW-IB. Close-up view of severe abrasive scoring of journal.

Figure 11. TB-NE-OB. General view of journal. Note bands of discoloration due to bronze embedment.

Figure 12. TB-NE-OB. Close-up view of abrasive wear, bronze embedment and surface breakdown.

The overall condition of the bearing journals was poor. The bottom bearing bushings, which were inaccessible for inspection, can be expected to be in similar poor condition. As a minimum, rehabilitation of journals and replacement of the bushings was required for continued long term service.

BINDING PROBLEM

Inspection of the span guides and operating rope deflector sheaves revealed that the movable span is effectively being bound and/or twisted in both the longitudinal and transverse planes, as illustrated in the figure below.

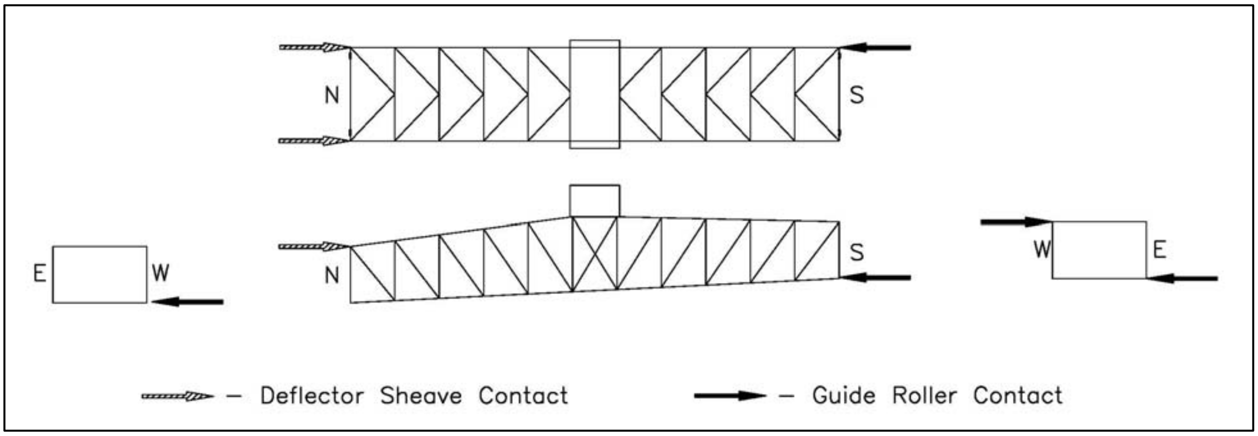


Figure 13. Schematic View of Lift Span depicting contact points from deflector sheaves and upper and lower guide rollers.

The deflector sheaves at the north end of the span were observed to be in hard contact with the tower legs with the span in the seated position. The contact has produced metal flow that has rounded the outside diameter (OD) of the sheave rims as well as producing grooves in the mating rivet heads on the tower legs. See Figures 14 and 15. As a result of the hard contact, both deflector sheaves at the north end of the bridge emit loud snapping noises during span operation near the fully seated position, which corresponds with the region of heaviest contact between the deflector sheaves and tower legs. To investigate the possibility that these noises could be indicative of cracks in the deflector axles, ultrasonic testing of the axles was the axles was performed. No defects were noted.

Figure 14. NW Deflector Sheave. General. The sheave rim has been rounded over due to contact.

Figure 15. NW Deflector Sheave. Close-up. Rim is rounded and rivets exhibit damage from contact.

In addition to the contact between the deflector sheaves and tower legs at the north end, there has been direct contact between the lift span and tower leg. There is evidence that the angles that run the length of the tower and form the guideway for the uphaul ropes have contacted the lift span during operation and worn grooves into the top chord of the truss. The ends of the outstanding legs of the angles have been flame cut back in the past to eliminate the contact.

Lift spans are typically designed to accommodate a certain amount of thermal expansion. Measurements were taken to quantify the position of the movable span relative to the tower at the expansion end of the bridge to quantify the relative shift necessary to result in the observed binding condition. An offset of 3" was measured between the center of the lateral guide roller and the center of the guide rail at the north end of the bridge. This offset was substantiated by the offset of the air buffer relative to the center of the strike plate.

While the source of the towers being 3" closer together than when originally installed was attributed to settlement of the south bank of the river, the opposing force necessary to generate the existing binding condition was produced by the guide rollers and was evident through their inspection. The contact has produced wear in the form of heavy metal flow and flaking metal at the contacting guide rails. See Figures 16 through 18.

Figure 16. Metal flakes on face of rail attest to heavy wear.

Figure 17. Metal flow has produced a ridge that stands proud of face.

Figure 18. Heavy metal flow has produced undulating surface.

In addition, the load induced by the contact was so substantial as to have resulted in failure of the SE longitudinal guide roller axle. The failed axle was verified through ultrasonic testing (UT) as part of this inspection. The City of Cleveland maintenance personnel were aware of this condition at the time of the inspection and were awaiting parts for replacement.

As a result of the above findings, the rehabilitation scope was expanded to address the settlement issue, eliminate the binding and make provision for future settlement.

MECHANICAL LOAD RATING

Mechanical load rating analyses were performed to assess the viability of the sheave assemblies for re-use and to determine the basic power requirements and machinery sizing for the new machinery options. The findings of the analyses are presented below.

All analyses were performed in accordance with the AASHTO LRFD Movable Highway Bridge Design Specifications, Second Edition 2007 with 2008 Interim Revisions, herein after referred to as "AASHTO". All information regarding existing machinery was obtained from the original design and shop drawings that were provided by the City of Cleveland.

SPAN SUPPORT ANALYSIS

The primary span support components including the sheave trunnion assemblies and main counterweight ropes must be sized to support the weight of the movable structure. Analysis was performed to determine the capacity of the existing components relative to the existing span weight as well as the proposed weight of the alternative span options.

Sheave Trunnions

Calculations were prepared to determine the fatigue life of the counterweight sheave trunnions for the existing span weight. The calculations indicate that the trunnions have a finite fatigue life and that the fatigue limit is 675,519 cyclic stress reversals. The bridge experiences 3.76 cyclic stress reversals during each full opening. Therefore, the bridge must undergo approximately 179,659 full openings to exceed the fatigue limit.

Based on a 68 year operating history, if the bridge has averaged 220.25 openings per month, the trunnions would have reached the end of their fatigue life. A prior report indicates that the bridge undergoes 350 to 400 openings per month. Based on this estimate, the trunnions have exceeded their fatigue life.

AASHTO requires sheave trunnions to be designed for an infinite fatigue life.

Therefore, although the physical inspection of the trunnions revealed no physical faults, reuse of the sheave trunnions was not a viable option.

Sheave Trunnion Bearings

The bearing stress for the existing plain bronze bushed trunnion bearings was calculated for the existing span weight as well as the alternate options considered as part of this rehabilitation work. The results are presented in the table below.

PLAIN TRUNNION BEARINGS	SPAN WEIGHT			
	Existing	Alternative 3 Steel Grid Deck with Concrete Fill at Curblines	Alternative 2 Steel Grid Deck Half Filled with Concrete	Alternative 1 Concrete Deck
	1800 kips	1980 kips	2300 kips	2600 kips
Bearing Stress (psi)	2010	2201	2550	2861
% allowable	134	147	170	191

Based on an AASHTO allowable stress of 1500 psi, the bearings were overstressed in the existing condition (indicated by yellow highlight) and any additional weight additions to the span would further increase the overstress.

Reuse of the trunnion bearings was not a viable option.

Counterweight Wire Ropes

The counterweight ropes were evaluated against AASHTO requirements for direct load and combined loading (i.e., direct load, rope bending, inertial effects) for the existing span weight as well as the alternate options considered as part of this rehabilitation work. Rope capacities were determined based on AASHTO allowable safety factors under each of the loading conditions, and the results are presented in the table below.

REQUIRED TENSILE STRENGTH	SPAN WEIGHT			
	Existing	**Alternative 3** Steel Grid Deck with Concrete Fill at Curblines	**Alternative 2** Steel Grid Deck Half Filled with Concrete	**Alternative 1** Concrete Deck
	1800 kips	**1980 kips**	**2300 kips**	**2600 kips**
Direct Loading	300 kips	330 kips	383 kips	433 kips
Combined Loading	313 kips	330 kips	360 kips	388 kips

The required rope tensile strength was then compared against the existing counterweight ropes as well as other options allowed by AASHTO.

ROPE OPTIONS		MINIMUM TENSILE STRENGTH (KIPS)			
		Fiber Core		IWRC	
		IPS	EIPS	IPS	EIPS
Diameter	1 7/8"	282	312	304	348
	2"	320	352	344	396
	2 1/8"	358	394	384	442
	2 1/4"	400	440	430	494

A review of the required tensile strength as compared to the manufacturer's published minimum tensile strength indicates that the existing ropes (indicated by yellow highlight) are overloaded in the present condition and cannot tolerate additional load. It will be necessary to use a higher strength and/or larger rope depending upon which alternative is selected for the rehabilitation.

Main Counterweight Sheaves

While a complete analysis of the main counterweight sheaves was beyond the scope of the load rating, a cursory review of the sheaves found that the sheave hubs did not meet AASHTO requirements with regard to minimum radial hub thickness.

In order to increase the fatigue life of the trunnions to meet AASHTO requirements, the trunnion diameter would increase and require a larger bore in the sheave resulting in a thinner hub. Since the hub was already thinner than required by AASHTO at the outset, a further reduction could not be tolerated. As a result, re-use of the main counterweight sheave was not a viable option.

SPAN DRIVE ANALYSIS

The analysis of the span drive machinery was limited to determining the effects of varying the system efficiency on the basic power requirements and sizing of the operating ropes for the existing span weight to provide a basis of comparison for the new machinery options.

Power Requirements

The external loads on the bridge were evaluated to determine the required motor capacity necessary to operate the bridge in accordance with AASHTO Article 5.4.4. Bridge information was obtained from the available drawings.

Two primary variations of the load cases were considered to evaluate the effects of plain vs. roller bearings on the system efficiency. Option 1 utilized plain bearings for the main counterweight sheave trunnion bearings. Option 2 utilized roller bearings for the main counterweight sheave trunnion bearings. These options were selected because the type of bearing used to support the main sheave assemblies has the greatest effect of all the evaluated loads on the required machinery capacity. For each of these variations, all other loading information was kept the same with the exception of the operating speed. The operating speed was varied from 60 feet per minute (FPM), which is the speed of operation based on the existing motor speed and gear ratios, to 40 FPM, which is the operating speed of the new West Third Street Bridge based on the available drawings. The results of the analysis are as follows:

REQUIRED MOTOR HORSEPOWER	SPEED OF OPERATION	
	60FPM	40FPM
Option 1. Plain Bearings	267 hp	178 hp
Option 2. Roller Bearings	183 hp	122 hp

The analysis indicates that the existing drive, which is powered by 2-100 hp motors, does not have sufficient power to operate the existing bridge at rated speed under AASHTO loading conditions (the existing power requirements are presented in yellow highlight). The analysis also indicates that there is a significant reduction in the power requirements by utilizing roller bearings and a slower operating speed.

Operating Wire Ropes

The operating ropes were evaluated against AASHTO requirements for direct load and combined loading (i.e., direct load, rope bending, inertial effects) for the two loading options discussed above. Note that the governing loads are independent of speed of operation. Rope capacities were determined based on AASHTO allowable safety factors under each of the loading conditions, and the results are presented in the table below.

REQUIRED TENSILE STRENGTH	LOADING CONDITION	
	Option 1	Option 2
	Plain Brgs	Roller Brgs
Direct Loading	65 kips	65 kips
Combined Loading	127 kips	113 kips

The required rope tensile strength was then compared against the existing operating ropes as well as other options allowed by AASHTO.

ROPE OPTIONS		MINIMUM TENSILE STRENGTH (KIPS)	
		Fiber Core	
		IPS	EIPS
DIAMETER	1 1/8"	105.2	115.6
	1 1/4"	129.2	142.2

A review of the required tensile strength as compared to the manufacturer's published minimum tensile strength indicates that the existing ropes (indicated by yellow highlight) are overloaded by the existing (plain bearing) machinery based on the combined loading condition. It will be necessary to use a higher strength and/or larger rope depending upon which alternative is selected for the rehabilitation.

SUMMARY OF MECHANICAL ANALYSIS

The stated intent of the mechanical analysis was to determine the suitability of the main counterweight sheave trunnion assemblies for reuse and to determine the basic power requirements and machinery sizing for the new machinery alternatives. The findings of the mechanical analysis with respect to each of these issues are as follows:

1. The existing span support components were overloaded based on existing conditions and were not suitable for continued usage.

2. Increasing the size and/or capacity of the span support components to accommodate an either in-kind replacement or rolling bearing replacement could be implemented with no modifications to the towers.

3. The span drive machinery was underpowered and the operating ropes did not have sufficient capacity based on current AASHTO requirements.

4. Increasing the system efficiency by utilizing roller bearings for the main counterweight trunnion bearings would result in a considerable reduction in the motor power requirements and operating rope loads, which in turn would result in smaller machinery.
    a. Reduced operating loads resulting in smaller operating ropes
    b. Reduced motor power requirements resulting in smaller motors
    c. Reduced machinery loads resulting in smaller machinery

5. Slowing the speed of operation to be consistent with other City bridges would result in similar benefits to item 4.

Due to the significant benefits provided by improving the system efficiency, all mechanical drive options were based on using roller bearings for the main counterweight trunnion bearings, as well as at other machinery components to the extent possible. In addition, the speed of operation was set to be consistent with other City bridges to minimize inertial loads.

PRELIMINARY DESIGN CONSIDERATIONS

SPAN DRIVE OPTIONS

Based on the findings from the Mechanical Load Rating, the three drive machinery options were presented for consideration.

Drive Option 1. Replace existing drive machinery with a similar more efficient drive that features a conventional parallel shaft reducer in lieu of the existing gear frame and roller bearings in lieu of plain bearings. See Figure 19. This drive configuration is similar to the new West Third Street bridge.

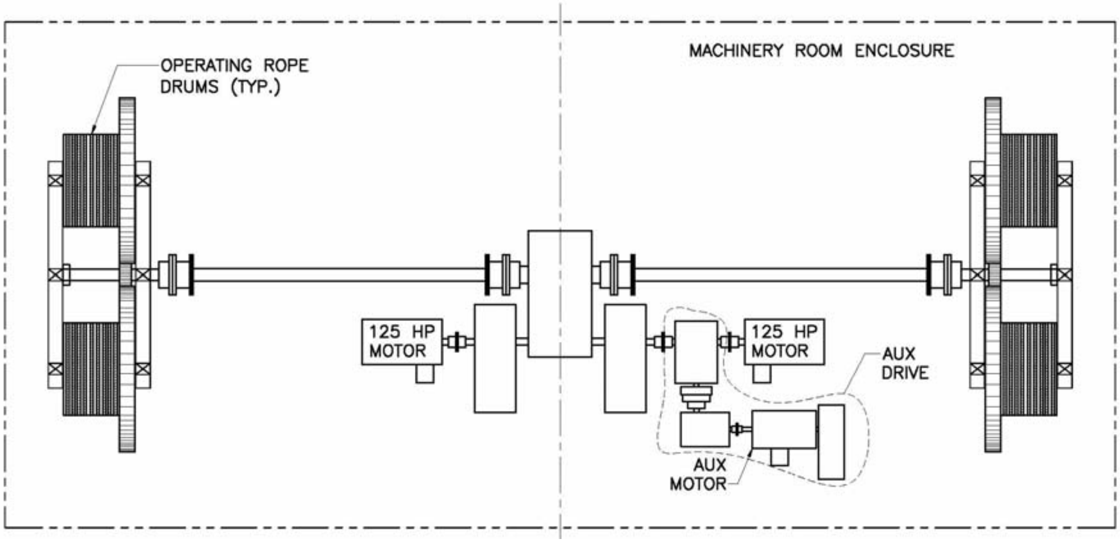


Figure 19. Drive Machinery Option 1. Span Drive.

Drive Option 2. Replace existing machinery with a more efficient drive that features a custom large parallel shaft reducer that provides all necessary reduction and direct drives each of the four operating rope drums. See Figure 20.

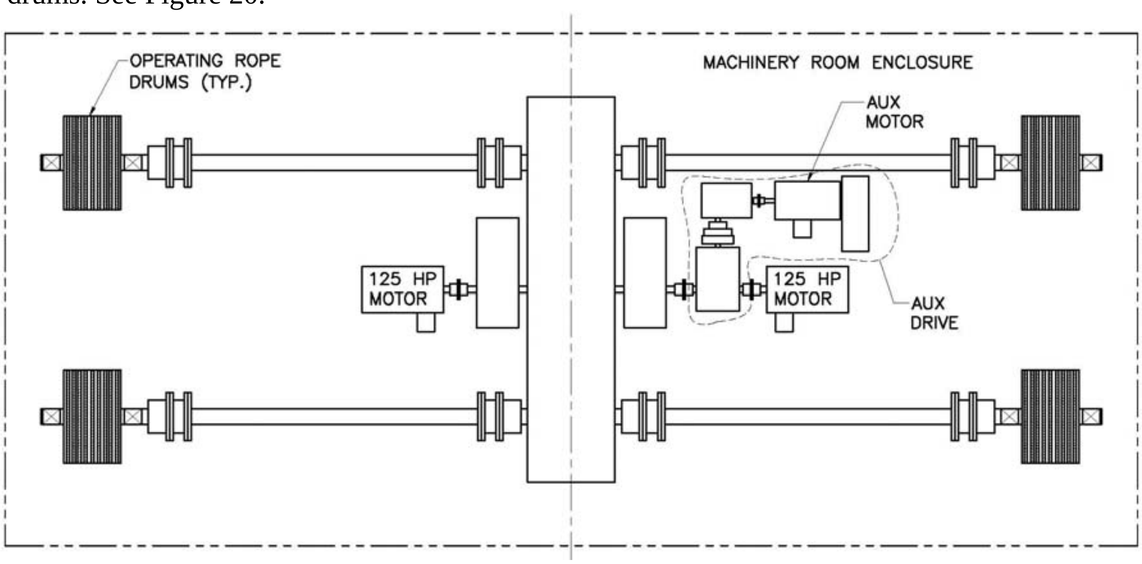


Figure 20. Drive Machinery Option 2. Span Drive

This drive configuration is not presently employed by any City of Cleveland bridge. However, it has demonstrated success in other areas of the country.

Drive Option 3. Provide conventional tower mounted drive machinery featuring enclosed reducers and roller bearings to minimize maintenance and improve efficiency. See Figure 21. Two drives required, one per tower.

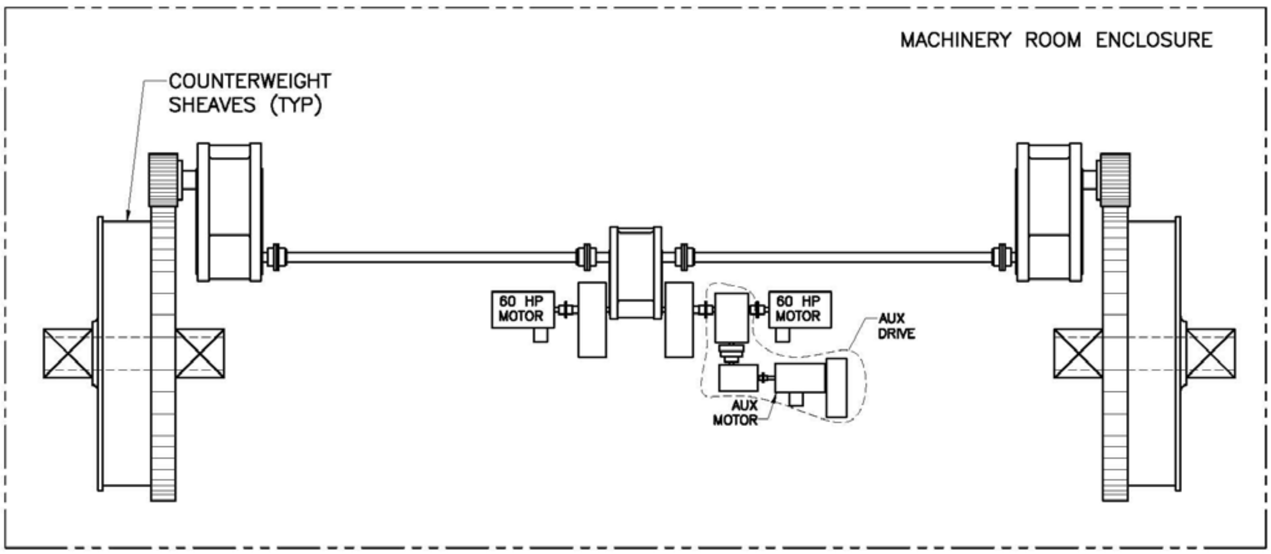


Figure 21. Drive Machinery Option 3. Tower Drive.

Options 1 and 2 applied to rehabilitation or replacement of the existing span drive lift span. Option 3 applied to replacement of the existing span drive lift span with a tower drive lift span. Each of these machinery options was intended to be powered by either of the two available motors. This provides complete redundancy of the prime mover. A separate auxiliary drive could also be provided if desired and was depicted in the attached schematics for reference.

SPAN SUPPORT OPTIONS

Based on the findings from the Scoping Inspection and Mechanical Load Rating, two span support options were presented for consideration:

Support Option 1. Replace existing sheave assembly with a similar assembly that features a rotating trunnion shaft and external roller bearings. The trunnion shaft will be designed for an infinite fatigue life. The trunnion bearings and counterweight ropes will be designed for the applied load.

Support Option 2. Replace existing sheave assembly with a new design that features a fixed trunnion shaft and roller bearings mounted at the I.D. of the sheave hub. This configuration eliminates the fatigue concern with the trunnion shaft. In addition this eliminates field alignment of the trunnion bearings and may allow for improved bearing seal design. The trunnion bearings and counterweight ropes will be designed for the applied load.

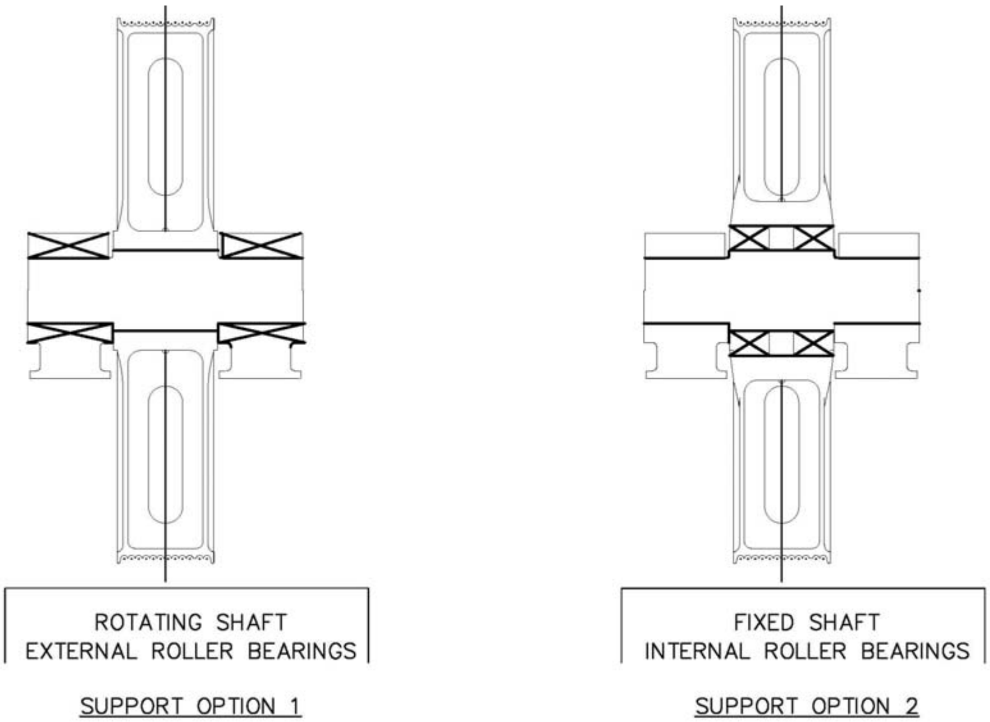


Figure 22. Span Support Options.

Note that with regard to either of the span support options, the applied load (which was yet to be determined based on the preferred roadway deck alternative) would govern the size of the counterweight ropes and sheave O.D. Additionally, determination of whether or not the tower were to be rehabilitated or replaced would impose space constraints on the size of the sheaves and ropes; in the case of rehabilitation, the existing space constructions would govern the maximum allowable load. In the case of replacement, the ropes and sheaves could be designed for the applied load and the new towers could be designed around them.

FINAL DESIGN CONSIDERATIONS

A comprehensive report was prepared to document the findings, options and preferences for either rehabilitating or replacing the existing machinery components in accordance with the scoping inspection and load rating. The owner's selected alternative was to replace the lift span outright but to rehabilitate the towers with the following impacts on the machinery.

SPAN DRIVE OPTIONS

Span drive machinery option 3 was eliminated due to the decision to reuse the existing towers as the towers could not be practically retrofit to accommodate tower drive machinery. Option 2 was chosen as the preferred alternative as this configuration offers the following benefits relative to option 1:

- Eliminates all open gearing and therefore requires less maintenance since all required reduction is enclosed in the central reducer.
- Simplifies installation.
- Provides for monitoring of the loading at each of the 4 operating rope drums, which in turn allows the balance of the bridge to be accurately determined as well as isolation of problems generating excessive loading.

In considering the design factor for the new drive machinery, consideration was given to actual operating loads measured at the existing bridge via dynamic strain gage testing.

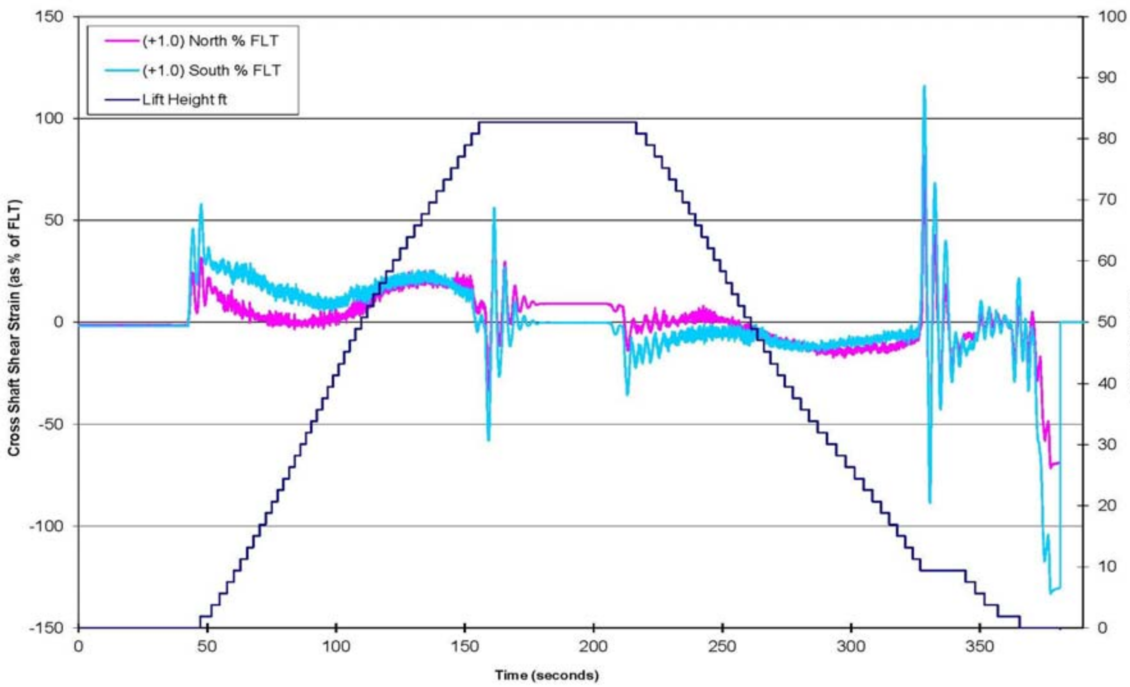


Figure 23. Chart recording of existing machinery operating loads as a percentage of full load motor torque, recorded April 14, 2010.

The testing indicates that the nominal operating loads are well below full load torque of the existing drive motors. The peak loads occurred during braking and seating and marginally exceeded 100% full load motor torque. These loads will be better controlled as part of the rehabilitation due to the benefits of speed control with the new electric drive. In addition, the seating torque will be limited to prevent excessive windup due to contact at the live load supports. Loading was also evaluated against a simulated wind and ice load in compliance with the maximum AASHTO loading conditions. Under these maximum loading conditions, the operating loads remain at or below 100% full load motor torque, with the exception of during acceleration, braking and seating. Again, these loads will be better regulated as part of the new electric drive. These findings substantiate that the sizing of the new drive machinery is appropriate for the intended load.

Based on the above, it should be expected, and required, that proper installation and erection of the machinery will result in equal loading of the machinery. Therefore, sizing the machinery based on 150% full load motor torque and assuming equal splits throughout the system should produce a machinery installation that is well sized for the applied loading.

To facilitate field installation and achieve the alignment required to ensure the anticipated load distribution, due consideration was given in the design of the machinery to simplify the mounting details to mitigate errors in installation and alignment to the extent possible. To that end, common baseplates were provided for the machinery so that the machinery could be shop mounted and aligned thereby minimizing the amount of field alignment required. However, given the size of the main drive components, consideration was also given to the practicality of handling large baseplates in the field. For these reasons, the design provided a common baseplate for the center high speed drive machinery, a baseplate for the secondary reducers, and a baseplate for the operating drum assemblies.

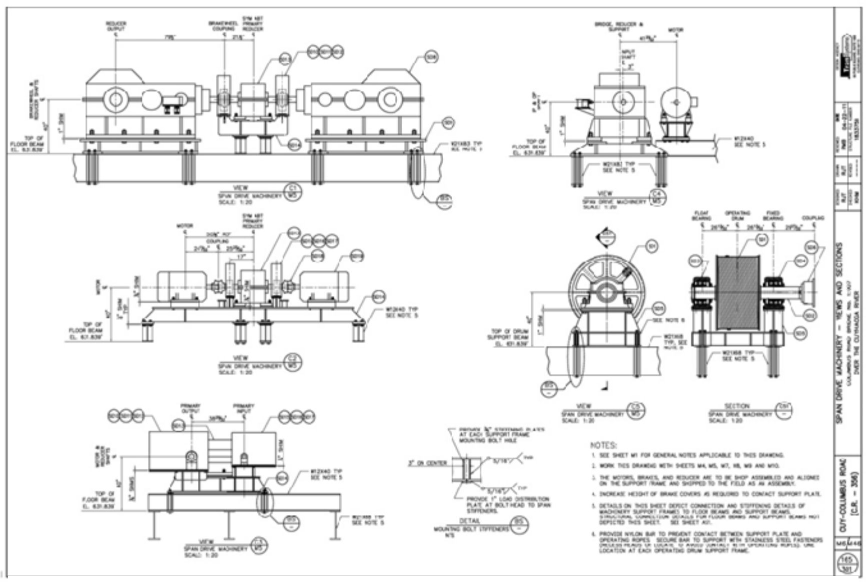


Figure 24. Drawing details for Span Drive Supports.

The framing details to mount these baseplates to the underlying support beams was closely coordinated with the structural engineers and full details of the intended mounting sequence were provided on the contract plans.

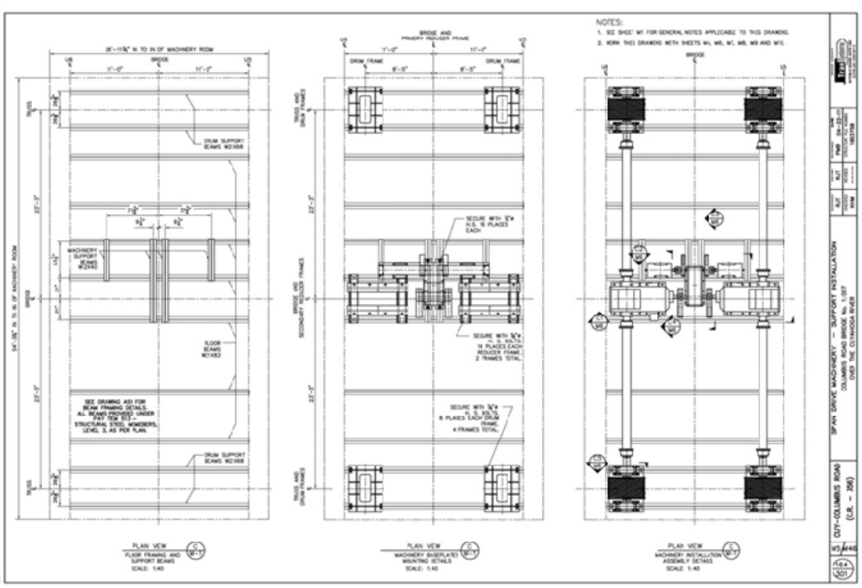


Figure 25. Framing details for Span Drive Supports.

SPAN SUPPORT OPTIONS

Span Support Option 2 was chosen as the preferred alternative.

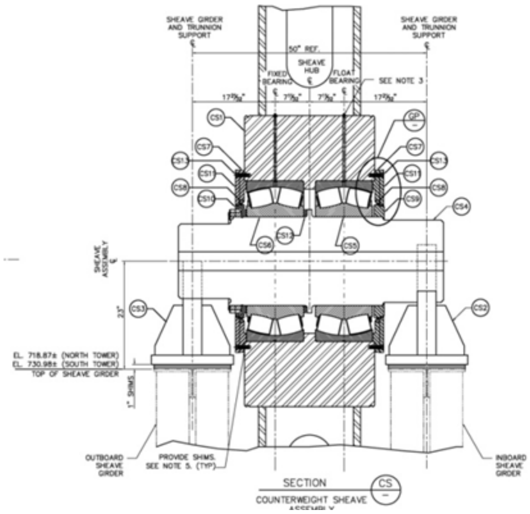


Figure 26. Trunnion Bearing Assembly Details

The mounting arrangements for the roller bearings was driven by the reuse of the existing tower. The support arrangement for the new counterweight sheaves needed to reuse the existing support beams, therefore the spacing of the bearing housings/supports was set. Once the new trunnion roller bearing size was established based on the design of the new lift span, it was determined that pillow block corresponding to the appropriate roller bearing was too large to fit in the available space. However, there was no problem designing supports for a static trunnion shaft to reuse the original bearing footprint and the new sheave could be designed to accommodate the internal roller bearings. The sheave design provides for positive positioning of the roller bearings and also provides a custom robust seal arrangement which has been proven through usage on another project to protect the bearings against ingress of debris.

Sizing of the new counterweight ropes is governed by the reuse of the existing towers and counterweights. Upsizing the counterweight ropes to provide greater capacity would require an increased sheave width to accommodate the larger rope diameters as well as greater spacing at the counterweight connections to accommodate the larger sockets. However, the existing sheave width must be maintained due to reuse of the towers, and the counterweight socket spacing is based on the reuse of the counterweights. The only practical way to provide greater capacity given these constraints was to select a rope with greater capacity. This was achieved by specifying ropes with independent wire rope cores that were manufactured from extra improved plow steel.

One other notable change for the counterweight ropes regarded the connection at the counterweight. At the counterweight connection, the ropes had been grouped into pairs based on how they aligned to the anchorage plate, and each pair of ropes was secured with a common pin. This is not in accordance with the AASHTO requirement that rope connections be detailed to permit replacement of any one rope without disturbing the other ropes. The new design provides a spacer to fill the through hole in the counterweight anchorage plate so that individual pins may be provided to secure each rope at the counterweight connection. The new spacers utilize a multi part threaded design to ensure they are positively secured to the anchorage block independent of the rope pins, and the spacers are fabricated from bronze to address concerns with corrosion over time. A through bolt is provided for each pair of pins to secure them against outward movement.

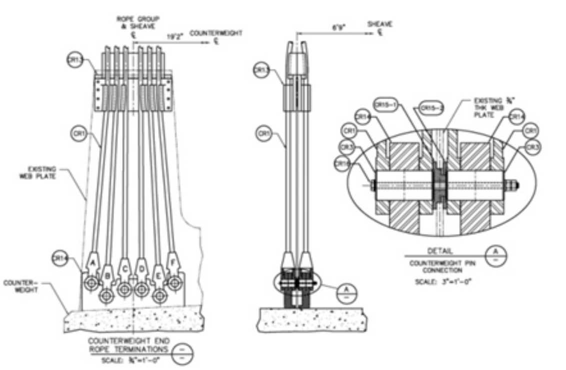


Figure 27. Counterweight Rope Anchorage Spacer Detail.

RESOLUTION OF BINDING ISSUE

In recognition of the binding problem identified in the scoping inspection, rehabilitation of the towers had to encompass jacking and re-plumbing the towers to accommodate the settlement which had occurred. This work would require that the counterweights be supported independent of the towers for the period that the towers were jacked. The original bridge design had provided a means to jack the counterweight to unload the sheave and to the hang the counterweights from the towers. However, that support method did not meet the needs of this rehabilitation.

To facilitate the tower re-plumbing, the final design repurposed the tower support beams and designed a new jacking frame to accommodate strand jacks so that the counterweights could be lowered to, and supported on, the piers. The mechanical plans provided full details for the new jacking frames and connection plans. The structural plans provided details for the pier cribbing system.

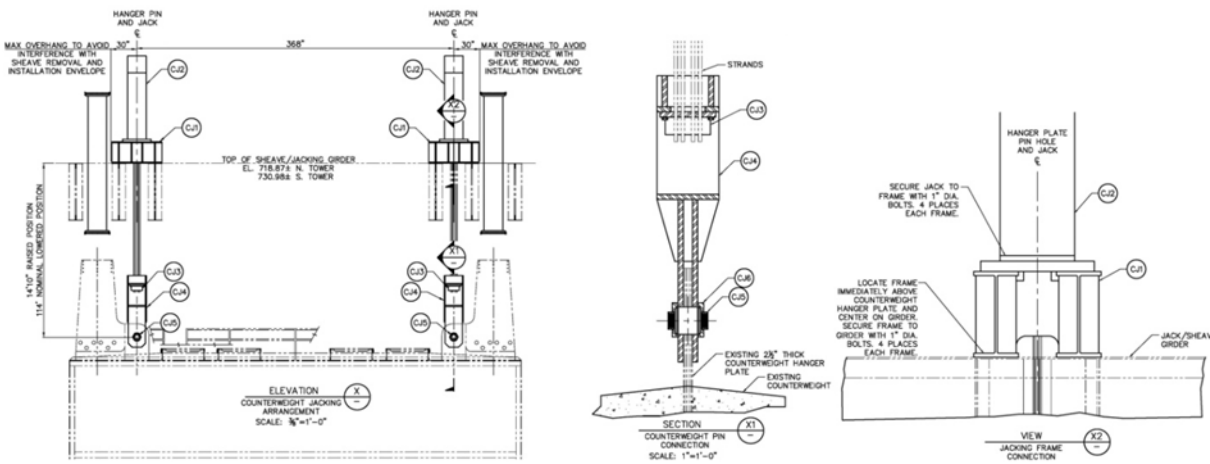
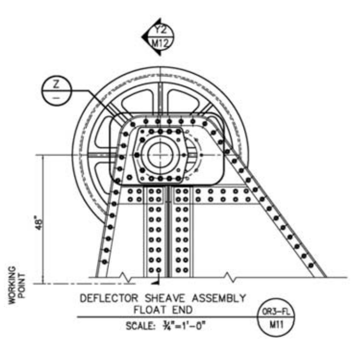


Figure 28. Strand Jacking Frame Details

The mechanical design plans also gave consideration to future settlement through a design detail at the deflector sheaves for the operating rope system which had previously experienced substantial wear due to past settlement. The supports for the new deflector sheaves at the South (Float) end of the bridge were designed to be indexable, so that if future settlement occurred, the deflector sheaves could be set back to relieve any contact with the tower leg and maintain proper function. A rudimentary procedure was provided on the design plan to indicate the intended functionality and sequence for this work.



DEFLECTOR SHEAVE INDEXING PROCEDURE:

FOUNDATION SETTLEMENT ISSUES AT THE SOUTH TOWER HAS PREVIOUSLY RESULTED IN HARD CONTACT BETWEEN THE NORTH END DEFLECTOR SHEAVES AND NORTH TOWER LEGS. THIS INDEXING PROCEDURE IS PROVIDED TO ACCOMMODATE FUTURE SETTLEMENT TO AVOID HARD CONTACT BETWEEN THE DEFLECTOR SHEAVES AND TOWER LEG.

THE DESIGN PROVIDES FOR CLEARANCE BETWEEN THE RIM OF THE DEFLECTOR SHEAVE AND THE FACE OF THE TOWER LEG. IF CONTINUED SETTLEMENT OCCURS AND RESULTS IN A REDUCTION OF THIS CLEARANCE OR CONTACT, THEN THIS INDEXING PROCEDURE SHOULD BE EMPLOYED. STARTING WITH THE LIFT SPAN IN THE SEATED POSITION:

1. OBTAIN A TEMPORARY NAVIGATION CLOSURE.
2. RELEASE ALL STORED ENERGY (I.E. WINDUP) IN THE OPERATING MACHINERY, INCLUDING OPERATING ROPES.
3. TEMPORARILY SUPPORT THE OPERATING ROPES AT THE DEFLECTOR SHEAVES.
4. REMOVE THE DEFLECTOR SHEAVES.
5. REMOVE THE MOUNTING BOLTS FOR SUPPORT-FL.
6. SHIFT SUPPORT-FL 3" SOUTHWARD IN THE SLOTTED GUIDES.
7. REINSTALL AND TORQUE THE MOUNTING BOLTS FOR THE SUPPORT-FL.
8. REINSTALL THE DEFLECTOR SHEAVES. SECURE THE DEFLECTOR PIN.
9. REINSTALL THE OPERATING ROPES.
10. TENSION THE OPERATING ROPES TO OBTAIN UNIFORM LOAD BETWEEN EACH PAIR OF OPERATING ROPES.
11. RETURN THE LIFT SPAN TO OPERATIONAL SERVICE.

NOTE THAT TWO POSITIONS OF ADJUSTMENT ARE PROVIDED WITH THE INTENTION THAT THEY BE UTILIZED, IF NEED BE, IN A STEPWISE SEQUENCE. FOLLOWING ANY ADJUSTMENT, BOTH THE MOUNTING BOLT THREADS AND THE SLOTTED SUPPORT SHOULD BE COATED WITH AN ANTI-SEIZE COATING TO PROTECT AGAINST CORROSION.

Figure 29. Deflector Sheave Indexing Procedure

CONCLUSION

The rehabilitation of the mechanical systems for the Columbus Road Lift Bridge meet the project objectives to provide new, more efficient components. The new mechanical systems are robust, efficient and require minimal maintenance, and should operate reliably for years to come. In addition, the design has addressed settlement issues identified during the scoping inspection and has provision to mitigate the effects of future settlement on the operating systems.

The contract plans were completed in August 2011. Construction is currently ongoing with anticipated completion late summer/early fall 2014.

Figure 30. The existing Columbus Road Lift Bridge in the sunset of its career in late 2011 prior to commencement of rehabilitation.

HEAVY MOVABLE STRUCTURES, INC.
FOURTEENTH BIENNIAL SYMPOSIUM

September 15-18, 2014

Niantic River Bridge Bascule Erection

Ronald Kief, P.E.
Cianbro, Inc.

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Overview

A new Amtrak Bascule span was installed by Cianbro, in Niantic CT, in February and March of 2012, with assistance from its joint venture partner Middlesex.

The Span itself was designed for Amtrak by Hardesty & Hanover of New York. , NY.
The span length, measured from Trunnion centerline to Rest Pier bearings is 138 feet. The large counterweight boxes set atop the heel girders extend 37 feet behind the trunnion centerline, giving the span an overall length of 175 feet. The total finished bridge span weight is on the order of five million pounds, or 2,500 Short Tons.

The work of assembly and erection was done almost entirely from barges on the water, and the large scale of the assembled pieces and narrow work space made this a particularly challenging project.

As an example, the largest single pick made was on the order of 138 short tons, and the weight of the float-in for the assembled toe section was measured as 421 short tons.

This presentation and paper provide a "deck-eye" view of the operation as it occurred.

The salient points I will address in this paper are the following.

- Bascule Fabrication
- Barge Unloading
- Bascule Heel Erection
- Bascule Toe Assembly
- Bascule Toe Float-In
- Bascule Toe/Heel Splice makeup

Pertinent to the job not discussed are the following:

- Heel Shoring Design
- Approach Spans Erection
- Trunnion Tower Erection
- Span Balance
- Strain Gauge Test Results
- Span Jacking Adjustment systems

If time allowed for discussion of the entire project, these would be included. For brevity, they are not.

Bascule Fabrication and Shipping

The bascule itself was completely fabricated and pre-assembled in the G & G facility in Alabama, adjacent to the Black River. The Bascule Heel girders are shown in the photo below, during the fabrication phase.

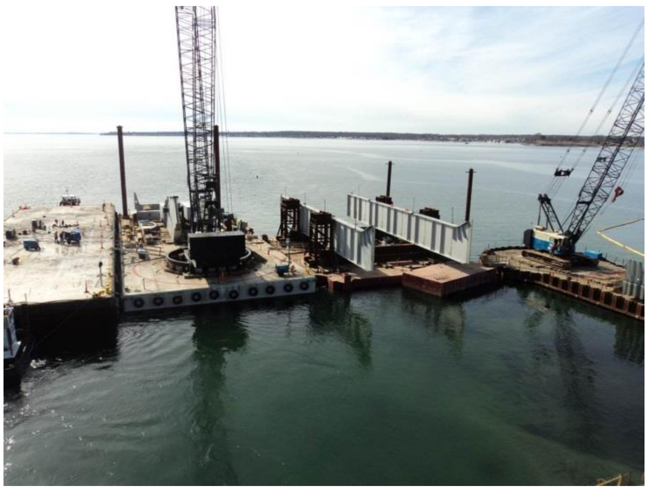
The assembly shown in the photograph below is at the stage after coatings have been applied and fit-up has been assured by assembly in the shop.

All the major components were rolled out directly onto the 60'x222' delivery barge, using a track system, saving the fabricator from employing a very large crane and saving considerable cost. Barge loading at the Black River began in late January, 2012.

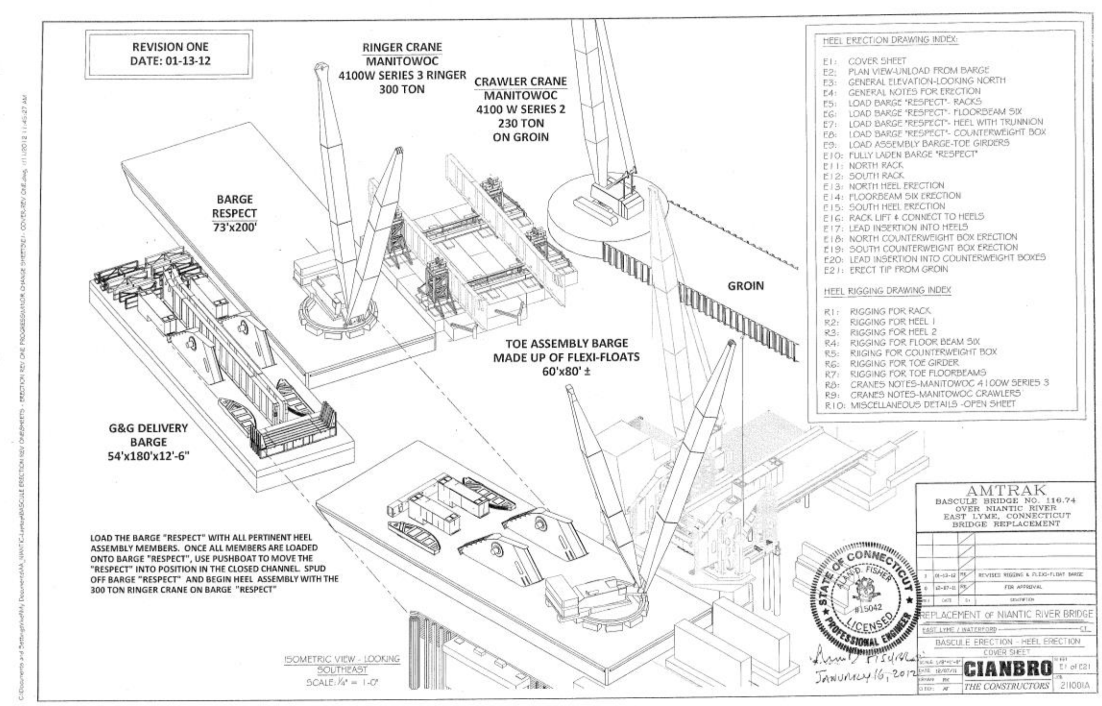
The barge was in transit from Alabama to Connecticut for several days, arriving at the Niantic River Site on Saturday February 11th, 2012.

Barge Unloading

The delivery barge was unloaded using barge-mounted ringer crane, a Manitowoc 4100W Series 3, mounted on the ringer on the 70 x 235 foot barge "RESPECT". This crane, mounted on a ringer with its associated counterweights, has a Safe Working Load of 300 short tons.



The delivery barge arrived on February 11th, 2013. The plan in place had to be revised with only two or three days to spare, as the actual location of the pieces on the delivery barge deck was changed the same week as the delivery was to be made. Such changes may seem minor, but when the crane used for unloading is near its limit, a few feet of radius can define what can be achieved at all.

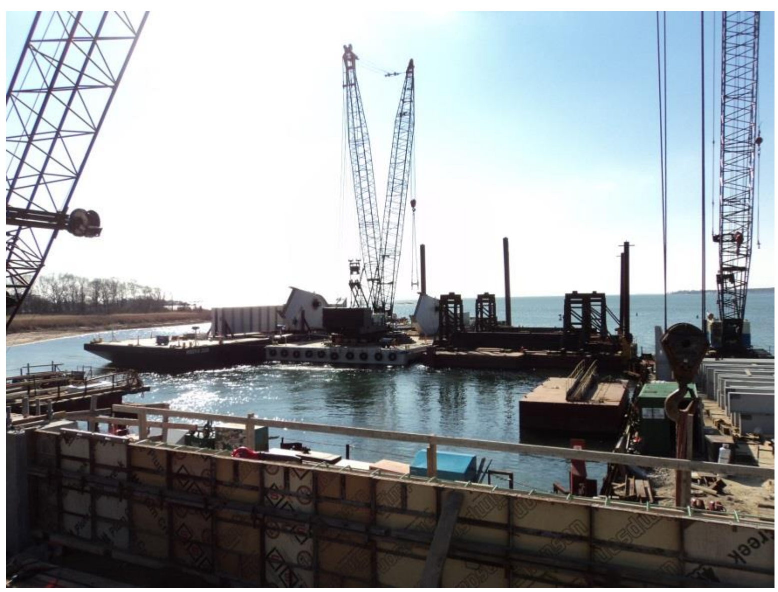


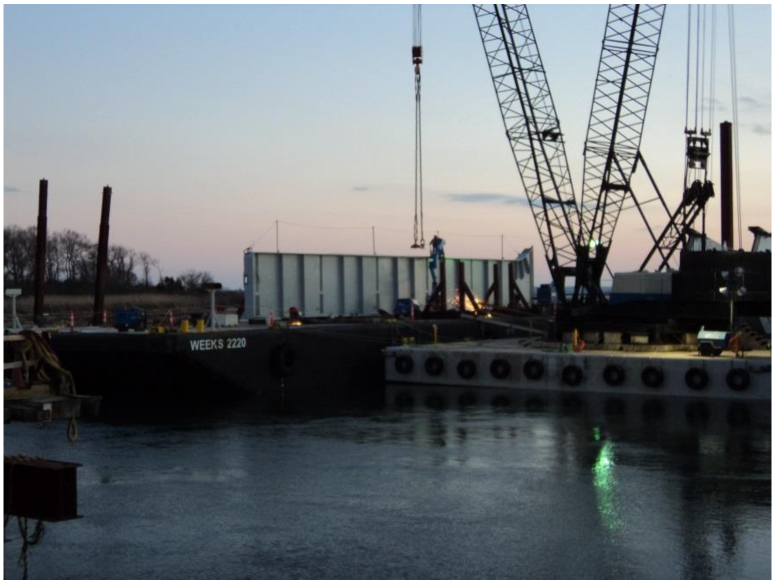
Fortunately in this case, all the components were offloaded without having to relocate the barges, except for in one instance. In that single instance, the delivery barge was rotated about its axis. This allowed the second heel girder to be offloaded by bringing it nearer to the Ringer Crane and make the pick at an allowable radius.

Minimizing the movements the delivery barge or the unloading barge greatly facilitated the unloading. In point of fact, all the major components were offloaded in two days of intensive operations.

The unloading had to proceed in a specific sequence, to allow all the major components to be secured on the deck of the Ringer without interfering with one another. Stability analyses were performed to gage the effect on the Ringer for each stage of the unloading.

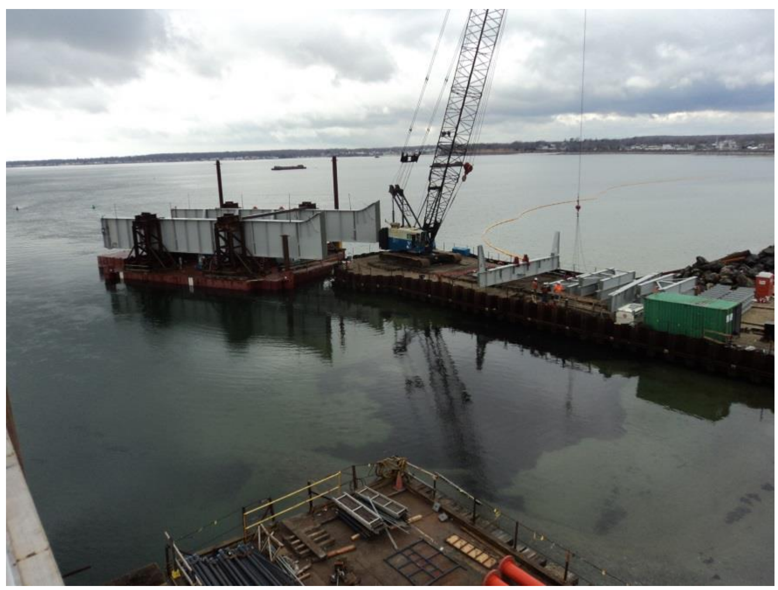
Using the Ringer Crane, the heel components were all off-loaded directly onto the deck of the Ringer, while the two fascia girders were offloaded directly on to the Flexi-Float barge and its attendant frame-work.





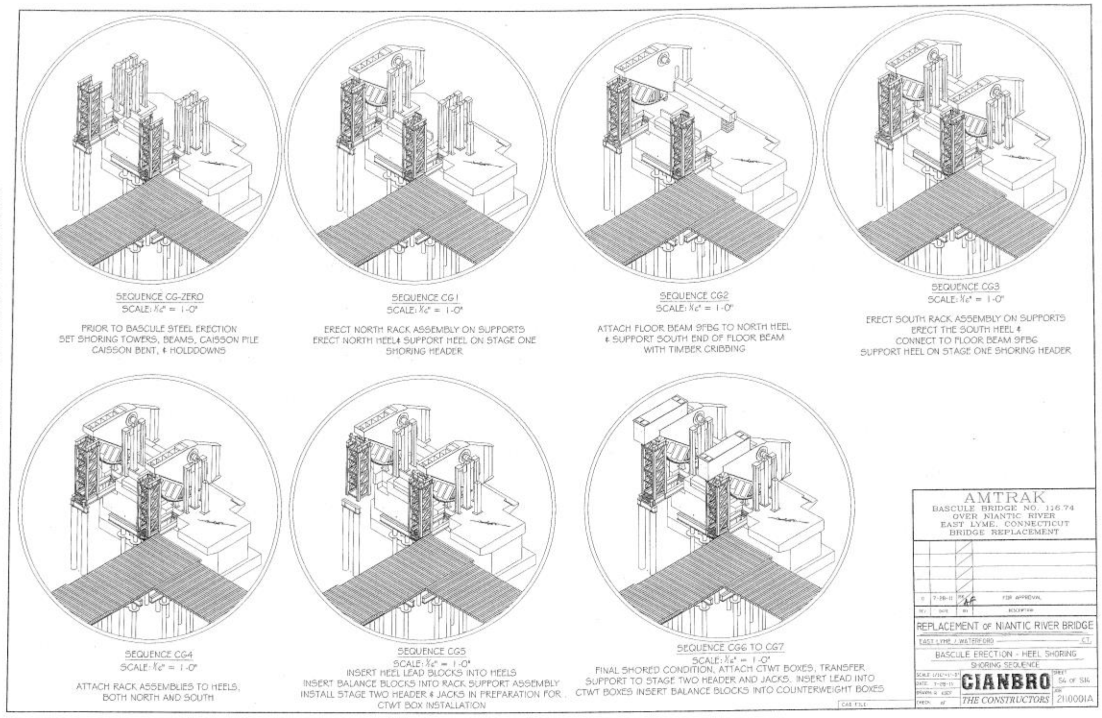
The delivery barge was then moved to a location near the groin, where a land based crane offloaded the stringer assemblies and floor-beams to a laydown area on the groin.

Once the delivery barge was removed from the groin, the Flexi-Float Barge was moved adjacent to the end of the groin so that the land based crane could begin assembling the Float-in. Bascule heel erection at the bascule pier proceeded immediately after the unloading was complete.



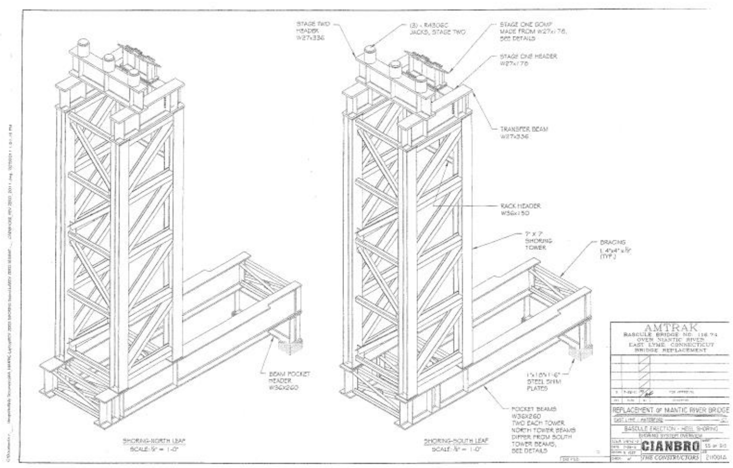
Bascule Heel Erection

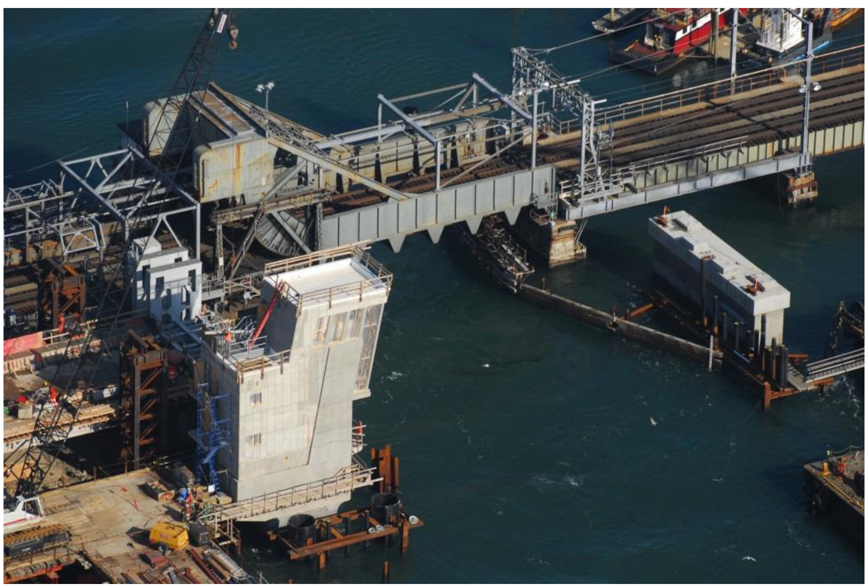
Heel erection began on Monday, February 13th, 2013. The first step was to move the barge from its location outside the channel, beginning the channel outage, which was to last five days. The complete sequence of erection is illustrated below.



Trunnion towers and shoring towers had all been assembled or emplaced prior to the arrival of the steel delivery, so the erection could proceed immediately.

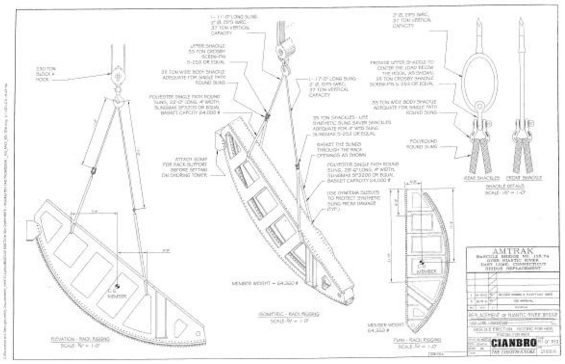
The shoring system was designed for a 600 short ton maximum load. The tower was fitted with support beams to accept the upper end of the rack, as well as the rear of the heel girder. Shoring system is illustrated in the drawing below.



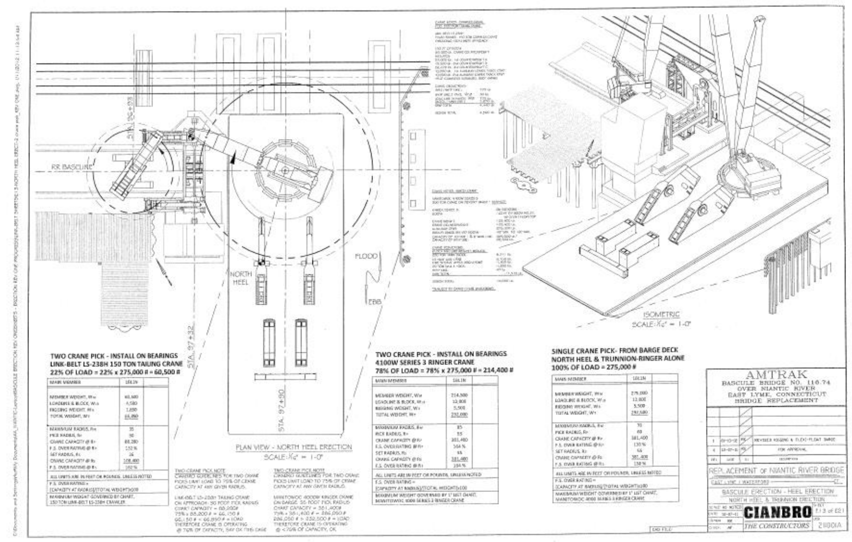


The first component erected was the rack assembly, on the north side. This component was supported on jack-stands and the shoring tower. The rack was set slightly below and to the rear of its final position, so that the heel could be set on the bearings without interference.

It should be noted that the control house was quite tall, standing four stories above the pier top. The face was battered and projected outwards towards the east or channel side. This presented a problem with swinging components into position. In theory all picks and swings could be made without interference from the control house, and the plans were laid out accordingly. In the practice in the field this proved to be "too close for comfort" for the operator and crew.

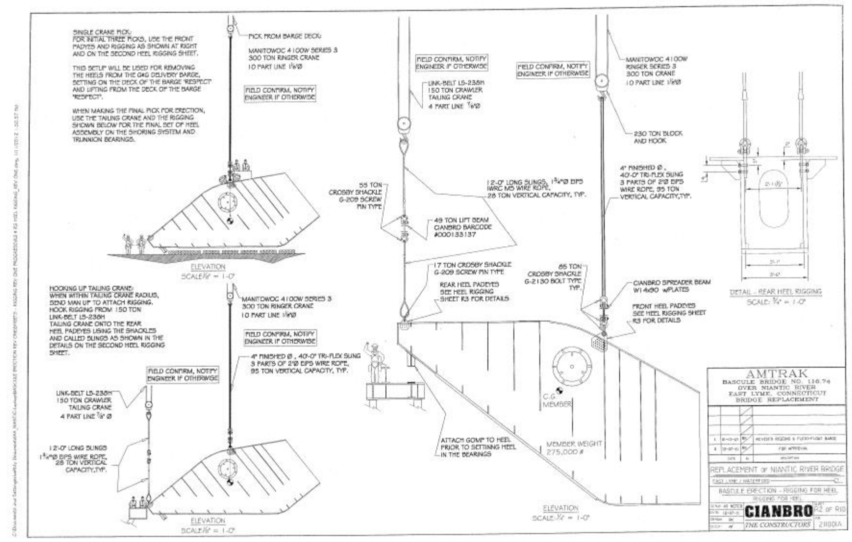


The crew opted to make the initial picks, move the barge slightly outboard, away from the pier, swing the load 90 degrees. Then, with the load still suspended from the hook, the tugs moved the barge back into position at the pier to make the set. This proved to be relatively simple and the tug crews and barge crew handled this movement with very little time spent. Safety was greatly enhanced and interference with the control house was thus avoided.



The next step was the heaviest pick of the process, the heel girder itself. The entire assembly weighed 275,000 pounds, or 137.5 short tons. The North heel was set on Wednesday, February 15th, 2012.

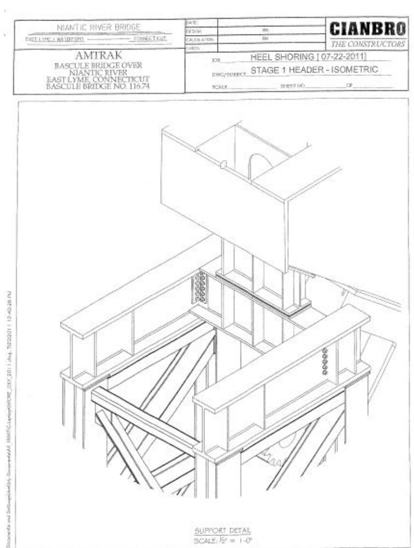
While the ringer could make the pick and set on its own, it could not upright the heel into the proper position to set it down on the bearings.



The rendering of the heel required the use of a tailing crane. This second crane was a Link-Belt LS-238, which was set up on crane mats on the approach span.

Once the Ringer had picked the north heel girder from the deck, the barge had to be moved a few feet to the eastwards, away from the pier. As described above, this allowed the swing to be made and the pick could clear the control house. Once the heel had been traversed ninety degrees, and the control house had been cleared, the barge was moved inboard again. The load remained suspended on the hook throughout this repositioning. Spuds were dropped to hold the barge in the channel as the work progressed.

The heel was aligned over the bearings, well above its final location. The tailing crane hooked on to the tail rigging, and the heel was gradually rendered upright. Once the heel was upright, both cranes lowered cable simultaneously so that the trunnion came to rest directly over the bearing.



The rear end of the heel girder was supported on the trunnion tower.

Next step was to install the Main floor-beam, Floor-beam six. This rectangular box beam connects the two heel girders. The Bottom chord of this floor-beam sits directly over the machinery enclosure. The connection to the north heel was made up, and the southern end of the floor-beam was temporarily supported on 20K shoring.

Following that, we installed the south rack on its supports, lower and to the rear of the final position. The South heel was then installed in the same manner as the north Heel, using the Ringer and the tailing crane, moving the barge outboard and inboard as required to clear the control house. The connection from south heel to floor-beam six was made up while the heel was still supported on the two crane hooks, and the trunnion was then set down into the bearing.

Once the heels and floor-beam connections were made up, and the bearings adjusted by the millwrights, but before the heel lead could be inserted, the next step could begin: Attaching the racks to the heels.

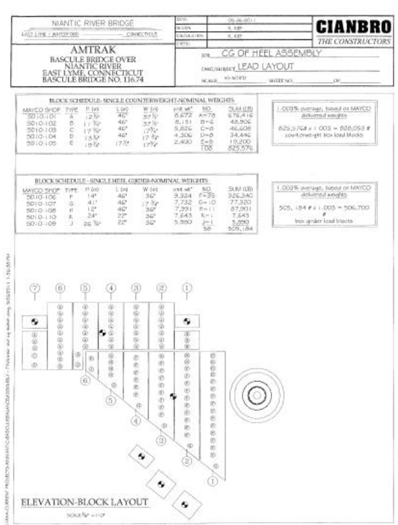
The completed erection for the closure week is shown in the photograph below.

For access to the rack bolts the crew was required to get down inside the heel pockets to make up the connection. The rack was rigged from above and jockeyed into position by the approach span crane. Pins were than inserted into a minimal number of holes, aligning the bolt holes. With several ironworkers going down inside the heel pockets to install bolts through the bottom chord of the rack, the shim pack plate, and the rack top flange, the bolting was completed.

Bascule Lead Installation and Counterweight Box

Once the rack connection was made up, insertion of the heel lead blocks could begin. These castings were provided with a handling loop by the vendor, weighing about 8,000 pounds on average. The blocks were quite closely fitted to the pocket, with a quarter inch all around for clearance. The process of insertion took about one week to complete.

When all the heel blocks were installed, grout was pumped in to fill the minimal void spaces around the blocks. The counterweight boxes were then swung into position by the ringer crane and the connections to the heel girders made up.



Note that while this process was ongoing, a separate crew of ironworkers were busy at the groin and on the flexi-float barge. This crew was simultaneously assembling the Float-in portion. This work is treated further down in the text.

After the counterweight boxes were installed, the counterweight pockets also had to be filled with lead castings. These castings weighed 9,000 pounds on average. The approach span crane and ironworker crews performed this work, which again took a week to complete.

At this stage, 55 pound lead pigs were installed in the balance pockets, following a pattern to provide a balanced condition at the initial rotation of the span. The shoring towers at this stage were as heavily laden as they were ever going to be, with a design load of 600 short tons per tower. All was in readiness for the float-in.

The complete heel assembly is illustrated below. This includes counterweight boxes and counterweight box lead.

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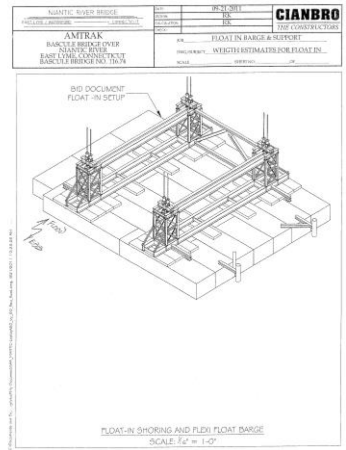
Bascule Toe Assembly

The initial step was on the day of steel offloading from the delivery barge. This was to install the fascia or "Toe" girders atop the pre-arranged lifting girders and jacking frame. These two large fascia girders were approximately 91 tons a piece. They were lifted directly from the G&G delivery barge using the 300 ton Ringer Crane, and set down on the flexi-float barge in the desired location on the frame. The unloading of the main toe girders all occurred on February 12th, 2012.

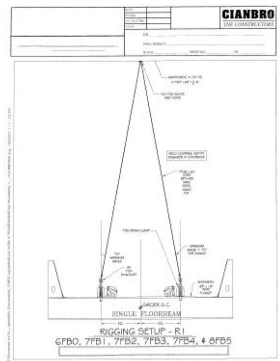


Note that the south Fascia girder was offset slightly to the south of its final position on the support system. Provision for lateral jacking was provided to slide the fascia girder into its final position when the time came.

Note also that the work described here occurred simultaneously with the lead installation of the heel as described above. After all the heel components except the counterweight boxes had been erected at the pier, the ringer was moved to a position outside the channel and moored.

The toe assembly barge, which was composed of assembled flexi-floats, was moored in a location adjacent to the groin. Here, the work could proceed using a land based crawler crane on the end of the groin. This crane was Manitowoc 4100W Series 2, with a safe working load of 230 short tons.

The floor-beams were installed, making up the connections on the north fascia girders as work progressed. The south end of the floor-beams was supported on solid blocking atop the flanges of the south fascia girder. Stringer assemblies were temporarily supported by a trapeze system using threaded rod as the work progressed, one bay at a time.



Once all floor-beams and stringer assemblies were in place, the south fascia girder was jacked laterally into position and the connections made up by the ironworkers. The Toe superstructure steel was basically complete at this point.

Ties, track, miter rails, and the majority of the galvanized catenary support frame were then installed. As the last lead blocks were being inserted into the counterweight pockets, the Flexi-Float barge was ready to move.

Bascule Toe Float-In

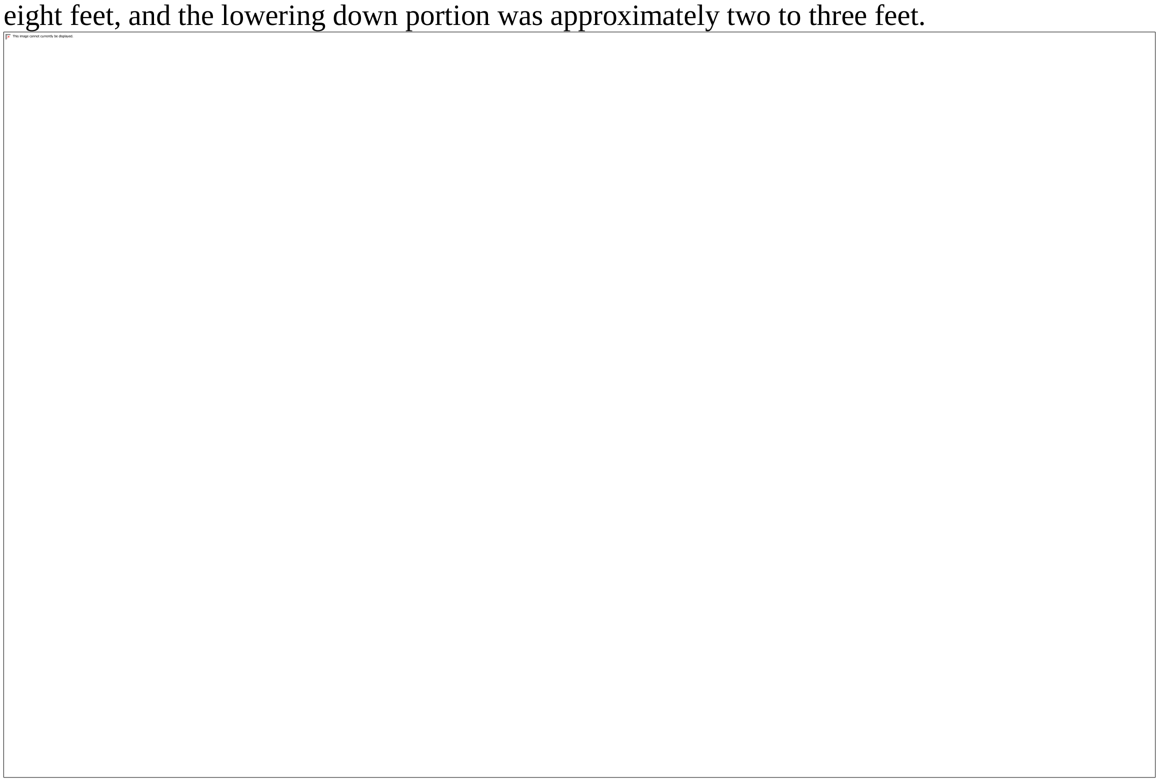
The float-in occurred on March 26, 2013.

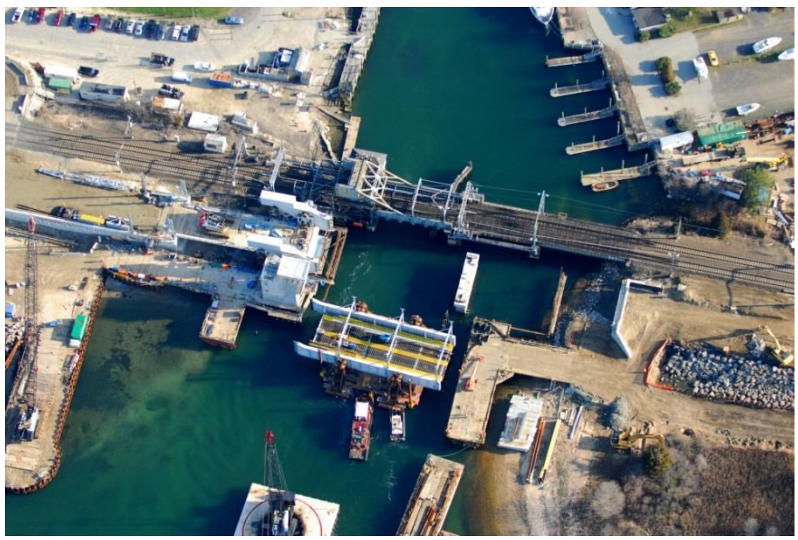
Ideally, one would float in at high tide, let the load down on its temporary supports, let the tide run out, and float the barge away. Recalling Archimedes, we know that the barge rises as it is unloaded. If we come in at the top of the tide, once the tide runs out sufficiently, the barge can be removed without difficulty. This is true so long as the tidal amplitude is sufficient. Where it is, this methodology is preferred.

Unfortunately, the expected tidal amplitude in the river mouth on Long Island Sound was small, on the order of two feet. An elementary analysis indicated that as the barge was unloaded, the increase in freeboard would match the decrease in water surface elevation due to the tide running out. Since the tide would drop two feet at its maximum and the barge would rise two feet as it was unloaded, the barge would remain stuck beneath the toe.

We needed to provide a system for jacking the beams up and letting them down and still allow the barge to be removed out from under the Toe assembly.

The Toe assembly was erected atop two lift beams, with the low chords of the fascia girders a few feet off the deck of the Flexi-float barge. The vertical jacking system consisted of four tower stations, 21'-6" high, each with two 200 short ton center-hole jacks. The jacks were used to raise the lift beams, employing 1 5/8" diameter dywidag rods to raise the Toe Assembly to a sufficient elevation to clear the piers. Total lift of the assemblage from the barge deck was approximately eight feet, and the lowering down portion was approximately two to three feet.



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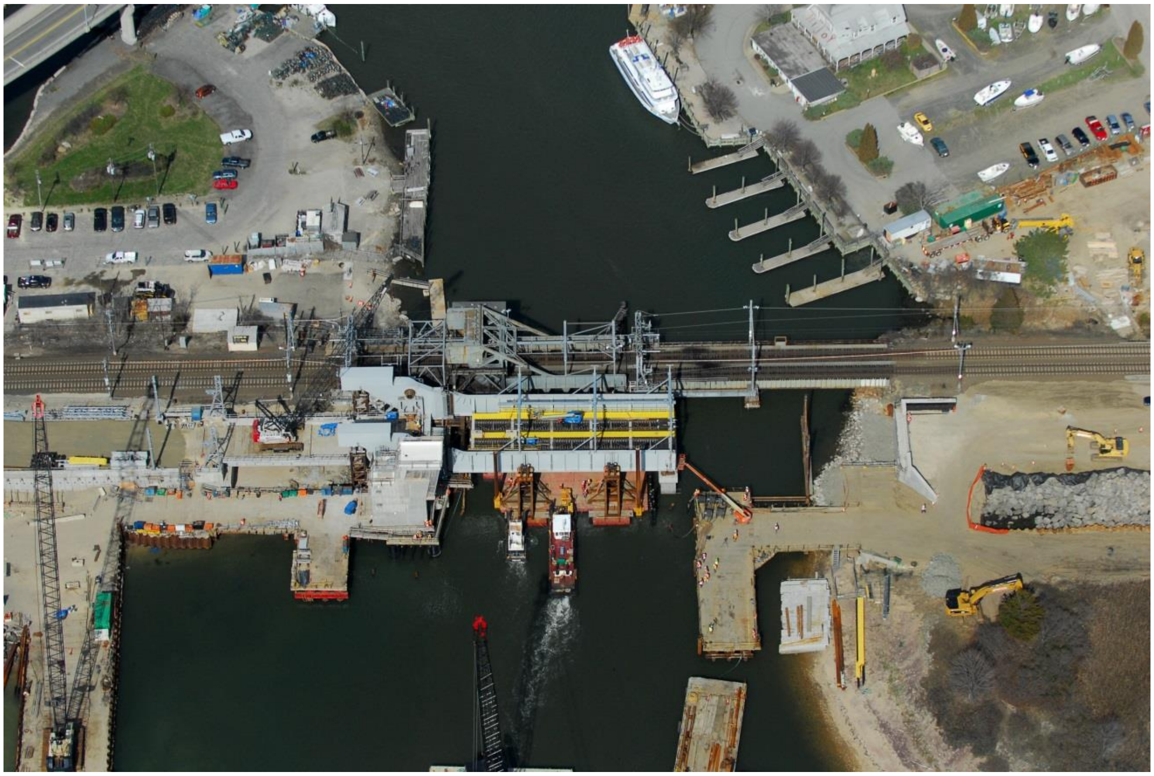


On the east end, at the rest pier, two temporary stainless steel-on-Teflon bearings were provided. On the west end, at the Bascule pier, a support system had been pre-assembled and was ready to accept the west end of the toe. The splice make-up system consisted of two large brackets, channels to act as tracks, and Hillman rollers, and assorted jacks and rods. Jacks and rollers were pre-positioned prior to the float-in for the splice make-up. The splice make-up will be addressed in the next section of this text.

As the tide was nearing its full, the barge was moved into the channel and maneuvered into alignment longitudinally, but slightly to the eastwards. The low chords hovering, as it were, a foot or so above the temporary bearing supports. At this point the entire asssemblage was approximately eight and a half feet above the barge deck.

With the Toe aligned longitudinally approximately three feet to the east of its final location, the process of lowering down could begin. By stages, coordinating the jacks at all four towers, the lifting beams were lowered onto the support points using the center-hole jacks. After lowering approximately eight feet, the lift beams were no longer in contact with the toe low chords, and the barge was effectively unloaded. At this point, the assemblage had been lowered two and a half to three feet from its maximum.

This did not mean that the flexi-float barge could be removed at once, however. The two towers on the north or upstream side of the span were effectively trapping the barge in place, as the tops of the towers projected above the top flange of the span.

Crews secured the span on its temporary supports, and left off for the night crew to remove the upstream side towers. The night crew worked diligently so that by morning, the towers were removed and the unloaded barge was removed from beneath the span and moored outside the channel.

Based on the jack pressure data collected from the four stations, the weight of the entire float-in assembly was estimated to be 842,000 pounds, or 421short Tons.

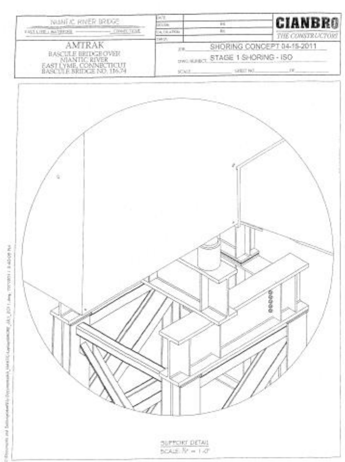
Bascule Toe/Heel Splice makeup

In this operation, completed on March 27, 2012, the toe assembly was systematically jacked and slid longitudinally so that the splice connection could be made up. This required a rather elaborate system of jacks and rollers, best illustrated in the accompanying figure.

Horizontal sliding in the longitudinal direction was accomplished by a paired set of horizontally mounted center-hole jacks. Threaded rod was used to pull the toe westwards, in the direction of the bascule pier.

Jacks were mounted on the rest pier as well, to allow for jockeying the Toe webs vertically as it was slid into the slot in the heel girder webs. Porta-powers on the rest pier allowed for horizontal adjustment of the toe webs.

In addition, three 430 short ton jacks beneath each of the counterweight boxes allowed a certain amount of vertical maneuver in the heel section.



It took most of the day, but in the end, we were able to slide, jack and make up the Main splices in both webs. The bascule span was essentially complete, save for the long labor of rattling all the bolts in the structure.

Much more work remained to be done, but at this point, the lion's share of the difficulties had been surmounted. After strain gauge testing and balance adjustments, the bascule was rotated up and out of the channel.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEENTH BIENNIAL SYMPOSIUM
SEPTEMBER 15-18, 2014

Norfolk Southern Railroad
Tombigbee River Bridge Cable Replacement

Michael Collier
Scott Bridge Company

Introduction

The Norfolk Southern Railroad Lift Bridge over the Tombigbee River in Jackson, AL carries approximately 7,300 trains a year. The bridge, installed in the late 1940's, consists of a 232'-10" vertical lift span over the main channel of the river. The operation of the lift span is vital to the transportation interests of the region in order to move products via the river. The bridge is operated by electric motors that power the drive shafts which rotate two operating drums located at the center of the span. Each operating drum has two uphaul ropes and two downhaul ropes. The uphaul and downhaul ropes had been replaced about 20 years prior to this project, but needed to be replaced again. The bridge has 24 counterweight ropes at each end of the span connected from the lift span to the counterweight box. The original counterweight ropes had been operational for over 60 years without incident.

Norfolk Southern Tombigbee River Bridge June 20, 2013

Norfolk Southern contracted with Modjeski and Masters to investigate the condition of both the counterweight ropes and operating ropes. The decision was made to replace all of the counterweight ropes and operating ropes. Modjeski and Masters then developed a set of construction specifications and plans for replacing the ropes.

Scott Bridge Company was awarded a contract to replace both the counterweight ropes and operating ropes. The work was to be completed during 2 separate 72 hour river outages. Prior to the outage, the dates and times that the river would be closed were provided to all of those with navigation interests on the Tombigbee River. Those impacted by the river outage were depending on the project team completing the project in the allotted time. The work was also to be coordinated with train traffic, so as to minimize the impact to the train schedule. This paper will discuss the preparatory operations and actual means and methods used to replace the operating ropes and counterweight ropes.

Outage Preparations

Due to the location of the bridge, all of the work was to be completed from barges. A Kobelco CK 1000 crane, 100 Ton capacity, was used to perform the work. The crane had 180' of boom installed to allow for the crane to reach both the upstream and downstream counterweight sheaves at one tower from the downstream side of the bridge. The reach would eliminate the need for barge movements during the outage. The crane was located on a 50'x120' spud barge. An additional deck barge was mobilized to the site in order to store material. The barge was also used as a work platform during replacement of the operating ropes.

The two river outages were to occur with only 4 days between the first and second outage. Therefore, the project team had to approach the pre-outage setup with the intention of not only getting setup for the first outage, but the team also made any preparations that could be performed for the second outage. The sheave covers at all four counterweight sheaves were removed prior to the outage. The counterweight rope keeper plates were prepared to be removed at all locations.

There were many obstacles to clear regarding access prior to the outage. There was no access to the top of the counterweight sheaves, which was required to clean the sheave grooves and to verify correct installation of the new ropes. SBC fabricated a work scaffold to install during the outage on top of each tower that would allow for access to the top of the sheave. Also, there was no access to the counterweight ropes on the front side of the counterweight sheaves. In order to attach rigging to the existing counterweight ropes, access was required to the ropes on the front side of the sheaves. Access to the front side of the sheaves was also necessary to remove the rigging as the new counterweight ropes were installed. SBC fabricated a work platform that cantilevered from the top of the tower. The design of this cantilevered platform allowed for the platform to be installed prior to the outage, which saved valuable time during the outage.

The new wire ropes were manufactured by Wireco. The original contract plans were used to determine the necessary rope lengths required. The counterweight ropes were supplied to the jobsite with the take-up sockets already installed. Wireco provided the fabricated length of all 48 counterweight ropes. SBC took these lengths and determined a best fit arrangement in an effort to get an equal average length at each of the 4 counterweight sheaves. Also, a predetermined thickness of shims was calculated at each location based off of the initial rope lengths.

The above picture shows the new counterweight ropes being installed at the lift span. The picture was taken as the jacks under the lift span were being lowered and the ropes tensioned.

A Change in Approach

The original bridge design included a W21x73 jacking structure beneath the counterweight box that would allow for the counterweight to be jacked upward, with span in lowered and locked position, in order to release the counterweight ropes. The W21x73 jacking structure was slid back and bolted to the tower when not in use to allow sufficient clearance for the counterweight box to operate. The stored location of the jacking frame created an additional challenge because it was not possible to use a crane to lift and slide into place. Therefore, a lifting frame was developed that would be attached to the tower prior to the outage. The lifting frame was not only able to vertically lift the jacking structure, but the frame also could be slid toward the front of the tower to place the structure beneath the counterweight box. For this project, the original plan was to move the counterweight jacking structure beneath the counterweight box and jack the counterweight box. The counterweight box would be jacked upward until the existing ropes had enough slack to be removed.

The above photo is a side view of the front and back tower legs. Just below the counterweight box, you can see the 4 beam W21x73 counterweight jacking frame in the stored position.

While making pre-outage preparations, it was discovered that the counterweight could only be jacked 11” before hitting the bottom flange of the sheave girder. It was also determined by field measurements that the new counterweight ropes were shorter than the existing ropes by almost 20”. The difference was not a misfabrication, but includes the elastic strain and construction stretch difference and shows that the ropes have elongated considerably over the years. There was sufficient clearance to jack the counterweights and remove the existing ropes, but it would not be possible to raise the counterweight an adequate height to install the new ropes due to the difference in lengths. Therefore, the team had to come up with a different approach to replacing the counterweight ropes. There were multiple options discussed for how to proceed. One option would require jacking both the counterweight box and the lift span. The problem with this option was that large jacks would be required at both locations. This option also would not allow for trains to pass during the replacement. The new plan had to be able to allow for the passing of trains concurrently with rope replacement. The decision was made to jack the lift span upward and seat the counterweight box on the W21x73 jacking frame. The ropes could be disconnected from the lift span and the span jacked down to the seated position. A 4-5 hours train outage would be required in order to disconnect the existing ropes from the lift span. This would allow rail traffic to pass while crews completed removal of the ropes from the counterweight box. Also, crews could install the new ropes to the counterweight box and drape over the sheave with the bridge in the closed position. The lift span would have to be jacked up again to allow for the ropes to be connected to the lift span. This would require another 5-6 hour shut down of the bridge to rail traffic.

Outage Operations

The first 72 hour river outage took place from Monday, July 8, 2013 through Thursday, July 11, 2013. During the first day of the outage, all 24 of the counterweight ropes were disconnected from the bridge and removed. The majority of the new counterweight ropes were also installed to the counterweight box on day one. On Tuesday, the crews completed installation of the new counterweight ropes, including adjusting the tension in the ropes. The two uphaul ropes at the North end of the bridge were replaced on Tuesday afternoon and Wednesday. The river was opened up to traffic prior to the 72 hour deadline

The second river outage took place 4 days later from Monday, July 15, 2013 through Thursday, July 18, 2013. On Monday of the second outage, all 24 counterweight ropes at the South end of the bridge were disconnected from the bridge and removed. Also, the majority of the new counterweight ropes were installed. On Tuesday, the installation of the new counterweight ropes was completed, including adjusting the tension in the ropes. The uphaul ropes on the South end of the bridge were replaced on Tuesday afternoon and Wednesday morning. The downhaul ropes, at both ends of the bridge, were replaced Wednesday afternoon and Thursday morning. The river was opened up to boats prior to the 72 hours deadline.

Counterweight Rope Replacement

The first task to be done in each outage was to simultaneously install the counterweight support structure beneath the counterweight and install the jacks beneath the lift span. In order to seat the counterweight box, the W21x73 jacking frame was slid under the counterweight box. Then, two W36x359 girders, fabricated prior to the outage, were installed between the W21 frame and the counterweight box.

Shown above is the counterweight box support frame and shims prior to the seating of the counterweight box on the frame.

In order to jack the lift span, crews installed 2 – 500 ton jacks under the end floorbeam of the lifttruss where the counterweight was being seated. On the opposite end of the bridge, crews used 2- 200 ton jacks to lift the bridge. The jacks were positioned on fabricated W14x550 jacking beams. As the lift span was raised, the counterweight box was seated on the frame and the ropes began to detension. The lift span was jacked a sufficient height, approximately 11", so that the counterweight ropes loosened from their supports at the lift span. Once all of the counterweight ropes were disconnected from the lift span, the jacks were retracted and the span was lowered and opened to rail traffic. The ropes were then disconnected from the counterweight box and removed from the bridge, while the bridge was open to rail traffic. A lifting beam was fabricated by SBC prior to the outage that would allow for 6 counterweight ropes to be removed at a time. By doing so, crews were able to save valuable time during the river outage.

The above picture shows SBC crews removing the first 6 counterweight ropes using the lifting beam. The SBC fabricated work platform is seen cantilevered from the front side of the tower.

After removal of the counterweight ropes, crews cleaned the grooves of the counterweight sheaves. Also, any burrs observed were removed. After cleaning, rope lube was applied to the grooves.

Once all surfaces were properly prepared, crews began installing the new counterweight ropes. The first step was to attach all 24 ropes to the counterweight box and drape the lift span end of the ropes over the counterweight sheave. The original plan was to use the same lifting beam to install the ropes as had been used to remove the ropes. The lifting beam was fabricated so that the ropes would hang at the same radius as the counterweight sheave. Thus, there was no risk to kinking the ropes during installation. The lifting beam allowed for 6 ropes to be installed at a time. The crews proceeded with using the lifting beam for the first 6 ropes. It was discovered during the first iteration that it was significantly more difficult and more time consuming to handle 6 ropes at a time.

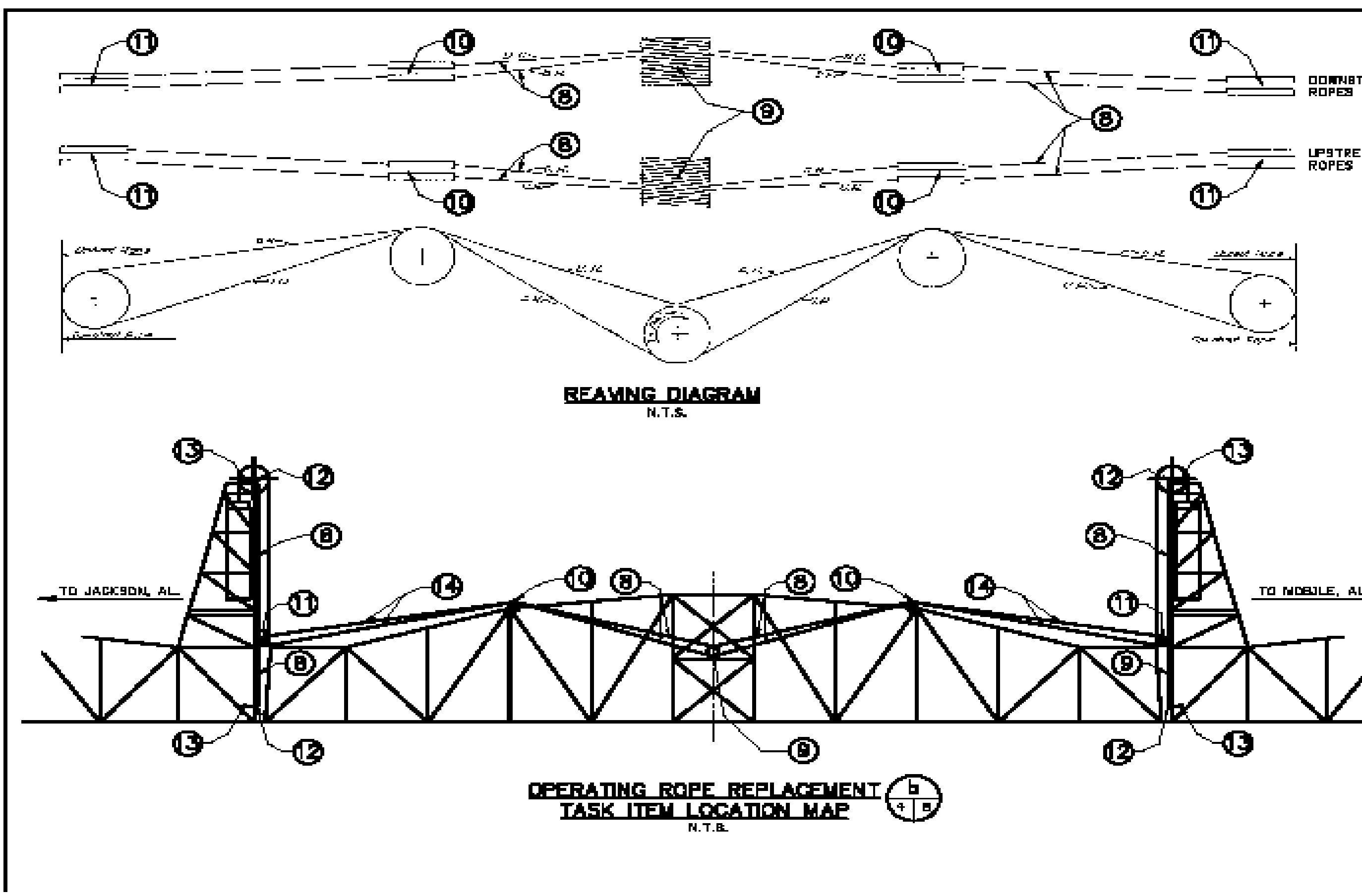
Shown in the picture is the counterweight rope lifting beam during a test lift prior to the outage. The beam was designed so that the radius of the ropes would match the radius of the counterweight sheave.

Crews continued to use the lifting beam to install the ropes, but installed only 2 ropes at a time. After all 24 of the ropes were connected to the counterweight box, the lift span would be jacked to a height, approximately 20”, that allowed for connection of the counterweight ropes to the lift span. During installation, care was taken to ensure that the white stripe, painted on each rope by the manufacturer, was facing outward from the lift span at every location. This was done to make sure that there was no twist in the ropes. When complete with installation of all 24 ropes, the lift span was then carefully lowered until all of the slack was removed from the new ropes and the sockets began to seat.

The lift span was operated at least 4 times after the new ropes were installed to equalize the tension in the ropes. Then, the tension in the new counterweight ropes was checked. The contractor was responsible for adjusting the tension in the ropes so that the tension in each counterweight rope was within 10% of the average for all of the 24 ropes on that counterweight. In order to accurately determine the tension in the cables, SBC used a precision tension measuring device, DynaTension P1000 Tension Meter. After a cable is plucked by hand, the P1000 is able to measure the fundamental frequency of vibration of the cable. This frequency is then computed into a tension, which is displayed on the machine’s screen. One alternate method to using the P1000, would have been to pluck the rope and manually count the number of oscillations in the rope for a given amount of time. By using the P1000, there would be no human error in calculating the exact number of oscillations in a given time period. Once the tension of each rope was measured, shims were installed as required and the tension rechecked. After installing shims, the bridge was operated 4 times in order to distribute the tension through all ropes prior to checking the tension. This process was repeated until all of the rope tensions were within the required range.

Uphaul Rope Replacement

The lift span has two operating drums located at the center of the lift span. Each operating drum has four ropes: 1 uphaul for the North tower, 1 uphaul for the South tower, 1 downhaul for the North tower, and 1 downhaul for the South tower. There was a take-up device located at the top of the tower for the uphaul ropes. The take-up for the downhaul ropes was located at track level on the tower front legs.



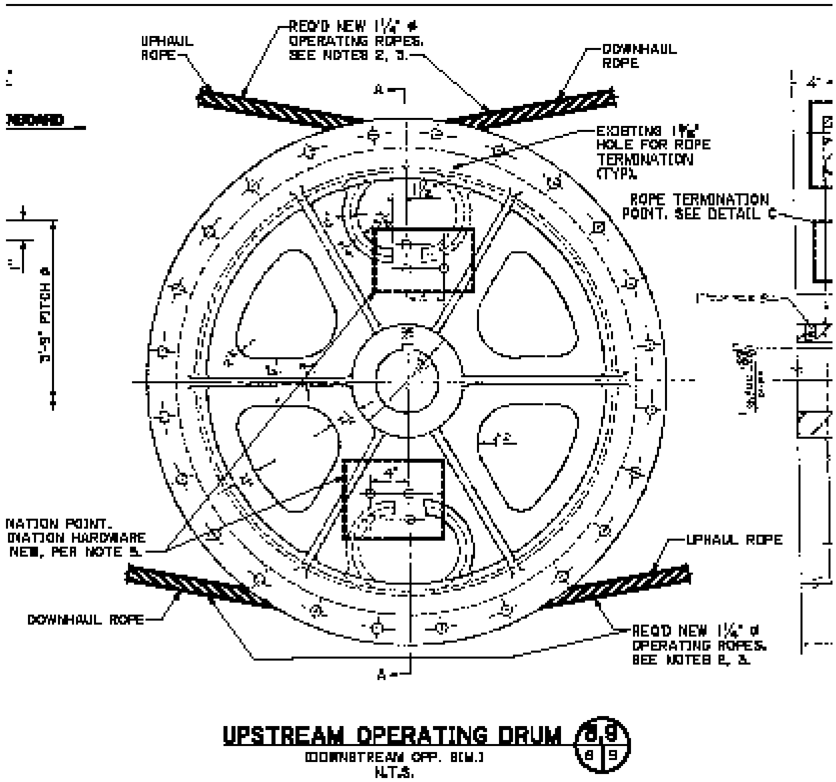
The above shows the operating rope layout on the lift span. The ropes must pass an idler sheave at quarter span. Then, the ropes go around a deflector sheave at the end of the span prior to going either to the top of the tower or bottom, depending on uphaul or downhaul rope.

The barges were positioned perpendicular to the flow of the river in order to perform the operating rope replacements. Each of the 8 operating ropes were pre-cut to plan lengths and individually spooled. The spools containing the new rope were placed on the barge deck directly below the operating drums, which were located at the center of the lift span. The uphaul rope take-up was loosened to allow enough slack in the rope to remove the rope from the operating drum. Crews removed the rope, by hand, from the operating drum. Once all of the rope was removed from the operating drum, crews pulled the new rope up from the barge deck to the operating drum platform. At this time, a Chinese finger was attached between the end of the new rope and the old rope. The crane was then connected to the old rope just above the truss end deflector sheave. The rope was disconnected from the take-up. The crane proceeded to pull the old rope up from the deflector sheave. As the old rope was being removed, the new rope was being installed. By allowing the new rope to be pulled into position by the old rope as it was removed, the new rope was installed correctly around both the idler sheave and deflector sheave. When a sufficient amount of the new rope had been pulled onto the bridge, crews manually installed the rope around the operating drum. Care was taken to ensure that the proper amount of wraps was placed around the drum. The crane was then used to pull the lead end of the new rope up to the take-up at the top of the tower. The new rope was tightened until it matched the pre-outage tension. This procedure was repeated for all four uphaul ropes.

Downhaul Rope Replacement

Similar to the uphaul ropes, the barges were positioned perpendicular to the flow of the river in order to perform the downhaul rope replacement. The spool of new rope was placed on the barge deck directly below the operating drum. A cable spooling machine was placed at the opposite end of the barge, directly below the downhaul take-up device. The rope was loosened up at the take-up device in order to allow sufficient slack for the rope to be removed from the operating drum. Crews manually removed the rope from the operating drum. Then, crews manually pulled the new rope up to the operating drum platform from the barge. A Chinese finger was installed between the new rope and the old rope. The old rope was removed from the tower take-up device. The take-up device end of the old rope was then fed down to the barge and wrapped around the cable spooling machine. The cable spooling machine was then used to pull the old rope off of the bridge. While the old rope was removed, it was pulling the new rope onto the bridge. When a sufficient amount of the new rope had been pulled onto the bridge, crews manually installed the rope around the operating drum. The new rope was then tightened until it matched the pre-outage tension.

While changing out the operating ropes, the project team discovered that the timing of the operating drums did not match the timing that was shown on the original drawings. The operating drums were designed so that the end take-ups for the ropes were located at 6 o'clock and 12 o'clock.



The above is a side view of the outside of the operating drum. The end take-ups are shown at the 6 o'clock and 12 o'clock position of the drum.

The rope lengths provided in the plans were based off of these take-up locations. During previous replacements of the operating ropes, the operating drum had not been timed back to the original location. The drums are designed so that as the shaft is rotated to lower the span, the uphaul rope is paying off of the drum and the downhaul rope is taking up on the drum. Conversely, as the span is raised, the uphaul rope is taking up on the drum while the downhaul rope is paying off of the drum. This means that as the timing of the operating drum is altered, the actual length of the ropes required is also changing. The operating drums were actually rotated approximately 240° from the design location. This resulted in the plan length of the uphaul ropes being 8' longer than required. The plan length of the downhaul ropes was 8' shorter than required. In retrospect, when replacing operating ropes, it would be best to order an excess length of rope that can be field cut to the in-place length required. By taking this approach, the risk of having incorrect length ropes can be avoided.

Conclusion

The cable replacement project was very successful. Norfolk Southern was provided with new operating and counterweight ropes which will serve their bridge for years to come. Also, the bridge will serve a countless number of transportation companies who use the river to move their products from one location to another. The project could not have been a success without the hard work and effort put forth by many individuals on the project team.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Planning, Design and Construction of Canadian National Railway Bascule Bridge 173.20 over the Fox River Oshkosh, WI

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INTRODUCTION

The bridge site is located at Canadian National (CN) Railway M.P. 173.20, Neenah Subdivision over the Fox River. The Fox River Bridge is located in close proximity to the popular Lake Winnebago. The old bridge, built in 1899, consisted of two approach trusses and a swing span truss in the main span. The single track Fox River Bridge was in need of rehabilitation or replacement in order to support safe, reliable service on Canadian National's core line between Winnipeg and Chicago. Located in downtown Oshkosh, Wisconsin, the bridge supports a train traffic level of approximately 40 trains per day, seven days a week, carrying grain, pot ash, sulfur and forest products at an operating speed of 25 mph. The swing span was required to open over 1,600 times per year during the recreational boating season that lasts from May to late October each year.

The project consisted of 3 phases. Phase 1: Project Planning and Conceptual Design, Phase 2: Preliminary Design, Final Design and NEPA documentation, and Phase 3: Complete reconstruction of the historic bridge on the existing alignment. This paper discusses the preliminary study on different feasible alternatives, the selected alternative, final design features and the construction processes of the project.

EXISTING BRIDGE

The Fox River Bridge was built in 1899. The bridge was an open deck, single track, three-span, through truss steel railroad bridge. The North and South approach spans were 148 feet. The through truss swing span over the navigation channel was 176 feet. The bridge length was 472 feet abutment to abutment. The abutments, rest piers and swing span pivot pier were of stone masonry construction, supported by driven timber piling. Strengthening measures were undertaken at both rest piers. During strengthening, each rest pier was connected to a reinforced concrete cap supported on four 54-inch drilled shafts with rock sockets. The swing span was designed to be operated by a bridge tender stationed at the bridge.

Photo1: The existing CN Fox River Bridge, built in 1899

PHASE 1

Phase 1 consisted of planning and Conceptual Design of recommended alternative. A detailed inspection and rating of the existing trusses were carried out during this phase. Investigation of various alternatives for the reconstruction or replacement of the bridge was conducted and guidance for the recommended alternative was provided.

INSPECTION AND RATING OF THE EXISTING BRIDGES:

HNTB performed a structural inspection of the bridge including structural, electrical and mechanical facilities that are integral with the operation of the movable span during the week of June 22, 2009. Collins Engineering performed the underwater inspection and survey. Helms & Associates performed a mussel survey, Hess, Roise and Company performed the historic assessment of the structure and Westbrook Associated Engineers, Inc. performed the survey and created the basemap for the bridge site. The following major observations were made during inspection:

Photo 2: The existing pivot pier

Structural: General section loss of approximately 10% and pack rust due to age of the structure was found on various members such as chords, diagonals and connections. In general, the inspection showed the structure to be in fair condition given its age. The key area of concern was the truss floor system. In addition to monitoring, a new floor system would be required to provide the structure with a longer service life.

Photo 3: The existing bridge cross section

Mechanical: The center bearing, rollers, and track needed to be replaced if a swing span rehabilitation was performed. The rack and pinion gears needed to be replaced.

Electrical: From an electrical and controls standpoint, the bridge was in relatively good condition. That said, during the inspection, several deficiencies were noted.

Photo 4: The existing rest pier

Underwater Inspection: The abutments were both in generally good condition with little mortar loss. The rest piers had been rehabilitated with drilled shafts and a cast in place concrete cap that show little deterioration; there was horizontal cracking in the pier caps that required routine monitoring. The original masonry aspects of piers 2 & 4 have widespread deficiencies including random areas of mortar loss (some 2' long with 18" penetration), moderate deterioration of the timber grillage, large undermining voids (up to 5') with exposed timber support piles, timber piles no longer in contact with the timber grillage, and grout bags present but not in contact with the timber grillage. The pivot pier was in overall satisfactory condition with only random and minor joint mortar section loss. The timber grillage around the pivot pier was exposed but the piles were not. Due to poor condition of the piers 2 & 4, it was recommended that inspection should be repeated in 2-3 years.

Thorough ratings of the existing swing span and approach span trusses were carried out after the physical inspections were done. Available as-built plans in conjuction with the field inspection measurements were used for section property calculation of the truss members. Based on the inspection and the rating analysis, it was concluded that, in general, the Fox River Bridge approach trusses and swing span rated as expected for vintage 1899 lattice truss spans. This structure type was economical in its days; however it is considered "light" by today's standards and the lack of hangers and posts complicates the rating of the diagonals. The floor system cracks were a concern, and it was recommended that those should be closely monitored in the event that the members were not replaced in the near future. Complete rehabilitation of these trusses was recommended in order for them to meet the CN design criteria of Cooper E-90. The existing substructure was adequate for Cooper E-80 loading conditions. To maintain this rating, repairs to the rest piers were recommended.

FEASIBLE ALTERNATIVES STUDY:

Various feasible alternatives for the reconstruction or replacement of the bridge that met CN and AREMA structural, track and signal design standards were considered. A Project Development Report was prepared which was the basis for the National Environmental Policy Act documentation, but made specific recommendations as to the best way to meet the stated objectives. Five possible alternatives as shown in the Table 1 were evaluated in this analysis phase.

Rehabilitation and strengthening of the existing bridge superstructure in Alternative 1 does include the repair of the mechanical and electrical system of the bridge. New superstructure in Alternative 2, 3 and 4 is considered with new mechanical and electrical system. Complete replacement of the existing bridge on a parallel or existing alignment by using an existing movable span having been retired and relocated to the project site was also considered as an alternative.

Alternative	Alignment	Primary Elements
1	Existing	▪Rehabilitate Superstructure ▪Rehabilitate Substructure
2	Existing	▪Replace Superstructure ▪Rehabilitate Substructure
3	New	▪Complete Bridge Replacement
4	Existing	▪Complete Bridge Replacement
5	Existing/New	▪Complete Bridge Replacement with a retired existing movable span

Table 1: Feasible alternatives considered

There were many possible arrangements and combinations of spans and span types; however, each alternative was designed to meet the following major external requirements: United State Coast Guard (USCG) vertical and horizontal clearance requirements, material availability and preference, railroad operational requirements, and historic and environmental constraints. Each alternative was presented to CN with respective detailed construction sequencing including track and marine outages, construction schedules and preliminary cost estimates.

A "Weighted Ranking" evaluation process was used to select the best alternative. The primary factors considered in the "Weighted Ranking" process are shown in Table 2. HNTB concluded that Alternative 3, construction of a new rolling lift bascule span on an offset alignment to the East of the existing structure met or exceeded CN selection criteria. HNTB was directed to proceed with this alternative. However, after 60% final design of the project was completed, due to the challenge to the permit process of the new bridge in a new alignment, CN decided to proceed with the Alternative 4: construction of a new rolling lift bascule span on the existing alignment. The span arrangement study and preliminary design were again carried out for the Alternative 4, aiming at using the already completed design as much as possible.

Criteria	Weighted Factor
Cost	25
Construction Difficulties	15
Environmental	10
Impacts to Rail (Construction)	40
Impact to navigation (Construction)	40
Impacts to navigation	25
Channel Maintenance	25
Impacts to Property owners	15
Bridge Operation & Maintenance	20
Construction risks	20

Table 2: Factors used in the "weighted ranking"

PHASE 2

Phase 2 consisted of Preliminary Design, Final Design and NEPA documentation, including preliminary design and refinement of the Phase 1 recommended alternative, development of final design and construction documents, and the completion of the Environmental Assessment and procurement of all necessary permits in the environmentally sensitive area.

PRELIMINARY DESIGN:

The design criteria were established. A replacement movable span must provide a clearance envelope as prescribed by the USCG: that is a 23' vertical clearance and 125' horizontal clearance. New structures would be designed per the AREMA specifications. The live load would be Cooper E-90 loading with diesel impact at a speed of 60 MPH. New structures will conform to CN guidelines for Track and Structures. HNTB evaluated two types of bascule spans, a Scherzer Rolling Lift bridge, commonly referred to a rolling bascule, as well as a Trunnion bascule. Both structures would effectively meet the clearance requirements, and each type has advantages and disadvantages. The rolling bascule structure has been in service on railroads for nearly 100 years. Single-leaf, through truss rolling bascule span is generally used for applications in which the horizontal clearance requirement is less than 175 feet. Positive aspects of a rolling bascule span over a swing span or trunnion bascule include:

- Live load transfer through structure supports and bearings, resulting in this application being more common in railroad bridge applications due to the railroad live and impact loads.
- Rail joints do not require articulated joint mechanisms found on swing spans.
- As the rolling lift translates horizontally away from the navigation channel during span openings, an unlimited vertical clearance is possible when the span is rotated open to approximately 60 degrees.
- There are no rail locks, easer bars, end lifts or center wedges; only the span locks at the leaf tip which results in few interlocked steps needed to engage the drive motors. This results in a somewhat simpler control system.
- Easy to inspect and maintain due to exposed track girder and rolling surface.

The alternative to the rolling bascule is a trunnion bascule structure. Instead of rolling back on a track frame, a trunnion bascule opens about a single point, a "pin" referred to as a trunnion. A trunnion bascule span does not translate, and can sometimes require less room to operate. That being said, the trunnion bearings themselves must absorb live load and need to be designed accordingly. Positive aspects of a trunnion bascule are similar to that of a rolling bascule with the main difference being the live load path. Owner preference plays a significant role in the selection of a rolling bascule over a trunnion bascule span. Having considered the advantages and disadvantages of each structural type, along with the site-specific constraints related to each bascule type, the decision was made to advance the concept of a rolling lift.

SPAN ARRANGEMENTS

Top of Rail is maintained at 756.50 (NAVD 88) and standard low water elevation as 746.21 (NAVD 88). The final span arrangements consist of one 147'-0" rolling lift bascule span, five 57'-2 ½" Deck plate girder span, one 40'-0" Deck plate girder span and a track girder.

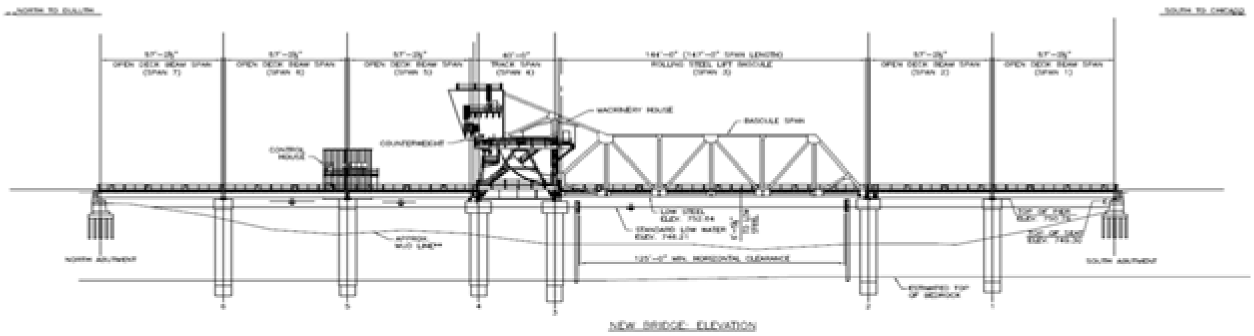

Figure 1: New Bridge Span arrangement

CROSS SECTION

The bascule truss was designed with a floor system consisting of floorbeams spaced at 24'-6" (maximum) and 4 stringers spaced at 2'-6" spanning between the floorbeams. Approach deck plate girders have 4 built-up sections spaced at 3'-0" spacing. After several iterations, it was concluded that these arrangements provided the most efficient structures considering the available structural depth. 24'-0" vertical clearance is provided from the top of rail. Trusses are spaced at 22'-0" apart.

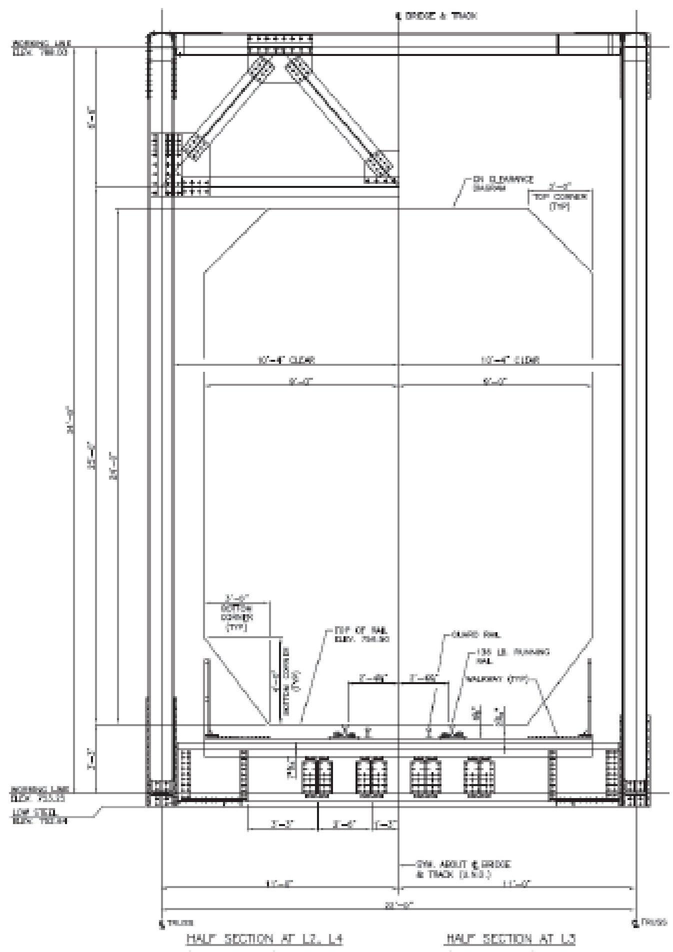

Figure 2: New Bascule Truss Cross Section

MATERIALS

High-strength Grade 50 weathering steel was selected for low maintenance requirements. Grade 70 was not economical due to excessive live-load deflections. High-strength steel bolts conforming to the requirements of ASTM A325 Type 3 were specified. Allowable bolt stresses is in accordance with AREMA specifications. Structural concrete was designed to have minimum 28 day strength of 5000 psi for the pier caps and drilled shafts. Open decks were used over ballast decks as ballasted decks would not be possible for a bascule span. An open timber deck was selected to achieve a lighter dead load. Walkways composed of sections of galvanized bar gratings were designed on both sides of the track. Walkways were supported on extended ties not exceeding 7 feet and designed for 100 psf loads. Galvanized handrails were provided at all locations.

BUILT-UP Vs. ROLLED SHAPED

A study was conducted in order to determine the most efficient section for the Deck Plate girders. Both built-up and standard rolled sections were considered in the study with 4 and 6 beams system. It was concluded that 4 beams with 3'- 5 ½" section depths would provide adequate clearances from top of standard water and 23% more efficient than a feasible rolled shape with 3'-6" section depth. Also, built-up sections would provide 14" clear distance between the flanges of the beams which was adequate to install the diaphragms. After a similar study for the floor system of the bascule truss, built-up sections were used for the floor beams and standard rolled shapes were used for the 4 stringer system.

FINAL DESIGN

Based on the preliminary study, a single leaf bascule warren truss span was designed with a segmental girder and a counterweight. The warren truss has 5 floor panels of 24'-6" long and 1 end panel of 21'-6" long. Floor beams are 36" deep built up sections. Stringers are 21" deep standard rolled sections. The AREMA fracture control plan outlined in Chapter 15 applies to tension chords, tension diagonals, hangers

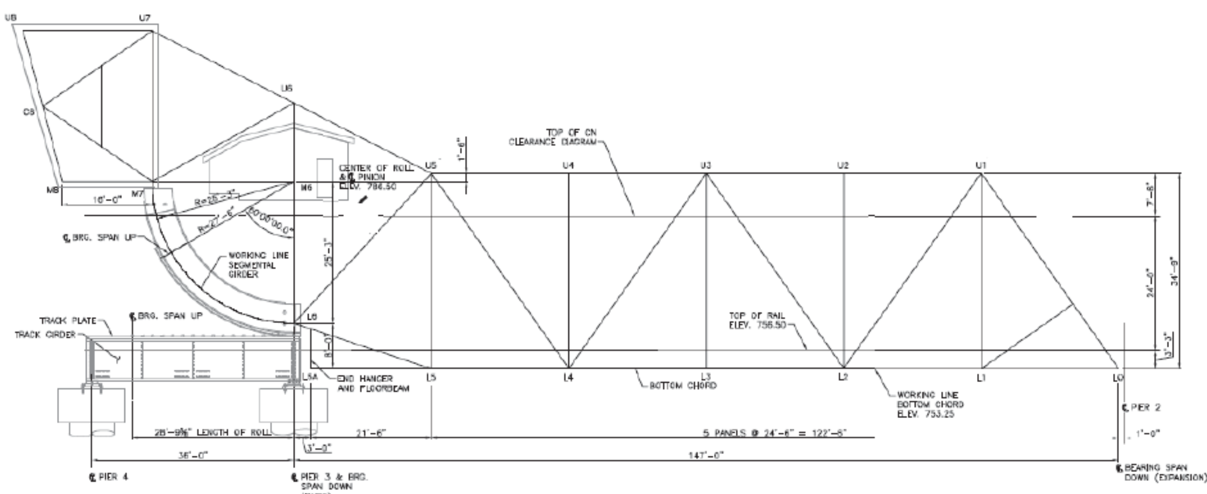

Figure 3: New Bascule Truss Span

and floorbeams. Stringers are not fracture critical members since a redundant 4 stringer system is used. Trusses are cambered for dead load plus a live load of 3000.0 lbs. per foot of track. 57'- 2 ½" approach span Deck Plate girders were designed as a built-up sections with 3'-5 ½" total section depth. 40'-0" Deck span was designed as a built-up section with 2'-3" section depth. The 40'-0" deck span goes over the 7'-6" deep track girder on top of which, the segmental girder of the bascule span rolls. Segmental girders were designed as built up section with working line radius of 25'-3". One end of the segmental girder is bolted to the node L6 of the truss and other end is connected to the counterweight.

Each of the counterweight is designed as a hollow closed cell box. Outer cells were designed with stiffeners, internal bracing system and tie-rods in order to facilitate concrete pours. One face of the counterweight box was made slanted in order to maintain adequate clearances between bottom corner of the counterweight box and the top of water when the bascule span is opened to 60 degree opening angle.

3-D MODELING OF THE TRUSS

3 dimensional model of the truss was created and analyzed for different loading conditions including the wind load effect on the truss at its different opening conditions. AREMA chapter 15 was used to design the members. Lateral bracing system was designed in order to resist and distribute the loads generated by

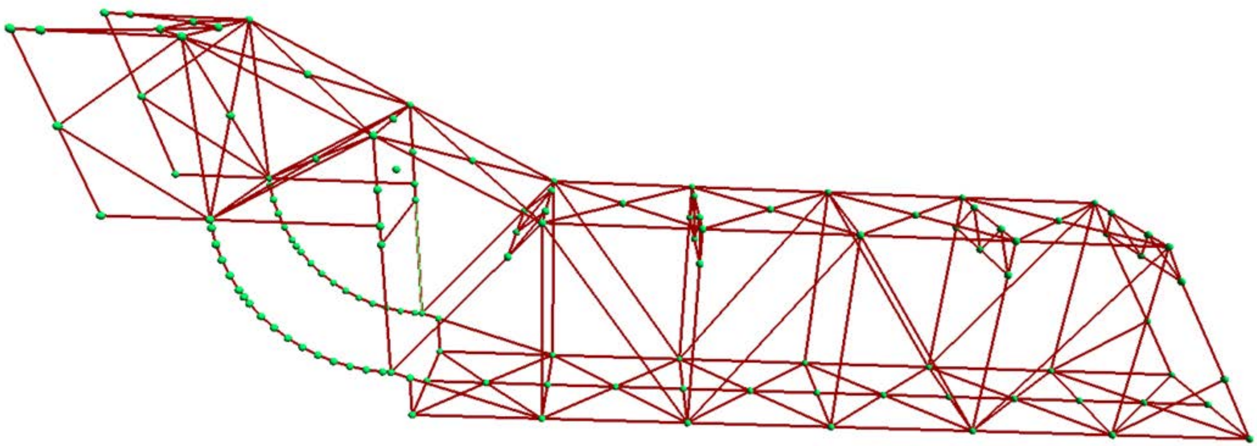

Figure 4: 3D model of the Truss

wind. Two traction frames, one at each end of the bridge were designed to distribute the longitudinal live loads. Reducing to one frame would require a heavier bottom bracing section which was not feasible without increasing the structure depth.

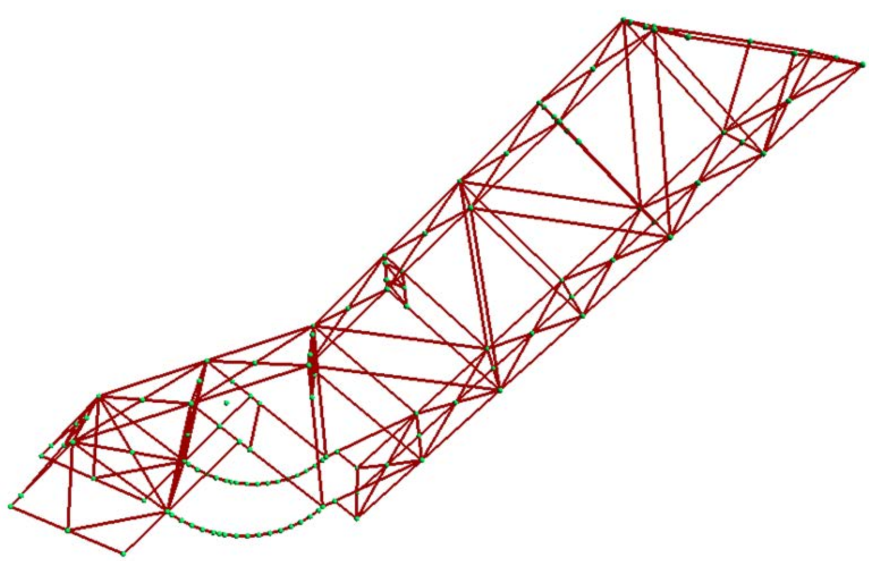

Figure 5: Truss model simulating open conditions

CLEARANCE CHECK AND BALANCE CALCULATIONS

Special attention to clearance is needed in designing movable spans in order to prevent interference of the movable span with any structural, mechanical, electrical components including walkways, rack frames, ties, and rails while span is operated. Similarly, thorough and accurate balance calculations are extremely important during design phase of the project. In order to ensure these two critical items, at final stage of the plan creation, two separate design teams were tasked to do the calculations independently and final results of both teams were compared iteratively until a deviation less than 3% was achieved. Also, HNTB's engineer's cost estimate came within 5% (on the low side) of the low-bidder that was awarded the construction contract.

BASCULE SPAN MACHINERY

The rolling lift span rolls on and is supported by two track girders. A counterweight is suspended at the aft end of the span to balance the dead weight of the structure. The machinery for operation is located in an enclosure on the movable span. Two electric motors are connected to a speed reducer. Output shafts on each side of the reducer are connected to a set of open gears that drive a pinion along a horizontal rack. As the pinion moves along the rack on each side of the span, the span opens for navigation by rolling on the track girders. A total of four (4) span lock assemblies are required: two Pier 2 Span Locks (Downlock) and two Rack Frame Span Locks (Uplock). Each span lock assembly includes a linear actuator with motor, gear box, motor brake and limit switch all within a waterproof housing that drives and withdraws a rectangular lock bar. The pier 2 span lock actuators are mounted to the pier and drive lock bars through pier mounted brackets and span mounted tongues to lock the span in the closed position. During the summer months, the span is kept in the normally opened condition. Thus, to aid the machinery in holding the span against the required wind loads, up-locks are used. The rack frame span lock actuators are mounted vertically on each rack frame and drive lock bars between guides and shoes mounted on the rack frame and bascule span. The span is normally operated using commercial electric power source and both main span drive motors. If either main drive is inoperable, the remaining motor can operate the span at reduced design loading. Additionally there is an auxiliary motor attached to the reducer in the event of failure of the normal commercial electric power source, the span is operable electrically by energizing the engine generator set provided for auxiliary drive operation.

Photo 5: New CN Fox River Bridge, in open condition

PHASE 3

Photo 6: New CN Fox River Bridge

Phase 3 included complete reconstruction of the historic bridge on the existing alignment, including environmental constraints for drilled shaft construction, waterline cap beams, two separate 10-hour change-out windows for the new approach span installations and a 36-hour change-out window for the new movable span. A 120-hour closure to navigation traffic was also set in place. On August 19, 2013, the 36-hour change-out schedule commenced in which the existing swing span was removed along with two temporary spans. To accommodate the installation of the final bascule span elements, the existing swing span pivot pier and rest piers were demolished. The bascule span elements that were installed during the final change-out included the fully assembled bascule span (complete with all mechanical and electrical components), 2 rack frames, a track girder, a 40’-0” DPG span and a 57’-2 ½” DPG span. Rail service across the completed structure was reinstated at the conclusion of the 36-hour change-out period. After another 36 hours, with the completion of counterweight concrete placement and final balance adjustments, the new bascule span opened for the first time. With additional fine-tuning of the span balance and bridge controls, the new bascule span was open to Fox River navigation traffic within the designated 120-hour navigation closure.

Photo 7: Bascule Span Cross Section, in Open Condition

Photo 8: Bascule Span

Photo 9: Segmental girder during fabrication

Photo 10: Energy absorbing Pier fender system

CONCLUSIONS

The replacement of the more than 100 year old swing span and approach trusses with a single leaf bascule span and deck plate girders was a very successful project. The achievement could not have happened without the hard work and effort from many different individuals on the project team. These would include those from Canadian National Railway, URS Construction Management team, on-site managing the day to day construction and the highly professional members of Edward Kraemer and Sons, Inc., the general contractor for the project.

ACKNOWLEDGEMENTS

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HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Repairs for Local Buckling of Sheave Support On Strauss Vertical Lift

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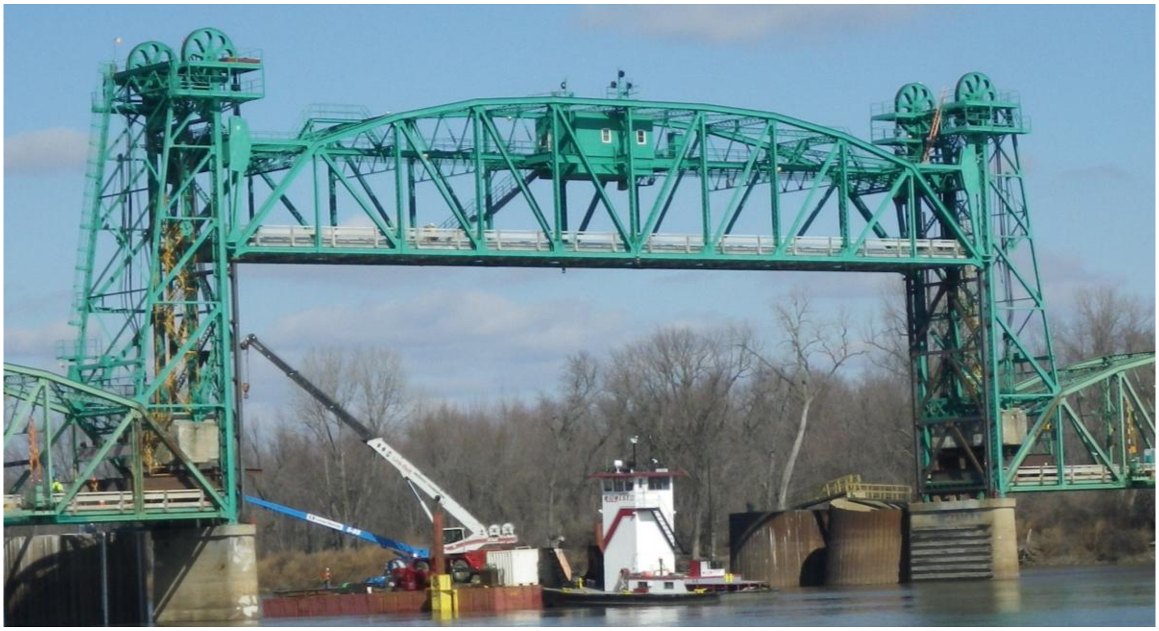
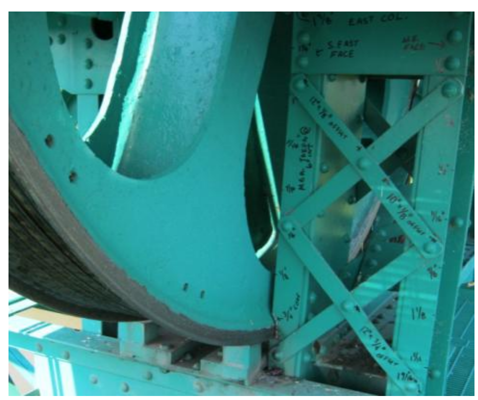
Bridge History and Existing Conditions



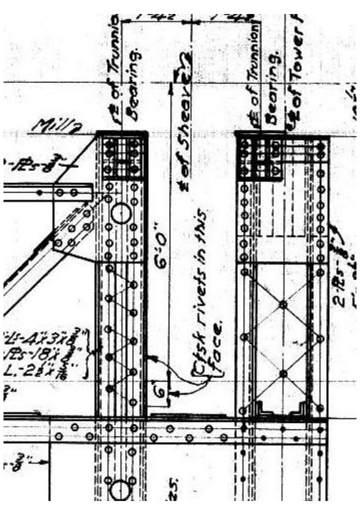
Florence Bridge Supported in the Open Position

The Florence Bridge includes a 215 foot long Strauss Vertical Lift Bridge (SVLB) originally constructed in 1926 and is owned by the Illinois Department of Transportation (IDOT). The bridge is one of two Strauss Vertical Lift Bridges built in the State of Illinois that remains in service. The bridge crosses over the Illinois River and is located in Florence, Illinois. Various repairs and rehabilitations have been performed over the years including a mechanical/electrical rehabilitation in 1982.

In June of 2012, a routine inspection was performed that revealed local buckling at one of the trunnion bearing (TB) support columns and movement of the sheave axially along the trunnion shaft. After this discovery, the bridge was closed to vehicular and barge traffic until an emergency in-depth evaluation was performed to confirm the bridge should remain closed to avoid further damage. In July of 2012, the emergency in-depth evaluation confirmed that the sheave had “walked” axially along the trunnion shaft, pushing the support column outward thus contributing to the buckling. The decision was made to leave the bridge in the fully raised position to allow barge traffic to continue uninterrupted until a repair could be performed.



Sheave Cutting into Damaged TB Column

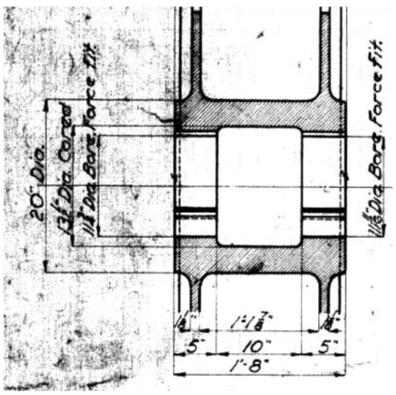


Original TB Support Column Design Detail

The repair design for this project incorporated the repurposing of four spare sheaves that were in storage from another decommissioned Strauss Vertical Lift Bridge in Illinois. A replacement tower column was designed to replace the buckled trunnion bearing support column. The decision was also made to replace the existing wire ropes and trunnion bearings during the repair. All of the repair work was performed with the lift span in the fully raised position and supported on temporary support columns. The repair work was completed in April of 2013 and the bridge was re-opened to vehicular traffic.

Field Investigation

The preliminary investigation confirmed that damage to the bridge extended beyond the buckled tower column. The buckled tower column provided direct support for one of the sheave bearings and the sheave in that corner. The sheave supported by the buckled column had shifted axially along the trunnion shaft and was cutting into the tower column steel. With this condition in mind, it was decided that continued operation of the bridge could result in further buckling of the column and ultimately failure of the entire column and movable span. Additional research for possible causes of the column buckling revealed that a major contributing factor was a mechanical design issue that is no longer used in the industry. The original trunnion shaft design was only fit into the sheave bore near the ends with a recess in the middle of the sheave bore. This design allows for deflection of the trunnion within the sheave bore, which in this case likely resulted in the sheave walking axially along the trunnion as it cycled between tension and compression during rotation.

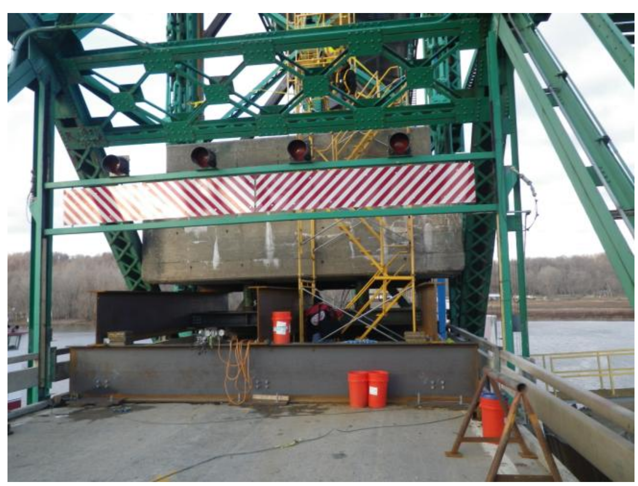


Original Sheave Hub Bore Design Detail

Structural Analysis and Design

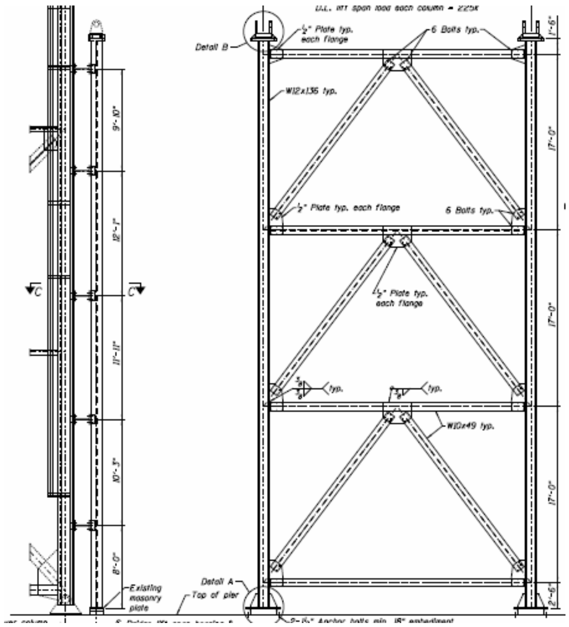
The findings of the field investigation showed that the buckling damage to the tower column was beyond a tolerable amount to attempt to repair by heat straightening methods, and therefore the only option was to replace the damaged portion. The section properties of the existing tower column were analyzed, and it was determined that as-built, there was adequate capacity in the tower columns, meaning that the tower column repairs could be of the "replace in-kind" type. The complexity of these repairs was greatly reduced by the fact that the work would be done with the sheaves removed from the towers, and therefore, the tower columns would be unloaded.

In order to perform the sheave and rope replacement, the lift span and the counterweights needed to be temporarily supported. In the case of the Florence Bridge, this presented two challenges: temporary support of the lift span and temporary support of the counterweight during the replacement operations.



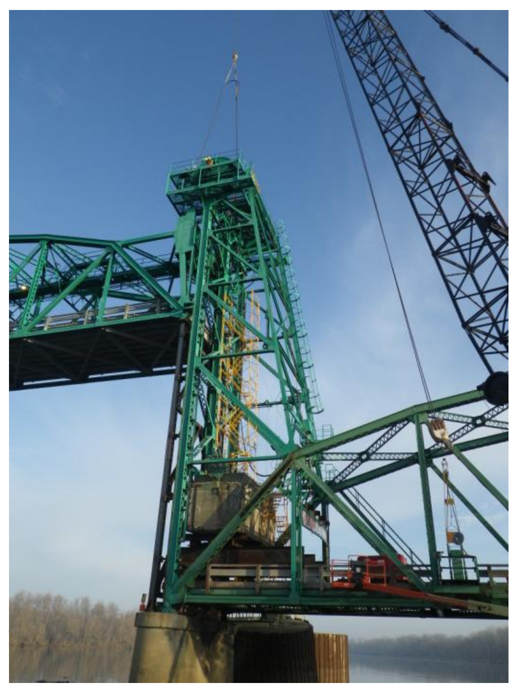
Counterweight Supported on Temporary Support Structure

Modern vertical lift bridges are designed with a provision to support the counterweight from the towers to help facilitate future rope replacements. The Florence Bridge was designed during an era prior to this provision becoming standard practice in movable bridge design. A brief analysis showed that there was not a feasible way to support the counterweights from the towers – the overall majority of the tower framing was far too light. Therefore, a method needed to be devised to support the 450,000 lb counterweights from the flanking truss spans, which were originally designed to support 40,000 lb trucks. After analyzing the flanking trusses, the floorsystem was deemed inadequate to support the counterweight, but the main truss members were found to have adequate capacity. A grillage system was designed that distributed the counterweight load directly to the main panel points of the truss, bypassing the floorsystem. In order to not impede marine traffic during the extended operational closure of the Florence Bridge, IDOT decided to keep the bridge in the open position. This decision required some means of supporting the lift span while the sheaves and ropes were removed for replacement. Analysis of the existing structure revealed the most feasible option was to erect temporary support towers to support the lift span while the sheaves and ropes were replaced.



Lift Span Support Column Design

The temporary towers supported the lift span at the existing bearings and transmitted the lift span weight down to the piers at the existing masonry plates. As there was no provision for temporary counterweight support, for rope replacement as an example, it was deemed prudent to verify that the existing bearings did have the capacity to support the full dead load of the span, since, during normal service, the bearings normally only carry a small amount of dead load (the span imbalance) and live load, with most of the dead load being counterbalanced by the counterweights. After finding that the bearings were adequate, the temporary towers were then designed to support the full dead load of the lift span along with wind load. With the small longitudinal footprint available, the temporary towers had a high slenderness ratio, requiring bracing to the existing tower in order to increase their critical buckling load. This was done via a series of bolted pieces of wide-flange beams. In the transverse direction, several tiers of K-braces effectively braced the tower legs and also served to carry transverse wind forces down to the substructure.


Lift Span Supported on Temporary Supports

Mechanical Analysis and Design

Repurposing Spare Sheaves and Trunnions

Spare Sheaves Located in IDOT Storage Lot

Several years prior to the incident occurring at the Florence Lift Bridge, IDOT had decommissioned and disassembled another Strauss Vertical Lift Bridge on the Illinois River. At the time of decommissioning and disassembly the decision was made by IDOT to store the main counterweight sheave assemblies and several other mechanical components in a storage lot instead of sending them to a scrap yard to be recycled. The spare sheave assemblies were part of a rehabilitation project in 1998 for the Shippingsport Lift Bridge and had plenty of life left in them. The critical dimensions of the stored sheave assemblies were checked to verify the feasibility of repurposing them for use on the Florence Lift Bridge. Once it was established that the Shippingsport sheave assemblies were dimensionally compatible with the current sheaves, a closer inspection was necessary to evaluate their condition after years of storage.

Wire Rope Grooves on Spare Sheaves

An on-site inspection of the Shippingsport sheaves was conducted to evaluate the overall condition of the sheaves. One of the major concerns after years of storage outside was the condition of the rope grooves and trunnion shaft journals. The inspection of the sheaves revealed that the rope grooves exhibited minor surface corrosion and light pitting with minimal groove wear. The trunnion shaft journals exhibited similar minor surface corrosion, which was easily cleaned up with a Scotch-Brite™ pad. This verification in the field ensured that the shaft journals could be machined with minimal material removal to mate with the trunnion bearings.

Trunnion Condition on Spare Sheaves

Based on the design drawings and verified by the on-site inspection, the Shippingsport sheave trunnions were 14 inch journal diameters, whereas the Florence Bridge was 10 inch journal diameters. On a vertical lift bridge with plain journal bearings supporting the trunnion shafts, the sheave bearings are a main source of friction in the drive system. Therefore, increasing the diameter from 10 inches to 14 inches would increase the friction in the system. The effect of this increased friction was evaluated to determine the increased motor requirements, and it was calculated to be less than 10 percent. This increase was not significant enough to require turning down the 14 inch shaft to a small diameter in order to reduce the friction. The trunnion shaft journals were to be cleaned up and polished to remove the layer of existing corrosion.

The existing trunnion bearings could not be reused due to the increased bore size required for the new trunnion shafts. New trunnion bearing housings complete with new bushings were designed to accommodate this increased trunnion shaft size. The new bearings had to maintain the same centerline height as the existing bearings and an identical mounting footprint to be a drop-in replacement. Outboard thrust plates bolted to each end of the trunnion shaft were also included in the new bearing design in order to share any axial thrust between both bearing housings and therefore sharing load between the TB support columns.

Machinery Fabrication and Rehabilitation

The stored sheave assemblies were sent to the machinery fabricator for cleaning and polishing of the rope grooves, trunnion journal surfaces and trunnion bearing thrust faces. Cleaning and polishing of rope grooves revealed surface pitting and gouges less than 1/64" deep that could not be removed without more aggressive machining methods that would result in material removal from the rope groove surface. Additional material removal from the grooves would also change the pitch diameter of the sheave, which could have resulted in operational issues of the lift span. The total indicator reading (TIR) of several rope grooves on each sheave was measured after the grooves were cleaned before the decision was made that additional groove machining was not required. Existing sheave measurements were taken with each sheave positioned on a vertical turret lathe (VTL). The TIR from groove to groove was within the specified tolerance of 0.010"and the surface finish was acceptable and would not cause any damage to the new wire rope.

Sheave Positioned on VTL
for Measurements

The trunnion journals and thrust faces were machined with the sheave assembly positioned on the VTL. All of the machining on the trunnion journals and thrust faces was performed with the trunnion shaft assembled in the sheave. This provided a unique challenge to achieve the appropriate surface finishes on the journal surface due to the reduced turning speed on the VTL. Because of this the surface finish requirement was relaxed from an 8 microinch finish to an 11 microinch finish in order to keep the project on schedule. Typically trunnion shafts are machined on a horizontal lathe which can turn at a higher speed and are not assembled in the sheave at the time the journals are machined. The shaft journals were turned down to a 13.75" diameter with the appropriate tolerance to achieve an RC6 fit with the new bearing bushings and were required to be concentric with the rope grooves within 0.005" TIR.

Construction

A laser survey was conducted to establish precise locations of the existing trunnion bearings and trunnion shafts. The position of each trunnion shaft journal center was accurately located with the dead load still on the ropes. Measurements were then taken again after the counterweight was jacked and

Precision Laser Survey of Existing
Trunnion Bearings

Turnbuckles and Equalizers at Counterweight Side Wire Rope Connection

the dead load was relieved from the ropes. These measurements were critical to establish a baseline location for the installation of the new trunnion bearings and the new tower column to optimize mechanical functionality

With the precision survey data recorded and the temporary span support towers installed, the counterweights were raised approximately 20" using hydraulic jacks to relieve the tension in the ropes. This lift more than compensated for the elastic relaxation of the counterweight ropes, and placed the counterweights at an elevation that was sure to allow the connection of the new ropes. Similar to other bridges of this vintage, the rope attachment to the counterweights uses a series of equalizer plate assemblies to distribute the tension equally to all of the ropes. These equalizers had practically seized to their pivot pins over the years and were taken to a local machine shop for disassembly, cleaning, and reassembly. In addition to the equalizers, turnbuckles were installed at the counterweight connection during the last counterweight rope replacement. Using a "belt and suspenders" approach, the Illinois DOT opted to keep the existing turnbuckles in use with the new ropes. That way if the equalizer plates were to seize again in the future, it may still be possible to adjust the rope tensions using the turnbuckles.

Another critical part of the installation was the fit between the new TB support column and the existing tower column. The splice for the new column portion was made within a large shear plate that connects the inboard and outboard columns. One of the shear plates was moved aside and the existing column was cut away. The new column was machined flat on the lower end during the shop fabrication, so to create a mill-to-bear fit, the existing tower was manually dressed with hand held grinders until the fit was acceptable. This involved repeatedly lowering the column into its correct plumb position, checking the fit by shining a flashlight through the joint and using feeler gauges to measure the gap, then raising the column to grind down high spots. Once complete, the two column portions were in contact for 50% of the bearing area (checked using a 0.003" feeler), and the rest of the bearing area had gaps no larger than 0.010".

Installation of New TB Support Column
(Inset Photo Shows Hand Dressed of Existing Column Mating With New Column)

When the existing trunnion bearings were removed, it was discovered that the original installation used stepped shims to level the trunnion bearings. The bearing surfaces at the tops of the column were not level, and examination of the precision survey data revealed that the surfaces would not have been level when the towers were loaded with the span dead load. The largest slope found was 1/8" across the 12" bearing width. To ensure the new sheaves were installed level, new stepped shims were made for each bearing location. Once the new stepped shims were installed, the column tops in a common corner were at the same elevation and level with 0.010" per foot. This was verified by using a precision level and a straight edge across both column tops.

Using traditional optical survey methods, the new trunnion bearings and repurposed sheaves on opposite towers were able to be located on the longitudinal centerline of the bridge within 1/16". Using the precision survey data, the trunnions for the sheaves on a common tower were aligned to a common axis within 1/16". With the full weight of the sheaves in the trunnion bearings, the bearings were still able to be moved fairly easily with pry bars and small jacks to obtain good alignment.

Installation of New Sheave Assembly

The trunnion bearings were bolted in place with undersized bolts, new counterweight ropes attached and the counterweight was lowered onto the ropes. With the full weight of the counterweight on the ropes, the sheave trunnions were rechecked for alignment. Measurements revealed the weight of the counterweight had caused greater than anticipated deflection in the tower, causing the trunnions to no longer be level within 0.010" per foot. Based on these measurements a new shim plan was created to compensate for the deflection. The weight of the counterweight was removed from the ropes and shimming was conducted on one bearing at a time, leaving the sheave's second bearing firmly bolted in place to maintain the sheave's position. Measurements also revealed the sheave bearing supports slid in closer to the sheave nearly 1/16", almost entirely eliminating the desired 1/16" gap left between the sheave and the bearing thrust surface. The gap between the sheave thrust face and the bearings was increased to 1/8" before being reloaded to compensate for the action. The counterweight was lowered onto the ropes and measurements showed the adjustments had the intended effect, and the alignment was acceptable. The undersized bolts were removed one at a time and the bearing supports were reamed in-place using the mounting holes in the bearings to align the reamer. To speed up reaming the bearing support holes an adjustable hand reamer was modified to be used with a magnetic drill press. While the holes in the supports were being reamed the counterweight jacking supports on the approaches were disassembled and removed and the temporary span supports were removed.

After the bearings were bolted in place the span was operated for the first time. IDOT and the Contractor positioned personnel at every critical location prior to this operation. After the span broke the friction caused by sitting dormant for several months the bridge operator reported that the span performed as well as it did before the buckled tower. Drive machinery strain gauge measurements and counterweight rope tension measurements confirmed the span was operating properly.

Conclusion

The accelerated timeline for the repair of the damaged Florence Bridge would not have been possible without the decision that was made to save four sheaves from a decommissioned bridge several years prior. A project of this nature typically will take two years to complete, which would include the time to fabricate all components including four new sheaves. The availability of the spare sheaves cut this time down to ten months from the time the bridge was shut down until the time the bridge was reopened to vehicular traffic.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Replacement of the Sir Ambrose Shea Lift Bridge and Ensuing Rehabilitation of the Existing Lift Bridge in Placentia, Newfoundland and Labrador – Canada

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Abstract

The Sir Ambrose Shea Vertical Lift Bridge is located in the town of Placentia, in the Province of Newfoundland and Labrador in Canada. This bridge is the only moveable bridge in the Province. Delcan Corporation was retained by the Department of Transportation and Works NL to perform a scoping study of the existing bridge with the objective of prioritizing rehabilitation, replacement and additions to the bridge as necessary to maintain safe and reliable bridge operation for at least the next 25 years.

The scoping study identified that replacement of the bridge was necessary to obtain the minimum required 25 year life. However, the scoping study also identified that immediate repairs were required to maintain the bridge in a safe operating condition for an additional service life of 5 years until a replacement bridge could be completed.

The harsh environment in which the structure is located led to deterioration that is not typically encountered even given the proximity of many movable bridges to harsh sea environments. The primary findings of the study, which required priority corrective action, resulted in the identification of deteriorated structural components on the lift span, deteriorated machinery components, with the most notable being the counterweight ropes and termination connections, and an aging electrical system. This paper will present the evaluation of the deteriorated structural components which led to a reduction in the load rating for the span and a subsequent interim rehabilitation to restore the load rating. The paper will present the extent of the deteriorated counterweight ropes and termination connections that led to development of interim rehabilitation plans to replace the counterweight ropes and the counterweight rope termination connection to the counterweight.

Introduction

Overview of Project

The Sir Ambrose Shea Lift Bridge is located in Placentia, in the province of Newfoundland and Labrador in Canada. The Department of Transportation and Works retained Delcan in September 2011 to complete an inspection of the structural, mechanical, and electrical components of the structure. The mechanical and electrical inspections were undertaken for Delcan by Stafford Bandlow Engineering, Inc.

The detailed inspection was carried out in October 2011 and based on the findings a follow-up inspection was performed in April 2012. The inspections consisted of a structural, mechanical, and electrical inspection. The purpose of the inspection was to assess the condition of the structural, mechanical, and electrical systems for immediate, short term, and long term improvements to maintain the bridge in good state and provide safe and reliable operation for the next 25 years. Following the inspection, a structural evaluation was carried out for the bridge taking into account deteriorated components. The structural evaluation resulted in the load posting of the bridge. This was followed by the structural rehabilitation of the existing bridge to achieve an improved single load posting of 25 tonne load restriction and keep the bridge safely and reliably in service over approximately four years, until which time a new bridge will be constructed and in service. The structural rehabilitation work was completed in 2012 and was followed with electrical and mechanical rehabilitation, which included the replacement of the counterweight ropes and associated counterweight anchorages.

Bridge Description

Built in 1961, the existing tower span vertical lift bridge consists of three simply supported spans: Two 29.72m approach spans and a 30.18m lift span in between. Two lift towers are supported on the approach span's main girders at the piers. Operation of the lift span is controlled by a mechanical drive system at the top of the towers. The bridge carries one lane of traffic in each direction with a sidewalk on each side. The superstructure consists of two main built-up steel plate I-girders, with transverse frames in between, supporting the deck. The transverse frames are spaced at 1200± mm at the lift span and at 2400± mm at the approach spans. The transverse frames consist of: a wide flange (WF) main floor beam supporting a 125mm thick grating at the lift span and the 165mm concrete deck at the approach spans; and the remaining members consist of double angles (back-to-back) connected together using rivets. The bracing members are connected to the main floor beam and the girders using gusset plates and riveted connections.

Figure 1. Overview of the bridge.

The sidewalks and railing system are supported on WF cantilevers, spaced at 2400± mm, overhanging the main girders. The railing posts are welded to the end of the cantilever and the cantilevers are bolted to the main girder. The sidewalk on the approach spans is comprised of a 114mm reinforced concrete section supported on the floor beams at the deck side and sitting on the cantilever end at the outside. The sidewalk on the lift span is comprised of a 3/8" checker plate stiffened by a 3/8" x 2" deep plate at 1200± mm spacing. This plate is bent to form the curb on the deck side and is supported by a channel, spanning between the railing posts, at the outside.

A typical section showing the components of the lift span deck is presented in Figure 2. The components of the approach span deck are similar except for the concrete sidewalk and deck.

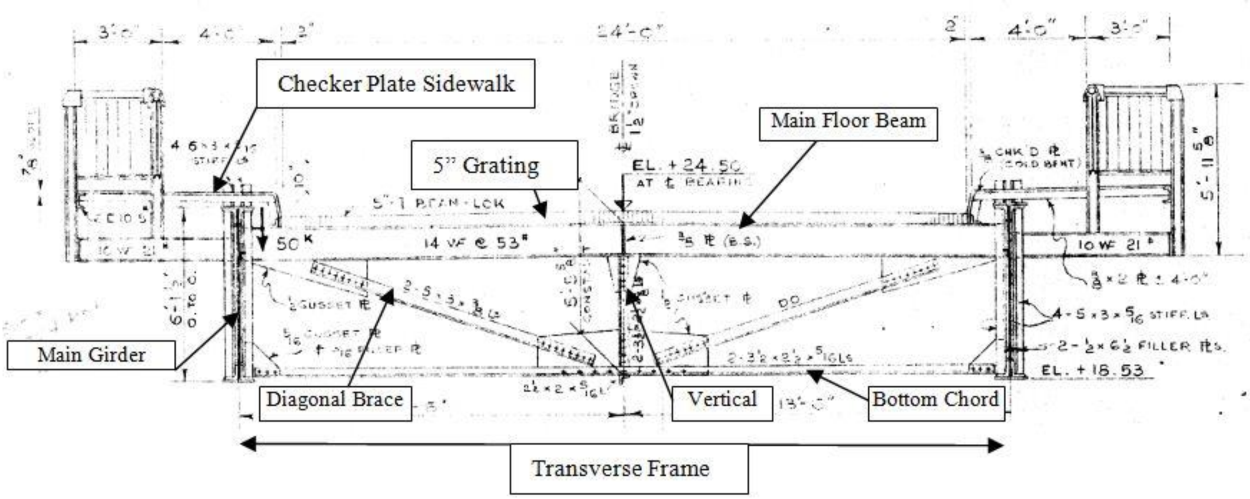

Figure 2. Typical cross-section at lift span.

The above deck superstructure consists of two lift towers, comprised of a three dimensional space truss, and connected at the top by a three dimension space truss housing the machine room, mechanical, and electrical equipment. The main tower elements are two built-up columns supported on the main girders of the approach span, which are in turn supported by the piers. The remaining tower members consist of double angles except for the main transverse member at the first level which is a WF section. The three dimensional space truss is comprised of: double angles for the two main outside trusses and most of the diagonal bracing; WF sections for the bottom members supporting the machine room; and double channel for the bottom transverse members outside the machine room.

The substructure consists of two abutments and two piers. The reinforced concrete abutments are supported on timber piles. The piers are comprised of mass concrete inside steel sheet piling and reinforced concrete above the sheet piling.

Mechanical Systems Description

The bridge is a tower span vertical lift bridge. Operation of the lift span is controlled by a mechanical drive system mounted on the span truss which connects the top of the two towers. The span drive motors, manual emergency drive, primary reducer, motor brakes, and control equipment are located in an enclosed room mid-span between the towers. Line shafting extends to a machinery brake, and secondary reducer at each tower. The secondary reducers have double extended output shafts each driving a main pinion which engages a ring gear mounted on the main counterweight rope sheaves. Each main counterweight rope sheave is mounted on a sheave trunnion. Each main counterweight sheave trunnion is straddle mounted in two sheave trunnion bearings. See Figure 3.

All of the span drive machinery bearings and main counterweight sheave trunnion bearings are pillow block spherical roller bearings.

There are four main counterweight sheaves; the sheaves are mounted at the top of the towers and one sheave serves each corner of the lift span. Each main counterweight sheave supports seven 1" diameter main counterweight ropes, for a total of twenty eight ropes. One end of each counterweight rope is attached to the lift span while the other end terminates at the counterweight. The span drive machinery is provided to rotate the counterweight sheaves thereby enabling the span to be raised and the counterweight to be lowered as the wire ropes pass over the sheaves. The weight of the span and the counterweights provides sufficient friction between the ropes and the sheaves to prevent the ropes from slipping during operation.

The lift span is equipped with a system of roller guides to constrain the longitudinal and transverse movement of the span during operation. Each corner of the span is equipped with a pair of transverse guides. At the fixed (south) end, there are also longitudinal guides.

The span is provided with four live load supports to transmit the imbalance load and the loads due to vehicular traffic from the movable span to the piers. The live load supports also function as centering devices to ensure that the lift span is aligned relative to the pier.

The bridge has traffic signals and traffic gates to halt vehicular traffic prior to span operation. Each approach to the lift span is equipped with one set of traffic gates. When lowered, the traffic gates cross the sidewalk and the oncoming/offgoing lanes of traffic.

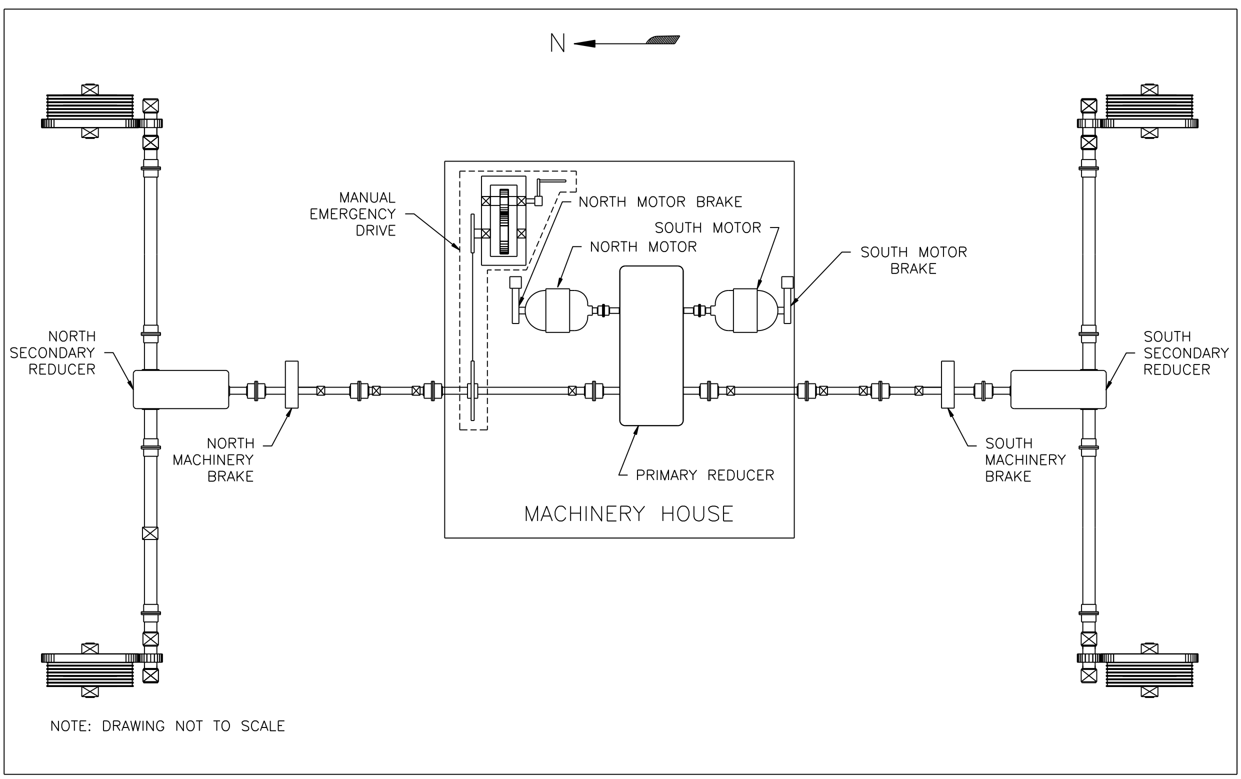

Figure 3. Plan View of Span Drive Machinery

Bridge Inspection

The objective of the inspection was to determine the status of the bridge's structural, mechanical, and electrical components and its conformance with codes and practices to operate safely and reliably. Additionally, another objective was to identify, prioritize, and scope rehabilitation, replacement, and additions to the bridge on an immediate, short term, and long term basis to maintain the bridge operating safely and reliably for at least the next 25 years, should replacement of the entire structure not be undertaken.

Structural Findings

The visual inspection indicated that the structure was in generally poor condition and has reached the end of its life expectancy. The structural steel below the deck was severely corroded with significant loss of section at the main girders and the transverse frame members. See Figures 4 and 5. The cantilevers supporting the bridge railing and sidewalk were severely deteriorated with significant section loss at several locations. The concrete deck has reached the end of its design life, as exhibited by delaminations, spalling with exposed corroded reinforcement, and cracks. The lift span sidewalk was in generally fair to poor condition and cannot accommodate the current Canadian Highway Bridge Design Code CAN/CSA S6-06 (CHBDC) pedestrian loading. The structural steel in the towers and machine room was in generally fair condition with localized areas in poor condition. A load evaluation of the superstructure was recommended to assess the residual capacity of the structure and the effect of the damaged towers. The substructure was in poor condition, with severe erosion and loss of concrete section at the piers and abutments. The bearings at the abutments and piers were in generally poor condition, exhibiting severe corrosion. The rocker bearings at the north abutment were seized and tilted towards the abutment.

Figure 4. Severe corrosion and loss of section at top flange of lift span main girder.

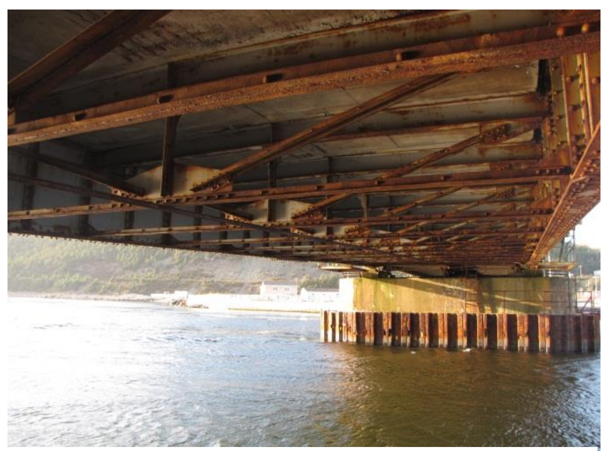

Figure 5. Deterioration at transverse frame members.

The counterweights were in generally poor condition with concrete spalling and exposed corroded rebar and wide cracks noted.

The bridge railing did not meet the crash test requirements of the CHBDC.

Based on the observed condition of the structure, the following structural work was recommended to be undertaken in the short term: miscellaneous steel repairs, miscellaneous concrete repairs, expansion joints and bearing repairs. At the time of inspection, the integrity of the superstructure could not be determined, as such, a structural evaluation of the superstructure was recommended to determine the extent of strengthening of steel members or the load posting required. The results of the structural evaluation will be presented in a subsequent section.

Mechanical Findings

Counterweight Wire Ropes

The counterweight rope design utilizes rope clamps at the span and counterweight terminations to position the ropes. Each clamp positions a group of seven ropes in a straight line with extremely tight spacing. Clamps are necessary due to the configuration of the terminations. The clamp arrangement has two adverse effects on the wire ropes.

1. The tight spacing has resulted in slapping wear from the ropes oscillating when subjected to frequent high winds.
2. Wide splay from the clamp to the terminations has resulted in contact wear at the clamp/rope interface.

The condition of the wire rope has been a concern over the life of the bridge due to the adverse effects of the wire rope clamps, with the following noted events:

1987 – Single rope failure, believed to have originated at a wire rope clamp. All ropes were replaced following the failure.
1994 – Single rope jumped the sheave. The single damaged rope was replaced.
Early 2000’s – All ropes were replaced.

At the time of the 2011 inspection, the wire ropes were in service approximately 10 years. Despite the relatively short period of service, severe abrasive wear was found on the main counterweight ropes with numerous ropes where the outer wire was completely worn through. The wear was found at approximately the middle of the unsupported length of the rope between the span side sheave tangent point and the span termination. Severe wear was present for up to 15 feet long sections of the rope, centered on the middle of the unsupported length. See Figure 6. The primary wear mode is resultant from the ropes slapping against each other during oscillation under wind loading. These sections of rope have the largest amplitude during oscillation. Contact abrades the crowns of the contacting wires; therefore for a group of seven ropes, each of the five central ropes has two areas of wear, one on either side where it contacts the adjacent rope. The wear on each side was found to be similar. The two sided wear evident on the ropes was

Figure 6. General View. Severe wear was found at approximately the middle of the unsupported length of counterweight wire rope.

the dominant wear present and governed the evaluation of the wire ropes. Wire rope wear due to contact with the sheave was not found to be significant, which is consistent with the ten year service life.

The worst wear was found in the North tower, where the rope group wear progressed to the point that the outer strand had worn through and the secondary (underlying) layer of wire was starting to wear flat. See Figure 7. Even though the secondary layer was present, the wear was evidence of how severe the wear was and that the wear was continuing through the secondary layer, further decreasing the strength of the rope. For evaluation purposes, each outer wire strand that was worn through was considered a broken wire. The maximum quantity of broken wires found at a single rope was 34 in a single lay, which grossly exceeds any wire rope replacement criteria, justifying replacement on priority basis.

Figure 7. Counterweight Wire Rope. Close-up. The outer strand of the rope is completely worn through at multiple locations.

Wire rope damage was also found at the wire rope group clamps near the span terminations. Wire rope clamps are installed to group the wire ropes near each termination to align the rope group to the sheave grooves. The span termination is located at the roadway level and has a circular connection pattern that results in different splay angles into the rope group clamp. See Figure 8. Damaged wires were found underneath the clamps at several ropes. See Figure 9. The damage occurred at the contact point of the wire rope and the bottom

Figure 8. General View. Counterweight Rope – Span Termination. Rope group clamp installed.

Figure 9. Northwest Rope Group. Wire damage from contact with the rope clamp affected wires in three specific strands.

edge of the clamp. The damage varied in severity from a displaced wire to broken wires in three specific strands at one location. This localized damage appears similar to what was described as the precipitating cause of the 1987 rope failure.

Clamps installed at the counterweight terminations were not removed for inspection due to reinstallation concerns. The counterweight wire rope group rotates 90° from the counterweight sheave grooves (transverse orientation) to mate with the counterweight termination (longitudinal orientation). See Figure 10. Due to the orientation of the wire rope group, the ropes are subjected to some sliding contact between adjacent ropes during operation. No significant wear was found at the contact areas. The rope group splay from the counterweight clamps is similar to the splay at the span termination. Due to the similar splay, the counterweight ropes contained within the counterweight group clamps are subjected to similar contact with the clamp edges that could result in wire damage.

Figure 10. Clamp installed at the counterweight rope.

Based on the observed wear on the unsupported length of rope, as well as the rope damage at the splay clamps, the wire ropes were considered worn beyond any acceptable criteria and replacement on a priority basis was warranted.

Counterweight Wire Ropes – Counterweight Terminations

The connection of the counterweight ropes to the counterweight was found to have been modified from the original design. The available plans depict the rope sockets connecting directly to the counterweight termination. The existing configuration utilizes turnbuckles and a connector plate to span between the socket and termination. While this common retrofit has been performed on a vertical lift bridge to provide adjustment, there is one aspect of the retrofit that is notable and potentially problematic. A single connector plate is provided for each rope and each connector plate is secured to the original termination connection with one ¾" bolt in single shear. See Figure 11. A visual assessment of this connection is that the bolted connection is undersized relative to the pinned rope connections. In addition to the relative capacity, the integrity of the connection was questionable due to degradation. The connection plate bolts were found buried in

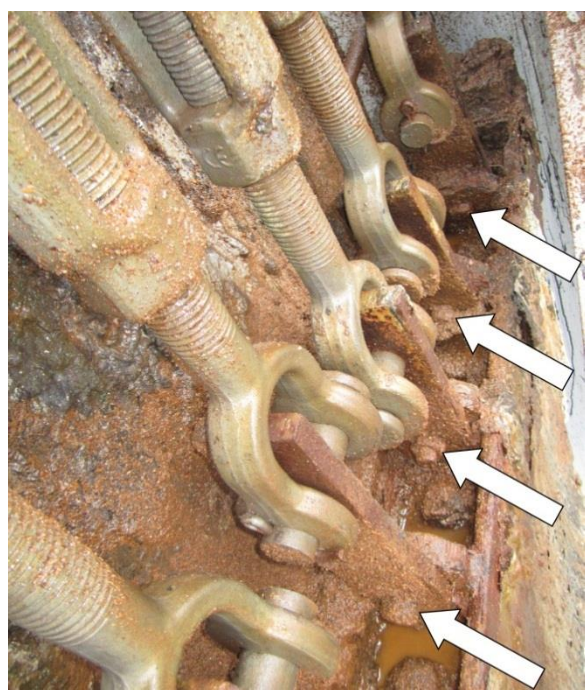

Figure 11. Counterweight Rope – Counterweight Termination. Each connection relies on a single bolted connection (arrows) to an extension plate.

debris and covered in standing water. See Figure 12.

Figure 12. Close-up of counterweight termination. Debris circled.

Upon evaluating the original strength of the bolted connection it was found that there was little excess capacity. With little excess capacity to begin with, the capacity is reduced by corrosion, section loss and a secondary offset loading induced by the prying action of impacted corrosion between the extension plate and original termination connection plate. The integrity of the connection was questionable and was deemed inadequate when considering any future service life. The connection was redesigned as part of the ensuing rehabilitation work. New turnbuckles were kept in line to facilitate adjustment. The counterweight termination was replaced and the connection of the turnbuckle to the counterweight termination utilizes the manufacturer's turnbuckle pin connected directly to the termination, avoiding any single shear connections. A drain hole was also designed into the termination to mitigate deterioration of the connection due to standing water.

Counterweight Wire Ropes – Lift Span Terminations

Severe deterioration was found at the counterweight rope termination connections to the lift span. Limited visual inspection of the connections revealed near complete loss of fastener heads as well as significant reduction in flange thicknesses due to extensive corrosion. See Figure 13. The deteriorated connection is an example of the how harsh the environment can be on structural components. Due to their atypical location, of just under the roadway surface, the rope terminations at the lift span are subjected to snow and ice melting solutions (e.g. rock salt, brine solution) for the duration of the winter months. These connections were evaluated and rehabilitated during the ensuing structural repairs which enabled additional access to determine the extent of the deterioration.

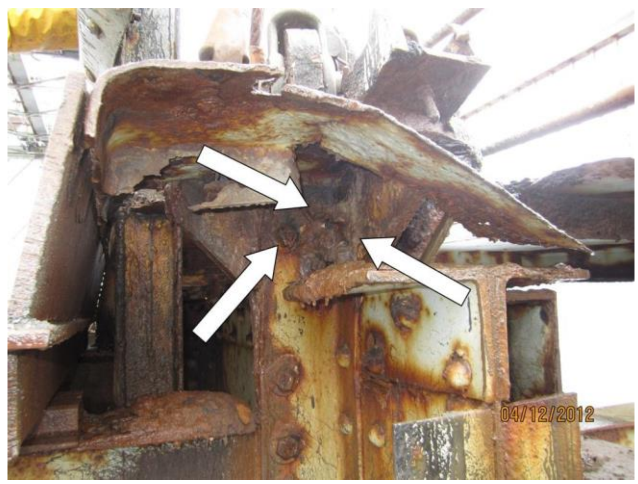

Figure 13. Counterweight Rope – Lift Span Termination Bolted Connection. Arrows indicate fasteners with significant section loss.

Counterweight Guides

The counterweight guides were found to have been heavily worn and the fasteners sheared. Without the counterweight guides, the counterweight could move longitudinally under wind loading. There was evidence that the counterweight impacted the structural elements of the tower resulting in damage to the counterweight and structure. The counterweight guides were originally part of the counterweight rope counterweight terminations. With new terminations designed, new counterweight guides were again incorporated into the termination to limit movement of the counterweight and prevent contact between the counterweight and surrounding structural components.

Span Drive Machinery

In general, the span drive machinery components were found to be serviceable for continued use for the remaining life of the bridge, with the exception of the machinery brakes. All machinery components located outside of the central machinery room were protected with metal covers. While the covers were well intentioned to protect the components, there were multiple instances where the covers trapped debris and moisture and appear to have accelerated corrosion resulting in significant section loss. In addition, the covers hindered access to the underlying machinery for routine lubrication/maintenance and the ability to visually assess degradation due to the harsh environment.

As part of the study, all covers were removed to assess component integrity. Components that had been covered and were severely corroded were the secondary reducer housings, supports, coupling bolts and bearing mounting bolts. One of the two secondary reducer supports exhibited complete section loss, resulting in a significant reduction in the support strength. During the April 2014 inspection, maintenance performed a weld repair to secure the reducer that will provide reliable continued use. The remaining reducer housing and mounting bolts, while severely corroded, exhibited enough remaining section to provide reliable operation for the remaining life of the bridge.

At the machinery brake, the cover was deteriorated and nest material had collected inside. The machinery brake was frozen in the released position and not able to be serviced. The combination of exposure to the elements and the cover hindering access resulted in degradation of the assembly. Without the machinery brakes, the braking torque provided by the two motor brakes was physically verified and determined to be adequate based on the ability of the brakes to hold a test load of approximately 80 kN (18 kips). Maintenance personnel periodically placed the test load on the lift span to simulate ice loading. In addition to the test load, the resulting total brake torque coincided with the total brake torque provided in the replacement bridge design.

The lesson learned is that environmental exposure may drive cover selection, but ease of maintenance and inspection should not be overlooked. Both requirements should be considered when evaluating appropriate covers for machinery.

Mechanical Rehabilitation

Based on the scoping study the following items were chosen to ensure safe and reliable bridge operation for the remaining life of the bridge:

- Replacement of the counterweight ropes.
- Replacement of the counterweight rope terminations and terminations counterweight connection.
- Provision of new counterweight guides.

Electrical Findings

The inspection determined that the bridge electrical power and control systems are operational but in need of upgrading to meet present day operational and safety standards.

A number of electrical deficiencies and non-conformances with current national, provincial and local codes were noted during the inspection. In addition, it was noted that the electrical systems were those originally installed when the bridge was constructed between 1958 and 1961. These systems are obsolete, have all well exceeded their design lives and due to their age, now inaccurately function to protect the electrical system under fault conditions.

The assessment concluded that although the bridge electrical system is aging, only limited immediate action needs to be taken to maintain a safe operational system. These actions were taken up by maintenance efforts. For continued safe and reliable operation over the next 5 years additional replacements and modifications will be necessary. To maintain the bridge in a safe and reliable state for the next 25 years, a complete electrical rehabilitation of the bridge electrical system will be necessary. Limited modifications to the electrical systems were included in an ensuing rehabilitation in order to provide continued service until the replacement bridge is complete.

Structural Evaluation

Evaluation Methodology

The live load evaluation of the structure was performed in accordance with Section 14 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06 and its supplements, which specifies methods for evaluating existing bridges. The bridge was evaluated at Ultimate Limit States (ULS) only, as no significant Serviceability Limit State (SLS) issues have been identified, nor were any observed during inspection. The live load distribution was performed using a "Sophisticated Method", with the software package MIDAS by the MIDAS Information Technology Company Limited. A three-dimensional model of the structure was created and all applicable loads were applied.

As the original structural drawings do not specify the structural steel grade used in construction, the yield strength and tensile strength of the main structural steel members were taken as 230 MPa and 420 MPa, respectively, per Section 14 of the CHBDC. The rivet tensile strength was taken to be 360 MPa, per Section 14 of the CHBDC. The specified tensile strength of bolts was taken to be 830 MPa. As the original drawings do not specify the compressive strength of the concrete or grade of reinforcing steel values of $f'c = 20$ MPa and $fy = 230$ MPa have been used, as per Cl. 14.7.4.3 and 14.7.4.4 of CHBDC.

The available structural drawings do not specify the traffic loading used during design. However, the maximum shear and moment diagrams provided appear to correspond to an HS20-44 truck load. As it appears that the structure was not designed for the CL625 vehicle now specified in the CHBDC, it is anticipated that load posting be required regardless of the state of the structure.

Evaluation Loads

The structure weights (dead loads) were computed based on the original unreduced sections, the geometry of the bridge, and the material densities specified in the CHBDC. The live load evaluation was performed to Evaluation Levels 1, 2, and 3. The loading that corresponds to Level 1 in accordance with the CHBDC is the CL1-625 Truck and CL1-625 Lane Load. The CL2-625 Truck and CL2-625 Lane Load correspond to Level 2, and the CL3-625 Truck and CL3-625 Lane Load correspond to Level 3, as per the CHBDC. Based on a vehicular traffic count carried out in May 2008, a Class C Highway was used for the analysis, resulting in a Lane Load of 7 kN/m. A Class C Highway has an Average Daily Traffic (ADT) per lane of between 100 and 1000 vehicles, and an Average Daily Truck Traffic (AADT) per lane of between 50 and 250 trucks.

A design return period of 50 years is specified in the CHBDC for the wind pressure on the towers, however a return period of 10 years was used in the evaluation, as the structure is expected be replaced during this time span. As a result, an hourly mean reference wind pressure of 570 Pa, based on the town of Argentia, was used. A wind pressure of 570 Pa is equivalent to a wind speed of approximately 110 km/h. A gust effect coefficient of 2.50; a wind exposure coefficient of 1.30; a horizontal wind drag coefficient of 1.70; and a vertical wind drag coefficient of 1.00 were used. This results in a horizontal wind load per unit exposed frontal area of 3.15 kPa and a vertical wind load per unit exposed frontal area of 1.85 kPa. The equivalent horizontal and vertical gusting wind speeds are approximately 258 km/h and 198 km/h, respectively.

Evaluation Scenarios

The lift span was evaluated based on its as-found condition from the inspection performed in October 2011. However, due to the extent of deterioration found during the inspection, consideration was given during the evaluation to the possibility of failure of some members of the floor beam truss system. As such, the following scenarios were investigated:

1. As-found condition.
2. Missing one end diagonal and the same diagonal in the adjacent truss.
3. Missing the vertical brace in the end truss and the vertical brace in the middle truss.
4. Missing the vertical brace in two adjacent trusses near the middle of the span.
5. Missing one diagonal near the middle of the span.
6. As-found condition, with single lane in centre of bridge.
7. As-found condition, with alternative vehicular loading over two lanes of traffic

The alternative vehicular loading consists of a two axle vehicle with axle loads of 90 kN and 60 kN. Two axle spacing scenarios were investigated, 3.0m and 4.5m. The evaluation of this alternative loading was completed as per Clause 14.9.1.6 of the CHBDC.

Reference Figure 2 for a typical section showing the components of the lift span deck.

Evaluation Findings

Based on the evaluation performed as per the CHBDC, the structure requires a triple load posting of 13, 16, and 25 tonnes for single unit vehicles, two-unit vehicles, and vehicle trains respectively, based on the governing lift span.

The bridge was evaluated under a single traffic lane along the centre of the bridge and the associated triple load posting requirement under this condition is 19, 23, and 35 tonnes for single unit vehicles, two-unit vehicles, and vehicle trains respectively, based on the governing lift span.

The bridge was evaluated for alternative loading of a two axle vehicle representing a school bus and a fire truck, with axle loads of 90 kN and 60 kN and two axle spacing scenarios, 3.0m and 4.5m. It was determined that the vertical braces in the lift span were not capable of supporting this loading over two lanes of traffic. However, sensitivity analysis showed that should the section loss of the vertical braces be limited to approximately 25% then the structure would be capable of supporting the alternative loading over two lanes.

The results of the lift towers evaluation indicated that, in their current condition, several members (14) of the lift towers do not have the required strength per the CHBDC. As wind loading governed the capacity/demand ratios for most members of the lift towers, different wind loading conditions were evaluated, from the 1 in 10 year wind speed of 110 km/h (258 km/h gusting), to a wind speed of 25 km/h (59 km/h gusting), to determine the possibility of operating the lift span under wind speed restrictions. The analysis results showed that the lift span could be operated under 50 km/h wind (117 km/h gusting).

Structural Rehabilitation

Based on the scoping study the following items were chosen to ensure safe and reliable bridge operation for the remaining life of the bridge:

- Removal and replacement of the main girder top cover plates. See Figure 14.
- Replacement of the approach span and lift span floor beam support trusses and associated connections. See Figure 14.
- Miscellaneous structural steel repairs including lift tower member strengthening, sidewalk channel replacement, railing anchorages and grating repairs.
- Bearing repairs and bearing stiffener replacement.

The structural strengthening work was carried out and completed in 2012 at a total cost of $1.7 million.

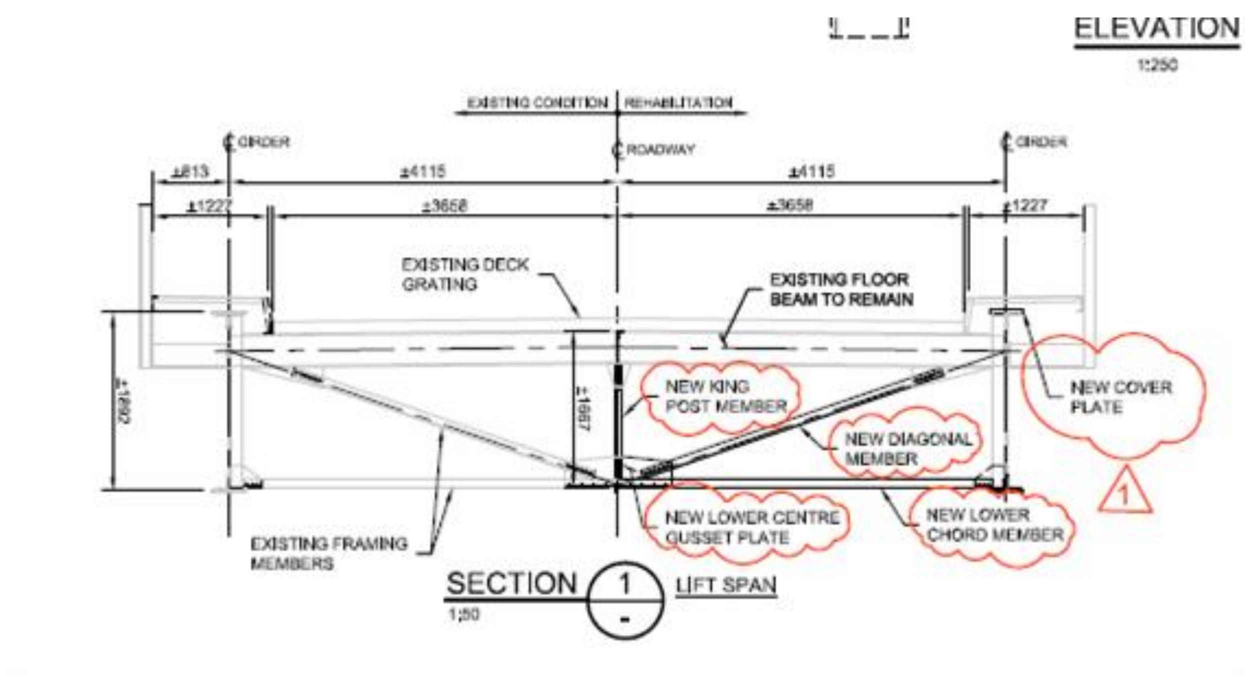

Figure 14. Structural Rehabilitation Schematic.

Conclusion

Many owners experience harsh environments that accelerate the degradation of their bridges. In the case of a movable bridge in Placentia, Newfoundland and Labrador, severe environmental conditions presented additional challenges compounded by the need to maintain multiple systems. The structural, electrical, and mechanical systems for a movable bridge each have specific maintenance challenges in such continual harsh weather. With prolonged exposure to the elements, movable bridge components are more likely to break down and may ultimately fail without extraordinary care taken to establish routine inspection and maintenance to suit specific requirements for each unique movable bridge situation. The deterioration of the Sir Ambrose Shea lift bridge is an example of accelerated wear on multiple components due to long term exposure to harsh environmental conditions. Even with a scheduled bridge replacement, the existing structure was found to require short term rehabilitation to provide continued reliable and safe service to the traveling public for the remainder of the useful life of the structure.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Replacement of the South Park Bridge

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

REPLACEMENT OF THE SOUTH PARK BRIDGE

Introduction

The South Park Bridge is located approximately four miles south of downtown Seattle, Washington. The bridge provides a critical connection, through the South Park community, between Seattle, southwest King County, and the Duwamish manufacturing and industrial center. The bridge carries 14th/16th Avenue South over the Duwamish Waterway, a roadway that is designated a truck route by King County due to its greater-than-average concentration of truck traffic. This relatively short stretch of road crosses the jurisdictional boundaries of City of Seattle, City of Tukwila, and unincorporated King County, all three of which are primary stakeholders.

The existing bridge was built in the late 1920's and opened to traffic in 1931. By many accounts, it has been well maintained by King County in recent decades, despite the influences of nature throughout its life. Construction quality, soil conditions, and intense seismic activity, among other things, have taken their toll on the bridge. The past few decades comprise a period of exhaustive deliberation over the future of the bridge. Actions to replace the span began in 1998 with a proposal to build a mid-level fixed-span bridge. Due to community displeasure and United States Coast Guard (USCG) requirements for navigational clearance at this location, the collective group of government agencies stepped back to re-evaluate the bridge's path forward. Feasible alternatives were developed and thoroughly evaluated, including permanent closure. The end result was a decision by King County in 2006 to replace the aging iconic structure that had served its community well for nearly 80 years.

Background

The main span of the original South Park Bridge, formerly known as the 16th Avenue South Bridge, was a double-leaf Scherzer rolling lift bascule bridge (Figure 1). At the end of its service life, it was the only functioning bridge of this type in the state of Washington, garnering a spot in the National Register of Historic Places (NRHP). In addition to NHRP recognition, the King County Landmarks Commission

Figure 1. Original South Park Bridge

(KCLC) further venerated the bridge by bestowing upon it “landmark” status in December of 1996. The approach spans were made up of a combination of concrete slab spans and steel truss spans, the latter spanning the edges of the waterway and framing into the bascule piers on both ends of the movable span.

Existing Conditions

The deteriorating condition of the bridge is well documented through various inspections, reports, studies, and tests. Major rehabilitation efforts were carried out in the 1970’s and 1980’s, including underpinning of the approach span foundations, post-tensioning of the bascule pier walls, and multiple instances of concrete repair and bascule machinery alignment efforts. Inspections in the late 1980’s and early 1990’s were followed up by costly recommendations for strengthening the bridge to marginally lengthen its expected service life. The reports and recommendations were mainly focused on pier strengthening and mechanical modifications to correct draw span misalignment due to excessive differential movement between the bascule piers.

In 1994, King County conducted a Life-Cycle Cost Analysis (LCCA) to evaluate potential courses of action for the aging structure. The LCCA considered rehabilitation strategies aimed at extending the life of the bridge 10 and 50 years, replacing the bridge, or doing nothing, which effectively equated to permanently closing the span. The replacement strategy was further expanded to include both mid- and high-level fixed-span options, as well as a movable span option. This analysis concluded that a mid-level fixed-span was both the preferred replacement option and the recommended course of action, subject to approval of proposed navigational clearances by the USCG.

Subsequent inspections continued to monitor the bridge’s rate of decline, consistently noting the condition of the submerged portions of the bascule piers as “fair to poor.” Large cracks and spalls were present. Concrete samples from the pier structures were tested and determined to exhibit significant compressive strength reduction and adverse chemical composition. In 2003, a concrete condition survey indicated that the piers warranted immediate attention and noted that conventional methods of arresting deterioration were not likely to improve the structure’s outlook due to multiple failure mechanisms working together to weaken the bridge.

Seismic Vulnerability

As noted in the previous section, the bridge has experienced excessive differential movement between the piers over its lifespan. The majority, if not all, of the movement has taken place at the north bascule pier, and this assertion was supported by the advanced deteriorated state of the north pier relative to the south pier. Large cracks in the pier walls and misalignment of the machinery at the track girders are a couple of indicators that the north pier was suffering more than its counterpart on the opposite side of the channel.

The underlying cause was determined to be lack of suitable soil strata supporting the pier structure. While the timber piles at the south pier were driven to very stiff, over-consolidated glacial material, the north pier foundation elements were tipped in soils with considerably less density and load-bearing capacity, despite the fact they were driven to an elevation approximately 25 feet lower than the south pier piles. Inspector log books show that timber piles under the north bascule pier were not driven to refusal, and many of the 315 timber piles experienced one to two inches of movement on the final blow of the pile driver. Such construction practice is believed to be caused by the limiting length of timber piling. Piles were also noted as having been driven into pre-jetted holes, with no reason given for this practice.

Multiple seismic vulnerability studies in the past two decades have highlighted the liquefaction potential of the higher-elevation, non-glacial strata beneath the bridge. Without penetration into reliable soil, the north pier is particularly vulnerable to movement and instability should the upper soil mass liquefy during an earthquake. In fact, following a 1949 earthquake, the northern approach span truss permanently shifted from its bearings, requiring jacking and installation of newly positioned bearings. Seismic retrofit recommendations in 1998 for the movable span included additional truss bracing, enhanced restraint at the interface of the moving leaves and their respective piers, crack repair and new foundations.

The Nisqually earthquake rattled the region in February, 2001, one of the largest recorded seismic events in Washington State history. The epicenter of this 6.8-magnitude earthquake was approximately 30 miles from the bridge. Despite its advanced state of decay, the South Park Bridge survived, but not without suffering nearly $1 million of additional damage, leading to increased concern over the safety of the bridge and its vulnerability to future seismic events.

Replacement

Following this most recent earthquake, attention became increasingly focused on laying out a plan for the future to provide for safety of motorist and pedestrians using the span, as well as freight mobility and stability of the local economy dependent upon the access provided by the bridge. Rehabilitation and replacement alternatives were formally compiled into a Draft Environmental Impact Statement (DEIS) and compared to identify the most feasible option. In early 2006, based on the detailed information in the DEIS, along with significant input from the community and other stakeholders, King County decided to replace the structure with a new bascule bridge. This option was selected because it had the least impact on a variety of disciplines, providing the necessary improvements for functionality, seismic resistance and safety, while minimizing impacts to the surrounding community, maintaining the navigation channel, and preserving the visual character of the original structure.

The decision to replace the span was subsequently underscored by a sufficiency rating of 4.0 (out of the possible 100 on the NBIS rating scale) following a routine NBI inspection. The South Park Bridge was also featured in an episode of The History Channel’s *Inspector America* after the bridge was permanently closed on June 30, 2010.

Preliminary Design

The replacement option was advanced, and a preliminary design was developed in 2007-2008. The intent of the proposed bridge was to enhance safety through improved roadway geometry and roadside features, augment the navigation channel to meet current USCG requirements, and accommodate pedestrians and cyclists while complying with the Americans with Disabilities Act (ADA). Seismic design resistance would be achieved in the preliminary design through earthquake drains, drilled shaft foundations deep enough to reach reliable bearing soils, and analytical optimization of the structure’s fundamental period and mass distribution.

The proposed piers were founded on sixteen 8 ft-diameter drilled shafts with composite steel casings. The pier structure was a unique eight-sided structure. The clipped corners allowed the piers to be positioned closer to the skewed channel, minimizing the span length. Two control towers were provided to mimic the appearance of the old bridge, although bridge operation was possible only from the north tower. The control towers were both heightened and upsized from the minimum floor space requirements to improve sight lines with marine traffic and to better architecturally proportion the towers with the large pier structures. Architectural finishes, such as rustication lines, brick and decorative railings, were selected to provide visual consistency with the original bridge.

The proposed main span was a double-leaf bascule bridge spanning 227 feet from trunnion to trunnion. Each leaf consisted of parallel steel trusses, floorbeams, and an Exodermic™ deck system. The roadway section was made up of four 11-foot lanes, a 4-foot median, and a 13-foot bicycle/pedestrian walkway along one side of the bridge.

The span drive was electro-mechanical gearing, with load equalization accomplished through mechanical load sharing. All drive machinery was supported on the floor of the bascule pier. The trunnions were supported on single bearings outboard of the bascule girders, connected by a stiff trunnion girder spanning between the trusses. Span locks were recommended to be jaw-type locks based on satisfactory performance under both traffic and seismic loading.

The proposed electrical service consisted of three-phase, 480-volt AC power feeding the north pier motor control center (MCC), along with a three-phase sub-feeder routed to the south pier MCC via a submarine cable. For emergency operations, the north pier was to house a pair of diesel-powered generators, one for operating the span drive system and the other for powering navigation lighting, control tower functions, and other ancillary needs to safely operate the span during a power outage. The proposed control system was a programmable logic controller (PLC) with hard-wired interlocks and controls for safety and redundancy. Additional manual features are proposed for emergency operations. Based on the preferences and level of familiarity of maintenance personnel, the main motor drives were recommended to be four-quadrant, regenerative silicone-controlled rectifier-type solid state drives. The design included two drives per leaf, only one of which would operate at a time. An automatic switch was proposed to alternate drives between bridge openings.

At the time of preliminary design, a two-level approach for seismic design of bridge structures had become common. This design approach had not yet achieved consistency across the industry, so criteria specific to the South Park Bridge were developed following a detailed review of several documents and anticipated revisions on the horizon for AASHTO LRFD. The upper-level event, or Design Earthquake, as it was ultimately termed, was defined as having a 7.5 percent probability of exceedance in 75 years (975-year return period). The lower-level event, or Operational Earthquake, was defined as having a 50 percent probability of exceedance in 75 years (108-year return period). The Design Earthquake was effectively a life safety, or “no collapse,” event, during which moderate damage would be expected, but the bridge would remain standing. When subjected to the Operational Earthquake, the bridge would be expected to respond nearly elastically, such that damage to the bridge was minimized and it would be functional following an earthquake for access by emergency operations.

Final Design

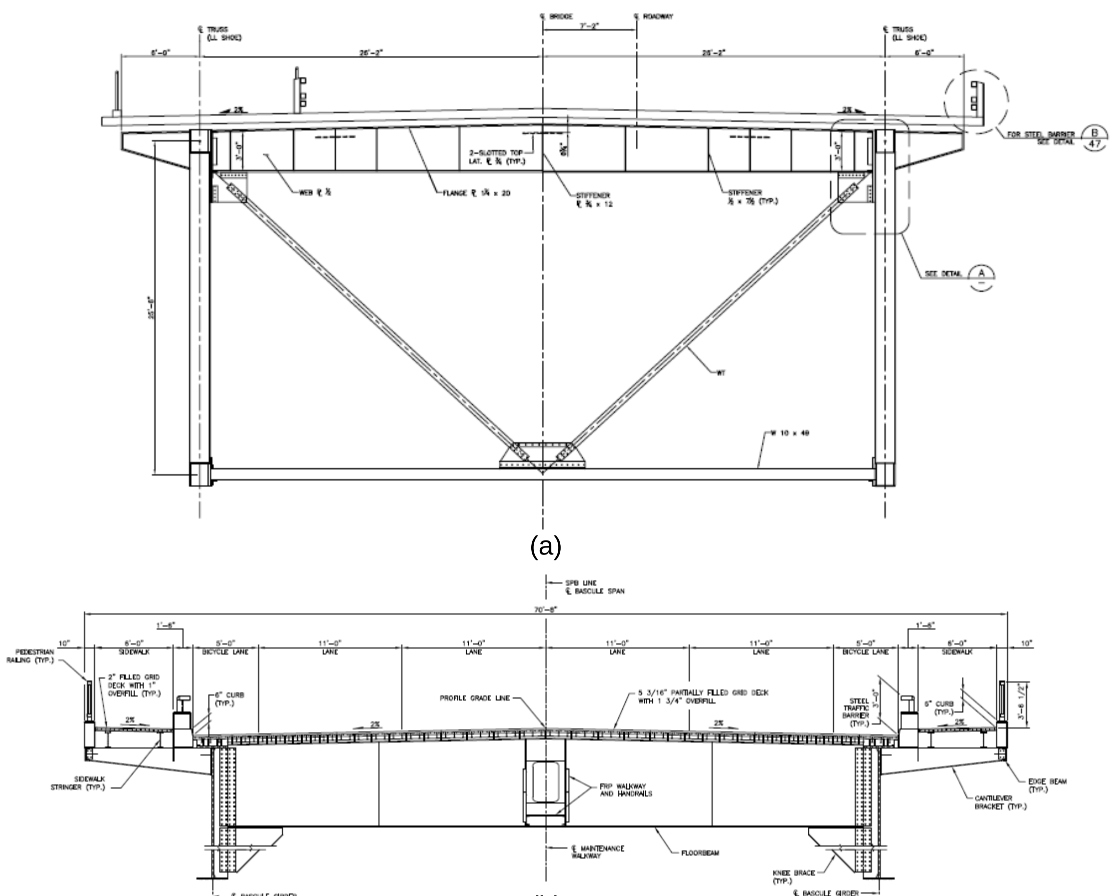
Following acceptance of the preliminary design, King County hired HNTB Corporation (HNTB) in 2008 to develop intermediate and final design and construction plans for the bridge. HNTB conducted a value engineering evaluation to identify modifications or alternatives to elements of the preliminary design that presented opportunities for initial or life-cycle cost benefits and improve the quality of the project. During intermediate design development, a foundation alternatives study was also performed to determine if the drilled shaft proposal was the most suitable foundation type for this structure. In the following paragraphs, concepts that were presented in the value engineering report and incorporated into the final bridge design are discussed.

Symmetric Cross-section

The use of a symmetric cross-section simplifies geometry, shop drawings, fabrication, erection, and balancing of the bascule leaves (Figure 2). All major structural steel elements, including girders, floorbeams, cantilever brackets, and bracing, are similar and have identical camber. This was accomplished by reducing the median to 2 feet wide, distributing the four 11-foot lanes symmetrically with respect to the bridge centerline and flanking the roadway on both sides by 5-foot bicycle lanes and 6-foot barrier-separated sidewalks. This configuration enhances safety over the preliminary design by permitting bi-directional cyclist and pedestrian traffic across the span without having to cross the street adjacent to the bridge at either end.

Plate Girder

The use of a truss superstructure (Figure 3) for this length for a bascule bridge was identified as inefficient. Truss fabrication is complex, requiring bolted connections and intricate framing at each panel point. The complex geometry of such framing promotes accumulation of debris, creating areas of the structure that are prone to deterioration. Replacing the truss with a plate girder reduced the depth of the superstructure, permitting the options of increasing navigational clearance and lowering the roadway profile, and it facilitated fabrication and erection while providing for enhanced maintainability. To retain



(a)

(b)

Figure 2. Preliminary (a) and final (b) cross-sections

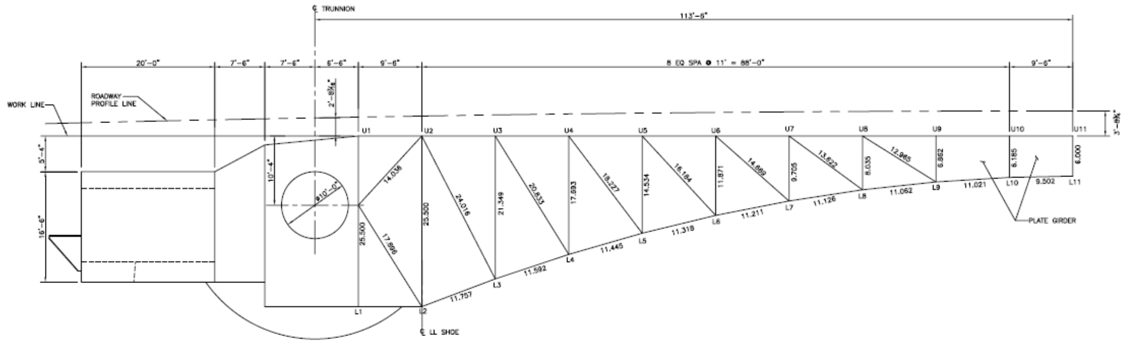


Figure 3. Conventional truss girder (preliminary)

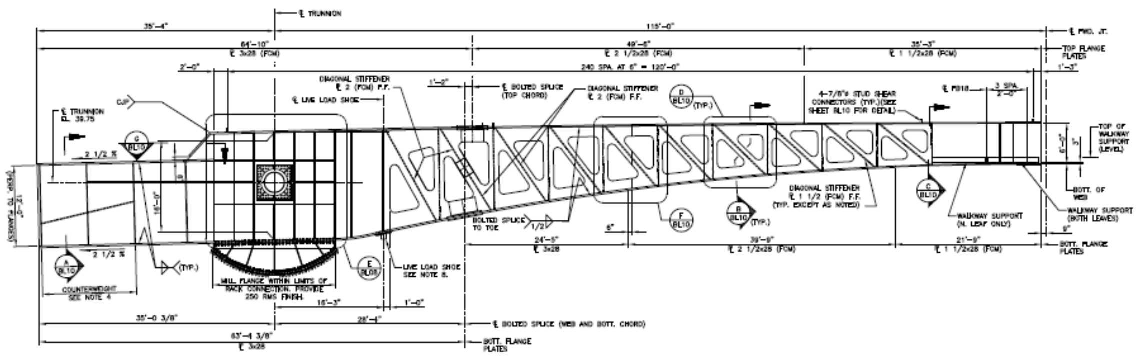


Figure 4. Perforated-web plate girder (final)

the truss-like appearance, the design of the girders incorporated triangular perforations between the floorbeam connections (Figure 4).

This concept was developed with the goals of improving safety, reducing long-term maintenance costs, extending the service life of the bridge, and expediting erection time. All of these goals were accomplished by a single feature of the design: eliminating the gusset-plated joints. Truss joints are, arguably, the most problematic elements of trusses because they consume extraordinary effort during fabrication and erection, and they have a tendency to collect dirt, debris and moisture, making them difficult and time-consuming to inspect and maintain. Gusset-plated connections are often to blame for deterioration that leads to expensive rehabilitation, replacement, and, in extreme cases, failure of steel truss spans. For this project, this design strategy achieves the safety, maintenance, and service-life goals while producing a pleasing structural form reminiscent of the existing bridge, enhancing the view for those living and working in the South Park community.

Reduced Floorbeam Spacing

Reducing the floorbeam spacing avoided the necessity of using an Exodermic™ deck system. A conventional partially-filled grid system provided the potential for cost savings by avoiding patent-licensing costs for the trademarked system and giving potential bidders the option of using a readily available product.

Lightweight Overfilled Grid Deck

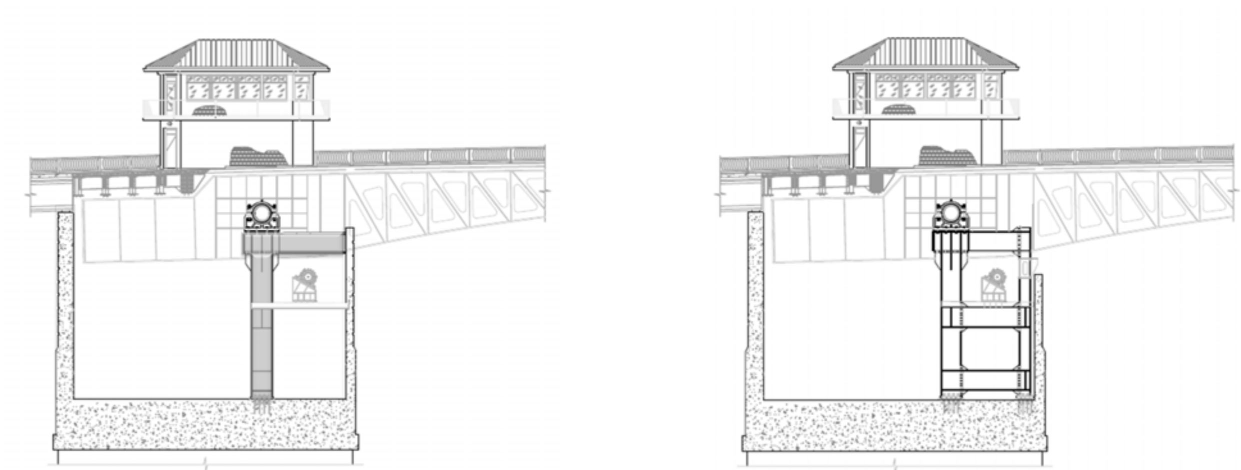
The use of lightweight concrete reduced the force demands and minimized the size of the counterweight, enhancing the overall seismic behavior of the superstructure and bascule pier. Based on the intense seismicity of the region, seismic demand was expected to govern the design of many, if not all, major load-carrying members in the bascule superstructure and piers. Therefore, reducing mass was an effective strategy, and the lighter superstructure effected positive impacts on the mechanical design by reducing the demands on the span drive system.

Sunken Caisson Foundations

During the early stages of final design development, sunken caissons were evaluated and compared to the drilled shaft arrangement preliminarily proposed. The deep soils required for bearing of the shafts resulted in large unsupported lengths in liquefiable soils, limiting their strength available for resistance to lateral loading combined with significant axial demand. The corresponding flexibility also yielded excessive lateral displacements. In the case of a movable span, where alignment between moving parts is critical, deformations are not necessarily desirable, as can be the case in other seismic design situations. The additional resistance of the large side-face area of the caisson foundation resulted in smaller, more manageable displacements than the drilled shaft foundation for a given level of lateral loading, making it more ideal in this instance.

Trunnion Frame

The interface between structure and machinery presents several unique issues to be considered in the design of a movable span, particularly in a high-seismic region. To ensure functionality following the Operational Earthquake, relative displacement between the machinery floor and the trunnion must be limited to retain alignment sufficient to operate. Further, following the Design Earthquake, possible re-alignment of individual machinery components was anticipated to restore the bridge's ability to operate. Each bearing, brake, gearbox, etc., would be individually subject to adjustment, the final position of each dependent upon all of the others for adequate alignment.



(a) (b)

Figure 5. Preliminary (a) and final (b) trunnion support

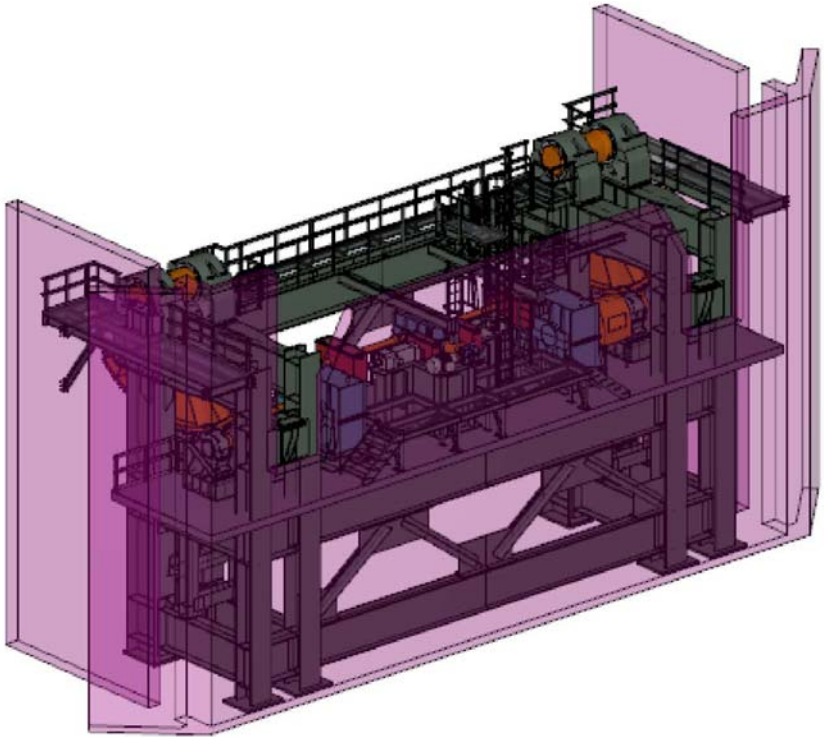


Figure 6. Three-dimensional model of trunnion frame and span drive machinery

To provide uniform support stiffness for the trunnion bearings and facilitate post-event re-alignment of the machinery, each bascule span, its supports, and its mechanical drive system are supported on a robust, free-standing steel frame, isolated from the bascule pier on all sides, and anchored to the pier floor (Figure 5 and Figure 6). The trunnion frame is excited by movement of the pier floor during an earthquake, but otherwise responds independently of the pier structure. Each trunnion frame is designed to resist the inertial forces induced during excitation within the elastic range of the steel, essentially retaining its geometry and the relative position between all components. Regardless of ground displacement, the machinery will remain in the same location relative to the trunnions. Should one or both piers experience permanent set in the form of global translation and/or rotation, each frame can be jacked uniformly or differentially at the four corners to reorient each bascule leaf and re-establish alignment of the span across the channel while maintaining internal alignment of the machinery components within each leaf.

Twin Trunnion Bearings

The preliminary design proposed a cantilevered trunnion with a single bearing at each support. By using a pair of bearings simply supporting the trunnion at each bascule girder, the trunnion girder is eliminated and the size of the bearings is reduced. Trunnion girders are non-redundant and vulnerable to permanent distortion during a seismic event, even from the Operational Earthquake, after which the bridge is expected to operate. Repair of this element is a major undertaking, and if repair is required due to trunnion misalignment following the lower-level event, the performance objective will not have been satisfied. Twin trunnion bearings supported on individual columns are much less sensitive to distortion from a large seismic event. Repair or replacement can be accomplished without significant disruption to bridge operations.

(a) (b)

Figure 7. Operating machinery: (a) 3D model view and (b) during construction

Operating Machinery

The operating machinery (Figure 7) follows a fairly conventional trunnion bascule machinery layout. As with most drive machinery, the high-speed, low-torque power of the prime mover is converted to low-speed, high-torque power through a series of speed reductions. The drive train for each leaf includes two 75 hp, 850 rpm, DC motors coupled to a differential central reducer. The two output shafts of the central reducer each drive a secondary reducer via a floating shaft. The output shafts of the secondary reducers each drive a pinion shaft integral with a pinion, which drives against a curved rack mounted to the bottom of the bascule girders. Normal operation is under use of both main motors operating in an overspeed condition. The spans are capable of being operated with either motor individually under reduced operational loading. Additionally, there is one auxiliary 50 hp, 1750 rpm, AC motor that is coupled to the central reducer via an auxiliary reducer and an electrically engaged clutch. The auxiliary motor provides operational redundancy independent of the PLC and drive controllers.

Design criteria required that the bridge remain operational immediately following the lower-level seismic event, and after minimal down time in the wake of an upper-level event. While differential displacement between machinery elements is a significant concern when accommodating seismic performance demands, the implementation of the integral trunnion frame and machinery platform mitigates seismically induced impact or disengagement of rack and pinion teeth, as well as post-event re-alignment of the entire leaf.

Collapsible Center Joint

Despite the improved foundation response with the sunken caissons, the structure was still expected to accommodate large transverse and longitudinal displacements during the two levels of seismic events. In conjunction with designing the trunnion frames to remain elastic, limiting the forces to be resisted presented a challenge. Without the ability to accommodate the large displacements expected, longitudinal impact forces between the two leaves would add to the inertial force of a supported leaf. Impact forces were significant, resulting in significant additional size in the trunnion frames and adding to congestion in the piers. To mitigate the force due to collision during an earthquake, a suitable gap between the tips of the bascule leaves was to be provided. Maximum computed relative longitudinal displacement at the upper-level event was 18 inches. Each bascule girder was held back 9 inches from the centerline of the center joint, leaving a large gap to be bridged to cross from one leaf to the other. Using the concept of a finger joint, a series of brackets project from the end floorbeam of each leaf, offset from one side of the joint to the other. Large segmented plates span between the brackets on each side of the joint to close the gap (Figure 8).

The objective is for the joint to act as a fuse, failing at a relative low longitudinal force and limiting the additional impact load resisted by the trunnion frames. These plates are minimally bolted to their supporting brackets to provide resistance to vertical loading while remaining susceptible to failure under lateral loading when impacted by the opposing leaf. Non-high-strength bolts are used where cyclical loading is not expected, and holes are strategically slotted to sequence bolt failures within a given plate segment. The plates are tapered and the supporting brackets are sloped to induce simultaneous tension, limiting the available shear capacity of the bolts at failure. Segmenting the plates serves two purposes: 1) limiting the failure force by isolating the fuse bolts to the vicinity of contact and 2) facilitating replacement of short segments of plate when damaged. The lateral and longitudinal offsets between "fingers" of the underlying support assembly provide clearances in accordance with the Design Earthquake displacements such that contact will not occur between the leaves once the surface plates are sacrificed.

Span Locks

The preliminary design recommended jaw-type devices based on past reliable performance on bridges in the region. The design strategy at the center joint to accommodate extreme displacements between the girders rendered this type of device impractical due to the reach required from one leaf to the other. Linearly actuated lock bar systems are better-suited for this application, but for the required stroke length on this bridge, even the largest of conventionally used lockbar systems was not up to the task.

The span lock system of choice for this bridge was the CushionLoks® system with an Earle EG-5 lockbar operator, manufactured by Steward Machine Company, Inc., in Birmingham, AL (Figure 10). This project required a larger-than-usual 7"x10" lockbar with a longer-than-usual stroke of 26 inches required to span the seismic performance-driven clear gap between the leaf tips. To maintain the seismic clearance between the end floorbeams of the leaves, the front guide and receiver assemblies were face-mounted entirely to the inside face of the floorbeams (Figure 9).

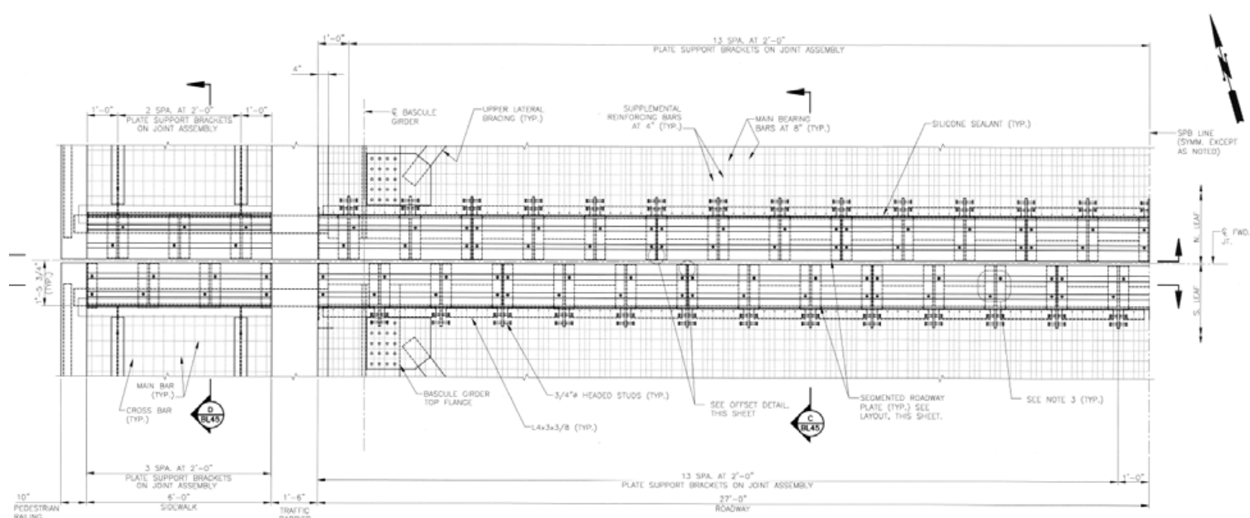


Figure 8. Partial joint plan at midspan

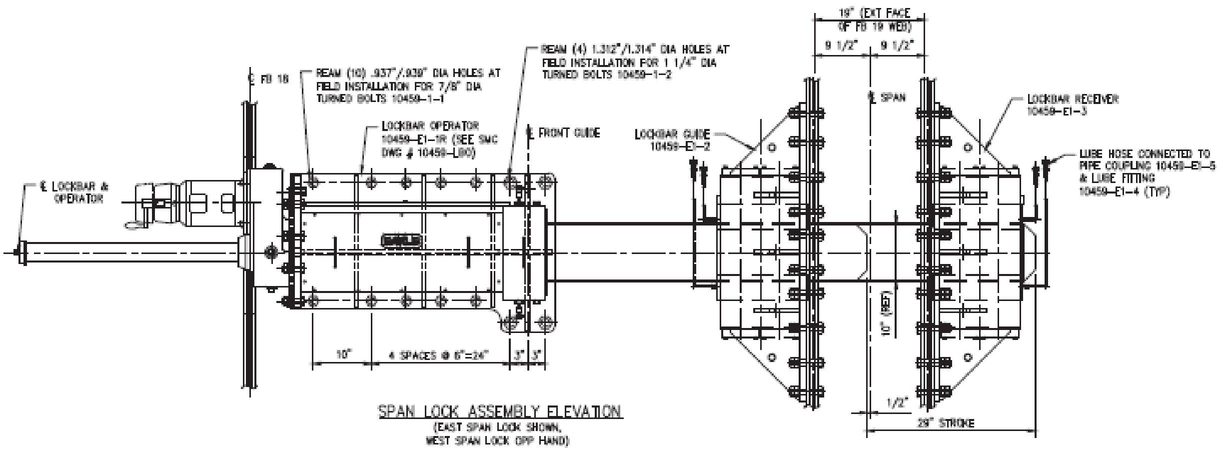


Figure 9. Span lock assembly detail

In conjunction with the additional longitudinal force due to leaf contact, the trunnion frames were also designed to resist additional transverse force, and the associated horizontal moment, required to fail the lockbars. An over-strength factor was used to determine the upper-bound lockbar failure load, and the width of the lockbar was kept to a practical minimum to minimize the transverse failure load.

Integral Counterweight

A counterweight that extends to the top of the structure and serves as part of the roadway offers multiple benefits to a bascule span. The additional depth increases the total mass for a given length of counterweight, reducing its length. A shortened counterweight reduces the longitudinal dimensional requirement of the pier structure. The floor elevation could potentially be raised, as well, due to the reduction in swing radius of the open leaf. The pier structure is simplified by eliminating the cover span over counterweight, and the transverse roadway joint is located at the back of the pier, away from the machinery, so joint leakage is significantly less problematic for maintenance.

(a) (b)

Figure 10. Installed span lock assembly: (a) actuator, lockbar and forward guide and (b) receiver

Despite the positive impacts on the design, this strategy does introduce disadvantages. The length of the bascule leaf behind the trunnion exposed to traffic loading dictates the need for tail locks to support live load. Also, because this fixed-trunnion bascule does not translate backwards as it opens (like its Scherzer predecessor), the counterweight pit is fully exposed when the bridge is in the opened position. Without a robust barrier system for vehicular traffic and fall protection for maintenance personnel, it was deemed too risky and removed from consideration approximately six months into the final design phase of the project.

Construction

The original plan was to build the replacement bridge adjacent to the existing structure on a parallel alignment in order to maintain traffic at the crossing for the duration of the project. The start of construction was delayed due to funding challenges, partly due to a failed attempt to obtain a grant through the initial offering of the Transportation Investment Generating Economic Recovery (TIGER I) program, created under the American Investment and Recovery Act of 2009. A second application was made for a TIGER II grant in mid-2010, but not before King County performed a condition and stability assessment of the existing bridge and decided it was no longer safe. HNTB subsequently concurred with the recommendation to close the bridge. It was officially taken out of service on June 30, 2010, and the superstructure of the bascule leaf was demolished in August of the same year.

The request for TIGER II funding received a positive response in October 2010, significantly reducing the funding gap that was delaying the start of construction of the new bridge. With the bridge out of service for the foreseeable future, the desire to commence rebuilding was heightened. The construction contract was advertised in March 2011, and a contract was awarded to the joint venture of Kiewit Construction Company and Massman Construction Company in April with a bid price of $97 million, 10 percent lower than estimated. Work at the site began in June 2011, with completion scheduled for December 2013. Due to unanticipated challenges during construction, e.g., installation of the deep caisson foundations, overweight leaves, and bascule span joint alignment, the overall duration was extended by approximately six months.

On June 30, 2014—exactly four years to the day after the bridge was taken out of service—the replacement South Park Bridge opened for business. The long-awaited opening-day celebration marked the re-establishment of this vital economic and transportation lifeline for the residents and businesses of South Park.

As with any bridge project, particularly one with a high degree of complexity, success is not attributed to a single individual, but to the collective work of many. King County took the lead for procurement of the span, and City of Seattle will ultimately become the owner. The bridge was designed by HNTB Corporation utilizing a nationwide network of engineers in Seattle, Kansas City, New York City, and Tampa. HDR was hired to lead the construction management effort, and URS was subcontracted to provide construction engineering and inspection services for the movable span. Kiewit-Massman assembled a robust team of contractors and suppliers to bring this bridge to fruition. The structural steel components of the bascule span were fabricated by Stinger Welding, Inc. (SWI), at their facility in Libby, MT, and Mountain States Steel (MSS) in Lindon, UT. SWI was responsible for fabrication, pre-assembly and alignment of the bascule leaves, and MSS performed similar activities for the trunnion frames, as well as fabrication of infill components for the bascule leaves. The steel deck grating was produced by L.B. Foster, of Pittsburgh, PA. Span drive machinery components were fabricated by Hardie-Tynes

Company, Inc., of Birmingham, AL, and Overton Chicago Gear Corporation of Chicago, IL. As previously noted, the span lock systems were provided by Steward Machine.

Conclusion

The South Park Bridge Replacement project is the result of a collaborative effort between agencies at all levels of government, design consultants, contractors, and the community whose livelihood is dependent upon the bridge. Millions of dollars have been spent over the past few decades to ensure a safe crossing while plans to build a stronger, safer, more and resilient and reliable, longer-lasting structure in this seismically volatile region were in the works. The sacrifices endured by thousands of residents and business owners due to the closure of the original bridge has, in no small part, contributed to the success of making this project a reality. The new South Park Bridge enhances the built environment of the community, providing an iconic asset that is certain to elicit pride and a sense of satisfaction that the hardship they endured was worthwhile. To the engineering community, this bridge may serve as a model of innovation in an environment where owners and designers are working together to increase their focus on maintainability, reliability, and durability in an effort to minimize the life-cycle costs associated with a new structure, and doing so without compromising the safety and well-being of the traveling public.

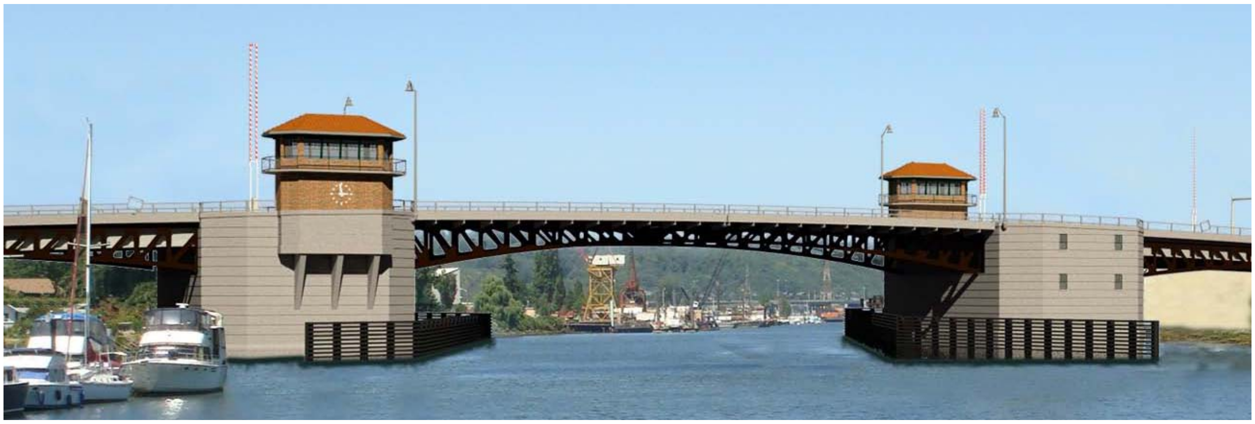


Figure 11.Rendered view of the replacement bridge (2009)

Figure 12. Replacement bridge near completion (2014)

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

REPLACEMENT OF TRUNNION BEARING – 92ND STREET BRIDGE

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Introduction

The 92nd Street Bridge in Chicago, Illinois has a double leaf bascule span located on the Calumet River and was built in 1914. The bascule leaves are of the Chicago type have simply supported trunnions and plain bronze bearings. Because of its location on the River, the bridge opens over 5,000 times a year and has been hit by ships on numerous occasions. The bridge had a major rehabilitation performed in the early 1990's. The machinery including the trunnion bearing assemblies date to original construction circa 1913. The inboard trunnions are supported from a deep trunnion girder that spans from one side of the counterweight pit to the other.

In late 2012 a City of Chicago maintenance worker discovered excess clearance at the northeast outboard trunnion bearing. As a result of this discovery the City of Chicago's Department of Transportation (CDOT) retained Wiss, Janney, Elstner Associates (WJE) to provide engineering services to rehabilitate all of the trunnion bearings on the east leaf of the bridge and to address the excess clearance problem on the northeast outboard trunnion bearing. WJE retained Stafford Bandlow Engineering (SBE) to provide the necessary mechanical engineering services. CODT's contractor, MQ Construction and its structural sub-contractor, Metropolitan Steel, retained Collins Engineers (Collins) to perform the engineering necessary to lift the span off the trunnion bearings and stabilize the leaf while the bearing work was being performed.

The necessary repairs led to a significant and interesting rehabilitation project that was completed in late September 2013. Along the way, there were numerous challenges and delays but, in the end, the project was a success. The bearing was rehabilitated and the bridge is operating satisfactorily.

Construction Engineering

Mechanical Design

Initial measurements indicate that the trunnion bearing clearance was approximately 5/8" at the northeast outboard trunnion bearing and that the other bearings on the same leaf were generally within acceptable limits given the age of the bridge. The maximum AASHTO specified clearance for a bearing of this size, 18" diameter, is 0.018". The measured clearance at the northeast bearing exceeded this value by a factor of nearly 35.

The measured clearance was great enough that it affected the alignment of the northeast inboard trunnion bearing where clearance existed at the bottom of the bearing at the inboard end. Therefore, the trunnion had rotated about the outboard end of the inboard bearing to accommodate the heavy wear at the outboard bearing. Clearly this was not a good situation and it was recommended that the bearing caps be removed for inspection of the wearing surfaces of the bearings. This work could not be accomplished prior to proceeding with the design because the bridge is located on a major route and needed to be kept in operation until work could commence.

As a result of the limited access to the bearing, is was not known if the excess clearance was due to bearing wear, trunnion wear or a combination of bearing and trunnion wear. Despite these unknowns, the design proceeded with what was believed to be adequate safeguards to deal with these unknown conditions. Another issue that needed to be dealt with in the design was determining and correcting the cause of the heavy wear. Fortunately this was fairly straight forward as it was discovered that at some time in the life of the bridge an electrical box was installed in a position that prevented access to the lube fitting for the northeast outboard bearing and therefore it was postulated that the bearing ran without lubrication for an extended period resulting in the heavy wear.

The plan was to field machine the trunnion journals and replace the bronze bushings in the pillow block housings to restore the AASHTO specified fit between the bearing journal and the bushing. As part of this work, the method of lubricating the bushings would be changed from a single lube fitting providing lube to multiple grease grooves in the trunnion to dedicated lube fittings for each grease groove in the bearing bushings. See Figures 1 and 2.

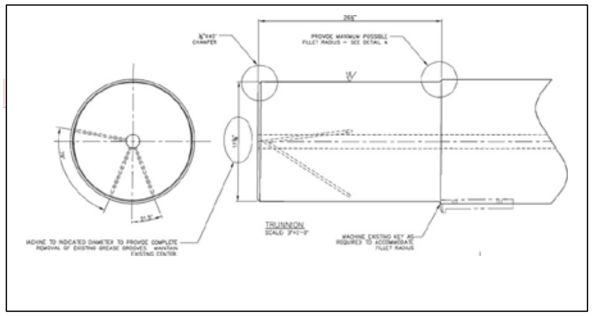

Figure 1. Trunnion will single lube fitting supplying three grease grooves.

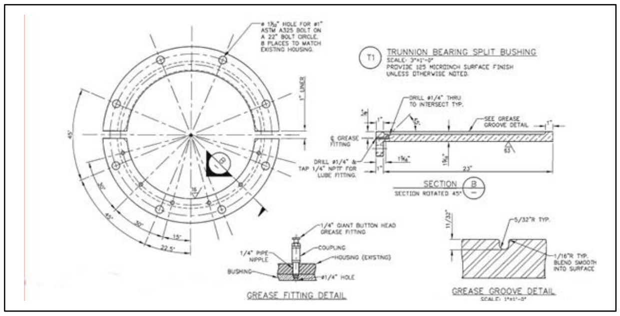

Figure 2. Bushing with six grease grooves, each with a dedicated lube fitting.

In order to remove the grease grooves from the trunnion, a minimum of 3/8” of material in diameter needed to be removed from the trunnion. To verify that this was acceptable, calculations were prepared to determine the following:

- Existing trunnion stress including fatigue analysis in accordance with AASHTO requirements
- Modified trunnion stress including fatigue analysis in accordance with AASHTO requirements
- Existing bearing pressure
- Modified bearing pressure

The trunnion fatigue calculations are highly dependent on the fillet radius in the trunnion where the trunnion steps up in diameter to the larger diameter that fits with the bascule truss. Although we had the shop drawings for the trunnion, the fillet radius was not called out. Therefore calculations were prepared for was we believed to be a likely fillet radius and the maximum possible radius based on the dimensions that were available. These values were then compared to the values that could be achieved with the modified trunnion. Table 1 shows the fatigue life of the trunnion based on the various options that were analyzed. The recommended material removal with the recommended fillet radius resulted in an adjusted stress at the fillet of 23,913 psi compared to an original adjusted stress of 26,873 psi using a 1/32” radius and 23,207 psi with the maximum possible fillet. All of these values exceed the endurance limit of 16,146 psi for the trunnion material indicating that the trunnion fatigue life was not infinite. Based on these calculations it was concluded that removing the 1/2” on diameter and providing the maximum possible fillet radius would produce a rehabilitated trunnion with a fatigue life that was most likely greater than the fatigue life of the original trunnion.

Table 1: Summary of Calculated Stresses at the Trunnion Fillet

92nd Street Bridge - Chicago, Illinois				
Trunnion Geometry Versus Adjusted Fillet Stress				
Journal Dia. (inches)	Material Removal (inches on dia.)	Fillet Radius (inches)	Stress at Fillet (psi)	Notes
17.875	0	.031	26,873	Original diameter with 1/32" fillet
17.875	0	.057	23,207	Original diameter with max possible fillet
17.625	.25	.125	25,849	
17.625	.25	.182	23,525	Recommended material removal
17.375	.50	.250	25,209	
17.375	.50	.307	23,913	
17.125	.75	.375	25,251	
17.125	.75	.432	24,322	

The bearing bushing material was called out on the drawings as having a chemistry of 80% copper, 10 % tin and 10% lead. This composition is nearly identical to ASTM B22 Alloy 937 which is a common alloy used in movable bridge bearings but not typically used in movable bridge trunnion bearings. Typically movable bridge trunnion bearings are the ASTM B22 Alloy 911 which contains a maximum of 0.25% lead and is a much harder alloy than alloy 937. The calculated bearing stress based on a span weight of 2,000,000 lbs. was 1,295 psi or 29.5% higher than the AASHTO allowable stress for alloy 937. Alloy 911 has an allowable stress of 1,500 psi and therefore this material would meet AASTHO requirements. The downside to using the harder alloy is that it is generally recognized that the journal should be considerably harder than the bronze. The trunnion material had a minimum ultimate strength of 58,000 psi according to the drawings. This meant that the steel was quite soft and was not a good match for the 911 alloy. It was concluded that alloy 937 would be used for the following reasons:

- The other bearing on the bridge lasted for nearly 100 years with minimal wear.
- The high lead content is desirable in a bronze bushing.
- The trunnion was not compatible with the other alloy and there was concern that this could damage the trunnion.

With these decisions made, the design for the rehabilitation of the bearings progressed and the signed and sealed documented were submitted to the CDOT on May 15, 2013. The completed design included the following work items at the northeast and southeast trunnion assemblies:

- Locate bearings by installing two dowel pins
- Survey bearing split line elevations to ± 0.001" at four locations
- Secure and jack span in the open position to allow for removal of the trunnion bearings while keeping the channel open to marine traffic.
- Remove and rehabilitate the trunnion bearings to include:
    - Verify location of bearing bore and provide specified fit with new bushing
    - Provide new split bronze bushing with grease grooves and dedicated fittings
    - Replace bearing cap bolts, base bolts, liners and shims
    - Replace split collar (thrust ring)

  - Replace steel collar (spacer between split collar and bascule truss)
- Machine trunnion journals to include:
  - Obtain 16 micro inch surface finish
  - Remove sufficient material to completely remove the existing grease grooves
  - Maintain existing center
  - Provide maximum possible fillet radius
- Return bearing to existing location by using dowels and survey data
- Lower span and verify alignment

Structural Design

The challenge for the Collins structural team was to determine a means to raise the bridge leaf so that the damaged northeast trunnion bearing could be removed and still maintain stability of the leaf during the removal process. As noted previously, the weight of the leaf was on the order of 2,000,000 pounds. Because of the level of boat traffic on the Calumet River and the need to minimize the roadway closure, it was directed that the leaf was to be jacked in the open position. WJE proposed an initial design that was later modified by Collins to meet the contractor's specific needs and to simplify construction.

The final design involved the use of four 500 ton short stroke hydraulic jacks that rested on the trunnion girder and raised the leaf with saddles designed to fit the exposed trunnion shafts between the bearings. See Photo 1. The saddles were designed to fully support the trunnion shafts and to resist wind loads of up to 20 pounds per square foot on the bridge leaf. Bolts with shear tabs were used to transfer the load from the leaf to the trunnion girder. The saddles needed to be able to accommodate a raise of approximately 2 inches in order to allow the northeast outboard bearing to be removed and accommodate machining. The trunnion girder was analyzed for the jacking loads and found to be structurally adequate for all loading conditions.
The counterweight required the installation of lock out braces to prevent the leaf from movement during the operation. Wide flange beams were installed between the counterweight bumpers and the pit backwall to stabilize the leaf. Beams were also installed at the front of the counterweight braced to the seawall to further stabilize the leaf.

Photo 1. Jacking stool under movable bridge girder at trunnion.

Jacking was accomplished through a centrally ported system that allowed each jack to be individually controlled so that the leaf could be leveled. Collins developed a procedure to guide the jacking of the bridge leaf prior to the removal of the damaged bearing.

Construction

CDOT retained a contractor early on in the process and the contractor was on board prior to the completion of the design to facilitate coordination. The construction was to take place over during a 10 week roadway closure with the knowledge that work needed to be complete by September 27, 2013 to avoid conflicts with other planned work in the City.

To avoid problems with material lead times, the castings for the bronze bushings and split collars were ordered in late April 2013 with additional stock based on worst case estimates regarding trunnion and bushing wear. This material would be provided to the machine shop selected to perform the work. The project encountered numerous delays and by August, 2013 it became apparent that it would be difficult if not impossible to complete the work within the allocated time. Realizing that the work would most likely not be completed as planned, CDOT decided to replace the bushing at the northeast outboard location (location with excessive clearance) without machining the trunnion and without removing the bearing housing.

WJE and SBE informed the City that this was not the best plan, however if this was their intention, we would assist them in their efforts. At this time, we insisted that the trunnion bearing caps be removed for internal inspection of the wearing surfaces to obtain as much information as possible for the fabrication of the bronze bushing.

Inspection

The inspection was limited to the northeast inboard and outboard trunnion bearings and included removal of the bearing caps to facilitate the inspection. At the inboard location the bearing cap could only be raised approximately 6” due to physical obstructions however this proved adequate to determine the condition. The inspection was conducted on Friday August 29, 2013.
The inspection included the following work and observations:

- Northeast Outboard Trunnion Bearing
  - The entire journal surface was covered with a heavy layer of grease.
  - Prior to cleaning the surface of the journal the journal surface was felt through the grease and the surface was very rough.
  - The journal surface was thoroughly cleaned using diesel fuel as a solvent.
  - After the grease was removed from the journal, the surface was found to have hardened lubrication deposits, scale build up due to heavy corrosion, heavy pitting, corrosion and scoring. See Photo 2. Scale and lubrication build up was approximately 3/16” over large portions of the journal in the area that remains above the split line of the bearing at all times.

Photo 2. Cleaned trunnion shaft exhibiting excessive corrosion.

- The bearing journal was measured at four locations along the length of the journal as follows:
  - Outboard of the bushing on an unworn surface. This measurement was taken as a reference dimension and represents the original diameter of the trunnion. The measured diameter was 17.842”.
  - Approximately 1” inboard of the outboard end of the bronze bushing. The measured diameter was 17.744”.
  - At the approximate longitudinal center of the journal. The measured diameter was 17.735”.
  - Approximately 1” outboard of the inboard end of the bronze bushing. The measured diameter was 17.554”.
- The taper in the journal measurements is consistent with observed conditions in that the original grease groove that was visible on the west side of the journal had very little remaining depth at the inboard end of the journal as a result of the wear. See Photo 3.

Photo 3. Note the limited depth of the grease groove.

- The outside diameter of the bronze bushing was measured at the approximate longitudinal center of the journal. The measured diameter was 19.963”.
- Measurements were also taken of the bushing length, flange OD, flange thickness and wall thickness for comparison with existing shop drawings. All dimensions were consistent with the shop drawings except for the overall length which was found to be 24 7/16” compared to 24” shown on the shop drawings.

- Northeast Inboard Trunnion Bearing
  - The journal was generally in good to fair condition. See Photo 4.
  - The journal surface was well lubricated.
  - There was light scoring and light pitting on the journal with more pitting and scoring at the end of the journal nearest the bascule truss.
  - There was no significant corrosion on the journal.

Photo 4. The journal is in good to fair condition.

  - There was light bronze embedment at the end of the journal nearest the truss. The end of the journal nearest the truss appeared to be more heavily loaded. This is consistent with the thought that the trunnion has rotated slightly due to wear at the outboard bearing.

- The journal measured 17.765” at the portion of the journal that extends beyond the inboard end of the bushing.

Based on these findings the following conclusions were made:

- The northeast outboard trunnion journal is in very poor condition and should not be returned to service until the journal surface is restored to an acceptable condition by machining. Drawings for this machining work were previously prepared and provide a suitable basis for this work. As with any precision work, it is imperative that this work is closely monitored by personnel familiar with the work and the associated acceptance requirements.
- The northeast inboard trunnion bearing is in serviceable condition and does not require rehabilitation. The end nearest the truss has suffered minor damage due to heavier loads that were caused by the excessive wear at the outboard bearing. Rehabilitation of the outboard bearing will redistribute the loads at the inboard bearing and eliminate the heavy loading at the end of the journal nearest the truss which should prevent further degradation of the journal.

The following recommendations were made:

- Rehabilitate the northeast outboard trunnion bearing assembly in general accordance with the rehabilitation plans dated May 15, 2013 and prepared by Stafford Bandlow Engineering, Inc. and Wiss, Janney, Elstner Associates, Inc. Note that this work was originally intended for all four bearings on the east leaf and therefore the plans may require minor modifications. The general scope of work at the northeast outboard location is consistent with the existing plans.
- Take detailed clearance and alignment measurements at the northeast inboard bearing as a baseline prior to jacking the bridge for the required work at the northeast outboard location.
- Remove the bearing caps at the southeast inboard and outboard trunnion bearings to inspect the condition of the trunnion journals.
- Take detailed clearance and alignment measurements at the southeast inboard and outboard trunnion bearings to provide information that may be useful in aligning the northeast inboard and outboard trunnion bearings.

Emergency Work

Now we had a real problem on our hands. The northeast trunnion bearing was in dire need of rehabilitation and we had 29 days to complete the work. To complicate matters, the bridge had yet to be secured in the raised position, there was no machine shop lined up to do any of the bearing work and there was no field machining contractor onboard to provide the necessary field machining work. Working with WJE and CDOT, SBE proposed that they take the lead in coordinating all of the required work to complete the trunnion rehabilitation portion of the work allowing the contractor to concentrate on the work required to secure the bridge in the raised position. CDOT and WJE agreed that this was a viable approach and the approach that had the best chance of success.

Since the necessary raw materials were on hand, we believed that it would be possible to get work done if in the available time if we could find the right people to complete the work. A good machine shop with the appropriate machining capabilities and available machine capacity and a skilled field machining company would be required to complete the work.

SBE had worked with Mountain Machine Works of Auburn, Maine on a recent project with great success and called to see if they would be interested in this project. After reviewing the drawings, discussing space constraints and going over the schedule, Mountain Machine Works agreed to take on the project. Initially we thought that a local machine shop within a few minutes of the bridge was going to complete all of the machine work and this would have been great as it would have kept transportation time to a minimum and in the event of a problem they would be close at hand. After meeting with the shop they decided that they did not have the machine availability to complete the trunnion bearing housing and bushing however they were willing to take on the other machine work and to be available throughout the project. There were more phone calls and more rejections until G&G Steel agreed to do the work within the required schedule. This complicated things somewhat as G&G Steel is located 620 miles from the bridge in Russellville, Alabama. They would have to get it right the first time in order for us to meet the schedule.

At this point everything was in place and if all went well we could meet the schedule.

The east leaf of the bascule span was jacked and secured in the fully open position on the morning of September 16, 2013. At this point we had 11 days to complete the work. The following is a brief chronology of events between September 16 and 27, 2013:

- September 16 – Bearing removed from bridge and transported to G&G Steel. See Photo 5.
- September 16 - 17 – Field machining equipment set up and aligned. See Photo 6

Photo 5. Note the poor condition of the bushing.

Photo 6. Field machining equipment in place.

- September 17 – Bearing arrives at G&G Steel and first chips are cut in the field.
- September 19 – Grease grooves removed from trunnion journal, rough machining complete.
- September 20 – SBE personnel on-site at G&G to inspect work progress.
- September 20 – Final trunnion OD determined and bushing ID provide to G&G.
- September 21 – Polishing of journal completed and Mountain Machine Works demobilizes. See Photo 7.

- September 24 – Rehabilitated bearing arrives on-site and installed on bridge. See Photo No. 8.
- September 25 – 26 – Bearing alignment and installation of bolts.
- September 26 – Removal of jacks and struts used to secure bridge in open position.
- September 26 – Initial operation of bridge following bearing rehabilitation.
- September 27 – Shimming of live load supports and clean-up.
- September 27 – Bridge opened to vehicular traffic on schedule.

Photo 7. Trunnion after machining and polishing.

Photo 8. Bearing base and bushing at installation.

The field machining proved more difficult than originally intended and it took several finish cuts to remove a taper. In the end the diameter was held between 17.231 and 17.234 along the entire length with a surface finish of 32 or better over the entire surface of the journal. Material removal was approximately 5/8” on diameter to provide complete cleanup of all corrosion indicating the severity of the corrosion.

Alignment and bearing shim adjustments were directed by SBE personnel. Alignment was checked with a precision level and feeler gages. At the completion of the alignment the outboard bearing and journal were parallel within 0.001”/ft. and level within 0.016”/ft. and the inboard bearing and journal were parallel within 0.0025”/ft. and level within 0.0025” per foot. At both bearings a 0.003” feeler could not be inserted between the bottom of the journal and the bushing. The alignment is considered very good.

Conclusions

The project did not go as originally envisioned however in the end the required worked was completed in the original time frame and to the satisfaction of the City and everyone else involved in the project.

Improvements were made to the design of the northeast trunnion bearing and lubrication of the bearing will no longer be an issue. With proper maintenance the trunnion bearings on the east leaf of the 92nd Street Bridge will provide reliable service in the long term.

Cooperation among all involved parties was critical to the success of this project. The City's trust in the Engineers and the Contractors involved in the project allowed work to proceed without interruption. Without this trust on the part of the City, the project would not have been completed in the allocated time.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Sheave Trunnion Fatigue and Replacement at Snohomish River Bridge in Everett, Washington

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NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Introduction

The Snohomish River Bridges are located near the town of Everett, Washington and carry Route 529 over the Snohomish River. There are two bridges, one carries northbound traffic from Everett to Marysville and the other carries southbound traffic from Marysville to Everett. This paper is going to concentrate on the rehabilitation performed on the bridge (529/10W) that carries southbound traffic. This bridge carries two vehicular lanes and a pedestrian walkway over the Snohomish River. The movable portion of the bridge is a tower drive vertical lift span with a length of 180 ft. and a width of 36ft. between the live load supports. The width of the channel is 105ft. The weight of the lift span is approximately 800 kips. The bridge was constructed in 1953.

The above photo shows three movable bridges. The subject bridge, 529/10W Snohomish River Bridge is in the middle and has an enclosed room on top of the tower. A railroad swing bridge is in the foreground. The 529/10E Snohomish River Bridge is in the background.

In the closed position, the movable span provides 35 ft. of clearance for marine traffic. The movable span can be raised 40 ft. to provide 75 ft. of clearance for marine traffic in the fully open position. Bridge operation is controlled from the operator's house on the adjacent 529/10E bridge. The point of operation for both bridges is combined due to the close proximity of the bridges. These bridges are owned and maintained by the Washington State Department of Transportation (WSDOT).

WSDOT performs in-depth inspections of the mechanical and electrical systems of movable bridges on a six year cycle, including disassembly and measurement of key components. Less intensive, routine inspections are performed on an annual basis. As part of an in-depth inspection, special inspections of the main sheave trunnion were recommended due to the original design of the trunnion and the time in service. Through these special inspections cracks in the trunnion fillet were discovered and it was determined that a rehabilitation was required. To perform this rehabilitation, the scope of the rehabilitation was developed based on the marine and vehicular outage requirements, feasibility of reuse of components, redesign required to meet current AASHTO requirements and cost. The rehabilitation consisted of , mechanical work, mechanical support work and electrical work. The work consisted of jacking the counterweight, replacing the sheaves, trunnions, trunnion bearings, ring gears and pinions, counterweight wire ropes, installing temporary dead load plus live load supports and rehabilitating the existing live load shoes, and supporting electrical work. Construction support services were also provided in the form of reviewing shop drawings, installation and alignment procedures, wire rope tensioning report and alignment measurements. As of the writing of this paper the construction is substantially completed and the machinery alignment is on-going.

Timeline

The following timeline summarizes the significant findings and rehabilitation work at the Snohomish River Bridge West – 529/10W:

- **August 2002**: An in-depth mechanical inspection was performed by Stafford Bandlow Engineering, Inc. (SBE), including visual and dye penetrant inspection of the sheave trunnion fillets. The fillet at the trunnion shoulder is a critical area on the sheave trunnions. This area is subject to the maximum stress as a result of the applied load and the stress riser created by the radius at the transition from the journal diameter to the larger sheave fit diameter. In addition, the trunnions are subject to complete stress reversal during operation of the bridge. The combination of high stress and stress reversal is a concern with regard to fatigue cracking and ultimately the development of cracks at this location.

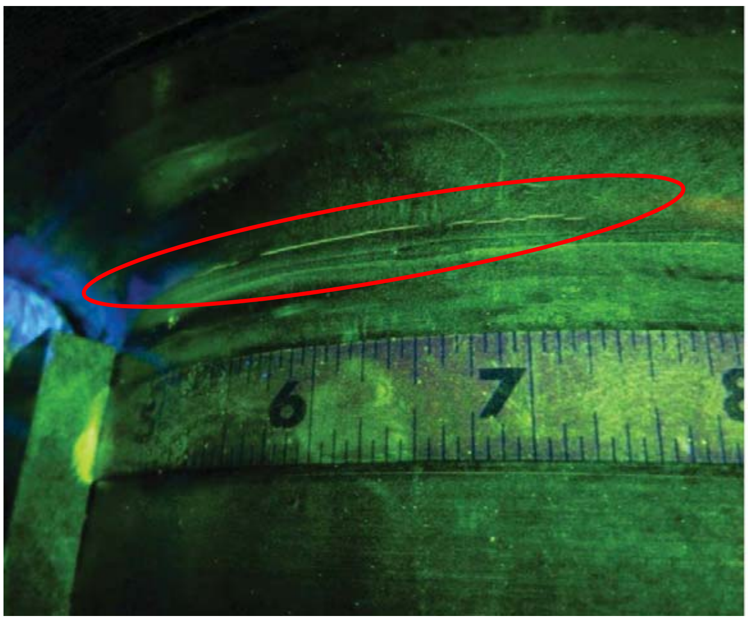
  To verify the integrity of the trunnions at the fillet area, the top 180 degrees of the fillet area was subjected to a dye penetrant test and close visual examination and the bottom 180 degrees was subjected to a close visual examination at all of the opened trunnion bearings. Access to the bottom 180 degrees of the fillet area requires raising the span and closing it to vehicular traffic. Traffic considerations did not allow for a dye check of the bottom 180 degrees of the fillet area. No cracks were found in the fillet area as part of this inspection

  Although dye penetrant and visual inspection provide some level of surety with regard to the condition of the fillet area, it is possible that cracks exist that were not picked up through these inspection methods. A more sensitive method of crack detection is wet fluorescent magnetic particle examination. This method of inspection is recommended to identify cracks in the early stages of development. Wet magnetic particle inspection was beyond the scope of this inspection.

  Calculations were prepared to determine the fatigue life of the counterweight sheave trunnions. The number of bridge operations over the life of the bridge was projected using the average number of openings per year based on data for the years 1979-2001 that was provided by WSDOT engineering personnel. All other information required to prepare the calculations was obtained from the original design drawings. The calculations indicate that the trunnions have a fatigue life of 19,732 cycles and that the actual number of cycles due to bridge operations is 34,632. Therefore the trunnions are well beyond their calculated fatigue life. Based on this information and our experience on other bridges we thought there was a high probability of finding fatigue cracks in these trunnions.

  The conclusion of this report was that although no cracks were found at the counterweight trunnions, using the methods employed during the 2002 inspection, **fatigue theoretical calculations indicate that the trunnions have exceeded the number of cycles to failure by 76%.** These calculations and our knowledge of calculated fatigue life and cracks in trunnions on other vertical lift bridges suggest that cracks are likely on this bridge. It was strongly recommended to perform follow-up inspection with more sensitive non-destructive examination such as ultrasonic testing of the fillet areas, including wet-magnetic particle inspection.

- **July 2008**: An in-depth mechanical inspection was performed by Scott Snelling, P.E. with non-destructive inspection, including through-bore ultra-sonic inspection and wet-magnetic particle inspection, of selected sheave trunnion fillets by Rob Gessel of Wiss Janney Elstner (WJE). Fatigue cracking in the sheave trunnion fillets was discovered. Several potential repair options were presented, including replacement of the sheaves, post-tensioning the sheave shafts, or in-place machining to excavate the cracks and increase the fillet radius, followed by peening. Calculations indicated that rehabilitation of the existing sheave shafts could add approximately 50 years of life before cracking would recur, but that infinite fatigue life was not possible with the existing trunnions.



CAPTION: Close up photo of fatigue cracks in the trunnion fillets as indicated during wet magnetic particle type non-destructive examination. Photo by R.Gessel of WJE

The original design of the subject sheave shafts, circa 1952 predated the application of stress concentration, metal fatigue and fracture mechanic principles within the AASHTO design codes. The current 2008 AASHTO LRFD specification is based on Soderberg fatigue failure theory. Fatigue calculations indicated a nominal bending stress at the fillet of 21 ksi. The 1938 AASHO, applicable at the time of design, has an allowable stress of 15 ksi for sheave shafts fabricated from heat-treated alloy steel. Therefore, the trunnions were apparently not designed in compliance with the standards in effect at the time. One explanation might be that the designer of the subject sheave shafts applied the 25 ksi allowable stress recommended by O.E. Hovey in his two-volume treatise on movable bridges, published in 1927.

The shafts in question were fabricated from ASTM A235 Class G, which is equivalent to ASTM A668 Class F, alloy steel forging with a heat treatment of quenched and tempered, with an ultimate stress of 82ksi. (For reference, using a typical safety factor of 1/5 used in the AASHTO 1988 specifications for other trunnion materials, the approximate allowable stress would be 16.4 ksi). Looking at the geometry of the shaft (D/d = 1.22, r/d = 0.055, r = 1/2 in) a stress concentration factor of 1.95 for a shaft with a shoulder fillet in bending (Kt) can be derived. From this the factored bending stress at the fillet was determined to be 42 ksi. Since sheave shaft rotations cause complete reversals in bending, the factored stress range is 84 ksi. Therefore the trunnions of the Snohomish River Bridge were the most overstressed, when compared to the eleven other known vertical lift bridges at which trunnion cracks have been discovered. Below is a list of a few of these bridges with known cracking, along with their factored stress ranges :

- Shippingsport Bridge in Illinois 75.7ksi Trunnion Collapse occurred
- Valleyfield Bridge in Quebec 56.2 ksi Trunnion Collapse occurred
- Carlton Bridge in Maine 72.8 ksi
- Duluth Aerial Bridge in Minnesota 55.2 ksi
- Calument River Bridge in Illinois 53.4 ksi
- PATH-Hackensack Bridge in New Jersey 44.7 ksi

Note that the stress ranges cited above include the stress concentration factor for both tension and compression. This is the typical practice when performing fatigue calculations. However, once the cracks have initiated and fracture analysis is being performed, standard practice is to only apply the stress concentration factor to the tensile stresses in the fillet and not the compressive stresses. Also, fracture analysis typically accounts for the decay of the stress concentration factor as the crack deepens.

The trunnions for Shippingsport Bridge in Illinois and Valleyfield Bridge in Quebec collapsed due to fracture induced by fatigue cracking at the trunnion fillet. At Valleyfield Bridge the factored stress range is lower than at Carlton Bridge, however the trunnion at Carlton Bridge did not collapse. An explanation for this can be that Valleyfield Bridge trunnions experienced more fatigue cycles than Carlton Bridge. Also note that temperature can affect the crack propogation if a crack has initiated. This is because the toughness of steel decreases with temperature.

The consequences of trunnion failure and collapse are very serious. As a worst case, risks include complete collapse of the bridge and associated potential loss of human lives. As a best case, the vertical-lift bridge risks being rendered inoperable for a months or longer while new sheaves are fabricated, most likely with the bridge closed to highway and marine traffic for the duration of the required repairs. The economic impacts of bridge closures to the local economy can be serious, depending on the location of the bridge and the availability of feasible detours. A failure of this serious nature would likely have political ramifications as well.

- **March 2010:** SBE worked with WJE to perform a complete non-destructive examination of all eight sheave trunnion fillets. The Northeast Inboard trunnion fillet location had the most advanced cracking, with continuous and intermittent cracking over 70% of its circumference. The maximum crack depth was estimated to be 0.125 inches. WSDOT maintenance staff used a “flapper wheel” to attempt to excavate the cracks, but the cracks were deeper than the amount of material that could practically be removed with a “flapper wheel” of approximately 1/16 inch. Dr. John Fisher performed material coupon testing and fracture analysis which concluded that if any trunnion cracks were allowed to extend to 0.5 inches deep, this would result in a safety factor of less than two against brittle fracture, rendering the movable span unsafe for operation. Dr. Fisher calculated that 4041 trunnion stress cycles, equivalent to 1585 movable span full openings (2.55 cycles per opening) could result in the cracks extending to thedepth of 0.5in. The average number of movable span openings in recent years was 600 each year, with a maximum number of

843 openings having occurred in 2005. In other words, WSDOT had about 2.5 years to design and implement a repair before it would be forced to lock the bridge in the open position and close the highway.

- **October 2010**: SBE worked with WJE to perform a follow-up non-destructive examination of the sheave trunnion fillets. No perceptible crack growth was indicated since the inspection six months earlier.

- **May 2011**: Scott Snelling, P.E. of Parsons Brinckerhoff and Norm Duke performed an in-depth inspection of the wire ropes. The 24 counterweight ropes dated from when the bridge was originally constructed in 1954. The wire ropes were 1-5/8 inch diameter 6x41 Warrington Seale construction with a fiber core. The ropes had moderate deterioration. The ropes had significant crown wear due to abrasion resulting in an estimated 9% reduction in ultimate breaking strength. Crown wear had increased measurably since the previous rope inspection in 2004. The calculated rope safety factors of 6.1 for direct static loads and 3.6 for dynamic loads did not comply with current AASHTO recommendations of 8.0 and 4.5, respectively. Light corrosion was found on the ropes underneath the old, hardened, accumulated lubricant. It was recommend to replace the counterweight ropes concurrently with upcoming sheave trunnion replacement work. Combining the rope replacement with the sheave trunnion replacement resulted in significant savings due to shared costs of the jacking and temporary support of the counterweights.

- **July 2011:** Parsons Brinckerhoff and SBE teamed to develop a set of plans and specifications to replace the sheaves, trunnions, trunnion bearings and wire ropes. The scope also included designing the temporary counterweight supports, temporary live load shoes, and evaluating the temporary stresses on the existing structure imposed by the temporary counterweight support. The new trunnions were designed such that they will have infinite fatigue life with a generous fillet radius, larger diameter and made with higher strength material. This dictated a redesign of the trunnion bearings to accommodate a larger diameter trunnion on the existing supports. As part of this design the sheaves were replaced with a new fabricated weldement design versus the previous cast steel sheave design. Also new ring gears and pinions were provided and the ring gears were pressfit into the fabricated sheave and the pinion was press fit onto the cross shaft.

- **March 2012:** Parsons Brinckerhoff worked with WJE to perform another non-destructive examination of the trunnion fillets using through-bore ultrasonic examination and wet-magnetic particle examination. Several new cracks were found, compared with the previous examination in October, 2010. The deepest crack was estimated to be 0.165 inches deep. Based on the March 2010 fracture analysis, there were 548 bridge openings remaining before a calculated crack depth of 0.5 inches was reached and the movable span would be categorized as “unsafe for operation.”

- **April, 2012:** The contract documents were put out to bid. The engineers cost estimate for the rehabilitation was $2.9 Million.

- **June, 2012:** PCL was awarded the contract with a low bid of $1.7 Million.

- **May, 2013:** The threshold of maximum movable span operations, based on the 2010 fracture analysis was reached. Parsons Brinckerhoff worked with WJE to perform yet another non-destructive examination. Parsons Brinckerhoff also worked with Dr. Fischer to update the fracture analysis, which estimated that an additional 1680 bridge openings were available before the cracks propagated to 0.5 inches deep and the movable span was designated “unsafe for operation.” Note that the estimated number of remaining bridge openings was larger after the 2013 analysis versus the 2010 analysis, this is due to difficulties with regards to estimating the rate of decay of the stress concentration factor with the depth of the crack. In 2010, Dr. Fisher used a conservative assumption. In 2013, the stress concentration decay could be less-conservatively calibrated based on the actual crack propagation rates measured in the intervening years.

- **September 2013:** Existing sheaves and ropes were removed and new sheaves and ropes were installed.

- **October 2013:** Construction was substantially completed and the machinery alignment is on-going.

Scope of the Rehabilitation

The scope of the rehabilitation was separated into three categories: mechanical support work, mechanical work, and electrical work.

Mechanical Work – Scope:
- Span Support Machinery: Replace the existing trunnions, sheaves, and trunnion bearings.
- Span Drive Machinery: Replace the existing pinion, ring gear, and coupling grids.
- Counterweight Ropes: Replace the existing counterweight ropes and pins, adjust tension in the new counterweight ropes.

Mechanical Support Work – Scope:
- Remove, rehabilitate and reinstall the live load supports.
- Provide and install temporary dead load plus live load (DL + LL) supports.
- Provide temporary counterweight supports
- Temporarily remove and reinstall existing machinery roof sections as needed.

Electrical Work – Scope:
- Create as-found electrical wiring diagrams for the rotary limit switch and position transmitter.
- Temporarily remove the rotary limit switch and position transmitter before the existing sheaves are removed.
- Replace the rotary limit switch and position transmitter after the new sheaves are installed.
- Protect electrical equipment, wiring, and conduits from physical damage during construction and damage due to weather while the machinery room roof is removed.

There were multiple factors that contributed to determining the scope of the replacement of the trunnions. These factors were the required marine and vehicular outage, the required redesign required to meet current AASHTO requirements and costs associated with all options. The option of replacing the trunnions only and salvaging the counterweight sheave and ring gear was considered. The advantage of this option is that the cost for manufacturing the parts would be less as the sheave, ring gear and pinion would be reused. The disadvantages of this option were as follows:

- The existing sheave hub thickness over the trunnion would be much smaller than recommended by AASHTO. This is because the new trunnion diameter through the sheave is larger by 2 ¼" then the existing.
- The removal of the existing trunnion from the existing sheave would add additional risk to the project as they have an FN2 fit and the removal may damage the bore in the sheave. Also a larger marine outage is required for replacing the new trunnions in the existing sheaves.
- Lastly a drop in replacement of the sheave assembly will require a smaller marine outage as the parts can be shop assembled and ready for installation prior to the start of the vehicular and marine outage.

Therefore the selected alternative was to replace the existing trunnions, sheaves, ring gear, pinons and trunnion bearings with new components of new design. This allowed the assembly to be assembled in the shop and essentially be a drop in replacement. With this approach the trunnions and sheaves can be fabricated ahead of time and the trunnions can be press fit into the sheave. Simultaneously, the ring gear can be fabricated and press fit into the sheave. Also the trunnion bearings and the pinions can be fabricated and stored prior to any interruptions to traffic at the bridge. Once all the parts are fabricated the bridge can be closed to vehicular and marine traffic as needed to remove the existing components and install the new components.

The main counterweight ropes were the original ropes installed when the bridge was construted. As part of this rehabilitation the counterweight ropes needed to be removed in order to facilitate the replacement of the sheaves. Considering that there is a potential of damaging the existing ropes in the removal and replacement process, wear found on the ropes and the service life of over 50 years it made sense to provide new wire ropes. Therefore the counterweight wire ropes and pin replacement was added to this rehabilitation. Once the new wire ropes were installed it was necessary to measure and adjust the tensions in the wire ropes such that they shared the load evenly and therefore this requirement was added to the scope.

In order to perform this sheave replacement work, it was necessary to temporarily support the counterweights in order to remove the dead load from the ropes and sheaves. Modern vertical lift bridge designs typically include provisions to temporarily support the counterweights directly from the bridge towers, typically using a steel pin and hydraulic jacks. The Snohomish River Bridge had no such provision. Therefore, the design of temporary counterweight supports was included in the scope of this project.

The four live load shoes for the lift span are all rocker-type with curved bearing surfaces. However, once the counterweight was to be jacked and the ropes removed, four rocker-type bearings is no longer a stable configuration. In addition, the shoes would temporarily be required to support the dead load of the movable span, in addition to the live load. The line-contact portion of the live load shoes would be overstressed by the added dead load. Therefore, the design of temporary dead load plus live load shoes was included in the scope of the project. In addition, the existing live load shoes were seized and in need of rehabilitation to free the shoes to allow rotation and provide for improved lubrication details to the pins to prevent recurrence of the seizing issue.

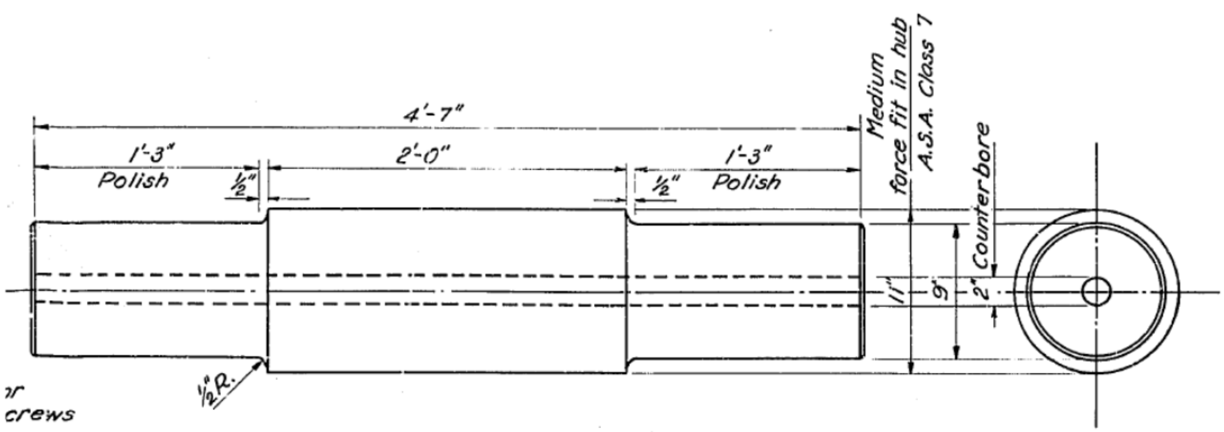
To allow for the removal of the existing sheaves and installation of the new sheaves, the temporary removal of the machinery room roofs and electrical limit switches were also required to be included in the contract documents.

Design Plans and Specifications

The development of the design plans and specifications began once the scope of the rehabilitation was finalized. This part of the work was a collaborative process with WSDOT. Milestones were established for various stages of development of the design plans and specifications. At each stage the design plans and specifications were reviewed by WSDOT and commented on and these comments were incorporated into the submittal. Below is a brief description of the components of this rehabilitation project and the improvements made from existing components.

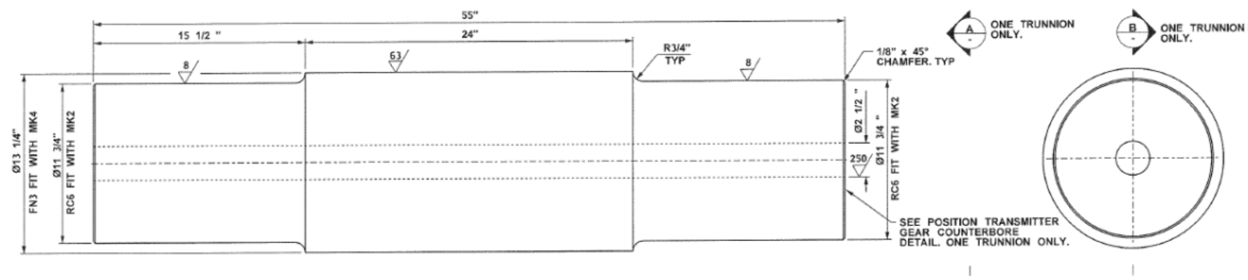
Span Support Machinery – Sheaves, Trunnions and Bearings

The rehabilitation of the span support machinery consisted of replacing the existing trunnion, sheave and trunnion bearings. The existing trunnions had fatigue cracks at the radius at which the trunnion transitions from the sheave fit to the bearing journal.



CAPTION: Design drawing of the original trunnion from 1953. The trunnion fillet radius is ½".

To prevent the new trunnions from cracking due to fatigue the new trunnion were designed to be larger in diameter,have a more generous radius at the transition from the sheave shrink fit to the bearing journal and were made of stronger material. The new trunnions were sized to meet the current AASHTO standards. The current AASHTO standard states that the trunnion shaft should have an infinite fatigue life. The new trunnion needed to be able to fit in the space constraints dicatated by the existing trunnion bearing supports. The new trunnion needed to have a 24in long trunnion hub and needed to have the same trunnion bearing spacing as existing. The new trunnion also needed to have an FN3 fit with the sheave to meet AASHTO requrements. All of these requirements have been added to the new trunnion as shown below.



CAPTION: Design drawing of the new trunnion. The trunnion fillet radius is ¾" and the trunnion is larger 13 ¼" vs 11" dia. and 11 ¾" vs 9" dia.

New sheaves were provided and meet the requirements of the current AASHTO standard and fit in the existing space constraints. The existing sheaves were made from steel casting; however few foundries remain in the USA capable of producing a ten-foot diameter casting. However, many fabricators continue to have the capability to fabricate a sheave of welded construction. Therefore a welded sheave design was developed to fit into the available space and match the critical geometric demission of the existing structure. The new sheave is designed to take the loads imparted on it, accommodates the internal gear ring gear and has a larger hub than the existing sheave to meet AASHTO requirement. It also has an FN3 fit with the trunnion and three 1 ½" pressfit dowels.

CAPTION: Original sheave, trunnion, and trunnion bearing.

CAPTION: New sheave, trunnion, and trunnion bearing.

The sheave trunnions are simply supported by two sheave trunnion plain bearings. To accommodate the new trunnion new trunnion bearings were developed that fit on the existing trunnion support and fit the new trunnions. The trunnion bearing base bolts were oversized so that the new bolts will have a turned bolt fit with the trunnion bearing support. The bearing bushing was designed to have axial lubrication grooves on the bottom half of the bushing and spherical lubrication grooves on the top half. The bottom lubrication grooves had a lube port and a purge port to allow for the old lubrication to be flushed through the bearing. The inboard edge of the bushing was provided with a larger chamfer to accommodate the larger radius on the trunnion.

Span Drive Machinery

To facilitate the drop-in replacement of the sheaves and trunnions, a new ring gear and pinion were added to the rehabilitation scope. The new ring gear and pinion were designed to meet the current AASHTO standards. AGMA spur gear design calculations in bending and pitting were performed to design the gearing as required by AASHTO. The gearing was designed to have a tip relief that will compensate for any deflection of the teeth at load and any manufacturing errors. The ring gear was designed to have a 0.005” to 0.010” interference fit with the sheave and have 32 – 1” diameter turned bolts that secure it to the sheave. The ring gear was made out of a ring forging that was 115” in diameter. The pinion was made out of a solid steel forging and was shrink fit on to the existing cross shaft. The existing pinion was removed from the existing cross shaft by torch cutting the pinion at the keyway to avoid damage to the shaft. At the other end of this cross shaft a grid coupling connects it to the output shaft of the reducer. The grids of this coupling were replaced as part of this rehabilitation.

A critical part of this rehabilitation was aligning the new machinery without moving the high sped end of the existing drive machinery. The intent was to return the pinions to the existing location and move the sheaves as required to obtain the desired alignment. As always the task of aligning large machinery is difficult and was challenging on this project for the following reasons:-

1) The alignment of both the pinion and rack are affected by the deflection of the tower under load and this deflection is not known until the full dead load is applied to the sheaves at which time adjustment to correct alignment are not practical. Therefore making alignment adjustments is an iterative process that involves loading and unloading the sheaves to obtain an acceptable alignment.

2) Due to the design of the ring gear and pinion, access to measure tip clearance and backlash at one end of the pinion is very limited and precludes conventional measurement methods.

3) As mentioned above, due to the limited scope of this rehabilitation, the Contractor was not allowed to move any of the existing remaining machinery. This necessitated that the Contractor control the alignment of the sheave trunnion assembly as well as the alignment of the ring gear to the cross shaft pinion without moving the position of the cross shaft pinion.

To help the Contractor align the new machinery with existing machinery given the above noted issues the following specification requirements were added to the contract:

1) The specification stated that “the alignment of the counterweight sheave trunnions relative to each other and relative to the survey line establishing the position of the existing trunnions is of secondary importance to the trunnion bearing alignment requirements and to the ring gear and pinion alignment requirements provided hearin. As such the alignment of the trunnions shall be recorded but will not dictate the final position of the trunnions.” This requirement prioritized the alignment of the trunnion bearing and the ring gear and pinion over the alignment of the counterweight sheave trunnions relative to each other and relative to the survey lines, hence giving the Contractor some leeway in aligning the sheaves.

2) The Contractor was advised that deflection of the bearing supports due to the counterweight load transferring from the temporary counterweight supports to the sheaves will affect the

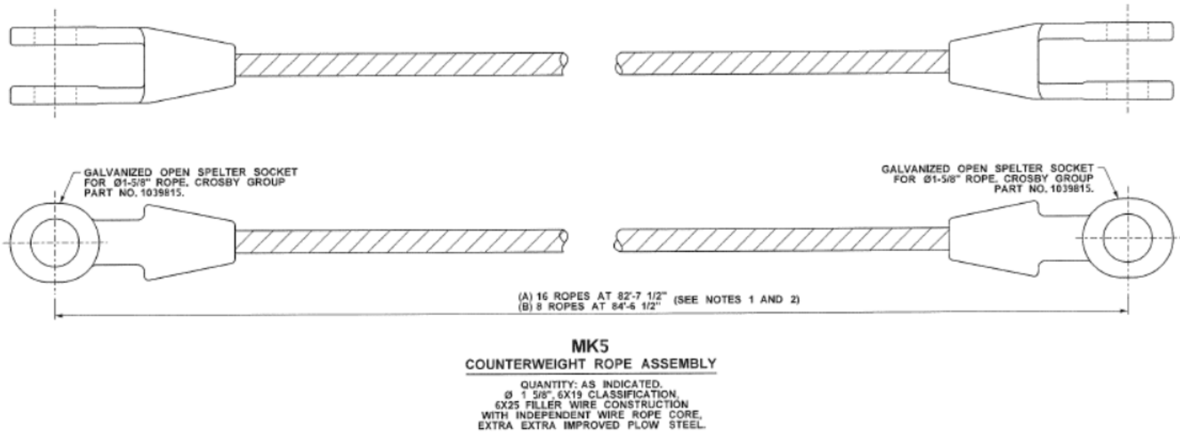
trunnion bearing alignment and that it may be necessary to jack the counterweights using the temporary counterweight supports and adjust the alignment or shims of the trunnion bearings multiple times to achieve the indicated alignment.

3) The specifications also required the Contractor to perform a detailed survey to locate the position of the existing machinery before and after the counterweight was jacked. This survey helps the Contractor predict the effect of the change in position of the machinery due to the counterweight being jacked and unjacked. This allows the Contractor to compensate for deflection when installing the machinery. The following were the requirements of the survey:-

- The alignment of the existing cross shaft, existing trunnion shafts and the existing ring gear and pinion shall be established.
- Permanent reference marks shall be established at the outboard side of each trunnion that can be relocated after the new trunnions are installed. This will be the reference line.
- The trunnion centerlines shall be located at both the inboard and the outboard side of each trunnion to a measurement accuracy of 0.002".
- The cross shaft shall be located with reference to the reference line at two locations 100 ft apart and measured to an accuracy of a 1/32".
- The ring gear and the pinion alignment shall be measured as follows:-
    - Tip clearance measured to an accuracy of ±1/64"
    - Axial alignment measured to an accuracy of ±1/64"
    - Backlash measured to an accuracy of ±0.002"
    - Gear tooth contact measured as determined by bluing at 4 locations 90 degrees apart and the length of contact shall be measured to ±1/16"
    - The trunnion bearing alignment with respect to level shall be measured. The bearing caps were removed at this bridge and hence this was possible by measuring at the bearing split. The level of the bearing base was measured in two directions. One is in the axial direction and the other is on the direction perpendicutlar to the axial direction. Precision blocks and level were used to step over the journal and the key in the base.

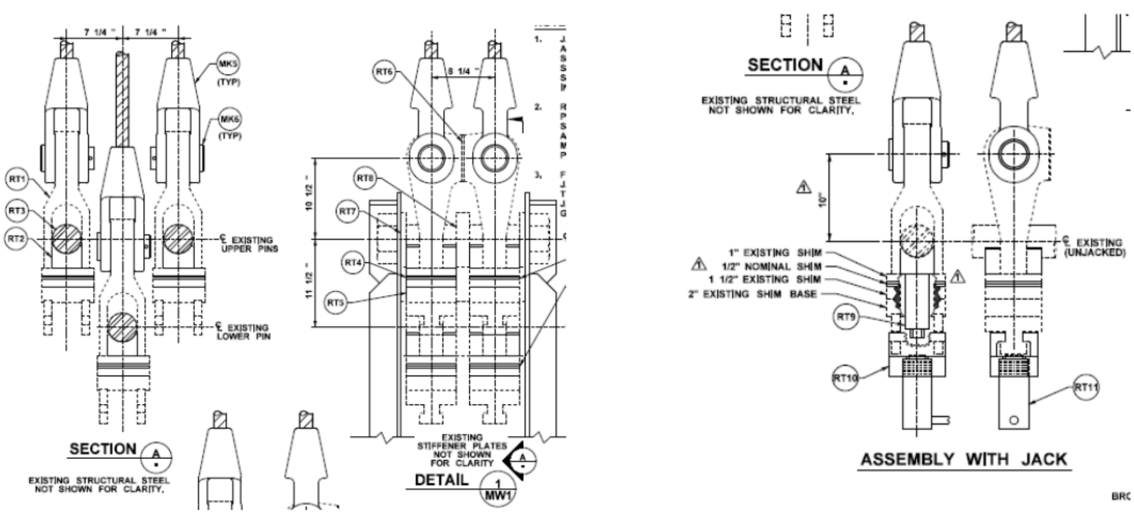
Counterweight Ropes

As part of the inspection of the counterweight wire ropes it was determined that they exhibited moderate wear. The existing counterweight ropes were the ropes originally install on the bridge in 1954. These factors combined with the savings associated with replacing the counterweight ropes along with the sheaves versus replacing the ropes as part of a separate contract dictated that the rope be replaced as part of this work. This bridge has a total of 24 counterweight wire ropes and 6 wire ropes on each sheave located at each corner of the bridge. The existing wire ropes were 1 5/8" diameter 6x41 "M", Purple, Regular Lay, Fiber Core ropes with a breaking strength of 214,000 lbs per rope. The replacement ropes were 1 5/8" 6x25 filler wire construction with independent wire rope core extra extra improved plow steel with a breaking strength of 292,000 lbs per rope. These new ropes are 36% stronger than the existing ropes which meets the direct load requirement of 2007 AASHTO. The ends of the ropes were fitted with Crosby Group galvanized open spelter socket. See below drawing.



CAPTION: The new counterweight wire rope.

The existing wire rope terminations were reused and included a system where adjustments to the effective length of the ropes can be made by adding or removing shims. See below.



CAPTION: The counterweight wire rope terminations which include a system of adjusting the effective length of the ropes.

The system works by jacking individual ropes using a hydraulic jack against a fixed pin on the structure. This removes the load from the shims and allows for the removal or addition of shims which effectively changes the length of the rope. Through this system the load taken by each rope can be changed and the load can be distributed evenly on all the ropes. This method of adjusting the load taken by each rope along with a method of measuring the rope tensions using their fundamental frequency was required for this project. The specification required that the rope tension at each rope group in all four corners shall be within ±5% of the average tension in each rope group. This work would ensure that all the ropes will equally share the load and decrease the changes of premature wear due to overloading.

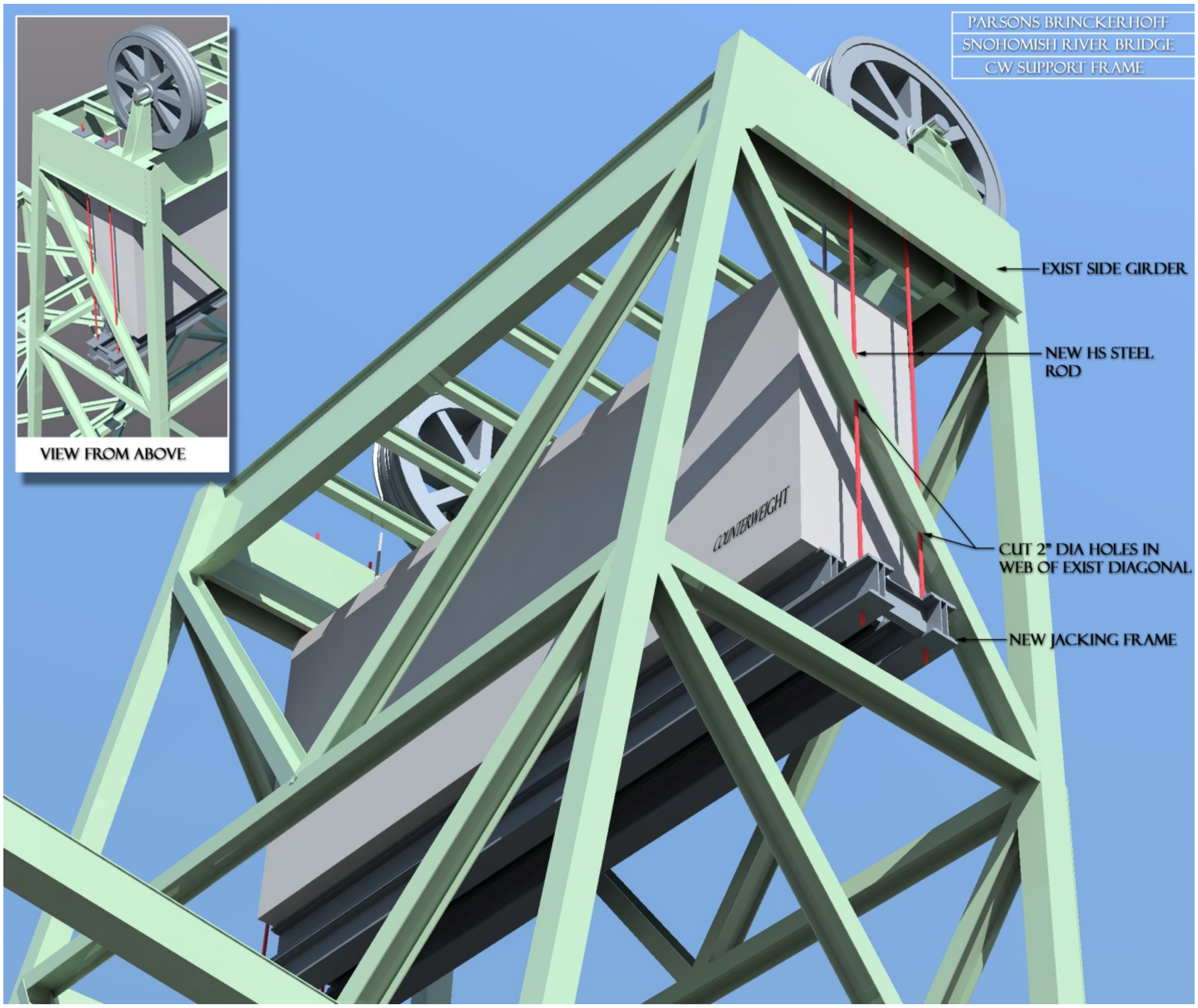
Determining the length of the new wire ropes that will replace the existing wire ropes should be considered carefully. This is because the new wire ropes have a different constructional and elastic stretch as they are of different construction than existing. Constructional rope stretch is the permanent increase in rope length that occurs over years of service. Elastic rope stretch is the increase in rope length that occurs while under load. The Contractor measured the lengths of these ropes and they were on average 4 ¼" longer than as noted on the original shop drawings. This stretch is constructional stretch as the original ropes were measured under load to the lengths noted on the shop drawings. Therefore the temporary counterweight jacking system needs to accommodate the difference in constructional and elastic stretch and the stretch due to service. Once it was determined that the jacking system could accommodate the required momement without any interferences, the new rope lengths were recommended to be the same as the length of the existing ropes noted on the shop drawings.

Temporary Counterweight Supports

The original design of the bridge did not include any provisions for the temporary support of the counterweight in order to facilitate unloading the existing ropes and sheaves for replacement. Therefore, the design of new temporary counterweight supports was included in the scope of this project.

Each of the two counterweights weighs approximately 400 kips. The counterweights are concrete with a structural steel frame inside. Since there is a sidewalk within the lift span through-truss, there are corresponding hollow-cavities on the West side of the counterweights.

Parsons Brinckerhoff developed a 3D model of the existing tower in order to facilitate the preliminary design process for the new temporary counterweight supports. After evaluating the geometry and capacities of the existing tower members, it became apparent that the existing side girder members had the capacity to temporarily support the counterweight, with the ability to transfer the temporary loads to the existing tower columns without any strengthening. See the image below showing the configuration that was proposed as a preliminary design.



CAPTION: Preliminary 3D Model of the Temporary Counterweight Support Design – Early in the Design Process (Existing Wind Guides and Temporary Lateral Support Not Shown)

Key attributes of the temporary counterweight support design included redundancy of the high strength rods and the ability to distribute the point loads imposed by the high strength rods. The quantity of four high strength rods used to provide temporary support, was the maximum feasible quantity, due to the limited geometric space envelope provided due to the close proximity of the tower diagonal and horizontal truss members. Given the opportunity, additional rods would have been used to provide additional redundancy. However, even with only four rods, it was possible to design for redundancy even in the extreme case of the complete failure of one rod. Note that the failure of one rod would result in zero load in the rod on the opposing corner and a doubling of the load in the other two remaining rods.

CAPTION: Counterweight temporarily supported during construction with ropes removed. From top to bottom, note: (1) machinery room roof partially removed with tarp for temporary weather protection, (2) machinery room wall cut to make room for bar jacks, (3) temporary lateral support, (4) temporary counterweight support beam.

Due to the tall and slender profile of the existing counterweight, as well as potential instability related to the considered rod failure case, it was necessary to also provide temporary lateral supports to prevent overturning of the counterweight. The lateral supports were conservatively designed to support 25% of the vertical dead load, which approximately translates to a rotation of 14 degrees from plumb.

The lateral supports were required to provide this overturning restraint while the counterweight was being jacked up, while the existing ropes were removed, and while jacking the counterweight down to transfer the lift span dead load to the new ropes. The total required vertical movement of the counterweight supports was conservatively on the order of two feet, to accommodate the constructional stretch of the existing ropes, as well as the elastic stretch of both the existing and new ropes.

The lateral support design was a steel weldment that was anchored the side of the counterweight and had slots allowing the high strength rods to pass through. The permanent wind guides for the counterweight were left in place and continued to provide additional support. The temporary counterweight supports did not directly utilize any of the adjacent existing tower diagonals to support loads. However, designing around the adjacent existing tower diagonals to avoid interferences was a key component of the design process.

As an additional measure to ensure that the counterweights remained plumb during jacking and were not allowed to rotate, the specifications required that the Contractor monitor and record, at one-minute intervals, the hydraulic pressures and correlated load in each jack, as well as the elevation and levelness of the counterweight.

During design, it was considered that the selected Contractor would be likely to propose design changes to the temporary supports in order to use on-hand materials. In fact, during the shop drawing phase, the Contractor proposed to use an alternative configuration for the jacking beam, while retaining the general configuration with four high strength rods operated by four hydraulic bar jacks. When evaluating the Contractor's proposed alternative design for the jacking beam, there was a focus on maintaining the key attributes of redundancy and the ability to distribute the high point loads imposed by the high strength rods.

Permanent Live Load Supports

CAPTION: Live Load Shoes disassembled in the shop

Unlike a typical simple span bridge with two fixed bearings and two free expansion bearings, each of the four existing live load shoes for the Snohomish River Bridge were rocker type to allow for free expansion. However, the pins were seized and no longer allowed for expansion. In addition, the grease grooves were plugged and were no longer accepting lubrication. Therefore, the scope of this project included removing the existing live load shoes, disassembling, improving the bearing and lubrication details, and re-installing the refurbished shoes.

Temporary DL + LL Supports

The existing live load shoes did not have the capacity to support the dead load of the lift span when the counterweights were jacket and the ropes were removed. Specifically, the overload that occurred was in the line contact created where the curved contact surface of the live load shoes meet the strike plates. The suggested construction sequence in the contract documents was to complete the rehabilitation work at one tower before proceeding to the next tower. Therefore, the new temporary DL+LL shoes were designed as a simple pedestal, with no capacity for free expansion.

CAPTION: Temporary DL+LL Shoe installed on the bridge.

During the shop drawing phase of the project, the Contractor instead proposed to simultaneously perform work at both towers, which necessitated alterations to the DL+LL shoes to allow for free-expansion. In the end, this was accomplished using PTFE (aka. Teflon) sheeting between the shoes and strike plates.

Temporary Removal of Existing Machinery Roof Section

Incidental to the sheave replacement work, it was necessary to temporarily, partially remove portions of the steel machinery house roof and walls to provide access.

Electrical Work

Incidental to the sheave replacement work, it was necessary to temporarily relocate existing limit switches and provide protection to existing electrical equipment.

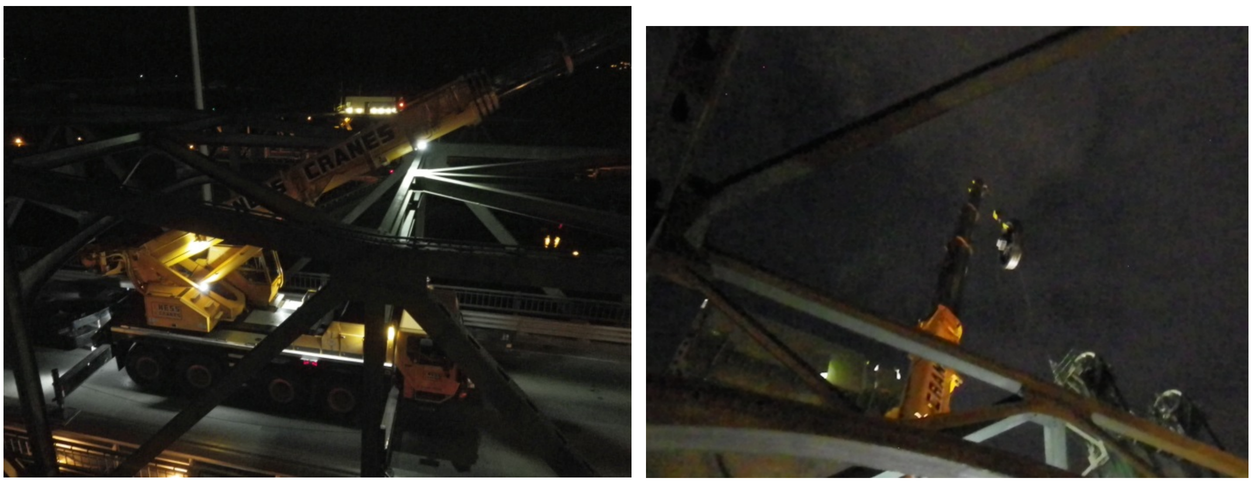
Construction Services and Highlights

This rehabilitation included replacement of large components, supporting heavy counterweights, precession manufacturing and installation of machinery and related work. To help contribute to the success of this project Parsons Brinckerhoff and SBE provided construction services for reviewing shop drawings, requests for deviation, request for information, work procedures, survey procedure and data and other submittals as requested by WSDOT. The goal of this review was to ensure that the Contractor was meeting the contract drawings and specifications and will be able to achieve the objectives of the project.

To perform the construction work, the contract documents allowed for two closure periods of one-weekend-long each to both vehicular traffic and marine traffic. In addition, two one-week-long closures to marine traffic were allowed. Balancing the acceptable length and season for the bridge closure periods was a key component of the project. Shorter closure periods would be physically possible to construct, but would increase the risk to the Contractor and therefore would be expected to increase the bid prices and cost to the State. Longer closure periods would place added burden on the traveling public.

Early in the design phase, WSDOT performed public outreach to bridge users, including the local mariner association as part of the effort to determine what length of closure periods, as well as which seasons were acceptable for closure. The peak mariner summer season, as well as falcon nesting periods in spring were determined to be un-acceptable seasons to perform the work. Winter season allowed for longer bridge closure periods without unduly interfering with marine traffic. An Incentive/Disincentive clause in the contract documents provided for financial rewards if the Contractor completed the work with shorter closure periods and financial penalties if the Contractor was not able to complete the work within the allowed windows. For this project, the Contractor performed the work within the allotted time, with no time related penalty or rewards realized.

During the design phase it was assumed that the replacement of these sheave trunnion assemblies will require the rental of a large barge mounted crane. This was a large part of the engineering cost estimate. However the Contractor was able to prove that this work can also be done using a boom crane that is setup on the span. See below.



CAPTION: The Contractor used a boom crane that is setup on the span to replace the sheave trunnion assemblies.

Being able to lift the sheave trunnion assemblies from roadway to tower level by the use of a rubber-tired boom crane from the bridge deck instead of a barge mounted crane allowed the Contractor to save a significant amount of construction cost.

Conclusion

The Snohomish River Bridges (529/10W) is a tower drive vertical lift bridge located in Everett, Washington and was built in 1954. This bridge is owned and maintained by the Washington State Department of Transportation (WSDOT). As part of maintaining the bridge WSDOT performs periodic in-depth inspections of this bridge. During one of these inspections, non-destructive testing was used and it was found that the main counterweight sheave trunnions exhibited cracks at the fillet area where the trunnion transitions from the sheave hub shrinkfit to the journal. These cracks form because the trunnions are subject to heavy loads and experience full reversal of those loads and the original design parameters did not include the consideration of fatigue. These cracks are due to fatigue and can increase in severity as the bridge is operated and can cause the trunnion to fail.

As these fatigue cracks were discovered, they were further documented by performing nondestructive testing. Dr. John Fisher was consulted to determine the severity of these cracks and he predicted that the cracks will grow and corrective action is necessary in the next couple of years. Several potential repair options were presented, including replacement of the sheaves, post-tensioning the sheave shafts, or in-place machining to excavate the cracks and increase the fillet radius, followed by peening. Calculations indicated that rehabilitation of the existing sheave shafts could add approximately 50 years of life before cracking would recur, but that infinite fatigue life was not possible with the existing trunnions. Therefore a set of contract plans and specifications were developed for the replacement of the main counterweight sheave assembly, counterweight ropes and other supporting work.

The new main counterweight trunnion was designed to fit in the same space envelope, however it was designed to meet 2007 AASHTO standards and has an infinite fatigue life. To accomplish this, the trunnion diameter was increased, stronger material was used and larger fillet radius was provided. This necessitated new sheave, ring gear and pinion. The partial replacement of the machinery required that the new machinery be aligned with the existing remaining machinery without moving it. Also the replacement of the main counterweight sheaves required that the main counterweights be jacked and supported from a different location than the sheaves. This causes the defection of the towers to change moving the sheave assemblies depending on if the load is transferred through the sheaves vs if the load is transferred through the temporary counterweight supports. To help the Contractor achieve the proper alignment in these challenges, the Contractor was asked to perform an extensive survey of the existing machinery prior to and after jacking of the counterweight. The Contractor was also advised that achieving the proper alignment of the machinery may require jacking the counterweight multiple times.

The main counterweight ropes were of original construction and had moderate wear and hence were also replaced at this time due to the savings in replacing them along with the sheaves. The new ropes are 36% stronger than the existing ropes which meets the direct load requirement of 2007 AASHTO. The existing wire rope terminations were reused and included a system where adjustments to the effective length of the ropes can be made by adding or removing shims. Using this adjustment method and measuring the rope tensions allowed the adjustments of the rope tensions at each rope group in all four corners to be within ±5% of the average tension in each rope group.

To facilitate this work, supporting structural and electrical work was necessary. This included installation of temporary support of the counterweight in order to facilitate unloading the existing ropes and sheaves for replacement, temporarily removing sections of the tower house, and temporarily removing electrical equipment and conduit. It was also necessary to remove the existing live load supports and send them to the shop for rehabilitation. Therefore a set of temporary DL+LL supports were provided that would take the loads when the counterweights were jacked.

HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

WELLS STREET BASCULE BRIDGE CONSTRUCTION ENGINEERING

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Walsh Construction Company

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Introduction

The Chicago Department of Transportation's (CDOT) Wells Street Bascule Bridge is located on the Main Branch of the Chicago River in downtown Chicago. The bridge is a two level, Chicago style trunnion bascule bridge that carries the Chicago Transit Authority's (CTA) Brown and Purple Transit Lines on the upper level and vehicular traffic on the lower level. The two CTA transit lines carry over 70,000 commuters a day over the bridge and the roadway is used by over 15,000 vehicle a day, has a bike lane and many thousands of pedestrians.

In 2012, CDOT awarded a $41.2 million contract to Walsh Construction Company (Walsh) to perform a major rehabilitation of the bridge. The project work included major rehabilitation to the movable trusses (including complete replacement of the first five bays of each truss starting at the centerbreak), replacement of the upper and lower floorbeams and stringers, replacement of the bridge sidewalks, extensive counterweight concrete and steel repairs, new roadway and track level decks, replacement of the bridge electrical and control systems and rehabilitation of the bridge houses. The contract provided two nine day windows in which the CTA transit service could be shut down to allow the replacement of the portions of the truss arms and to complete the CTA level structural steel and trackwork. Walsh retained Collins Engineers (Collins) to perform construction engineering associated with the rehabilitation and replacement of the trusses and bridge balancing to maintain operations.

History

The history of movable spans at Wells Street dates back to 1841 when a floating bridge was used to span the river. This span was subsequently replaced on three occasions with swing type structures. The last swing span was completed in 1888. Initially a single level structure, this bridge was converted to a two level swing bridge in 1896 to accommodate the newly built Northwestern Elevated Railroad (a forerunner company of the CTA) tracks entering Chicago's Loop. It is perhaps interesting to note that two famous individuals were involved in the construction of the Northwestern Elevated Railway – Charles Yerkes who developed the lines and J.A.L. Waddell who designed the track supporting structure[1].

At the turn of the twentieth century, the United States Department of War deemed all swing bridges on the Chicago River a hazard to navigation and ordered their removal[2]. This order affected both the Wells Street and Lake Street Bridges, both of which carried transit trains on their upper levels. The order gave rise to the Chicago Department of Public Works' development of the Chicago Type trunnion bascule bridge. The order also posed a challenge to the City's Chief Bridge Engineer, Thomas G. Pihlfeldt. It fell to Pihlfeldt to develop a means to replace the Lake and Wells swing bridges with new bascule bridges without prolonged closures that would impact transit operations.

For the Lake Street Bridge in 1909, in a twelve hour period the bridge carried 3.180 motorized vehicles, 1,000 elevated trains, 850 horse teams and 7,000 pedestrians[3]. The procedure that Pihlfeldt developed to replace the swing bridge consisted of building the new bascule leaf around the ends of the swing bridge with the bascule leaves in the open position, then, as construction neared completion, cut the old swing bridge away, lower the new bascule leaves and complete the remaining construction work with all work completed within a week[4].

In the case of the Wells Street Bridge, Pihlfeldt sought to reduce the duration of the bridge closure even further. The process of construction for the Wells Street Bridge replicated that of Lake Street with one major exception. Starting on Friday December 2, 1921, crews stopped the transit trains and roadway traffic over the existing Wells Street swing bridge and worked around the clock to remove the existing swing span, lower the new bascule leaves and complete the construction of the upper level track and lower level roadway structures. Amazingly, Pihlfeldt was able to complete the work in time to allow resume the elevated train service for the Monday morning rush hour[5].

Wells Street Rehabilitation

Since the completion of the "new" Wells Street Bridge in 1922, there has been little in the way of rehabilitation or repairs to the bridge. Work has included installation of open grating decks to replace the old timber decks, updating of the bridge controls to allow for one person operation and other minor repairs. This is a testimony to the durability of the design of the original bridge.

After over ninety years of reliable service, CDOT realized that the Wells Street Bridge needed a major rehabilitation. The urgency of the need for rehabilitation was revealed during a routine inspection where the truss bottom chord was found to be heavily deteriorated due to road salts. Emergency repairs were completed on the truss chord, but they were interim in nature and the preparation of plans for the complete rehabilitation of the bridge were advanced. The work included the complete replacement of the upper track level floorbeams and stringers supporting the CTA and the lower level deck, floorbeams and stringers as well as a substantial portion of each truss arm. Complete replacement of major portions of the truss arms was the only satisfactory option to effectively rehabilitate the truss arms due to the level of deterioration.

The challenge that CDOT was presented with was how best to perform a major rehabilitation of the bridge truss arms while minimizing the impact to the CTA's train service that crossed the bridge. Pihlfeldt had the advantage ninety years ago of having an existing bridge that could remain in service while he built a new bridge around it. CDOT's design team had no such option. The work had to be staged in such a way that it could be completed within short duration closures of the CTA. Fortunately, CTA had scheduled two major track outages on the lines that crossed the bridge in order to perform needed track work. CDOT and CTA agreed to two nine day service outages over the bridge which allowed CTA to complete needed trackwork and CDOT to replace a substantial portion of the bridge truss arms. The outages were fixed windows spaced about two months apart. The challenge to the successful contractor performing the work was to ensure that the advance work was completed on time to meet the track outages and that when those outages occurred, the work could be completed within the nine day closure. CDOT set heavy penalties of $250,000 per day for failure to complete the work within the track outage windows.

Construction Engineering

Collins was retained by Walsh to provide pre-bid and construction engineering services for the project. The bid documents provided a suggested methodology for the replacement of the truss arms. The suggested methodology included the float in of a portion of the truss arm with much of the floor beams

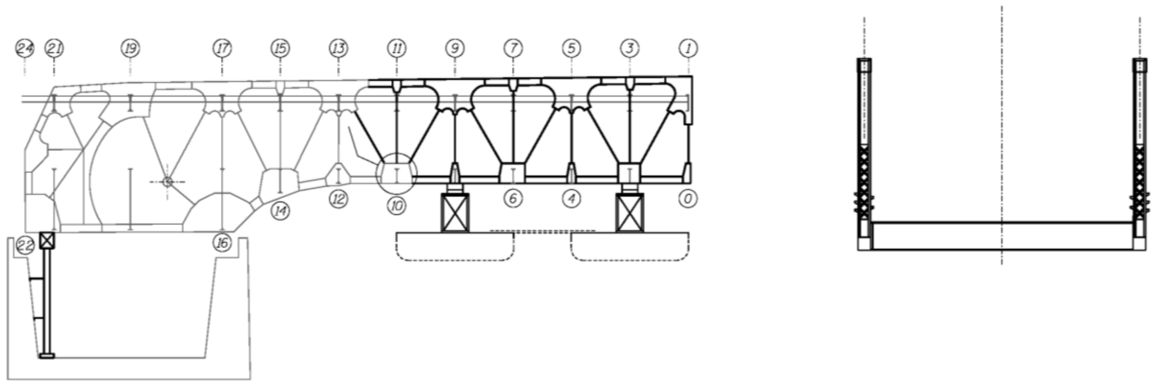

Figure 1: As designed configuration of truss arm float in. Elevation and Section

and bracing omitted. The intent was to ensure flexibility within the truss arm assembly in order to allow the connection of the new to existing truss since it was felt that making four truss points between the new and existing truss arms would be difficult to achieve.

During pre-bid, Collins reviewed the proposed scope of steel work with Walsh to determine the best way to maximize the pre-assembly of the truss prior to installation and still ensure that the work could be completed within the nine day track closure windows. Collins and Walsh agreed that it would be feasible to float in the entire truss thereby minimizing work during the track outage. Collins also reviewed the shoring of the counterweight and the other repairs principally to the truss arms that remained to determine if the work could be safely performed while CTA rail traffic used the bridge. These insights allowed Walsh to refine their bid and allowed Walsh to quantify the risks associated with the work.

Truss Modelling

From experience with other bascule bridge rehabilitations, Collins' first task was to develop an analytic model for the truss. This model was essential to providing CDOT with stresses in the truss associated with various stages of construction and also to determine the deflections of the truss arm at truss float out or float in. Collins modelled the truss in STAAD. Members were modelled as line elements at the neutral axis of the member. The Wells Street Bascule bridge is different from most typical bascules in that the bridge has heel locks at the back of the counterweight box handle the roadway and railroad loads that occur between the trunnion and the rear of the bridge. Typical bascule bridges try to keep the roadway of the movable span ahead of the trunnion so that all of the live loads are taken by the live load bearings. The heel locks complicated the analysis since live loads had to be accounted for in the analysis of load both ahead and behind the trunnion. The model was used to determine the members required to safely support CTA live loads during portions of the rehabilitation work since one of the objectives of Walsh was to minimize the work required at the bridge site in order to reopen the bridge to CTA traffic. Finally, many existing truss members to remain were slated for repairs. The model was used to determine whether or not doubler members were required for the repair of compression members and to rehabilitate the bottom truss chords under CTA live load between the trunnion/live load and the counterweight.

Truss Float-Out/Float-In

After contract award, Collins worked with Walsh to finalize the approach to be used for the existing truss float out and new truss float in. Walsh established a yard on Chicago's near South Side along the South Branch of the Chicago River to be used for staging the work. Working with Walsh, Collins developed a procedure to pre-assemble as much of the truss arms as possible on the barge for float in to the bridge site (Panel Points 0-10 of the bottom chord and 1-9 of the top chord). We recommended that Walsh perform a detailed three dimensional survey of the existing truss arm matching work points at Panel Points 10 on the bottom chord and 9 on the top chord and that these work points be transferred over to the make-up points of the new truss arms. Only with accurate three dimensional survey could we ensure that the new field assembled trusses would mate up with the existing trusses.

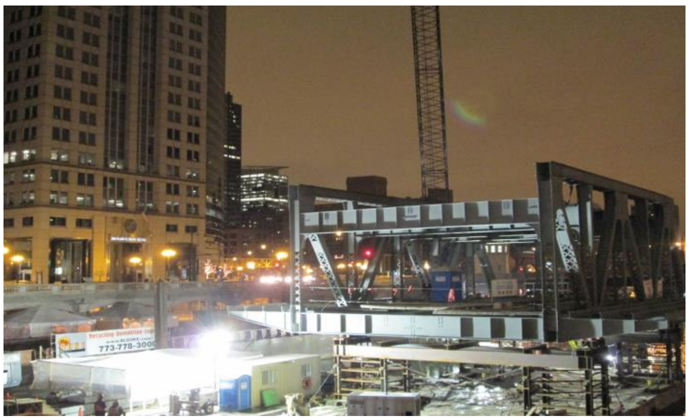

Figure 2: As erected new truss arm assembly awaiting installation

As part of the truss float in and removal, Collins also verified the stability of the barges with the truss loads. Metacenter calculations were run for the truss assembly float out as well as float in and ballasting scenarios developed to ensure that the elevations of the new truss assembly could be adjusted to match the existing. Since the existing portion of the truss arm assembly was to be burned free from the truss arms to remain, the calculations were complicated by the need to ensure that the loading of the barge with the existing truss assembly did not result in a sudden load transfer when cut free. The ultimate arrangement required that the nose of the truss be fully supported by the barge through ballasting with an additional upward force so that when the existing truss arm assembly was cut free, the truss assembly would raise slightly to accept the full load. The float out barge was equipped with container boxes reinforced with steel supports to distribute the load of the truss. This arrangement relied solely on the ballasting of the barge to support the load and there was no other provision, other than barge ballasting, provided to adjust the elevation of the truss support shoring. The float in barge was provided with adjustable shoring towers with centrally ported jacks that were used to fine tune the elevation in combination with barge ballasting.

Helium-acetylene torches were used to cut the existing truss free. Prior to shutdown, Walsh removed the roadway level deck, floorbeam, laterals and stringers on either side of Panel Point 10 where the truss arm assembly was to be cut. Additionally, Walsh prepped the gussets at Panel Points – and – by removing the existing rivets and installing black bolts to facilitate replacement of the gussets and final fit up. Work began with the replacement of the south leaf truss arm assembly. After shutdown of the CTA at 2200 hours Friday evening, the north leaf was raised and locked out. Work began on the south leaf to remove the CTA electrified contact rail and running rails and demolition of the track supporting structure floor beam and associated stringers at Panel Point 13. All prep work associated with the truss arm assembly removal was completed by Saturday afternoon and work began on the cutting of the truss arms. Cutting began on the top chord and diagonals to allow the nose of the truss to transfer load to the barge. As the load transferred to the barge, water was pumped out of the barge compartments near the nose to allow more load to be picked up by the barge. The truss arm assembly was fully removed by late afternoon on Saturday and floated off for further demolition.

Figure 3: Cutting existing truss chords

Work continued around the clock preparing the connection points for the new truss assembly. Collins developed guide support beams to be used to guide and support the new truss arm assembly until the connections to the existing truss could be made. Collins also provided Walsh with the minimum number of bolts required per connection in order to cut the float in barge free. By prior arrangement with CDOT, Walsh was allowed to sub-drill the fit up holes required to support the dead load of the truss arm assembly. This significantly sped up the bolt up of the truss arm assembly. All other bolts for make-up were drilled full size using the gusset plates as templates. In total, over 4,000 bolts were required to fully make up the connections between the new and existing truss arms.

Figure 4: New to existing truss arm connection at gusset plate

The existing truss was ready to accept the new truss arm assembly by Monday afternoon. The ironworker crews arranged a winching scheme to guide in the truss arm assembly without the benefit of needing the barge tug. This allowed for accurate alignment of the truss using three dimensional survey. The bottom chord was aligned and leveled with jacking to allow bolting to begin. Concurrent with the bolting of the bottom chord, diagonal member 10-13 and a “tee” section consisting of the top chord between Panel Points 9 and 13 with plumb post 10-11 was flown in by crane and made up the new truss arm assembly. The new truss arm assembly was then adjusted in elevation and alignment to ensure fit up the top chord connections. The procedure was repeated for the north leaf approximately two months after the rehabilitation of the south leaf.

Counterweight Shoring

Critical to the removal of the truss arm assemblies was the shoring of the counterweight. Although the trusses have rear heel locks and attempts in the past have been made to try to utilize the heel locks to stabilize the truss arm during reconstruction, CDOT prudently elected to require full shoring of the counterweight. Since the lower chord of the truss between the live load bearings and the counterweight had to be rehabilitated, Collins elected to design the counterweight shoring to carry the full load of the counterweight. As designed, the counterweight shoring coupled with the live load bearings allowed us to make the lower chord a zero force member to allow easy rehabilitation. Walsh wanted the shoring to be easily installed and removed quickly in order to allow partial operation of the leaf when necessary. Collins designed the shoring to be easily jacked and spliced so that the lower portion of the shores could remain in place while the upper portion was removed to allow bridge operation and the counterweight to clear the shoring without requiring complete removal.

Bridge Balancing

The phasing of the construction required that at least one bridge leaf of the bridge be maintained in operating balance throughout the construction. Since Walsh wanted to minimize the work required to open the bridge leaves to transit service after replacement of the truss arm assemblies, this meant the many different interim balance scenarios were required to maintain balance for safe operations. Balance was further complicated due to the initial unbalance of the bridge leaves associated with temporary structural repairs performed by CDOT prior to the letting of the construction contract. Collins recommended to Walsh that strain gage testing of the leaves be performed to establish a base line for the balance calculations. Unfortunately, the leaves were so far out of balance that the strain gage readings only provided as estimate of the state of bridge unbalance.

Figure 5: Jersey Barriers used to temporarily balance bridge leaf.

The final arrangement of the bridge included a half concrete filled steel grating. For the interim condition of the south leaf, the deck was not in place and temporary ballasting of the leaf was required to maintain balance for operation. Precast Jersey type barriers were used to provide the necessary additional weight to allow the bridge to be safely operated. Collins devised a lashing system to hold the barriers in place during operations. The advantage of the barriers was that the balance could be easily adjusted as the work progressed. Interim and final strain gage reading were taken to verify balance and to final balance the bridge. Final balance was established to provide a “nose heavy” condition of approximately 2,500 pounds.

Other Construction Engineering

During the course of construction, other issues arose not necessarily covered by the design documents that required additional construction engineering.

- Temporary centerlocks. The project required that the existing centerlocks and alignment casting be rehabilitated. Collins devised a temporary locking system that employed a wide flange beam lock bar and a jacking pedestal to assist with the insertion of the lock bar. A jacking pedestal was employed due the changes in the deflection of the leaves during phases of the work.
- Steel repair. Collins reviewed the scope of steel repair required for the project. Some of the called out repairs were associated with structural steel that was used for stability during the initial construction of the bridge. Although records do not exist, apparently the trunnion girders and anchor columns were installed prior to the construction of the bridge pits. These elements where braced to the pit floor to provide temporary stability until the pit work could be completed. Collins performed an analysis of the bridge trunnion girder and anchor column and determined that the rehabilitation of some of the bracing members was not required.

Conclusion

Working with Walsh, Collins was able to demonstrate that the joining of a partially pre-assembled truss to an existing truss on a movable bridge is feasible and, if properly executed, can significantly reduce the duration of construction required to rehabilitate a movable bridge. When there is an opportunity for significant rehabilitation of a movable truss arm, it is believed that this methodology can be applied to the other movable bridge rehabilitations.

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[1] Wells Street Bridge Drawing File, Chicago Department of Transportation, Division of Engineering
[2] Charles Scott, Frances Alexander, and John Nicolay, 1986; Matthew T. Sneddon, 1999. “Historic American Engineering Record: Chicago River Bascule Bridge, Wells Street.” HAER No. IL-52.
[3] Thomas Pihlfeldt, “The Wells Street Bridge” *Journal of the Western Society of Engineers* 27 (February 1922): 59.
[4] Charles Scott, Frances Alexander, and John Nicolay, 1986; Matthew T. Sneddon, 1999. “Historic American Engineering Record: Chicago River Bascule Bridge, Wells Street.” HAER No. IL-52.
[5] “Handling Traffic on Chicago ‘L’ During Bridge Replacement”, *Electric Railway Journal* 58 (24 December 1921): 1113.