Monitoring of a Bascule Bridge during Construction

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ABSTRACT

The risk of member/connection distress or even overall collapse of bridge structures under significant rehabilitation efforts is generally greater than when subject to normal operating conditions and hazards. This is particularly evident for movable structures. Currently, visual inspection during construction is the most common source of assurance a bridge owner has that the structure is functioning properly. Complex structures like movable bridges may benefit from additional oversight beyond visual inspection because of the elevated risks. Structural health monitoring of critical elements can help reduce these risks. A case study is presented for the first monitoring effort of a rolling lift bascule bridge under rehabilitation. During an opening of the span for ship traffic, a 460 µε (92MPa or 13.3ksi) shift in strain response was measured in the truss lower chord. To identify the root cause of this behavior, a detailed investigation was performed including field studies, interviews and model-experiment correlation. It was concluded that a connection slip occurred at a partially completed repair. Fortunately the structural system utilized on this bascule span includes a redundant truss which allowed for redistribution of forces. In this scenario, the primary impact of the monitoring was a change to the construction sequence allowed by the contractor. This investigation shows the potential benefit of instrumentation and monitoring in conjunction with traditional visual assessment methods, specifically during construction and retrofit of a movable bascule bridge.

INTRODUCTION

The state of infrastructure in the United States is a topic of constant conversation and debate. The majority of the national bridge population is reaching or has surpassed its intended design life. Replacement of these structures can be hindered by historical, social, environmental, and economic drivers [1]. This problem is much more prevalent within the subset of the total bridge population comprised of movable spans. Roughly 900 movable bridges are in the United States inventory as of 2012 with 27% designated structurally deficient [2]. The average age of these structures is over 60 years [2]. To complicate matters movable spans are often located in densely populated areas with substantial vehicle and ship traffic. Businesses and residences may have developed organically around the span over the years. These structures become ingrained in the fabric of the community and part of the local skyline. There can be substantial resistance to altering this fabric by replacing an icon with a new structural form that bypasses the movable span requirement, even if it will ease congestion and bring an influx of toll revenue. There are also right-of-way issues, ownership disagreement, management rights, and toll revenue division. In these scenarios, indefinite preservation, or maintaining the status quo, often emerges as the most acceptable compromise.

Indefinite preservation by its nature implies a more aggressive maintenance and retrofit approach than traditional fixed-life bridge management. NYCDOT, for example, has implemented an aggressive preservation and maintenance plan of their 25 movable structures [3]. This includes roughly one billion dollars of rehabilitation and construction for many of their movable bridges. Typically, retrofits and repairs as extensive as individual component or span replacement are possible. A common constraint of this type of construction is that the spans must remain operable throughout – both to vehicle and ship traffic. The success of the retrofit not only depends on the resulting structural integrity of the bridge, but also on the magnitude of the impact on the traveling public. Interventions of this nature on movable or fixed spans alike require innovative engineering in design, construction staging, and execution. With that comes a commensurately increased level of risk. One tool to mitigate this risk is structural health
monitoring (SHM) which is the practice of identifying and tracking of quantitative performance metrics through measured data and analytical simulation.

**SHM JUSTIFICATION THROUGH RISK-BASED DECISION MAKING**

The decision-making process for implementation of a SHM system requires an intelligent foundation, such as a risk-based approach. Risk explicitly includes the hazards to which a bridge is subject, the vulnerability of the structure to that hazard, and the exposure or consequences that would result if the bridge failed [4]. An uncertainty premium is also included to account for our inability to precisely quantify the hazard and vulnerability associated with a risk. While some hazards can be reliably quantified due to a steady rate of occurrence, the probability of failure given a hazard relies on numerical analysis, subject to assumptions and limitations.

If the risk level for a given hazard is high, and it is shown that a monitoring system can sufficiently reduce that risk by lowering the uncertainty premium, then implementation of the system is justified. Rehabilitation of a movable bridge adds high risk scenarios to the structure due to new hazards (construction loads, temporary configurations, etc.) and elevated vulnerability (removal of members/connections) present at various stages of construction. The exposure is high due to the potential loss of life, loss of toll revenue for closing the bridge, and impact to the economy due to stoppage of ship traffic. In addition, the uncertainty present at each construction stage is increased with the multitude of contractors and construction methods that can be utilized.

A properly targeted monitoring system can help reduce the level of risk associated with a movable bridge rehabilitation project through directly reducing the uncertainty premium. To date many bridges have been monitored during major rehabilitation projects [5-8]. While limited, SHM systems have also been implemented on movable spans [9-11]. However, the case study presented herein is believed to be the first detailed monitoring effort of a rolling lift bascule bridge under construction.

**SHM OF A BASCULE BRIDGE**

A construction monitoring effort was performed on a 79m (260ft) two-leaf rolling lift bascule bridge (Scherzer type) that has been in service for over 80 years. Gradual corrosion and deterioration of the structure necessitated a substantial rehabilitation project. This included repairs to the truss members / connections, replacement of the steel grid deck and supporting stringers along with painting of the superstructure. Overall the structure has two primary functions. The first is to safely carry vehicular traffic across the river (service condition). The second is to open periodically allowing for ship traffic to pass (operating condition). The resulting structural system has different load paths when in service versus operation. The primary components of the structural system are illustrated in Figure 1.
During service, the bridge carries vehicular live load from the steel grid deck and distributes the forces to the supporting stringers. These stringers rest on the transverse floorbeams which carry the live load from the stringers into the longitudinal trusses. At this point the load path becomes redundant. A unique design is present which utilizes dual trusses on each side of the roadway. The outside truss (Truss B) provides continuity of the span transferring the live load through arching action to the substructures (Figure 2). However, in parallel with this truss is a partial length inside truss (Truss A) specifically designed to carry the self-weight of the structure when the span is open.

To operate the span a mechanical system is present on each leaf. The span is opened by releasing the brakes at which time a motor drives a series of gears that torques the two main pinions. These pinions run along a pair of racks (Figure 3a), applying a horizontal force to the structure above the supporting segmental girder (Figure 3b). The span then rolls away from the shipping channel as it rotates up, providing maximum clearance. The segmental girder is connected directly with Truss A which carries the weight of the span during an opening.
Construction monitoring of the bascule bridge consisted of two phases. First the baseline was established through numerical modeling, instrumentation, and data processing / interpretation. This informed the second phase of targeted construction monitoring.

**Phase 1: Baseline Evaluation**

The bascule span baseline evaluation began with a visual assessment and review of all existing documents (as-built drawings, rehabilitation plans, inspection reports, etc.). From this a history was established, tracking the span from construction to present day as there have been numerous changes to the structure. This information was utilized for development of a-priori finite element (FE) models to gain insight into the force distribution throughout various configurations of the bascule and under different loading scenarios. A full beam-shell element level 3D model was developed for the entire span using SAP2000 software (Figure 4a). In addition, sub-structuring was used for development of a single-leaf portion of the truss system (Figure 4b). The material properties were obtained from the existing construction drawings. Analysis of the FE models was performed under both live load and dead load cases in the closed position and dead load cases for varying opening angles. The analysis allowed for a better understanding of the structural behavior and aided in design of the experiment and instrumentation plan, which was later used to verify measured observations.
The experiment and instrumentation design was a critical step in the evaluation process. The approach for the bascule span was to use the openings for the experiment due to the ideal input-output relationship (structure orientation versus member force variation). This could be obtained through measurement of the opening angles along with the member strains. In addition, the intrinsic force variation was substantial allowing for significant signal-to-noise ratios. This approach was selected over conventional truck load testing to avoid traffic shutdowns and acquiring test vehicles. Vibration-based testing (ambient or forced impact / excitation) was also considered but not selected due to the complexity of the structure, the inevitable violation of the inherent assumptions of vibration testing (linearity, stationarity, etc.), and higher cost (sensors, equipment, and data storage).
The instrumentation design focused on the primary load carrying members. The intent was to characterize the force redistribution as the orientation of the span changes during opening. However, characterizing the vehicular live load distribution from ambient traffic was also a priority. It was decided to minimize the number of sensors installed due to the fact that the rehabilitation included blast cleaning and painting of the structure likely destroying the sensors by the end of construction. Truss members at select locations were instrumented with electrical resistance strain gages. This included two gages on the Truss A lower chord and two gages on the Truss B lower chord near the support (upstream and downstream sides of each leaf - total of 16 gages). Strain gages were also attached to the upper chord of Truss B near the tip of each leaf (upstream and downstream side - total of 4 gages). In addition, a tilt sensor was installed on each leaf for measurement of the opening angles. Figure 5 graphically illustrates the instrumentation plan along each leaf.

All sensors were controlled with a National Instruments CompactRIO (cRIO) data acquisition system (one on each leaf). A custom data acquisition program was developed using LabView software that allowed for both manual and threshold triggers. Manual triggers were utilized for measurement of several openings and subsets of live traffic response. The data was processed and interpreted. Direct data interpretation included the magnitude of force distribution throughout bascule openings and under typical live loading. For example the Truss A lower chord experienced roughly 250 $\mu$ε compression release during openings and around 30 $\mu$ε compression from typical truck loads. An example of a typical lower chord measured response from Truss A is provided in Figure 6. Six distinct stages occur during each opening. Each stage is briefly described below.

Stage A: The bridge is closed to traffic and the span is still in the continuous state.

Stage B: The brakes are released (removing the preload at the tip) and each leaf is slowly rotated apart breaking the continuity and transitioning from arching action (Figure 2) to a cantilever state. This applies an additional compressive force to the Truss A lower chord.

Stage C: The leaf is rotated and stopped after 50 degrees. This is a safety mechanism built-in to the mechanical system. Notice the substantial compressive strain release in the lower chord.
Stage D: The leaf is at the fully open position which is roughly 70 degrees.
Stage E: The leaf is lowered to nearly the horizontal position; however the leaves are not fully engaged with one another.
Stage F: The leaves are driven together and the brakes are locked (preloading each leaf).
Note a span lock is not present at the tip.

The information obtained from Phase 1 of the study allowed for error screening and refinement of the FE model, updating the demand versus capacity ratios, and future scenario analyses (discussed below).

Phase 2: Construction Monitoring
The second phase was to monitor the bascule span throughout construction. The intent was to track forces in critical members to identify any significant changes to the overall structural integrity of the span. Specific scenarios were not targeted due to the limitless possibilities during a major rehabilitation project. It was decided the core instrumentation used for the earlier baseline evaluation (Phase 1) was adequate for this phase. The primary sensor additions were a weather station at the top of the northeast tower and four network IP cameras (AXIS 221 day/night) that were installed halfway through construction. This included programming of a graphical interface for visualization of the images and data. For further information refer to Glisic [12] and Yarnold [13].

An event-driven monitoring approach was applied throughout construction. Two categories of events were recorded. The first category was bascule openings triggered with the tilt sensors (5 degree change). The opening behavior is a signature of the structure that allows for continual assessment.

The second category of events was excessive live load or anomalies that exceeded min/max threshold values. A threshold was set at ±60 µε so that legal truck load response was not captured, however still relatively low enough to capture the desired events. The data from these events were manually reviewed by an engineer to ensure the performance of the structure. This was conducted through comparison of the triggered event with prior measured data along with a check for recovery of the structure (ensuring elastic deformation). The previously
determined demand versus capacity ratios were also used as a means to ensure overstress had not occurred. Note all the triggered events of this type showed adequate recovery and were well below overstress of any members. Therefore, they were considered no harm to the structure.

**Measured Force Redistribution**

Four recorded openings from June 22nd to June 29th indicated typical signature behavior. Figure 7 shows the Truss A upstream (west leaf) measurements for each opening. The slight variation in peak strain results from differences in the angle of opening. Note the strain measurements do not recover perfectly due to the imprecise nature of manual application of preload applied by the operator in Stage F of the opening sequence.

![Figure 7 - Truss A Lower Chord (Upstream) Time History Plots from 6/22 to 6/29](image)

On June 30th a significant change in behavior was observed at the end of an opening prior to reseating (Stage E). A $235 \mu\varepsilon (47 \text{ MPa or 6.8 ksi})$ shift equivalent to 3200 kN (720 kips) in the upstream Truss A (west leaf) strain gages was recorded (Figure 8). It should be mentioned that at Stage E substantial compression is present within the Truss A lower chord. As a result, the positive shift in strain indicates that compression was released. Also recorded at that same moment was a shift in the upstream Truss B (west leaf) strain gage measurements. A $460 \mu\varepsilon (92 \text{ MPa or 13.3 ksi})$ shift equivalent to 2490 kN (560 kips) was observed (Figure 9). This shift occurred in the opposite sign as that from Truss A (negative strain shift). Therefore, Truss B absorbed additional compression force. Upon observing this response, the research team made diagnosis of this situation a top priority.
Note data quality issues were ruled out due to the fact that two independent strain gages measured nearly identical response for both the upstream Truss A and Truss B lower chord cross-sections (west leaf). The measurements from the downstream Truss A and Truss B members (west leaf) indicated no variation from typical response. This uncoupled behavior between the upstream and downstream truss pairs was consistent with that observed from earlier live load data.

**Diagnosis**

The assessment of the measured anomaly started with direct data interpretation as described above. Then, an interview was conducted with the resident engineer overseeing the construction along with a visual inspection of the structure. This information was used with the FE model to run various simulations and identify the root cause.
Interview and Visual Assessment

The target of the interview and visual inspection was to identify any changes to the structural system that would cause a significant variation in behavior to Truss A and Truss B on the upstream side of the west leaf. Due to the nature of the measured event the connections and supports were the primary focus. However, members were also considered for potential fracture or buckling. The resident engineer indicated that at several locations rivets had been punched (removed) and replaced with bolts. This is typically done in preparation for steel repairs to allow for faster installation. However, if the bolts are not tightened sufficiently the connection has the potential to slip. Note no members or connections were observed to show any visible signs of distress.

After the completion of the visual inspection, four primary locations were identified along Truss A as potential causes for connection slip and redistribution of force (Figure 10). These locations were selected due to the steel repairs currently in-progress. In addition, these locations were at original field splice locations resulting in potential for relative movement or slip. Truss B was inspected for condition more than potential causes of force redistribution due to the fact that Truss B absorbed additional force rather than releasing it.

![Figure 10 – Scenario Locations (Truss A)](image)

It should be mentioned that other construction activities were being performed that potentially influenced (or activated) the variation of intrinsic forces. The grid deck and stringers were recently replaced on the upstream side of the bascule span. This was the same side as the measured strain shift. In addition, counterweight adjustment was performed a week prior to the event.

Numerical Simulation

Numerical simulations were performed on four potential scenarios of the measured strain shift. The simulations were conducted using the FE model previously developed during Phase 1. The reduced sub-structure model was utilized due to the lack of coupling between the upstream and downstream truss pairs. In addition, the shift occurred just prior to re-seating the bridge so each leaf was independent (cantilever state), precluding the necessity to model the interaction between each leaf.

The FE models were independently error-screened through an extensive process that included review of the geometry, section properties, material properties, boundary / continuity condition, loading application, etc. The FE models were then checked against measured responses from bascule openings / truck live loads and updated accordingly. These steps increased the confidence in the model allowing it to better represent the behavior of the structure.
For each of the scenarios investigated the FE model was first analyzed in the cantilever state. The axial force diagram is shown in Figure 11a. Then an axial release was applied for each of the four scenario locations shown in Figure 10. The axial force diagram for Scenario 1 is provided in Figure 11b. The net difference in simulations was calculated and compared to the measured strain shift values (Table 1). Scenario 1 was clearly more representative than the other scenarios with a 10% and 2% error for the Truss A and Truss B (upstream), respectively. Note all other connections and members were evaluated for possible connection slip and/or member fracture / buckling in the event the visual assessment missed a potential scenario. However, the percent errors for these cases were all orders of magnitude larger than that obtained from Scenario 1.

Table 1 – Scenario Analysis Results

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<tbody>
<tr>
<td>Measured Response</td>
<td>235</td>
<td>-</td>
<td>-460</td>
<td>-</td>
</tr>
<tr>
<td>1) Truss A, PP9 (L11-L9)</td>
<td>212</td>
<td>10%</td>
<td>-471</td>
<td>2%</td>
</tr>
<tr>
<td>2) Truss A, PP13 (U13-U10)</td>
<td>-26</td>
<td>111%</td>
<td>-411</td>
<td>11%</td>
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<tr>
<td>3) Truss A, PP9 (L9-L7)</td>
<td>66</td>
<td>72%</td>
<td>-151</td>
<td>67%</td>
</tr>
<tr>
<td>4) Truss A, PP14 (L14-U10)</td>
<td>11</td>
<td>95%</td>
<td>-166</td>
<td>64%</td>
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Diagnosis Summary
The interview, visual assessment, and numerical simulations conclusively showed the measured strain shift was consistent with a connection slip at Truss A, panel point 9 (L11-L9 side) (Figure 12). Additional calculation of the total movement at panel point 9 was approximately 1.5mm (1/16in). Therefore, it was concluded the temporary bolts recently placed at this connection were not sufficiently tightened providing inadequate slip restraint which caused the connection to go into bearing. This slip released significant force in Truss A that was primarily absorbed by Truss B. Fortunately the structural system has the redundancy to allow for redistribution of forces.

Figure 12 – Connection Slip Location

A demand versus capacity analysis (analogous to load ratings) of the entire span was performed to ensure the safety of the bridge. The controlling location was identified as the Truss B lower chord with a demand versus capacity ratio of 0.75. As a result, the calculations indicated sufficient reserve capacity is present and no additional retrofit was recommended outside the current construction project. However, procedural changes were implemented for future structural repairs. The primary modifications were to the allowable repair sequencing. The owner no longer permits removal of existing rivets until the repair is being performed.

CONCLUSIONS AND RECOMMENDATIONS
This paper addresses monitoring of movable bridges during construction. Due to the unique nature of many movable spans indefinite preservation is implemented, often subjecting these structures to significant rehabilitation projects. An argument has been made with regard to the importance of construction monitoring as a result of the elevated risks associated with such rehabilitations. A case study was presented for monitoring of a rolling lift bascule bridge throughout construction. The bascule study concluded:

- Substantial force redistribution was measured as a result of a connection slip from insufficient bolt tightening during the intermediate stage of a steel repair.
- The safety of the bridge was adequate due to the redundancy of the structure and no additional repairs were necessary. However, procedural changes were implemented for future retrofits.
Monitoring throughout the rehabilitation process is as important as monitoring before and after construction. As a result, of the overall movable bridge construction monitoring study, the following general conclusions were drawn:

- Movable bridges are highly vulnerable during construction due to the uncertain temporary configurations. As a result, the risk of distress or collapse of the structure is elevated.
- Intrinsic force variations resulting from construction can be significantly greater than force variations during regular service and operation (i.e. opening). These variations can be “locked-in” resulting in a drastically different structure during and after construction that should be accounted for in refined analysis (e.g. load ratings).
- There is benefit to instrumentation and monitoring in conjunction with traditional visual assessment methods, specifically during construction and rehabilitation of a movable bridge structure.
- Anomalies identified during construction monitoring should be evaluated through a comprehensive process such as:
  1. Verification of data quality to ensure the measurements are as a result of the structure.
  2. Visual assessment (if video is not included in the monitoring system) to understand the temporary configuration of the structure.
  3. Interview of the involved personnel (i.e., contractor, resident engineer, construction inspectors) for any additional information.
  4. Numerical simulations to identify / verify the root cause(s).

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