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Considerations during Gravity Lowering of Movable Structures Stickel Bridge, Newark, New Jersey

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INTRODUCTION

This paper discusses considerations which arose during the gravity lowering of a movable structure as part of the rehabilitation of the William A. Stickel vertical lift bridge. The Stickel Bridge is a tower drive vertical lift bridge that carries six lanes of interstate I-280 traffic over the Passaic River in Newark, New Jersey.

As part of an ongoing rehabilitation project that is nearing its completion, the grid deck was replaced, as were all main counterweight ropes. Although the bridge is equipped with hanger plates and pins to hang the counterweights for rope changeout, the towers/bridge were apparently not provided with a system to jack the counterweights. The project plan to hang the counterweights as detailed on the contract plans and implemented by the contractor was to jack the lift span, make the counterweight pin connections, sever the ropes and then lower the bridge back to its seats. Once the ropes and deck were replaced, the plan to return the bridge to service was to jack the lift span to make the rope terminations at the lift girder and then to lower the span until the ropes took the full weight of the counterweight. With the ropes carrying the load of the counterweight, the counterweight pins could then be pulled, and the lift span lowered back to its seats.

On the night of December 1, 2007, the attempt to lower the rehabilitated lift span after connecting the newly installed ropes was not successful. This is a recounting of the steps that were ultimately required to seat the lift span.

PROJECT MEMBERS

The primary project members involved in the portion of the project discussed in this paper are as follows. New Jersey Department of Transportation is the bridge owner, and provided resident and support engineering services. The Engineer of Record for the work was PB Americas of New York, NY. HDR, Inc. of Newark, New Jersey provided construction engineering support for the State. The Prime Contractor was IEW Construction Group of Trenton, New Jersey. IEW utilized White Marine of Perth Amboy, New Jersey for the mechanical work. W.B. Equipment of Wood Ridge, New Jersey provided the hydraulic jacks and calibration. Stafford Bandlow Engineering, Inc. initially provided balance testing and weighing services for IEW, and then provided construction engineering support on an as needed basis to assist in troubleshooting the field issue that is the source of this paper.

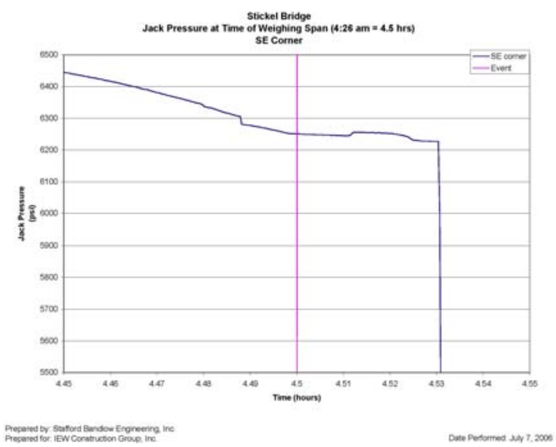
PRELIMINARY PREPARATIONS

In order to facilitate the above work, the contract documents required that the contractor measure the imbalance of the lift span at the outset of the work, weigh the lift span following the removal of the counterweight ropes, weigh the lift span again at the completion of all rehabilitation work but prior to reconnecting the ropes, and then perform a final imbalance test at the completion of the work. The imbalance and weighing tests were significant factors considering that the project approach was predicated on the lift span lowering under the influence of gravity.

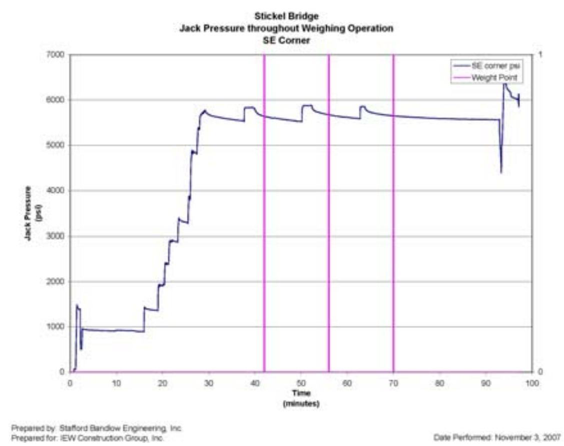
SBE provided testing services for IEW and performed the required tests per the following timeline:

1. Initial Strain Gage Balance Test - June 24, 2006
2. Initial Weighing of Lift Span - July 7, 2006
3. Final Weighing of Lift Span - November 3, 2007
4. Final Strain Gage Balance - "to be completed"

The initial balance of the lift span was determined via dynamic strain gage bridge balancing. The initial and final weighing of the lift span was conducted by monitoring the pressure at the four jacks used to raise the lift span; the pressures were recorded via electronic pressure transducers. A graphical depiction of the initial and final weighing of the SE corner are presented below; the red bars indicate the weigh points. For the initial weighing, only one weigh point was utilized due to the necessity to lower the bridge to clear traffic. The final weighing was conducted as part of its own bridge closure prior to the initial rope termination attempt. Therefore, there was sufficient time to utilize three separate weigh points with the bridge raised to a different height at each point. The average of these three weigh points was then utilized to determine the final span weight.



Initial Jacking Pressure, SE Corner



Final Jacking Pressure, SE Corner

The results of these tests are summarized in the following table:

		CORNER WEIGHTS +/- 28,250 (lbs.)			
	TEST	**SW**	**NW**	**SE**	**NE**
	Initial Span Weight	**608,432**	**619,495**	**680,345**	**663,750**
	Final Span Weight	**634,738**	**641,538**	**631,738**	**639,238**
	Delta	**+26,306**	**+22,042**	**-48,608**	**-24,512**
	Imbalance (Initial Strain Gage Result)	**10,721**	**5,705**	**6,096**	**9,022**
	Final Corner Reactions (Net)	**+37,027**	**+27,747**	**-42,512**	**-15,490**
Construction Staging	**4 Man Lifts (lbs.)**	**36,000**			
	2 Light Stands (lbs.)	**3,520**			
	4 Work Trucks (lbs.)	**50,000**			
	*** Corner Reactions at time of Jacking**	**+59,407**	**+50,127**	**-20,132**	**+6,890**
	vs				
	Friction (Initial Strain Gage Result)	**5,174**	**7,470**	**5,624**	**6,581**

*** Based on Equal Distribution of Construction Staging**

The test results indicated that the refurbished span was in fact span heavy and therefore corroborated the basic project approach.

FIELD WORK

Prior to the attempting to reconnect the ropes at the completion of the project, field personnel raised a concern with a potential interference at the counterweights.

The newly installed ropes had been pinned at the counterweight terminations, draped over the sheaves, and were free hanging at the lift girder. Due to constructional and elastic stretch, the unloaded ropes were not long enough to make the termination at the lift girder. The lift span rope connections could not be made until the lift span was jacked high enough for the rope take-ups to pass through the lift girder billet so that the take-up nuts could be installed. Field personnel determined that the lift span would need to be jacked 21 inches to make the connection. However, field personnel also noted that there was a limiting clearance of 19.5 inches between the top of the counterweight and the underside of a tower support beam. Therefore, if the ropes did not stretch during the loading process, the counterweight would be pulled into contact with the tower as the span was lowered.

IEW asked SBE to look into whether at least 3” of rope stretch could be expected during loading in order to avoid the interference problem as well as to provide sufficient engagement of the nuts on the take-up rods. SBE performed rope stretch calculations to determine the theoretical elastic rope stretch based on the fundamental equation:

$$\delta = PL/AE$$

where,

δ = Rope Stretch (inches)
P = Applied Load, Tension (lbs.)
L = Rope Length (inches)
A = Metallic Cross Sectional Area of Rope (in^2)
E = Modulus of Elasticity of Rope (psi)

A second factor that must also be considered is the constructional stretch of the rope: when the ropes are spooled after fabrication, they experience a certain amount of “shortening” due to relaxation of the strands. Rope manufacturer’s provide a range for constructional stretch, however it is not as definitive as the elastic stretch calculations. We cross checked the range with direct data we have obtained as part of prior rope change-out projects. We calculated that the ropes would see 3.8 inches of elastic stretch, and that an additional 3 to 4 inches of constructional stretch could be expected. Therefore, we did not anticipate a problem with interference. However, the duration of the roadway outage allotted for the work was extremely limited. Due to the extremely high volume of traffic that this bridge carries due to its location in the New York metropolitan area, the closure had to be performed over the weekend, at night, and was limited to 5 hours for a full closure; lane restrictions were allowed for the 3 hours prior to and following the closure to allow for staging. Therefore, in order to complete the work in the allotted time, it was essential to limit any delays to the work. To safeguard against a disconnect between the rope stretch calculations and physical reality, we also suggested that IEW proof load one or more of the free hanging ropes to verify that the actual elastic stretch agreed with the theoretical calculations. IEW agreed and performed the proof test. Due to logistics, the load that was applied to the ropes fell short of the anticipated 40 kip load; however, at an applied load of 28.8 kips, a rope stretch of 5¼ inches was recorded. This stretch was consistent with the calculated values and exceeded the minimum requirement, providing sufficient justification to proceed with the work.

On the night of December 1, 2007, an attempt was made to connect the West Tower ropes. Rope stretch was monitored as the ropes were loaded. The ropes stretched as anticipated and thereby eliminated any concern with an interference when the lift span was lowered. However, even after the ropes were loaded with the full weight of the counterweight as evidenced by the jacks that had been supporting the weight of the lift span dropping out from under the span, the counterweight did not raise up off its support pins. Several steps were taken to troubleshoot the problem on-site, including verifying that all brakes were released (which they initially were not) as well as attempting to lower the span by manually rotating the auxiliary motor shaft. However, as the troubleshooting work soon exceeded the allotted bridge closure without producing any movement of the span, the troubleshooting efforts had to be abandoned so that the ropes could be disconnected in order to lower the lift span and clear the roadway for traffic.

ASSESSMENT OF FIELD WORK

Following the failed rope connection attempt, IEW held a meeting of its project team to discuss how to resolve the stalemate that had arisen. The basic project approached relied on gravity to lower the lift span. Therefore, we identified the following areas as sources that might have contributed to the problem:

- If the lift span were not span heavy
- If the lift span was not span heavy enough to overcome friction
- If the counterweight pins were bound on the hanging plates
- Allotted time for the work

As part of the meeting we discussed troubleshooting measures and/or solutions to each of these problems. A subsequent meeting was then held at HDR’s office on December 28, 2007 in which the proposed actions were presented to all project parties for discussion and comment.

Span Balance

Although the direct results of the jacking indicated a span heavy condition, this did not account for the tolerance on the weighing results. In addition, any increase in friction, as discussed below, would need to be compensated for through a comparable addition of weight. Therefore, it was agreed that it would be prudent to plan on having sufficient additional weight on site to fully compensate for the jacking tolerance.

To guard against overloading the jacks, which had been sized based on the weight of the lift span, this additional weight needed to be added <u>after</u> the ropes had been connected and fully loaded so that the ropes, and not the jacks, would carry the additional load. A discussion then ensued regarding the form of the additional weight. Various suggestions included ballast weight which could be added to the deck and a water tank, which would utilize water pumped out of the river. A critical factor in choosing a method was to ensure that it could be speedily implemented so that it would have the least impact on the allotted closure window.

Friction

The sole assessment of how friction factored into this problem came from the strain gage balance testing that was conducted in July 2006. As part of balance testing, the system friction from the location of the gages out to the movable span can be calculated if it is assumed that friction is the same in both directions of travel. The strain gage testing did not show the system friction to be a significant factor. However, it is noted that the friction determined through the strain gage test is dynamic friction as opposed to static friction which can be expected to be higher. In addition, the friction determined through the strain gage test does not account for the inefficiency of the machinery from the gage back to the motor, which must be back driven by the descending span.

As strain gages were already installed, we proposed to turn the machinery at the motor shaft and measure the loading at the rack pinion shafts. This testing had two purposes; first we would be able to obtain the frictional load due to the machinery, and second, we would be rotating the counterweight sheaves, and therefore could verify that the sheaves turned freely and had not bound up. The sheaves would also be lubricated during this work to ensure adequate and recent lubrication of the entire journal

Binding at Counterweight Pins

The initial work plan had been to manually remove the counterweight pins with hand tools. Following the noted problem, team members voice concern that the pins may have been bound in the hanger plates and thereby prevented the counterweight from raising up to unload the pin. This concern was based on the fact that the NW pin had been driven into place during installation due to a slight misalignment between the hanger and counterweight plates. Therefore, we proposed to design and implement a jacking frame to retract the pin during the next attempt.

Allotted Roadway Closure

In addition to the above concerns, it was discussed that the allotted 5 hour roadway closure did not afford adequate time to perform the required work. As part of the field work on December 1, nearly the entire closure period had been used to jack the lift span and making the rope connections. Even if the counterweight pin removal had gone flawlessly, it would have been challenging to lower the span and open the roadway to traffic within the 5 hour timeframe. In light of the additional work tasks (i.e. jacking the counterweight pins and adding weight to the lift span) now being added to the original work plan, the contractor asked for an extension on the roadway closure.

IMPLEMENTATION OF TROUBLESHOOTING

At the conclusion of the project meeting on December 28, a consensus was reached to pursue the following course of action:

1. Frictional testing of the drive machinery and sheaves was performed.

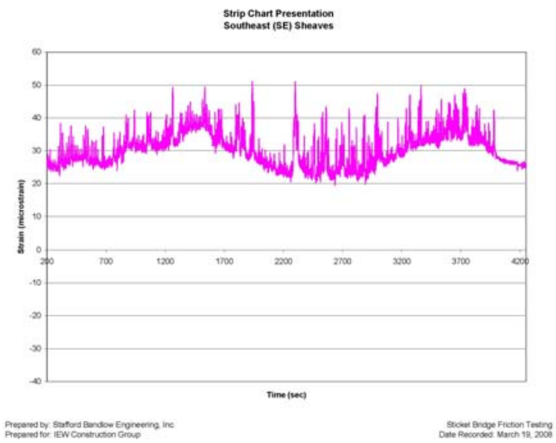
The friction due to the span drive machinery was assessed by employing a torque wrench to measure the torque input at the auxiliary motor shaft that was required to initiate rotation of the unloaded rack pinion shafts. The following results were obtained:

West Tower: 110 ft-lbs at auxiliary motor shaft.

East Tower : 80 to 100 ft-lbs at auxiliary motor shaft.

Based on the above results, the torque required to overcome the machinery friction alone far exceeded the torque of 28 ft-lbs that was calculated to rotate the unloaded sheaves. However, this work also demonstrated that the machinery did rotate without obstruction, and therefore the machinery could be ruled out as an obstacle to the rope termination work.

The friction due to the counterweight sheaves was assessed by measuring the strain in the rack pinion shafts as the unloaded sheaves were rotated in each direction under both “unlubricated” and lubricated conditions. [Note that the trunnion bearings had been rotated and lubricated through the course of the rehabilitation work, however the lubrication had taken place approximately 6 months prior to the March field work; therefore, the starting condition for the friction testing is considered "unlubricated" as opposed to the subsequent test which would be directly lubricated.] The findings of the investigation revealed that there was no significant difference in the frictional value from the lubricated to the unlubricated condition, that variations in the relative rotation of the pairs of sheaves at three out of four corners of the bridge attested to frictional issues due to either a sticky differential or bearing friction, and finally, that the acquired data took the form of a sinusoid (as presented in the figure below), indicating that the sheaves had varying amounts of eccentricity. The table below presents the measured frictional strain as compared to the calculated strain utilizing the AASHTO prescribed coefficient for moving friction and the dead weight of the sheave and trunnion.



Friction Testing, SE Sheave

		CORNER			
		NE	SE	NW	SW
SHEAVE FRICTION	Actual Strain	22.2	30.3	28.6	28.1
	AASHTO Calculated Strain	12.5	12.5	12.5	12.5

The results presented in the above table indicate that the actual frictional loading results in a frictional coefficient that is 2.24 times greater than that recommended by AASHTO. When considering the sheaves in their fully loaded condition, this would result in cumulative trunnion friction of 47,000 lbs per tower. Therefore, a comparable amount of weight would need to be added to the lift span to offset this friction as part of the forthcoming rope termination work.

2. The contractor would need to be prepared to add additional weight to the lift span to compensate for the jacking tolerances, as well as the results of the frictional testing, as presented in the table below.

ADJUSTMENT	CORNERS			
	SW	NW	SE	NE
Friction	23,350	23,350	23,350	23,350
Initial Jacking Tolerance	28,050	28,050	28,050	28,050
Final Jacking Tolerance	21,050	21,050	21,050	21,050
Total Per Corner	72,450	72,450	72,450	72,450
Total Per End	144,900		144,900	

Note:
Friction based on Friction Testing conducted March 2008
Jacking Tolerance based on initial and final reports dated August 2006 and November 2007.

It was recommended that the contractor be prepared to add a total of 145 kips to the lift span. IEW chose to use concrete barrier segments for the additional weight. Each segment weighed 6,000 lbs and could be loaded on and off waiting flatbed trailers with a front end loader. It was thought that this method provided the most efficient and timely weight transfer.

3. A jacking assembly was designed to retract the counterweight pins. The jacking assembly was sized for a load of 75,000 lbs, based on the assumption that deformation had resulted in a light interference fit (.00015 to .0003") between the pin and hanger plates with a frictional coefficient for clean steel (0.8). The jacking assembly utilized a 50 ton hollow core jack (which would be limited to 7500 psi) seated on top of a pipe stand which would bear against the counterweight hanger plates; the 1 ¾" diameter jacking rod would extend through and bear against the backside of the pin. As the existing pins were not fabricated for this purpose, the pins needed to have a 2" diameter hole drilled along their axis to accommodate the jacking rod, and this needed to be performed in place. Calculations were performed to verify that the material removal would not compromise the integrity of the pins. The calculations indicated that the pins had adequate capacity and the holes were drilled without event.

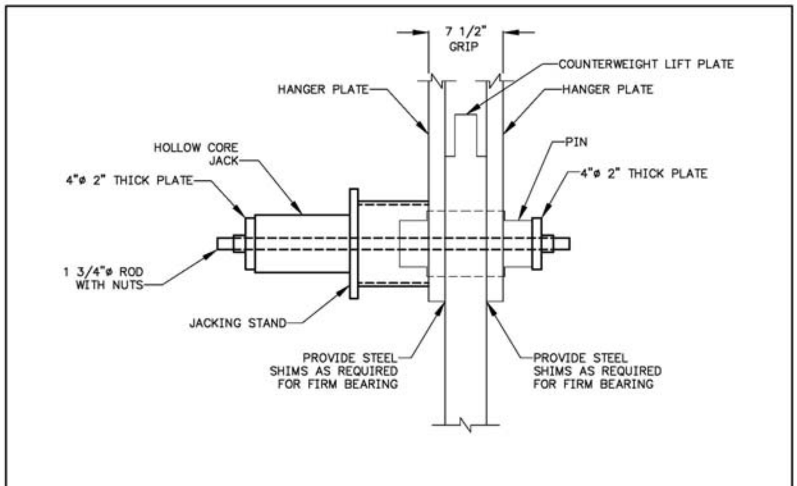


Figure 1. Hydraulic Jacking Setup For Counterweight Pin Removal.

Project: William A. Stickel Bridge Rehabilitation Newark, New Jersey

Prepared by: Stafford Bandlow Engineering, Inc.

4. Project management coordinated the request for a longer roadway closure with state traffic control. While the work still needed to be performed over the weekend at night, the request was granted and a 10 hour closure was obtained which double the original work time.

SECOND ATTEMPT AT ROPE CONNECTIONS

After the completion of the above recommendations, a second attempt was made to connect the ropes.

On the evening of March 29, 2008 a second attempt was made to connect the ropes at the West Tower. Upon loading the ropes, movement was immediately noted at the counterweight pins so that, when the ropes were fully loaded, clearance was visibly present at the underside of each pin. This eliminated the prior concern with the pins not being fully unloaded. The pin jacking assembly, which was already in place, was then utilized to pull the pins. At the NW pin, despite the clearance at the underside of the pin indicating that it had been unloaded, a significant force

Clearance at underside of Counterweight Pin

was still required to retract the pin. Therefore, the jacking assembly was clearly a prudent choice.

Once the pins were pulled, the span was lowered via the jacks without much effort. The additional weight of the concrete barriers was not required. The work was completed before the end of the allotted closure. The East Tower work was then performed during a separate night closure on April 5 with similar successful results.

CONCLUSIONS

While the ultimate source of the failed span lowering during the rope connection attempt in March 2007 was not definitively established, the likely source may be attributed to sheave or counterweight pin friction. To that end, the following points should be considered as part of any future endeavor.

1. The effects of sheave and machinery friction must be amply accounted for when attempting to lower structures via gravity.

2. Lubrication of the trunnion bearings immediately prior to the movement attempt would be prudent.

3. Providing a pin jacking assembly can eliminate problems with manual removal.

4. Providing additional weight to compensate for jacking and/or friction tolerances may be prudent.

Several corollary points may also be drawn, including:

5. Wire rope stretch calculations exhibit good correlation with actual stretch. However, constructional stretch must be considered when unspooling new ropes.

6. Frictional testing of the drive machinery revealed that the unloaded machinery exhibited considerable frictional resistance. Ample allowance should be provided for large drive trains that utilize open gearing and sleeve bearings

7. Frictional testing of the unloaded main counterweight sheave bearings demonstrated that the measured friction was well in excess of calculated values. However, when the measured coefficient was used to determine the system friction based on the full weight of the bridge, the resultant value was well in excess of the frictional value determined through the initial balance test. The final balance test has not yet been conducted to determine if there has in fact been a change in the system friction, or if there may be a size to weight factor that has affected the frictional testing. Based on the fact that the bridge was lowered without the addition of any weight, the latter scenario appears more likely.