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Wilson Piggot Bridge Instrumentation and Load Testing Evaluation

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Introduction

Background

Throughout the United States, there are countless examples of bridges subject to significantly greater loading than was originally anticipated. As a result, owners such as the Florida Department of Transportation are given the unenviable task of evaluating the impact of increased loading on the load carrying capacity of older bridges and corresponding safety to the traveling public and service life of the these facilities. This paper discusses one such instance and is noteworthy given that it applies to a double-leaf trunnion bascule bridge commonly found throughout Florida.

State Road (SR) 31 over the Okeechobee Waterway in Lee County Florida (locally known as the Wilson Piggot Bridge) was constructed in 1958 and is one of more than a hundred double-leaf trunnion bascule bridges of a common design built in Florida from the mid 1950's to the early 1970's. SR 31 is a two lane rural road several miles inland from the Gulf of Mexico on the west coast of southern Florida (see Fig. 1.) The original contract plans specified a design traffic volume ADT of 1,200 vehicles per day with 5% truck traffic, which was consistent with the relatively remote location of the site and the relatively low original anticipated future growth of this area. At the time SR 31 was constructed, and for many years to follow, this stretch of highway primarily serviced low-density residential areas and agricultural industries including cattle and citrus with low volumes of traffic.

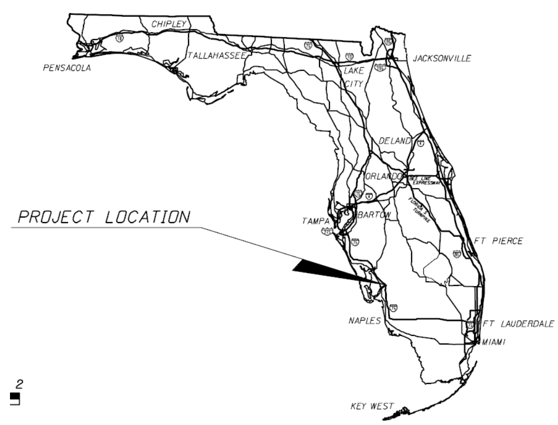


FIGURE 1: Project Location

Throughout the 1990's and early 2000's, because of a number of factors including favorable weather and socio-economic environment, coastal Florida exploded as a retirement Mecca with significant corresponding real estate development. During this timeframe, there were periods in which nearly one-thousand new residents were added to Florida each day. Throughout Florida, coastal agricultural acreage, including the area near the Wilson Piggot Bridge, was rezoned from agricultural to real estate development. As a result of unbridled development, traffic volumes on rural roads such as SR 31 increased exponentially.

In 2007, the high volumes of vehicular traffic were taking its toll on the Wilson Piggot Bridge. Traffic volumes had increased to more than 12,500 vehicles per day (6400 southbound and 6100 northbound) with 33% truck traffic which equates to more than 2,100 trucks per day in each direction. Furthermore, a large borrow pit located north of the bridge, servicing several major developments located south of the bridge, produced a significant portion of the truck traffic crossing the bridge (i.e. a significant number of trucks traveling north-south were fully loaded, 35 ton or greater, four axle dump trucks.) Misadjusted bascule leaf center locks and live load shoes exacerbated the loading, which increased the live load dynamic impact on the span. This extreme loading resulted in premature failure of a number of bridge elements including:

- Cracking and deformation of the steel open grid roadway flooring
- Cracking in the coped areas of the stringers
- Excessive wear and fretting corrosion between the plies of the riveted built-up main girder bottom flange plates and bearing stiffeners at the live load shoes resulting in a small gap (i.e. less than 1/32" total between these elements) that deformed, closing the gap, under repeated heavy loading
- Cracking and spalling of the bascule pier concrete front wall below the live load shoes
- Excessive wear of the span lock guide and receiver shoes.

In response to the above concerns, truck traffic was removed from the bridge until emergency repairs could be implemented. Although emergency repairs would address these relatively localized concerns in the short term, the Florida Department of Transportation, District 1, had greater concerns over the affect that this increased loading would have on the long-term structural capacity, safety to the traveling public and service life of the bridge. Specifically, the Department was concerned over the impact the increased loading had on the fatigue life of the riveted built-up main girders, load capacity of the main girder bearing stiffeners, and the affect continuing wear to the span lock guide and receiver shoes would have on the main girder stresses and deflections. Preliminary analysis and evaluation of these concerns indicated a potential cause for concern. However, as is often the case of such problems, conservatism inherent in the methodologies used in analysis, which are typically derived from conservative provisions in the design specifications, left room for skepticism of the preliminary results. The repercussions of accepting the results of the preliminary analysis included permanent load posting of the bridge and/or expensive additional major repairs. As such, the Department made the decision to evaluate the structure in greater detail by implementing an in-depth instrumentation and load testing program with the possibility of mitigating load posting and avoiding the expensive additional repairs.

This paper discusses the details of the instrumentation and load-testing program and the results and decisions made as a result of this program. The paper also discusses some of the challenges inherent in implementing an in-depth instrumentation and load-testing program and interpreting and evaluating the results. The author is unaware of a similar instrumentation, load testing and evaluation on a double-leaf trunnion bascule span of this configuration and thus this paper offers information of potential educational value for future use.

FIGURE 2: Wilson Piggot Bridge

Bridge Description

FIGURE 3: Bascule Leaf Framing

The Wilson Piggot Bridge (see Figs. 2 thru 4) is typical double-leaf trunnion bascule bridge commonly found throughout Florida and constructed during a period from the late 1950's to early 1970's. The bridge carries two lanes of traffic with a clear roadway width of 28'-0" and 3'-6" wide access walkways behind raised brush curbs each side of the roadway with an overall width of 35'-9". The bascule leaf structure consists of a traditional steel framework with two main girders and a floor system of floorbeams and stringers supporting steel open grid roadway flooring. Cantilevered floorbeam brackets outboard the main girders support the access walkways. The main girders are deck girders of riveted built-up construction. Each bascule leaf is balanced by a concrete counterweight located below the roadway deck. Each bascule leaf is supported on and pivots about a pair of Hopkins trunnion assemblies (i.e. trunnion with a single bearing on the outboard side of the main girder with the inboard end of the trunnion shaft supported by a trunnion girder that frames between the counterweight and a floorbeam.) Live load shoes mounted on the underside of the main girders and that bear on the front wall of the bascule piers forward of the trunnions stabilize the bascule leaves when vehicles are on the bascule span. Span lock assemblies, consisting of a rectangular lock bar driven through a set of guides on one leaf and engage a receiver on the opposite leaf, maintain equal deflection of the two bascule leaves and continuity of the bridge deck. The bascule span has a length of 122'-6" center-to-center of trunnions and 104'-6" center-to-center of live load shoes.

FIGURE 4: Bascule Span Deck

Instrumentation and Load Testing Program

The Florida Department of Transportation (FDOT), District One Structures Maintenance Office sought the assistance of the FDOT Structures Research Center (FDOT SRC), Tallahassee, Florida and EC Driver and Associates, Tampa, Florida to develop and implement the instrumentation program, evaluate the data and make recommendations regarding the disposition of the structure.

Instrumentation

FDOT SRC installed instrumentation throughout the bascule span (See Figs. 5 and 6) to measure strains and deflections during loading from test trucks that simulated observed truck loading. The measured strains and deflections were used to evaluate the structural behavior during the loading. Specifically, the results were used to assess whether:

- Fatigue stress range in the main girder flanges is a concern
- Live load shoe bearing stiffeners are subject to excessive compressive and bearing stresses,
- Differential vertical displacement of the two bascule leaves at the center of the bascule span, caused by poor adjustment of the span lock guides and receivers, is significant and if these displacements have a significant affect on the main girder stresses
- Vertical displacements of the main girders at the live load shoes, due to poorly adjusted live load shoes and/or deformations in the stacks of main girder cover plates, live load shoes and shims, are significant.

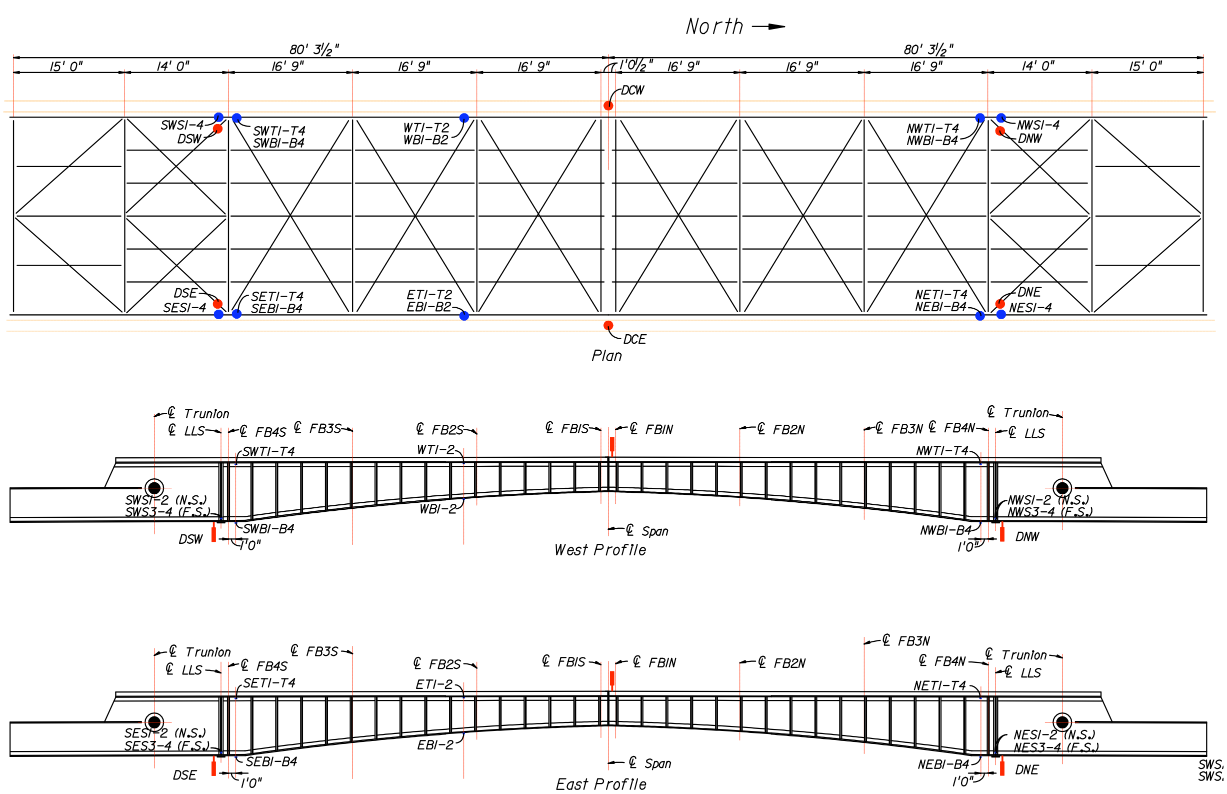


FIGURE 5: Instrumentation Layout

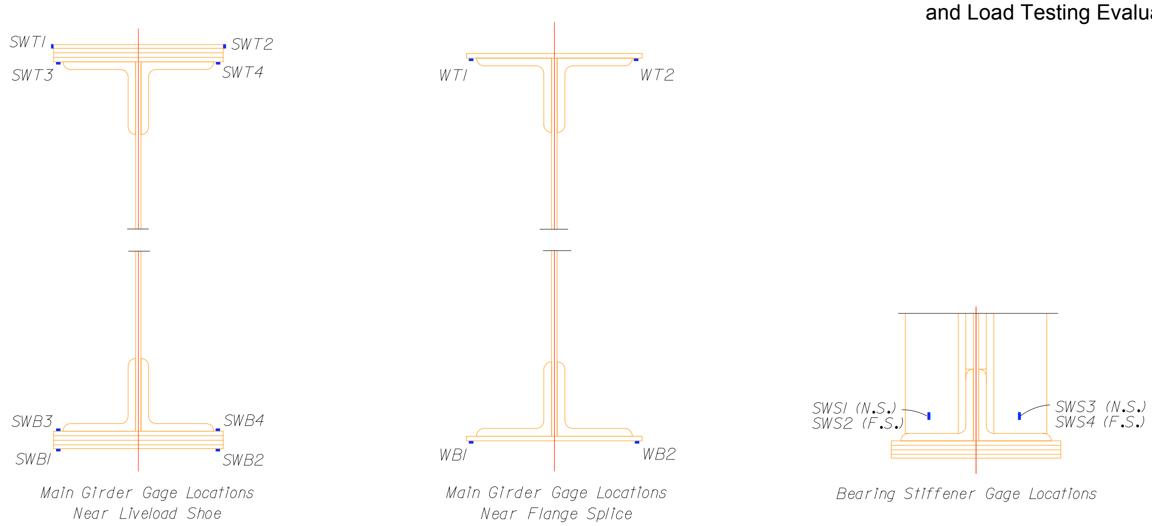


FIGURE 6: Instrumentation Details

Specifically, the instrumentation included the following:

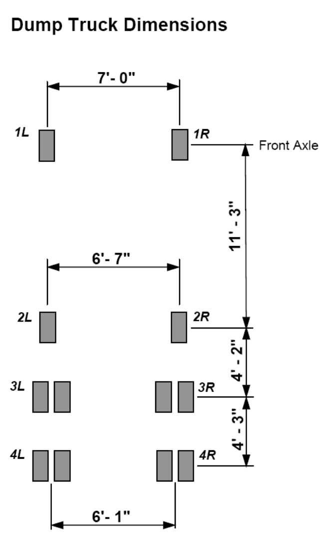
- A total of forty (40) foil strain gages were mounted on the bascule leaf structural steel at the top and bottom flanges of the main girders at specific locations throughout the bascule span. The locations for the strain gages were strategically selected as they were pre-determined by calculation to be the controlling locations for fatigue stress range. The selected locations were immediately forward (leaf tip side) of the live load shoes (eight strain gages per location) and immediately forward (weak side) of the riveted field splice of one leaf (four strain gages at each location) at approximately the third point of the bascule span. The strain gages were used to measure strains in the flanges created by the test truck loading. Foil strain gages were 30 mm except for a few locations where 5 mm foil strain gages were used due to limited available space. Four (4) of the strain gages were found to have malfunctioned and thus the results from these gages were disregarded.
- A total of sixteen (16) 30 mm foil strain gages were mounted on the live load shoe bearing stiffeners (four at each live load shoe) near the bottom of the stiffeners. The strain gages were used to measure strains in the bearing stiffeners created by the test truck loading.
- Four (4) displacement gages (LVDTs) were mounted on the bascule piers at the live loads shoes (one at each live load shoe.) The LVDTs were used to measure differential vertical movement between the bascule leaf and bascule pier at the live load shoes to determine the magnitude of deformation between the gaps in the plies of the riveted built-up main girder bottom flange plates, live load shoe and shims created by the test truck loading.
- Displacement gages (LVDTs) were mounted on the bridge railing at the joint between the bascule leaves (one at each railing.) The LVDTs were used to measure the differential vertical movement between the two leaves created by the test truck loading.
- Two (2) pivoting lasers were mounted on the bascule pier sidewalk with digital targets mounted on the bascule leaf near the joint between the leaves (one at each sidewalk.) The lasers were used to measure maximum total deflection of the bascule leaf tips created by the test truck loading.

Loading

Data was recorded as two trucks, each weighing approximately 70,000 pounds and configured in a single-unit four-axle configuration (approximately equivalent to Florida SU4 Legal Load), were loaded on the bascule span in a variety of scenarios including both static (non-moving load) and dynamic (moving load) cases. Figures 7 and 8 depict the truck configuration used in the load testing including axle and wheel line spacing and the truck positioning on the bascule span. The weight of each wheel was measured using weigh scales. Figure 9 includes a list of the various loading scenarios used during the load testing.

The measured values were compared with calculated values based on AASHTO Standard Specifications for Highway Bridges, which was used in lieu of AASHTO LRFD Specifications for Highway Bridges for consistency with original design loading. The measured wheel loads and vehicle dimensions were used to calculate the stresses in the main girder flanges and live load shoe bearing stiffeners at the strain gage locations using methodology in the AASHTO Standard Specifications for Highway Bridges. A two-dimensional frame analysis was performed using GT-STRUDL to compute moments in the main girders and reactions at the bearing stiffeners accounting for appropriate distribution and impact factors. Stresses were computed using spreadsheet calculations using calculated main girder section properties.

The measured strains for the various loading conditions were summarized in spreadsheets with the strains converted to stresses and plotted on graphs. For the dynamic load cases, in order to illustrate the change in stress as the test vehicles cross the span, stresses were plotted versus the relative position of the truck to the rear joint of the bascule leaf (i.e. the point at which the struck first loads the bascule span.) Calculated stresses were included on the same graphs as the measured stresses for direct comparison. The measured stress values for the four strain gages at each flange location were averaged to produce a single curve per flange to eliminate the affects of torsion and other anomalies in the results. Interestingly, the strain in the outer most cover plate of the flanges was found to be consistently slightly lower than the strain at the inner most cover plate of the flanges, which is thought to be a result of very small slip deformation between the plies. In addition, the measured stresses for the four strain gages at the live load shoe bearing stiffeners were also averaged.



Dump Truck Axle Weights (lbs)

Axle	Truck 2 Left	Truck 2 Right	Truck 5 Left	Truck 5 Right
1	6720	7040	7140	7060
2	7940	6540	4700	4700
3	8820	9540	11260	11260
4	8480	8940	10700	12600
Total	64020		70340	

FIGURE 7: Test Truck Configuration

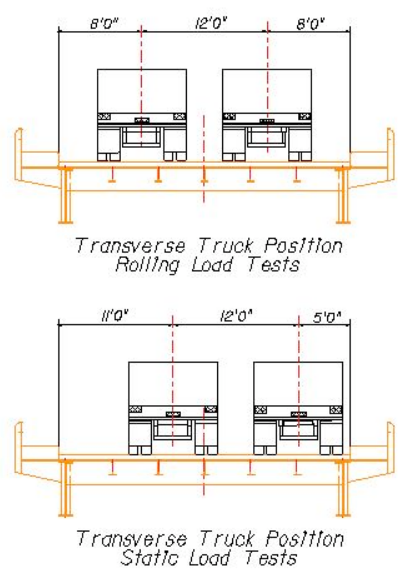


FIGURE 8: Test Truck Positioning

Loading Cases				
Load #	**File Name**	**Number of Trucks**	**Direction**	**Speed**
1	1.lvm	One Truck	South to North	10 mph
2	2.lvm	One Truck	North to South	10 mph
3	3.lvm	Two Trucks	South to North	10 mph
4	4b.lvm	Two Trucks	North to South	10 mph
5	5.lvm	One Truck	South to North	35 MPH
6	6.lvm	One Truck	North to South	35 MPH
7	7.lvm	Two Trucks	South to North	35 MPH
8	8.lvm	Two Trucks	North to South	35 MPH
9	9a.lvm	One Truck	South to North	45 MPH
10	9_55mph.lvm	One Truck	South to North	50 MPH
11	10_55mph.lvm	One Truck	North to South	55 MPH
12	11_55mph.lvm	Two Trucks	South to North	50 MPH
13	12_55mph.lvm	Two Trucks	North to South	50 MPH
14	17_35mph	Back-to-Back	South to North	35 MPH
15	18_35mph	Back-to-Back	North to South	35 MPH
16	13.lvm	One Truck	South to North	Static
17	13a.lvm	Two Trucks	South to North	Static
18	14.lvm	One Truck	South to North	Static
19	14a.lvm	Two Trucks	South to North	Static
20	15.lvm	One Truck	South to North	Static
21	15a.lvm	Two Trucks	South to North	Static

FIGURE 9: Table of Test Truck Loading Scenarios

Due to the voluminous amount of data from the load testing and evaluation, only a few samples of the graphs are shown for illustration purposes (see Figs. 10 thru 18.)

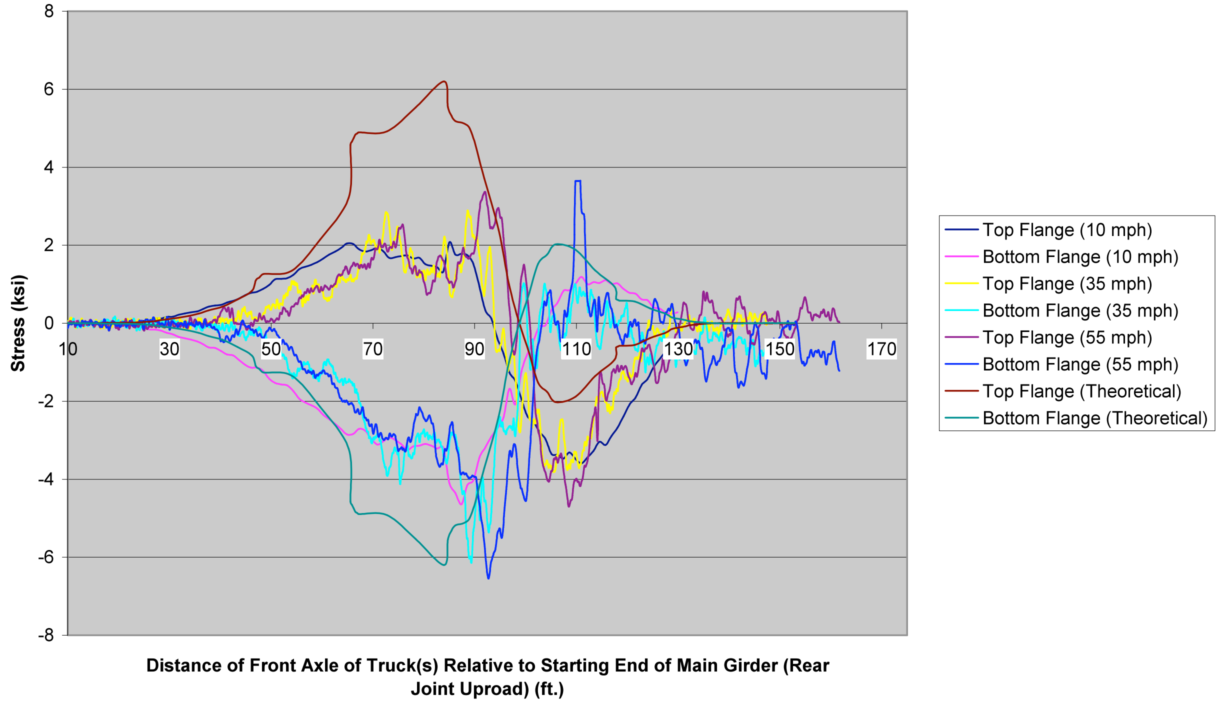


FIGURE 10: One Truck Southbound – SW Main Girder at Field Splice

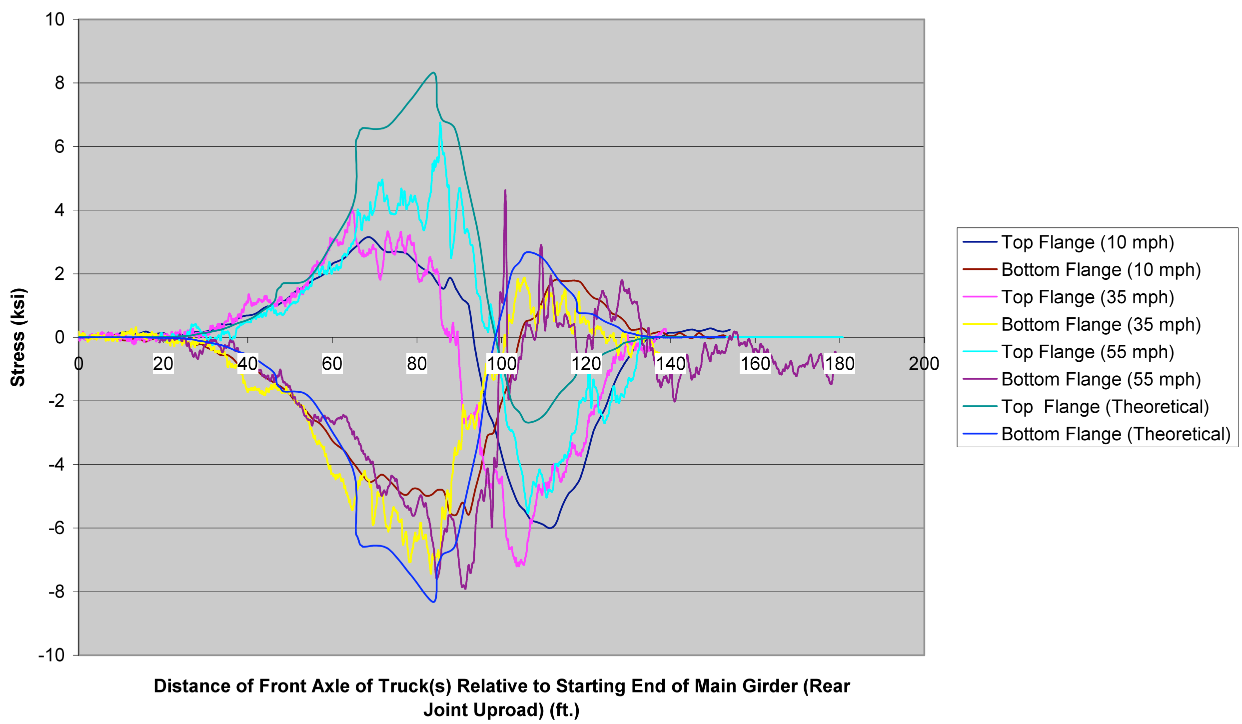


FIGURE 11: Two Trucks Side by Side Southbound – SW Main Girder at Field Splice

FIGURE 12: Two Trucks Back to Back Southbound – SW Main Girder at Field Splice

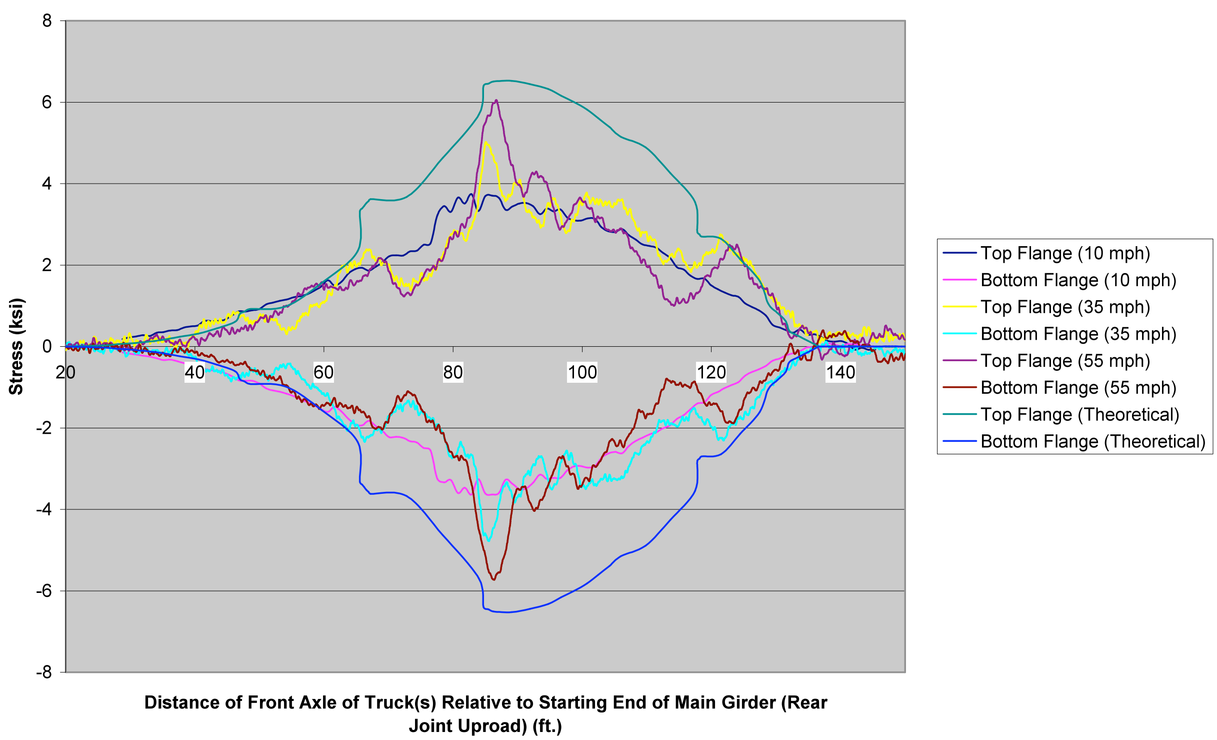


FIGURE 13: One Truck Southbound – SW Main Girder at Live Load Shoe

FIGURE 14: Two Trucks Back to Back Southbound – SW Main Girder at Live Load Shoe

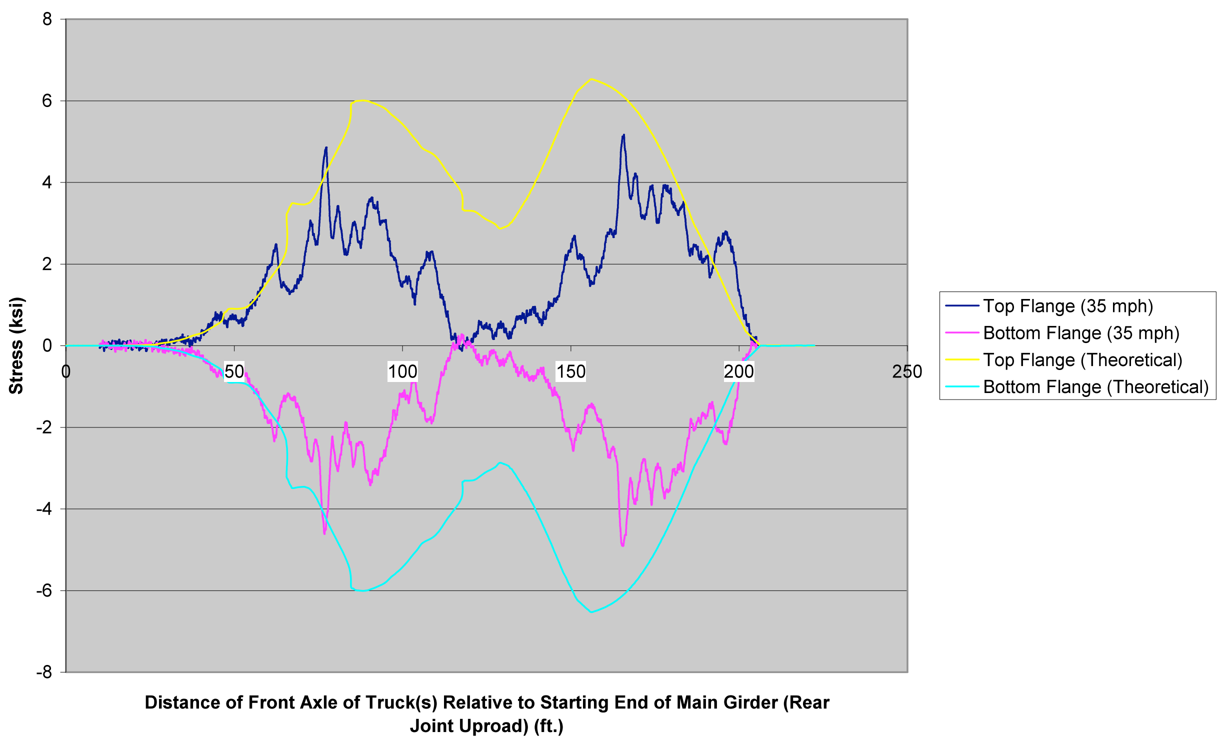


FIGURE 15: Two Trucks Back to Back Southbound – SW Main Girder at Live Load Shoe

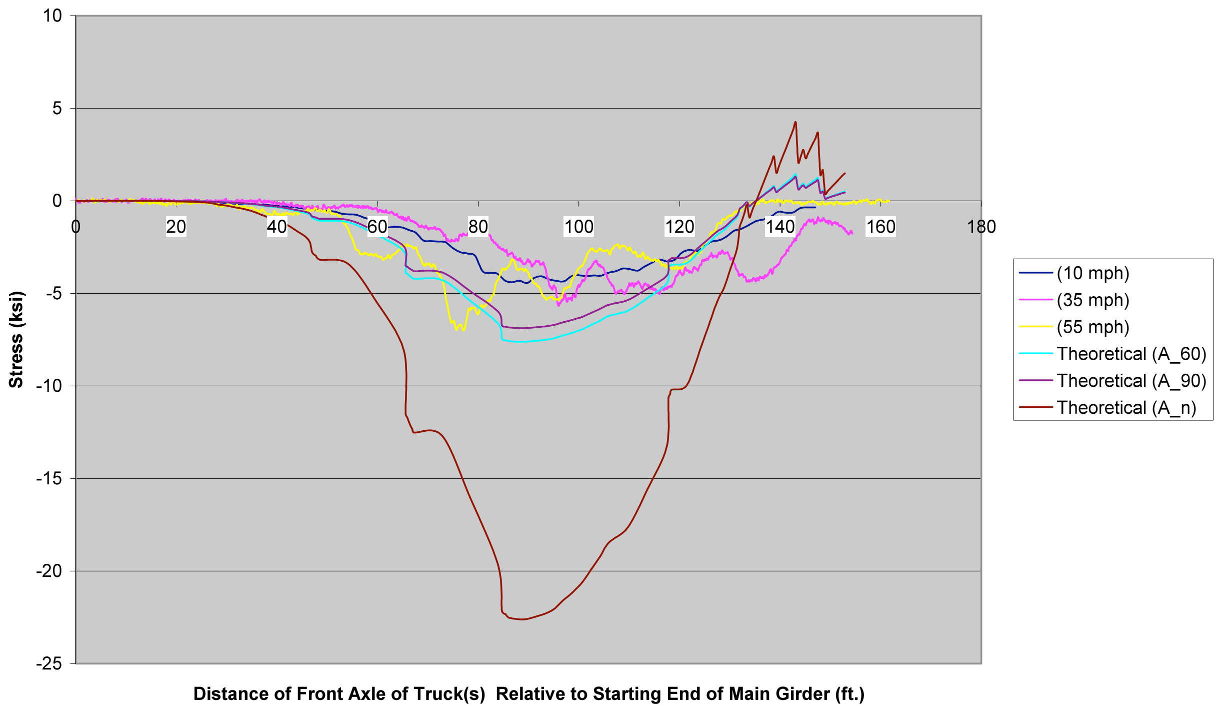


FIGURE 16: Two Trucks Side by Side Southbound – SW Live Load Shoe Bearing Stiffener

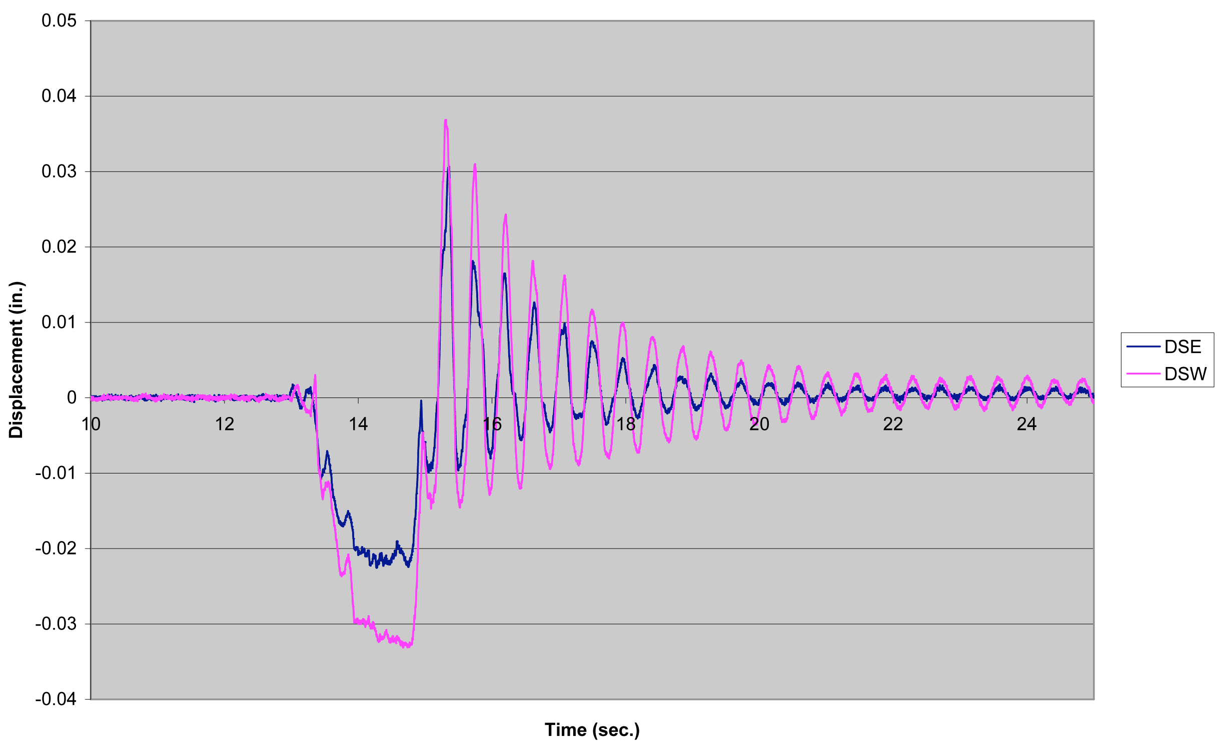


FIGURE 17: Vertical Deformation at South Leaf Bearing Stiffeners

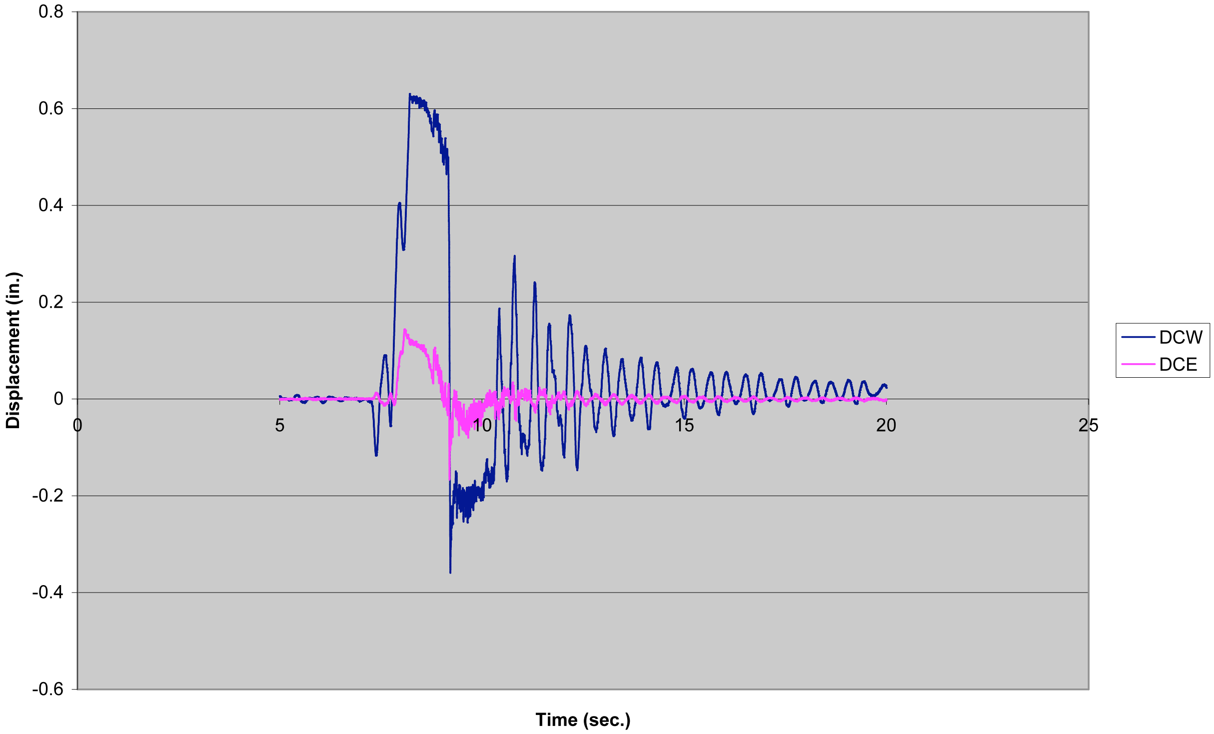


FIGURE 18: Differential Vertical Deformation at Joint between Bascule Leaves

Evaluation of Results

Main Girder Fatigue Stress Range

Based on the findings from the instrumentation and load testing, fatigue stress in the main girder flanges under legal loads (SU4 truck loading) was found not to be an immediate concern. However, it was found that fatigue stress could become a concern in the future if the bridge were maintained in poor condition with misaligned span locks and live load shoes, and if significant numbers of trucks in excess of legal loads were allowed to continue using the bridge over a prolonged period of time.

Figure 19 summarizes the measured and calculated stress ranges at each of the strain gage locations where either tensile stress or stress reversal was present. In general, the maximum stress range was found at the weak side of the field splice near the third point of the bascule span. This location is subject to stress reversal (i.e. the main girder is subject to both negative and positive flexure.) Also included in table are the dead load stresses, and maximum and minimum stresses measured from the strain gages for one truck, two trucks side by side, and two trucks back to back. It should be noted that where the magnitude of the computed dead load flange compression stresses are greater than the magnitude of the measured (live load) flange stresses in tension at a gage location, then the flange remains in compression thus eliminating the concern of fatigue stress and crack development at that location. As the lateral position of the trucks is different for the static and moving load cases, different load distribution factors were used to calculate the stresses.

Instrumentation and load testing yielded the following results. The maximum observed measured fatigue stress range in the main girder flanges was 12.3 ksi for two SU4 trucks located side by side and 7.9 ksi for one SU4 truck. For all load cases, the measured values of live load stress range were less than the calculated values, with the exception of one location where the results were found to be in error as a result of faulty strain gages. The results of the malfunctioned strain gages were disregarded.

Based on fatigue testing of different fatigue details performed by Lehigh University in the 1970s, a Category D fatigue detail, such as the riveted, built-up connections of the main girder flanges, can be subject to an infinite number of cycles, provided that the stress range does not exceed approximately 7 ksi (see Fig. 20, from *Bridge Fatigue Guide by John Fisher, Lehigh University, dated 1977*.) At the measured stress range of 7.9 ksi for a single-truck, the detail can be subject to approximately 4.0×10^6 cycles before fatigue crack development and propagation is expected, while at the measured stress range of 12.3 ksi for two-trucks side by side, the detail can be subject to only 1.2×10^6 cycles.

FIGURE 19: SUMMARY OF MAXIMUM STRESS RANGE VALUES AT MAIN GIRDER FLANGE STRAIN GAGE LOCATIONS (1), (2), (3)

Gage Name	Gage Location	# of Trucks	Measured	Theoretical (Calculated)	HS-20 (Calculated)	Allowable Stress Range 500,000 Cycles	Allowable Stress Range ∞ Cycles	Measured Live Load Stresses, σ_{LL}		Dead Load Stress(4)
			Max. Stress Range (ksi)	Max. Stress Range (ksi)	Max. Stress Range (ksi)	(F_{sr}) (ksi)	(F_{sr}) (ksi)	max. (+) (ksi)	min. (-) (ksi)	σ_{DL} (ksi)
SET	Top Flange of the SE Main Girder near the LLS	1	2.7	6.3	7.6	13	7	2.7	0	-4.4
		2(a)	3.5	8.5	11.7	13	7	3.5	0	-4.4
		2(b)	3.0	6.3	7.6	13	7	3.0	0	-4.4
ET	Top Flange of the SE Main Girder near the FS	1	5.1	8.2	8.6	13	7	3.3	-1.8	-2.2
		2(a)	7.6	10.9	13.3	13	7	4.6	-3	-2.2
		2(b)	5.0	8.2	8.6	13	7	3.0	-2	-2.2
EB	Bottom Flange of the SE Main Girder near the FS	1	7.1	8.2	8.6	13	7	1.8	-5.3	2.2
		2(a)	10.1	10.9	13.3	13	7	2.9	-7.2	2.2
		2(b)	6.4	8.2	8.6	13	7	1.8	-4.6	2.2
NET	Top Flange of the NE Main Girder near the LLS	1	5.0	6.5	7.3	13	7	5.0	0	-4.4
		2(a)	6.0	8.7	11.4	13	7	6.0	0	-4.4
		2(b)	4.0	6.5	7.3	13	7	4.0	0	-4.4
SWT	Top Flange of the SW Main Girder near the LLS	1	6.0	6.5	7.6	13	7	6.0	-1.8	-4.4
		2(a)	7.5	8.7	11.7	13	7	7.5	-3	-4.4
		2(b)	5.0	6.5	7.6	13	7	5.0	-2	-4.4
WT	Top Flange of the SW Main Girder near the FS	1	7.9	8.2	8.6	13	7	3.3	-4.6	-2.2
		2(a)	12.0	11.0	13.3	13	7	6.6	-5.4	-2.2
		2(b)	6.8	8.2	8.6	13	7	3.2	-3.6	-2.2
WB	Bottom Flange of the SW Main Girder near the FS	1	7.6	8.2	8.6	13	7	1.2	-6.4	2.2
		2(a)	12.3	11.0	13.3	13	7	4.6	-7.7	2.2
		2(b)	8.9	8.2	8.6	13	7	1.9	-7	2.2
NWT	Top Flange of the NW Main Girder near the LLS	1	4.0	6.3	7.3	13	7	4.0	0	-4.4
		2(a)	5.3	8.4	11.4	13	7	5.3	0	-4.4
		2(b)	4.0	6.3	7.3	13	7	4.0	0	-4.4

Notes: (1) Positive stress indicates tension; negative stress indicates compression. (2) The stresses at the flanges were obtained by averaging the stresses measured from the load test at each flange location (e.g. The stresses at SET1, SET2, SET3 and SET4 were averaged to obtain one stress at the top flange of the SE main girder). (2) 2(a) represents load case of two trucks side by side while 2(b) represents load case of two trucks back to back. (3) Only locations where the flanges that are subject to net tension or stress reversal are shown. Locations where the flanges are subject to net compression are not expected to develop or propagate fatigue cracks. (4) If the magnitude of the DL stress in compression is greater than the measured live load stress in tension at a flange location, then that flange is not subject to net tension.

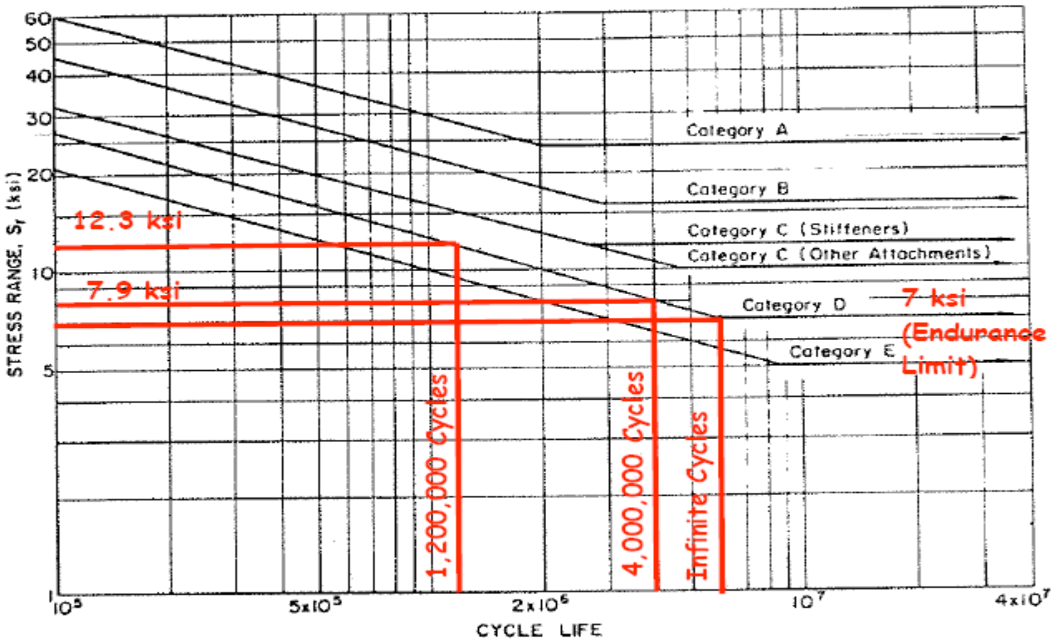


FIGURE 20: Design Stress Range Curves from Lehigh Fatigue Testing.

However, of the more than 4,200 trucks crossing the bridge on a daily basis, not all trucks are heavy enough to cause fatigue damage. The traffic counting equipment used by the Department records all vehicles in excess of 5,000 pounds as a truck. In 1970, a nationwide loadometer survey was performed by FHWA to determine the frequency distribution of trucks traveling the highways throughout the United States and proportion of trucks causing fatigue damage (see Figs. 21 and 22, from *Bridge Fatigue Guide by John Fisher, Lehigh University, dated 1977*.)

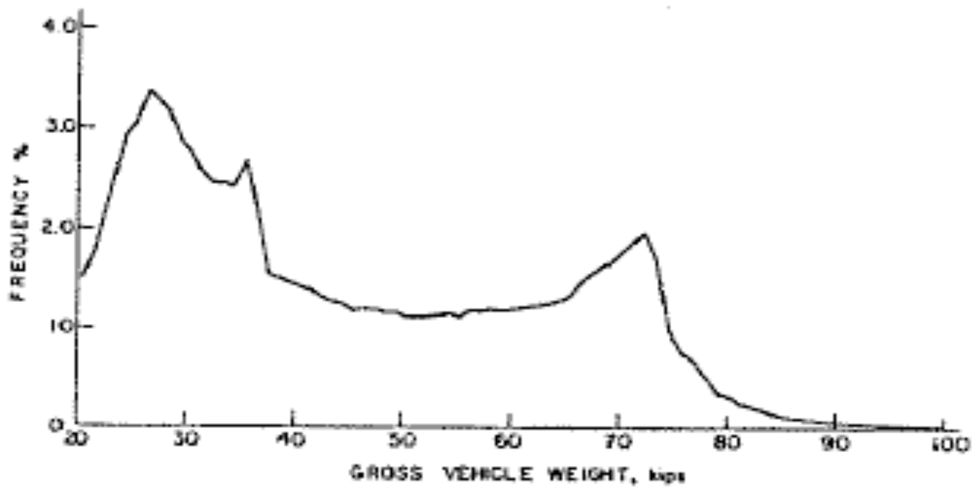


FIGURE 21: 1970 Nationwide Loadometer Survey.

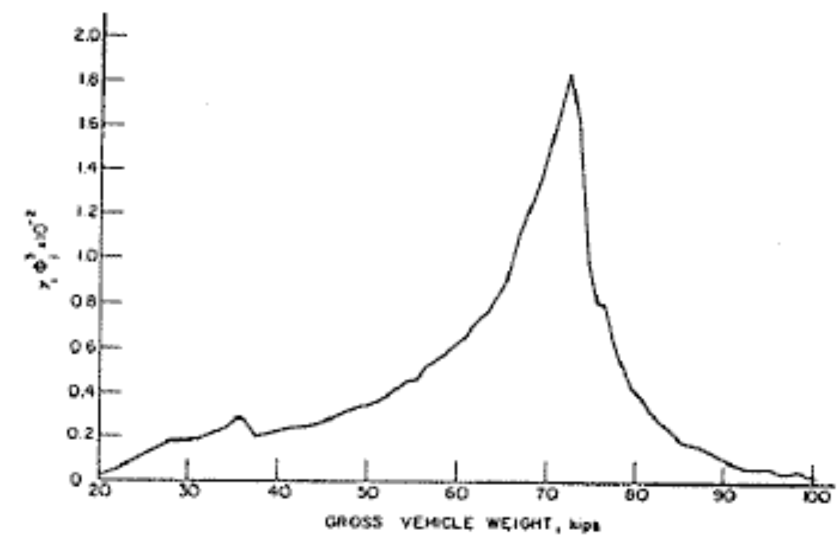


FIGURE 22: Probable Fatigue Damage by Truck Weight.

The research by Lehigh determined that generally only 10% to 15% of the total trucks actually cause the majority of fatigue damage. However, it is generally recognized that since 1970, the character of vehicular traffic has changed significantly and as such may not be representative of vehicle frequency weight distribution today. Furthermore, the results of the loadometer study may not be representative of all highways. Without a current site-specific loadometer survey it is not possible with certainty to determine the actual fatigue damage caused by current loading. Furthermore, without a loading history, it is not possible to determine with certainty the fatigue damage that has occurred to date, nor the remaining fatigue life.

Despite the lack of available information, a qualitative assessment of the measured stress range provided useful information in making the decision whether to restore truck traffic to the bridge. For illustration, if it were assumed that the heavy trucks (i.e. trucks similar in weight to the SU4 test trucks that produce stresses around 8 ksi) made up approximately 10% of the total trucks, the bridge would potentially be subject to approximately 200 heavy truck cycles per day or approximately 75,000 heavy truck cycles per year. At this frequency, the fatigue life (4,000,000 cycles) would not be reached until approximately 50 years of use. Conversely, if the heavy trucks made up approximately 25% of the total trucks, the bridge would be subject to approximately 500 heavy truck cycles per day or 200,000 heavy truck cycles per year. At this frequency, the fatigue threshold would be reached in approximately only 20 years of use.

As fatigue damage is cumulative and higher stress ranges produce more fatigue damage, heavier single trucks (i.e. trucks in excess of legal loads) and load cases where two heavy trucks are on the span in a position to produce higher stress range will reduce the remaining fatigue life and shorten the time to which fatigue cracks might develop. Although two heavy trucks on the span at the same time produce higher stress range, the number of cycles of this load case is significantly lower than that for a single truck. Furthermore, most of the observed trucks that are expected to be producing the fatigue damage usually return empty and thus the likelihood of two heavy trucks positioned for maximum affect is likely not as severe as that tested. The asset management contractor responsible for maintaining and operating the bridge, had reported that prior to removing trucks from the bridge, dump trucks crossing the bridge, similar to the test trucks, were observed to be loaded significantly more than the test trucks and as such were believed to be overloaded. As such, it is possible that the single trucks were causing more fatigue damage than that measured. Furthermore, if trucks were restored to the bridge, an aggressive program to curtail illegal overload trucks would need to be implemented to ensure that further fatigue damage was minimized.

Although, the bridge had been in service for approximately 50 years, the borrow pit producing the recent heavy truck traffic had reportedly been in service for less than 10 years and the observed damage caused by heavy loading had only recently appeared (i.e. within the last five years.) Based on the load test results, single trucks with a gross vehicle weight of 35 tons produce stress range at our below the endurance limit and thus did not cause fatigue damage. Prior to 10 years ago, the traffic volumes and corresponding truck traffic was reportedly low and thus the likelihood of two heavy trucks on the span at the same time was low and the volume of single overload trucks was also low.

Higher values of stress range were observed on the west main girders, which had span locks that were found to be significantly more out of adjustment than the span locks on the east main girders. The maximum stress range on the east main girders was 7.1 ksi, which was close to the fatigue endurance limit (i.e. the stress limit at which the detail can be subject to an infinite number of cycles.) Although the maximum stress range for the west main girders was slightly higher at around 7.9 ksi, the stress range for both main girders was expected to decrease to below 7 ksi once the span lock guides and receivers were replaced with pre-loaded and self-adjusted proprietary Cushionlok guides and receivers (manufactured by Steward Machine Co., Birmingham, Alabama.)

The AASHTO Standard Specifications recognizes the risk of developing fatigue cracks in non-redundant members whose fracture could result in the catastrophic collapse of the span. For these members, AASHTO arbitrarily reduced the allowable fatigue stress range by 20%, which reduced the endurance limit from 7 ksi to 5 ksi for non-redundant members. However, although the main girders on bascule spans of this configuration are generally considered non-redundant, the riveted built-up fabrication introduces internal redundancy (i.e. a fatigue crack in one cover plate would not propagate into the other cover plates, flange angles or web and thus a fatigue crack would not expect to initiate a catastrophic collapse.) As such, the flanges of the riveted built-up main girders can be considered redundant and the arbitrary 20% reduction in allowable fatigue stress range for non-redundant members not applied.

Even though the main girders did not show evidence of fatigue damage, based on the lack of accurate available historical data on the bridge loading and because a relatively economical means to improve the fatigue resistance of the main girders, without strengthening, was available, the decision was made to improve the fatigue resistance of the main girders by replacing the main girder top and bottom flange rivets with high strength bolts. This improved the fatigue detail from a Category D to a Category B with corresponding significant increase in endurance limit (7 ksi to 16 ksi), thus eliminating the concern of fatigue of the main girders.

Bearing Stiffener Loading and Displacements

The measured compression stresses in the live load shoe bearing stiffeners were found to be significantly lower than those computed using AASHTO design methodology. This is likely because the live load shoe bearing reaction is being resisted by a much larger effective bearing area than assumed in design. A portion of the area of the web of the main girder along with the area of the bearing stiffener angles and fill plates is likely resisting the bearing reaction instead of just the bearing stiffeners as assumed in the AASHTO methodology. Stress values at the bearing stiffeners were recomputed using an assumed length of web with the stresses distributing upward from the live load shoe line contact at a 60° to 90° angle. Using this distribution area, the recomputed stresses were much closer to the measured stresses. Distribution of the reaction into the web of the girders takes place by way of bearing of the web onto the bottom flange cover plates and by way of shear transfer through the bottom flange angle rivets.

The vertical displacements of the main girders at the live load shoes were generally found to be small (i.e. from 0.010” at NW, 0.015” at NE, 0.020” at SE and 0.030” at SW Live Load Shoes respectively.) The vertical displacement is a result of a combination of poorly adjusted live load shoes, compression of the slight gaps between the bottom flange cover plates, live load shoe and associated shims, and compression of the gap between the live load shoe bearing stiffener and the top of the bottom flange (see Fig. 23.) Slight closing of these gaps can be visually observed as traffic crosses the span. The gaps are a result of wear between these components following repeated heavy loading. The lack of proper pretension of the live load shoe mounting bolts was found to contribute

FIGURE 23: Live Load Shoe Typical Condition

to the vibration and corresponding fretting corrosion between these components.

It was recommended that the worn bearing stiffeners be replaced with new heavier bearing stiffeners designed to resist current loads. Furthermore, to prevent future fretting corrosion, it was recommended that the base of the stiffeners include a tab plate bolted to the bottom flange. The significantly worn main girder bottom flange cover plates and live load shoe assembly also be replaced. Once the bottom flange rivets were replaced with high strength bolts, the pretension of the bolts prevent future fretting between the various components.

Leaf Differential Displacements

The measured differential vertical displacements between the two leaves at the center of the bascule span were indicative of the worn and misadjusted span lock guide and receiver shoes. The west side span locks (southbound side) were found to be significantly more out of adjustment than the east side span locks. The southbound loading was observed to be generally greater than the northbound loading as the numerous heavy trucks leaving the borrow pit north of the bridge would cross the bridge fully loaded in the southbound direction and return empty in the northbound direction. The maximum differential deflection at the west side span lock was approximately 5/8” while the differential deflection at the east side span lock was approximately 1/8”. The large difference in vertical displacements between the west and east sides of the leaf explains the consistently larger stress range in the west main girders than the east main girders.

The deflection of the lock bar under the load transfer reaction on the lock bar was computed and was found to be small (i.e. 0.01” which is an order of magnitude smaller than the measured displacements) and thus the differential deflection was found to be primarily the result of poorly adjusted span lock guides and receivers.

The difference in stress range between the east and west main girders due to the difference in differential deflection at the east and west span lock assemblies was found to be significant. Stress range at both the live load shoes and field splice were found to be 25% to 50% greater for the west main girders than the east main girders.

It was recommended that the span lock assemblies be replaced with new assemblies that include new preloaded, self-adjusting guides and receivers, such as the proprietary Cushionlok guide and receivers.

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