Report of Findings

Load Testing, Model Calibration and Analysis of the Oceanic Bridge Over the Navesink River

Prepared For:

Monmouth County Engineering Department
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1 Introduction

1.1 Background

The Oceanic Bridge spans the Navesink River in Monmouth County New Jersey and was constructed in 1939 and rehabilitated in 1970. The structure consists of 57 spans with a 98 foot double leaf bascule span over the navigable channel (Figure 1). Due to significant deterioration throughout the bridge it is currently posted for three (3) tons, which is a detriment to the public as it significantly hampers the movement of emergency vehicles, school buses and commerce in the surrounding areas. Given its condition it is slated for replacement; however, the length of the project delivery process is such that it will be required to remain in service for at least 5 to 10 years.

Although Monmouth (County) is fully aware of the need for some level of rehabilitation to increase the posting to a more acceptable level, given the bridge's finite remaining life it is important that only interventions necessary to maintain safe operation for a decade be undertaken. The County fully appreciates that the significant conservatism implicit in conventional engineering assessment approaches (which recognizes various uncertainties) may result in unnecessary, costly and in some cases, even counterproductive rehabilitation efforts. As a result, the County has elected to have a 'best practices' structural identification (St-Id) carried out – in parallel with conventional assessment approaches – to more precisely characterize the bridge's load carrying capacity and key vulnerabilities. Such an approach will offer the most reliable establishment of the actual performance and safety of the bridge and will allow for more effective and efficient repair/retrofit interventions to guarantee a safe life of at least a decade. To add a further measure of safety during the remaining service life, the County has also expressed an interest in employing contemporary sensor and health monitoring technology. Such an approach will allow the operational loads and safety to be definitively established and tracked, and real-time alerts to be provided to the County Engineer if any anomalies in either the loading or behavior occur.

Figure 1 – Oceanic Bridge

1.2 Structural Identification (St-Id)

The paradigm of St-Id aims to develop reliable estimates of the performance and vulnerability of structural systems through the correlation of mathematical/simulation models with experimental observations/data. To organize the diverse paradigm of St-Id, the ASCE St-Id of Constructed Systems Committee adopted the six steps shown in Figure 2 (Aktan and Moon 2005). As evidenced by these diverse steps, a team of multi-disciplinary experts is often required to fully achieve the potential of any application related to decision-support or vulnerability and risk assessment. The application must be driven by experts with domain knowledge related to constructed systems and their asset management (applied systems analysis, heuristics), and should be built around a set of carefully designed objectives that are both attainable and will be of direct and demonstrable benefit. Although this is a necessary condition, successful applications of St-Id to constructed systems require additional expertise related to: modeling (analytical, numerical); experimentation (observations, sensing, and data acquisition); data processing (error screening, feature extraction, etc.); correlation of model and experiment (model selection, parameter identification.); and decision-support (parametric studies, scenario analyses, risk assessment, etc.).

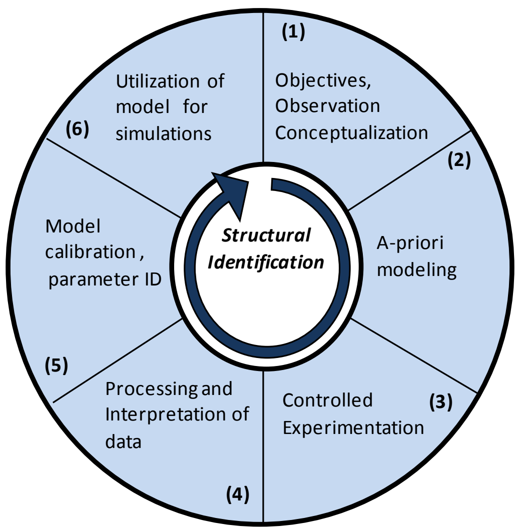


Figure 2– Six Steps of Structural Identification

1.3 Objectives and Scope

The objective of this project was to perform a 'best practices' St-Id of the Oceanic Bridge to:

1) Reliably establish the current as-is safe load carrying capacity
2) Identify key vulnerabilities that may lead to unacceptable performance in the next decade
3) Make recommendations related to appropriate retrofit/repair approaches and long-term monitoring to ensure safe operation throughout the next decade.

It is understood that the County is interested in carrying out the minimum required interventions to maintain the safe operation of the Oceanic Bridge for 5 to 10 years with a posting that will permit the crossing of school buses and emergency vehicles (responding to a call). As guided by the County, the performance of the grid deck, purlins, stringers, and bridge railings (including the supporting cantilever brackets) are outside the scope of the detailed assessment.

2 Bridge Condition

The condition of the primary and secondary members of the Oceanic Bridge's superstructure was investigated and documented by Hardesty and Hannover (H&H) in the form of field notes, and provided to Pennoni Associates Inc. (Pennoni) and Intelligent Infrastructure Solutions, LLC (IIS). To better conceptualize this information, the deterioration was plotted by IIS on a 3D CAD model of the superstructure and keyed to representative photographs. Figure 3 and Figure 4 show these models for the east and west bascule girders, respectively. In these models, the sections indicated in yellow refer to section where some level of deterioration was noted by H&H. In addition to the deterioration of the superstructure members, it is particularly important to note that it was observed that the northeast live load bearing was not properly seating (approximately ¾ in gap) when the Pennoni-IIS team performed their cursory visual inspection. Figure 5 shows a photograph of this bearing in the condition observed during the visual inspection. The presence of this gap significantly influences the load paths of the leaf, induces potentially damaging torsional actions, and drastically increases the risk of overstressing of the bascule girder which was properly seating. As a result, the Pennoni-IIS team immediately informed the County of this issue and the County promptly mitigated the vulnerability by shimming the live load bearing to ensure proper seating.

Figure 3 - Mapping of deterioration and representative photographs for the east bascule girders

Figure 4 - Mapping of deterioration and representative photographs for the west bascule girders

Figure 5 - Gap in the northeast live load bearing observed during visual inspection

3 Geometric Modeling

In order to fully understand the geometry of the Oceanic Bridge, a three dimensional model was created based on the available construction drawings obtained from the County Engineering office. The 3D model was built in both AutoCAD and Google Sketchup to allow for proper error screening. These models served as excellent tools to understand and conceptualize the interaction of the components and load paths of the structure. In addition, these models were employed to (1) visualize deterioration (see Figure 3 and Figure 4), (2) aid in instrumentation planning, and (3) establish the geometry of the preliminary and refined finite element models. The models are shown in Figure 6 and Figure 7.

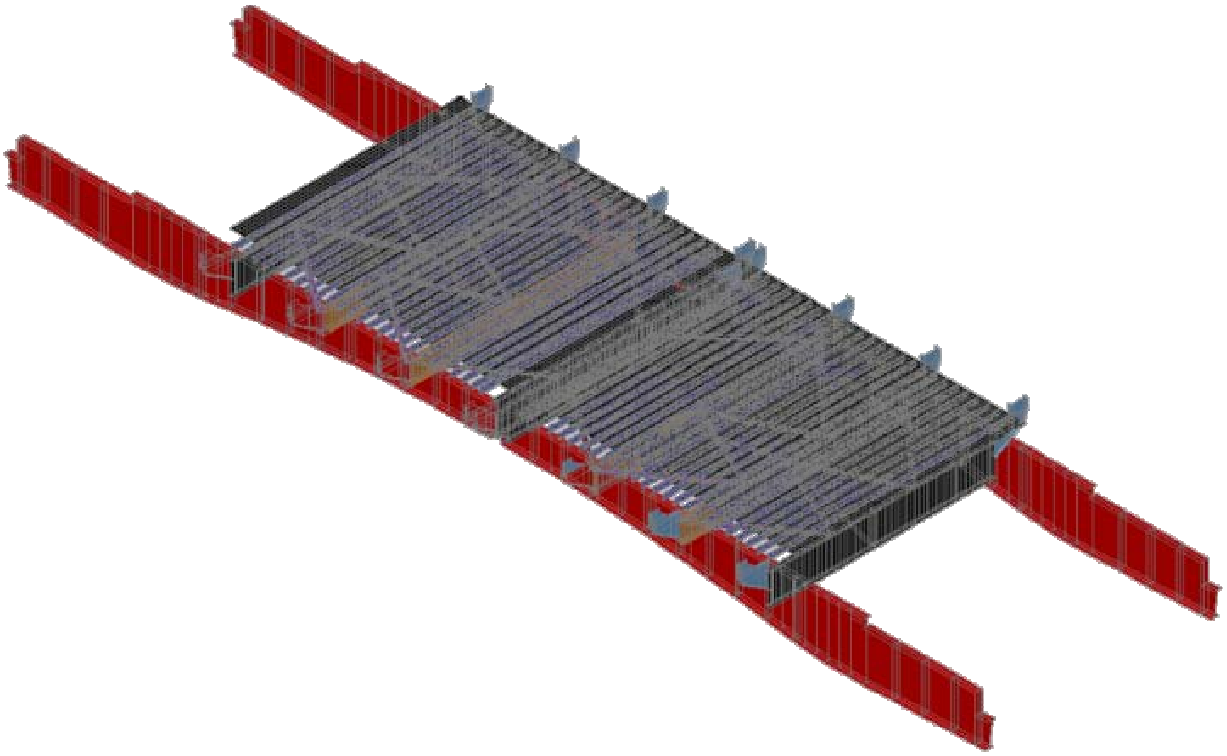


Figure 6 - 3D AutoCAD Model

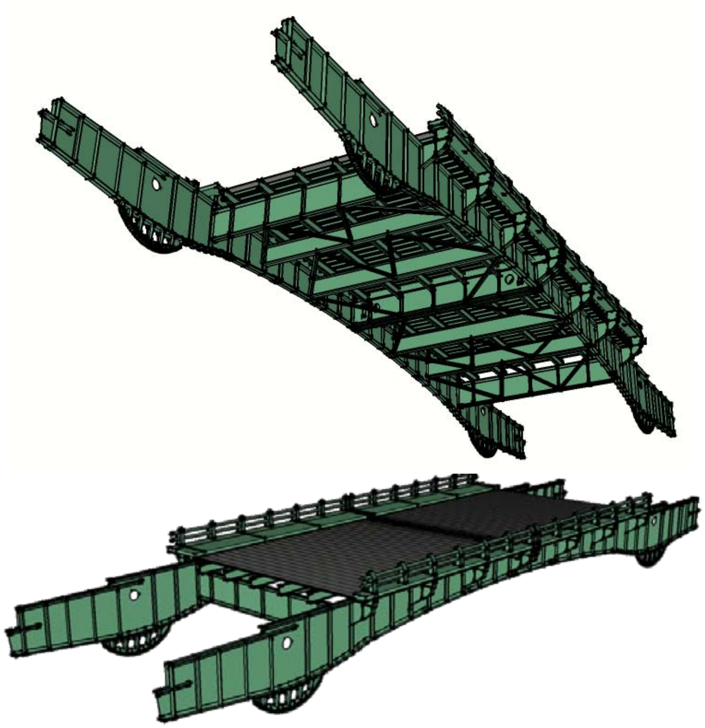


Figure 7 - 3D Sketchup Model

4 Preliminary Finite Element Modeling

4.1 Goals and Methodology

The goal of any a-priori model is to roughly approximate the responses from a structure at given load stages in order to (1) identify critical elements, (2) guide the instrumentation planning, and (3) provide a baseline for comparison during loading testing to ensure safety. This approach includes three primary steps. First a benchmark finite element (FE) model, using the proposed modeling technique, is developed and compared to closed-form solutions. This allows both the general modeling approaches ability to simulate the relevant responses (flexure, shear, etc.) and an appropriate mesh size to be selected. Once the modeling approach and mesh have been validated, a model of the structure of interest is developed. This model is then subjected to a series of simple loading cases (point load, uniform load, etc.) and the resulting stress and displacement patterns are examined to identify and cleanse any blatant input errors (e.g. model connectivity). The final step in the process is to perform a series of parametric studies to identify and bound critical responses. This is carried out by first identifying the principal sources of uncertainty (e.g. boundary/continuity conditions, geometric properties of deteriorated members, etc.) and then varying associated parameters to establish their influence on the structure's response.

Once completed the preliminary model and the results of the parametric studies provide a powerful tool to guide the remainder of the St-Id. For example, identifying the most critical responses and sources of uncertainty within the structure allows for the development of a robust instrumentation plan. In addition, having approximate response quantities (and their bounds) available during the load test provides a means to compare and assess whether the bridge is responding within the expected range throughout all testing phases. This ensures safety of both the engineers on site and the structure. Finally, this a-priori model serves as an ideal starting point for further refinement and calibration following the experimentation, data processing and data interpretation phases.

4.2 Model Construction

Using simple benchmark models together with closed-closed-form solutions, the use of four node quadrilateral shell elements with six degrees of freedom per node (employed using SAP2000) was shown to sufficiently capture both the in-plane and out-of-plane response of the primary members of the Oceanic Bridge. The nominal mesh size selected for the model was 4 in. by 4 in. While this mesh was far more refined than required to simulate the intact response of the structural members, this level of refinement was desired so that the deterioration observed could be approximated with reasonable accuracy.

To construct an FE model of the Oceanic Bridge, the geometry was established in AutoCAD (using the geometric models previously discussed) and imported into SAP2000. Once the basic 3D geometry was imported, the model was meshed using the nominal mesh size. In addition, the model was examined to locate areas of stress concentrations (member connections, stiffeners, boundary conditions, etc.) and the mesh in these regions was further refined down to 0.5 in. by 0.5 in. to allow the expected complex stress distributions to be properly simulated. Once the meshing was completed, the material properties, boundary and continuity (where applicable) conditions were assigned and several simple analysis cases were conducted to error screen the model. The basic geometry of the north leaf model is shown in Figure 8.

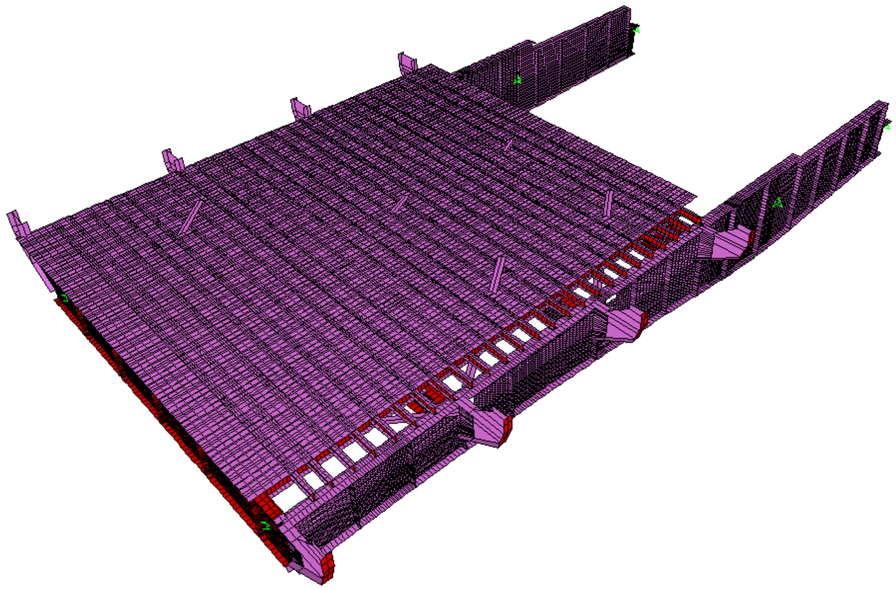


Figure 8 - 3D SAP2000 Model – North Leaf

4.3 Preliminary Simulations

To guide the parametric studies to identify the influence of various uncertainties, a live load consisting of two 36 ton tri-axle dump trucks was employed. The tire loads were represented as point loads on the surface of the shell element grid deck and the spacing of the truck tires was estimated for the preliminary model based on past experience (following the test, the actual geometry of the tri-axle trucks employed in the test was used). The 36 ton trucks were positioned on the model in order to produce the maximum flexural and shear responses in the primary members, and the member geometry (web thickness) and boundary conditions (live load bearings, shear transfer between spans) were varied to assess their influence. These analyses indicated that the significant uncertainty present dominated the simulation results, and thus the bounds for the critical responses were quite broad and were not sufficient to guide the load test. As a result, a judgment was made to refine the preliminary FE model prior to conducting the load test.

Figure 9 - UB40 access truck

5 Finite Element Refinement

To refine the preliminary FE model two sources of information were employed. First the results of the H&H condition assessment were directly incorporated to modify the primary and secondary structural members. Second, preliminary strain data was used to provide information about the shear transfer between spans and the performance of the live load bearings. Following the installation of strain gages (see Section 7.1 for instrumentation plan) a cursory load test was performed using the 40 ton UB40 access truck (Figure 9), which was on site to aid in the instrumentation. The following sections outline the modifications made to the preliminary FE model based on the observations of H&H and the cursory load test.

5.1 General Calibration Procedure – Heuristic Method

The traditional method of FE model calibration is conducted by manually minimizing the error between the experimental results and model output. Based on the initial discrepancies between results and model output, adjustments to the model are made, varying material properties, geometry, boundary/continuity conditions or part interaction to attempt to reduce the error. These adjustments are based on heuristic knowledge as well as the in-situ properties of the structure being calibrated. In the case of the Oceanic Bridge, uncertainty was present in regards to the (1) condition and activation of the span locks, (2) the performance of the live load anchors on both bascule leaves, and (3) the deterioration of primary and secondary members.

5.2 Modeling Deterioration

The deterioration of the Oceanic Bridge was established by H&H through visual inspection and provided to Pennoni and IIS via a series of member elevation views with areas of deterioration drawn by hand. In addition, H&H also provided an estimate of the percentage of section loss for each area of deterioration. As discussed in Section 2, these areas of section loss and corrosion were mapped onto a 3D CAD model to aid in conceptualization of the current state of the bridge. Using this 3D CAD model, the locations of deterioration were transferred to the FE model.

To simulate the deterioration within the model, stiffness modifiers were applied to shell elements associated with deteriorated regions. The stiffness multipliers included were based on the information provided by H&H, i.e., the stiffness reduction factors were applied for 25%, 50%, and 75% section loss. This approach essentially reduces the moment of inertia and cross sectional area in accordance with the amount of section loss. For areas where 100% section loss was noted, the entire shell element was removed from the model in an area approximately the size of the area noted in the field notes. Figure 10 and Figure 11 show the deteriorated sections as well as the elements that were removed from the FE model for the north leaf of the Oceanic Bridge.

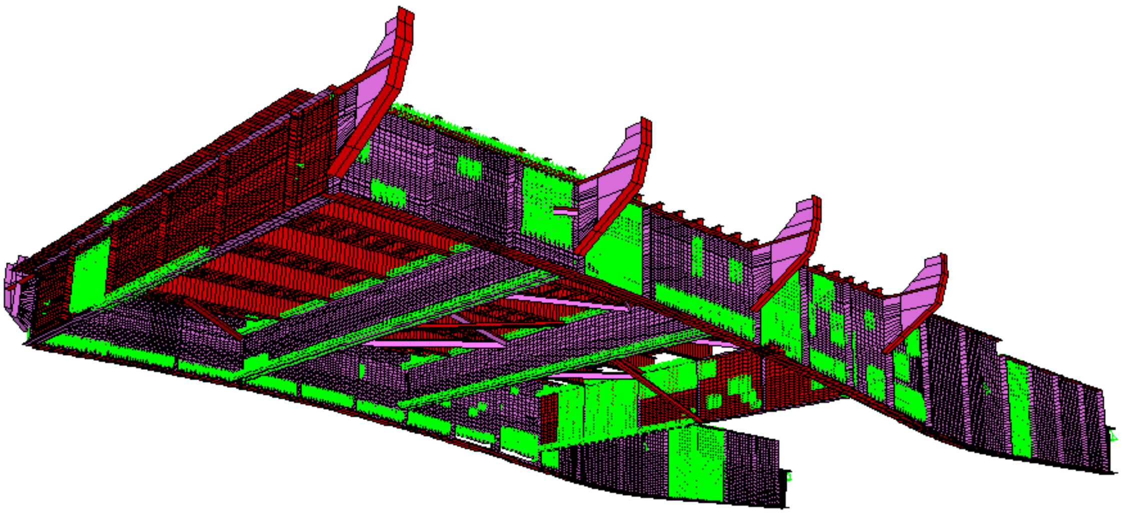


Figure 10 - Deteriorated FE Model – Stiffness Modifiers

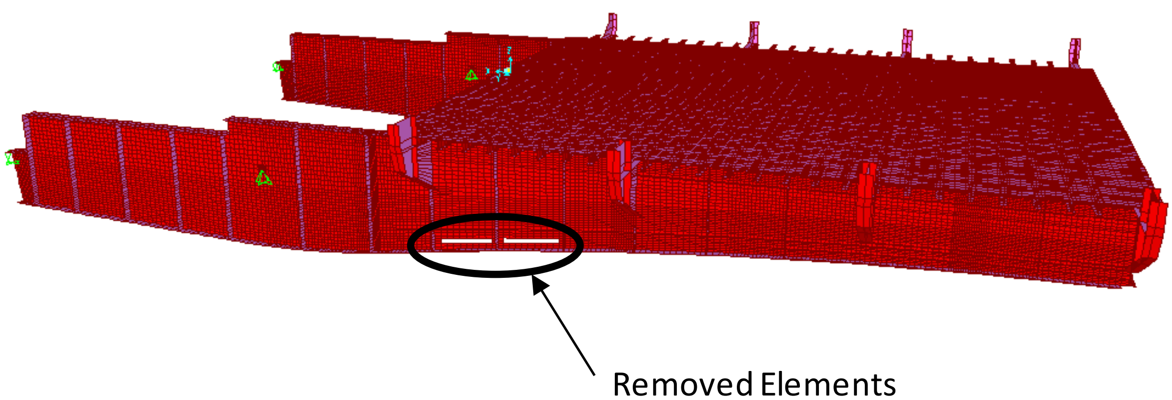


Figure 11 - Deteriorated FE Model – Removed Elements

5.3 Boundary and Continuity Conditions

The second modification made to the preliminary as built model involved how the boundary conditions at the live load supports and the continuity conditions at the span locks were modeled. In the as built model, one of the live load support conditions employed was a pin, which restrained translation in the x, y, z directions but allowed rotation around these axes. The restraint in the x-direction induced large compressive forces into the bascule girder that could not be justified after inspecting the live load supports, and were not consistent with the results of the cursory load test. Removing the restraint in the x-direction alleviated the large compressive stresses and drastically reduced the bounds of the responses in the rear area of the bascule girders. Also, the preliminary analyses indicated that the bascule girder stresses were very sensitive to the level of shear transfer simulated between the two leafs. Based on the strains measured during the cursory load test, the continuity conditions at the span locks were calibrated. The resulting continuity condition allowed for a maximum shear transfer of slightly less than 20% between spans.

6 Analytical Load Ratings Using the Refined FE Model

Once the FE model was refined based on the cursory load test and visual inspection results, it was employed to develop analytical ratings using the Load Resistance Factor Rating (LRFR), Load Factor Rating (LFR) and Allowable Stress Rating (ASR) methodologies. The following sections detail the procedures employed to develop these ratings.

6.1 AASHTO LRFR Procedure

Equation 1 was used in calculating the flexural rating factor and Equation 2 is used to calculate the shear rating factor as per the AASHTO LRFR manual.

$$RF = \frac{(\phi * \phi c * \phi s * Mn) - (\gamma c * M_{DC}) - (\gamma_w * M_{DW})}{\gamma_{LL} * M_{LL}(1+I)} \quad \textbf{(1)}$$

$$RF = \frac{(\phi * \phi c * \phi s * Vc) - (\gamma c * V_{DC}) - (\gamma_w * V_{DW})}{\gamma_{LL} * V_{LL}(1+I)} \quad \textbf{(2)}$$

where,

- *RF* is defined as either the inventory or operating rating factor depending on the modification factors chosen, i.e. γ_{DC}, γ_{DW}, and γ_{LL}.
- *I* is the impact factor to account for dynamic loading of trucks and truck-bridge interaction.
- γ_{DC}, γ_{DW}, and $\gamma_{LL,}$ are factors which increase the dead load and live load actions (moments and shears) to account for random uncertainty and other factors such as average daily truck traffic (provided in Appendix A.6.2 Table 6-3 and 6-4 of the AASHTO LRFR manual).
- φ_c and φ_s are factors which reduce the resistance based on the observed condition of the structure and the structural system, respectively.

It is important to note that the many factors included within the LRFR procedure to either increase the demand actions or decrease the estimated capacity are intended to account, in part, for the uncertainty associated with very simplistic mechanistic models and the difficulty in simulating the influence of deterioration with such models. As a result, the use of such factors together with more refined modeling approaches (as employed here) can be expected to be quite conservative.

In addition, since the AASHTO LRFR load rating procedure was intended for use with mechanistic models, it is based on actions (moments and shears) and not on stresses. Since the refined FE model developed does not readily output actions, this load rating procedure was modified to use stresses. Specifically, the maximum stress within the flange of the member was substituted for moment demand and the maximum stress within the web of the member was substituted for shear demand. Similarly the capacity terms were taken as the AASHTO defined material strengths. This modification was also made for the LFR procedure described in the following section. Since this approach does not rely on assumed strain distributions within the members (e.g. plane section remain plane) it is well-suited for the rating of highly deteriorated members such as those found within the Oceanic Bridge.

6.2 LFR Load Rating Procedure

The LFR procedure differs from the LRFR procedure in that it only uses modifiers for the load or demand terms. The rating factors from this method were calculated using the following equations as described in NCHRP (1998)

$$RF = \frac{(Mn) - (\gamma_C * M_{DC}) - (\gamma_W * M_{DW})}{\gamma_{LL} * M_{LL}(1 + I)} \tag{3}$$

$$RF = \frac{(Vc) - (\gamma_C * V_{DC}) - (\gamma_W * V_{DW})}{\gamma_{LL} * V_{LL}(1 + I)} \tag{4}$$

6.3 ASR Load Rating Procedure

The ASR procedure uses only an impact factor modification on the demand stresses. The rating factor from this method is calculated using the following equation as per NCHRP (1998)

$$RF = \frac{(\text{Allowable maximum stress}) - (\text{Nominal dead load stress})}{\text{Nominal live load stress plus impact}} \tag{5}$$

6.4 Analytical Load Ratings

The Oceanic Bridge bascule girders and floor beams were rated using two (one per lane) 36 ton trucks with the configuration shown in Figure 12. These load ratings were carried out using both the AASHTO recommended impact factor of 1.33 and the theoretical maximum impact factor of 2.0. Table 1 through Table 6 provides the ratings factors for LRFR (flexure and shear), LFR (flexure and shear) and ASR (flexure and shear), respectively. The calculations used to obtain these rating factors are detailed in Appendices A and B.

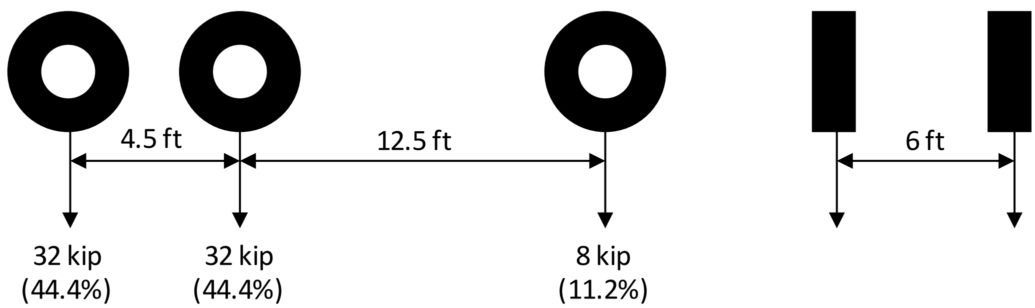


Figure 12 – Truck configuration used for analytical ratings

Table 1 - Load Ratings (tons) for Oceanic Bridge – Flexure – LRFR – Legal Load

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	42	28
NE Bascule Girder	53	35
SW Bascule Girder	53	35
SE Bascule Girder	71	46
North Leaf Floor beam 1	30	20
North Leaf Floor beam 2	34	22
North Leaf Floor beam 3	34	22
North Leaf Floor beam 4	31	21

Table 2 - Load Ratings (tons) for Oceanic Bridge – Shear – LRFR – Legal Load

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	55	36
NE Bascule Girder	64	42
SW Bascule Girder	77	51
SE Bascule Girder	77	51
North Leaf Floor beam 1	110	73
North Leaf Floor beam 2	147	97
North Leaf Floor beam 3	147	97
North Leaf Floor beam 4	110	73

Table 3 - Load Ratings (tons) for Oceanic Bridge – Flexure – LFR – Legal Load

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	50	33
NE Bascule Girder	63	41
SW Bascule Girder	63	41
SE Bascule Girder	83	55
North Leaf Floor beam 1	36	24
North Leaf Floor beam 2	40	27
North Leaf Floor beam 3	40	27
North Leaf Floor beam 4	37	25

Table 4 - Load Ratings (tons) for Oceanic Bridge – Shear – LFR – Legal Load

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	66	44
NE Bascule Girder	77	52
SW Bascule Girder	93	61
SE Bascule Girder	93	61
North Leaf Floor beam 1	130	87
North Leaf Floor beam 2	174	116
North Leaf Floor beam 3	174	116
North Leaf Floor beam 4	130	87

Table 5 - Load Ratings (tons) for Oceanic Bridge – Flexure – ASR - Operating

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	91	61
NE Bascule Girder	114	76
SW Bascule Girder	114	76
SE Bascule Girder	153	101
North Leaf Floor beam 1	48	32
North Leaf Floor beam 2	54	36
North Leaf Floor beam 3	54	36
North Leaf Floor beam 4	50	33

Table 6 - Load Ratings (tons) for Oceanic Bridge – Shear – ASR - Operating

	Impact Factor - 1.33	**Impact Factor - 2.00**
NW Bascule Girder	123	82
NE Bascule Girder	144	96
SW Bascule Girder	173	115
SE Bascule Girder	173	115
North Leaf Floor beam 1	102	68
North Leaf Floor beam 2	137	91
North Leaf Floor beam 3	137	91
North Leaf Floor beam 4	102	68

7 Experimental Design

The Oceanic Bridge was tested using diagnostic level loads in order to validate the predictions of the refined (deteriorated) FE model. The load was applied in several configurations using two 36 ton tri-axle dump trucks (with only two rear axles engaged) as shown in Figure 13. Each bascule girder was instrumented to track the critical strain responses within these members under the applied loads. An example of the installation process employed is detailed in Figure 14.

The measured response data was principally used to validate the accuracy of the refined (deteriorated) FE model, and thus provide an additional level of confidence that it is in fact a conservative representation of the actual performance of the Oceanic Bridge. In addition, the results were used to perform diagnostic load ratings in accordance with the specifications and guidance provided in NCHRP report 234 "*Manual for Bridge Rating through Load Testing*" (NCHRP 1998). Although the procedure outlined in this report is widely applied, the authors believe that such an approach is potentially unreliable. As a result, while the load ratings derived from the method promoted by NCHRP (1998) is included in this report, it is included only for completeness and it is not recommended that ratings derived in this manner be adopted.

Figure 13 – Side by Side Static Load Case

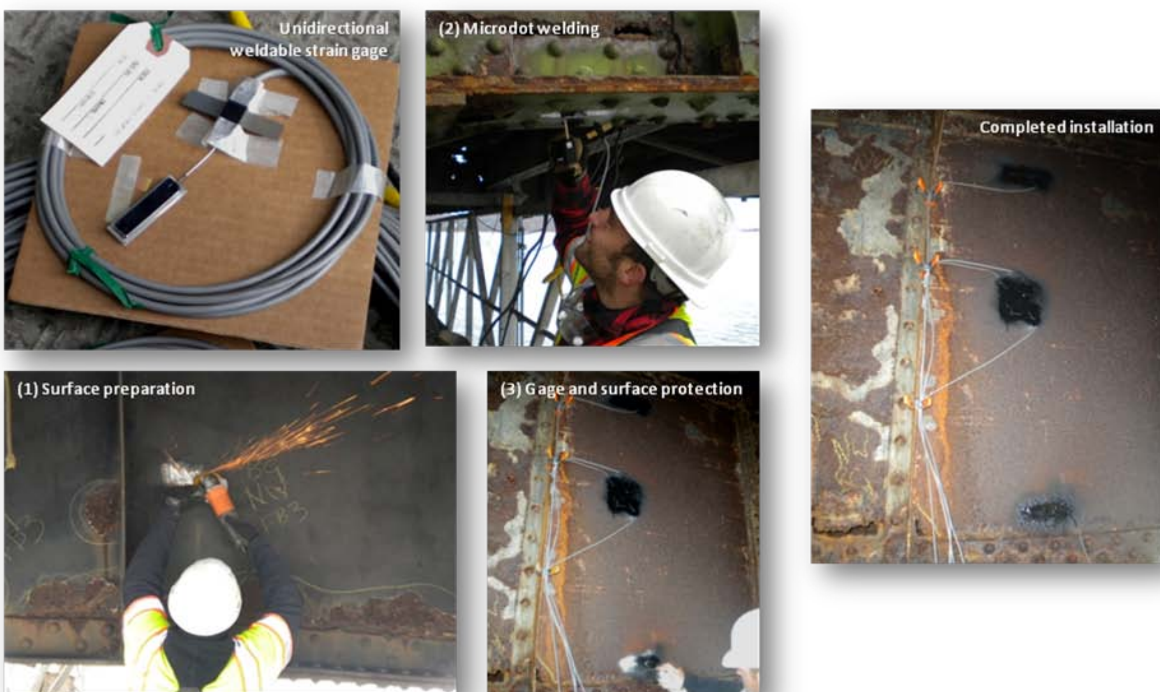


Figure 14 – Installation of Strain Gages

7.1 Instrumentation Plan

Based on the results of the FE modeling study and the fact that the bascule girders are the most critical structural members within the superstructure, it was decided to focus the instrumentation on these four elements. Based on the visual inspection report developed by H&H it was clear that the north leaf of the bascule span exhibited more significant deterioration and thus was instrumented more heavily than the south leaf. The general goal of the instrumentation was twofold: (1) identify and quantify the load sharing of the four bascule girders to inform the performance of both the live load bearings and the magnitude of shear transfer between the spans, and (2) quantify the strain distributions within the most deteriorated elements to identify whether strain compatibility exists. Given the objectives and the budget constraints, the north leaf was instrumented with 22 strain gages, while the south leaf was instrumented using 10 strain gages. Figure 15 shows the sensor locations for the north leaf. The south leaf instrumentation included only the sensors labeled as Location 3 in Figure 15.

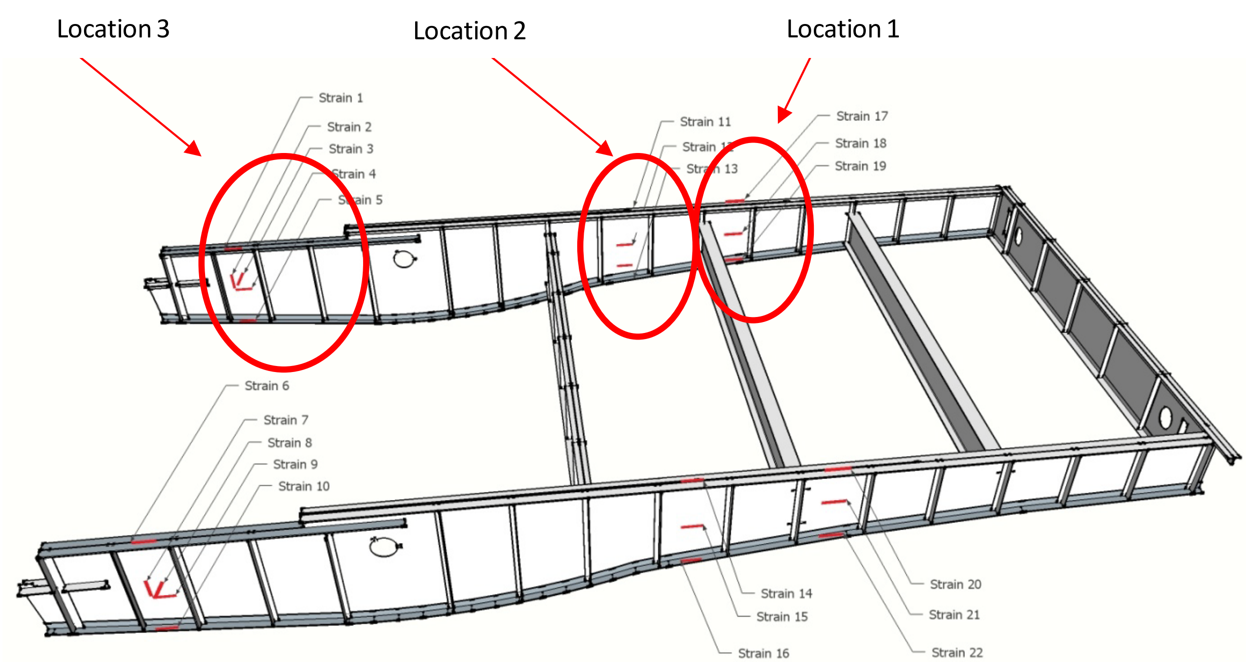


Figure 15 – North Leaf Instrumentation Plan

7.2 Sensor Selection

The sensors used for the load test were weldable type strain gages procured from Hitec Products. The specific sensor model was HBW-35-1000-6-10GP-TR, which is comprised of a one inch strain gage mounted on a two inch shim and encapsulated in a weather resistant assembly. The installation procedure involves microdot welding the shim to the prepared steel surface and protecting the entire sensor with corrosion-resistant paint. Given the County's interest in long-term monitoring, this sensor was selected as it has proven quite durable and thus it is well-suited for both short-term and long-term monitoring.

7.3 Data Acquisition

The Optim Electronics Megadac 3415 AC was employed for data acquisition, which includes signal conditioning, analog filtering, analog to digital conversion, data sampling and recording, and real-time data visualization. The Optim utilizes a chassis and removable/interchangeable signal conditioning cards. The cards utilized during the load testing were Model AD305QB quarter bridge completion cards. Each card accommodates 8 channels of data and interfaces with the sensors through STB modules, which provide screw terminal blocks. The Optim is configured with a 16 bit analog to digital converter, which provides greater than one microstrain resolution. The Optim can sample data at rates up to 25 kHz; however, given the frequency of responses observed during truck load tests, the sampling frequency was set to 50 Hz for the static and crawl-speed tests and 400 Hz for the dynamic tests (impact factor characterization).

7.4 Installation Difficulties and Malfunctions

Several of the cross sections selected for instrumentation exhibited significant levels of deterioration. As a result, in some cases the actual sensor location differed from the initial instrumentation plan in order to ensure that the sensor was attached to a sound portion of the cross-section that could be expected to provide meaningful response measurements. For example, in some cases the gages that were intended to be installed on flanges were installed on adjacent webs due to the significant deterioration observed in the flanges. In all cases, the exact location of the sensors was documented and used for both the model-experimental correlation/validation and diagnostic-level load ratings.

8 Static Load Test Results

8.1 Test Summary

The static and crawl-speed load test was carried out on Thursday December 17th, 2009. Before the static cases were started, the trucks were run back and forth over the bridge with traffic to take some measurements of the response under moving loads and to 'loosen' up the bridge in case any appreciable stick-slip conditions existed. Next, traffic was stopped for 15 minute intervals and the static load cases were carried out. The last static test was completed around 3 pm. The following load cases were completed during this test:

- Load Case 1: A single 19 ton truck crawled along the centerline (north)
- Load Case 2: A single 19 ton truck crawled along the centerline (south)
- Load Case 3: A single 18 ton truck crawled along the centerline (north)
- Load Case 4: A single 18 ton truck crawled along the centerline (south)
- Load Case 5: Two 18 ton trucks crawled along the bridge, one in each lane (north)
- Load Case 6: Two 18 ton trucks crawled along the bridge, one in each lane (south)
- Load Case 7: Two 36 ton trucks positioned side by side statically on the bridge
    - Midpoint of the south leaf
    - At the midpoint of the bascule span

- Midpoint of the north leaf

- Load Case 8: Two 36 ton trucks positioned back to back at the midpoint of the bascule in the northbound traffic lane
- Load Case 9: Two 36 ton trucks positioned back to back at the midpoint of the bascule in the southbound traffic lane
- Load Case 10: Two 36 ton trucks crawled along the bridge, one in each lane (south)
- Load Case 11: Two 36 ton trucks crawled along the bridge, one in each lane (north)

8.2 Overall Response

Throughout the experimental program, the bridge exhibited responses well within the expected bounds and performed in a consistent manner with the refined FE model. Of particular interest was the observed shear transfer between the spans, which appeared with the FE model (a maximum shear transfer of 20% between leafs). In comparing the various load cases with different total live loads, it was observed that the bridge exhibited linear response throughout the entire test. In addition, the results indicated that the northeast live load bearing, which was shimmed prior to the load test, was performing properly. This resulted in very symmetrical responses of the east-west bascule girders and a noticeable increase in stiffness at the mid-span of the bridge.

8.3 Measured Strains

To interpret the measured strain data captured during the load test, they were first plotted on the 3D CAD model in order to establish their spatial distributions. Figure 16 and Figure 17 show the strain responses captured during Load Cases 9 and 10, respectively. Based on these figures it can be observed that all cross-sections displayed nearly linearly varying strain, which indicates that although significant deterioration is present, the cross-section maintained a high degree of strain compatibility. Further the moment reversals apparent between Locations 1 and 2 of the north bascule girders in Figure 17 clearly demonstrate the active shear transfer mechanism between the spans. A structure acting as a pure cantilever would not experience such a stress reversal. Therefore, it is accurate to model a hinge at the bascule leaf tips. Figure 16 illustrates the load case which caused the largest stress recorded – 7.05 ksi along the bottom flange of the northwest bascule girder at Location 2.

In addition to the 3D visualization, the measured strain distributions at each cross-section, and for each Load Case, were examined to investigate the linearity and stationarity of the bridge. Figure 18 through Figure 20 show the measured strain distributions for the northeast bascule girder at Locations 1 and 3, and the northwest bascule birder at Location 1, respectively. Although some minor nonlinearity in the strain responses of the instrumented cross-sections were noted, these figures illustrate the profiles captured were consistent between Load Cases and the location of the neutral axes were constant. In addition, the strain magnitudes appear proportional to actions induced by the various load cases, which suggests linear and stationary response. The stress reversals observed in Figure 18 through Figure 20 are associated with the moment reversal illustrated in Figure 17. A complete set of measured strain profiles are given in Appendix C.

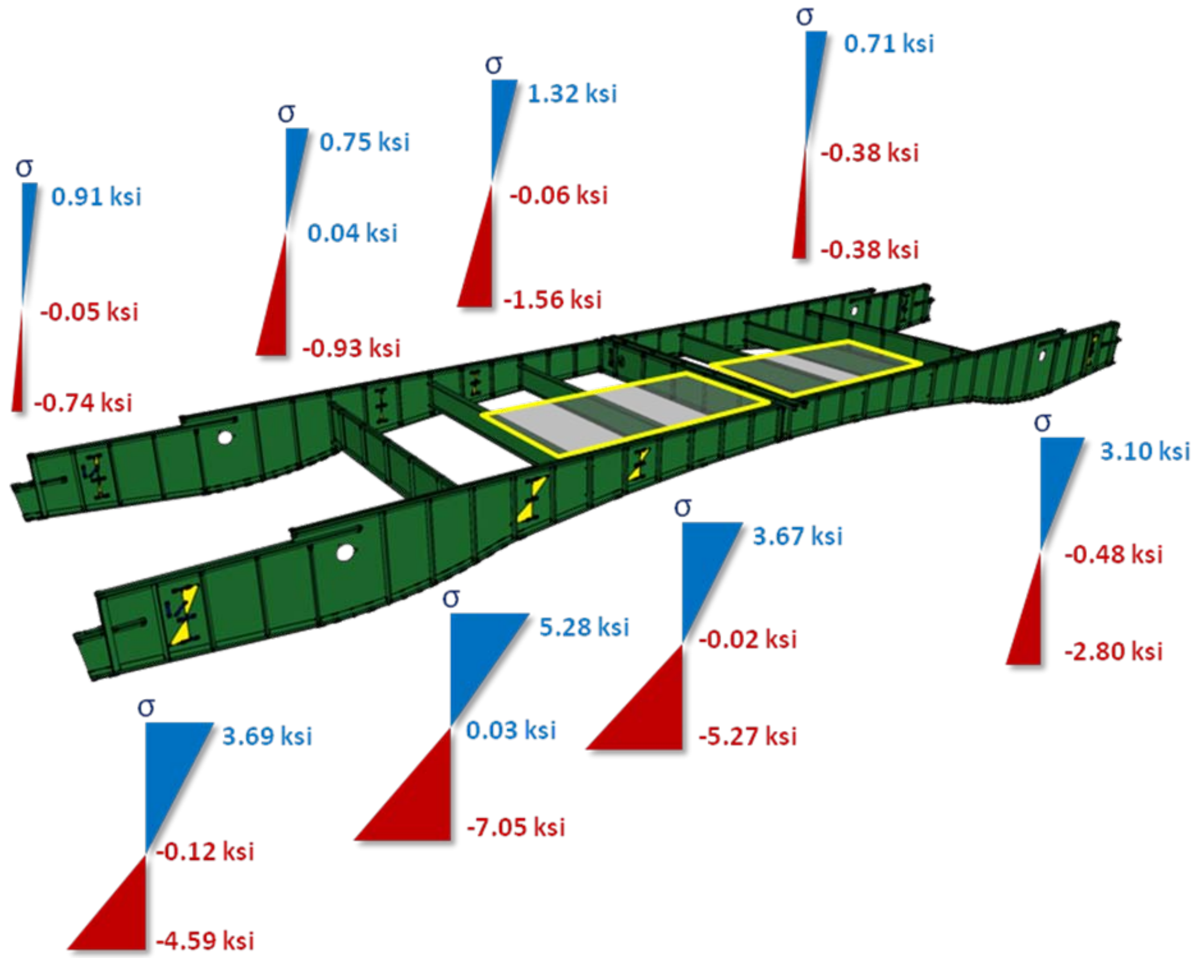


Figure 16 – Strain distributions captured during Load Case 9, which included two 36 ton trucks placed back-to-back at the mid-span of the southbound lane

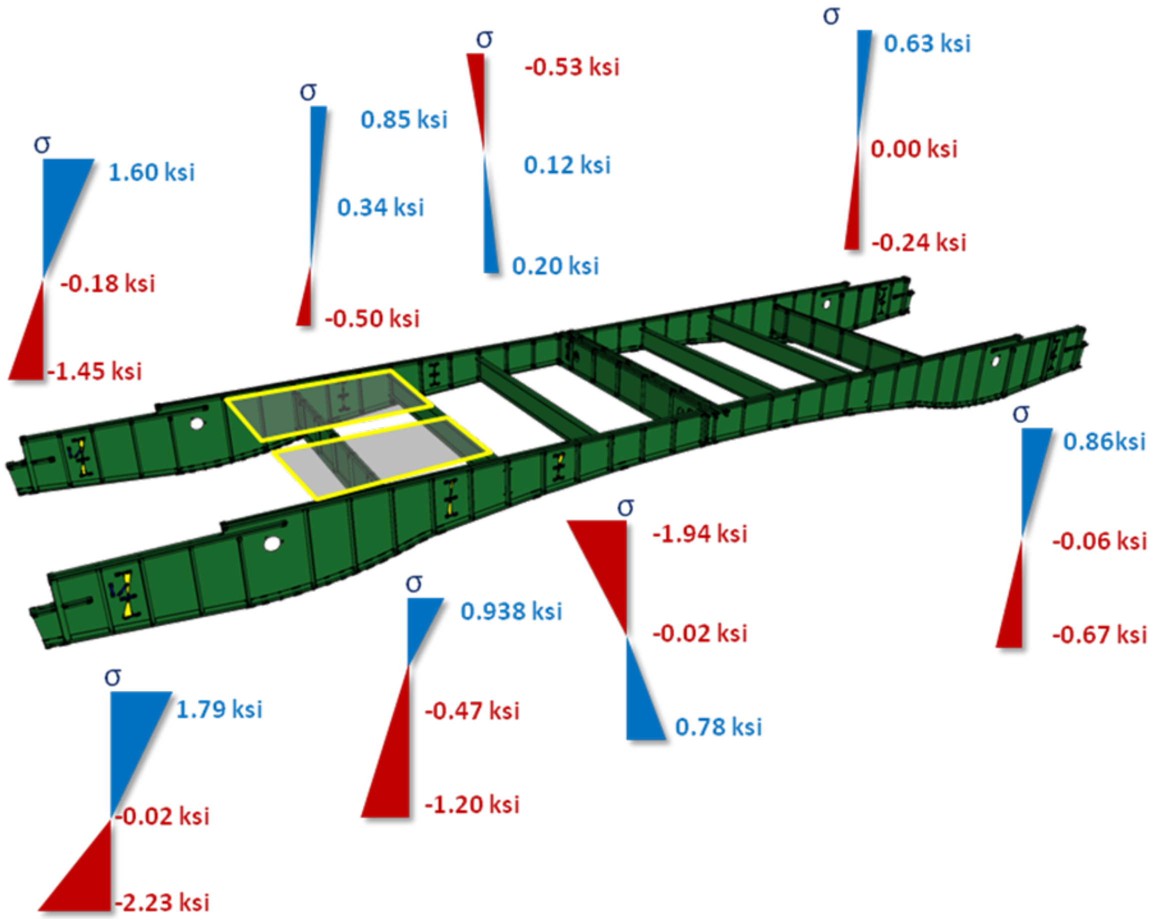


Figure 17 – Strain distributions captured during Load Case 10, which included two 36 ton trucks crawling south across the bridge

Figure 18 – Strain profiles measured at Location 1 of the northeast bascule girder

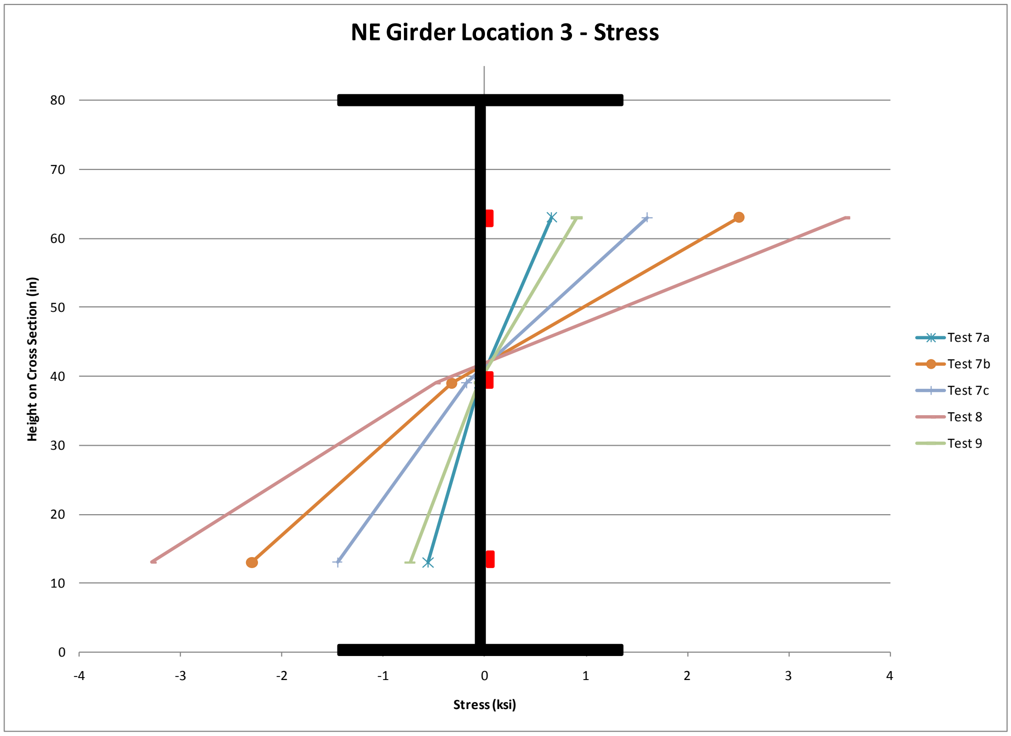


Figure 19 – Strain profiles measured at Location 3 of the northeast bascule girder

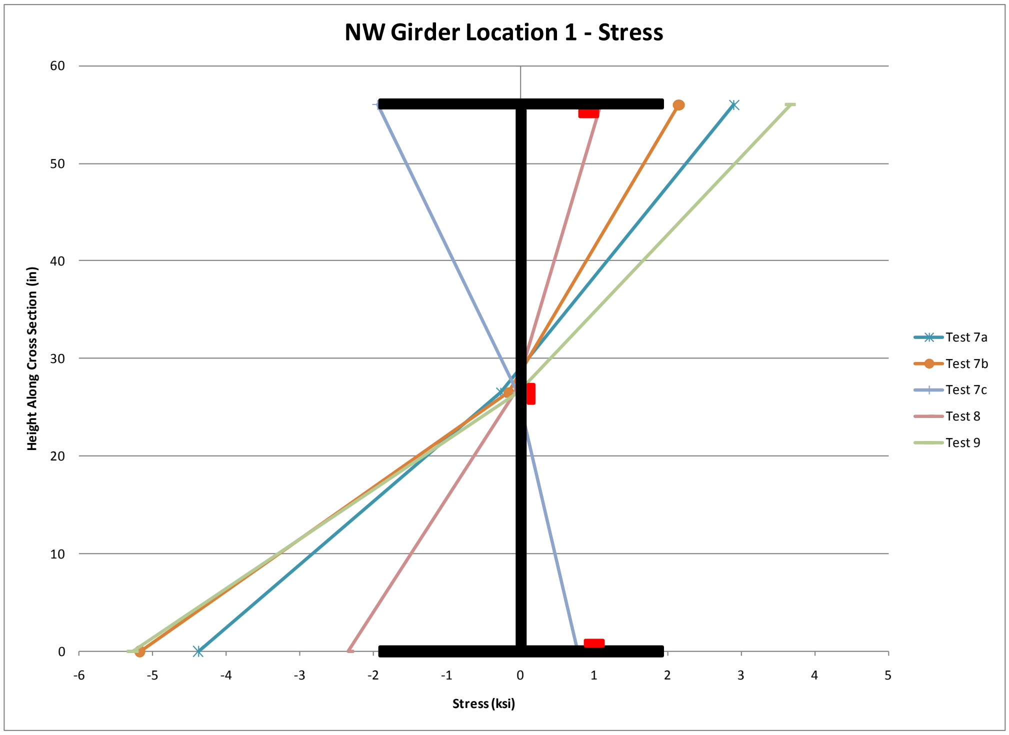


Figure 20 – Strain profiles measured at Location 1 of the northwest bascule girder

9 Finite Element Correlation

Following the conclusion of the static truck load test, the strain readings from the test were compared with the values obtained with the refined FE model. The measured strains/stresses compared reasonably well and demonstrated that the FE model in all cases was conservative. This observed conservatism is likely a result of geometric simplifications made during the construction of the FE model. Specifically, the refined FE model neglected the contribution of the vertical legs of the angles (stiffeners) between the web and flanges of the built-up members and the stiffness contribution of the bridge railing and sidewalks. It is interesting to note that the experimentally determined locations of the neutral axis at the instrumented cross-section were mostly above the mid-height of the member, indicating that some level of composite action with the floor system and/or railings/sidewalk may be present. Overall, the observed deterioration showed little influence on the strain distribution/strain compatibility in both the experiment and FE model simulations. Figure 21 through Figure 23 illustrate the correlation between the measured and calculated stress distributions.

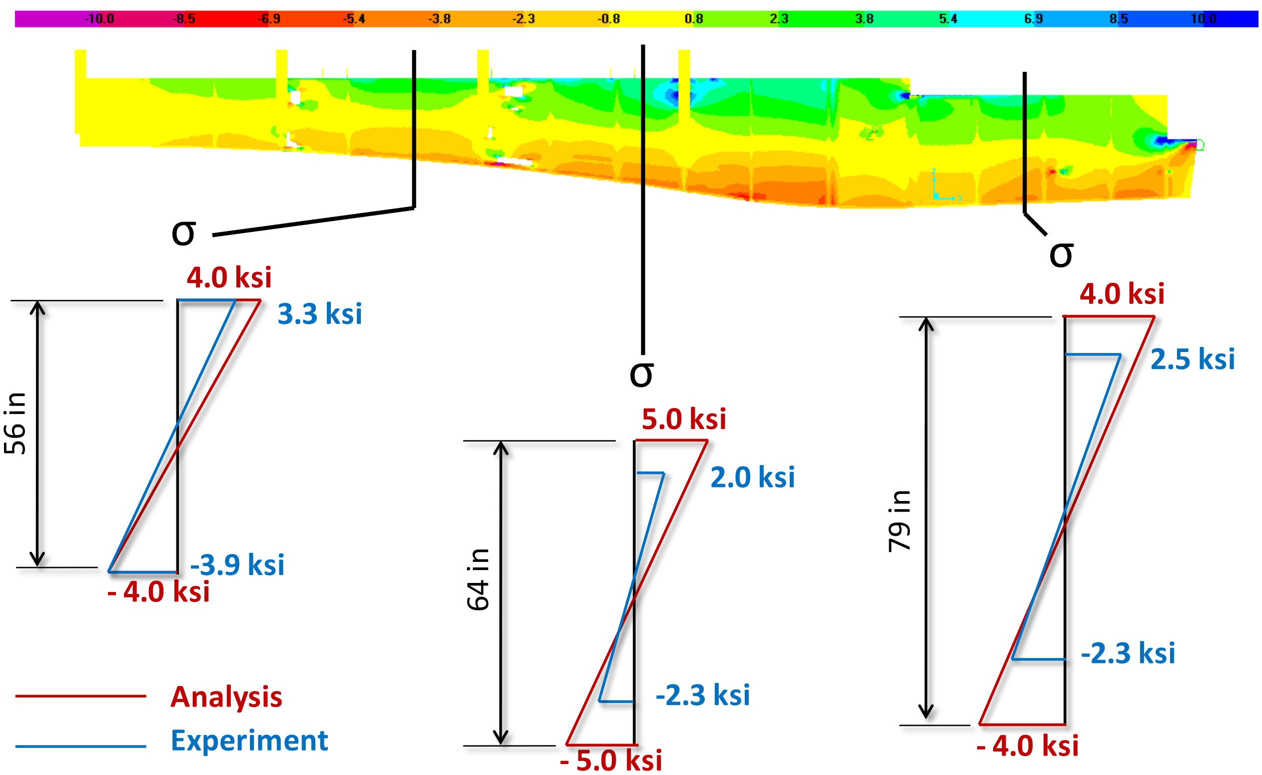


Figure 21 – FE model correlation for the northeast bascule girders

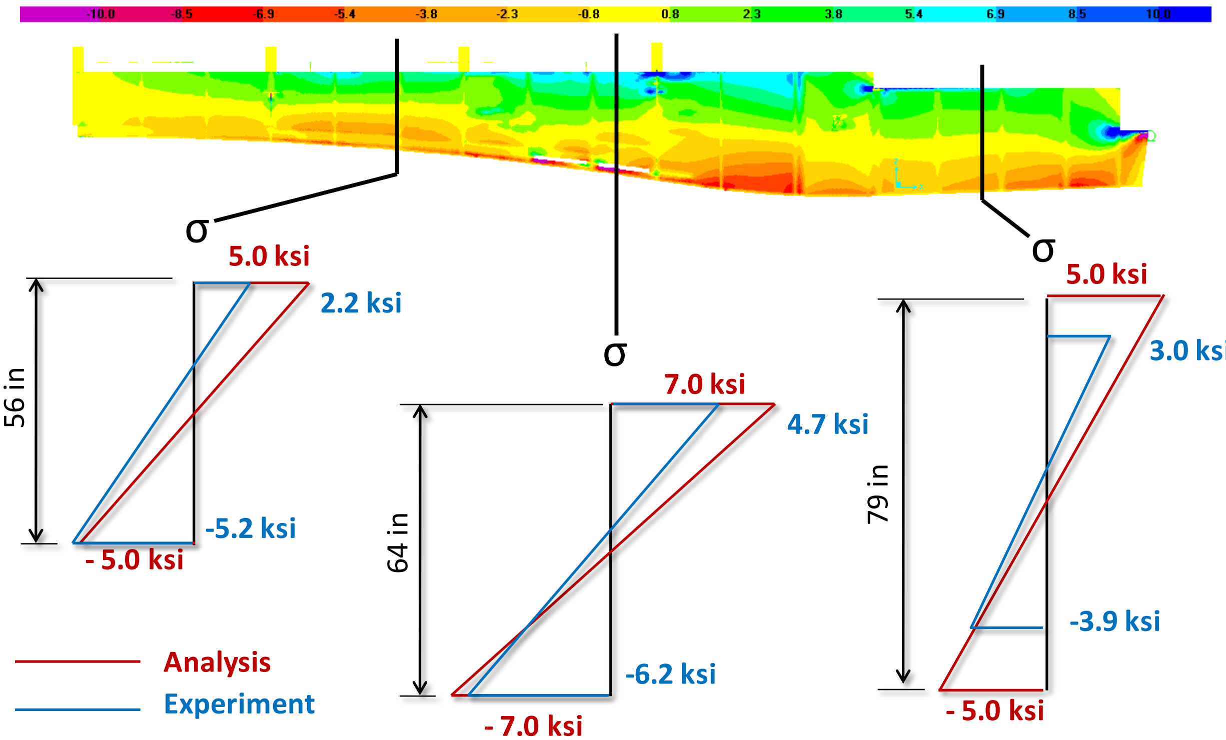


Figure 22 – FE model correlation for the northwest bascule girders

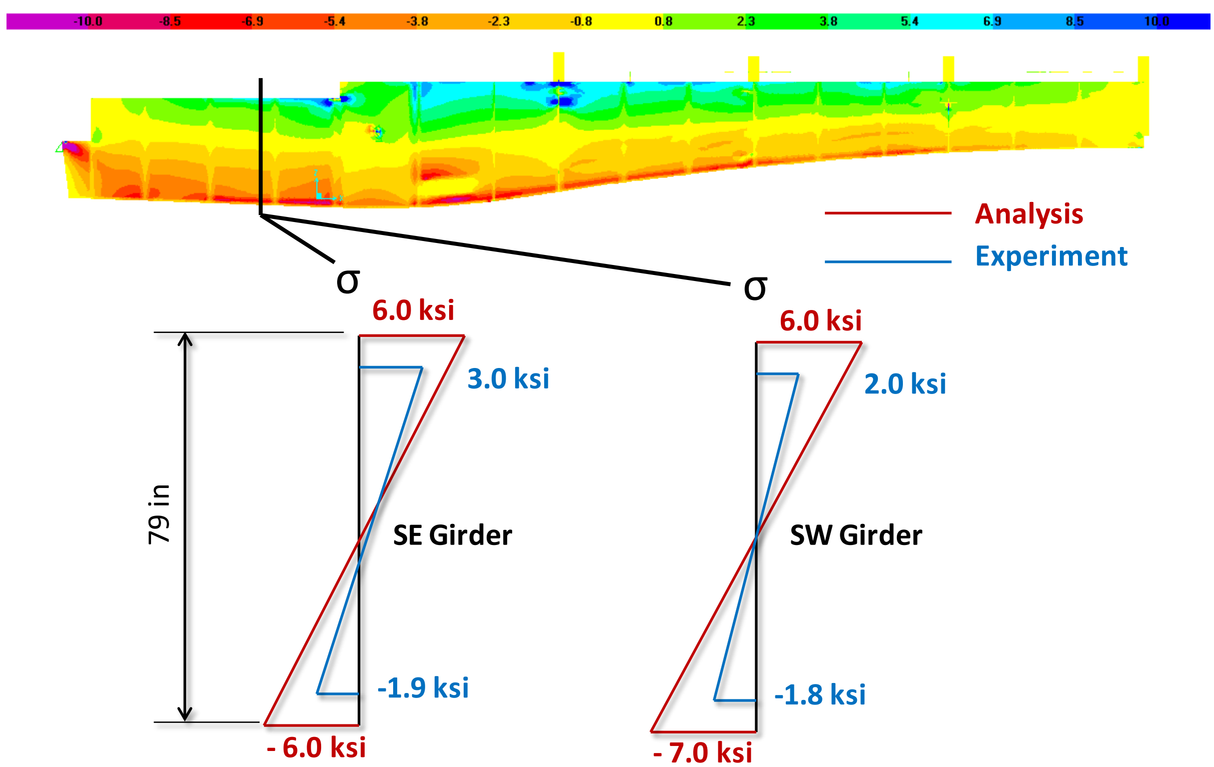


Figure 23 – FE model correlation for the south bascule girders

10 Experimental Load Rating and Impact Factor

10.1 NCHRP Load Rating

Load ratings, utilizing the experimental load test results, were calculated for the Oceanic Bridge in accordance with the guidelines presented in NCHRP (1998). The rating values are based on one truck per lane, using the truck configuration shown in Figure 12. The "theoretical" load effects were obtained at the corresponding instrumentation locations using the refined FE model. The general approach promoted by NCHRP (1998) is based on the ratio of "theoretical" strain and measured strain. This ratio is then modified by a series of empirical factors that takes into account the frequency of visual inspection, fracture critical details, extrapolation of test results to higher loads, and the engineer's knowledge of the behavior of the structure and their evaluation and explanation of the load test results.

Based on experience, the authors believe that the simplistic approach given by NCHRP (1998) is unreliable and fails to properly address the different benefits garnered through the many different approaches to load testing. In addition, this approach suffers from the fact that it is exclusively focused on locations where instrumentation is installed. Although one would typically aim to locate sensors at the most critical locations, in some cases (either due to access issues or due to uncertainties that make it difficult to identify critical locations in an a-priori sense) this is not possible. Nonetheless, this procedure is applied widely and the resulting rating factors are included in Table 7 for completeness. The underlying calculations used to obtain these rating factors are given in Appendix D.

Table 7– Experimental Load Ratings (tons per truck) for Oceanic Bridge - Flexure

	NCHRP Load Rating
NW Bascule Girder	56
NE Bascule Girder	55
SW Bascule Girder	91
SE Bascule Girder	115

10.2 Experimental Quantification of the Dynamic Impact Factor

Crawl speed tests as well as dynamic truck load tests (at both 25 mph and 35 mph) were conducted on January 12, 2010 to quantify the impact factor used in the rating of the structure. As is typically the case, it was found that the impact factor varied with location of interest, truck speed, and the direction the truck was traveling (due to the misalignment of the two bascule leafs). In addition, based on the experience of the authors, the dynamic impact factor is generally extremely dependent on the specific truck or trucks used in the test. For example, the authors have found impact factors that vary by more than 50% depending on the suspension system of the trucks being used. Accurately capturing the complete live load environment [LL*(1+I)] of the Oceanic Bridge requires a long-term monitoring study to quantify the significant variability associated with such a demand model. Table 8 shows the maximum, mean and standard deviation of the impact factors for the northeast and northwest girders for 25 mph and 35 mph. The impact factors provided in this table represent a ratio of dynamic strain captured during the 25 mph and 35 mph test runs and the static strains captured during crawl-speed test runs.

Table 8– Experimentally established dynamic impact factors

	Impact Factor – 25 mph			**Impact Factor – 35 mph**		
	Max	Mean	St Dev	Max	Mean	St Dev
NW Bascule Girder	1.45	1.25	0.15	1.22	1.11	0.09
NE Bascule Girder	1.29	1.20	0.09	1.22	1.15	0.06

11 Additional Simulation to Justify the Measured Reserve Capacity

To explain why the bridge is able to carry relatively large loads in spite of the extensive deterioration, an extreme case of structural deterioration was simulated by removing the webs of a section of the bascule girders and representing only the flanges and stiffeners, i.e., a so-called Vierendeel truss. The analyses with this model (Figure 24) revealed that even as a Vierendeel truss, the main girders were able to carry two 30 ton trucks (neglecting the load and resistance factors provided by the various load rating methodologies). This exercise was useful in revealing just how conservative the original design of the bascule girders was. It is fortunate that most of the extreme deterioration in the bridge was limited to the web plates, leaving the heavy angles that made up portions of the flanges and the stiffeners in relatively good shape. In addition, it is important to note that even the most deteriorated webs were demonstrated (both through experiment and model-based simulation) to be sufficient to enforce strain compatibility throughout the built-up cross-sections.

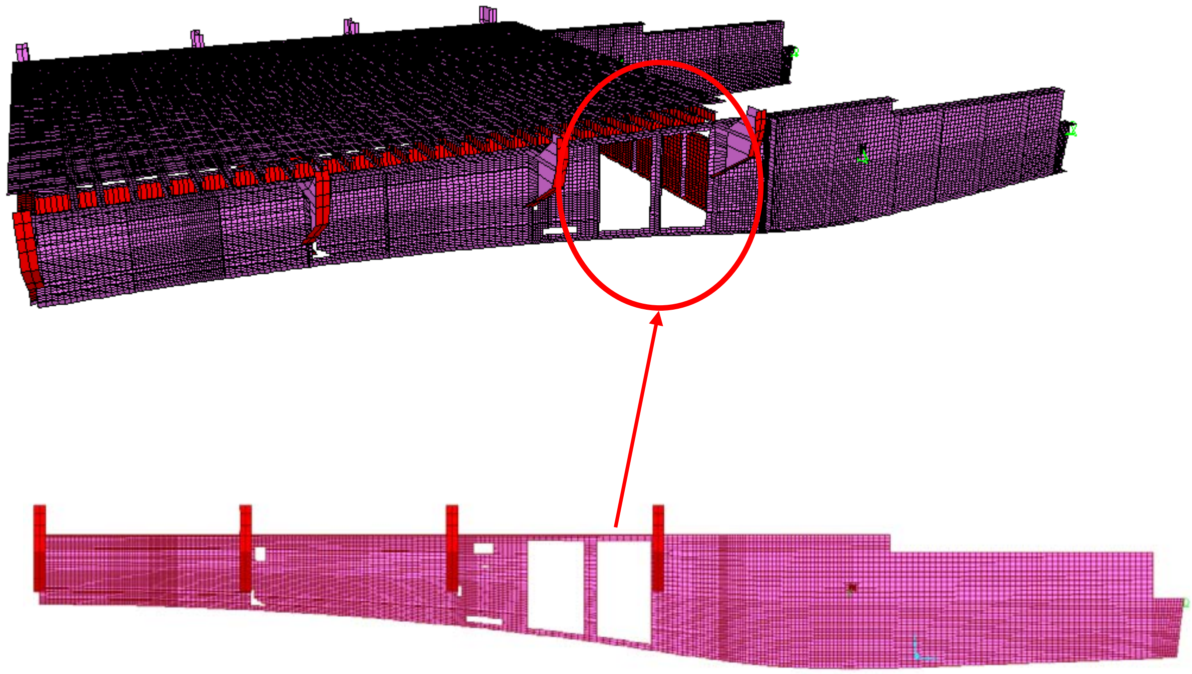


Figure 24 – Vierendeel FE Model

12 Conclusions and Recommendations

The following sections outline a set of conclusions and recommendations reached after the St-Id of the Oceanic Bridge was performed as detailed throughout this report.

12.1 Safe Load Carrying Capacity and Posting

Based on the St-Id conducted, it is concluded that the safe load carrying capacity of the Oceanic Bridge exceeds 25 tons. However, given the extensive deterioration and uncertainty associated with the stationarity of the bridge's boundary and continuity conditions, it is recommended that the County only raise the posting to 10 tons until the interim repairs are completed and the long-term monitoring system is designed and installed. In addition to raising the posting, if the County was able to control the speed limit of school buses and emergency vehicles to below 25 mph, such vehicles with gross vehicle weights of up 15 tons should be allowed to traverse the bridge. Following the interim repairs and the installation of a long-term monitoring system, the posting of the bridge should be re-evaluated based on (1) the effectiveness of the interim repairs (established through sensing) to improve bridge performance, and (2) the measured live load environment the bridge is exposed to under normal operating conditions.

12.2 Vulnerabilities

Several vulnerabilities of the Oceanic Bridge were identified during the St-Id. The primary vulnerability identified was related to the operational safety associated with a truck impacting the bridge railings. Although this may be somewhat unlikely due to the alignment of the structure, given the condition of the railing supports the railing system does not appear capable of restraining a truck impact. From the structural safety standpoint, there are two principal vulnerabilities. The first is related to the boundary conditions supplied by the live load bearings. During the initial inspection of the bridge, it was noted that a ¾ in gap was present at the northeast live load bearing. The simulation modeling and initial live load strain monitoring both indicated that such a condition results in erratic stress distributions in the primary members and the potential for overstressing. As a result, it was recommended that the County shim the bearing to reengage the support under self weight of the bridge, which they promptly did. Following this, the stresses reduced and the stress distributions became far more stable.

The second structural safety vulnerability is related to the performance of the span locks which transfer shear between the two bascule leafs. As discussed, by correlating the experimental data and the simulation model, it appears that a maximum shear transfer of approximately 20% can be expected between spans. While this reduces the stresses in the bascule girders and thus helps to improve their ratings, it also induces higher stresses in the outermost floor beam (which supports the span locks). As apparent from the live load ratings, under certain cases, these elements govern the rating. It appears that under the current configuration (i.e. without any retrofitting of the outermost floor beam), the load sharing between spans is somewhat optimal. Having said that, this mechanism represents a key vulnerability as either increasing or decreasing the amount of shear transfer between spans may reduce the overall rating of the bridge.

12.3 Repair Strategies

The proposed repairs to the bascule girders, sidewalk brackets and superstructure replacement as outlined in previous retrofit reports should be reviewed based on the insight from the load test and analytical studies. Once a consensus is reached, the recommended repairs should be completed as soon as possible. During the completion of the repairs the structure should be monitored to establish the effectiveness of the repairs and assess what affect they may have on the safe load-carrying capacity of the bridge. For example, retrofitting a throw-away bridge as deteriorated as the Oceanic Bridge cannot be effective if it is based on simply replacing deteriorated plates with new plate material. Rather, given the investment the County has made into performing a St-Id, a great deal of insight into the current performance of the bridge is available and may be used to develop more efficient and cost effective approaches. One example can be envisioned to replace the additional plates to be added to the webs of the bascule girders with diagonal bracing between the top and bottom flange angles between two vertical web stiffeners. This retrofit will efficiently couple the flanges by developing a truss system.

In addition, it is not clear that the planned floor system replacement (grid deck, purlins, and stringers) is necessarily required. The largest vulnerability of this system is associated with the buckling of the stringers (obviated by the timber bracing) and a punch through of the deck. It is possible that a more targeted approach to enhancing the performance of this system is possible, but this would require a detailed visual inspection and cost study associated with different options. Given the potential cost of over $2 million for a bridge intended to stay in service for only 10 years, such a study may be warranted. In addition, based on the experimental data, it appears that the current bridge structure is benefiting from the composite action, lateral confinement and load redistribution provided by the floor system. Removing this system may cause significant changes to the intrinsic forces of the structure (similar to the unwinding of a tight spring), and there is no assurance that a new floor system will become as effective as the current one.

12.4 Long-term Monitoring

It is recommended that a long-term monitoring system be designed and installed immediately to track the performance and safety of the Oceanic Bridge. This long term monitoring system should be design to (1) capture the influence and effectiveness of any repairs/retrofit during their installation, (2) track responses directly related to key vulnerabilities such as the performance of boundary and continuity conditions, (3) capture the actual live load environment (LL*(1+I)) the bridge is exposed to under normal operating conditions, and (4) develop a baseline of performance that can be used for comparative purposes for the rest of the life of the bridge.

In addition, the monitoring system should be capable of identifying important “events” through the establishment of thresholds for critical responses. Once an event is triggered an alert should be sent to appropriate personnel in the form of an email or text message informing them that action may be required. During the initial implementation of the system, alarms would be monitored in order to work out any electronic anomalies and to calibrate the thresholds that are used to trigger alarms. Finally and most importantly, the long term monitoring will greatly help in determining any corrective actions in regard to the load posting/speed limit, and taking such operational measures in an optimal and efficient manner, if necessary.

Appendix A– Analytical Load Ratings of the Bascule Girders

Load Rating (LRFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$	ksi	Bridge constructed prior to 1940 - Material capacity specified on construction drawings

Model Stresses

NW Girder

$\sigma_{DL1} := 2$	ksi	Maximum flexural dead load stress from FE model - NW Girder
$\sigma_{LL1} := 10$	ksi	Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NW Girder

NE Girder

$\sigma_{DL2} := 2$	ksi	Maximum flexural dead load stress from FE model - NE Girder
$\sigma_{LL2} := 8$	ksi	Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NE Girder

SW Girder

$\sigma_{DL3} := 2$	ksi	Maximum flexural dead load stress from FE model - SW Girder
$\sigma_{LL3} := 8$	ksi	Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SW Girder

SE Girder

$\sigma_{DL4} := 2$ ksi — Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 6$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SE Girder

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Capacity Reduction Factors

$\phi_c := 0.85$ — Condition factor is poor - **AASHTO LRFR Table 6-2**

$\phi_s := 1$ — System factor for a redundant system - **AASHTO LRFR Table 6-3**

$\phi := 1$ — LRFD Resistance Factor - **AASHTO LRFD Design Specifications**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.174$$

NE Girder

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.467$$

SW Girder

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.467$$

SE Girder

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.956$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

NW Girder

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 0.781$$

NE Girder

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 0.976$$

SW Girder

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 0.976$$

SE Girder

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.301$$

Load Rating (LFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi — Bridge constructed prior to 1940 - Material capacity specified on construction drawings

Model Stresses

NW Girder

$\sigma_{DL1} := 2$ ksi — Maximum flexural dead load stress from FE model - NW Girder

$\sigma_{LL1} := 10$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NW Girder

NE Girder

$\sigma_{DL2} := 2$ ksi — Maximum flexural dead load stress from FE model - NE Girder

$\sigma_{LL2} := 8$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NE Girder

SW Girder

$\sigma_{DL3} := 2$ ksi — Maximum flexural dead load stress from FE model - SW Girder

$\sigma_{LL3} := 8$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SW Girder

SE Girder

$\sigma_{DL4} := 2$ ksi — Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 6$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SE Girder

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.399$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.749$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.749$$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 2.332$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 0.931$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.163$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.163$$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.551$$

Load Rating (ASD) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 0.75 \cdot 36\ \text{ksi}$ — Bridge constructed prior to 1940 - Material capacity specified on construction drawings -Allowable stress for a member at the operating level

Model Stresses

NW Girder

$\sigma_{DL1} := 2\ \text{ksi}$ — Maximum flexural dead load stress from FE model - NW Girder

$\sigma_{LL1} := 10\ \text{ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NW Girder

NE Girder

$\sigma_{DL2} := 2\ \text{ksi}$ — Maximum flexural dead load stress from FE model - NE Girder

$\sigma_{LL2} := 8\ \text{ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NE Girder

SW Girder

$\sigma_{DL3} := 2\ \text{ksi}$ — Maximum flexural dead load stress from FE model - SW Girder

$\sigma_{LL3} := 8\ \text{ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SW Girder

SE Girder

$\sigma_{DL4} := 2$ ksi — Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 6$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SE Girder

Obtain the ASD Legal Load Rating Factors for each Bascule Girder

$I := 0.333$

Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1}\cdot(1 + I)\right]} \qquad RF1 = 1.875$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2}\cdot(1 + I)\right]} \qquad RF2 = 2.344$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3}\cdot(1 + I)\right]} \qquad RF3 = 2.344$$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 3.126$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.25$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.563$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.563$$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 2.083$$

Load Rating (LRFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Shear Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi Bridge constructed prior to 1940

Model Stresses - Shear

NW Girder

$\sigma_{DL1} := 4$ ksi Maximum dead load stress due to shear from FE model - NW Girder

$\sigma_{LL1} := 7$ ksi Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

NE Girder

$\sigma_{DL2} := 4$ ksi Maximum dead load stress due to shear from FE model - NE Girder

$\sigma_{LL2} := 6$ ksi Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SW Girder

$\sigma_{DL3} := 4$ ksi Maximum dead load stress due to shear from FE model - SW Girder

$\sigma_{LL3} := 5$ ksi Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SE Girder

$\sigma_{DL4} := 4$ ksi — Maximum dead load stress due to shear from FE model - SE Girder

$\sigma_{LL4} := 5$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Capacity Reduction Factors

$\phi_c := 0.85$ — Condition factor is poor - **AASHTO LRFR Table 6-2**

$\phi_s := 1$ — System factor for a redundant system - **AASHTO LRFR Table 6-3**

$\phi := 1$ — LRFD Resistance Factor - **AASHTO LRFD Design Specifications**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.528$$

NE Girder

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.782$$

SW Girder

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 2.139$$

SE Girder

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 2.139$$

Rating Factors for an Impact Factor of 2

$I := 1$ Impact Factor

NW Girder

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.016$$

NE Girder

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.185$$

SW Girder

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.422$$

SE Girder

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.422$$

Load Rating (LFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Shear Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi — Bridge constructed prior to 1940

Model Stresses - Shear

NW Girder

$\sigma_{DL1} := 4$ ksi — Maximum dead load stress due to shear from FE model - NW Girder

$\sigma_{LL1} := 7$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

NE Girder

$\sigma_{DL2} := 4$ ksi — Maximum dead load stress due to shear from FE model - NE Girder

$\sigma_{LL2} := 6$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SW Girder

$\sigma_{DL3} := 4$ ksi — Maximum dead load stress due to shear from FE model - SW Girder

$\sigma_{LL3} := 5$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SE Girder

$\sigma_{DL4} := 4$ ksi — Maximum dead load stress due to shear from FE model - SE Girder

$\sigma_{LL4} := 5$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Obtain the LFD Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.85$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 2.158$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]}$$

$RF3 = 2.59$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]}$$

$RF4 = 2.59$

Rating Factors for an Impact Factor of 2

$I := 1$

Impact Factor

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]}$$

$RF1 = 1.23$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]}$$

$RF2 = 1.435$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]}$$

$RF3 = 1.722$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]}$$

$RF4 = 1.722$

Load Rating (ASD) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Shear Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 0.45 \cdot 36 \text{ ksi}$ Bridge constructed prior to 1940 - Allowable stress for shear in girder webs - Operating level

Model Stresses - Shear

NW Girder

$\sigma_{DL1} := 4 \quad \text{ksi}$ Maximum dead load stress due to shear from FE model - NW Girder

$\sigma_{LL1} := 7 \quad \text{ksi}$ Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

NE Girder

$\sigma_{DL2} := 4 \quad \text{ksi}$ Maximum dead load stress due to shear from FE model - NE Girder

$\sigma_{LL2} := 6 \quad \text{ksi}$ Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SW Girder

$\sigma_{DL3} := 4 \quad \text{ksi}$ Maximum dead load stress due to shear from FE model - SW Girder

$\sigma_{LL3} := 5 \quad \text{ksi}$ Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

SE Girder

$\sigma_{DL4} := 4$ ksi — Maximum dead load stress due to shear from FE model - SE Girder

$\sigma_{LL4} := 5$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Obtain the Legal Load Rating Factors for each Bascule Girder

Live Load Magnification

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

ASD Rating Factors for an Impact Factor of 1.33

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.31$$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.529$$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.835$$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]}$$ $RF4 = 1.835$

Rating Factors for an Impact Factor of 2

$I := 1$ Impact Factor

NW Girder

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1} \cdot (1 + I)\right]}$$ $RF1 = 0.871$

NE Girder

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2} \cdot (1 + I)\right]}$$ $RF2 = 1.017$

SW Girder

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3} \cdot (1 + I)\right]}$$ $RF3 = 1.22$

SE Girder

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]}$$ $RF4 = 1.22$

Appendix B– Analytical Load Ratings of the Floor Beams

Load Rating (LRFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi — Bridge constructed prior to 1940 - Material capacity specified on construction drawings

Model Stresses

Floorbeam 1

$\sigma_{DL1} := 1$ ksi — Maximum flexural dead load stress from FE model - Floorbeam 1

$\sigma_{LL1} := 14.5$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf directly over FB1

Floorbeam 2

$\sigma_{DL2} := 1$ ksi — Maximum flexural dead load stress from FE model - NE Girder

$\sigma_{LL2} := 13$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB2

Floorbeam 3

$\sigma_{DL3} := 1$ ksi — Maximum flexural dead load stress from FE model - SW Girder

$\sigma_{LL3} := 13$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB3

Floorbeam 4

$\sigma_{DL4} := 1$ ksi

Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 14$ ksi

Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB4

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$

Dead load magnification - **AASHTO LRFR Table 6-1**

Live Load Magnification

$\gamma_{LL} := 1.8$

Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$

Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.001$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.117$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.117$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.037$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 0.666$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 - I)\right]} \qquad RF2 = 0.743$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 - I)\right]} \qquad RF3 = 0.743$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 0.689$$

Load Rating (LFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi — Bridge constructed prior to 1940 - Material capacity specified on construction drawings

Model Stresses

Floorbeam 1

$\sigma_{DL1} := 1$ ksi — Maximum flexural dead load stress from FE model - Floorbeam 1

$\sigma_{LL1} := 14.5$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf directly over FB1

Floorbeam 2

$\sigma_{DL2} := 1$ ksi — Maximum flexural dead load stress from FE model - NE Girder

$\sigma_{LL2} := 13$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB2

Floorbeam 3

$\sigma_{DL3} := 1$ ksi — Maximum flexural dead load stress from FE model - SW Girder

$\sigma_{LL3} := 13$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB3

Floorbeam 4

$\sigma_{DL4} := 1$ ksi — Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 14$ ksi — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB4

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 1.001$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1.117$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1.117$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.037$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 0.666$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 0.743$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 0.743$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 0.689$$

Load Rating (ASD) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum Flexural Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 0.75 \cdot 36 \text{ ksi}$ — Bridge constructed prior to 1940 - Material capacity specified on construction drawings - Allowable stress

Model Stresses

Floorbeam 1

$\sigma_{DL1} := 1 \quad \text{ksi}$ — Maximum flexural dead load stress from FE model - Floorbeam 1

$\sigma_{LL1} := 14.5 \text{ ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf directly over FB1

Floorbeam 2

$\sigma_{DL2} := 1 \quad \text{ksi}$ — Maximum flexural dead load stress from FE model - NE Girder

$\sigma_{LL2} := 13 \quad \text{ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB2

Floorbeam 3

$\sigma_{DL3} := 1 \quad \text{ksi}$ — Maximum flexural dead load stress from FE model - SW Girder

$\sigma_{LL3} := 13 \quad \text{ksi}$ — Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB3

Floorbeam 4

$\sigma_{DL4} := 1 \quad ksi$

Maximum flexural dead load stress from FE model - SE Girder

$\sigma_{LL4} := 14 \quad ksi$

Maximum flexural live load stress due to 2 36 ton trucks placed with their rear axles directly over FB4

Live Load Magnification

$I := 0.33$

Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1}\cdot(1 + I)\right]} \qquad RF1 = 1.348$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2}\cdot(1 + I)\right]} \qquad RF2 = 1.504$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3}\cdot(1 + I)\right]} \qquad RF3 = 1.504$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 1.396$$

$I := 1$ Impact Factor

Rating Factors for an Impact Factor of 2

Floorbeam 1

$$RF1 := \frac{\left[\sigma_y - \left(\sigma_{DL1}\right)\right]}{\left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 0.897$$

Floorbeam 2

$$RF2 := \frac{\left[\sigma_y - \left(\sigma_{DL2}\right)\right]}{\left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 1$$

Floorbeam 3

$$RF3 := \frac{\left[\sigma_y - \left(\sigma_{DL3}\right)\right]}{\left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 1$$

Floorbeam 4

$$RF4 := \frac{\left[\sigma_y - \left(\sigma_{DL4}\right)\right]}{\left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 0.929$$

Load Rating (LRFR) of Oceanic Bridge Using stresses obtained from 3D finite element model

1) Obtain Maximum shear Stresses from SAP 2000 FE Model

Material Capacity at Yield Point

$\sigma_y := 36$ ksi — Bridge constructed prior to 1940

Model Stresses - Shear

Floorbeam 1

$\sigma_{DL1} := 1$ ksi — Maximum dead load stress due to shear from FE model - Floorbeam 1

$\sigma_{LL1} := 4$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Floorbeam 2

$\sigma_{DL2} := 1$ ksi — Maximum dead load stress due to shear from FE model - Floorbeam 2

$\sigma_{LL2} := 3$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

FLoorbeam 3

$\sigma_{DL3} := 1$ ksi — Maximum dead load stress due to shear from FE model - Floorbeam 3

$\sigma_{LL3} := 3$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Floorbeam 4

$\sigma_{DL4} := 1$ ksi — Maximum dead load stress due to shear from FE model - Floorbeam 4

$\sigma_{LL4} := 4$ ksi — Maximum live load stress (shear) due to 2 36 ton trucks positioned to produce maximum shear

Obtain the Legal Load Rating Factors for each Bascule Girder

Dead Load Factors

$\gamma_{DL} := 1.25$ — Dead load magnification - **AASHTO LRFR Table 6-1**

Capacity Reduction Factors

$\phi_c := 0.85$ — Condition factor is poor - **AASHTO LRFR Table 6-2**

$\phi_s := 1$ — System factor for a redundant system - **AASHTO LRFR Table 6-3**

$\phi := 1$ — LRFD Resistance Factor - **AASHTO LRFD Design Specifications**

Live Load Magnification

$\gamma_{LL} := 1.8$ — Legal Load factor for an unknown ADTT - **AASHTO LRFR Table 6-5**

$I := 0.33$ — Impact Factor - **AASHTO LRFR Section 6.4.4.3**

Rating Factors for an Impact Factor of 1.33

Floorbeam 1

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 3.065$$

Floorbeam 2

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 4.087$$

Floorbeam 3

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 4.087$$

Floorbeam 4

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 3.065$$

Rating Factors for an Impact Factor of 2

$I := 1$ Impact Factor

Floorbeam 1

$$RF1 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL1}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL1} \cdot (1 + I)\right]} \qquad RF1 = 2.038$$

Floorbeam 2

$$RF2 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL2}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL2} \cdot (1 + I)\right]} \qquad RF2 = 2.718$$

Floorbeam 3

$$RF3 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL3}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL3} \cdot (1 + I)\right]} \qquad RF3 = 2.718$$

Floorbeam 4

$$RF4 := \frac{\left[\phi_c \cdot \phi_s \cdot \phi \cdot \sigma_y - \left(\gamma_{DL} \cdot \sigma_{DL4}\right)\right]}{\gamma_{LL} \cdot \left[\sigma_{LL4} \cdot (1 + I)\right]} \qquad RF4 = 2.038$$

Appendix C– Stress Data and Plots

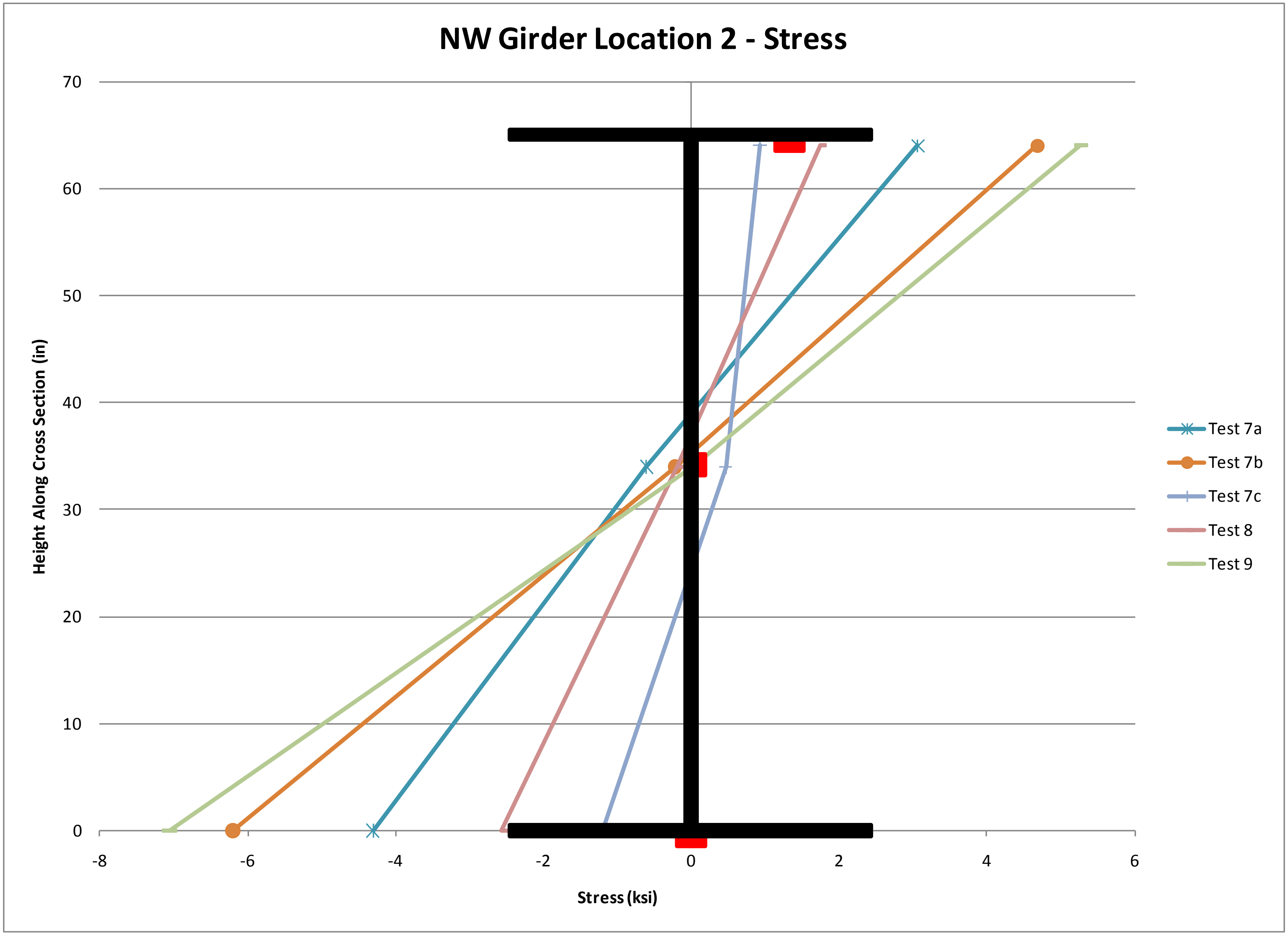
Table C1 – Maximum Stress Measurements for Static Load Test – North Leaf

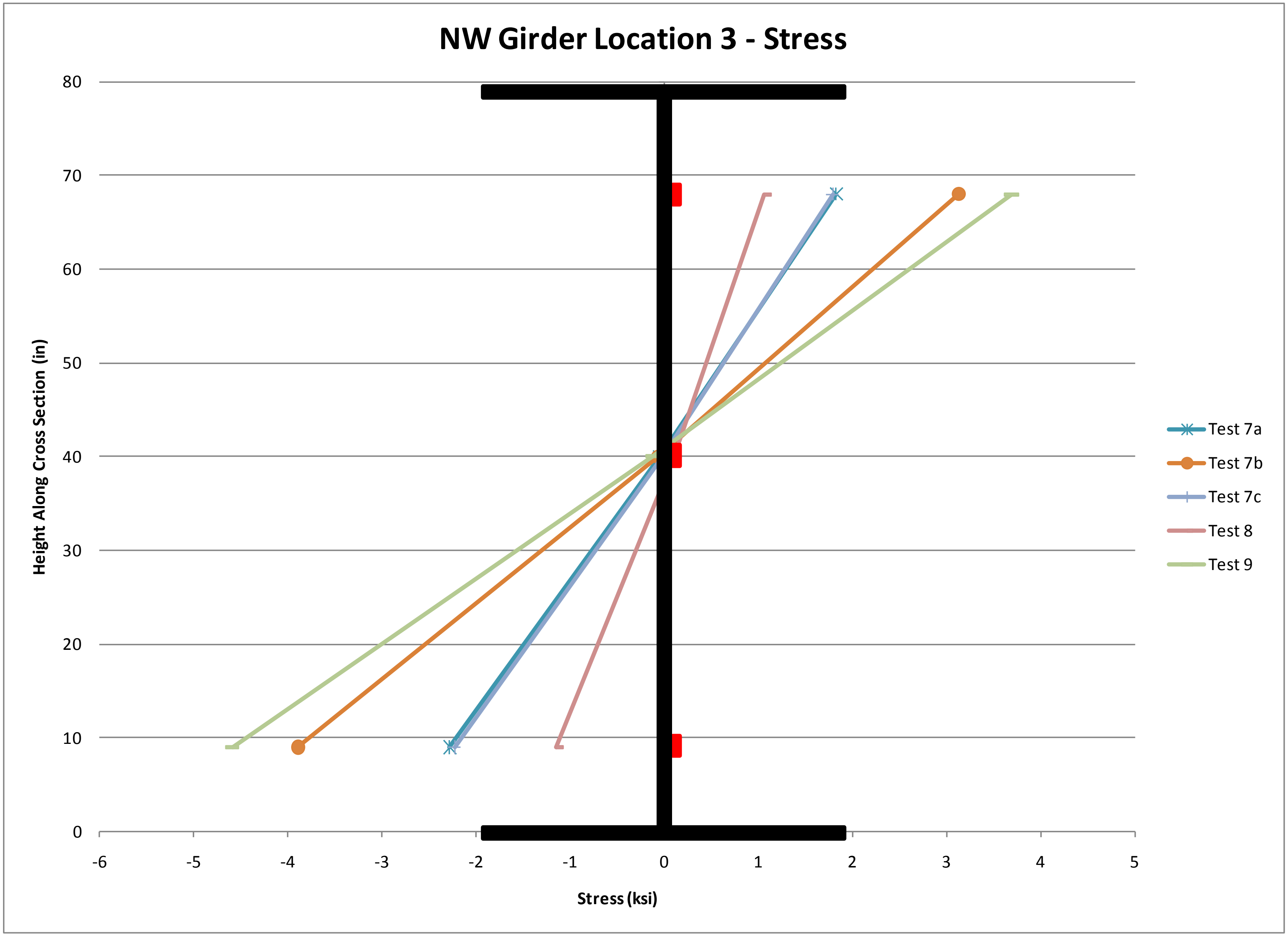
	Test 7a	**Test 7b**	**Test 7c**	**Test 8**	**Test 9**	**Units**
NW Girder - Loc1 - Top Flange	2.90	2.15	-1.94	1.09	3.67	ksi
NW Girder - Loc1 - Web	-0.26	-0.16	-0.04	-0.07	-0.02	ksi
NW Girder - Loc1 - Bottom Flange	-4.38	-5.18	0.78	-2.34	-5.27	ksi
NW Girder - Loc2 - Top Flange	3.07	4.68	0.93	1.74	5.28	ksi
NW Girder - Loc2 - Web	-0.61	-0.22	0.47	-0.20	0.03	ksi
NW Girder - Loc2 - Bottom Flange	-4.31	-6.20	-1.20	-2.57	-7.05	ksi
NW Girder - Loc3 - Top Flange	1.83	3.13	1.79	1.07	3.69	ksi
NW Girder - Loc3 - Rosette - 0 deg	-0.05	-0.07	-0.02	0.11	-0.12	ksi
NW Girder - Loc3 - Rosette - 45 deg	-1.45	-2.80	-1.62	-1.49	-3.06	ksi
NW Girder - Loc3 - Rosette - 90 deg	0.00	0.00	0.04	-0.03	0.02	ksi
NW Girder - Loc3 - Bottom Flange	-2.28	-3.90	-2.23	-1.14	-4.59	ksi
NE Girder - Loc1 - Top Flange	1.59	3.29	-0.53	4.15	1.32	ksi
NE Girder - Loc1 - Web	-0.11	-0.10	0.12	-0.06	-0.06	ksi
NE Girder - Loc1 - Bottom Flange	-1.66	-3.59	0.20	-4.49	-1.56	ksi
NE Girder - Loc2 - Top Flange	0.66	1.95	0.85	2.51	0.75	ksi
NE Girder - Loc2 - Web	-0.04	0.14	0.34	0.23	0.04	ksi
NE Girder - Loc2 - Bottom Flange	-0.89	-2.31	-0.50	-2.95	-0.93	ksi
NE Girder - Loc3 - Top Flange	0.66	2.51	1.60	3.55	0.91	ksi
NE Girder - Loc3 - Rosette - 0 deg	-0.05	-0.32	-0.18	-0.49	-0.05	ksi
NE Girder - Loc3 - Rosette - 45 deg	-0.76	-2.32	-1.43	-2.63	-1.03	ksi
NE Girder - Loc3 - Rosette - 90 deg	-0.15	-0.44	-0.28	-0.47	-0.21	ksi
NE Girder - Loc3 - Bottom Flange	-0.56	-2.30	-1.45	-3.29	-0.74	ksi
NW Girder - Maximum Principal Strain	1.40	2.74	1.64	1.57	2.95	ksi
NW Girder - Minimum Principal Strain	-1.45	-2.80	-1.62	-1.49	-3.06	ksi
NW Girder - Maximum Principal Stress	0.89	1.76	1.06	1.04	1.88	ksi
NW Girder - Minimum Principal Stress	-0.95	-1.83	-1.04	-0.94	-2.01	ksi
NW Girder - Principal Angle	-45.53	-45.36	-45.49	-43.73	-45.68	degrees
NE Girder - Maximum Principal Strain	0.56	1.56	0.97	1.68	0.77	ksi
NE Girder - Minimum Principal Strain	-0.76	-2.32	-1.43	-2.63	-1.04	ksi
NE Girder - Maximum Principal Stress	0.31	0.80	0.50	0.82	0.43	ksi
NE Girder - Minimum Principal Stress	-0.55	-1.71	-1.05	-1.96	-0.74	ksi
NE Girder - Principal Angle	-42.86	-44.16	-43.79	-45.14	-42.58	degrees

Table C2 – Maximum Stress Measurements for Static Load Test – South Leaf

	Test 7a	Test 7b	Test 7c	Test 8	Test 9	Units
SW - Girder-Loc3-TF	2.12	2.24	0.86	1.00	3.10	ksi
SW - Girder-Loc3-BF	-1.84	-1.97	-0.67	-0.76	-2.80	ksi
SW - Girder-Loc3-0 deg	-0.25	-0.28	-0.06	0.01	-0.48	ksi
SW - Girder-Loc3-45 deg	-2.64	-2.70	-0.93	-1.57	-3.34	ksi
SW - Girder-Loc3-90 deg	-0.44	-0.50	-0.22	-0.25	-0.42	ksi
SE - Girder-Loc3-TF	2.92	2.29	0.63	3.60	0.71	ksi
SE - Girder-Loc3-BF	-1.82	-1.31	-0.24	-2.29	-0.38	ksi
SE - Girder-Loc3-0 deg	-0.12	-0.08	0.00	-0.13	0.05	ksi
SE - Girder-Loc3-45 deg	-2.30	-1.78	-0.46	-2.37	-1.06	ksi
SE- Girder-Loc3-90 deg	-0.19	-0.28	-0.20	0.08	-0.14	ksi
SW Girder - Maximum Principal Stress	1.20	0.82	0.11	1.50	0.60	ksi
SW Girder - Minimum Principal Stress	-1.57	-1.25	-0.36	-1.55	-0.71	ksi
SW Girder - Principal Angle	-44.55	-43.22	-36.73	-46.29	-42.27	degrees
SE Girder - Maximum Principal Stress	1.08	1.02	0.34	0.80	1.33	ksi
SE Girder - Minimum Principal Stress	-1.90	-1.97	-0.68	-1.09	-2.41	ksi
SE Girder - Principal Angle	-43.80	-43.65	-42.13	-42.48	-45.31	degrees

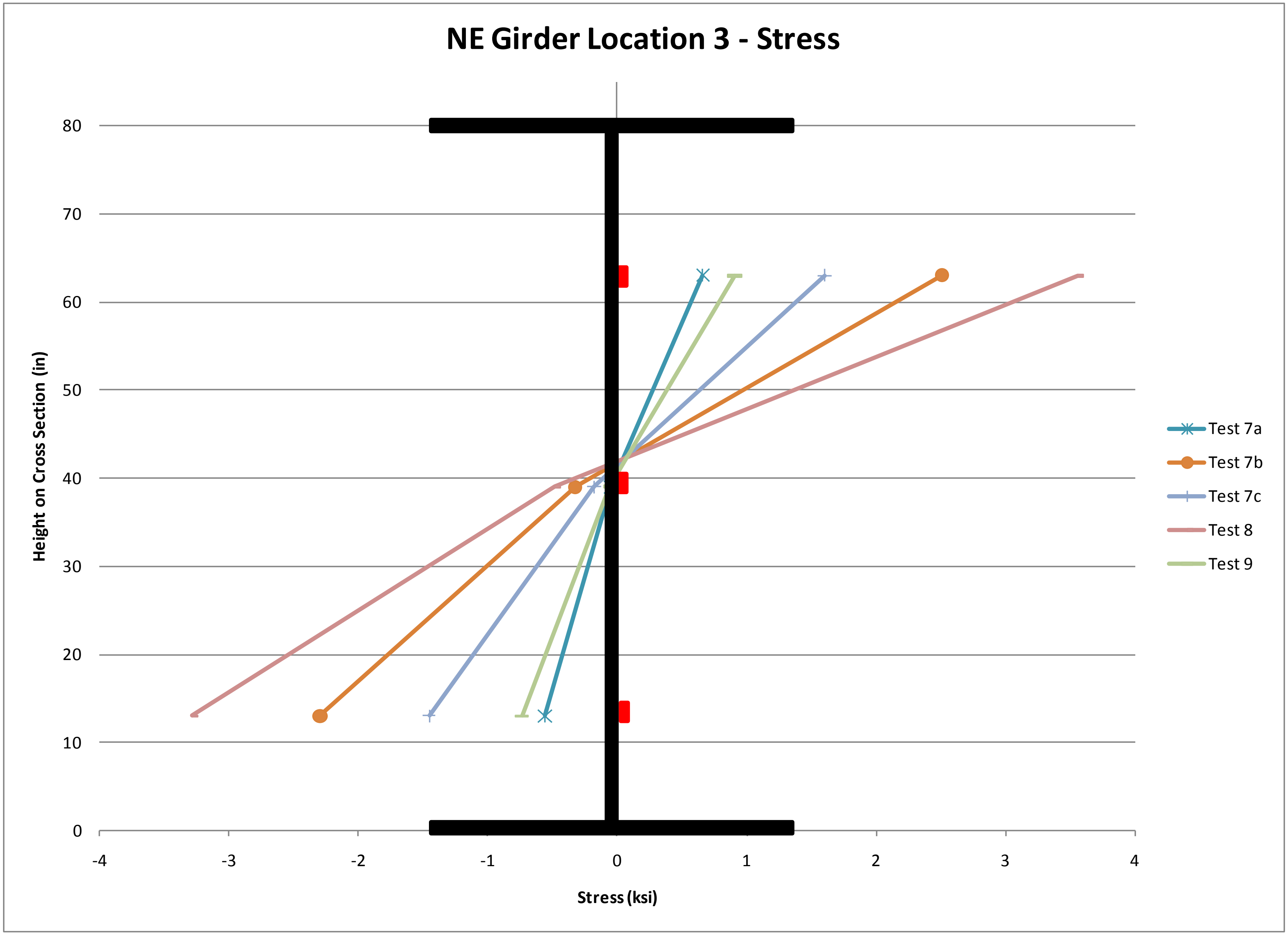

NW Girder Location 1 - Stress
Height Along Cross Section (in)
60
50
40
30
20
10
0
-6
-5
-4
-3
-2
-1
0
1
2
3
4
5
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9

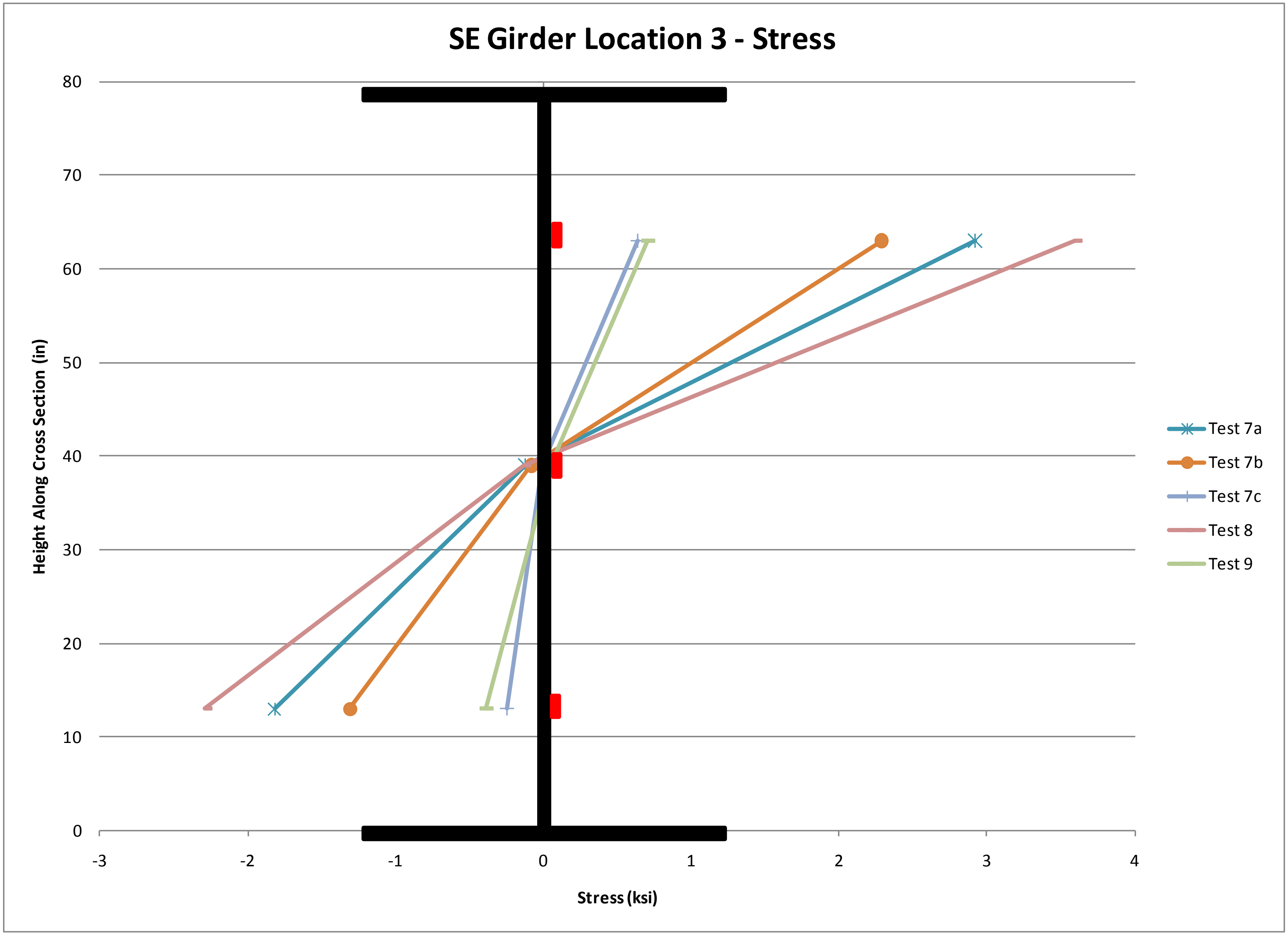

NW Girder Location 2 - Stress
Height Along Cross Section (in)
70
60
50
40
30
20
10
0
-8
-6
-4
-2
0
2
4
6
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9

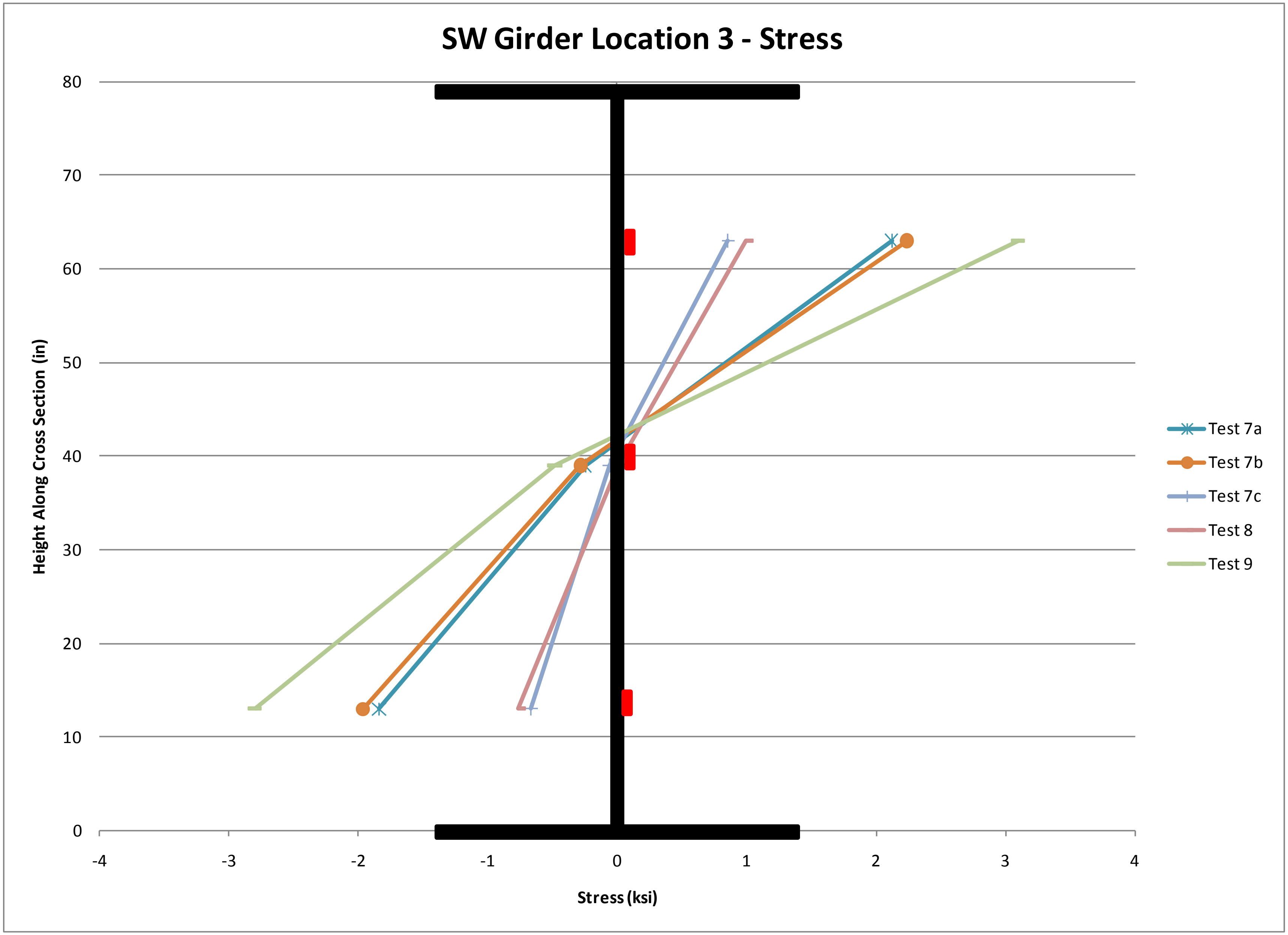

NW Girder Location 3 - Stress
80
70
60
50
40
30
20
10
0
-6
-5
-4
-3
-2
-1
0
1
2
3
4
5
Height Along Cross Section (in)
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9


NE Girder Location 1 - Stress
60
50
40
30
20
10
0
Height Along Cross Section (in)
-5
-4
-3
-2
-1
0
1
2
3
4
5
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9


NE Girder Location 2 - Stress
70
60
50
40
30
20
10
0
-4
-3
-2
-1
0
1
2
3
Height Along Cross Section (in)
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9


NE Girder Location 3 - Stress
Height on Cross Section (in)
80
70
60
50
40
30
20
10
0
-4
-3
-2
-1
0
1
2
3
4
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9


SE Girder Location 3 - Stress
80
70
60
50
40
30
20
10
0
-3
-2
-1
0
1
2
3
4
Height Along Cross Section (in)
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9


SW Girder Location 3 - Stress
Height Along Cross Section (in)
80
70
60
50
40
30
20
10
0
-4
-3
-2
-1
0
1
2
3
4
Stress (ksi)
Test 7a
Test 7b
Test 7c
Test 8
Test 9

Appendix D– Experimental Load Ratings

Load Rating of Oceanic Bridge Using the Results of the Load Test - Updating LRFR Load Ratings

1) Obtain Maximum Flexural Strains (Live Load Only) from SAP 2000 FE Model

Model Stresses/Strains

NW Girder

$\sigma_{LL1} := 10$ ksi

Maximum flexural live load stress at instrumentation location due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NW Girder

$$\varepsilon_{C1} := \frac{\sigma_{LL1} 1000000}{29000}$$

$\varepsilon_{C1} = 344.828$ microstrain

NE Girder

$\sigma_{LL2} := 8$ ksi

Maximum flexural live load stress at instrumentation location due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - NE Girder

$$\varepsilon_{C2} := \frac{\sigma_{LL2} 1000000}{29000}$$

$\varepsilon_{C2} = 275.862$ microstrain

SW Girder

$\sigma_{LL3} := 8$ ksi

Maximum flexural live load stress at instrumentation location due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SW Girder

$$\varepsilon_{C3} := \frac{\sigma_{LL3} 1000000}{29000}$$

$\varepsilon_{C3} = 275.862$ microstrain

SE Girder

$\sigma_{LL4} := 8$ ksi

Maximum flexural live load stress at instrumentation location due to 2 36 ton trucks placed with their rear axles at the tip of the bascule leaf - SE Girder

$$\varepsilon_{C4} := \frac{\sigma_{LL4} 1000000}{29000}$$

$\varepsilon_{C4} = 275.862$ microstrain

Determine the benefit derived from the field test (Obtain a value of K) in the following equation

$RF_T = RF_C \times K$ where:

RF_T = Rating factor for the live load capacity based on test results

RF_C = The rating factor predicted analytically

K = Adjustment factor resulting from the comparison of measured and analytical behavior

Determine K where $K = 1 + (K_a \times K_b)$

Calculate K_a

$K_a = (\varepsilon_c/\varepsilon_T) - 1$

$K_b = K_{b1} + K_{b2} + K_{b3}$

NW Girder

$\varepsilon_{T1} := 112$ Measured maximum strain

$K_{a1} := \frac{\varepsilon_{C1}}{\varepsilon_{T1}} - 1$ $K_{a1} = 2.079$

$K_{b1\_NW} := 1$ The member behavior can be extrapolated to 1.33 times the rating load and the ratio of the test load to the rating load is great than 0.7

Table 6-1 - Section 6.5.4 NCHRP Results Digest 234

$K_{b2\_NW} := 0.8$ Routine inspections are performed every 1 to 2 years

Table 6-2 - Section 6.5.4 NCHRP Results Digest 234

$K_{b3\_NW} := 0.8$ Fatigue controls this strucutre, however it has redundancy

Table 6-3 - Section 6.5.4 NCHRP Results Digest 234

$K_{B\_NW} := K_{b1\_NW} \cdot K_{b2\_NW} \cdot K_{b3\_NW}$

$K_{NW} := K_{a1} \cdot K_{B\_NW}$

$RF_{C\_NW} := 1.174$

$RF_{T\_NW} := RF_{C\_NW} \cdot K_{NW}$ $RF_{T\_NW} = 1.562$

NE Girder

$\varepsilon_{T2} := 104$ Measured maximum strain

$K_{a2} := \frac{\varepsilon_{C2}}{\varepsilon_{T2}} - 1$ $K_{a2} = 1.653$

$K_{b1\_NE} := 1$ The member behavior can be extrapolated to 1.33 times the rating load and tł ratio of the test load to the rating load is great than 0.7

Table 6-1 - Section 6.5.4 NCHRP Results Digest 234

$K_{b2\_NE} := 0.8$ Routine inspections are performed every 1 to 2 years

Table 6-2 - Section 6.5.4 NCHRP Results Digest 234

$K_{b3\_NE} := 0.8$ Fatigue controls this strucutre, however it has redundancy

Table 6-3 - Section 6.5.4 NCHRP Results Digest 234

$K_{B\_NE} := K_{b1\_NE} \cdot K_{b2\_NE} \cdot K_{b3\_NE}$

$K_{NE} := K_{a2} \cdot K_{B\_NE}$

$RF_{C\_NE} := 1.467$

$RF_{T\_NE} := RF_{C\_NE} \cdot K_{NE}$ $RF_{T\_NE} = 1.552$

SW Girder

$\varepsilon_{T3} := 78$ Measured maximum strain

$K_{a3} := \frac{\varepsilon_{C3}}{\varepsilon_{T3}} - 1$ $K_{a3} = 2.537$

$K_{b1\_SW} := 1$ The member behavior can be extrapolated to 1.33 times the rating load and th
ratio of the test load to the rating load is great than 0.7

Table 6-1 - Section 6.5.4 NCHRP Results Digest 234

$K_{b2\_SW} := 0.8$ Routine inspections are performed every 1 to 2 years

Table 6-2 - Section 6.5.4 NCHRP Results Digest 234

$K_{b3\_SW} := 0.8$ Fatigue controls this strucutre, however it has redundancy

Table 6-3 - Section 6.5.4 NCHRP Results Digest 234

$K_{B\_SW} := K_{b1\_SW} \cdot K_{b2\_SW} \cdot K_{b3\_SW}$

$K_{SW} := K_{a3} \cdot K_{B\_SW}$

$RF_{C\_SW} := 1.467$

$RF_{T\_SW} := RF_{C\_SW} \cdot K_{SW}$ $RF_{T\_SW} = 2.382$

SE Girder

$\varepsilon_{T4} := 84$ Measured maximum strain

$K_{a4} := \frac{\varepsilon_{C4}}{\varepsilon_{T4}} - 1$ $K_{a4} = 2.284$

$K_{b1\_SE} := 1$ The member behavior can be extrapolated to 1.33 times the rating load and the ratio of the test load to the rating load is great than 0.7

Table 6-1 - Section 6.5.4 NCHRP Results Digest 234

$K_{b2\_SE} := 0.8$ Routine inspections are performed every 1 to 2 years

Table 6-2 - Section 6.5.4 NCHRP Results Digest 234

$K_{b3\_SE} := 0.8$ Fatigue controls this strucutre, however it has redundancy

Table 6-3 - Section 6.5.4 NCHRP Results Digest 234

$K_{B\_SE} := K_{b1\_SE} \cdot K_{b2\_SE} \cdot K_{b3\_SE}$

$K_{SE} := K_{a3} \cdot K_{B\_SE}$

$RF_{C\_SE} := 1.98$

$RF_{T\_SE} := RF_{C\_SE} \cdot K_{SE}$ $RF_{T\_SE} = 3.214$