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# Protocol to Evaluate and Load Rate Existing Bridges

Maria Szerszen

Ali Alhajami

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**Final Report**
**March 2017**

**UNIVERSITY OF NEBRASKA-LINCOLN**
Nebraska Transportation Center

**Sponsored by**
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## Disclaimer Notice

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# PROTOCOL TO EVALUATE AND LOAD RATE EXISTING BRIDGES

**Final Report**

**March 2017**


**Principal Investigator**
Maria M. Szerszen

**Co-Principal Investigators**
Joshua Steelman, Richard, L. Wood

**Research Assistant**
Ali Alhajami

**Authors**
Maria M. Szerszen and Ali Alhajami



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## Introduction

In general, load rating involves determination the safe carrying capacity of a bridge. With load rating of an existing structure, bridge engineers face uncertainty in the amount of structural resistance. For load rating, results are generally expressed in terms of a rating factor for considered live load model. Rating factors greater than one indicate that the bridge is safe for the loads tested. The actual performance of most bridges is more favorable than conventional theory dictates. When a structures' computed theoretical load capacity is less than desirable, it may be beneficial to take advantage of some of bridge's inherent extra capacity that might been ignored in conventional computations. Load tests can be used to verify component and system performance under a known live load and provide an alternative evaluation methodology to analytically computed the load rating of a bridge. Procedures for nondestructive load testing, which involves field observations and measurements of in-service bridges and field testing often can revel additional capacity since analytical bridge rating tend to be fairly conservative.

The goal of presented research was to develop a data-driven rating process to assess and rate bridges. This procedure would be beneficial for problematic bridges, which are rated based on visual inspections and analytical computations only. The visual inspections may be very conservative in nature due to the fact that any significant sense of distress in the bridge will result in the inspector requesting the bridge be posted. While this distress may exist, it may not significantly affect the overall load carrying capacity of the bridge, and therefore the posting of the bridge may not be warranted. This data-driven rating system will reduce the subjective judgment of the bridge inspectors through the use of computer models and field load testing, and result in a numerically calculated load rating.

The report discusses procedures to complete a successful field test and acquire the data needed to analyze the results. The procedures to develop computer models used to analyze the bridges are also discussed. In an attempt to show that the procedures that had been developed worked for different types of bridges, four bridges of different types of structure were tested and rated based on field measurements.

## Procedure of field load testing

Prior to field load testing, a few important steps need to be performed in a preparation for actual testing. These include review of a bridge, analytical evaluation of its condition and capacity, selection of testing equipment and instrumentation plan and selection of testing load.

<u>Step 1 – Site visit and review of a bridge</u>

The first step is needed to collect all important information about bridge structure, dimensions, actual condition including any distress, supporting conditions, height of bridge, presence of water under the bridge and surrounding vegetation. These information is necessary to analytically evaluate the bridge and plan the test. The length and width of spans (determination of number of lanes), thickness of the slab, and dimensions of supporting elements (girders) need to be measured. Depending of type of a superstructure, it includes: depth of steel girders, width and thickness of flanges, spacing between girders, existence of transverse stiffeners and their location; dimensions of planks and stringers with spacing for timber bridges, thickness of concrete bridge slab etc. Availability of bridge documentation (drawings) is very helpful in this step since field measurements will potentially confirm drawings. In a case of very old bridges, where documentation is not accessible, exact and detailed measurements are very important. Information about the year bridge was built helps in determining type and strength of construction materials. Non destructive material testing should be performed in this step, for example estimation of concrete strength using rebound hammer. Any signs of deterioration of steel elements (corrosion, cracking), cracking of concrete slab (leaking cracks), moldered or charred timber elements should be documented. Actual dimensions and condition of superstructure are required to analyze bridge prior to loading test. Height of spans and accessibility is needed to plan instrumentation and conduct the test. If testing is to be performed in late autumn or winter, it needs consideration how low temperatures will affect performance of testing equipment (strain transducers for example).

Step 2 – Analysis of a bridge prior to testing

To safely, efficiently perform load bridge testing and properly collect data, it is crucial to perform bridge analysis before testing. The response of superstructure to load, shearing of load by structural elements, its capacity is needed to plan instrumentation and to select testing load. Analytical analysis can give answers to such questions but it can result in some simplifications, for example assuming to big load distribution factors (for closely spaced girders) or difficulty in estimation influence of transverse stiffeners. The better way of analysis is to model bridge using finite elements method. The accurately developed FE model will present more realistic bridge behavior under the load. CSI Bridge 2016 software was used to model investigated bridges presented in this report. CSI Bridge is versatile and productive program for bridge modeling and analysis. Complex bridge geometries, boundary conditions and load cases can be easily defined. Spine, shell or solid objects can be modeled and updated as the bridge definition parameters are changed.
Program allows for a specified spatial distribution of forces, helps to define moving loads and gives resulting internal forces under moving load.

There are three factors as the main contributors to the accuracy of the bridge models: the materials properties, load distribution and possible member end continuity. The recommendations for the creation of the models can be summarized as follows:

1. Strength of materials should be used based on field measurements (compressive strength of concrete) or assumed accordingly to the year of construction (yielding of steel). The modulus of elasticity of concrete can be evaluated on the assumption of concrete weight equal to 150 pcf and ACI equation $E_c = 33w_c1.5(f_c')^{1/2}$ or $E_c = 57{,}000(f_c')^{1/2}$. The modulus of elasticity of steel should be set to 29,000 ksi. For timber bridges, if spices of timber are not known, Douglas Fir should be assumed. BIM proposes to use timber density of 50 lb/ft$^3$, and allowable stresses for bending, $F_b$ = 1,450 psi, for shear $F_v$ = 90 psi and E = 1,000 ksi (for purpose of lateral stability). AASHTO LRFD proposes $F_b$ = 1,500 psi for 2-4 in. wide planks, $F_b$ = 1,250 psi for 5-6 in. wide planks, E = 1,600 ksi and shear parallel to grains = 175 psi. Because of orthotropic nature of wood (with three main axes: longitudinal,

L, radial, R, and tangential, T), during the FE modeling, it is important to pay attention to the orientation of grains in planks and supporting beams since it will result in different values of modulus of elasticity, $E_L$, $E_R$, $E_T$, shear modulus, $G_{LR}$, $G_{LT}$, $G_{RT}$, and Poisson ratio, $\upsilon_{LR}$, $\upsilon_{LT}$, $\upsilon_{RT}$. Values of these parameters can be adopted based on published test results, but lab values may need to be lowered because of the quality of timber (lab tests are performed on clear wood) and moisture content. This was a case during modeling of timber bridge in presented research.

2. For concrete members, gross moment of inertia, cracked moment of inertia and effective moment of inertia should be considered according to the condition of concrete. For steel members, the gross moment of inertia should be used for non-composite members and transformed moment of inertia should be used for composite members. For modeling, girders should be modeled as non-composite unless the existing documentation clearly states that they were designed and constructed as composite with deck. If upper flange of girder is embedded in the bridge deck, the cross section should be also modeled as composite. Some possible, unintended composite action can be determined based on measured strains (during testing) what will be presented on examples of investigated bridges (in Appendix).
3. Transverse load distribution percentages depend on number of girders and their spacing, number of lanes, flexural stiffness and span length.
4. Longitudinal and transverse distribution of load can be modeled as a series of point loads.
5. Most simply supported members exhibit some degree of constraints on supports. Typically, they are modeled as simply supported and after testing, the supporting conditions are calibrated by introducing horizontal spring elements of adjusted stiffness to represent support constrains.

<u>Step 3 – Selection of testing load</u>

In general, load rating vehicles should be representative of trucks typically using roads in the US. Load ratings are normally completed for three types of trucks as well as for design loading as required by AASHTO. Each state determines by statue the maximum legal axle weight and spacing for vehicles in their particular state. Three Legal Trucks are considered for rating: Type 3, Type 3S2 and Type 3-3. Nebraska Legal Trucks were used for rating in this study. Type 3 Nebraska Legal Truck is the same as Type 3 AASHTO Legal Truck; the other two types of Nebraska Legal Trucks have slightly larger weight on front axle if compared to AASHTO Legal Trucks, but the same configuration of axles. Nebraska Legal Rating Trucks are presented in Fig. 1 below.

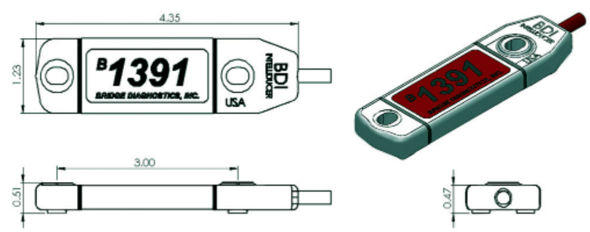
FE model of a bridge was used to select testing load for particular investigated bridge. Each type of legal rating truck was run on model for three transverse positions of the truck: left lane, right lane and on the center. Resulting strains in girders were examined to find if bridge response is safe, means in linear range of behavior. This also determined expected internal moments and shears. Test load was selected on the basis of largest load created internal forces in structural members. Since tested bridges were of simply supported short spans, the shortest Type 3 Legal Truck was selected as the testing load because it created the largest moments and shears. Prior to loading test, the loading truck needs to weighted on axis to determine the actual test load. Spacing between axes and distance between front and rare wheels also need to be measured.

Step 4 – Bridge instrumentation and testing

Instrumentation of a bridge for testing should include following considerations:

1. Selection of testing equipment – if possible modern strain transducers should be used for testing. They need to be calibrated for linear range performance. For testing performed in this study, wireless transducers from Bridge Diagnostics were used. Transducers are connected to STS nodes with signal antennas; they are operated by churched batteries. The nodes send signal to STS Base station which needs to be connected to power supply. Signal from the base station is wirelessly transmitted to PC and all data is recorded. The testing system consists of:

BDI Strain Transducers:




with specifications

| | |
|---|---|
| Type | 350Ω, Full Wheatstone bridge configuration |
| Range | ±4,000με |
| Effective Length | 3.0 In. (76.2mm) |
| Accuracy | ±1% |
| Sensitivity (approximate) | 500με/mV/V |
| Excitation | +1 to +10VDC |
| Temperature Range | -58°F to +185°F (-50°C to +85°C) |
| Size | 4.35 in x 1.23 in x 0.51 in (110mm x 31mm x 13mm) |
| Weatherproof | IP67 |

Data Acquisition System:
STS4-4 Nodes


STS4-Base Station


2. LVDTs are typically used to measure deflections. Modern type of LVDT working independently requires supporting structure, which can undergo vibrations transmitted through the ground (for smaller bridges), which disturb in accurate deflection readings, especially if load travels with a higher speed. Traditional LVDTs with spring attached to moving part are more reliable in authors opinion since they can be attached to tested structural member and spring connected by wire with heavy load positioned under the bridge. Nebraska Legal Rating Trucks are shown in Fig. 1.

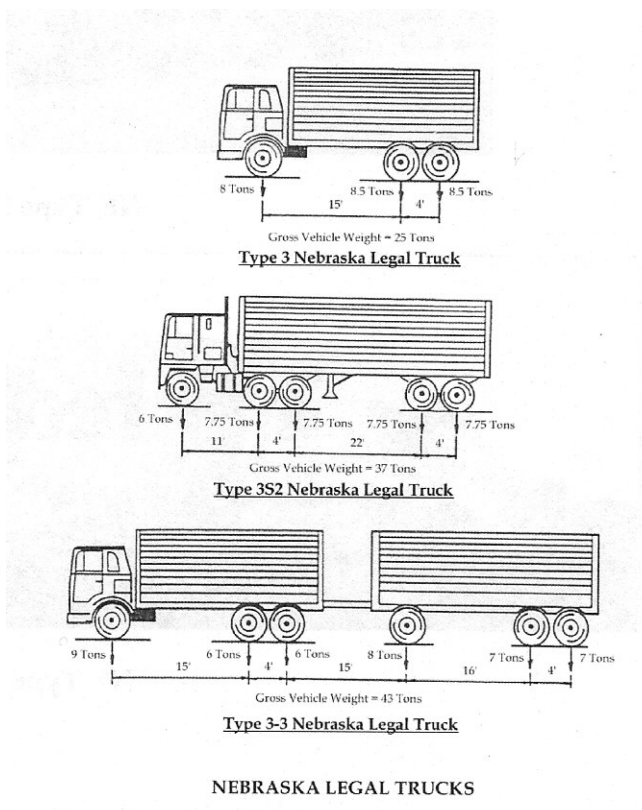



Fig. 1. Nebraska Legal Rating Trucks.

3. Plan for instrumentation – strain transducers and LVDTs should be positioned in locations of expected highest moments and deflections. FE model of bridge is helpful in localization of such places. Traditionally, for simply supported spans, strain transducers are placed in

the midspan, quarter spans and close to supports (1 foot from support). If girders are instrumented, transducers are attached centrally to bottom flanges. In the midspan and close to supports, transducers should be also attached to upper flanges (on one side) to decide of possible unintentional composite action and calibration of supporting conditions. The important requirement is that transducers are placed parallel to longitudinal axes of girders and span in general. If the concrete slab bridge is tested, transducers are also placed longitudinally to span in the midspan, quarters and close to supports (at the bottom of slab), in at least three locations transversally. Transducers can be also placed perpendicular to the span to detect load shearing in the slab. LVDTs to measure deflections are typically placed at the midspan.

4. Attachment of transducers – firm attachment requires full bonding between transducer and tested member. If attached to steel girders, paint or rust need to be gently grinded to uncover clear steel surface and transducers can be glued to girder by connecting bolts using fast acting glue. After transducers are removed, grinded surface must be spray painted to prevent corrosion. For timber bridges, transducers can be firmly attached by screws adequate for timber. For concrete members, transducers can be also glued to surface. However, typically used elongated base for concrete transducers, may result in not firm grip. In this situation, shut in steel pins can be used. LVDTs can be attached by clumps directly to the bottom flanges of girders or to supporting structure (depending of the LVDT type). Clear notes about position of each (numbered) transducer or LVDT are needed for data analysis. Picture transducer installation is presented in Fig. 2. In a case of high bridge, building scaffolding or using so called boom truck needs to be considered, Fig. 3. In presence of deep water under the bridge, snooper truck needs to be used to reach under the bridge, Fig. 4 and 5.

Fig. 2. Instrumentation of the bridge

Fig. 3. Boom truck operating under the bridge

Fig. 4. Snooper truck

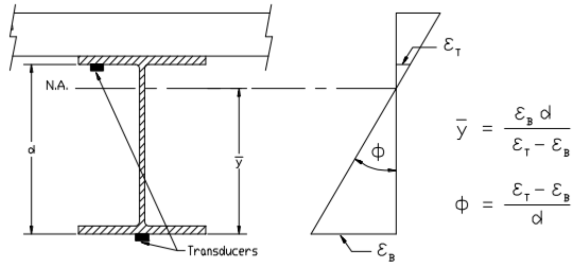
Fig. 5. Snooper operating on the bridge

5. Conduct of the test – performed at two loading truck speeds and various transverse positions of the truck. If two lanes bridge is tested, the loading truck should be positioned two feet from the curb. If more lanes, truck should travel centrally to the lane. Regardless of the number of lanes, the truck should also run positioned in the center of slab. Two speeds are used during the test: crawling speed (3, 4 mph) and higher speed (30 mph or more) adequate to the category of the road. Each run of truck is repeated at least twice to obtain consistency of results. Truck runs are monitored using computer software and recorded. Written notes taken during the test about truck position and speed for each run are very helpful in later data analysis.

## Bridge rating based on field test results

<u>Step 5 – Analysis of test results</u>

BDI program was used to record and analyze data. The software provides Data Analysis and Conversion program that allows for quicker data conversion during the analysis phase of bridge rating. Distribution of longitudinal strains measured at the midspans were plotted for all girders depending on transverse position of loading truck (lane 1, lane 2 or at the center). Such diagrams allow for computation of Girder Distribution Factor (GDF). GDFs for three truck positions were plotted in a separate diagram. Strains were converted to live loading moments. For mostly loaded girders, strains distributions along the length of span were also plotted versus time (time the truck was traveling on the bridge). These plots were prepared for strains measured under truck moving with very low speed (crawling speed) and when truck speed was higher. Ratio of dynamic portion of strains to static portion of strains allowed to estimate dynamic load factor for each bridge (see details for each tested bridge in Appendices). In general, these factors were lower than provided by AASHTO specifications. For the bridge with truss supporting the middle span, measured strains in lower and upper chords, and vertical, diagonal elements were converted into forces. Measured strains in floor beams in this span were also converted to live load moments. Strains measured at the bottom and upper flange of girder (at the same location at midspan) allow to compute the position of neutral axis in the cross-section and identify possible, unintended some composite action.




In a case when strains in upper girders can not be measured (upper flanges to narrow to attach transducers or upper flanges embedded in concrete slab), the other method to detect such action will be described below.

The next step in analysis of test results was to calibrate FE models. The calibration was performed on the basis that measured live load moment on mostly loaded girders should be very close to moment obtained from model under the actual loading truck (with specified load on each axle and known spacing between them). Springs with adjusted stiffness were introduced at the ends of girders, representing some constrains at supports. Once the stiffness was selected, the same spring elements were used for all beams in the model. The idea of this procedure is that calibrated model should reflect the real bridge behavior (measured) under the specified load. Calibrated bridge model was used to generate shear live forces in the beams, and moments and shears under the dead load.

Calibrated bridge FE model can be also used to estimate possible composite behavior of girders. In the model, it can be declared non-composite cross-section or fully composite. Live load moments (under the test truck) can be generated for these two options and compared to measured live load moment (at the same location). Obtained three curves for live load moment distribution for a given girder show position of actual moment. If actual, measured live load moment is positioned between moments generated for non-composite and full- composite sections, some composite behavior of girders can be determined. The procedure is illustrated in Appendix.

Step 6 – Rating based on test results

The idea of bridge rating performed based on the field test results is to use measured values of strains under a test load. The test is considered a diagnostic one, but important thing is to use the legal rating truck as a test load since results are to be used in rating of a bridge. The legal rating truck producing the largest internal forces in bridge elements should be selected as a test load, which usually depends on on the length of a span. The conversion of live load strains to the internal loading forces depends on a type of structure and evaluated structural elements, be it steel girders, timber beams, floor beams, solid concrete slabs or truss elements. The cross-sections of these elements are used with their moments of inertia, section modulus, and strength of materials like concrete, steel or timber with adequate moduli of elasticity. The dead load internal forces are best to be evaluated from calibrated FE model developed in the preliminary analysis. Nominal capacity of structural elements, for example the nominal moment capacity for steel girders should be computed based on parameters of a cross-section and assumed yielding strength of material. In a case when some or full composite action was detected during the test, it should be included in rating. The nominal moment capacity should be computed including composite cross-section with a part of concrete slab. The position of neutral axis will be computed for a transformed cross section, transforming concrete part to steel using ratio of elastic moduli of both materials. Than, the internal forces in tension and compression in the cross-section can be computed and finally, the internal moment capacity. The capacity of axially loaded structural elements is computed using a size of cross-section and a strength of material. The dynamic factor (IM) should be evaluated during the test comparing strains measured at the same location under crawling and high speed of test truck traveling on the same lane (typically at the center of slab). The difference between dynamic and static strains is measured by static strains to evaluate dynamic factor. This factor can be also evaluated based on measured deflections at the same location on the same principle. Such prepared values of loading forces (live load and dead load), nominal capacities and dynamic factor can be used in rating formula.

Rating of the bridge can be performed using Load Factor Method (LFR) according to formula:

$$RF = \frac{\emptyset C - \gamma DL \times D}{\gamma LL \times LL(1 + IM)}$$

C = nominal member resistance
D = unfactored dead loads
LL= unfactored live loads
$\emptyset$ = resistance factor (based on construction material)
IM = impact
$\gamma DL$ = dead load factor
$\gamma LL$ = live load factor

Adopted values for factors:

$\gamma DL$ = 1.3
$\gamma LL$ = 2.17 (inventory)
= 1.30 (operating)
$\emptyset$ = 1.00 (steel members in flexure)

For a reinforced concrete elements resistance factor needs to be selected according to the magnitude of strains in the most tensioned reinforcing bars. If these strains are larger or equal to 0.005, a tension control failure mode can be assumed and resistance factor will be equal 0.9. If the strains would be smaller or equal to yielding strains, a compression control failure mode needs to be assumed and resistance factor equal to 0.65. For strains in between (transition zone), resistance factor is computed based on actual strains.

In a case when preliminary analysis indicates that legal rating truck is to heavy, unsafe load for a test which can be situation for very old and heavily deteriorated bridges, not fully loaded truck can be used for the test. Finite element model needs to be calibrated using measured results for such a load. With the assumption that calibrated model will respond to load as a real bridge, live load internal forces can be obtained from the model running the fully loaded legal rating truck on the bridge model. Dead load results can be also computed from such model. These values can be used in the bridge rating.

**Closing remarks**

Performing the bridge rating based on field test results can show still existing capacity in an older structure which was for example posted based on the analytical rating. Measured values will show real behavior of a bridge with all its signs of deterioration or restrained supporting conditions, which can be difficult to analytically evaluate. If an older bridge was designed and constructed as non-composite, it would be analytically rated like that. During the field test, it is possible to detect some not intended composite action created by friction or partially embedded upper flanges of girders in concrete slab. This situation is changing bridge response to the load (live load and dead load as well) and is also changing a bridge capacity. The next benefit coming from load testing is using actual values of loading moments or shears instead of using girder distribution factors according to the code provisions, which give unrealistic values for bridges with closely spaced girders. Dynamic or impact factor, IM, evaluated using measured strains under very slaw and high speed of loading truck gives a realistic value of this factor specific for a tested bridge and typically much lower than 0.30 proposed in the code provisions (for highway bridges). There are a few important steps to perform successful bridge rating using the results of field testing. One of them is the initial analysis prior to testing. This helps in selection of loading truck for test. Typically, it would be a legal rating truck but prior analysis should answer a question if this load will be safe for a bridge. Very helpful is also development of finite element model of a bridge because when it is calibrated using test results allows for estimation of dead load internal forces, which is more precise than analytical calculations only. Finally, the important part is to properly compute live load effects using measured values of strains. Including unintentional, but possible some composite action in a cross-section can improve values of rating factors.

## Appendix -Examples of bridge rating based on field testing

Several bridges were reviewed as potential candidates and finally five bridges were selected, analyzed, field tested and rated as a part of project program. Selected bridges were of different types of structure: concrete deck supported on steel girders, timber bridge, steel truss supporting steel floor beams and steel grid deck, concrete slab bridge and concrete deck supported on steel girders with thick layer of soil on top of the deck. Each of these bridges required specific approach to analysis, instrumentation and testing procedure. Evaluation of these bridges is presented as a separate case below.

### Bridge 1 – one span steel bridge located close to Weston, Sounders County, NE (ID #C007803635)

One span, simply supported bridge carried concrete deck with gravel wearing surface supported on steel girders. Information on bridge dimensions and posting is as follows:

Length of span = 24 feet
Width of deck = 20 feet
Thickness of reinforced concrete slab = 5 inches
Top layer of gravel = 1.5 inch
Number of girders = 10

Interior and exterior girders differed in the cross section, Fig. 7 and 8. Spacing between interior girders was 2.4 ft, and between exterior and the first interior girders was 2.2 ft.

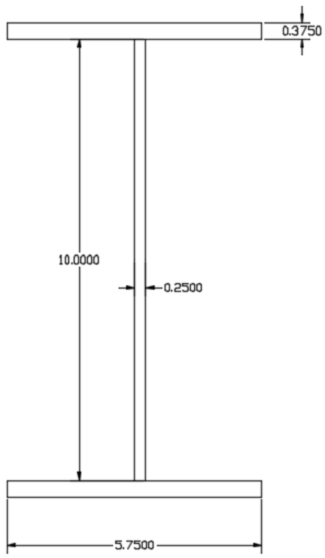



Fig. 7. Interior girders

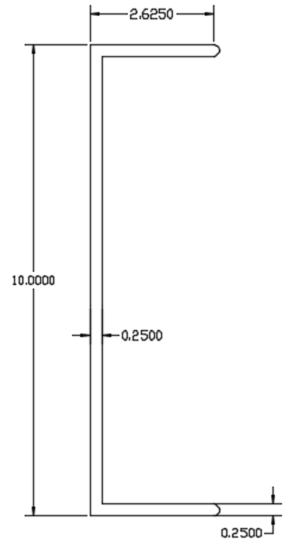



Fig. 8. Exterior girders

Posting information:

21 T 

33 T 

43 T 

View of the bridge is presented in photos below.


The first step in bridge evaluation was site inspection to determine bridge dimensions and compare to available bridge drawings. Compressive strength of concrete in deck was evaluated using rebound (Schmidt) hammer as $f_c' = 4000$ psi; yielding strength of steel was assumed, $f_y = 33$ ksi (based on the year bridge was built). Modulus of elasticity of concrete was assumed, $E_c = 57000(f_c')^{0.5}$ and steel modulus, $E_s = 29000$ ksi.

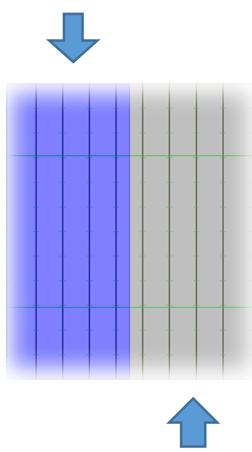
Preliminary analysis

The analysis of the bridge structure conducted prior to the load testing is discussed in this part. The goal of this analysis was to identify the beam's design parameters and to determine the maximum load that could be safely applied during the test, estimate expected strains, moments and shears in girders.

Analytical analysis was performed to estimate dead and live load moments and resulting possible stresses in the girders. Section properties were calculated based the data in Fig. 7 and 8. Dead load moment was calculated using the weight of the beam, weight of part of slab supported on girder and top layer of gravel. Live load moment was calculated using legal rating truck Type 3 with axial load of 16 kips on front axle and 17 kips on both rear axis (total load 50 kips), with spacing 15 ft and 4 ft between axis. Load distribution factor of S/4.7, where S is the beam spacing, GDF = 0.46, and impact factor, IM = 0.33 (from AASHTO formula) were used in computations. Dead load moment, $M_D$ = 140.5 in-k, and live load moment, $M_L$ = 794.9 in-k were used to analytically rate the bridge; resulting inventory rating factor, RF = 0.29. It is understood that so low value is the effect of GDF computed from formula which applies to girder spacing 3.6 ft and larger. With assumption of total live load moment evenly distributed to girders, RF = 1.18.

The next step of the preliminary analysis was to model the bridge using CSI Bridge 2016 software. Bridge was modeled according to drawings and field measurements with the assumption of simply supported span. Computer model was used to predict moments and shears resulting of planned testing truck (Type 3) traveling on the bridge model. Other legal rating trucks, Fig. 1, were not used in preliminary analysis since they did not produce larger internal forces on a short span bridge.

Results from finite element preliminary analysis are presented below.



- The first run was on the 1st lane (gray)
- The second run was on the 2nd lane (blue)
- Truck speed = 5 mph

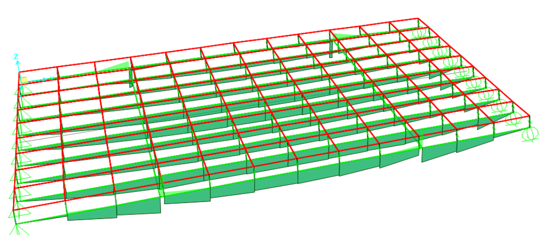
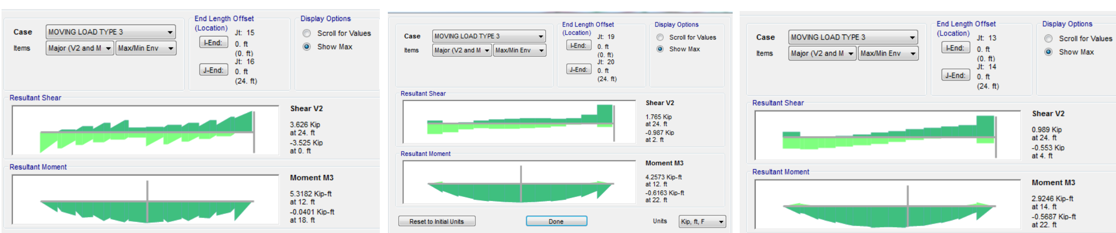


Fig. 9. Results due to truck on lane 1st

Table 1. Preliminary results for the truck running on lane 1st

| Truck on 1st Lane | | | |
|---|---|---|---|
| Girder # | Shear (K) | Moment (K-ft) | Location (ft ) |
| 1 | 1.22 | 2.9466 | 5.833 |
| 2 | 1.765 | 4.2573 | 12 |
| 3 | 14.072 | 8.8953 | 16.8 |
| 4 | 3.626 | 5.3182 | 12 |
| 5 | 0.989 | 2.9246 | 14 |
| 6 | 0.61 | 2.07 | 12 |
| 7 | 0.412 | 1.4991 | 12 |
| 8 | 0.276 | 1.1924 | 12 |
| 9 | 0.349 | 1.0913 | 10 |
| 10 | 0.74 | 5.6293 | 18.1667 |



Girder 2 Girder4 Girder 5

Fig. 10. Examples of moment and shear diagrams for mostly loaded girders (truck on lane 1st)

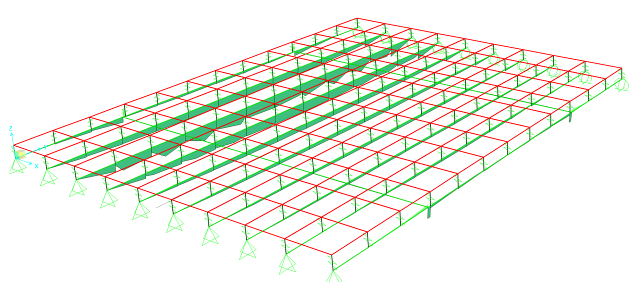
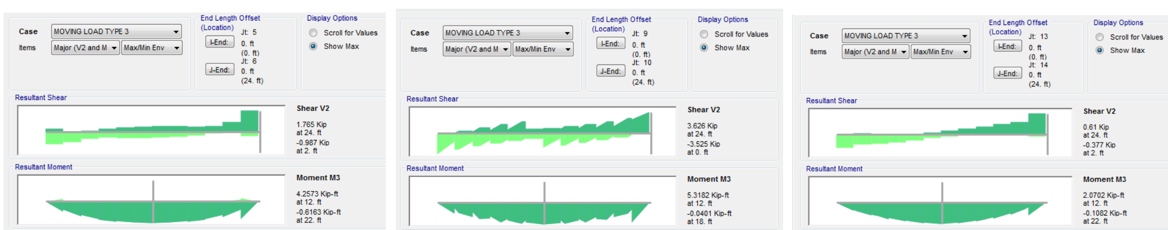


Fig. 11. Results due to truck on lane 2nd

Table 2. Preliminary results for the truck running on lane 2nd

| Truck on 2nd Lane | | | |
|---|---|---|---|
| Girder # | Shear (K) | Moment (K-ft) | Location (ft ) |
| 10 | 1.141 | 2.9466 | 18.1667 |
| 9 | 1.765 | 4.2573 | 12 |
| 8 | 14.072 | 8.8953 | 16.8 |
| 7 | 3.626 | 5.3182 | 12 |
| 6 | 0.989 | 2.9246 | 14 |
| 5 | 0.61 | 2.07 | 12 |
| 4 | 0.412 | 1.4991 | 12 |
| 3 | 0.276 | 1.1924 | 12 |
| 2 | 0.349 | 1.0913 | 10 |
| 1 | 0.74 | 5.6293 | 18.1667 |



Girder 9 Girder 7 Girder 5

Fig. 12. Examples of moment and shear diagrams for mostly loaded girders (truck on lane 2nd)

The bridge model was also used to estimate resulting forces of the self weight of bridge.

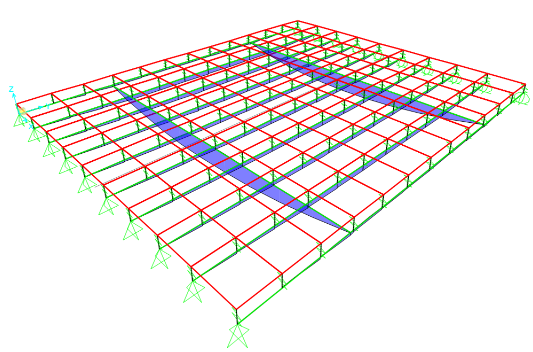


Fig. 13. Results due to self weight of bridge

Table 3. Preliminary results for the self weight of bridge

| Girder | Shear (K) | Moment (K-ft) | Location (ft) |
|---|---|---|---|
| 1 | 3.894 | 2.9653 | 5.8 |
| 2 | 2.445 | 6.4115 | 12 |
| 3 | 2.055 | 6.0461 | 12 |
| 4 | 1.796 | 5.711 | 12 |
| 5 | 1.694 | 5.4849 | 12 |
| 6 | 1.694 | 5.4849 | 12 |
| 7 | 1.796 | 5.711 | 12 |
| 8 | 2.055 | 6.0461 | 12 |
| 9 | 2.445 | 6.4115 | 12 |
| 10 | 3.894 | 2.9653 | 18.16 |
| Stiffener 1 | 4.9 | 24.9339 | 10 |
| Stiffener 2 | 4.9 | 24.9318 | 10 |

Based on these preliminary results, maximum stresses in girders were evaluated.

Steel internal girders (max moments)
$M_L = 106.8$ *Kip-in*
$M_D = 72.6$ *Kip-in*
$M_T = 179.4$ *Kip-in*
$\sigma = 7.04$ *ksi*

Steel external girders (max moments)
$M_L = 67.56$ *Kip-in*
$M_D = 35.64$ *Kip-in*
$M_T = 103.2$ *Kip-in*
$\sigma = 10.52$ *ksi*

Based on preliminary results for maximum stresses in girders under loading truck and self weight much smaller than yielding strength of steel, $f_y = 33$ ksi, truck Type 3 as the legal rating truck, was selected as the test load.

Bridge instrumentation

The bridge was instrumented with wireless Bridge Diagnostics strain transducers. Transducers were attached to the bottom flanges of the girders at the midspan and 1 ft from the supports (parallel to longitudinal axis of girders), Fig. 14. Normally, transducers would be attached to the upper flanges of girders, usually to the most loaded ones (based on preliminary analysis). Unfortunately, width of the upper flanges in this bridge was to narrow to install instruments. Strain transducers were attached using steel bolts glued to the flanges using instant adhesive Loctite 410 with Loctite accelerator 7452™. Surface of flanges was grinded using electric grinder to clean the surface of paint and possible rust and allow for good grip of instruments. After the test, all grinded surfaces were painted to protect against corrosion. Location of transducers is shown below, Fig.15.

Fig. 14. Transducers attached to girders

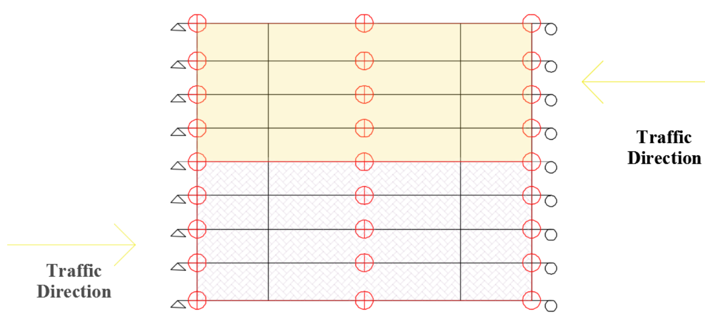



Fig. 15. Location of transducers

Signal receiving station was located on the site of the river bank to read the signal and transmit it to recording computer, Fig. 16.

Fig. 16. Receiving station

Bridge test

Based on the prior analysis, rating truck Type 3 was selected as a test load. Before the test, truck was loaded and actual axial load was measured on the police load station. Front axle was loaded to 15.14 kips and two rear axles were loaded to 17.71 kips each, with total load of 50.56 kips. Spacing between axles was 15 ft and 4.75 ft.

Bridge test was performed by running the test truck on lanes 1 and 2 positioned one ft from the curb and at the center of the slab. The truck was driving with a crawling speed about 4 mph. Each run was repeated to confirm readings. Strains measured by transducers were recorded using Bridge Diagnostics computer software. The last two runs were with truck traveling with higher speed 35 mph on the center of slab. Readings under higher speed allowed to estimate dynamic factor for this bridge.

Test results

During the test, strains were monitored and recorded. Strain values were converted to stresses using properties of girders cross sections. Measured strains also allowed to compute real distribution factors for live load. Table 4 presents recorded results. Fig. 17 shows girder distribution factors for each girder depending on the transverse position of the load.

Based on measured strain values, FE model was calibrated correcting supporting conditions by introducing horizontal springs with adjusted stiffness to represent some restrictions on the supports. Two options of cross-section were considered in the model: non-composite section and fully composite section.
Measured strains if compared to two strains obtained from calibrated model for these two options (composite or non-composite) show that measured strains were between, means that in fact bridge behaved as with some composite action, Fig. 18.
Measured strains/stresses allowed to compute live load moment. Dead load stresses for non-composite and composite actions were obtained from calibrated model, presented in Table 5.

Table 4. Measured strains and resulting stresses on midspan of girders depending on position of load

| E | 0.029 | Center Lane | | | First lane | | | Second Lane | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Girder # | Stress | Strain | DF | Stress | Strain | DF | Stress | Strain | DF |
| Ext1(south) | 1 | 2.66 | 91.72413793 | 0.063351736 | 7.45 | 256.8965517 | 0.171833988 | 1.73 | 59.65517241 | 0.040669519 |
| | 2 | 3.36 | 115.862069 | 0.080023245 | 7.22 | 248.9655172 | 0.166529046 | 2.35 | 81.03448276 | 0.055244722 |
| | 3 | 3.98 | 137.2413793 | 0.094789439 | 6.28 | 216.5517241 | 0.144847979 | 3 | 103.4482759 | 0.070525177 |
| | 4 | 4.827 | 166.4482759 | 0.114961965 | 5.562 | 191.7931034 | 0.128287334 | 3.84 | 132.4137931 | 0.090272227 |
| | 5 | 5.03 | 173.4482759 | 0.119796703 | 5.17 | 178.2758621 | 0.119245868 | 4.87 | 167.9310345 | 0.114485871 |
| | 6 | 5.133 | 177 | 0.122249796 | 3.95 | 136.2068966 | 0.091106611 | 5.09 | 175.5172414 | 0.119657718 |
| | 7 | 5.28 | 182.0689655 | 0.125750813 | 2.964 | 102.2068966 | 0.068364556 | 5.42 | 186.8965517 | 0.127415487 |
| | 8 | 4.48 | 154.4827586 | 0.10669766 | 2.16 | 74.48275862 | 0.049820324 | 5.83 | 201.0344828 | 0.137053928 |
| | 9 | 3.97 | 136.8965517 | 0.094551274 | 1.57 | 54.13793103 | 0.036211995 | 5.43 | 187.2413793 | 0.127650571 |
| Ext2 (north) | 10 | 3.2678 | 112.6827586 | 0.077827369 | 1.0298 | 35.51034483 | 0.023752301 | 4.978 | 171.6551724 | 0.117024778 |
| | Tot | | 1447.855172 | | | 1495.027586 | | | 1466.827586 | |

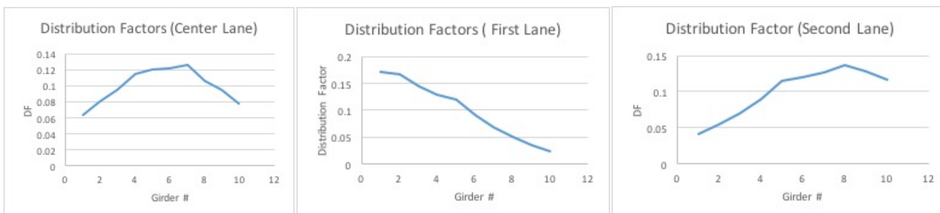



Fig. 17. Girder Distribution Factors based on measured strains in the midspan depending on the transverse position of load

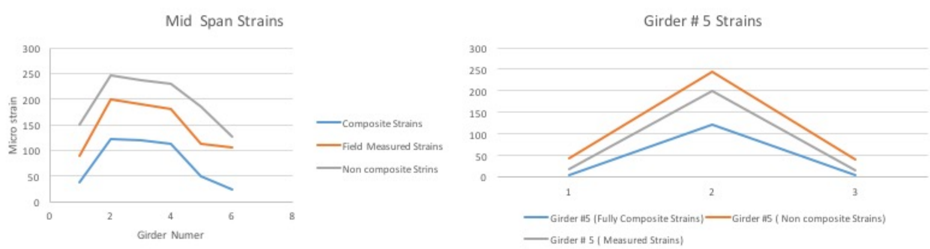



Fig. 18. Comparison of measured strains and computed for non-composite and composite cross-section

Dynamic factor was evaluated based on measured dynamic strains under high speed truck run and static strains measured at crawling speed and was estimated as 0.166. Example of measured strain values used for this evaluation are presented in Fig. 19.

Table 5. Dead load stresses on midspan of girders computed based on calibrated model for non-composite and composite cross-sections

| Girder # | Composite Dead Load Stress (Ksi) | Non Composite Dead Load Stress (Ksi) |
|---|---|---|
| 1 | 2.7251 | 1.8431 |
| 2 | 2.746 | 3.6513 |
| 3 | 2.6692 | 3.5641 |
| 4 | 2.6671 | 3.5138 |
| 5 | 2.6845 | 3.491 |
| 6 | 2.6845 | 3.4911 |
| 7 | 2.6671 | 3.5118 |
| 8 | 2.6692 | 3.5644 |
| 9 | 2.746 | 3.6514 |
| 10 | 2.7258 | 1.8429 |

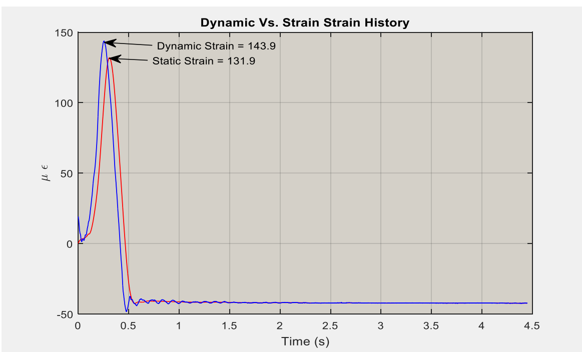



Fig. 19. Measured dynamic and static strains for evaluation of Dynamic Factor

<u>Bridge rating based on test results</u>

Rating was performed for the interior and the exterior girders considering three possible conditions of cross-section behavior:

- Non-composite cross-section
- Fully composite cross-section
- Some composite cross-section action

For each case, live load moment was calculated based on measured maximum strains under the truck load and adequate cross-section parameters (area, moment of inertia and position of neutral axis). For full composite action, the cross-section was transformed (concrete to steel) using moduli ratio. Width of the slab working with internal and external girders was computed according to AASHTO specifications. Position of neutral axis and moment of inertia for transformed cross-sections were computed for each case.

For some composite action, the width of corresponding slab was decreased based on estimated percentage of composite action. The estimation was based on comparison of strains: measured and computed for non-composite and fully composite cross-section, Fig. 16. 60% of composite action was assumed for interior girders and 40% for exterior girders. More precise way to estimate some composite action would be by finding position of the neutral axis based on measured strains at the bottom and top flanges of girders assuming linear distribution. As mentioned above, strains on upper flanges were not measured, so the possible composite action was estimated based on comparison of strains at the bottom flanges as already explained. Dead load moment was computed using maximum stresses in girders (from calibrated model) using parameters of adequate cross-sections (three options).

**Rating of interior girder W10x22, non-composite:**
$I_x = 136.93\ in^4$, $f_y = 33$ ksi, $y = 5.375$ in
$M_n = (33 \times 136.96)/5.375 = 840.87$ in-k
$M_{LL} = 194.54$ in-k (max on girder#2)
IM = 0.166 (measured), (if computed IM = 50/(125+24) = 0.335)
$M_D = (3.65 \times 136.96)/5.375 = 93$ in-k

Inventory rating

$$RF = \frac{(1.0)(840.87) - 1.3(93)}{2.17(194.54)(1 + 0.166)} = 1.46$$

Operating rating:

$$RF = \frac{(1.0)(840.87) - 1.3(93)}{1.30(194.54)(1 + 0.166)} = 2.44$$

**Rating of exterior girder C10x15.3, non-composite:**
$I_x = 49.06\ in^4$, $f_y = 33$ ksi, $y = 5.0$ in
$M_n = (33 \times 49.06)/5 = 323.8$ in-k
$M_{LL} = 51.5$ in-k (max on girder#10)
IM = 0.12 (measured)
$M_D = (1.84 \times 49.06)/5 = 18.05$ in-k

Inventory rating:

$$RF = \frac{(1.0)(323.8) - 1.3(18.05)}{2.17(51.5)(1 + 0.166)} = 2.3$$

Operating rating:

$$RF = \frac{(1.0)(323.8) - 1.3(18.05)}{1.3(51.5)(1 + 0.166)} = 3.9$$

**Rating of interior girder W10x22, full composite:**
b = 26 in (width of concrete slab)
Transformed cross-section properties:
$I_T = 548.82\ in^4$, $f_y = 33$ ksi, $y = 11.55$ in

Nominal moment capacity based on equilibrium of compression and tension forces and internal arm:
$M_n = 2270$ in-k
$M_{LL} = (7.2 \times 548.82)/11.55 = 342.1$ in-k (max on girder#2)
IM = 0.166 (measured) (if computed IM = 50/(125+24) = 0.335)
$M_D = (2.746 \times 548.82)/11.55 = 130.48$ in-k

Inventory rating:

$$RF = \frac{(1.0)(2270) - 1.3(130.48)}{2.17(342.1)(1 + 0.166)} = 3.30$$

Operating rating:

$$RF = \frac{(1.0)(2270) - 1.3(130.48)}{1.3(342.1)(1 + 0.166)} = 5.50$$

**Rating of exterior girder C10x15.3, full composite:**

b = 14.5 in (width of concrete slab)

Transformed cross-section properties:
$I_T$ = 251.63 in$^4$, $f_y$ = 33 ksi, y = 10.96 in
Nominal moment capacity based on equilibrium of compression and tension forces and internal arm:
$M_n$ = 1188.27 in-k
$M_{LL}$ = (1.84 x 251.63)/10,96 = 114.8 in-k (max on girder#10)
IM = 0.12 (measured)
$M_D$ = (2.73 x 251.63)/10.96 = 62.68 in-k

Inventory rating:

$$RF = \frac{(1.0)(1188.27) - 1.3(62.68)}{2.17(114.8)(1 + 0.166)} = 3.8$$

Operating rating:

$$RF = \frac{(1.0)(1188.27) - 1.3(62.68)}{1.3(114.8)(1 + 0.166)} = 6.36$$

**Rating of interior girder W10x22, with some composite action:**

b = 15.6 in (width of concrete slab)
Transformed cross-section properties:
$I_T$ = 477.15 in$^4$, $f_y$ = 33 ksi, y = 10.7 in

Nominal moment capacity based on equilibrium of compression and tension forces and internal arm:
$M_n$ = 2080 in-k
$M_{LL}$ = (7.2 x 477.15)/10.7 = 321.1.1 in-k (max on girder#2)
IM = 0.166 (measured) (if computed IM = 50/(125+24) = 0.335)
$M_D$ = (2.746 x 477.15)/10.7 = 122.45 in-k

Inventory rating:

$$RF = \frac{(1.0)(2080) - 1.3(122.45)}{2.17(321.1)(1 + 0.166)} = 2.36$$

Operating rating:

$$RF = \frac{(1.0)(2080) - 1.3(122.45)}{1.3(321.1)(1 + 0.166)} = 3.95$$

**Rating of exterior girder C10x15.3, with some composite action:**
b = 5.8 in (width of concrete slab)
Transformed cross-section properties:
$I_T$ = 189.29 $in^4$, $f_y$ = 33 ksi, y = 9.3 in

Nominal moment capacity based on equilibrium of compression and tension forces and internal arm:
max stress = 5 ksi
$M_n$ = 929.2 in-k
$M_{LL}$ = (5.0 x 189.29)/9.3 = 101.77 in-k (max on girder#10)
IM = 0.12 (measured)
$M_D$ = (2.73 x 189.29)/9.3 = 55.57 in-k

Inventory rating:

$$\mathrm{RF} = \frac{(1.0)(929.3) - 1.3(55.6)}{2.17(101.77)(1 + 0.166)} = 3.32$$

Operating rating:

$$RF = \frac{(1.0)(929.3) - 1.3(55.6)}{1.3(101.77)(1 + 0.166)} = 5.55$$

Comparison of rating factors for this bridge is summarized in Table 6.

Table 6. Summary of rating factors

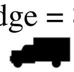
| | Non-composite | | Fully composite | | Some composite action | |
|---|---|---|---|---|---|---|
| **RF** | Inventory | Operating | Inventory | Operating | Inventory | Operating |
| **Internal girder** | 1.46 | 2.44 | 3.30 | 5.50 | 2.36 | 3.95 |
| **External girder** | 2.30 | 3.90 | 3.80 | 6.36 | 3.32 | 5.55 |

It can be seen that bridge rating performed based on field diagnostic testing results in much improved rating factors if compared to analytical rating discussed in the preliminary analysis part of this document. The measured values of strains caused by the legal rating truck, Type 3, allow for realistic estimation of live load moments. It is especially beneficial if girder spacing is smaller than 3.6 ft and code formula gives higher, not true values for GDF. The dynamic factor estimated from test measurements, lower than computed according to code also results in increased values of rating factors. If possible, some composite action between girders and concrete deck due to friction or partially embedded upper flange in concrete, as it was a case of this bridge, can be detected during the field test and included in rating procedure, it can also increase values of rating factors.

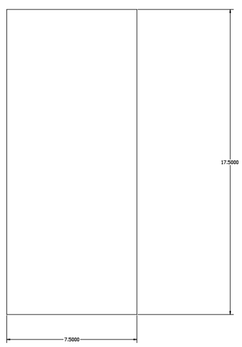
## Bridge 2 – Timber bridge located in Platte County, NE (ID # C007101805)

One span bridge built with timber girders supporting deck made of timber planks carries a rural road over a small creek. Photos of side view of the bridge are shown below. The bridge was built in 1935 and reconstructed in 1981. During reconstruction, rotted timber piles on one side of support were strengthen with new steel H-piles creating the new support and shortening the original span length from 31 ft to 29 ft.

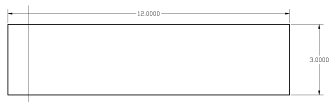

Specification and dimensions of the bridge are as follows:

One span simply supported bridge
Length of span = 29 ft
Width of deck = 16 ft
Girders: 11 timber beams
Slab: timber planks
Transverse bracing between girders
Height of bridge = 8 ft
Posting: 20 T 

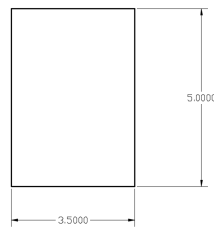
Dimensions of cross sections (in inches):




girders 17.5x7.5




planks 12x3




bracing 5x3.5

During inspection of the bridge, dimensions of structural elements were measured, also the length of the span and width of the slab. Spacing between girders (on axis) was 18.6 inches. It was noticed that the the length of the span was shortened by constructing new steel support on one side of the bridge due to deterioration of an original timber abutment, Fig 20. Quality of timber was estimated as not of the highest grade, Fig. 21, which was included in material strength parameters assumed for prior analysis and finite element modeling.

Fig. 20. New steel support

Fig. 21. View under the bridge

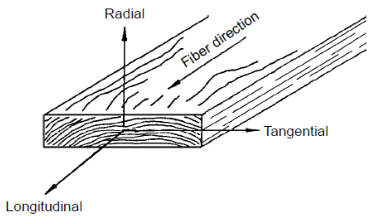
<u>Preliminary analysis</u>

The analysis of the bridge structure conducted prior to the load testing was performed for this bridge. The goal of this analysis was to identify bridge design parameters and to determine the maximum load that could be safely applied during the test, estimate expected strains, moments and shears in girders.

Analytical analysis was performed to estimate dead and live load moments and resulting possible stresses in the girders. Section properties of the girders were calculated based the measured dimensions. Since the information on wood spices and its mechanical properties was not available, the Douglas Fir was assumed with 12% of moisture and modulus of rupture, $F_b$ = 1,800 psi. This value was lowered 20%, because of assumed lower grade lumber, to $F_b$ = 1,450 psi. Density of lumber was assumed as 50 lb/ft$^3$. Analytically estimated moments in girders with $I_x$ = 3349.6 in$^4$ were: dead load moment, $M_D$ = 90.92 k-in, live load moment for Type 3 truck, $M_L$ = 246.55 k-in, total moment, $M_T$ = 337.47 k-in, resulting in tensile stress = 826.5 psi. It should be mentioned that formula for distribution factor DF = S/5.5 could not be used for closely spaced girders and one lane bridge and live load moment was estimated by dividing total live load moment by 11 (number of girders).

The next step of the preliminary analysis was to model the bridge using CSI Bridge 2016 software. Bridge was modeled according to drawings and field measurements with the assumption of simply supported span. Computer model was used to predict moments and shears resulting of planned testing truck traveling on the bridge model.

Since timber is an anisotropic material, its properties like moment of elasticity, shear modulus and Poisson ratio depend on the orientation of axis (three major axis) in reference of wood grains. Values of these mechanical properties adopted for FE model are presented below.




| Modulus of Elasticity | | |
|---|---|---|
| $E_L$ | 1.95E+06 | Psi |
| $E_R$ | 1.33E+05 | Psi |
| $E_T$ | 9.75E+04 | Psi |

| Shear Modulus | | |
|---|---|---|
| $G_{LR}$ | 1.25E+05 | Psi |
| $G_{LT}$ | 1.52E+05 | Psi |
| $G_{RT}$ | 1.37E+04 | Psi |

| Poisson Ratio | |
|---|---|
| $\nu_{LR}$ | 0.292 |
| $\nu_{LT}$ | 0.449 |
| $\nu_{RT}$ | 0.39 |

CSI Bridge software was used to model the bridge. Frame elements were used to model the timber girders and the deck planks. The dead load moment based on FE model was very close to hand calculated. Values of dead load, DL, moment resulting from model are presented in Table 7 and Fig. 22.

Table 7. FEM DL moments

| Girder | $M_{DL}$ (K-ft) |
|---|---|
| 1 | 6.2857 |
| 2 | 6.8349 |
| 3 | 7.0276 |
| 4 | 7.0713 |
| 5 | 7.0786 |

| | |
|---|---|
| 6 | 7.0793 |
| 7 | 7.0786 |
| 8 | 7.0713 |
| 9 | 6.8349 |
| 10 | 6.8375 |
| 11 | 6.2857 |

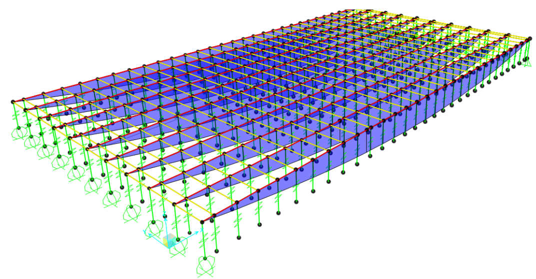


Fig. 22. DL moments

Each legal rating truck, Type 3, Type 3S2 and Type 3-3 was run on the model to choose which type will produce the highest moments. The Tables 8, 9 and 10, below show the results of preliminary analysis. It was found that Type 3 as the shortest truck produced the highest values and was selected as the testing truck.

Table 8. Type 3 truck moments

| Girder # | Moment (K-in) |
|---|---|
| 1 | 170.528 |
| 2 | 212.501 |
| 3 | 281.261 |
| 4 | 384.466 |
| 5 | 335.597 |
| 6 | 322.985 |
| 7 | 339.283 |
| 8 | 372.098 |
| 9 | 272.752 |
| 10 | 204.746 |
| 11 | 163.071 |

Table 9. Type 3S2 truck moments

| Girder # | Moment (K-in) |
|---|---|
| 1 | 165.784 |
| 2 | 209.062 |
| 3 | 272.537 |

| | |
|---|---|
| 4 | 365.975 |
| 5 | 322.746 |
| 6 | 310.92 |
| 7 | 325.644 |
| 8 | 354.135 |
| 9 | 264.465 |
| 10 | 201.488 |
| 11 | 158.264 |

Table 10. Type 3-3 truck moments

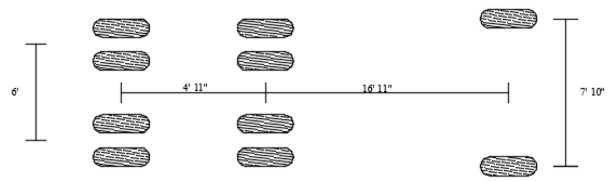
| Girder # | Moment (K-in) |
|---|---|
| 1 | 132.963 |
| 2 | 162.895 |
| 3 | 203.847 |
| 4 | 273.831 |
| 5 | 237.975 |
| 6 | 229.139 |
| 7 | 240.394 |
| 8 | 268.866 |
| 9 | 197.888 |
| 10 | 157.287 |
| 11 | 126.954 |

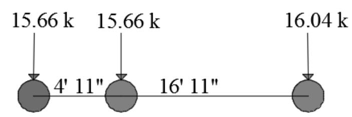
<u>Bridge instrumentation</u>

The bridge was instrumented with wireless Bridge Diagnostics strain transducers. Transducers were attached to the bottom flanges of the girders at the midspan and 1 ft from the supports (parallel to longitudinal axis of girders). Strain transducers were attached using timber steel screws.

<u>Bridge test</u>

Based on the prior analysis, rating truck Type 3 was selected as a test load. Before the test, truck was loaded and actual axial load was measured on the police load station. Front axle was loaded to 16.04 kips and two rear axles were loaded to 15.66 kips each, with total load of 47.36 kips. Spacing between axles was 12.9 ft, 4 ft and 4.9 ft.






Because bridge was very narrow (16 ft), bridge test was performed by running the test truck in the center lane. The truck was driving with a crawling speed about 4-5 mph. Two runs at the central position were recorded to confirm readings. Strains measured by transducers were recorded using Bridge Diagnostics computer software. The last two runs were with truck traveling with higher speed 20-22 mph also on the center of slab. Readings under higher speed allowed to estimate dynamic factor for this bridge.

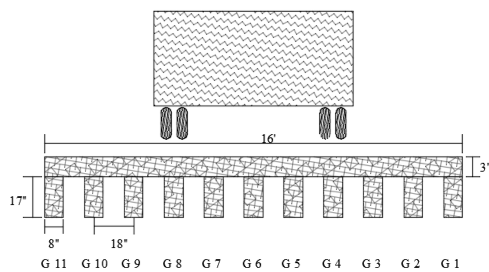



Table 11 presents stresses (based on measured strains) resulted from test truck running with crawling speed. Dead load stresses obtained from FE model are also included in the table.

Table 11. Live load and dead load stresses

| | Live Load Stress (Ksi) | | | DL Stress (Ksi) |
|---|---|---|---|---|
| Girder # | Model Stress | | | Model Stress |
| 1 | 0.1689 | | | 0.1276 |
| 2 | 0.2802 | | | 0.1305 |
| 3 | 0.4295 | | | 0.1331 |
| 4 | 0.5861 | | | 0.1349 |
| 5 | 0.4841 | | | 0.1359 |
| 6 | 0.4683 | | | 0.1362 |
| 7 | 0.5261 | | | 0.1359 |
| 8 | 0.5435 | | | 0.1349 |
| 9 | 0.3416 | | | 0.1331 |
| 10 | 0.2183 | | | 0.1305 |
| 11 | 0.11 | | | 0.1278 |

Measured live load strains allowed for computation of girder distribution factors, presented in Table 12.

Table 12. Girder Distribution Factors

| | Field Strain | GDF |
|---|---|---|
| 1 | 39.066 | 0.020725251 |
| 2 | 90.94 | 0.048245388 |
| 3 | 186.82 | 0.09911154 |
| 4 | 274.78 | 0.145775982 |
| 5 | 240.43 | 0.127552658 |
| 6 | 267.32 | 0.141818311 |
| 7 | 173.08 | 0.09182221 |
| 8 | 271.35 | 0.143956302 |
| 9 | 154.79 | 0.08211902 |
| 10 | 134.76 | 0.071492726 |
| 11 | 51.611 | 0.027380611 |
| Tot | 1884.947 | |

Values of measured strains in midspan and close to supports (one ft) compared to results from model with assumption of simply supported or fixed span, indicated some restrains on supports (partial fixity), Fig. 22.

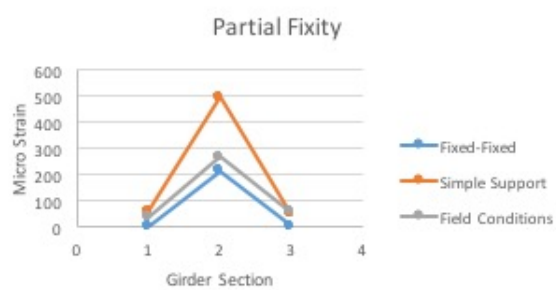
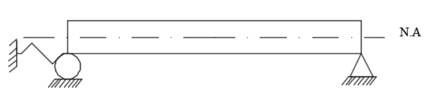



Fig. 22. Comparison of strains along the length of girder

Similarly, as described in the first bridge case, FE model was calibrated to adjust supporting conditions. Gauges close to supports (1 ft) show small negative strains indicating the presence of restraining force at the support.

| | Girder # 4 | Girder #8 | Girder # 6 |
|---|---|---|---|
| West Support | -20.114 | -0.51746 | -2.3848 |
| Mid Span | 274.78 | 271.35 | 267.32 |
| East Support | -22.679 | -10.552 | -6.9341 |

FE model was adjusted to account for the restraining force by adding the translation spring, which stiffness value was obtained by manual tuning.



The FE model values were very close to field measured values after accounting for the restraining force, shown in Table 13.

Table 13. Field test data and FEM after calibration

| Girder # | Model Stress | Model Strain | Field Strain |
|---|---|---|---|
| 1 | 0.1564 | 80.20512821 | 39.066 |
| 2 | 0.2671 | 136.974359 | 90.94 |
| 3 | 0.4157 | 213.1794872 | 186.82 |
| 4 | 0.5718 | 293.2307692 | 274.78 |
| 5 | 0.4695 | 240.7692308 | 240.43 |
| 6 | 0.4539 | 232.7692308 | 267.32 |
| 7 | 0.5121 | 262.6153846 | 173.08 |
| 8 | 0.5302 | 271.8974359 | 271.35 |
| 9 | 0.3293 | 168.8717949 | 154.79 |
| 10 | 0.2071 | 106.2051282 | 134.76 |
| 11 | 0.1027 | 52.66666667 | 51.611 |

Results of calibration by comparison of modeled and measured strains distribution to girders is also shown in Fig. 23.


Fig. 23. Comparison of modeled and measured strains

Field test data were analyzed using matlab using data recorded as strain history for each one of the strain gauges. With each run, a new strain history was recorded. Strain histories were plotted against position of the front wheel or time. Results of bridge calibration are also presented in time history of recorded and modeled strains for selected , most loaded girders, Fig. 24.

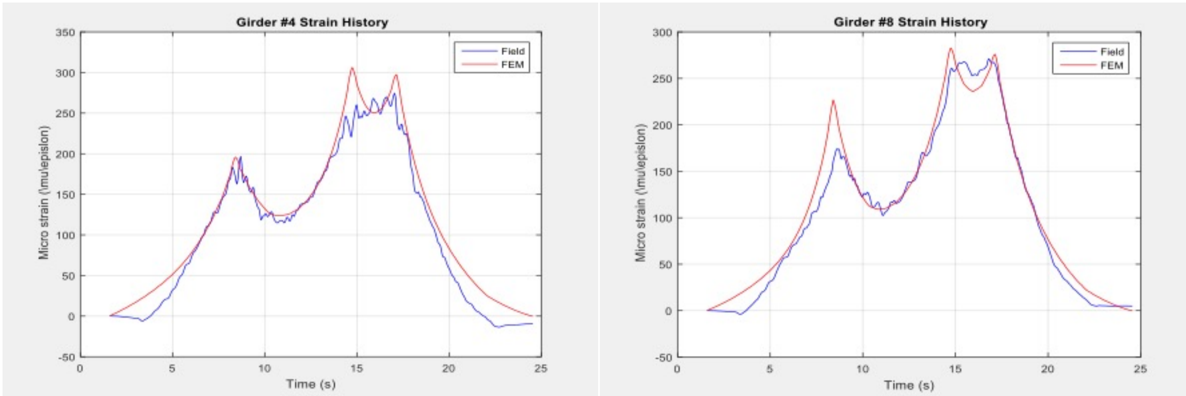



Fig. 24. Time strain history for selected girders

Measured strains under load truck traveling with crawling and high speed allowed for estimation of dynamic factor for this bridge = 0.08, Fig 25.

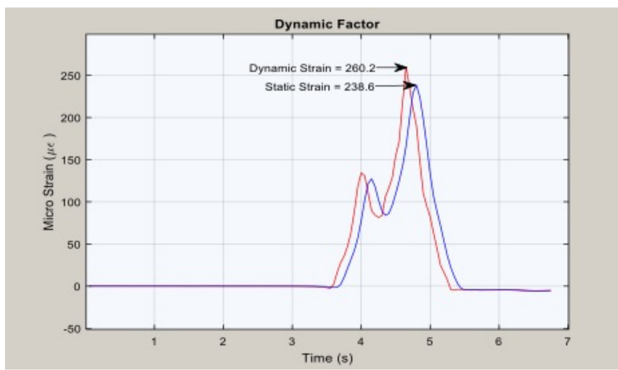



Fig. 25. Static and dynamic strains measured during the test

Bridge rating based on test results

Following parameters were adopted for bridge rating:

Timber girders were 17.5 in x 7.5 in with spacing 18.6 in on axis.
Timber – assumed Douglas Fir
$F_b$ = 1,450 psi (No 2 Grade from AASHTO Standard Spec.)
= 800 psi (No 3 Grade)
I = 3349.6 $in^4$
y = 8.75 in
Nominal moment capacity, $M_n$ = (1450 x 3349.6)/8.75 = 555 in-k
max LL stress on girder #8 = 0.5435 ksi
Live load moment, $M_{LL}$ = (0.5435 x 3349.6)/8.75 = 208.06 in-k
max DL stress on girder #8 = 0.1349 ksi (from calibrated model)
Dead load moment, $M_{DL}$ = (0.1349 x 3349.6)/8.75 = 51.64 in-k

Allowable stress rating (ASR) was used for the timber bridge. Rupture modulus for No 3 Grade lumber was used for rating due to rather low quality of timber.

$$RF = \frac{Fa + Fb}{Fl}$$

$F_a$ = allowable stress
$F_b$ = dead load stress
$F_l$ = live load stress

Inventory rating:

$$RF = \frac{800 + 134.9}{543.5} = 1.7$$

Operating, allowable stress was increased 33% (according to AASHTO Standard Spec.):

$$RF = \frac{800 \times 1.33 + 134.9}{543.5} = 2.2$$

## <u>Bridge 3 – pony truss bridge, Thayer County, NE (ID #S005 00446)</u>

The third analyzed and tested was three span bridge located on No 5 Road, North of Deshler over Little Blue River. The first and the third spans consisted of concrete decks supported on five steel girders, and the middle span consisted of two steel pony trusses supporting floor steel beams and open grate steel grid deck. Spans 1 and 3 were 62 ft multiple steel girder approach spans. The middle span was a 100 ft pony steel truss. Deck in all three spans was 29.5 ft wide. Spans were simply supported. The bridge was built in 1976 using elements from several salvaged bridges. Bridge was posted for Type 3 – 20 T, Type 3S2 – 32 T and Type 3-3 – 42 T. View of the bridge can be seen in photos below.


The bridge was visited by the research team and inspection revealed some corrosion deterioration and deformations on parts of truss. Also steel girders supporting the other two spans have perforation holes in webs. The understanding is that these parts of bridge (trusses and girders and also floor beams) were originally used in an older bridge and were used in construction of currently investigated bridge. Fig. 26 and 27 show photos of deformations of elements of truss and examples of corrosion. Strength of concrete in decks in the first and the third spans was evaluated using rebound hammer as $f_c$' = 3700 psi with modulus of elasticity, $E_c$ =57000($f_c$')$^{0.5}$. Yielding strength of steel was assumed $f_y$ = 33 ksi and modulus of elasticity, $E_s$ = 29000 ksi.

Fig. 26. Corrosion on truss elements

Fig. 27. Deformations of truss element

Based on NDOR Fracture Critical Inspection Report form June 2013 and our observations, it was understood that middle truss span is of a concern in this bridge because of the fracture critical members in truss and floor beams. Floor beams were cut and extended in a span length by welding new part to fit actual width of the bridge.

Preliminary analysis

The first part of preliminary analysis was analytical evaluation of bridge girders, floor beams and truss elements followed by the second part with finite element modeling.

The girders in spans 1 and 3 were W33x221 with $I_x$ = 12,900 $in^4$ and S = 759 $in^3$. The preliminary evaluation of stresses was performed for HS20 truck and the self weight with total stress for mostly loaded girder about 12 ksi.

Floor beams in the second span, supporting 7.5 in thick grid, were W30x108 with $I_x$ = 4,470 $in^4$ and S = 299 $in^3$. Analytical evaluation resulted in stress about 5.2 ksi also under HS20 truck.

Elements of the truss differed in cross section. Top chords consisted of two channels 10 in x 20 in and 3/8 in lacing on the top and bottom of cross section with total cross section of 18.135 $in^2$. Maximum force in compression was estimated as 62.2 kips. Bottom chords were also built with two channels 10 in x 20 in or 10 in x 35 in, depending on location and battens on top and bottom of cross section. Maximum force in tension was estimated as 51.2 kips. Vertical and diagonal elements were built with W10x45 and maximum vertical force was evaluated about 1.4 kips and diagonal force 22.9 kips. It was understood that complicated structure especially of the second span with pony truss did not allow for precise estimation of the internal forces. Forces obtained from FE model were far more reliable.

Three spans of the bridge were modeled using the same software as for the other bridges, CS Bridge. Existing drawings of the bridge help with modeling details. Model was used to find preliminary forces and stresses in elements of the bridge. Because of the longer spans (comparing to two previously presented bridges), analysis was performed for three legal rating trucks: Type 3, Type 3S2 and Type 3-3 to find out which load will produce the largest internal forces. Layout of the bridge, plan for the truss and finite element model are presented in Fig. 28, 29 and 30.

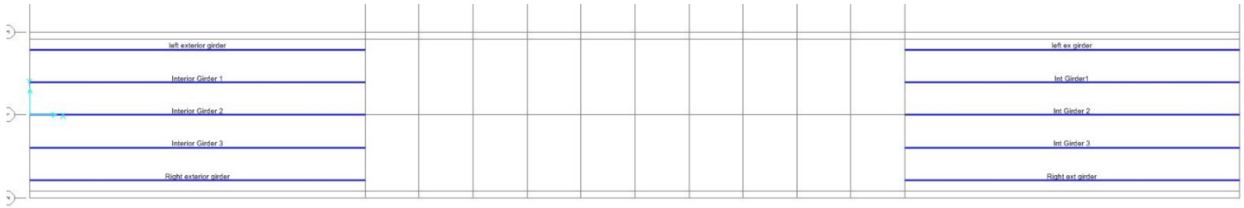



Fig. 28. Layout of the bridge

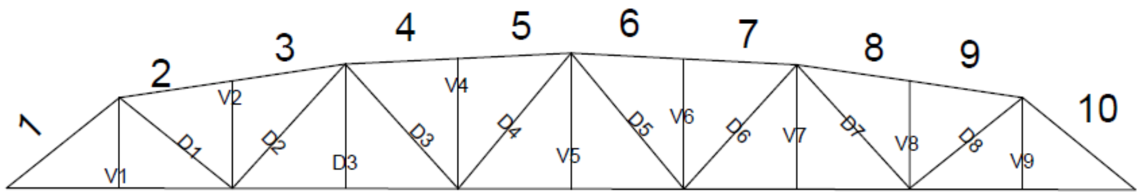



Fig. 29. Plan of the truss

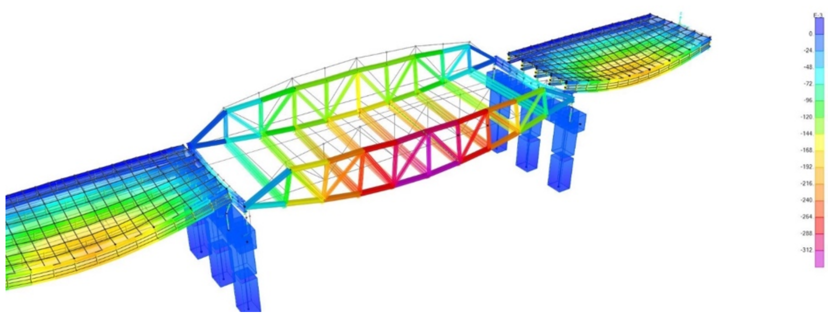


Fig. 30. Finite element model

Results of primary FE analysis will be presented in form of graphs and tables for three considered loading trucks.

<u>Internal forces under Type 3 truck</u>

Moments and shears in mostly loaded floor beams are presented in Fig. 31.

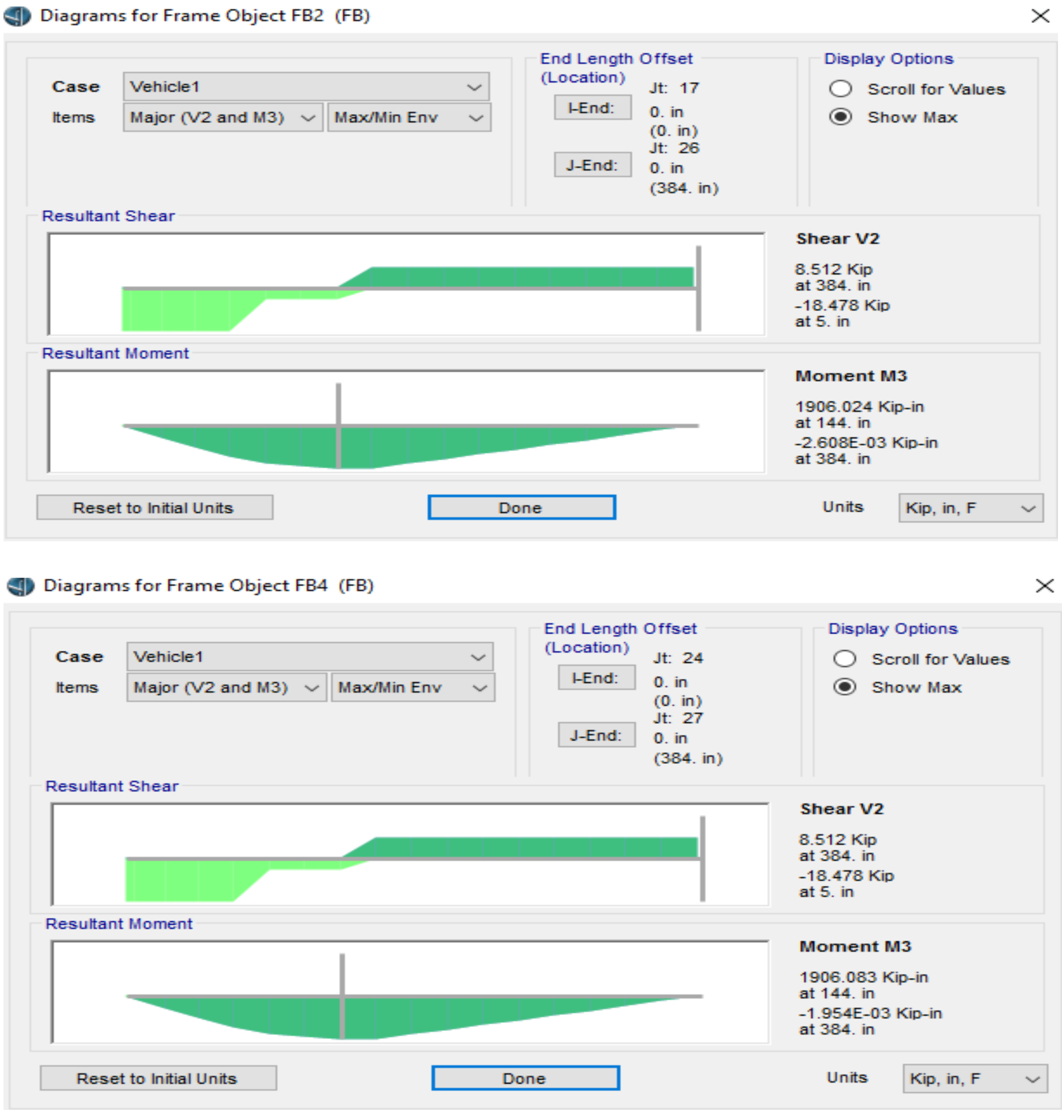



Fig. 31. Internal forces under Type 3 truck

Moments and shears in mostly loaded floor beams are presented in Fig. 31. Moments in mostly loaded girder in spans 1 or 3 are presented in Fig. 32.

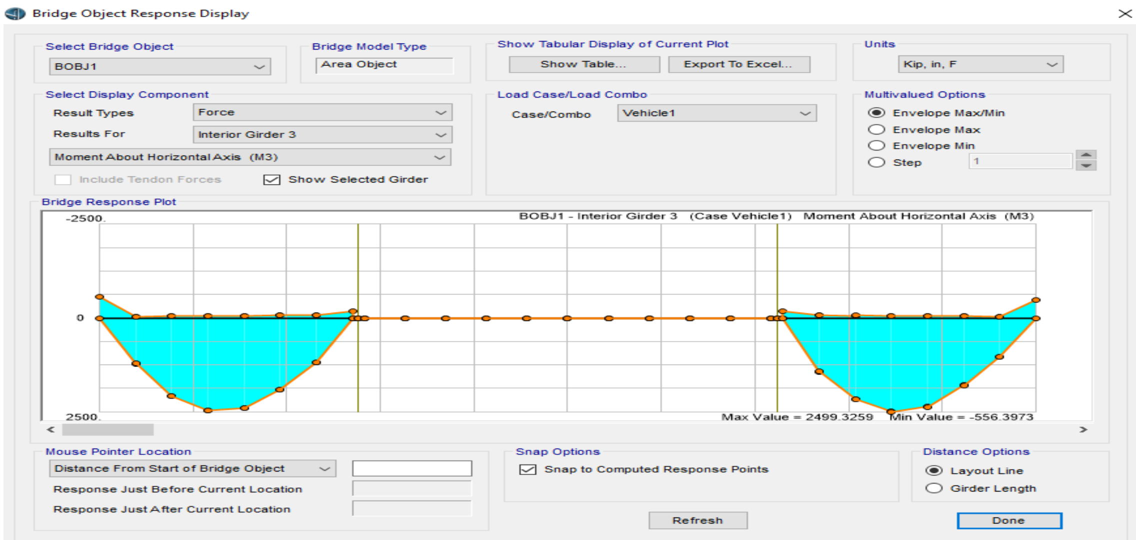



Fig. 32. Moments in girder 3 of span 1and 3

Live load and dead load moments in floor beams and forces in truss elements depending on the position of truck (lane 1 or 2) are shown in Table 14.

Table 14. Internal forces in floor beam and truss elements under Type 3 truck

| TYPE 3 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1st Lane | | | | | | 2nd Lane | | | |
| Floor Beam | Moment (Kip-in) | DL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) | LL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) |
| FB2 | 1906.024 @ 144 | 644.193 @ 192 | WT1 | 0 | -47.042 | 1906.011 @ 240 | ET1 | 0 | -42.056 |
| FB3 | 1906.062 @ 144 | 644.291 @ 192 | WT2 | 0.018 | -52.464 | 1906.039 @ 240 | ET2 | 0.019 | -47.573 |
| FB4 | 1906.083 @ 144 | 644.298 @ 192 | WT3 | 0.018 | -52.456 | 1906.089 @ 240 | ET3 | 0.019 | -47.565 |
| FB5 | 1906.044 @ 144 | 644.293 @ 192 | WT4 | 0.02 | -62.227 | 1906.047 | ET4 | 0.02 | -60.582 |
| FB6 | 1906.085 @144 | 644.291@ 192 | WT5 | 0.02 | -62.223 | 1906.086 | ET5 | 0.02 | -60.578 |
| FB7 | 1906.048 @ 144 | 644.293 @ 192 | WT6 | 0.02 | -60.578 | 1906.044 | ET6 | 0.02 | -62.223 |
| FB8 | 1906.088 | 644.298 @ 192 | WT7 | 0.02 | -60.582 | 1906.082 | ET7 | 0.02 | -62.226 |
| FB9 | 1906.039 | 644.291@ 192 | WT8 | 0.019 | -47.565 | 1906.062 | ET8 | 0.018 | -52.456 |
| FB10 | 1906.011 | 644.193 @ 192 | WT9 | 0.019 | -47.573 | 1906.024 | ET9 | 0.018 | -52.464 |
| | | | WT10 | 0 | -42.056 | | ET10 | 0 | -47.042 |
| | | | WD1 | 29.8 | -3.752 | | ED1 | 25.667 | -7.403 |
| | | | WD2 | 7.859 | -22.858 | | ED2 | 15.579 | -19.136 |
| | | | WD3 | 22.913 | -9.953 | | ED3 | 18.785 | -15.333 |
| | | | WD4 | 15.55 | -18.424 | | ED4 | 15.55 | -18.424 |
| | | | WD5 | 20.753 | -14.525 | | ED5 | 22.118 | -22.95 |
| | | | WD6 | 18.785 | -15.333 | | ED6 | 22.913 | -9.953 |
| | | | WD7 | 15.579 | -19.136 | | ED7 | 7.859 | -22.858 |
| | | | WD8 | 25.667 | -7.403 | | ED8 | 29.8 | -3.752 |
| | | | WV1 | 18.349 | -3.749 | | EV1 | 19.217 | -2.039 |
| | | | WV2 | 0.053 | -1.84E-05 | | EV2 | 0.048 | -1.98E-05 |
| | | | WV3 | 18.596 | -3.94 | | EV3 | 0.018 | -3.729 |
| | | | WV4 | 0.064 | -2.93E-05 | | EV4 | 0.062 | -2.11E-05 |
| | | | WV5 | 18.647 | -3.818 | | EV5 | 18.647 | -3.818 |
| | | | WV6 | 0.062 | -2.10E-05 | | EV6 | 0.064 | -2.10E-05 |
| | | | WV7 | 18.821 | -3.729 | | EV7 | 18.596 | -3.94 |
| | | | WV8 | 0.048 | -1.98E-05 | | EV8 | 0.053 | -1.84E-05 |
| | | | WV9 | 19.217 | -2.039 | | EV9 | 18.349 | -3.749 |
| | | | WBC | 51.249 @ 698 | -1.84E+00 | | EBC | 51.249 @ 582.003 | -1.84E+00 |

<u>Internal forces under Type 3S3 truck</u>

Moments and shears in mostly loaded floor beams are presented in Fig. 33.

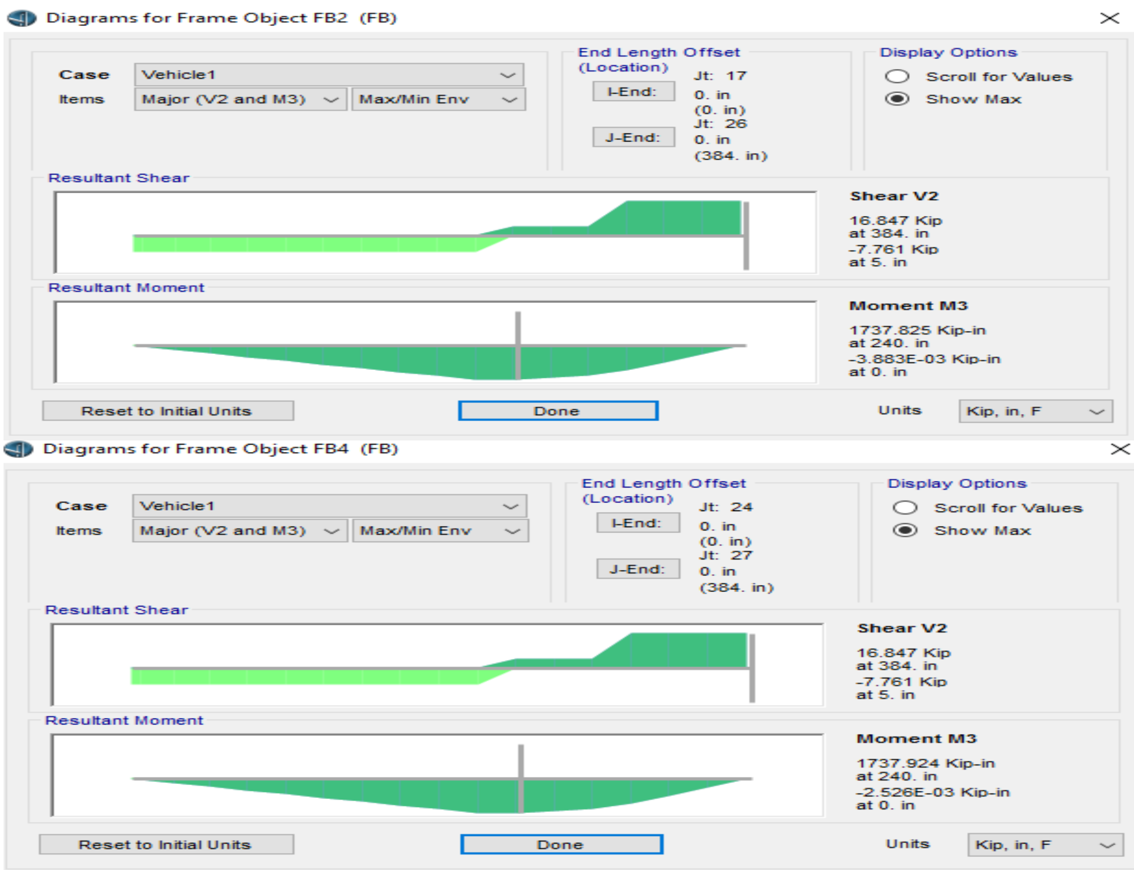



Fig. 33. Moments and shears in selected floor beams

Example of interior girder moments in spans 1 and 3 is presented in Fig. 34.

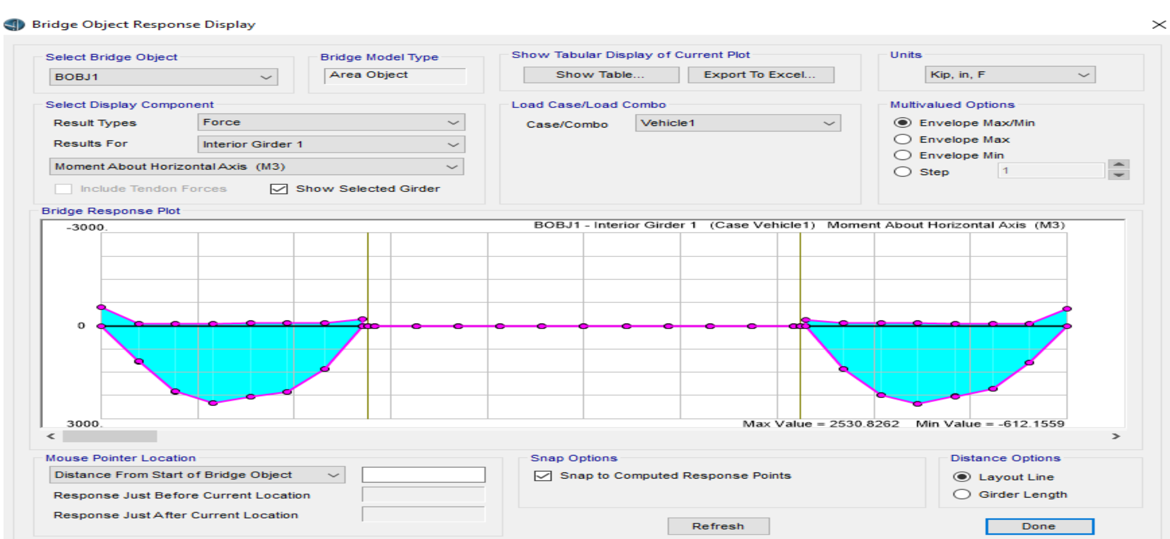



Fig. 34. Moments on interior girder in spans 1 and 3

Live load and dead load moments in floor beams and forces in truss elements depending on the position of truck (lane 1st or 2nd) are shown in Table 15.

Table 15. Internal forces in floor beam and truss elements under Type 3S2 truck

| | | | | | TYPE 3S2 | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1st Lane | | | | | | | 2nd Land | | | | |
| Floor Beam | Moment (Kip-in) | DL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) | | LL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) | |
| FB2 | 1737.884 @ 144 | 644.193 @ 192 | WT1 | 0 | -57.018 | | 1737.825 @ 240 | ET1 | 0 | -52.467 | |
| FB3 | 1737.896 @ 144 | 644.291 @ 192 | WT2 | 0.02 | -63.458 | | 1737.903 @ 240 | ET2 | 0.039 | -63.564 | |
| FB4 | 1737.919 @ 144 | 644.298 @ 192 | WT3 | 0.02 | -63.448 | | 1737.924 @ 240 | ET3 | 0.039 | -63.554 | |
| FB5 | 1737.908 @ 144 | 644.293 @ 192 | WT4 | 0.023 | -72.681 | | 1737.909 | ET4 | 0.023 | -75.431 | |
| FB6 | 1737.921 @144 | 644.291@ 192 | WT5 | 0.023 | -72.677 | | 1737.922 | ET5 | 0.023 | -75.427 | |
| FB7 | 1737.91 @ 144 | 644.293 @ 192 | WT6 | 0.023 | -75.427 | | 1737.907 | ET6 | 0.023 | -72.677 | |
| FB8 | 1737.923 | 644.298 @ 192 | WT7 | 0.023 | -75.431 | | 1737.919 | ET7 | 0.023 | -72.681 | |
| FB9 | 1737.903 | 644.291@ 192 | WT8 | 0.039 | -63.554 | | 1737.896 | ET8 | 0.02 | -63.448 | |
| FB10 | 1737.825 | 644.193 @ 192 | WT9 | 0.039 | -63.564 | | 1737.884 | ET9 | 0.02 | -63.458 | |
| | | | WT10 | 0 | -52.467 | | | ET10 | 0 | -57.018 | |
| | | | WD1 | 34.653 | -2.357 | | | ED1 | 31.058 | -6.814 | |
| | | | WD2 | 11.47 | -25.471 | | | ED2 | 14.375 | -21.456 | |
| | | | WD3 | 24.666 | -9.927 | | | ED3 | 21.462 | -12.772 | |
| | | | WD4 | 16.414 | -18.243 | | | ED4 | 18.92 | -15.099 | |
| | | | WD5 | 18.92 | -15.099 | | | ED5 | 16.414 | -18.243 | |
| | | | WD6 | 21.462 | -12.772 | | | ED6 | 24.666 | -9.927 | |
| | | | WD7 | 14.375 | -21.456 | | | ED7 | 11.47 | -25.471 | |
| | | | WD8 | 31.058 | -6.814 | | | ED8 | 34.653 | -2.356 | |
| | | | WV1 | 18.092 | -3.139 | | | EV1 | 18.027 | -2.864 | |
| | | | WV2 | 0.0665 | -2.06E-05 | | | EV2 | 0.065 | -4.01E-05 | |
| | | | WV3 | 18.898 | -5.484 | | | EV3 | 18.682 | -5.586 | |
| | | | WV4 | 0.075 | -2.34E-05 | | | EV4 | 0.078 | -2.34E-05 | |
| | | | WV5 | 18.85 | -5.503 | | | EV5 | 18.85 | -5.503 | |
| | | | WV6 | 0.078 | -2.34E-05 | | | EV6 | 0.075 | -2.34E-05 | |
| | | | WV7 | 18.682 | -5.586 | | | EV7 | 18.989 | -5.484 | |
| | | | WV8 | 0.065 | -4.00E-05 | | | EV8 | 0.065 | -5.21E-04 | |
| | | | WV9 | 18.027 | -2.864 | | | EV9 | 18.092 | -3.139 | |
| | | | WBC | 60.056 @ 582.003 | -2.76E+00 | | | EBC | 60.056 @ 398 | -2.76E+00 | |

<u>Internal forces under Type 3-3 truck</u>

Since graphs of internal forces produced by Type 3-3 truck are similar in shape to presented above (for different trucks), the results of analysis are only presented in Table 16.

Table 16. Internal forces in floor beam and truss elements under Type 3-3 truck

| | | | | | | | | | | | | TYPE 3-3 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1st Lane | | | | | | | 2nd Land | | | | |
| | Floor Beam | Moment (Kip-in) | DL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) | | LL Moment (Kip-in) | Truss Member | Tension Force (Kip) | Compression (Kip) | |
| | FB2 | 1569.658 @ 144 | 644.193 @ 192 | WT1 | 0 | -58.033 | | 15.69.656 @ 240 | ET1 | 0 | -51.921 | |
| | FB3 | 1569.697 @ 144 | 644.291 @ 192 | WT2 | 0.02 | -64.193 | | 1569.671 | ET2 | 0.029 | -59.544 | |
| | FB4 | 1569.709 @ 144 | 644.298 @ 192 | WT3 | 0.02 | -64.186 | | 1569.714 | ET3 | 0.029 | -59.534 | |
| | FB5 | 1569.678 @ 144 | 644.293 @ 192 | WT4 | 0.023 | -73.257 | | 1569.678 | ET4 | 0.023 | -74.012 | |
| | FB6 | 1569.711 @144 | 644.291@ 192 | WT5 | 0.023 | -73.253 | | 1569.712 | ET5 | 0.023 | -74.008 | |
| | FB7 | 1569.678 @ 144 | 644.293 @ 192 | WT6 | 0.023 | -74.008 | | 1569.678 | ET6 | 0.023 | -73.253 | |
| | FB8 | 1569.714 | 644.298 @ 192 | WT7 | 0.023 | -74.012 | | 1569.709 | ET7 | 0.023 | -73.257 | |
| | FB9 | 1569.671 | 644.291@ 192 | WT8 | 0.029 | -59.534 | | 1569.697 | ET8 | 0.02 | -64.186 | |
| | FB10 | 1569.656 | 644.193 @ 192 | WT9 | 0.029 | -59.544 | | 1569.658 | ET9 | 0.02 | -64.196 | |
| | | | | WT10 | 0 | -51.921 | | | ET10 | 0 | -58.033 | |
| | | | | WD1 | 34.542 | -2.835 | | | ED1 | 28.339 | -6.115 | |
| | | | | WD2 | 5.886 | -25.019 | | | ED2 | 12.824 | -19.281 | |
| | | | | WD3 | 23.201 | -7.34 | | | ED3 | 16.51 | -12.593 | |
| | | | | WD4 | 11.423 | -17.254 | | | ED4 | 17.616 | -11.916 | |
| | | | | WD5 | 17.616 | -11.916 | | | ED5 | 11.426 | -17.254 | |
| | | | | WD6 | 16.51 | -12.593 | | | ED6 | 23.201 | -7.34 | |
| | | | | WD7 | 12.824 | -19.281 | | | ED7 | 5.886 | -25.019 | |
| | | | | WD8 | 28.339 | -6.115 | | | ED8 | 34.542 | -2.834 | |
| | | | | WV1 | 15.399 | -2.957 | | | EV1 | 15.84 | -1.383 | |
| | | | | WV2 | 0.065 | -2.09E-05 | | | EV2 | 15.831 | -3.06E+00 | |
| | | | | WV3 | 15.629 | -3.185 | | | EV3 | 15.831 | -3.06 | |
| | | | | WV4 | 0.075 | -2.38E-05 | | | EV4 | 0.076 | -2.36E-05 | |
| | | | | WV5 | 15.645 | -3.149 | | | EV5 | 15.645 | -3.149 | |
| | | | | WV6 | 0.076 | -2.37E-05 | | | EV6 | 0.075 | -2.37E-05 | |
| | | | | WV7 | 15.831 | -3.06 | | | EV7 | 15.629 | -3.185 | |
| | | | | WV8 | 0.061 | -2.94E-05 | | | EV8 | 0.065 | -2.08E-05 | |
| | | | | WV9 | 15.84 | -1.383 | | | EV9 | 15.399 | -2.957 | |
| | | | | WBC | 58.935 @ 698 | -2.82E+00 | | | EBC | 58.935 @ 582 | -2.83E+00 | |

Looking at the results of FE analysis, Tables 14, 15 and 16, it could be concluded that truck Type 3 produced the largest internal forces in the bridge. Stresses in the floor beams, the girders and truss elements were checked and it was concluded that Type 3 load would be safe as testing load. Internal stresses in bridge elements resulting from preliminary analysis:

Floor beams (max moments)
$M_L = 1906$ *Kip-in*
$M_D = 644.29$ *Kip-in*
$M_T = 2550.29$ *Kip-in*
$\sigma = 8.7$ *ksi*
Steel girders (max moment)
$M_L = 2499.3$ *Kip-in*
$M_D = 4745.08$ *Kip –in*
$M_T = 7244.38$ *Kip-in*
$\sigma = 10.3$ *ksi*
Truss elements (max forces)
$T = 25.667$ *Kips* (in tension)
$\sigma = 3$ *ksi*
$C = 62.227$ *Kips* (in compression)
$\sigma = 2.8$ *ksi*
$F_y = 33$ *ksi*

Bridge instrumentation

Presented truss bridge was 30 feet high and a ladder for instrumentation was not an option. To reach girders and floor beams under the bridge, the snooper truck was used as presented in Fig. 4 and 5. The bridge was instrumented with wireless transducers the same as the other bridges presented above. The transducers were attached by steel tabs glued to girders and floor beams. Typically, on simply supported girders of spans 1 and 3, transducers should be located at the midspan, quarter span and 1 ft close to supports on the bottom flanges and close to supports on the upper flanges. However, the was no access to locations on quarter spans and close to the external supports on both 1 and 3 spans; on other locations instruments were attached. Prior to attachment, flanges need to be cleaned of paint and rust (using electric hand grinder) to insure the full grip of transducers. The selection of floor beams for instrumentation was performed based on initial FE analysis which revealed the mostly loaded beams for truck traveling on 1st, 2nd spans or at the center of bridge. These beams were the 2nd, 5th, 7th and the 9th out of 11 floor beams. Selected floor beams were instrumented following the same standard procedure using steel tabs and glue for attachment. Snooper truck arm was long enough to reach the midspan of beams so transducers could be attached there to measure a live load strains/moments. They were also attached 1 ft from the supports. Selection of the truss elements for instrumentation was also done based on preliminary FE model results. The mostly loaded elements (upper and bottom chords, diagonals and vertical components) were located and prepared for instrumentation. The plan for instrumentation of two trusses is shown in Fig. 35. There was found a symmetry in the location of maximum forces in both trusses depending on the direction of passing truck.
After the test, all grinded locations where transducers were attached were painted to protect steel elements against corrosion.

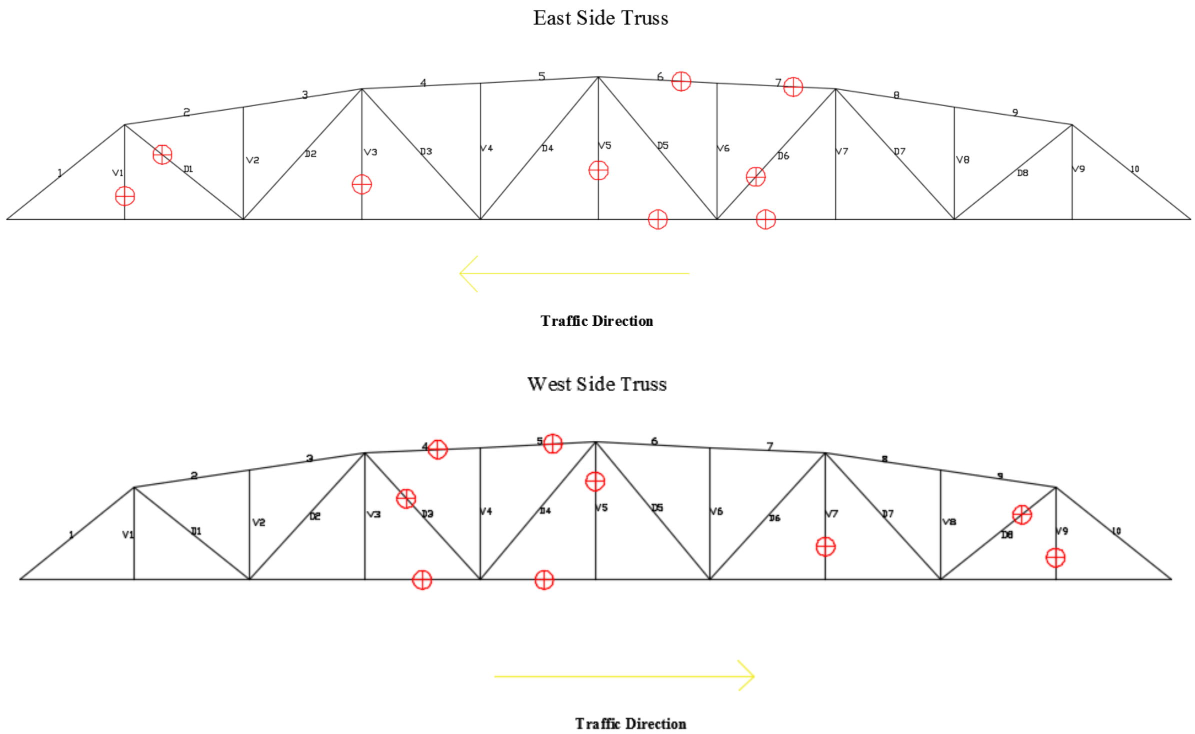



Fig. 35. Plan for transducers location on both trusses.

Bridge test

The test was performed using Type 3 loading, truck which was found based on preliminary FE analysis to produce the largest internal forces in steel girders, floor beams and truss elements. The testing truck was loaded to maximum load of 50 kips prior to test and axial loads were measured on the police load station. Also geometry of the truck was measured.

The test was conducted running the truck on lanes 1 and 2, and center of the bridge, with very slow crawling speed about 1 mph. Each run was repeated two or three times to compare consistency of readings. The last two runs were at the higher speed of the truck traveling at the center of the bridge. Because of the category of the road, the higher speed was 40 and 50 mph. Results of all runs were recorded using computer software as usually.

Test results

Recorded test results were analyzed to find the maximum moments and forces in bridge components resulting from testing truck load. The results are presented in tables and in form of graphs. Based on measured strain values, FE model of the bridge was calibrated using principle that measured values of strains and resulting from real test truck traveling on FE model are as closed as possible. It is an important part of post-test analysis since dead load moment and forced can be obtained from the calibrated bridge model, which is considered more precise than analytical hand calculations only.

Tables below present measured live load internal forces in truss elements in the second span, and moments and shears for floor beams and girders in the third span. Since the results for the first and the third spans were very similar (spans have the same geometry, number of girders and span length), one of spans was selected for analysis. Table 17 presents axial measured forces in truss elements for both trusses denoted as in Fig.35, moments and shears for selected floor beams (mostly loaded).

Table 17. Live load internal forces in trusses and floor beams

| Second Span (Live Load) | | | | | |
|---|---|---|---|---|---|
| Structural Member | Axial Force (k) | | Axial Force (k) | Moment (k-in) | Shear (k) |
| Bottom Chord | 37.436 @ 600 | | | | |
| Top Chord | -38.208 | | | | |
| West V1 | 10.742 | East V1 | 15.226 | | |
| West V2 | 0 | East V2 | 0 | | |
| West V3 | 10.087 | East V3 | 14.456 | | |
| West V4 | 0 | East V4 | 0 | | |
| West V5 | 10.143 | East V5 | 14.505 | | |
| West V6 | 0 | East V6 | 0 | | |
| West V7 | 10.191 | East V7 | 14.498 | | |
| West V8 | 0 | East V8 | 0 | | |
| West V9 | 10.576 | East V8 | 15.009 | | |
| West D1 | 15.623 | East D1 | 22.021 | | |
| West D2 | -10.878 | East D2 | -15.667 | | |
| West D3 | 11.098 | East D3 | 16.181 | | |
| West D4 | 9.62 | East D4 | 12.423 | | |
| West D5 | 9.116 | East D5 | 14.155 | | |
| West D6 | 12.022 | East D6 | 14.878 | | |
| West D7 | -11.833 | East D7 | -14.706 | | |
| West D8 | 16.211 | East D8 | 20.707 | | |
| FB2 | | | | 429.344 | 15.903 |
| FB5 | | | | 430.536 | 15.948 |
| FB7 | | | | 387.774 | 15.911 |
| FB9 | | | | 420.868 | 15.589 |

Table 18 presents dead load internal forces in trusses elements, and floor beams estimated based on calibrated FE model.

Table 18. Dead load internal forces in trusses and floor beams

| Second Span (Live Load) | | | | | |
|---|---|---|---|---|---|
| Structural Member | Axial Force (k) | | Axial Force (k) | Moment (k-in) | Shear (k) |
| Bottom Chord | 190.587 | | 203.679 | | |
| Top Chord | -231.664 | | -238.511 | | |
| West V1 | 21.42 | East V1 | 21.462 | | |
| West V2 | -1.049 | East V2 | -1.049 | | |
| West V3 | 21.098 | East V3 | 21.189 | | |

| | | | | | |
|---|---|---|---|---|---|
| West V4 | -1.1 | East V4 | -1.1 | | |
| West V5 | 21.544 | East V5 | 21.564 | | |
| West V6 | -1.1 | East V6 | -1.1 | | |
| West V7 | 21.928 | East V7 | 21.424 | | |
| West V8 | -1.049 | East V8 | -1.049 | | |
| West V9 | 22.168 | East V8 | 21.264 | | |
| West D1 | 76.064 | East D1 | 81.528 | | |
| West D2 | -34.199 | East D2 | -38.948 | | |
| West D3 | 25.753 | East D3 | 31.337 | | |
| West D4 | 6.446 | East D4 | 0.904 | | |
| West D5 | -7.939 | East D5 | 0.79 | | |
| West D6 | 39.414 | East D6 | 31.306 | | |
| West D7 | -46.712 | East D7 | -39.286 | | |
| West D8 | 88.783 | East D8 | 81.541 | | |
| FB2 | | | | 1175.138 | 20.593 |
| FB5 | | | | 1133.61 | 21.11 |
| FB7 | | | | 1132.983 | 20.639 |
| FB9 | | | | 1142.126 | 19.794 |

Table 19 presents measured strains in girders of the first span, strains values obtained from calibrated model, load distribution factor and estimated error.

Table 19. Strains in the first span girders

| | Center lane | | | |
|---|---|---|---|---|
| | FEM Strain | Field Strain | DF | Error |
| G1 | 18.08 | 18.25 | 0.14 | 0.001 |
| G2 | 37.52 | 31.44 | 0.25 | 0.047 |
| G3 | 49.90 | 43.71 | 0.34 | 0.048 |
| G4 | 31.10 | 15.97 | 0.12 | 0.118 |
| G4 | 14.31 | 18.66 | 0.15 | 0.034 |
| Total | 150.91 | 128.04 | | 0.249 |

| | First Lane | |
|---|---|---|
| | Field Strain | DF |
| G1 | 63.99 | 0.42 |
| G2 | 66.87 | 0.43 |
| G3 | 17.15 | 0.11 |
| G4 | 4.57 | 0.03 |
| G5 | 1.53 | 0.01 |
| Total | 154.10 | |

Table 20 presents moments and shears in the first span girders caused by dead load. Dead load values were obtained from calibrated FE model of the bridge.

Table 20. Dead load moments and shears on the first span

| | First Span Span (Dead Load) | |
|---|---|---|
| **Structural Member** | **Moment (K-in)** | **Shear (K)** |
| **G1** | 4391.148 | 31.295 |
| **G2** | 4366.537 | 18.672 |
| **G3** | 4783.423 | 20.327 |
| **G4** | 4364.353 | 18.672 |
| **G5** | 4385.043 | 31.295 |

Figures below represent the strain history measured and resulting from FE calibrated model for the top and bottom chords in truss and mostly loaded floor beam and girder. Values from the model were obtained by running the test truck on calibrated model. Fig. 36 shows strain history for one of the floor beams versus position of the front axle; there are measured strains and computed from calibrated FE model. The strain time history for bottom and top chords of the truss is presented in Fig. 37 and 38, respectively. Fig. 39 presents history of strains versus the position of the front axle for mostly loaded 3rd girder in the first span.

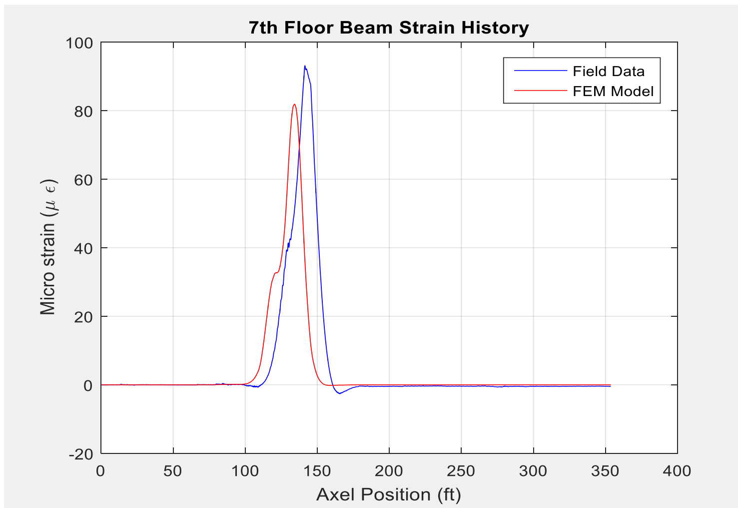



Fig. 36 Strain history for one of the floor beams

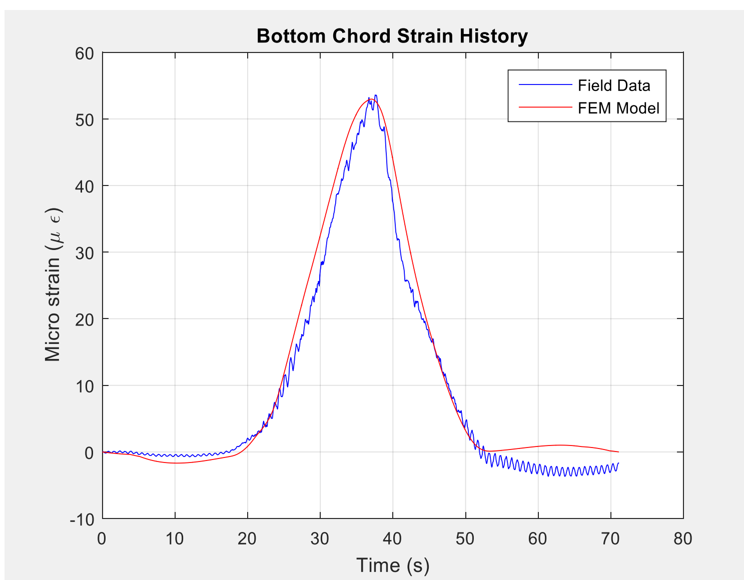



Fig. 37. Strain time history for the bottom chord of the truss

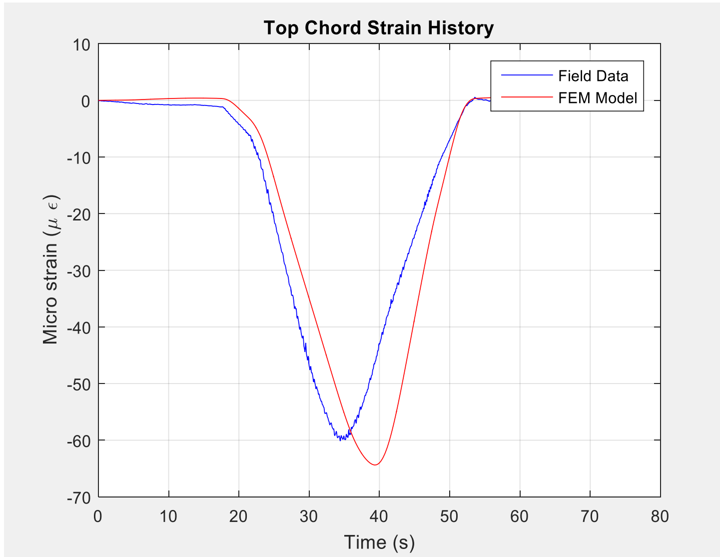



Fig. 38. Strain time history for the top chord of the truss

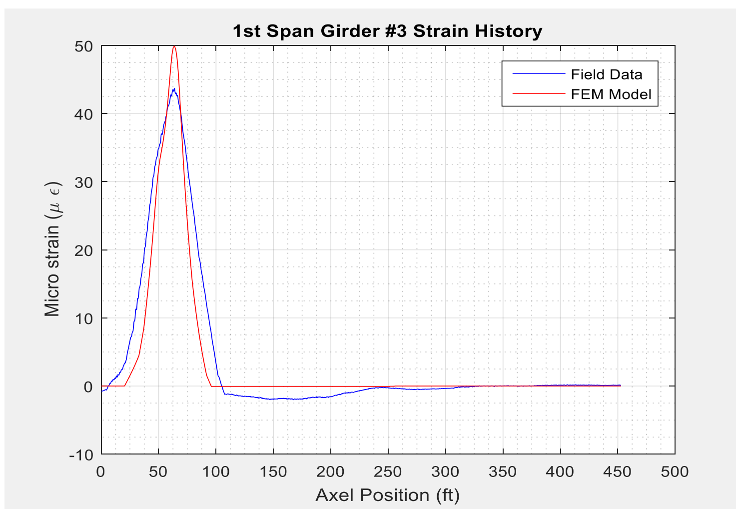



Fig. 39. Strain history for 3rd girder in the first span

Fig. 40 and 41 present comparisons of measured strains under crawling and high speed of the test truck in the mostly loaded girder and mostly loaded floor beam, respectively. For these measurements, the truck was traveling on the center lane. These values were used to estimate dynamic factor (IM) for the first and the third spans, and for the second span. Estimated value of the dynamic factor was, IM = 0.09 for girders (1st and 3rd spans), and IM = 0.12 for floor beams (2nd span). There was a difference in this factor between the outward and middle spans, which was expected due to different type of the structure and the span length.

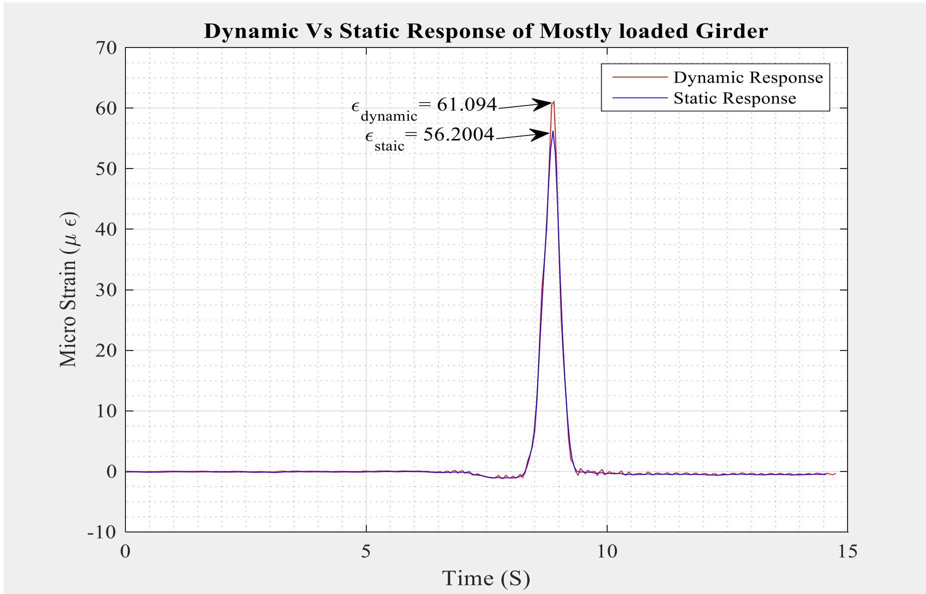



Fig. 40. Dynamic and static strains measured in mostly loaded girder

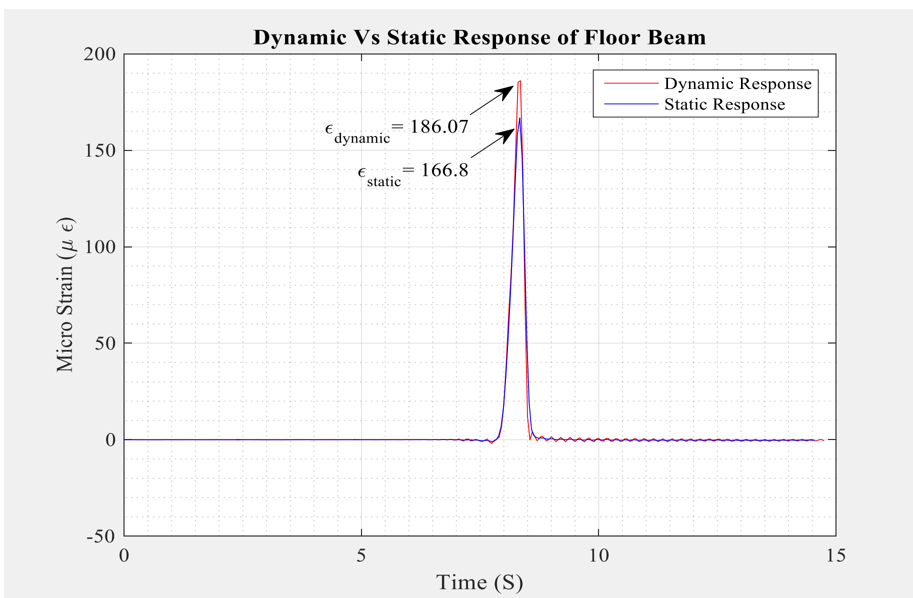



Fig. 41. Dynamic and static strains measured in mostly loaded floor beam

Bridge rating based on test results

Bridge rating was performed for steel girders in the external spans, and the floor beams and truss elements in the second span. The highest values of the live load moment or axial forces (converted from measured strains) were used in the computations. The dead load moments were obtained from calibrated FE model. The highest values of both moments were on girder 3 in the first span.

Rating for steel girders

For steel girders, W33x221, with section modulus S = 759 in$^3$, maximum measured live load moment was $M_L$ = 1409 k-in and deal load moment, $M_D$ = 4783.42 k-in.
Nominal moment capacity, $M_n$ = 25047 k-in. Dynamic factor, estimated as IM = 0.09 (Fig.40) was used in calculations.

Inventory rating

$$RF = \frac{(1.0)(25047) - 1.3(4783.42)}{2.17(1409)(1 + 0.09)} = 5.6$$

Operating rating

$$RF = \frac{(1.0)(25047) - 1.3(4783.42)}{1.30(1409)(1 + 0.09)} = 9.4$$

Rating for floor beams

For floor beams, W30x108 with section modulus S = 299 in$^3$, maximum live load moment (from converted measured strain) was $M_L$ = 1613.4 k-in, and dead load moment, $M_D$ = 1142.13 k-in. Nominal capacity of the floor beams was, $M_n$ = 9867 k-in. Dynamic factor measured for floor beams as presented in Fig. 41, was IM = 0.12.

Inventory rating:

$$RF = \frac{(1.0)(9867) - 1.3(1142.13)}{2.17(1613.4)(1 + 0.12)} = 2.14$$

Operating rating:

$$RF = \frac{(1.0)(9867) - 1.3(1142.13)}{1.30(1613.4)(1 + 0.12)} = 3.6$$

Rating for truss elements

Bottom chord:

Area of the top and bottom cord was 18.135 in$^2$, and its capacity estimated using $f_y$ = 33 ksi was 598.45 kips. Maximum live load force based on measured strain was 37.44 kips in tension. The dead load force in the bottom chord estimated from calibrated FE model was 203.68 kips in tension. The same dynamic factor IM = 0.12 was used for truss elements.

Inventory rating:

$$RF = \frac{1.0(598.45) - 1.3(203.68)}{2.17(37.44)(1 + 0.12)} = 3.6$$

Operating rating:

$$RF = \frac{1.0(598.45) - 1.3(203.68)}{1.30(37.44)(1+0.12)} = 6.1$$

Top chord:

Maximum live load force based on measured strain was 38.21 kips in compression. The dead load force in the top chord estimated from calibrated FE model was 238.51 kips also in compression.

Inventory rating:

$$RF = \frac{1.0(598.45) - 1.3(238.51)}{2.17(38.21)(1+0.12)} = 3.1$$

Operating rating:

$$RF = \frac{1.0(598.45) - 1.3(238.51)}{1.30(38.21)(1+0.12)} = 5.2$$

Diagonals:

Mostly loaded diagonal, D1, was loaded with tensile force $F_{LL}$ = 22.02 kips. Force resulting from the dead load was estimated from FE calibrated model, $F_{DL}$ = 81.53 kips. Using the cross-section area = 8.66 in$^2$, the capacity the axial force, $F_n$ = 285.78 kips.

Inventory rating:

$$RF = \frac{1.0(285.78) - 1.3(81.53)}{2.17(22.02)(1+0.12)} = 3.3$$

Operating rating:

$$RF = \frac{1.0(285.78) - 1.3(81.53)}{1.30(22.02)(1+0.12)} = 5.6$$

Mostly loaded in compression was diagonal D2 with compression force $F_{LL}$ = 15.67 kips and $F_{DL}$ = 38.95 kips.

Inventory rating:

$$RF = \frac{1.0(285.78) - 1.3(38.95)}{2.17(15.67)(1+0.12)} = 6.17$$

Operating rating:

$$RF = \frac{1.0(285.78) - 1.3(38.95)}{1.30(15.67)(1+0.12)} = 10.3$$

## Bridge 4 – concrete slab bridge (ID # C007821635), Saunders County, NE

One span simply supported concrete slab bridge carries a rural road over a small creek. View of the bridge is presented below.

Dimensions of the concrete slab were: width 28 ft, span length 31 ft, thickness of the slab 12 inches. Slab was constructed with 34-inch-wide precast reinforced concrete panels connected by shear keys filled with grouting. Panels were reinforced with seven bars #9 spaced at 4.5 in. (based on available drawing), Grade 60. During a site inspection, compressive strength of slab concrete was estimated using rebound hammer to be, $f_c$' = 4500 psi.

Preliminary analysis

As a part of preliminary analysis, analytical analysis was performed to estimate nominal moment capacity of the precast panel. Cross-section 34 in. wide and 12 in. deep made of concrete with compressive strength, $f_c$' = 4500 psi, and reinforced with 7 bars #9 with, $f_y$ = 60 ksi was considered. With effective depth of d = 10.44 in. and position of neutral axis a = 3.23 in., the nominal moment capacity was $M_n$ = 308.875 k-ft. Considering a linear distribution of strains in the cross-section, strain in tension steel was estimated as $\varepsilon_t = 0.005$, which allowed to use strength reduction factor, $\phi = 0.90$. Resulting total moment capacity, $\phi M_n$ = 278 k-ft.

Bridge was also modeled using CSI Bridge software as solid slab simply supported. Stiffness of the reinforcement was included by transferring layer of reinforcement into concrete by ratio of moduli of materials. A view of modeled bridge deflected under the dead load and the live load is shown below in Fig. 42 and 43, respectively.

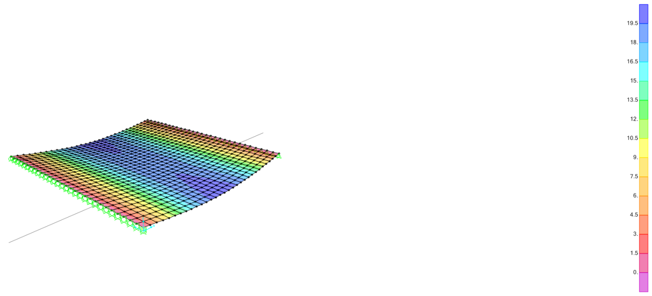


Fig. 42. View of deflected slab under the dead load

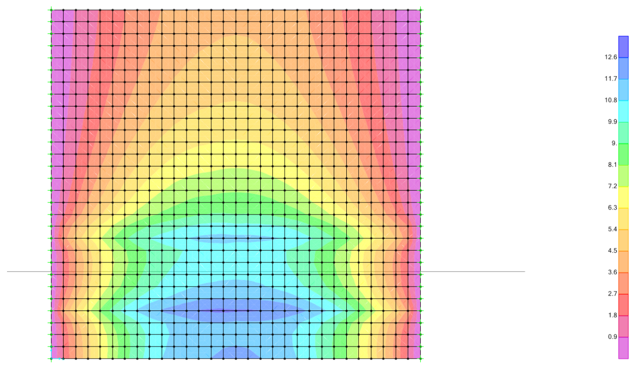


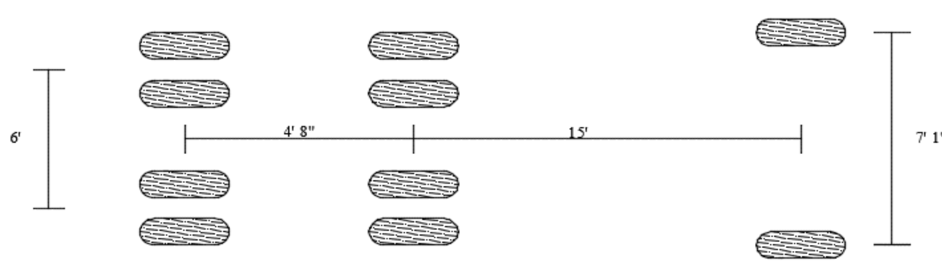
Fig. 43. View of strain distribution under the live load

Bridge instrumentation

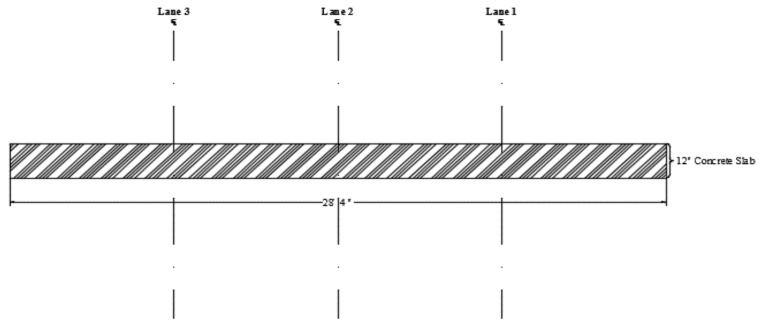
Bridge was instrumented with the wireless transducers attached parallel to the longitudinal axis of the slab at the midspan and 1 ft. from supports. Transversally, transducers were placed 2 ft from the edge of slab and spaced at 6 ft at the width of slab. Transducers were attached using the same glue as for other bridges.

Bridge test

The test was performed using legal rating truck Type 3 with geometry shown below. The axial load was 15.3 kips on the front axle and 17.54 kips on each of rear axles, with total load of 50.38 kips.




The loading truck was running on two lanes and at the center of slab as shown on a sketch below.




First runs of truck were on lanes 1 and 3 with low speed about 4-3 mph. Each run was repeated. Next, there were two runs also with low speed 3 mph on the center of slab, lane2. The last two runs were with a higher speed, 30 mph, performed to measure dynamic factor for the bridge. Strains from all runs were recorded using the computer software.

<u>Test results</u>

Longitudinal strains measured at the bottom of the slab were converted to live load moments. Example of slab response to the load is presented in Fig. 44. This figure also shows results from calibrated FE model, which are in good agreement with measured values.

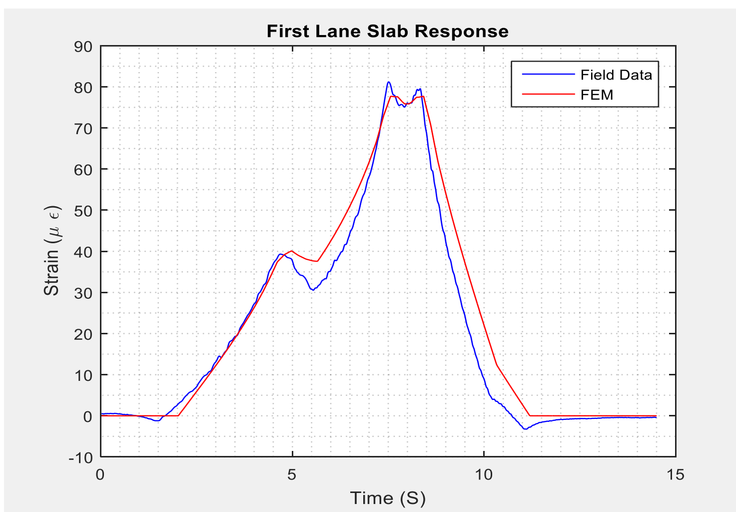



Fig. 44. Example of slab response to load on lane 1

Fig. 45 shows measured strains under slow crawling speed and higher speed. These values were used to evaluate dynamic factor for the bridge, which was estimated equal to 0.03. This value of impact factor was considerably lower than measured for other bridges and could be influenced by higher flexural stiffness of the slab and better dumping properties of concrete if compared to steel girders.

Measured longitudinal strains at the bottom of slab were converted to stresses using modulus of elasticity of concrete, $E_c = 57000(4500)^{0.5} = 3,823,676$ psi. After calibration of the FE model, strain/stress from the self weight (dead load) was also obtained. These values were used in bridge rating.

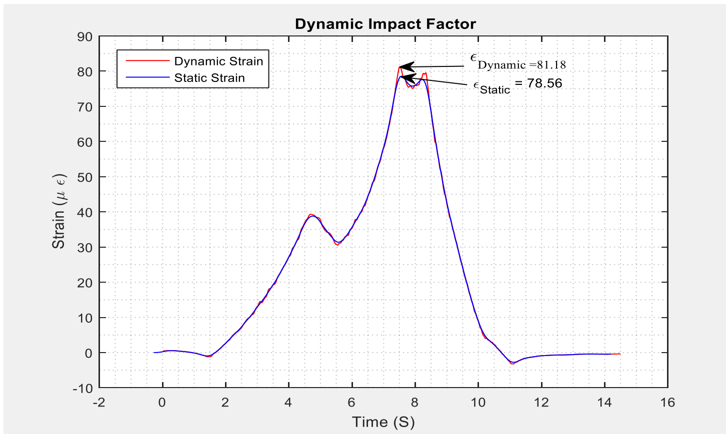



Fig. 45. Static and dynamic strains

Bridge rating based on test results

The maximum strain measured under the loading truck was $\varepsilon_{LL}$ = 81.18x10$^{-6}$ which gave the stress $\sigma_{LL}$ = 310.4 psi. This stress resulted in the loading moment, $M_{LL}$ = 253.286 k-in computed for one precast panel. Maximum stress from dead load was obtained from calibrated model, $\sigma_{DL}$ = 600 psi, which resulted in the dead load moment, $M_{DL}$ = 489.6 k-in. Factored moment capacity of the panel was computed in preliminary analysis, $\phi M_n$ = 3336 k-in. Measured dynamic impact factor, IM = 0.03 was used in rating.

Inventory rating:

$$RF = \frac{3336 - 1.3(489.6)}{2.7(253.3)(1 + 0.03)} = 3.83$$

Operating rating:

$$RF = \frac{3336 - 1.3(489.6)}{1.3(253.3)(1 + 0.03)} = 7.9$$

## Bridge 5 – J-143 in Lancaster County, NE

The structure was a hybrid steel stringer bridge with cast in place concrete box culvert extensions. The original structure was 15 ft long by 24 ft wide steel stringer superstructure with a concrete deck of unknown thickness. The deck was supported on 9 internal I- girders spaced 2.5 ft. and 2 external channels. The bridge was extended 15 ft on both sides with 14 ft x 6 ft concrete culvert with inlet and outlet aprons and wing walls. Because of a scour holes across the inlet of the culvert boxes and settlement of wing footing, there were longitudinal cracks separating the main span from extension boxes. It was clear that the 24 ft. wide deck supports traffic load and the extensions, in form of two culverts boxes, were added later to allow for leveling the riding surface with the actual level of the road. To accomplish this task, the top of the slab was covered with 4 ft dirt and 4 in. asphalt layer on the top. The bridge was originally built in 1936 and drawings were not available. Analytical rating performed earlier with the assumption of simply supported girders, girder distribution factor, S/5.5, no composite action and impact factor, IM = 0.30, indicated that bridge does not have capacity for the live load and was closed to traffic. There was a need to rate the bridge based on field testing. The view of the bridge is presented below.

During the field inspection, it was noticed that steel girders were in different stages of corrosion. External channels were corroded to the degree they could not be assumed to take any load. In that situation, for the prior analysis and FE modeling it was assumed that the deck was supported only on the internal girders. The compressive strength of deck's concrete was estimated using the rebound hammer as, $f_c' = 5{,}000$ psi. The concrete deck was drilled through to find the thickness equal to 8 in. It was also noticed that at the supports, girders were embedded in the abutment walls, which suggested some restrictions to the movement or rotations on the supports, Fig. 46.

Fig. 46. Supporting conditions for girders

Preliminary analysis

Analytical analysis was performed to find out if the legal rating Type 3 truck would be safe as a testing load. The other two available rating trucks were not considered because of a short span of this bridge. Hand calculations resulted in finding stress at the bottom of girders under the dead load, which was substantial with 4 ft. of dirt and asphalt layers on the top of slab, and standard Type 3 truck. From field inspection, it was understood that the span is working with some partial fixity on supports. Also it was observed that the upper flanges of girders were partially embedded in concrete deck indicating possible some composite action. These findings were considered in analysis. The bridge was also modeled using CSI Bridge software with two options of support conditions, simply supported and fixed. As was found after testing, the real behavior was between these two theoretical conditions.

Results from preliminary FE model analysis are presented below. They were computed with the assumptions of simply supported girders and non-composite action, Table 21, 22 and 23.

Table 21. Preliminary results of dead load moments

| **Girder #** | DL Moment (k-in) | **Stress (Ksi)** | Sx (in^3) | A (in^2) |
|---|---|---|---|---|
| 1 | 466.148 | 32.371 | 14.4 | 5.41 |
| 2 | 475.415 | 33.015 | | |
| 3 | 479.257 | 33.282 | | |
| 4 | 478.172 | 33.206 | | |
| 5 | 478.245 | 33.211 | | |
| 6 | 478.079 | 33.200 | | |
| 7 | 477.24 | 33.142 | | |
| 8 | 473.542 | 32.885 | | |
| 9 | 458.247 | 31.823 | | |

Table. 22. Preliminary results of live load moments under fully loaded truck Type 3, 50% loaded and empty truck

| | Full Truck Capacity | | | | | |
|---|---|---|---|---|---|---|
| Girder # | LL (K-in) - Lane 1 | Stress (Ksi) | LL (K-in) - lane 2 | Stress (Ksi) | LL (K-in) - lane3 | Stress (Ksi) |
| 1 | 234.735 | 16.301 | 7.46 | 0.518 | 0.222 | 0.015 |
| 2 | 108.684 | 7.548 | 24.159 | 1.678 | 0.715 | 0.050 |
| 3 | 69.369 | 4.817 | 118.913 | 8.258 | 2.795 | 0.194 |
| 4 | 238.859 | 16.587 | 173.424 | 12.043 | 11.088 | 0.770 |
| 5 | 44.298 | 3.076 | 62.997 | 4.375 | 44.298 | 3.076 |
| 6 | 11.088 | 0.770 | 173.424 | 12.043 | 238.859 | 16.587 |
| 7 | 2.795 | 0.194 | 118.913 | 8.258 | 69.369 | 4.817 |
| 8 | 0.715 | 0.050 | 24.159 | 1.678 | 108.684 | 7.548 |
| 9 | 0.222 | 0.015 | 7.46 | 0.518 | 234.735 | 16.301 |
| | 50 % Truck Capacity | | | | | |
| Girder # | LL (K-in) - Lane 1 | | LL (K-in) - lane 2 | Stress (Ksi) | LL (K-in) - lane3 | Stress (Ksi) |
| 1 | 199.818 | 13.876 | 6.351 | 0.441 | 0.189 | 0.013 |
| 2 | 92.517 | 6.425 | 20.565 | 1.428 | 0.609 | 0.042 |
| 3 | 59.05 | 4.101 | 101.225 | 7.030 | 2.38 | 0.165 |
| 4 | 203.328 | 14.120 | 147.627 | 10.252 | 9.439 | 0.655 |
| 5 | 37.709 | 2.619 | 53.626 | 3.724 | 37.709 | 2.619 |
| 6 | 9.439 | 0.655 | 147.627 | 10.252 | 203.328 | 14.120 |
| 7 | 2.38 | 0.165 | 101.225 | 7.030 | 59.05 | 4.101 |
| 8 | 0.609 | 0.042 | 20.565 | 1.428 | 92.517 | 6.425 |
| 9 | 0.189 | 0.013 | 6.351 | 0.441 | 199.818 | 13.876 |
| | Empty Truck | | | | | |
| Girder # | LL (K-in) - Lane 1 | | LL (K-in) - lane 2 | Stress (Ksi) | LL (K-in) - lane3 | Stress (Ksi) |
| 1 | 164.901 | 11.451 | 5.241 | 0.364 | 0.156 | 0.011 |
| 2 | 76.35 | 5.302 | 16.971 | 1.179 | 0.502 | 0.035 |
| 3 | 48.732 | 3.384 | 83.536 | 5.801 | 1.964 | 0.136 |
| 4 | 167.798 | 11.653 | 121.83 | 8.460 | 7.789 | 0.541 |
| 5 | 31.119 | 2.161 | 44.255 | 3.073 | 31.119 | 2.161 |
| 6 | 7.789 | 0.541 | 121.83 | 8.460 | 167.798 | 11.653 |
| 7 | 1.964 | 0.136 | 85.536 | 5.940 | 48.732 | 3.384 |
| 8 | 0.502 | 0.035 | 16.971 | 1.179 | 76.35 | 5.302 |
| 9 | 0.156 | 0.011 | 5.241 | 0.364 | 164.901 | 11.451 |

Bridge instrumentation

Interior girders were instrumented with wireless transducers placed in the midspan and 1 ft from supports at the bottom of girders, as in Fig.47.

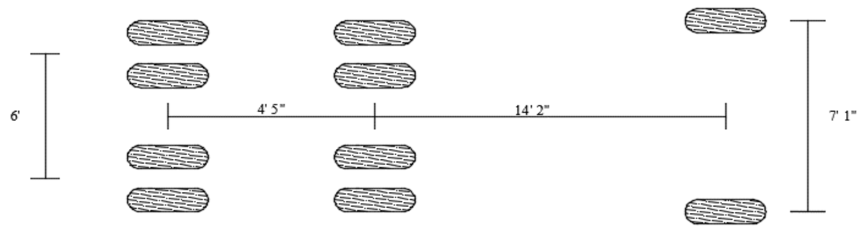

Fig. 47. Transducers attached to the bottom of girders

To detect restrains at the supports, transducers were also placed at the top part of girders webs at 1 ft. from supports on the other girder starting with the most external on both sides. This location for upper transducers was used because the girder flanges were too narrow and partially embedded in slab. To estimate some possible composite action, upper transducers were also located at the midspan on the top parts of girder webs, Fig. 48.


Fig. 48. Transducer attached to the top part of girder web at the same location as the bottom transducer

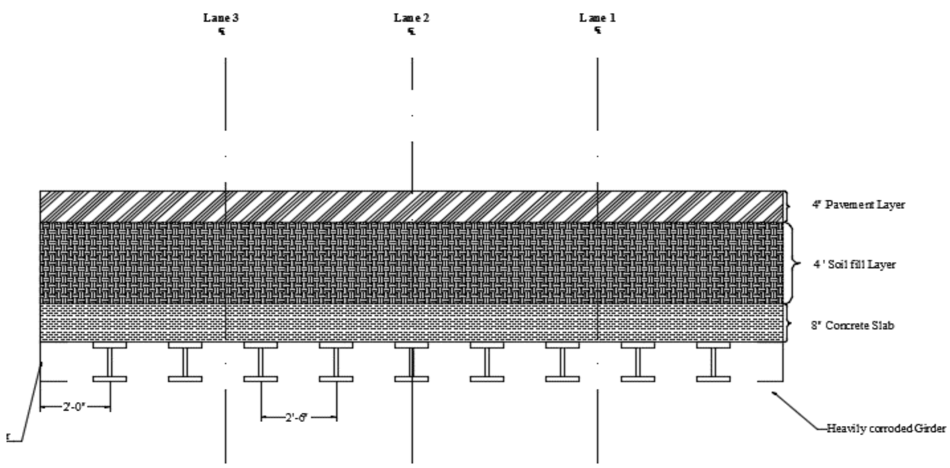
<u>Bridge test</u>

The legal rating truck Type 3 was selected as the test load, Fig.49. Geometrical configuration of the truck is shown in a sketch below.




The truck was loaded to 51.55 kips with 14.4 kips on the front axle and 18.9kips and 18.25 kips on two rear axles. Three lanes were considered when running test truck as shown below.


Fig. 49. Test loading truck




Because of the condition of the bridge, the first three runs on lanes 1, 2 and 3 were conducted using the empty truck, when the recorded strains were monitored. Maximum strains recorded in the bottom flanges at the midspan and in the top of the webs close to supports, converted to stresses for all three runs are presented below, Table 22.

Table 22. Initial readings under empty truck

| | At midspan | Close to support | At midspan | Close to support |
|---|---|---|---|---|
| 1st run | $\varepsilon = 4.89 \times 10^{-6}$ | $\varepsilon = -2.23 \times 10^{-6}$ | $\sigma = 0.142$ ksi | $\sigma = -0.065$ ksi |
| 2nd run | $\varepsilon = 6.67 \times 10^{-6}$ | $\varepsilon = -4.01 \times 10^{-6}$ | $\sigma = 0.193$ ksi | $\sigma = -0.116$ ksi |
| 3rd run | $\varepsilon = 3.93 \times 10^{-6}$ | $\varepsilon = -2.93 \times 10^{-6}$ | $\sigma = 0.114$ ksi | $\sigma = -0.085$ ksi |

The negative values of strains measured close to supports indicated restrains and partial fixity at the supports. After this initial test, it was decided that bridge can be tested with a fully loaded truck. The next runs were performed with fully loaded truck (51.55 kips) traveling on lanes 1, 2 and 3 with a slow speed of 3 mph. Each run was repeated. Last three runs were on lane 2 (center) with higher speed: 25 mph, 33 mph and 37 mph. As before, the high speed runs were used to estimate the dynamic factor for the bridge. All measured strains were recorded by computer software.

Test results

Strains measured at the bottom of girders at the midspan allowed for estimation of the live load moments and the girder distribution factor, Table 23 and Fig. 50.

Table 23. Live load strains at the midspan and GDF

**FIELD DATA**

| Girder | 1st Lane | | Center Lane | | 2nd Lane | |
|---|---|---|---|---|---|---|
| # | Strain | DF | Strain | DF | Strain | DF |
| 1 | 13.5061 | 0.081985683 | 6.6217 | 0.034158498 | 1.8391 | 0.011519379 |
| 2 | 36.2319 | 0.21993744 | 17.3929 | 0.089722479 | 4.4648 | 0.027965703 |
| 3 | 35.2554 | 0.21400982 | 24.9 | 0.128448375 | 7.3886 | 0.046279205 |
| 4 | 30.9117 | 0.187642386 | 32.5787 | 0.16805948 | 13.6082 | 0.085236266 |
| 5 | 21.0655 | 0.127873287 | 32.3848 | 0.167059234 | 20.2752 | 0.12699566 |
| 6 | 13.6341 | 0.082762677 | 31.3973 | 0.161965147 | 28.5496 | 0.178823158 |
| 7 | 7.5237 | 0.045670895 | 23.1433 | 0.119386316 | 29.921 | 0.187413053 |
| 8 | 4.259 | 0.025853283 | 15.8131 | 0.081572972 | 28.9977 | 0.181629875 |
| 9 | 2.3499 | 0.014264529 | 9.6204 | 0.0496275 | 24.6085 | 0.1541377 |
| Sum | 164.7373 | | 193.8522 | | 159.6527 | |

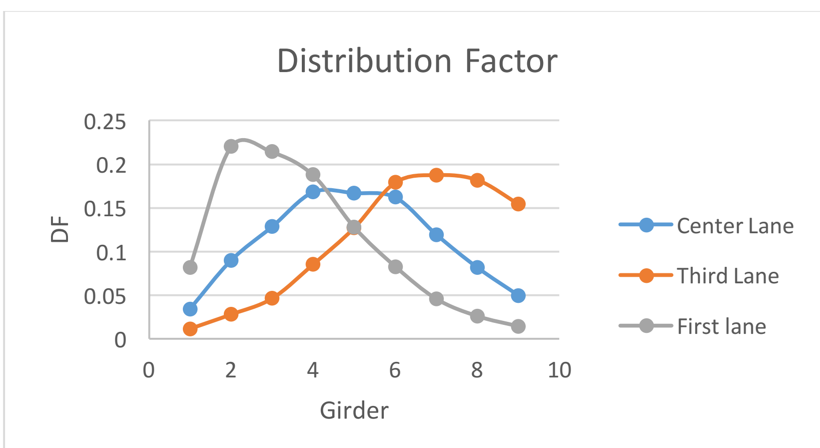



Fig. 50. Girder distribution factors based on measured strains

FE model was calibrated based on the measured strains adjusting supporting conditions for a partial fixity. Example of the history of measured strains and obtained from FE calibrated model for one of the girders is presented in Fig. 51.

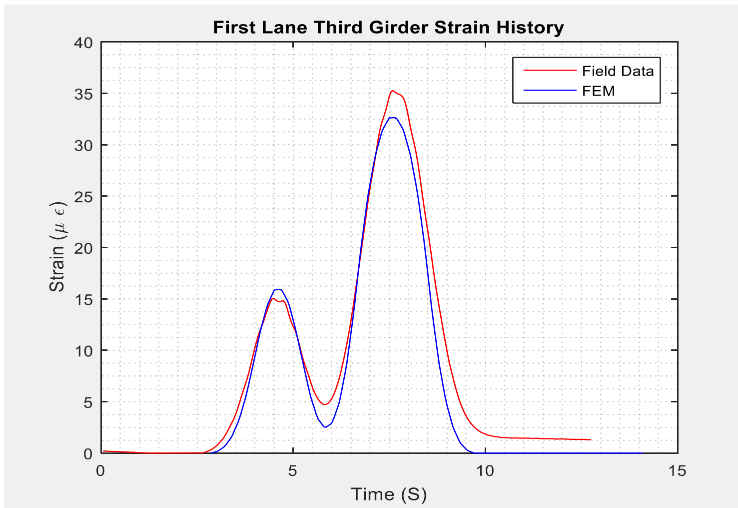



Fig. 51. Strain time history during one of runs

Comparison of strains measured under crawling and higher speed allowed for evaluation of dynamic factor for this bridge, Fig. 52. This value was estimated equal to IM = 0.104.

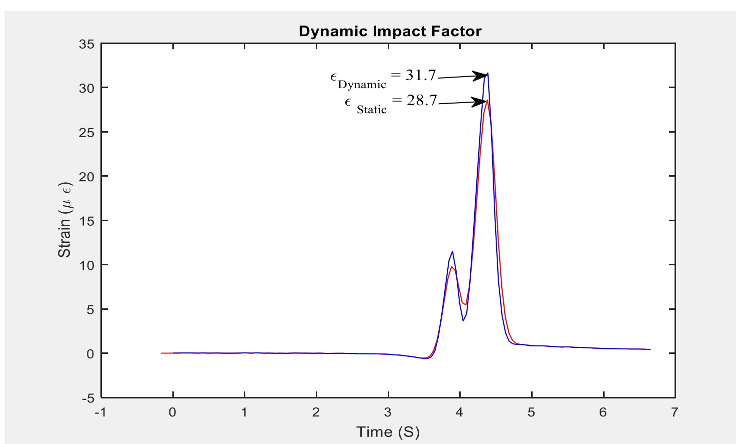



Fig. 52. Comparison of strains for evaluation of dynamic factor

Values of dead load moment were computed using calibrated FE model and these values are shown in Table 24.

Table 24. Dead load moments obtained from FE model

| CALIBRATED MODEL | |
|---|---|
| Girder | DL Moment |
| # | (Kip-in) |
| 1 | 427.397 |
| 2 | 237.508 |
| 3 | 263.973 |
| 4 | 273.124 |
| 5 | 273.823 |
| 6 | 273.159 |
| 7 | 264.442 |
| 8 | 237.153 |
| 9 | 416.969 |

Strains measured at midspan for mostly loaded girder at the bottom flange and close to top flange allowed for estimation of the position of neutral axis, 7.46 in. from the bottom of the cross-section indicating some composite action with slab. This fact was included in rating of the bridge.

Bridge rating based on test results

Moment capacity of the girder was computed assuming some composite action detected during the test. Computed moment equals 868.56 k-in. Rating was performed for the most loaded girder. The live load moment, $M_L$ = 36.54 k-in, was computed using measured live load strain, $\varepsilon = 36.23 \times 10^{-6}$ converted to stress by modulus of elasticity, $E_s$ = 29,000 ksi, using transformed moment of inertia including steel girder and part of the slab. Dead load moment obtained from calibrated FE model, $M_{DL}$ = 427.40 k-in. This was the largest dead load moment for the first interior girder since external girders were so badly corroded that they were considered as not taking load elements and not included in the analysis.

Inventory rating:

$$RF = \frac{(1.0)(868.56) - 1.3(427.4)}{2.17(36.54)(1 + 0.104)} = 3.6$$

Operating rating

$$RF = \frac{(1.0)(868.56) - 1.3(427.4)}{1.30(36.54)(1 + 0.104)} = 5.9$$

Rating performed base on field testing in difference to analytical rating showed that the bridge has enough capacity to be open to traffic.

**Bibliography**

AASHTO LRFD, "Bridge Design Specifications" 7th Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2014.

AASHTO, Manual for Bridge Evaluation, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2010.

Nebraska Department of Roads, Bridge Division, Bridge Inspection Program Manual, 2013.

NCHRP Report 222, "Bridges on Secondary Highways and Local Roads, Rehabilitation and Replacement", Transportation Research Board, National Research Council, Washington, D.C., 1980.

NCHRP Report 243, "Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads", Transportation Research Board, National Research Council, Washington, D.C., 1981.

NCHRP Synthesis 327, "Cost-Effective Practices for Off-System and Local Interest Bridges", Transportation Research Boards of the National Academies, Washington, D.C., 2004.

NCHRP Research Results Digest-No 234, Manual for Bridge Rating Through Load Testing, Nov. 1998.

"Standard Guide for Evaluating Mechanical and Physical Properties of Wood-Plastic Composite Products". ASTM D7031, 2011.

U.S. Department of Agriculture Forest Service Forest Products Laboratory, Madison, WIS., Mechanical Properties of 23 Species of Eastern Hardwoods, FPL – 0230, 1975.

ODOT, Manual of Bridge Inspection, ORC 5501.47, 2010.

SCDOT Research Project No. 655, Load Testing for Assessment and Rating of Highway Bridges, Jan. 2006.

TXDOT, Bridge Inspection Manual, 2013.

WSDOT, Bridge Design Manual, Bridge Load Rating, M 23-50.12, 2012.

Gralund, M.S. and Puckett, J.A., "System for Bridge Management in a Rural Environment", Journal of Computing in Civil Engineering, Vol. 10, No 2, 1996:97-105.

Lichtenstein, A.G., "Bridge Rating Through Nondestructive Load Testing", NCHRP Project 12-28 (13) A, Transportation Research Board, National Research Council, Washington, D.C., 1993.

Long Qiao, "Structural Evaluation Methods on an Existing Concrete Bridge", American Journal of Engineering and Technology Research, Vol. 12, No 2, 2012.

Michael, C. Mertz, D.R. and Commander, B., “Experimental Load Rating of a Posted Bridge”, Journal of Bridge Engineering, 2.1. (1997): 1-10.

Nowak, A.S., “Closure to Load Distribution and Impact Factors for I-Girder Bridges”, Journal of Bridge Engineering, 4.4 (1999).

Santini, B.E. Lefebvre, P.J. Sanayei, M. Brenner, Sipple, B. and Peddle, J., “Objective Load Rating of a Steel-Girder Bridge Using Structural Modeling and Health Monitoring”, Journal of Structural Engineering, 139.10 (2013): 1771-779.