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Protocol to Evaluate and Load Rate Existing Bridges

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MATC

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16. Abstract
The project presents protocols for evaluation of existing bridges. The first part of project is focused on review of analytical methods of bridge rating. The second part is devoted to field-testing procedures, which are presented and described. The protocols include description of the required equipment and operational guides. The document also includes the assessment of accuracy, potential problems, and best practice observations. The field tests include the measurement of strain and deflection, with the objective of determining/verifying girder distribution factor, and minimum load carrying capacity. Field-testing is applied to pre-selected representative structural types and materials, so that the obtained results can be applied to a wider population of bridges. Field-testing involves instrumentation using strain gages (wireless transducers) and linear variable differential transformers (LVDT's), signal nodes and signal receiver, STS VIEW computer software to record test data and a power generator. The test load is considered in the form of test trucks. The developed protocols are demonstrated on three selected bridges. The structures were selected in cooperation with the Nebraska Department of Roads (NDOR). Performed field tests were diagnostic type to verify the live load distribution factors (GDF) and proof load test to find the minimum load carrying capacity.

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Disclaimer

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Abstract

The project presents protocols for evaluation of existing bridges. The research involves identification of problems to be addressed — for example, verification of the load distribution factors, actual live load (weight of trucks), dynamic load factors, minimum load carrying capacity, or fatigue load spectra. The first part of project is focused on review of analytical methods of bridge rating. The second part is devoted to field-testing procedures, which are presented and described. The protocols include description of the required equipment and operational guides. The document also includes the assessment of accuracy, potential problems, and best practice observations. The field tests include the measurement of strain and deflection, with the objective of determining/verifying girder distribution factor, and minimum load carrying capacity. Field-testing is applied to pre-selected representative structural types and materials so that the obtained results can be applied to a wider population of bridges. Field-testing involves instrumentation using strain gages (wireless transducers) and linear variable differential transformers (LVDT's), signal nodes and signal receiver, STS VIEW computer software to record test data and a power generator. The test load is considered in the form of test trucks.

The developed protocols are demonstrated on three selected bridges. The structures were selected in cooperation with the Nebraska Department of Roads (NDOR). Performed field tests were diagnostic type to verify the live load distribution factors (GDF) and proof load test to find the minimum load carrying capacity.

Chapter 1 Introduction

Many existing bridges need to be evaluated to verify their ability to carry traffic. Recent FHWA Long-Term Bridge Performance Program (LTBP) guidelines provide a good basis for the rational evaluation of existing bridges. In rural areas, there are many county bridges that require non-conventional methods of evaluation to verify their adequacy or to upgrade their postings. Evaluation and rating procedures are listed in the AASHTO Manual for Bridge Evaluation. Strict requirements apply to state-owned structures. Federal requirements are not necessarily applicable to off-system bridges, including those that are owned by counties. The AASHTO specified procedures involve analysis to determine load carrying capacity and load effects (such as moments and shear forces). The analytical approach is based on assumptions regarding material properties, the role of connections, and idealized load models. Even the most advanced analytical methods, such as the finite element method, are only as accurate as the input data (or boundary conditions). Because of these hard-to-predict uncertainties, the practical analytical procedures are specified to provide conservative results. However, many county bridges are marginal; their analytically determined capacity can be smaller than analytically predicted loads. Many bridges that are found through analysis to be marginal (on the borderline of being acceptable or deficient) appear to be better than analytically predicted. Field-testing procedures and equipment have reached a level of maturity that is sufficient for effective applications. However, the major problem is the selection of the best technique and equipment for the problem at hand. The available field testing procedures vary in terms of accuracy, ease of installation and operation, initial cost and cost of operation, power supply requirements, qualifications of the operators, data processing capability, and so on. Therefore, there is a need to develop a practical guide for field-testing procedures for bridge owners.

Chapter 2 Literature Review

The documents related to processes and methods of bridge evaluation were reviewed. This included methods of analytical evaluation and rating, and types and procedures of field-testing. The main documents reviewed include AASHTO (American Association of State Highway and Transportation Officials) documents like the AASHTO Manual for Condition Evaluation of Bridges, AASHTO LRFD Bridge Design Specifications, and AASHTO Manual for Bridge Evaluation. The other reviewed documents related to procedures of bridge evaluations were manuals prepared by State Departments of Transportation including:

- NDOR (Nebraska Department of Roads) Bridge Inspection Program Manual
- ODOT (Ohio Department of Transportation) Manual of Bridge Inspection
- DelDOT (Delaware Department of Transportation) Bridge Design Manual
- TXDOT (Texas Department of Transportation) Bridge Inspection Manual

Reports of AASHTO technical committees were also reviewed to find out new propositions to analytical bridge rating. This included two recent reports from 2012 AASHTO Bridge Committee meetings on subject of The Manual for Bridge Evaluation, the report from Technical Committee T-18 Bridge Management, Evaluation and Rehabilitation, and report from Technical Committee T-10 Concrete Design.

Chapter 3 Analytical Bridge Evaluation and Rating

3.1 Methods of Bridge Evaluation

Bridge evaluation methods depend on the type of structure, construction materials, and the category of road and bridge conditions. Types and methods of load rating should be adequate for evaluated structures, which includes traffic volume and the year bridge was built.

Highway bridges are categorized as short-span, mid-span, or long-span bridges. The live load depends on the category of road and is expressed as ADTT (Average Daily Truck Traffic). The construction materials consist of steel, concrete, timber, or masonry. In bridge evaluation, specifically load rating, the current status of structure is important. Signs of bridge aging and deterioration are described in two-year bridge inspection reports, which are mandated by federal and state law. Critical inspections of bridge components should be also considered.

There are six types of inspections:

- Inventory inspections, which document the condition of the bridge, usually after rehabilitation or construction;
- Routine inspection, which determine the condition/safety of bridge components;
- High load hit or vehicle damage bridge inspections, which take place if a vehicle collides with the bridge;
- Underwater inspections, which detect the extent of scour damage;
- Fracture-critical inspections, which detect steel fractures; and
- Special feature inspections, in which any unique feature on the structure that requires inspection at an increased cycle is examined.

An inspection beyond a routine inspection may be called a detailed inspection, which is a close-up, hands-on, inspection of elements to detect deficiencies not readily detected during a routine inspection. This inspection will typically require lift equipment to get the inspector close to the elements being inspected. Non-destructive tests are conducted as part of this inspection.

Prior to rating an existing bridge, the engineer should review results of a recent detailed inspection. It is also needed to review a complete description of a bridge, as-built plans, any modifications since it was built, and its present condition. The bridge condition is evaluated according to the NBIS standard (rated on a scale of 0-9), and by AASHTO guidelines (rated within 1-5 condition states for each identified element) for Pontis data. The three major bridge elements considered for ratings and evaluations are: deck, superstructure and substructure. The condition of an individual bridge element can be classified according to its rating:

"poor" = condition rating 4

"fair" = condition rating 5 or 6

"good" = condition rating 7

Routine inspections also identify maintenance and bridge rehabilitation issues.

3.2 Load Rating Methods

Load rating methods include theoretical analysis and load testing. Depending on the information on a bridge, the load rating method can be selected. The load testing method may be used in certain circumstances, like lack of documentation of the bridge or unusual type of bridge. It can be also used to help with analytical methods, especially with modeling a bridge with the finite element method, which requires calibration.

Generally, methods of analysis are: load factor method (LFD), working stress method (ASD), and load and resistance factor method (LRFD). Ratings according to these methods are determined as follows:

- In the LFD (ultimate strength) method, ratings are determined by calculating the ratio of the yield strength of a member to the factored loads.
- The working stress method (ASD) is used in rating if the LFD method is not possible. The working stress method should be used to load rate timber bridges. Other types of bridges may be rated by the working stress design method if approved by Bridge Management Engineer in the State.
- The load and resistance factor method (LRFD) is the most desirable method according to AASHTO standards. It can be performed using a computer program, be it BRASS or STAAD or any finite element model, but it is understood that the computer model needs to be calibrated, preferably by load test results.

Load test methods provide the answer to the actual performance and capacity of the bridge, but this testing needs to be proceeded by analytical analysis. We cannot simply put a load on a bridge that was not validated by analytical analysis, which is especially important for proof load testing.

The load rating is a measure of bridge live load capacity and it has three commonly used categories:

- Inventory rating, as defined by the current AASHTO Manual for Bridge Evaluation, is the load, including loads in multiple lanes, that a bridge can safely utilize for an indefinite period of time.
- Operating rating, defined by the same manual, is the maximum permissible live load that can be placed on a bridge. This load rating also includes the same load in multiple lanes. Allowing unlimited usage at the operating rating level will reduce the life of the bridge.

- Posting: a bridge will need posting if the load effects exceed the maximum permissible level of load capacity, i.e. operating level. States posting policies can vary and may be at a level between inventory and operating.

Load ratings are normally completed for three types of trucks and for design loading as required by AASHTO. Other types of load can be used to load rate the bridge if a special permit load is required to transport a heavy load or a vehicle has unique axle spacing. AASHTO requires the load ratings to be completed with AASHTO standard rating vehicles (HS20) and AASHTO design vehicle loads. Load rating vehicles are representative of trucks typically using US roads. Each state also determines the maximum legal axial load and spacing, which is representative for their particular state. Some State Legal Trucks can differ from AASHTO Legal Trucks by axial tonnage or spacing. The inventory rating can be initially estimated to be at least equal to the design loading if there is no damage or deterioration, and the original design was using HS or HL-93 (LRFD) load pattern. Traditionally, inventory or operating ratings were determined using either Load Factor (LF) or Allowable Stress (AS) methods. Since 2000, LF is to be used for all on-system bridges, except for timber bridges because of difficulty to assign an ultimate strength to timber. Timber bridges are rated using AS methods. If the bridge was designed using the AASHTO LRFD method, the same method and load should be used for rating.

On the AASHTO Bridge Committee meeting in 2012, revisions to Legal Load Factors were proposed, based on findings of NCHRP Project 12-78, to refine the LRFD methods in the AASHTO Manual for Bridge Evaluation (MBE). Extensive data analysis was performed on 1,500 bridges of varying material types and structure configurations (redundant systems) to maintain an acceptable level of bridge reliability without unnecessary restrictions on commerce. For AASHTO Legal Loads, the load factors for live load included in the MBE appear to produce higher reliability index than that assumed in the development of the MBE. Reducing the load factors to correspond to the target reliability index of 2.5 results in reducing the number of bridges not passing the LRFD rating for these loads and still satisfy the target level of reliability.

The proposed, generalized live load factors for the STRENGTH I limit state for routine commercial traffic are shown in table 3.1 below.

Table 3.1 Generalized live load factors for routine commercial traffic

Traffic Volume (one direction)	Load Factor
Unknown	1.45
ADTT 5000	1.45
ADTT 1000	1.30

Linear interpolation is permitted for other ADTT values between 5000 and 1000

If in the engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in the MBE when determining the safe legal load, the factors from table 3.1 may be increased, but not to exceed the value of the factor multiplied by 1.3. Where adequate information on the traffic is available, site, route, or region-specific load factors may be developed and if accepted by the owner of bridge, these factors may be used in lieu of the values in the MBE. Weight-In-Motion (WIM) data collected at specific a site or region may be used to perform the load calibration to determine these site-specific factors. Depending on the traffic pattern and truck counts, these load factors may be higher or lower than those listed in the manual.

The LRFD methodology in the AASHTO MBE includes provisions for evaluating permit requests for Routine and Special permits. The current permit load factor calibration for permits is tied to the LRFD distribution analysis method and does not provide guidance to States that want to use refined methods of analysis for heavy vehicles and non-standard widths vehicles permits. Additionally, the target reliability level for different permit types, which is currently established at either reliability index equal to 2.5 or 3.5, may not be consistent with current permit issuance practices. This reliability index was verified by comparing with reliability indices used in current permit practices. Revised permit load factors are presented in table 3.2. In order to better reflect the load effects from the different truck types, the routine permits were categorized based on a combination of their gross vehicle weights (in kip) and first to rear axle lengths (in ft). The load factors for routine permits can be reduced for cases where the permit truck's gross weight and load effect are high to reflect the lower probability of having a random truck of equal or higher weight and load effect crossing alongside the permit truck.

Table 3.2 Permit load factors

Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight Ratio[b]	
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	Up to 100 kips	≥ 150 kips
				=1000	1.80	1.30
				<100	1.60	1.20
					1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

DF_a = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

$_b$ Permit Weight Ratio = GVW/AL; GVW Gross Vehicle Weight; AL = Front axle to rear axle Length; Use only axles on the bridge.

According to AASHTO LRFD Bridge Design Specifications, computer programs used for structural analysis (based on finite element method for example) should be verified against the results of universally accepted closed-form solutions, other previously verified computer programs, or physical testing. It also says that existing bridges can be instrumented and test results can be obtained under various conditions of traffic and/or environmental loads or loads tested with special purpose vehicles to establish force effects and/or the load-carrying capacity of the bridge. These measured force effects may be used to project fatigue life, to serve as a basis for similar designs, to establish permissible weight limits, to aid in issuing permits, or to establish a basis for prioritizing rehabilitation or retrofitting.

Chapter 4 Field Testing

Chapter 4.1 Types of Field Testing

There are a few popular field tests performed on bridges to find specific information that cannot be obtained from analytical analysis.

Weight-in-Motion (WIM) Measurement of Trucks

A weight-in-motion test attempts to gather truck traffic data, which includes axle weight, axle spacing, vehicle speed, multiple truck presence on the bridge, and average daily truck traffic (ADTT). Beneath the deck, the WIM system is invisible to the truck drivers, so overloaded trucks would not avoid the bridge. Unbiased results can thus be obtained. The system is portable and easily installed to obtain site-specific traffic data. Sensors measure strains in girders, and these data are than used to calculate the truck parameters at the given traffic speed. The bridge WIM system consists of three basic components: strain transducers, axle detectors (tape switches or infrared sensors), and the data acquisition and processing system.

Diagnostic Testing

One type of diagnostic bridge test is a live (truck) load distribution test. The objective of this test is to determine the distribution of live load to each girder. One or two trucks of known weight (total, on axles) and axle spacing, are used as the test load and the resulting strains are collected from all girders. To determine the distribution of load transversely on the bridge, at least one strain value is taken to determine the longitudinal load distribution; strain values at the ends and quarter points of the girders are also necessary. The number and placement of instruments on the bridge may vary according to test objective, but in general, transducers are placed to determine the distribution of load to the girders transversally and longitudinally, and to find the maximum load effects. Strain transducers are attached to the lower flanges of each girder at midspan, assuming that that the bridge is a simple span. Although midspan is not the location of maximum stress, it is sufficiently close for test measurements. For continuous spans, the locations of the maximum effect should be estimated analytically, and the transducers should be placed there. Field tests have shown that in general, the girder distribution factors (GDFs) specified in the AASHTO Code are too conservative for longer spans and larger girder spacing, and in some cases can be not conservative enough for short span bridges.

Dynamic Load Spectra

The objective of dynamic tests is to verify or determine the actual dynamic load. The dynamic load can be a significant component of the live load. Not only is it time variant and random in nature, but also it depends on the vehicle type, vehicle weight, axle configuration, bridge span length, road roughness, and transverse truck position on the bridge. In the dynamic test, strain transducers are attached to the bridge girders, and strain values are recorded under actual moving traffic loads. The weight-in-motion truck measuring system can also be used for dynamic load tests. If gross vehicle weight (GVW) and axle loads are not required, then axle detectors are not needed. The procedure for equipment installation is identical to that of the WIM test. This test can be carried out simultaneously with the weight-in-motion test. Field measurements are taken by the WIM system to determine the actual dynamic load effects and to verify the available analytical models. For each truck passage, the dynamic response can be monitored by recording strain data. The truck weight, speed, axle configuration, and lane occupancy can be determined and recorded from WIM measurements. Field tests have indicated that dynamic load is independent of bridge span, and for heavily loaded vehicles on roads of reasonable quality, typically does not exceed 10% of the static load.

Fatigue Load Spectra

Development of a fatigue load model requires the collection of actual dynamic stress time histories for various members and components. Following a collection of time histories, the data must be processed into a usable form. The expected fatigue life of a component can then be calculated using the rainflow method. The strain measuring system (SMS) is required to collect component strain histories produced by actual traffic loads. The stress cycle histograms can be assembled by the rainflow method of cycle counting. The rainflow method counts the number of cycles, n, in each predetermined stress range, S_i, for a given stress history. The SMS can record up to 4 billion cycles per channel for extended periods in an unattended mode. Strain transducers are attached to the lower mid-span flanges of a bridge. Since the strains are to be recorded over an extended period of time, the data acquisition system must be attached to a reliable and secure location on the bridge. Strain histories usually must be collected continuously for periods of at least one week long, although this time can be reduced using the rainflow algorithm. Data should be collected for each bridge girder. As a means of comparison of fatigue live load, the equivalent stress, s_{eq}, can be calculated for each girder using the root mean cube (RMC) formula. Needed for solution are: S_i = midpoint of the stress interval i, and p_i = the relative frequency of cycle counts for interval i. The stress, S_i, is calculated as a product of strain and modulus of elasticity of steel. Stress spectra vary considerably from girder to girder (component specific). Therefore, the expected fatigue life is different depending on girder location. Test results show that exterior girders experience the lowest load spectra.

Proof Load Testing

Proof load testing is most often used to verify the capacity of a bridge to avoid traffic restrictions, and the results can also help to determine whether to replace or repair a bridge. The proof load test can be used either to find the yield capacity of the structure or to check its ability to carry a specified live load. Usually, the yield capacity of a bridge is very high and requires exceptionally heavy loads. In this case, proof load tests are carried out to verify if the bridge can safely carry the maximum allowable legal load. Before the proof load tests, the target proof load needs to be estimated by analytical methods (preferably finite elements model). The type and placement of load as well as the instrumentation and data acquisition setup would depend on the target proof load level. Most field tests have indicated that bridges have a great amount of reserve capacity beyond that which they are assumed to have in design. Usually, the actual experienced stresses are significantly lower than those expected. Proof load tests can reveal that even deteriorated structures are capable of carrying loads in excess of their design values without distress.

4.2 Field-testing Procedure

The field-testing procedure requires a few steps that are mandatory for successful bridge test. After selection of a candidate bridge for testing, the analytical evaluation of structure needs to follow. In general, the capacity of bridge superstructure should be estimated. In the case of diagnostic testing, proposed test load effect needs to be checked against bridge capacity; usually there is no problem because this type of load will not be proposed larger than a legal load. Recent bridge rating documents (if available) should be studied to find out the actual operating and inventory ratings. Drawings of the bridge are very helpful in analytical evaluations and also finite element modeling. If bridge documentation is not available (as can be the case with older bridges), a detailed bridge inspection, field measurements to find dimensions, and materials evaluation will be needed. Additionally, analytical bridge rating should be performed prior to testing. Rebound hammer or coring can be used to estimate the strength of the concrete. Strength of other materials, such as structural and reinforcing steel or timber, should be assumed appropriate for the year of construction (if material testing is not possible). Finite element modeling is very helpful in preparations of the field-test and later analysis of the test results, however, to obtain reasonable information on how superstructure is shearing the load, the FE model needs to be calibrated using field test results. In the case of proof load testing, special attention should be on capacity estimation (moments and shears), not only of girders, but also bridge deck and supporting transversal beams (cap beams on supports). Any deterioration of main bridge components needs to be estimated and included in analytical computations and FE modeling. The importance of this step cannot be overestimated since the proof load is usually about 1.5 times the legal load and often much more. A good practice is to perform proof loading in steps, starting with smaller loads (results can be used to calibrate FE model) and gradually increasing. Once again, a calibrated FE model can be useful in selecting a load close to the end of linear performance. The maximum proof load that did not initiate the plastic stage in the bridge components can be considered a minimum capacity of the bridge and a safe load for special permits.

4.3 Examples of Field Testing

A detailed testing procedure is illustrated using three examples of bridge testing performed as part of the project. Descriptions of tested bridges, materials, testing equipment and their installation, analytical estimations and FE modeling, and test results and their meanings are presented below, separately for each test.

In cooperation with the Nebraska Department of Roads (NDOR) Bridge Division, two bridges were selected for diagnostic field-testing. Both bridges were located on rural roads in Lancaster County in Nebraska. Permission for testing was granted by the Office of the Lancaster

County Bridge Engineer. The first bridge (C005504105) scheduled for testing was inspected and reviewed by the project team. The bridge is a single span, 24 feet long, built in 1961, and located on an unpaved road. Measurements, which were taken on the site, were in agreement with drawings of the bridge. Bridge superstructure was built with 11 steel girders supporting concrete deck. The compressive strength of deck concrete was evaluated using a nondestructive method (rebound hammer). Yielding strength of steel girders was assumed based on the year the bridge was built. Analytical analysis was performed to estimate the capacity of the bridge and rating factors. Bridge superstructure was also modeled using CSi Bridge finite element software. The finite element model was calibrated using the test results. Based on analytical calculations and primary modeling, a loading truck was selected for the test. The first two field tests were planned as diagnostic type of test.

In preparation for field-testing, analytical analysis using close-form solutions was performed on the first bridge. This single span bridge consisted of 9 interior steel girders, S 12x31.8, and two exterior girders, C 12x20.7, with a 22.8 ft span length and a 6 inch thick concrete deck. The interior girders were spaced 2.54 ft apart and exterior girders were 1.375 ft apart. The curb size was 12.5 inch wide and 9.25 inch deep. The roadway was 22 ft wide with typical railing on both sides. The 5 stiffeners in form of transversal bars were spaced 68 inches apart with a 7-inch distance from supports for the first and last stiffener. The bridge span was simply supported and non-composite action was assumed between deck and girders. The compressive strength of concrete was estimated as 3.75 ksi (using Schmidt hammer), and the yield strength of girder steel was assumed equal to 36 ksi. Moment capacity of internal and external girders was computed.

Inventory and operating rating was also performed for the bridge using the AASHTO LRFD method. Because of the short span of the bridge and category of the road, an AASHTO Type 3 truck was used for rating. This rating truck is also listed as a possible rating vehicle in the Bridge Inspection Program Manual prepared by the Nebraska Department of Roads. The model truck has three axles: the first is 8 tons and two rear axles are 8.5 tons each with 15 ft distance between first the second axle, and 4 ft between two rare axles; the gross weight is 25 tons. The girder distribution factor was computed according to the AASHTO LRFD formula and the value was very similar to the older Newman's formula based only on the space between girders. The understanding was that an actual load distribution factor would differ since code provisions are for 3.5 ft or more girder spacing. The expectation was that bridge test results should give a more realistic load distribution factors. Strength Limit State I was considered for rating. The impact factor was calculated from the code formula. The condition factor was assumed to be 0.95 for fair condition; the system factor and LRFD resistance factor were equal to 1.0. With the above assumptions, the inventory and operating rating factors were both smaller than 1.0.

The truck used as a test load was a Type 3 truck with a first axle load of 14,060 lb and two rear axles at 13,130 lb each with 15 ft distance between first the second axles, and 4.7 ft between the two rear axles, with a gross weight of 40,320 lb. Figure 4.1 shows a photo of this loading truck approaching the bridge.

Figure 4.1 Test load for the first bridge

Twenty-eight strain wireless transducers were attached to the bottom of all girders at the midspan and four girders at 12 inches from supports (on both ends) at the bottom and top of girders. Transducers were attached by screw tabs glued to steel. Fast acting glue with accelerator was used to install transducers, as seen in figure 4.2.

Figure 4.2 Strain transducer attached to the bottom flange of girder

Four LVDTs (linear variable differential transformers) were located at the midspan of girders to measure deflections, as shown in figure 4.3.

Figure 4.3 LVDT and node signal collector

Cables connected the strain transducers and LVDTs to nodes. Each node provided eight channels to collect data. Two main wireless signal receivers were placed outside of the bridge to transmit signals to a computer, as seen in figure 4.4. All nodes and receivers were battery operated with a working capacity of about eight hours, however, portable power generator was used to run the computer. Data was registered using STS View computer software.

The test truck was placed in three positions transversally on the bridge: at the center and one foot from the left and right curbs. The tests were performed at crawling speed and high speed (35mph for this road). The FE model showing the strains under the test truck when it was running in the center (transversally) of the bridge is presented in figure 4.5. The model was calibrated using test data. Supporting conditions were adjusted (introduced springs with certain stiffness) to obtain strains in girders similar to measured strains. Moments from the test (computed based on strains) were also compared to moments from the FE model. Calibration was performed for three transverse tuck positions. Figures 4.6, 4.7, and 4.8 illustrate the calibration results.

Figure 4.4 Main signal receiver

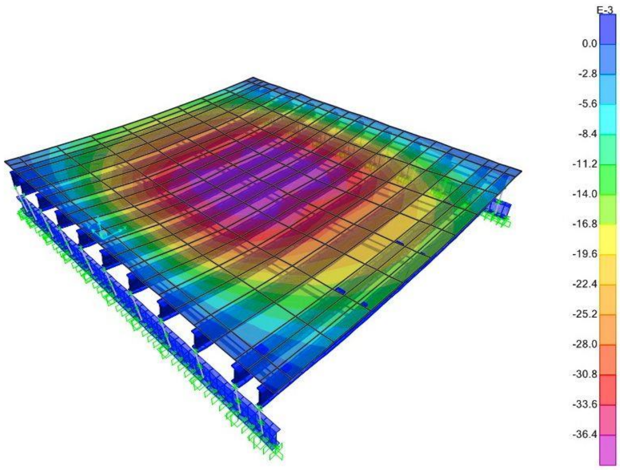

Figure 4.5 FE model of the bridge presenting strains under passage of the test truck centrally located

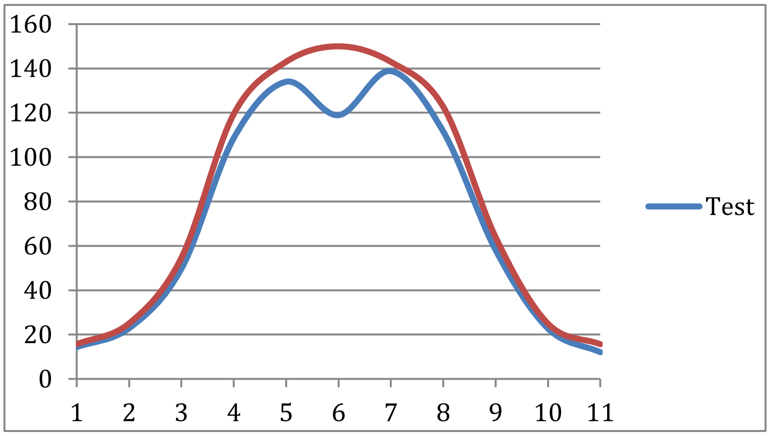

Figure 4.6 Measured and modeled strains (x 10^{-6}) after calibration for central position of the truck

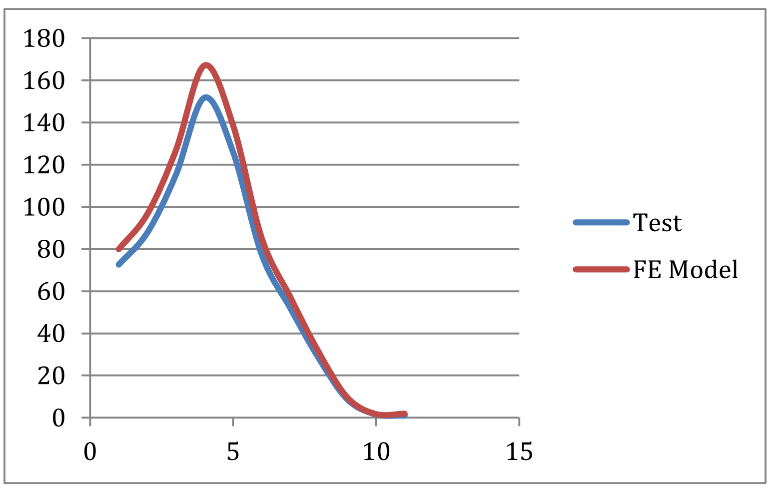

Figure 4.7 Measured and modeled strains (x 10^{-6}) after calibration for right lane position of the truck

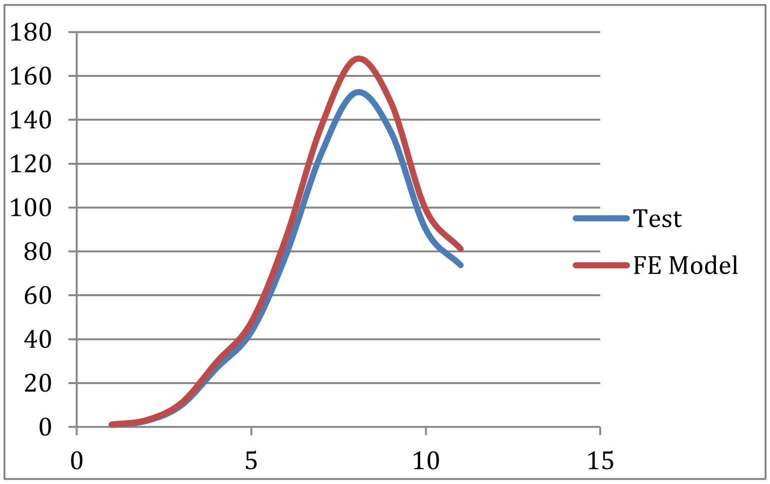

Figure 4.8 Measured and modeled strains (x 10^{-6}) after calibration for left lane position of the truck

Girder distribution factors (GDF) that were computed based on test results and the FE model for the central position of the truck, as well as left and right lanes, are presented in figures 4.9 and 4.10.

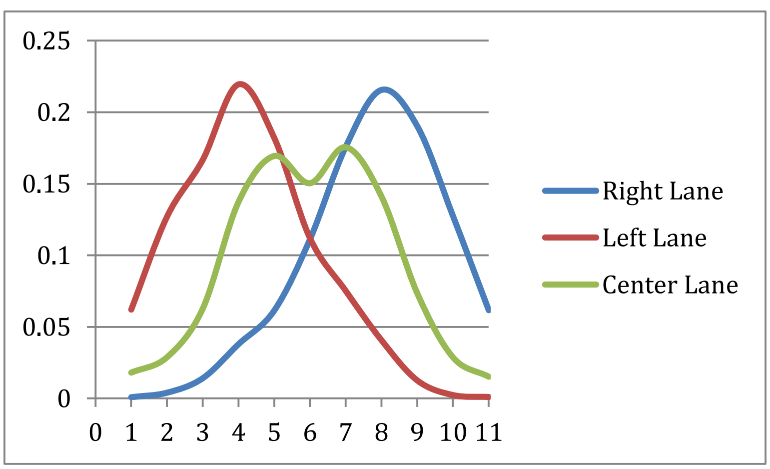

Figure 4.9 GDF computed for center, left lane, and right lane positions of the truck, based on test results

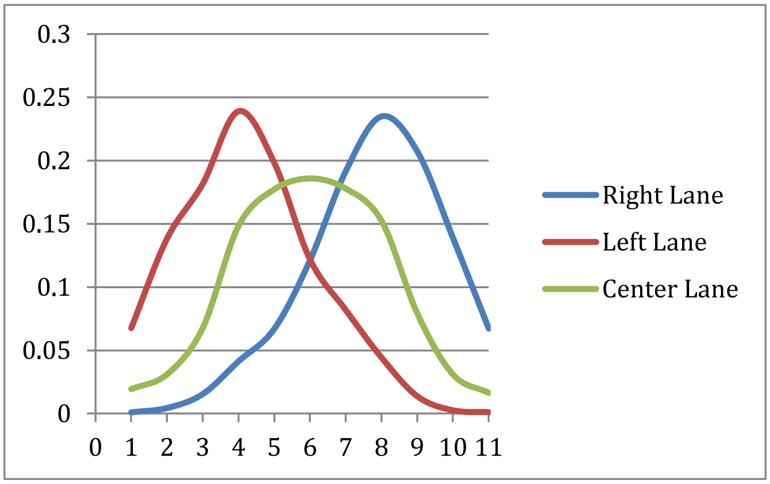

Figure 4.10 GDF computed for center, left lane and right lane positions of the truck, based on FE model

It can be observed that live load distribution factor (GDF) computed for interior girders according to the AASHTO formula was higher (0.264) than GDFs obtained from test results. This was expected since the AASHTO formula was developed for girder spacing more than 3.5 ft, and the actual girder spacing for the tested bridge was 2.54 ft for the interior girders.

If the AASHTO Bridge Committee proposition to reduce the live load factor for rating (as described earlier) was adopted as a 1.3 load factor (since ADTT on the rural road is much less than 1000 trucks), the rating factor's value was increased to 1.32 for inventory rating and 1.27 for operating rating. The major factor influencing the rating of this bridge is the shearing of live load between girders. The load distribution factors from code may not be very realistic for this bridge. Using load distribution factors computed from test results (presented in the figures above), both rating factors are much higher. The inventory rating factor is equal to 3.16 and the operating rating factor equals 3.04. The live load factor was decreased from 1.75 to 1.30 for inventory rating; the operating rating used the same as original in the AASHTO code, which was equal to 1.35. For both rating factor estimations, the condition factor $ø_c = 0.95$ for a fair condition of the bridge and the system factor $ø_s = 1.0$ for multiple girders spaced 2.54 ft apart.

The second tested bridge (C005506410) was located in Kramer, which is close to Crete, NE. The superstructure of this bridge was similar to the first one: 22 ft long with a 24 ft wide span, 9 interior girders (S 12x31.8) spaced 2.46 ft apart, and 2 exterior girders (C12x20.7) spaced 2 ft from interior girder. The reinforced concrete slab was 5 inches thick with a 6 in x 6 in curb and 6 in gravel overlay. The bridge had 4 transverse stiffeners spaced 7.25 ft and 0.58 ft from supports. The compressive strength of the concrete was estimated as 4,000 psi and the yielding strength of steel was assumed to be 33 ksi because of the age of the bridge. Analytical analysis prior to testing was performed, similar to the first bridge, finding bridge capacity (girders) and the AASHTO rating using an AASHTO Type 3 truck. A finite element model of the bridge was prepared.

For field-testing, bridge girders were instrumented with strain transducers and LVDTs as in the first tested bridge. The loading 3-axle truck used for test was the same as before, but with larger axle loads. The front axle load was 16,240 lb and two rear axles were 18,250 lb each, with a total gross weight of 52,380 lb. The truck was also run in three transverse positions and test data was recorded. Figures below show the results of FE model calibration (figs. 4.11, 4.12, and 4.13) and distribution of GDFs for 11 girders, obtained from test results and the calibrated FE model of the bridge (figs. 4.14 and 4.15).

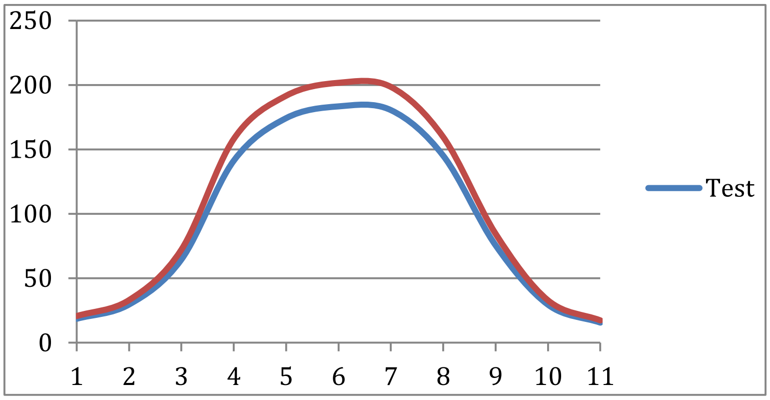

Figure 4.11 Measured and modeled strains (x 10^{-6}) after calibration for central position of the truck

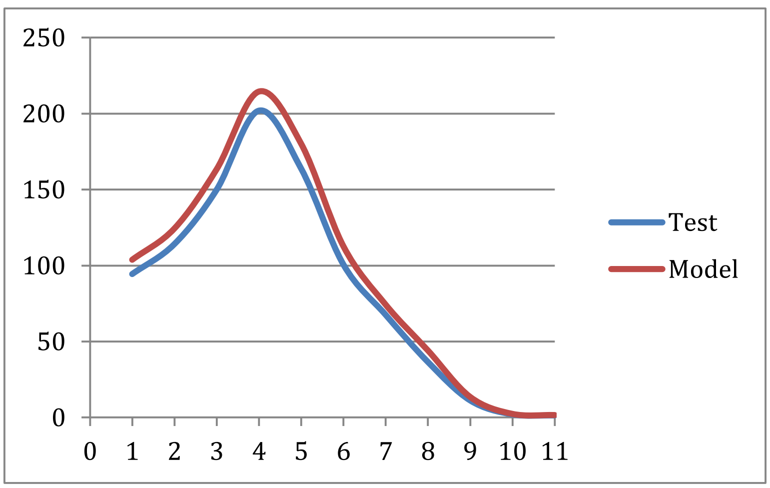

Figure 4.12 Measured and modeled strains (x 10^{-6}) after calibration for right lane position of the truck

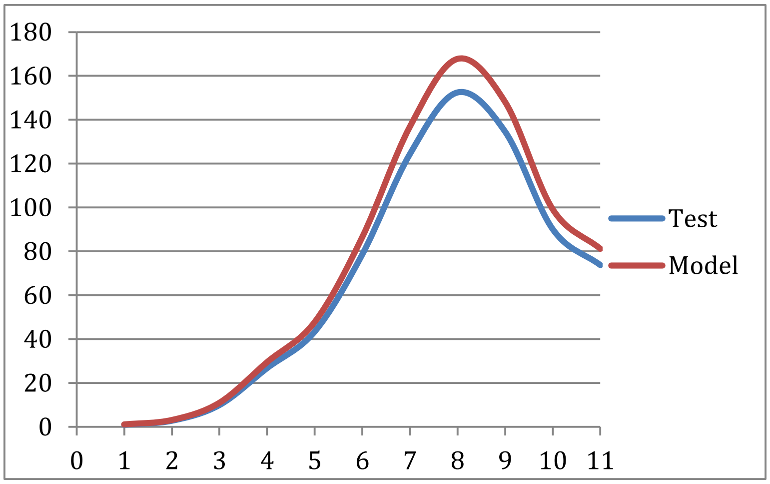

Figure 4.13 Measured and modeled strains (x 10^{-6}) after calibration for left lane position of the truck

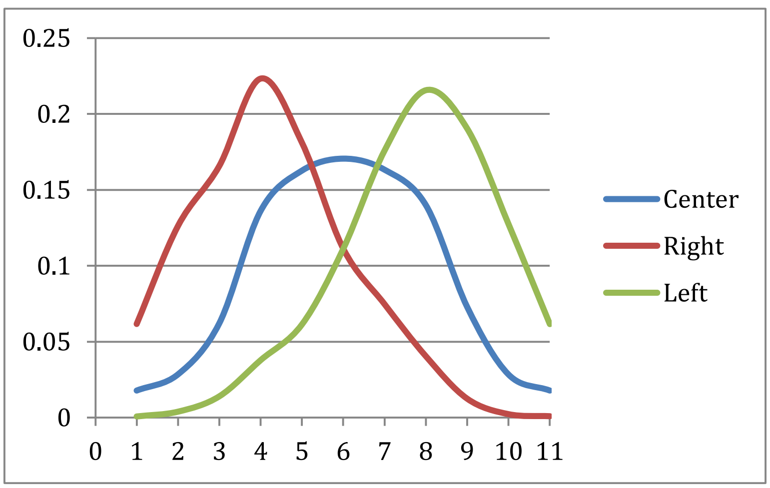

Figure 4.14 GDFs computed for center, left lane, and right lane positions of the truck based on test results

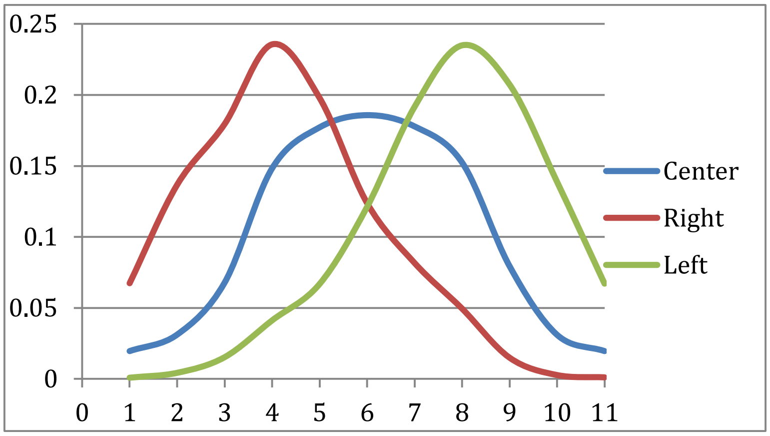

Figure 4.15 GDFs computed for center, left lane, and right lane positions of the truck based on FE model

The bridge rating was performed analytically (ASSHTO LRFD method) and based on test results. The analytical inventory factor was equal to 1.57 and the operating factor was equal to 1.51. These rating factors, estimated based on test results, were 2.99 and 2.88 respectively. Test-based rating factors were computed for the most loaded internal girder (selected from three positions of the loading truck).

The third type of test was proof loading. Proof load testing is typically used to verify components and system performance under a known external load and is normally intended to provide a complementary assessment methodology to the theoretical assessment. The use of such tests, due to the risk of collapse or damaging essential elements of the structure, must be restricted to bridges that have failed to pass advanced theoretical assessment and are therefore condemned to be posted, closed to traffic, or demolished. It is also important that the bridge has a high level of redundancy to carry the load in order to be a good candidate for proof loading. Monitoring of critical locations should be performed during the test to determine the possibility of non-linear behavior. The strains in the materials in the most critical sections should be measured in order to guarantee the elastic limit is not reached. The load is applied in accordance to a loading scheme and loads must be moved to different positions to check all load path components. After execution of proof load test and load removal, the bridge should be inspected to see that no damage has occurred, which could be excessive crack width, deflection, or opening cracks in prestressed concrete structures. The personnel in charge of the execution of proof load test should have a proved qualification and experience in the execution of similar tests. The proof load level should be sufficiently higher to ensure the desired level of safety if the bridge passes the test. Typically, a load higher than the one the bridge is expected to carry along its entire service life is placed on the bridge. This accounts for uncertainties, in particular the possibility that a bridge can be overloaded during the normal operation, and the impact factor, since proof load tests are normally executed in a static way.

The two historic (built in 1933) identical parallel Penny Bridges, located on Sheridan Boulevard and 33rd St, located in City of Lincoln, NE were considered for testing. The eastbound bridge posted and in worse condition was selected for proof load testing. This 23 degree skewed bridge consisted of three spans: 42 ft, 50 ft, and 42 ft. The longest middle span, 50 ft, was tested. A reinforced concrete bridge deck 22 ft wide (roadway), 5 ft wide (sidewalk), and 7.5 inches thick with two inches of asphalt overlay was supported on six simply supported steel girders. The curb, approximately 10 inches by 20 inches, was located on both sides of the deck and between roadway and sidewalk. Three types of girders were used in superstructure: W30x108, W30x124 and W30x132. Concrete railing was constructed on both sides of the deck. The middle span was more than 20 ft high, passing over a bike path located in the park below the bridge. The bridge was in a poor condition; steel girders were corroded on the top and bottom flanges, with up to 30% loss of flange thickness on some girders according to NDOR’s last evaluation in 2009. The deck concrete was cracked through the thickness of deck. The bottom of the tested middle span is shown in figure 4.16.

Figure 4.16 Bottom view of the tested span

The last load rating performed by NDOR (Load Factor Rating based on a Type 3 truck) shows very low inventory rating factor of 0.31 and operating rating factor 0.52. The bridge was recommended for posting to 17 tons and a permit load of 22.2 tons. Because of poor condition and very low ratings, the bridge was scheduled for replacement shortly after the test (construction starting the next day). The bridge was selected for testing in cooperation with NDOR and City of Lincoln.

The goal of the test was to check if the real capacity of a very deteriorated bridge could be higher than estimated based on theoretical analysis and current methods of rating. Since the bridge was scheduled for replacement, the idea was to find out how big of a load would be needed to start plasticity in steel girders. In preparation for testing, the bridge superstructure was analytically evaluated to establish its theoretical capacity. Two external and four internal girders were analyzed to find the moment capacity and shears. The girders were analyzed as non-composite with deck. Two reinforced concrete transverse (cap) beams supporting the middle span on both sides were also evaluated for their moment and shear capacities. Considering the condition of the bridge and the year when it was build (1933), the following materials properties were assumed for analysis: for steel girders – structural or unknown grade prior to 1954 yielding strength = 33,000 psi (Article 6.6.2.3, AASHTO Manual for Condition Evaluation) and modulus of elasticity = 29,000 ksi; for concrete deck – concrete Class AA (according to the original drawings) should have compressive strength of 4,000 psi (Section 520, Portland Cement Concrete), however because of very cracked concrete, its strength was assumed = 2,500 psi with modulus of elasticity = 2,850 ksi; for cap beam – compressive strength of concrete was assumed = 3,200 psi; the reinforcing steel for bridge deck and cap beams – yielding strength was assumed = 33,000 psi with modulus of elasticity = 29,500 ksi (for reinforcing steel grade of the period of bridge construction, no information was provided on bridge drawings). The bridge was also modeled using the finite element program CSiBridge 2014 with the assumption of simple supports of three spans and material parameters as listed above (fig. 4.17). The loss of girder cross-section due to bad corrosion was included in the FE model. The proof load was selected for testing based on this preparation part of the test.

Two Type 3 trucks and a CAT 328D LCR Hydraulic Excavator were used as a proof load. Three axles truck has a distance of 15 ft between first and the second axle and 4.7 ft between two rear axles. The transverse distance between axles was 6 ft. The first truck was loaded to 15,620 lb on front axle and 36,680 lb on rear dual axels (total 52,300 lb); the second truck was loaded to 15,120 lb on the front axle and 34,820 lb on rear dual axles (total 49,940 lb). The CAT excavator specifications were: length of tracks 16.5 ft, width of tracks on axis 8.5 ft, and a total load of 87,850 lb. The roadway was considered as two lanes. The load was applied gradually starting with one truck on the first lane (2 ft from the curb) and the second lane separately, two trucks side-by side on both lanes, one truck following the other at a distance of 4 ft, positioned in the center of the roadway, and finally with the CAT Excavator also positioned in the center of the road way, which was considered as the most concentrated heavy load. Figures 4.18 and 4.19 show the loading trucks. It should be noticed that selected load was much heavier than allowed for this bridge (based on rating 22.2 tons); each truck was weighted about 25 tons and CAT excavator over 40 tons.

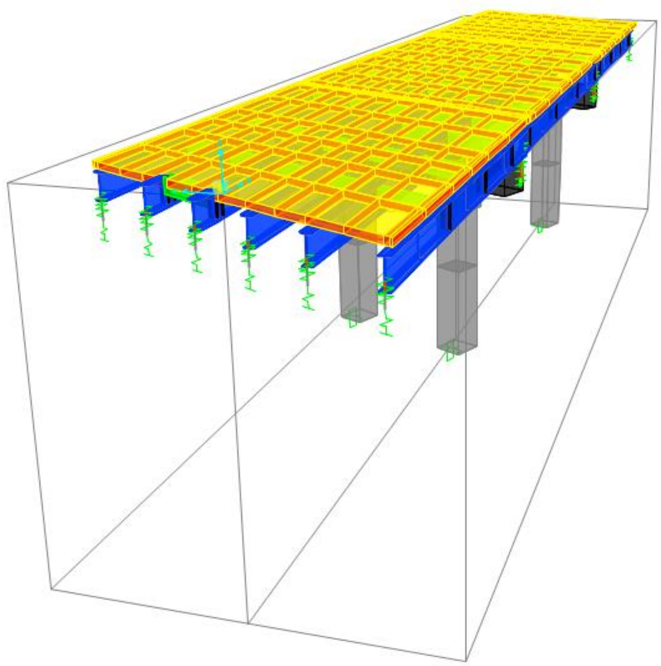

Figure 4.17 Finite Element Model of the bridge

Figure 4.18 Loading truck on the one lane

Figure 4.19 Two loading truck side-by-side

The heavier load was not allowed to travel through the city to the location of bridge. The analytical estimations indicated that none of these loads would result in initiating plasticity in steel girders.

The boom truck positioned on the path under the bridge was used to reach the girders to install testing equipment, as shown in figure 4.20. Wireless strain transducers (the same as described in the first two tests) were attached to the bottom flanges of girders at the midspan and one foot from the supports. Transducers were also attached to the upper flanges one foot away from the supports on the three internal girders, on the more loaded external girder on both supports, and on two interior girders at the midspan.

Figure 4.20 Installation of transducers

Strains, especially at the midspan of girders, were monitored during all passes of the load versus analytically predetermined values, resulting in the reaching of the end of elastic limit (including the strains from the dead load not measured during the tests). All measured strains were not close to strains initiating plasticity in girders. The FE model was calibrated based on tests results. The figures below show the measured moments in the midspan of all girders and moments from the calibrated FE model. Figure 4.21 presents maximum midspan moments on all six girders computed based on measured strains and from the calibrated bridge model. The calibration of the FE model was performed by adjusting support conditions accounting for restrictions to rotations, which were mostly caused by corrosion of girders and bearings. It can be noticed that there is a good agreement between measured and model computed moments.

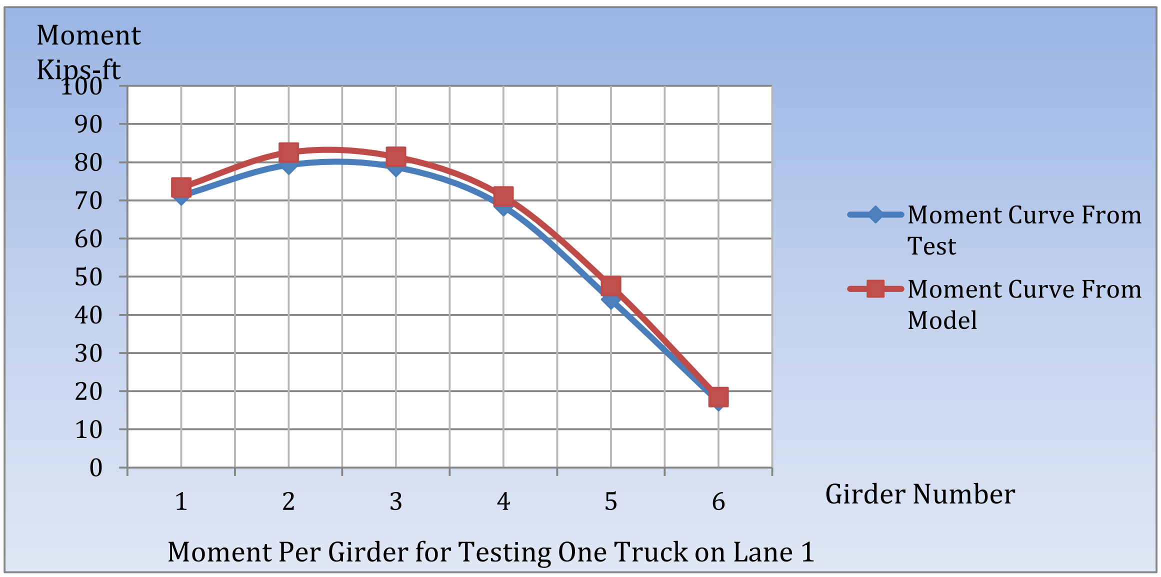

Figure 4.21 Measured and FE model computed maximum midspan moments for one truck positioned on one lane

Values of the girder distribution factor (GDF) for this position of the testing truck are shown in figure 4.22.

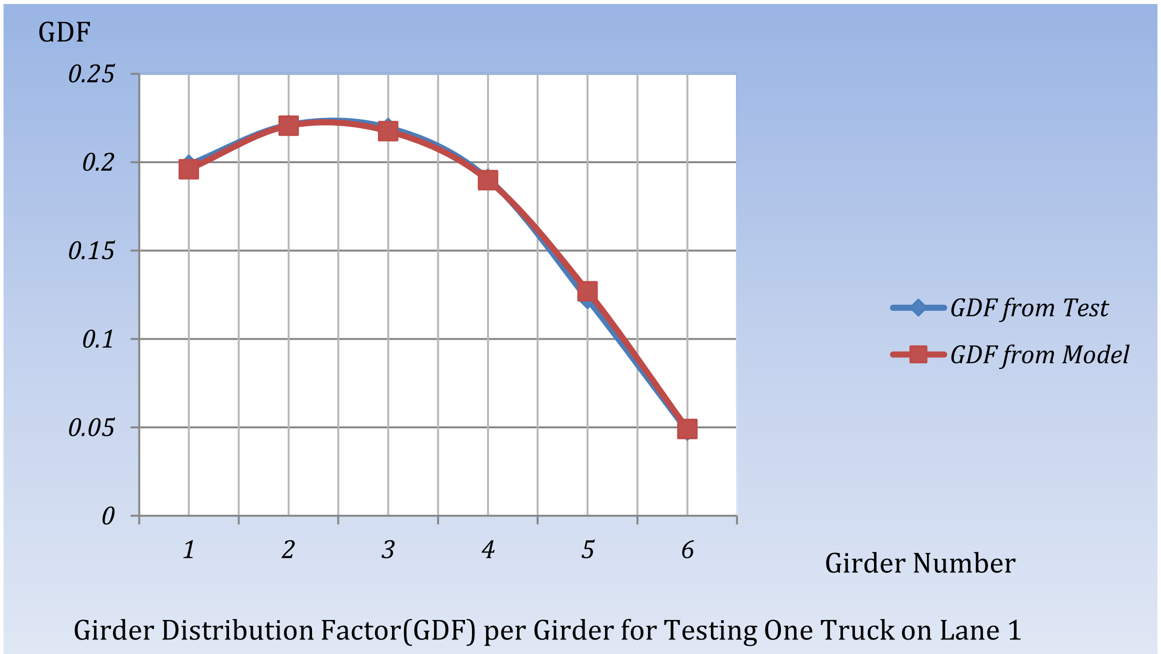

Figure 4.22 GDF distribution for one truck positioned on one lane

Larger moments on girders resulted from using two loading trucks with total load of 102.24 kips driving parallel on two lanes, as shown in figure 4.23.

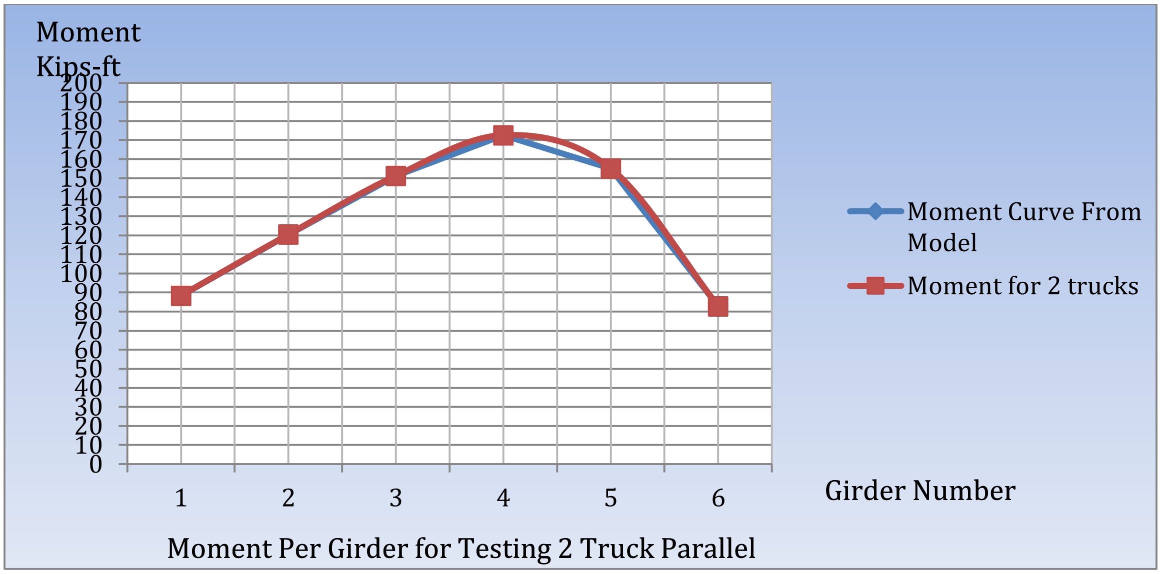

Figure 4.23 Measured and FE model computed maximum midspan moments for two trucks parallel on two lanes

Finally, the CAT excavator weighting 87.85 kips was positioned in the middle of the roadway; the resulting maximum moments at the midspan of girders are presented in figure 4.24.

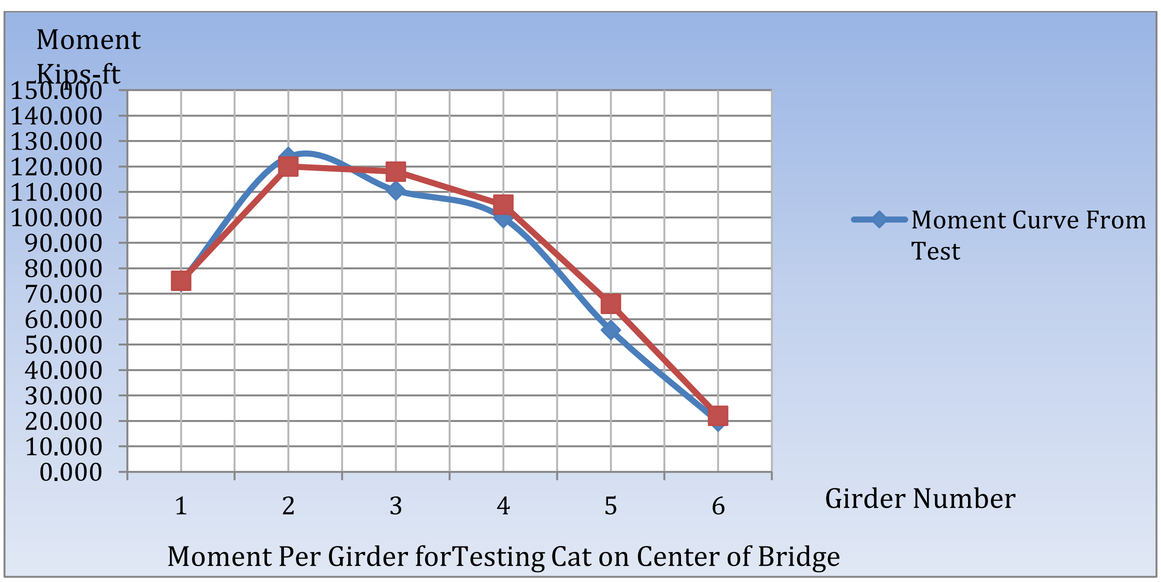

Figure 4.24 Maximum midspan moments for CAT excavator

Dead load maximum span moments for girders were evaluated based on the calibrated FE model, as shown in figure 4.25.

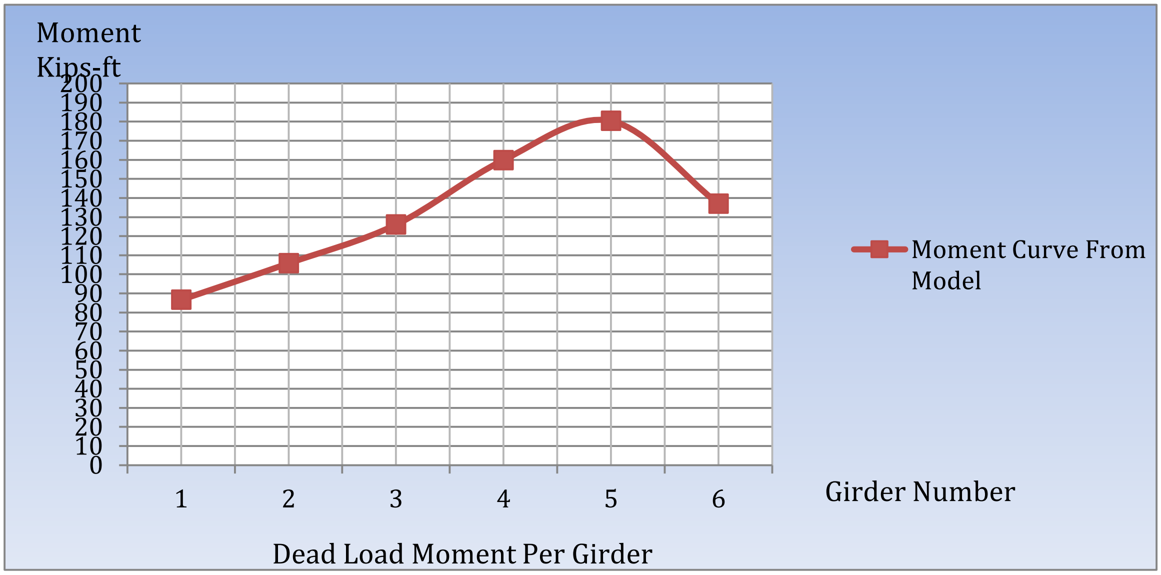

Figure 4.25 Maximum dead load moments per girder

Total maximum moments per girder resulted from two tracks parallel on two lanes and dead load together are shown in figure 4.26.

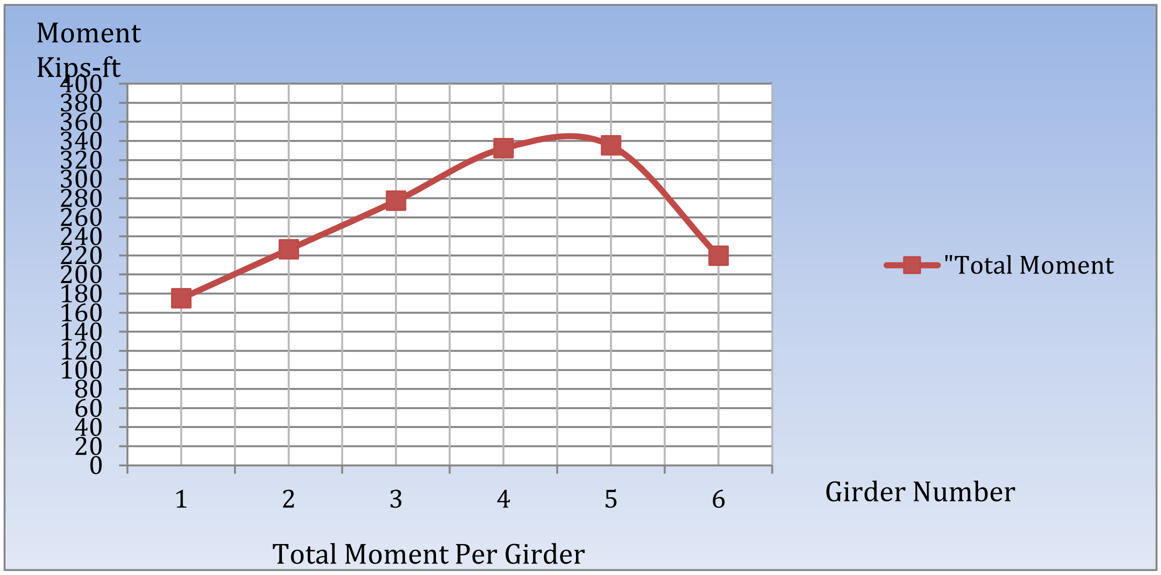

Figure 4.26 Total maximum moments at midspan per girder

Test results show that the total strains in the most loaded girder (number 5) estimated for dead load and measured for live load, which were equal to 360 x 10^{-6} (microstrains), were only about 30% of yielding strains for girders (1138 x 10^{-6}). It needs to be noticed that total strains (from dead load and live load) would be slightly higher since the dead load strains were evaluated based on the full cross-section of girders and in reality they were corroded. However, it can be stated that total strains in girders were not more than 50% of yielding strains. The maximum used testing load of 102.24 kips (two parallel trucks) was 2.3 times larger than analytically estimated 44.4 kips as the allowed load (based on LRF rating), and it did not start a plastic state in the girders. Results of these proof load tests show that analytical evaluations by hand calculations or commercial rating programs are overly conservative. It comes from the difficulty of properly estimating how bridge superstructure shears the load. Also, the stiffening influence of railing, curbs, haunches, and restrictions on supports cannot be estimated in theoretical analysis. The proof load test results can provide an answer about the real capacity of a bridge.

Chapter 5 Conclusions

A review of AASHTO Bridge Committee works resulted in finding changes in Live Load Factors for routine commercial traffic. The refined values are based on changes in target reliability factor. When the new factors are implemented in analytical bridge evaluation, they will reduce the number of bridges not passing the LRFD rating for these loads while still satisfying the target level of reliability.

It was found that live load distribution factors (GDFs) are in reality different (smaller) when compared to those computed from the AASHTO formula. This formula was prepared for girder spacing larger/equal to 3.5 ft and gives higher distribution factors for closer spaced girders; lower values were computed based on diagnostic tests results. Rating factors, both inventory and operating, were substantially higher when evaluated based on measured strains (converted to loading moments) for each girder. Values from the FE model calibrated on test data were similar to those from tests (this was basis for model calibration). Finite element modeling is a very helpful method of structure analysis, however, a model needs to be calibrated using test results, otherwise the model will present the general way the load is sheared by bridge components, but the results will not be accurate. The load results are usually higher because of ideal supporting conditions assumed for the model and possible partial composite action between girders and deck that is not assumed in the model.

Finally, proof load test results indicated that the real capacity of even an old, deteriorated and corroded bridge was much higher than was analytically predicted. It shows the real safety margin with which bridges were designed and constructed, especially the older ones. Proof load test results reflect the actual (can be deteriorated) status of the structure while theoretical analysis and ratings need to rely on assumptions, which can be subjective. Proof load testing can be a good and proven method to evaluate the types of bridges in question, and can allow for a special (higher) permit load or for removing posting.

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