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ILLINOIS-SPECIFIC LIVE-LOAD FACTORS BASED ON TRUCK DATA

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A report of the findings of

ICT PROJECT R27-171
Refinement of Load Factors for Illinois-Specific Load and Resistance Factored Rating (LRFR) Bridge Load Rating Using Weigh-in-Motion (WIM) Data

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14. Abstract

This research project has a focus on the load and resistance factored rating (LRFR) live-load factors for load rating bridges in Illinois. The study's objectives were to examine the adequacy of available Illinois weigh-in-motion (WIM) data and to develop refined live-load factors for Illinois LRFR practice, based on recorded truck loads in Illinois.

There are currently 20 operating WIM sites in Illinois, each next to a weigh station. Initially, only one WIM site was providing two lanes of truck-weight data simultaneously recorded, while the remaining 19 were collecting data for the driving lane only. Two-lane WIM data are important for live-load factor refinement because it is the cluster events involving trucks in different lanes that induce maximum load effects in primary bridge components such as girders. Thus, such data are critical to live-load factors. Upon recommendation from this project, the capability of passing-lane recording was promptly added to two more of the 20 sites. An additional effort was made in this study to simulate the passing lane's data for the remaining 17 sites, to maximize the use of Illinois-relevant WIM data for covering the entire state. This simulation used the probability of multiple trucks in a cluster, based on WIM data from eight states including Illinois. It also used truck-weight-demography information and headway distances of trucks in cluster from all available Illinois sites. This simulation method was tested and proven in the present project to be reliable for calibration here for Illinois.

The resulting truck records of these 17 sites and those recorded at the other 3 sites capable of providing two lanes of truck-weight data from 2013 to 2017 were then used to develop refined live-load factors for LRFR in Illinois. Illinois trucks are seen in these WIM data to be less severe than those weighed in Canada, which were used in calibrating the current AASHTO *LRFD Bridge Design Specifications* (*BDS*) (2017). Illinois trucks recorded in the WIM data were also found to have behaved with little or no influence from the nearby weigh station. Four load-rating cases are addressed in this project in calibrating LRFR live-load factors for Illinois: design load, legal load, routine-permit load, and special-permit load. Based on calibration using Illinois truck-weight records, no change for the design load rating is recommended. Lower live-load factors are recommended for the other three cases for Illinois than those prescribed in the current *MBE,* by about 8% to 14%, depending on average daily truck traffic (ADTT). Illustrative examples using the recommended live-load factors have been prepared and presented in this report.

It is also recommended that Illinois Department of Transportation (IDOT) continue to keep the WIM stations well-maintained, including periodical calibration of the weight sensors and systems; gather more truck-weight-data; review them at least biennially; and focus on possible growth of truck load in both magnitude and volume. When funding becomes available, passing-lane recording is recommended to be added to those WIM sites that currently do not have this capability. Truck-data gathering is also recommended for sites where congested truck traffic is often observed, given adequate funding for such facilities.

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EXECUTIVE SUMMARY

This research project has a focus on the load and resistance factored rating (LRFR) live-load factors for load rating bridges in Illinois. The study's objectives were to examine the adequacy of available Illinois weigh-in-motion (WIM) data and to develop refined live-load factors for Illinois LRFR practice, based on recorded truck loads in Illinois.

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CONTENTS

CHAPTER 1: INTRODUCTION AND BACKGROUND

US highway bridge design and evaluation have been moving towards structural-reliability-based practice with the mandated AASHTO load-and-resistance-factor (LRFD) and load-and-resistance-factor-rating (LRFR) specifications (Fu 2013b). The states also have entered a new era of accordingly adjusting their practice and actively implementing jurisdiction-specific practice advocated therein. This research project is timely to that end. Nationwide progress of this movement is highlighted below, with a more detailed review presented in next chapter along with discussion on a number of remaining issues to be addressed.

The US specifications for highway bridge design and evaluation based on truck-weight data started with NCHRP Project 12-33 (Nowak 1999) and its update work (Kulicki *et al.* 2007). This work represents a remarkable milestone for the new era of structural-reliability-based bridge design and evaluation.

The structural reliability levels associated with AASHTO's calibrated live-load factors are different for bridge design and evaluation specifications. It is higher for design than for evaluation. This difference has included consideration of a balance between tolerable uncertainty and required cost. The planned design life is 75 years. The time period focused on for evaluation has been 5 years for reliability assessment in calibration.

The 75-year design life involves a higher uncertainty than the 5-year horizon for evaluation, hence the higher target reliability index of 3.5. This target was established as the average of reliability level embedded in the previous bridge design practice, which also had variation over span length that has been significantly reduced via *LRFD* calibration. In addition, this higher reliability target beta costs much less in new construction to add load-carrying capacity than to strengthen an existing structure by the same amount. For example, research has shown that an additional 25% higher strength in bridge components costs about 2% to 3% more for new bridge construction (Fu and van de Lindt 2006). For existing bridges, this 25% additional capacity could cost significantly more, depending on the material type. For concrete primary beams, for example, this additional cost can be prohibitively high so that bridge replacement would be more economical compared to strengthening. This consideration leads to the strategy of a lower target reliability index 2.5 for bridge evaluation, so that the number of bridges and/or their components that would be rated below standard and required to be strengthened or replaced would not be excessive.

Furthermore, the above two target reliability indices are embedded in the national specifications, intended to be applicable to all roadway bridges in the country. For a region, such as a state or a county, the truck loading can be very different from the national level (*e.g.*, Fu and van de Lindt 2006). The AASHTO *Manual for Bridge Evaluation* (*MBE*) (2018) thus allows the use of local load information to be used in adjusting the live-load factors. This study is accordingly to develop live-load factors for the state of Illinois using weigh-in-motion (WIM) records gathered from roads within the state.

Even before the LRFD and LRFR specifications, efforts had been made to develop such local load models and/or live-load factors (Fu and Moses 1991; Fu and Hag-Elsafi 1997, 2000). They have contributed to LRFR development, as discussed and referenced in NCHRP Report 454 (Moses 2001). In particular, Fu and Hag-Elsafi (1997) based on New York overweight truck data was cited there as the basis for permit live-load factors in the national specifications because no truck weight were used in NCHRP Report 454 (Moses 2001).

After the AASHTO LRFD and LRFR specifications were first issued in 1994 and 2003, respectively, additional research efforts have been developing new load factors according to the locally experienced truck loads. These efforts are listed here chronologically: (1) design live-load factors (Fu and van de Lindt 2006) and rating live-load factors (Curtis and Till 2008) for Michigan Department of Transportation (DOT); (2) design live-load factors for specific sites (Nassif *et al.* 2008; Mertz 2008); (3) rating live-load factors for Oregon DOT (Pelphrey *et al.* 2008 ; Kinney and Higgins 2009); (4) recalibrated AASHTO LRFR live-load factors for permit load (NCHRP 20-07/285, Sivakumar and Ghosn 2011); (5) LRFR live-load factors for New York State DOT (Ghosn *et al.* 2011); (6) LRFD live-load factors for Missouri DOT (Kwon *et al.* 2011); (7) LRFR live-load factors for Alabama DOT (Uddin *et al.* 2011). There is also at least one more project in the same direction to be completed: Caltrans' LRFD and LRFR specifications for permits and fatigue truck loads (Fu 2012, 2013a). More details about these efforts are presented below in Chapter 4 as part of a review of the state of the art.

Note that although the federal bridge formula (FBF) has been used as a federally applicable criterion for determining whether a vehicle is legal or otherwise overweight, states as bridge owners may have their own definition of legal weights for commercial vehicles. Namely, permits may have different definitions in different jurisdictions. This situation has also contributed to the justification for different live-load factors, depending on the bridge owner.

CHAPTER 2: REVIEW OF THE STATE OF THE ART AND PRACTICE

2.1 SURVEY

A nationwide survey was conducted in this project to understand the status of developing jurisdiction-specific live-load models and/or factors for bridge design and/or evaluation. A simplified approach was used in designing the questionnaire for this survey. It included only two questions, to minimize the effort in providing answers and thus to maximize the response rate. The questionnaire was sent to 49 state transportation departments, excluding Illinois. A total of 40 states responded, a response rate of 82%.

The first question asked if the agency had funded and/or conducted any study of developing LRFR live-load factors for the jurisdiction, whether for the entire state, a district, *etc.* Eight of those responding states answered *yes* and provided either the study report or contact information to acquire a copy of the report or further information, except one reported an ongoing study. The review of these projects/reports is presented in the next section.

The second question was about the time-stamp resolution in the agency's available WIM data. A 0.01-second resolution is required for acceptable statistics used in calibration about the probability of two trucks in a cluster and in different lanes on the bridge span. Using a 70-mph speed as an example, 0.01 second of time is translated to 1.02 ft of distance traveled. This resolution allows structural analysis to distinctively quantify the load effects of the two trucks to a bridge span. Many states use a 1-second time stamp in their WIM data, which translates to about 102 ft of travel distance. Due to rounding off in recording, this lower resolution can make two trucks not on the same span be recorded as both on the span (with the two different time stamps rounded to the same second). Vice versa, two trucks on the same span may be identified as not on the same span due to rounding off to two different seconds. Both cases cause significant error in the resulting load effect, moment or shear.

Nineteen states reported having 0.01-second or higher resolution, out of 33 that responded to this question. Later on, when followed up for providing 0.01-second or higher resolution WIM data, a number of them corrected their answer because their data actually did not have resolution this high. Gathered WIM data were used to address inadequacy in Illinois WIM data, as discussed below in Chapters 4 and 5.

2.2 LITERATURE REVIEW

New Your State DOT Study 1997

Fu and Hag-Elsafi (1997, 2000) conducted the earliest research effort developing state-specific live-load factors for overweight permit trucks, using truck-weight data and reliability-based calibration. Available New York permit truck-weight data were used to develop the live-load factors for annual and trip permits. It is important to note that it was this effort that historically introduced the concept of lower live-load factors for heavier permit trucks. This concept was then adopted in the AASHTO

LRFR specifications calibrated in NCHRP Report 454 (Moses 2001), adopted in the national specifications in 2003, and further carried over to current *MBE* (2018). Fu and Hag-Elsafi (1997) was also the first time when real permit loads were treated separately to derive load-factors for permit checking. This concept has been used in another project for AASHTO NCHRP 20-07/285 (Sivakumar and Ghosn 2011). The report of this New York State DOT research project (Fu and Hag-Elsafi 1997) was cited in the LRFR calibration report NCHRP Report 454 (Moses 2001).

NCHRP Projects 12-46 and 20-07/285

Moses conducted calibration of live-load factors for load rating, as documented in NCHRP Report 454 (Moses 2001). Although live-load factors for permit loads were also included in the LRFR code then, such recommendation was based rather on judgement and reference to earlier experiences in Fu and Hag-Elsafi (1997). NCHRP Report 454 reported the original calibration for the AASHTO LRFR specifications, based on the same Canadian truck-weight data used in the calibration of the AASHTO LRFD specifications (Nowak 1999; Kulicki *et al.* 2007). Therefore, the same assumptions had to be used. They included but were not limited to the side-by-side probability of 1/15 that was later shown to be excessively over-conservative by WIM measurement data in NCHRP Project 12-63 (Sivakumar *et al.* 2007). A simple model was used in this calibration in NCHRP Report 454, including two independent random variables, each representing one lane of truck-weight (not its load effect). Several years later, NCHRP 20-07/285 (Sivakumar and Ghosn 2011) was established to recalibrate the LRFR specifications for permit rating.

Note also that the approach to recalibration in NCHRP 20-07/285 (Sivakumar and Ghosn 2011) is different from that in NCHRP Report 454 (Moses 2001). This approach used WIM data with the permit load separated as done in Fu and Hag-Elsafi (1997, 2000). It also used a very different maximum value projection (to predict 5-year future maximum load) from those in NCHRP Report 454 (Moses 2001) and the LRFD calibration (Nowak 1999; Kulicki *et al.* 2007). This approach's main concepts are based on the protocols recommended by NCHRP Project 12-76 (Sivakumar *et al.* 2008, 2011), to be discussed below, although the protocols are for bridge design, not explicitly for evaluation (load rating). This change of calibration method also highlights the limitations of those original calibration methods for the AASHTO LRFD and LRFR specifications.

Michigan DOT Study

Fu and van de Lindt (2006) completed the second effort in the country to calibrate state-specific live-load factors, after the first one for New York State DOT (Fu and Hag-Elsafi 1997, 2000). This Michigan DOT study targeted bridge design, while the first one was on bridge load rating for permit loads. WIM data were available for this Michigan DOT project, with the occupied lane recorded. They were compared with the Canadian truck-weight data used in the calibration of the AASHTO LRFD specifications. Based on calibration, a 25% increase in the design load factor was recommended for the Metro Region in Michigan where severe truck loads were recorded. It was also estimated that the induced incremental cost for new bridges is only about 2% to 3% of the total bridge cost and thus a much lower percentage of the state's entire construction cost.

NCHRP Project 12-63

NCHRP Project 12-63 is another project relevant to the present one in understanding how heavy trucks may simultaneously appear on a bridge span, referred to here as *cluster appearing.* However, in the literature, the phrase "side-by-side" has been used for this phenomenon but without explicit definition. The words "side-by-side" appear to refer to the loading situation when two trucks in different lanes have their headway distance equal to zero. However, this situation of side-by-side has been rarely, or never, recorded in WIM data. The real situation of concern is when two or more trucks are in a cluster simultaneously on the same span and with small headway-distance from one another. This subject is focused on here because cluster appearing represents the critical loading for strength limit states in design and evaluation of bridge spans.

NCHRP Report 575 for NCHRP Project 12-63 (Sivakumar *et al.* 2007) documented measured data collected from highways by Fu, one of the co-authors of the report and the principal investigator of the present project. WIM data with 0.01-second time-stamp resolution was gathered and analyzed for the first time in history. The data are critical in understanding the truck load occurrences in cluster. Highways in Idaho, Michigan, and Ohio were specially instrumented to acquire such data. However, the so-called side-by-side loading with zero headway distance was never recoded because the time-stamp resolution was 0.01 second; and no two time stamps of heavy trucks were ever identical. In other words, no two tucks ever arrived at the same cross section of a bridge span at the same time, up to the resolution of 0.01 second. The multiple-presence data then were arranged in terms of headway distance to quantitatively describe the real behavior, as discussed next.

Table 15 in NCHRP Report 575 prepared by Fu shows a typical example of the measurement result for one of the three sites in Michigan. It shows that if headway less than 5 ft is defined as a "side-by-side" occurrence, its probability is averaged at only 0.045% for an average daily truck traffic (ADTT) of 4,214. This value is negligible compared with the 1/15 (6.7%) value used in the original calibration for LRFD *Bridge Design Specifications (BDS)* for a maximum ADTT of 5,000. Further, if headway of less than 15 ft is accepted as a "side-by-side" occurrence, then an averaged 0.10% probability is observed. Moreover, if the acceptable headway is increased to 60 ft, as in the most generous case (also conservative due to an overestimated load effect), an averaged 2.11% probability was observed, still much lower than 1/15, or 6.7%.

Note that this 60-ft headway should not be accepted as the "side-by-side" occurrence for all spans and all load effects. For example, when the first truck is in the mid-span area of a simple span inducing a maximum moment, a second truck with a 60-ft headway will be off the span if the span length is 90 ft or shorter. Thus, the second truck contributes nothing to the total mid-span moment; and therefore, the so-called side-by-side configuration does not form at all. The "generous" 60-ft headway in NCHRP Report 575's Table 15 was used to make a point that the 1/15 side-by-side probability was an obvious overestimate, while extremely conservative. It was not meant to be the definition for the so called "side-by-side" configuration.

Nevertheless, this 60-ft headway since then has been misused, unfortunately, as the definition of side-by-side loading in many reports (*e.g.*, Sivakumar *et al.* 2008, 2011; Nassif *et al.* 2008; Sivakumar 2010). Note also that the Idaho and Ohio data obtained in NCHRP Project 12-63 have shown the same behavior as in Table 15 (Sivakumar *et al.* 2007) for the Michigan site on US-23.

NCHRP Project 12-76

NCHRP Project 12-76 (Sivakumar *et al.* 2008, 2011) had an objective to develop a set of protocols for calibrating live-load factors for bridge design using truck-weight data. The subject is relevant to the present study, although load rating was outside its scope. The recommended protocols have made important progress from the very first calibration effort for the AASHTO LRFD specifications. On the other hand, a number of features of the protocols are still not based on conclusive research but on judgment. Some of the observed major issues are commented on below, while other details still remain to be addressed.

Live-load models and live-load factors are required to cover at least two subjects that the bridge design or evaluation engineer faces to address live load: (1) spatial arrangement of the load to induce the maximum possible load effect and (2) temporal projection to address credible future maximum load effect over the intended time span, 75 years for design (Nowak 1999; Kulicki *et al.* 2007) and 5 years for load rating (Moses 2001).

In practice, truck load's spatial arrangement is addressed by the so-called multiple-presence factor (and then the lateral-distribution factor or structural analysis to design individual components) (Fu *et al.* 2013). The temporal projection is addressed by providing live-load factors. For the fatigue limit state, the latter is relatively less critical because it is the repeated routine cyclic load, not the maximum load that is believed to control failure, according to the assumed damage mechanism described in Miner's law. Traditionally, the live-load model (*e.g.,* the permit load in LRFR) is set forth to reasonably approximate the corresponding load for the most commonly seen trucks on the conservative side (overestimating). Then, the live-load factor is made to appear to cover uncertainty associated with individual vehicles and over the intended time span (75 or 5 years, as mentioned above, respectively, for design and evaluation). Modifying the multiple-presence factors (and the live-load-distribution factors) in current AASHTO LRFD specifications was not a focus for NCHRP Project 12-76 (Sivakumar *et al.* 2008, 2011). Rather, its recommended protocols attempt to provide a procedure for calibration using WIM data, focusing on the live-load-factor.

There are two versions of the final report for NCHRP 12-76 (Sivakumar *et al.* 2008, 2011). The differences between the two have not been well explained. Some of these differences are very important to the current study, such as how to estimate/project future maximum load effects based on limited WIM data of a year or two. These issues actually have been studied by Fu and You (2009, 2011). The findings and experiences can be used to advantage in the present project to advance the state of the art and the practice.

Schuyler Heim Bridge Replacement and State Route 47 Extension Study

Nassif *et al.* (2008) reported an effort of recommending load factors for bridges in the Schuyler Heim Bridge Replacement and State Route 47 Extension project. As a result, for spans longer than 60 ft, an adjusted design live-load factor of 2.15 was recommended for the Strength I limit state. For shorter spans, the adjusted load factor of 2.65 was recommended. When future load and traffic growth was considered, an additional increase of 11% in the live-load factor was recommended. These proposed load factors are significantly higher than the value of 1.75 in the AASHTO LRFD specifications. Nevertheless, the derivation arriving at these recommendations included obvious errors in calculation for the projected future-maximum load effect (Fu 2012). This effort used the future-maxima projection method in the calibration of the AASHTO LRFD specifications (Nowak 1999), using extension of the normal probability paper plot. This approach was later excluded from the protocols recommended by NCHRP Project 12-76 (Sivakumar *et al.* 2008, 2011). These observed issues warn us that refinement for live-load factors based on WIM data is not as simple as filling in a formula or a graph. Thorough understanding of the aspects involved is critical, including but not limited to legislation on truck-weight limits, associated implementation procedures, mathematical estimation and projection methods, developments of these methods and their associated issues, to mention a few.

Oklahoma DOT Study

The Oklahoma DOT was implementing a statewide load-rating program for in-service bridges using the LRFR methodology. The objective of the project was to define state-specific live loads and/or load factors using recent truck-weight data collected from both Interstate and non-Interstate WIM sites in Oklahoma for use with the LRFR methodology (Sivakumar 2011). WIM data from nine sites were used in this study, with the longest history of 640 days and the shortest 59 days. The overall average was 478 days. The approach of NCHRP 12-76 (Sivakumar *et al.* 2011) was used for calibration. It was recommended that for Interstate highways, the three AASHTO legal trucks (Types, 3, 3S2, and 3-3) be used for load rating. For state routes, these vehicles were recommended for load rating: Types 3S2 and 3-3, SU4, SU5, and SU6. With these recommendations, no change to the AASHTO live-load factors was needed, as recommended in the report. It is interesting to note that the AASHTO live-load factors for legal load rating have been decreased since then. In addition, the relative calibration recommended by Moses (2001) was used in the study. This approach will be used in present study as presented below in Chapter 3 Calibration Approach.

Gerald Desmond Bridge Replacement Study

Mertz (2008) recommended site-specific live-load factors for the design of the Gerald Desmond Bridge Replacement Project. The projected increases in truck traffic were attributed for the multiple-presence-factors (MPFs) to increase by 5%, but with no supporting analysis presented in the report. MPF's effect in design or evaluation actually is the same as the live-load factor because it is a multiplication factor to the live-load effect in proportioning or evaluating a bridge member. More progress in the state of the art and the practice regarding MPF can be found in Fu *et al.* (2013) as a result of NCHRP Project 12-81 (Bowman *et al.* 2012). Mertz (2008) also mentioned that the recommendation was based on Monte Carlo simulation, however without providing any details about the simulation. The Monte Carlo simulation method needs to be applied with care (Fu 1987, 1994; Fu and Moses 1987). Unfortunately, errors have been commonly observed in using Monte Carlo simulation, which is elaborated next.

A code-calibration problem usually involves more than one random variable. For example, the resistance R, dead-load effects DC and DW, and live-load effect (LL) are often the basic random variables in the problem. The dynamic impact factor and load-distribution factor are often also treated as additional random variables in the same problem. For estimating statistics such as the future-maximum live-load effect's mean and variance, the random variables involved may include truck gross weight, axle configuration, axle weights, headway distance between two trucks, *etc.*

However, no matter how many variables are used in Monte Carlo simulation to solve the problem, there is only one single pseudo-random-number generator in the computer software to generate all samples of these random variables assumed to be independent of one another. Nevertheless, tests have shown that this assumption is often untrue (Fu 2012). These samples are then used to compute the failure probability or the required statistical parameters such as the mean and variance of the maximum load effect. For the former, the failure probability is estimated as the ratio between the numbers of the failed and the total cases computed. For the latter, a number of maximum values of interest are generated for the given future and then their mean and variance are computed as the estimated results. Note that when another simulation is performed using new pseudo-random samples generated by the computer, the estimate result will change. Therefore, more such simulations need to be performed to reach a stable final solution/answer for the problem.
In almost all Monte Carlo simulations presented in the literature for bridge specification calibration, the commonly observed issue of these pseudo-random samples being correlated to one another has never been studied. This issue stems from the fact that all these samples are generated from one single generator (Fu 1994).

New York State DOT Study 2011
Ghosn *et al.* (2011) conducted a project for New York State DOT to recommend state-specific live-load factors for load rating. Five sites of WIM data were used in the study, but no further information was given as to where and why these five sites were chosen. It is known though that there were many more sites of WIM data available in New York State. The final report also includes no information on how the WIM data were analyzed specifically for this project, as to how future-maximum load effects were extracted/predicted or how the multiple-presence factor was determined, *etc.* It is important that these details are documented so that implementation can be pursued with adequate justification. In addition, future adjustment to the live-load factors, when needed, can be performed with a good foundation.

Missouri DOT Study

Kwon *et al.* (2011) conducted a study for Missouri DOT to calibrate the state's live-load factor for the Strength I limit state of LRFD, although LRFR was not within its scope. Twenty-four stations or sites of WIM data were used in the study. "It was found that most representative bridges in Missouri have reliability indices β slightly lower than the target 3.5, mainly due to the adopted projection method to predict 75 year load." (Kwon *et al.* 2011, cover page, abstract) This statement refers to the method of extension from the normal probability paper plot used in the original calibration for the AASHTO LRFD and LRFR specifications. This method was found to be not reliable in the study. Note that the method used about 10,000 Canadian trucks over a period of 2 weeks, as truck-weight data were not as readily available as today.

It is of importance that Moses noted in NCHRP Report 454 (2001, p. 15) the following, reporting calibration for the AASHTO bridge evaluation specifications: "Other parts of the Ontario data that should be kept in mind in considering the accuracy of load projections are as follows:

- The data recorded is a 2-week sample. Any other 2-week sample would have a different outcome because of statistical variability and also seasonal influences on truck movements.
- Heavy trucks avoid static weigh stations, and the degree to which this avoidance occurred in the recorded sampling is unknown.
- Truck weights have changed over time. A repeat of the Ontario trial recently, some 20 years after the first weighings, showed increased truck weights in terms of the maximum bridge loadings (Ontario General Report, 1997)."

Alabama DOT Study

Uddin *et al.* (2011) conducted another study on live-load factors for Alabama DOT LRFR, based on six sites of WIM data within the state. Two years of WIM data were used. This analysis of WIM data in Alabama resulted in lower live-load factors being recommended for each of the six sites than those presented in the LRFR *Manual for Bridge Evaluation.* It is recommended that Alabama DOT consider using these lower live-load factors to more accurately represent the load rating of bridges across the state. It should be noted that the calibration approach in NCHRP Report 454 (Moses 2001) was used in this Alabama study. The work of NCHRP 12-76 (Sivakumar *et al.* 2008, 2011) was not cited in the report. That work excluded the calibration and maximum load projection methods used in calibration of *BDS* and *MBE*, along with the Canadian truck-weight data. At that time, WIM data were not used.

Louisiana DOTD Study

This project sponsored by the Louisiana Department of Transportation and Development (LADOTD) was to verify the adequacy of LADOTD's LRFD design load, LRFR rating and posting procedures, and permit rating methods utilizing recent Louisiana WIM data and reliability methods. One goal was to ensure that the design, rating, and permit procedures provide acceptable structural reliability levels for Louisiana traffic. This study was conducted by Sivakumar in association with Ghosn (HNTB 2016).

Louisiana WIM data were used from ten permanent and three temporary sites, and were between 2007 and 2012. All permanent sites were located on Interstates (I-10, I-12, and I-20), and the three temporary sites were located on state routes (LA-1 SB, US-84, and US-61). Calibration of live-load models was performed following the NCHRP 12-76 protocols (Sivakumar *et al.* 2008, 2011).

For bridge design load, the study recommended LADOTD use a modification factor from 1.15 to 1.45 depending on span length to be applied to the design load moment. It is because the LADV-11 design live-load model does not meet the target reliability criteria (β = 3.5) in some of the span ranges and certain load effects for Strength I loads. For legal load rating for one-lane bridges with width less than 18 ft, an increased live-load factor between 1.65 and 2.00 was recommended, compared with current *MBE*. However, for bridges of two or more lanes, no change from the current *MBE* was recommended. For permit load rating, an increase in the load model was also recommended for single-lane loading by including an additional lane load of 200lb/ft but a decrease in the live-load factor by 0.10. For multiple-lane loading, a uniform reduction by 0.10 from those in *MBE* was recommended in the live-load factor.

Alabama DOT Study 2017

This study was identified by Alabama DOT, when responding to our survey early in the present study, as an ongoing research effort. It was then completed in 2017 prior to the present project's conclusion (Iatsko and Nowak 2017). Although live load factors for bridge design and evaluation were outside its scope, it had a focus on Alabama truck records using the WIM technology. The objective of the study was to review available WIM data for Alabama and assess the degree of damage in highway bridges, depending on traffic volume (ADTT) and weight of heavy vehicles. The WIM database for Alabama included 97 million vehicles. After filtering to eliminate vehicles lighter than 20 kips and questionable records, data were reduced to 57 million. The collected records were provided from 13 WIM stations and covered 9 years (2006 to 2014).

It was observed that traffic load is strongly site-specific. On average, about 10% of all recorded vehicles are heavier than 80 kips. The percentage of overweight vehicles is less than 0.1% for most locations. It was confirmed that for each WIM location, it is possible to pinpoint which types of vehicles make a significant contribution to bridge damage. A load model was developed for the state of Alabama based on extrapolation of the upper tail of the probability distributions of moment and shear.

Caltrans Study

California DOT (Caltrans) has a research project, LRFD & LRFR Specifications for Permits & Fatigue Truck Loads, yet to be completed (Fu 2012, 2013a). Its objective is to develop statewide LRFD and LRFR live-load models and factors for permit, fatigue, and wheel loads for respectively relevant bridge components; 117 sites of WIM data are being used. The duration of data available for a site varies from 3 to 11 years, and all data are included in the analysis. The sites have been ranked according to load severity in order to determine which ones are to be included in the calibrations for which case. Preliminary work has been reported in interim reports to Caltrans (Fu 2012, 2013a).

CHAPTER 3: CALIBRATION APPROACH

3.1 RELATIVE CALIBRATION

Calibration of the load and resistance factor rating (LRFR) in the AASHTO *Manual for Bridge Evaluation (MBE)* was performed using a concept of relative calibration (Moses 2001). This approach focuses on the live load relevant to the live-load factor, without changing the parts involving dead load, resistance, *etc.* The same concept is proposed to be used here for calibrating the live-load factors for Illinois truck loads as follows

$$\frac{\gamma_L LE_n}{\overline{LE}} = \frac{\gamma_{L,ref} LE_{n,ref}}{\overline{LE}_{ref}} \tag{3-1}$$

In general, the right-hand side of the above equation refers to an existing case as reference and the left-hand side to the case of interest for which the live-load factor is being sought. Accordingly, γ_L is the live-load factor for Illinois loads to be determined using this calibration process; and $\gamma_{L,ref}$ is a known live factor for a reference case. For example, γ_L can be the live-load factor for Illinois legal loads' span moment to be developed and recommended from this study; and $\gamma_{L,ref}$ is the current AASHTO *MBE* live-load factor for legal load rating and for the same load effect of span moment.

Furthermore, LE_n in Eq. 3-1 is the nominal load effect for the case of interest. $LE_{n,ref}$ is the nominal load effect for the corresponding reference case. For the same above example of Illinois legal load, LE_n is then the bridge span's (spatial) maximum moment of the Illinois legal load trucks. $LE_{n,ref}$ then is the (spatial) maximum load effect of the reference case, AASHTO legal loads (Types 3, 3S2-2, and 3-3, and the special hauling vehicles).

The phrase "spatial" maximum above is used to contrast with "temporal" maximum to be discussed next. The latter refers to time projection to a future maximum load effect, while the former refers to the maximum load effect in a bridge member with the vehicle crossing the span such as the maximum moment of a simple span in its mid-span area. The temporal maximum is based on time projection of the spatial maxima. For the same example of Illinois legal load's bending moment above, the spatial maximum moment is expected to occur in the mid-span area of a simple primary beam. The temporal maximum is this spatial maximum moment's future maximum based on statistical projection to the 5-year future for load rating. In other words, the temporal maximum is actually a double maximum: temporal maximum of the spatial maxima. Note that this projection has to be performed on a statistical basis; as such, a sample of the spatial maximum values is needed. WIM data are to provide these values for this temporal projection.

Further, $\overline{LE}$ in Eq. 3-1 is the mean value of the (temporal) maximum live-load effect projected to the 5-year future for the loads of interest, based on WIM-measured vehicles' (spatial) maximum live-load effects. $\overline{LE_{ref}}$ in Eq. 3-1 is correspondingly the mean value of the (temporal) maximum load effect projected to the 5-year future for the reference case, also based on measured vehicles. This 5-year horizon has been consistently used in previous studies of live-load factor calibration for load rating (Moses 2001; Fu and You 2009). For the same example of Illinois legal load for bending moment above, $\overline{LE}$ is the mean value of the 5-year maximum moment in the same primary beam of the bridge span. $\overline{LE_{ref}}$ is the mean value of the 5-year maximum moment of the same beam of bridge span for legal load vehicles under the AASHTO definition. Both $\overline{LE}$ and $\overline{LE_{ref}}$ are to be obtained from WIM data.

The left-hand side of Eq. 3-1 accounts for the safety margin for Illinois loads' case, and the right-hand side for the reference loads' case. Namely, Eq. 3-1 requires the same reliability level by maintaining the safety margins at the same level for both sides of the equation. It can be rewritten as follows to explicitly show how the recommended live-load factor for Illinois load rating can be found:

$$\gamma_L = \frac{\gamma_{L,ref} LE_{n,ref}}{\overline{LE}_{ref}} \frac{\overline{LE}}{LE_n} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}} \tag{3-2}$$

Eq. 3-2 shows that the Illinois live-load factor is derived as a product of the referenced live-load factor and two ratios of the load effects. The first is the ratio of the deterministic, or nominal, load effects, which are calculated using live-load models for bridge load rating. For the earlier example of legal load rating for bending moment, this is the moment ratio between the notional models of the AASHTO legal load models to those of Illinois. The second ratio in Eq. 3-2 is the ratio of the means of 5-year future maximum load effects, which are to be obtained by projecting the maxima using WIM data. The process of temporal projection is presented below in Section 3.2.

Note that this relative calibration approach was used by Moses (2001) in calibrating the live-load factors for load rating. The results were adopted by AASHTO in *MBE*. Thus, it is proposed here to be consistent. It is also rational to focus on the live-load here for calibration because all other items in load rating remain unchanged between the case of interest and the reference case, such as the load-carrying capacity of the bridge component or the level of deterioration to the capacity if any.

3.2 SPATIAL MAXIMUM LOAD EFFECTS OF TRUCKS BASED ON WIM RECORDS

A stream of trucks observed crossing a bridge span can be viewed as a series of truck clusters, as recorded in the WIM data set. If a truck crosses a bridge span without any other trucks simultaneously being on the span, it still can be treated as a cluster that has just one truck in it. This truck's or this cluster of trucks' maximum spatial load effect is then computed here using the bridge span's influence line, treating the span as a beam. This process is also referred to as girder-line analysis in the literature.

When the cluster has more than one truck, the analysis will be more involved but still straightforward. Each truck's contribution to the total load effect is found according to its position on the span (*i.e.*, according to its position on the span's influence line) and then is superimposed onto the total. For the basic time period defined (a month in this study, considering available data duration of about 4 years), all clusters' maximum load effects are compared to find this time period's maximum (*i.e*., monthly maximum). Along with other maxima found in other basic time periods (other different months' maxima), the future maximum value's statistics (mean and standard deviation) are found, as discussed below in Section 3.3.

As can be seen, this truck-by-truck approach does not require any assumption about every two trucks' correlation, the probability of clustering, every two trucks' headway distance, *etc*., if the WIM site provides adequate information on all lanes of trucks. In that, this approach is different from those in many previous studies reported in the literature, relying on various assumptions. This method realizes the maximized use of available WIM data. The computer program implementing this algorithm was developed and successfully used in previous studies (Fu and You 2009, 2011).

3.3 TEMPORAL PROJECTION FOR FUTURE MAXIMUM LOAD EFFECTS

Projection for temporal maximum value starts from a basic period of time for which sample data are available. For the case of projecting for temporal maximum moment using WIM data, for example, this basic period can be a week, a month, a quarter, a year, depending on availability of WIM data. Note that the available data need to provide samples to allow statistically significant extrapolation for estimating their mean and standard deviation.

For an example of Illinois vehicles for the span bending moment, if a month is selected as the basic period, a total of 12 months of data results in 12 monthly maximum moments for a bridge span, allowing estimation for the monthly maximum's mean μ_1 and standard deviation σ_1. The subscript 1 here designates one basic period of time. For this example of a basic period of one month, μ_1 and σ_1 are for one month. If the basic period is one-quarter or one year, accordingly, μ_1 and σ_1 are for one-quarter or one year, respectively.

Apparently, the more WIM data available, the longer the basic period can be; and the more reliable a projection can be accomplished. It is equivalent to saying that the more past behavior data are available, the better a prediction or projection for future behavior can be exercised, based on the available behavior data.

Using the theory of statistical projection (Fu and You 2011), this basic period's maximum value can be modeled as an Extreme I (maximum) random variable. Its maximum value of N periods in the future is also an Extreme I random variable. It has the following mean μ_N and standard deviation σ_N, for N basic periods:

$$\mu_N = \mu_1 + \frac{Ln(N)}{\pi}\sqrt{6}\sigma_1 \quad (3\text{-}3)$$

$$\sigma_N = \sigma_1 \quad (3\text{-}4)$$

It can be seen in Eq. 3-3 that the future maximum value's mean μ_N increases from the basic period's maximum value's mean μ_1, as a function of number of time periods N to the future and the basic period's standard deviation σ_1. In other words, the more uncertain the basic period's maximum value (the larger σ_1), the larger the future maximum's mean or the future maximum will be. In other words, the more remote the future is (the larger N) the larger the future maximum will be. Eq. 3-3 shows that the future maximum value's variation (standard deviation) σ_N remains as the maximum value's variation (standard deviation) σ_1 over the basic period. In other words, the future variation does not diminish.

As stated earlier, the Illinois load-rating live-load factor calibration herein will consistently use the 5-year future, as used for other cases of load rating currently included in the *MBE*. Depending on what basic period is to be selected, N will then be accordingly determined for 5 years. For example, if the basic period is selected as one month, 5 years will consist of N=60 basic periods (5 years times 12 months per year). If the basic period is selected as one quarter, 5 years will be N=20 basic periods (5 years times 4 quarters per year). Again, the length of the basic period depends on the availability of WIM data.

CHAPTER 4: WIM DATA COLLECTED FROM ILLINOIS SITES

4.1 ILLINOIS WIM SITES AND AVAILABLE WIM DATA

Illinois Department of Transportation (IDOT) has established WIM stations or sites to gather truck-weight data for various purposes and applications. Figure 4.1-1 shows their locations in the state, along with their respective identifications. The circles or ellipses indicate the stations or sites that provided data for this study. More details about these sites are provided in Table 4.1-1.

Note that these stations or sites are also used in prescreening for weight enforcement. As such, the WIM scale is installed ahead of the adjacent weigh station, so that a truck can be weighed using WIM first and then guided into the weigh station for stationary weighing when further inspection or a double check is needed. Figure 4.1-2 shows a computer screen in a WIM station that reads the WIM data from sensors in the pavement. Figure 4.1-3 shows a truck being directed into the weigh station for stationary weighing. Figure 4.1-4 displays the computer screen reading the weight of the truck to the stationary scale.

The Illinois WIM data set used in this project contains approximately four years of records, from 2013 to 2017. They continuously recorded trucks on their axle weights and configurations, along with arrival times and speeds. Twenty sets of WIM data, each containing about 1.4 to 5.7 million trucks, have been obtained from these sites that cover the main truck lines within the state of Illinois.

The Illinois WIM data were delivered to the research team in the Excel format. They were then converted to text format for computer analysis, including simulation, to be discussed in the next chapter. The recorded variables for each truck include the year, month, day, hour, minute, second, millisecond, travel lane, speed, and individual axle weights and axle distances.

The data were first scrubbed before being used for analysis, to exclude errors in measurement and/or recording, as well as those vehicles that are apparently not trucks. The following criteria were used in this process, which was developed based on a combination of experiences from Illinois and other states in dealing with WIM data.

1. Number of axles fewer than 2 or greater than 13
2. Time stamps not in chronological order
3. Trucks' overlapping with each other in the same lane
4. Axle load smaller than 2 kips or greater than 35 kips
5. If the number of axles is 2, gross vehicle weight (GVW) below 8 kips
6. If the number of axles is 3, GVW below 12 kips
7. Axle distance shorter than 2.5 ft
8. Steering axle weight below 4 kips
9. First axle distance under 5 ft

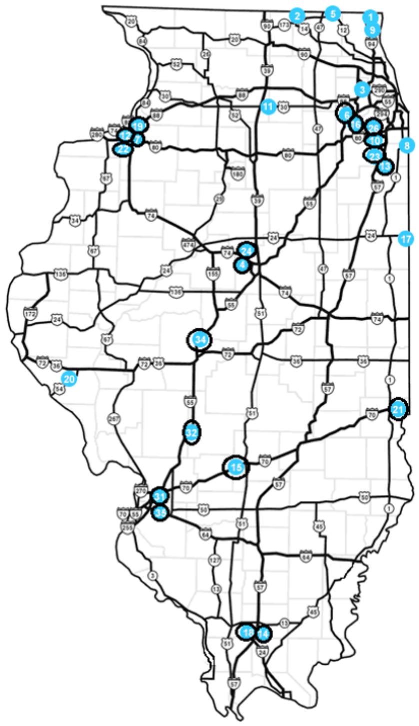

Figure 4.1-1. Locations of WIM sites in Illinois.

Table 4.1-1. Locations and Routes of Illinois WIM Sites

WIM Site ID	Route	Location	Near City
4	I-74	East of Carlock, 122-mile marker	Bloomington
6	I-55	West of IL-53, 265-mile marker	Chicago
7	I-80	East Moline EAST BOUND, 2-mile marker	Davenport
10	I-80	East of US-45, 147-mile marker	Chicago
12	I-74/280	East of US-6, 7.5-mile marker	Davenport
13	I-57	North of Peotone, 330-mile marker	Chicago
14	I-57	South of Marion, 47-mile marker	Marion
15	I-70	East of US-51, 71-mile marker	Springfield
16	I-55	West of IL-43, 267-mile marker South of Marion, 47-mile marker	Chicago
18	I-57	South of Marion, 47-mile marker	Marion
19	I-80	2-mile marker	Davenport
21	I-70	5 miles east of IL-1, 151-mile marker	Terre Haute
22	I-74/280	East of US-6, 5.5-mile marker	Davenport
23	I-57	North of Peotone, 330-mile marker	Chicago
24	I-74	East of Carlock, 122-mile marker	Bloomington
26	I-80	West of US-45, 143-mile marker	Chicago
31	I-55/70	1 mile west of IL-59, 14-mile marker	St. Louis
32	I-55	North of Litchfield, 54.5-mile marker	Springfield
34	I-55	South of Williamsville, 107-mile marker	Springfield
35	I-64	West of IL-158, 18-mile marker	St. Louis

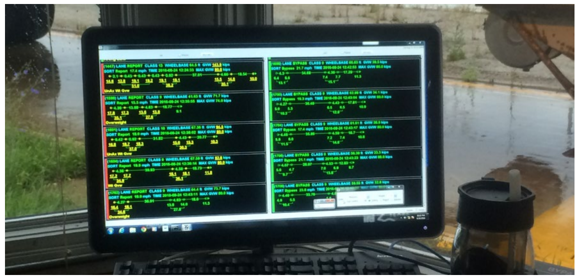

Figure 4.1-2. WIM data of trucks shown on computer screen at Site 16.

Figure 4.1-3. A truck being diverted onto the stationary weigh scale at Site 16.

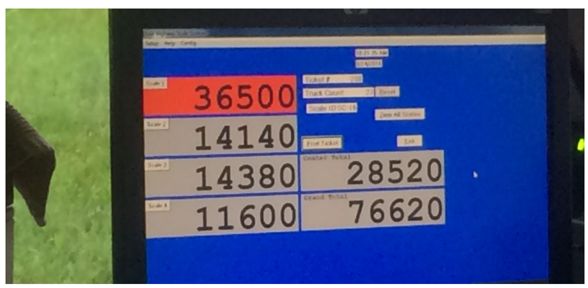

Figure 4.1-4. Truck weights measured by stationary scale at Site 16.

Table 4.1-2 below provides more characteristic information on the 20 Illinois WIM sites used in this study. The fourth column, "Number of Lanes Recorded," indicates the total number of lanes recorded in one traffic direction. If a value "1" is given, only one lane (the driving lane) is recorded there for this two-lane road. If a "2" is given, then both lanes are recorded. Also, as can be seen in this table, these WIM sites continuously recorded for approximately more than four years of data between 2013 and 2017, with the ADTT ranging from about 1,900 to 6,000.

Note that each of the listed ADTT values is based on a real count in the data set over the time period, after the initial data scrubbing discussed earlier. Occasional WIM equipment downtimes, if any, were also subtracted. As such, they are not nominal ADTT and can be reliably used in regression analysis when needed to statistically model truck behavior on the road, such as the probability of trucks in cluster related to ADTT. A *cluster* is defined as a group of trucks with short headway distances among them, thus possibly on the same bridge span, depending on span length.

Table 4.1-2. Characteristic Information on Illinois WIM Sites Providing Data to Present Study

WIM Site ID	Time Period (Month/Year)	Duration (Months)	Number of Lanes Recorded	ADTT
4	3/13 to 5/17	47	1	1944
6	5/13 to 5/17	48	1	3869
7	8/13 to 5/17	39	1	3029
10	5/13 to 5/17	49	1	5641
12	9/13 to 5/17	33	1	1904
13	11/13 to 8/16	31	1	2521
14	5/13 to 5/17	47	1	4842
15	5/13 to 5/17	43	2/1[a]	3982
16	5/13 to 4/17	48	2	4512
18	5/13 to 5/17	39	2/1[a]	5078
19	3/13 to 5/17	48	1	3713
21	1/16 to 12/16	12	1	5060
22	3/13 to 5/17	39	1	2128
23	5/13 to 5/17	49	1	3013
24	3/13 to 5/17	48	1	2011
26	5/13 to 5/17	46	1	6059
31	4/13 to 5/17	49	1	3611
32	5/13 to 5/17	49	1	3063
34	5/13 to 5/17	49	1	3153
35	4/13 to 5/17	50	1	3356

a: Two lanes recorded from 11/16 to 5/17, and one lane recorded for other months

4.2 ILLINOIS WIM DATA QUALITY

As mentioned earlier, each Illinois WIM site is located near a weigh station, which offers an opportunity to study the accuracy of the collected WIM weights by comparing them with those measured by the stationary scale at the weigh station. To that end, 20 trucks passing Site 16 on I-55 near Chicago were randomly selected and diverted onto the stationary scale for weighing. The result then was compared with the WIM-recorded weights. Figures 4.1-3 and 4.1-4 above show one of these trucks being directed into the weight station and the axle weights being read. Tables 4.2-1 to 4.2-3 display the comparisons of the gross, axle, and tandem weights, respectively.

Table 4.2-1. Comparison of Gross Vehicle Weights (GVW) by WIM and Scale

WIM Weight (Kips)	Stationary-Scale Weight (Kips)	Difference
62.30	64.26	-3.05%
34.60	32.84	5.36%
74.90	79.00	-2.66%
50.20	52.32	-4.05%
58.60	59.42	-1.38%
49.20	52.24	-5.82%
78.10	77.94	0.21%

73.80	77.32	-4.55%
28.00	25.84	8.36%
41.90	39.34	4.51%
64.30	70.40	-5.82%
34.40	34.50	5.51%
49.20	53.76	-8.48%
37.50	34.28	9.39%
54.40	52.68	7.06%
20.00	18.30	9.29%
65.90	72.62	-9.25%
37.60	34.30	9.62%
41.40	41.74	-0.81%
69.20	68.66	0.79%

Table 4.2-2. Comparison of Axle Weights by WIM and Scale

WIM Weight (Kips)	Stationary-Scale Weight (Kips)	Difference
11.40	11.50	-0.87%
14.80	14.40	2.78%
14.50	14.22	1.97%
9.80	10.64	-7.89%
14.60	14.20	2.47%
9.50	10.32	-7.95%
7.20	4.76	4.51%
7.80	7.12	9.55%
10.90	12.42	-12.24%
15.00	15.72	-4.58%
13.40	15.36	-12.76%
9.50	10.40	-8.65%
11.20	11.00	1.82%
11.90	11.54	3.12%
12.20	11.36	7.39%
10.40	10.66	-2.44%
7.40	7.40	0.00%
7.40	7.02	5.41%
10.70	11.50	-4.96%
10.10	10.36	-2.51%
10.90	10.46	4.21%
10.90	11.54	-5.55%
8.20	4.80	20.59%
7.90	4.34	24.61%
10.90	11.46	-4.89%
15.20	14.76	2.98%
15.70	14.52	8.13%
11.80	12.22	-3.44%
8.50	7.88	7.87%
8.30	7.80	4.41%
9.40	10.90	-13.76%
4.30	5.10	23.53%
5.60	4.90	14.29%
10.90	11.66	-4.52%
14.70	14.16	3.34%
17.10	15.76	8.50%
11.50	12.28	-4.35%
17.30	17.64	-1.93%
18.70	17.30	8.09%

10.50	10.70	-1.87%
11.30	10.82	4.44%
11.50	10.52	9.32%
9.00	9.32	-3.43%
13.90	12.14	14.50%
13.10	11.80	11.02%
9.80	10.52	-4.84%
11.40	9.18	24.18%
9.20	8.56	7.48%
11.40	11.72	-2.73%
17.30	14.14	7.19%
17.00	14.30	4.29%
11.00	11.04	-0.36%
4.30	4.34	-0.63%
4.90	5.98	15.38%
10.80	11.32	-4.59%
12.90	12.60	2.38%
13.00	12.60	3.17%

Table 4.2-3. Comparison of Tandem Weights by WIM and Scale

WIM Weight (Kips)	Stationary-Scale Weight (Kips)	Difference
25.50	27.74	-8.07%
10.30	9.48	8.65%
31.20	34.84	-10.45%
19.80	24.06	-17.71%
22.60	24.16	-13.61%
15.90	20.20	-21.29%
30.60	30.72	-0.39%
29.10	33.74	-13.75%
4.80	4.94	37.65%
13.20	11.44	15.38%
24.50	29.66	-17.40%
9.50	9.82	-3.26%
17.50	21.44	-18.38%
12.30	9.20	33.70%
21.00	18.78	11.82%
24.60	29.12	-8.65%
13.10	10.10	29.70%
15.00	14.90	0.67%
28.50	28.52	-0.07%

The comparison in Table 4.2-1 shows that the differences in GVW between WIM and the stationary scale are within the range of -9.25% to 9.62%, with a mean of 0.81% and a standard deviation of 4.15%. The axle weights in Table 4.2-2 have the largest differences, of which the mean is 2.58% and standard deviation is 9%. The differences of tandem weights in Table 4.2-3 have a mean of 0.24% and a standard deviation of 18%.

The differences of axle and tandem weights between the two weighing methods are usually larger than those of GVW. This phenomenon has been observed in other studies on WIM data accuracy.

Nevertheless, the mean of differences of the Illinois WIM data are generally within 5%. The above observed differences between WIM and stationary weighing are also within the ranges reported in previous studies.

4.3 POSSIBLE INFLUENCE OF ADJACENT WEIGH STATIONS

Because the Illinois WIM stations are adjacent to weigh stations, there was a concern as to whether the WIM data are biased due to the publicly known weigh stations' being next to the WIM sensors. A study was conducted to address this concern. The WIM data during the weigh station's open hours and closed hours are compared in this study to identify possibly noticeable differences, if any, in order to understand if there were any behavior change of trucks in the two time periods. WIM data from five Illinois WIM sites (Sites 4, 7, 12, 22, and 35 in Tables 4.1-1 and 4.1-2) are used for this study. These sites were selected because their historical open and closed hours could be confirmed for the durations of the data provided. Over the data durations of about 4 years, as identified in Table 4.1-2, there have been numerous changes of schedules for a number of reasons. They included but were not limited to seasonal changes, personnel shift changes, and employee changes, along with their individual working hour changes. Tracking these changes for 4 years or so in the past was very challenging if not impossible.

Accordingly, for the five WIM sites used in this study on possible influence of an adjacent weigh station, the provided truck-weight data for each site are separated into two sets, A and B, according to the data recording hour. Data Set A contains the trucks recorded when the adjacent weigh station was open for operation, and Data Set B when closed. Tables 4.3-1 to 4.3-5 exhibit the results of this comparison between Data Sets A and B, along with their difference in percentage, respectively, for the five selected sites. For both Data Sets A and B, the mean of monthly maximum load effect (mid-span moment or end shear of simple span) is focused on for this comparison. These load effects are of interest for the calibration's temporal projection, as discussed earlier in Section 3.2.

In Tables 4.3-1 to 4.3-5, a negative difference means that the mean of maximum load effect decreased from open hours to closed hours, and a positive difference means it increased. It is interesting to notice that these five sites' comparison results show more negative than positive, except Site 22. At Site 22, the maximum increase is 7.48% for the maximum span moment at the 70-ft span. Its corresponding mean increase for shear for the same span length of 70 ft is 1.98%. Overall, these results indicate that the closed hours do not experience noticeably more severe truck loads, according to the mean values of monthly maximum load effects. While truck-volume density (trucks per hour) is expected to decrease during closed hours, this observation also seems to indicate that there have not been perceived illegally heavy trucks avoiding weighing enforcement or weigh station during closed hours. Note that in Illinois, each truck has been issued a mandatory wireless device for identification so that it can be identified while in motion without the need to be stopped and weighed. This device may have effectively reduced any intention for illegal overweighting.

Table 4.3-1. Comparison of Open (A) and Closed (B) Hours with Gap H=0 (Site 4)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	392.83	398.11	1.34%
	50	816.24	821.20	0.61%
	70	1406.39	1369.25	-2.64%
	100	2484.97	2317.71	-6.73%
	130	3585.10	3313.65	-7.57%
	160	4731.79	4497.46	-4.95%
	190	5936.61	5779.71	-2.64%
	220	7211.49	7110.40	-1.40%
Mean of Maximum Monthly End Shear (Kips)	30	57.50	58.90	2.43%
	50	71.88	72.37	0.68%
	70	86.52	83.76	-3.19%
	100	104.02	100.73	-3.17%
	130	115.17	113.98	-1.03%
	160	124.64	125.28	0.52%
	190	132.36	133.54	0.90%
	220	138.27	140.35	1.50%

Table 4.3-2. Comparison of Open (A) and Closed (B) Hours with Gap H=0 (Site 7)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	383.79	396.62	3.34%
	50	837.68	806.46	-3.73%
	70	1333.82	1284.19	-3.72%
	100	2129.69	2080.66	-2.30%
	130	3095.38	3042.08	-1.72%
	160	4321.38	4273.28	-1.11%
	190	5697.42	5626.16	-1.25%
	220	7109.67	6986.81	-1.73%
Mean of Maximum Monthly End Shear (Kips)	30	59.84	56.72	-5.21%
	50	74.83	71.80	-4.05%
	70	85.10	81.74	-3.95%
	100	97.61	97.30	-0.32%
	130	110.65	114.01	3.03%
	160	123.78	126.65	2.32%
	190	133.62	135.30	1.26%
	220	141.08	141.59	0.36%

Table 4.3-3. Comparison of Open (A) and Closed (B) Hours with Gap H=0 (Site 12)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	395.40	375.86	-4.94%
	50	812.29	763.17	-6.05%
	70	1359.43	1264.94	-6.95%
	100	2327.33	2161.54	-7.12%
	130	3360.93	3199.50	-4.80%
	160	4531.85	4479.95	-1.15%
	190	5900.44	5883.74	-0.28%
	220	7317.80	7330.31	0.17%
Mean of Maximum Monthly End Shear (Kips)	30	57.84	54.93	-5.04%
	50	73.35	68.61	-6.46%
	70	86.92	79.91	-8.07%
	100	100.55	97.21	-3.32%
	130	113.63	114.79	1.02%
	160	127.27	128.73	1.15%
	190	137.68	138.86	0.86%
	220	145.60	146.41	0.56%

Table 4.3-4. Comparison of Open (A) and Closed (B) Hours with Gap H=0 (Site 22)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	399.38	417.24	4.47%
	50	811.44	851.63	4.95%
	70	1331.58	1431.12	7.48%
	100	2243.34	2403.52	7.14%
	130	3343.20	3570.57	6.80%
	160	4641.18	4909.01	5.77%
	190	6070.71	6381.18	5.11%
	220	7504.23	7875.71	4.95%
Mean of Maximum Monthly End Shear (Kips)	30	58.47	61.62	5.40%
	50	73.83	77.25	4.63%
	70	88.27	90.01	1.98%
	100	106.44	107.55	1.04%
	130	122.74	125.05	1.88%
	160	135.36	138.68	2.45%
	190	144.55	148.30	2.60%
	220	151.50	155.69	2.77%

Table 4.3-5. Comparison of Open (A) and Closed (B) Hours with Gap H=0 (Site 35)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	427.93	399.87	-6.56%
	50	850.43	777.65	-8.56%
	70	1361.26	1338.40	-1.68%
	100	2290.90	2325.84	1.52%
	130	3316.30	3417.16	3.04%
	160	4442.66	4559.22	2.62%
	190	5712.88	5747.02	0.60%
	220	7081.55	6963.18	-1.67%
Mean of Maximum Monthly End Shear (Kips)	30	60.55	57.71	-4.68%
	50	72.24	70.90	-1.86%
	70	86.35	85.41	-1.09%
	100	102.81	102.69	-0.11%
	130	114.77	114.75	-0.02%
	160	125.06	123.43	-1.30%
	190	134.63	129.71	-3.65%
	220	141.86	134.44	-5.24%

Furthermore, in real-time operation, the distinction in trucking behavior between a weigh station's open and closed hours may not be exactly synchronized with the weigh station's opening and closing. Namely, there may be a transition period of time in which this distinction develops its momentum. In addition, the real opening and closing of the weigh station may not have been exactly executed as scheduled. This transition period is referred to as H hours hereafter, for studying its effect. In other words, the comparison exhibited in Tables 4.3-1 to 4.3-5 is under an assumption that H=0 hours, without any transition time in trucking-behavior development due to the weigh station's opening or closing. As such, a sensitivity analysis was also conducted here considering H=1 hour, as presented next.

In this sensitivity analysis, the data during the gap of H hours are omitted because it cannot be certain whether the weigh station was indeed open or closed; and/or whether truck drivers had identified the weight station's open or closed status, and the truck traffic accordingly changed behavior in weight compliance, until H hours elapsed. Taking Site 4 as an example, for H=0 hour, Data Set A contains recorded trucks during 6:00–14:00 on weekdays. Data Set B contains data recorded during 0:00–6:00 and 14:00–24:00 on weekdays, as well as 0:00–24:00 on weekend days. For H=1 hour, Set A contains trucks recorded during 7:00–13:00 on weekdays. Set B contains recorded trucks during 0:00–5:00 and 15:00–24:00 on weekdays, as well as 0:00–24:00 on weekend days.

Tables 4.3-6 to 4.3-10 display comparisons between the means of monthly maximum load effects for Data Sets A and B, like Tables 4.3-1 to 4.3-5 but for H=1 hour. Namely, they are for the real open and closed hours when 1 hour of data is neglected around the opening and closing times for both Sets A and B. The "Difference" in these tables is defined in the same way as in Tables 4.3-1 to 4.3-5.

Table 4.3-6. Comparison of Open (A) and Closed (B) Hours with Gap H=1 Hour (Site 4)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	392.57	395.35	0.71%
	50	816.24	818.52	0.28%
	70	1405.81	1359.29	-3.31%
	100	2456.25	2292.10	-6.68%
	130	3534.50	3307.00	-6.44%
	160	4656.79	4494.21	-3.49%
	190	5832.03	5775.44	-0.97%
	220	7056.07	7105.10	0.69%
Mean of Maximum Monthly End Shear (Kips)	30	57.48	58.62	1.99%
	50	71.88	71.78	-0.15%
	70	86.52	83.46	-3.54%
	100	103.72	100.25	-3.35%
	130	114.38	112.88	-1.31%
	160	123.27	124.30	0.84%
	190	130.28	132.70	1.85%
	220	135.53	139.60	3.00%

Table 4.3-7. Comparison of Open (A) and Closed (B) Hours with Gap H=1 Hour (Site 7)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	380.43	396.07	4.11%
	50	837.68	799.03	-4.61%
	70	1333.82	1254.06	-5.98%
	100	2129.50	2055.88	-3.46%
	130	3051.65	2961.38	-2.96%
	160	4182.56	4069.13	-2.71%
	190	5430.20	5233.28	-3.63%
	220	6718.84	6397.43	-4.78%
Mean of Maximum Monthly End Shear (Kips)	30	59.84	56.72	-5.21%
	50	74.83	70.99	-5.13%
	70	85.10	81.74	-3.95%
	100	96.58	96.64	0.06%
	130	108.09	108.39	0.28%
	160	119.34	117.17	-1.81%
	190	127.72	123.18	-3.55%
	220	136.05	127.55	-6.24%

Table 4.3-8. Comparison of Open (A) and Closed (B) Hours with Gap H=1 Hour (Site 12)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	393.67	373.28	-5.18%
	50	796.19	758.91	-4.68%
	70	1292.82	1261.10	-2.45%
	100	2193.15	2100.97	-4.20%
	130	3209.86	3077.46	-4.12%
	160	4394.38	4258.36	-3.10%
	190	5735.35	5593.44	-2.47%
	220	7123.10	7003.67	-1.68%
Mean of Maximum Monthly End Shear (Kips)	30	57.41	54.57	-4.95%
	50	70.38	67.64	-3.90%
	70	82.52	77.96	-5.53%
	100	96.63	93.27	-3.47%
	130	109.94	108.84	-1.00%
	160	123.11	122.59	-0.42%
	190	133.50	132.97	-0.40%
	220	141.27	140.52	-0.53%

Table 4.3-9. Comparison of Open (A) and Closed (B) Hours with Gap H=1 Hour (Site 22)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	399.38	415.90	4.14%
	50	802.46	837.18	4.33%
	70	1300.37	1387.97	6.74%
	100	2210.06	2306.66	4.37%
	130	3294.37	3433.59	4.23%
	160	4545.37	4736.83	4.21%
	190	5935.38	6174.68	4.03%
	220	7327.79	7636.28	4.21%
Mean of Maximum Monthly End Shear (Kips)	30	58.37	60.55	3.74%
	50	72.87	75.58	3.71%
	70	87.97	86.40	-1.79%
	100	105.86	103.94	-1.81%
	130	119.89	121.40	1.26%
	160	131.89	134.68	2.12%
	190	140.26	144.15	2.77%
	220	146.35	151.06	3.22%

Table 4.3-10. Comparison of Open (A) and Closed (B) Hours with Gap H=1 Hour (Site 35)

Load Effect	Span Length (ft)	Data Set A	Data Set B	Difference
Mean of Maximum Monthly Mid-Span Moment (Kip-ft)	30	415.87	387.72	-6.77%
	50	820.50	753.50	-8.17%
	70	1319.68	1265.99	-4.07%
	100	2211.80	2172.11	-1.79%
	130	3225.66	3211.20	-0.45%
	160	4348.64	4321.82	-0.62%
	190	5589.83	5470.08	-2.14%
	220	6871.30	6629.29	-3.52%
Mean of Maximum Monthly End Shear (Kips)	30	58.87	56.20	-4.52%
	50	70.37	68.75	-2.31%
	70	83.69	82.43	-1.51%
	100	99.66	98.86	-0.80%
	130	112.99	110.42	-2.27%
	160	123.20	118.56	-3.77%
	190	132.04	124.65	-5.60%
	220	138.54	129.30	-6.67%

Tables 4.3-1 to 4.3-10 show that the observed increase and decrease in the mean of maxima during and out of weigh station operation hours appear to be random, without a recognizable trend one way or the other. Namely, these random changes are not due to illegal overweight trucks intentionally avoiding the weigh station. Overall, it was therefore concluded that the Illinois WIM stations' truck-weight data are not biased by the presence of a weigh station next to the WIM sensor. Again, the wireless truck-identification device may have positively contributed to this behavior.

4.4 ILLINOIS WIM DATA COMPARED WITH CANADIAN WEIGH STATION DATA

The focus of this study is on the truck load and its associated live-load factors for LRFR practice in the state of Illinois. As such, one of the requirements for this study was to compare the recently recorded truck loads of Illinois with those in Canada in the 1970s that were used to calibrate the live-load factors in the AASHTO LRFD *Bridge Design Specifications (BDS)* and *Manual for Bridge Evaluation (MBE).*

Note that the Canadian truck load data of about 10,000 trucks were acquired at weigh stations in 1975, not using the WIM technology (Nowak 1999). As a result, the data set does not include information on the trucks' behavior in motion, such as occupying which lane and having what headway distances. Thus, the comparison in this section will not address these behaviors related to the trucks in motion. The next section will deal with those subjects as to how two trucks may be related in terms of random weights and positions on a bridge span.

The Canadian data set is equivalent to approximately two weeks of truck traffic. According to Moses (2001), the top 20% of the data was used to develop the live-load factors in *BDS* and *MBE*. They are plotted in Figure B-5 in Nowak (1999), shown on the normal probability paper. The bold extensions there from the data set project to the future maxima up to 75 years as the AASHTO-required bridge life, from about 1 day. They are compared below with corresponding curves of the Illinois WIM truck-weight data. Two sites with the lowest and highest ADTT are shown in Figures 4.4-1 to 4.4-12 below as typical cases. The remaining sites were also plotted and compared. They are included in Appendix A.

In each figure, only one curve is for the Canadian data set, as there is only one set for about two weeks of duration including approximately 10,000 trucks. The Illinois data set includes 20 sites, each having recorded millions of trucks, as summarized in Table 4.1-2. As such, four randomly selected 2-week samples of Illinois trucks from one site are used in each such figure for comparison. The samples are designated as Samples A to D. Note that Samples A to D do not change if they are from the same site when used to compute the load effects for the three different span lengths, but they are randomly different for different sites.

All these comparison figures show that the maximum load effects of Illinois trucks are lower or much lower than those of the Canadian trucks. For example, for Site 12 and 30-ft span end shear in Figure 4.4-2, the Canadian trucks' maximum is about 1.75 of the HL93 design truck, while the Illinois maximum is only about 1.25, or about 29% lower. Such figures for the other Illinois sites and span lengths are included in Appendix A.

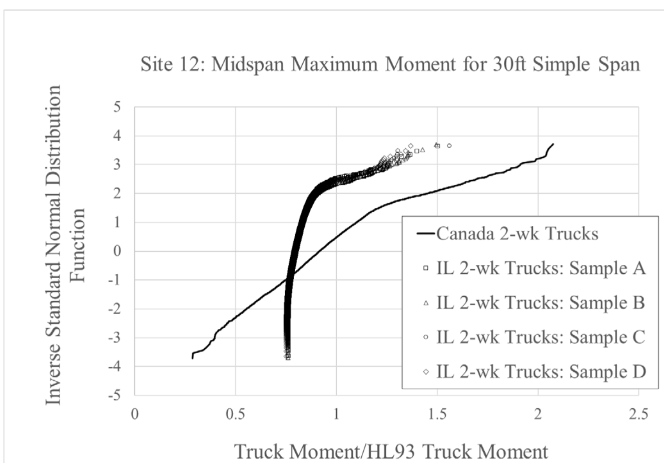

Figure 4.4-1. Moments of Canadian and Illinois trucks at Site 12 for 30-ft span.

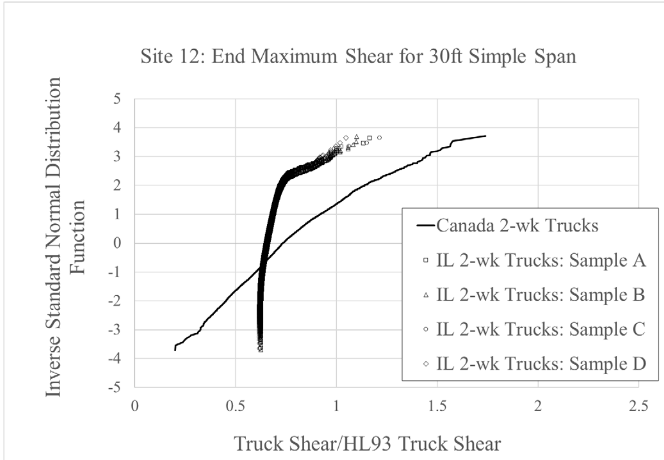

Figure 4.4-2. Shears of Canadian and Illinois trucks at Site 12 for 30-ft span.

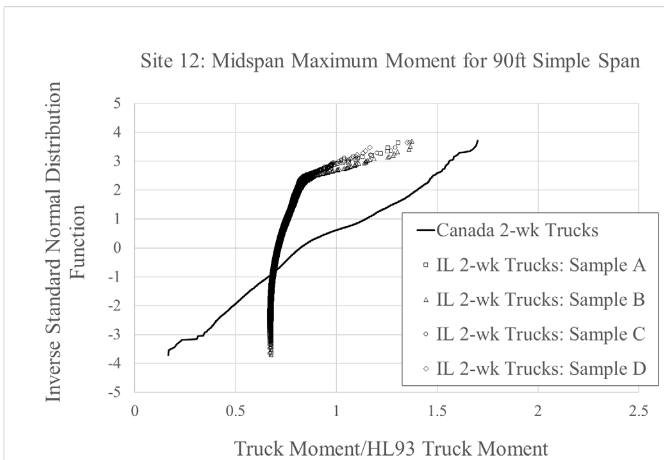

Figure 4.4-3. Moments of Canadian and Illinois trucks at Site 12 for 90-ft span.

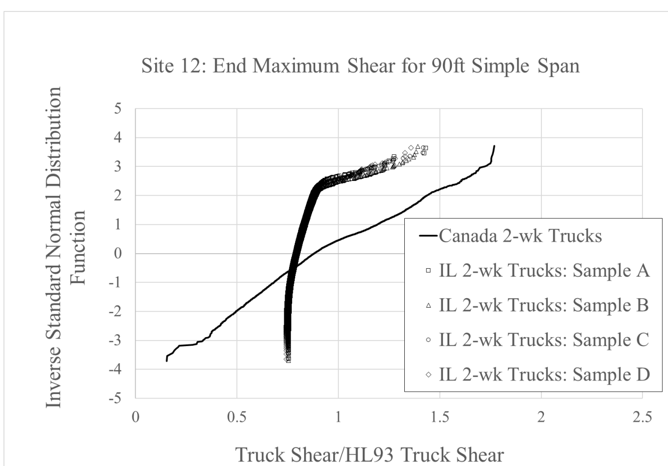

Figure 4.4-4. Shears of Canadian and Illinois trucks at Site 12 for 90-ft span.

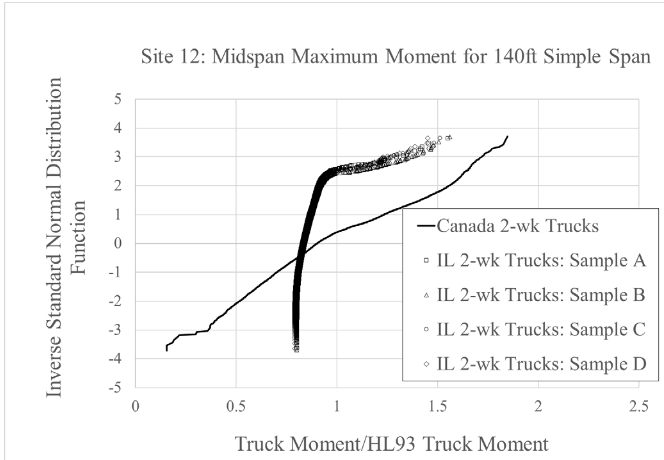

Figure 4.4-5. Moments of Canadian and Illinois trucks at Site 12 for 140-ft span.

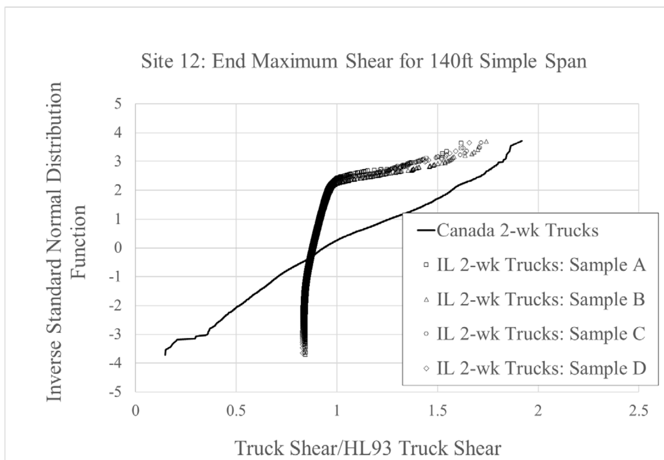

Figure 4.4-6. Shears of Canadian and Illinois trucks at Site 12 for 140-ft span.

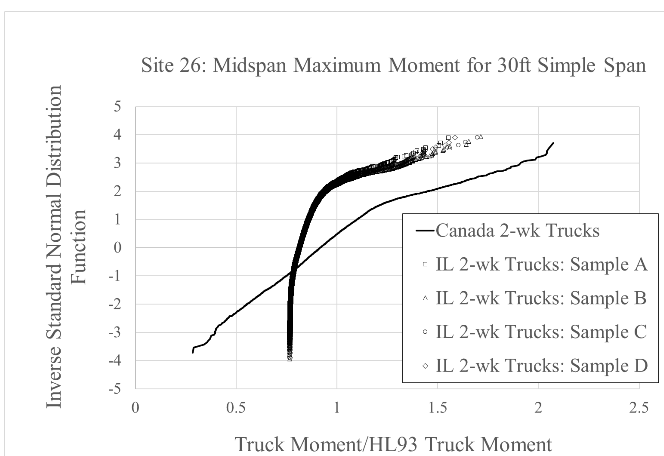

Figure 4.4-7. Moments of Canadian and Illinois trucks at Site 26 for 30-ft span.

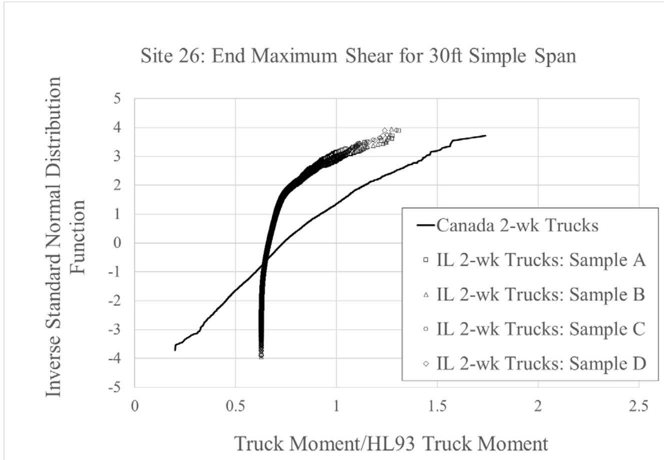

Figure 4.4-8. Shears of Canadian and Illinois trucks at Site 26 for 30-ft span.

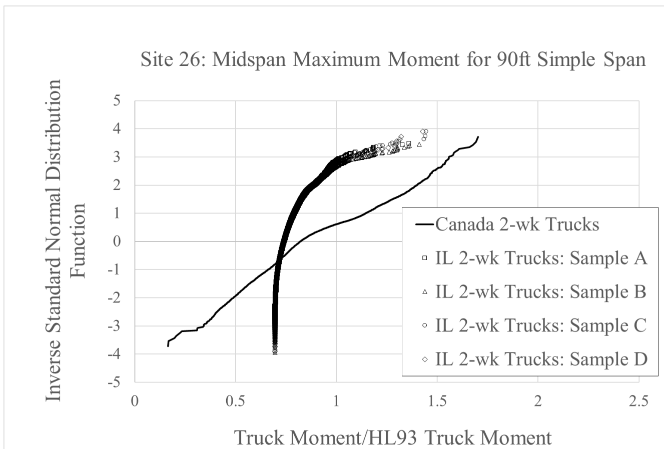

Figure 4.4-9. Moments of Canadian and Illinois trucks at Site 26 for 90-ft span.

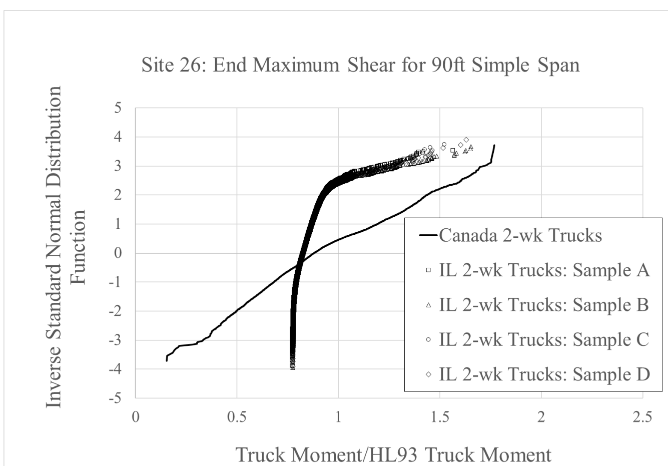

Figure 4.4-10. Shears of Canadian and Illinois trucks at Site 26 for 90-ft span.

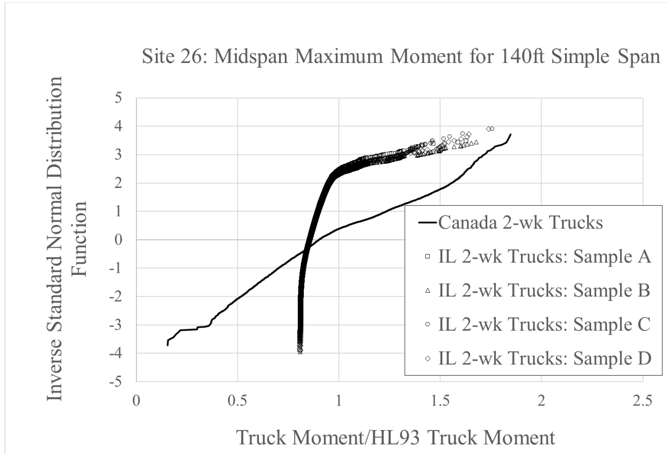

Figure 4.4-11. Moments of Canadian and Illinois trucks at Site 26 for 140-ft span.

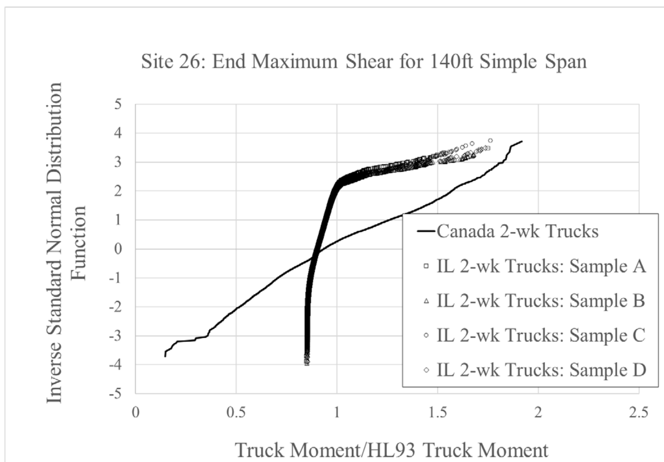

Figure 4.4-12. Shears of Canadian and Illinois Trucks at Site 26 for 140-ft span.

4.5 WIM TRUCK BEHAVIORS AND ASSUMPTIONS IN *BDS* CALIBRATION

Comparison in Section 4.4 focused on individual trucks' gross vehicle weight (GVW) and configuration (axle weights and axle spacings). These parameters dictate truck load effects in bridge components, as used above for comparison between the Illinois WIM-observed and Canadian weigh-station-weighed truck loads. In addition, the available Illinois WIM data allowed comparison of behaviors of trucks in motion with the assumed behaviors used in the calibration of *BDS* as documented in NCHRP Report 368. This section presents the approaches and results towards the requirement of this project on comparing Illinois truck data with the Canadian data set used in *BDS* calibration in the 1990s.

4.5.1 WIM Data from 8 States Including Illinois Used in Study

As mentioned in the Section 2.1 survey, WIM data from other states were gathered in this project. The following states' data are used here for truck-behavior comparison, along with Illinois data: California, Kentucky, Michigan, Minnesota, New York, Oregon, and Pennsylvania. More states' WIM data were received, but their data did not have the time-stamp resolution that could permit headway distance analysis required for traveling-truck behavior relevant to bridge safety. Data from above seven states, besides Illinois data, have time-stamp resolution at or higher than 0.01 second and two lanes simultaneously recorded, along with vehicle speed. These important features allow comparison of recorded truck behaviors on bridges with those relevant assumptions in NCHRP Report 368 about the Canadian trucks weighed in the 1970s and used for *BDS* calibration in the 1990s. It should be emphasized that the calibration did not use WIM data, and thus a number of assumptions were needed to model the critical loading cases to bridge spans. These assumptions are the focus here.

The 0.01-second time-stamp resolution permits identification of headway distance between any two trucks in different lanes to an approximately 1-ft resolution at 70 mph. For contrast, if the time-stamp resolution is 1 second, as many other states use in WIM data collection, this headway-distance resolution becomes about 100 ft. In other words, any real headway distance shorter than 100 ft is rounded to either 0 or 100 ft. When rounded to 0, the two trucks are treated as if their load effects double if they have similar axle weights and spacings. If rounded to 100 ft, the second truck may be off the span if the span length is shorter than about 150 ft depending on which load effect; and its contribution to the total load effect is zero. As such, this rounding can cause the resulting load effect to be so much different between 0 and 100 ft. When a 0.01-second resolution is used, this issue is significantly mitigated to between 0 and 1 ft.

Table 4.5.1-1 below displays these sites from which WIM data are used in this study. In Table 4.5.1-1, the column “Number of Lanes” indicates the total number of available lanes in one traffic direction for which WIM truck data were made available to this project.

Table 4.5.1-1. Information Used for WIM Sites of Eight States

State	WIM Site ID	Time Period (Month/Year)	Duration (Months)	Number of Lanes	ADTT	ADTT in Lane 1
California	CA 002	1/13 to 12/15	36	2	2806	2556
	CA 005	1/13 to 12/15	36	2	4064	3681
	CA 007	1/13 to 12/15	36	2	3522	3180
	CA 020	1/13 to 12/15	36	2	706	606
	CA 022	1/13 to 12/15	36	2	762	701
	CA 025	1/13 to 12/15	36	2	2351	2164
	CA 066	1/13 to 12/15	36	2	3411	3067
	CA 115	1/13 to 12/15	36	2	849	716
Illinois	IL 15	11/16 to 5/17	7	2	3982	3843
	IL 16	5/13 to 4/17	48	2	4512	4368
	IL 18	11/16 to 5/17	7	2	5078	4848
Kentucky	022P47	7/14 to 11/16	26	2	1478	1244
	025P20	4/14 to 6/16	27	2	415	381
	047P07	4/14 to 11/16	32	2	244	176
	056P98	4/14 to 11/16	26	2	2447	1851
	057P65	4/14 to 9/16	30	2	433	356
	120P60	6/14 to 1/16	20	2	355	311
Michigan	MI 2229	1/13 to 12/15	36	2	369	348
	MI 5059	1/13 to 12/15	36	2	1209	1142
	MI 6119	1/13 to 12/15	36	2	1890	1454
	MI 6429	1/13 to 12/15	36	2	690	661
	MI 6449	1/13 to 12/15	36	2	2248	2134
	MI 6469	1/13 to 12/15	36	2	1368	1294

	MI 7159	1/13 to 12/15	36	2	5061	4621
	MI 7219	1/13 to 12/15	36	2	3914	3610
	MI 7269	1/13 to 12/15	36	2	2483	2379
	MI 7319	1/13 to 12/15	36	2	2339	2222
Minnesota	MN 026	12/15 to 8/16	9	2	2242	1985
	MN 029	12/15 to 8/16	9	2	399	363
	MN 030	12/15 to 8/16	9	2	302	271
	MN 037	1/15 to 8/16	16	2	3814	3090
	MN 038	12/15 to 8/16	9	2	1080	872
	MN 040	12/15 to 8/16	9	2	1464	1112
	MN 042	12/15 to 8/16	9	2	730	464
	MN 043	12/15 to 8/16	9	2	614	537
New York	NY 2680	1/13 to 12/15	36	2	251	233
	NY 3311	1/13 to 12/15	36	2	1762	1590
	NY 5183	1/13 to 12/15	36	2	644	605
	NY 5280	1/07 to 12/09	36	2	635	575
	NY 6282	1/13 to 12/15	36	2	175	158
	NY 9121	1/13 to 12/15	36	2	1998	1803
	NY 9580	1/07 to 12/09	36	2	1942	1699
Oregon	N/A	2/06 to 11/07	21	2	4506	4026
Pennsylvania	PA 158	5/15 to 10/16	17	2	5141	4259
	PA 501	5/15 to 10/16	17	2	1454	1344
	PA 502	1/15 to 10/16	22	2	5251	4401

4.5.2 Assumptions for One-Lane Moment and Shear

NCHRP Report 368 states, "On the basis of this limited data it is assumed that, on average, about every 50th truck is followed by another truck with the headway distance less than 100 ft, about every 150th truck is followed by a partially correlated truck, and about every 500th truck is followed by a fully correlated truck (Nowak 1999, p. B-17). It then further defines: "Three degrees of correlation between truck weights are considered: ρ=0 (no correlation), ρ=0.5 (partial correlation) and ρ=1 (full correlation), where ρ is the coefficient of correlation (Nowak 1999, p. B-17)."

Following Probability for Trucks with Headway Distance Shorter Than 100 ft

The headway distance for one-lane load events was defined in Figure B-13 in NCHRP Report 368, as the distance from the rear axle of the truck in the front to the front axle of the following truck.

The driving lane's trucks in these WIM data are used herein to find the probability of the recorded headway distance being shorter than 100 ft, to be compared with its assumed values in NCHRP Report 368. The driving lane has recorded many more trucks, compared with the passing lane, and therefore is used here for a more statistically valid comparison.

The headway distance $H_{one\text{-}lane}$ of every truck pair in the passing lane is computed as follows:

$$H_{one-lane} = (TimeStamp_2 - TimeStamp_1)(\frac{Speed_2 + Speed_1}{2}) - WheelBase_1 \qquad (4.5.2\text{-}1)$$

in which, subscripts 2 and 1, respectively, refer to the later truck and earlier truck in the pair. TimeStamp is the time stamp, and its subscript 2 or 1 respectively identifies the later or earlier truck.

$WheelBase_1$ is the wheel base of the earlier truck (in front), or the distance from its first axle to its last axle.

When $H_{one\text{-}lane}$ is found to be less than 100 ft between any two trucks in the passing lane in the WIM data set, it is then identified as an event meeting the criterion, also referred to as a following-truck event. Such pair identification and headway distance checking repeat until the entire WIM data set is exhausted. The probability of following trucks $P_{following}$ is then accordingly computed as the ratio between the number of following events and the total number of trucks.

$P_{following}$ is computed for all the WIM sites and data sets in Table 4.5.1-1. The results are tabulated in Table 4.5.2-1, in comparison with the assumed value of 2% (1/50) in NCHRP Report 368. Figure 4.5.2-1 below provides this comparison in a graph for a more intuitive review.

Table 4.5.2-1. Following Probabilities Based on WIM Observation and Assumption

WIM Site ID	ADTT in Lane 1	Observed Probability in WIM Data	Assumed Probability in NCHRP 368
CA 002	2556	1.72%	2.00%
CA 005	3681	1.63%	2.00%
CA 007	3180	1.72%	2.00%
CA 020	606	0.39%	2.00%
CA 022	701	0.40%	2.00%
CA 025	2164	0.74%	2.00%
CA 066	3067	1.75%	2.00%
CA 115	716	0.90%	2.00%
IL 15	3843	1.10%	2.00%
IL 16	4368	1.58%	2.00%
IL 18	4848	1.63%	2.00%
KY 022P47	1244	0.47%	2.00%
KY 025P20	381	0.41%	2.00%
KY 047P07	176	0.53%	2.00%
KY 056P98	1851	1.22%	2.00%
KY 057P65	356	0.34%	2.00%
KY 120P60	311	0.42%	2.00%
MI 2229	348	0.11%	2.00%
MI 5059	1142	0.85%	2.00%
MI 6119	1454	0.39%	2.00%
MI 6429	661	0.31%	2.00%
MI 6449	2134	1.35%	2.00%
MI 6469	1294	0.80%	2.00%
MI 7159	4621	2.15%	2.00%
MI 7219	3610	1.73%	2.00%
MI 7269	2379	0.92%	2.00%
MI 7319	2222	0.97%	2.00%
MN 026	1985	0.34%	2.00%
MN 029	363	0.22%	2.00%
MN 030	271	0.27%	2.00%
MN 037	3090	0.98%	2.00%
MN 038	872	0.59%	2.00%
MN 040	1112	1.14%	2.00%
MN 042	464	0.47%	2.00%
MN 043	537	0.15%	2.00%
NY 2680	233	0.32%	2.00%
NY 3311	1590	0.51%	2.00%
NY 5183	605	0.18%	2.00%
NY 5280	575	0.25%	2.00%

NY 6282	158	0.15%	2.00%
NY 9121	1803	0.79%	2.00%
NY 9580	1699	0.78%	2.00%
OR Data	4026	1.87%	2.00%
PA 158	4259	1.58%	2.00%
PA 501	1344	0.61%	2.00%
PA 502	4401	1.78%	2.00%

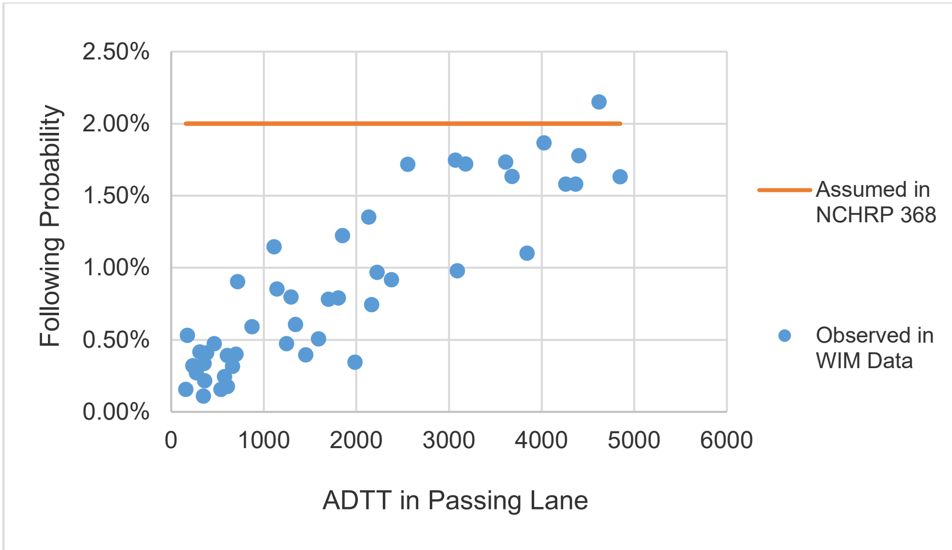

Figure 4.5.2-1. Comparison of observed and assumed following probability ($P_{following}$).

As can be seen in Table 4.5.2-1 and Figure 4.5.2-1, for the sites with driving-lane ADTT below 4,500, the recorded following probabilities are lower or much lower than the assumed 2% in NCHRP Report 368. For those with ADTT below 2,000, this following probability is about 1% or even lower. It is shown clearly that the 2% assumption for headway distance shorter than 100 ft is an overestimate and thus conservative. In Figure 4.5.2-1, this following probability can be seen as a function of $ADTT_{SL}$ (and also total ADTT for both lanes). For sites with ADTT below 1,000, this 2% assumption can be excessively over-conservative.

More critically, the definition of *following probability* here, based on headway distance up to 100 ft, can become irrelevant for bridges with a span length shorter than 100 ft, because the following truck is not on the same span and makes no contribution to the total load effect. There is a very large number of bridge spans in the country that are shorter than 100 ft. Illinois has many as well. For them, two trucks that are 100 ft apart in the same lane are not of concern because one of them will be off the span when the other is on the span; and thus they would not constitute a critical load to shorter spans. In other words, this threshold of headway distance should have been a function of span length to be relevant for shorter bridge spans. Specifically, which truck and trucks may become a critical load needs to be defined with consideration to bridge span length as well.

Using the results in Table 4.5.2-1, a regression relation is found as follows between $P_{following}$ with $ADTT_{SL}$ with an R^2 of 0.79:

$$P_{following} = 3.6 \times 10^{-6} ADTT_{SL} + 0.0021 \qquad (4.5.2\text{-}2)$$

GVW Correlation Coefficient 0.5 for 150th Trucks with Headway Distance Shorter Than 100 ft

For the partial GVW correlation coefficient ρ=0.5 assumed for the 150th trucks and with headway distance less than 100 ft in the same lane, the following approach was used to compare it with recorded corresponding truck behavior. This 150th compared with 50th truck of headway distance shorter than 100 ft is equivalent to one-third of the events used in computing $P_{following}$ above. Thus, one-third of the qualified events for $P_{following}$ were randomly selected; and then, the paired trucks' GVW values were used to compute the correlation coefficient of the sampled truck pairs, one following the other within 100 ft.

To reduce chances of random results, this procedure of randomly selecting one-third was repeated 15 times to obtain a mean of 15 correlation coefficients for each site. Their mean value is reported in Table 4.5.2-2, directly compared with the assumed 0.5 for each WIM site listed there. In order to understand the level of random nature in the reported mean, the 15 samples' standard deviation for each site is also reported in Table 4.5.2-2.

As can be seen in Table 4.5.2-2 and Figure 4.5.2-2, the GVW correlation between two trucks, one following the other within 100 ft, is about a third of the assumed 0.5. The mean values used for this comparison are statistically valid, with a standard deviation at about 10% of the mean. Namely the coefficient of variation is at a level of about 10%.

Table 4.5.2-2. GVW Correlation Coefficients (CC) for 1/3 of Following Truck Events (150th Trucks)

WIM Site ID	ADTT in Lane 1	Mean of Observed CC	Standard Deviation of Observed CC	Assumed CC
CA 002	2556	0.1794	0.0098	0.5000
CA 005	3681	0.0941	0.0098	0.5000
CA 007	3180	0.1096	0.0080	0.5000
CA 025	2164	0.1907	0.0168	0.5000
CA 066	3067	0.1287	0.0065	0.5000
IL 15	3843	0.0993	0.0187	0.5000
IL 16	4368	0.0462	0.0049	0.5000
IL 18	4848	0.1227	0.0134	0.5000
KY 022P47	1244	0.1753	0.0279	0.5000
KY 056P98	1851	0.1407	0.0099	0.5000
MI 5059	1142	0.1489	0.0163	0.5000
MI 6119	1454	0.1575	0.0208	0.5000
MI 6449	2134	0.0948	0.0137	0.5000
MI 6469	1294	0.1439	0.0262	0.5000
MI 7159	4621	0.0569	0.0053	0.5000
MI 7219	3610	0.0963	0.0051	0.5000
MI 7269	2379	0.0898	0.0113	0.5000
MI 7319	2222	0.1316	0.0441	0.5000
MN 026	1985	0.1135	0.0385	0.5000
MN 037	3090	0.0978	0.0143	0.5000
MN 040	1112	0.1438	0.0333	0.5000
NY 3311	1590	0.2074	0.0215	0.5000
NY 9121	1803	0.1566	0.0281	0.5000

NY 9580	1699	0.1415	0.0167	0.5000
OR Data	4026	0.1053	0.0088	0.5000
PA 158	4259	0.1774	0.0081	0.5000
PA 501	1344	0.1634	0.0257	0.5000
PA 502	4401	0.1607	0.0105	0.5000

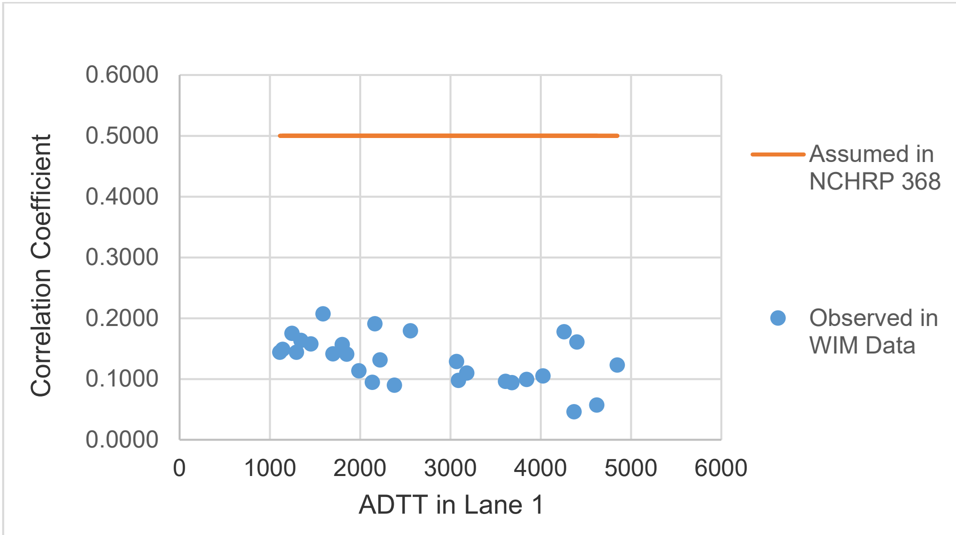

Figure 4.5.2-2. Comparison of observed and assumed GVW correlation coefficients for one-third of following truck events (150th trucks).

GVW Correlation Coefficient 1.0 for 500th Trucks with Headway Distance Shorter Than 100 ft

For the assumed full correlation coefficient ρ=1 for 500th trucks or one-tenth of those following-truck events, the same approach as above was used for comparison with WIM-recorded behavior. The only difference is the change from one-third to one-tenth of the qualified events to be used for calculating the correlation coefficient. Table 4.5.2-3 displays the mean value of 15 random correlation coefficients, along with their standard deviation for each site used.

Table 4.5.2-3. GVW Correlation Coefficients (CC) for 1/10 of Following-Truck Events (500th Trucks)

WIM Site ID	ADTT in Lane 1	Mean of Observed CC	Standard Deviation of Observed CC	Assumed CC
CA 002	2556	0.1786	0.0081	1.0000
CA 005	3681	0.1054	0.0047	1.0000
CA 007	3180	0.1104	0.0112	1.0000
CA 025	2164	0.1677	0.0325	1.0000
CA 066	3067	0.1293	0.0108	1.0000
IL 15	3843	0.1030	0.0256	1.0000
IL 16	4368	0.0474	0.0082	1.0000
IL 18	4848	0.1272	0.0259	1.0000
KY 022P47	1244	0.1846	0.0377	1.0000
KY 056P98	1851	0.1440	0.0282	1.0000
MI 5059	1142	0.1479	0.0310	1.0000
MI 6119	1454	0.1543	0.0537	1.0000
MI 6449	2134	0.0960	0.0202	1.0000
MI 6469	1294	0.1404	0.0414	1.0000
MI 7159	4621	0.0541	0.0096	1.0000
MI 7219	3610	0.0968	0.0132	1.0000
MI 7269	2379	0.0917	0.0214	1.0000

MI 7319	2222	0.1481	0.0271	1.0000
MN 026	1985	0.1277	0.0249	1.0000
MN 037	3090	0.0984	0.0330	1.0000
MN 040	1112	0.1548	0.0547	1.0000
NY 3311	1590	0.2055	0.0323	1.0000
NY 9121	1803	0.1618	0.0152	1.0000
NY 9580	1699	0.1550	0.0262	1.0000
OR Data	4026	0.1077	0.0148	1.0000
PA 158	4259	0.1818	0.0181	1.0000
PA 501	1344	0.1792	0.0476	1.0000
PA 502	4401	0.1680	0.0155	1.0000

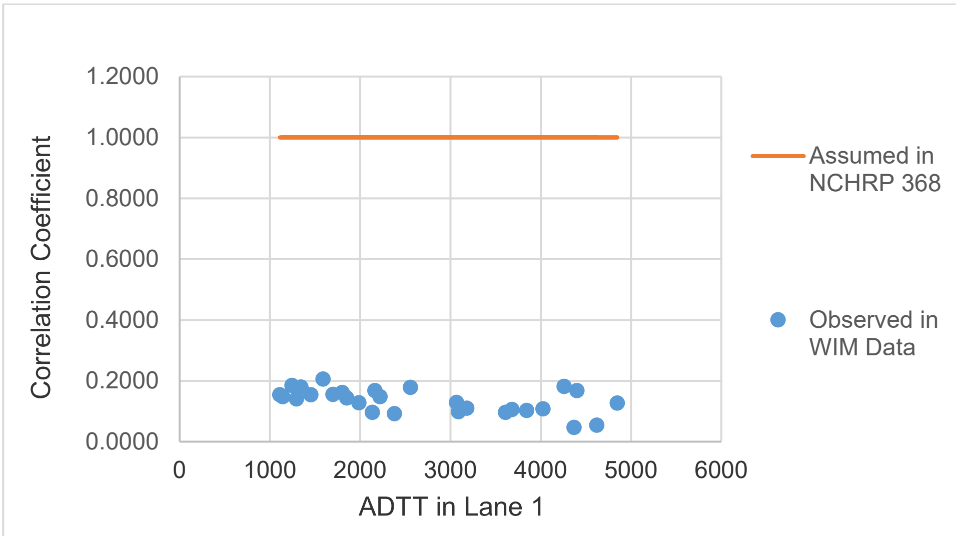

Figure 4.5.2-3. Comparison of observed and assumed GVW correlation coefficients for one-tenth of following-truck events (500th trucks).

Table 4.5.2-3 and Figure 4.5.2-3 show that the GVW correlation coefficients observed in WIM data are much lower than the assumed 1.0 in NCHRP Report 368. In other words, the assumed 1.0 is clearly an overestimate, which is about six to seven times higher than the observed. This overestimation may lead to an over-conservative estimation for the critical loading when two trucks are on the same span with a small headway distance between them.

4.5.3 Assumptions for Two-Lane Moment and Shear

NCHRP Report 368 also states, "It has been observed that, on average, about every 15th truck is on the bridge simultaneously with another truck (side-by-side). For each such simultaneous occurrence, it is assumed that every 10th time the trucks are partially correlated and every 30th time they are fully correlated (with regard to weight)." (Nowak 1999, p. B-25) The event of the side-by-side presence of two trucks in different lanes is not explicitly defined in NCHRP Report 368, particularly with respect to the headway distance between them. Thus, a headway distance between the front axles of earlier and later trucks shorter than 50 ft is used here. This definition was used by Professor Nowak as defined in Figure 2-1 in Nowak *et al.* (1994).

Note that this headway-distance definition is different from that used in Section 4.5.2 earlier for trucks in the same lane. That definition is for two trucks in the same lane, and this one is for being in different lanes. That one measures the headway distance from the earlier truck's rear axle to the later truck's front axle, and this one from the earlier truck's front axle to the later truck's front axle.

The WIM data listed in Table 4.5.1-1 are used to find the side-by-side probability for comparison with the assumed value in NCHRP Report 368. The headway distance $H_{two\text{-}lane}$ defined in Figure 2-1 in Nowak *et al.* (1994) is computed as follows

$$H_{two-lane} = (TimeStamp_2 - TimeStamp_1)(\frac{Speed_2 + Speed_1}{2}) \quad (4.5.3\text{-}1)$$

in which all the items on the right-hand side are identical to those in Eq. 4.5.2-1 earlier, except that $Truck_1$ and $Truck_2$ are in different lanes, while Eq. 4.5.2-1 has them in the same lane. When $H_{two\text{-}lane}$ is 50 ft or shorter, a side-by-side event is positively identified. The total number of such events divided by the total number of trucks in the WIM data set is an estimate for the side-by-side probability. This observed side-by-side probability is compared with the assumed 6.67% (1/15) in Table 4.5.3-1 for the WIM data sets listed in Table 4.5.1-1. Figure 4.5.3-1 offers a graphic comparison, to provide a more intuitive view of the difference between the observed and assumed.

Table 4.5.3-1. Comparison of Assumed and Observed Side-by-Side Probabilities

WIM Site ID	ADTT	Side-by-Side Probability from WIM Data	Side-by-Side Probability in NCHRP 368
CA 002	2806	1.85%	6.67%
CA 005	4064	1.95%	6.67%
CA 007	3522	2.03%	6.67%
CA 020	706	0.73%	6.67%
CA 022	762	0.82%	6.67%
CA 025	2351	1.34%	6.67%
CA 066	3411	2.06%	6.67%
CA 115	849	0.80%	6.67%
IL 015	3982	1.26%	6.67%
IL 016	4512	1.05%	6.67%
IL 018	5078	1.29%	6.67%
KY 022P47	1478	0.91%	6.67%
KY 025P20	415	0.31%	6.67%
KY 047P07	244	0.19%	6.67%
KY 056P98	2447	1.41%	6.67%
KY 057P65	433	0.31%	6.67%
KY 120P60	355	0.43%	6.67%
MI 2229	369	0.12%	6.67%
MI 5059	1209	0.51%	6.67%
MI 6119	1890	1.03%	6.67%
MI 6429	690	0.31%	6.67%
MI 6449	2248	0.67%	6.67%
MI 6469	1368	0.50%	6.67%
MI 7159	5061	1.81%	6.67%
MI 7219	3914	1.53%	6.67%
MI 7269	2483	0.78%	6.67%
MI 7319	2339	1.00%	6.67%
MN 026	2242	1.20%	6.67%
MN 029	399	0.47%	6.67%

MN 030	302	0.28%	6.67%
MN 037	3814	1.55%	6.67%
MN 038	1080	0.61%	6.67%
MN 040	1464	0.67%	6.67%
MN 042	730	0.82%	6.67%
MN 043	614	0.36%	6.67%
NY 2680	251	0.22%	6.67%
NY 3311	1762	0.88%	6.67%
NY 5183	644	0.33%	6.67%
NY 5280	635	0.30%	6.67%
NY 6282	175	0.09%	6.67%
NY 9121	1998	1.06%	6.67%
NY 9580	1942	0.84%	6.67%
OR Data	4506	1.89%	6.67%
PA 158	5141	2.23%	6.67%
PA 501	1454	0.85%	6.67%
PA 502	5251	2.68%	6.67%

The comparison in Table 4.5.3-1 and Figure 4.5.3-1 shows that the assumed value 6.67% (1/15) for side-by-side probability in NCHRP Report 368 is much higher than the ones observed in the WIM data sets. It is thus very conservative. Nevertheless, it can become excessively over-conservative, especially for shorter bridge spans, which translates to higher cost. For bridge design, this additional cost due to over-conservative estimation may not be significant, compared with the total cost of a bridge (Fu and van de Lindt 2006). For existing bridge evaluation, however, this additional cost can become excessive, especially when triggering bridge replacement.

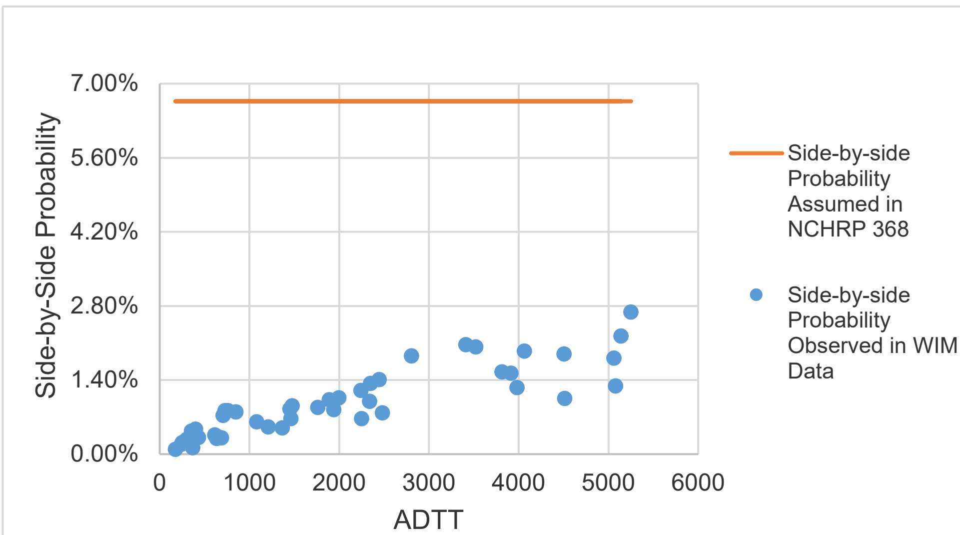

Figure 4.5.3-1. Comparison of observed and assumed side-by-side probabilities.

The side-by-side probability $P_{side\text{-}by\text{-}side}$ can be seen in Figure 4.5.3-1 as a function of ADTT. The following equation obtained from regression analysis shows this relation in a statistical sense, with an R^2 of 0.78.

$$P_{side\text{-}by\text{-}side} = 3.56 \times 10^{-6}\, ADTT + 0.0024 \qquad (4.5.3\text{-}2)$$

GVW Correlation Coefficient 0.5 for 10th Side-by-Side Trucks

For the assumed partial correlation coefficient ρ=0.5 for every tenth truck of those in side-by-side configuration, one-tenth of the events identified above are randomly selected to estimate this correlation. Then the GVWs of the trucks in the selected events are used to compute a correlation coefficient. This sampling of one-tenth of the population is repeated 15 times. The resulting 15 correlation coefficients are used to find their mean value and standard deviation. Both are tabulated in Table 4.5.3-2 for the WIM sites used, compared with the assumed 0.5 value. The assumed 0.5 appears to overestimate the GVW correlation of two trucks in a side-by-side configuration.

Table 4.5.3-2. Comparison of Observed and Assumed Correlation Coefficients (CC) for 1/10 of Side-by-Side Trucks

WIM Site ID	ADTT	Mean of Observed CC	Standard Deviation of Observed CC	Assumed CC
CA 002	2806	0.1183	0.0340	0.5000
CA 005	4064	0.0703	0.0099	0.5000
CA 007	3522	0.0683	0.0106	0.5000
CA 025	2351	0.0889	0.0134	0.5000
CA 066	3411	0.1131	0.0090	0.5000
IL 015	3982	0.2250	0.0327	0.5000
IL 016	4512	0.1042	0.0135	0.5000
IL 018	5078	0.1641	0.0222	0.5000
KY 022P47	1478	0.0864	0.0211	0.5000
KY 056P98	2447	0.0661	0.0123	0.5000
MI 5059	1209	0.1238	0.0319	0.5000
MI 6119	1890	0.0949	0.0308	0.5000
MI 6449	2248	0.0801	0.0290	0.5000
MI 6469	1368	0.1004	0.0344	0.5000
MI 7159	5061	0.0552	0.0258	0.5000
MI 7219	3914	0.0518	0.0101	0.5000
MI 7269	2483	0.0686	0.0167	0.5000
MI 7319	2339	0.0745	0.0355	0.5000
MN 026	2242	0.1072	0.0304	0.5000
MN 037	3814	0.1118	0.0136	0.5000
MN 040	1464	0.1031	0.0333	0.5000
NY 3311	1762	0.0948	0.0326	0.5000
NY 9121	1998	0.1373	0.0240	0.5000
NY 9580	1942	0.1178	0.0216	0.5000
OR Data	4506	0.0720	0.0130	0.5000
PA 158	5141	0.1125	0.0101	0.5000
PA 501	1454	0.1218	0.0098	0.5000
PA 502	5251	0.0805	0.0094	0.5000

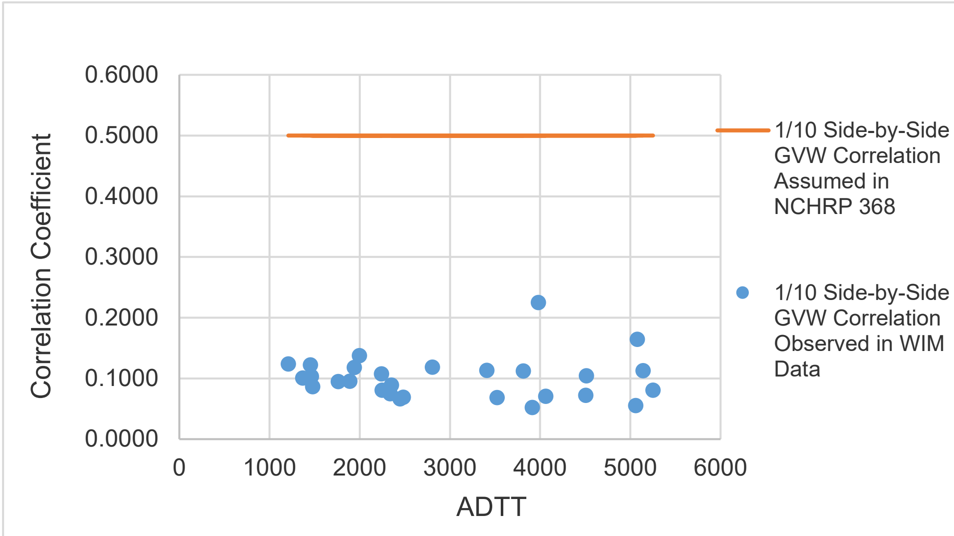

Figure 4.5.3-2. Comparison of observed and assumed GVW correlation coefficients for one-tenth of side-by-side truck events.

Figure 4.5.3-2 above shows the same comparison as Table 4.5.3-2 but in a graphic way. It is intended to quantitatively exhibit the difference between the observed and assumed. The former is shown at about one-fifth to one-fourth of the latter. Both Table 4.5.3-2 and Figure 4.5.3-2 indicate that the assumed 0.5 correlation coefficient is an overestimate and conservative because a higher correlation overestimates the chances of having two very heavy trucks simultaneously on a bridge span.

It is this load configuration that induces the maximum load effect to a bridge span. This overestimation can become excessive for very short spans, shorter than 70 ft or so. There are still a large number of these in Illinois. For example, for maximum end shear in these short spans, the second heavy truck while 50 ft behind the first heavy truck by headway distance contributes negligibly to the total end shear. However, this overestimated correlation leads to a more significant contribution instead. For mid-span moment, when the first heavy truck is in the mid-span area inducing the maximum moment, the second truck can actually be off the span if up to 50 ft away, not contributing at all to the total maximum moment. Nevertheless, the assumed 0.5 correlation coefficient increases the likelihood of the second heavy truck also being on the span and, accordingly, increases the total moment by about 100% when two trucks have almost the same load effect.

GVW Correlation Coefficient 1.0 for 30th Side-by-Side Trucks

For the assumed full correlation with correlation coefficient ρ=1, the comparison with the observed is shown in Table 4.5.3-3. Its graphic version is presented in Figure 4.5.3-3. The procedure for arriving at these correlation coefficients is similar to that used above in Table 4.5.3-2 and Figure 4.5.3-2, except 1/30 of the side-by-side events is being used here.

Table 4.5.3-3. GVW Correlation Coefficients (CC) for 1/30 of Side-by-Side Trucks

WIM Site ID	ADTT	Mean of Observed CC	Standard Deviation of Observed CC	Assumed CC
CA 002	2806	0.1065	0.0165	1.0000
CA 005	4064	0.0633	0.0204	1.0000
CA 007	3522	0.0671	0.0166	1.0000
CA 025	2351	0.0945	0.0356	1.0000
CA 066	3411	0.1189	0.0181	1.0000
IL 015	3982	0.2339	0.0639	1.0000
IL 016	4512	0.1098	0.0207	1.0000
IL 018	5078	0.1658	0.0499	1.0000
KY 022P47	1478	0.0878	0.0461	1.0000
KY 056P98	2447	0.0679	0.0218	1.0000
MI 5059	1209	0.1283	0.0578	1.0000
MI 6119	1890	0.0966	0.0298	1.0000
MI 6449	2248	0.0832	0.0327	1.0000
MI 6469	1368	0.1058	0.0293	1.0000
MI 7159	5061	0.0582	0.0265	1.0000
MI 7219	3914	0.0504	0.0160	1.0000
MI 7269	2483	0.0623	0.0147	1.0000
MI 7319	2339	0.0781	0.0367	1.0000
MN 026	2242	0.1037	0.0447	1.0000
MN 037	3814	0.1112	0.0179	1.0000
MN 040	1464	0.1027	0.0345	1.0000
NY 3311	1762	0.0972	0.0281	1.0000
NY 9121	1998	0.1449	0.0332	1.0000
NY 9580	1942	0.1082	0.0536	1.0000
OR Data	4506	0.0833	0.0154	1.0000
PA 158	5141	0.1200	0.0177	1.0000
PA 501	1454	0.1223	0.0097	1.0000
PA 502	5251	0.0807	0.0147	1.0000

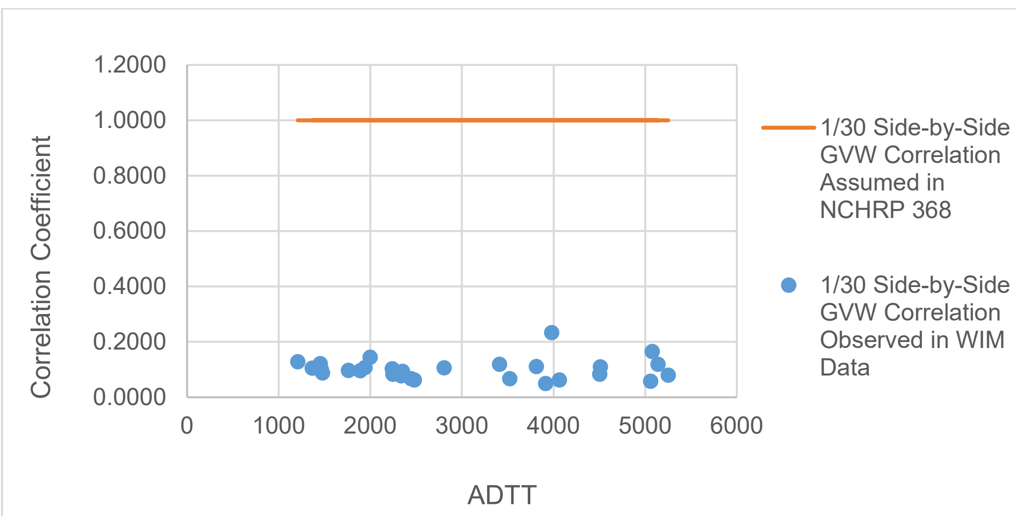

Figure 4.5.3-3. Comparison of observed and assumed GVW correlation coefficients for 1/30 of side-by-side truck events.

Figure 4.5.3-3 shows that the 1.0 correlation coefficient assumption overestimates by five to ten times, which is conservative. As discussed above for the assumed 0.5 correlation coefficient, such overestimation leads to overestimation for the maximum load effects in bridge spans. For shorter spans, such overestimation can be very significant. For bridge load rating, this overestimation may translate to extra cost for strengthening and/or replacing bridges that should have been rated as adequate.

CHAPTER 5: INADEQUACY IN ILLINOIS WIM DATA AND MITIGATION

For bridge design and evaluation with respect to strength, the critical live load consists of trucks in different lanes with a very small headway distance between them. WIM data recording two or more lanes of trucks can capture such loading, which is of interest in this study. In addition, the recorded time stamps need to have a resolution at or higher than 0.01 second for a resolution of headway distance at 1 ft or better (smaller), as explained earlier. Nevertheless, only three of the 20 WIM sites in Illinois have two lanes simultaneously recorded, namely Sites 15, 16, and 18, as indicated in Table 4.1-2. The rest of the Illinois sites have only the driving lane instrumented, providing truck-weight data for only that lane. Sites 15 and 18's passing lanes' sensors were not installed until this project's recommendation was implemented to mitigate the issue. As a result, two-lane data from these two sites are for about 6 months, shorter than for Site 16, which had been the only site in Illinois having both lanes instrumented for some years.

It was felt that the three WIM sites with two-lane data are inadequate to cover the entire state, especially the Davenport, St. Louis, and Terre Haute areas as ports from and to the neighboring states. An effort was thus made in this project to artificially generate trucks in the passing (second) lane in order to mitigate the situation. This approach was based on the following assumptions:

1. The passing lane's trucks belong to the same statistical population as trucks in the driving lane in terms of configuration (axle weights and axle spacings). WIM-recorded configuration data for the driving lane are available, so the passing lane's trucks can be simulated by random sampling from the driving lane's trucks.
2. The probability for trucks in the two lanes to become a cluster simultaneously on the same bridge span is a function of ADTT and span length. As such, this function can be found from WIM data.
3. There may be a correlation between the weights of the trucks in a cluster, which needs to be examined.
4. The headway distance between two trucks in a cluster in different lanes is a random variable, whose realizations are available in the trucks recorded at other Illinois sites that have both lanes recorded.

The following Sections 5.1, 5.2, and 5.3 elaborate and investigate these assumptions and present the results to be used in Section 5.4 for simulation. This simulation is to generate trucks for the second lane at 17 Illinois sites, identified in Table 4.1-2, whose WIM data have only the driving lane recorded.

5.1 LONGITUDINAL AND TRANSVERSE MULTIPLE-PRESENCE PROBABILITIES

Multiple presence is defined here as an event when two or more trucks are simultaneously on the span as a cluster. They both contribute to the total load effect. Their respective contributions are computed here according to the influence line of bridge span as a beam. In other words, the first truck of a potential cluster is placed at a position on the influence line to induce maximum load effect; and then the second truck of the potential cluster is placed on the same influence line according to the headway distance of the two trucks. If the second truck is off the span (*i.e.*, where its influence line ordinate is all zero for all of its axles), the second truck is omitted; and the potential cluster is denied and dismissed. This process continues until all trucks have been examined and maximum load effects identified accordingly.

In addition, if these vehicles are in the same lane, this cluster event is referred to as a *longitudinal multiple presence,* as can be seen in Figure 5.1-1. When they are in different lanes, the event is a *transverse multiple presence,* as shown in Figure 5.1-2. Note also that the latter is different from multiple presence as used in the *BDS,* where a zero headway distance of multiple identical trucks (HL93 design truck) is implicitly assumed, leading to double, triple, quadruple, … of the load effect for two, three, four, ... lanes of roadway configuration. Instead, the headway distance here is later dealt with explicitly. Note that the transverse multiple-presence probability is also different from the so-called side-by-side probability in the literature in that the latter has not been clearly or explicitly defined but is perceived to mean zero headway distance. The headway distance referred here is defined in Figure 5.1-3.

Figure 5.1-2 provides an example of longitudinal multiple presence when two trucks, whose wheel bases are depicted as two boxes, are in the same lane on the same bridge span, one ahead of the other. Figure 5.1-3 shows another example but for transverse multiple presence when they are in different lanes. Note that not all axles of the two trucks need to be fully on the bridge span to qualify for multiple presence. As long as part of the wheel base of a truck is on the span, it is counted as a multiple presence when another truck is fully or partially also on the span. How much this fully or partially present truck contributes to the total load effect depends on where it is with respect to the span's load-effect influence line. This approach will be used in load-effect computation, to be discussed later.

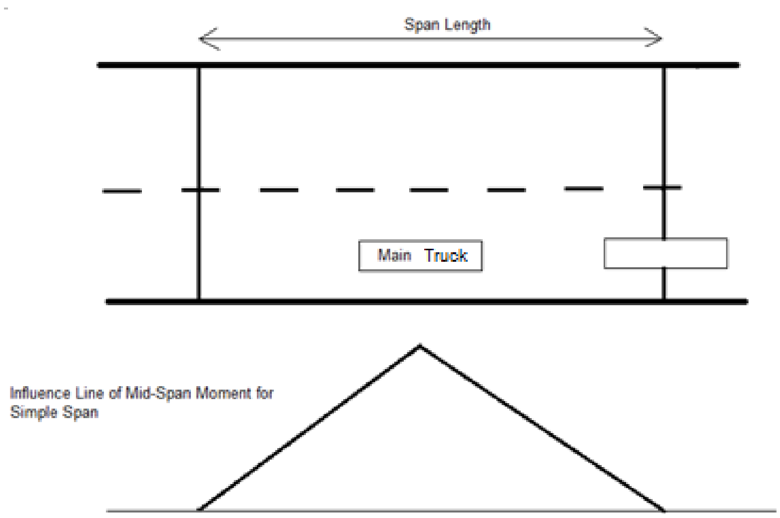

Figure 5.1-1. An example of longitudinal multiple presence.

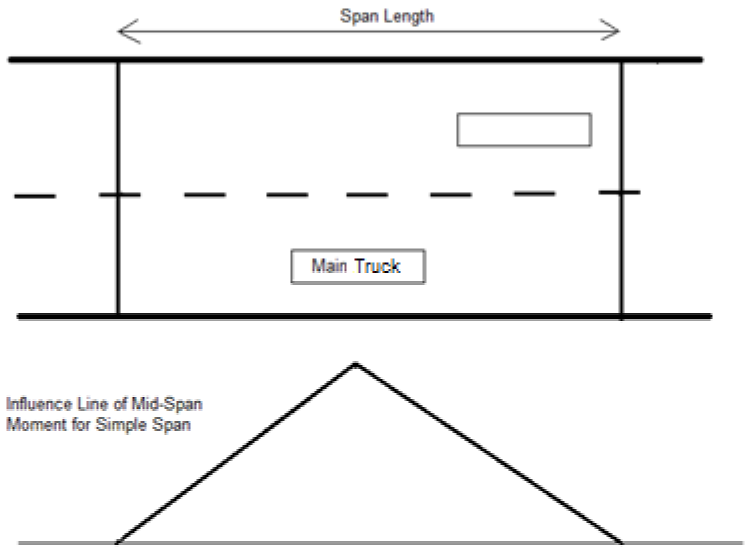

Figure 5.1-2. An example of transverse multiple presence.

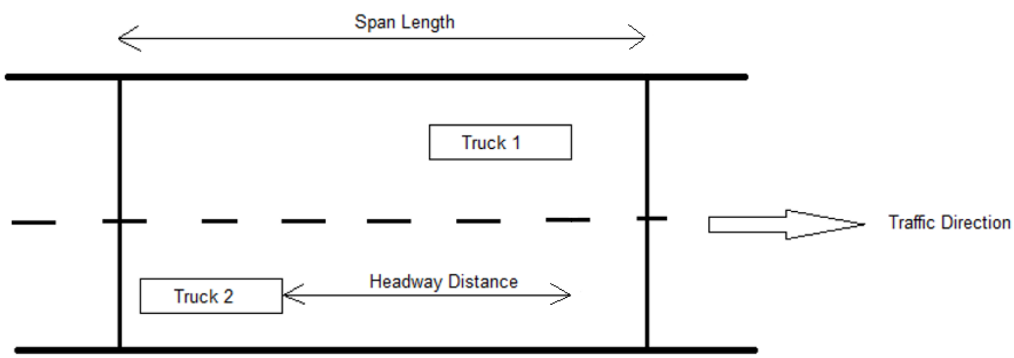

Figure 5.1-3. Headway distance between two trucks on the same bridge span.

In order to understand the behavior of trucks in motion in terms of longitudinal and transverse multiple-presence probabilities, the Illinois WIM data with both lanes recorded are used here (Sites 15, 16, and 18). In addition, several other states have also kindly provided their WIM data. All of these WIM records summarized in Table 4.5.1-1 have a time stamp-resolution of 0.01 second or better that allows such analysis of headway distance to the resolution of about 1 ft, depending on vehicle speed. Table 4.5.1-1 also shows their ADTT values for Lane 1 (driving lane) and for both lanes. The longitudinal and transverse multiple-presence probabilities are calculated from several sites in Table 4.5.1-1 as follows, using a truck-by-truck approach for the entire WIM record of each site:

$$P_L = \frac{N_{LongitudinalMultiplePresence}}{N_{AllTrucks}} \tag{5-1}$$

$$P_T = \frac{N_{TransverseMultiplePresence}}{N_{AllTrucks}} \qquad (5\text{-}2)$$

where P_L and P_T are respectively longitudinal and transverse multiple-presence probabilities. *N* is the number of events, with $N_{LongitudinalMultiplePresnece}$ and $N_{TransverseMultiplePresence}$, respectively, for the two cases here, longitudinal and transverse multiple presences. The ADTT range of WIM data used is between 1,800 and 5,000. It is in the same range as the Illinois sites, therefore relevant to the present study for Illinois.

Example results and their comparisons in Figures 5.1-4 and 5.1-5 below are shown for simple spans with length ranging from 30 to 220 ft. The span range is considered comprehensive for short to medium span lengths as a vast majority of the targeted bridge population in Illinois.

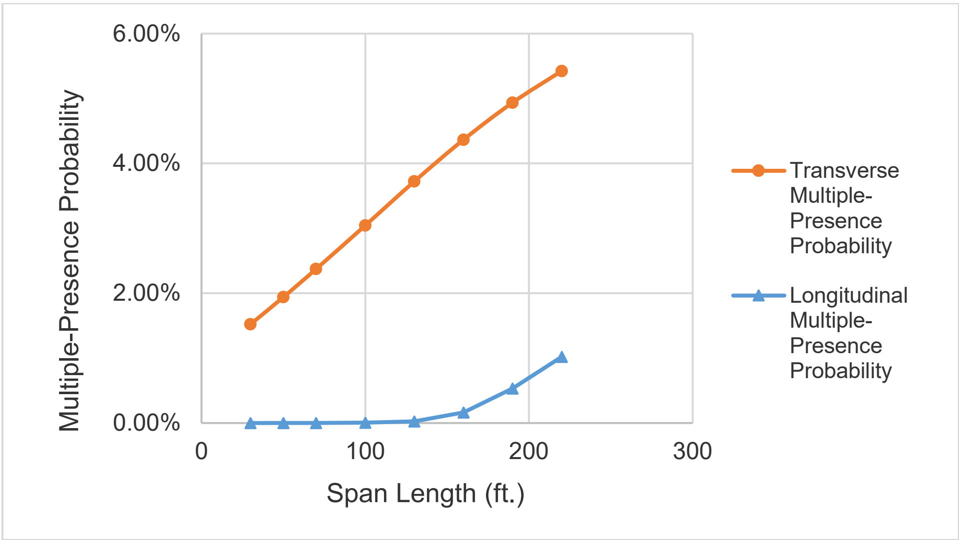

Figure 5.1-4. Multiple-presence probabilities for WIM data of MI 7159, ADTT=5,061.

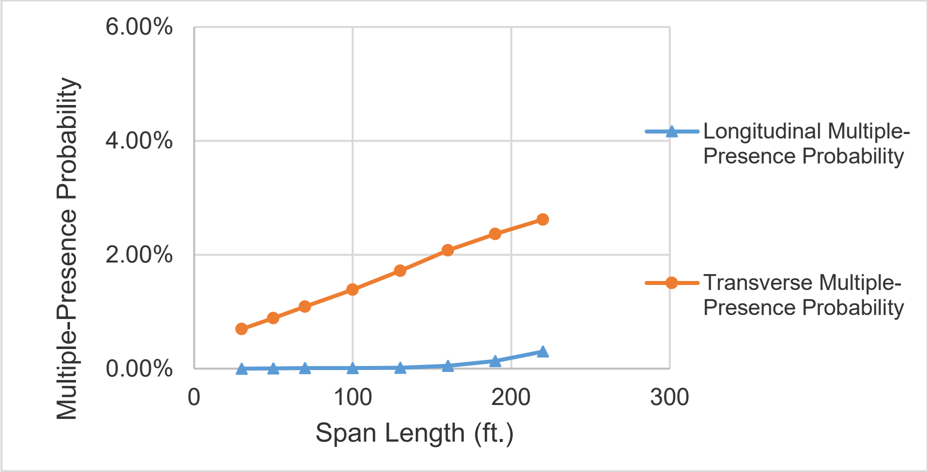

Figure 5.1-5. Multiple-presence probabilities for WIM data of NY 3311, ADTT=1,762.

For the ADTT range used, the behavior of longitudinal and transverse multiple-presence probabilities compared in Figures 5.1-4 and 5.1-5 indicates the following features that should be used in simulation for the missing second lane of trucks. These behaviors are also seen in WIM data from the remaining states, which are included in Appendix B.

1. For span length shorter than 100 ft, there are very few or no trucks in the same lane simultaneously on the same span for the observed ADTT range. Beyond 100 ft, the probability of longitudinal multiple presence starts to increase with span length. For the longest span length, 220 ft in this study, the longitudinal multiple-presence probabilities are always lower than 1%.
2. Transverse multiple-presence probability is about six or more times larger than the longitudinal.
3. The longitudinal multiple-presence probability may be ignored in the simulation of clustered trucks, as discussed in Section 5.4 below due to its negligibly low level, especially for shorter spans. In addition, the second truck's contribution to the total load effect of the cluster of trucks is expected to be negligibly lower as well, compared to the first truck at the maximum load effect position on the influence line. When the second truck is ignored, its own maximum load effect with regard to the influence line (mid-span moment and end shear) is not neglected but included as a single truck not in a cluster with other trucks. In case it is clustered with another following truck, then these two are treated as a new cluster to go through confirmation computation, depending on their respective contributions to the total load effect.

5.2 STATISTICAL CORRELATION BETWEEN CLUSTERED TRUCKS' LOAD EFFECTS

Correlation between the weights and load effects of trucks in a cluster on the same bridge span can be critical because two heavy trucks simultaneously on the span can induce the maximum load effect for the span, depending also on the headway distance between them. The headway distance between two trucks in different lanes was defined in Figure 5.1-3.

This correlation is studied here in order to provide guidance to the simulation for randomly generating trucks in the second lane for those Illinois sites that do not have the second lane recorded. When the headway distance of two trucks is smaller than the span length plus the first truck's wheel base, then the event is confirmed as a cluster. Also, their load effects are calculated according to their respective positions in that load effect's influence line. Again, this is not the definition for the so-called side-by-side configuration used in the literature. An apparent assumption of side-by-side configuration is that the headway distance is zero (so the total load effect of two trucks is simply twice that of one truck). It is equivalent to placing the two trucks at the same maximum load-effect position on the same influence line, which is not used here in dealing with a cluster of trucks because it is an overestimate.

For a span length range from 30 to 220 ft and load effects of mid-span moment and support shear, the correlation coefficients of a cluster of trucks are tabulated in Tables 5.2-1 to 5.2-4 as examples. Such tables for other sites could not be included because of the limit on the number of pages for this report, but similar behaviors have been observed. The WIM data site is identified in each table's title. These sites were identified in Table 5.1-1 earlier.

Table 5.2-1. Correlation Coefficients between Cluster Trucks' Load Effects for IL 16 with ADTT=4,512

In Terms of Mid-Span Moment	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0124	0.0110	0.0100	0.0104	0.0109	0.0111	0.0113	0.0114
In Terms of Support Shear	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0114	0.0113	0.0118	0.0119	0.0119	0.0119	0.0119	0.0119

Table 5.2-2. Correlation Coefficients between Cluster Trucks' Load Effects for IL 18 with ADTT=5,078

In Terms of Mid-Span Moment	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0143	0.0168	0.0188	0.0092	0.0058	0.0047	0.0042	0.0040
In Terms of Support Shear	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0159	0.0093	0.0052	0.0077	0.0059	0.0053	0.0048	0.0046

Table 5.2-3. Correlation Coefficients between Cluster Trucks' Load Effects for KY056P98 with ADTT= 2,447

In Terms of Mid-Span Moment	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0318	0.0322	0.0326	0.0328	0.033	0.0331	0.0331	0.0331
In Terms of Support Shear	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0320	0.0324	0.0324	0.0327	0.0328	0.0329	0.0330	0.0330

Table 5.2-4. Correlation Coefficients between Cluster Trucks' Load Effects for PA000158 with ADTT=5,141

In Terms of Mid-Span Moment	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0365	0.0326	0.0333	0.0361	0.0404	0.0421	0.0430	0.0438
In Terms of Support Shear	**Span Length (ft)**							
	30	50	70	100	130	160	190	220
	0.0332	0.0342	0.0341	0.0371	0.0392	0.0408	0.0419	0.0426

The results demonstrate that

1. Within each site, the correlation coefficients for trucks in a cluster are similar between mid-span moment and support shear, regardless of span lengths.
2. For all the WIM sites of the eight states used, this correlation coefficient for truck load effects when the vehicles are in a cluster on the same span are negligible. They are mostly below 0.050. A few are higher but still below 0.091.
3. Practically speaking, there is no correlation between the load effects of trucks in a cluster on the same bridge span, for the observed rage of ADTT.

Therefore, in the simulation to be discussed below in Section 5.4, this correlation can be ignored. In other words, the trucks in the second lane can be sampled from the first lane's recorded trucks, ignoring correlation.

5.3 CLUSTER PROBABILITY

Cluster probability here is defined as the probability of two trucks in different lanes and possibly on the same bridge span, when their headway distance satisfies the following condition:

$$\text{Headway Distance} < \text{Span Length} + \text{First Truck's Wheel Base} \tag{5-3}$$

Based on observed transverse multiple-presence probability in Section 5.1, this cluster probability is to be expressed as a function of ADTT and span length. This expression allows a reliable estimation based on ADTT and span length in the simulation application to be presented in the next section. In other words, a relation of the cluster probability to these two parameters is sought here, using the available WIM data. For this relation to be reliable and generally applicable in this project, WIM data from eight states with ADTT ranged similarly with Illinois WIM sites are used, as listed in Table 5.1-1. Their ADTT range is similar to those of the Illinois WIM sites whose data are used in this study.

Note that this cluster probability is different from the so-called side-by-side probability in the literature, while the latter actually has not been fully defined. The cluster probability quantifies the chance or likelihood for two trucks in different lanes on the same bridge span. It does not indicate the headway distance between them, which is a different random variable dealt with separately below. The headway distance determines how much each truck contributes to the total load effect according to the bridge's influence line.

The side-by-side probability used in the literature instead mixes the likelihood of two trucks on the same bridge span, with an implicit assumption of zero-headway distance. As a result, the two trucks' load effects are treated or computed in the same manner, assuming their being in the same position on the bridge span's influence line and inducing the same maximum load effect. As such, the load effects of two lanes of trucks are overestimated and sometimes excessively overestimated. This situation is particularly true for shorter spans because two trucks' simultaneously being on a short bridge span requires the headway distance to be very small. Such short headway distance very rarely happens, as shown in Table 15 in (Sivakumar et al. 2007) for sites of three states, and Section 5.1 for a total of eight states' WIM data.

Tables 5.3-1 to 5.3-3 below show example cluster probabilities for 22 sites of Illinois and seven other states with ADTT in the range similar to the Illinois WIM sites. Other tables could not be included here due to the limit on number of pages, but they are included in Appendix C. They all display a general trend that clustering is more likely when ADTT increases or/and when the span is longer.

Table 5.3-1. Cluster Probabilities for IL 15

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
3843	30	2.04%
3843	50	2.50%
3843	70	2.93%
3843	100	3.50%
3843	130	3.92%
3843	160	4.22%
3843	190	4.46%
3843	220	4.68%

Table 5.3-2. Cluster Probabilities for IL 16

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
4338	30	4.07%
4338	50	5.15%
4338	70	5.58%
4338	100	6.65%
4338	130	7.18%
4338	160	7.69%
4338	190	8.19%
4338	220	8.70%

Table 5.3-3 Cluster Probabilities for IL 18

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
4026	30	2.16%
4026	50	2.67%
4026	70	3.14%
4026	100	3.79%
4026	130	4.31%
4026	160	4.72%
4026	190	5.06%
4026	220	5.35%

The results of the tables in Appendix C, like those of Tables 5.3-1 to 5.3-3, are used to develop a regression relation between the cluster probability to $ADTT_{SL}$ and bridge span length, where $ADTT_{SL}$ is the ADTT in the driving lane. $ADTT_{SL}$, instead of total ADTT, is used here because the Illinois WIM sites that have only one lane recorded have only $ADTT_{SL}$ available to be used to estimate the cluster probability and then randomly generate trucks in the second lane, as discussed in Section 5.4.
The following regression relation was derived using the results, as in Tables 5.3-1 to 5.3-3, for the cluster probability ($P_{cluster}$):

$$P_{cluster} = 1.32\text{x}10^{-5}\ ADTT_{SL} + 1.86\text{x}10^{-4}\ \text{Span Length} - 0.016 \qquad (5\text{-}4)$$

where $ADTT_{SL}$ should be in terms of the average number of trucks per day for the driving lane and span length in ft. The R^2 for this regression is 0.74. This cluster probability is for simple span lengths ranging from 30 to 220 ft.

5.4 SIMULATION FOR MITIGATING UNRECORDED SECOND LANE'S TRUCKS

A simulation method is presented in this section. It was used to artificially generate trucks for the second (passing) lane for those Illinois WIM sites that do not have this lane instrumented. These 17 sites were identified in Table 4.1-2. This simulation used the assumptions identified earlier, as well as the analysis results of WIM data presented in Sections 5.1 to 5.3. The following procedure was used in this simulation.

1. For a given site with known $ADTT_{SL}$ and a span length, use Eq. 5-4 to find the cluster probability.
2. Find the number of clusters $N_{cluster}$ = Cluster Probability x Total number of trucks in the WIM data set.
3. For Cluster i of $N_{cluster}$, generate a uniformly distributed random ID to identify one of all headway distances available from the two-lane WIM data. Also, generate another uniformly distributed random ID to identify a truck from Lane 1's trucks, which will be treated as a truck in Lane 2. Further generate a third uniformly distributed ID to identify a truck in Lane 1 to form a cluster with the truck artificially generated in Lane 2.
4. Set i=i+1, go to 3) until Cluster $N_{cluster}$ is reached.

This simulation method was tested first using WIM data from 19 sites of 7 states for verification and validation. During this step, a site's WIM data's second lane was treated as unavailable; then, the above simulation method was applied to generate the second lane's data. This set of simulated second-lane trucks, along with the available first-lane trucks, was then used to estimate the following statistics.

$$R = \frac{\overline{LE}}{\overline{LE}_{ref}} \qquad (5\text{-}5)$$

The results were compared with the same statistics, using the original two lanes of WIM data for verification and validation. The above ratio of two mean values is used in calibration, as defined in Eq. 3-2. This test criterion was used here because R is one of the two ratios involved in calibration calculation, while the other ratio is deterministic, not involving WIM data at all. In other words, the validity of the simulation affects the accuracy of this ratio but not the other (deterministic) one in calibration. The validation results are displayed in Table 5.4-1 below as an example, where R_T is the R defined in Eq. 5-5 for the original two lanes of WIM data; and R_S is the same, but for one lane of WIM data plus a simulated second lane of data. Similar results could not be included here due to the limit on number of pages for this report, but they are included in Appendix D.

In these results, a month was selected as the basic period; and the projection was to 5 years; namely, N in Eqs. 3-3 and 3-4 is 60 for 60 one-month periods, or 5 years. The same basic period of one month and projection length of 5-years are used in the calibration presented in Chapter 5. Mid-span moments and end shears of simple spans for various span lengths ranging from 30 to 220 ft are induced to produce the maximum load effects.

These results show that the mean of difference is within $\pm$3.36% and standard deviation within 5.53%, for all the span lengths considered and both load effects of span moment and end shear. It is thus concluded that this simulation method is able to restore the second lane's trucks that were not recorded at the 17 Illinois sites that do not have that lane instrumented. The difference between the simulated data and the real data is within an acceptable level, with regard to calibration results.

Table 5.4-1. Simulation Verification Results for IL 16, $ADTT_{SL}$=4,124

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.85	2.25%
	50	0.71	0.68	-4.18%
	70	0.73	0.73	0.26%
	100	0.79	0.78	-1.49%
	130	0.83	0.85	2.56%
	160	0.86	0.86	0.09%
	190	0.87	0.89	2.27%
	220	0.89	0.90	1.48%
End Shear	30	0.78	0.78	-0.46%
	50	0.73	0.70	-4.75%
	70	0.79	0.76	-4.14%
	100	0.85	0.81	-4.89%
	130	0.88	0.86	-2.97%
	160	0.90	0.87	-3.28%
	190	0.92	0.89	-2.66%
	220	0.93	0.88	-4.92%
Mean of Difference				-1.42%
Standard Deviation of Difference				2.78%

CHAPTER 6: CALIBRATION OF LRFR LIVE-LOAD FACTORS

This calibration covers LRFR load rating for Illinois bridges. It includes cases of (1) design load rating, (2) legal load rating, (3) routine-permit load rating, and (4) special-permit load rating. The concept of live-load calibration used in this project was presented earlier in Eq. 3-2 in Chapter 3. This equation is then specifically defined below for each load-rating case identified above.

6.1 CALIBRATION FOR DESIGN LOAD RATING

Design load rating in *MBE* requires the design load HL93 be the nominal load. There are two levels of design load rating required, inventory and operating. The inventory rating, or equivalently the inventory-level rating, for LRFR allows comparisons with the capacity for new structures or the design life of 75 years for the bridges to be used indefinitely. It prescribes a live-load factor of 1.75 in *MBE*, the same for Strength I limit state design in *BDS*. The operating rating, or equivalently the operating-level rating, for LRFR generally describes the maximum permissible live load to which the structure may be subjected. Its live-load factor is 1.35. Accordingly, Eq. 3-2 becomes

$$\gamma_{L,inventory-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}} = 1.75 \times \frac{LE_{HL93}}{LE_{HL93}} \times \frac{\overline{LE}_{uptoFBF,5-year-projected}}{\overline{LE}_{uptoFBF,5-year-projected}} \tag{6.1-1}$$

$$\gamma_{L,operating-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}} = 1.35 \times \frac{LE_{HL93}}{LE_{HL93}} \times \frac{\overline{LE}_{uptoFBF,5-year-projected}}{\overline{LE}_{uptoFBF,5-year-projected}} \tag{6.1-2}$$

The above equations are applicable for both one-lane loading and multiple-lane loading situations. As can be seen in these equations, the new live-load factors for both cases of inventory and operating levels remain unchanged because the two ratios multiplied to the reference live-load factor 1.75 or 1.35 are both equal to 1.0. This situation is partially because relative calibration is used here. It is also consistent with current Illinois bridge design practice using HL93 load and current *BDS* live-load factors, with which the design load rating is required to be consistent. Specifically for Strength I limit state design, the live-load factor is 1.75.

6.2 CALIBRATION FOR LEGAL LOAD RATING

For the legal load rating, *MBE* prescribes vehicles Types 3, 3S2, and 3-3 and the special hauling vehicles SU4, SU5, SU6, and SU7, along with the notional rating load (NRL), as the rating vehicles. The corresponding live-load factor is given as a function of ADTT as follows. Linear interpolation may be used to obtain $\gamma_{L,ref}$ for other ADTT values, according to *MBE*.

$$\gamma_{L,ref} = \begin{cases} 1.45 & for\ ADTT unknown \\ 1.45 & for\ ADTT \geq 5,000 \\ 1.30 & for\ ADTT \leq 1,000 \end{cases} \tag{6.2-1}$$

In the state of Illinois starting from 2017, IDOT (2017) has been using its own posting vehicles below in Figure 6.2-1 for legal load screening of its bridges. They are intended to envelope legal loads in Illinois, which are defined according to state statutes. Accordingly, the general calibration equation Eq. 3-2 becomes

for a one-lane loading situation:

$$\gamma_{L,legal-load-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}}$$
$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-posting-trucks}} \times \frac{OneLaneLoad's\overline{LE}_{uptoIL-posting-trucks,5-year-projected}}{OneLaneLoad's\overline{LE}_{uptoNRL,5-year-projected}} \quad (6.2\text{-}2)$$

and for a two-lane loading situation:

$$\gamma_{L,legal-load-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}}$$
$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-posting-trucks}} \times \frac{TwoLaneLoad's\overline{LE}_{uptoIL-posting-trucks,5-year-projected}}{TwoLaneLoad's\overline{LE}_{uptoNRL,5-year-projected}} \quad (6.2\text{-}3)$$

In the above equations, $\gamma_{L,ref}$ is the live-load factor in the *MBE* for legal load rating, which has been identified in Eq. 6.2-1 for this legal load rating case for calibration. As discussed in Chapter 3, the mean value $\overline{LE}$ of the temporal maximum is obtained by projection to the 5-year future as formulated in Eqs. 3-3 and 3-4 using the basic time period equal to one month and N=60, as 5 years is equal to 60 months.

The notional rating load (NRL) envelopes the AASHTO legal loads Types 3, 3S2, and 3-3, and the special hauling vehicles (SHV), which meets FBF and thus can travel without a permit. Comparison between the NRL and Illinois posting vehicles is displayed in Table 6.2-1, in terms of mid-span moment and end shear of the listed simple spans. The span-length range is typical for Illinois. Table 6.2-1 shows that the AASHTO NRL is less severe than the Illinois posting loads by 3% to 19%, depending on span length and load effect.

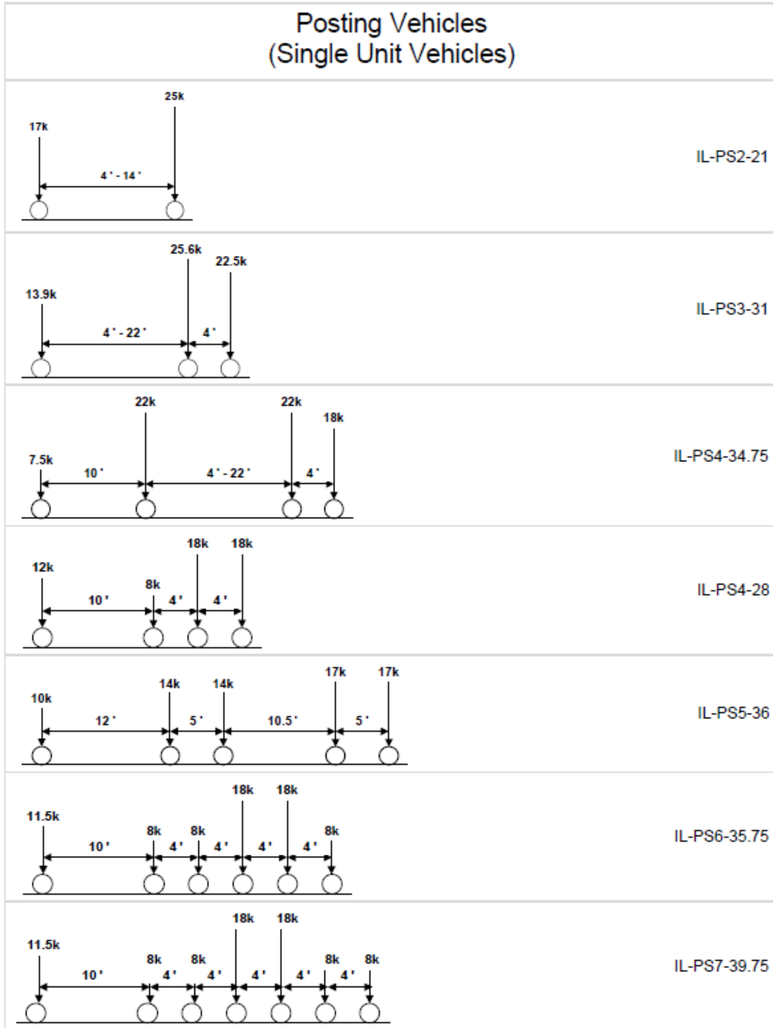

Figure 6.2-1a. Illinois posting trucks for legal loads—single units.

Tables 6.2-2 to 6.2-5 below exhibit the calibrated live load factors for Illinois legal load rating over the range of ADTT observed at two of the 20 WIM sites in Table 4.1-2. Similar tables for the remaining sites are included in Appendix E. The first set of two tables (6.2-2 and 6.2-3) are for the one-lane loading case according to Eq. 6.2-2 and the second set (Tables 6.2-4 and 6.2-5) for the two-lane loading case according to Eq. 6.2-3. These two sites (12 and 26) have the lowest and highest ADTTs of all these 20 sites. The WIM site IDs referred to in these tables were given in Table 4.1-2. These results show that the current AASHTO live-load factors in Eq. 6.2-1 are over-conservative by about 10% for legal load rating.

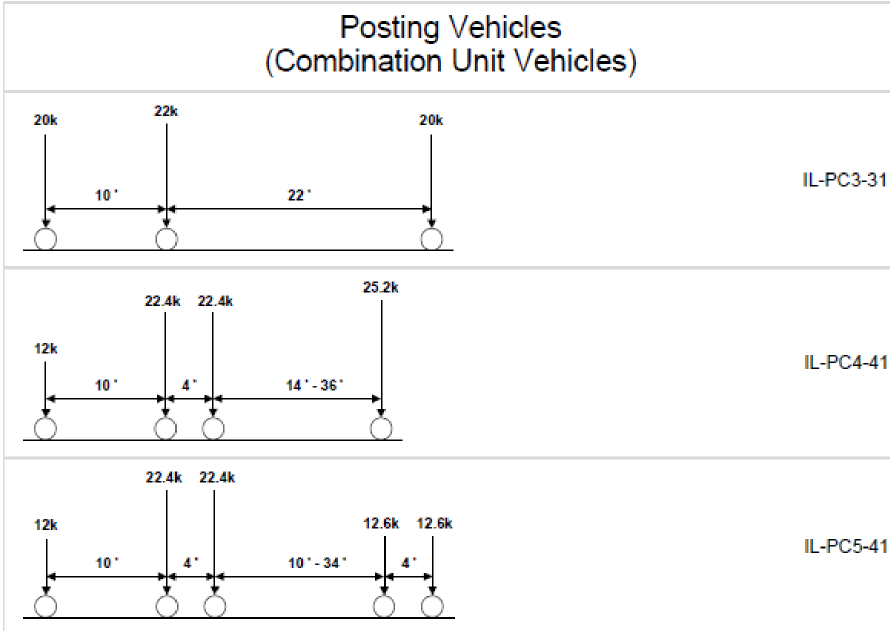

Figure 6.2- 1b. Illinois posting trucks for legal loads—combination units.

Table 6.2-1. Load-Effect Ratios of NRL to Illinois Posting Loads

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.88	0.81	0.86
50	0.99	0.94	0.95
70	0.98	0.96	0.95
100	0.98	0.96	0.96
130	0.98	0.97	0.96
160	0.98	0.97	0.96
190	0.98	0.97	0.97
220	0.98	0.97	0.97

Table 6.2-2. Live-Load Factors for Legal Load Rating for IL-12 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.04	0.98	1.06
50	1.21	1.14	1.14
70	1.19	1.18	1.13
100	1.20	1.21	1.16
130	1.19	1.26	1.16
160	1.21	1.27	1.19
190	1.26	1.25	1.21
220	1.27	1.25	1.22

Table 6.2-3 Live-Load Factors for Legal Load Rating for IL-26 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.16	1.09	1.23
50	1.37	1.26	1.28
70	1.35	1.29	1.27
100	1.34	1.20	1.27
130	1.31	1.19	1.23
160	1.31	1.24	1.25
190	1.31	1.26	1.27
220	1.29	1.25	1.28

Table 6.2-4. Live-Load Factors for Legal Load Rating for IL-12 for Two-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	0.99	1.02
50	1.26	1.18	1.21
70	1.28	1.24	1.18
100	1.21	1.19	1.18
130	1.21	1.20	1.17
160	1.18	1.22	1.16
190	1.19	1.17	1.17
220	1.21	1.19	1.19

Table 6.2-5 Live-Load Factors for Legal Load Rating for IL-26 for Two-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	0.97	1.02
50	1.18	1.17	1.19
70	1.18	1.22	1.19
100	1.30	1.29	1.31
130	1.30	1.29	1.29
160	1.30	1.30	1.30
190	1.33	1.29	1.29
220	1.31	1.30	1.29

The tabulated results, as in Tables 6.2-2 to 6.2-5, for all 20 sites are plotted in Figure 6.2-2 below as a summary, comparing the calibrated and referenced AASHTO live-load factors. It also includes a regression expression for the calibrated live-load factor for the legal load rating case as follows. The calibrated-live-load factor is given in this empirical relation as a function of ADTT. It is similar to what is prescribed in the *MBE*. This relation is intended to be used for both one-lane and two-or-more-lane loading situations, whichever governs.

$$\gamma_{L,legal-load-rating} = \begin{cases} 1.34 & for\ ADTT > 6,500 \quad or\ unknown \\ 3.25*10^{-5}\ ADTT + 1.13 & for\ 1,500 \le ADTT \le 6,500 \\ 1.18 & for\ ADTT < 1,500 \end{cases} \tag{6.2-4}$$

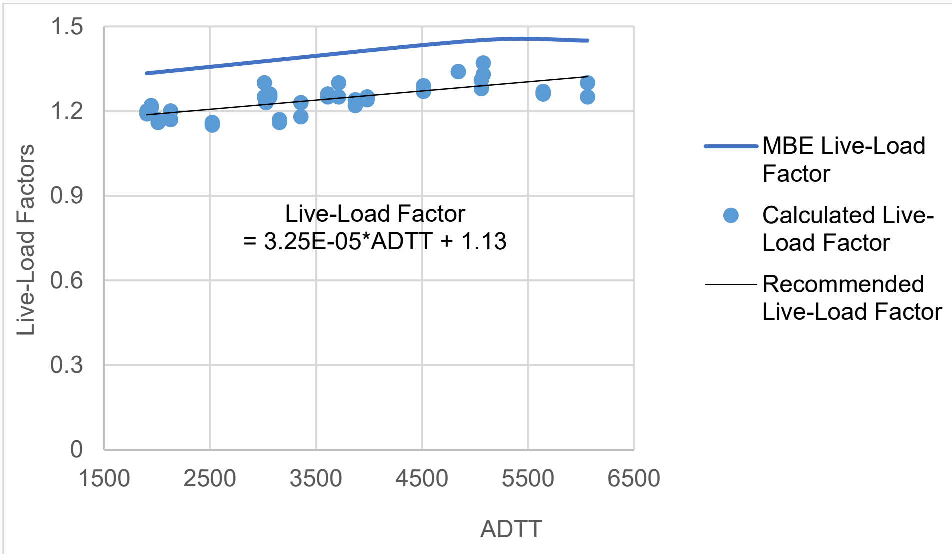

Figure 6.2-2. Comparison of calibrated and reference live-load factors for legal load rating.

Note also that regression is based on averaging over the variation, which is consistent with the target reliability index as the average embedded in previous practice. A sensitivity analysis was also conducted herein, which indicated that these results are not sensitive to input data's possible random variation, including that of the WIM data and simulated data.

In addition, the current *MBE* uses a lower-limit ADTT at 1,000 and high at 5,000, which was implemented in AASHTOWare BrDR that IDOT uses for bridge load rating. In order to facilitate implementation with this software program, the following Table 6.2-6 is recommended as an alternative to Eq. 6.2-4 to fit within the *MBE*'s lower and upper limits.

Table 6.2-6. Recommended Live-Load Factor for Legal Load Rating in Illinois

ADTT	Live-Load Factor
<1,000	1.18
1,000*	1.18*
5,000*	1.34*
>5,000	1.34

* Linear interpolation is permitted for ADTT values between 1,000 and 5,000.

6.3 CALIBRATION FOR ROUTINE-PERMIT LOAD RATING

A routine-permit load has a gross weight, axle weight, or distance between axles not conforming to state statutes for legally configured vehicles, authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis (IDOT 2017).

Routine permits are issued in Illinois according to the *Illinois Vehicle Code* (IDOT 2014). Permit vehicles need to comply with the following requirements.

1. Steering-axle load shall not exceed 20,000 lb.
2. Any single-axle load shall not exceed 24,000 lb.
3. Group and gross axle load shall be under the following limits:

  - 6-axle tractor semitrailer combinations: maximum of 120,000 lb gross; maximum of 48,000 lb on drive tandem and maximum of 60,000 lbs on semitrailer tridem
  - 5-axle tractor semitrailer combinations: maximum of 100,000 lb gross; maximum of 48,000 lb on either tandem
  - 4-or-more-axle vehicle: maximum of 76,000 lb gross; maximum of 34,000 lb on one tandem and 44,000 lb on the other. The wheelbase must be 23 ft or more.
  - 3-or-more-axle vehicle: maximum of 68,000 lb gross; maximum of 20,000 lb on one axle and 48,000 lb on the tandem. The wheelbase must be 18 ft or more.

The calibration equation for Illinois routine-permit load rating is then accordingly derived as follows using Eq. 3-2,

$$\gamma_{L,routine-permit-rating} = \gamma_{L,ref}\,\frac{LE_{n,ref}}{LE_n}\,\frac{\overline{LE}}{\overline{LE}_{ref}}$$
$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-routine-permit-trucks}} \times \frac{OneLaneLoad's\,\overline{LE}_{uptoIL-IL-routine-permit-trucks,5-year-projected}}{OneLaneLoad's\,\overline{LE}_{uptoNRL,5-year-projected}}$$

for one-lane loading (6.3-1)

$$\gamma_{L,routine-permit-rating} = \gamma_{L,ref}\,\frac{LE_{n,ref}}{LE_n}\,\frac{\overline{LE}}{\overline{LE}_{ref}}$$
$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-routine-permit-trucks}} \times \frac{TwoLaneLoad's\,\overline{LE}_{uptoIL-IL-routine-permit-trucks,5-year-projected}}{TwoLaneLoad's\,\overline{LE}_{uptoNRL,5-year-projected}}$$

for two-lane loading (6.3-2)

Eqs. 6.3-1 and 6.3-2 are similar to Eqs. 6.2-2 and 6.2-3, respectively; the latter are for the legal load rating presented in Section 6.2. The only difference is the case of interest now being routine permit and then legal load. Accordingly, the nominal load effect LE in the right-hand side's first ratio's denominator in the above equations now is that of the Illinois routine-permit trucks. They are displayed in Figure 6.3-1 below. These live-load models were adopted in 2017 by IDOT (2017) for bridge screening and permit checking and issuance. In addition, the mean of the temporal maximum $\overline{LE}$ of real trucks in Eqs. 6.3-1 and 6.3-2 are obtained from WIM data using the same approach used in the case of legal load rating, except the upper limit for trucks to be included. The concept and procedure were presented in Chapter 3.

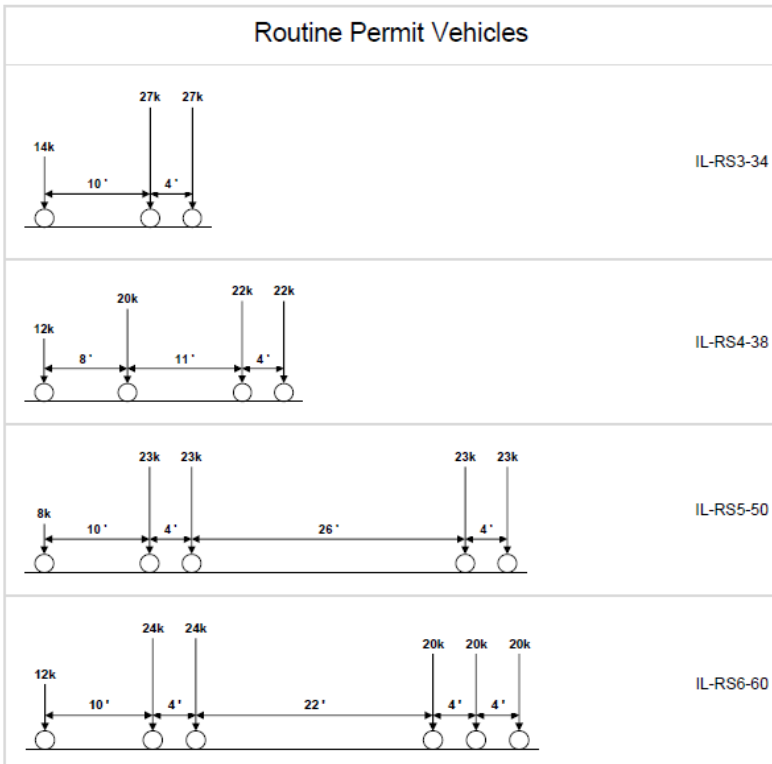

Figure 6.3-1. Illinois routine-permit trucks.

To understand the relative relation of the AASHTO NRL and the Illinois routine-permit trucks in Figure 6.3-1, Table 6.3-1 below displays the moment and shear ratios between the two load models for simple spans typical in Illinois. These values are used in Eqs. 6.3-1 and 6.3-2 as the first ratio on the right-hand side.

Table 6.3-1. Load-Effect Ratios of NRL and Illinois Routine-Permit Trucks

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.89	0.78	0.89
50	1.01	0.78	0.81
70	0.95	0.74	0.76
100	0.83	0.71	0.74
130	0.78	0.70	0.72
160	0.76	0.69	0.71
190	0.74	0.69	0.70
220	0.73	0.68	0.70

The example calibration results using Eqs. 6.3-1 and 6.3-2 are displayed in Tables 6.3-2 to 6.3-5 for two of the 20 Illinois WIM sites. The first set of two tables (Tables 6.3-2 and 6.3-3) are for one-lane loading using Eq. 6.3-1, and the second set of two (Tables 6.3-4 and 6.3-5) for two-lane loading according to Eq. 6.3-2. Similar tables for the other sites are included in Appendix F. The WIM sites were identified in Table 4.1-2.

Table 6.3-2. Live-Load Factors for Routine-Permit Load Rating for IL-12 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.09	1.31	1.05
50	1.28	1.39	1.08
70	1.26	1.33	1.05
100	1.11	1.32	1.03
130	1.06	1.24	1.33
160	1.04	1.09	1.24
190	1.01	1.02	1.14
220	0.94	1.01	1.10

Table 6.3-3. Live-Load Factors for Routine-Permit Load Rating for IL-26 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.22	1.19	1.23
50	1.57	1.25	1.29
70	1.51	1.14	1.24
100	1.32	0.95	1.63
130	1.17	1.13	1.29
160	0.98	1.01	1.11
190	0.85	1.01	1.06
220	0.76	1.02	1.06

Table 6.3-4. Live-Load Factors for Routine-Permit Load Rating for IL-12 for Two-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.12	1.11	1.27
50	1.40	1.14	1.17
70	1.32	1.09	1.14
100	1.04	1.00	1.07
130	1.08	1.03	1.14
160	0.97	1.06	1.05
190	0.93	1.04	1.06
220	0.96	1.08	1.08

Table 6.3-5. Live-Load Factors for Routine-Permit Load Rating for IL-26 for Two-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.34	1.20	1.34
50	1.40	1.23	1.24
70	1.27	1.15	1.23
100	1.19	1.18	1.28
130	1.12	1.18	1.19
160	1.13	1.19	1.21
190	1.15	1.18	1.21
220	1.09	1.18	1.18

The tabulated results as in Tables 6.3-2 to 6.3-5 are plotted in Figure 6.3-2 below, compared with current AASHTO live-load-factors for routine-permit load rating. It can be seen that the current live-load factors are over-conservative by about 5% for routine-permit load rating.

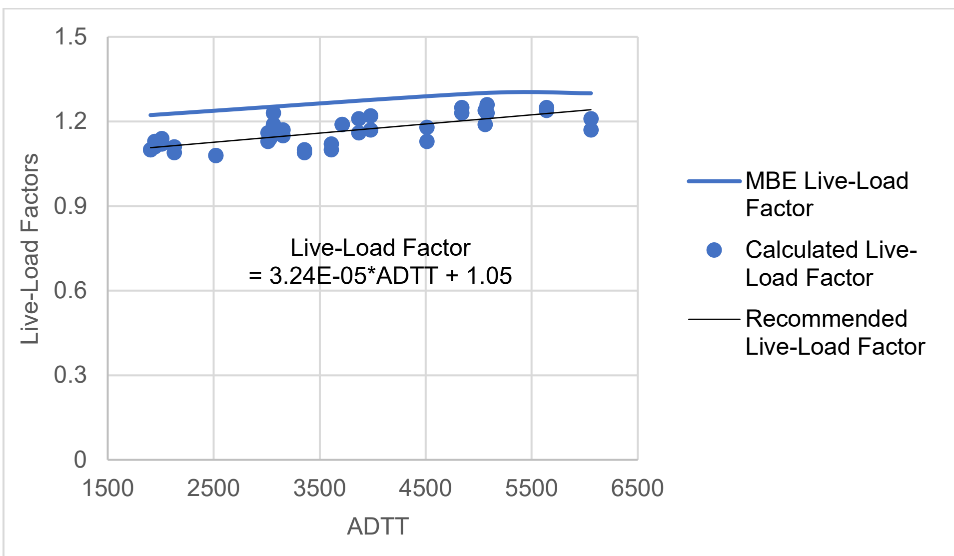

Figure 6.3-2. Comparison of calibrated and current live-load factors for routine-permit load rating.

Figure 6.3-2 also shows a regression analysis result for the obtained calibrated live-load factors for the case of routine-permit load rating. This regression relation is recommended as follows. It is intended to be applied with one-lane and two-or-more-lanes loading for bridge evaluation for routine-permit loads in Illinois, whichever governs.

$$\gamma_{L,routine-permit-load-rating} = \begin{cases} 1.26 & for\ ADTT > 6,500 \quad or\ unknown \\ 3.24 * 10^{-5}\ ADTT + 1.05 & for\ 1,500 \le ADTT \le 6,500 \\ 1.10 & for\ ADTT < 1,500 \end{cases} \tag{6.3-3}$$

Note that the AASHTO *MBE* also uses GVW over AL (front axle to rear axle length) as a parameter to categorize routine-permit trucks and correspondingly prescribes live-load factors. The Illinois WIM data were found to have overwhelmingly more routine-permit trucks with GVW/AL smaller than 2, with very few trucks in the other two categories (2<GVW/AL<3 and GVW/AL>3). Thus categorizing according to GVW/AL appears to be unnecessary for Illinois.

As discussed above for the legal load-rating case, regression based on the average is consistent with the target reliability index selection as the average embedded safety level in previous practice. A sensitivity analysis has also shown that these results are not sensitive to input data's possible random variation including the WIM data and simulated data.

In addition, the following Table 6.3.-6 is recommended as an alternative to Eq. 6.3-3, assisting IDOT in implementation via the AASHTOWare BrDR, which has the MBE's lower and upper limits of 1,000 and 5,000 for ADTT.

Table 6.3-6. Recommended Live-Load Factor for Routine-Permit Load Rating in Illinois

ADTT	Live-Load Factor
<1,000	1.10
1,000*	1.10*
5,000*	1.26*
>5,000	1.26

* Linear interpolation is permitted for ADTT values between 1,000 and 5,000.

6.4 CALIBRATION FOR SPECIAL-PERMIT LOAD RATING

A special permit is issued by a bridge owner to a vehicle, allowing a permit load of a specific configuration, axle weights, and gross vehicle weight for a limited number of specified bridge crossings (IDOT 2017). The so called superloads are required to have such a special permit to travel on state-owned routes and bridges. Issuance of a permit could also require special restrictions such as reduced speed, load positioning, or traffic prohibitions. To ensure compliance with the permit, restriction such as a police escort or other means of supervision for the bridge crossing may be specified within the issued permit.

Note that those trucks allowed to travel more than a few times, such as the category in *MBE* referred to as *multiple trips* (less than 100 times), should be categorized as routine permits not special permits according to the IDOT definition above. In other words, the differentiation between routine and special permits should be in accordance with the contents, namely frequency of travel, not the name only.

As such, special permits are meant to be issued for a limited number of trips and thus to a limited number of vehicles. Accordingly, there is a very high likelihood that this vehicle will not be adjacent to another truck in the traffic in a cluster. Therefore, the calibration below is formulated for one-lane loading only.

$$\gamma_{L,special-permit-rating} = \gamma_{L,ref}\frac{LE_{n,ref}}{LE_n}\frac{\overline{LE}}{\overline{LE}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-special-permit-truck}} \times \frac{OneLaneLoad's\,\overline{LE}_{uptoIL-IL-special-permit-truck,5-year-projected}}{OneLaneLoad's\,\overline{LE}_{uptoNRL,5-year-projected}} \tag{6.4-1}$$

Figure 6.4-1 displays an example special-permit truck from the *Structural Services Manual* of IDOT (2015). It is used here in Eq. 6.4-1 in the denominator of the first ratio on the right-hand side. Table 6.4-1 below displays a comparison between the AASHTO legal (NRL) and this Illinois special-permit load. The span range remains the same as for the legal and routine-permit load ratings typical in Illinois. These ratio values are used as the first ratio in Eq. 6.4-1 on the right-hand side.

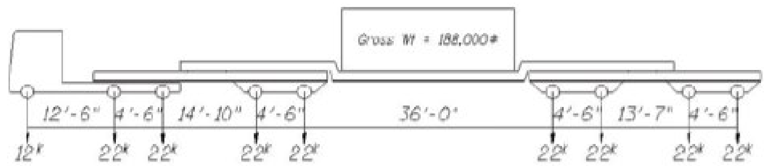

Figure 6.4-1. Example of Illinois special-permit truck (IDOT 2015).

Table 6.4-1. Load Effect Ratios of NRL and Illinois Special-Permit Load

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.16	0.84	0.86
50	1.02	0.82	0.84
70	0.93	0.81	0.80
100	0.85	0.66	0.66
130	0.71	0.58	0.59
160	0.63	0.54	0.55
190	0.58	0.52	0.53
220	0.55	0.50	0.51

The calibration results using Eq. 6.4-1 are displayed in Tables 6.4-2 and 6.4-3 as examples for all Illinois WIM sites. Appendix G includes results for the other sites. Figure 6.4-2 illustrates the comparison of the calibrated and the current AASHTO live-load factors for Illinois special-permit load rating over the range of the observed ADTT. This figure includes those sites in Appendix G. The comparison shows that the current AASHTO live-load factors are over-conservative by about 10% for special-permit load rating.

Table 6.4-2. Live-Load Factors for Special-Permit Load Rating for IL-12 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.10	0.24	0.91
50	1.14	0.99	1.07
70	1.15	1.45	1.06
100	1.16	1.32	1.01
130	1.08	1.29	1.00
160	1.07	1.16	1.28
190	1.04	1.08	1.17
220	0.96	1.05	1.12

Table 6.4-3. Live-Load Factors for Special-Permit Load Rating for IL-26 for One-Lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.15	1.13
50	1.29	1.13	1.18
70	1.25	1.15	1.18
100	1.25	1.35	1.17
130	1.23	1.17	1.28
160	1.10	1.07	1.13
190	0.99	0.99	1.03
220	0.89	0.93	0.96

Figure 6.4-2 below summarizes the calibration results in a plot. It also contrasts the calibrated with the current AASHTO LRFR live-load factors for special-permit load rating. It appears that, for Illinois, a lower live-load factor is justified considering the truck loads in the state.

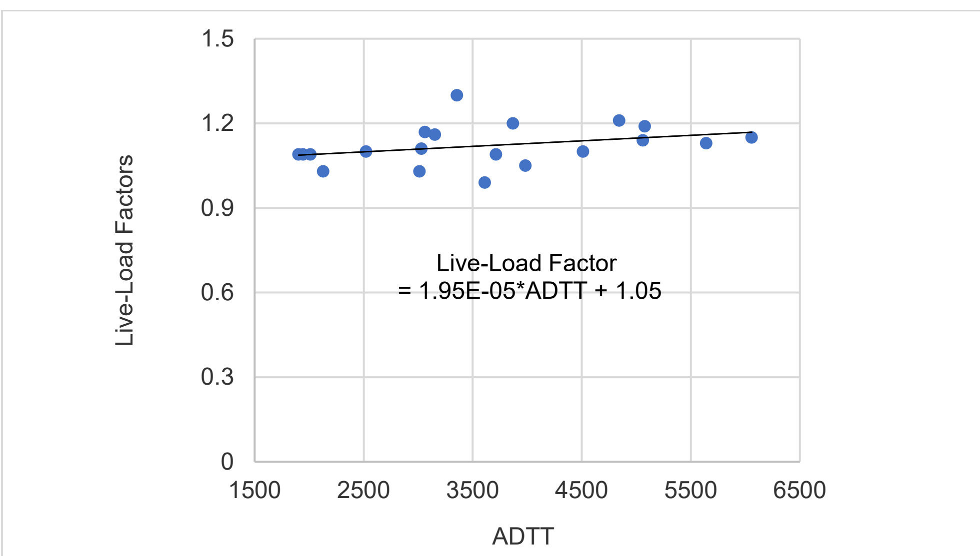

Figure 6.4-2. Comparison of calibrated and current live-load factors for special-permit load rating.

Figure 6.4-2 above also shows a regression analysis result expressing the live-load factor for special-permit load rating in Illinois as follows. Note that this relation is intended to be applied for one-lane loading.

$$\gamma_{L,special-permit-load-rating} = \begin{cases} 1.18 & for\ ADTT > 6,500 \quad or\ unknown \\ 1.95*10^{-5}\ ADTT + 1.05 & for\ 1,500 \leq ADTT \leq 6,500 \\ 1.08 & for\ ADTT < 1,500 \end{cases} \tag{6.4-2}$$

For the purpose of implementation with AASHTOWare BrDR, the average value of Eq. 6.4-2, 1.13, may be used for this category of special permit.

CHAPTER 7: ILLUSTRATIVE APPLICATION EXAMPLES

In this chapter, one illustrative load-rating example is presented with the proposed and current live-load factors for comparison. A total of ten such examples were worked out, but only one could be presented here due to the limit on the number of pages for this report. The remaining nine are included in Appendix H. Existing bridges built for lower design live loads are the focus here. By computing rating factors, the impact of the proposed live-load factors is shown for these existing highway structures.

7.1 NONCOMPOSITE STEEL BEAM BRIDGE WITH CONCRETE DECK

Bridge Data :

Span: 50 ft 3 in. = 50.25 ft
Year Built: 1942
Material:
Concrete: $f'_c = 2.5$ Ksi
Structural Steel: $F_Y = 33$ Ksi
Reinforcing Steel: $f_Y = 33$ Ksi
Condition: Fair
Traffic: Two Lanes
ADTT (one direction): Unknown
Skew: 0°

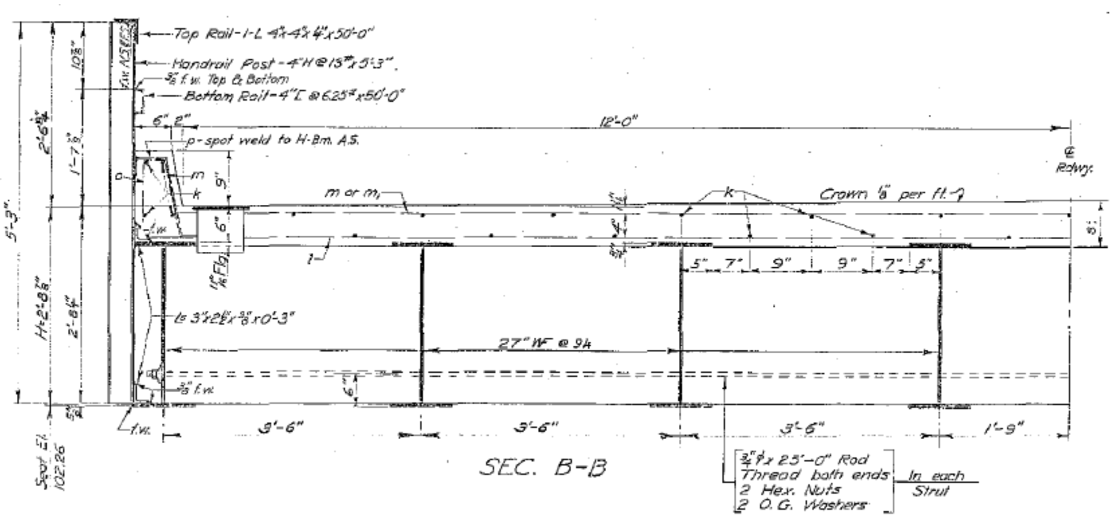

Figure 7.1-1 Cross section of steel I-girder bridge.

Number of Beams: 8
Beam Spacing: 3 ft 6 in. = 3.5 ft
Beam Section: W27×94
$\mathrm{A} = 27.6 \text{ in.}^2$
$\mathrm{I_z} = 3270 \text{ in.}^4$
$\mathrm{S_z} = 243 \text{ in.}^3$

Thickness of Deck: 6 in.
Thickness of Concrete Overlay: 2 in.
Depth of Curb: 9 in.
Width of Top Curb: 6 in.
Width of Bottom Curb: 8 in.
Width of Clear Roadway: 24 ft
Total Width of Deck: 25 ft 4 in = 25.33 ft

Load Rating for Steel Beam:

Dead-Load Analysis:

Components and Attachments, DC

$$\mathrm{Deck} = 3.5 \times \frac{6}{12} \times 0.15 = 0.26\ \mathrm{Kip/ft}$$

Beam = $0.094 \times 1.06 = 0.10\ \mathrm{Kip/ft}$ 6% increase for connections

$$\mathrm{Curb} = \frac{(6+8) \times 9}{2} \times \frac{1}{144} \times 0.15 \times \frac{2\ \mathrm{curbs}}{8\ \mathrm{beams}} = 0.016\ \mathrm{Kip/ft}$$

Railings:

Railings are composed of one L4 $\times$ 4 $\times$ 1/4, one MC4 $\times$ 13 and one C4 $\times$ 6.25

$$(0.0066 + 0.013 + 0.00625) \times 1.06 \times \frac{2\ \mathrm{Railings}}{8\ \mathrm{beams}} = 0.0069\ \mathrm{Kip/ft}$$

Total per Beam = $0.26 + 0.10 + 0.016 + 0.0069 = 0.38\ \mathrm{Kip/ft}$

$$\mathrm{M_{DC}} = \frac{1}{8} \times 0.38 \times 50.25^2 = 119.94\ \mathrm{Kip-ft}$$

Wearing Surface

$$\mathrm{DW} = 24 \times \frac{2}{12} \times 0.14 \times \frac{1}{8\ \mathrm{beams}} = 0.07\ \mathrm{Kip/ft}$$

$$\mathrm{M_{DW}} = \frac{1}{8} \times 0.07 \times 50.25^2 = 22.09\ \mathrm{Kip-ft}$$

Live-Load Analysis:

Distribution Factor

$K_g = \mathrm{n}(\mathrm{I} + \mathrm{Ae_g^2})$

$$\mathrm{n} = \frac{\mathrm{E_B}}{\mathrm{E_D}}$$

E_B=29000 Ksi

$E_D = 33000(\mathrm{w_c})^{1.5}\sqrt{\mathrm{f'_c}} = 33000 \times (0.150)^{1.5} \times \sqrt{2.5} = 3031.24\ \mathrm{Ksi}$

I = 3270 $\mathrm{in^4}$

e_g =0 for non-composite construction

$$\mathrm{K_g} = \frac{\mathrm{E_B}}{\mathrm{E_D}} \times \mathrm{I} = \frac{29000}{3031.24} \times 3270 = 31284.23\ \mathrm{in^4}$$

$$\frac{\mathrm{K_g}}{12\mathrm{Lt_s^3}} = \frac{31284.23}{12 \times 50.25 \times 6^3} = 0.24$$

One Lane Loaded:

$$g_1 = 0.06 + (\frac{S}{14})^{0.4}(\frac{S}{L})^{0.3}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.06 + (\frac{3.5}{14})^{0.4}(\frac{3.5}{50.25})^{0.3}(0.24)^{0.1} = 0.28$$

Multiple Lanes Loaded:

$$g_m = 0.075 + (\frac{S}{9.5})^{0.6}(\frac{S}{L})^{0.2}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.075 + (\frac{3.5}{9.5})^{0.6}(\frac{3.5}{50.25})^{0.2}(0.24)^{0.1} = 0.35$$

Multiple-lane loading controls.

Undistributed Live-Load Effects

Use the undistributed load effects due to the Illinois design, legal and permit live loads in Table 7.1-1. Dynamic load allowance (IM) of 33% is included in the calculation.

Table 7.1-1. Undistributed Live-Load Effects

Load Rating	Live Loads	Live-Load Effects (Kip-ft)
Design	HL-93	1026.61
Legal	IL-PS2-21	653.03
	IL-PS3-31	933.93
	IL-PS4-34.75	979.21
	IL-PS4-28	750.12
	IL-PS5-36	754.78
	IL-PS6-35.75	896.75
	IL-PS7-39.75	896.75
	IL-PC3-31	605.15
	IL-PC4-41	957.33
	IL-PC5-41	990.85
Routine Permit	IL-RS3-34	965.58
	IL-RS4-38	907.06
	IL-RS5-50	783.37
	IL-RS6-60	891.10

Distributed Live-Load Effects

The live-load effects are distributed with multiple-lane loading factor, 0.35. Table 7.1-2 displays the distributed mid-span moments for an interior beam.

Table 7.1-2. Distributed Live-Load Effects (LL)

Load Rating	Live Loads	Live-Load Effects (Kip-ft)
Design	HL-93	359.31
Legal	IL-PS2-21	228.56
	IL-PS3-31	326.87
	IL-PS4-34.75	342.72
	IL-PS4-28	262.54
	IL-PS5-36	264.17
	IL-PS6-35.75	313.86

	IL-PS7-39.75	313.86
	IL-PC3-31	211.80
	IL-PC4-41	335.07
	IL-PC5-41	346.80
Routine Permit	IL-RS3-34	337.95
	IL-RS4-38	317.47
	IL-RS5-50	274.18
	IL-RS6-60	311.89

***Nominal resistance*:**

For W27×94:

t_w = 0.49 in.

b_f = 10 in.

D = 26.9 in.

t_f = 0.745 in.

D_w = D - 2t_f = 26.9 − 2×0.745 = 25.41 in.

Web Slenderness Limit

$$\frac{2D_c}{t_w} = \frac{D_w}{t_w} = \frac{25.41}{0.49} = 51.86 < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7 \times \sqrt{\frac{29000}{33}} = 168.97 \quad \text{OK}$$

Flange Limit

$$\frac{I_{yc}}{I_{yt}} = 1 > 0.3 \quad \text{OK}$$

Location of Plastic Neutral Axis (PNA)

$$\overline{Y} = \frac{D_w}{2} = \frac{25.41}{2} = 12.71\ \text{in}, \quad \text{from bottom of the top flange to PNA.}$$

Top and Bottom Flanges

$$P_c = P_t = F_y b_f t_f = 33 \times 10 \times 0.745 = 245.85\ \text{Kips}$$

$$d_t = d_c = \frac{(t_f + D_w)}{2} = \frac{(0.745 + 25.41)}{2} = 13.08\ \text{in.}$$

Web

$$P_w = F_y D_w t_w = 33 \times 25.41 \times 0.49 = 410.88\ \text{Kips}$$

Plastic Moment

$$M_p = \frac{P_w}{2D_w}[\overline{Y}^2 + (D_w - \overline{Y})^2] + P_c d_c + P_t d_t$$

$$M_p = \{\frac{410.88}{2 \times 25.41}[12.71^2 + (25.41 - 12.71)^2] + 2 \times 245.85 \times 13.08\} \times \frac{1}{12}$$

$$M_p = 753.46\ \text{Kip} - \text{ft}$$

Web Compactness

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

$$\frac{2D_{cp}}{t_w} = \frac{D_w}{t_w} = \frac{25.41}{0.49} = 51.86$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{(0.54\frac{M_p}{R_h M_y} - 0.09)^2} \leq \lambda_{rw}\frac{D_{cp}}{D_c}$$

where:

$$\lambda_{rw}\frac{D_{cp}}{D_c} = 5.7\sqrt{\frac{E}{F_{yc}}}\left(\frac{D_{cp}}{D_c}\right) = 5.7 \times \sqrt{\frac{29000}{33}} \times (1) = 168.97$$

$$R_h = 1$$

$$M_y = F_y S_z = 33 \times 243 \times \frac{1}{12} = 668.25 \text{ Kip} - \text{ft}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{33}}}{(0.54\frac{753.46}{1 \times 668.25} - 0.09)^2} = 110.12$$

Use 110.12

$$\frac{2D_{cp}}{t_w} = 51.86 \leq \lambda_{pw(D_{cp})} = 110.12 \quad \text{OK}$$

The section is compact.

Web Plastification Factor, R_{pc}:

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{753.46}{668.25} = 1.13$$

Nominal resistance:

Sections are considered as continuously being braced at compression flanges.

$$R_n = \varphi_f R_{pc} M_{yc} = \varphi_f \frac{M_p}{M_{yc}} M_{yc} = \varphi_f M_p = 1 \times 753.46 = 753.46 \text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 1.0, for Flexure in Steel Girders

Condition Factor, φ_C = 0.95, for Fair Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridge

Dead-Load Factor, γ_{DC} = 1.25

Wearing-Surface Factor, γ_{DW} = 1.50

The live-load factors used are displayed in Table 7.1-3.

Table 7.1-3. Live-Load Factors (γ_L)

Load Rating	*MBE* Live-Load Factors	Recommended Live-Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:
The rating factors (RF) calculated according to the general equation are displayed in Table 7.1-4.

Table 7.1-4. Rating Factors (RF)

Load Rating	Live Loads	Using *MBE* Live-Load Factors	Using Recommended Live-Load Factors
Design	HL-93	0.85	0.85
		1.10	1.10
Legal	IL-PS2-21	1.60	1.75
	IL-PS3-31	1.13	1.21
	IL-PS4-34.75	1.07	1.15
	IL-PS4-28	1.40	1.51
	IL-PS5-36	1.39	1.51
	IL-PS6-35.75	1.17	1.27
	IL-PS7-39.75	1.17	1.27
	IL-PC3-31	1.74	1.88
	IL-PC4-41	1.09	1.19
	IL-PC5-41	1.06	1.14
Routine Permit	IL-RS3-34	0.95	1.21
	IL-RS4-38	1.02	1.29
	IL-RS5-50	1.17	1.49
	IL-RS6-60	1.03	1.31

Load Rating for Concrete Deck :
Dead-Load Analysis: (Unit Width)
Components and Attachments, DC

Concrete Slab:

$$1.0 \times \frac{6}{12} \times 0.15 = 0.075 \text{ Kip/ft}$$

Curb:

$$\frac{(6+8) \times 9}{2} \times \frac{1}{144} \times 1.0 \times 0.15 \times \frac{2}{25.33} = 0.0052 \text{ Kip/ft}$$

Railings:

$$(0.0066 + 0.013 + 0.00625) \times 1.0 \times 1.06 \times \frac{2}{25.33} = 0.0022 \text{ Kip/ft}$$

The decks are modeled as simply supported beam with the effective span as same as the distance of the supporting girders because the construction is non-composite.

$$\mathrm{M_{DC}} = \frac{1}{8} \times (0.075 + 0.0052 + 0.0022) \times 3.5^2 = 0.13 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\mathrm{DW} = 1 \times \frac{2}{12} \times 0.14 = 0.023 \text{ Kip/ft}$$

$$\mathrm{M_{DW}} = \frac{1}{8} \times 0.023 \times 3.5^2 = 0.035 \text{ Kip} - \text{ft}$$

Live-Load Analysis:

Undistributed Live-Load Effects

The undistributed live-load effects, which are displayed in Table 7.1-5, are calculated with axle loads of Illinois design-, legal-, and permit-trucks. Dynamic load allowance of 33% is included in the calculation.

Table 7.1-5. Undistributed Live-Load Effects

Load Rating	Live Loads	Live-Load Effects (Kip-ft)
Design	HL-93	18.62
Legal	IL-PS2-21	14.55
	IL-PS3-31	14.90
	IL-PS4-34.75	12.80
	IL-PS4-28	10.47
	IL-PS5-36	9.89
	IL-PS6-35.75	10.47
	IL-PS7-39.75	10.47
	IL-PC3-31	12.80
	IL-PC4-41	14.66
	IL-PC5-41	13.03
Routine Permit	IL-RS3-34	15.71
	IL-RS4-38	12.80
	IL-RS5-50	13.38
	IL-RS6-60	13.97

Equivalent Lane Width

Equivalent Strip Width:

E_s = 26 +6.6S

S= Spacing of Supporting Components (ft) = 3.5 ft

E_s = 26 + 6.6(3.5) = 49.1 in. =4.09 ft

Distributed Live-Load Effects

The calculated axle loads are converted over transverse equivalent strip width and displayed in Table 7.1-6. In this conversion, the multiple-presence factor for one-lane loading 1.2 is included.

Table 7.1-6. Distributed Live-Load Effects

Load Rating	Live Loads	Live-Load Effects (Kip-ft)
Design	HL-93	5.46
Legal	IL-PS2-21	4.27
	IL-PS3-31	4.37
	IL-PS4-34.75	3.76
	IL-PS4-28	3.07
	IL-PS5-36	2.90
	IL-PS6-35.75	3.07
	IL-PS7-39.75	3.07
	IL-PC3-31	3.76
	IL-PC4-41	4.30
	IL-PC5-41	3.82
Routine Permit	IL-RS3-34	4.61
	IL-RS4-38	3.76
	IL-RS5-50	3.93
	IL-RS6-60	4.10

Nominal resistance:

$$c = \frac{A_s\, f_y}{0.85 f_c' \beta_1 b}$$

1/2 in. Diameter @ 0.5 ft. For unit width:

$A_s = 0.2 \times 2 = 0.4\ \text{in.}^2/\text{ft}$

$f_y = 33\ \text{Ksi}$

$f_c' = 2.5\ \text{Ksi}$

$\beta_1 = 0.85$

b = be = 12 in., Rectangular Section Behavior Assumed

$$c = \frac{0.4\ \times\ 33}{0.85 \times 2.5 \times 0.85 \times 12} = 0.61\ \text{in.}$$

$a = \beta_1 c = 0.85 \times 0.61 = 0.52\ \text{in.}$

a < Slab Thickness=6 in., the assumption of the rectangular section behavior is valid.

Distance from extreme compression fiber to C.G. of Steel, d_s:

$$d_s = \text{Slab Thickness} - \text{Deck bottom cover} - \frac{\text{Rebar Diameter}}{2} = 6 - 0.75 = 5.25\ \text{in}$$

Nominal Flexure Resistance, M_n:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[0.4 \times 33 \times \left(5.25 - \frac{0.52}{2}\right)\right] \times \frac{1}{12} = 5.49\ \text{Kip} - \text{ft}$$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$\varepsilon_c = 0.003$

$d = d_s = 5.25\ \text{in.}$

$$\varepsilon_t = \frac{(5.25 - 0.61) \times 0.003}{0.61} = 0.023 > 0.005$$

The section is tension-controlled and the Resistance factor φ shall be taken as 0.9.

Minimum Reinforcement

Amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$$M_r = \varphi M_n = 0.9 \times 5.49 = 4.94\ \text{Kip} - \text{ft}$$

1.33Mu :

$$1.33M_u = 1.33(1.75M_{HL-93} + 1.25M_{DC} + 1.50M_{DW})$$

$$1.33M_u = 1.33 \times (1.75 \times 5.46 + 1.25 \times 0.13 + 1.50 \times 0.035) = 12.99\ \text{Kip} - \text{ft}$$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

A non-composite section is designed to resist all the loads; S_{nc} is substituted for Sc. In this case, f_{cpe} = 0.

$$M_{cr} = S_{nc} f_r$$

$$S_{nc} = \frac{I}{y_t}$$

Moment of Inertia of Uncracked Section (Neglecting Reinforcement Steel)

$$I = \frac{1}{12} \times 12 \times 6^3 = 216\ \text{in.}^4$$

Distance from the neutral axis of the uncracked section to the extreme tension fiber

$$y_t = \frac{6}{2} = 3\ \text{in.}$$

$$S_{nc} = \frac{I}{y_t} = \frac{216}{3} = 72\ \text{in.}^3$$

$$f_r = 0.37\sqrt{f'_c} = 0.37 \times \sqrt{2.5} = 0.59\ \text{Ksi}$$

$$M_{cr} = S_{nc} f_r = \frac{1}{12} \times 72 \times 0.59 = 3.54\ \text{Kip} - \text{ft}$$

$$1.2M_{cr} = 1.2 \times 3.54 = 4.25\ \text{Kip} - \text{ft} < M_r = 4.94\ \text{Kip} - \text{ft}$$

The section meets the requirements for minimum reinforcement.

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.9, for Tension-Controlled Reinforced-Concrete Slab in Flexure

Condition Factor, φ_C = 0.95, for Fair Condition

System Factor, φ_S = 1.0, for Reinforced-Concrete Slab

Dead-Load Factor, γ_{DC} = 1.25

Wearing-Surface Factor, γ_{DW} = 1.50
The used live-load factors are displayed in Table 7.1-7.

Table 7.1-7. Live-Load Factors (γ_L)

Load Rating	*MBE* Live-Load Factors	Recommended Live-Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:
The rating factors (RF) calculated according to the general equation are displayed in Table 7.1-8. As shown in Table 7.1-8, the RF for legal load rating is less than 1.0. Therefore, the permit-load rating is not applicable.

Table 7.1-8. Rating Factors (RF)

Load Rating	Live Loads	Using *MBE* Live-Load Factors	Using Recommended Live-Load Factors
Design	HL-93	0.47	0.47
		0.61	0.61
Legal	IL-PS2-21	0.72	0.77
	IL-PS3-31	0.71	0.76
	IL-PS4-34.75	0.82	0.88
	IL-PS4-28	1.01	1.09
	IL-PS5-36	1.07	1.14
	IL-PS6-35.75	1.01	1.09
	IL-PS7-39.75	1.01	1.09
	IL-PC3-31	0.82	0.88
	IL-PC4-41	0.72	0.77
	IL-PC5-41	0.81	0.87
Routine Permit	IL-RS3-34	N/A	N/A
	IL-RS4-38	N/A	N/A
	IL-RS5-50	N/A	N/A
	IL-RS6-60	N/A	N/A

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

1. Illinois WIM stations are recording truck-weight and configuration data in motion to the 0.01-second time-stamp resolution. This time stamp is satisfactory for calibrating specifications for bridge load rating. However, when funding becomes available, more sites are recommended to have the second lane added for simultaneous recording so that critical loading of trucks in a cluster with short headway can be captured and recorded.
2. WIM stations in Illinois are recommended to be well-maintained with regular calibration for their weighing systems in order to regularly provide high-quality truck-weight data.
3. Illinois truck weights recorded from the 20 current WIM sites are less severe than the trucks weighed in Canadian weigh stations in the 1970s and used for calibration of current AASHTO *BDS* (LRFD) and *MBE* (LRFR).
4. Based on these truck loads recorded at Illinois WIM sites, no change is recommended for the design-load load rating's live-load factors, to be consistent with the live-load factors for bridge design.
5. The live-load factors for legal load rating are recommended in Eq. 6.2-4, for routine-permit load in Eq. 6.3-3, and for special-permit load rating in Eq. 6.4-2, along with their alternative forms to be immediately implementable with current AASHTOWare BrDR. These live-load factors are based on recorded truck loads in Illinois.
6. It is also recommended to continue monitoring truck weights and configurations recorded at the Illinois WIM stations, for their possible changes in both magnitude and volume. These changes in the future may need to trigger further changes to the recommended load rating live-load factors and possibly further localized live-load factors for load rating, for example, for a district. An interval of two years is recommended for such review.
7. Periodical review of issued permits is recommended to monitor possible load growth at least on a biennial basis.
8. WIM data collection at more often congested areas with significant truck traffic is recommended when funding becomes available.

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APPENDIX A: COMPARISON OF ILLINOIS AND CANADIAN TRUCKS

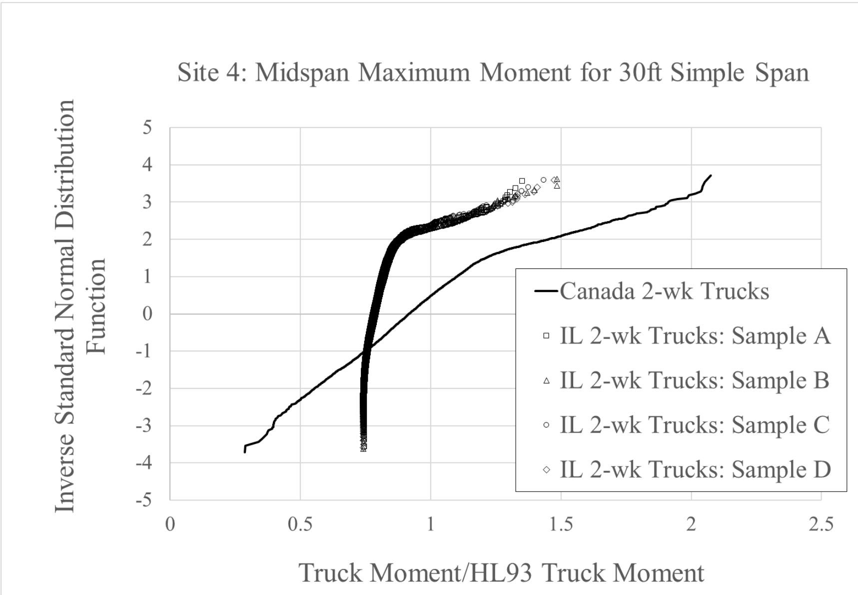


Figure A-1 Moments of Canada and Illinois Trucks at Site 4 for 30ft Span

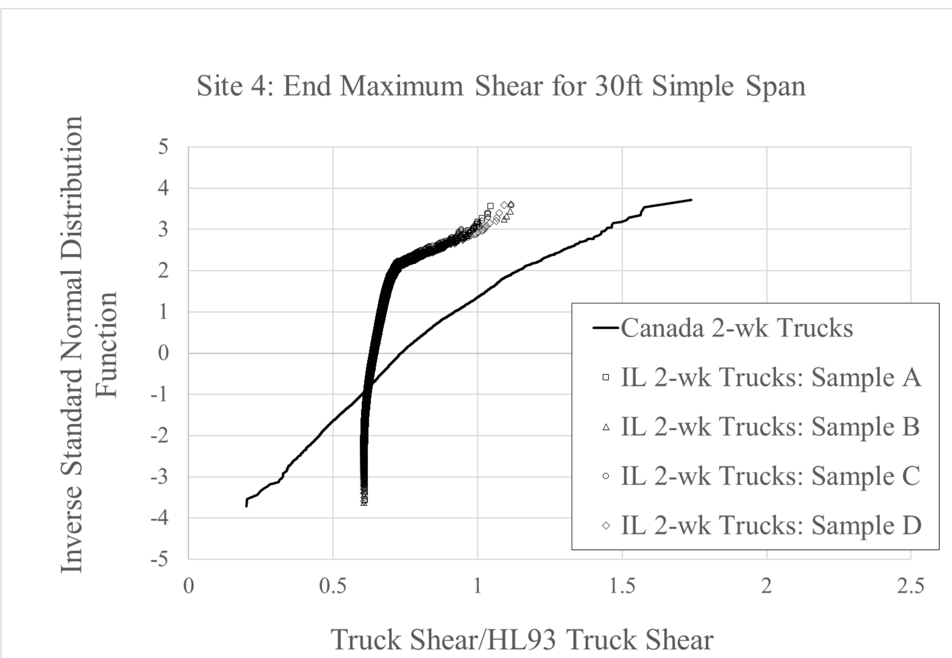


Figure A-2 Shears of Canada and Illinois Trucks at Site 4 for 30ft Span

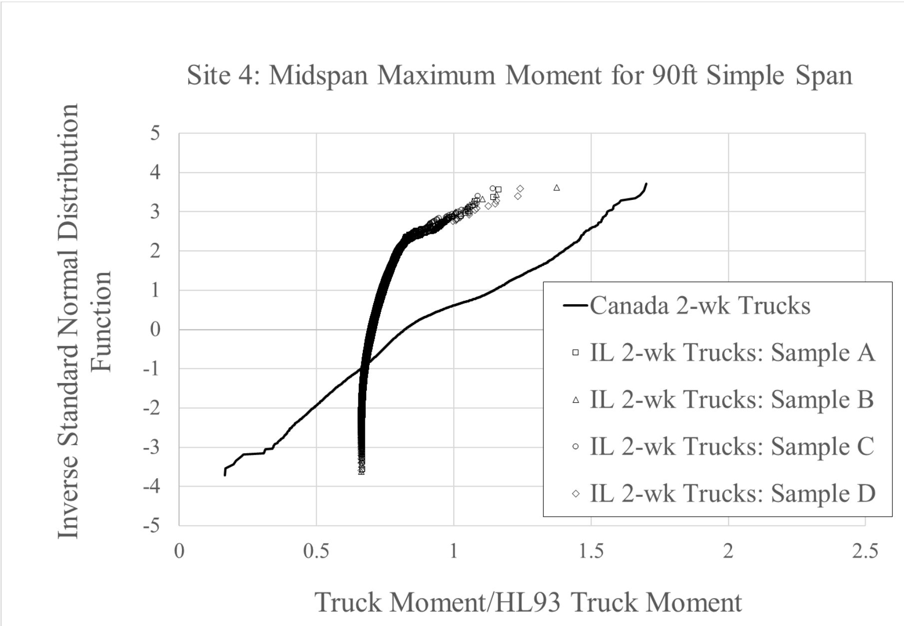


Figure A-3 Moments of Canada and Illinois Trucks at Site 4 for 90ft Span

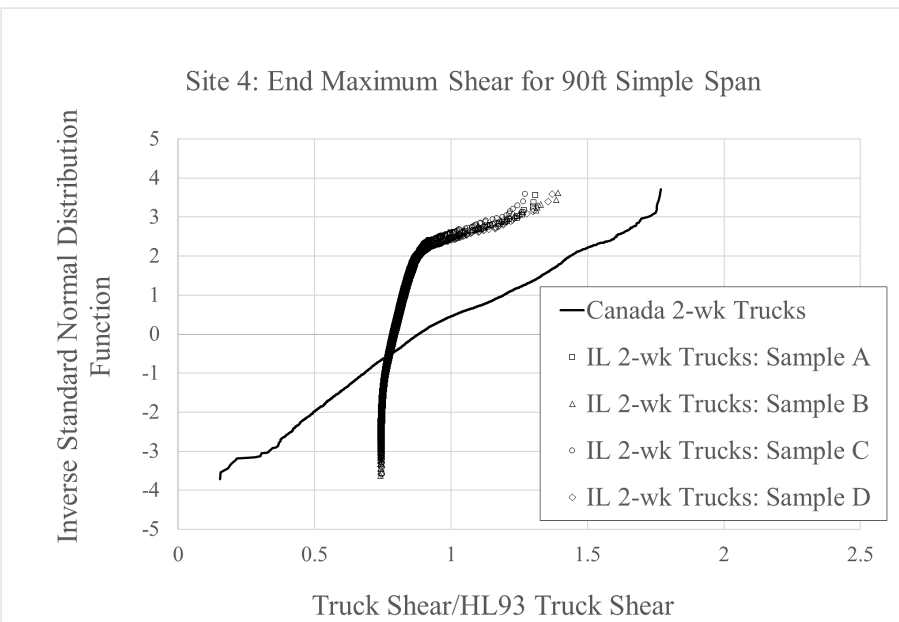


Figure A-4 Shears of Canada and Illinois Trucks at Site 4 for 90ft Span

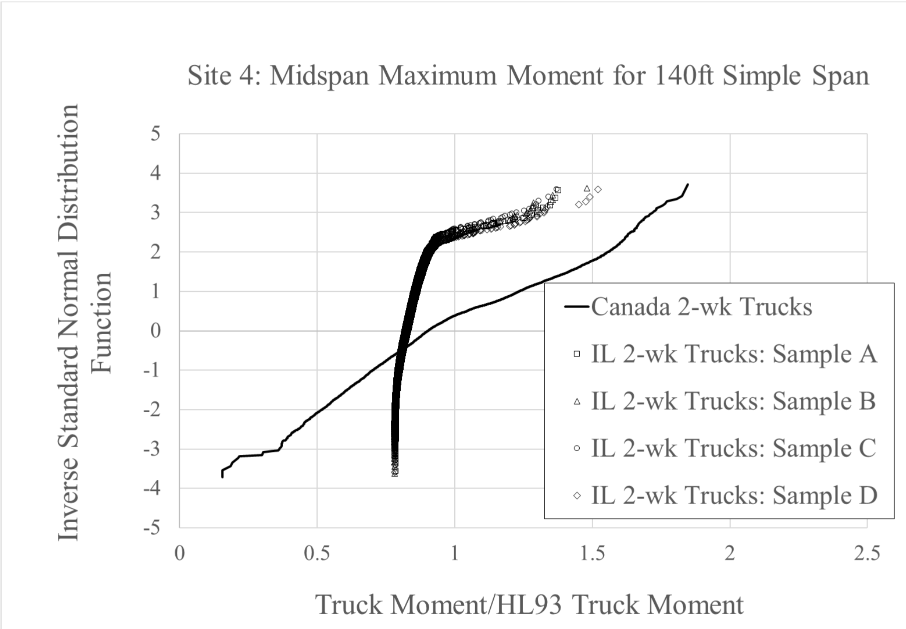


Figure A-5 Moments of Canada and Illinois Trucks at Site 4 for 140ft Span

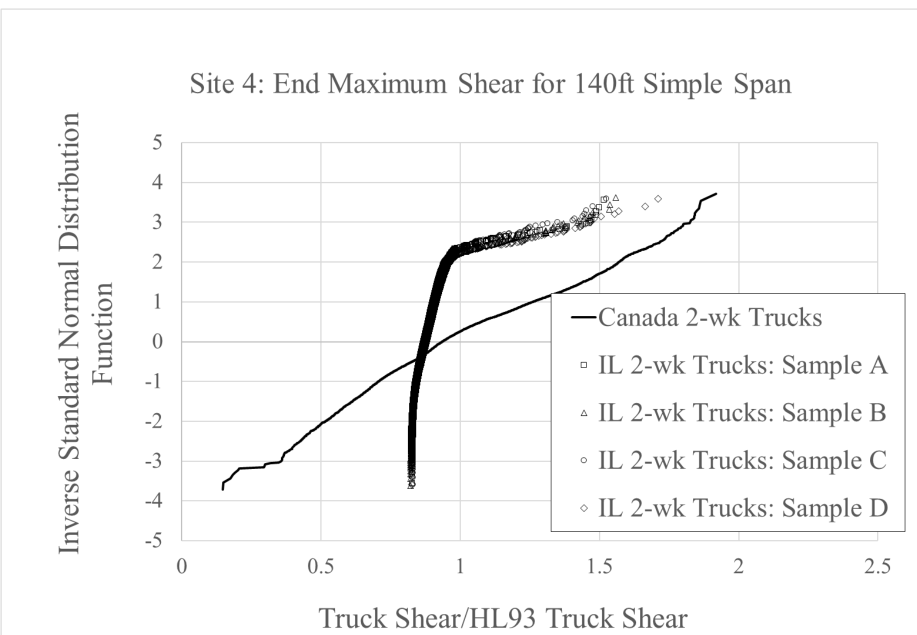


Figure A-6 Shears of Canada and Illinois Trucks at Site 4 for 140ft Span

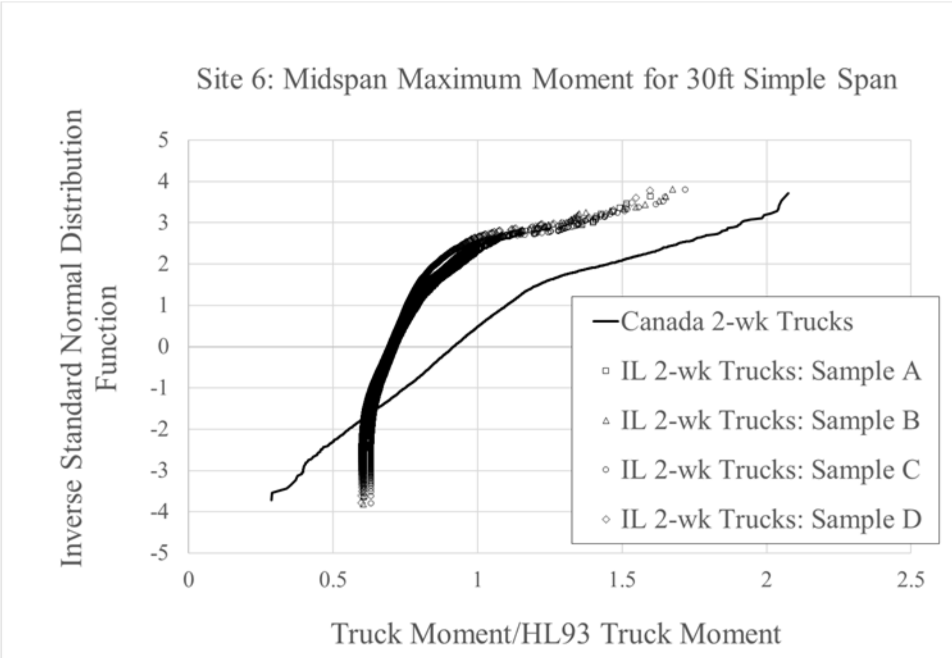


Figure A-7 Moments of Canada and Illinois Trucks at Site 6 for 30ft Span

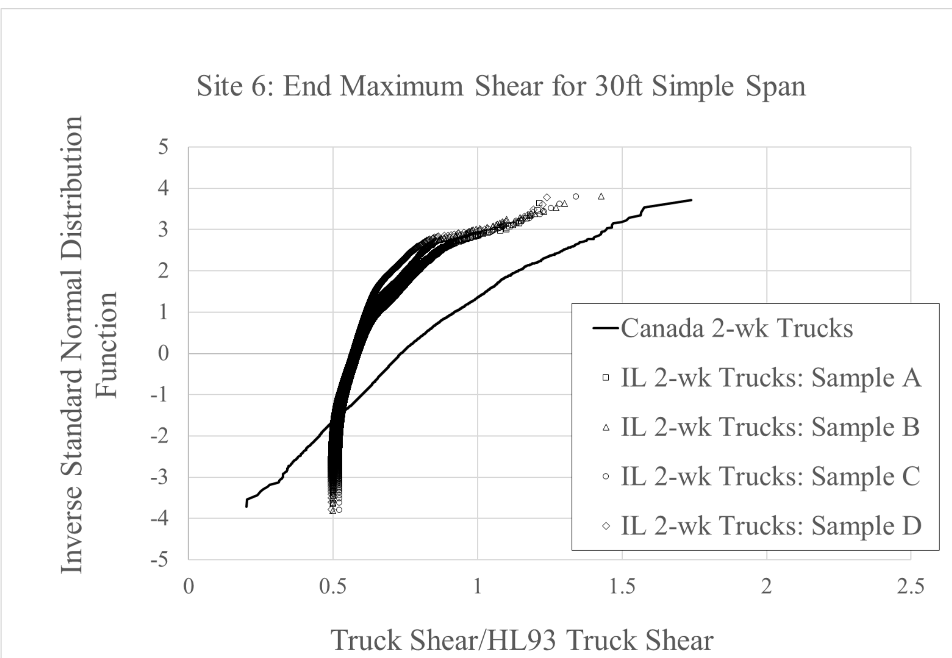


Figure A-8 Shears of Canada and Illinois Trucks at Site 6 for 30ft Span

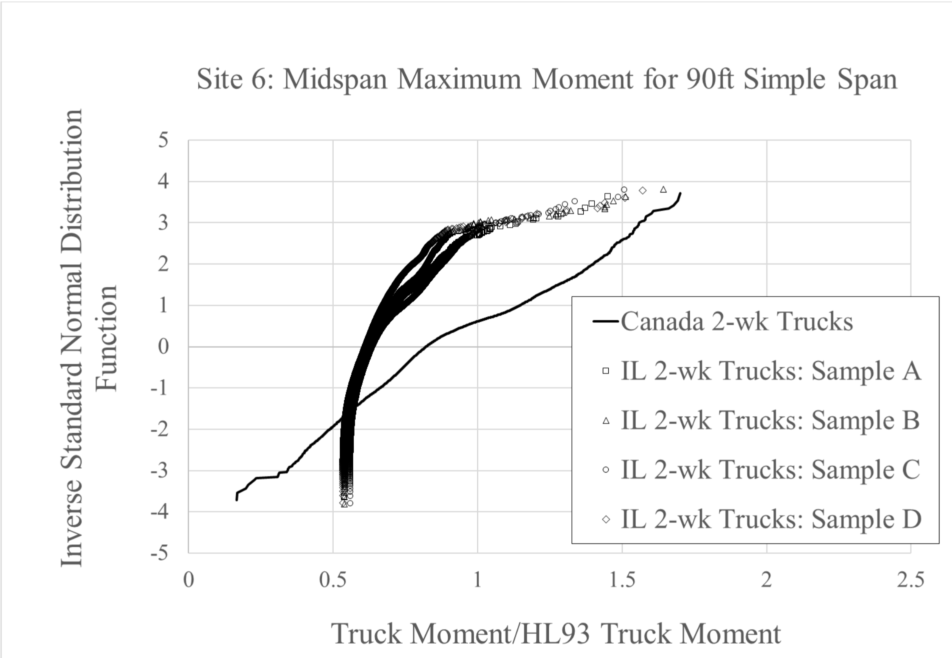


Figure A-9 Moments of Canada and Illinois Trucks at Site 6 for 90ft Span

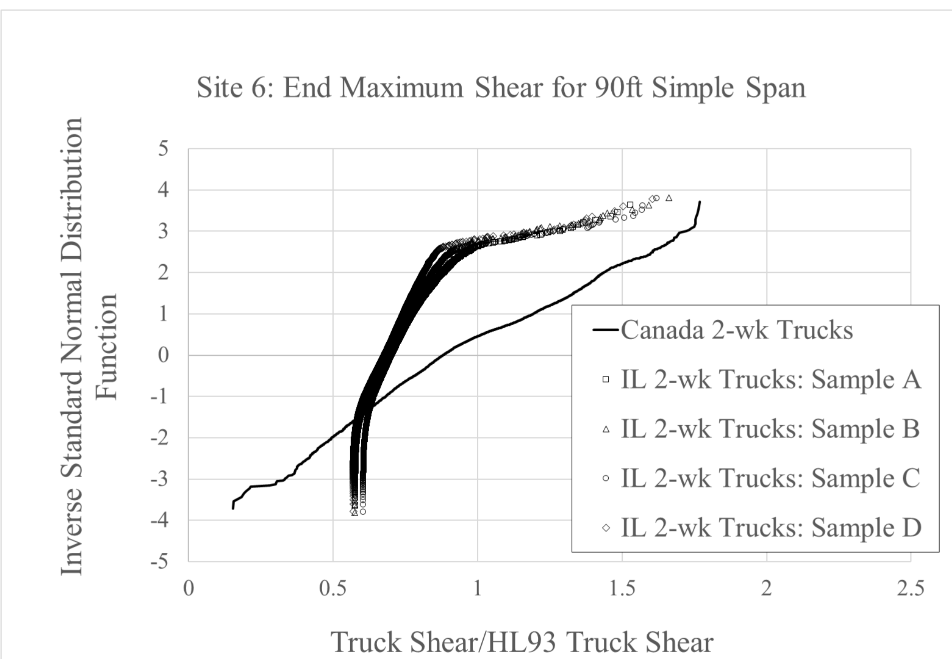


Figure A-10   Shears of Canada and Illinois Trucks at Site 6 for 90ft Span

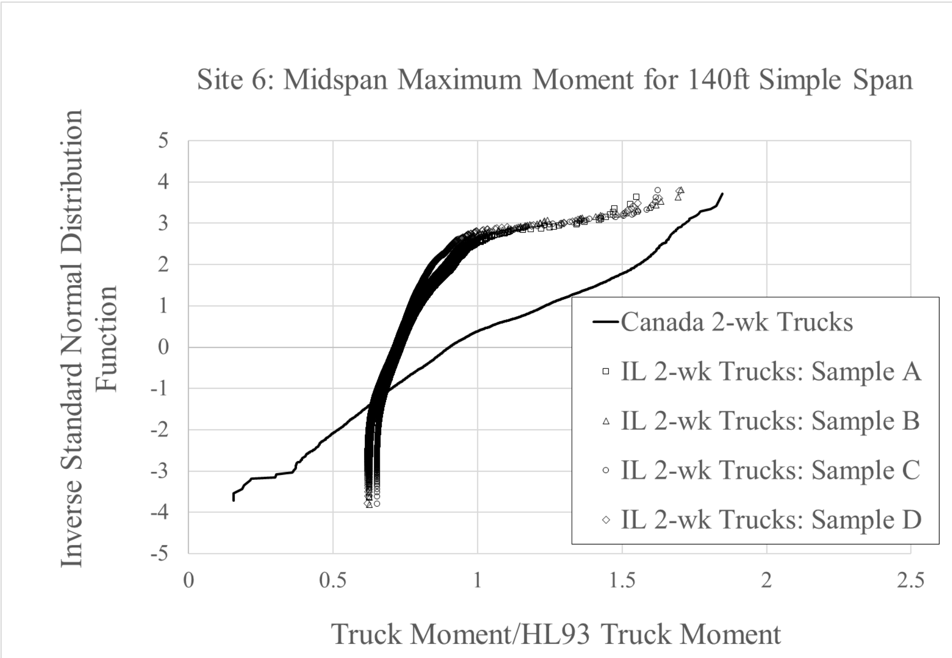


Figure A-11 Moments of Canada and Illinois Trucks at Site 6 for 140ft Span

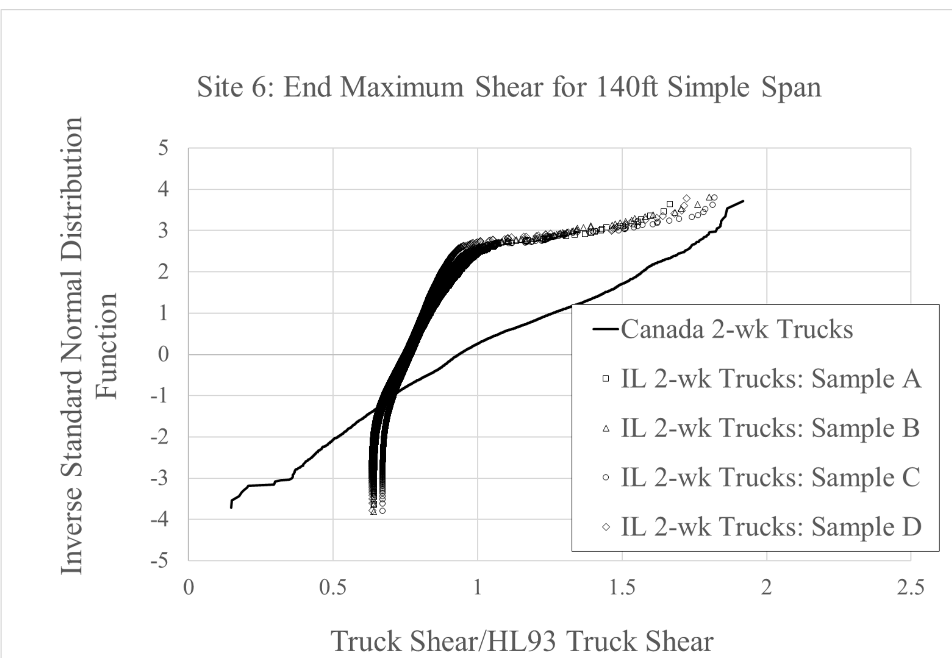


Figure A-12 Shears of Canada and Illinois Trucks at Site 6 for 140ft Span

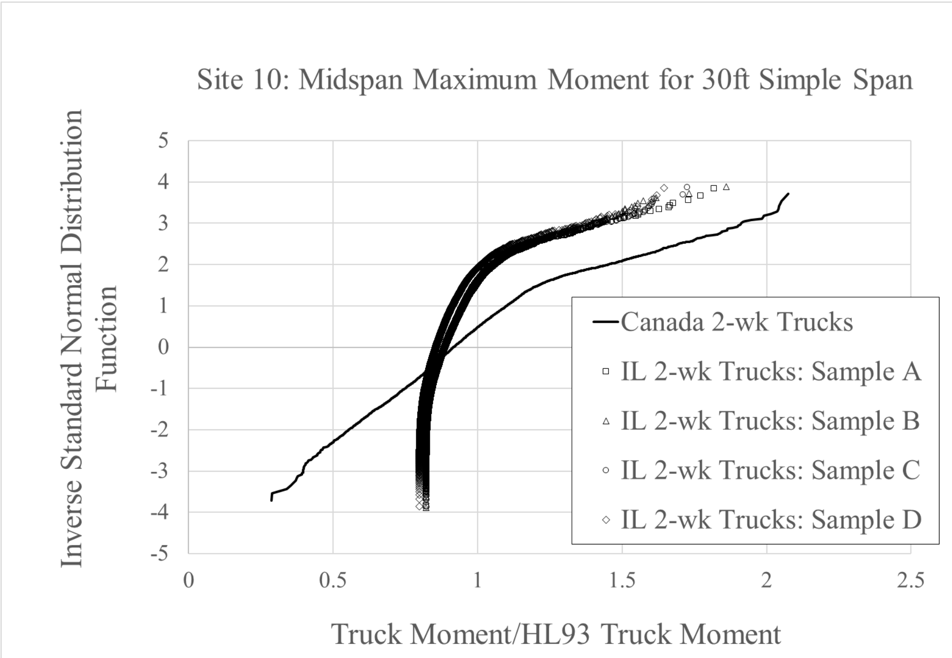


Figure A-13 Moments of Canada and Illinois Trucks at Site 10 for 30ft Span

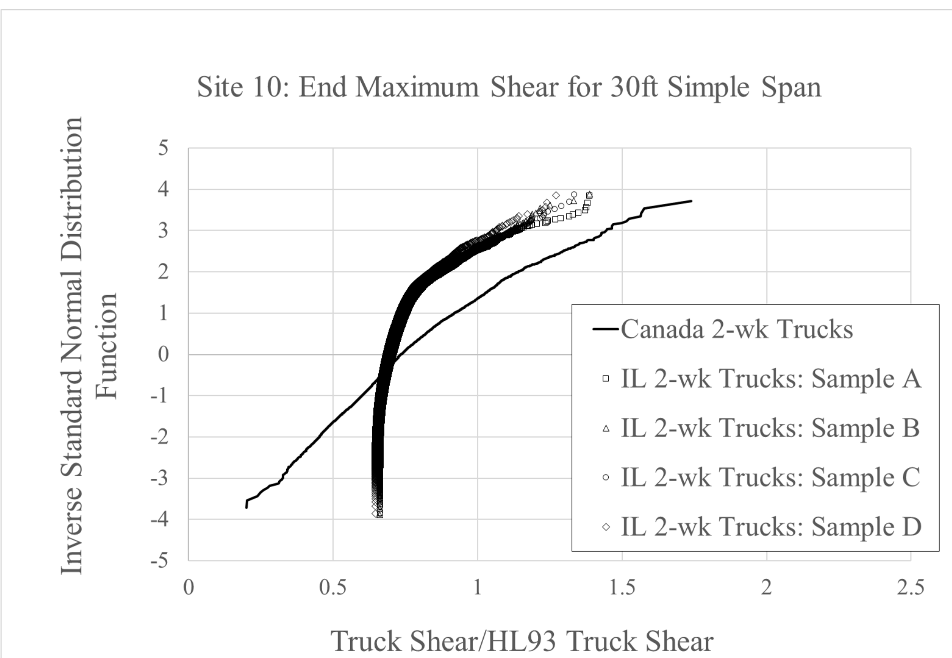
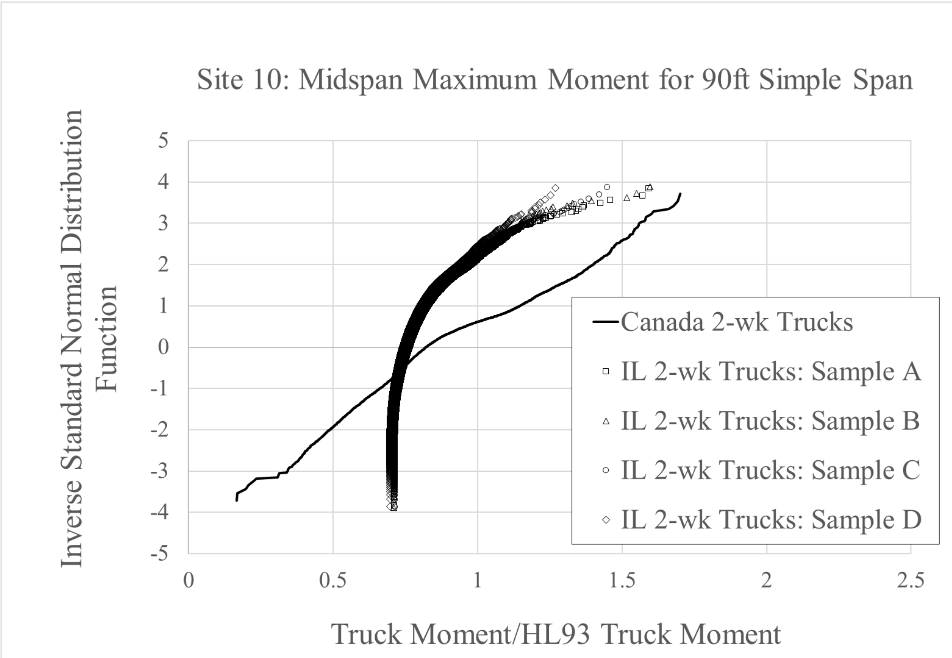


Figure A-14 Shears of Canada and Illinois Trucks at Site 10 for 30ft Span



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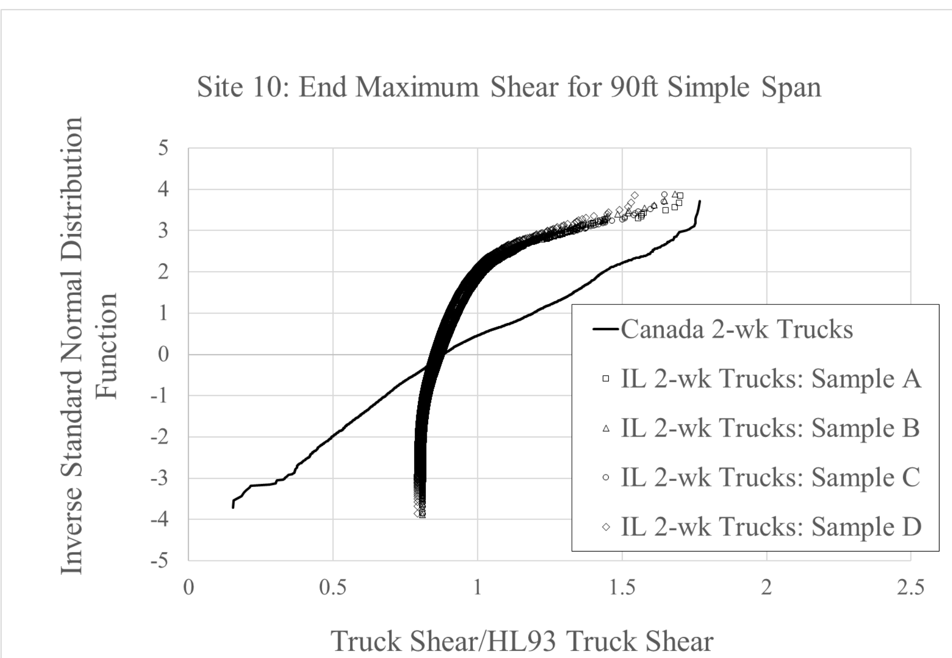


Figure A-16 Shears of Canada and Illinois Trucks at Site 10 for 90ft Span

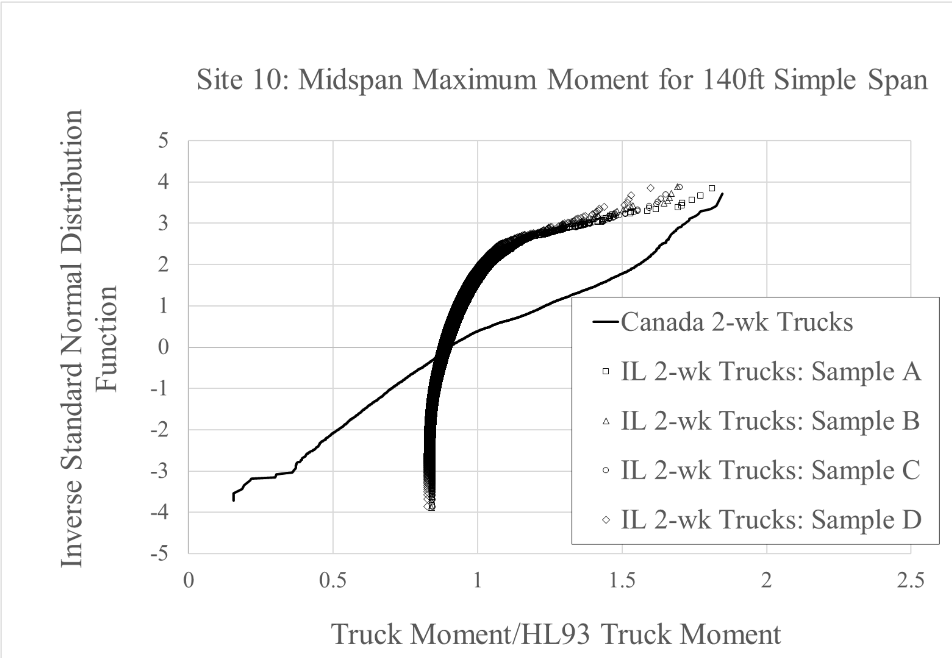
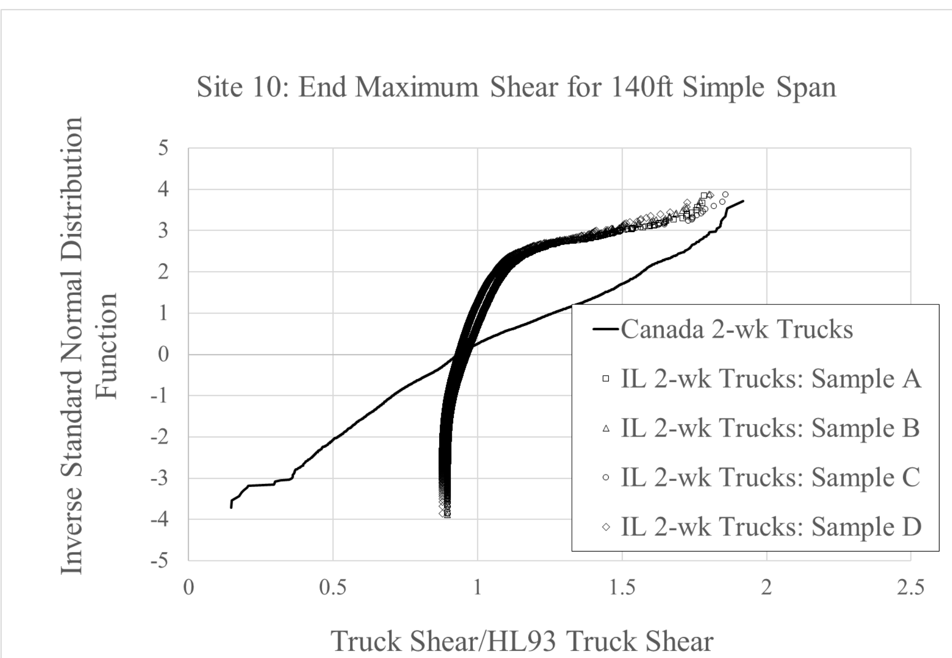


Figure A-17 Moments of Canada and Illinois Trucks at Site 10 for 140ft Span



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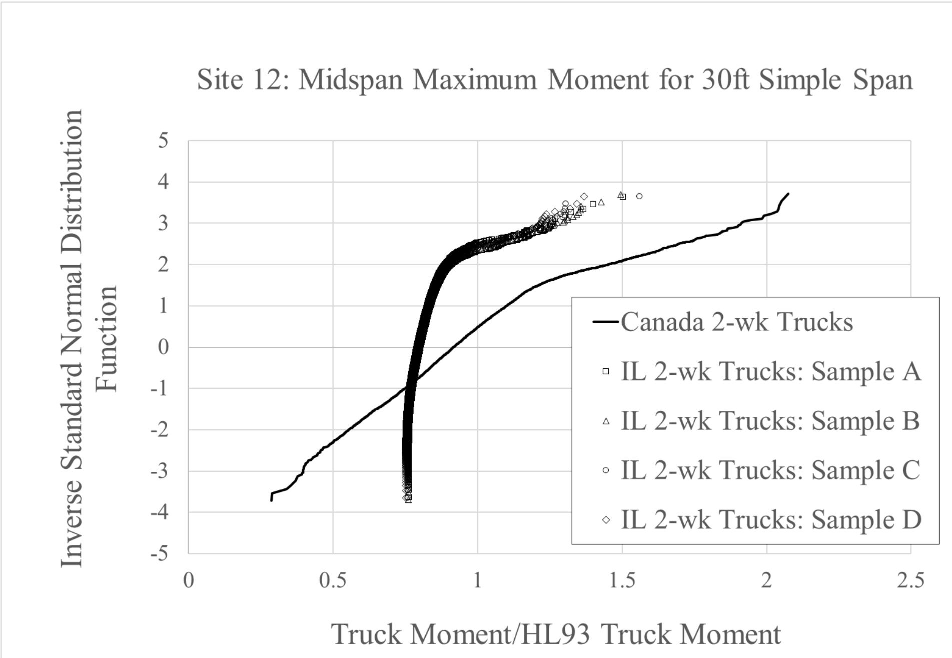


Figure A-19 Moments of Canada and Illinois Trucks at Site 12 for 30ft Span

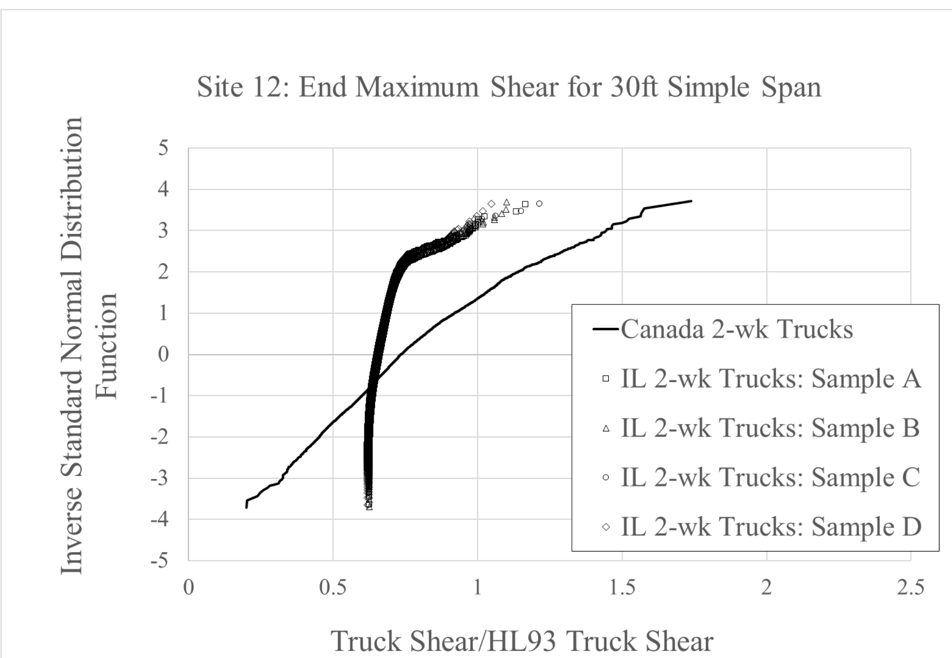
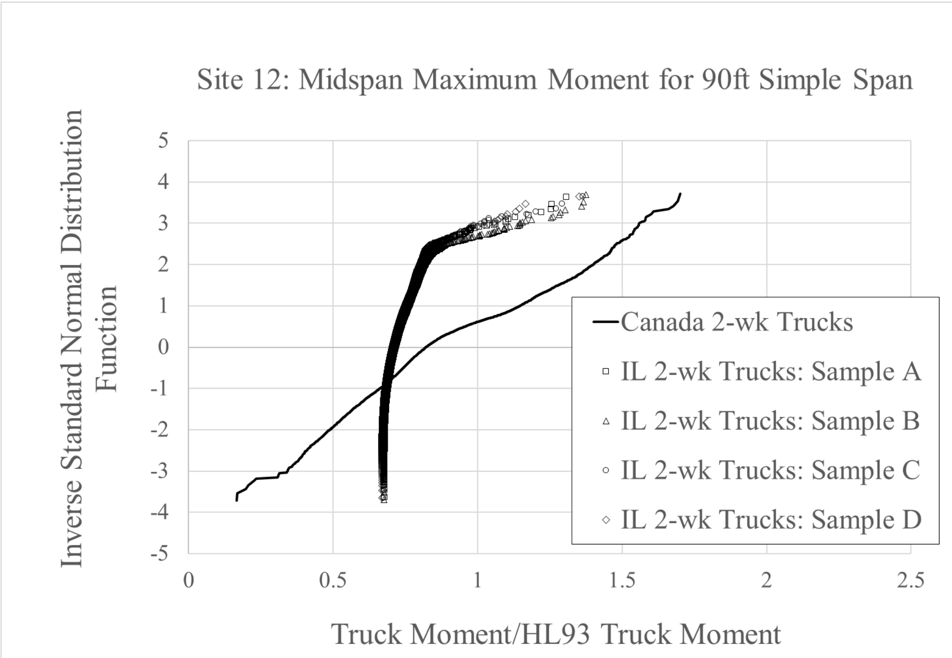


Figure A-20 Shears of Canada and Illinois Trucks at Site 12 for 30ft Span



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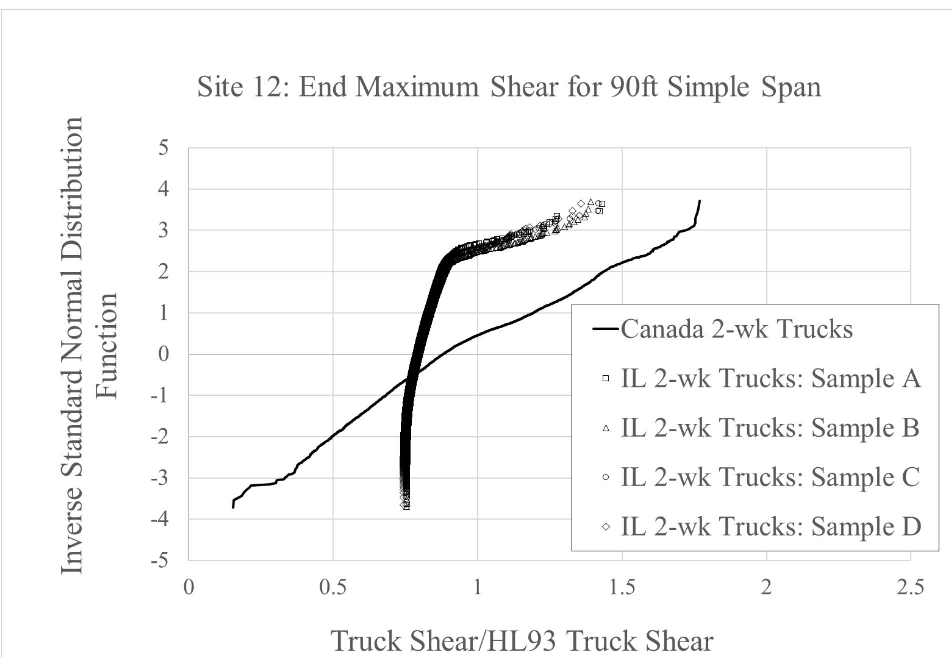

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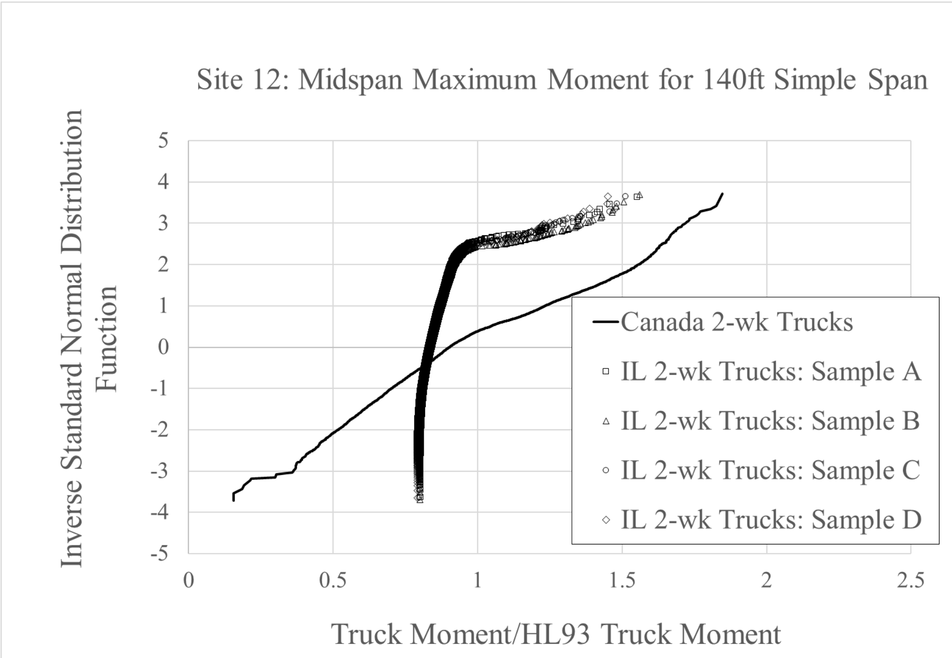


Figure A-23 Moments of Canada and Illinois Trucks at Site 12 for 140ft Span

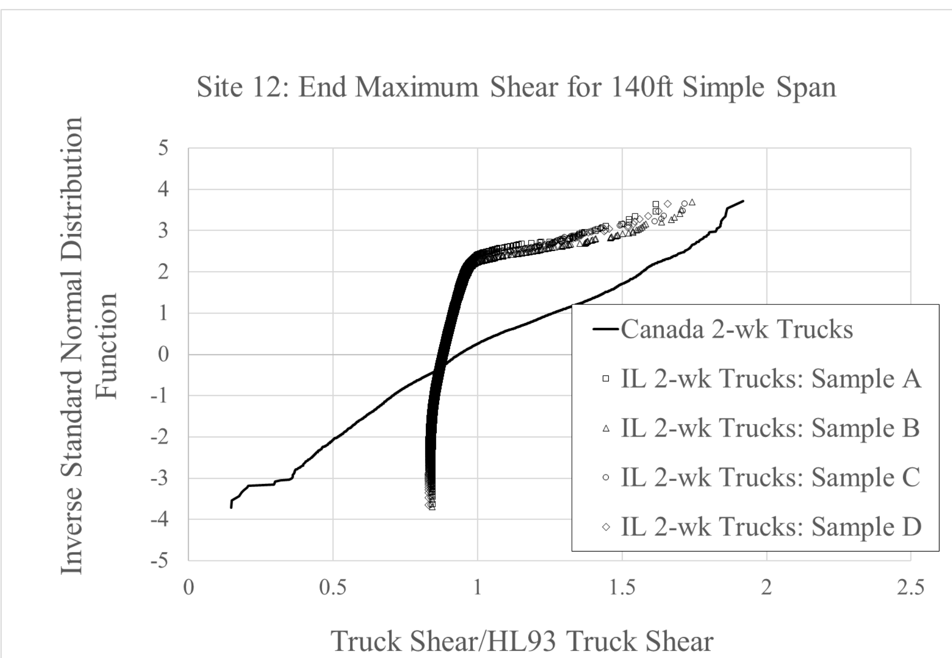


Figure A-24 Shears of Canada and Illinois Trucks at Site 12 for 140ft Span

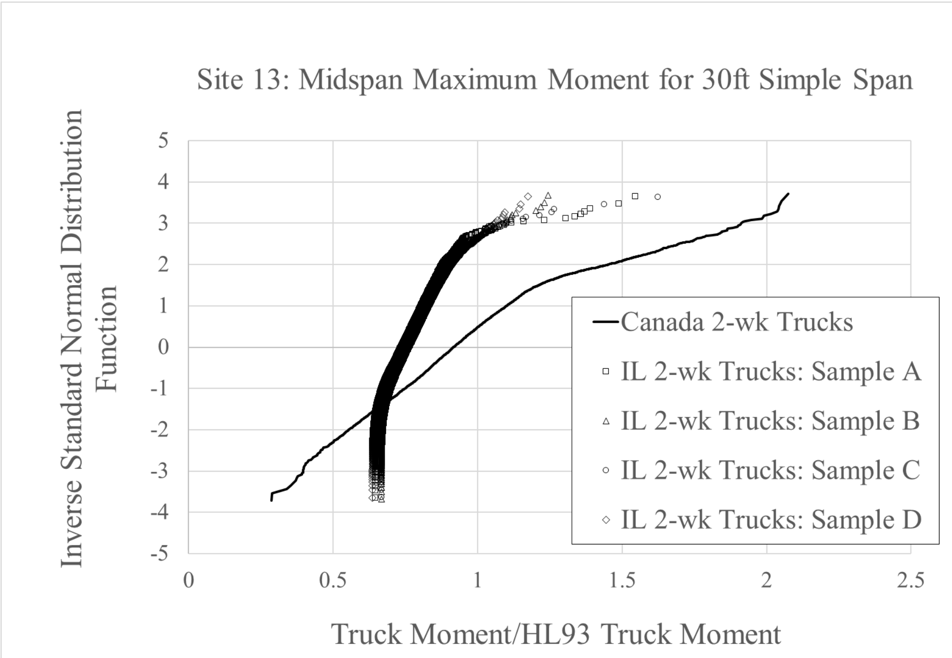


Figure A-25 Moments of Canada and Illinois Trucks at Site 13 for 30ft Span

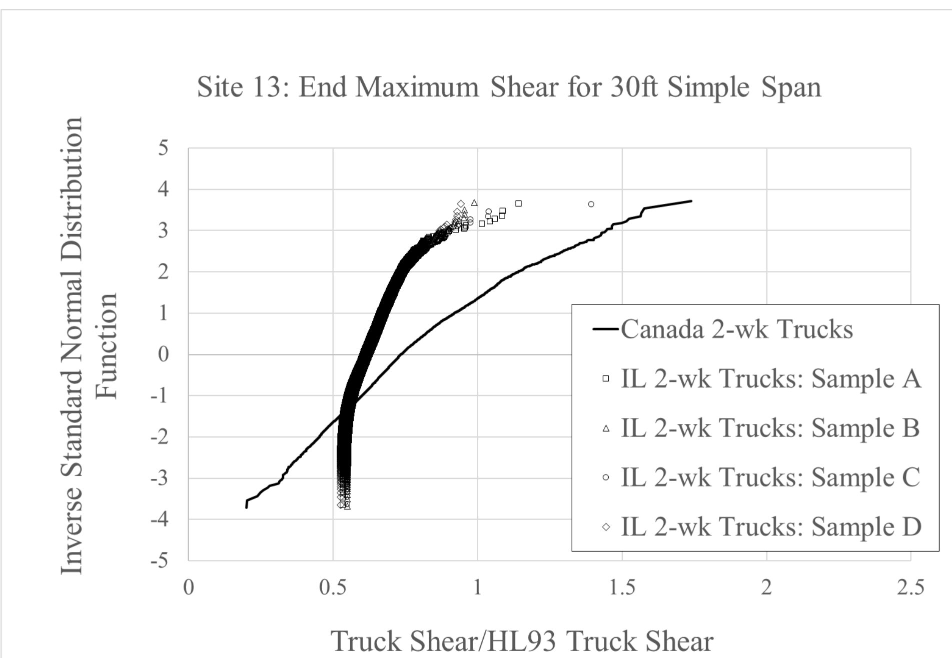


Figure A-26 Shears of Canada and Illinois Trucks at Site 13 for 30ft Span

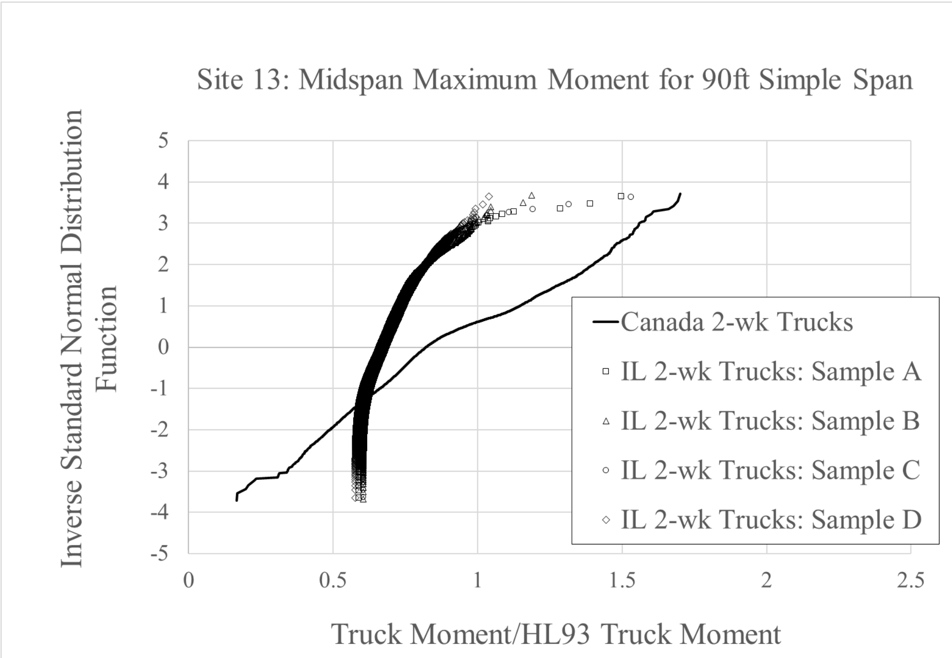
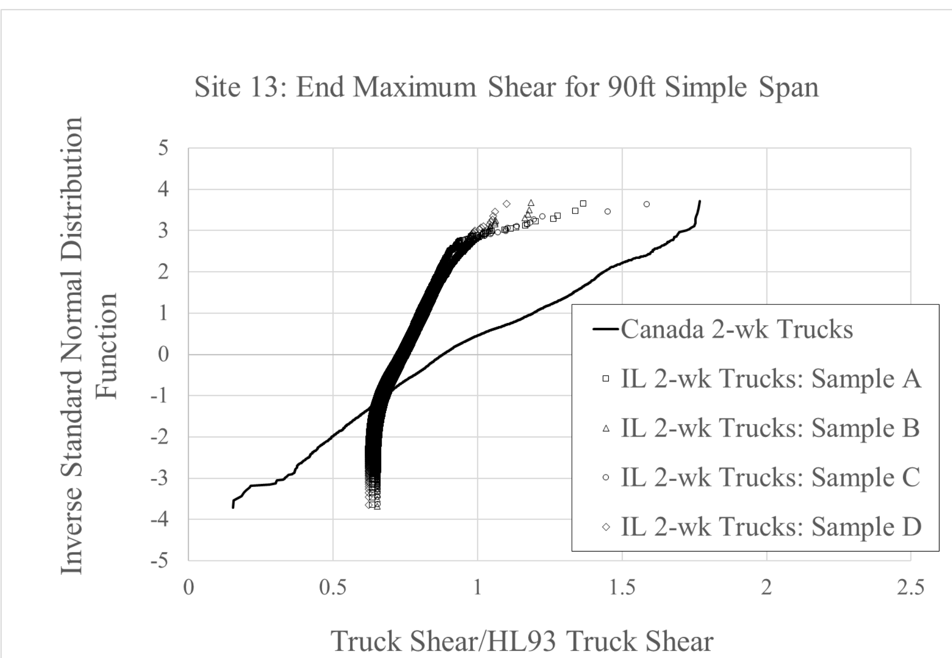


Figure A-27 Moments of Canada and Illinois Trucks at Site 13 for 90ft Span



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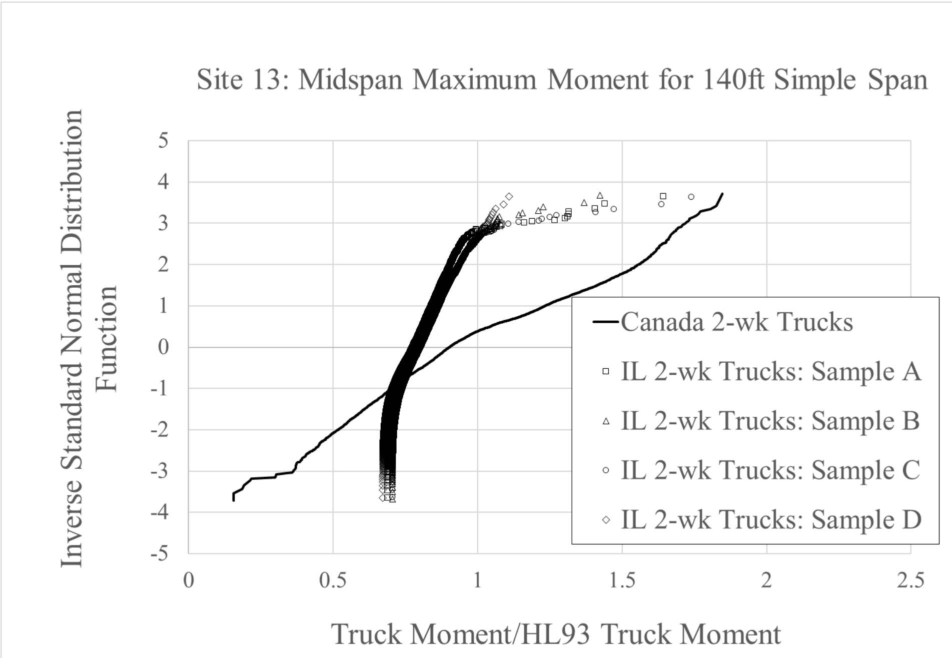


Figure A-29 Moments of Canada and Illinois Trucks at Site 13 for 140ft Span

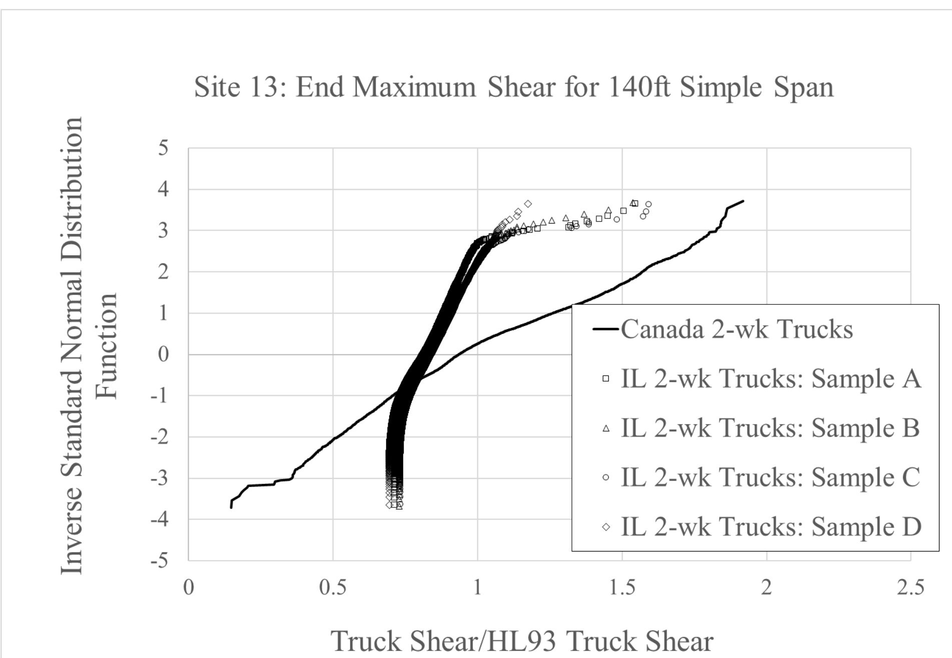


Figure A-30 Shears of Canada and Illinois Trucks at Site 13 for 140ft Span

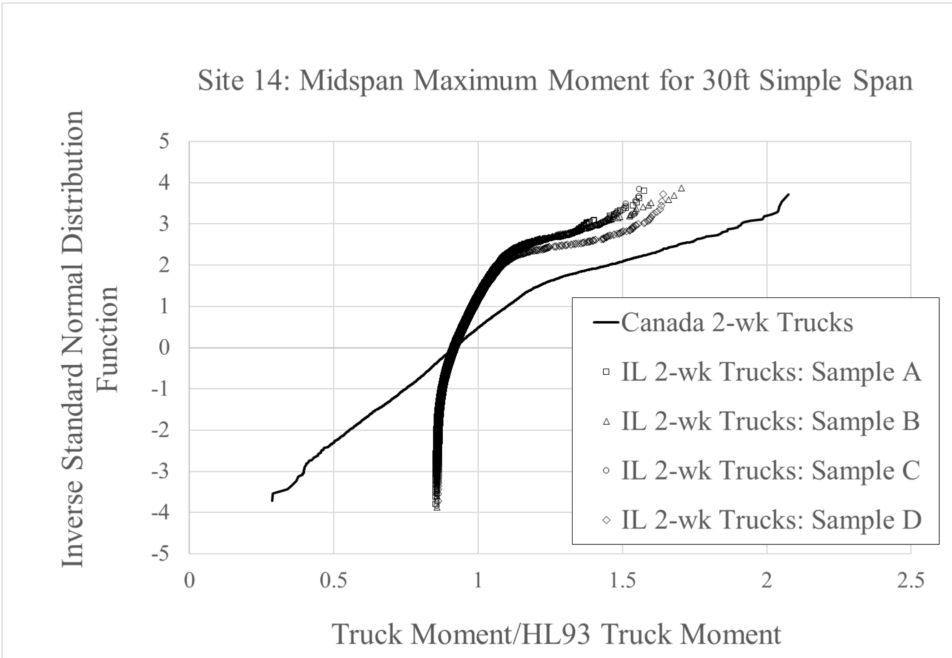
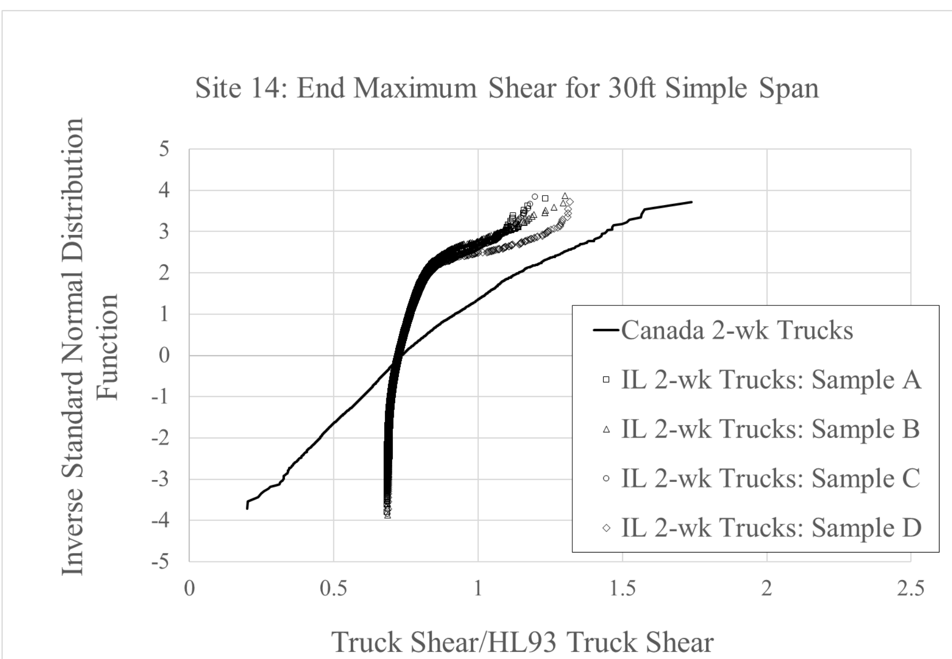


Figure A-31 Moments of Canada and Illinois Trucks at Site 14 for 30ft Span



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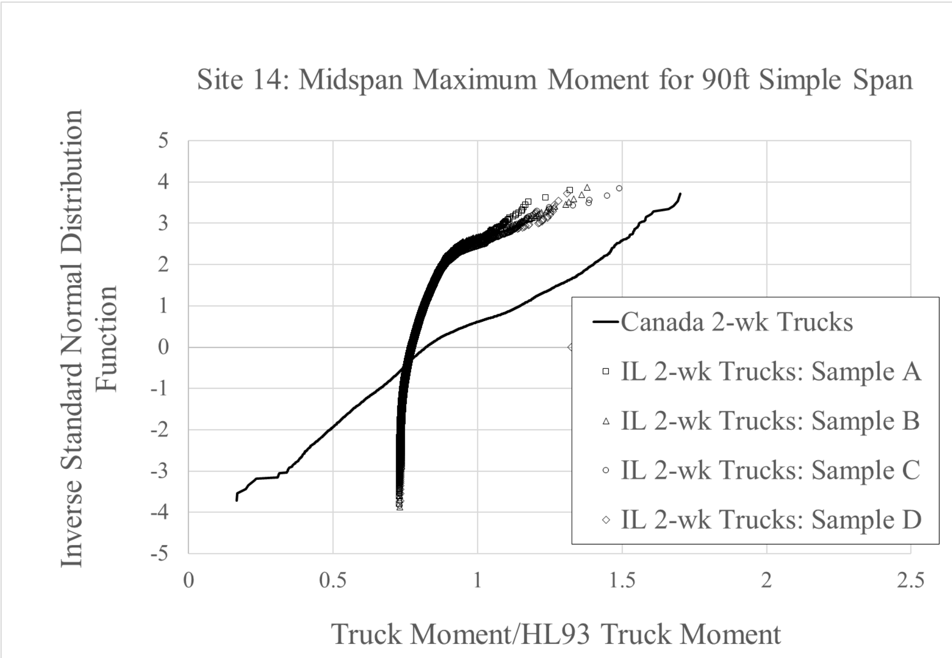

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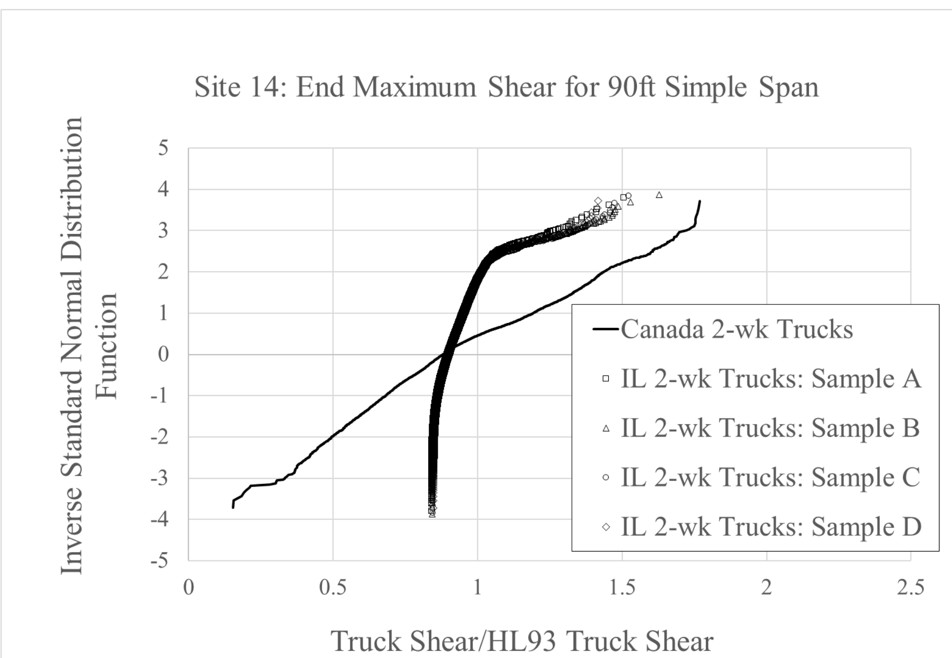


Figure A-34 Shears of Canada and Illinois Trucks at Site 14 for 90ft Span

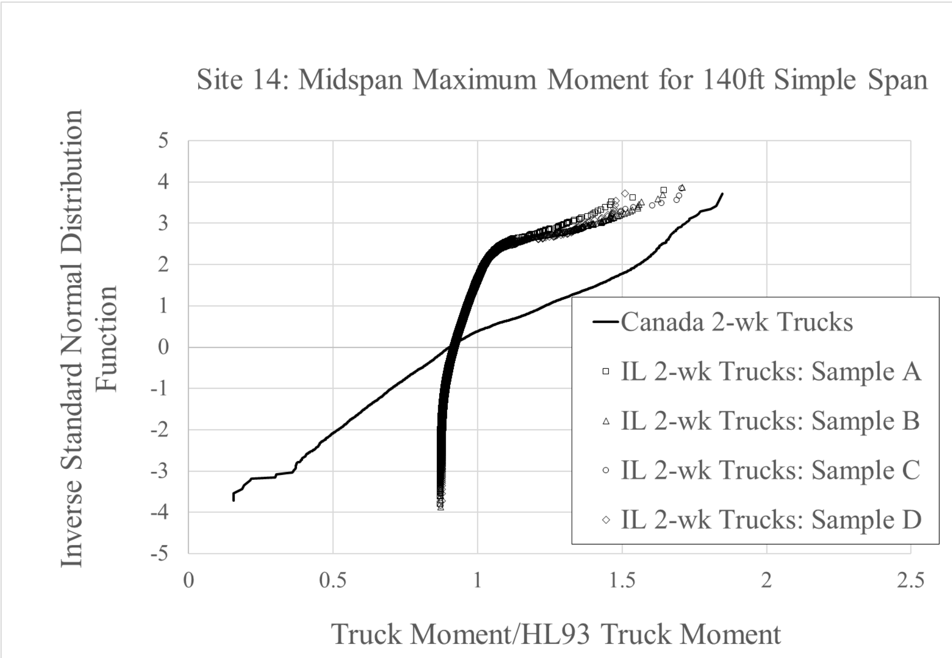


Figure A-35 Moments of Canada and Illinois Trucks at Site 14 for 140ft Span

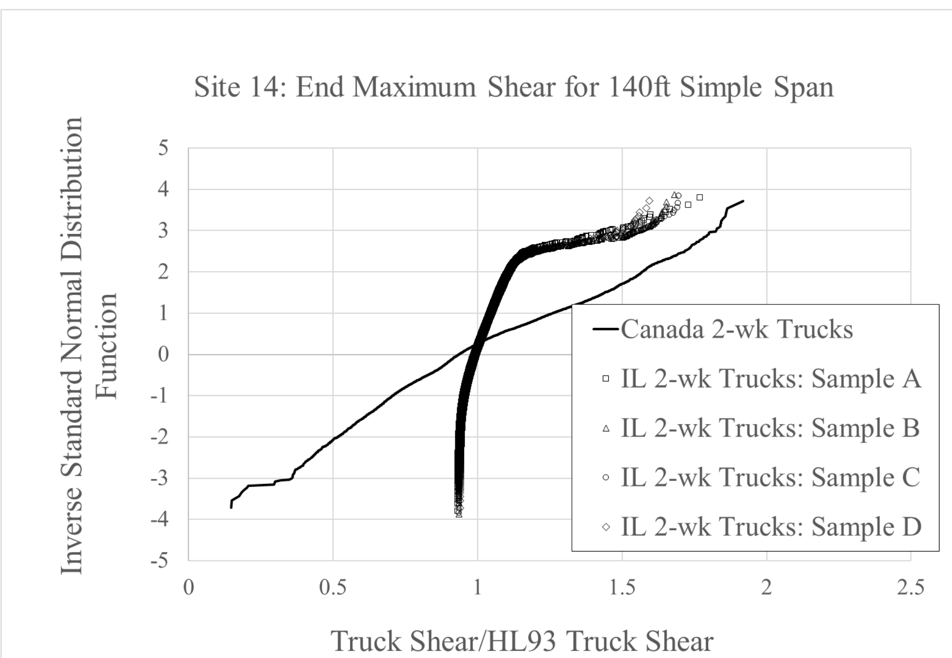
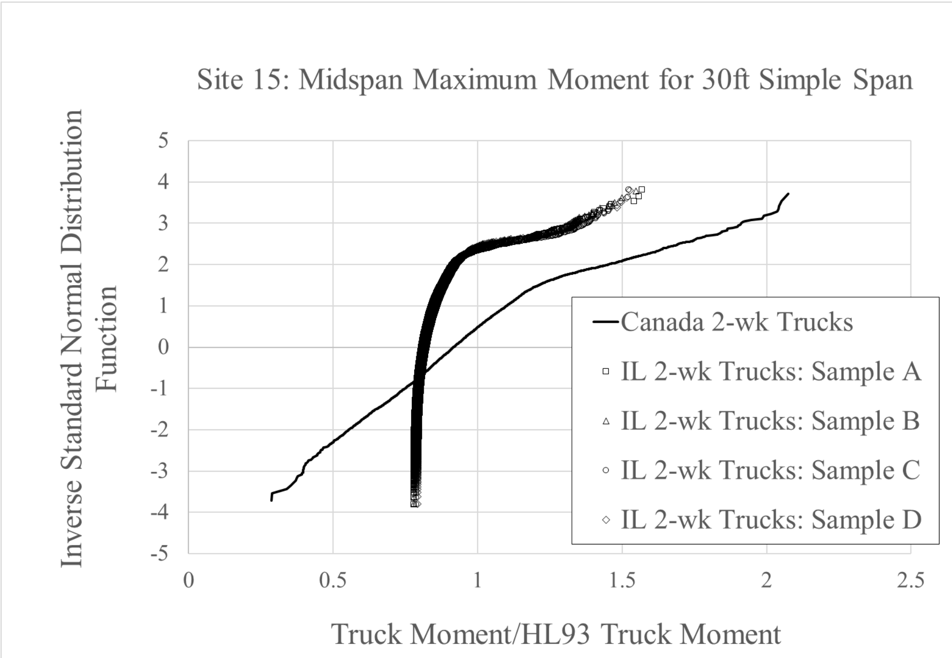


Figure A-36 Shears of Canada and Illinois Trucks at Site 14 for 140ft Span



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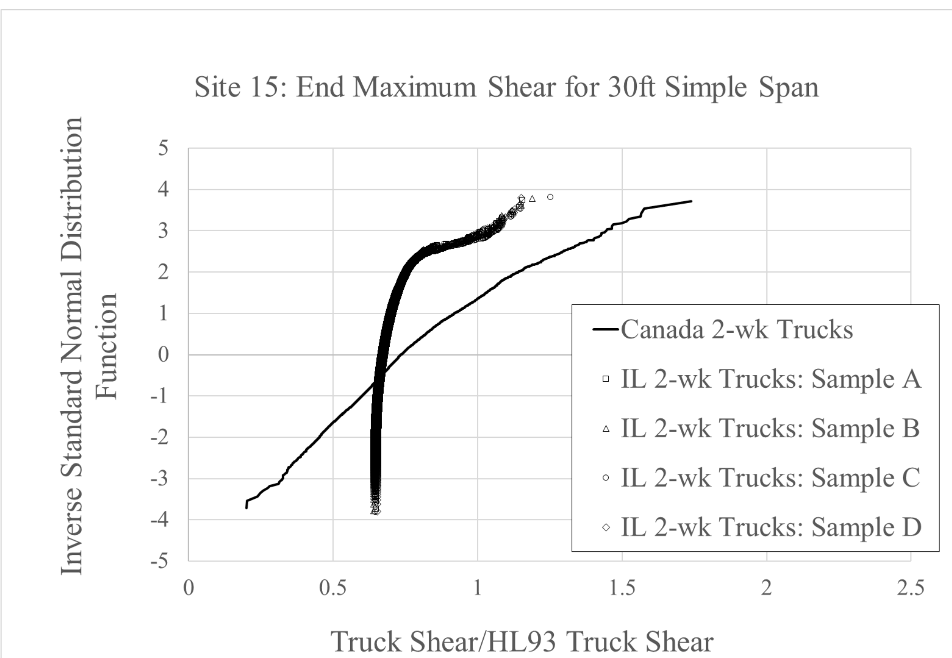

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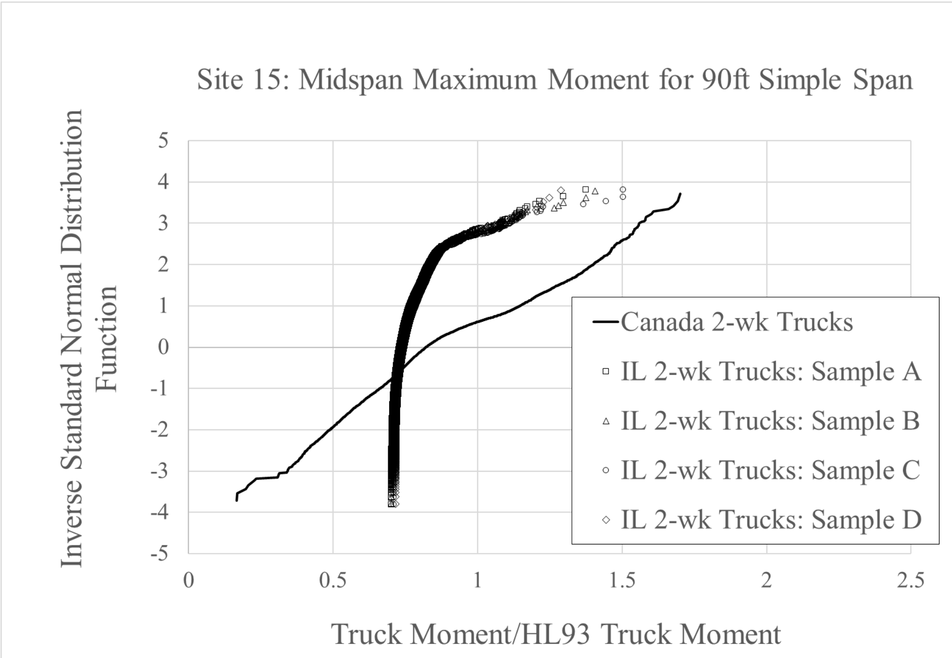

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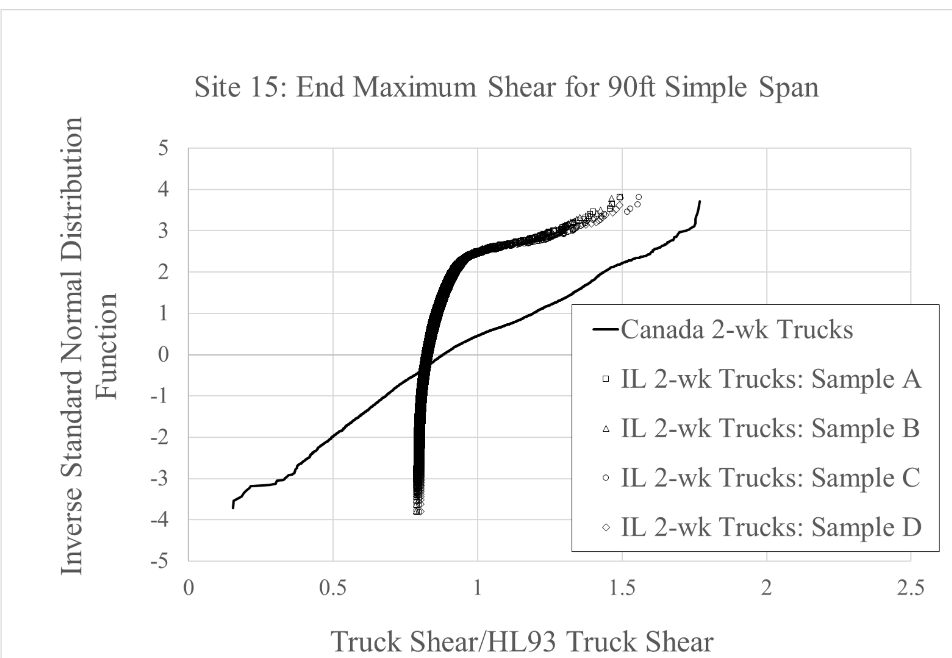


Figure A-40 Shears of Canada and Illinois Trucks at Site 15 for 90ft Span

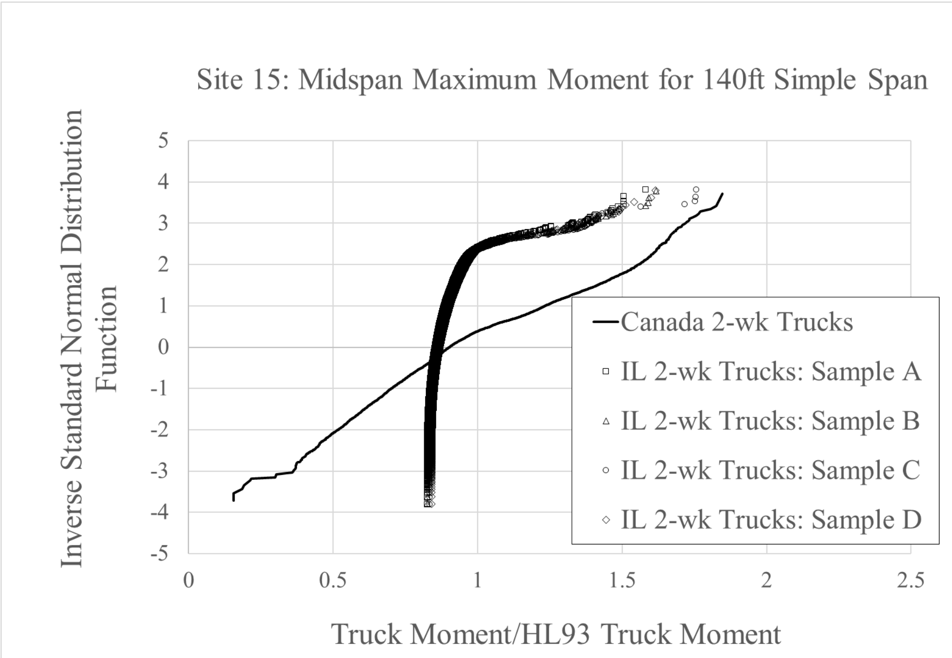


Figure A-41 Moments of Canada and Illinois Trucks at Site 15 for 140ft Span

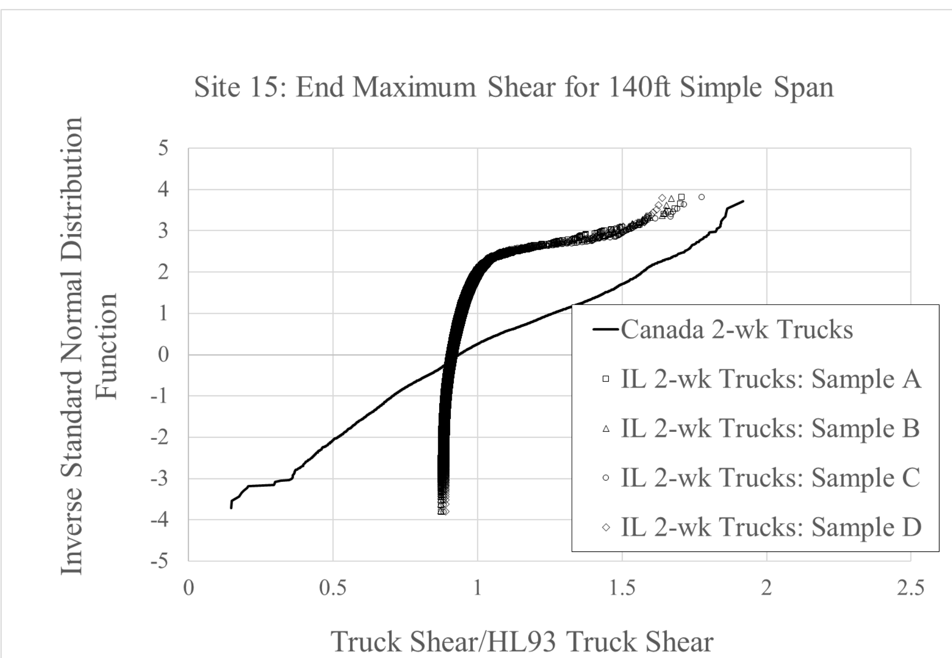
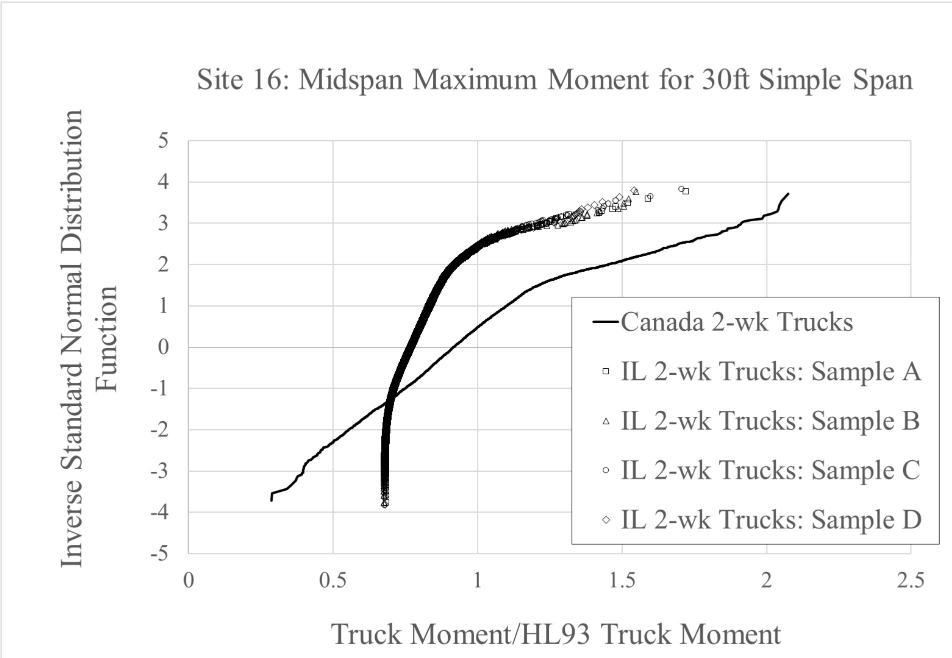


Figure A-42 Shears of Canada and Illinois Trucks at Site 15 for 140ft Span



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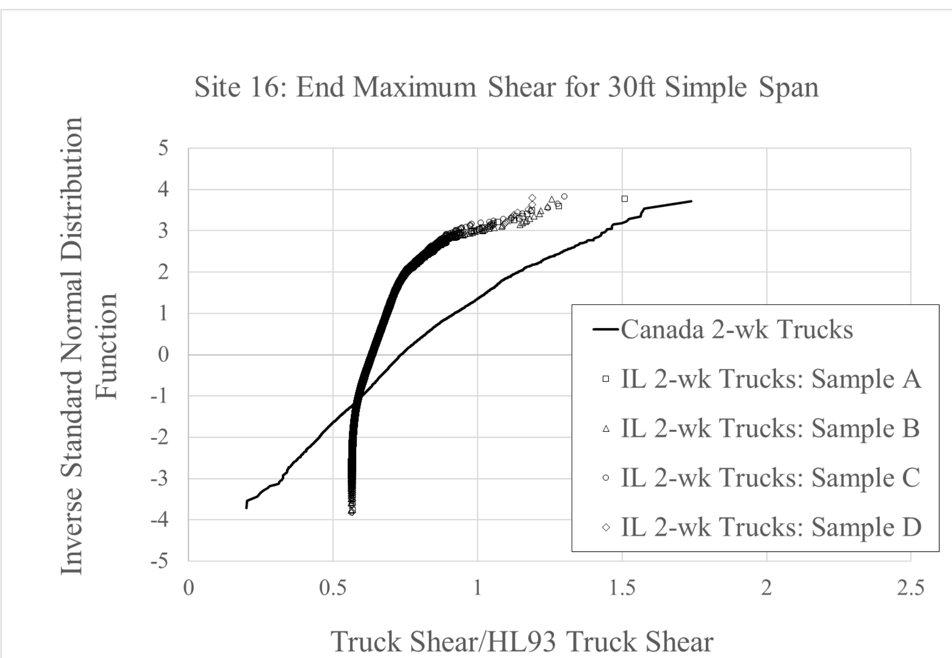


Figure A-44 Shears of Canada and Illinois Trucks at Site 16 for 30ft Span

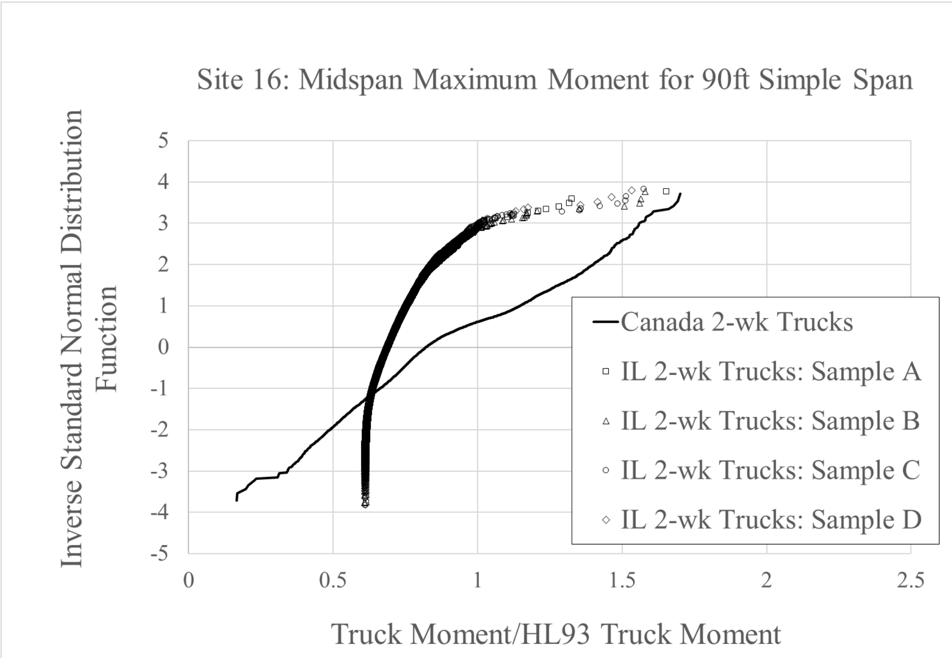


Figure A-45 Moments of Canada and Illinois Trucks at Site 16 for 90ft Span

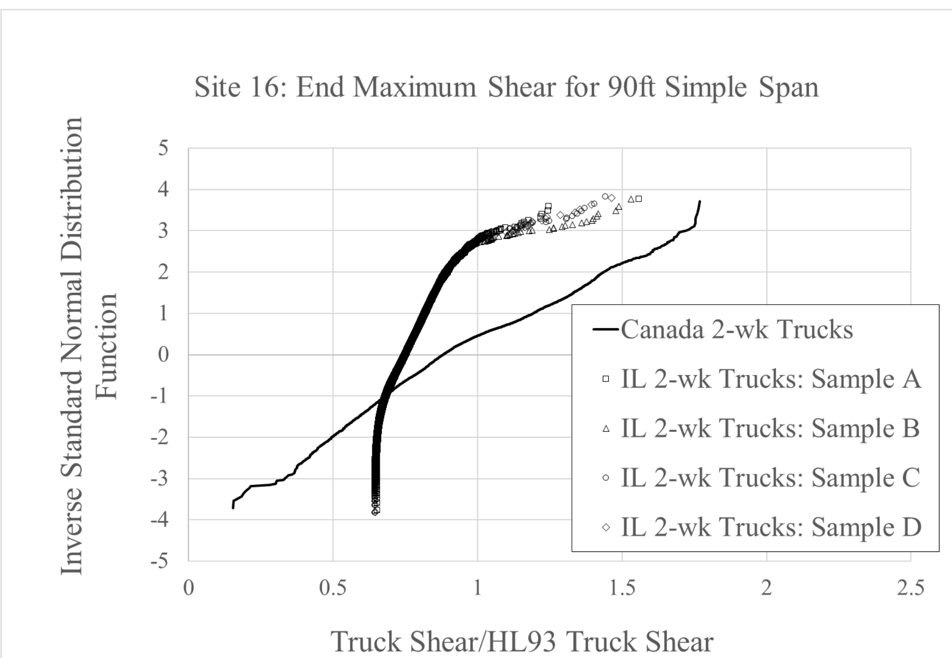


Figure A-46 Shears of Canada and Illinois Trucks at Site 16 for 90ft Span

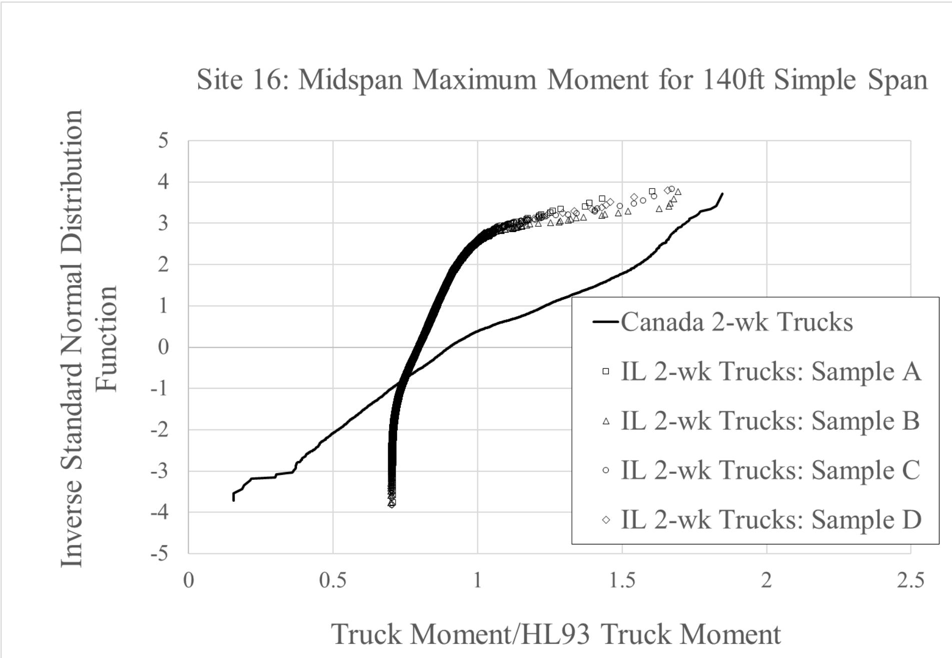


Figure A-47 Moments of Canada and Illinois Trucks at Site 16 for 140ft Span

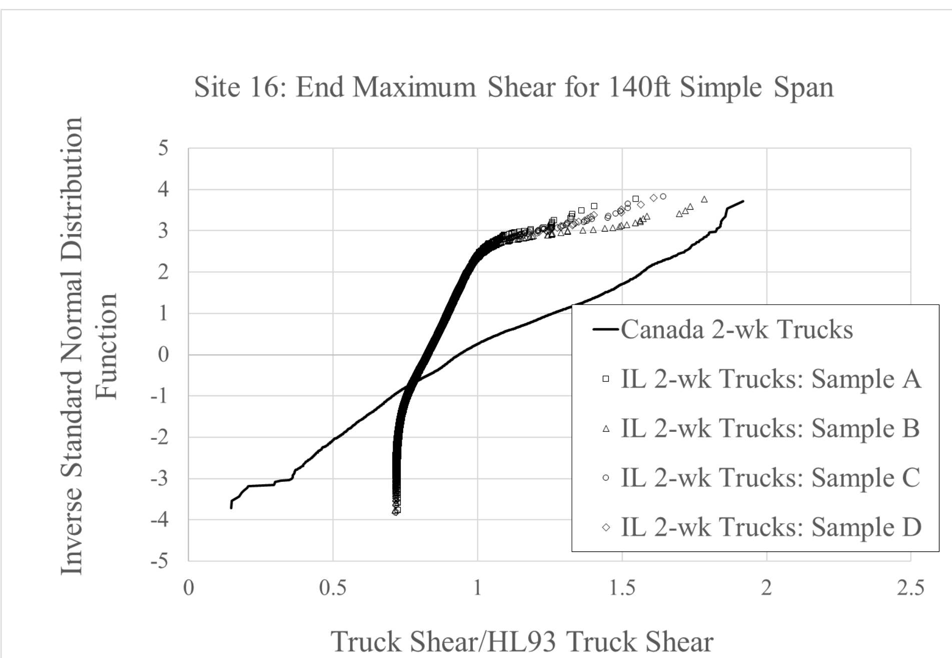


Figure A-48 Shears of Canada and Illinois Trucks at Site 16 for 140ft Span

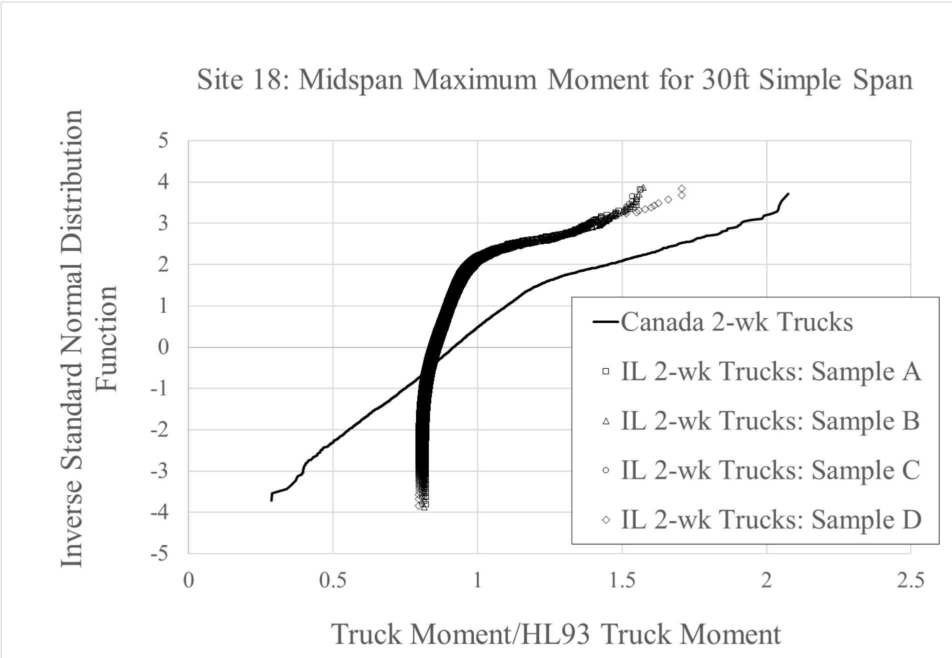


Figure A-49 Moments of Canada and Illinois Trucks at Site 18 for 30ft Span

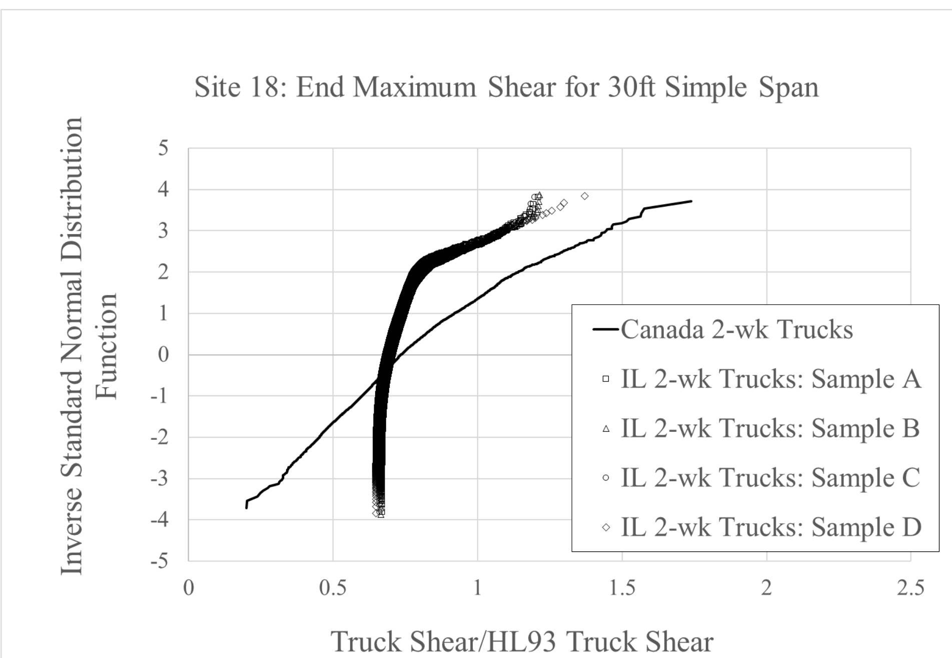


Figure A-50 Shears of Canada and Illinois Trucks at Site 18 for 30ft Span

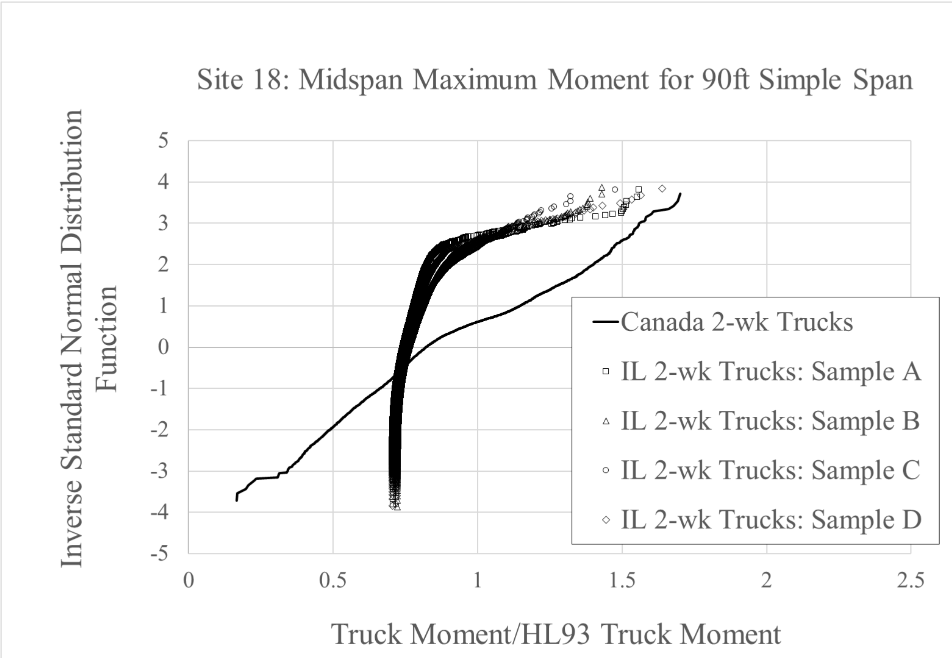


Figure A-51 Moments of Canada and Illinois Trucks at Site 18 for 90ft Span

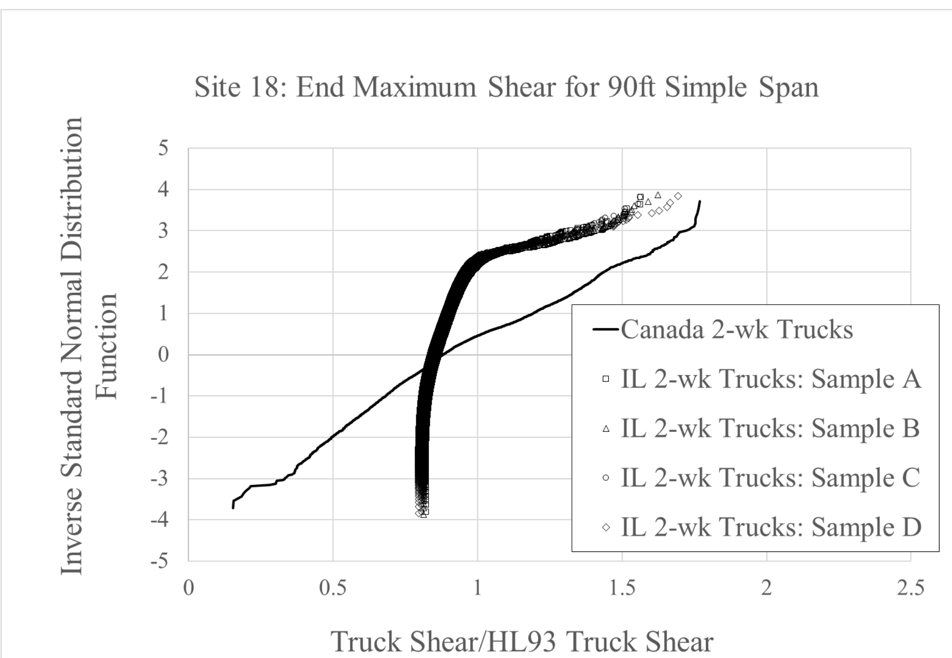


Figure A-52 Shears of Canada and Illinois Trucks at Site 18 for 90ft Span

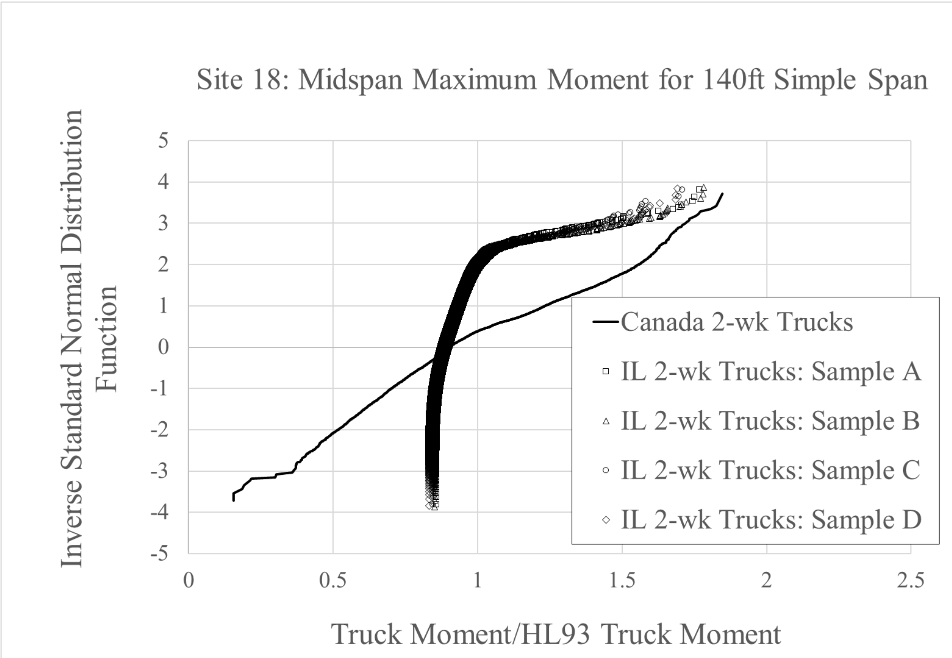


Figure A-53 Moments of Canada and Illinois Trucks at Site 18 for 140ft Span

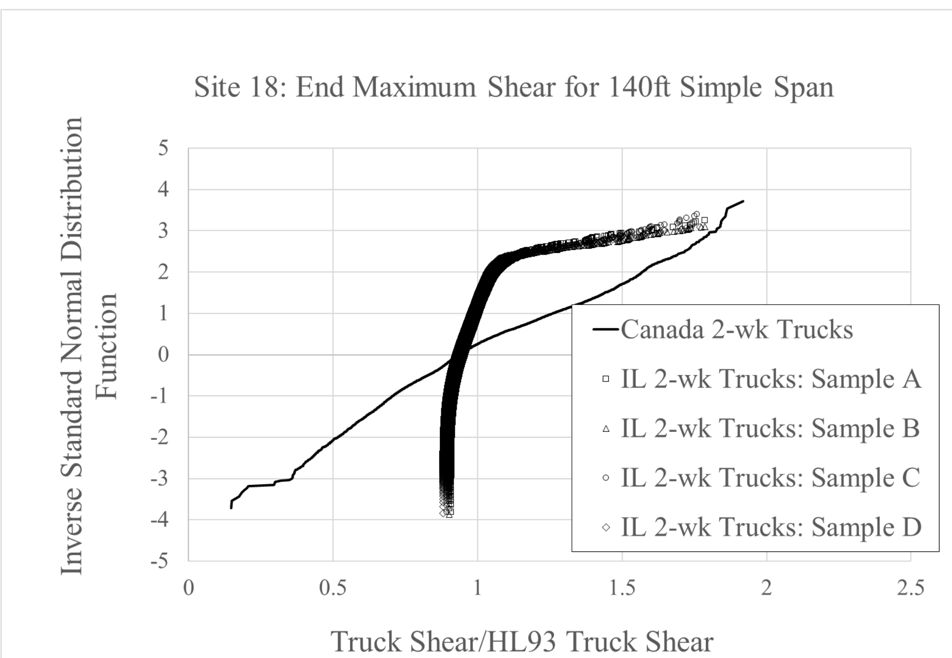
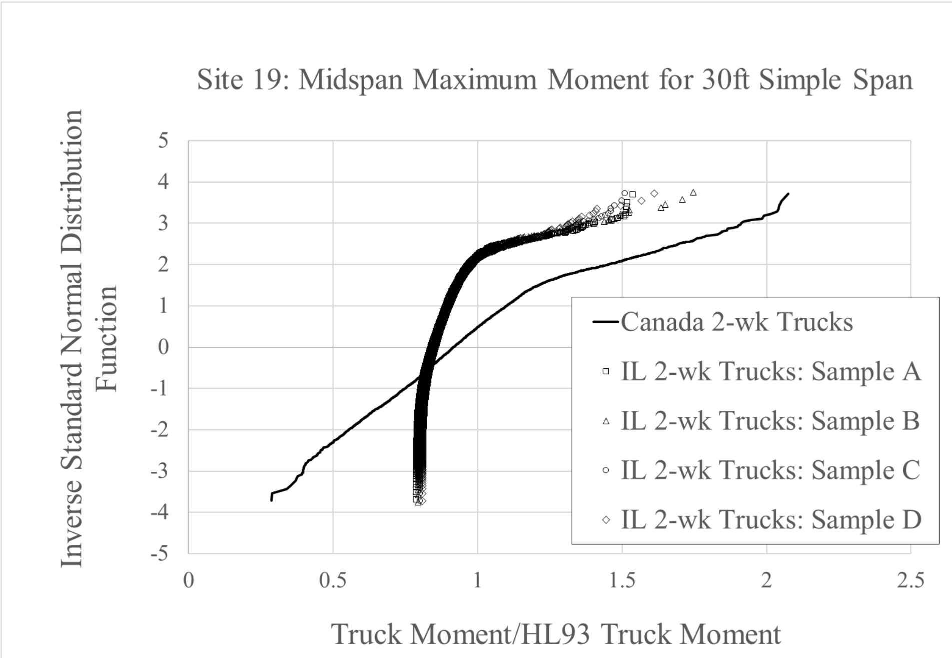


Figure A-54 Shears of Canada and Illinois Trucks at Site 18 for 140ft Span



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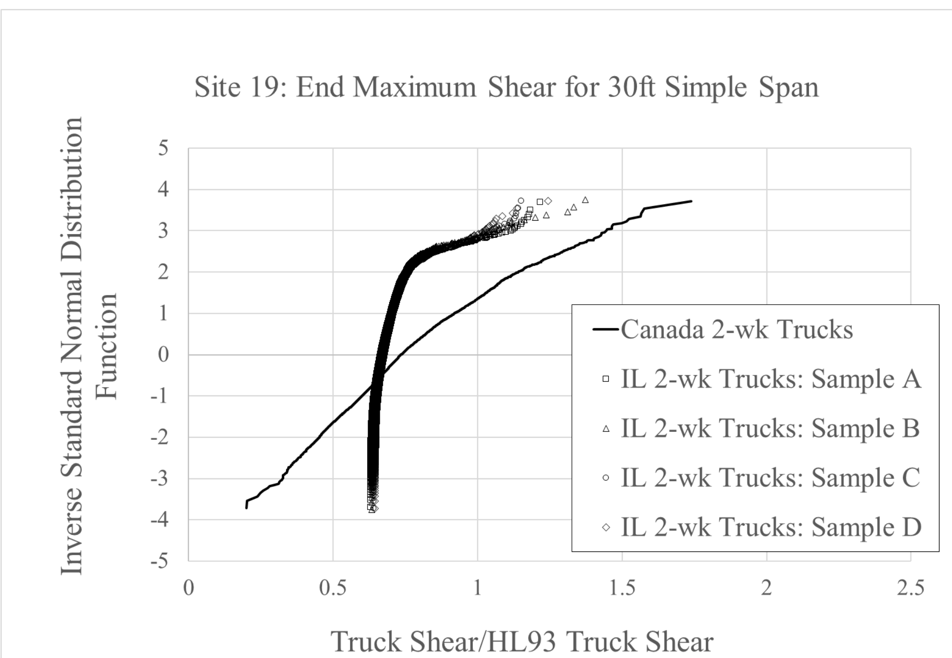

Figure A-56 Shears of Canada and Illinois Trucks at Site 19 for 30ft Span

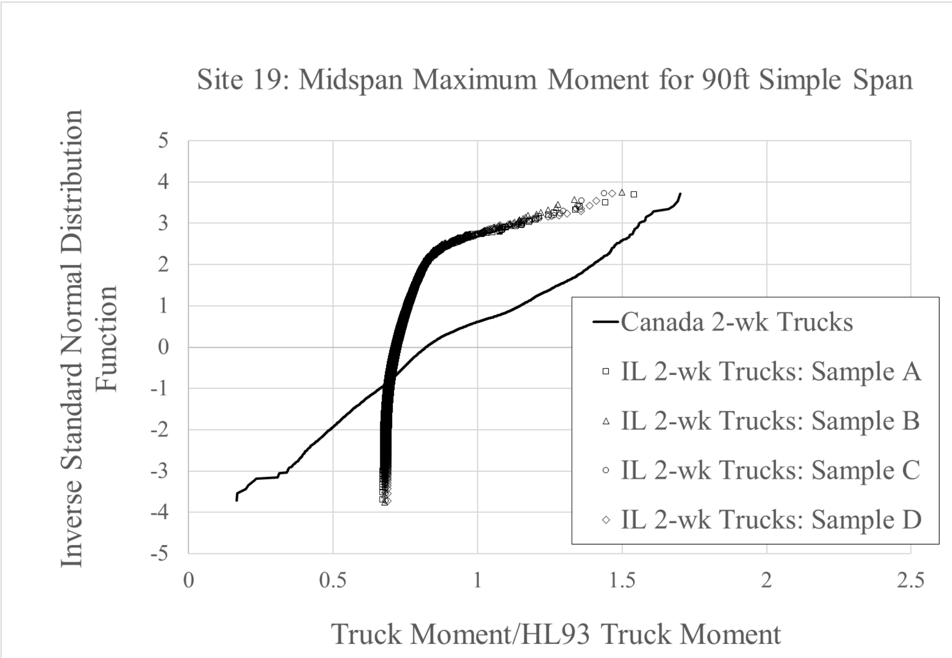

Figure A-57 Moments of Canada and Illinois Trucks at Site 19 for 90ft Span

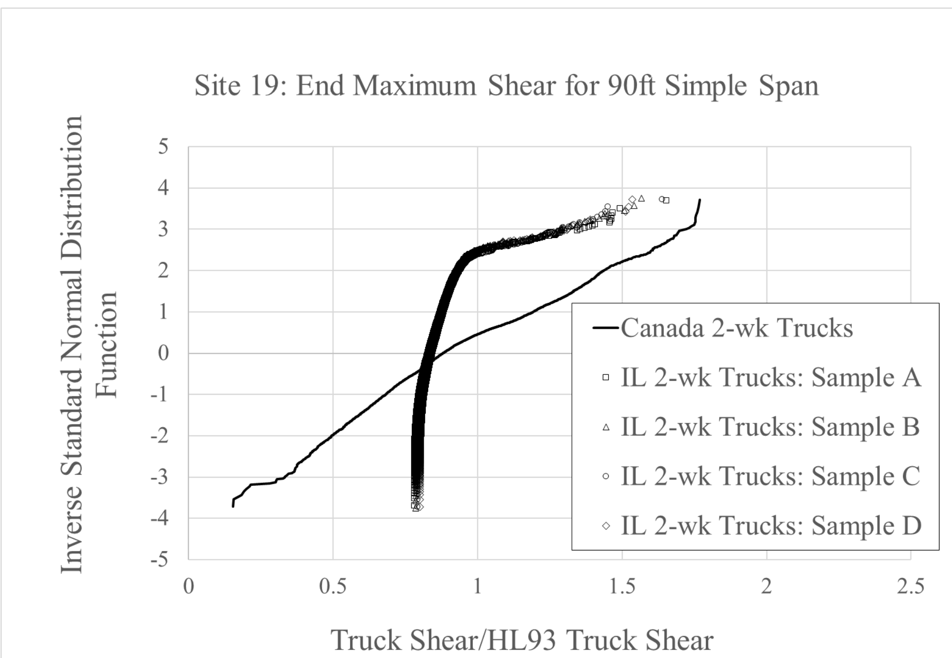

Figure A-58 Shears of Canada and Illinois Trucks at Site 19 for 90ft Span

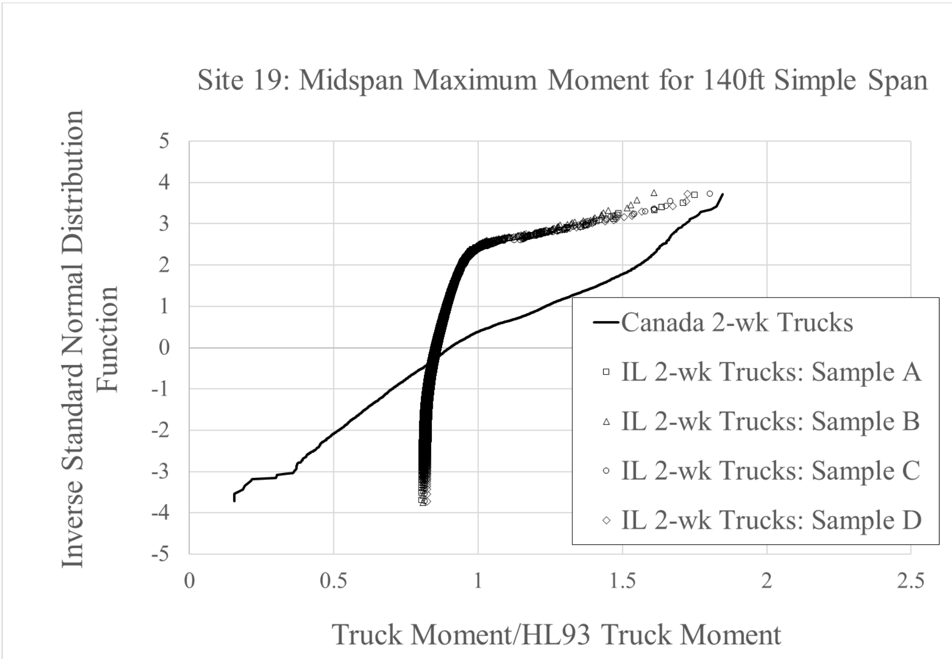

Figure A-59 Moments of Canada and Illinois Trucks at Site 19 for 140ft Span

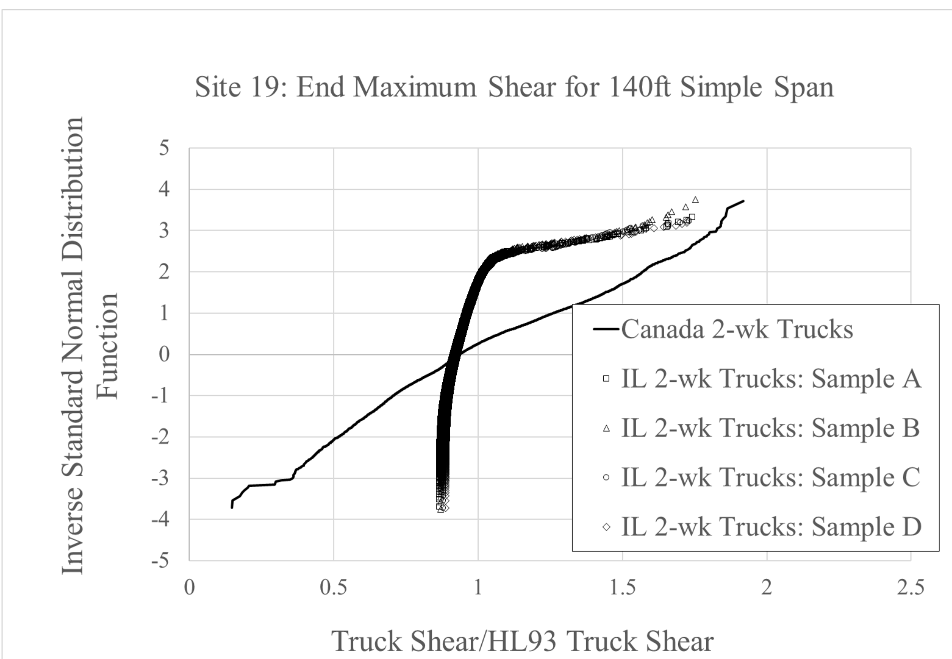

Figure A-60 Shears of Canada and Illinois Trucks at Site 19 for 140ft Span

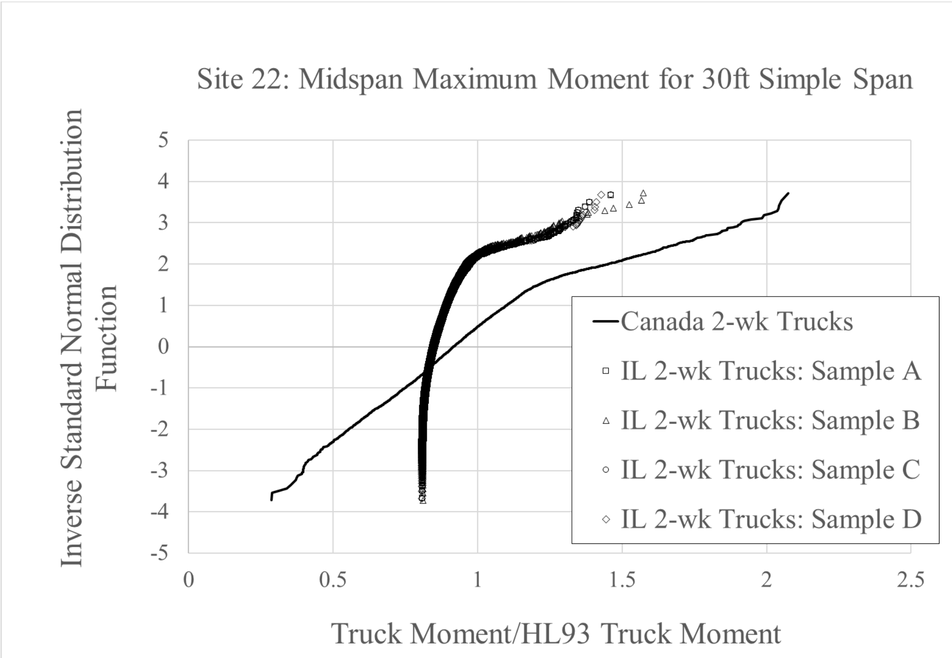

Figure A-61 Moments of Canada and Illinois Trucks at Site 22 for 30ft Span

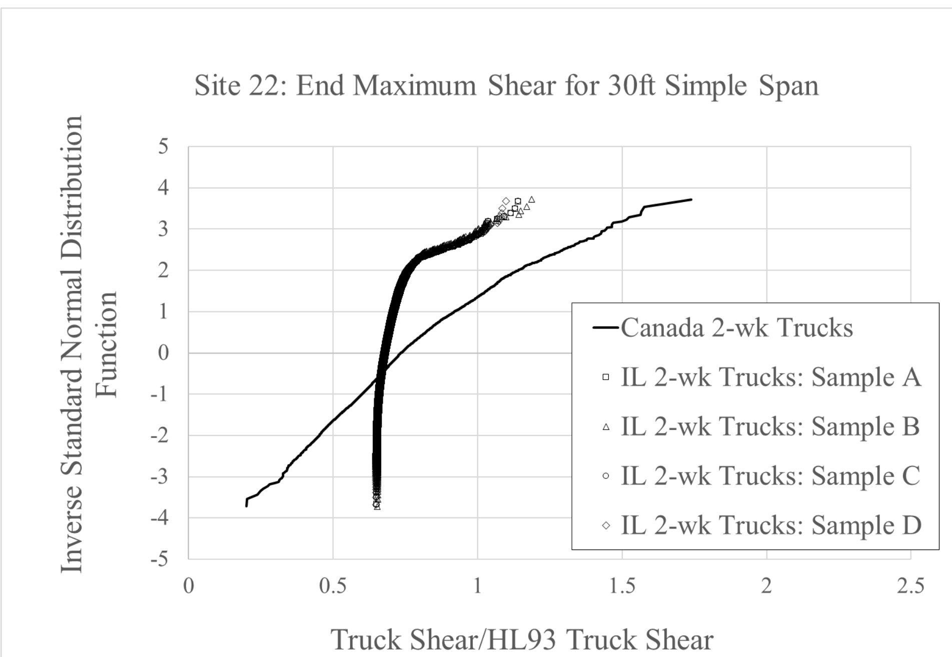

Figure A-62 Shears of Canada and Illinois Trucks at Site 22 for 30ft Span

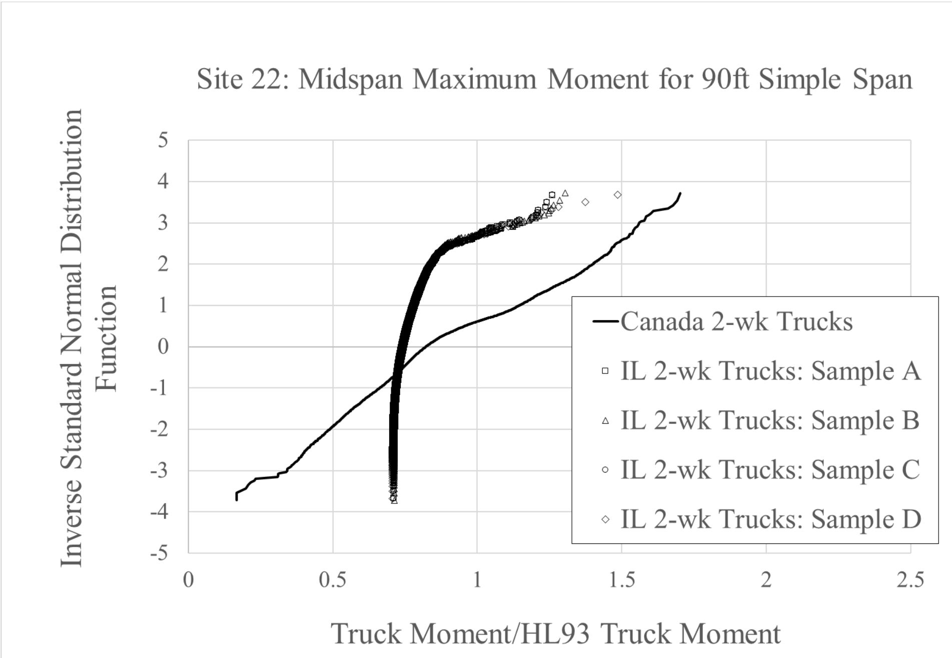

Figure A-63 Moments of Canada and Illinois Trucks at Site 22 for 90ft Span

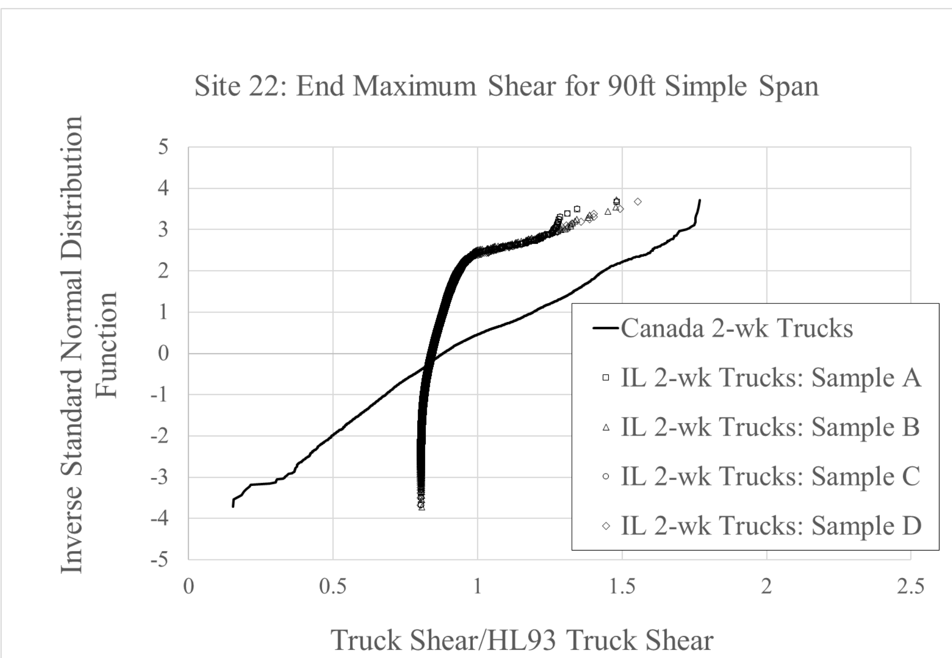

Figure A-64 Shears of Canada and Illinois Trucks at Site 22 for 90ft Span

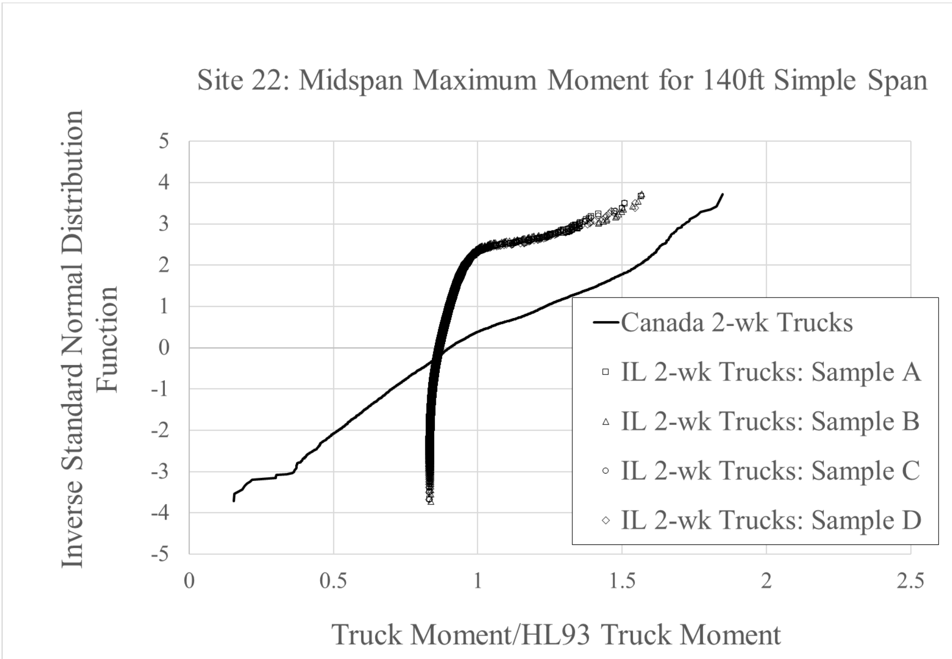

Figure A-65 Moments of Canada and Illinois Trucks at Site 22 for 140ft Span

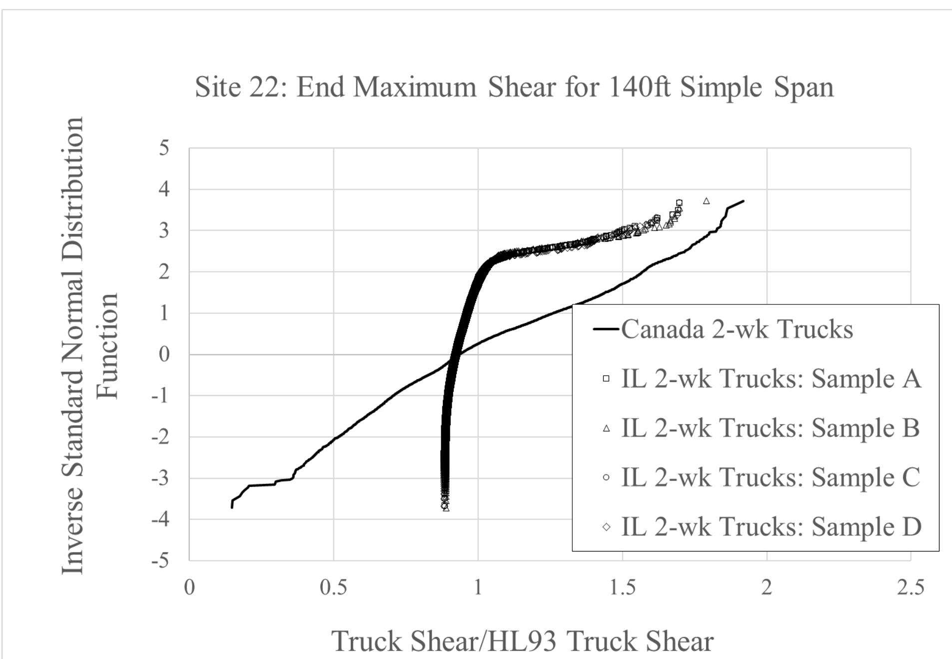

Figure A-66 Shears of Canada and Illinois Trucks at Site 22 for 140ft Span

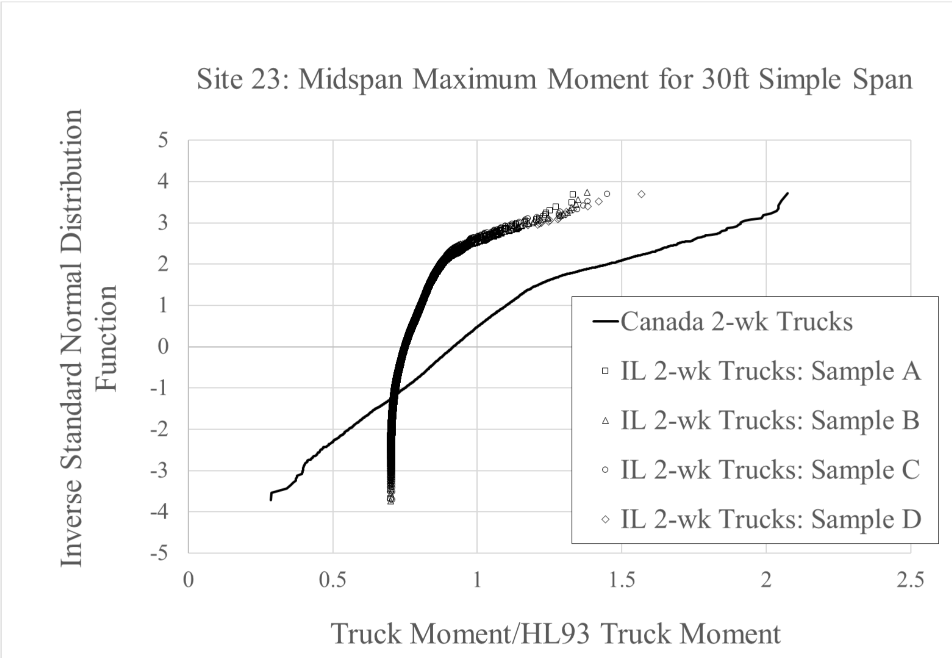

Figure A-67 Moments of Canada and Illinois Trucks at Site 23 for 30ft Span

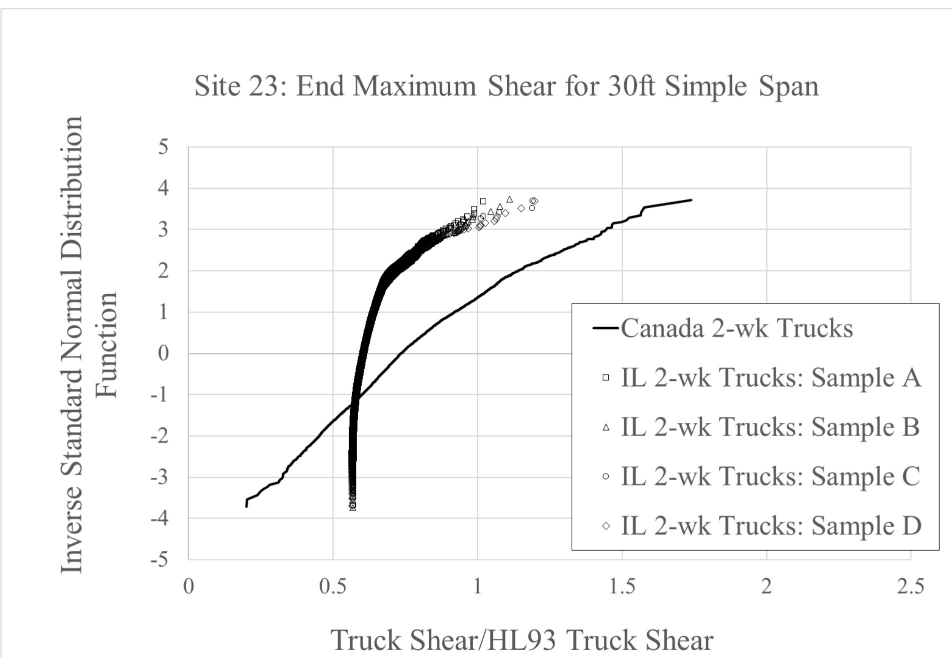

Figure A-68 Shears of Canada and Illinois Trucks at Site 23 for 30ft Span

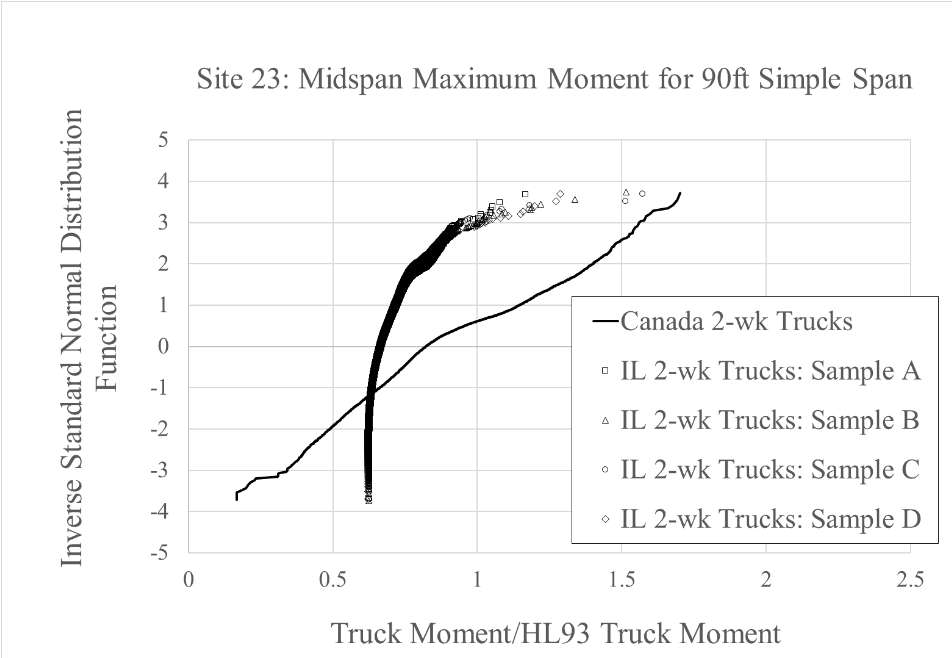

Figure A-69 Moments of Canada and Illinois Trucks at Site 23 for 90ft Span

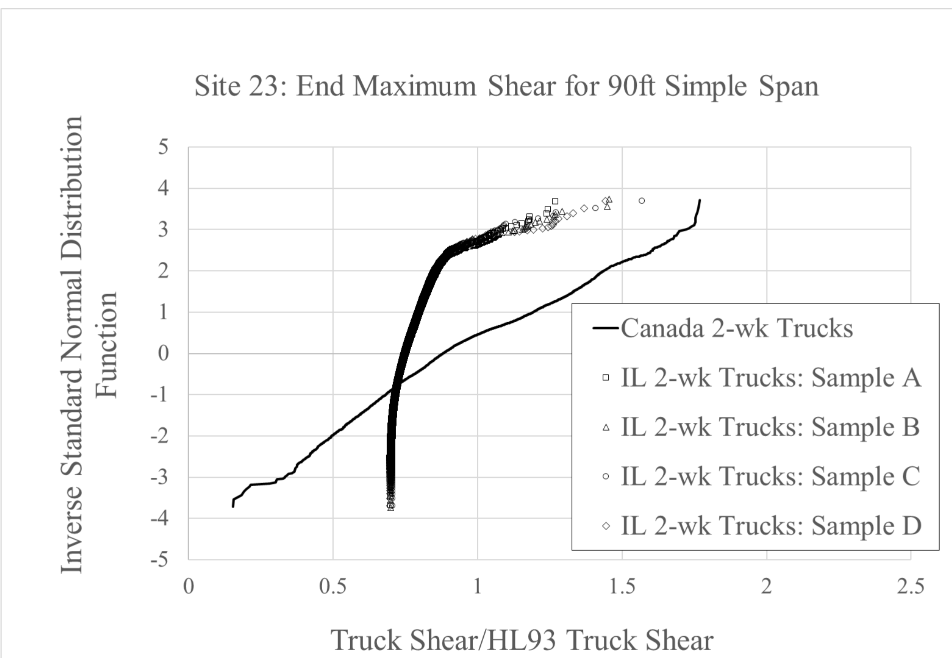

Figure A-70 Shears of Canada and Illinois Trucks at Site 23 for 90ft Span

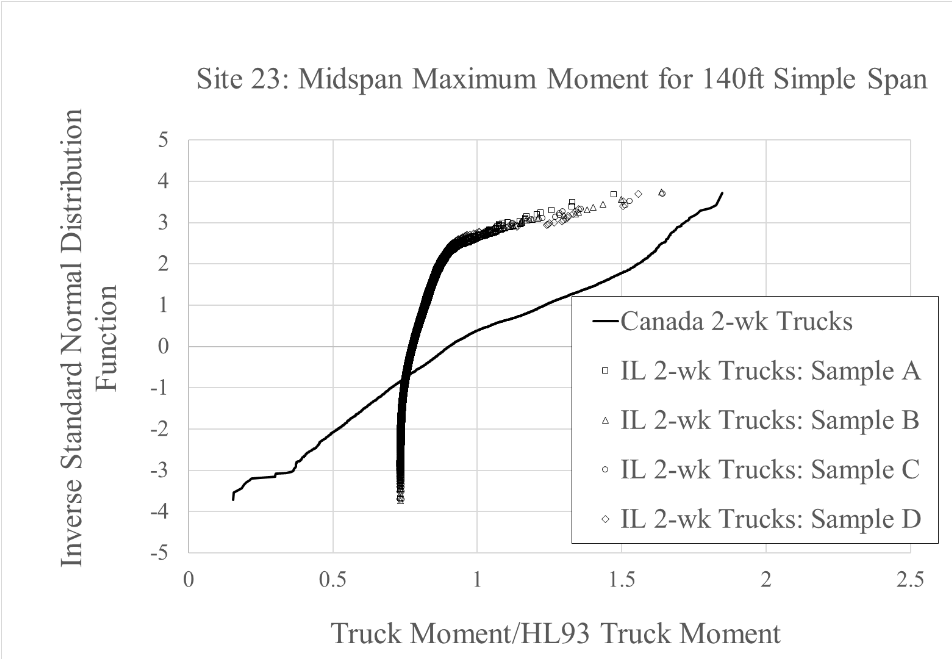

Figure A-71 Moments of Canada and Illinois Trucks at Site 23 for 140ft Span

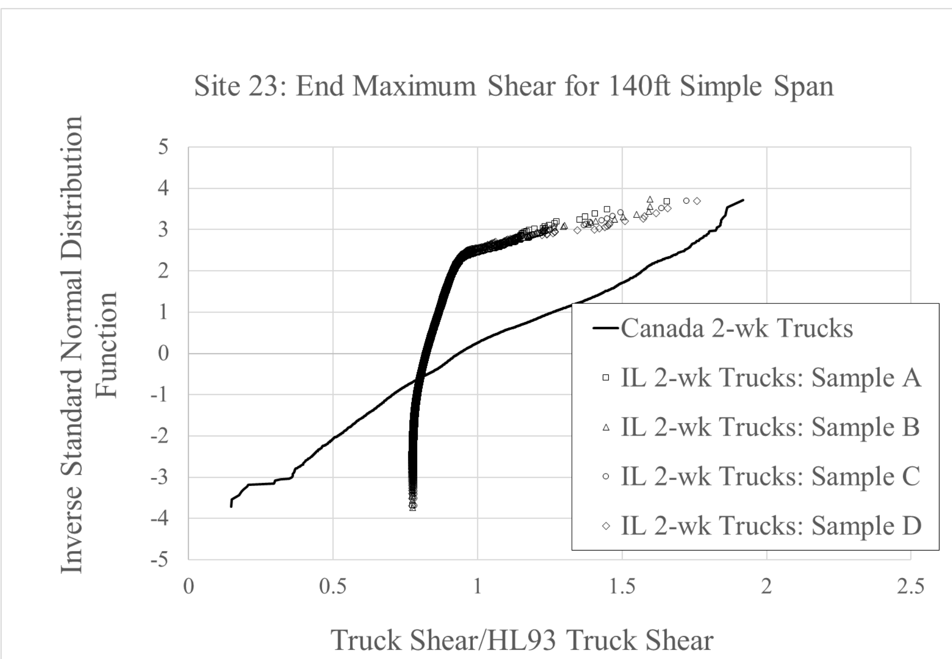

Figure A-72 Shears of Canada and Illinois Trucks at Site 23 for 140ft Span

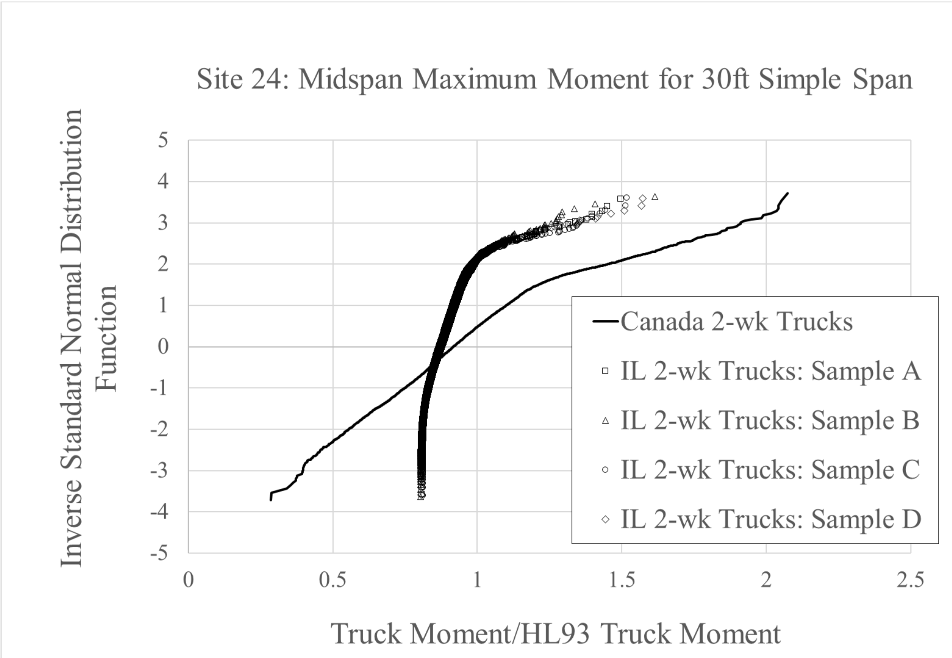

Figure A-73 Moments of Canada and Illinois Trucks at Site 24 for 30ft Span

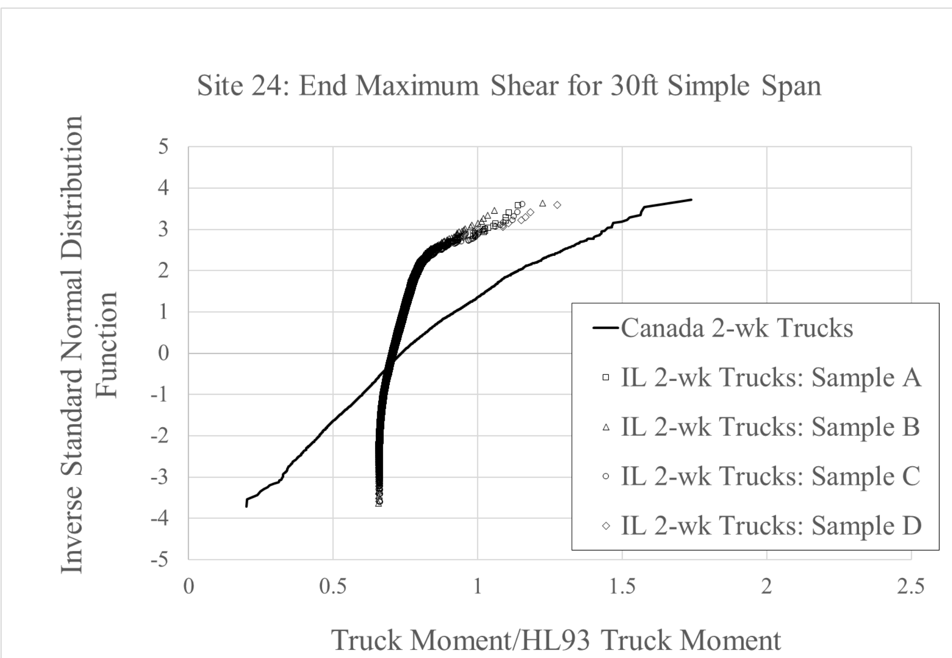

Figure A-74 Shears of Canada and Illinois Trucks at Site 24 for 30ft Span

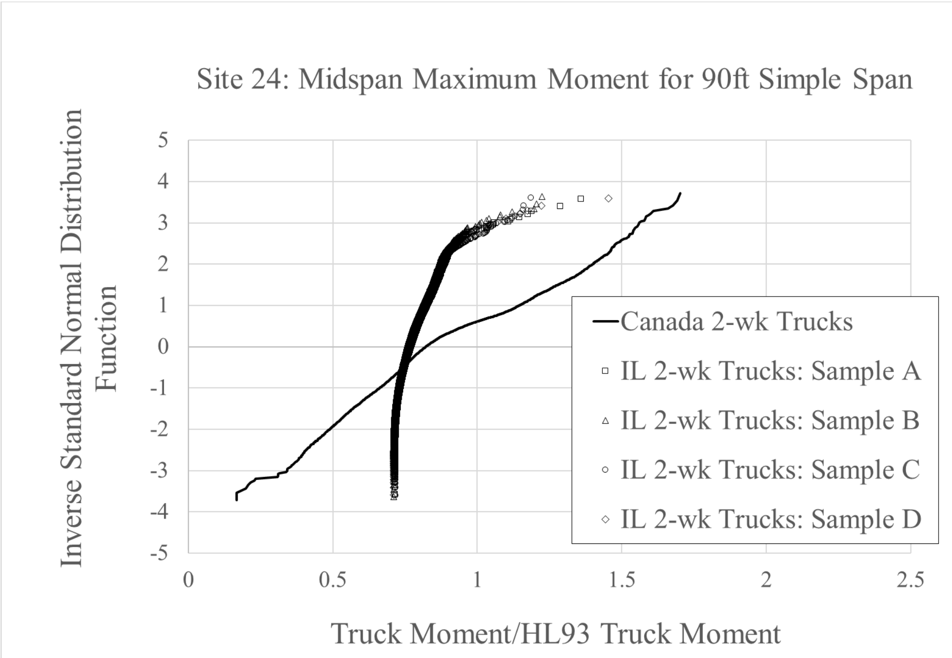

Figure A-75 Moments of Canada and Illinois Trucks at Site 24 for 90ft Span

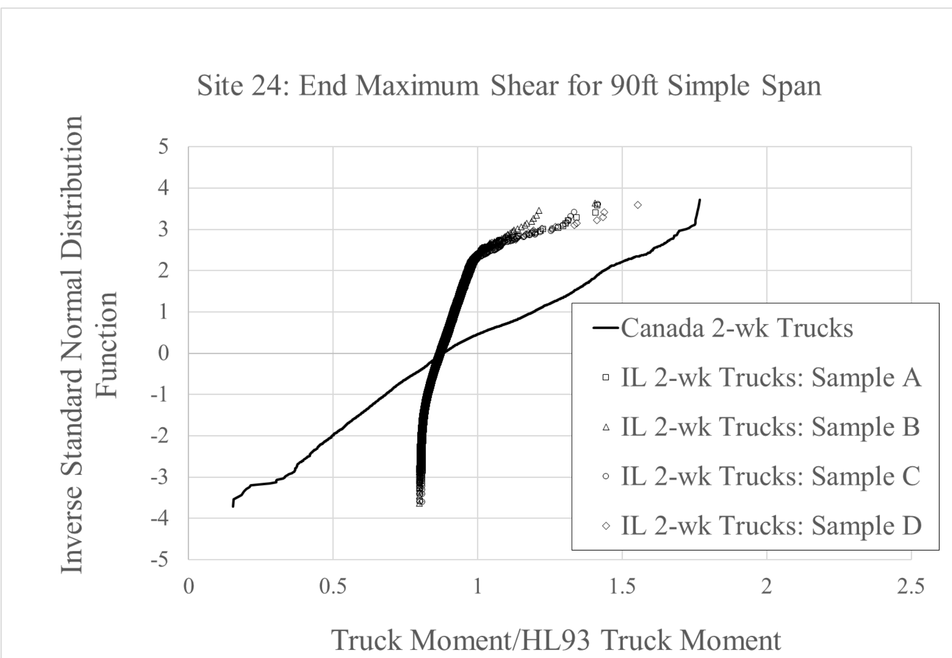

Figure A-76 Shears of Canada and Illinois Trucks at Site 24 for 90ft Span

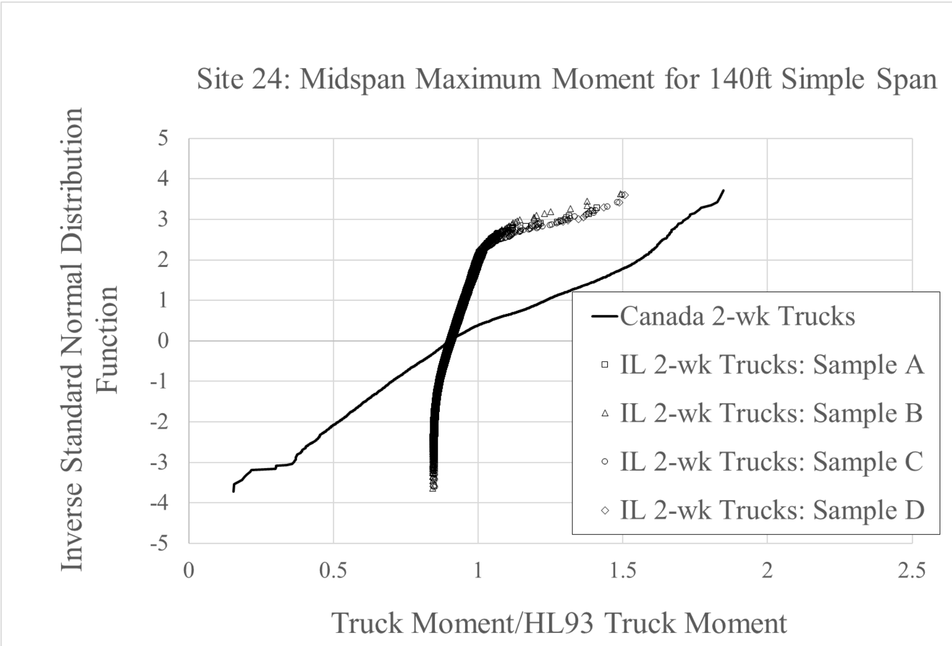

Figure A-77 Moments of Canada and Illinois Trucks at Site 24 for 140ft Span

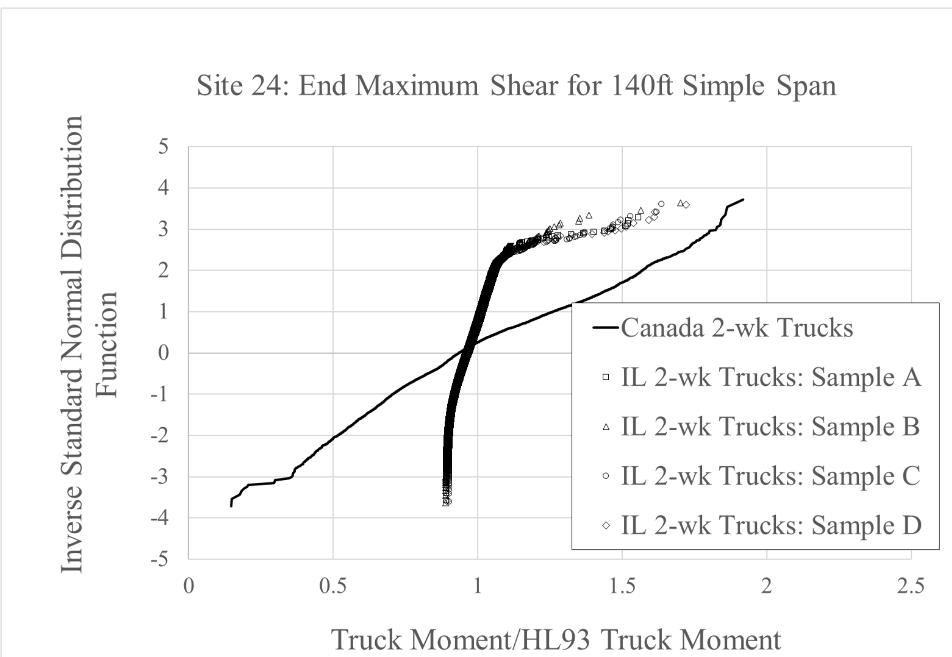

Figure A-78 Shears of Canada and Illinois Trucks at Site 24 for 140ft Span

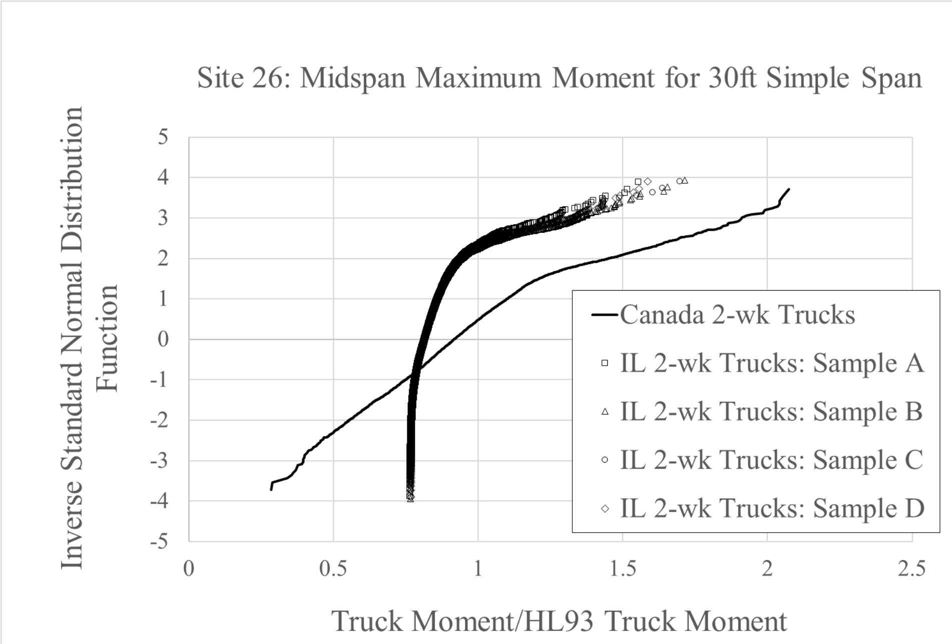

Figure A-79 Moments of Canada and Illinois Trucks at Site 26 for 30ft Span

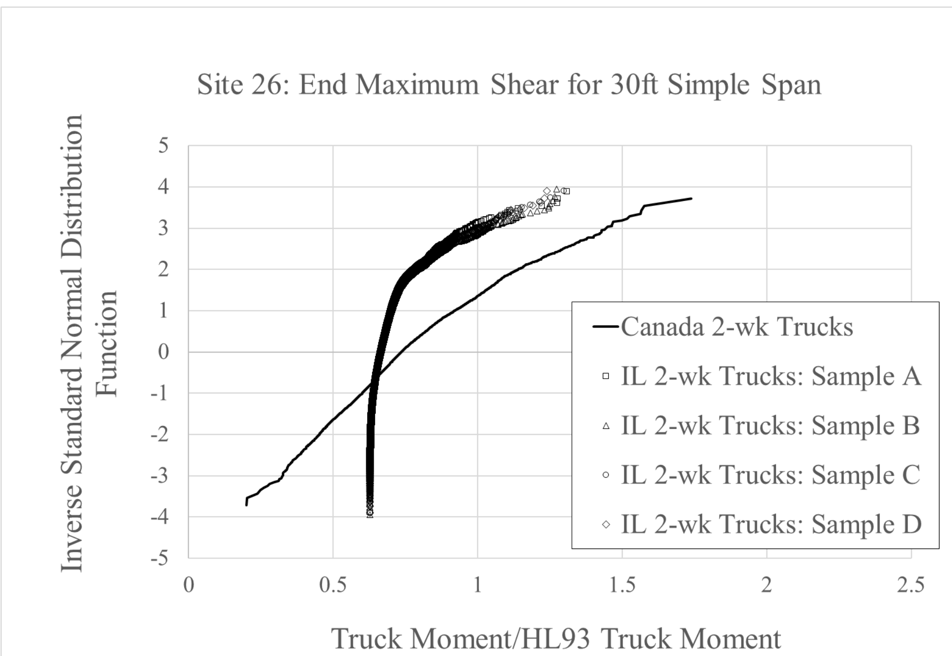

Figure A-80 Shears of Canada and Illinois Trucks at Site 26 for 30ft Span

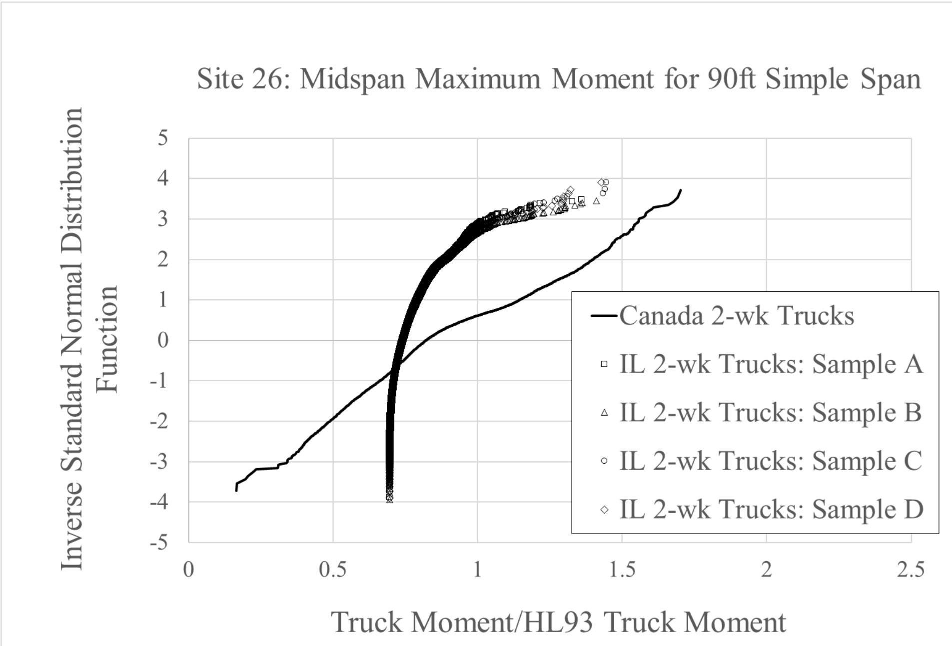

Figure A-81 Moments of Canada and Illinois Trucks at Site 26 for 90ft Span

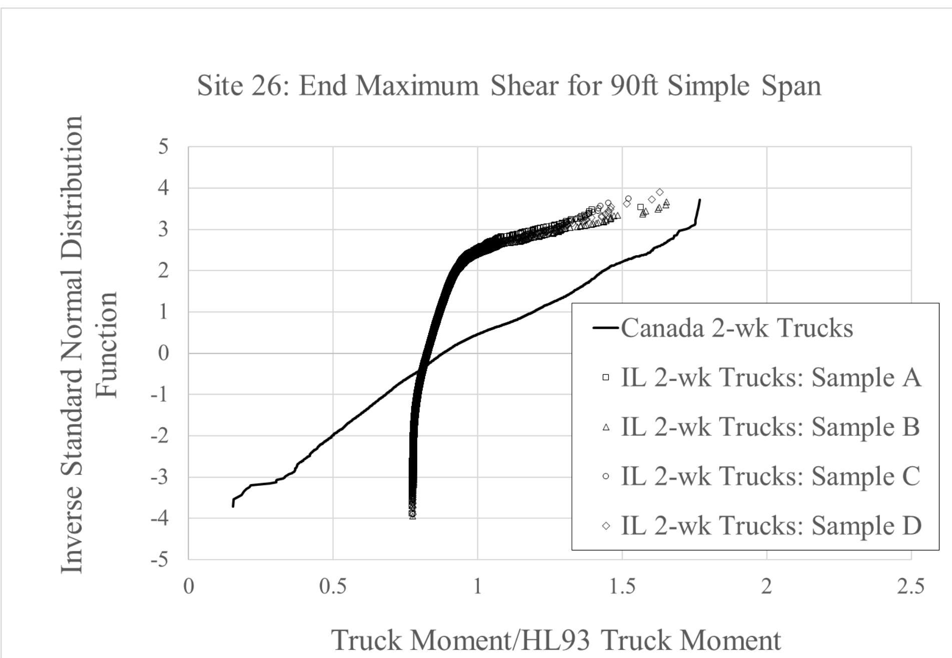

Figure A-82 Shears of Canada and Illinois Trucks at Site 26 for 90ft Span

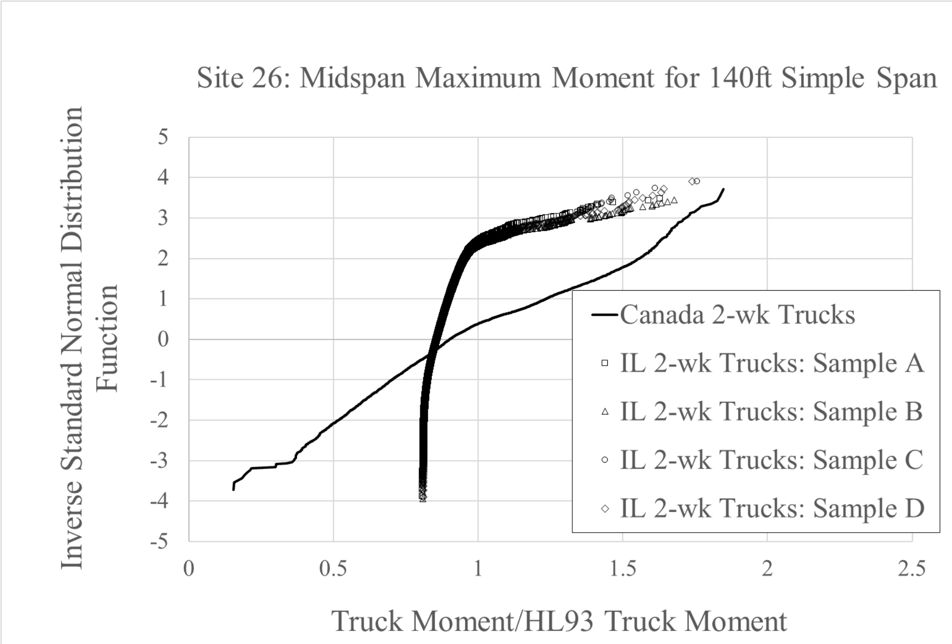

Figure A-83 Moments of Canada and Illinois Trucks at Site 26 for 140ft Span

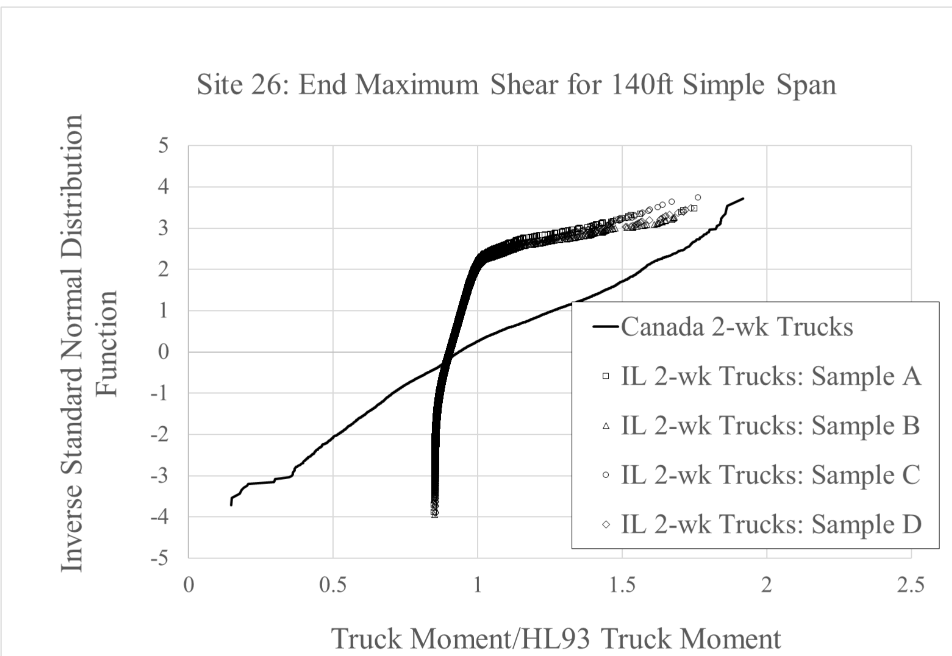

Figure A-84 Shears of Canada and Illinois Trucks at Site 26 for 140ft Span

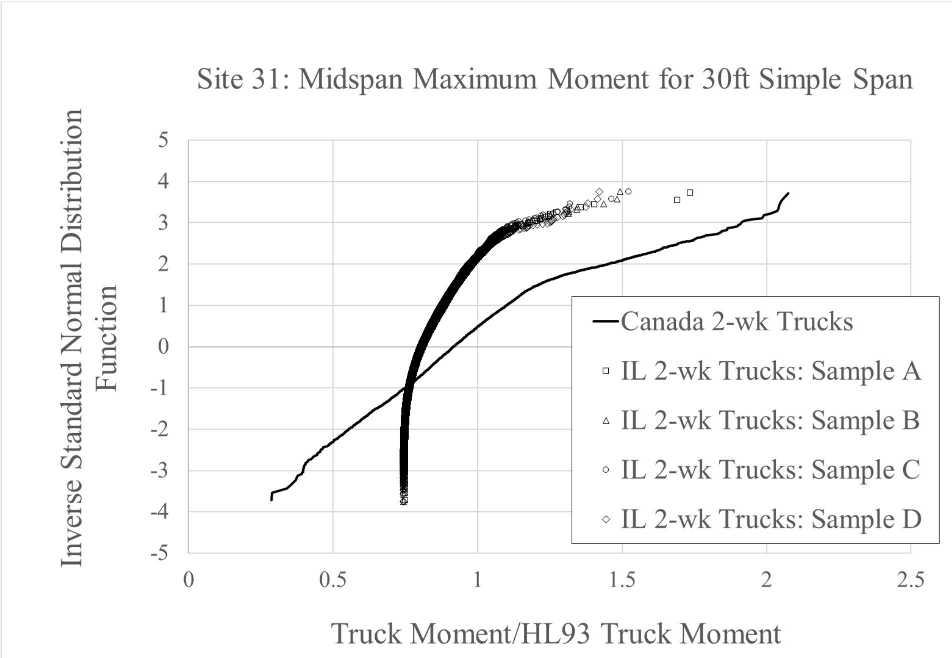

Figure A-85 Moments of Canada and Illinois Trucks at Site 31 for 30ft Span

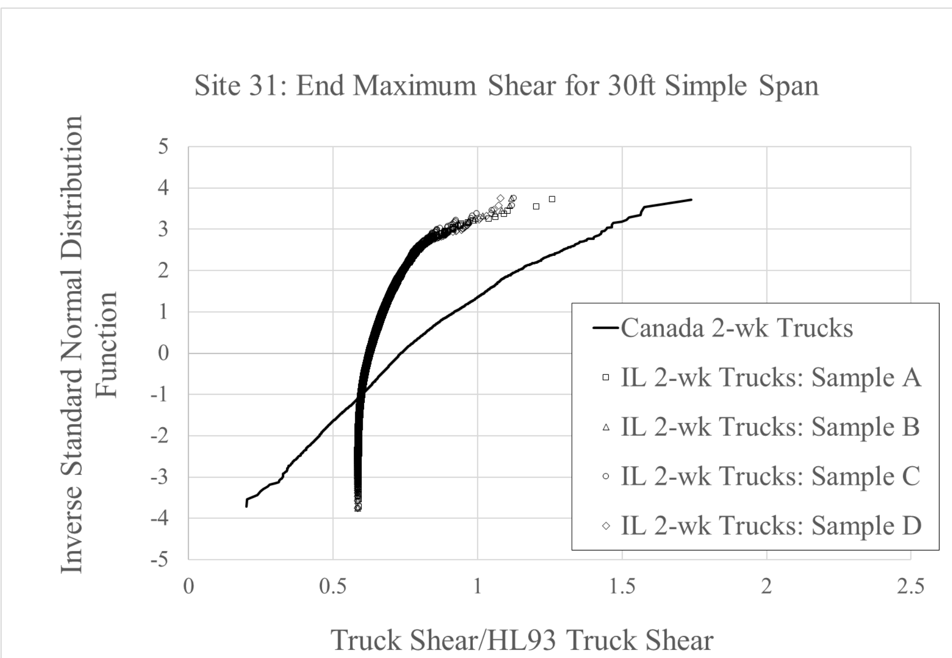

Figure A-86 Shears of Canada and Illinois Trucks at Site 31 for 30ft Span

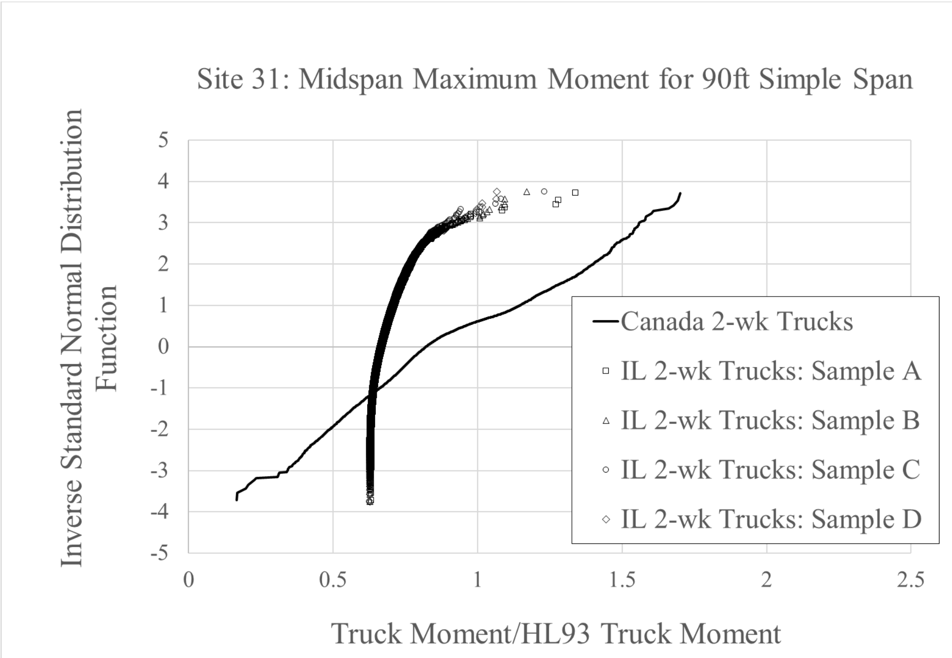

Figure A-87 Moments of Canada and Illinois Trucks at Site 31 for 90ft Span

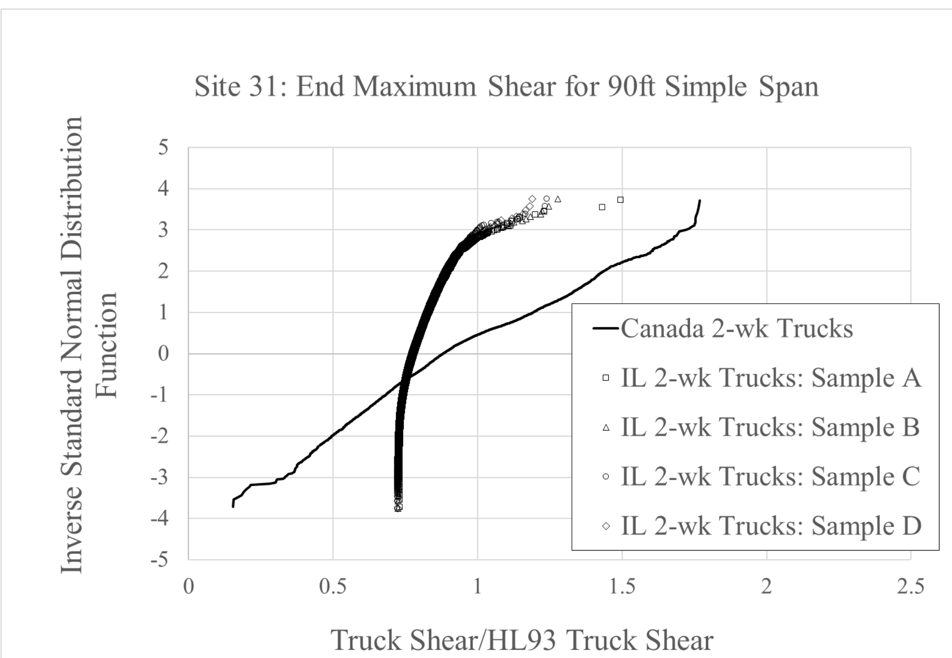

Figure A-88 Shears of Canada and Illinois Trucks at Site 31 for 90ft Span

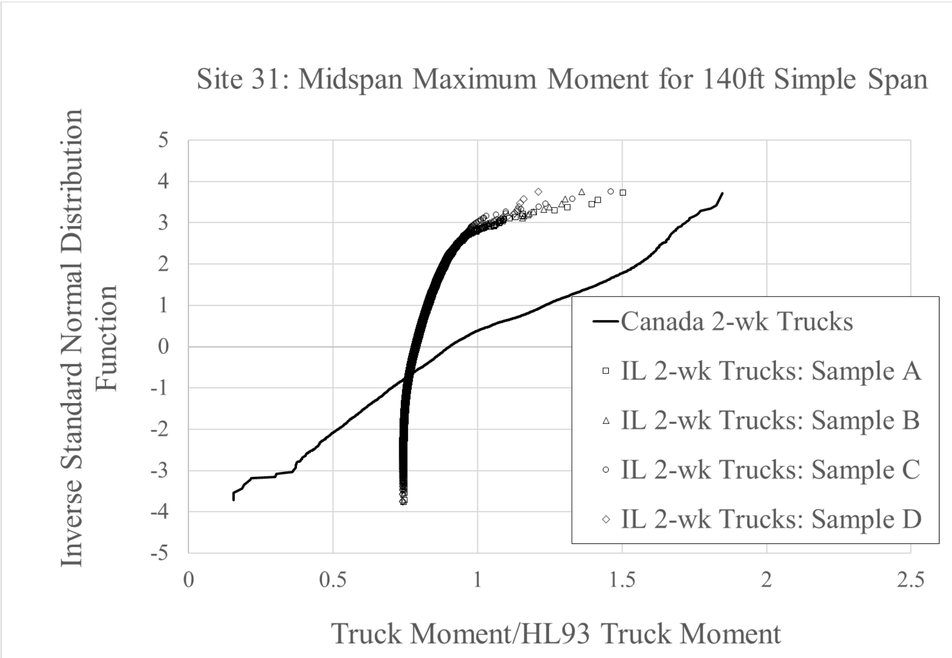

Figure A-89 Moments of Canada and Illinois Trucks at Site 31 for 140ft Span

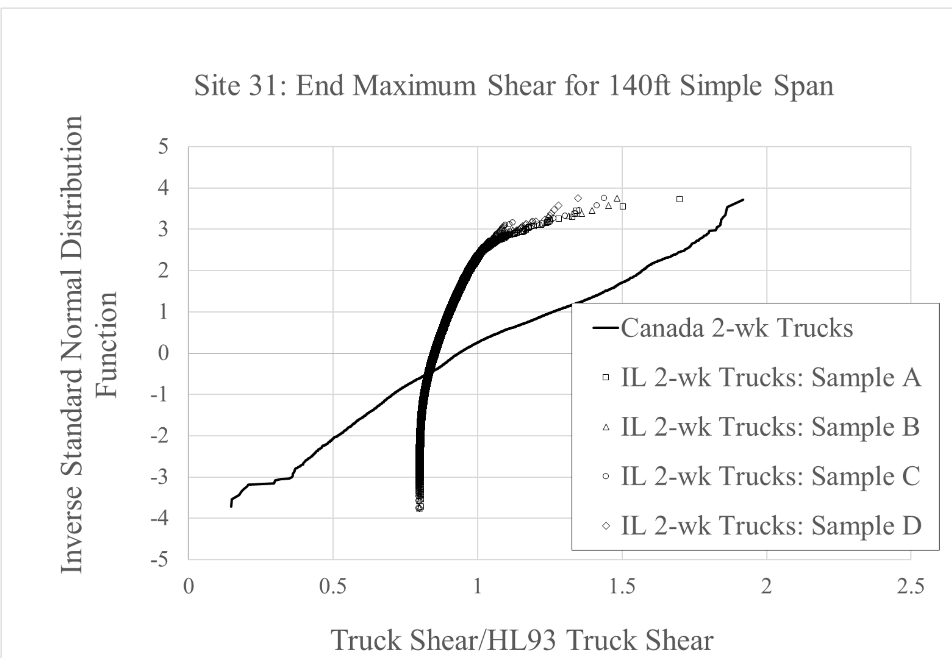

Figure A-90 Shears of Canada and Illinois Trucks at Site 31 for 140ft Span

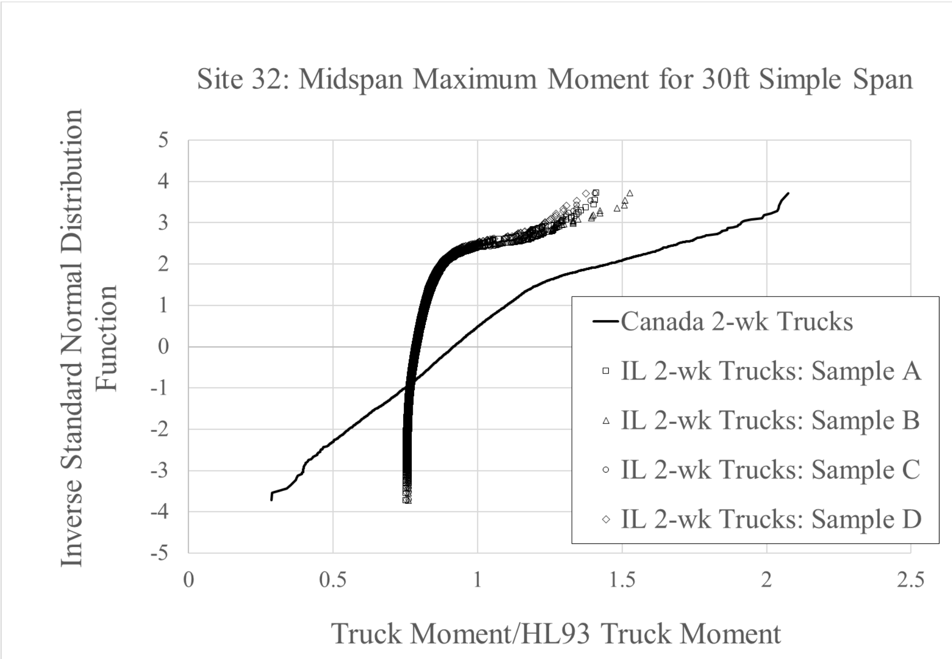

Figure A-91 Moments of Canada and Illinois Trucks at Site 32 for 30ft Span

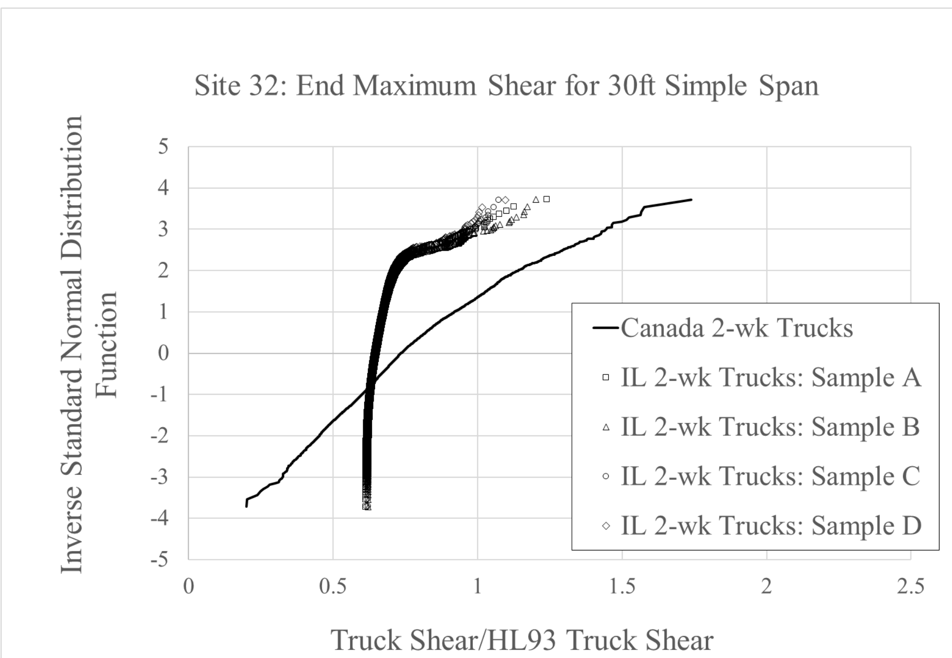

Figure A-92 Shears of Canada and Illinois Trucks at Site 32 for 30ft Span

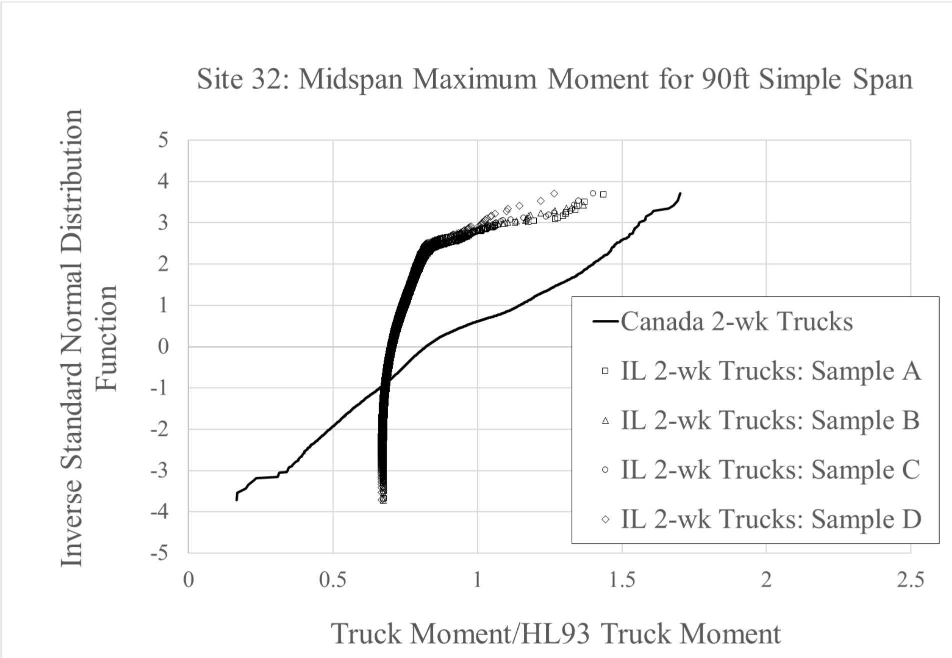

Figure A-93 Moments of Canada and Illinois Trucks at Site 32 for 90ft Span

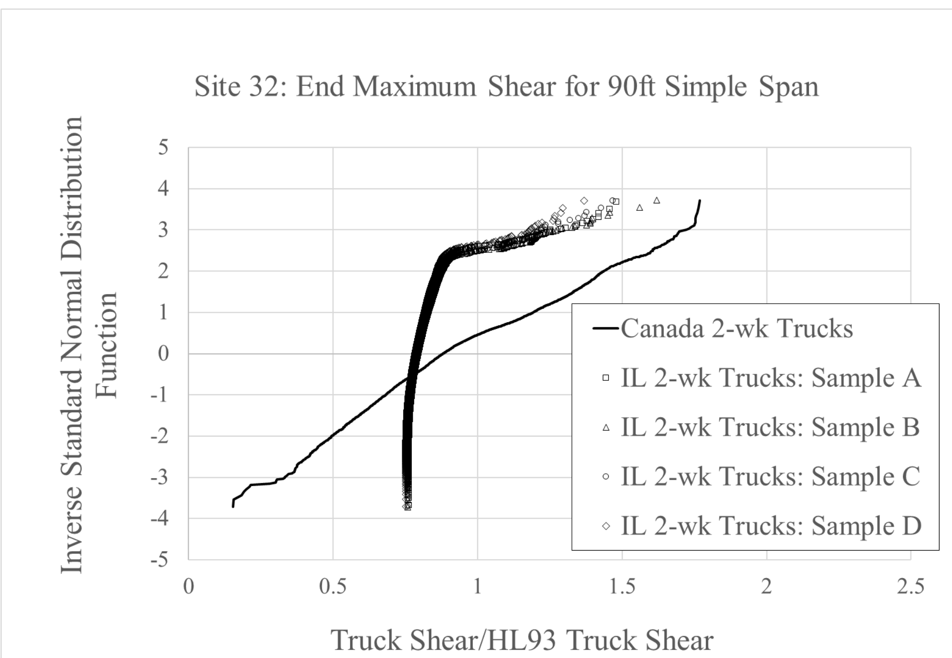

Figure A-94 Shears of Canada and Illinois Trucks at Site 32 for 90ft Span

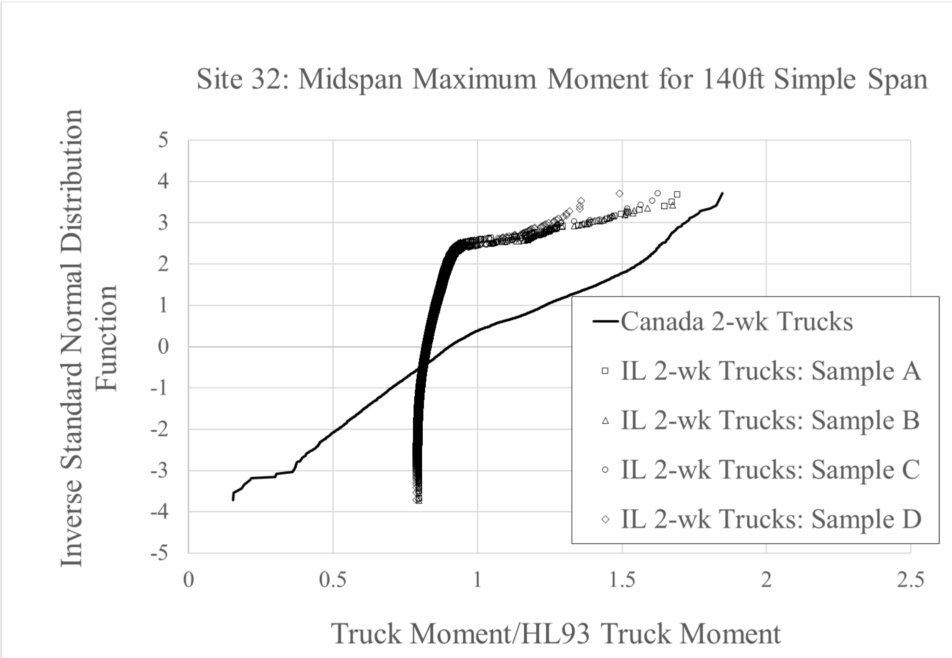

Figure A-95 Moments of Canada and Illinois Trucks at Site 32 for 140ft Span

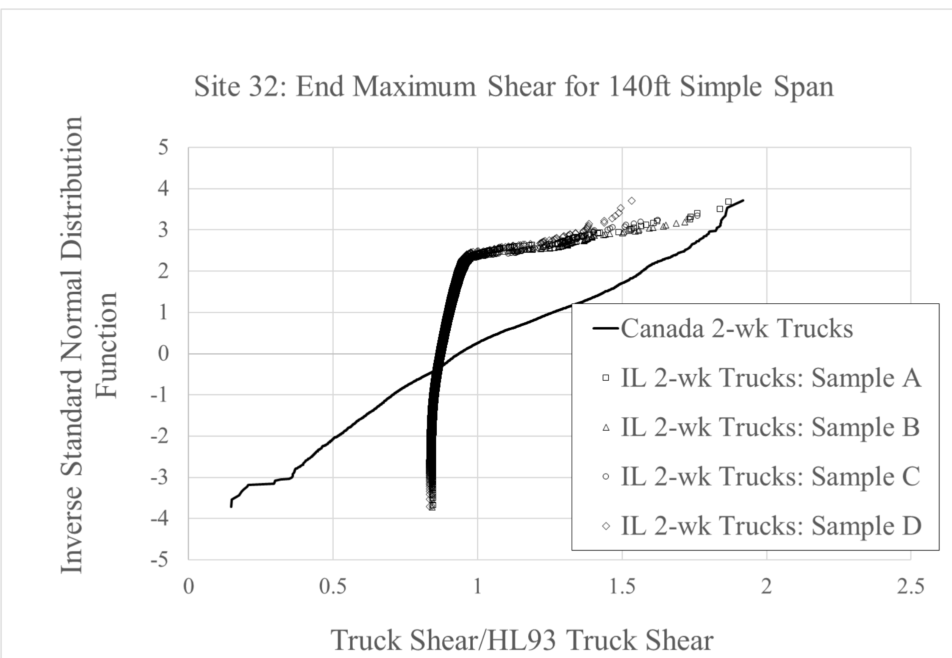

Figure A-96 Shears of Canada and Illinois Trucks at Site 32 for 140ft Span

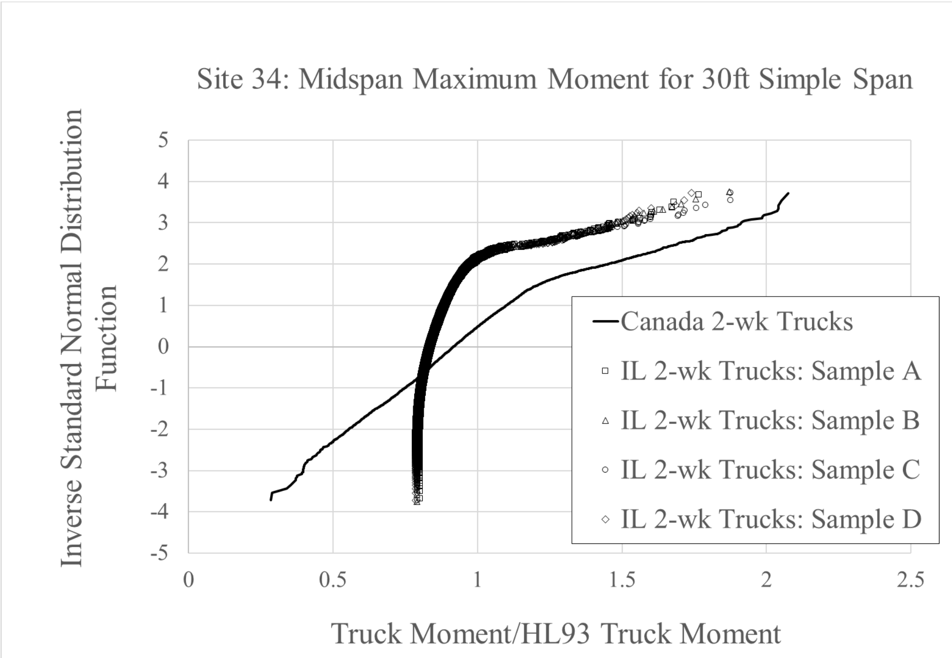

Figure A-97 Moments of Canada and Illinois Trucks at Site 34 for 30ft Span

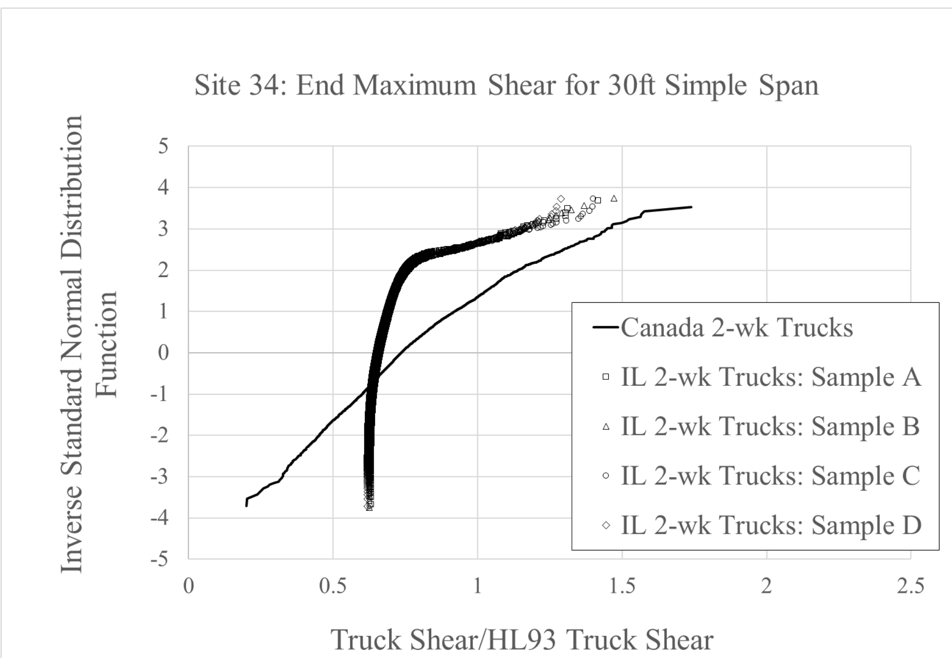

Figure A-98 Shears of Canada and Illinois Trucks at Site 34 for 30ft Span

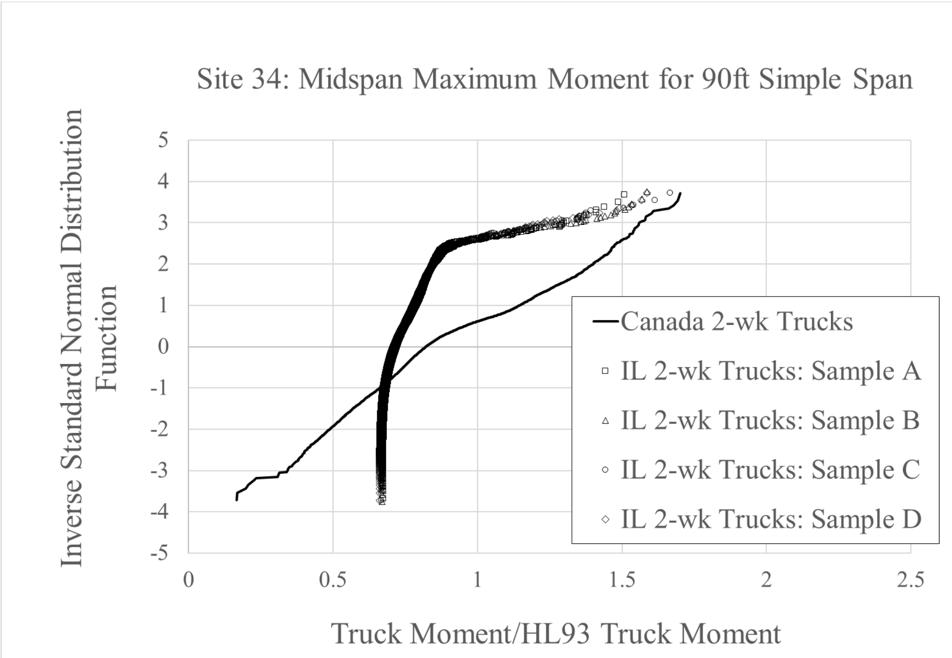

Figure A-99 Moments of Canada and Illinois Trucks at Site 34 for 90ft Spa

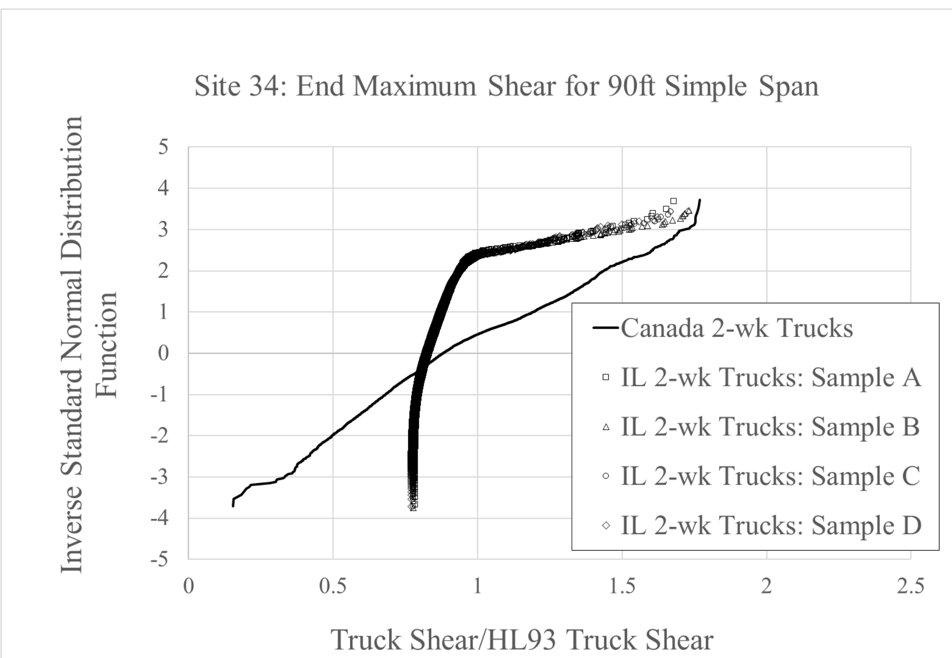

Figure A-100 Shears of Canada and Illinois Trucks at Site 34 for 90ft Span

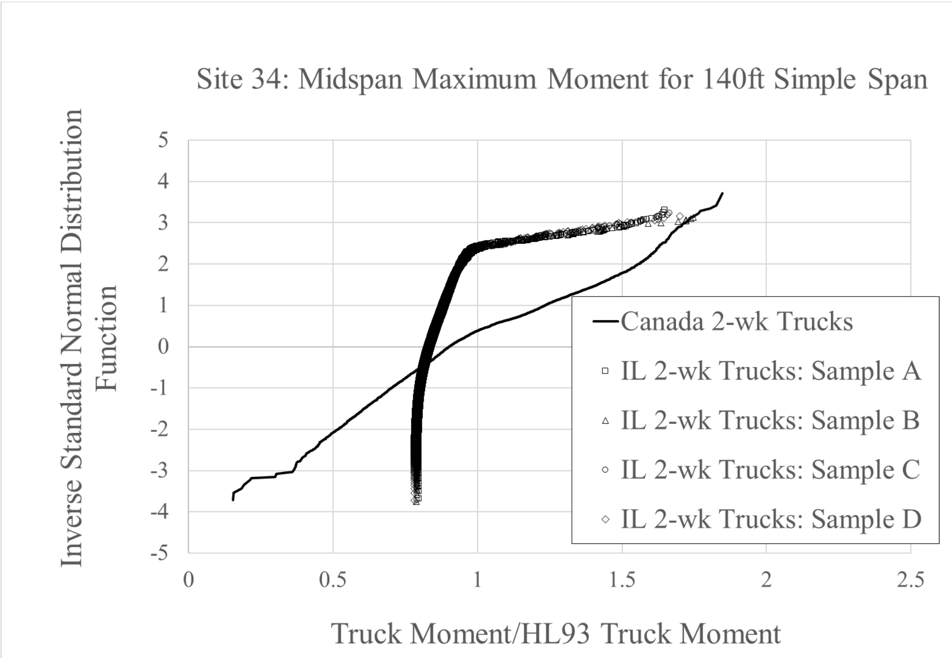

Figure A-101 Moments of Canada and Illinois Trucks at Site 34 for 140ft Span

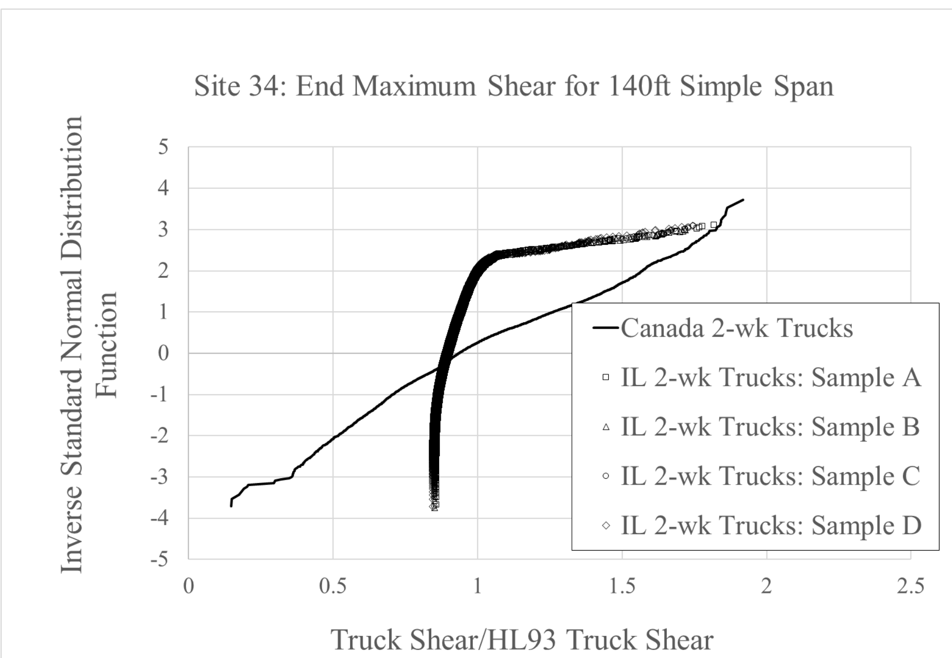

Figure A-102 Shears of Canada and Illinois Trucks at Site 34 for 140ft Span

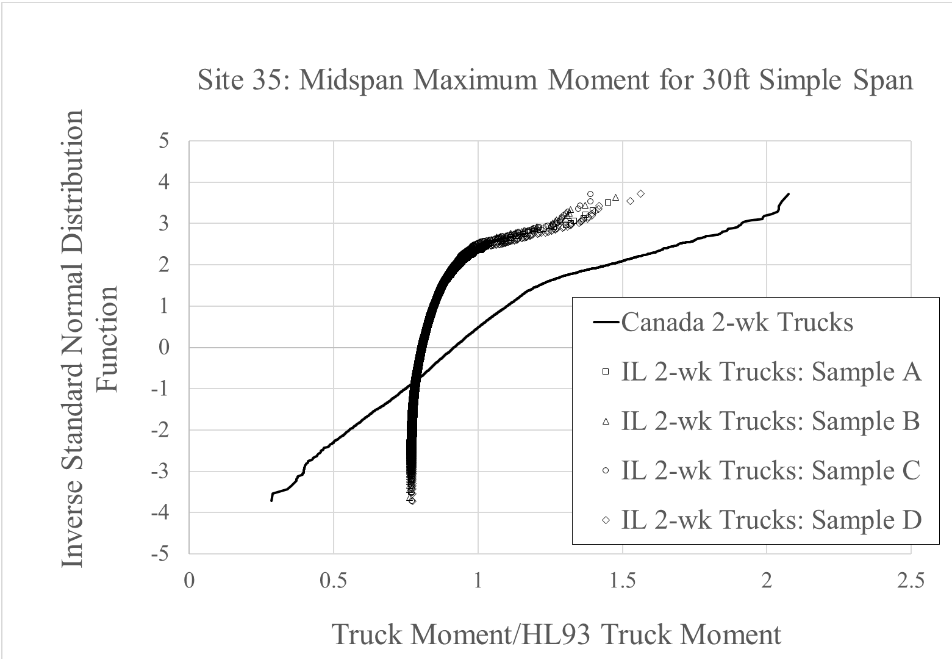

Figure A-103 Moments of Canada and Illinois Trucks at Site 35 for 30ft Span

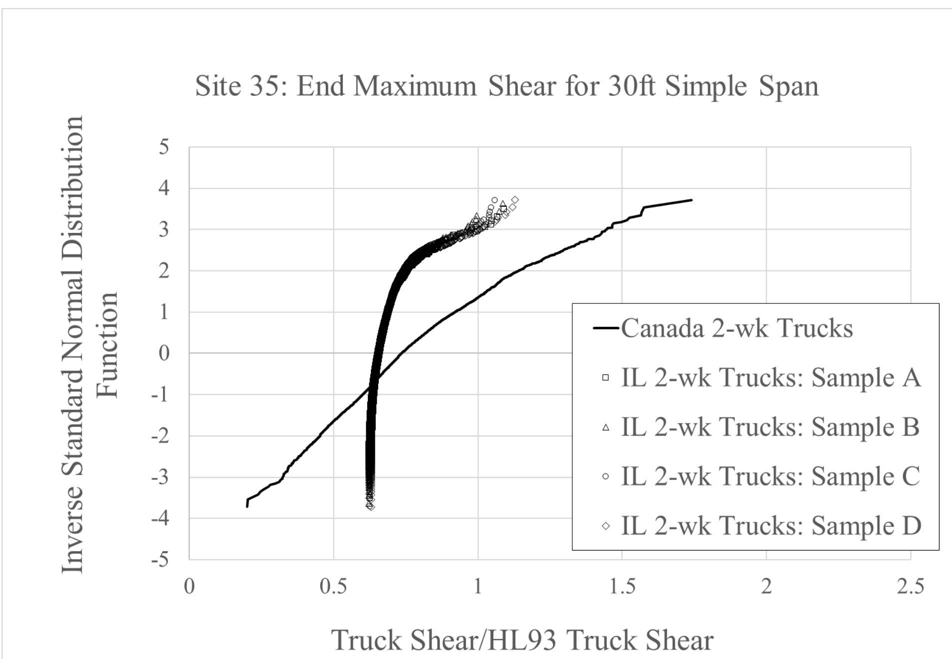

Figure A-104 Shears of Canada and Illinois Trucks at Site 35 for 30ft Span

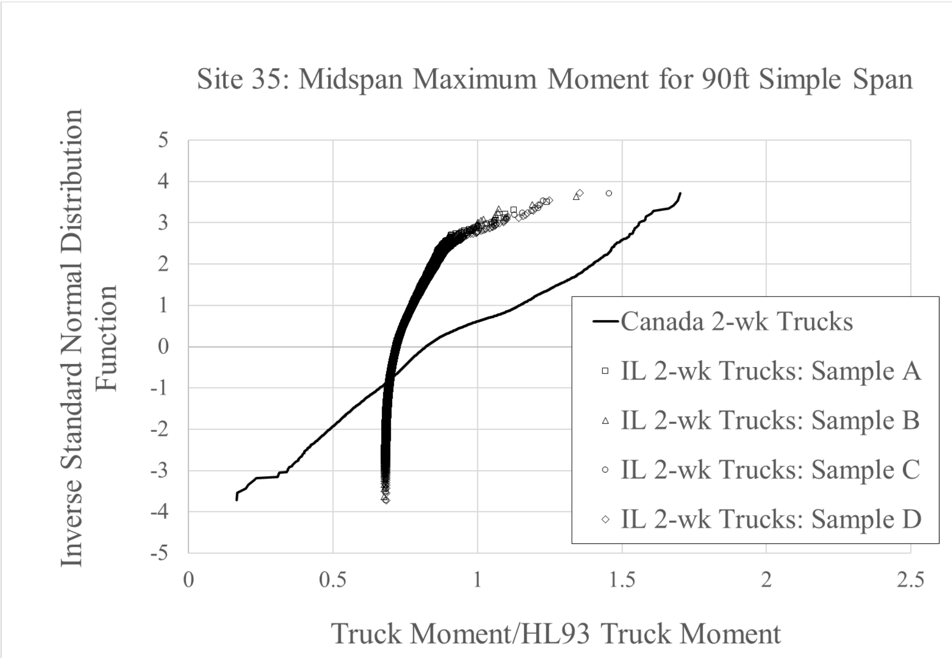

Figure A-105 Moments of Canada and Illinois Trucks at Site 35 for 90ft Span

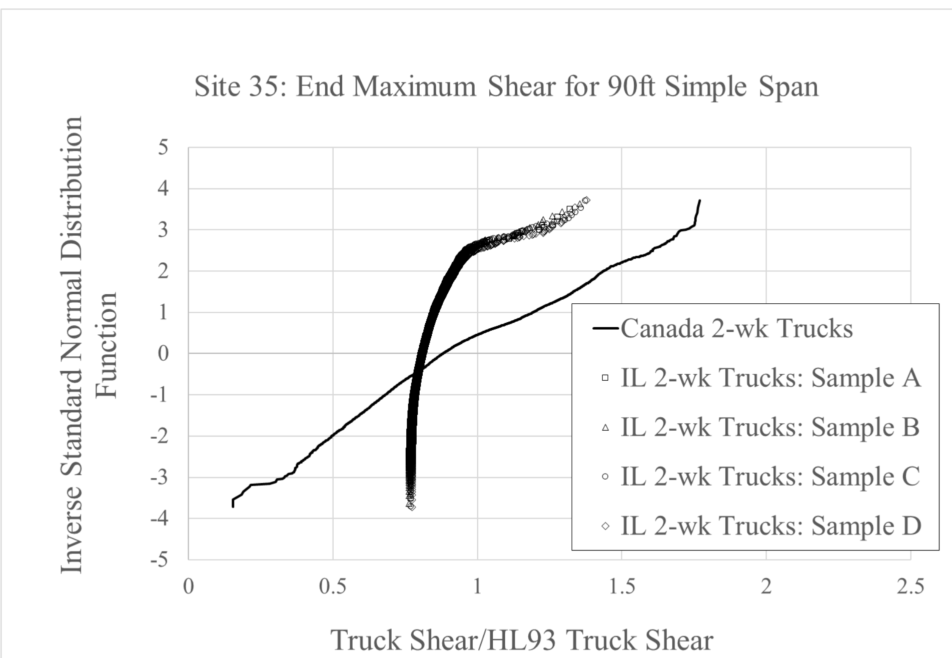

Figure A-106 Shears of Canada and Illinois Trucks at Site 35 for 90ft Span

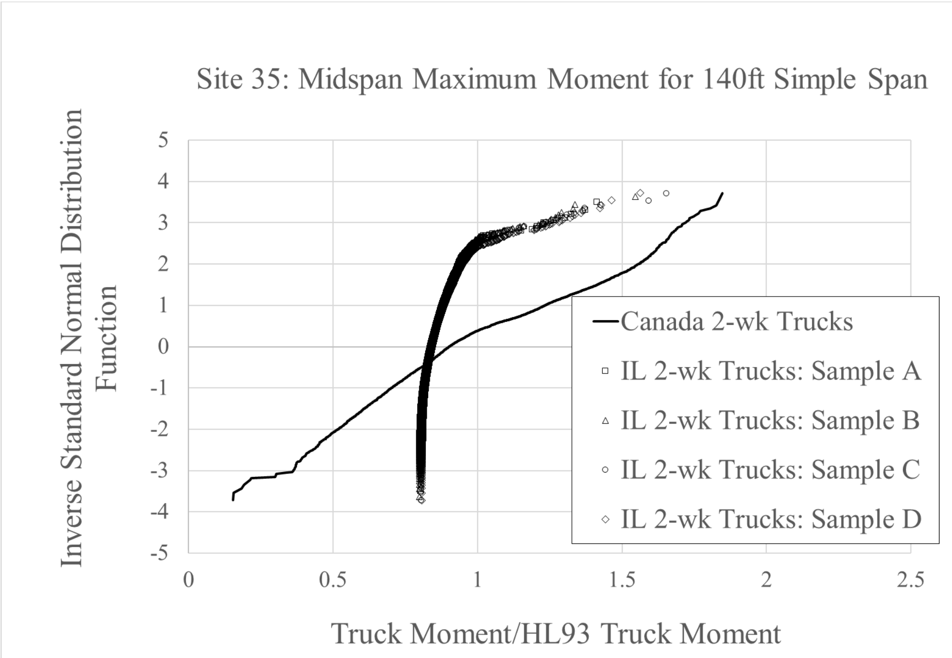

Figure A-107 Moments of Canada and Illinois Trucks at Site 35 for 140ft Span

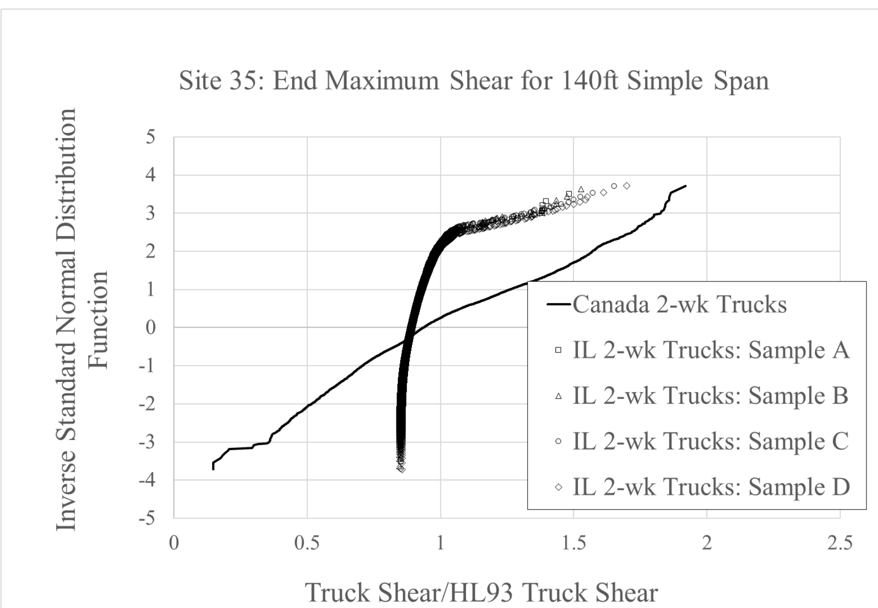

Figure A-108 Shears of Canada and Illinois Trucks at Site 35 for 140ft Span

APPENDIX B: MULTIPLE PRESENCE PROBABILITIES RECORDED IN WIM DATA OF STATES

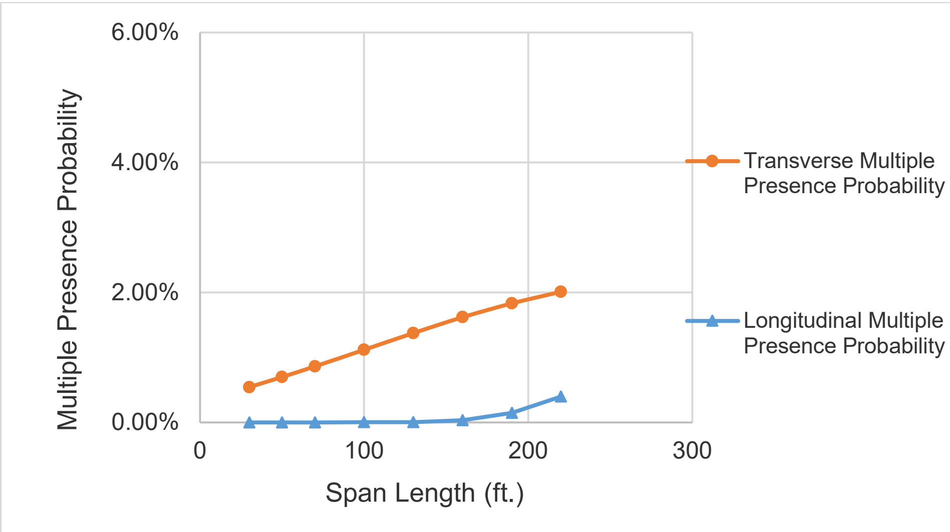

Figure B-1 Multiple Presence Probabilities for WIM Data of MI 6449, ADTT=2,248

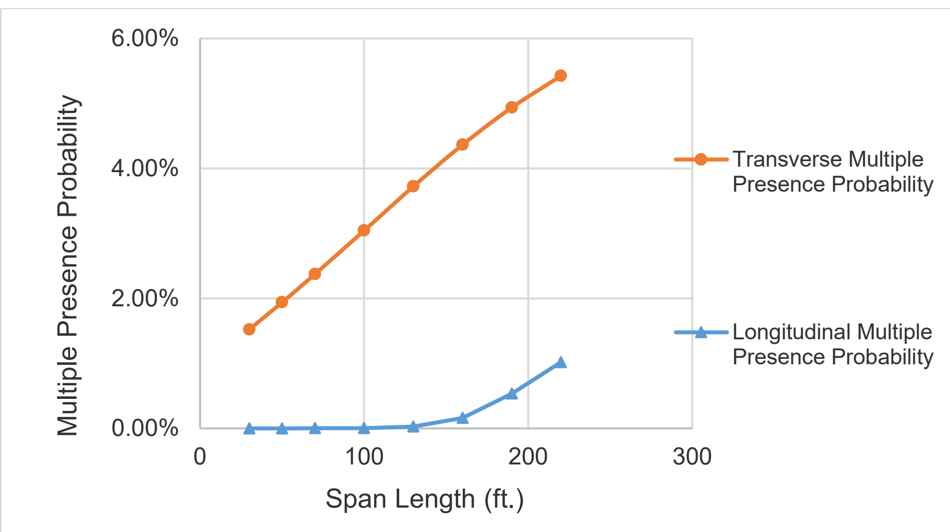

Figure B-2 Multiple Presence Probabilities for WIM Data of MI 7159, ADTT= 5,061

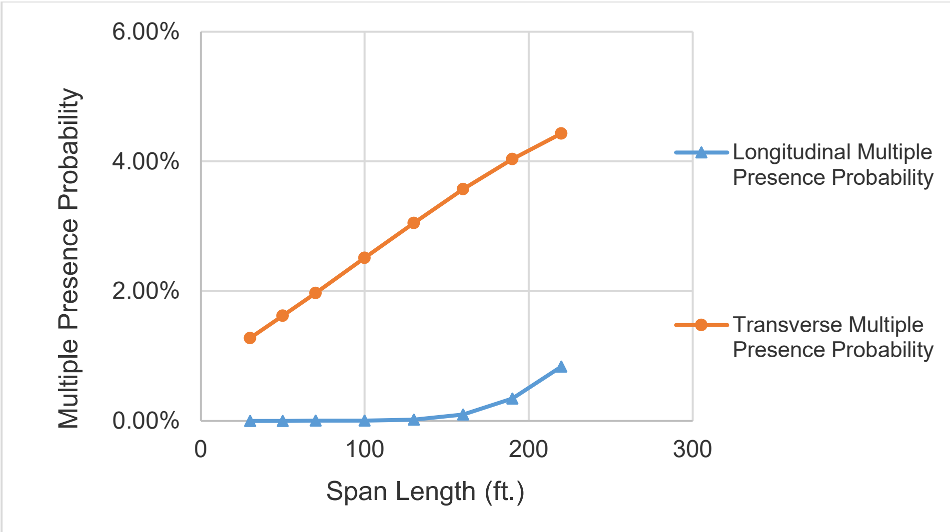

Figure B-3 Multiple Presence Probabilities for WIM Data of MI 7219, ADTT= 3,914

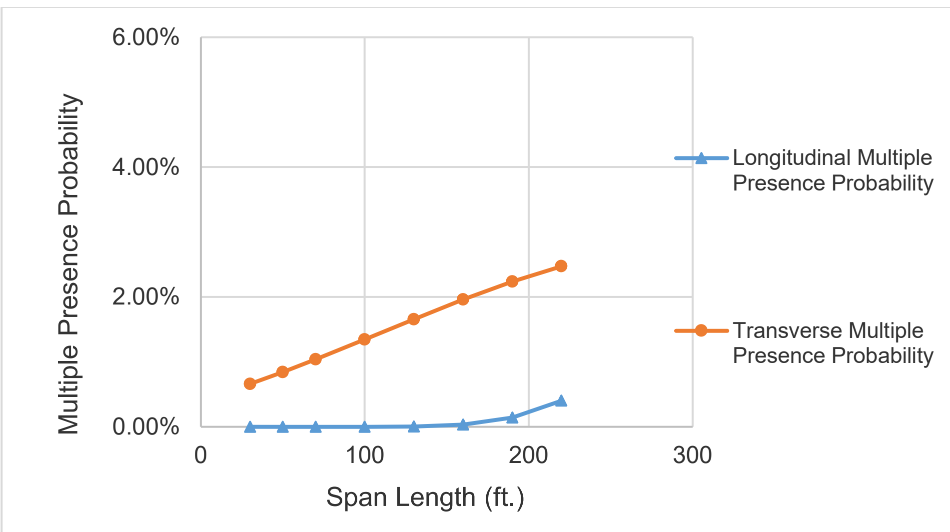

Figure B-4 Multiple Presence Probabilities for WIM Data of MI 7269, ADTT= 2,483

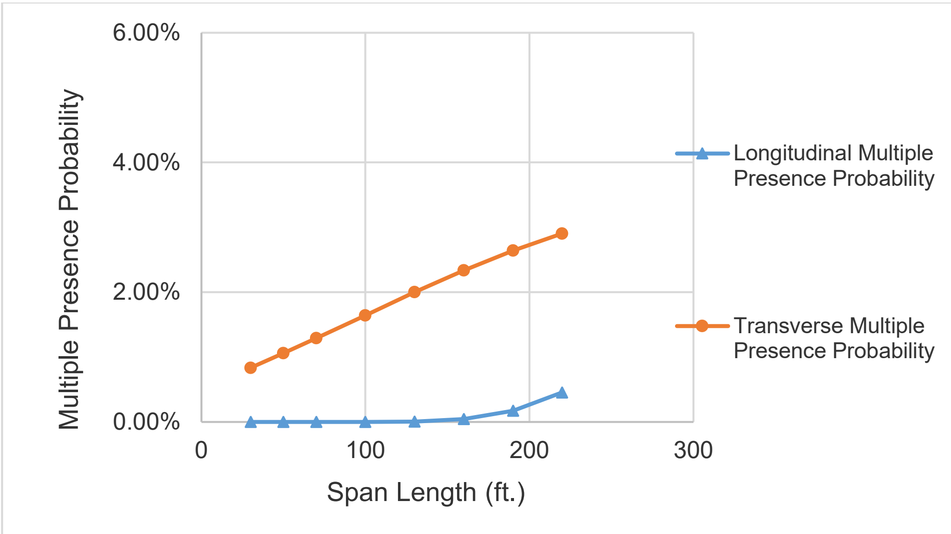

Figure B-5 Multiple Presence Probabilities for WIM Data of MI 7319, ADTT= 2,339

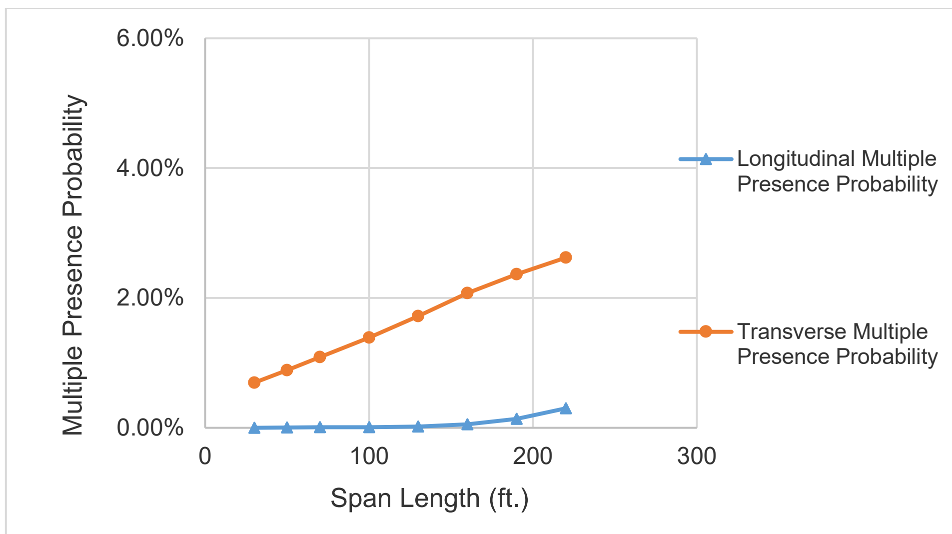

Figure B-6 Multiple Presence Probabilities for WIM Data of NY 3311, ADTT= 1,762

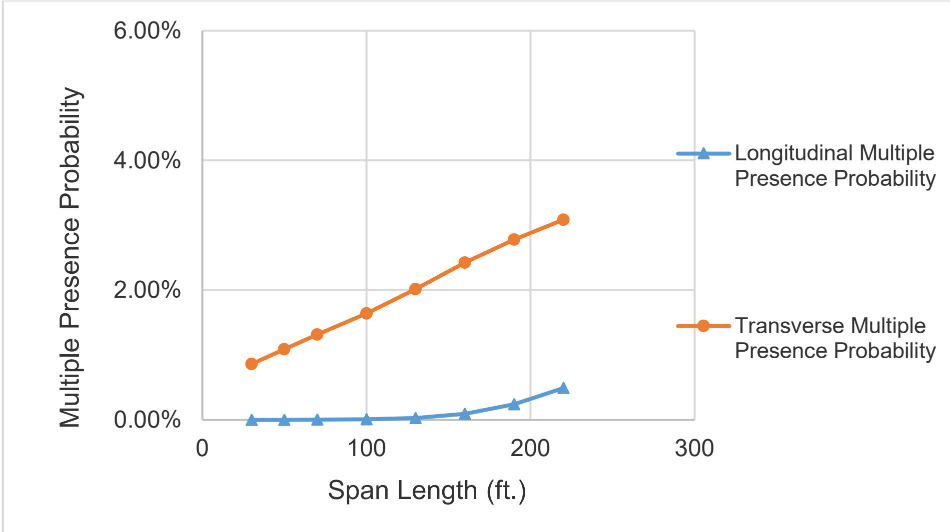

Figure B-7 Multiple Presence Probabilities for WIM Data of NY 9121, ADTT= 1,998

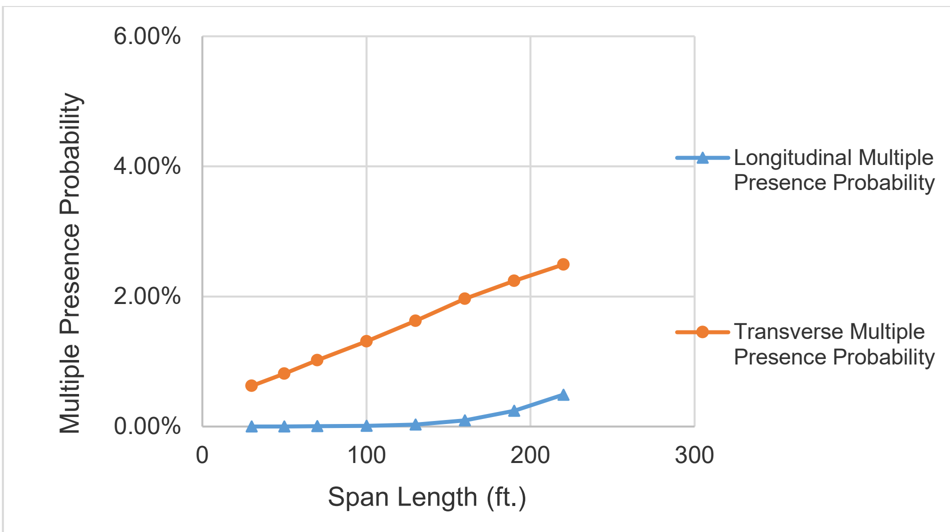

Figure B-8 Multiple Presence Probabilities for WIM Data of NY 9580, ADTT= 1,942

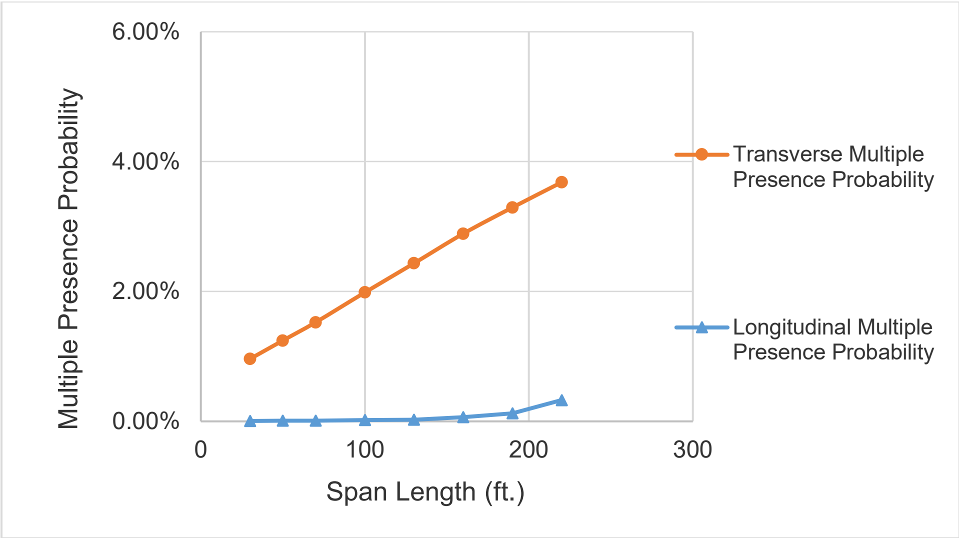

Figure B-9 Multiple Presence Probabilities for WIM Data of MN 026, ADTT= 2,242

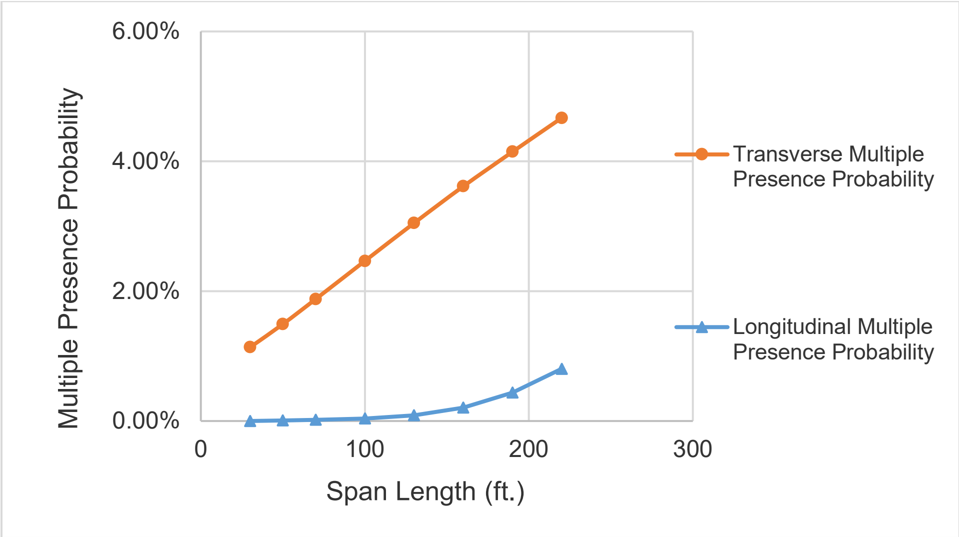

Figure B-10 Multiple Presence Probabilities for WIM Data of MN 037, ADTT= 3,814

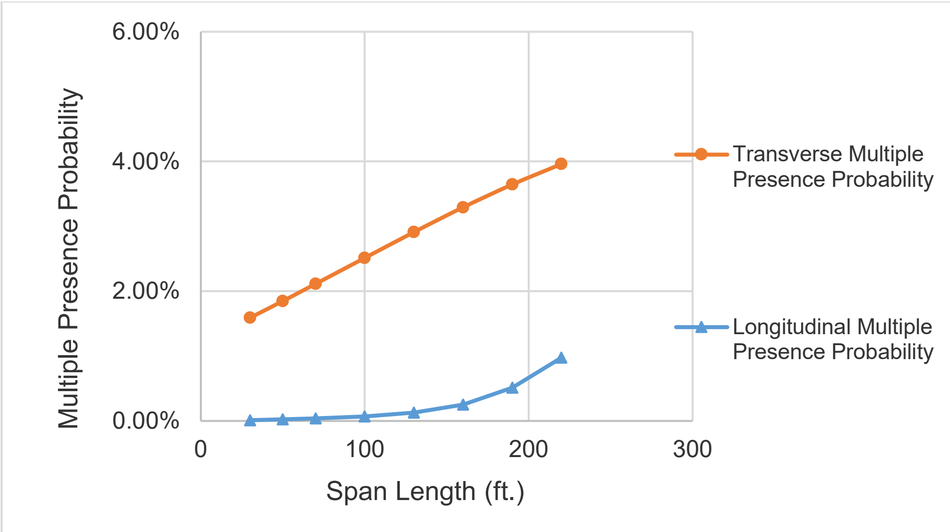

Figure B-11 Multiple Presence Probabilities for WIM Data of CA 002, ADTT= 2,806

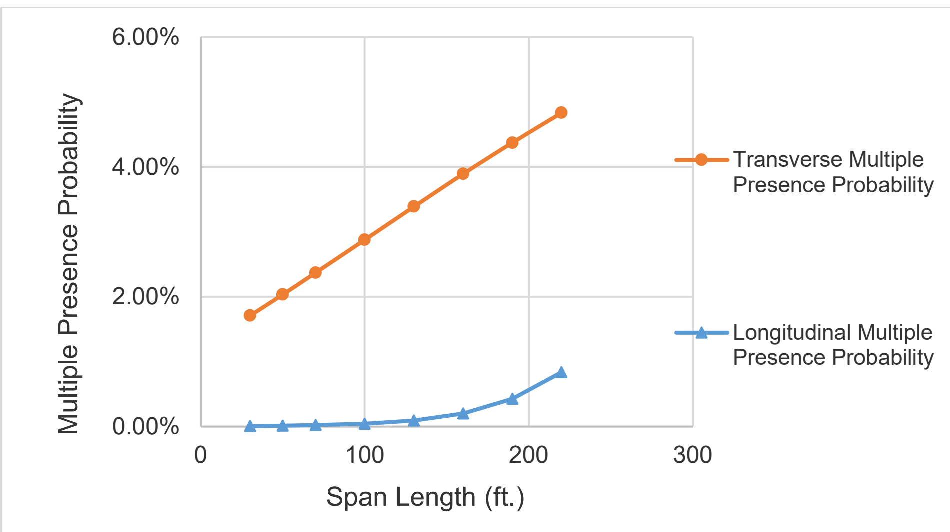

Figure B-12 Multiple Presence Probabilities for WIM Data of CA 005, ADTT= 4,064

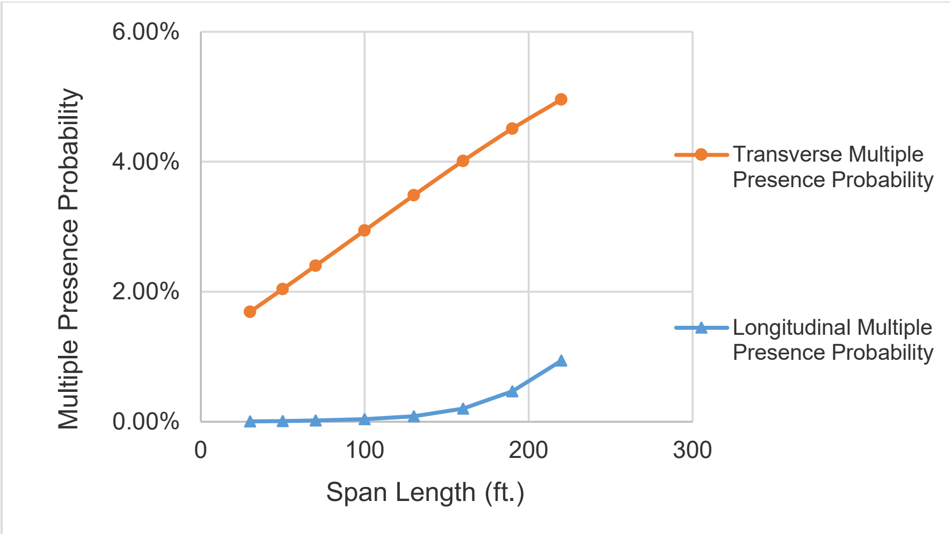

Figure B-13 Multiple Presence Probabilities for WIM Data of CA 007, ADTT= 3,522

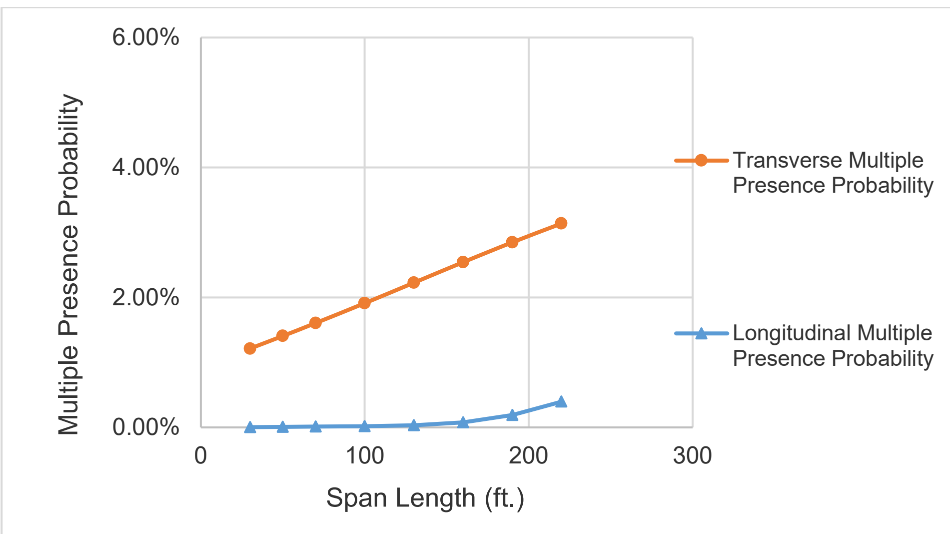

Figure B-14 Multiple Presence Probabilities for WIM Data of CA 025, ADTT= 2,351

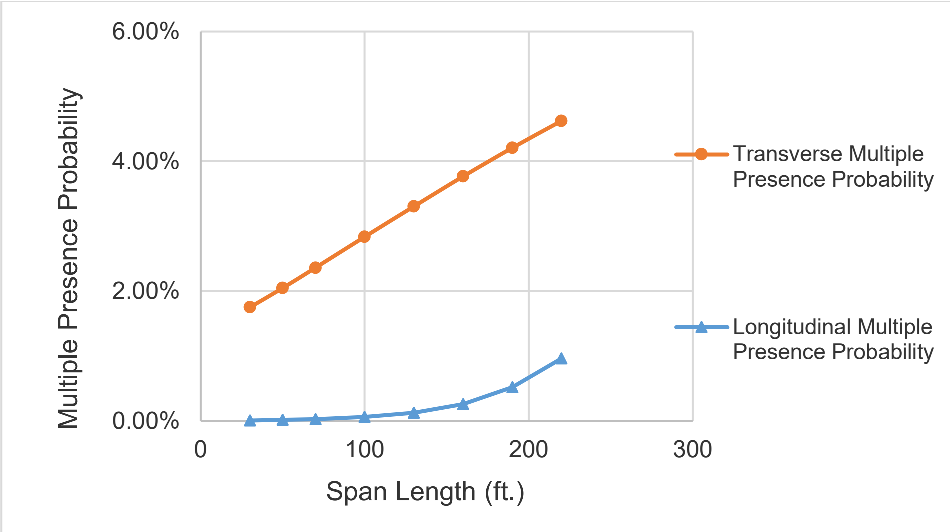

Figure B-15 Multiple Presence Probabilities for WIM Data of CA 066, ADTT= 3,411

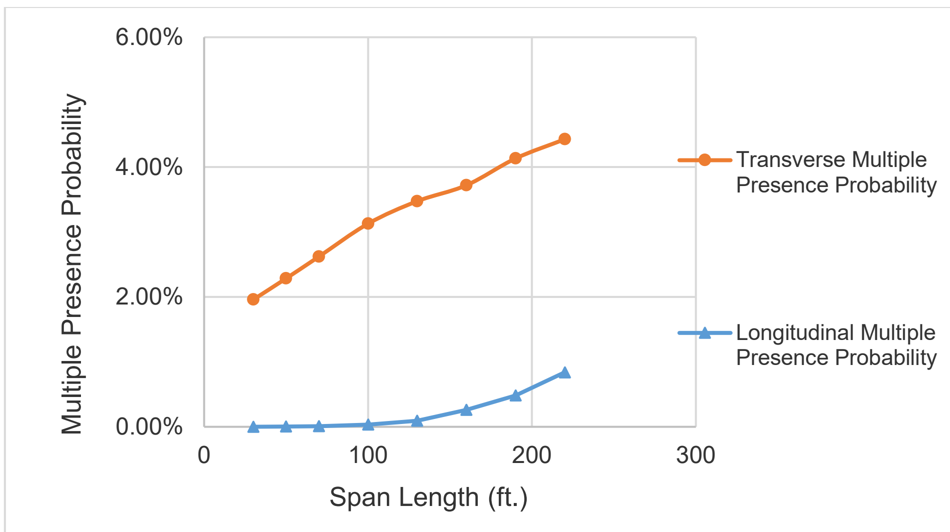

Figure B-16 Multiple Presence Probabilities for WIM Data of IL 16, ADTT= 4,512)

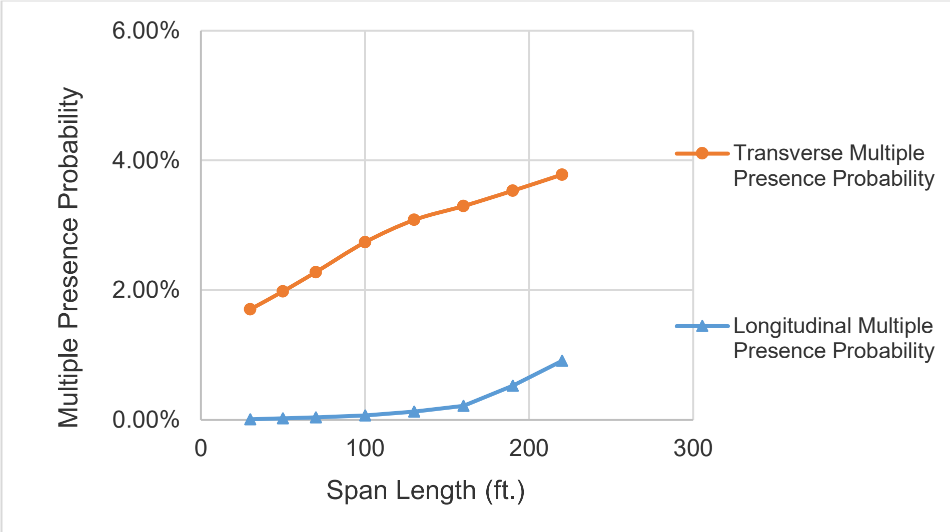

Figure B-17 Multiple Presence Probabilities for WIM Data of Oregon, ADTT= 4,506

APPENDIX C: CLUSTER PROBABILITES RECORDEDD IN WIM DATA OF STATES

Table C-1 Cluster Probabilities for MI 6449

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
2134	30	1.23%
2134	50	1.58%
2134	70	1.86%
2134	100	2.16%
2134	130	2.36%
2134	160	2.52%
2134	190	2.66%
2134	220	2.77%

Table C-2 Cluster Probabilities for MI 7159

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
4621	30	3.40%
4621	50	4.29%
4621	70	5.05%
4621	100	5.88%
4621	130	6.46%
4621	160	6.94%
4621	190	7.35%
4621	220	7.70%

Table C-3 Cluster Probabilities for MI 7219

$ADTT_{SL}$	Span Length (ft)	Cluster Probability
3610	30	2.79%
3610	50	3.50%

3610	70	4.12%
3610	100	4.80%
3610	130	5.28%
3610	160	5.66%
3610	190	5.97%
3610	220	6.25%

Table C-4 Cluster Probabilities for MI 7269

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
2379	30	1.51%
2379	50	1.93%
2379	70	2.30%
2379	100	2.69%
2379	130	2.96%
2379	160	3.13%
2379	190	3.28%
2379	220	3.40%

Table C-5 Cluster Probabilities for MI 7319

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
2222	30	1.82%
2222	50	2.29%
2222	70	2.70%
2222	100	3.14%
2222	130	3.44%
2222	160	3.66%
2222	190	3.83%
2222	220	3.98%

Table C-6 Cluster Probabilities for NY 3311

ADTT_{SL}	Span Length (ft)	Cluster Probability
1590	30	1.46%
1590	50	2.00%
1590	70	2.38%
1590	100	2.85%
1590	130	3.20%
1590	160	3.48%
1590	190	3.71%
1590	220	3.92%

Table C-7 Cluster Probabilities for NY 9121

ADTT_{SL}	Span Length (ft)	Cluster Probability
1803	30	1.73%
1803	50	2.35%
1803	70	2.81%
1803	100	3.36%
1803	130	3.76%
1803	160	4.08%
1803	190	4.36%
1803	220	4.59%

Table C-8 Cluster Probabilities for NY 9580

ADTT_{SL}	Span Length (ft)	Cluster Probability
1699	30	1.36%
1699	50	1.86%
1699	70	2.24%
1699	100	2.71%
1699	130	3.11%
1699	160	3.44%
1699	190	3.72%

1699	220	3.98%

Table C-9 Cluster Probabilities for MN 026

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
1985	30	2.14%
1985	50	2.73%
1985	70	3.30%
1985	100	4.01%
1985	130	4.58%
1985	160	5.05%
1985	190	5.44%
1985	220	5.78%

Table C-10 Cluster Probabilities for MN 037

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
3090	30	2.60%
3090	50	3.37%
3090	70	4.09%
3090	100	5.07%
3090	130	5.94%
3090	160	6.69%
3090	190	7.35%
3090	220	7.96%

Table C-11 Cluster Probabilities for CA 002

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
2556	30	2.63%
2556	50	3.14%
2556	70	3.57%

2556	100	4.10%
2556	130	4.49%
2556	160	4.81%
2556	190	5.08%
2556	220	5.32%

Table C-12 Cluster Probabilities for CA 005

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
3681	30	3.12%
3681	50	3.80%
3681	70	4.42%
3681	100	5.22%
3681	130	5.88%
3681	160	6.42%
3681	190	6.88%
3681	220	7.27%

Table C-13 Cluster Probabilities for CA 007

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
3180	30	3.15%
3180	50	3.85%
3180	70	4.48%
3180	100	5.23%
3180	130	5.80%
3180	160	6.24%
3180	190	6.62%
3180	220	6.95%

Table C-14 Cluster Probabilities for CA 025

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
2164	30	2.08%
2164	50	2.50%
2164	70	2.90%
2164	100	3.40%
2164	130	3.81%
2164	160	4.14%
2164	190	4.41%
2164	220	4.65%

Table C-15 Cluster Probabilities for CA 066

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
3067	30	2.98%
3067	50	3.60%
3067	70	4.15%
3067	100	4.87%
3067	130	5.45%
3067	160	5.93%
3067	190	6.33%
3067	220	6.67%

Table C-16 Cluster Probabilities for IL 15

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
3843	30	2.04%
3843	50	2.50%
3843	70	2.93%
3843	100	3.50%
3843	130	3.92%

3843	160	4.22%
3843	190	4.46%
3843	220	4.68%

Table C-17 Cluster Probabilities for IL 16

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
4338	30	4.07%
4338	50	5.15%
4338	70	5.58%
4338	100	6.65%
4338	130	7.18%
4338	160	7.69%
4338	190	8.19%
4338	220	8.70%

Table C-18 Cluster Probabilities for IL 18

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
4026	30	2.16%
4026	50	2.67%
4026	70	3.14%
4026	100	3.79%
4026	130	4.31%
4026	160	4.72%
4026	190	5.06%
4026	220	5.35%

Table C-19 Cluster Probabilities for Oregon

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
4026	30	4.23%
4026	50	4.49%
4026	70	4.77%
4026	100	5.22%
4026	130	5.70%
4026	160	6.15%
4026	190	6.59%
4026	220	7.03%

Table C-20 Cluster Probabilities for KY 056P98

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
1851	30	2.00%
1851	50	2.61%
1851	70	3.19%
1851	100	3.90%
1851	130	4.52%
1851	160	5.14%
1851	190	5.68%
1851	220	6.19%

Table C-21 Cluster Probabilities for PA 000158

$\mathrm{ADTT_{SL}}$	Span Length (ft)	Cluster Probability
4260	30	3.93%
4260	50	4.93%
4260	70	5.88%
4260	100	7.18%
4260	130	8.26%

4260	160	9.15%
4260	190	9.93%
4260	220	10.60%

Table C-22 Cluster Probabilities for PA 000502

$\mathrm{ADTT}_{\mathrm{SL}}$	Span Length (ft)	Cluster Probability
4402	30	4.70%
4402	50	5.85%
4402	70	6.90%
4402	100	8.24%
4402	130	9.29%
4402	160	10.14%
4402	190	10.86%
4402	220	11.49%

APPENDIX D: VERIFICATION FOR SIMULATION

Table D-1 Simulation Verification Results for MI 6449, $ADTT_{SL}$=2,134

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.69	0.68	-1.13%
	50	0.56	0.53	-4.43%
	70	0.56	0.56	-1.11%
	100	0.60	0.60	-0.56%
	130	0.64	0.61	-4.90%
	160	0.66	0.62	-5.91%
	190	0.67	0.64	-5.31%
	220	0.69	0.65	-4.93%
End Shear	30	0.65	0.64	-0.89%
	50	0.57	0.58	1.62%
	70	0.62	0.60	-2.77%
	100	0.69	0.66	-3.68%
	130	0.70	0.68	-2.67%
	160	0.71	0.68	-4.13%
	190	0.71	0.65	-1.09%
	220	0.73	0.67	-1.69%
Mean of Difference				-2.72%
Standard Deviation of Difference				2.06%

Table D-2 Simulation Verification Results for MI 7159, $ADTT_{SL}$ = 4,621

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.72	0.76	5.39%
	50	0.57	0.56	-1.46%
	70	0.57	0.57	-0.77%
	100	0.59	0.62	5.08%
	130	0.62	0.67	7.77%

	160	0.64	0.64	-0.41%
	190	0.66	0.67	0.82%
	220	0.68	0.66	-3.88%
End Shear	30	0.76	0.78	3.16%
	50	0.62	0.64	2.35%
	70	0.65	0.68	4.91%
	100	0.69	0.73	5.41%
	130	0.71	0.75	4.70%
	160	0.73	0.71	-3.48%
	190	0.79	0.77	-3.22%
	220	0.86	0.81	-5.18%
Mean of Difference				1.33%
Standard Deviation of Difference				3.92%

Table D-3 Simulation Verification Results for MI 7219, $ADTT_{SL}$ = 3,610

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.70	0.69	-1.57%
	50	0.54	0.53	-1.57%
	70	0.56	0.59	5.59%
	100	0.62	0.64	2.91%
	130	0.65	0.67	2.57%
	160	0.67	0.61	-8.17%
	190	0.68	0.66	-2.20%
	220	0.69	0.68	-1.16%
End Shear	30	0.73	0.74	1.88%
	50	0.59	0.57	-3.35%
	70	0.66	0.68	4.07%
	100	0.74	0.72	-1.83%
	130	0.75	0.74	-0.40%
	160	0.74	0.72	-2.87%

	190	0.78	0.79	0.92%
	220	0.86	0.89	3.69%
Mean of Difference				-0.09%
Standard Deviation of Difference				3.37%

Table D-4 Simulation Verification Results for MI 7269, $ADTT_{SL}$ = 2,379

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.78	-4.97%
	50	0.69	0.71	2.51%
	70	0.71	0.76	6.47%
	100	0.74	0.72	-2.89%
	130	0.78	0.76	-2.50%
	160	0.80	0.84	4.85%
	190	0.83	0.88	6.37%
	220	0.85	0.85	-0.19%
End Shear	30	0.76	0.77	1.19%
	50	0.73	0.75	2.99%
	70	0.76	0.76	0.43%
	100	0.83	0.78	-6.09%
	130	0.86	0.85	-1.58%
	160	0.88	0.91	3.27%
	190	0.90	0.93	3.08%
	220	0.92	0.87	-4.80%
Mean of Difference				0.51%
Standard Deviation of Difference				3.88%

Table D-5 Simulation Verification Results for MI 7319, $ADTT_{SL}$ = 2,222

Load Effect	Span Length (ft)	R_T	R_S	Difference

Mid-Span Moment	30	0.82	0.84	3.23%
	50	0.64	0.69	7.87%
	70	0.64	0.69	9.09%
	100	0.69	0.68	-2.45%
	130	0.73	0.80	8.53%
	160	0.76	0.71	-5.39%
	190	0.77	0.71	-7.09%
	220	0.78	0.73	-6.63%
End Shear	30	0.79	0.79	0.01%
	50	0.68	0.66	-2.31%
	70	0.71	0.75	6.47%
	100	0.80	0.76	-4.71%
	130	0.85	0.87	2.52%
	160	0.86	0.84	-2.31%
	190	0.87	0.82	-5.84%
	220	0.90	0.86	-4.11%
Mean of Difference				-0.20%
Standard Deviation of Difference				5.53%

Table D-6 Simulation Verification Results for NY 3311, $ADTT_{SL}$ = 1,590

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.84	0.79	-5.41%
	50	0.71	0.70	-1.55%
	70	0.72	0.71	-1.41%
	100	0.78	0.79	0.47%
	130	0.83	0.85	2.23%
	160	0.85	0.86	1.10%
	190	0.87	0.84	-4.00%
	220	0.88	0.89	0.84%

End Shear	30	0.78	0.77	-0.65%
	50	0.74	0.74	0.59%
	70	0.80	0.78	-2.87%
	100	0.87	0.82	-4.93%
	130	0.90	0.88	-1.42%
	160	0.91	0.90	-1.75%
	190	0.92	0.87	-5.43%
	220	0.92	0.92	-0.82%
Mean of Difference				-1.56%
Standard Deviation of Difference				2.32%

Table D-7 Simulation Verification Results for NY 9121, $ADTT_{SL}$ = 1,803

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.82	-0.79%
	50	0.71	0.71	1.21%
	70	0.73	0.70	-3.38%
	100	0.78	0.79	0.79%
	130	0.83	0.82	-1.55%
	160	0.86	0.86	-0.11%
	190	0.87	0.88	0.61%
	220	0.89	0.81	-8.31%
End Shear	30	0.78	0.77	-1.35%
	50	0.73	0.75	2.38%
	70	0.77	0.76	-1.29%
	100	0.83	0.83	0.11%
	130	0.87	0.84	-2.91%
	160	0.89	0.89	-0.29%
	190	0.91	0.88	-3.18%

	220	0.92	0.87	-5.07%
Mean of Difference				-1.45%
Standard Deviation of Difference				2.57%

Table D-8 Simulation Verification Results for NY 9580, $ADTT_{SL}$ = 1,699

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.82	0.84	2.66%
	50	0.70	0.71	1.78%
	70	0.72	0.77	6.53%
	100	0.79	0.78	-0.58%
	130	0.84	0.82	-1.78%
	160	0.86	0.86	-0.53%
	190	0.87	0.88	0.55%
	220	0.87	0.87	-0.11%
End Shear	30	0.78	0.79	1.74%
	50	0.74	0.74	1.18%
	70	0.78	0.81	2.82%
	100	0.85	0.85	0.33%
	130	0.88	0.89	0.84%
	160	0.90	0.88	-2.07%
	190	0.91	0.89	-2.41%
	220	0.92	0.88	-4.33%
Mean of Difference				0.41%
Standard Deviation of Difference				2.45%

Table D-9 Simulation Verification Results for MN 026, $ADTT_{SL}$ = 1,985

Load Effect	Span Length (ft)	R_T	R_S	Difference

Mid-Span Moment	30	0.84	0.80	-5.30%
	50	0.70	0.65	-6.78%
	70	0.71	0.67	-5.27%
	100	0.73	0.72	-1.23%
	130	0.77	0.78	1.65%
	160	0.79	0.76	-4.15%
	190	0.81	0.85	5.00%
	220	0.82	0.86	4.12%
End Shear	30	0.78	0.77	-1.29%
	50	0.70	0.69	-0.65%
	70	0.77	0.77	0.49%
	100	0.88	0.85	-2.97%
	130	0.90	0.87	-3.16%
	160	0.91	0.92	1.12%
	190	0.91	0.89	-1.63%
	220	0.93	0.91	-1.58%
Mean of Difference				-1.35%
Standard Deviation of Difference				3.20%

Table D-10 Simulation Verification Results for MN 037, $ADTT_{SL}$ = 3,090

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.84	0.87	3.99%
	50	0.71	0.76	7.42%
	70	0.73	0.73	-0.03%
	100	0.81	0.76	-5.80%
	130	0.85	0.85	-0.10%
	160	0.86	0.80	-7.88%
	190	0.84	0.91	8.41%

	220	0.82	0.81	-1.78%
End Shear	30	0.78	0.80	3.42%
	50	0.74	0.73	-0.23%
	70	0.79	0.79	0.51%
	100	0.85	0.83	-2.73%
	130	0.88	0.86	-2.92%
	160	0.86	0.89	3.03%
	190	0.87	0.83	-3.51%
	220	0.89	0.92	3.00%
Mean of Difference				0.30%
Standard Deviation of Difference				4.31%

Table D-11 Simulation Verification Results for CA 002, $ADTT_{SL}$ = 2,556

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.83	-0.43%
	50	0.71	0.71	0.20%
	70	0.73	0.70	-3.91%
	100	0.80	0.83	4.12%
	130	0.84	0.84	-0.71%
	160	0.87	0.88	2.00%
	190	0.89	0.87	-2.05%
	220	0.91	0.90	-0.71%
End Shear	30	0.78	0.78	0.27%
	50	0.73	0.76	3.64%
	70	0.79	0.77	-1.51%
	100	0.85	0.82	-3.23%
	130	0.88	0.87	-1.56%
	160	0.90	0.87	-3.91%

	190	0.92	0.91	-0.99%
	220	0.93	0.96	3.79%
Mean of Difference				-0.31%
Standard Deviation of Difference				2.49%

Table D-12 Simulation Verification Results for CA 005, $ADTT_{SL}$ = 3,681

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.82	0.84	2.60%
	50	0.70	0.74	5.18%
	70	0.72	0.71	-2.21%
	100	0.79	0.79	0.39%
	130	0.83	0.84	0.80%
	160	0.86	0.85	-1.09%
	190	0.88	0.86	-2.12%
	220	0.90	0.88	-2.63%
End Shear	30	0.78	0.78	0.22%
	50	0.74	0.74	0.27%
	70	0.79	0.77	-1.50%
	100	0.85	0.86	0.76%
	130	0.89	0.88	-1.01%
	160	0.91	0.88	-2.39%
	190	0.92	0.91	-0.98%
	220	0.93	0.91	-1.50%
Mean of Difference				-0.32%
Standard Deviation of Difference				1.99%

Table D-13 Simulation Verification Results for CA 007, $ADTT_{SL}$ = 3,180

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.82	0.83	0.69%
	50	0.70	0.72	2.28%
	70	0.72	0.73	1.12%
	100	0.80	0.79	-1.29%
	130	0.84	0.84	-0.96%
	160	0.87	0.85	-3.26%
	190	0.89	0.89	-0.49%
	220	0.91	0.90	-0.76%
End Shear	30	0.78	0.80	3.08%
	50	0.73	0.73	-0.37%
	70	0.78	0.80	1.60%
	100	0.85	0.85	-0.43%
	130	0.88	0.88	-0.63%
	160	0.90	0.89	-1.06%
	190	0.91	0.92	0.64%
	220	0.92	0.93	1.57%
Mean of Difference				0.11%
Standard Deviation of Difference				1.53%

Table D-14 Simulation Verification Results for CA 025, $ADTT_{SL}$ = 2,164

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.80	-3.37%
	50	0.71	0.71	1.20%
	70	0.73	0.71	-2.18%
	100	0.80	0.84	5.40%
	130	0.85	0.82	-3.19%
	160	0.87	0.87	-0.33%
	190	0.89	0.88	-1.37%

	220	0.91	0.89	-1.35%
End Shear	30	0.78	0.78	-0.20%
	50	0.73	0.74	1.20%
	70	0.79	0.79	1.03%
	100	0.85	0.85	0.41%
	130	0.88	0.86	-2.78%
	160	0.90	0.87	-4.30%
	190	0.92	0.92	0.66%
	220	0.93	0.92	-0.23%
Mean of Difference				-0.59%
Standard Deviation of Difference				2.30%

Table D-15 Simulation Verification Results for CA 066, $ADTT_{SL}$ = 3,067

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.82	0.81	-0.84%
	50	0.68	0.70	3.99%
	70	0.70	0.73	4.54%
	100	0.76	0.80	4.83%
	130	0.80	0.86	6.43%
	160	0.83	0.87	4.60%
	190	0.85	0.88	3.10%
	220	0.87	0.88	1.09%
End Shear	30	0.78	0.78	-0.70%
	50	0.72	0.73	2.54%
	70	0.76	0.81	5.95%
	100	0.81	0.83	2.28%
	130	0.84	0.88	4.85%
	160	0.86	0.91	5.04%

	190	0.88	0.91	4.00%
	220	0.89	0.91	1.99%
Mean of Difference				3.36%
Standard Deviation of Difference				2.09%

Table D-16 Simulation Verification Results for IL 16, $ADTT_{SL}$ = 4,124

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.85	2.25%
	50	0.71	0.68	-4.18%
	70	0.73	0.73	0.26%
	100	0.79	0.78	-1.49%
	130	0.83	0.85	2.56%
	160	0.86	0.86	0.09%
	190	0.87	0.89	2.27%
	220	0.89	0.90	1.48%
End Shear	30	0.78	0.78	-0.46%
	50	0.73	0.70	-4.75%
	70	0.79	0.76	-4.14%
	100	0.85	0.81	-4.89%
	130	0.88	0.86	-2.97%
	160	0.90	0.87	-3.28%
	190	0.92	0.89	-2.66%
	220	0.93	0.88	-4.92%
Mean of Difference				-1.42%
Standard Deviation of Difference				2.78%

Table D-17 Simulation Verification Results for KY 056P98, $ADTT_{SL}$ = 1,851

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.81	-1.93%
	50	0.71	0.70	-0.44%
	70	0.73	0.69	-6.37%
	100	0.79	0.79	-0.55%
	130	0.84	0.83	-0.84%
	160	0.87	0.80	-7.93%
	190	0.89	0.83	-6.67%
	220	0.90	0.92	1.74%
End Shear	30	0.78	0.78	-0.48%
	50	0.74	0.70	-4.76%
	70	0.79	0.80	1.53%
	100	0.86	0.82	-4.78%
	130	0.89	0.90	1.15%
	160	0.90	0.85	-5.97%
	190	0.92	0.87	-4.97%
	220	0.93	0.96	2.81%
Mean of Difference				-2.40%
Standard Deviation of Difference				3.36%

Table D-18 Simulation Verification Results for PA 000158, $ADTT_{SL}$ = 4,260

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.76	-8.87%
	50	0.71	0.71	0.96%
	70	0.73	0.75	2.30%
	100	0.80	0.81	2.39%
	130	0.85	0.85	0.52%
	160	0.87	0.90	3.12%
	190	0.89	0.92	3.39%

	220	0.91	0.93	3.00%
End Shear	30	0.78	0.78	0.42%
	50	0.73	0.74	0.95%
	70	0.79	0.81	3.68%
	100	0.85	0.88	3.87%
	130	0.88	0.91	2.92%
	160	0.90	0.93	2.69%
	190	0.92	0.94	2.19%
	220	0.93	0.96	3.08%
Mean of Difference				1.66%
Standard Deviation of Difference				2.92%

Table D-19 Simulation Verification Results for PA 000502, $ADTT_{SL}$ = 4,402

Load Effect	Span Length (ft)	R_T	R_S	Difference
Mid-Span Moment	30	0.83	0.81	-2.90%
	50	0.70	0.69	-1.90%
	70	0.72	0.70	-3.72%
	100	0.80	0.80	0.22%
	130	0.85	0.87	3.15%
	160	0.87	0.87	-0.19%
	190	0.89	0.90	0.77%
	220	0.91	0.92	1.62%
End Shear	30	0.78	0.77	-0.82%
	50	0.73	0.70	-4.69%
	70	0.79	0.77	-1.40%
	100	0.85	0.87	1.82%
	130	0.88	0.89	0.11%
	160	0.90	0.91	0.37%
	190	0.92	0.93	0.74%

220	0.93	0.92	-0.70%
Mean of Difference			-0.47%
Standard Deviation of Difference			2.01%

APPENDIX E: RESULTS FOR LEGAL LOAD RATING CALIBRATION

Table E-1 Live Load Factors for Legal Load Rating for IL-4 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.01	0.98	1.03
50	1.17	1.18	1.15
70	1.22	1.22	1.21
100	1.27	1.25	1.25
130	1.28	1.24	1.25
160	1.29	1.23	1.26
190	1.27	1.21	1.23
220	1.25	1.21	1.21

Table E-2 Live Load Factors for Legal Load Rating for IL-6 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	1.01	0.99
50	1.18	1.13	1.12
70	1.18	1.19	1.16
100	1.24	1.24	1.24
130	1.28	1.23	1.24
160	1.29	1.25	1.27
190	1.29	1.28	1.28
220	1.30	1.31	1.30

Table E-3 Live Load Factors for Legal Load Rating for IL-7 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	1.04	1.16
50	1.32	1.22	1.20
70	1.31	1.24	1.14
100	1.29	1.21	1.20
130	1.24	1.17	1.22
160	1.19	1.15	1.18
190	1.19	1.16	1.18
220	1.18	1.16	1.18

Table E-4 Live Load Factors for Legal Load Rating for IL-10 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.07	1.08	1.14
50	1.29	1.26	1.29
70	1.31	1.31	1.27
100	1.35	1.21	1.22
130	1.33	1.21	1.22
160	1.26	1.25	1.28
190	1.26	1.28	1.32
220	1.29	1.29	1.32

Table E-5 Live Load Factors for Legal Load Rating for IL-12 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.04	0.98	1.06
50	1.21	1.14	1.14
70	1.19	1.18	1.13
100	1.20	1.21	1.16
130	1.19	1.26	1.16
160	1.21	1.27	1.19
190	1.26	1.25	1.21
220	1.27	1.25	1.22

Table E-6 Live Load Factors for Legal Load Rating for IL-13 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.99	0.95	0.97
50	1.12	1.09	1.11
70	1.14	1.15	1.16
100	1.19	1.13	1.22
130	1.20	1.12	1.24
160	1.20	1.14	1.20
190	1.18	1.16	1.20
220	1.17	1.18	1.21

Table E-7 Live Load Factors for Legal Load Rating for IL-14 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	1.15	1.20
50	1.32	1.30	1.24
70	1.33	1.32	1.26
100	1.34	1.34	1.36
130	1.37	1.33	1.34
160	1.40	1.34	1.37
190	1.40	1.36	1.38
220	1.41	1.37	1.36

Table E-8 Live Load Factors for Legal Load Rating for IL-15 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.04	1.04	0.98
50	1.25	1.19	1.17
70	1.27	1.22	1.21
100	1.28	1.21	1.26
130	1.27	1.21	1.29
160	1.27	1.24	1.26
190	1.26	1.26	1.28
220	1.26	1.27	1.28

Table E-9 Live Load Factors for Legal Load Rating for IL-16 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	1.05	1.03
50	1.21	1.18	1.20
70	1.28	1.29	1.27
100	1.36	1.32	1.30
130	1.35	1.27	1.27
160	1.33	1.27	1.28
190	1.32	1.28	1.29
220	1.32	1.30	1.31

Table E-10 Live Load Factors for Legal Load Rating for IL-18 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.19	1.11	1.24
50	1.39	1.28	1.33
70	1.42	1.36	1.22
100	1.40	1.39	1.34
130	1.40	1.38	1.38
160	1.41	1.37	1.36
190	1.38	1.35	1.33
220	1.38	1.34	1.33

Table E-11 Live Load Factors for Legal Load Rating for IL-19 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.04	1.13
50	1.33	1.20	1.26
70	1.31	1.27	1.23
100	1.35	1.28	1.23
130	1.33	1.27	1.27
160	1.31	1.27	1.25
190	1.30	1.28	1.26
220	1.31	1.29	1.28

Table E-12 Live Load Factors for Legal Load Rating for IL-21 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.07	0.95	1.01
50	1.27	1.19	1.19
70	1.30	1.27	1.23
100	1.31	1.30	1.25
130	1.30	1.29	1.28
160	1.32	1.32	1.32
190	1.35	1.33	1.31
220	1.37	1.31	1.31

Table E-13 Live Load Factors for Legal Load Rating for IL-22 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.11	1.02	1.13
50	1.31	1.17	1.18
70	1.29	1.17	1.08
100	1.24	1.10	1.09
130	1.22	1.05	1.09
160	1.16	1.10	1.12
190	1.13	1.14	1.13
220	1.15	1.16	1.17

Table E-14 Live Load Factors for Legal Load Rating for IL-23 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.22	1.10	1.19
50	1.37	1.28	1.21
70	1.35	1.30	1.24
100	1.35	1.29	1.29
130	1.34	1.23	1.25
160	1.31	1.24	1.24
190	1.27	1.24	1.25
220	1.26	1.25	1.25

Table E-15 Live Load Factors for Legal Load Rating for IL-24 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.01	0.99	0.95
50	1.18	1.12	1.10
70	1.17	1.13	1.18
100	1.19	1.17	1.15
130	1.17	1.16	1.15
160	1.17	1.16	1.14
190	1.18	1.19	1.15
220	1.17	1.19	1.15

Table E-16 Live Load Factors for Legal Load Rating for IL-26 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear

30	1.16	1.09	1.23
50	1.37	1.26	1.28
70	1.35	1.29	1.27
100	1.34	1.20	1.27
130	1.31	1.19	1.23
160	1.31	1.24	1.25
190	1.31	1.26	1.27
220	1.29	1.25	1.28

Table E-17 Live Load Factors for Legal Load Rating (Data Source: IL-31 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.03	1.02	1.08
50	1.23	1.19	1.16
70	1.27	1.24	1.19
100	1.32	1.28	1.25
130	1.31	1.29	1.28
160	1.30	1.30	1.27
190	1.29	1.30	1.26
220	1.28	1.28	1.24

Table E-18 Live Load Factors for Legal Load Rating (Data Source: IL-32 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.99	1.00	0.97
50	1.15	1.13	1.11
70	1.26	1.18	1.20
100	1.27	1.24	1.28
130	1.29	1.27	1.30
160	1.33	1.30	1.30
190	1.34	1.32	1.31
220	1.34	1.33	1.32

Table E-19 Live Load Factors for Legal Load Rating (Data Source: IL-34 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear

30	1.03	0.98	0.95
50	1.18	1.13	1.08
70	1.15	1.18	1.17
100	1.17	1.19	1.22
130	1.17	1.16	1.22
160	1.18	1.18	1.23
190	1.18	1.21	1.24
220	1.20	1.21	1.23

Table E-20 Live Load Factors for Legal Load Rating (Data Source: IL-35 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.04	1.06	0.98
50	1.24	1.25	1.19
70	1.24	1.23	1.23
100	1.19	1.12	1.15
130	1.18	1.14	1.13
160	1.18	1.16	1.18
190	1.18	1.18	1.20
220	1.19	1.20	1.20

Table E-21 Live Load Factors for Legal Load Rating for IL-4 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.08	1.01	1.03
50	1.23	1.16	1.14
70	1.20	1.24	1.23
100	1.24	1.24	1.22
130	1.25	1.24	1.25
160	1.23	1.23	1.22
190	1.25	1.25	1.24
220	1.22	1.22	1.20

Table E-22 Live Load Factors for Legal Load Rating for IL-6 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.05	1.00	1.05
50	1.27	1.24	1.20
70	1.22	1.27	1.19
100	1.23	1.25	1.23
130	1.25	1.28	1.26
160	1.32	1.32	1.30
190	1.33	1.34	1.34
220	1.27	1.28	1.28

Table E-23 Live Load Factors for Legal Load Rating for IL-7 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.11	1.03	1.07
50	1.20	1.18	1.19
70	1.19	1.26	1.26
100	1.25	1.27	1.28
130	1.28	1.27	1.27
160	1.29	1.27	1.28
190	1.30	1.29	1.29
220	1.29	1.27	1.28

Table E-24 Live Load Factors for Legal Load Rating for IL-10 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	1.02	1.04
50	1.22	1.12	1.17
70	1.21	1.26	1.23
100	1.26	1.29	1.31
130	1.28	1.33	1.32
160	1.31	1.32	1.31
190	1.35	1.35	1.34
220	1.34	1.35	1.33

Table E-25 Live Load Factors for Legal Load Rating for IL-12 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	0.99	1.02
50	1.26	1.18	1.21
70	1.28	1.24	1.18
100	1.21	1.19	1.18
130	1.21	1.20	1.17
160	1.18	1.22	1.16
190	1.19	1.17	1.17
220	1.21	1.19	1.19

Table E-26 Live Load Factors for Legal Load Rating for IL-13 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.07	0.99	1.01
50	1.10	1.06	1.09
70	1.05	1.12	1.15
100	1.20	1.24	1.23
130	1.17	1.22	1.20
160	1.25	1.23	1.23
190	1.27	1.22	1.24
220	1.22	1.22	1.20

Table E-27 Live Load Factors for Legal Load Rating for IL-14 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.15	1.09	1.08
50	1.27	1.25	1.31
70	1.29	1.37	1.33
100	1.32	1.37	1.38
130	1.40	1.41	1.41
160	1.43	1.41	1.42
190	1.42	1.41	1.41
220	1.43	1.41	1.41

Table E-28 Live Load Factors for Legal Load Rating for IL-15 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.12	1.03	1.04
50	1.22	1.21	1.21
70	1.26	1.27	1.23
100	1.26	1.25	1.24
130	1.26	1.26	1.26
160	1.28	1.26	1.26
190	1.29	1.28	1.27
220	1.28	1.27	1.26

Table E-29 Live Load Factors for Legal Load Rating for IL-16 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.15	1.06	1.11
50	1.27	1.22	1.22
70	1.20	1.27	1.25
100	1.33	1.28	1.30
130	1.36	1.35	1.32
160	1.33	1.29	1.31
190	1.33	1.29	1.30
220	1.34	1.31	1.31

Table E-30 Live Load Factors for Legal Load Rating for IL-18 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.11	1.08	1.09
50	1.28	1.21	1.30
70	1.34	1.34	1.34
100	1.42	1.31	1.37
130	1.38	1.34	1.35
160	1.40	1.38	1.39
190	1.36	1.33	1.34
220	1.38	1.36	1.36

Table E-31 Live Load Factors for Legal Load Rating for IL-19 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.06	1.04	1.09
50	1.26	1.20	1.24
70	1.25	1.22	1.23
100	1.29	1.28	1.24
130	1.30	1.28	1.28
160	1.30	1.30	1.29
190	1.29	1.25	1.26
220	1.26	1.25	1.25

Table E-32 Live Load Factors for Legal Load Rating for IL-21 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	1.10	1.09
50	1.28	1.24	1.29
70	1.39	1.33	1.42
100	1.33	1.27	1.28
130	1.32	1.30	1.26
160	1.31	1.33	1.35
190	1.33	1.35	1.31
220	1.36	1.35	1.34

Table E-33 Live Load Factors for Legal Load Rating for IL-22 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	0.98	0.99
50	1.20	1.15	1.15
70	1.12	1.18	1.17
100	1.19	1.17	1.19
130	1.19	1.20	1.19
160	1.24	1.21	1.21
190	1.20	1.21	1.21
220	1.20	1.20	1.19

Table E-34 Live Load Factors for Legal Load Rating for IL-23 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.06	1.02	1.00
50	1.21	1.17	1.21
70	1.29	1.22	1.25
100	1.35	1.25	1.28
130	1.31	1.25	1.27
160	1.27	1.26	1.24
190	1.26	1.24	1.25
220	1.25	1.24	1.22

Table E-35 Live Load Factors for Legal Load Rating for IL-24 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.02	0.98	0.95
50	1.13	1.09	1.12
70	1.15	1.19	1.17
100	1.20	1.20	1.19
130	1.21	1.20	1.17
160	1.21	1.19	1.19
190	1.21	1.19	1.18
220	1.23	1.23	1.22

Table E-36 Live Load Factors for Legal Load Rating for IL-26 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	0.97	1.02
50	1.18	1.17	1.19
70	1.18	1.22	1.19
100	1.30	1.29	1.31
130	1.30	1.29	1.29
160	1.30	1.30	1.30
190	1.33	1.29	1.29
220	1.31	1.30	1.29

Table E-37 Live Load Factors for Legal Load Rating for IL-31 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.95	0.97	0.93
50	1.24	1.21	1.30
70	1.32	1.25	1.24
100	1.34	1.25	1.25
130	1.36	1.27	1.30
160	1.31	1.28	1.30
190	1.28	1.24	1.25
220	1.27	1.24	1.26

Table E-38 Live Load Factors for Legal Load Rating for IL-32 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.09	1.05	1.04
50	1.23	1.27	1.25
70	1.31	1.28	1.26
100	1.26	1.25	1.26
130	1.32	1.30	1.30
160	1.32	1.29	1.28
190	1.27	1.27	1.26
220	1.27	1.25	1.26

Table E-39 Live Load Factors for Legal Load Rating for IL-34 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.00	0.98	0.96
50	1.11	1.12	1.15
70	1.25	1.17	1.16
100	1.23	1.20	1.20
130	1.19	1.20	1.18
160	1.19	1.20	1.19
190	1.20	1.19	1.18
220	1.22	1.21	1.20

Table E-40 Live Load Factors for Legal Load Rating for IL-35 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.14	1.05	1.06
50	1.15	1.16	1.21
70	1.20	1.22	1.22
100	1.25	1.25	1.22
130	1.27	1.23	1.24
160	1.29	1.26	1.26
190	1.27	1.26	1.25
220	1.26	1.24	1.24

APPENDIX F: RESULTS FOR ROUTINE PERMIT LOAD RATING CALIBRATION

Table F-1 Live Load Factors for Routine Permit Load Rating for IL-4 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.06	1.27	1.11
50	1.39	1.52	1.33
70	1.42	1.37	1.28
100	1.23	1.25	1.15
130	1.10	1.17	1.04
160	1.01	1.04	0.94
190	0.93	0.96	0.85
220	0.87	0.94	0.83

Table F-2 Live Load Factors for Routine Permit Load Rating for IL-6 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.18	1.67	1.14
50	1.54	1.62	1.11
70	1.41	1.46	1.07
100	1.20	1.36	1.09
130	1.13	1.17	0.98
160	1.05	1.04	0.85
190	0.93	0.99	0.77
220	0.83	0.98	0.73

Table F-3 Live Load Factors for Routine Permit Load Rating for IL-7 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.32	1.13
50	1.44	1.42	1.14
70	1.42	1.32	1.14
100	1.22	1.15	1.40
130	1.09	0.98	1.27
160	0.97	0.92	1.10

190	0.90	0.93	1.02
220	0.85	0.96	1.02

Table F-4 Live Load Factors for Routine Permit Load Rating for IL-10 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.30	1.69	1.31
50	1.70	1.72	1.21
70	1.60	1.61	1.22
100	1.38	1.34	1.12
130	1.23	1.11	1.24
160	1.02	1.05	1.10
190	0.89	1.07	1.10
220	0.84	1.06	1.11

Table F-5 Live Load Factors for Routine Permit Load Rating for IL-12 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.09	1.31	1.05
50	1.28	1.39	1.08
70	1.26	1.33	1.05
100	1.11	1.32	1.03
130	1.06	1.24	1.33
160	1.04	1.09	1.24
190	1.01	1.02	1.14
220	0.94	1.01	1.10

Table F-6 Live Load Factors for Routine Permit Load Rating for IL-13 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.08	1.37	1.12
50	1.40	1.46	1.21
70	1.35	1.38	1.19
100	1.18	1.24	1.15
130	1.06	1.06	1.28
160	0.94	0.98	1.07

190	0.84	0.97	1.01
220	0.76	0.98	1.00

Table F-7 Live Load Factors for Routine Permit Load Rating for IL-14 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.17	1.37	1.15
50	1.52	1.52	1.31
70	1.58	1.41	1.36
100	1.40	1.23	1.26
130	1.25	1.04	1.33
160	1.11	0.99	1.19
190	1.01	1.01	1.10
220	0.93	1.02	1.06

Table F-8 Live Load Factors for Routine Permit Load Rating for IL-15 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.33	1.46	1.51
50	1.57	1.42	1.51
70	1.48	1.36	1.41
100	1.30	1.22	1.39
130	1.18	1.05	1.20
160	1.06	0.97	1.04
190	0.96	1.00	1.01
220	0.89	1.03	1.03

Table F-9 Live Load Factors for Routine Permit Load Rating for IL-16 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.26	1.53	1.56
50	1.63	1.54	1.61
70	1.54	1.45	1.53
100	1.28	1.27	1.30
130	1.05	1.05	1.11
160	0.94	0.98	1.03

190	0.88	0.97	1.00
220	0.84	0.99	0.99

Table F-10 Live Load Factors for Routine Permit Load Rating for IL-18 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.15	1.50	1.10
50	1.59	1.68	1.33
70	1.62	1.62	1.36
100	1.42	1.34	1.24
130	1.28	1.18	1.09
160	1.11	1.12	0.95
190	0.98	1.11	0.88
220	0.92	1.10	0.84

Table F-11 Live Load Factors for Routine Permit Load Rating for IL-19 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.32	1.31	1.37
50	1.53	1.37	1.51
70	1.51	1.28	1.54
100	1.29	1.08	1.36
130	1.12	0.96	1.17
160	0.99	0.95	1.02
190	0.91	0.99	0.99
220	0.87	1.02	1.02

Table F-12 Live Load Factors for Routine Permit Load Rating for IL-21 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.37	1.32	1.60
50	1.46	1.33	1.35
70	1.33	1.38	1.39
100	1.25	1.45	1.44
130	1.24	1.33	1.35
160	1.20	1.11	1.16

190	1.10	1.04	1.03
220	0.98	1.04	1.02

Table F-13 Live Load Factors for Routine Permit Load Rating for IL-22 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.12	1.28	1.38
50	1.37	1.36	1.36
70	1.34	1.28	1.37
100	1.18	1.16	1.28
130	1.11	1.00	1.11
160	1.01	0.95	1.04
190	0.91	0.98	1.02
220	0.86	0.99	1.03

Table F-14 Live Load Factors for Routine Permit Load Rating for IL-23 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.10	1.09	1.46
50	1.45	1.18	1.29
70	1.45	1.11	1.25
100	1.28	1.02	1.19
130	1.18	1.18	1.03
160	1.06	1.10	1.18
190	0.93	1.07	1.06
220	0.86	1.06	1.02

Table F-15 Live Load Factors for Routine Permit Load Rating for IL-24 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.11	1.43	1.50
50	1.49	1.59	1.25
70	1.47	1.49	1.24
100	1.28	1.37	1.14
130	1.12	1.15	0.96

160	0.97	0.99	1.12
190	0.86	0.95	1.01
220	0.79	0.96	0.97

Table F-16 Live Load Factors for Routine Permit Load Rating for IL-26 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.22	1.19	1.23
50	1.57	1.25	1.29
70	1.51	1.14	1.24
100	1.32	0.95	1.63
130	1.17	1.13	1.29
160	0.98	1.01	1.11
190	0.85	1.01	1.06
220	0.76	1.02	1.06

Table F-17 Live Load Factors for Routine Permit Load Rating for IL-31 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.27	1.45	1.21
50	1.46	1.37	1.10
70	1.34	1.27	1.31
100	1.13	1.21	1.30
130	1.04	1.08	1.21
160	0.96	1.01	1.06
190	0.89	1.03	1.01
220	0.84	1.04	1.00

Table F-18 Live Load Factors for Routine Permit Load Rating for IL-32 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.21	1.25	1.21
50	1.47	1.33	1.25
70	1.49	1.32	1.25
100	1.33	1.28	1.48

130	1.24	1.10	1.33
160	1.14	0.99	1.13
190	1.02	0.97	1.05
220	0.94	0.99	1.03

Table F-19 Live Load Factors for Routine Permit Load Rating for IL-34 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.28	1.24	1.34
50	1.34	1.33	1.36
70	1.30	1.28	1.38
100	1.18	1.25	1.31
130	1.13	1.16	1.17
160	1.08	1.06	1.06
190	0.99	1.08	1.07
220	0.93	1.10	1.08

Table F-20 Live Load Factors for Routine Permit Load Rating for IL-35 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	0.89	1.40	1.28
50	1.24	1.34	1.16
70	1.28	1.25	1.43
100	1.12	1.08	1.30
130	1.07	0.97	1.13
160	1.08	0.94	1.02
190	1.07	0.95	0.98
220	1.05	0.96	0.97

Table F-21 Live Load Factors for Routine Permit Load Rating for IL-4 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.26	1.16	1.40
50	1.34	1.08	1.12
70	1.35	1.08	1.14

100	1.00	0.99	1.07
130	0.96	1.02	1.05
160	1.03	1.10	1.11
190	0.93	1.00	1.03
220	0.98	1.03	1.08

Table F-22 Live Load Factors for Routine Permit Load Rating for IL-6 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.19	1.21	1.36
50	1.49	1.20	1.19
70	1.30	1.07	1.12
100	1.16	1.04	1.10
130	1.15	1.07	1.10
160	1.09	1.05	1.06
190	1.15	1.03	1.05
220	1.17	0.96	1.02

Table F-23 Live Load Factors for Routine Permit Load Rating for IL-7 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.26	1.15	1.46
50	1.38	1.22	1.26
70	1.28	1.06	1.14
100	1.14	1.11	1.22
130	1.04	1.09	1.11
160	1.01	1.08	1.09
190	1.06	1.11	1.15
220	1.01	1.10	1.12

Table F-24 Live Load Factors for Routine Permit Load Rating for IL-10 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.41	1.31	1.42
50	1.44	1.20	1.29

70	1.32	1.23	1.24
100	1.20	1.17	1.25
130	1.10	1.19	1.20
160	1.18	1.24	1.27
190	1.15	1.23	1.24
220	1.21	1.28	1.28

Table F-25 Live Load Factors for Routine Permit Load Rating for IL-12 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.12	1.11	1.27
50	1.40	1.14	1.17
70	1.32	1.09	1.14
100	1.04	1.00	1.07
130	1.08	1.03	1.14
160	0.97	1.06	1.05
190	0.93	1.04	1.06
220	0.96	1.08	1.08

Table F-26 Live Load Factors for Routine Permit Load Rating for IL-13 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.17	1.18	1.31
50	1.29	1.10	1.19
70	1.15	1.05	1.06
100	1.05	0.98	1.07
130	0.97	1.02	1.06
160	1.10	1.08	1.14
190	0.97	0.99	1.02
220	0.95	1.03	1.03

Table F-27 Live Load Factors for Routine Permit Load Rating for IL-14 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.29	1.20	1.36
50	1.36	1.20	1.21

70	1.38	1.22	1.22
100	1.22	1.18	1.29
130	1.14	1.19	1.21
160	1.12	1.21	1.20
190	1.19	1.24	1.26
220	1.09	1.14	1.16

Table F-28 Live Load Factors for Routine Permit Load Rating for IL-15 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.18	1.36
50	1.40	1.22	1.25
70	1.42	1.23	1.25
100	1.14	1.12	1.14
130	1.06	1.06	1.10
160	1.05	1.09	1.11
190	1.06	1.08	1.09
220	1.06	1.06	1.12

Table F-29 Live Load Factors for Routine Permit Load Rating for IL-16 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.22	1.19	1.40
50	1.38	1.21	1.24
70	1.25	1.10	1.15
100	1.15	1.09	1.16
130	1.07	1.10	1.11
160	1.01	1.03	1.08
190	1.01	1.07	1.08
220	0.97	1.02	1.06

Table F-30 Live Load Factors for Routine Permit Load Rating for IL-18 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.31	1.17	1.52
50	1.52	1.34	1.31

70	1.47	1.21	1.19
100	1.24	1.37	1.43
130	1.08	1.18	1.12
160	1.11	1.08	1.20
190	1.09	1.09	1.17
220	1.05	1.13	1.16

Table F-31 Live Load Factors for Routine Permit Load Rating for IL-19 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.27	1.20	1.43
50	1.54	1.28	1.30
70	1.32	1.13	1.16
100	1.15	1.10	1.16
130	1.08	1.10	1.15
160	1.04	1.11	1.11
190	1.10	1.13	1.15
220	1.03	1.13	1.12

Table F-32 Live Load Factors for Routine Permit Load Rating for IL-21 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.27	1.26	1.36
50	1.43	1.18	1.15
70	1.40	1.14	1.23
100	1.16	1.09	1.15
130	1.11	1.12	1.13
160	1.02	1.07	1.05
190	1.09	1.12	1.11
220	1.03	1.03	1.05

Table F-33 Live Load Factors for Routine Permit Load Rating for IL-22 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.18	1.13	1.32
50	1.34	1.18	1.18

70	1.25	1.07	1.09
100	1.07	1.02	1.09
130	1.00	1.02	1.03
160	1.02	1.06	1.07
190	0.95	1.01	1.02
220	0.93	1.01	1.00

Table F-34 Live Load Factors for Routine Permit Load Rating for IL-23 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.22	1.21	1.32
50	1.40	1.25	1.24
70	1.20	1.06	1.12
100	1.13	1.07	1.14
130	1.08	1.07	1.12
160	1.06	1.04	1.09
190	0.97	1.01	1.05
220	1.00	1.02	1.06

Table F-35 Live Load Factors for Routine Permit Load Rating for IL-24 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.13	1.10	1.26
50	1.33	1.15	1.16
70	1.24	1.14	1.20
100	1.13	1.12	1.16
130	1.04	1.05	1.08
160	1.03	1.06	1.09
190	1.04	1.10	1.09
220	1.00	1.08	1.06

Table F-36 Live Load Factors for Routine Permit Load Rating for IL-26 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.34	1.20	1.34

50	1.40	1.23	1.24
70	1.27	1.15	1.23
100	1.19	1.18	1.28
130	1.12	1.18	1.19
160	1.13	1.19	1.21
190	1.15	1.18	1.21
220	1.09	1.18	1.18

Table F-37 Live Load Factors for Routine Permit Load Rating for IL-31 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.08	1.12	1.24
50	1.42	1.21	1.28
70	1.21	1.05	1.09
100	1.11	1.10	1.14
130	1.01	1.02	1.06
160	1.01	1.01	1.07
190	0.98	1.00	1.04
220	0.97	1.02	1.06

Table F-38 Live Load Factors for Routine Permit Load Rating for IL-32 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.19	1.13	1.29
50	1.61	1.34	1.33
70	1.25	1.09	1.12
100	1.11	1.11	1.19
130	1.10	1.11	1.14
160	1.09	1.15	1.17
190	1.07	1.15	1.13
220	1.09	1.13	1.18

Table F-39 Live Load Factors for Routine Permit Load Rating for IL-34 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear

30	1.21	1.23	1.36
50	1.42	1.28	1.29
70	1.33	1.18	1.25
100	1.16	1.17	1.22
130	1.11	1.17	1.20
160	1.06	1.16	1.18
190	1.03	1.15	1.14
220	1.00	1.13	1.13

Table F-40 Live Load Factors for Routine Permit Load Rating for IL-35 for Two-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.18	1.12	1.29
50	1.27	1.10	1.16
70	1.20	1.05	1.09
100	1.07	1.05	1.09
130	1.02	1.03	1.08
160	0.99	1.02	1.05
190	1.02	1.04	1.06
220	1.01	1.06	1.08

APPENDIX G: RESULTS FOR SPECIAL PERMIT LOAD RATING CALIBRATION

Table G-1 Live Load Factors for Special Permit Load Rating for IL-4 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.02	0.96
50	1.19	1.06	1.09
70	1.18	1.06	1.11
100	1.16	1.25	1.30
130	1.05	1.21	1.29
160	1.03	1.11	1.19
190	0.99	1.01	1.06
220	0.92	0.95	0.98

Table G-2 Live Load Factors for Special Permit Load Rating for IL-6 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.32	1.39	1.36
50	1.35	1.36	1.16
70	1.31	1.40	1.49
100	1.31	1.32	1.47
130	1.21	1.17	1.36
160	1.15	1.05	1.19
190	1.04	0.98	1.07
220	0.93	0.93	0.98

Table G-3 Live Load Factors for Special Permit Load Rating for IL-7 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.34	1.32	1.03
50	1.29	1.34	1.08
70	1.25	1.34	1.41
100	1.20	1.15	1.31
130	1.07	0.99	1.20

160	0.96	0.92	1.06
190	0.89	0.89	0.98
220	0.84	0.87	0.93

Table G-4 Live Load Factors for Special Permit Load Rating for IL-10 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.21	1.14	1.12
50	1.32	1.11	1.15
70	1.23	1.47	1.49
100	1.21	1.37	1.46
130	1.20	1.19	1.25
160	1.07	1.07	1.14
190	0.96	1.01	1.06
220	0.87	0.96	0.99

Table G-5 Live Load Factors for Special Permit Load Rating for IL-12 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.10	0.24	0.91
50	1.14	0.99	1.07
70	1.15	1.45	1.06
100	1.16	1.32	1.01
130	1.08	1.29	1.00
160	1.07	1.16	1.28
190	1.04	1.08	1.17
220	0.96	1.05	1.12

Table G-6 Live Load Factors for Special Permit Load Rating for IL-13 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.15	1.32	0.99
50	1.16	1.29	1.03
70	1.09	1.31	1.03
100	1.04	1.11	1.36
130	0.97	1.02	1.24

160	0.93	0.94	1.08
190	0.86	0.90	0.98
220	0.79	0.86	0.91

Table G-7 Live Load Factors for Special Permit Load Rating for IL-14 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.44	1.34	1.31
50	1.35	1.33	1.34
70	1.32	1.32	1.46
100	1.27	1.17	1.38
130	1.19	1.03	1.23
160	1.10	0.98	1.15
190	1.03	0.97	1.09
220	0.98	0.96	1.02

Table G-8 Live Load Factors for Special Permit Load Rating for IL-15 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.10	0.99	0.99
50	1.17	0.99	1.05
70	1.14	1.02	1.05
100	1.13	1.25	1.11
130	1.07	1.15	1.40
160	1.01	1.06	1.22
190	0.92	1.00	1.10
220	0.85	0.95	1.00

Table G-9 Live Load Factors for Special Permit Load Rating for IL-16 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.24	1.45	1.41
50	1.35	1.38	1.48
70	1.30	1.41	1.42
100	1.24	1.19	1.23
130	1.03	1.04	1.10

160	0.95	0.96	1.01
190	0.88	0.92	0.94
220	0.84	0.88	0.88

Table G-10 Live Load Factors for Special Permit Load Rating for IL-18 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.31	1.46	1.36
50	1.37	1.40	1.43
70	1.34	1.51	1.54
100	1.33	1.28	1.37
130	1.23	1.09	1.27
160	1.09	1.03	1.15
190	0.97	1.00	1.07
220	0.90	0.98	1.02

Table G-11 Live Load Factors for Special Permit Load Rating for IL-19 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.30	1.34	1.06
50	1.29	1.33	1.15
70	1.21	1.35	1.21
100	1.16	1.18	1.12
130	1.06	1.04	1.23
160	0.97	0.99	1.10
190	0.90	0.96	1.04
220	0.86	0.93	1.01

Table G-12 Live Load Factors for Special Permit Load Rating for IL-21 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.28	1.27	1.02
50	1.24	1.32	1.12
70	1.20	1.45	1.17
100	1.23	1.41	1.15
130	1.15	1.30	1.43

160	1.09	1.10	1.23
190	1.01	0.98	1.04
220	0.90	0.90	0.93

Table G-13 Live Load Factors for Special Permit Load Rating for IL-22 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.11	0.96	0.88
50	1.05	0.94	0.96
70	1.04	0.96	1.01
100	1.04	1.24	1.03
130	1.08	1.13	1.33
160	1.05	1.06	1.23
190	0.96	1.00	1.11
220	0.89	0.94	1.02

Table G-14 Live Load Factors for Special Permit Load Rating for IL-23 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.18	1.00	0.95
50	1.16	0.97	1.06
70	1.10	0.99	1.03
100	1.06	1.28	1.01
130	1.02	1.13	1.33
160	0.98	1.05	1.18
190	0.89	0.99	1.07
220	0.83	0.94	0.99

Table G-15 Live Load Factors for Special Permit Load Rating for IL-24 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.19	1.37	1.31
50	1.27	1.33	1.40
70	1.25	1.35	1.45
100	1.21	1.30	1.37
130	1.11	1.13	1.20

160	1.00	1.00	1.06
190	0.90	0.93	0.97
220	0.83	0.89	0.92

Table G-16 Live Load Factors for Special Permit Load Rating for IL-26 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.20	1.15	1.13
50	1.29	1.13	1.18
70	1.25	1.15	1.18
100	1.25	1.35	1.17
130	1.23	1.17	1.28
160	1.10	1.07	1.13
190	0.99	0.99	1.03
220	0.89	0.93	0.96

Table G-17 Live Load Factors for Special Permit Load Rating for IL-31 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.12	0.99	0.95
50	1.19	1.00	1.02
70	1.13	0.99	1.00
100	1.05	0.84	0.98
130	0.95	1.06	1.31
160	0.88	0.99	1.15
190	0.81	0.94	1.03
220	0.76	0.88	0.93

Table G-18 Live Load Factors for Special Permit Load Rating for IL-32 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.28	1.31	1.35
50	1.29	1.31	1.39
70	1.26	1.41	1.44
100	1.26	1.28	1.40
130	1.17	1.14	1.33

160	1.12	1.05	1.16
190	1.03	1.02	1.09
220	0.95	0.99	1.04

Table G-19 Live Load Factors for Special Permit Load Rating for IL-34 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.23	1.36	1.10
50	1.30	1.35	1.10
70	1.23	1.39	1.13
100	1.21	1.41	1.47
130	1.19	1.32	1.38
160	1.16	1.11	1.19
190	1.05	1.00	1.05
220	0.94	0.94	0.96

Table G-20 Live Load Factors for Special Permit Load Rating for IL-35 for One-lane Loading

Span Length (ft)	With Respect to Mid-Span Moment	With Respect to Left-Support Shear	With Respect to Right-Support Shear
30	1.44	1.36	1.30
50	1.56	1.34	1.37
70	1.56	1.34	1.45
100	1.44	1.09	1.24
130	1.21	0.97	1.15
160	1.12	0.94	1.07
190	1.05	0.92	1.00
220	1.00	0.89	0.95

APPENDIX H: ADDITIONAL ILLUSTRATIVE APPLICATION EXAMPLES

Table H.0-1 Summary of Rated Bridges

Section	Year Built	Design Load	Span Length (ft)	Type	Rated Members
H.1	1983	HS20	26	Timber Slab Bridge	Interior Strip
H.2	1942	H20	180	Steel Truss Bridge with Timber Deck	Top and Bottom Chords, Diagonal, Vertical, a Floor Beam and Timber Deck
H.3	1929	H20	31	Timber Stringer Bridge with Timber Deck	Interior Stringer and Timber Deck
H.4	1975	HS20	36	Prestressed Concrete I-Girder Bridge with Concrete Deck	Interior Beam and Concrete Deck
H.5	1938	HS20	20	Steel Stringer Bridge with Timber Deck	Interior Stringer and Timber Deck
H.6	1983	HS20	26	Timber Slab Bridge	Interior Strip
H.7	1968	HS20	60	Prestressed Concrete Adjacent Box-Beam Bridge with Concrete Deck	Interior and Exterior Box Beams, Concrete deck
H.8	1920	H10	60	Steel Truss Bridge with Timber Deck	Timber Deck and a Floor Beam
H.9	1939	H20	23.5	Reinforced Concrete Slab Bridge	Interior Strip

H.1 Timber Slab Bridge

Bridge Data:

Span: 26 ft

Year Built: 1983

Material: Douglas Fir-Larch No. 1

Condition: Good

Traffic: Two Lanes

ADTT (one direction): Approximately 100

Skew: 0◦

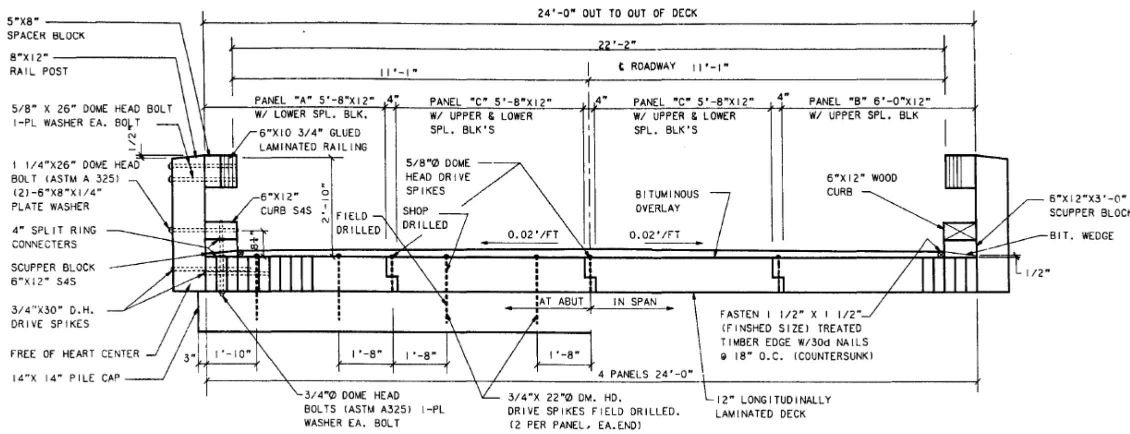

Figure H.1-1 Cross Section of Timber Slab Bridge

Deck Thickness: 12 in

Post Type: 8 in x 12 in x 4 ft

Type of Scupper Block: 6 in x 12 in

Type of Spacer Block: 5 in x 8 in x 10.75 in

Guard Type: 6 in x 10.75 in

Curb Type: 6 in x 12 in

Total Width of Slab: 24 ft

Thickness of Asphalt Overlay: 0.75 in

Width of Clear Roadway: 22 ft 2 in = 22.16 ft

Dead Load Analysis:

Components and Attachments, DC

$$\text{Deck} = 1 \times \frac{12}{12} \times 0.05 = 0.05\ \text{Kip/ft}$$

$$\text{Post} = \frac{8 \times 12}{144} \times 4 \times 0.05 \times (10\ \text{posts})/26\text{ft} \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0021\ \text{Kip/ft}$$

$$\text{Scupper Block} = \frac{6 \times 12}{144} \times 0.05 \times (2\ \text{Scupper Blocks}) \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0021\ \text{Kip/ft}$$

$$\text{Spacer Block} = \frac{5 \times 8 \times 10.75}{1728} \times 0.05 \times \frac{10\ \text{Blocks}}{26\ \text{ft}} \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0002\ \text{Kip/ft}$$

$$\text{Guard} = \frac{6 \times 10.75}{144} \times 0.05 \times (2\ \text{Guards}) \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0019\ \text{Kip/ft}$$

$$\text{Curb} = \frac{6 \times 12}{144} \times 0.05 \times (2\ \text{Curbs}) \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0021\ \text{Kip/ft}$$

Total per Deck Strip:

$$0.05 + 0.0027 + 0.0021 + 0.0012 + 0.0019 + 0.0021 = 0.06\ \text{Kip/ft}$$

$$\text{M}_{\text{DC}} = \frac{1}{8} \times 0.06 \times 26^2 = 5.07\ \text{Kip} - \text{ft}$$

Wearing Surface

$$\text{DW} = 22.16 \times \frac{0.75}{12} \times 0.14 \times \frac{1\ \text{ft}}{24\ \text{ft}} = 0.0081\ \text{Kip/ft}$$

$$\text{M}_{\text{DW}} = \frac{1}{8} \times 0.0081 \times 26^2 = 0.68\ \text{Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed load effects due to the Illinois design, legal and routine permit loads are calculated and showed in Table H.1-1. Dynamic load allowance does not need to be considered for wood components.

Table H.1-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	329.08
Legal	IL-PS2-21	239.00
	IL-PS3-31	330.20
	IL-PS4-34.75	323.00
	IL-PS4-28	234.00
	IL-PS5-36	199.50
	IL-PS6-35.75	274.00
	IL-PS7-39.75	274.00
	IL-PC3-31	173.00
	IL-PC4-41	264.40
	IL-PC5-41	265.30
Routine Permit	IL-RS3-34	318.00
	IL-RS4-38	262.00
	IL-RS5-50	265.00
	IL-RS6-60	310.00

Equivalent Lane Width

Equivalent Lane Width for Single Lane Loaded:

$E_s = 10 + 5\sqrt{L_1 W_1}$

Span = 26 ft < 60 ft: L_1 = 26 ft

Total Width of Slab = 24 ft < 30 ft: W_1 = 24 ft

$E_s = 10 + 5\sqrt{26 \times 24} = 134.90 \text{ in} = 11.24 \text{ ft}$

Equivalent Lane Width for Multiple Lanes Loaded:

$$E_m = 84 + 1.44\sqrt{L_1 W_1} < 12\frac{W}{N_L}$$

Span = 26 ft < 60 ft: L_1 = 26 ft

Total Width of Slab = 24 ft < 60 ft: W_1 = 24 ft

$$E_m = 84 + 1.44\sqrt{26 \times 24} = 119.97\text{in} = 10.00\ \text{ft} < 12 \times \frac{24}{2} = 144\ \text{in} = 12\ \text{ft}$$

Two-lane loading controls.

Distributed Live Load Effects

The distributed live load effects are calculated by converting undistributed load effects into 1-ft format. In other words, Values in Table H.1-2 are the products of the corresponding values in Table H.1-1 and the inverse of the Equivalent Lane Width.

Table H.1-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	32.91
Legal	IL-PS2-21	23.90
	IL-PS3-31	33.02
	IL-PS4-34.75	32.30
	IL-PS4-28	23.40
	IL-PS5-36	19.95
	IL-PS6-35.75	27.40
	IL-PS7-39.75	27.40
	IL-PC3-31	17.30
	IL-PC4-41	26.44
	IL-PC5-41	26.53
Routine Permit	IL-RS3-34	31.80
	IL-RS4-38	26.20
	IL-RS5-50	26.50
	IL-RS6-60	31.00

Nominal resistance:

Section Properties for Stringer

$$S_x = \frac{bd^2}{6} = \frac{12 \times 12^2}{6} = 288.00\ \text{in}^3$$

Design Values

$F_b = F_{bo} C_{KF} C_M \, (C_F \text{ or } C_V) C_{fu} C_i C_d C_\lambda$

Reference Design Value, F_{bo}:

$F_{bo} = 1.20$ Ksi

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\phi}$$

$\phi = 0.85$, for flexure in wood structures

$$C_{KF} = \frac{2.5}{0.85} = 2.94$$

Wet Service Factor, C_M:

$C_M = 1.0$

Size Effect Factor, C_F:

$C_F = 1.0$

Flat Use Factor, C_{fu}:

$C_{fu} = 1.0$

Incising Factor, C_i:

$C_i = 0.8$ for Douglas Fir-Larch

Deck Factor, C_d:

$C_d = 1.15$

Time-Effect Factor, C_λ:

$$C_\lambda = \begin{array}{l} 0.8 \quad \text{Time Effect Factor for Strength I} \\ 1.0 \quad \text{Time Effect Factor for Strength II} \end{array}$$

$F_b = 1.20 \times 2.94 \times 1.0 \times 1.0 \times 1.0 \times 0.8 \times 1.15 \times 0.8 = 2.60$ Ksi for Strength I

$F_b = 1.20 \times 2.94 \times 1.0 \times 1.0 \times 1.0 \times 0.8 \times 1.15 \times 1.0 = 3.25$ Ksi for Strength II

Adjusted Design Value = F_b = 2.60 Ksi for Design and Legal Load Rating

Adjusted Design Value = F_b = 3.25 Ksi for Routine Permit Load Rating

Nominal Resistance = $R_n = F_b S C_L$

C_L = 1.0

$R_n = 2.60 \times 288.00 \times \frac{1}{12} = 62.40 \text{ Kip} - \text{ft}$ for Design Load Rating

$R_n = 2.60 \times 288.00 \times \frac{1}{12} = 62.40 \text{ Kip} - \text{ft}$ for Legal Load Rating

$R_n = 3.25 \times 288.00 \times \frac{1}{12} = 78.00 \text{ Kip} - \text{ft}$ for Routine Permit Load Rating

General Load-Rating Equation:

$$\text{RF} = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.85, for Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Slab Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DC} = 1.50

The used live load factors are shown in Table H.1-3.

Table H.1-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors, RF, are calculated using the general load-rating equation with the corresponding parameter values. Table H.1-4 shows the rating factors with the current and proposed live load factors.

Table H.1-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.79 1.03	0.79 1.03
Legal	IL-PS2-21	1.48	1.62
	IL-PS3-31	1.07	1.17
	IL-PS4-34.75	1.09	1.20
	IL-PS4-28	1.50	1.66
	IL-PS5-36	1.77	1.94
	IL-PS6-35.75	1.28	1.41
	IL-PS7-39.75	1.28	1.41
	IL-PC3-31	2.04	2.24
	IL-PC4-41	1.32	1.46
	IL-PC5-41	1.32	1.46
Routine Permit	IL-RS3-34	N/A	1.69
	IL-RS4-38	N/A	2.05
	IL-RS5-50	N/A	2.02
	IL-RS6-60	N/A	1.73

H.2 Steel Truss Bridge with Plank Deck

Bridge Data:

Span: 180 ft

Year Built: 1942

Material:

Steel: F_y = 33 Ksi (Nominal Yield Strength)

F_u = 66 Ksi (Nominal Ultimate Strength)

Condition: Poor Condition

Distance between Trusses: 21 ft 5 in = 21.42 ft

Width of Clear Roadway: 18 ft 9 in = 18.75 ft

Traffic: One Lane

ADTT (one direction): 29

Skew: 0◦

Thickness of Floor Plank: 4 in

Thickness of Timber Overlay: 3.5 in

Distance between Stringers: 2ft 3 ¾ in = 2.3125 ft

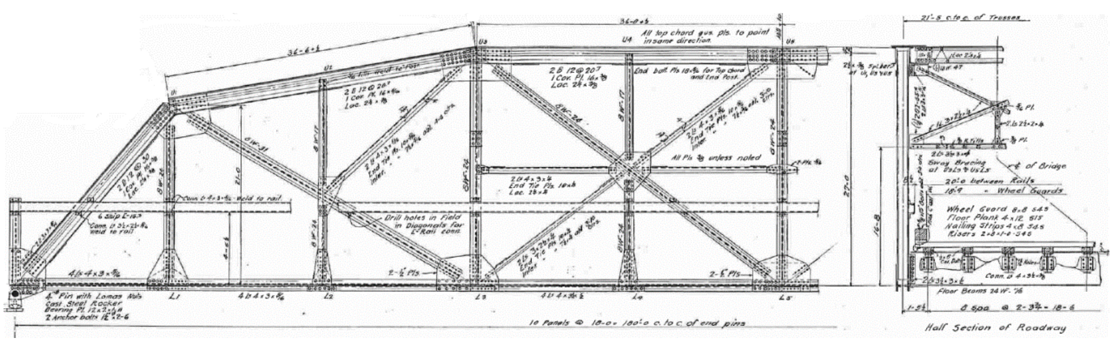

Figure H.2-1 Plan of Steel Truss Bridge

Member Properties

The properties of the rated members are displayed in Table H.2-1.

Table H.2-1 Properties of Rated Members

Member	Section	A, in^2	r, in	S_z, in^3

Top Chord U4U5	Built-up Section 2 Channels-2C12x20.7 1 Top Plate 16x3/8 Bottom Bar 2 1/4x3/8	19.89	5.10	N/A
Bottom Chord L4L5	4 Angles 4L4x3 1/2x1/2	14.00	N/A	N/A
Diagonal U1M2	W8x31	9.13	N/A	N/A
Vertical U1L1	W8x24	7.08	N/A	N/A
Floor Beam	W24x76	N/A	N/A	176
Timber Deck Strip	4”x12”	48	N/A	32

Dead Load Analysis:

The dead load effects (DC=Component, DW=Wearing Surface) are exhibited in Table H.2-2.

Table H.2-2 Dead Load Effects

Member	P_{DC}, Kips	P_{DW}, Kips	M_{DC}, Kip-ft	M_{DW}, Kip-ft
Top Chord U4U5	-76.87	-0.30	N/A	N/A
Bottom Chord L4L5	29.99	0.11	N/A	N/A
Diagonal U1M2	45.79	0.17	N/A	N/A
Vertical U1L1	6.74	0.04	N/A	N/A
Floor Beam	N/A	N/A	46.59	15.05
Timber Deck Strip	N/A	N/A	0.011	0.0097

Live Load Analysis:

Distribution Factor for Truss Members

Use level rule to distribute one-lane live loads to one side of the truss members. The truss is analyzed as a planar structure with a standard live load applied within a lane, as shown in Figure H.2-2.

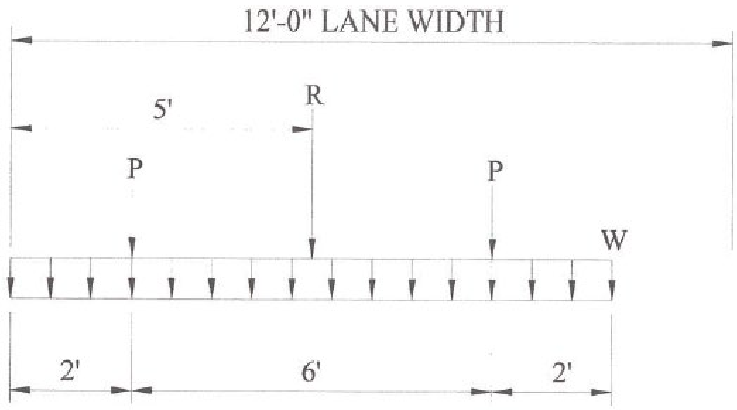

Figure H.2-2 Load Placement within a Lane

In Figure H.2-2, R represents the resultant of lane and wheel loads. W and P stand for lane and wheel loads, respectively.

Width of Clear Roadway = 18 ft 9 in = 18.75 ft

Distance between Trusses = 21 ft 5 in = 21.42 ft

$$\text{Edge Distances} = (21.42 - 18.75) \times \frac{1}{2} = 1.34 \text{ ft}$$

One Lane Loaded:

Multiple Presence Factor = 1.2

Distribution Factor, g_1:

$$g_1 = \left(\frac{21.42 - 1.34 - 5}{21.42}\right) \times 1.2 = 0.84$$

Live Load Effects for Truss Members

The live load effects for truss members are displayed in Tables H.2-3 to H.2-6. The distribution factor of 0.84 and dynamic load allowance of 33% are included in the calculation.

Table H.2-3 Live Load Effects (LL) for Top Chord U4U5

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	198.75
Legal	IL-PS2-21	76.20
	IL-PS3-31	111.04
	IL-PS4-34.75	122.32
	IL-PS4-28	97.24
	IL-PS5-36	117.95
	IL-PS6-35.75	121.87
	IL-PS7-39.75	134.37
	IL-PC3-31	100.03
	IL-PC4-41	136.86
	IL-PC5-41	137.89
Routine Permit	IL-RS3-34	119.55
	IL-RS4-38	128.73
	IL-RS5-50	154.75
	IL-RS6-60	184.82

Table H.2-4 Live Load Effects (LL) for Bottom Chord L4L5

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	193.09
Legal	IL-PS2-21	72.54
	IL-PS3-31	104.87
	IL-PS4-34.75	116.18
	IL-PS4-28	92.77
	IL-PS5-36	114.08
	IL-PS6-35.75	116.96
	IL-PS7-39.75	129.03
	IL-PC3-31	97.12
	IL-PC4-41	132.35

	IL-PC5-41	132.99
Routine Permit	IL-RS3-34	113.61
	IL-RS4-38	123.78
	IL-RS5-50	149.99
	IL-RS6-60	179.83

Table H.2-5 Live Load Effects (LL) for Diagonal U1M2

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	124.44
Legal	IL-PS2-21	49.11
	IL-PS3-31	71.56
	IL-PS4-34.75	78.91
	IL-PS4-28	63.33
	IL-PS5-36	77.64
	IL-PS6-35.75	78.48
	IL-PS7-39.75	85.59
	IL-PC3-31	66.25
	IL-PC4-41	88.32
	IL-PC5-41	87.91
Routine Permit	IL-RS3-34	77.92
	IL-RS4-38	84.42
	IL-RS5-50	102.52
	IL-RS6-60	122.98

Table H.2-6 Live Load Effects (LL) for Vertical U1L1

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	44.29

Legal	IL-PS2-21	40.09
	IL-PS3-31	52.16
	IL-PS4-34.75	51.02
	IL-PS4-28	37.98
	IL-PS5-36	31.90
	IL-PS6-35.75	40.47
	IL-PS7-39.75	40.47
	IL-PC3-31	31.90
	IL-PC4-41	43.94
	IL-PC5-41	44.05
Routine Permit	IL-RS3-34	52.88
	IL-RS4-38	43.45
	IL-RS5-50	44.32
	IL-RS6-60	48.41

Live Load Effects for Floor Beams:

Spacing of Floor Beams: 18 ft

Figure H.2-3 shows the critical live load position for reactions on an intermediate floor beam.

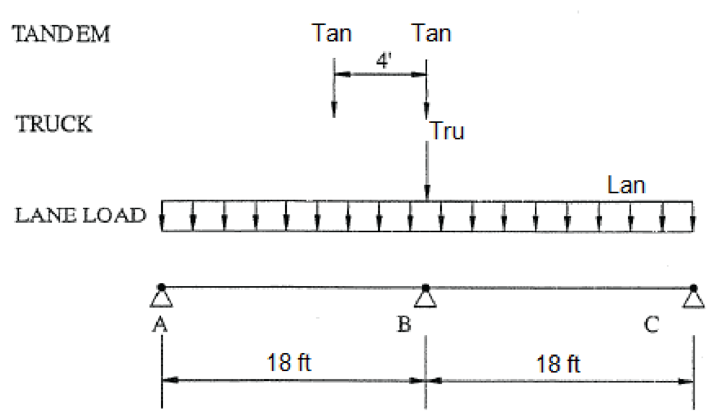

Figure H.2-3 Critical Live Load Position for Reactions on Floor Beam

The floor beams are modeled as hinges to support deck.

Reaction at Floor-beam B:

IM = 33%

Truck + Lane

$$\mathrm{R_{Tru+Lan}} = \mathrm{Tru} \times 1.33 + \mathrm{Lan} \times 18$$

Tandem + Lane

$$\mathrm{R_{Tan+Lan}} = \left(\mathrm{Tan} + \mathrm{Tan} \times \frac{18-4}{18}\right) \times 1.33 + \mathrm{Lan} \times 18$$

$$\mathrm{R_{Wheel}} = \frac{\mathrm{R_{Tan\ or\ Tru}}}{2} = \mathrm{P}$$

$$\mathrm{R_{Lane\ per\ foot\ width}} = \frac{\mathrm{R_{Lan}}}{10} = \mathrm{w}$$

For Illinois Design Load (HL-93):

Tan = 25 Kips

Tru = 32 Kips

Lan = 0.64 Kip/ft

$$\mathrm{R_{Tru+Lan}} = 32 \times 1.33 + 0.64 \times 18 = 54.08\ \mathrm{Kips}$$

$$\mathrm{R_{Tan+Lan}} = \left(25 + 25 \times \frac{18-4}{18}\right) \times 1.33 + 0.64 \times 18 = 70.63\ \mathrm{Kips}$$

Tandem Load Governs.

For Illinois Legal and Routine Permit Loads, the maximum axle loads of the model trucks "Tru", along with 33% dynamic load allowance are used for the calculation.

The reactions on the floor beams "R" are converted into wheel loads "P" and lane load per foot width "w". Tables H.2-7 and H.2-8.

Table H.2-7 Wheel Loads P

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	29.56
Legal	IL-PS2-21	16.63
	IL-PS3-31	17.02
	IL-PS4-34.75	14.63
	IL-PS4-28	11.97
	IL-PS5-36	11.31
	IL-PS6-35.75	11.97
	IL-PS7-39.75	11.97
	IL-PC3-31	14.63
	IL-PC4-41	16.76
	IL-PC5-41	14.90
Routine Permit	IL-RS3-34	17.96
	IL-RS4-38	14.63
	IL-RS5-50	15.30
	IL-RS6-60	15.96

Table H.2-8 Lane Loads w

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	1.15
Legal	IL-PS2-21	N/A
	IL-PS3-31	N/A
	IL-PS4-34.75	N/A
	IL-PS4-28	N/A
	IL-PS5-36	N/A
	IL-PS6-35.75	N/A
	IL-PS7-39.75	N/A
	IL-PC3-31	N/A
	IL-PC4-41	N/A

	IL-PC5-41	N/A
Routine Permit	IL-RS3-34	N/A
	IL-RS4-38	N/A
	IL-RS5-50	N/A
	IL-RS6-60	N/A

Maximum load effects due to wheel and lane loads:

Figure H.2-4 illustrates the critical positions of one design lane to produce the maximum mid-span moment in the floor beam. The truss members are treated as pinned supports. Table H.2-9 displayed the maximum load effects due to the Illinois design, legal and permit loads. The Multiple presence factor 1.2 is included in the calculation.

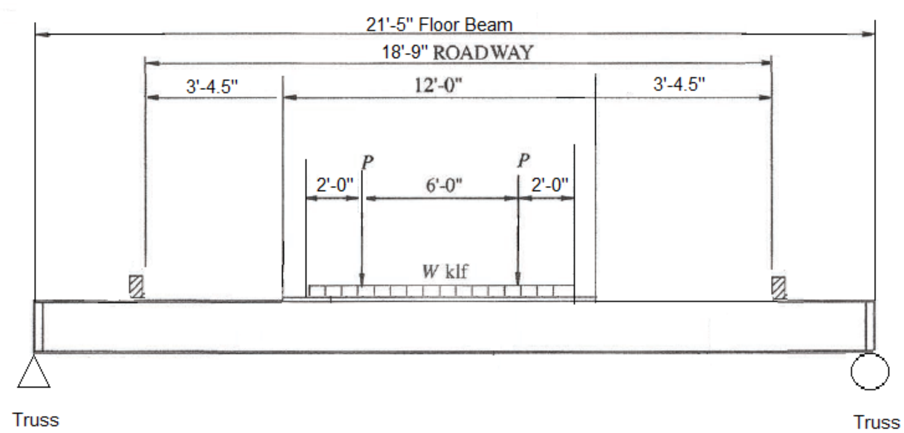

Figure H.2-4 Critical Lane Position for Maximum Moment in the Floor Beam

Table H.2-9 Live Load Effects (LL) for a Floor Beam

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	330.10
Legal	IL-PS2-21	153.81
	IL-PS3-31	157.51
	IL-PS4-34.75	135.36
	IL-PS4-28	110.75

	IL-PS5-36	104.59
	IL-PS6-35.75	110.75
	IL-PS7-39.75	110.75
	IL-PC3-31	135.36
	IL-PC4-41	155.05
	IL-PC5-41	137.82
Routine Permit	IL-RS3-34	166.12
	IL-RS4-38	135.36
	IL-RS5-50	141.51
	IL-RS6-60	147.66

Live Load Effects for Timber Deck Strips

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Table H.2-10, are calculated with axle loads of Illinois design, legal and permit trucks. Dynamic load allowance is not considered for timber components.

Table H.2-10 Undistributed Live Load Effects for Deck Strips

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	9.25
Legal	IL-PS2-21	7.23
	IL-PS3-31	7.40
	IL-PS4-34.75	6.36
	IL-PS4-28	5.20
	IL-PS5-36	4.91
	IL-PS6-35.75	5.20
	IL-PS7-39.75	5.20
	IL-PC3-31	6.36
	IL-PC4-41	7.28

	IL-PC5-41	6.48
Routine Permit	IL-RS3-34	7.80
	IL-RS4-38	6.36
	IL-RS5-50	6.65
	IL-RS6-60	6.94

Equivalent Lane Width

Equivalent Strip Width:

$E_s = 4.0h + 40$

h= Thickness of Deck = 4.0 in

$E_s = 4.0 \times 4.0 + 40 = 56$ in

Distributed Live Load Effects

The calculated axle loads are converted over transverse equivalent strip width. In this conversion, multiple presence factor for one-lane loading 1.2 is included.

Table H.2-11 Distributed Live Load Effects (LL) for Deck Strips

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	2.38
Legal	IL-PS2-21	1.86
	IL-PS3-31	1.90
	IL-PS4-34.75	1.64
	IL-PS4-28	1.34
	IL-PS5-36	1.26
	IL-PS6-35.75	1.34
	IL-PS7-39.75	1.34
	IL-PC3-31	1.64
	IL-PC4-41	1.87
	IL-PC5-41	1.67
	IL-RS3-34	2.01

Routine Permit	IL-RS4-38	1.64
	IL-RS5-50	1.71
	IL-RS6-60	1.78

Nominal Resistance of Rated Members:

Top Chord U4U5 (Compression Member)

Area, A = 19.89 in^2

r = 5.10 in

Length, L = 18.25 ft

Limiting Slenderness Ratio:

$$\frac{\mathrm{KL}}{\mathrm{r}} = \frac{0.875 \times 18.25 \times 12}{5.10} = 37.57 < 120 \text{ for Main Members, OK}$$

K=0.875 for pinned ends

Column slenderness term, λ:

$$\lambda = \left(\frac{\mathrm{KL}}{\mathrm{r}\pi}\right)^2 \frac{\mathrm{F_y}}{\mathrm{E}} = \left(\frac{37.57}{\pi}\right)^2 \frac{33}{29000} = 0.16 < 2.25 \text{ Intermediate Length Column}$$

Limiting Width/Thickness Ratios:

$$\frac{\mathrm{b}}{\mathrm{t}} \le \mathrm{k}\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}}$$

k = Plate Bucking Coefficient

Top Plate, k=1.40

$$\frac{\mathrm{b}}{\mathrm{t}} \le 1.40\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}}$$

b = 10 in, (Distance between Back-to-Back Channels)

$$\frac{b}{t} = \frac{10}{3/8} = 26.67 \le 1.40\sqrt{\frac{E}{F_y}} = 1.40 \times \sqrt{\frac{29000}{33}} = 41.50 \quad \text{OK}$$

Web Plates, k=1.49

$$\frac{h}{t_w} \le 1.49\sqrt{\frac{E}{F_y}}$$

$$\frac{h}{t_w} = \frac{11}{5/16} = 35.2 < 1.49 \times \sqrt{\frac{29000}{33}} = 44.17 \quad \text{OK}$$

Bottom Plate, k=0.45

$$\frac{b}{t} \le 0.45\sqrt{\frac{E}{F_y}}$$

$$\frac{b}{t} = \frac{2.25}{\frac{3}{8} + \frac{1}{2}} = 2.57 < 0.45 \times \sqrt{\frac{29000}{33}} = 13.34 \quad \text{OK}$$

The built-up section meets limiting width/thickness ratios. Local buckling will not occur prior to yielding. Therefore, the nominal resistance is calculated as follows:

R_n = $-0.66^{\lambda} F_y A = -0.66^{0.15} \times 33 \times 19.89 = -616.71 \text{ Kips}$

$\varphi = 0.9$ for Compression Member

$\varphi R_n = 0.9 \times (-616.71) = -555.04$ kips

Bottom Chord L4L5 (Tension Member)

Area, A = 14.00 in^2

R_n = $F_y A = 33 \times 14.00 = 462.00$ Kips

$\varphi = 0.95$ for Tension Member

$\varphi R_n = 0.95 \times 462.00 = 438.90$ kips

Diagonal Member U1M2

Area, A = 9.13 in^2

$R_n = F_y A = 33 \times 9.13 = 301.29$ Kips

$\varphi = 0.95$ for Tension Member

$\varphi R_n = 0.95 \times 301.29 = 286.23$ kips

Vertical Member U1L1

Area, A = 7.08 in^2

$R_n = F_y A = 33 \times 7.08 = 233.64$ Kips

$\varphi = 0.95$ for Tension Member

$\varphi R_n = 0.95 \times 233.64 = 221.96$ kips

Floor Beam

For W24x76:

D = 23.9 in

t_w = 0.44 in

b_f = 8.99 in

t_f = 0.68 in

I_z = 2100 in^4

S_z = 176 in^3

$D_w = D - 2t_f = 23.9 - 2\times0.68 = 22.54$ in

$$D_c = D_t = \frac{D_w}{2} = \frac{22.54}{2} = 11.27 \text{ in}$$

Web Slenderness Limit

$$\frac{2D_c}{t_w} = \frac{2 \times 11.27}{0.44} = 51.23 < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7 \times \sqrt{\frac{29000}{33}} = 168.97 \quad \text{OK}$$

$$\frac{I_{yc}}{I_{yt}} = 1.0 > 0.3 \quad \text{OK}$$

Location of Plastic Neutral Axis (PNA)

$$\overline{\mathrm{Y}} = \frac{\mathrm{D_w}}{2} = \frac{22.54}{2} = 11.27 \text{ in},$$

from bottom of the top flange to PNA.

Top and Bottom Flanges

$$\mathrm{P_c} = \mathrm{P_t} = \mathrm{F_y b_f t_f} = 33 \times 8.99 \times 0.68 = 201.74 \text{ Kips}$$

$$\mathrm{d_t} = \mathrm{d_c} = \frac{(\mathrm{t_f} + \mathrm{D_w})}{2} = \frac{(0.68 + 22.54)}{2} = 11.61 \text{ in}$$

Web

$$\mathrm{P_w} = \mathrm{F_y D_w t_w} = 33 \times 22.54 \times 0.44 = 327.28 \text{ Kips}$$

Plastic Moment

$$\mathrm{M_p} = \frac{\mathrm{P_w}}{2\mathrm{D_w}}[\overline{\mathrm{Y}}^2 + (\mathrm{D_w} - \overline{\mathrm{Y}})^2] + \mathrm{P_c d_c} + \mathrm{P_t d_t}$$

$$\mathrm{M_p} = \{\frac{327.28}{2 \times 22.54}[11.27^2 + (22.54 - 11.27)^2] + 2 \times 201.74 \times 11.61\} \times \frac{1}{12}$$

$$\mathrm{M_p} = 544.05 \text{ Kip} - \text{ft}$$

Web Compactness

$$\frac{2\mathrm{D_{cp}}}{\mathrm{t_w}} \leq \lambda_{\mathrm{pw(D_{cp})}}$$

$$\frac{2\mathrm{D_{cp}}}{\mathrm{t_w}} = \frac{2 \times 11.27}{0.44} = 51.23$$

$$\lambda_{\mathrm{pw(D_{cp})}} = \frac{\sqrt{\frac{\mathrm{E}}{\mathrm{F_{yc}}}}}{(0.54\frac{\mathrm{M_p}}{\mathrm{R_h M_y}} - 0.09)^2} \leq \lambda_{\mathrm{rw}}\frac{\mathrm{D_{cp}}}{\mathrm{D_c}}$$

where:

$$\lambda_{\mathrm{rw}}\frac{\mathrm{D_{cp}}}{\mathrm{D_c}} = 5.7\sqrt{\frac{\mathrm{E}}{\mathrm{F_{yc}}}}\left(\frac{\mathrm{D_{cp}}}{\mathrm{D_c}}\right) = 5.7 \times \sqrt{\frac{29000}{33}} \times (1) = 168.97$$

$R_h = 1$

$$M_y = F_y S_z = 33 \times 176 \times \frac{1}{12} = 484.00 \text{ Kip} - \text{ft}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{33}}}{(0.54\frac{544.05}{1 \times 484.00} - 0.09)^2} = 110.91$$

Use 110.91

$$\frac{2D_{cp}}{t_w} = 51.23 \le \lambda_{pw(D_{cp})} = 110.91 \quad \text{OK}$$

The section is compact.

Web Plastification Factor, R_{pc}:

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{544.05}{484.00} = 1.12$$

Nominal Resistance for Floor Beams:

Sections are considered as continuously being braced at compression flanges.

$$R_n = \varphi_f R_{pc} M_{yc} = \varphi_f \frac{M_p}{M_{yc}} M_{yc} = \varphi_f M_p = 1 \times 544.05 = 544.05 \text{ Kip} - \text{ft}$$

$\varphi = 1.0$, for flexure in steel beams.

$\varphi R_n = 1 \times 544.05 = 544.05 \text{ Kip} - \text{ft}$

Deck Strips

Section Properties for Stringer

$S = 32 \text{ in}^3$

Design Values

$F_b = F_{bo} C_{KF} C_M \ (C_F \text{ or } C_V) C_{fu} C_i C_d C_\lambda$

Reference Design Value, F_{bo}:

$F_{bo} = 0.9 \text{ Ksi}$

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\emptyset} = \frac{2.5}{0.85} = 2.94$$

Size Effect Factor for Sawn Lumber, C_F:

Structural Grade: NO.2; b=12 in; d=4 in

C_F = 1.1

Wet Service Factor, C_M:

$$F_{bo}C_F = 0.9 \times 1.1 = 0.99 \text{ Ksi} \leq 1.15 \text{ Ksi}$$

$C_M = 1.0$

Flat Use Factor, C_{fu}:

C_{fu} = 1.0

Incising Factor (only apply to dimension lumber), C_i:

C_i = 0.8

Deck Factor, C_d:

C_d = 1.50

$C_\lambda = \begin{matrix} 0.8 & \text{Time Effect Factor for Strength I} \\ 1.0 & \text{Time Effect Factor for Strength II} \end{matrix}$

$F_b = 0.9 \times 2.94 \times 1.0 \times 1.1 \times 1.0 \times 0.8 \times 1.50 \times 0.8 = 2.79$ Ksi for Strength I

$F_b = 0.9 \times 2.94 \times 1.0 \times 1.1 \times 1.0 \times 0.8 \times 1.50 \times 1.0 = 3.49$ Ksi for Strength II

Adjusted Design Value = F_b = 2.79 Ksi for Design and Legal Load Rating

Adjusted Design Value = F_b = 3.49 Ksi for Permit Load Rating

Nominal Resistance = $R_n = F_b S C_L$

C_L = 1.0

For Design Load Rating :

$$R_n = 2.79\text{ Ksi} \times 32\text{ in}^3 \times \frac{1}{12} = 7.44\text{ Kip} - \text{ft}$$

For Legal Load Rating :

$$R_n = 2.79\text{ Ksi} \times 32\text{ in}^3 \times \frac{1}{12} = 7.44\text{ Kip} - \text{ft}$$

For Permit Load Rating :

$$R_n = 3.49\text{ Ksi} \times 32\text{ in}^3 \times \frac{1}{12} = 9.31\text{ Kip} - \text{ft}$$

φ = 0.85, for Wood Component in Flexure

For Design and Legal Load Rating :

$$\varphi R_n = 0.85 \times 7.44 = 6.32\text{ Kip} - \text{ft}$$

For Permit Load Rating :

$$\varphi R_n = 0.85 \times 9.31 = 7.91\text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Condition Factor, φ_C = 0.85, for Poor Condition

System Factor, φ_S = 0.9, for Two-Truss Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The live load factors in Table H.2-12 are used.

Table H.2-12 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level

Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors are calculated and displayed in Tables H.2-13 to H.2-18.

Table H.2-13 Rating Factors (RF) for Top Chord U4U5

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.94 1.22	0.94 1.22
Legal	IL-PS2-21	3.32	3.64
	IL-PS3-31	2.27	2.50
	IL-PS4-34.75	2.07	2.27
	IL-PS4-28	2.60	2.86
	IL-PS5-36	2.14	2.36
	IL-PS6-35.75	2.07	2.28
	IL-PS7-39.75	1.87	2.07
	IL-PC3-31	2.52	2.78
	IL-PC4-41	1.84	2.03
	IL-PC5-41	1.83	2.02
Routine Permit	IL-RS3-34	2.29	2.49
	IL-RS4-38	2.12	2.32
	IL-RS5-50	1.77	1.93
	IL-RS6-60	1.48	1.61

Table H.2-14 Rating Factors (RF) for Bottom Chord L4L5

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.88 1.14	0.88 1.14
Legal	IL-PS2-21	3.17	3.48
	IL-PS3-31	2.19	2.41
	IL-PS4-34.75	1.97	2.17
	IL-PS4-28	2.48	2.72
	IL-PS5-36	2.01	2.21
	IL-PS6-35.75	1.96	2.16
	IL-PS7-39.75	1.78	1.96
	IL-PC3-31	2.36	2.60
	IL-PC4-41	1.73	1.91
	IL-PC5-41	1.72	1.90
Routine Permit	IL-RS3-34	2.19	2.39
	IL-RS4-38	2.00	2.19
	IL-RS5-50	1.66	1.81
	IL-RS6-60	1.39	1.51

Table H.2-15 Rating Factors (RF) for Diagonal U1M2

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.74 0.96	0.74 0.96
Legal	IL-PS2-21	2.53	2.79
	IL-PS3-31	1.73	1.91
	IL-PS4-34.75	1.57	1.73
	IL-PS4-28	1.96	2.16
	IL-PS5-36	1.60	1.76
	IL-PS6-35.75	1.58	1.74

	IL-PS7-39.75	1.45	1.60
	IL-PC3-31	1.87	2.07
	IL-PC4-41	1.41	1.55
Routine Permit	IL-PC5-41	1.41	1.56
	IL-RS3-34	1.72	1.88
	IL-RS4-38	1.59	1.74
	IL-RS5-50	1.31	1.43
	IL-RS6-60	1.09	1.19

Table H.2-16 Rating Factors (RF) for Vertical U1L1

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	2.08 2.70	2.08 2.70
Legal	IL-PS2-21	3.09	3.41
	IL-PS3-31	2.38	2.62
	IL-PS4-34.75	2.43	2.68
	IL-PS4-28	3.26	3.60
	IL-PS5-36	3.89	4.29
	IL-PS6-35.75	3.07	3.38
	IL-PS7-39.75	3.07	3.38
	IL-PC3-31	3.89	4.29
	IL-PC4-41	2.82	3.11
	IL-PC5-41	2.82	3.10
Routine Permit	IL-RS3-34	2.55	2.77
	IL-RS4-38	3.10	3.38
	IL-RS5-50	3.03	3.31
	IL-RS6-60	2.77	3.03

Table H.2-17 Rating Factors (RF) for Floor Beams

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.58 0.75	0.58 0.75
Legal	IL-PS2-21	1.68	1.85
	IL-PS3-31	1.64	1.80
	IL-PS4-34.75	1.91	2.10
	IL-PS4-28	2.33	2.56
	IL-PS5-36	2.47	2.72
	IL-PS6-35.75	2.33	2.56
	IL-PS7-39.75	2.33	2.56
	IL-PC3-31	1.91	2.10
	IL-PC4-41	1.67	1.83
	IL-PC5-41	1.87	2.07
Routine Permit	IL-RS3-34	1.68	1.84
	IL-RS4-38	2.07	2.25
	IL-RS5-50	1.97	2.15
	IL-RS6-60	1.90	2.06

Table H.2-18 Rating Factors (RF) for Deck Strips

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	1.16 1.50	1.16 1.50
Legal	IL-PS2-21	1.99	2.19
	IL-PS3-31	1.95	2.14
	IL-PS4-34.75	2.26	2.49
	IL-PS4-28	2.77	3.05
	IL-PS5-36	2.93	3.23
	IL-PS6-35.75	2.77	3.05
	IL-PS7-39.75	2.77	3.05

	IL-PC3-31	2.26	2.49
	IL-PC4-41	1.97	2.17
	IL-PC5-41	2.22	2.45
Routine Permit	IL-RS3-34	2.50	2.73
	IL-RS4-38	3.07	3.35
	IL-RS5-50	2.94	3.20
	IL-RS6-60	2.82	3.07

H.3 Timber Stringer Bridge with Timber Deck

Bridge Data:

Span: 31 ft

Year Built: 1929

Material: Southern Pine

Cross Section: See Figure H.3-1

Condition: Good

Traffic: Two Lanes

ADTT (one direction): Approximately 100

Skew: 0

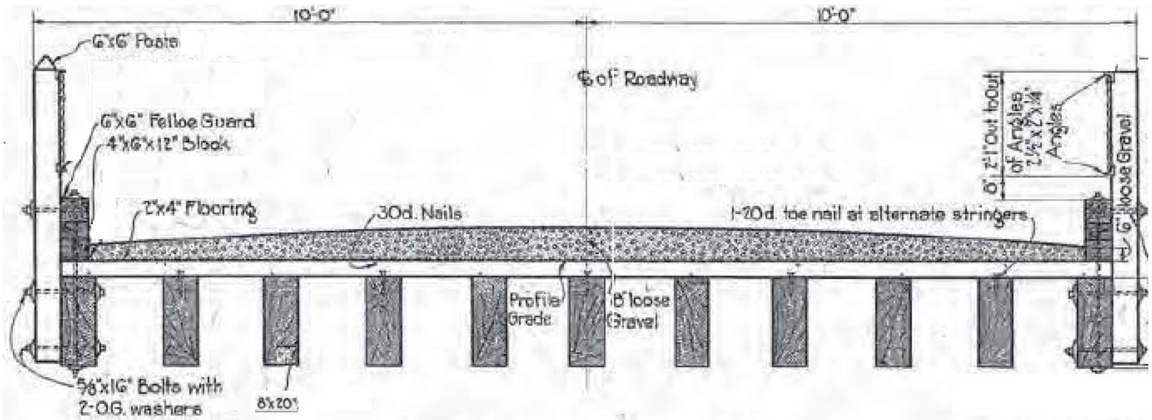

Figure H.3-1 Cross Section of Timber Stringer Bridge

Deck Thickness: 4 in

Beam Spacing: 1 ft 10 in = 1.83 ft

Beam Type: 8 in x 20 in

Post Type: 6 in x 6 in x 5 ft-9 in

Guard Type: 6 in x 6 in

Block Type: 4 in x 6 in x 12 in

Thickness of Asphalt Overlay: 2 in

Width of Clear Roadway: 18 ft

Load Rating for Interior Stringer :

Dead Load Analysis:

Components and Attachments, DC

$$\text{Deck} = 1.83 \times \frac{4}{12} \times 0.05 = 0.031\ \text{Kip/ft}$$

$$\text{Stringer} = \frac{8 \times 20}{144} \times 0.05 = 0.056\ \text{Kip/ft}$$

$$\text{Post} = \frac{6 \times 6}{144} \times 5.75 \times 0.05 \times \frac{10\ \text{Posts}}{31\ \text{ft}} \times \frac{1}{11\ \text{Beams}} = 0.0021\ \text{Kip/ft}$$

$$\text{Guard} = \frac{6 \times 6}{144} \times 0.05 \times \frac{2 \text{ Guards}}{11 \text{ Beams}} = 0.0023 \text{ Kip/ft}$$

$$\text{Block} = \frac{6 \times 4}{144} \times 0.05 \times \frac{10 \text{ Blocks}}{31 \text{ ft}} \times \frac{1}{11 \text{ Beams}} = 0.0002 \text{ Kip/ft}$$

Total per stringer = $0.031 + 0.056 + 0.0021 + 0.0023 + 0.0002 = 0.092 \text{ Kip/ft}$

$$\text{M}_{\text{DC}} = \frac{1}{8} \times 0.092 \times 31^2 = 13.05 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\text{DW} = (18 \times \frac{2}{12} \times 0.14) \times \frac{1}{11 \text{ Beams}} = 0.038 \text{ Kip/ft}$$

$$\text{M}_{\text{DW}} = \frac{1}{8} \times 0.038 \times 31^2 = 4.56 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

AASHTO LRFD Type *l* cross section

One Lane Loaded:

$$\text{g}_1 = \frac{\text{S}}{6.7} = \frac{1.83}{6.7} = 0.27$$

Two or More Lanes Loaded:

$$\text{g}_\text{m} = \frac{\text{S}}{7.5} = \frac{1.83}{7.5} = 0.24$$

One-lane loading governs.

Undistributed Live Load Effects

The undistributed live load effects due to the Illinois design, legal and permit loads are displayed in Table H.3-1. For wood components, according to AASHTO Manual for Bridge Evaluation, dynamic load allowance (IM) needs not to be applied.

Table H.3-1 Undistributed Live Load Effects Table 7.1.1. Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	401.88
Legal	IL-PS2-21	281.00
	IL-PS3-31	392.20
	IL-PS4-34.75	388.75
	IL-PS4-28	284.00
	IL-PS5-36	247.50
	IL-PS6-35.75	334.00
	IL-PS7-39.75	334.00
	IL-PC3-31	215.00
	IL-PC4-41	321.20
	IL-PC5-41	335.00
Routine Permit	IL-RS3-34	386.00
	IL-RS4-38	326.00
	IL-RS5-50	319.00
	IL-RS6-60	370.00

Distributed Live Load Effects

The live load effects for one interior stringer are calculated by multiplying values in Table H.3-1 by one-lane distribution factor, g_1. The distributed load effects are displayed in Table H.3-2.

Table H.3-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	108.51
Legal	IL-PS2-21	75.87
	IL-PS3-31	105.89
	IL-PS4-34.75	104.96
	IL-PS4-28	76.68

	IL-PS5-36	66.83
	IL-PS6-35.75	90.18
	IL-PS7-39.75	90.18
	IL-PC3-31	58.05
	IL-PC4-41	86.72
	IL-PC5-41	90.45
Routine Permit	IL-RS3-34	104.22
	IL-RS4-38	88.02
	IL-RS5-50	86.13
	IL-RS6-60	99.90

Section Properties

$$S_x = \frac{bh^2}{6} = \frac{8 \times 20^2}{6} = 533.33 \text{ in}^3$$

Design Values

$F_b = F_{bo}C_{KF}C_M\,(C_F \text{ or } C_V)C_{fu}C_iC_dC_\lambda$

Reference Design Value, F_{bo}:

F_{bo} = 1.35 Ksi

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\emptyset}$$

$\emptyset = 0.85$, for flexure in wood structures

$$C_{KF} = \frac{2.5}{0.85} = 2.94$$

Wet Service Factor, C_M:

C_M = 1.0, for southern pine in the specific size

Size Effect Factor for Sawn Lumber, C_F:

$C_F = (\frac{12}{d})^{\frac{1}{9}} = (\frac{12}{20})^{\frac{1}{9}} = 0.94$

Flat Use Factor, C_{fu}:

C_{fu} = 1.0

Incising Factor, C_i:

C_i = 1.0

Deck Factor, C_d:

C_d = 1.0

Time-Effect Factor, C_λ:

$C_\lambda = \begin{matrix} 0.8 & \text{Time Effect Factor for Strength I} \\ 1.0 & \text{Time Effect Factor for Strength II} \end{matrix}$

F_b = $1.35 \times 2.94 \times 1.0 \times 0.94 \times 1.0 \times 1.0 \times 1.0 \times 0.8 = 2.98$ Ksi for Strength I

F_b = $1.35 \times 2.94 \times 1.0 \times 0.94 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 3.73$ Ksi for Strength II

Adjusted Design Value = F_b =2.98 Ksi for Design and Legal Load Rating

Adjusted Design Value = F_b =3.73 Ksi for Permit Load Rating

Nominal Resistance

$R_n = F_b S C_L$

C_L = 1.0

$R_n = (2.98 \times 533.33 \times 1.0) \times \frac{1}{12} = 132.44$ Kip − ft for Design Load Rating

$R_n = (2.98 \times 533.33 \times 1.0) \times \frac{1}{12} = 132.44$ Kip − ft for Legal Load Rating

$R_n = (3.73 \times 533.33 \times 1.0) \times \frac{1}{12} = 165.78$ Kip − ft for Permit Load Rating

General Load-Rating Equation :

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.85, for Wood Component in Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.3-3.

Table H.3-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors, RFs, are calculated using the general load-rating equation with the corresponding parameter values. Table H.3-4 shows the rating factors with the current and proposed live load factors. As shown in Table H.3-4, the RF for legal load rating is less than 1.0. Therefore, the permit load rating is not applicable.

Table H.3-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.48 0.62	0.48 0.62
Legal	IL-PS2-21	0.94	1.03

	IL-PS3-31	0.67	0.73
	IL-PS4-34.75	0.67	0.74
	IL-PS4-28	0.93	1.02
	IL-PS5-36	1.07	1.17
	IL-PS6-35.75	0.79	0.87
	IL-PS7-39.75	0.79	0.87
	IL-PC3-31	1.22	1.35
	IL-PC4-41	0.82	0.91
	IL-PC5-41	0.79	0.87
Routine Permit	IL-RS3-34	N/A	N/A
	IL-RS4-38	N/A	N/A
	IL-RS5-50	N/A	N/A
	IL-RS6-60	N/A	N/A

Load Rating for Timber Deck:

Dead Load Analysis:

Components and Attachments, DC

$$\text{Deck} = 1 \times \frac{4}{12} \times 0.05 = 0.017 \text{ Kip/ft}$$

$$\text{M}_{\text{DC}} = \frac{1}{8} \times 0.017 \times 1.83^2 = 0.0071 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\text{DW} = 1 \times \frac{2}{12} \times 0.14 = 0.023 \text{ Kip/ft}$$

$$\text{M}_{\text{DW}} = \frac{1}{8} \times 0.023 \times 1.83^2 = 0.0096 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Table H.3-5, are calculated with axle loads of Illinois design, legal and routine permit trucks. Dynamic load allowance is not considered for timber components.

Table H.3-5 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	3.15
Legal	IL-PS2-21	4.00
	IL-PS3-31	4.25
	IL-PS4-34.75	3.50
	IL-PS4-28	2.75
	IL-PS5-36	3.00
	IL-PS6-35.75	2.50
	IL-PS7-39.75	2.90
	IL-PC3-31	2.90
	IL-PC4-41	5.00
	IL-PC5-41	3.15
Routine Permit	IL-RS3-34	3.00
	IL-RS4-38	3.50
	IL-RS5-50	3.00
	IL-RS6-60	2.90

Equivalent Lane Width

Equivalent Strip Width:

$E_s = 4.0h + 40$

h= Thickness of Deck = 4.0 in

$E_s = 4.0 \times 4.0 + 40 = 56$ in

Distributed Live Load Effects

The calculated axle loads are converted over transverse equivalent strip width. In this conversion, multiple presence factor for one-lane loading 1.2 is included.

Table H.3-6 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	0.81
Legal	IL-PS2-21	1.03
	IL-PS3-31	1.09
	IL-PS4-34.75	0.90
	IL-PS4-28	0.71
	IL-PS5-36	0.77
	IL-PS6-35.75	0.64
	IL-PS7-39.75	0.75
	IL-PC3-31	0.75
	IL-PC4-41	1.29
	IL-PC5-41	0.81
Routine Permit	IL-RS3-34	0.77
	IL-RS4-38	0.90
	IL-RS5-50	0.77
	IL-RS6-60	0.75

Nominal Resistance:

Section Properties for Stringer

$$S_x = \frac{bd^2}{6} = \frac{12 \times 4.0^2}{6} = 32 \text{ in}^3$$

Design Values

$F_b = F_{bo} C_{KF} C_M \, (C_F \text{ or } C_V) C_{fu} C_i C_d C_\lambda$

Reference Design Value, F_{bo} :

$F_{bo} = 1.45$ Ksi

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\emptyset} = \frac{2.5}{0.85} = 2.94$$

Size Effect Factor for Sawn Lumber, C_F:

$C_F = 1.0$

Wet Service Factor, C_M:

$F_{bo}C_F = 1.45 \times 1.0 = 1.45 \text{ Ksi} > 1.15 \text{ Ksi}$

$C_M = 0.85$

Flat Use Factor, C_{fu}:

$C_{fu} = 1.0$

Incising Factor (only apply to dimension lumber), C_i:

$C_i = 1.0$

Deck Factor, C_d:

Decks are nail-laminated.

$C_d = 1.15$

$$C_\lambda = \begin{matrix} 0.8 & \text{Time Effect Factor for Strength I} \\ 1.0 & \text{Time Effect Factor for Strength II} \end{matrix}$$

$F_b = 1.45 \times 2.94 \times 0.85 \times 1.0 \times 1.0 \times 1.0 \times 1.15 \times 0.8 = 3.33$ Ksi for Strength I

$F_b = 1.45 \times 2.94 \times 0.85 \times 1.0 \times 1.0 \times 1.0 \times 1.15 \times 1.0 = 4.17$Ksi for Strength II

Adjusted Design Value = $F_b = 3.33$ Ksi for Design and Legal Load Rating

Adjusted Design Value = $F_b = 4.17$ Ksi for Permit Load Rating

Nominal Resistance = $R_n = F_b S C_L$

$C_L = 1.0$

For Design Load Rating :

$$R_n = 3.33 \times 32 \times \frac{1}{12} = 8.88 \text{ Kip} - \text{ft}$$

For Legal Load Rating :

$$R_n = 3.33 \times 32 \times \frac{1}{12} = 8.88 \text{ Kip} - \text{ft}$$

For Routine Permit Load Rating :

$$R_n = 4.17 \times 32 \times \frac{1}{12} = 11.12 \text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.85, for Wood Component in Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Slab in Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The live load factors in Table H.3-7 are used.

Table H.3-7 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.3-8.

Table H.3-8 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	5.31 5.42	5.31 5.42
Legal	IL-PS2-21	5.30	5.84
	IL-PS3-31	6.43	7.09
	IL-PS4-34.75	8.18	9.01
	IL-PS4-28	7.51	8.26
	IL-PS5-36	9.00	9.92
	IL-PS6-35.75	7.76	8.55
	IL-PS7-39.75	7.76	8.55
	IL-PC3-31	4.50	4.96
	IL-PC4-41	7.15	7.87
	IL-PC5-41	7.51	8.26
Routine Permit	IL-RS3-34	8.73	9.52
	IL-RS4-38	10.18	11.11
	IL-RS5-50	10.54	11.49
	IL-RS6-60	10.18	11.11

H.4 Prestressed Concrete I-Girder Bridge with Concrete Deck

Bridge Data:

Span: 36 ft

Year Built: 1975

Material:

Concrete: $f'_c = 3.0$ Ksi, for Deck

$f'_c = 4.0$ Ksi, for P/S Beam

$f'_c = 3.0$ Ksi, for P/S Beam at Transfer

Non-Prestressed Reinforcing Steel: $f_y = 36$ Ksi

Prestressing Steel: ½ in. Diameter, 250 Ksi, Low-Relaxation Strands

$A_{ps} = 0.153$ in^2, per Strand

14 Prestressing Strands

Condition: Fair

Traffic: Multiple Lanes

ADTT (one direction): Unknown

Skew: 0°

Additional Information: Diaphragms at every 12 ft

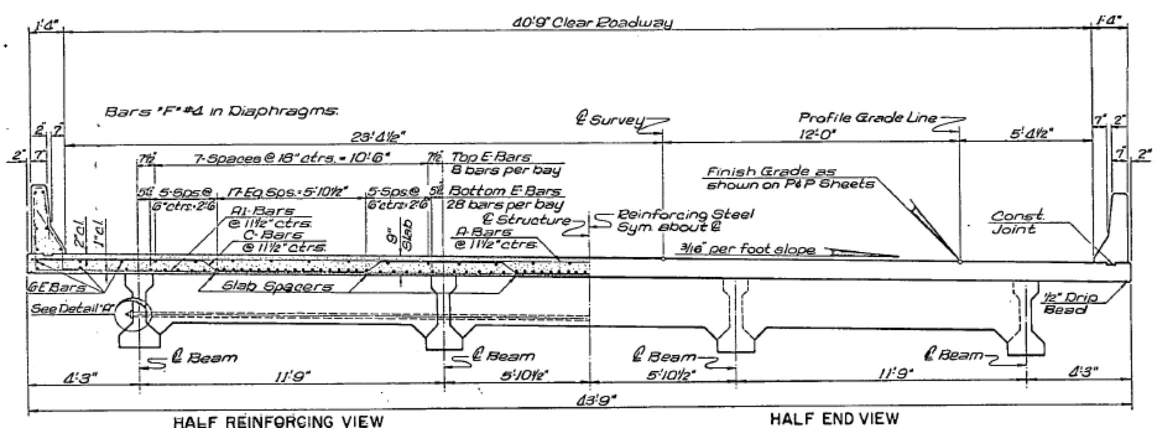

Figure H.4-1 Cross Section of P/S I-Beam Bridge

Number of Beams: 4

Beam Spacing: 11 ft 9 in = 11.75 ft

Thickness of Deck: 9 in

Width of Concrete Deck: 40 ft 9 in = 40.75 ft

Thickness of Concrete Overlay: 2.5 in

Width of Clear Roadway: 43 ft 5 in = 43.42 ft

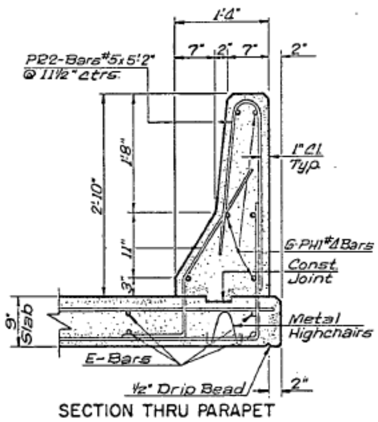

Figure H.4-2 Cross Section of Parapet

$$\text{Area of Parapet} = \left[\frac{7+9}{12} \times \frac{20}{12} + \frac{9+16}{12} \times \frac{11}{12}\right] \times \frac{1}{2} + \frac{16}{12} \times \frac{3}{12} = 2.40 \text{ ft}^2$$

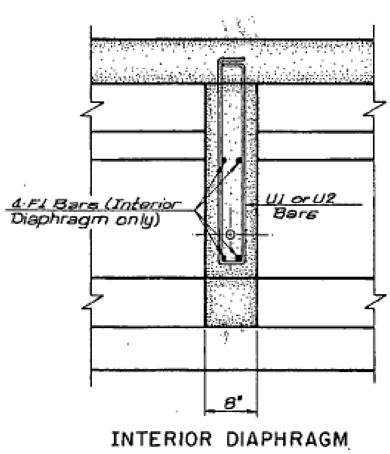

Figure H.4-3 Dimension of Diaphragm

$$\text{Volume of One Diaphragm} = \frac{30}{12} \times 11.25 \times \frac{8}{12} = 18.75 \text{ ft}^3$$

Effective Flange Width b_e

Minimum of:

i. $\frac{1}{4}\mathrm{L} = \frac{1}{4} \times 36 \times 12 = 108 \text{ in}$ Governs

ii. $12\mathrm{t_s} + \text{greater of } \mathrm{t_w} \text{or } \frac{1}{2}\mathrm{b_{f\,top}} = 12 \times 9 + 6 = 114 \text{ in}$

iii. $\mathrm{S} = 11.75 \times 12 = 141 \text{ in}$

Effective Flange Width b_e = 108 in

E_c = $33000(\mathrm{w_c})^{1.5}\sqrt{\mathrm{f_c'}}$

For Deck, E_{deck} = $33000 \times (0.15)^{1.5} \times \sqrt{3.0} = 3320.56 \text{ Ksi}$

For P/S Beam, E_{beam} = $33000 \times (0.15)^{1.5} \times \sqrt{4.0} = 3834.25 \text{ Ksi}$

Modular Ratio, n:

$$\mathrm{n} = \frac{\mathrm{E_{deck}}}{\mathrm{E_{beam}}} = \frac{3320.56}{3834.25} = 0.87$$

Transformed Width, $\mathrm{b_{trans}} = \mathrm{nb_e} = 0.87 \times 108 = 93.96 \text{ in}$

Summary of Section Properties

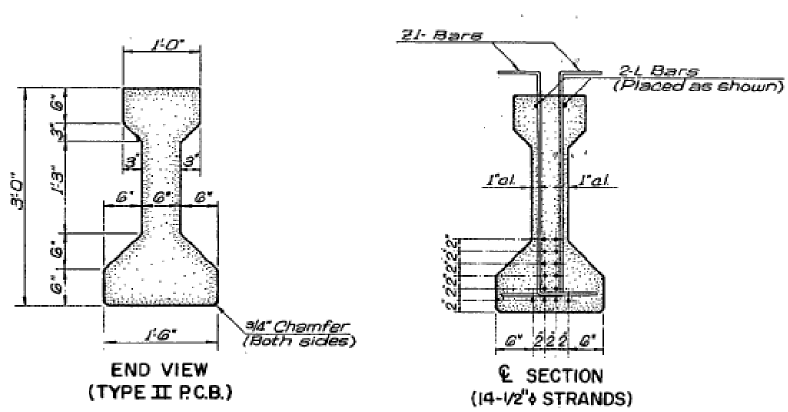

Figure H.4-4 Cross Section of AASHTO Type 2 I-Girder

Type 2 Girder:

h = 36 in

A = 369 in^2

$I = 50980 \text{ in}^4$

$Y_b = 15.83$ in

$S_b = 3220 \text{ in}^3$

$S_t = 2528 \text{ in}^3$

Composite Section:

Table H.4-1 Composite Section Properties

	Area, in^2	Y, in	AY	d	Ad^2, in^4	I_0, in^4
P/S Beam	369	15.83	5,841	17.18	108,911	50,980
Slab	846	40.50	34,263	7.49	47,461	5,708
Totals	1,215		40,104		156,372	56,688

Area of Slab = $t_s b_{trans} = 9 \times 93.96 = 846 \text{ in}^2$

$$\text{Y for Slab} = h + \frac{t_s}{2} = 36 + \frac{9}{2} = 50.50 \text{ in}$$

$$\overline{Y} = \frac{\sum AY}{A} = \frac{40104}{1215} = 33.01 \text{ in}$$

$$d = |Y - \overline{Y}|$$

$$y_{bottom} = \overline{Y} = 33.01 \text{ in}$$

$$y_{top} = h - \overline{Y} = 36 - 33.01 = 2.99 \text{ in}$$

$$I_{0,slab} = \frac{bh^3}{12} = \frac{b_{trans}{t_s}^3}{12} = \frac{93.96 \times 9^3}{12} = 5708 \text{ in}^4$$

$$I_{comp} = \sum I_0 + \sum Ad^2 = 56688 + 156372 = 213060 \text{ in}^4$$

Bottom of Beam, S_b:

$$S_b = \frac{I}{y_{bottom}} = \frac{213060}{33.01} = 6454 \text{ in}^3$$

Top of Beam, S_t:

$$S_t = \frac{I}{y_{top}} = \frac{213060}{2.99} = 71258 \text{ in}^3$$

Dead Load Analysis:

Components and Attachments, DC

Noncomposite Dead Loads, DC_1

Girder Self Weight = 0.384 Kip/ft

$$\text{Diaphragms} = \frac{0.15 \times 18.75 \times 3 \text{ Diaphragms}}{36} = 0.23 \text{ Kip/ft}$$

$$\text{Slab} = 11.75 \times \frac{9}{12} \times 0.15 = 1.32 \text{ Kip/ft}$$

Total per Girder DC_1 = $0.384 + 0.23 + 1.32 = 1.93 \text{ Kip/ft}$

$$M_{DC1} = \frac{1}{8} \times 1.93 \times 36^2 = 312.66 \text{ Kip} - \text{ft}$$

Composite Dead Loads, DC_2

$$\text{Parapet} = 0.15 \times 2.40 \times \frac{2 \text{ Parapets}}{4 \text{ Beams}} = 0.18 \text{ Kip/ft}$$

$$M_{DC2} = \frac{1}{8} \times 0.18 \times 36^2 = 29.16 \text{ Kip} - \text{ft}$$

Wearing Surface, DW

$$\text{Concrete Overlay} = 0.15 \times 40.75 \times \frac{2.5}{12} \times \frac{1}{4 \text{ Beams}} = 0.32 \text{ Kip/ft}$$

$$M_{DW} = \frac{1}{8} \times 0.32 \times 36^2 = 51.84 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

AASHTO LRFD Type *k* Cross Section

$K_g = n(I + Ae_g^2)$

$$n = \frac{E_B}{E_D} = \frac{3834.25}{3320.56} = 1.15$$

I = 50980 in^4

A = 369 in^2

$$e_g = \frac{1}{2}\, t_s + (h - Y_b) = \frac{1}{2} \times 9 + (36 - 15.83\,) = 24.67 \text{ in}$$

$K_g = 1.15 \times (50980 + 369 \times 24.67^2) = 316890.19 \text{ in}^4$

$$\frac{K_g}{12Lt_s^3} = \frac{316890.19}{12 \times 36 \times 9^3} = 1.01$$

One Lane Loaded:

$$g_1 = 0.06 + (\frac{S}{14})^{0.4}(\frac{S}{L})^{0.3}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.06 + (\frac{11.75}{14})^{0.4}(\frac{11.75}{36})^{0.3}(1.01)^{0.1}$$

$g_1 = 0.73$

Multiple Lanes Loaded:

$$g_m = 0.075 + (\frac{S}{9.5})^{0.6}(\frac{S}{L})^{0.2}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.075 + (\frac{11.75}{9.5})^{0.6}(\frac{11.75}{36})^{0.2}(1.01)^{0.1}$$

$g_m = 0.98$

Multiple-lane loading controls.

Undistributed Live Load Effects

The maximum mid-span moments induced by the Illinois design, legal and routine permit live loads are displayed in Table H.4-2. Dynamic load allowance (IM) of 33% is included in the calculation.

Table H.4-2 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	635.68
Legal	IL-PS2-21	457.52
	IL-PS3-31	645.32
	IL-PS4-34.75	655.69
	IL-PS4-28	489.44
	IL-PS5-36	452.87
	IL-PS6-35.75	563.92
	IL-PS7-39.75	563.92
	IL-PC3-31	369.74
	IL-PC4-41	575.62
	IL-PC5-41	609.14
Routine Permit	IL-RS3-34	649.04
	IL-RS4-38	561.26
	IL-RS5-50	532.00
	IL-RS6-60	611.80

Distributed Live Load Effects

The live load effects are distributed with multiple-lane loading factor, 0.98. Table H.4-3 displays the distributed mid-span moments for an interior beam.

Table H.4-3 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	622.97

Legal	IL-PS2-21	448.37
	IL-PS3-31	632.41
	IL-PS4-34.75	642.58
	IL-PS4-28	479.65
	IL-PS5-36	443.81
	IL-PS6-35.75	552.64
	IL-PS7-39.75	552.64
	IL-PC3-31	362.35
	IL-PC4-41	564.11
	IL-PC5-41	596.96
Routine Permit	IL-RS3-34	636.06
	IL-RS4-38	550.03
	IL-RS5-50	521.36
	IL-RS6-60	599.56

Nominal resistance:

Average stress in prestressing steel:

$$f_{ps} = f_{pu}(1 - k\frac{c}{d_p})$$

k=0.28, for Low Relaxation Strand

$$f_{pu} = 250 \text{ Ksi}$$

dp, distance from extreme compression fiber to the centroid of the prestressing tendons. Table H.4-4 displays the arrangement of the prestressing tendons.

Table H.4-4 Placement of Prestressing Tendons

	Number of Strands	y, in	Number of Strands * y
Layer 1	4	2	8
Layer 2	2	4	8
Layer 3	2	6	12

Layer 4	2	8	16
Layer 5	2	10	20
Layer 6	2	12	24
Total	14		88

Distance from Bottom of Girder to Centroid of Prestressing Strands:

$$\bar{y} = \frac{\sum \text{Number of Strands} \times y}{\sum \text{Number of Strands}} = \frac{88}{14} = 6.29 \text{ in}$$

$$d_p = (h + t_s) - \bar{y} = (36 + 9) - 6.29 = 38.71 \text{ in}$$

Distance from the Neutral Axis to the Compressive Face (Rectangular Section Behavior Assumed):

$$c = \frac{A_{ps}\, f_{pu}}{0.85 f_c' \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

$$A_{ps} = 0.153 \times 14 = 2.14 \text{ in}^2$$

$$f_{pu} = 250 \text{ Ksi}$$

$$f_c' = 3.0 \text{ Ksi}$$

β_1 = 0.85

b = be = 108 in

$$c = \frac{2.14 \times 250}{0.85 \times 3.0 \times 0.85 \times 108 + 0.28 \times 2.14 \times \frac{250}{38.71}} = 2.25 \text{ in}$$

$$a = \beta_1 c = 0.85 \times 2.25 = 1.91 \text{ in}$$

a < $t_s = 9$ in, the assumption of the rectangular section behavior is valid.

$$f_{ps} = 250 \times \left(1 - 0.28 \times \frac{2.25}{38.71}\right) = 245.93 \text{ Ksi}$$

Nominal Resistance :

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) = [2.14 \times 245.93 \times \left(38.71 - \frac{1.91}{2}\right)] \times \frac{1}{12}$$

$M_n = 1655.84 \text{ Kip} - \text{ft}$

Maximum Reinforcement

Net Tensile Strain, ε_t:

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_p = 38.71 \text{ in}$$

$$\varepsilon_t = \frac{(38.71 - 2.25) \times 0.003}{2.25} = 0.049 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 1.0.

Minimum Reinforcement

Amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$$M_r = \varphi M_n = 1.0 \times 1655.84 = 1655.84 \text{ Kip} - \text{ft}$$

1.33Mu :

$$1.33M_u = 1.33[(1.75M_{HL-93} + 1.25(M_{DC1} + M_{DC2}) + 1.50M_{DW}]$$

$$1.33M_u = 1.33[(1.75 \times 622.97 + 1.25(312.66 + 29.16) + 1.50 \times 51.84]$$

$$1.33M_u = 2121.66 \text{ Kip} - \text{ft}$$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

$$M_{dnc} = M_{DC1} = 312.66 \text{ Kip} - \text{ft}$$

$$S_c = 6454 \text{ in}^3$$

$$S_{nc} = 3220 \text{ in}^3$$

Modulus if Rupture, fr:

$$f_r = 0.37\sqrt{f'_c} = 0.37 \times \sqrt{4} = 0.74 \text{ Ksi}$$

Compressive Stress in Concrete due to Effective Prestress Force (after Allowance for All Prestress Losses) at Extreme Fiber of Section Where Tensile Stress is Caused by Externally Applied Loads, f_{cpe}:

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

Effective Prestress Force, P_{pe}:

$$P_{pe} = A_{ps}f_{pe}$$

$$f_{pe} = 0.75f_{pu} - \Delta f_{pT}$$

Total Prestress Losses, Δf_{pT}:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

Loss Due to Elastic Shortening and/or External Loads, Δf_{pES} :

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_d e}{I}$$

$$P_i = f_i A_{ps} = (0.9 \times 0.75 \times 250) \times\ 2.14 = 361.13\ \text{Kips}$$

$$M_d = \frac{\text{Girder Self Weight } \times \text{Span}^2}{8} = \frac{0.384\ \times 36^2}{8} = 62.21\ \text{Kips} - \text{ft}$$

$$e = Y_b - \bar{y} = 15.83\ - 6.29 = 9.54\ \text{in}$$

$$A = A_{Beam} = 369\ \text{in}^2$$

$$I = I_{Beam} = 50980\ \text{in}^4$$

$$f_{cgp} = \frac{361.13}{369} + \frac{361.13 \times 9.54^2}{50980} - \frac{62.21 \times 12 \times 9.54}{50980} = 1.48\ \text{Ksi}$$

$$E_p = 28500\ \text{Ksi}$$

$$E_{ct} = 33000(w_c)^{1.5}\sqrt{f'_{ci}} = 33000 \times (0.15)^{1.5}\sqrt{3} = 3320.56\ \text{Ksi}$$

$$\Delta f_{pES} = \frac{28500}{3320.56} \times 1.48 = 12.70\ \text{Ksi}$$

Approximate Lump Sum Estimate of Time-Dependent Losses, Δf_{pLT} :

Time-dependent losses include shrinkage of concrete, creep of concrete and relaxation of steel. For I-Girders, time-dependent losses can be approximated by:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}r_h r_{st} + 12.0 r_h r_{st} + \Delta f_{pR}$$

$$r_h = 1.7 - 0.01H$$

The relative humidity H is 72.5% for Illinois.

$$r_h = 1.7 - 0.01 \times 72.5 = 0.98$$

$$r_{st} = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 3} = 1.25$$

The estimate of relaxation loss Δf_{pR} is taken as 2.4 Ksi for low-relaxation strands.

$$A_g = A_{Beam} = 369\ \text{in}^2$$

$$\Delta f_{pLT} = 10.0 \times \frac{(0.75 \times 250) \times 2.14}{369} \times 0.98 \times 1.25 + 12.0 \times 0.98 \times 1.25 + 2.4$$

$$\Delta f_{pLT} = 30.42\ \text{Ksi}$$

Total Prestress Losses, Δf_{pT}:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 12.70 + 30.42 = 43.12\ \text{Ksi}$$

Effective Prestress Force, P_{pe}:

$$P_{pe} = f_{pe}A_{ps}$$

$$f_{pe} = f_{pi} - \Delta f_{pT} = 0.75 \times 250 - 43.12 = 144.38\ \text{Ksi}$$

$$P_{pe} = 144.38 \times 2.14 = 308.97\ \text{Kips}$$

Substitute in:

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b} = \frac{308.97}{369} + \frac{308.97 \times 9.54}{3220} = 1.75\ \text{Ksi}$$

M_{cr}:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

$$S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) = \frac{1}{12}[6454 \times (0.74 + 1.75)] - 312.66 \times \left(\frac{6454}{3220} - 1\right)$$

$$S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) = 1025.19 \text{ Kip} - \text{ft}$$

$$S_c f_r = \frac{1}{12} \times 6454 \times 0.74 = 398.00 \text{ Kip} - \text{ft}$$

Therefore, Mcr is taken as 1025.19 Kip-ft.

$$M_r = 1655.84 \text{ Kip} - \text{ft} > 1.2M_{cr} = 1.2 \times 1025.19 = 1230.23 \text{ Kip} - \text{ft} \quad \text{OK}$$

$$\varphi R_n = M_r = 1655.84 \text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 1.0, for Flexure

Condition Factor, φ_C = 0.95, for Fair Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.4-5.

Table H.4-5 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.4-6.

Table H.4-6 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.98 1.27	0.98 1.27
Legal	IL-PS2-21	1.64	1.78
	IL-PS3-31	1.17	1.25
	IL-PS4-34.75	1.14	1.24
	IL-PS4-28	1.54	1.67
	IL-PS5-36	1.66	1.80
	IL-PS6-35.75	1.33	1.44
	IL-PS7-39.75	1.33	1.44
	IL-PC3-31	2.04	2.19
	IL-PC4-41	1.30	1.42
	IL-PC5-41	1.23	1.33
Routine Permit	IL-RS3-34	1.05	1.33
	IL-RS4-38	1.22	1.55
	IL-RS5-50	1.28	1.62
	IL-RS6-60	1.11	1.41

Load Rating for Concrete Deck :

Dead Load Analysis: (Unit Width)

Components and Attachments, DC

Concrete Slab:

$$1.0 \times \frac{9}{12} \times 0.15 = 0.11 \text{ Kip/ft}$$

Curb:

$$2.40 \times 1.0 \times 0.15 \times \frac{2}{43.42} = 0.017 \text{ Kip/ft}$$

The deck is modeled as a continuous beam with overhang of 4ft-3in and the effective span as same as the distance of the supporting girders. The moment diagram for unit uniformly distributed load is shown in Figure H.4-5.

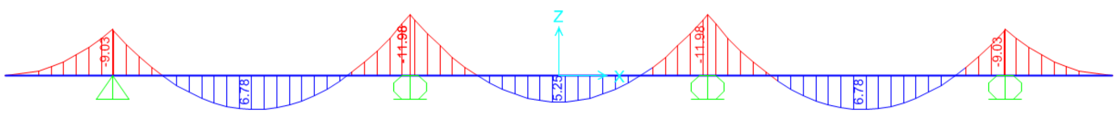

Figure H.4-5 Moment Diagram for Unit Uniformly Distributed Load

$$M_{DCPositive} = (0.11 + 0.017) \times 6.78 = 0.86 \text{ Kip} - \text{ft}$$

$$M_{DCNegative} = (0.11 + 0.017) \times (-11.98) = -1.52 \text{ Kip} - \text{ft}$$

Wearing Surface

$$DW = 1 \times \frac{2.5}{12} \times 0.14 = 0.029 \text{ Kip/ft}$$

$$M_{DWPositive} = 0.029 \times 6.78 = 0.20 \text{ Kip} - \text{ft}$$

$$M_{DWNegative} = 0.029 \times (-11.98) = -0.35 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Tables H.4-7 and H.4-8, are calculated with wheel loads of Illinois design, legal and routine permit trucks. Dynamic load allowance of 33% is included in the calculation. The minimum distance from the center of vehicle wheel to the inside face of parapet equals 1 ft. For two-lane loading, the distance between wheels of the two vehicles equals 4 ft. Figures H.4-6 and H.4-7 show the moment envelopes induced by the unit wheel loads in one lane and two lanes, respectively. Figures H.4-6 and H.4-7 show that the two-lane loading controls.

Multiple Presence Factor, m:

With one lane loaded, m=1.2

With two lanes loaded, m=1.0

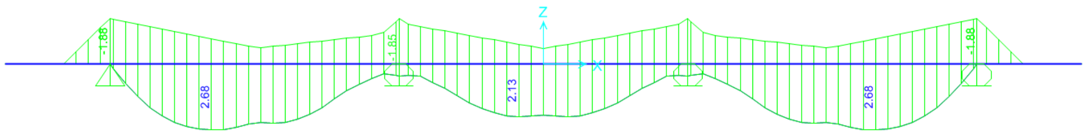

Figure H.4-6 Moment Envelope Diagram for Unit Wheel Loads in One Lane

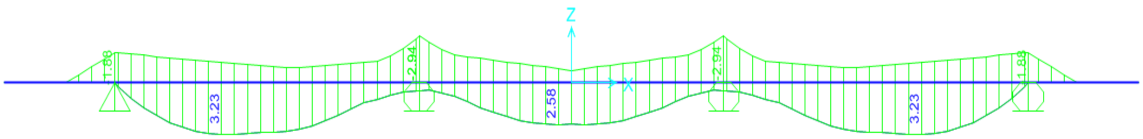

Figure H.4-7 Moment Envelope Diagram for Unit Wheel Loads in Two Lanes

Table H.4-7 Undistributed Positive Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	68.73
Legal	IL-PS2-21	53.70
	IL-PS3-31	54.99
	IL-PS4-34.75	47.25
	IL-PS4-28	38.66
	IL-PS5-36	36.52
	IL-PS6-35.75	38.66
	IL-PS7-39.75	38.66
	IL-PC3-31	47.25
	IL-PC4-41	54.13
	IL-PC5-41	48.11
Routine Permit	IL-RS3-34	57.99
	IL-RS4-38	47.25
	IL-RS5-50	49.40
	IL-RS6-60	51.55

Table H.4-8 Undistributed Negative Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	62.56
Legal	IL-PS2-21	48.88
	IL-PS3-31	50.05
	IL-PS4-34.75	43.01
	IL-PS4-28	35.19
	IL-PS5-36	33.24
	IL-PS6-35.75	35.19
	IL-PS7-39.75	35.19
	IL-PC3-31	43.01
	IL-PC4-41	49.27
	IL-PC5-41	43.79
Routine Permit	IL-RS3-34	52.79
	IL-RS4-38	43.01
	IL-RS5-50	44.97
	IL-RS6-60	46.92

Equivalent Lane Width

Equivalent Strip Width for Positive Moment:

$E_s = 26 + 6.6S$

S= Spacing of Supporting Components (ft) = 11.75 ft

$E_s = 26 + 6.6 \times 11.75 = 103.55 \text{ in} = 8.63 \text{ ft}$

Equivalent Strip Width for Negative Moment:

$E_s = 48 + 3.0S$

S= Spacing of Supporting Components (ft) = 11.75 ft

$E_s = 48 + 3.0 \times 11.75 = 83.25 \text{ in} = 6.94 \text{ ft}$

Distributed Live Load Effects

The calculated load effects in Table H.4-7 and H.4-8 are converted over transverse equivalent strip width and displayed in Tables H.4-9 and H.4-10.

Table H.4-9 Distributed Positive Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	7.96
Legal	IL-PS2-21	6.22
	IL-PS3-31	6.37
	IL-PS4-34.75	5.48
	IL-PS4-28	4.48
	IL-PS5-36	4.23
	IL-PS6-35.75	4.48
	IL-PS7-39.75	4.48
	IL-PC3-31	5.48
	IL-PC4-41	6.27
	IL-PC5-41	5.58
Routine Permit	IL-RS3-34	6.72
	IL-RS4-38	5.48
	IL-RS5-50	5.72
	IL-RS6-60	5.97

Table H.4-10 Distributed Negative Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	9.01
Legal	IL-PS2-21	7.04
	IL-PS3-31	7.21
	IL-PS4-34.75	6.20
	IL-PS4-28	5.07
	IL-PS5-36	4.79

	IL-PS6-35.75	5.07
	IL-PS7-39.75	5.07
	IL-PC3-31	6.20
	IL-PC4-41	7.10
	IL-PC5-41	6.31
Routine Permit	IL-RS3-34	7.61
	IL-RS4-38	6.20
	IL-RS5-50	6.48
	IL-RS6-60	6.76

Nominal resistance:

Positive Flexure

$$c = \frac{A_s\, f_y}{0.85 f_c' \beta_1 b}$$

#6 Rebar @ 1.0 ft. For unit width:

$A_s = 0.44\ \mathrm{in}^2/\mathrm{ft}$

$f_y = 36\ \mathrm{Ksi}$

$f_c' = 3.0\ \mathrm{Ksi}$

β_1 = 0.85

b = be = 12 in, Rectangular Section Behavior Assumed

$$c = \frac{0.44\ \times\ 36}{0.85 \times 3.0 \times 0.85 \times 12} = 0.61\ \mathrm{in}$$

$$a = \beta_1 c = 0.85 \times 0.61 = 0.52\ \mathrm{in}$$

a < Slab Thickness=9 in, the assumption of the rectangular section behavior is valid.

Distance from extreme compression fiber to C.G. of Steel, d_s:

$$d_s = \text{Slab Thickness} - \text{Deck bottom cover} - \frac{\text{Rebar Diameter}}{2}$$

$d_s = 9 - 1 = 8 \text{ in}$

Nominal Flexure Resistance, M_n:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[0.44 \times 36 \times \left(8.00 - \frac{0.52}{2}\right)\right] \times \frac{1}{12} = 10.22 \text{ Kip} - \text{ft}$$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_s = 8.00 \text{ in}$$

$$\varepsilon_t = \frac{(8.00 - 0.61) \times 0.003}{0.61} = 0.036 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

Negative Flexure

$$c = \frac{A_s\, f_y}{0.85 f'_c \beta_1 b}$$

#6 Rebar @ 1.0 ft. For unit width:

$$A_s = 0.44 \text{ in}^2/\text{ft}$$

$$f_y = 36 \text{ Ksi}$$

$$f'_c = 3.0 \text{ Ksi}$$

β_1 = 0.85

b = be = 12 in, Rectangular Section Behavior Assumed

$$c = \frac{0.44\ \times\ 36}{0.85 \times 3.0 \times 0.85 \times 12} = 0.61 \text{ in}$$

$$a = \beta_1 c = 0.85 \times 0.61 = 0.52 \text{ in}$$

a < Slab Thickness=9 in, the assumption of the rectangular section behavior is valid.

Distance from extreme compression fiber to C.G. of Steel, d_s:

$$d_s = \text{Slab Thickness} - \text{Deck Top cover} - \frac{\text{Rebar Diameter}}{2}$$

$$d_s = 9 - 2 = 7 \text{ in}$$

Nominal Flexure Resistance, M_n:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[0.44 \times 36 \times \left(7.00 - \frac{0.52}{2}\right)\right] \times \frac{1}{12} = 8.90 \text{ Kip} - \text{ft}$$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_s = 7.00 \text{ in}$$

$$\varepsilon_t = \frac{(7.00 - 0.61) \times 0.003}{0.61} = 0.031 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ=0.90, for tension controlled reinforced concrete slab in flexure

Condition Factor, φ_C = 0.95, for Fair Condition

System Factor, φ_S = 1.0, for Reinforced Concrete Slab

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.4-11.

Table H.4-11 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.4-12. The minimum RF between positive and negative flexures is displayed in Table H.4-12. Based on the calculation, RF for legal load rating is less than 1. Therefore, permit load rating is not applicable.

Table H.4-12 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.33 0.43	0.33 0.43
Legal	IL-PS2-21	0.51	0.56
	IL-PS3-31	0.50	0.55
	IL-PS4-34.75	0.57	0.64
	IL-PS4-28	0.71	0.76
	IL-PS5-36	0.74	0.81
	IL-PS6-35.75	0.71	0.76
	IL-PS7-39.75	0.71	0.76
	IL-PC3-31	0.57	0.64
	IL-PC4-41	0.51	0.55
	IL-PC5-41	0.57	0.63
Routine Permit	IL-RS3-34	N/A	N/A
	IL-RS4-38	N/A	N/A
	IL-RS5-50	N/A	N/A
	IL-RS6-60	N/A	N/A

H.5 Steel Stringer Bridge with Plank Deck

Bridge Data:

Span: 20 ft

Year Built: 1938

Year Reconstructed: 2007

Material:

Steel: $F_y = 33$ Ksi

Douglas Fir-Larch: No. 2

Condition: Good

Traffic: One Lane

ADTT (one direction): 52

Skew: 0°

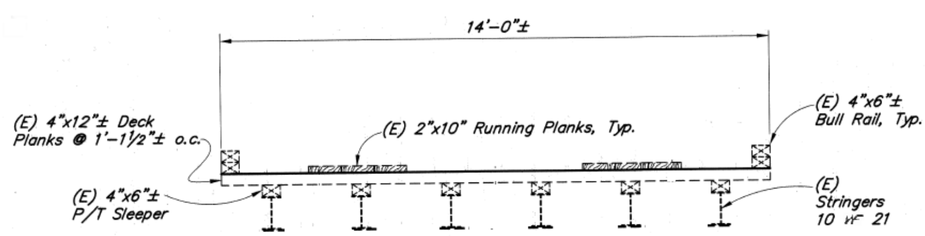

Figure H.5-1 Cross Section of Steel Stringer Bridge

Thickness of Deck: 4 in

Number of Beams: 6

Beam Spacing: 2ft 3in = 2.25 ft

Beam Type: W10x21

Rail Type: 4 in x 6 in

Running Plank Type: 2 in x 10 in

Timber Plank Sleeper Type: 4 in x 6 in

Thickness of Timber Overlay: 3.5 in

Width of Clear Roadway: 13 ft

Dead Load Analysis:

Components and Attachments, DC

$$\text{Deck} = 2.25 \times \frac{4}{12} \times 0.05 = 0.038\ \text{Kip/ft}$$

$\text{Stringer} = 0.021 \times 1.06 = 0.022\ \text{Kip/ft}$, 6% increase for connections

$$\text{Rail} = \frac{4 \times 6}{144} \times 0.05 \times \frac{2\ \text{Rails}}{6\ \text{Beams}} = 0.0028\ \text{Kip/ft}$$

$$\text{Running Planks} = \frac{2 \times 10}{144} \times 0.05 \times \frac{2\ \text{Planks}}{6\ \text{Beams}} = 0.0023\ \text{Kip/ft}$$

$$\text{Sleeper} = \frac{4 \times 6}{144} \times 0.05 = 0.0083\ \text{Kip/ft}$$

Total per stringer = $0.038 + 0.022 + 0.0028 + 0.0023 + 0.0083 = 0.073\ \text{Kip/ft}$

$$M_{DC} = \frac{1}{8} \times 0.073 \times 20^2 = 3.65\ \text{Kip} - \text{ft}$$

Wearing Surface

$$DW = \left(13 \times \frac{3.5}{12} \times 0.05\right) \times \frac{1}{6\ \text{Beams}} = 0.032\ \text{Kip} - \text{ft}$$

$$M_{DW} = \frac{1}{8} \times 0.032 \times 20^2 = 1.6\ \text{Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

AASHTO LRFD Type *a* cross section

One Lane Loaded:

$$g_1 = \frac{S}{8.8} = \frac{2.25}{8.8} = 0.26$$

Undistributed Live Load Effects

The undistributed live load effects due to the Illinois design, legal and permit loads in Table H.5-1 are used. The dynamic load allowance (IM) of 33% is applied for the calculation.

Table H.5-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	298.00
Legal	IL-PS2-21	234.08
	IL-PS3-31	315.48
	IL-PS4-34.75	305.90
	IL-PS4-28	223.44
	IL-PS5-36	169.58
	IL-PS6-35.75	244.72
	IL-PS7-39.75	244.72
	IL-PC3-31	146.30
	IL-PC4-41	238.34
	IL-PC5-41	238.34
Routine Permit	IL-RS3-34	287.28
	IL-RS4-38	234.08
	IL-RS5-50	244.72
	IL-RS6-60	292.60

Distributed Live Load Effects

The live load effects for one interior stringer are calculated by multiplying the one-lane distribution factor, g_1. The distributed load effects are displayed in Table H.5-2.

Table H.5-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	77.48
Legal	IL-PS2-21	60.86
	IL-PS3-31	82.02
	IL-PS4-34.75	79.53
	IL-PS4-28	58.09
	IL-PS5-36	44.09
	IL-PS6-35.75	63.63
	IL-PS7-39.75	63.63
	IL-PC3-31	38.04
	IL-PC4-41	61.97
	IL-PC5-41	61.97
Routine Permit	IL-RS3-34	74.69
	IL-RS4-38	60.86
	IL-RS5-50	63.63
	IL-RS6-60	76.08

Nominal resistance:

For W10x21:

$D = 10.2$ in

$t_w = 0.24$ in

$b_f = 5.75$ in

$t_f = 0.36$ in

$I_z = 118\ in^4$

$S_z = 23.2\ in^3$

$D_w = D - 2t_f = 10.2 - 2\times0.36 = 9.48$ in

$$D_c = D_t = \frac{D_w}{2} = \frac{9.48}{2} = 4.74 \text{ in}$$

Web Slenderness Limit

$$\frac{2D_c}{t_w} = \frac{2 \times 4.74}{0.24} = 39.5 < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7 \times \sqrt{\frac{29000}{33}} = 168.97 \quad \text{OK}$$

$$\frac{I_{yc}}{I_{yt}} = 1.0 > 0.3 \quad \text{OK}$$

Location of Plastic Neutral Axis (PNA)

$$\overline{Y} = \frac{D_w}{2} = \frac{9.48}{2} = 4.74 \text{ in},$$

from bottom of the top flange to PNA.

Top and Bottom Flanges

$$P_c = P_t = F_y b_f t_f = 33 \times 5.75 \times 0.36 = 68.31 \text{ Kips}$$

$$d_t = d_c = \frac{(t_f + D_w)}{2} = \frac{(0.36 + 9.48\ )}{2} = 4.92 \text{ in}$$

Web

$$P_w = F_y D_w t_w = 33 \times 9.48 \times 0.24 = 75.08 \text{ Kips}$$

Plastic Moment

$$M_p = \frac{P_w}{2D_w}[\overline{Y}^2 + (D_w - \overline{Y})^2] + P_c d_c + P_t d_t$$

$$M_p = \{\frac{75.08}{2 \times 9.48}[4.74^2 + (9.48 - 4.74)^2] + 2 \times 68.31 \times 4.92\} \times \frac{1}{12}$$

$$M_p = 70.84 \text{ Kip} - \text{ft}$$

Web Compactness

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

$$\frac{2D_{cp}}{t_w} = \frac{2 \times 4.74}{0.24} = 39.5$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{(0.54\frac{M_p}{R_h M_y} - 0.09)^2} \le \lambda_{rw}\frac{D_{cp}}{D_c}$$

where:

$$\lambda_{rw}\frac{D_{cp}}{D_c} = 5.7\sqrt{\frac{E}{F_{yc}}}\left(\frac{D_{cp}}{D_c}\right) = 5.7 \times \sqrt{\frac{29000}{33}} \times (1) = 168.97$$

$$R_h = 1$$

$$M_y = F_y S_z = 33 \times 23.2 \times \frac{1}{12} = 63.80 \text{ Kip} - \text{ft}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{33}}}{(0.54\frac{70.84}{1 \times 63.80} - 0.09)^2} = 114.16$$

Use 114.16

$$\frac{2D_{cp}}{t_w} = 39.5 \le \lambda_{pw(D_{cp})} = 114.16 \quad \text{OK}$$

The section is compact.

Web Plastification Factor, R_{pc}:

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{70.84}{63.80} = 1.11$$

Nominal resistance:

Sections are considered as continuously being braced at compression flanges.

$$R_n = \varphi_f R_{pc} M_{yc} = \varphi_f \frac{M_p}{M_{yc}} M_{yc} = \varphi_f M_p = 1 \times 70.84 \quad = 70.84 \quad \text{Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 1.0, for Flexure

Condition Factor, φ_C = 1.0, for Fair Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.5-3.

Table H.5-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine	1.20	1.10
Permit		

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.5-4. Based on the calculation, RF for legal load rating is less than 1. Therefore, permit load rating is not applicable.

Table H.5-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.47 0.61	0.47 0.61
Legal	IL-PS2-21	0.81	0.89
	IL-PS3-31	0.60	0.66
	IL-PS4-34.75	0.61	0.68
	IL-PS4-28	0.85	0.94
	IL-PS5-36	1.11	1.23

	IL-PS6-35.75	0.78	0.85
	IL-PS7-39.75	0.78	0.85
	IL-PC3-31	1.29	1.42
	IL-PC4-41	0.80	0.88
	IL-PC5-41	0.80	0.88
Routine Permit	IL-RS3-34	N/A	N/A
	IL-RS4-38	N/A	N/A
	IL-RS5-50	N/A	N/A
	IL-RS6-60	N/A	N/A

Load Rating for Timber Deck:

Dead Load Analysis:

Components and Attachments, DC

$$\text{Deck} = 1 \times \frac{4}{12} \times 0.05 = 0.017\ \text{Kip/ft}$$

$$\text{Rail} = \frac{4 \times 6}{144} \times 1 \times 0.05 \times \frac{2}{14} = 0.0012\ \text{Kip/ft}$$

$$\text{Running Planks} = \frac{2 \times 10}{144} \times 1 \times 0.05 \times \frac{2}{14} = 0.00099\ \text{Kip/ft}$$

$$\text{M}_{\text{DC}} = \frac{1}{8} \times (0.017 + 0.0012 + 0.00099) \times 2.25^2 = 0.012\ \text{Kip} - \text{ft}$$

Wearing Surface

$$\text{DW} = 1 \times \frac{3.5}{12} \times 0.05 = 0.015\ \text{Kip/ft}$$

$$\text{M}_{\text{DW}} = \frac{1}{8} \times 0.015 \times 2.25^2 = 0.0095\ \text{Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Table H.5-5, are calculated with wheel loads of Illinois design, legal and routine permit trucks. Dynamic load allowance is not considered for timber components.

Table H.5-5 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	9.00
Legal	IL-PS2-21	7.03
	IL-PS3-31	7.20
	IL-PS4-34.75	6.19
	IL-PS4-28	5.06
	IL-PS5-36	4.78
	IL-PS6-35.75	5.06
	IL-PS7-39.75	5.06
	IL-PC3-31	6.19
	IL-PC4-41	7.09
	IL-PC5-41	6.30
Routine Permit	IL-RS3-34	7.59
	IL-RS4-38	6.19
	IL-RS5-50	6.47
	IL-RS6-60	6.75

Equivalent Lane Width

Equivalent Strip Width:

$E_s = 4.0h + 40$

h= Thickness of Deck = 4.0 in

$E_s = 4.0 \times 4.0 + 40 = 56 \text{ in}$

Distributed Live Load Effects

The calculated axle loads are converted over transverse equivalent strip width. In this conversion, multiple presence factor for one-lane loading 1.2 is included.

Table H.5-6 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	2.31
Legal	IL-PS2-21	1.81
	IL-PS3-31	1.85
	IL-PS4-34.75	1.59
	IL-PS4-28	1.30
	IL-PS5-36	1.23
	IL-PS6-35.75	1.30
	IL-PS7-39.75	1.30
	IL-PC3-31	1.59
	IL-PC4-41	1.82
	IL-PC5-41	1.62
Routine Permit	IL-RS3-34	1.95
	IL-RS4-38	1.59
	IL-RS5-50	1.66
	IL-RS6-60	1.74

Nominal resistance:

Section Properties for Stringer

$$S_x = \frac{bd^2}{6} = \frac{12 \times 4.0^2}{6} = 32 \text{ in}^3$$

Design Values

$F_b = F_{bo} C_{KF} C_M \, (C_F \text{ or } C_V) C_{fu} C_i C_d C_\lambda$

Reference Design Value, F_{bo} :

$F_{bo} = 0.90$ Ksi

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\emptyset} = \frac{2.5}{0.85} = 2.94$$

Size Effect Factor for Sawn Lumber, C_F:

C_F = 1.0

Wet Service Factor, C_M:

$$F_{bo}C_F = 0.9 \times 1.0 = 0.90 \text{ Ksi} \leq 1.15 \text{ Ksi}$$

$C_M = 1.00$

Flat Use Factor, C_{fu}:

C_{fu} = 1.0

Incising Factor (only apply to dimension lumber), C_i:

C_i = 0.80

Deck Factor, C_d:

C_d = 1.00

$C_\lambda = \begin{matrix} 0.8 & \text{Time Effect Factor for Strength I} \\ 1.0 & \text{Time Effect Factor for Strength II} \end{matrix}$

F_b= $0.90 \times 2.94 \times 1.00 \times 1.00 \times 1.00 \times 0.80 \times 1.00 \times 0.80 = 1.69 \text{Ksi}$ for Strength I

F_b=$0.90 \times 2.94 \times 1.00 \times 1.00 \times 1.00 \times 0.80 \times 1.00 \times 1.00 = 2.12 \text{ Ksi}$ for Strength II

Adjusted Design Value = F_b = 1.69 Ksi for Design and Legal Load Rating

Adjusted Design Value = F_b = 2.12 Ksi for Permit Load Rating

Nominal Resistance = R_n = F_bSC_L

C_L = 1.0

For Design Load Rating :

$$R_n = 1.69 \times 32 \times \frac{1}{12} = 4.51 \text{ Kip} - \text{ft}$$

For Legal Load Rating :

$$R_n = 1.69 \times 32 \times \frac{1}{12} = 4.51 \text{ Kip} - \text{ft}$$

For Permit Load Rating :

$$R_n = 2.12 \times 32 \times \frac{1}{12} = 5.65 \text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.85, for Wood Component in Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Slab in Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are shown in Table H.5-7.

Table H.5-7 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine	1.20	1.10
Permit		

Rating Factors:

The rating factors (RF) calculated using the general equation are displayed in Table H.5-8.

Table H.5-8 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.94 1.22	0.94 1.22
Legal	IL-PS2-21	1.62	1.78
	IL-PS3-31	1.58	1.74
	IL-PS4-34.75	1.84	2.03
	IL-PS4-28	2.25	2.48
	IL-PS5-36	2.38	2.62
	IL-PS6-35.75	2.25	2.48
	IL-PS7-39.75	2.25	2.48
	IL-PC3-31	1.84	2.03
	IL-PC4-41	1.60	1.77
	IL-PC5-41	1.81	1.99
Routine Permit	IL-RS3-34	2.04	2.22
	IL-RS4-38	2.50	2.73
	IL-RS5-50	2.39	2.61
	IL-RS6-60	2.30	2.50

H.6 Reinforced Concrete T-Beam Bridge with Concrete Deck

Bridge Data:

Span: 41 ft

Year Built: 1958

Material:

Concrete: f'_c=2.5 Ksi

Reinforcing Steel: f_y=33 Ksi

Condition: Good

Cross Section: See Figure 9.2-1

Traffic: Multiple Lanes

ADTT (one direction): Unknown

Skew: 20° 41'3"= 20.68°

Beam cross section details: See Figure 9.2-2

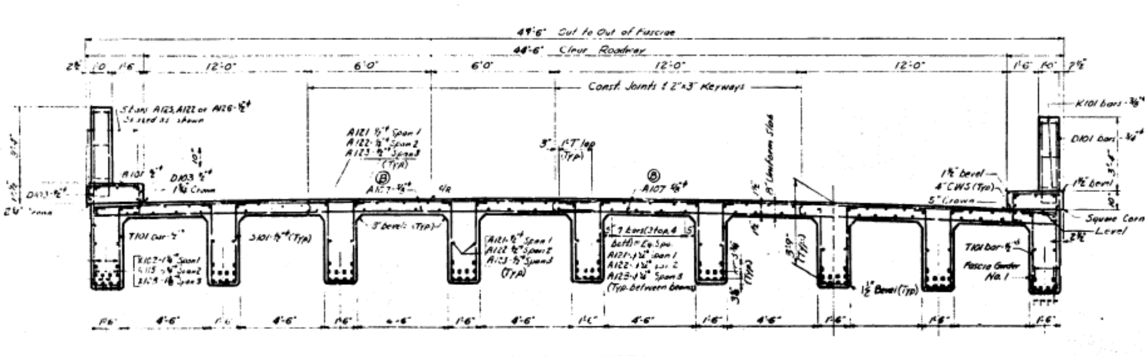

Figure H.6-1 Cross Section of Reinforced Concrete T-Beam Bridge

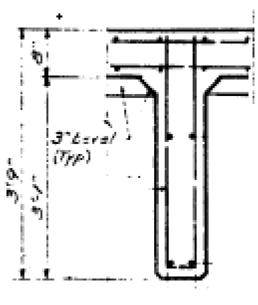

Figure H.6-2 Typical Beam Dimension

Number of Beams: 9

Beam Spacing: 6 ft

Depth of Beam: 3 ft 1 in = 3.08 ft

Width of Beam: 1 ft 6 in = 1.5 ft

Thickness of Deck: 8 in

Depth of Parapet: 3 ft 4 in = 3.33 ft

Width of Parapet: 1 ft

Depth of Curb: 10 in = 0.83 ft

Width of Curb: 2 ft 6 in = 2.5 ft

Thickness of Asphalt overlay: 5 in

Width of Clear Roadway: 44 ft 6 in = 44.5 ft

Load Rating for Interior Beam :

Dead Load Analysis:

Components and Attachments, DC

Structural Concrete: Consisting of deck + stem + haunches

$$[6 \times \frac{8}{12} + 1.5 \times 3.08 + (\frac{1}{2} \times \frac{3}{12} \times \frac{3}{12}) \times 2] \times 0.15 = 1.30 \text{ Kip/ft}$$

Parapets and Curbs:

$$(1 \times 3.33 + 2.5 \times 0.83) \times \frac{2 \text{ Parapets and Curbs}}{9 \text{ Beams}} \times 0.15 = 0.18 \text{ Kip/ft}$$

Total per Beam = $1.30 + 0.18 = 1.48 \text{ Kip/ft}$

$$\mathrm{M_{DC}} = \frac{1}{8} \times 1.48 \times 41^2 = 310.99 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\mathrm{DW} = 44.5 \times \frac{5}{12} \times 0.14 \times \frac{1}{9 \text{ Beams}} = 0.29 \text{ Kip/ft}$$

$$\mathrm{M_{DW}} = \frac{1}{8} \times 0.29 \times 41^2 = 60.94 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

AASHTO LRFD Type *e* cross section

Longitudinal Stiffness Parameter, K_g

$K_g = n(I + Ae_g^2)$

n=1

$$I = \frac{1}{12} \times 18 \times 37^3 = 75979.50 \text{ in}^4$$

A = $18 \times 37 = 666.00 \text{ in}^2$

$$e_g = \frac{1}{2} \times (8 + 37) = 22.50 \text{ in}$$

$K_g = 1 \times (75979.5 + 666 \times 22.5^2) = 413142.00 \text{ in}^4$

$$\frac{K_g}{12Lt_s^3} = \frac{413142}{12 \times 41 \times 8^3} = 1.64$$

One Lane Loaded:

$$g_1 = 0.06 + (\frac{S}{14})^{0.4}(\frac{S}{L})^{0.3}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.06 + (\frac{6}{14})^{0.4}(\frac{6}{41})^{0.3}(1.64)^{0.1} = 0.48$$

Multiple Lanes Loaded:

$$g_m = 0.075 + (\frac{S}{9.5})^{0.6}(\frac{S}{L})^{0.2}(\frac{K_g}{12Lt_s^3})^{0.1} = 0.075 + (\frac{6}{9.5})^{0.6}(\frac{6}{41})^{0.2}(1.64)^{0.1} = 0.62$$

Multiple-lane loading controls.

Correction Factor

$$C = 1 - 0.25(\frac{K_g}{12Lt_s^3})^{0.25}(\frac{S}{L})^{0.5}(\tan\theta)^{1.5}$$

$\theta = 20.68^\circ$

$$C = 1 - 0.25(1.64)^{0.25}(\frac{6}{41})^{0.5}(\tan 20.68)^{1.5} = 0.97$$

Undistributed Live Load Effects

The undistributed load effects due to the Illinois design, legal and permit live loads are displayed in Table H.6-1. Dynamic load allowance (IM) is included in the calculation.

Table H.6-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	732.98
Legal	IL-PS2-21	513.38
	IL-PS3-31	727.78
	IL-PS4-34.75	748.13
	IL-PS4-28	563.92
	IL-PS5-36	535.33
	IL-PS6-35.75	659.02
	IL-PS7-39.75	659.02
	IL-PC3-31	425.60
	IL-PC4-41	684.68
	IL-PC5-41	718.20
Routine Permit	IL-RS3-34	739.48
	IL-RS4-38	654.36
	IL-RS5-50	603.82
	IL-RS6-60	691.60

Distributed Live Load Effects:

The distributed live load effects for one interior beam are computed with multiple-lane loading, considering the skew effect. Table H.6-2 displays the distributed mid-span moments for an interior beam.

Table H.6-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	440.81
Legal	IL-PS2-21	308.75
	IL-PS3-31	437.68
	IL-PS4-34.75	449.92
	IL-PS4-28	339.14

	IL-PS5-36	321.94
	IL-PS6-35.75	396.33
	IL-PS7-39.75	396.33
	IL-PC3-31	255.96
	IL-PC4-41	411.77
	IL-PC5-41	431.93
Routine Permit	IL-RS3-34	444.72
	IL-RS4-38	393.53
	IL-RS5-50	363.14
	IL-RS6-60	415.93

Nominal Resistance:

Effective Flange Width

The minimum of the following values is used as the effective flange width.

i. $\frac{1}{4}\mathrm{L} = \frac{1}{4} \times 41 \times 12 = 123 \text{ in}$

ii. $12\mathrm{t_s} + \text{greater of } \mathrm{t_w} \text{ or } \frac{1}{2}\mathrm{b_{f\,top}} = 12 \times 8 + 18 = 114 \text{ in}$

iii. $\mathrm{S} = 6 \times 12 = 72 \text{ in}$

Use 72 in.

Distance to Neutral Axis, c

Rectangular section behavior is assumed.

$$c = \frac{A_s f_y}{0.85 f_c' \beta_1 b}$$

10 bars with diameters of $1^{1/4}$" are used as reinforcing steels.

$$A_{bar} = \frac{3.14 \times (1.25)^2}{4} = 1.23 \text{ in}^2$$

$$A_s = 1.23 \times 10 = 12.30 \text{ in}^2$$

$$f_y = 33 \text{ Ksi}$$

$$f_c' = 2.5 \text{ Ksi}$$

$$\beta_1 = 0.85, \text{for } f_c' < 4 \text{ Ksi}$$

$$b = 72 \text{ in}$$

$$c = \frac{12.30 \times 33}{0.85 \times 2.5 \times 0.85 \times 72} = 3.12 \text{ in} < \text{Thickness of Deck} = 8 \text{ in}$$

The neutral axis is within slab. Therefore, there will be rectangular section behavior.

$$a = c\beta_1 = 3.12 \times 0.85 = 2.65 \text{ in}$$

Distance from Bottom of Section to C.G. of Reinforcement, $\bar{y}$:

$$\bar{y} = \frac{2 \times 3.75 \times 3 + 4 \times 3.75 \times 2 + 4 \times 3.75}{10} = 6.75 \text{ in}$$

Nominal Resistance:

$$R_n = A_s f_y \left(d_s - \frac{a}{2}\right)$$

$A_s = 12.30 \text{ in}^2$

$f_y = 33 \text{ Ksi}$

$$d_s = h - \bar{y} = (3 \times 12 + 9) - 6.75 = 38.25 \text{ in}$$

$$a = 2.65 \text{ in}$$

$$M_n = 12.3 \times 33 \times \left(38.25 - \frac{2.65}{2}\right) \times \frac{1}{12} = 1248.99 \text{ Kip} - \text{ft}$$

Minimum Reinforcement

The amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$$M_r = \varphi R_n = 0.9 \times 1248.99 = 1124.09 \text{ Kip} - \text{ft}$$

1.33Mu :

$$1.33M_u = 1.33(1.75M_{HL-93} + 1.25M_{DC} + 1.50M_{DW})$$

$$1.33M_u = 1.33 \times (1.75 \times 440.81 + 1.25 \times 310.99 + 1.50 \times 60.94)$$

$$1.33M_u = 1664.58 \text{ Kip} - \text{ft}$$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

$$S_c = \frac{I_c}{Y_c}$$

Distance from bottom of beam to centroid of uncracked section, Y_c

$$Y_c = \frac{\sum(A_i y_i)}{\sum(A_i)} = \frac{18 \times 37 \times \frac{37}{2} + 72 \times 8 \times (37 + \frac{8}{2})}{18 \times 37 + 72 \times 8} = 28.93 \text{ in}$$

Moment of Inertia of composite section, I_c:

$$I_c = \sum (I_i + A_i d_i^2)$$

$$I_c = \frac{18 \times 37^3}{12} + 18 \times 37 \times (28.93 - \frac{37}{2})^2 + \frac{72 \times 8^3}{12} + 72 \times 8 \times (37 + \frac{8}{2} - 28.93)^2$$

$$I_c = 235416.75 \text{ in}^4$$

$$S_c = \frac{235416.75}{28.93} = 8137.46 \text{ in}^3$$

$$f_r = 0.37\sqrt{f'_c} = 0.37 \times \sqrt{2.5} = 0.59 \text{ Ksi}$$

$$f_{cpe} = 0, \text{for non} - \text{prestressed concrete structures}$$

F_{cpe} = 0, for non-prestressed concrete structures

$$M_{dnc} = 0, \text{for monolithic structures}$$

$$M_{cr} = S_c f_r = (8137.46 \times 0.59) \times \frac{1}{12} = 400.09 \text{ Kip} - \text{ft}$$

$$1.2M_{cr} = 1.2 \times 400.09 = 480.11 \text{ Kip} - \text{ft} < M_r = 1124.09 \text{ Kip} - \text{ft}$$

The section meets the requirements for minimum reinforcement.

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$d = d_s = 38.25 \text{ in}$$

$$c = 3.12 \text{ in}$$

$$\varepsilon_c = 0.003$$

$$\varepsilon_t = \frac{(38.25 - 3.12) \times 0.003}{3.12} = 0.034 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.90

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridges

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.6-3

Table H.6-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors, RF, are displayed in Table H.6-4 Since the RF for legal load rating is less than 1.0 when the current live load factors are used, the permit load rating is not applicable.

Table H.6-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.83 1.08	0.83 1.08
Legal	IL-PS2-21	1.44	1.56
	IL-PS3-31	1.02	1.09
	IL-PS4-34.75	0.99	1.08
	IL-PS4-28	1.30	1.42
	IL-PS5-36	1.38	1.50
	IL-PS6-35.75	1.12	1.21
	IL-PS7-39.75	1.12	1.21
	IL-PC3-31	1.74	1.89

	IL-PC4-41	1.08	1.16
	IL-PC5-41	1.03	1.11
Routine Permit	IL-RS3-34	N/A	1.14
	IL-RS4-38	N/A	1.31
	IL-RS5-50	N/A	1.41
	IL-RS6-60	N/A	1.24

Load Rating for Exterior Beam:

Dead Load Analysis:

Components and Attachments, DC

Structural Concrete: Consisting of deck + stem + haunches

$$[3 \times \frac{8}{12} + 1.5 \times 3.08 + (\frac{1}{2} \times \frac{3}{12} \times \frac{3}{12})] \times 0.15 = 1.00 \text{ Kip/ft}$$

Parapets and Curbs:

$$(1 \times 3.33 + 2.5 \times 0.83) \times \frac{2 \text{ Parapets and Curbs}}{9 \text{ Beams}} \times 0.15 = 0.18 \text{ Kip/ft}$$

Total per Beam = $1.00 + 0.18 = 1.18$ Kip/ft

$$M_{DC} = \frac{1}{8} \times 1.18 \times 41^2 = 247.95 \text{ Kip} - \text{ft}$$

Wearing Surface

$$DW = 44.5 \times \frac{5}{12} \times 0.14 \times \frac{1}{9 \text{ Beams}} = 0.29 \text{ Kip/ft}$$

$$M_{DW} = \frac{1}{8} \times 0.29 \times 41^2 = 60.94 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

One Lane Loaded:

Lever rule is applied to calculate the distribution factor for one-lane loading. The multiple presence factor, m=1.2 is included in the calculation.

For $S + d_e = 6ft + 0ft < 8ft$, one wheel acting upon the beam.

$$g_1 = m\left(\frac{S + d_e - 2}{2S}\right) = 1.2 \times \left(\frac{6 + 0 - 2}{2 \times 6}\right) = 0.33$$

Multiple Lanes Loaded:

$$g_m = eg_{m,interior}$$

$$e = 0.77 + \frac{d_e}{9.1} = 0.77$$

$$g_m = 0.77 \times 0.62 = 0.48$$

Special Analysis for Exterior Beams with Diaphragms or Cross-Frames.

Roadway Layout: Four 11-ft wide lanes

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum_1^{N_L} e}{\sum_1^{N_b} x^2}$$

$$g_{special} = mR$$

One Lane Loaded:

$$R = \frac{1}{9} + \frac{24 \times 16.75}{2 \times (24^2 + 18^2 + 12^2 + 6^2)} = 0.30$$

$$g_{special1} = 1.2 \times 0.3 = 0.36$$

Two Lanes Loaded:

$$R = \frac{2}{9} + \frac{24 \times (16.75 + 5.75)}{2 \times (24^2 + 18^2 + 12^2 + 6^2)} = 0.47$$

$$g_{special2} = 1.0 \times 0.47 = 0.47$$

Three Lanes Loaded:

$$R = \frac{3}{9} + \frac{24 \times (16.75 + 5.75 - 5.75)}{2 \times (24^2 + 18^2 + 12^2 + 6^2)} = 0.52$$

$$g_{special3} = 0.85 \times 0.52 = 0.44$$

Four Lanes Loaded:

$$R = \frac{4}{9} + \frac{24 \times (16.75 + 5.75 - 5.75 - 16.75)}{2 \times (24^2 + 18^2 + 12^2 + 6^2)} = 0.44$$

$$g_{\mathrm{special4}} = 0.65 \times 0.44 = 0.29$$

Governing Distribution Factor:

Used $g_m = 0.48$

The correction factor, C=0.97, is the same as for the interior beam.

Undistributed Live Load Effects

The undistributed load effects due to the Illinois design, legal and permit live loads are as same as for the interior beam, which are displayed in Table H.6-1.

Distributed Live Load Effects

The distributed live load effects for one exterior beam are computed with multiple-lane loading, considering the skew effect. Table H.6-5 displays the distributed mid-span moments for an interior beam.

Table H.6-5 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	341.28
Legal	IL-PS2-21	239.03
	IL-PS3-31	338.85
	IL-PS4-34.75	348.33
	IL-PS4-28	262.56
	IL-PS5-36	249.25
	IL-PS6-35.75	306.84
	IL-PS7-39.75	306.84
	IL-PC3-31	198.16
	IL-PC4-41	318.79
	IL-PC5-41	334.39

Routine Permit	IL-RS3-34	344.30
	IL-RS4-38	304.67
	IL-RS5-50	281.14
	IL-RS6-60	322.01

Nominal Resistance:

Effective Flange Width

$b_e = \frac{1}{2}\text{Interior}b_e + \text{Minimum of:}$

$\frac{1}{8}L = \frac{1}{8} \times 41 \times 12 = 61.5 \text{ in}$

$$6t_s + \text{greater of } \frac{1}{2}t_w \text{or } \frac{1}{4}b_{f\,top} = 6 \times 8 + 9 = 57 \text{ in}$$

$$\text{Overhang} = 0 \text{ in Controls}$$

$b_e = \frac{1}{2}\text{Interior}b_e + 0 = \frac{1}{2} \times 72 = 36 \text{ in}$

Distance to Neutral Axis, c

Rectangular section behavior is assumed.

$c = \frac{A_s f_y}{0.85 f_c' \beta_1 b}$

10 bars with diameters of $1^{1/4''}$ are used as reinforcing steels.

$A_{bar} = \frac{3.14 \times (1.25)^2}{4} = 1.23 \text{ in}^2$

$A_s = 1.23 \times 10 = 12.30 \text{ in}^2$

$f_y = 33 \text{ Ksi}$

$f_c' = 2.5 \text{ Ksi}$

$\beta_1 = 0.85, \text{for } f_c' < 4 \text{ Ksi}$

$b = 72 \text{ in}$

$$c = \frac{12.30 \times 33}{0.85 \times 2.5 \times 0.85 \times 72} = 3.12 \text{ in} < \text{Thickness of Deck} = 8 \text{ in}$$

The neutral axis is within slab. Therefore, there will be rectangular section

behavior.

$$a = c\beta_1 = 3.12 \times 0.85 = 2.65 \text{ in}$$

Distance from Bottom of Section to C.G. of Reinforcement, $\bar{y}$:

$$\bar{y} = \frac{2 \times 3.75 \times 3 + 4 \times 3.75 \times 2 + 4 \times 3.75}{10} = 6.75 \text{ in}$$

Nominal Resistance:

$$R_n = A_s f_y \left(d_s - \frac{a}{2}\right)$$

$A_s = 12.30 \text{ in}^2$

$f_y = 33 \text{ Ksi}$

$$d_s = h - \bar{y} = (3 \times 12 + 9) - 6.75 = 38.25 \text{ in}$$

$$a = 2.65 \text{ in}$$

$$M_n = 12.3 \times 33 \times \left(38.25 - \frac{2.65}{2}\right) \times \frac{1}{12} = 1248.99 \text{ Kip} - \text{ft}$$

Minimum Reinforcement

The amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$$M_r = \varphi R_n = 0.9 \times 1248.99 = 1124.09 \text{ Kip} - \text{ft}$$

1.33Mu :

$$1.33M_u = 1.33(1.75M_{HL-93} + 1.25M_{DC} + 1.50M_{DW})$$

$$1.33M_u = 1.33 \times (1.75 \times 341.28 + 1.25 \times 247.95 + 1.50 \times 60.94)$$

$$1.33M_u = 1328.12 \text{ Kip} - \text{ft}$$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

$$S_c = \frac{I_c}{Y_c}$$

Distance from bottom of beam to centroid of uncracked section, Y_c

$$Y_c = \frac{\sum(A_i y_i)}{\sum(A_i)} = \frac{18 \times 37 \times \frac{37}{2} + 36 \times 8 \times (37 + \frac{8}{2})}{18 \times 37 + 36 \times 8} = 25.29 \text{ in}$$

Moment of Inertia of composite section, I_c:

$$I_c = \sum(I_i + A_i d_i^2)$$

$$I_c = \frac{18 \times 37^3}{12} + 18 \times 37 \times (25.29 - \frac{37}{2})^2 + \frac{36 \times 8^3}{12} + 36 \times 8 \times (37 + \frac{8}{2} - 25.29)^2$$

$$I_c = 179300.41 \text{ in}^4$$

$$S_c = \frac{179300.41}{25.29} = 7089.78 \text{ in}^3$$

$$f_r = 0.37\sqrt{f_c'} = 0.37 \times \sqrt{2.5} = 0.59 \text{ Ksi}$$

$$f_{cpe} = 0, \text{for non} - \text{prestressed concrete structures}$$

$$M_{dnc} = 0, \text{for monolithic structures}$$

$$M_{cr} = S_c f_r = (7089.78 \times 0.59) \times \frac{1}{12} = 348.58 \text{ Kip} - \text{ft}$$

$$1.2M_{cr} = 1.2 \times 348.58 = 418.30 \text{ Kip} - \text{ft} < M_r = 1124.09 \text{ Kip} - \text{ft}$$

The section meets the requirements for minimum reinforcement.

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$d = d_s = 38.25 \text{ in}$$

$$c = 3.12 \text{ in}$$

$$\varepsilon_c = 0.003$$

$$\varepsilon_t = \frac{(38.25 - 3.12) \times 0.003}{3.12} = 0.034 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

General Load-Rating Equation:

$$\mathrm{RF} = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.90

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridges

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.6-6

Table H.6-6 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors, RF, are displayed in Table H.6-7.

Table H.6-7 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	1.21 1.57	1.21 1.57
Legal	IL-PS2-21	2.09	2.26
	IL-PS3-31	1.46	1.59
	IL-PS4-34.75	1.43	1.55
	IL-PS4-28	1.90	2.05
	IL-PS5-36	2.00	2.16
	IL-PS6-35.75	1.63	1.76
	IL-PS7-39.75	1.63	1.76
	IL-PC3-31	2.52	2.73
	IL-PC4-41	1.56	1.69
	IL-PC5-41	1.49	1.61
Routine Permit	IL-RS3-34	1.61	1.67
	IL-RS4-38	1.82	1.88
	IL-RS5-50	1.98	2.04
	IL-RS6-60	1.72	1.79

H.7 Prestressed Concrete Adjacent Box-Beam Bridge with Concrete Deck

Bridge Data:

Span: 60 ft

Year Built: 1968

Material:

Concrete: f'_c = 4.0 Ksi, for P/S Beam

f'_c = 3.0 Ksi, for P/S Beam at Transfer

f'_c = 3.0 Ksi, for Deck

Reinforcing Steel: f_y = 36 Ksi

Prestressing Steel: 3/8 in. Diameter, 250 Ksi, Stress-relieved strand

A_{ps} = 0.085 in^2, per Strand

28 Prestressing Strands

Condition: Good, the beams are transversely post-tensioned to act as a unit.

Traffic: Multiple Lanes

ADTT (one direction): Unknown

Skew: 0°

Number of Beams: 15

Thickness of Overlay: 5.25 in

Width of Clear Roadway: 42 ft 6 in = 42.5 ft

Width of Deck: 46 ft 5 in = 46.42 ft

Area of Parapet = $1 \times 2.25 + 1.96 \times 1 = 4.21$ ft^2

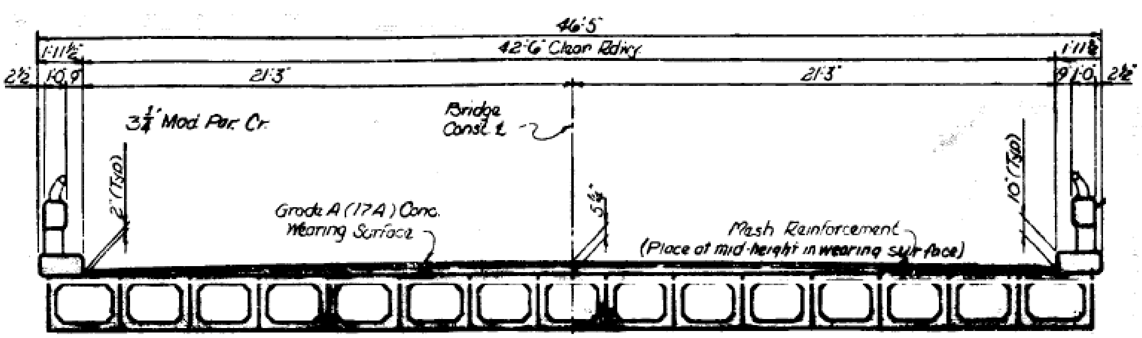

Figure H.7-1 Cross Section of P/S Adjacent Box Beam Bridge

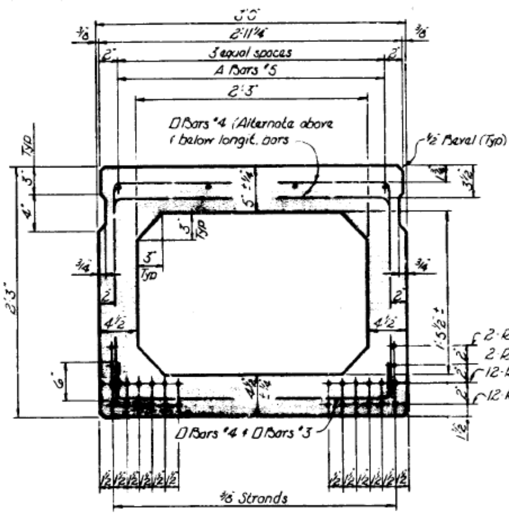

Figure H.7-2 Cross Section of Box-beam

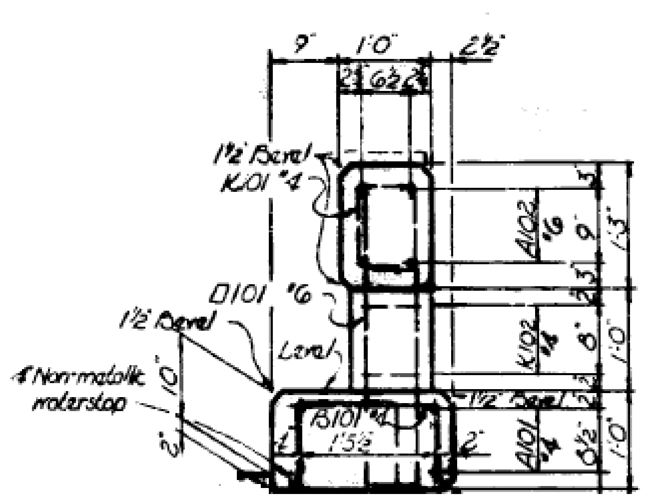

Figure H.7-3 Cross Section of Parapet

Summary of Box-Beam Section Properties

$b = 36$ in

$h = 27$ in

$A = 561$ in^2

$I = 50334$ in^4

$Y_b = 13.15$ in

S_b = 3770 in^3

S_t = 3687 in^3

Load Rating for Interior Box Beam:

Dead Load Analysis:

Components and Attachments, DC

Girder Self Weight = 0.584 Kip/ft

$$\text{Parapet} = 4.21 \times 0.15 \times \frac{2\text{ Parapets}}{15\text{ Beams}} = 0.08\text{ Kip/ft}$$

Total per Girder DC = $0.584 + 0.08 = 0.66\text{ Kip/ft}$

$$\text{M}_{\text{DC}} = \frac{1}{8} \times 0.66 \times 60^2 = 297.00\text{ Kip} - \text{ft}$$

Wearing Surface, DW

$$\text{Overlay} = 42.5 \times \frac{5.25}{12} \times 0.14 \times \frac{1}{15\text{ Beams}} = 0.17\text{ Kip/ft}$$

$$\text{M}_{\text{DW}} = \frac{1}{8} \times 0.17 \times 60^2 = 76.5\text{ Kip} - \text{ft}$$

Live Load Analysis:

Distribution Factor

AASHTO LRFD Type *g* Cross Section

$$\text{k} = 2.5(\text{N}_\text{b})^{-0.2} \geq 1.5$$

$$\text{N}_\text{b} = 15$$

$$\text{k} = 2.5 \times (15)^{-0.2} = 1.45 < 1.5$$

use k = 1.5

I = 50334 in^4

b = 36 in^2

$$J = \frac{4A_o^2}{\sum \frac{s}{t}}$$

Area enclosed by the centerlines of elements:

$$A_o = (36 - 4.5) \times (27 - 4.75) = 700.88 \text{ in}^2$$

s = Length of a side element

$$J = \frac{4 \times 700.88^2}{\frac{36 - 4.5}{5} + \frac{36 - 4.5}{4.5} + \frac{(27 - 4.75) \times 2}{4.5}} = 84735.89 \text{ in}^4$$

One Lane Loaded:

$$g_1 = k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25} = 1.5 \times \left(\frac{36}{33.3 \times 60}\right)^{0.5} \times \left(\frac{50334}{84735.89}\right)^{0.25} = 0.18$$

Multiple Lanes Loaded:

$$g_m = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$$

$$g_m = 1.5 \times \left(\frac{36}{305}\right)^{0.6} \times \left(\frac{36}{12 \times 60}\right)^{0.2} \times \left(\frac{50334}{84735.89}\right)^{0.06}$$

$$g_m = 0.22$$

Multiple-lane loading controls.

Undistributed Live Load Effects

The maximum mid-span moments induced by the Illinois design, legal and routine permit live loads are displayed in Table H.7-1. Dynamic load allowance (IM) of 33% is included in the calculation.

Table H.7-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	1352.00
	IL-PS2-21	792.68
	IL-PS3-31	1140.08
Legal	IL-PS4-34.75	1210.30
	IL-PS4-28	936.32
	IL-PS5-36	994.18

	IL-PS6-35.75	1134.49
	IL-PS7-39.75	1134.49
	IL-PC3-31	811.30
	IL-PC4-41	1229.98
	IL-PC5-41	1263.50
Routine Permit	IL-RS3-34	1191.68
	IL-RS4-38	1159.76
	IL-RS5-50	1002.82
	IL-RS6-60	1228.92

Distributed Live Load Effects

The live load effects are distributed with multiple-lane loading factor, 0.22. Table H.7-2 displays the distributed mid-span moments for an interior beam.

Table H.7-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	297.44
Legal	IL-PS2-21	174.39
	IL-PS3-31	250.82
	IL-PS4-34.75	266.27
	IL-PS4-28	205.99
	IL-PS5-36	218.72
	IL-PS6-35.75	249.59
	IL-PS7-39.75	249.59
	IL-PC3-31	178.49
	IL-PC4-41	270.60
	IL-PC5-41	277.97
Routine Permit	IL-RS3-34	262.17
	IL-RS4-38	255.15
	IL-RS5-50	220.62

IL-RS6-60	270.36

Nominal resistance:

Average Stress in Prestressing Steel:

$$f_{ps} = f_{pu}(1 - k\frac{c}{d_p})$$

k=0.38, for Stress-relieved Strand

$$f_{pu} = 250 \text{ Ksi}$$

dp, distance from extreme compression fiber to C.G. of prestressing tendons.

$$d_p = h - \bar{y} = 27 - \frac{12 \times 1.5 + 12 \times 3.5 + 2 \times 5.5 + 2 \times 7.5}{12 + 12 + 2 + 2} = 23.93 \text{ in}$$

Distance from the Neutral Axis to the Compressive Face:

$$c = \frac{A_{ps}\, f_{pu}}{0.85 f_c' \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

$$A_{ps} = 0.085 \ \times 28 = 2.38 \text{ in}^2$$

b = 36 in, Rectangular Section Behavior Assumed

$$f_c' = 4.0 \text{ Ksi}$$

β_1 = 0.85

$$c = \frac{2.38\ \times\ 250}{0.85 \times 4.0 \times 0.85 \times 36 + 0.38 \times 2.38 \times \frac{250}{23.93}} = 5.24 \text{ in}$$

$$a = \beta_1 c = 0.85 \times 5.24 = 4.45 \text{ in}$$

a < $t_s = 5$ in, the assumption of the rectangular section behavior is valid.

$$f_{ps} = 250 \times \left(1 - 0.38 \times \frac{5.24}{23.93}\right) = 229.20 \text{ Ksi}$$

Nominal Resistance :

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) = [2.38 \times 229.20 \times \left(23.93 - \frac{4.45}{2}\right)] \times \frac{1}{12}$$

$M_n = 986.67 \text{ Kip} - \text{ft}$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$d = d_p = 23.93 \text{ in}$

c = 5.24 in

$\varepsilon_c = 0.003$

$$\varepsilon_t = \frac{(23.93 - 5.24) \times 0.003}{5.24} = 0.01 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 1.0.

Minimum Reinforcement

Amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$M_r = \varphi M_n = 1.0 \times 986.67 \; = 986.67 \;\text{ Kip} - \text{ft}$

1.33Mu :

$1.33M_u = 1.33[(1.75M_{HL-93} + 1.25M_{DC} + 1.50M_{DW}]$

$1.33M_u = 1.33[(1.75 \times 297.44 + 1.25 \times 297.00 \; + 1.50 \times 76.5]$

$1.33M_u = 1338.67 \text{ Kip} - \text{ft}$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

For a monolithic section designed to resist all the loads, S_{nc} is substituted for S_c.

$M_{cr} = S_{nc}(f_r + f_{cpe}) \geq S_{nc} f_r$

$S_{nc} = S_b = \; 3770 \text{ in}^3$

Modulus of Rupture, fr:

$$f_r = 0.37\sqrt{f'_c} = 0.37 \times \sqrt{4} = 0.74 \text{ Ksi}$$

Compressive Stress in Concrete due to Effective Prestress Force (after Allowance for All Prestress Losses) at Extreme Fiber of Section Where Tensile Stress is Caused by Externally Applied Loads, f_{cpe}:

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

Effective Prestress Force, P_{pe}:

$$P_{pe} = A_{ps}f_{pe}$$

$$f_{pe} = 0.7f_{pu} - \Delta f_{pT}$$

Total Prestress Losses, Δf_{pT}:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

Loss Due to Elastic Shortening and/or External Loads, Δf_{pES} :

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_d e}{I}$$

$$P_i = f_i A_{ps} = (0.9 \times 0.70 \times 250) \times\ 2.38 = 374.85 \text{ Kips}$$

$$M_d = \frac{\text{Girder Self Weight } \times \text{Span}^2}{8} = \frac{0.584\ \times 60^2}{8} = 262.8 \text{ Kips} - \text{ft}$$

$$e = Y_b - \bar{y} = 13.15 - \frac{12 \times 1.5 + 12 \times 3.5 + 2 \times 5.5 + 2 \times 7.5}{12 + 12 + 2 + 2} = 10.08 \text{ in}$$

$$A = A_{Beam} = 561 \text{ in}^2$$

$$I = I_{Beam} = 50334 \text{ in}^4$$

$$f_{cgp} = \frac{374.85}{561} + \frac{374.85 \times 10.08^2}{50334} - \frac{262.8 \times 12 \times 10.08}{50334} = 0.79 \text{ Ksi}$$

$$E_p = 28500 \text{ Ksi}$$

$$E_{ct} = 33000(w_c)^{1.5}\sqrt{f'_{ci}} = 33000 \times (0.15)^{1.5}\sqrt{3} = 3320.56 \text{ Ksi}$$

$$\Delta f_{pES} = \frac{28500}{3320.56} \times\ 0.79 = 6.78\ \text{Ksi}$$

Approximate Lump Sum Estimate of Time-Dependent Losses, Δf_{pLT} :

Time-dependent losses include shrinkage of concrete, creep of concrete and relaxation of steel. Time-dependent losses can be approximated by:

$$\Delta f_{pLT} = 10.0 \frac{f_{pi} A_{ps}}{A_g} r_h r_{st} + 12.0 r_h r_{st} + \Delta f_{pR}$$

$$r_h = 1.7 - 0.01H$$

The relative humidity H is 72.5% for Illinois.

$$r_h = 1.7 - 0.01 \times 72.5 = 0.98$$

$$r_{st} = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 3} = 1.25$$

The estimate of relaxation loss Δf_{pR} is taken as 10 Ksi.

$$\Delta f_{pLT} = 10.0 \times \frac{(0.70 \times 250) \times 2.38}{561} \times 0.98 \times 1.25 + 12.0 \times 0.98 \times 1.25 + 10$$

$$\Delta f_{pLT} = 33.79\ \text{Ksi}$$

Total Prestress Losses, Δf_{pT}:

$$\Delta f_{pT} = \ \Delta f_{pES} + \Delta f_{pLT} = 6.78\ + 33.79 = 40.57\ \text{Ksi}$$

Effective Prestress Force, P_{pe}:

$$P_{pe} = \ f_{pe} A_{ps}$$

$$f_{pe} = f_{pi} - \Delta f_{pT} = 0.70 \times 250 - 40.57 = 134.43\ \text{Ksi}$$

$$P_{pe} = \ 134.43 \times 2.38 = 319.94\ \text{Kips}$$

Substitute in:

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe} e}{S_b} = \frac{319.94}{561} + \frac{319.94\ \times 10.08}{3770} = 1.43\ \text{Ksi}$$

M_{cr}:

$$M_{cr} = S_{nc}(f_r + f_{cpe}) \geq S_{nc}f_r$$

$$S_{nc}\left(f_r + f_{cpe}\right) = \frac{1}{12}[3770 \times (0.74 + 1.43)] = 681.74 \text{ Kip} - \text{ft}$$

$$S_{nc}f_r = \frac{1}{12} \times 3770 \times 0.74 = 232.48 \text{ Kip} - \text{ft}$$

Therefore, Mcr is taken as 681.74 Kip-ft.

$$M_r = 986.67 \text{ Kip} - \text{ft} > 1.2M_{cr} = 1.2 \times 681.74 = 818.09 \text{ Kip} - \text{ft} \quad \text{OK}$$

$$\varphi R_n = M_r = 986.67 \quad \text{Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c\varphi_s\varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 1.0, for Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.7-3.

Table H.7-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors (RF) calculated using the general equation are displayed in Table H.7-4.

Table H.7-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.96 1.25	0.96 1.25
Legal	IL-PS2-21	1.97	2.14
	IL-PS3-31	1.38	1.50
	IL-PS4-34.75	1.29	1.41
	IL-PS4-28	1.68	1.82
	IL-PS5-36	1.58	1.71
	IL-PS6-35.75	1.38	1.50
	IL-PS7-39.75	1.38	1.50
	IL-PC3-31	1.94	2.08
	IL-PC4-41	1.28	1.39
	IL-PC5-41	1.24	1.34
Routine Permit	IL-RS3-34	1.46	1.52
	IL-RS4-38	1.51	1.55
	IL-RS5-50	1.75	1.80
	IL-RS6-60	1.43	1.47

Dead Load Analysis:

For adjacent box beams, the dead loads carried by an exterior beam are as same as the interior beam. Therefore,

$$\mathrm{M_{DC}} = 297.00\ \mathrm{Kip - ft}$$

$$\mathrm{M_{DW}} = 76.5\ \mathrm{Kip - ft}$$

Live Load Analysis:

Distribution Factor

One Lane Loaded:

$$g_1 = eg_{\text{Interior 1}}$$

$$e = 1.125 + \frac{d_e}{30} \geq 1.0$$

Horizontal Distance from the Centerline of the Exterior Web of Exterior Beam at the Deck Level to the Interior Edge of Curb or Traffic Barrier, d_e:

$$d_e = -16.25 \text{ in} = -1.35 \text{ ft} < 2 \text{ ft, OK}$$

$$e = 1.125 + \frac{-1.35}{30} = 1.08 \geq 1.0$$

$$g_{\text{Interior 1}} = 0.18$$

$$g_1 = 1.08 \times 0.18 = 0.19$$

Multiple Lanes Loaded:

$$g_m = eg_{m,\text{interior}}$$

$$e = 1.04 + \frac{d_e}{25} = 1.04 + \frac{-1.35}{25} = 0.99 < 1$$

$$e = 1$$

$$g_m = 1 \times 0.22 = 0.22$$

Multiple-lane loading controls.

Undistributed Live Load Effects

The undistributed load effects due to the Illinois design, legal and routine permit live loads are as same as in the interior beam, which are displayed in Table H.7-1.

Distributed Live Load Effects

The distributed live load effects for one exterior box beam are computed with multiple-lane loading. Table H.7-5 displays the distributed live load effects for an exterior box beam.

Table H.7-5 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	297.44
Legal	IL-PS2-21	174.39
	IL-PS3-31	250.82
	IL-PS4-34.75	266.27
	IL-PS4-28	205.99
	IL-PS5-36	218.72
	IL-PS6-35.75	249.59
	IL-PS7-39.75	249.59
	IL-PC3-31	178.49
	IL-PC4-41	270.60
	IL-PC5-41	277.97
Routine Permit	IL-RS3-34	262.17
	IL-RS4-38	255.15
	IL-RS5-50	220.62
	IL-RS6-60	270.36

Nominal resistance:

The flexure resistance for the exterior box beam is as same as in the interior beam, which is:

$$\varphi R_n = M_r = 986.67 \quad \mathrm{Kip-ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 1.0

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Multi-Girder Bridges

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.7-6.

Table H.7-6 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Permit	1.30	1.26

Rating Factors:

The rating factors, RF, are displayed in Table H.7-7.

Table H.7-7 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.96 1.25	0.96 1.25
Legal	IL-PS2-21	1.97	2.14
	IL-PS3-31	1.38	1.50
	IL-PS4-34.75	1.29	1.41
	IL-PS4-28	1.68	1.82
	IL-PS5-36	1.58	1.71
	IL-PS6-35.75	1.38	1.50
	IL-PS7-39.75	1.38	1.50
	IL-PC3-31	1.94	2.08
	IL-PC4-41	1.28	1.39
	IL-PC5-41	1.24	1.34
	IL-RS3-34	1.46	1.52

Routine Permit	IL-RS4-38	1.51	1.55
	IL-RS5-50	1.75	1.80
	IL-RS6-60	1.43	1.47

Load Rating for Concrete Deck :

Dead Load Analysis: (Unit Width)

Components and Attachments, DC

Concrete Slab:

$$1.0 \times \frac{(5.25 + 5.125)}{12} \times 0.15 = 0.13 \text{ Kip/ft}$$

Parapet:

$$4.21 \times 1.0 \times 0.15 \times \frac{2}{46.42} = 0.027 \text{ Kip/ft}$$

The deck is modeled as 15 continuous beams with the effective span lengths of 2.25 ft (Distance between inside of the box webs). The moment diagram for unit uniformly distributed load is shown in Figure H.7-4.

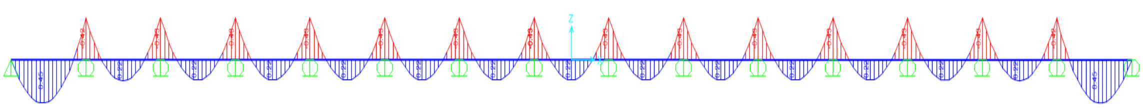

Figure H.7-4 Moment Diagram for Unit Uniformly Distributed Load

$$\mathrm{M_{DCPositive}} = (0.13 + 0.027) \times 0.45 = 0.071 \text{ Kip} - \text{ft}$$

$$\mathrm{M_{DCNegative}} = (0.13 + 0.027) \times (-0.43) = -0.068 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\mathrm{DW} = 1 \times \frac{5.25}{12} \times 0.14 = 0.061 \text{ Kip/ft}$$

$$\mathrm{M_{DWPositive}} = 0.061 \times 0.45 = 0.027 \text{ Kip} - \text{ft}$$

$$M_{DWNegative} = 0.061 \times (-0.43) = -0.026 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Tables H.7-8 and H.7-9 for positive and negative moments, respectively, are calculated with wheel loads of Illinois design, legal and routine permit trucks. Dynamic load allowance of 33% is included in the calculation. The minimum distance from the center of vehicle wheel to the inside face of parapet equals 1 ft. For two-lane loading, the distance between wheels of the two vehicles equals 4 ft. Figures H.7-5 and H.7-6 show the moment envelop diagrams induced by the unit wheel loads in one lane and two lanes, respectively. Figures H.7-5 and H.7-6 show that one-lane loading with the multiple presence factor of 1.2 controls.

Multiple Presence Factor, m:

With one lane loaded, m=1.2

With two lanes loaded, m=1.0

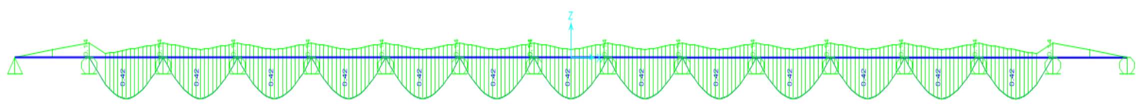

Figure H.7-5 Moment Envelope for Unit Wheel Loads in One Lane

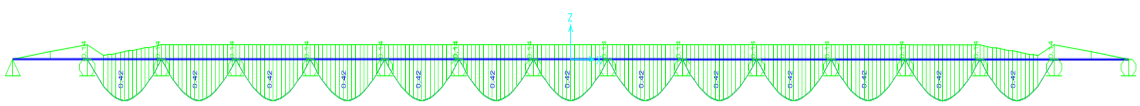

Figure H.7-6 Moment Envelope for Unit Wheel Loads in Two Lanes

Table H.7-8 Undistributed Positive Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	10.73
	IL-PS2-21	8.38
	IL-PS3-31	8.58
Legal	IL-PS4-34.75	7.37
	IL-PS4-28	6.03
	IL-PS5-36	5.70

	IL-PS6-35.75	6.03
	IL-PS7-39.75	6.03
	IL-PC3-31	7.37
	IL-PC4-41	8.45
	IL-PC5-41	7.51
Routine Permit	IL-RS3-34	9.05
	IL-RS4-38	7.37
	IL-RS5-50	7.71
	IL-RS6-60	8.04

Table H.7-9 Undistributed Negative Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	3.58
Legal	IL-PS2-21	2.79
	IL-PS3-31	2.86
	IL-PS4-34.75	2.46
	IL-PS4-28	2.01
	IL-PS5-36	1.90
	IL-PS6-35.75	2.01
	IL-PS7-39.75	2.01
	IL-PC3-31	2.46
	IL-PC4-41	2.82
	IL-PC5-41	2.50
Routine Permit	IL-RS3-34	3.02
	IL-RS4-38	2.46
	IL-RS5-50	2.57
	IL-RS6-60	2.68

Equivalent Lane Width

Equivalent Strip Width for Positive Moment:

$E_s = 26 + 6.6S$

S= Spacing of Supporting Components (ft) = 2.25 ft

$E_s = 26 + 6.6 \times 2.25 = 40.85 \text{ in} = 3.40 \text{ ft}$

Equivalent Strip Width for Negative Moment:

$E_s = 48 + 3.0S$

S= Spacing of Supporting Components (ft) = 2.25 ft

$E_s = 48 + 3.0 \times 2.25 = 54.75 \text{ in} = 4.56 \text{ ft}$

Distributed Live Load Effects

The calculated load effects in Table H.7-8 and H.7-9 are converted over transverse equivalent strip width and displayed in Tables H.7-10 and H.7-11.

Table H.7-10 Distributed Positive Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	3.15
Legal	IL-PS2-21	2.46
	IL-PS3-31	2.52
	IL-PS4-34.75	2.17
	IL-PS4-28	1.77
	IL-PS5-36	1.68
	IL-PS6-35.75	1.77
	IL-PS7-39.75	1.77
	IL-PC3-31	2.17
	IL-PC4-41	2.48
	IL-PC5-41	2.21
Routine Permit	IL-RS3-34	2.66
	IL-RS4-38	2.17
	IL-RS5-50	2.27

	IL-RS6-60	2.37

Table H.7-11 Distributed Negative Moments

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	0.78
Legal	IL-PS2-21	0.61
	IL-PS3-31	0.63
	IL-PS4-34.75	0.54
	IL-PS4-28	0.44
	IL-PS5-36	0.42
	IL-PS6-35.75	0.44
	IL-PS7-39.75	0.44
	IL-PC3-31	0.54
	IL-PC4-41	0.62
	IL-PC5-41	0.55
Routine Permit	IL-RS3-34	0.66
	IL-RS4-38	0.54
	IL-RS5-50	0.56
	IL-RS6-60	0.59

Nominal resistance:

Positive Flexure

$$c = \frac{A_s\, f_y}{0.85 f_c' \beta_1 b}$$

#4 Rebar @ 1.0 ft. For unit width:

$$A_s = 0.20\ \mathrm{in}^2/\mathrm{ft}$$

$$f_y = 36\ \mathrm{Ksi}$$

$$f_c' = 3.0\ \mathrm{Ksi}$$

β_1 = 0.85

b = be = 12 in, Rectangular Section Behavior Assumed

$$c = \frac{0.20 \times 36}{0.85 \times 3.0 \times 0.85 \times 12} = 0.28 \text{ in}$$

$$a = \beta_1 c = 0.85 \times 0.28 = 0.24 \text{ in}$$

Total Deck Thickness = Slab Thickness + Thickness of Top Flange of Box Beam

Total Deck Thickness = 5.25 + 5.125 = 10.38 in

a < Total Deck Thickness = 10.38 in, the assumption of the rectangular section behavior is valid.

Distance from extreme compression fiber to C.G. of Steel, d_s:

$$d_s = \text{Total Deck Thickness} - \text{Deck bottom cover} - \frac{\text{Rebar Diameter}}{2}$$

$$d_s = 10.38 - 1 - \frac{0.5}{2} = 9.13 \text{ in}$$

Nominal Flexure Resistance, M_n:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[0.20 \times 36 \times \left(9.13 - \frac{0.24}{2}\right)\right] \times \frac{1}{12} = 5.41 \text{ Kip} - \text{ft}$$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_s = 9.13 \text{ in}$$

$$\varepsilon_t = \frac{(9.13 - 0.28) \times 0.003}{0.28} = 0.095 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

Negative Flexure

$$c = \frac{A_s\, f_y}{0.85 f_c' \beta_1 b}$$

#4 Rebar @ 1.0 ft. For unit width:

$A_s = 0.20 \text{ in}^2/\text{ft}$

$f_y = 36 \text{ Ksi}$

$f'_c = 3.0 \text{ Ksi}$

$\beta_1 = 0.85$

b = be = 12 in, Rectangular Section Behavior Assumed

$$c = \frac{0.20 \times 36}{0.85 \times 3.0 \times 0.85 \times 12} = 0.28 \text{ in}$$

$$a = \beta_1 c = 0.85 \times 0.28 = 0.24 \text{ in}$$

a < Slab Thickness = 10.38 in, the assumption of the rectangular section behavior is valid.

Distance from extreme compression fiber to C.G. of Steel, d_s:

$$d_s = \text{Slab Thickness} - \text{Deck Top cover} - \frac{\text{Rebar Diameter}}{2}$$

$$d_s = 10.38 - 2.5 - \frac{0.5}{2} = 7.63 \text{ in}$$

Nominal Flexure Resistance, M_n:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[0.20 \times 36 \times \left(7.63 - \frac{0.24}{2}\right)\right] \times \frac{1}{12} = 4.51 \text{ Kip} - \text{ft}$$

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_s = 7.63 \text{ in}$$

$$\varepsilon_t = \frac{(7.63 - 0.28) \times 0.003}{0.28} = 0.079 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

General Load-Rating Equation:

$$\mathrm{RF} = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.90, for Tension Controlled Reinforced Concrete Slab in Flexure

Condition Factor, φ_C = 1.00, for Good Condition

System Factor, φ_S = 1.0, for Reinforced Concrete Slab

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.7-12.

Table H.7-12 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.45	1.34
Routine Permit	1.30	1.26

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.7-13. The minimum RF between positive and negative flexures is displayed in Table H.7-13.

Table H.7-13 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.86 1.11	0.86 1.11
Legal	IL-PS2-21	1.33	1.44

	IL-PS3-31	1.29	1.41
	IL-PS4-34.75	1.50	1.63
	IL-PS4-28	1.84	1.99
	IL-PS5-36	1.95	2.11
	IL-PS6-35.75	1.84	1.99
	IL-PS7-39.75	1.84	1.99
	IL-PC3-31	1.50	1.63
	IL-PC4-41	1.32	1.43
	IL-PC5-41	1.48	1.60
Routine Permit	IL-RS3-34	1.37	1.41
	IL-RS4-38	1.69	1.74
	IL-RS5-50	1.61	1.66
	IL-RS6-60	1.54	1.59

H.8 Steel Truss Bridge with Timber Deck

Bridge Data:

Span: 60 ft

Year Built: 1920

Material: Douglas Fir-Larch No. 1

Structural Steel: F_y = 30 Ksi

Condition: Good

Traffic: One Lane

ADTT (one direction): 151

Skew: 0°

Thickness of Deck: 3 in

Distance between Stringers: 27 in = 2.25 ft

Distance between Floor Beams: 15 ft

Width of Deck: 17.2 ft

Additional Information: The Deck is not analyzed as continuous since the spikes holding the deck down are not tight.

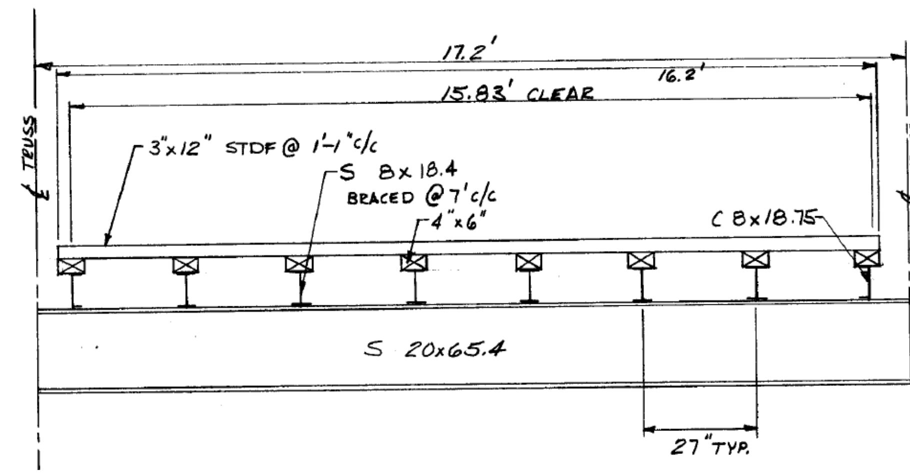

Figure H.8-1 Cross Section of Timber Deck

Load Rating for Timber Deck :

Dead Load Analysis:

Components and Attachments, DC

$$\mathrm{Deck} = 1 \times \frac{3}{12} \times 0.05 = 0.013\ \mathrm{Kip/ft}$$

$$\mathrm{M_{DC}} = \frac{1}{8} \times 0.013 \times 2.25^2 = 0.0082\ \mathrm{Kip-ft}$$

Wearing Surface

DW=0

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects, which are displayed in Table H.8-1, are calculated with axle loads of Illinois design, legal and routine permit trucks. Dynamic load allowance is not considered for timber components.

Table H.8-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	9.00
Legal	IL-PS2-21	7.03
	IL-PS3-31	7.20
	IL-PS4-34.75	6.19
	IL-PS4-28	5.06
	IL-PS5-36	4.78
	IL-PS6-35.75	5.06
	IL-PS7-39.75	5.06
	IL-PC3-31	6.19
	IL-PC4-41	7.09
	IL-PC5-41	6.30
Routine Permit	IL-RS3-34	7.59
	IL-RS4-38	6.19
	IL-RS5-50	6.47
	IL-RS6-60	6.75

Equivalent Lane Width

Equivalent Strip Width:

$E_s = 4.0\text{h} + 40$

h= Thickness of Deck = 3.0 in

$E_s = 4.0 \times 3.0 + 40 = 52 \text{ in}$

Distributed Live Load Effects

The calculated axle loads are converted over transverse equivalent strip width. In this conversion, multiple presence factor for one-lane loading 1.2 is included.

Table H.8-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	2.49
Legal	IL-PS2-21	1.95
	IL-PS3-31	1.99
	IL-PS4-34.75	1.71
	IL-PS4-28	1.40
	IL-PS5-36	1.32
	IL-PS6-35.75	1.40
	IL-PS7-39.75	1.40
	IL-PC3-31	1.71
	IL-PC4-41	1.96
	IL-PC5-41	1.74
Routine Permit	IL-RS3-34	2.10
	IL-RS4-38	1.71
	IL-RS5-50	1.79
	IL-RS6-60	1.87

Nominal resistance:

Section Properties for Stringer

$$S_x = \frac{bd^2}{6} = \frac{12 \times 3.0^2}{6} = 18 \text{ in}^3$$

Design Values

$F_b = F_{bo} C_{KF} C_M \, (C_F \text{ or } C_V) C_{fu} C_i C_d C_\lambda$

Reference Design Value, F_{bo}:

$F_{bo} = 1.0 \text{ Ksi}$

Format Conversion Factor, C_{KF}:

$$C_{KF} = \frac{2.5}{\emptyset} = \frac{2.5}{0.85} = 2.94$$

Size Effect Factor for Sawn Lumber, C_F:

b=12 in, d=3 in

C_F = 1.0

Wet Service Factor, C_M:

$$F_{bo}C_F = 1.0 \times 1.0 = 1.0 \text{ Ksi} \leq 1.15 \text{ Ksi, and } d = 3 \text{ in} \leq 4 \text{ in}$$

$$C_M = 1.0$$

Flat Use Factor, C_{fu}:

C_{fu} = 1.0

Incising Factor (only apply to dimension lumber), C_i:

C_i = 0.8

Deck Factor, C_d:

C_d = 1.0

$C_\lambda = \begin{matrix} 0.8 & \text{Time Effect Factor for Strength I} \\ 1.0 & \text{Time Effect Factor for Strength II} \end{matrix}$

$F_b = 1.0 \times 2.94 \times 1.0 \times 1.0 \times 1.0 \times 0.8 \times 1.0 \times 0.8 = 1.88$ Ksi   For Strength I

$F_b = 1.0 \times 2.94 \times 1.0 \times 1.0 \times 1.0 \times 0.8 \times 1.0 \times 1.0 = 2.35$ Ksi   For Strength II

Adjusted Design Value = F_b = 1.88 Ksi   for Design and Legal Load Rating

Adjusted Design Value = F_b = 2.35 Ksi   for Permit Load Rating

Nominal Resistance = $R_n = F_b S C_L$

C_L = 1.0

For Design Load Rating :

$$R_n = 1.88 \times 18 \times \frac{1}{12} = 2.82 \text{ Kip} - \text{ft}$$

For Legal Load Rating :

$$R_n = 1.88 \times 18 \times \frac{1}{12} = 2.82 \text{ Kip} - \text{ft}$$

For Permit Load Rating :

$$R_n = 2.35 \times 18 \times \frac{1}{12} = 3.53 \text{ Kip} - \text{ft}$$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.85, for Wood Component in Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Slab Bridge

Dead Load Factor, γ_{DC} = 1.25

The live load factors are tabulated in Table H.8-3.

Table H.8-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.8-4. Based on the calculation, RF for legal load rating is less than 1 when the current live load factor is used. Therefore, permit load rating is not applicable.

Table H.8-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.55 0.71	0.55 0.71
Legal	IL-PS2-21	0.94	1.04
	IL-PS3-31	0.92	1.01
	IL-PS4-34.75	1.07	1.18
	IL-PS4-28	1.31	1.44
	IL-PS5-36	1.39	1.52
	IL-PS6-35.75	1.31	1.44
	IL-PS7-39.75	1.31	1.44
	IL-PC3-31	1.07	1.18
	IL-PC4-41	0.93	1.03
	IL-PC5-41	1.05	1.16
Routine Permit	IL-RS3-34	N/A	1.29
	IL-RS4-38	N/A	1.58
	IL-RS5-50	N/A	1.51
	IL-RS6-60	N/A	1.45

Load Rating for a Floor Beam:

Dead Load Analysis:

The floor beam is modeled as a simple beam supported by truss members with span length of 17.2 ft. The dead loads are uniformly distributed on the beam.

Components and Attachments, DC

$$\text{Deck} = 15 \times \frac{3}{12} \times 0.05 = 0.19 \text{ Kip/ft}$$

$$\text{Sleeper} = \frac{4 \times 6}{144} \times 15 \times 0.05 \times \frac{8 \text{ Sleepers}}{17.2} = 0.058 \text{ Kip/ft}$$

$$\text{Stringer} = 0.01875 \ \times 15 \times \frac{8 \text{ Stringers}}{17.2} = 0.13 \text{ Kip/ft}$$

$$M_{DC} = \frac{1}{8} \times (0.19 + 0.058 + 0.13) \times 17.2^2 = 13.98 \text{ Kip} - \text{ft}$$

Wearing Surface

DW=0

Live Load Effects for Floor Beams

Spacing of Floor Beams: 15 ft

Figure H.8-2 shows the critical live load position for reactions on an intermediate floor beam.

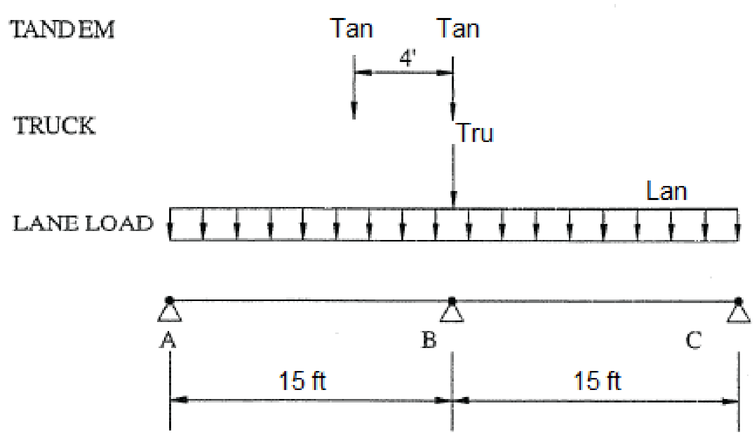

Figure H.8-2 Critical Live Load Position for Reactions on Floor Beam

The floor beams are modeled as hinges to support deck.

Reaction at Floor-beam B:

IM = 33%

Truck + Lane

$$R_{Tru+Lan} = \text{Tru} \times 1.33 + \text{Lan} \times 15$$

Tandem + Lane

$$R_{Tan+Lan} = \left(\text{Tan} + \text{Tan} \times \frac{15-4}{15}\right) \times 1.33 + \text{Lan} \times 15$$

$$R_{\text{Wheel}} = \frac{R_{\text{Tan or Tru}}}{2} = P$$

$$R_{\text{Lane per foot width}} = \frac{R_{\text{Lan}}}{10} = w$$

For Illinois Design Load (HL-93):

Tan = 25 Kips

Tru = 32 Kips

Lan = 0.64 Kip/ft

$$R_{\text{Tru+Lan}} = 32 \times 1.33 + 0.64 \times 15 = 52.16 \text{ Kips}$$

$$R_{\text{Tan+Lan}} = \left(25 + 25 \times \frac{15-4}{15}\right) \times 1.33 + 0.64 \times 15 = 67.23 \text{ Kips}$$

Tandem Load Governs.

For Illinois Legal and Permit Loads, the maximum axle loads of the model trucks "Tru", along with 33% dynamic load allowance are used for the calculation.

The reactions on the floor beams "R" are converted into wheel loads "P" and lane load per foot width "w". Tables H.8-5 and H.8-6.

Table H.8-5 Wheel Loads P

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	28.82
Legal	IL-PS2-21	16.63
	IL-PS3-31	17.02
	IL-PS4-34.75	14.63
	IL-PS4-28	11.97
	IL-PS5-36	11.31
	IL-PS6-35.75	11.97
	IL-PS7-39.75	11.97
	IL-PC3-31	14.63
	IL-PC4-41	16.76
	IL-PC5-41	14.90

Routine Permit	IL-RS3-34	17.96
	IL-RS4-38	14.63
	IL-RS5-50	15.30
	IL-RS6-60	15.96

Table H.8-6 Lane Loads w

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	0.96
Legal	IL-PS2-21	N/A
	IL-PS3-31	N/A
	IL-PS4-34.75	N/A
	IL-PS4-28	N/A
	IL-PS5-36	N/A
	IL-PS6-35.75	N/A
	IL-PS7-39.75	N/A
	IL-PC3-31	N/A
	IL-PC4-41	N/A
	IL-PC5-41	N/A
Routine Permit	IL-RS3-34	N/A
	IL-RS4-38	N/A
	IL-RS5-50	N/A
	IL-RS6-60	N/A

Maximum load effects due to wheel and lane loads:

Figure H.8-3 illustrates the critical positions of one design lane to produce the maximum mid-span moment in the floor beam. Tables H.8-7 displayed the maximum load effects due to the Illinois design, legal and permit loads. The Multiple presence factor 1.2 is included in the calculation.

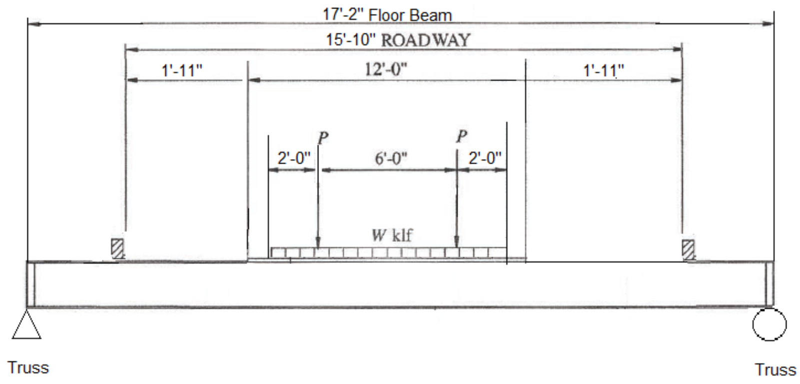

Figure H.8-3 Critical Lane Position for Maximum Moment in the Floor Beam

Table H.8-3 Live Load Effects (LL) for a Floor Beam

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	228.78
Legal	IL-PS2-21	111.72
	IL-PS3-31	114.40
	IL-PS4-34.75	98.31
	IL-PS4-28	80.44
	IL-PS5-36	75.97
	IL-PS6-35.75	80.44
	IL-PS7-39.75	80.44
	IL-PC3-31	98.31
	IL-PC4-41	112.61
	IL-PC5-41	100.10
Routine Permit	IL-RS3-34	120.66
	IL-RS4-38	98.31
	IL-RS5-50	102.78
	IL-RS6-60	107.25

Nominal resistance:

Floor Beam

For S20x65.4:

$D = 20.0$ in

$t_w = 0.505$ in

$b_f = 8.26$ in

$t_f = 0.795$ in

$I_z = 1190\ in^4$

$S_z = 119\ in^3$

$D_w = D - 2t_f = 20.0 - 2 \times 0.795 = 18.41$ in

$$D_c = D_t = \frac{D_w}{2} = \frac{18.41}{2} = 9.21 \text{ in}$$

Web Slenderness Limit

$$\frac{2D_c}{t_w} = \frac{2 \times 9.21}{0.505} = 36.48 < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7 \times \sqrt{\frac{29000}{30}} = 177.22 \quad \text{OK}$$

$$\frac{I_{yc}}{I_{yt}} = 1.0 > 0.3 \quad \text{OK}$$

Location of Plastic Neutral Axis (PNA)

$$\overline{Y} = \frac{D_w}{2} = \frac{18.41}{2} = 9.21 \text{ in,}$$

from bottom of the top flange to PNA.

Top and Bottom Flanges

$$P_c = P_t = F_y b_f t_f = 30 \times 8.26 \times 0.795 = 197.00 \text{ Kips}$$

$$d_t = d_c = \frac{(t_f + D_w)}{2} = \frac{(0.795 + 18.41)}{2} = 9.60 \text{ in}$$

Web

$$P_w = F_y D_w t_w = 30 \times 18.41 \times 0.505 = 278.91 \text{ Kips}$$

Plastic Moment

$$M_p = \frac{P_w}{2D_w}[\bar{Y}^2 + (D_w - \bar{Y})^2] + P_c d_c + P_t d_t$$

$$M_p = \{\frac{278.91}{2 \times 18.41}[9.21^2 + (18.41 - 9.21)^2] + 2 \times 197.00 \times 9.60\} \times \frac{1}{12}$$

$$M_p = 422.17 \text{ Kip} - \text{ft}$$

Web Compactness

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

$$\frac{2D_{cp}}{t_w} = \frac{2 \times 9.21}{0.505} = 36.48$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{(0.54\frac{M_p}{R_h M_y} - 0.09)^2} \leq \lambda_{rw}\frac{D_{cp}}{D_c}$$

where:

$$\lambda_{rw}\frac{D_{cp}}{D_c} = 5.7\sqrt{\frac{E}{F_{yc}}}\left(\frac{D_{cp}}{D_c}\right) = 5.7 \times \sqrt{\frac{29000}{30}} \times (1) = 177.22$$

$$R_h = 1$$

$$M_y = F_y S_z = 30 \times 119 \times \frac{1}{12} = 297.50 \text{ Kip} - \text{ft}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{30}}}{(0.54\frac{422.17}{1 \times 297.50} - 0.09)^2} = 67.98$$

Use 67.98

$$\frac{2D_{cp}}{t_w} = 36.48 \leq \lambda_{pw(D_{cp})} = 67.98 \quad \text{OK}$$

The section is compact.

Web Plastification Factor, R_{pc}:

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{422.17}{297.50} = 1.42$$

Nominal Resistance for Floor Beams:

Sections are considered as continuously being braced at compression flanges.

$$R_n = \varphi_f R_{pc} M_{yc} = \varphi_f \frac{M_p}{M_{yc}} M_{yc} = \varphi_f M_p = 1 \times 422.17 = 422.17 \text{ Kip} - \text{ft}$$

$\varphi = 1.0$, for flexure in steel beams.

$\varphi R_n = 1 \times 422.17 = 422.17 \text{ Kip} - \text{ft}$

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 0.9, for Two-Truss Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.8-8.

Table H.8-8 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors are calculated and displayed in Tables H.8-9.

Table H.8-9 Rating Factors (RF) for Floor Beams

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.91 1.17	0.91 1.17
Legal	IL-PS2-21	2.49	2.75
	IL-PS3-31	2.44	2.68
	IL-PS4-34.75	2.83	3.13
	IL-PS4-28	3.47	3.81
	IL-PS5-36	3.67	4.04
	IL-PS6-35.75	3.47	3.81
	IL-PS7-39.75	3.47	3.81
	IL-PC3-31	2.83	3.13
	IL-PC4-41	2.47	2.72
	IL-PC5-41	2.79	3.07
Routine Permit	IL-RS3-34	2.50	2.73
	IL-RS4-38	3.07	3.35
	IL-RS5-50	2.94	3.21
	IL-RS6-60	2.82	3.07

H.9 Reinforced Concrete Slab Bridge

Bridge Data:

Span: 23.5 ft

Year Built: 1939

Material:

Concrete: $f'_c = 2.5$ Ksi

Main Reinforcement: 1 in. Diameter @ 6 in. CC

$A_s = 0.79$ in^2

$f_y = 36$ Ksi

Condition: Good

Traffic: Multiple Lanes

ADTT (one direction): 100

Skew: 0°

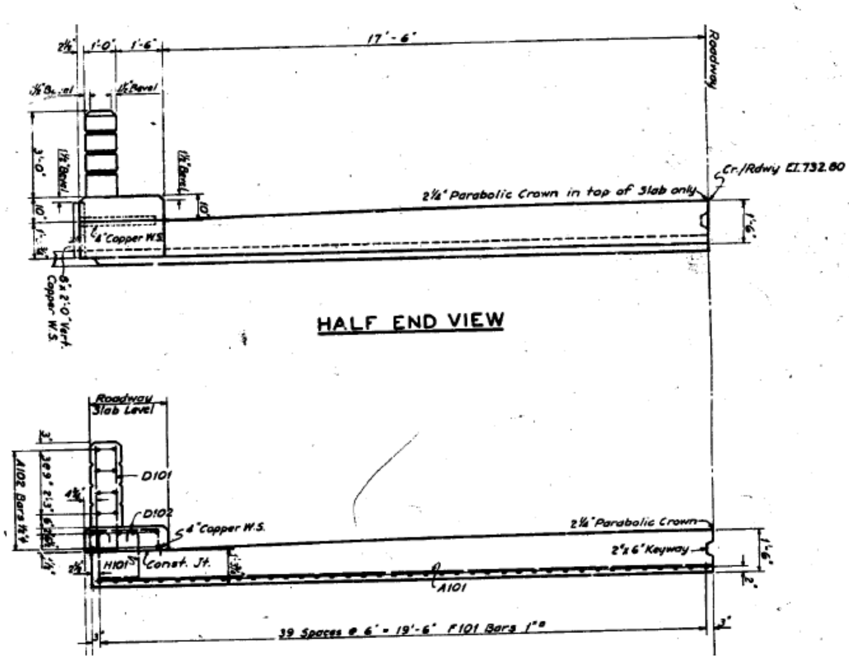

Figure H.9-1 Cross Section of Reinforced Concrete Slab Bridge

Width of Slab: 40 ft

Depth of Slab: 1 ft 6 in = 1.5 ft

Width of Clear Roadway: 35 ft

Depth of Overlay: 2.25 in

Width of Parapet: 1 ft

Depth of Parapet: 3 ft

Width of Curb: 2 ft 6 in = 2.5 ft

Depth of Curb: 10 in = 0.83 ft

Dead Load Analysis: (Unit Width)

Components and Attachments, DC

Concrete Slab:

$$1.0 \times 1.5 \times 0.15 = 0.23\ \mathrm{Kip/ft}$$

Parapet and Curb:

$$\frac{(1 \times 3 + 2.5 \times 0.83) \times 0.15 \times 2}{40} = 0.038 \text{ Kip/ft}$$

$$\text{DC} = 0.23 + 0.038 = 0.27 \text{ Kip/ft}$$

$$\text{M}_{\text{DC}} = 0.27 \times \frac{23.5^2}{8} = 18.64 \text{ Kip} - \text{ft}$$

Wearing Surface

$$\text{DW} = 1 \times \frac{2.25}{12} \times 0.14 = 0.026 \text{ Kip/ft}$$

$$\text{M}_{\text{DW}} = 0.026 \times \frac{23.5^2}{8} = 1.79 \text{ Kip} - \text{ft}$$

Live Load Analysis:

Undistributed Live Load Effects

The undistributed live load effects caused by the Illinois design, legal and permit live loads are calculated and showed in Table H.9-1. Dynamic load allowance, IM of 33% is included in the calculation.

Table H.9-1 Undistributed Live Load Effects

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	376.68
Legal	IL-PS2-21	289.94
	IL-PS3-31	397.94
	IL-PS4-34.75	388.36
	IL-PS4-28	281.96
	IL-PS5-36	233.42
	IL-PS6-35.75	324.52
	IL-PS7-39.75	324.52
	IL-PC3-31	202.16
	IL-PC4-41	313.88
	IL-PC5-41	314.68
Routine Permit	IL-RS3-34	377.72
	IL-RS4-38	305.90
	IL-RS5-50	316.54
	IL-RS6-60	372.40

Equivalent Lane Width

Equivalent Lane Width for Single Lane Loaded:

$E_s = 10 + 5\sqrt{L_1 W_1}$

L_1 = Span = 23.5 ft < 60 ft: L_1 = Span = 23.5 ft

W_1 = Lesser of Slab Width or 30 ft.

Slab Width = 40 ft > 30 ft: W_1 = 30 ft

$E_s = 10 + 5\sqrt{23.5 \times 30} = 142.76 \text{ in} = 11.90 \text{ ft}$

Equivalent Lane Width for More Than One Lane Loaded:

$$E_m = 84 + 1.44\sqrt{L_1 W_1} < 12\frac{W}{N_L}$$

L_1 = Span = 23.5 ft < 60 ft: L_1 = Span = 23.5 ft

W_1 = Lesser of Slab Width or 60 ft.

Slab Width = 40 ft < 60 ft: W_1 = 40 ft

$E_m = 84 + 1.44\sqrt{23.5 \times 40} = 128.15 \text{ in} = 10.68 \text{ ft}$

$$N_L = \frac{40}{12} = 3 \text{ Design Lanes}$$

$$12\frac{W}{N_L} = 12 \times \frac{40}{3} = 160 \text{ in} > E_m = 128.15 \text{ in} \quad \text{OK}$$

Use E_m = 10.68 ft

Distributed Live Load Effects

The distributed live load effects are calculated and displayed in Table H.9-2.

Table H.9-2 Distributed Live Load Effects (LL)

Load Rating	Live Loads	Live Load Effects (Kip-ft)
Design	HL-93	35.27
Legal	IL-PS2-21	27.15
	IL-PS3-31	37.26
	IL-PS4-34.75	36.36
	IL-PS4-28	26.40
	IL-PS5-36	21.86
	IL-PS6-35.75	30.39
	IL-PS7-39.75	30.39
	IL-PC3-31	18.93

	IL-PC4-41	29.39
	IL-PC5-41	29.46
Routine Permit	IL-RS3-34	35.37
	IL-RS4-38	28.64
	IL-RS5-50	29.64
	IL-RS6-60	34.87

Nominal resistance:

$$c = \frac{A_s\, f_y}{0.85 f_c' \beta_1 b}$$

1 in. Diameter @ 6 in..

$$A_s = 0.79 \times 2 = 1.58\ \text{in}^2/\text{ft}$$

$$f_y = 36\ \text{Ksi}$$

$$f_c' = 2.5\ \text{Ksi}$$

β_1 = 0.85

b = be = 12 in, Rectangular Section Behavior Assumed

$$c = \frac{1.58\ \times\ 36}{0.85 \times 2.5 \times 0.85 \times 12} = 2.62\ \text{in}$$

$$a = \beta_1 c = 0.85 \times 2.62 = 2.23\ \text{in}$$

a < Slab Thickness = 18 in, the assumption of the rectangular section behavior is valid.

$d_s = \text{Slab Thickness} - 2 = 18 - 2 = 16\ \text{in}$, Distance from extreme compression fiber to C.G. of steel

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) = \left[1.58 \times 36 \times \left(16 - \frac{2.23}{2}\right)\right] \times \frac{1}{12} = 70.55\ \text{Kip} - \text{ft}$$

Minimum Reinforcement

Amount of reinforcement must be sufficient to develop Mr equal to the lesser of 1.2 Mcr or 1.33 Mu.

$$M_r = \varphi M_n = 0.9 \times 70.55 = 63.50\ \text{Kip} - \text{ft}$$

1.33Mu :

$$1.33M_u = 1.33(1.75M_{HL-93} + 1.25M_{DC} + 1.50M_{DW})$$

$$1.33M_u = 1.33 \times (1.75 \times 35.27 + 1.25 \times 18.64 + 1.50 \times 1.79) = 116.65\ \text{Kip} - \text{ft}$$

1.2Mcr :

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$

A non-composite section is designed to resist all the loads, S_{nc} is substituted for Sc. In this case, f_{cpe} = 0.

$$M_{cr} = S_{nc} f_r$$

$$S_{nc} = \frac{I}{y_t}$$

Moment of Inertia of Uncracked Section (Neglecting Reinforcement Steel)

$$I = \frac{1}{12} \times 12 \times 18^3 = 5832 \text{ in}^4$$

Distance from the neutral axis of the uncracked section to the extreme tension fiber

$$y_t = \frac{18}{2} = 9 \text{ in}$$

$$S_{nc} = \frac{I}{y_t} = \frac{5832}{9} = 648 \text{ in}^3$$

$$f_r = 0.37\sqrt{f_c'} = 0.37 \times \sqrt{2.5} = 0.59 \text{ Ksi}$$

$$M_{cr} = S_{nc} f_r = \frac{1}{12} \times 648 \times 0.59 = 31.86 \text{ Kip} - \text{ft}$$

$$1.2M_{cr} = 1.2 \times 31.86 = 38.23 \text{ Kip} - \text{ft} < M_r = 63.50 \text{ Kip} - \text{ft}$$

The section meets the requirements for minimum reinforcement.

Maximum Reinforcement

Net Tensile Strain :

$$\varepsilon_t = \frac{(d - c)\varepsilon_c}{c}$$

$$\varepsilon_c = 0.003$$

$$d = d_s = 16 \text{ in}$$

$$\varepsilon_t = \frac{(16 - 2.62) \times 0.003}{2.62} = 0.015 > 0.005$$

The section is tension controlled and the Resistance factor φ shall be taken as 0.9.

General Load-Rating Equation:

$$RF = \frac{\varphi_c \varphi_s \varphi R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL)}$$

Evaluation Factors

Resistance Factor, φ = 0.9, for Reinforced Concrete Slab in Flexure

Condition Factor, φ_C = 1.0, for Good Condition

System Factor, φ_S = 1.0, for Slab Bridge

Dead Load Factor, γ_{DC} = 1.25

Wearing Surface Factor, γ_{DW} = 1.50

The used live load factors are displayed in Table H.9-3.

Table H.9-3 Live Load Factors (γ_L)

Load Rating	MBE Live Load Factors	Recommended Live Load Factors
Design	1.75 at Inventory Level 1.35 at Operating Level	1.75 at Inventory Level 1.35 at Operating Level
Legal	1.30	1.18
Routine Permit	1.20	1.10

Rating Factors:

The rating factors (RF) calculated according to the general equation are displayed in Table H.9-4. Based on the calculation, several RFs for legal load rating are less than 1. Therefore, permit load rating is not applicable.

Table H.9-4 Rating Factors (RF)

Load Rating	Live Loads	Using MBE Live Load Factors	Using Recommended Live Load Factors
Design	HL-93	0.61 0.79	0.61 0.79
Legal	IL-PS2-21	1.07	1.31
	IL-PS3-31	0.78	0.96
	IL-PS4-34.75	0.80	0.97
	IL-PS4-28	1.09	1.34
	IL-PS5-36	1.32	1.62
	IL-PS6-35.75	0.95	1.16
	IL-PS7-39.75	0.95	1.16
	IL-PC3-31	1.53	1.87
	IL-PC4-41	0.98	1.21
	IL-PC5-41	0.98	1.21
Routine Permit	IL-RS3-34	N/A	N/A
	IL-RS4-38	N/A	N/A

IL-RS5-50	N/A	N/A
IL-RS6-60	N/A	N/A

APPENDIX I: LIST OF EQUATIONS

$$\frac{\gamma_L LE_n}{\overline{\overline{LE}}} = \frac{\gamma_{L,ref} LE_{n,ref}}{\overline{\overline{LE}}_{ref}}$$ Eq.(3-1): Concept of calibration to maintain level of structural safety.

$$\gamma_L = \frac{\gamma_{L,ref} LE_{n,ref}}{\overline{\overline{LE}}_{ref}} \frac{\overline{\overline{LE}}}{LE_n} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{\overline{LE}}}{\overline{\overline{LE}}_{ref}}$$

Eq.(3-2): An alternative form of Equation (3-1) to find live load factor.

$$\mu_N = \mu_1 + \frac{Ln(N)}{\pi}\sqrt{6}\sigma_1$$

$$\sigma_N = \sigma_1$$

Eqs.(3-3) and (3-4): To find respectively the mean and standard deviation of N-period-future maximum-load-effects.

$$H_{one-lane} = (TimeStamp_2 - TimeStamp_1)(\frac{Speed_2 + Speed_1}{2}) - WheelBase_1$$

Eq.(4.5.2-1): To find spacing of two trucks in the same lane (between front truck's last axle to rear truck's first axle).

$P_{following} = 3.6 \times 10^{-6} ADTT_{SL} + 0.0021$

Eq.(4.5.2-2): Regression for probability of two trucks in one lane and less than 100ft apart to ADTT in that lane (compared with assumed 2%).

$$H_{two-lane} = (TimeStamp_2 - TimeStamp_1)(\frac{Speed_2 + Speed_1}{2})$$

Eq.(4.5.3-1): To find headway of two trucks in different lanes (between the steering axles of the two trucks).

$P_{side-by-side} = 3.56 \times 10^{-6} ADTT + 0.0024$

Eq.(4.5.3-2): Regression for probability of two trucks in different lanes and less than 50ft apart to total ADTT (compared with assumed 6.67%).

$$P_L = \frac{N_{LongitudinalMultiplePresence}}{N_{AllTrucks}} \tag{5-1}$$

Eq.(5-1): Definition for probability of longitudinal multiple-truck-presence on a bridge span.

$$P_T = \frac{N_{TransverseMultiplePresence}}{N_{AllTrucks}} \tag{5-2}$$

Eq.(5-2): Definition for probability of transverse multiple-truck-presence on a bridge span.

Headway Distance < Span Length + First Truck's Wheel Base

Eq.(5-3): Definition for a cluster of trucks on a bridge span.

$P_{cluster}$ = 1.32x10^{-5} ADTT$_{SL}$ + 1.86x10^{-4} Span Length – 0.016

Eq.(5-4): WIM-data-based regression relation of cluster probability on a bridge span to ADTT and span length.

$$R = \frac{\overline{LE}}{\overline{LE}_{ref}}$$

Eq.(5-5): Definition of the ratio between two mean values in the calibration Eqs.(3-1) and (3-2), numerator for the case of interest and denominator for the reference case.

$$\gamma_{L,inventory-rating} = \gamma_{L,ref}\frac{LE_{n,ref}}{LE_n}\frac{\overline{LE}}{\overline{LE}_{ref}} = 1.75 \times \frac{LE_{HL93}}{LE_{HL93}} \times \frac{\overline{LE}_{uptoFBF,5-year-projected}}{\overline{LE}_{uptoFBF,5-year-projected}}$$

Eq.(6.1-1): Calibration equation for inventory rating of design load rating.

$$\gamma_{L,operating-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{\overline{LE}}_{ref}} = 1.35 \times \frac{LE_{HL93}}{LE_{HL93}} \times \frac{\overline{LE}_{uptoFBF,5-year-projected}}{\overline{\overline{LE}}_{uptoFBF,5-year-projected}}$$

Eq.(6.1-2): Calibration equation for operating rating of design load rating.

$$\gamma_{L,ref} = \begin{cases} 1.45 & for\ ADTT unknown \\ 1.45 & for\ ADTT \geq 5,000 \\ 1.30 & for\ ADTT \leq 1,000 \end{cases}$$

Eq.(6.2-1): AASHTO BME live load factor for legal load rating.

$$\gamma_{L,legal-load-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{\overline{LE}}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-posting-trucks}} \times \frac{OneLaneLoad's\overline{LE}_{uptoIL-posting-trucks,5-year-projected}}{OneLaneLoad's\overline{\overline{LE}}_{uptoNRL,5-year-projected}}$$

Eq.(6.2-2): Calibration equation for legal load rating for one-lane loading.

$$\gamma_{L,legal-load-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{\overline{LE}}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-posting-trucks}} \times \frac{TwoLaneLoad's\overline{LE}_{uptoIL-posting-trucks,5-year-projected}}{TwoLaneLoad's\overline{\overline{LE}}_{uptoNRL,5-year-projected}}$$

Eq.(6.2-3): Calibration equation for legal load rating for two-lane loading.

$$\gamma_{L,legal-load-rating} = \begin{cases} 1.34 & for\ ADTT > 6,500 \quad or\ unknown \\ 3.25 * 10^{-5}\ ADTT + 1.13 & for\ 1,500 \leq ADTT \leq 6,500 \\ 1.18 & for\ ADTT < 1,500 \end{cases}$$

Eq.(6.2-4): Recommended live load factor for legal load rating in Illinois.

$$\gamma_{L,routine-permit-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{\overline{LE}}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-routine-permit-trucks}} \times \frac{OneLaneLoad's\overline{LE}_{uptoIL-IL-routine-permit-trucks,5-year-projected}}{OneLaneLoad's\overline{\overline{LE}}_{uptoNRL,5-year-projected}}$$

Eq.(6.3-1): Calibration equation for routine permit load rating for one-lane loading.

$$\gamma_{L,routine-permit-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-routine-permit-trucks}} \times \frac{TwoLaneLoad's\overline{LE}_{uptoIL-IL-routine-permit-trucks,5-year-projected}}{TwoLaneLoad's\overline{LE}_{uptoNRL,5-year-projected}}$$

Eq.(6.3-2): Calibration equation for routine permit load rating for two-lane loading.

$$\gamma_{L,routine-permit-load-rating} = \begin{cases} 1.26 & for\ ADTT > 6,500 \quad or\ unknown \\ 3.24 * 10^{-5}\ ADTT + 1.05 & for\ 1,500 \le ADTT \le 6,500 \\ 1.10 & for\ ADTT < 1,500 \end{cases}$$

Eq.(6.3-3): Recommended live load factor for routine permit load rating in Illinois.

$$\gamma_{L,special-permit-rating} = \gamma_{L,ref} \frac{LE_{n,ref}}{LE_n} \frac{\overline{LE}}{\overline{LE}_{ref}}$$

$$= \gamma_{L,ref} \times \frac{LE_{NRL}}{LE_{IL-special-permit-truck}} \times \frac{OneLaneLoad's\overline{LE}_{uptoIL-IL-special-permit-truck,5-year-projected}}{OneLaneLoad's\overline{LE}_{uptoNRL,5-year-projected}}$$

Eq.(6.4-1): Calibration equation for special permit load rating for one-lane loading.

$$\gamma_{L,special-permit-load-rating} = \begin{cases} 1.18 & for\ ADTT > 6,500 \quad or\ unknown \\ 1.95 * 10^{-5}\ ADTT + 1.05 & for\ 1,500 \le ADTT \le 6,500 \\ 1.08 & for\ ADTT < 1,500 \end{cases}$$

Eq.(6.4-2): Recommended live load factor for special permit load rating in Illinois.

APPENDIX J: SEQUENCE OF ANALYSIS STEPS FOR CALIBRATION

1) Scrub WIM data.

2) Select a case of interest, for example, legal load rating.

3) Establish the calibration equation, such as Eqs.(6.2-2) and (6.2-3) for legal load rating.

4) Analyze a month of a site's WIM record, truck by truck on a specified span length, to identify the maximum load effect of moment and shear for both one-lane and two-lane loading cases for the month.

5) Repeat 4) until all available months of data have been exhausted for all interested span lengths.

6) Use the acquired monthly maximum load effects respectively for all considered spans, perform temporal projection to the 5-year future using Eqs.(3-3) and (3-4).

7) Use the calibration equation from Step 3) above with the data input from Step 6) to find the live load factor of interest for that site.

8) Repeat Step 7) until all sites have been exhausted.

9) Perform a regression analysis for all the resulting live load factors from Step 8).

10) Go to Step 2) for another case of interest, until all cases (legal load, routine permit, special permit) have been completed.

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