Recalibration of LRFR Live Load Factors in the AASHTO Manual for Bridge Evaluation

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ABSTRACT

The Object of this study is to use recent national WIM data to recalibrate the LRFR live load factors for Permit Load Ratings given in the AASHTO MBE and to adjust these live load factors based on the method of analysis used during the load rating process. This study has also taken into consideration current WIM data to verify that the AASHTO LRFR produce acceptable and uniform levels of reliability for typical U.S. bridges under current loading conditions. The target reliability index set for the permit load recalibration is β_{target}=2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50. A minimum live load factor of 1.10 was also maintained for all permit ratings. The protocols established in NCHRP Project 12-76 were adapted to the application here of bridge rating recalibration. This report presents the results of the reliability calibration of the permit live load factors for use in proposed revisions to the AASHTO LRFR. Permit load factors are provided for routine permits and special or trip permits for use with the AASHTO LRFD distribution factors and with refined methods of distribution analysis. These new load factors will result in improved permit load ratings in most cases when used in conjunction with the LRFR methodology.

EXECUTIVE SUMMARY

Recalibration of LRFR Live Load Factors in the AASHTO Manual for Bridge Evaluation

The Object of this NCHRP 20-07 Task 285 project is to use recent national WIM data to recalibrate the LRFR live load factors for permit load ratings and to adjust these live load factors based on the method of analysis used during the load rating process. This study also reviewed the reliability indices represented by the current LRFR legal load ratings, using recent WIM data. The first two tasks of this project developed and recommend a calibration approach and safety targets better aligned with current permit operations for routine and special permit calibrations. Tasks 3 and 4 of this project pertain to the recalibration of the LRFR permit load factors. These recalibrated LRFR permit live load factors allow for the use of either the LRFD distribution formulas or refined methods of analysis.

To achieve this goal, it was critical to use the most representative statistical information on truck weights, truck configurations, and multiple presence data. For the purposes of this study, data collected in each direction at six U.S. WIM sites as part of NCHRP project 12-76 was analyzed to obtain projections for the maximum bridge load effects. Multiple presence probabilities were assembled form a representative site in New York. The protocols established in NCHRP Project 12-76 were adapted to the application here of bridge rating recalibration.

The recalibration of the live load factors for permits considered the following four cases:

I. Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.
II. Unlimited crossings of multiple trip permits where two permit trucks could cross a bridge simultaneously side-by-side.
III. Unlimited crossings where a permit truck mixes with other random vehicles.
IV. Single Permit trips where the permit truck could mix with other random vehicles.

This Report presents the results of the reliability calibration of the permit live load factors for use in proposed revisions to the AASHTO LRFR specifications. The target reliability index set for the calibration is β_{target}=2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50. The minimum reliability index always governed the recalibration, which raised the average beta values to be mostly greater than 2.5. The reliability analysis performed in this report covered the bending and shear loading effects of bridges with simple span lengths having parallel T-beams, prestressed I-beams, composite and non-composite steel I-beams. Reliability index values, multiple presence probabilities and estimation of the load effect of the random trucks that will cross a bridge alongside a permit truck are calculated using the actual live load effects of the trucks recorded by WIM from several sites throughout the U.S.

Routine Permits: In order, to better reflect the load effects from the different truck types, the trucks are categorized based on a combination of their gross vehicle weights and their first to rear axle lengths. For the cases of Routine Permits mixed with random traffic in the adjacent lane, where data shows that some permit loads may exceed the permit weight limits, live load factors varying from γ_L=1.40 for sites with ADTT=5000, γ_L=1.35 for sites with ADTT=1000, and γ_L=1.30 for sites with ADTT=100 will increase the reliability index values so that the minimum

value remains above β=1.50. These factors are specifically recommended for trucks with Gross Vehicle Weight (GVW) divided by the front axle to rear axle length (AL) ratios (GVW/AL) < 2.0. The above live load factors for Routine Permits can be reduced for the cases where the Permit truck's Gross vehicle weight is high to reflect the lower probability of having a random truck of equal or higher weight crossing alongside the Permit truck as given in Table A. These checks should be performed with the two-lane AASHTO LRFD load distribution factors.

For trucks with GVW/AL between 2.0 and 3.0 the recommended live load factors are γ_L=1.35 for sites with ADTT=5000, γ_L=1.25 for sites with ADTT=1000, and γ_L=1.20 for sites with ADTT=100. For trucks with GVW/AL above 3.0, the live load factors γ_L=1.30 for sites with ADTT=5000, γ_L=1.20 for sites with ADTT=1000, and γ_L=1.15 for sites with ADTT=100. For the most heavy trucks with GVW/AL>3.0 the live load factors are reasonably similar to those in the current AASHTO LRFR. For the lower truck weight category, the live load factors are lower than those in the AASHTO LRFR.

Table A. Recommended Live Load Factors for Routine Permits Mixed with Random Traffic

Unlimited crossings of Routine Permits	AASHTO LRFD two or more lane D.F.	Truck Volume	GVW/AL<2.0 (kip/ft)	2.0<GVW/AL<3.0 (kip/ft)	GVW/AL>3.0 (kip/ft)
		ADTT=5000	γ_L=1.40	γ_L=1.35	γ_L=1.30
		ADTT=1000	γ_L=1.35	γ_L=1.25	γ_L=1.20
		ADTT=100	γ_L=1.30	γ_L=1.20	γ_L=1.15

Special Permits: The calculations performed in this report demonstrate that using live load factors γ_L=1.10 for escorted Special permit loads will provide average reliability index values greater than the target β_{target}=2.5 when the single lane AASHTO LRFD load distribution factors are used after removing the multiple presence factor MP=1.2.

For Special permits mixed with traffic, a live load factor γ_L=1.40 leads to an average conditional reliability index of $\beta_{average}$=3.00 with a minimum value of 1.54. Single lane AASHTO LRFD load distribution factors are used after removing the multiple presence factor MP=1.2. The reliability analysis was executed assuming that there are a total of 100 crossings of a Special permit over the rating period and that the permit will always cross the bridge alongside a random truck.

Refined Analysis: When performing a refined analysis of the bridge for special permits, it is recommended to use the same γ_L=1.10 for escorted Special Permits with no other vehicles on the bridge. Special permits travelling over bridges at crawl speed should still be checked with a dynamic allowance factor of 1.05 to satisfy the minimum value of β_{min}=1.50.

If the permit check with no traffic restriction is to be executed using a rigorous analysis, it is important for the rating engineer to know what live load factor to apply on the permit truck and what truck load and what live load factor to apply to a random truck in the adjacent lane.

For the case when a refined analysis is performed for special permits, a live load factor γ_P=1.0 is applied on the permit truck while a live load factor γ_L=1.10 is applied on the governing AASHTO legal truck placed in the adjacent lane. These load factors for Special Permit crossings are recommended for all ADTT values.

When routine permit checks are executed using a refined analysis, the load factors as given in Table A shall be increased by 0.10 and applied to the two permit trucks placed in adjacent lanes .

CHAPTER 1

BACKGROUND

PROBLEM STATEMENT AND RESEARCH OBJECTIVE

The calibration of the AASHTO LRFR live load factors was performed using a structural reliability framework based on the models and a generic database described in NCHRP Report 454 (Moses, 2001). The live load model is an essential part of the reliability calibration process. The AASHTO LRFR calibration made several assumptions on the truck weight spectra in order to produce a live load model that matches the one developed by Nowak (1999) for the calibration of the AASHTO LRFD specifications. The database used during the calibration of the AASHTO LRFD specifications was adopted from a truck survey conducted in the 1970's in Ontario Canada. Kulicki et al (2007) explain that the Ontario database was biased in the sense that only the trucks that appeared to be heavy were flagged for weighing. In his review of the Nowak (1999) results, Moses (2001) concluded that the Ontario data approximately represented the heaviest 20% of the trucks that crossed the highway at the survey site. By assuming that the average weight of these heavy trucks is 68 kips and the standard deviation is 18 kips and that the weight spectrum follows a Normal probability distribution, a good match between the AASHTO LRFR and LRFD load models was obtained.

Recognizing that the current truck loads in the U.S. may be different than the generic Ontario data that was originally used, the AASHTO LRFR provides enough flexibility to modify the LRFR live load factors to reflect current state-specific or site-specific loading conditions based on truck Weigh-In-Motion (WIM) data. However, the simplified approach proposed in the AASHTO LRFR for modifying the live load factors preserves several assumptions about the target reliability level and the shape of the truck weight spectra that may not be consistent with observed truck weight data collected from recent Weigh-In-Motion systems. It assumes that the heaviest trucks follow a Normal distribution and that 1 in 15 trucks will cross the bridge side–by–side. Studies performed in NCHRP 12-76 have shown that these simplifying assumptions may not be valid in all cases. NCHRP Project 12-76 has proposed a more consistent approach for using WIM data for live load modeling, which takes into consideration the actual distribution of the truck traffic data, including the actual configurations and the actual percentage of side-by-side crossings.

The estimation of the maximum load effect L_{max} expected over a 5-year bridge evaluation period can be executed through a variety of methods. The one implemented in this study is based on the assumption that the tail end of the histogram of the maximum load effect over a given return period approaches a Gumbel distribution as the return period increases (NCHRP 12-76). The method assumes that the WIM data is assembled over a sufficiently long period of time to ensure that the data is representative of the tail end of the truck weight histograms. Typically WIM data collected over a full year at a site is utilized.

Furthermore, the AASHTO LRFD and LRFR assume that the load distribution factors of the AASHTO LRFD tables provide on the average consistent results with the actual load distribution on typical highway bridge configurations and accordingly apply the same live load factors when the rating is performed using the tabulated load distributions, those obtained from advanced analyses, or those obtained from field measurements. Recent observations have however indicated that the AASHTO LRFD load distribution tables provide conservative envelopes to those obtained from advanced analyses or field measurements.

The Object of this NCHRP 20-07 Task 285 project is to use recent national WIM data to recalibrate the LRFR live load factors for permit load ratings and to adjust these live load factors based on the method of analysis used during the load rating process. This study also reviewed the

reliability indices represented by the LRFR legal load ratings, using recent WIM data. This report provides a review of the basic concepts of structural reliability, outlines the approach that will be followed to calibrate the live load factors, and uses recent Weigh-In-Motion data to develop the live loads that will be used during the course of this study.

PROJECT OBJECTIVES

1. Develop and recommend reliability indices better aligned with current permit operations for routine and special permit calibrations.
2. Recalibrate LRFR permit live load factors for the recommended reliability indices
3. Review the LRFR calibration for legal loads using current national WIM data.
4. Recalibration of LRFR live load factors for permits should allow for the use of either the LRFD distribution formulas or refined methods of analysis such as finite element analysis or measured field data.

The aim of the code calibration process is not to propose new and higher safety levels (unless warranted by the rate and frequency of bridge failures) but simply to harmonize the discrepancies in the safety levels observed for different bridge categories and classifications and loadings in past ratings so that future LRFR permit ratings would produce uniform safety levels across the board.

RESEARCH TASKS

The work is to be accomplished in accordance with the following tasks:

Task 1: *Investigate reliability indices inherent in current permit operations for Routine and Special permits.*

Task 2: *Recommend target reliability indices for LRFR Routine and Special permit calibrations*

Task 3: *Recalibrate LRFR permit live load factors for the target reliability indices suitable for use with LRFD distribution formulas*

Task 4: *Recalibration of LRFR permit live load factors for the target reliability indices suitable for use with refined methods of analysis such as finite element analysis.*

Task 5: *Prepare LRFR Specification and Commentary suitable for inclusion in the AASHTO Manual for Bridge Evaluation, Section 6. Prepare a final report.*

INTRODUCTION TO THE FINAL REPORT

This Final Report prepared in accordance with Task 5 requirements for this project, documents the findings of Tasks 1 through 5. It contains four chapters and one Appendix. Chapter 1 gives a review of the problem statement, the research objective and scope of study. Chapter 2 describes the basic concepts of structural reliability, bridge resistance models, analysis of representative Weigh-In-Motion data, and reliability analysis for legal loads and for permit loads.

Chapter 3 provides the current reliability levels for LFR legal load ratings and permit load ratings, target reliability indices for LRFR recalibrations, and recalibration of LRFR permit load factors. Chapter 4 contains the conclusions, recommended LRFR permit load factors, and recommendations for future research. The Appendix shows reliability plots for legal loads and permit loads.

CHAPTER 2

RESEARCH APPROACH

RESEARCH TASKS

The research effort was organized according to the following five tasks:

The work is to be accomplished in accordance with the following tasks:

Task 1: *Investigate reliability indices inherent in current permit operations for Routine and Special permits.*
Task 2: *Recommend target reliability indices for LRFR Routine and Special permit calibrations*
Task 3: *Recalibrate LRFR permit live load factors for the target reliability indices suitable for use with LRFD distribution formulas*
Task 4: *Recalibration of LRFR permit live load factors for the target reliability indices suitable for use with refined methods of analysis such as finite element analysis.*
Task 5: *Prepare LRFR Specification and Commentary suitable for inclusion in the AASHTO Manual for Bridge Evaluation, Section 6. Prepare a final report.*

BASIC CONCEPTS OF STRUCTURAL RELIABILITY

The aim of structural reliability theory is to account for the uncertainties encountered while evaluating the safety of structural systems or during the calibration of load and resistance factors for structural design and evaluation codes. To account for the uncertainties associated with predicting the load carrying capacity of a structure, the intensities of the loads expected to be applied, and the effects of these loads as well as the capacity of structural members may be represented by random variables.

The value that a random variable can take is described by a probability distribution function. That is, a random variable may take a specific value with a certain probability and the ensemble of these values and their probabilities are described by the probability distribution function. The most important characteristics of a random variable are its mean value or average, and the standard deviation that gives a measure of dispersion or a measure of the uncertainty in estimating the variable. The standard deviation of a random variable R with a mean $\overline{R}$ is normally represented by σ_R. A dimensionless measure of the uncertainty is the coefficient of variation (COV) which is the ratio of the standard deviation divided by the mean value. For example the COV of the random variable R is represented by V_R such that:

$$V_R = \frac{\sigma_R}{\overline{R}} \tag{1}$$

Codes often specify nominal or characteristic values for the variables that are used in design or load rating equations. These nominal values may be conservative or unconservative estimates of the true value of the variable and are related to the means through a bias. The bias is defined as the ratio of the mean to the nominal value used during the design or evaluation process. For example, if R is the member resistance, the mean of R, namely, $\overline{R}$ can be related to the nominal or design value, R_n, using a bias factor such that:

$$\overline{R} = b_r R_n \tag{2}$$

where: b_r is the resistance bias, and R_n is the nominal value as specified by the design code. For example, Grade A50 steel has a nominal design yield stress of 50 ksi but coupon tests show an actual average value close to 56 ksi. Hence the bias of the yield stress is 56/50 or 1.12.

In structural analysis, safety may be described as the situation where capacity (member strength or resistance) exceeds demand (applied load, moment, or stress). Probability of failure, i.e., probability that capacity is less than applied load effects, may be formally calculated; however, its accuracy depends upon detailed data on the probability distributions of load and resistance variables. Since such data are often not available, approximate models are often used for calculation.

Let the reserve margin of safety of a bridge component be defined as, Z, such that:

$$Z = R - S \tag{3}$$

Where R is the resistance or member capacity, S is the total load effect. Probability of failure, P_f, is the probability that the resistance R is less than or equal to the total applied load effect S or the probability that Z is less or equal to zero. This is symbolized by the equation:

$$P_f = \Pr [\, R \leq S \,] \tag{4}$$

Where Pr is used to symbolize the term probability. If R and S follow independent Normal (Gaussian) distributions, then the probability of failure can be obtained based on the mean of Z and its standard deviation which can be calculated from the mean of R and S and their standard deviations:

$$P_f = \Phi\left(\frac{0-\overline{Z}}{\sigma_Z}\right) = \Phi\left(-\frac{\overline{R}-\overline{S}}{\sqrt{\sigma_R^2+\sigma_S^2}}\right) \tag{5}$$

where Φ is the normal probability function that gives the probability that the normalized random variable is below a given value. $\overline{Z}$ is the mean safety margin and σ_Z is the standard deviation of the safety margin. Thus, Equation 5 gives the probability that Z is less than 0 (or R less than S). The reliability index, β, is defined such that:

$$P_f = \Phi(-\beta) \tag{6}$$

For example, if the reliability index is β=3.5, then the implied probability of failure is obtained from the Normal Distribution tables given in most books on statistics as P_f=2.326x10^{-4}. If β=2.5 then P_f=6.21x10^{-3}. A β=2.0 implies that P_f=2.23x10^{-2}. One should note that these P_f values are only notional measures of risk giving the probability by which the limit state represented by Equation 3 is exceeded but are not actuarial values corresponding to actual probability of bridge collapse. This is because the limit sates used in checking the safety of structures include many simplifying assumptions regarding the behavior of bridge systems. The basic equations usually check the safety of an individual member and ignore the system's effects. In addition, contributions from nonstructural members such as curbs barriers and other components to the stiffness and the strength of the system as well as unintended end fixities and other factors lead to a significant increase in the reserve margins of bridge structures that are not include in the analysis process. For these reasons, the original developers of structural reliability methods have

preferred to use the reliability index, β, as a comparative measure of risk and safety rather than using the probability of failure.

For the Normal distribution case and based on Eq. (5) and (6), the reliability index is obtained from:

$$\beta = \frac{\overline{Z}}{\sigma_Z} = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{7}$$

Thus, the reliability index, β, which is used as a measure of structural safety, gives in this instance the number of standard deviations that the mean margin of safety falls on the safe side as represented in Figure 1.

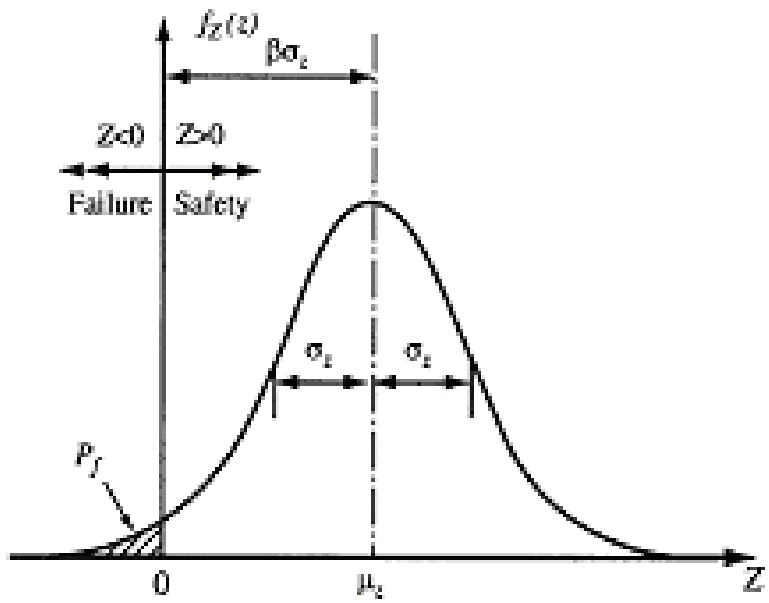

Figure 1. Graphical Representation of Reliability Index

The reliability index, β, defined in Equations 6 and 7 provides an exact evaluation of risk (failure probability) if R and S follow normal distributions. Although β was originally developed for normal distributions, similar calculations can be made if R and S are Lognormally distributed (i.e. when the logarithms of the basic variables follow normal distributions). In this case, the reliability index can be calculated as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\frac{\sqrt{1+V_S^2}}{\sqrt{1+V_R^2}}\right)}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_S^2\right)\right]}} \tag{8}$$

Which, for small values of V_R and V_S on the order of 20% or less can be approximated as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} \tag{9}$$

"Level II" methods have also been developed to obtain the reliability index for the cases when the basic variables are neither normal nor lognormal. Level II methods, often referred to as FORM (First Order Reliability Methods) or FOSM (First Order Second Moment) involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure equation (i.e. when Z=0) by a tangent multi-dimensional plane at the point on the failure surface closest to the mean value. For example, during the calibration of the AASHTO LRFD code, Nowak (1999) used the FORM algorithm developed by Rackwitz and Fiessler (1978) to calculate the reliability index values when R is assumed to follow a lognormal distribution and S is a normal random variable. More advanced techniques including SORM (Second Order Methods) have also been developed.

Alternatively, Monte Carlo simulations can be used to provide estimates of the probability of failure and the reliability index. Monte Carlo simulations are suitable for any random variable distribution type and failure equation. In essence, a Monte Carlo simulation creates a large number of "experiments" through the random generation of sets of resistance and load variables. Estimates of the probability of failure are obtained by comparing the number of experiments that produce failure to the total number of generated experiments. Given values of the probability of failure, P_f, the reliability index, β, is calculated from Equation 6 and used as a measure of structural safety even for non-normal distributions. Kulicki et al (2007) used the Monte Carlo simulation while reviewing the code calibration effort reported by Nowak (1999) and verified that the results of the FORM method with the Rackwitz-Fiessler algorithm and those of the Monte Carlo simulation are essentially similar. More detailed explanations of the principles discussed in this section can be found in published texts on structural reliability (e.g. Thoft-Christensen & Baker; 1982, Nowak & Collins; 2000, Melchers, 1999).

The reliability index has been used by many code writing groups throughout the world to express structural risk. Reliability index values, β, in the range of 2 to 4 are usually specified for different structural applications. For example, the calibration of the Strength I limit state in AASHTO LRFD Specifications aimed to achieve a uniform target reliability index β_{target}=3.5 for a range of typical bridge span lengths, beam spacing and materials (Nowak, 1999). A reliability index β_{target}=2.5 was used by Moses (2001) for the calibration of the Operating Rating in the AASHTO LRFR. These values usually correspond to the failure of a single component. If there is adequate redundancy, overall system reliability indices will be higher (Ghosn and Moses, 1986).

Reliability-based Code Calibration Approach

The reliability index β is seldom used in practice for making decisions regarding the safety of a new bridge design or an existing structure but it is rather used by code writing groups for recommending appropriate load and resistance safety factors for new structural design or evaluation specifications. One commonly used calibration approach is based on the principle that each type of structure should have uniform or consistent reliability levels over the full range of applications. For example, load and resistance factors should be chosen to produce similar β values for the shear and bending of steel and concrete bridges of different span lengths, number of lanes, number of beams and beam spacing, simple or continuous spans, and roadway categories. Thus, a single target β must be achieved for all applications. Some engineers and

researchers on the other hand are suggesting that higher values of β should be used for more important structures such as bridges with longer spans, bridges that carry more traffic, or bridges that, according to AASHTO, are classified as critical for "social/survival or security/defense requirements". Since higher β levels would require higher construction costs, the justification should be based on a cost-benefit analysis whereby target β values are chosen to provide a balance between cost and risk (Aktas, Moses and Ghosn, 2001). However, there is currently no consensus on how this can be achieved. Therefore, traditional and recent code calibration efforts have been based on the principle of maintaining uniform reliability levels leaving it up to the bridge owners to apply additional safety factors for special circumstances.

In most code calibration efforts, appropriate target β values are deduced based on the performance of a sample population of existing designs that have performed in a satisfactory manner. That is, if the safety performance of bridges designed according to current standards has generally been found satisfactory, then the average reliability index obtained from current designs is used as the target that the new code should satisfy. The aim of the calibration procedure is to minimize designs that deviate from the target reliability index.

The calibration based on past performance has been found to be robust in the sense that it minimizes the effects of any inadequacies in the database as reported by Ghosn & Moses (1986). Ghosn and Moses (1986) found that the load and resistance factors obtained following a calibration based on "safe existing designs" are relatively insensitive to errors in the statistical data base as long as the same statistical data and criteria used to find the target reliability index are also used to calculate the load and resistance factors for the new code. In fact, a change in the load and resistance statistical properties (e.g. in the coefficients of variation) would affect the computed β values for all the bridges in the selected sample population of existing bridges and consequently their average β value. Assuming that the performance history of these bridges is satisfactory, then the target reliability index would be changed to the new "average" and the calibrated load and resistance factors that would be used for new designs would remain approximately the same.

The calibration of resistance and live load factors for a new bridge code is usually executed by code writing groups as follows:

- A representative sample of bridges that have been designed to efficiently satisfy existing codes and that have shown good safety record is assembled.
- Reliability indices are calculated for each bridge of the representative sample. The calculation is based on statistical information about the randomness of the strength of members, the statistics of load intensities, and their effects on the structures.
- In general, there will be considerable scatter in such computed reliability indices. A target β is selected to correspond to the average reliability index of the representative bridge sample.
- For the development of the new code, load and resistance factors as well as nominal loads are selected by trial and error to satisfy the target β as closely as possible for the whole range of applications.

Resistance Modeling

To execute the calculation of the reliability index, one needs to obtain the statistical data for all the random variables that affect the safety margin Z of Equation 3 including all the uncertainties in estimating the variables that describe the member resistances and the load effects. Experimental and simulation studies have developed statistical estimates of member resistances for different types of bridge structural members. These models have accounted for the variability

and uncertainties in estimating the material properties; modeling errors; differences between predicted member capacities and measured capacities; human error and construction control. For example, Nowak (1999) followed the approach of Ellingwood et al. (1980) and represented a bridge member resistance capacity by a variable R that is the product of several underlying parameters, such that:

$$R = M\ F\ P\ R_n \tag{10}$$

Where M= material factor representing the variability in properties such as strength, modulus of elasticity, capacity to resist cracking, and chemical composition; F= fabrication factor representing the variability in geometric properties including dimensions, moments of inertia, and section modulus; P= analysis factor representing the variations of the actual strength when compared to the approximate code specified models for estimating member capacity; R_n= predicted member capacity using code specified methods. Equation (10) can be used to find the mean of R using Equation (2) if the total resistance bias, b_r, is set to be equal to the product of the mean values of the parameters M, F and P.

Note that the resistance model of Equation (10) does not account for member deterioration or other changes with time. Thus, all the resistance variables are usually assumed to be time-independent random variables. Research studies on modeling member deterioration have been ongoing for a number of years. However, these have not been implemented in code calibration practice pending more studies to verify the validity of these models when compared to actual behavior of bridge members over time.

Live Load Modeling

For a bridge member (or structural system) to be safe, the resistance should be large enough to withstand the maximum load effect that could occur within the structure's service life. Estimating the effects of the maximum loads involves a number of random variables, which are often associated with large levels of modeling uncertainties. The permanent loads of a bridge are constant over time and studies have been conducted to compare field verified as-built permanent loads to the nominal dead loads estimated during the bridge design or load capacity evaluation processes. Accordingly, the dead load is usually represented as a random variable that follows a normal distribution having a mean value and a standard deviation that can be expressed using a bias and COV that can be presented in a similar manner to that used for the resistance as shown in Eq. (1) and (2).

On the other hand, the intensities of the maximum live loads are time-dependent random variables in the sense that longer service lives imply higher chances that the structure will be subjected to a given extreme load level. However, it is generally not possible to collect live load data for extended periods of time corresponding to the service life of the structure. Therefore, statistical projections of live load data collected in the past over a limited period of time must be performed. This assumes that past data will also be valid in the future and that data collected at some representative locations are valid for the location of the bridge to be analyzed. It should be noted, that the projection of limited load intensity data, collected from previous measurements over short periods of time, to future return periods is associated with various levels of statistical modeling uncertainties. In addition, modeling the structure's response to the applied loads and estimating the variables that control the effects of the loads on the structure are also associated with high levels of structural modeling uncertainty. The structural modeling uncertainties are independent of the return period.

To find the probability distribution for the maximum loading event in a period of time T we have to start by assuming that N loading events occur during this period of time T. These events are designated as S_1, S_2, … S_N. The maximum of these N events, call it $S_{max,N}$, is defined as:

$$S_{max,N} = \max (S_1, S_2, \ldots S_N) \tag{11}$$

We are interested in finding the probability distribution of the maximum live load event that will control whether the structure will be safe or unsafe. The probability distribution of the load can be represented by the cumulative probability distribution of $S_{max,N}$. This cumulative probability distribution, $F_{s\ max\ N}(S)$, gives the probability that $S_{max,N}$ is less than or equal to a value S. If the maximum of N events, i.e. $S_{max,N}$ is less than S, this implies that each one of these N events is less than S. Therefore, S_1 is less than S, S_2 is less than S, … and S_N is less than S. Hence, assuming that the loading events are independent, using the basics concepts of the theory of probability, the probability that $S_{max,N} \leq S$ can be calculated from:

$$F_{s_{\max N}}(S) = F_{s_1}(S).F_{s_2}(S)...F_{s_N}(S) \tag{12}$$

where $F_{s_i}(S)$ is the cumulative distribution of event S_i.

If S_1, S_2 ... S_N are independent random loading events that are drawn from the same probability distribution, then:

$$F_{s_1}(S) = F_{s_2}(S) = ... = F_{s_N}(S) = F_s(S) \tag{13}$$

Therefore, Equation (12) reduces to

$$F_{s_{\max N}}(S) = \left[F_s\ (S)\right]^N \tag{14}$$

The cumulative distributions of the load effects, F(S), can be assembled by sending the truck weight and axle configuration data assembled at a WIM site through appropriate influence lines and the data from all the trucks are assembled into cumulative distribution histograms. This could be done for individual trucks and for multi-truck loading events whether these multi-truck events consist of a series of trucks in a single lane (trucks following each other) or trucks in multi-lanes (side-by-side or staggered).

The number of events expected in a return period can be obtained based on information from WIM data on the Average Daily Truck Traffic (ADTT) as well as WIM headway data. Generally speaking, WIM data has shown that Interstate highways can be subjected to up to 5000 trucks per day, more than 85% of which travel in the main driving lane and about 1% to 2% of which can be close to each other in two contiguous lanes to be considered side-by-side. Note that equation 14 assumes that the number of events, N, is a known deterministic value. The sensitivity analysis performed by Sivakumar, Ghosn & Moses (2008) has however demonstrated that the results of Equation 14 are not highly sensitive to variations in the number of events, N, as N becomes large.

The probability distribution of the maximum live load intensity using Eq. (14) can be used to find the mean and the standard deviation of the maximum intensity, S_{max}, expected in a return period.

Besides the uncertainties associated with estimating the maximum laod effect, S_{max}, obtained from Eq. (14) given the histogram for the WIM data, F(S), collected at a particular WIM site, the total load effect must also account for other types of modeling uncertainties. One type of modeling uncertainties is related to the data collection process and the sufficiency of the quantity of WIM data in adequately describing the true distribution of the load F(S). This modeling uncertainty will be referred to as data variability. Another modeling uncertainty is related to the variability in the WIM data collected from different sites. This type of modeling uncertainty is referred to as site-to-site variability. In addition, the structural analysis process involves a level of uncertainty that should be included when assessing the safety of bridges. Particularly important are the analysis of lateral distribution of the load in multi-girder bridges and the dynamic impact analysis.

BRIDGE CONFIGURATIONS, RESISTANCE MODEL AND DEAD LOAD MODEL

Bridge Configurations

The reliability calibration of load rating specifications requires that the bridge load rating process leads to uniform reliability levels for the applicable bridge configurations. Hence, the calibration has to be performed on a sample set of bridges that are most representative of the bridges to which the specifications will apply. During the calibration of the AASHTO LRFD, Nowak (1999) used a representative sample of multi-girder concrete T-beam, prestressed concrete and steel bridges having span lengths varying between 30-ft to 200-ft and beam spacings varying between 4-ft and 12-ft. The analysis was performed for shear and bending moment effects. Table 1 gives an example of the steel bridge configurations to be used during the calibration process.

In addition to the basic bridge configuration database, the reliability analysis requires as input information on the statistics of all the random variables that are used in the safety assessment of a bridge member. Specifically, for each bridge configuration, the probability distributions as well as the means and coefficients of variation of the member strength, dead loads and live loads are required. In order to be consistent with the current bridge design process, the same database for member resistance and dead load statistics used by Nowak (1999) during the calibration of the AASHTO LRFD will be used in this study to represent the resistance and the dead loads on typical bridge configurations.

On the other hand, recent observations made on truck weight data collected from Weigh-In-Motion stations (WIM) at representative U.S. sites (Sivakumar et al, 2008) have shown that trucks travelling over the U.S. highway system can be significantly different than the biased Ontario truck weight data used during the calibration of the AASHTO LRFD specifications. In this study, truck weight, configuration and headway data collected in each direction of six different WIM stations will be used to model the live load effects on typical bridges as will be described further below.

Dead Load Model

Following Nowak (1999)'s approach, the total dead load, DL is divided into the dead load of pre-fabricated members, D_{C1}, the dead load of cast-in-place members, D_{C2}, and the dead load of the wearing surface, D_w, such that the mean total dead load is given by:

$$\overline{DL} = \overline{D_{c1}} + \overline{D_{c2}} + \overline{D_w} \tag{15}$$

The standard deviation of the total dead load, σ_{DL}, is expressed as a function of the standard deviations of each dead load component:

$$\sigma_{DL} = \sqrt{\sigma_{DC1}^2 + \sigma_{DC2}^2 + \sigma_{DW}^2} \tag{16}$$

The relationship between the standard deviation, σ_{DL}, mean, $\overline{DL}$, and the coefficient of variation (COV) of the dead load, V_{DL}, is obtained as:

$$V_{DL} = \frac{\sigma_{DL}}{\overline{DL}} \tag{17}$$

Following Nowak (1999), the dead load effects are assumed to follow Normal probability distributions where the mean values and the COV's of each dead load component are given as:

$$\begin{aligned} \overline{D_{C1}} &= 1.03\ D_{C1} \qquad & V_{DC1} &= 8\% \\ \overline{D_{C2}} &= 1.05\ D_{C2} \qquad & V_{DC2} &= 10\% \\ \overline{D_{W}} &= 1.0\ D_{W} \qquad & V_{DW} &= 25\% \end{aligned} \tag{18}$$

Where D_{c1}, D_{c2} and D_w are respectively the nominal values of the dead load of pre-fabricated members, cast-in-place members, and wearing surface. Table 1 provides typical nominal values for the moment effect of each dead load component for a typical set of simple span composite multi-girder steel bridges. These data are obtained from Nowak (1999) for the 30-ft, 60-ft, 120-ft and 200-ft spans. For the other spans, the moment effects of the dead weights are obtained from estimates of the dead weights per unit length. These estimates are obtained by interpolation from the weight per unit length values obtained from the report by Nowak (1999).

Table 1. Nominal dead weight moment effects for typical composite steel girders.

Span (ft)	Spacing (ft)	D_{C1} (kip-ft)	D_{C2} kip-ft)	D_W kip-ft)
40 – ft	4	14	109	21
	6	15	149	32
	8	20	185	43
	10	23	231	54
	12	27	284	64
60 –ft	4	39	245	49
	6	48	335	73
	8	70	414	97
	10	84	521	122
	12	103	639	146
100 –ft	4	329	681	135
	6	361	931	203
	8	386	1150	270
	10	407	1447	337
	12	458	1775	405
120 – ft	4	502	981	194
	6	607	1341	292
	8	650	1656	389
	10	681	2083	486
	12	773	2556	583
200 – ft	4	2780	2725	540
	6	3303	3725	810
	8	3790	4600	1080
	10	4190	5788	1350
	12	4875	7100	1620

Resistance Model

The nominal resistance of an existing bridge member depends on the dead load and the rating of the member. Specifically, the relationship between the Rating Factor, R.F., and the nominal member resistance, R_n, can be expressed by:

$$R.F. = \frac{\varphi_n R_n - \gamma_{DC}\left(D_{c1} + D_{c2}\right) - \gamma_{DW} D_W}{\gamma_L L_L} \tag{19}$$

where R_n is the nominal resistance, D_{C1} is the dead load effect of pre-fabricated components, D_{C2} is the dead load effect of cast-in-place components and attachments, D_W is the nominal dead load effect for the wearing surface, L_L is the live load effect of the nominal load used to calculate the Rating Factor including dynamic allowance and load distribution factor, φ_n is the resistance factor, and γ are the load factors.

The resistance factor, φ_n and load factors γ depend on the specifications used and the type of load effect being considered. For example, according to the AASHTO LRFR specifications φ_n =1.0 for the bending moment capacities of steel and prestressed concrete members, γ_{DW}=1.50, γ_{DC}=1.25. The Inventory Rating live load factor is given as γ_L=1.75 and γ_L=1.35 for the Operating Rating using the HL-93 live load model. The AASHTO LRFR Operating Rating for the AASHTO legal loads is given as γ_L=1.80 for ADTT $\geq$ 5000, γ_L=1.65 for 1000<ADTT <5000, and γ_L=1.40 for ADTT $\leq$ 100. In the AASHTO LRFR, the dynamic allowance factor is 1.33 times the static truck moment effect and the load distribution factor is calculated as a function of span length and beam spacing for different numbers of loaded lanes as provided in the AASHTO LRFD load distribution tables. The format of Eq. (19) is also applicable for standard load rating methods although different nominal loads, dynamic impact and load distribution factors as well as resistance and load factors are used for AASHTO Load Factor Ratings (LFR) or Allowable Stress Ratings (ASR).

Equation (19) can be used to find the nominal resistance of a bridge member for different values of the rating factor. Nowak (1999) assumed that the member resistances can be modeled by lognormal probability distributions where the mean and COV of the moment resistance of bridge girders are related to the nominal values by:

$$\begin{aligned}
&\overline{R} = 1.12\ R_n && V_R = 10\% && \text{For steel beams} \\
&\overline{R} = 1.05\ R_n && V_R = 7.5\% && \text{For prestressed concrete beams} \\
&\overline{R} = 1.14\ R_n && V_R = 13\% && \text{For reinforced concrete beams}
\end{aligned} \tag{20}$$

For the shear resistance, the mean and COV are given by Nowak (1999) as:

$$\begin{aligned}
&\overline{R} = 1.14\ R_n && V_R = 10.5\% && \text{For steel beams} \\
&\overline{R} = 1.15\ R_n && V_R = 14\% && \text{For prestressed concrete beams} \\
&\overline{R} = 1.20\ R_n && V_R = 15.5\% && \text{For concrete beams with steel} \\
&\overline{R} = 1.40\ R_n && V_R = 17\% && \text{For concrete beams without steel}
\end{aligned} \tag{21}$$

ANALYSIS OF REPRESENTATIVE WIM DATA

The object of this study is to recalibrate the AASHTO LRFR live load factors taking into consideration current WIM data to verify that the AASHTO LRFR produce acceptable and uniform levels of reliability for typical U.S. bridges under current loading conditions. To achieve this goal, it is critical to use the most representative statistical information on truck weights, truck configurations, and multiple presence data. For the purposes of this study, data collected by Sivakumar et al (2008) in each direction of six WIM U.S. sites as part of NCHRP project 12-76 is analyzed to obtain projections for the maximum bridge load effects. The WIM six sites are located in: New York (NY 9121), Mississippi (MS 2606), Indiana (IN 9512), Florida (FL 9926), California (CA 0001), Texas (TX0526). The data was gathered in 2005 and 2006. The WIM data from each direction of these sites contained the number of axles for each truck, the axle spacings and the axle weights. Multiple presence probabilities were assembled form a representative site in New York based on the analysis performed by Sivakumar et al (2008). This Section of the report presents the method used to analyze the WIM data.

WIM Data Sites

The WIM Data sites on Interstate highways were selected to represent high traffic sites with representative distribution across the United States. Table 2 lists descriptive information for each site including in the Average Daily Truck Traffic (ADTT) and the number of truck records for each site after scrubbing the data.

Table 2. WIM Data Sites for LRFR Recalibration

Site	State	Interstate Route / Dir	# Trucks Recorded	ADTT	Mean GVW (kips)	Mean of top 10% GVW (kips)
0001	CA	I-5 E/N	1,537,613	5,058	56.0	80.6
		I-5 W/S	1,470,924	4,839	53.3	81.9
0526	TX	I-20 E/N	1,330,799	4,070	55.6	80.8
		I-20 W/S	1,174,954	3,593	56.7	81.5
2606	MS	I-55 N	564,393	1,622	66.5	108.7
		I-55 S	604,919	1,733	63.2	83.5
9121	NY	I-81 N	531,042	1,715	57.2	101.7
		I-81 S	525,733	1,614	57.8	98.3
9512	IN	I-74 E	931,971	2,596	60.7	82.5
		I-74 W	1,003,443	2,795	60.1	87.3
9926	FL	I-75 N	1,096,076	4,136	48.4	84.1
		I-75 S	1,032,680	3,897	53.5	84.6

The protocols established in NCHRP Project 12-76 were adapted to the application here of bridge rating recalibration. The recommended protocols for using traffic data in bridge design are:

STEP 1	DEFINE WIM DATA REQUIREMENTS FOR LIVE LOAD MODELING
STEP 2	SELECTION OF WIM SITES FOR COLLECTING TRAFFIC DATA FOR BRIDGE DESIGN
STEP 3	QUANTITIES OF WIM DATA REQUIRED FOR LOAD MODELING
STEP 4	WIM CALIBRATION & VERIFICATION TESTS
STEP 5	PROTOCOLS FOR DATA SCRUBBING, DATA QUALITY CHECKS & STATISTICAL ADEQUACY OF TRAFFIC DATA
STEP 6	GENERALIZED MULTIPLE-PRESENCE STATISTICS FOR TRUCKS AS A FUNCTION OF TRAFFIC VOLUME
STEP 7	PROTOCOLS FOR WIM DATA ANALYSIS FOR ONE-LANE LOAD EFFECTS FOR SUPERSTRUCTURE DESIGN
STEP 8	PROTOCOLS FOR WIM DATA ANALYSIS FOR TWO-LANE LOAD EFFECTS FOR SUPERSTRUCTURE DESIGN
STEP 9	ASSEMBLE AXLE LOAD HISTOGRAMS FOR DECK DESIGN
STEP 10	FILTERING OF WIM SENSOR ERRORS / WIM SCATTER FROM WIM HISTOGRAMS
STEP 11	ACCUMULATED FATIGUE DAMAGE AND EFFECTIVE GROSS WEIGHT FROM WIM DATA
STEP 12	LIFETIME MAXIMUM LOAD EFFECT Lmax FOR SUPERSTRUCTURE DESIGN
STEP 13	DEVELOP AND CALIBRATE VEHICULAR LOAD MODELS FOR BRIDGE DESIGN

The first 8 of these steps, except step 4 which provides guidance to the WIM site owner, were followed in this study to prepare data to support the LRFR bridge recalibration. The remaining steps are intended for use in new bridge design and are not applicable to load rating except an alteration to the design load protocol in step 12.

Step 1 requires defining the traffic data needs. Lane by lane truck type distribution, total weights, and axle weights and spacings for each truck record are required. Accurate time history information is useful for modeling multiple vehicle events but is frequently unavailable. An alternate approach to addressing multiple presence probability was presented in the protocols in step 8 and is utilized in this study.

Step 2 requires selecting sites that both meet the requirements of quality WIM data and where the loads are representative of the objective. The WIM data used in this study from NCHRP 12-76 represent sites with unbiased data, good calibration maintenance, and sensor information from at least two lanes in each direction and both directions. The sites selected for this study are diverse enough to represent a national load rating calibration effort with at least one site in each of the five U.S. regions defined by FHWA.

Step 3 is satisfied by the quantity of truck records at the recorded sites and by the fact that each site has approximately one year's worth of continuous data recently recorded.

Step 5 followed the data scrubbing protocols by removing trucks that did not meet any of the 13 tests on reasonable truck configurations through a combination of programmed tests and manual review of the data records. This step is executed at the stage where the state specifically formatted WIM data files are converted to a common truck data format for subsequent processing.

Step 6 addresses the lack of high resolution time stamp information in most WIM data. The multiple presence probabilities determined in NCHRP 12-76 for New York sites were used as described in the following section. A second truck data file was generated for each site in the common truck format using configurations taken from the raw WIM data but arranged with time stamps determined using random sampling. This provides the data that can be used for determining force effects due to multiple vehicles both in the same lane and in adjacent lanes.

Steps 7 and 8 were performed by using the two truck data files generated for each site. The sections on Single lane loading and Two-lane loading describe the process in this study.

Multiple Presence Probability

The truck arrival data collected at several New York WIM sites were analyzed by Sivakumar et al (2008). The daily truck traffic volume was classified into three categories: 1) Light with less than 1000 trucks per day; 2) Average with more than 1000 trucks but less than 2500 trucks per day; and 3) Heavy with more than 2500 trucks but less than 5000 trucks per day. Although a very heavy volume category was defined for cases where more than 5000 trucks crossed a WIM site on a given day, there were not enough cases of such situations to obtain any useful information.

When considering multiple trucks on a given span, a multiple presence event occurs if the gap between two trucks, that is the distance between the last axle of the leading truck and the first axle of the trailing truck, is less than the span length. Multiple presence probabilities were compiled for two trucks in adjacent lanes side-by-side, two trucks in adjacent lanes staggered, and two trucks in the same lane. Multiple presence probabilities for trucks in two adjacent lanes were compiled for headway separations up to 300 feet, in 20-foot increments as shown in Table 3.

The number of multiple presence events that occurred in a given day is recorded as a percentage of the total truck count for that day. The average multiple presence percentage is then calculated for days with light truck volume, average truck volume, heavy truck volume. Each direction of traffic was considered separately. The maximum multiple side-by-side presence cumulative percentages for each truck volume category are summarized in Table 3. Following the assumption made in NCHRP 12-76, it will be conservatively assumed that trucks in adjacent lanes within 60-ft head-to-head are actually side-by-side. Accordingly, for light volume sites, the percentage of side-by-side trucks is obtained as P_{sxs}=0.54%. This indicates that 0.54% of the trucks were found to be in adjacent lanes where the headways were 60-ft head to head. On average volume sites, 1.25% of the trucks were found to be in adjacent lanes within 60-ft headways. For heavy truck traffic sites 1.95% of the trucks were found to be in adjacent lanes within 60-ft head-to-head.

The probability of side-by-side events as calculated in this study are compared to the values used by Moses (2001) during the calibration of the AASHTO LRFR. Specifically, Moses (2001) used a side-by-side probability P_{sxs}=0.5% for light volume sites with ADTT=100, P_{sxs}=1% for average sites with ADTT=1,000 and P_{sxs}=6.67% for heavy traffic sites with ADTT=5000. It is observed that the values used by Moses (2001) are reasonably similar to the values obtained from the New York WIM data for the low volume and average volume sites. However, the 6.67% side-by-side probability used by Moses (2001) that was adopted from Nowak (1999) is larger than observed in New York. Furthermore, in order to match the load projection of Nowak (1999), Moses (2001) applied the side-by-side probabilities on only the heaviest 20% of the trucks while the data in Table 3 is for all the trucks. Thus, the Moses (2001) and Nowak (1999) assumptions add a significant level of conservatism to the maximum values of side-by-side percentages observed on New York sites. These comparisons are highlighted in Table 4. The last row of Table 4 provides the rounded values that are proposed for use in this study.

Table 3. Upper envelope for percentage of side-by-side trucks on New York bridges

Maximum Side-by-Side Truck Multiple Presence Cumulative Probabilities				
	Site Truck Traffic			
Headway H (ft)	Light: ADTT < 1k	Average: 1k < ADTT < 2.5k	Heavy: 2.5k < ADTT < 5k	Very Heavy: ADTT > 5k
H ≤ 20	0.19	0.41	0.61	0.00
H ≤ 40	0.33	0.84	1.27	0.00
H ≤ 60	0.54	1.25	1.95	0.00
H ≤ 80	0.80	1.60	2.57	0.00
H ≤ 100	1.00	2.13	3.33	0.00
H ≤ 120	1.21	2.54	4.14	0.00
H ≤ 140	1.45	2.88	4.80	0.00
H ≤ 160	1.62	3.18	5.41	0.00
H ≤ 180	1.80	3.47	5.97	0.00
H ≤ 200	1.99	3.73	6.49	0.00
H ≤ 220	2.09	3.97	6.97	0.00
H ≤ 240	2.23	4.21	7.42	0.00
H ≤ 260	2.35	4.43	7.85	0.00
H ≤ 280	2.49	4.64	8.26	0.00
H ≤ 300	2.60	4.84	8.66	0.00

Table 4. Comparison of ASSHTO LRFR side-by-side data and maximum measured New York data.

Traffic Volume	Light	Average	Heavy
AASHTO LRFR classification	ADTT $\leq$ 100	ADTT=1000	ADTT $\geq$ 5000
AASHTO LRFR Percent side-by-side P_{sxs}	0.5% of heaviest 20% of trucks	1% of heaviest 20% of trucks	6.67% of heaviest 20% of trucks
NCHRP 12-76 classification	ADTT<1000	1000<ADTT<2500	2500<ADTT<5000
NCHRP 12-76 Percent side-by-side P_{sxs}	0.54% of all trucks	1.25% of all trucks	1.95% of all trucks
Proposed ADTT classification for calibration study	ADTT=100	ADTT=1000	ADTT=5000
Proposed for NYSDOT LRFR Load Simulation	0.5% of all trucks	1.25% of all trucks	2% of all trucks

The analysis of the reliability of bridges requires statistical information on the maximum live load effects expected to be witnessed by the bridge within its service period. For the design of new bridges, AASHTO LRFD assigns a design life of 75 years. For the evaluation of existing bridges that are inspected on a regular basis, a rating period of 5 years has been recommended by Moses (2001).

The evaluation of the safety of a single lane bridge requires the evaluation of the maximum load effect in the lane. For multi-lane bridges, the bridge must be able to sustain the maximum load effect expected when a single lane is loaded as well as the maximum load effect from multi-lanes. The next two sections describe the process used in this study to obtain the statistics for the maximum load for single lane and multi-lane loading.

Single lane loading

The analysis of the safety of existing bridges is based on ensuring that bridges maintain an acceptable level of reliability to sustain the maximum load effect expected within a pre-set return period. The return period for the load rating of an existing bridge is taken to be 5 years following the recommendation of Moses (2001) for the calibration of the AASHTO LRFR code. It should be clearly stated that it is not possible to obtain exact values for the maximum expected 5-year load due to the limitations in the available database. In fact, to obtain accurate results, one would need several cycles of WIM data collected over 5 years for each cycle which is not possible at this stage due to the relative recent adoption of WIM technology in the U.S. even if one assumes that the load spectra are stationary and do not change over time. Hence, some form of statistical projection will be needed for any practical load modeling effort as will be described in this section.

A single lane truck loading event is defined as the occurrence of a single truck or following trucks in a single lane of the bridge. The analysis presented in this report assumes that only one truck is in a lane. Some WIM systems are capable of providing axle weights and axle spacings for each truck crossing the site of an installation and are also capable of taking continuous uninterrupted data at normal highway traffic speeds with accurate time stamps. For

these sites the WIM data are able to identify single lane and multi-lane loading events involving several trucks. Such WIM systems can thus provide the axle weights, axle spacings and relative positions of all the trucks involved in each single-lane or multilane loading event. Since not al the WIM sites used in this study possess this capability, a simulation will be used to consider multi-truck loadings in a single lane or multi-lane loading. The next paragraph outlines the process used to study the side-by-side loading in two-lane bridges. A similar simulation can be used for the multi-truck loading on a continuous bridge. Multi-truck loadings on a simple span bridge are not expected to be significantly different than single truck loadings because in this study the focus is on short span bridges of up to 200-ft in length, where the probability of having a heavy truck following the main truck on the bridge is relatively low and the effect of the following truck is small to the positioning of the following truck on the influence line.

Lacking WIM data from single lane bridges, in this analysis we will assume that single lane bridges will be exposed to the same truck loads as those in the main driving lane of multi-lane bridges. The process of obtaining the maximum effect in a return period T for a single truck on the bridge is based on the implementation of Eq. (11) through (14) and consists of the following steps:

- Given the time of arrival of trucks and their axle weight and axle spacing, obtain the load effect of each truck by sending each truck through the appropriate influence line.
- Assemble the single lane load effects into a histogram such as the one shown in Figure 2.
- Assemble the load effects into a cumulative distribution function, $F_s(S)$ such as the one plotted in black in Figure 3.

Ideally, the next step would require applying $F_s(S)$ into Equation 14 to obtain the cumulative distribution function for $F_{S_{\max N}}(S)$ which is the maximum effect in a given return period, T, where within the return period we will have N bridge loading events. For example, given an ADTT=2000 in one lane and a return period of 5 years, N=2000x365x5=3,650,000 events. Equation 14 assumes the truck weight and other truck properties for each loading event are independent but they are sampled from the same probability distribution that does not change over time.

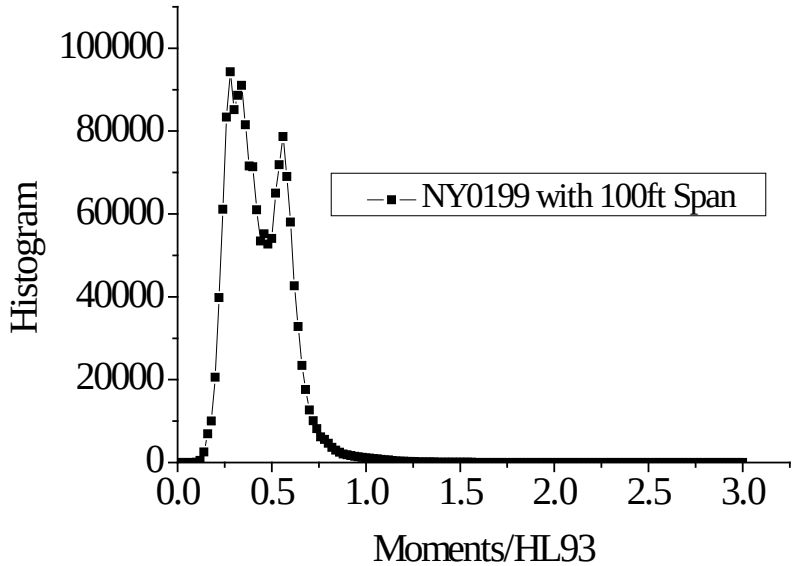

Figure 2 - Histogram of normalized moment effect for a single lane events

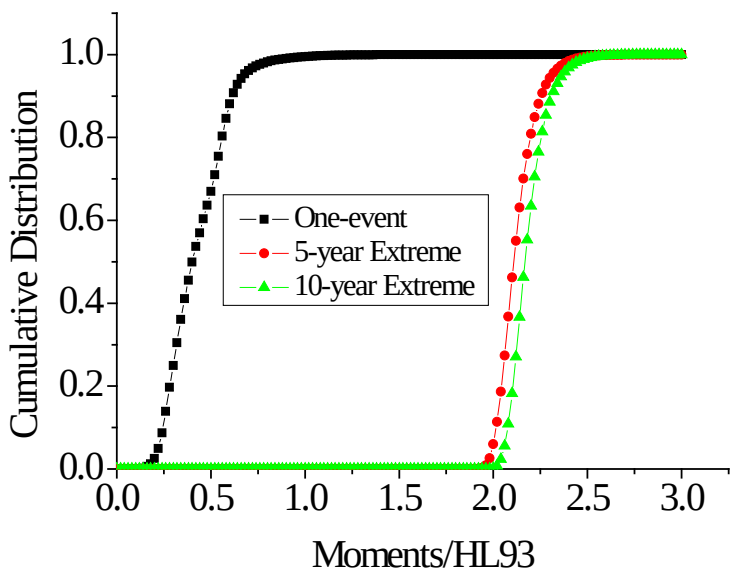

Figure 3 - Cumulative distribution for single lane for a single event, 5-year maximum and 10-year maximum load effects

In most instances, we do not have data for a whole period, T, and the available WIM data in the tail end of $F_s(S)$ is usually not sufficient to provide accurate values of $F_{s_{\max N}}(S)$ when Eq. (14) is applied with very high values of N. Therefore, some sort of statistical projection is required to extend the range of the cumulative distribution $F_s(S)$. It would be possible to apply Eq. (14) if the probability distribution function of a single loading event, $F_s(S)$, is known. However, the probability distribution of one loading event as shown in Figure 2 does not follow any known probability distribution type. On the other hand, careful observations of the tail ends

of the WIM data histograms for single lane events assembled from several National sites have indicated that the tail end of the histograms of single loading events may approach the tail ends of Normal probability distributions. For example, Figure 4 shows the plot of the data collected at WIM site 0199 in upstate New York on a Normal probability scale. A Normal probability plot is executed by taking the standard deviate which is the Normal inverse of $F_s(S)$ represented by $\Phi^{-1}[F_s(S)]$ and plotting it versus the magnitude of the load effect, S. The plot would produce a straight line if S follows a Normal distribution. In this case, the mean of S would correspond to the abscissa for which $\Phi^{-1}[F_s(S)]$ is zero. The mean plus one standard deviation would correspond to the abscissa for which $\Phi^{-1}[F_s(S)]$ is equal to 1.0.

The plots of the WIM data in Figure 2 and Figure 4 show that the data as a whole (plotted in black) does not follow a Normal distribution as the curve in Figure 4 does not follow a straight line. However, Figure 4 shows that the upper 5% of the data does approach a straight line indicating that the tail end of the data resembles the tail end of a hypothetical Normal distribution. A linear fit (shown in red) of the Normal probability plot of the upper 5% of the data collected at this 0199 WIM site will produce a slope, m, and an intercept, n, which will give the mean of the equivalent Normal distribution that best fits the tail end as μ_{event}= -n/m. The standard deviation of the best-fit Normal is σ_{event}=1/m. As an example, the analysis of the WIM data of site 0199 for a 100-ft simply supported bridge shows that the upper 5% matches that of a normal distribution with an equivalent mean value μ_{event}= -0.18522, and a standard deviation σ_{event}=0.4363. Figure 5 shows the match of the tail between the original histogram and the normal histogram. Figure 6 zooms in on the tail end to better show the match.

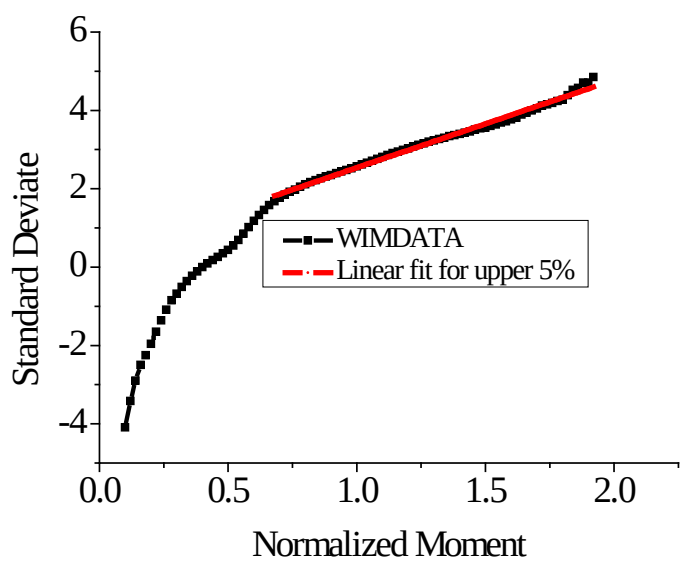

Figure 4 - Normal Probability Plot for Moment Effect of Single Lane Events

The application of Eq. 14 can be executed numerically for any parent probability distribution including a hybrid distribution where the lower 95% of the data is taken from the actual histogram of $F_s(S)$ and the upper 5% taken from the Normal probability distribution that fits the tail end. However, the fact that the tail end of the WIM data matches that of a Normal distribution allows for the application of extreme value theory to obtain the statistics of the maximum load effect in closed form. The approach is based on the following known concept as

provided in Ang and Tang (2007): "If the parent distribution of the initial variable, S, has a general Normal distribution with mean□ $_{event}$ and standard deviation □$_{event}$, then the maximum value after N repetitions approaches asymptotically an Extreme Value Type I (Gumbel) distribution" with an inverse measure of dispersion □$_N$ given by:

$$\alpha_N = \frac{\sqrt{2\ln(N)}}{\sigma_{event}} \tag{22}$$

and a most probable value u_N given by:

$$u_N = \mu_{event} + \sigma_{event}\left(\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}}\right) \tag{23}$$

α_N and u_N can be used to find the mean of the maximum load effect, L_{max}, its standard deviation, σ_{Lmax}, and its Coefficient of Variation (COV) V_{Lmax} for any return period having N repetitions as:

$$\overline{L}_{\max} = \mu_{\max} = u_N + \frac{0.577216}{\alpha_N} \tag{24}$$

$$\sigma_{L\max} = \frac{\pi}{\sqrt{6}\alpha_N} \tag{25}$$

$$V_{L\max} = \frac{\sigma_{L\max}}{\overline{L}_{\max}} \tag{26}$$

The most probable value u_N and the inverse dispersion coefficient α_N, can also be used to describe the probability distribution function, $f_{smax}(S)$ and the cumulative distribution function, $F_{smax}(S)$, of the maximum load effect S_{max} by:

$$f_{s\max}(S) = \alpha_N e^{-\alpha_N (S-u_N)} e^{-e^{-\alpha_N (S-u_N)}} \tag{27}$$

$$F_{s\max}(S) = e^{-e^{-\alpha_N (S-u_N)}} \tag{28}$$

As an example, to find the cumulative distribution of the maximum effect in 5 years, we take the cumulative distribution of one event obtained from the WIM data as shown in Figure 3 and then adjust its tail end by using the cumulative Normal distribution with mean μ_{event}= -0.18522, and a standard deviation σ_{event}=0.4363 for the cases when Fs(S) is greater than 95%. The new extended composite distribution of $F_s(S)$ is raised to the N's power. For ADTT=5000, in a 5-year return period, the number of truck loading events becomes N=5year*365days/year*5000 trucks/day =9,125,000. The distribution of $F_{smax}(S)$ is obtained as shown in Figure 7. The mean for L_{max} is found to be 2.13 and the standard deviation is 0.010.

Since the tail end of the WIM data matches that of a Normal distribution, this allows for the application of extreme value theory to obtain the statistics of the maximum load effect in closed form. By applying Eq. (25) and (26) the extreme value distribution of the maximum five-year load effect is found to have a mean equal to 2.125, and a standard deviation σ_{Lmax}= 0.0988. Figure 7 shows a comparison between the results of the projection using Eq. (14) and those obtained by plotting the Extreme distribution of Eq. (27). The implementation of Eq. (28) for the moment effects on a 100-ft span for the data of WIM site 0199 for 5-year and 10-year projections are also plotted in Figure 3.

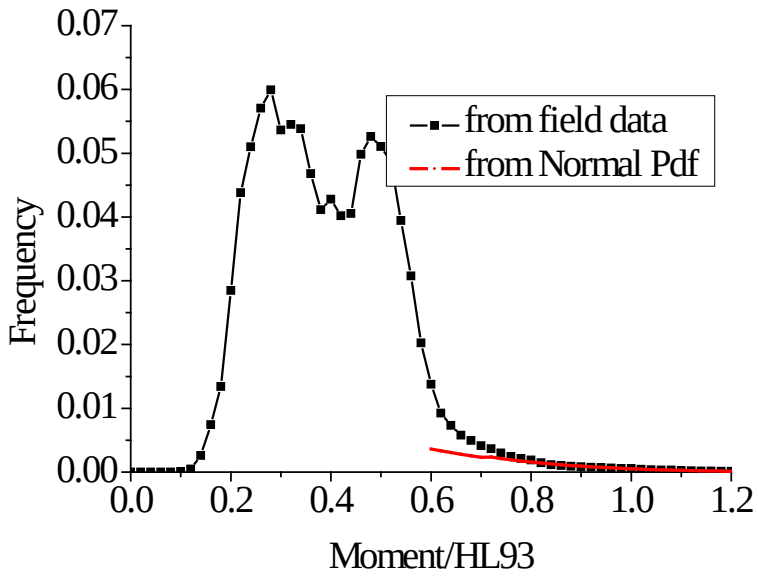

Figure 5 - Comparison of WIM data and Normal distribution that best match the tail end of the Normalized Moment Histogram for Site NY0199

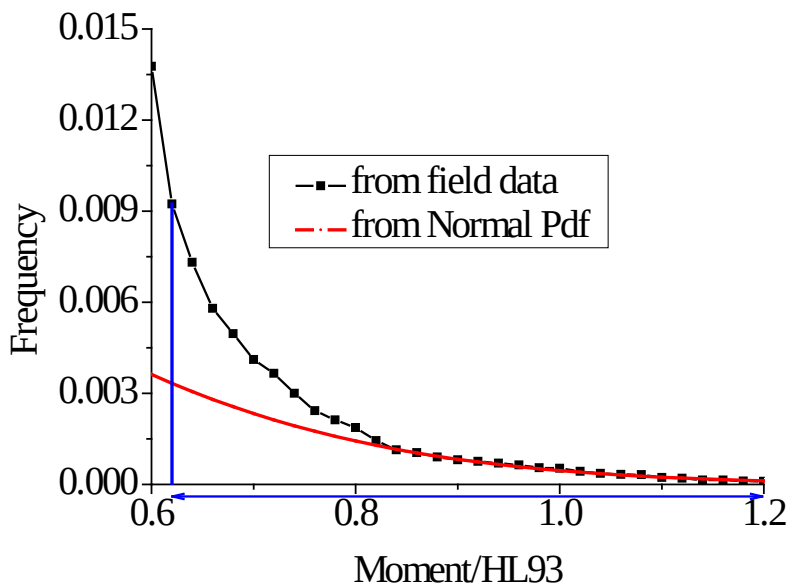

Figure 6 - Zoom on the tail end of the WIM data and the corresponding Normal probability distribution.

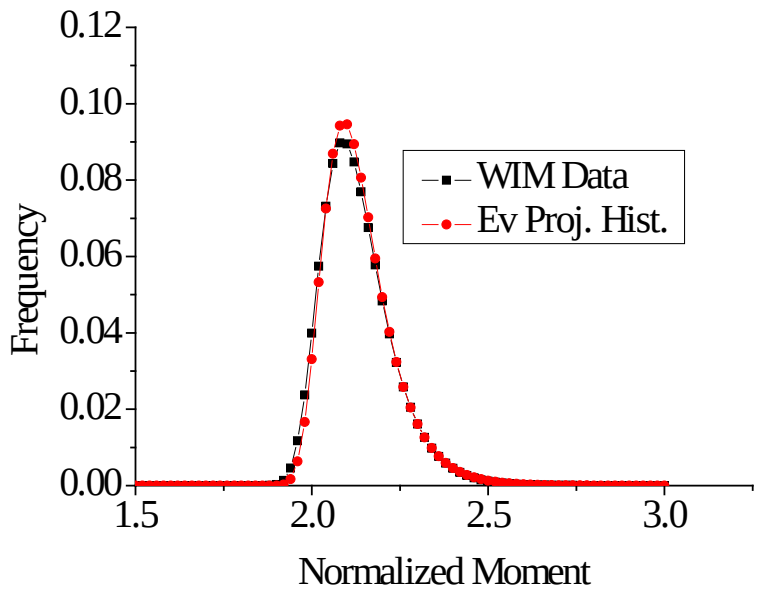

Figure 7 - Comparison of Histogram for 5-year Maximum with Simulated WIM data and Extreme Value Model

Two-lane loading

The maximum load on a multi-lane bridge could occur due to a single truck, multiple trucks in one lane or when several lanes are loaded. Due to the lack of data and the low probability of having several trucks simultaneously in three or more lanes, this study following the approach taken by Nowak (1999) and Moses (2001) will focus on two-lane loads. Although some of the available WIM data provides sufficient information to obtain the location of multi-lane truck events, it is herein decided to use a convolution approach to obtain the load effect when two adjacent bridge lanes are simultaneously loaded by heavy trucks. The convolution approach will provide the flexibility of addressing multi-lane loading events for low volume sites with low ADTT as well as high volume and regular sites and will be consistent with the approach that will be followed when studying the load effects when permit trucks could be simultaneously on a bridge with random truck traffic. In this process, it is conservatively assumed that the truck weights and statistics on the number of following trucks in each of the lanes belong to the same truck weight and headway populations.

In a first step, using the WIM data files, the shear force or bending moment effect of each truck loading event in the WIM record is calculated for a given bridge span length by passing the sequence of trucks through the proper influence line. The shear or moment for each truck load event is then normalized by dividing the calculated value by the shear or moment of the HL-93 load model. The shear and moment data for the single lane loading are collected into separate percent frequency histograms. Each histogram provides a discretized form of the probability density function (PDF) of the shear or moment effects for the site. The histogram is designated as $H_x(X)$ while the PDF is designated as $f_x(X)$. The relation between $H_x(X)$ and $f_x(X)$ is given by:

$$H_x(X) = \int_{X_l}^{X_u} f_x(x)dx \qquad (29)$$

where X_l and X_u give the upper and lower bounds of the bin within which X lies. If the bin size is small, then $f_x(X)$ can be assumed to be constant within the range of X_l to X_u. For example, Figure 8 shows in red the moment load effect histogram for a single lane obtained from the WIM data collected at site 9121 for a 100-ft simple span. The bin size in this figure and the data assembled in this study is $\Delta X=X_u-X_l=0.02$.

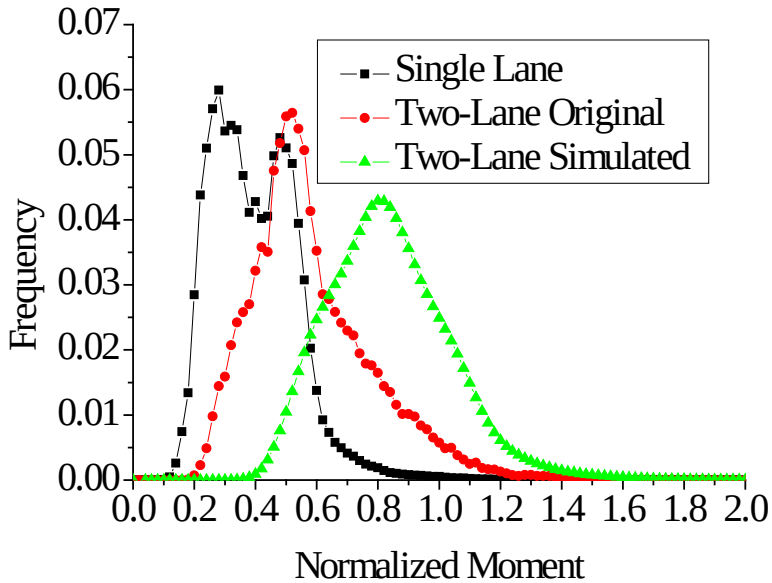

Figure 8. Normalized 100-ft moment histogram for trucks in a single lane and both lanes of WIM site 9121.

The total moment effect when two lanes are loaded is obtained from the single lane load effect as $S=x_1+x_2$ where x_1 is the effect of the truck in the drive lane and x_2 is the effect of the truck in the passing lane. Assuming independence between truck effects, the probability density function of the effect of side-by-side trucks $f_s(S)$ can be calculated using a convolution approach. The convolution equation is presented as:

$$f_s(S) = \int_{-\infty}^{+\infty} f_{x2}(S - x_1) f_{x1}(x_1) dx_1 \tag{30}$$

where $f_s(S)$ is the probability distribution of the multi-lane effects, $f_{x1}(\ldots)$ is the probability distribution of the effects of trucks in lane 1, $f_{x2}(\ldots)$ is the probability distribution of the effects of trucks in lane 2. In these calculations we assume that $f_{x2}(\ldots)$ and $f_{x1}(\ldots)$ are actually the same distribution function.

Equation 30 can be interpreted as follows: Given that the effect of the truck in lane 1 is equal to x_1, then the probability that the effects of two side-by-side trucks will take a value S, is equal to the probability that the effect of the truck in lane 1 is x_1, times the probability that the effect of the truck in lane 2 is equal to $X_2=S-x_1$. This will lead to the following expression: $f_{x2}(S - x_1) f_{x1}(x_1)$. The integration is executed to cover all possible values of x_1. Equation 28 gives the probability density function (PDF) for one particular value of S. Thus,

equation 28 must be repeated for each possible value of S. Equation 27 is then used to convert the PDF's into equivalent histograms.

Figure 8 shows the histogram obtained from the WIM data for the single lane events in red and the histogram for the two-lane loading events obtained from applying Eq. (28) in blue. The figure compares the latter to the two-lane loading events obtained directly from the WIM data shown in black. The results confirm that the convolution approach yields more conservative values. This due to the assumption that trucks in two lanes that are within $\pm$60-ft head to head are compressed so that they are placed side-by-side.

To find the cumulative distribution of the maximum effect for multi-lane loading in 5 years, we take the cumulative distribution of one event, $F_s(S)$, and raise it to the N's power as shown in Eq. 14. For a site with ADTT=5000, the percentage of side-by-side loading events has been estimated as 2% as per Table 3. Accordingly, in a 5-year return period, N=5000$\times$2%$\times$365 days/year x 5 years=182,500. For ADTT = 1000, the number of events becomes N=1000$\times$1.25%$\times$365x 5=22813 and for ADTT =100, N=913.

Since the distribution of multi-lane loading events have been obtained from the convolution and because of the relative small values of N for the number of multi-lane events in the 5-year return period, it would be possible to find $F_{s_{\max N}}(S)$ directly from Equation (14). Alternatively, the same approach followed for a single lane event which consists of fitting the tail end of the data into an equivalent normal distribution can also be used to obtain the mean and COV of the Gumbel distribution that describes the maximum load effect in a 5-year return period. Figure 9 shows a comparison between the results of the projection using Eq. (14) and those obtained by plotting the Gumbel distribution using Eq. (27). A good match is observed from both procedures.

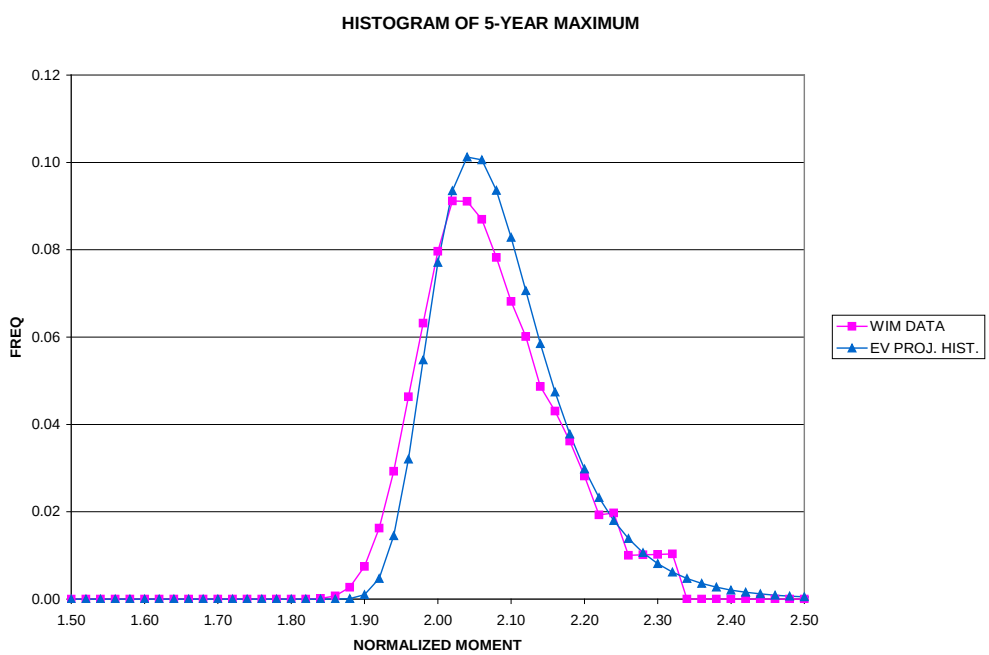

Figure 9. Comparison of Histogram for 5-year Maximum from Eq. (14) and Extreme Value Model

MAXIMUM LIVE LOAD EFFECTS USING WIM DATA

The statistical analysis of the WIM data from each direction of the six National sites has been performed to project for the maximum live load effects expected on simple span bridges for five-year projection periods. The load effects studied are for the moment at midspan of the bridges and for the shear near the support for a single truck on the bridge and for side-by-side trucks.

As an example, Tables 5 and 6 gives the average results for $\overline{L}_{max}$ for 5-years and $V_{site\text{-}to\text{-}site}$ obtained for one truck loading and two-lane loading by applying Eq. (24) and Eq. (26) for the moment and shear effects on simple span beams for the data collected from all the WIM sites studied and for each of the three ADTT categories. All the results are normalized by the corresponding effect of the HL-93 load.

Figures 10, 11, 12 and 13 give the plot of the L_{max} values for each direction of the six National WIM sites. The plots show large variability in L_{max} between the sites with a site-to-site COV reaching as high as 20% for the shear effect of side-by-side trucks as summarized in Table 6. Tables 6A thru 6D give average Lmax values for each of the six states.

The Appendix to this report provides detailed plots describing the beta indices for various bridge types, loading types and span lengths.

Table 5. Summary of average L_{max} for maximum load effects for 5 years

ADTT 5000											
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.780	1.777	1.641	1.601	1.615	1.626	1.609	1.552
			Average COV	0.035	0.038	0.039	0.039	0.038	0.039	0.040	0.041
		MULTI LANE	Average Lmax	2.483	2.409	2.198	2.165	2.201	2.208	2.164	2.060
			Average COV	0.037	0.039	0.041	0.041	0.040	0.041	0.043	0.043
	SIMPLE SHEAR	1-LANE	Average Lmax	1.799	1.706	1.716	1.760	1.785	1.786	1.722	1.642
			Average COV	0.036	0.037	0.037	0.038	0.039	0.040	0.041	0.042
		MULTI LANE	Average Lmax	2.495	2.324	2.342	2.402	2.414	2.385	2.282	2.157
			Average COV	0.037	0.039	0.039	0.039	0.040	0.041	0.043	0.044

ADTT 1000											
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.697	1.688	1.556	1.518	1.533	1.542	1.524	1.468
			Average COV	0.039	0.042	0.044	0.043	0.043	0.043	0.045	0.046
		MULTI LANE	Average Lmax	2.319	2.242	2.041	2.010	2.046	2.051	2.002	1.904
			Average COV	0.044	0.047	0.049	0.049	0.048	0.048	0.051	0.052
	SIMPLE SHEAR	1-LANE	Average Lmax	1.714	1.621	1.630	1.673	1.694	1.692	1.629	1.551
			Average COV	0.039	0.042	0.042	0.042	0.043	0.044	0.046	0.047
		MULTI LANE	Average Lmax	2.330	2.167	2.182	2.238	2.244	2.212	2.110	1.992
			Average COV	0.044	0.046	0.046	0.046	0.048	0.049	0.051	0.052

ADTT 100											
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.568	1.552	1.425	1.392	1.408	1.415	1.393	1.338
			Average COV	0.046	0.050	0.052	0.052	0.051	0.052	0.054	0.055
		MULTI LANE	Average Lmax	2.025	1.943	1.760	1.733	1.769	1.770	1.714	1.624
			Average COV	0.062	0.066	0.069	0.069	0.068	0.069	0.073	0.074
	SIMPLE SHEAR	1-LANE	Average Lmax	1.582	1.491	1.500	1.538	1.554	1.547	1.486	1.410
			Average COV	0.047	0.050	0.050	0.050	0.051	0.053	0.055	0.057
		MULTI LANE	Average Lmax	2.034	1.887	1.897	1.944	1.939	1.903	1.804	1.696
			Average COV	0.062	0.064	0.065	0.065	0.068	0.070	0.073	0.076

Table 6. Average L_{max} values and $V_{site\text{-}to\text{-}site}$.

				ADTT 5000							
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.780	1.777	1.641	1.601	1.615	1.626	1.609	1.552
			site to site STD	0.398	0.311	0.226	0.221	0.243	0.241	0.210	0.191
			site to site COV	0.224	0.175	0.138	0.138	0.150	0.148	0.130	0.123
		MULTI LANE	Average Lmax	2.483	2.409	2.198	2.165	2.201	2.208	2.164	2.060
			site to site STD	0.519	0.448	0.363	0.373	0.395	0.401	0.383	0.363
			site to site COV	0.209	0.186	0.165	0.172	0.180	0.182	0.177	0.176
	SIMPLE SHEAR	1-LANE	Average Lmax	1.799	1.706	1.716	1.760	1.785	1.786	1.722	1.642
			site to site STD	0.381	0.267	0.274	0.289	0.263	0.233	0.213	0.189
			site to site COV	0.212	0.156	0.160	0.164	0.147	0.130	0.124	0.115
		MULTI LANE	Average Lmax	2.495	2.324	2.342	2.402	2.414	2.385	2.282	2.157
			site to site STD	0.491	0.402	0.422	0.431	0.415	0.401	0.376	0.354
			site to site COV	0.197	0.173	0.180	0.179	0.172	0.168	0.165	0.164
				ADTT 1000							
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.697	1.688	1.556	1.518	1.533	1.542	1.524	1.468
			site to site STD	0.377	0.298	0.219	0.215	0.236	0.236	0.208	0.188
			site to site COV	0.222	0.176	0.141	0.142	0.154	0.153	0.136	0.128
		MULTI LANE	Average Lmax	2.319	2.242	2.041	2.010	2.046	2.051	2.002	1.904
			site to site STD	0.485	0.426	0.353	0.359	0.379	0.384	0.367	0.347
			site to site COV	0.209	0.190	0.173	0.178	0.185	0.187	0.183	0.182
	SIMPLE SHEAR	1-LANE	Average Lmax	1.714	1.621	1.630	1.673	1.694	1.692	1.629	1.551
			site to site STD	0.361	0.257	0.263	0.278	0.255	0.228	0.208	0.185
			site to site COV	0.210	0.158	0.161	0.166	0.150	0.134	0.128	0.119
		MULTI LANE	Average Lmax	2.330	2.167	2.182	2.238	2.244	2.212	2.110	1.992
			site to site STD	0.458	0.386	0.400	0.410	0.397	0.384	0.361	0.341
			site to site COV	0.196	0.178	0.183	0.183	0.177	0.174	0.171	0.171
				ADTT 100							
			SPAN LENGTH	20	40	60	80	100	120	160	200
AVERAGE OF ALL STATES	SIMPLE MOMENT	1-LANE	Average Lmax	1.568	1.552	1.425	1.392	1.408	1.415	1.393	1.338
			site to site STD	0.347	0.279	0.211	0.208	0.229	0.230	0.206	0.188
			site to site COV	0.221	0.180	0.148	0.150	0.163	0.163	0.148	0.140
		MULTI LANE	Average Lmax	2.025	1.943	1.760	1.733	1.769	1.770	1.714	1.624
			site to site STD	0.435	0.395	0.341	0.339	0.356	0.358	0.343	0.321
			site to site COV	0.215	0.203	0.194	0.196	0.201	0.202	0.200	0.198
	SIMPLE SHEAR	1-LANE	Average Lmax	1.582	1.491	1.500	1.538	1.554	1.547	1.486	1.410
			site to site STD	0.332	0.243	0.249	0.262	0.245	0.223	0.204	0.183
			site to site COV	0.210	0.163	0.166	0.171	0.158	0.144	0.137	0.130
		MULTI LANE	Average Lmax	2.034	1.887	1.897	1.944	1.939	1.903	1.804	1.696
			site to site STD	0.411	0.365	0.370	0.381	0.372	0.359	0.339	0.322
			site to site COV	0.202	0.193	0.195	0.196	0.192	0.189	0.188	0.190

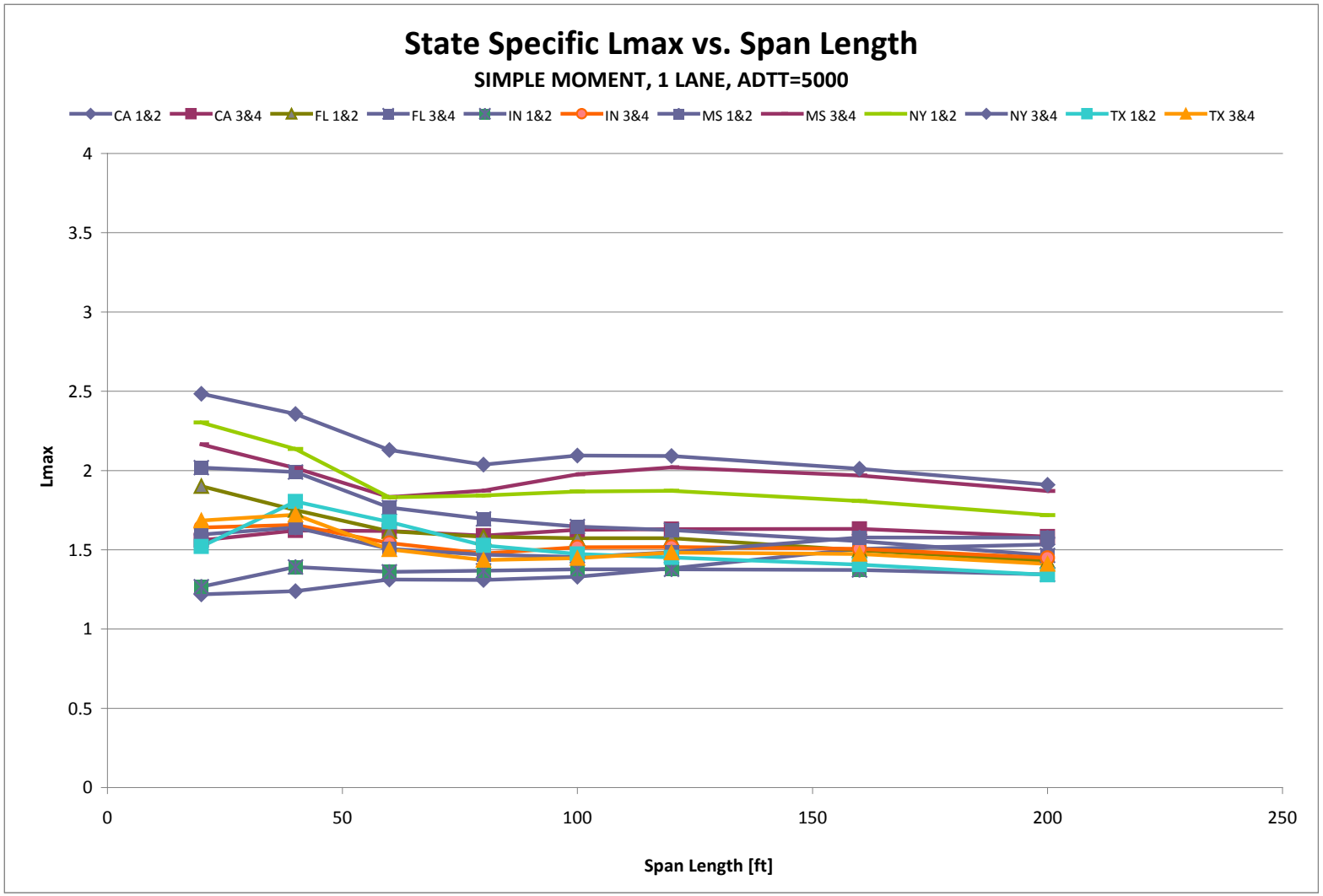

Figure 10. Plot of L_{max} for single truck moment from all sites.

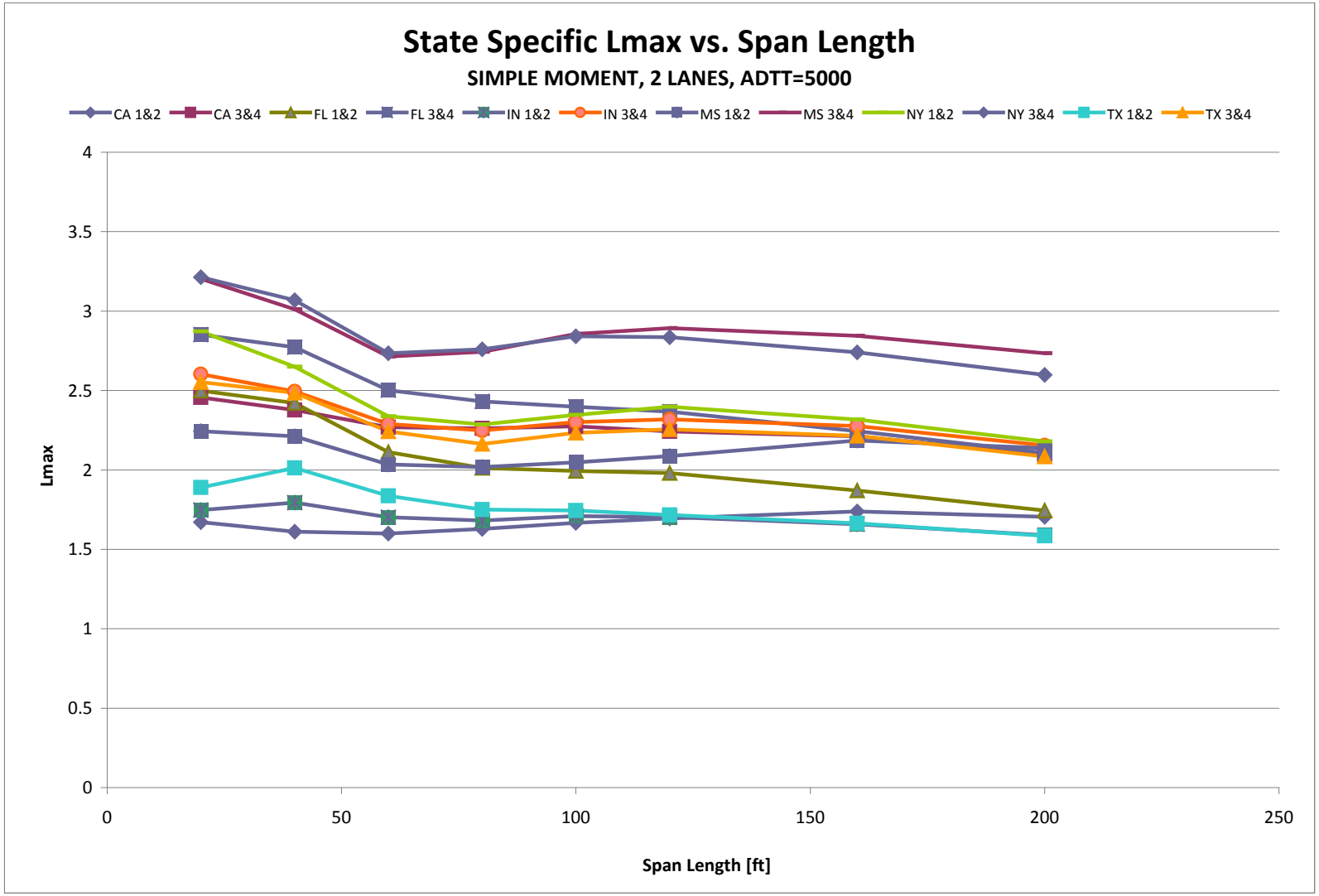

Figure 11. Plot of L_{max} for side-by-side truck moment from all sites.

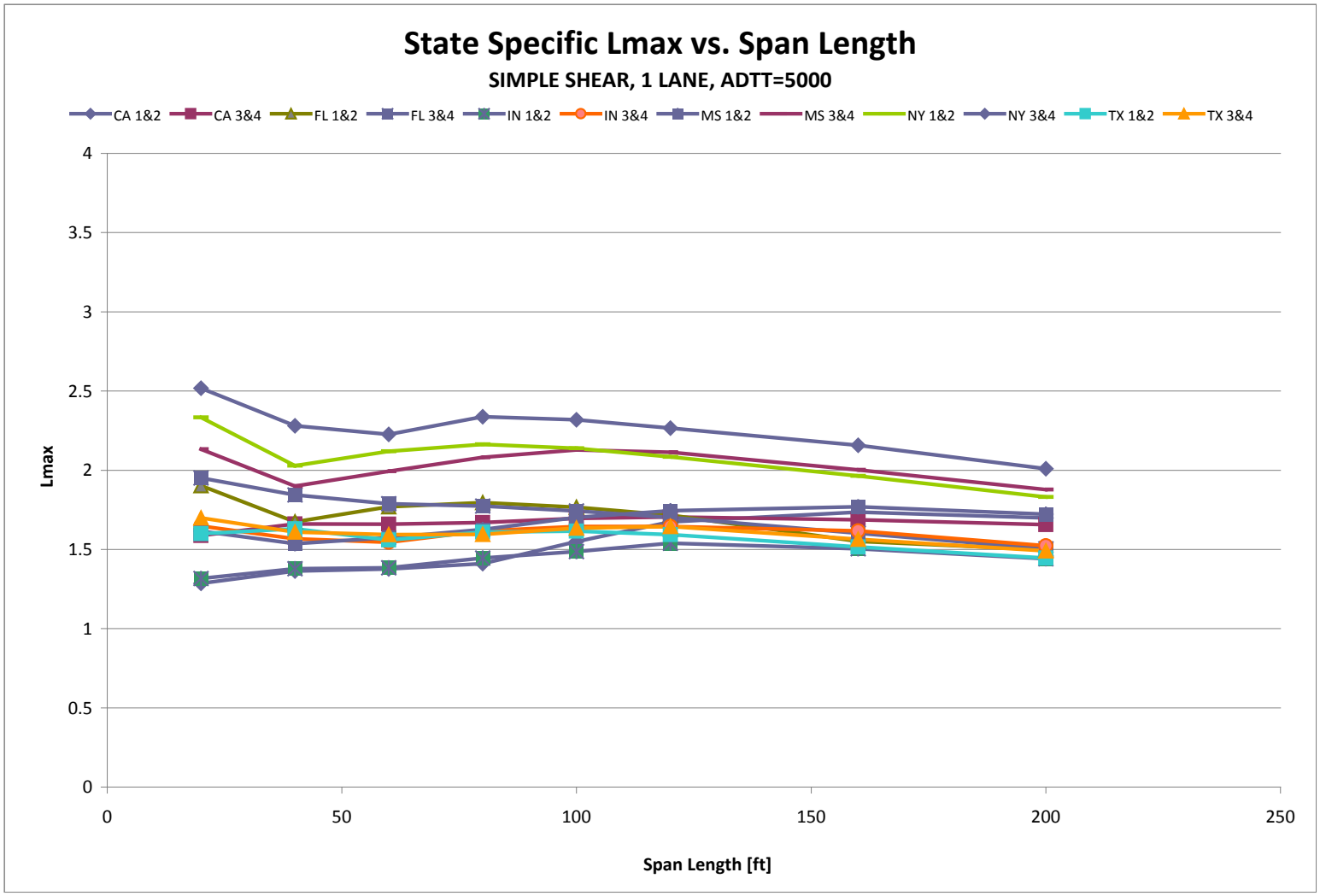

Figure 12. Plot of L_{max} for single truck shear from all sites.

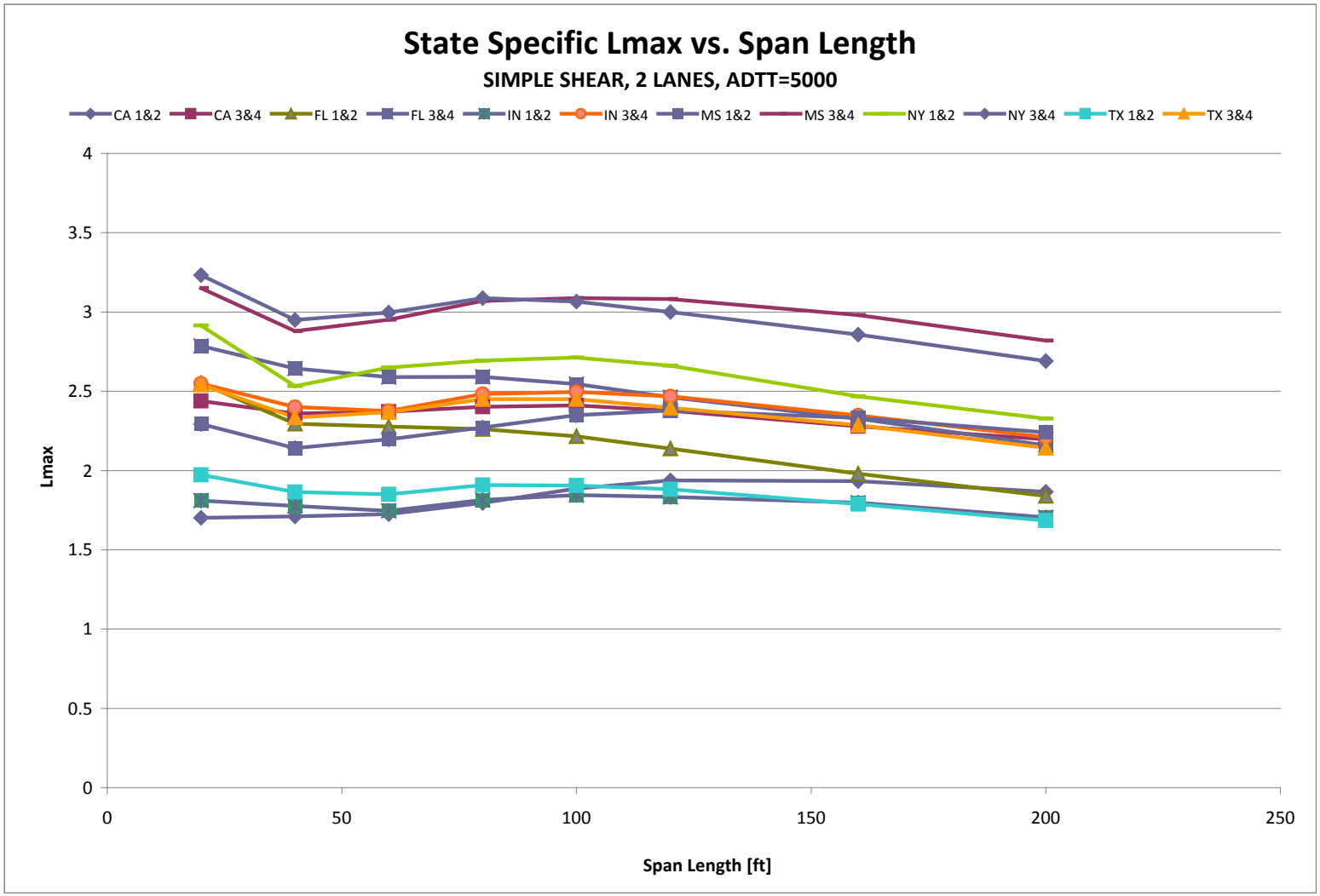

Figure 13. Plot of L_{max} for side-by-side truck shear from all sites.

Table 6A. L_{max} for single lane moment from all sites. (See Plot Fig 10)

Lmax SIMPLE MOMENT 1-LANE		Span Length [ft]							
		20	**40**	**60**	**80**	**100**	**120**	**160**	**200**
Direction 1&2	**CA**	1.219	1.239	1.313	1.310	1.330	1.382	1.505	1.534
	FL	1.901	1.750	1.617	1.582	1.573	1.573	1.497	1.428
	IN	1.267	1.392	1.361	1.368	1.377	1.377	1.372	1.344
	MS	1.597	1.640	1.506	1.469	1.454	1.484	1.578	1.576
	NY	2.303	2.136	1.831	1.843	1.867	1.872	1.807	1.718
	TX	1.521	1.805	1.675	1.528	1.473	1.452	1.406	1.342
Direction 3&4	**CA**	1.560	1.622	1.619	1.589	1.626	1.630	1.632	1.583
	FL	2.018	1.991	1.766	1.694	1.646	1.625	1.554	1.464
	IN	1.641	1.656	1.543	1.476	1.516	1.518	1.505	1.448
	MS	2.165	2.015	1.833	1.874	1.976	2.020	1.969	1.870
	NY	2.484	2.356	2.130	2.038	2.094	2.092	2.011	1.910
	TX	1.684	1.721	1.503	1.436	1.447	1.482	1.474	1.410

Table 6B. L_{max} for two lane moment from all sites. (See Plot Fig 11)

Lmax SIMPLE MOMENT 2-LANES		Span Length [ft]							
		20	**40**	**60**	**80**	**100**	**120**	**160**	**200**
Direction 1 & 2	**CA**	1.671	1.611	1.599	1.628	1.667	1.695	1.739	1.705
	FL	2.499	2.421	2.112	2.012	1.994	1.980	1.870	1.745
	IN	1.748	1.794	1.702	1.681	1.709	1.703	1.658	1.589
	MS	2.244	2.211	2.035	2.019	2.048	2.087	2.185	2.137
	NY	2.872	2.649	2.337	2.286	2.346	2.398	2.317	2.180
	TX	1.889	2.012	1.837	1.750	1.744	1.716	1.664	1.585
Direction 3 & 4	**CA**	2.456	2.377	2.269	2.261	2.275	2.243	2.211	2.097
	FL	2.851	2.772	2.501	2.431	2.398	2.367	2.244	2.108
	IN	2.602	2.494	2.290	2.248	2.301	2.318	2.276	2.154
	MS	3.203	3.011	2.714	2.744	2.856	2.893	2.845	2.734
	NY	3.213	3.068	2.735	2.759	2.842	2.836	2.740	2.599
	TX	2.553	2.485	2.242	2.164	2.234	2.256	2.215	2.084

Table 6C . L_{max} for single lane shear from all sites. (See Plot Fig 12)

Lmax SIMPLE SHEAR 1-LANE		Span Length [ft]							
		20	**40**	**60**	**80**	**100**	**120**	**160**	**200**
Direction 1 & 2	**CA**	1.286	1.363	1.377	1.411	1.549	1.674	1.735	1.699
	FL	1.903	1.674	1.770	1.795	1.766	1.717	1.551	1.498
	IN	1.317	1.378	1.384	1.445	1.487	1.540	1.504	1.441
	MS	1.615	1.536	1.574	1.626	1.702	1.745	1.769	1.723
	NY	2.333	2.028	2.119	2.164	2.139	2.084	1.964	1.831
	TX	1.599	1.630	1.560	1.609	1.615	1.594	1.516	1.445
Direction 3 & 4	**CA**	1.588	1.661	1.659	1.669	1.696	1.705	1.687	1.657
	FL	1.951	1.844	1.789	1.773	1.743	1.702	1.602	1.504
	IN	1.648	1.567	1.545	1.617	1.645	1.645	1.617	1.523
	MS	2.133	1.900	1.993	2.082	2.128	2.113	2.002	1.878
	NY	2.518	2.281	2.226	2.337	2.318	2.266	2.157	2.009
	TX	1.699	1.609	1.593	1.595	1.634	1.645	1.564	1.491

Table 6D. Plot of L_{max} for two lane shear from all sites.(See Plot 13)

Lmax SIMPLE SHEAR 2-LANES		Span Length [ft]							
		20	**40**	**60**	**80**	**100**	**120**	**160**	**200**
Direction 1 & 2	**CA**	1.702	1.710	1.725	1.795	1.887	1.938	1.934	1.866
	FL	2.551	2.295	2.278	2.262	2.216	2.139	1.980	1.840
	IN	1.811	1.776	1.746	1.814	1.845	1.834	1.797	1.705
	MS	2.294	2.142	2.197	2.271	2.349	2.378	2.332	2.241
	NY	2.915	2.533	2.650	2.694	2.713	2.662	2.468	2.328
	TX	1.973	1.865	1.850	1.908	1.905	1.882	1.789	1.684
Direction 3 & 4	**CA**	2.438	2.359	2.372	2.402	2.411	2.377	2.279	2.197
	FL	2.786	2.643	2.590	2.591	2.546	2.462	2.328	2.159
	IN	2.548	2.401	2.376	2.482	2.496	2.468	2.348	2.210
	MS	3.150	2.879	2.952	3.070	3.088	3.081	2.981	2.819
	NY	3.233	2.950	2.996	3.088	3.066	3.000	2.858	2.691
	TX	2.541	2.335	2.369	2.450	2.450	2.396	2.286	2.143

RELIABILITY ANALYSIS FOR LEGAL LOADS

The reserve margin of safety of a bridge component is defined as, Z, such that:

$$Z = R - S = R - DL - LL \tag{3}$$

Where R is the resistance or member capacity, S is the total load effect. Probability of failure, P_f, is the probability that the resistance R is less than or equal to the total applied load effect S or the probability that Z is less or equal to zero. This is symbolized by the equation:

$$P_f = Pr\ [\ R \leq S\ ] \tag{4}$$

Where Pr is used to symbolize the term probability. The reliability index, β, is defined such that:

$$P_f = \Phi(-\beta) \tag{6}$$

Where Φ is the normal probability function that gives the probability that the normalized random variable is below a given value.

The statistical analysis of the WIM data as described in the previous section leads to the calculating the mean value of the maximum normalized total static live load effect, $\overline{L}_{\max}$, applied on the bridge. To analyze the safety of a single member under actual traffic conditions, the distribution of the total load to the most heavily loaded member must be considered along with the dynamic amplification due to the moving vehicles. Accordingly, the mean live load effect on the most critical girder of two-lane and one-lane multi-girder bridges can be respectively obtained from:

For two loaded lanes $$\overline{LL} = \overline{L}_{\max} \times HL_{93} \times \overline{IM} \times \overline{D.F.}/2 \tag{31}$$

For a single loaded lane $$\overline{LL} = \overline{L}_{\max} \times HL_{93} \times \overline{IM} \times \overline{D.F.}/1.2 \tag{32}$$

Where HL_{93} is the effect of the HL-93 vehicle, $\overline{IM}$ is the mean dynamic amplification factor and $\overline{D.F.}$ is the mean of the load distribution factor. Dividing the D.F. of two lanes by 2 is done to account for the fact that the $\overline{L}_{\max}$ values for two lanes while the D.F. values are applied on a single lane of load. The one-lane distribution factor is divided by 1.2 to remove the multiple presence factor already included in the LRFD equations.

To obtain the mean values of the load distribution factors in Eq. (31) and (32), Nowak (1999) and Moses (2001) assume that the D.F. values given by the AASHTO LRFD Specifications are the actual mean values of the distribution factors. This is not strictly speaking correct because the LRFD equations for the distribution factors include some level of conservatism as explained by Zokaie et al (1991). In fact, Puckett et al (2007) found large variations in the distribution factors they calculated using advanced methods when compared to the values of the AASHTO LRFD tables.

For bending of typical reinforced and prestressed concrete and steel girder bridges loaded by one lane of traffic, the LRFD load distribution factor equation is given as (AASHTO, LRFD, 2007):

$$D.F. = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{33}$$

Where S is the beam spacing, L is the span length, t_s is the deck thickness, and K_g is a beam stiffness parameter. Not having enough information to calculate the term within the last parenthesis, it is taken as 1.0 as recommended in the AASHTO LRFD specifications. Note that Equation 33 already includes a multiple presence factor MP=1.2 which accounts for the higher probability of having one heavy truck in one lane as compared to the probability of having two side-by-side heavy trucks in two adjacent lanes. Since in this study we calculated L_{max} directly, the multiple presence factor will have to be removed when calculating the maximum applied live load as shown in Equation (32).

For two lanes loaded, the load distribution factor equation for bending becomes (AASHTO, LRFD, 2007):

$$D.F. = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{34}$$

According to the AASHTO LRFD, Equation (34) should be applied on the effect of a single lane. Since, the Lmax calculations are for two lanes of loading, we need to divide the load effect by 2 as shown in Eq, 31.

The distribution factor for shear in typical reinforced and prestressed concrete and steel girder bridges for one lane loaded is given by the AASHTO LRFD as:

$$D.F. = 0.36 + \frac{S}{25} \tag{35}$$

For two lanes loaded, the LRFD shear distribution factor becomes:

$$D.F. = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \tag{36}$$

If the results of advanced analyses are believed to be more representative of the actual load distribution as compared to the AASHTO load distribution factors, then the mean distribution factor $\overline{DF}$ can be obtained by dividing the AASHTO values with the ratios calculated by Puckett et al (2007) a sample of which is presented in Table 7 for multi-girder steel and prestressed girder bridges. Table 7 shows that the AASHTO LRFD tables can over-predict the actual DF by as much as 29% for single lanes and 11% for two loaded lanes of multi-girder steel and prestressed concrete bridges. The variability of the AASHTO LRFD values is also high on the order of 10 to 11% for the two lane loading.

Table 7. Ratio of AASHTO load distribution factor to factor from refined analysis as per Puckett et al (2007).

Ratio of AASHTO LRFD/Refined Analysis	Concrete deck on interior steel I-girders		Concrete deck on interior prestressed concrete I-girders	
	One lane	Two lanes	One lane	Two lanes
Average	1.29	1.11	1.29	1.11
COV	7.2%	11.5%	8.3%	10.4%
Count	73	73	107	112

The coefficient of variation (COV = Std Deviation / Mean) of the maximum live load effect on a single girder should account for the site-to-site variability in the maximum load effect L_{max} represented by $V_{site\text{-}to\text{-}site}$, the variability within a site represented by V_{Lmax}, and also must account for the uncertainty associated with the limited WIM data sample size. In NCHRP 12-76, Sivakumar et al suggested that the effect of data limitation be represented by an additional Coefficient of Variation, V_{data}. The variability in the dynamic amplification factor, V_{IM} and the variability in the load distribution factor V_{DF} should also be included. Thus, the final COV for the applied live load effect on a single beam can be obtained from:

$$V_{LL} = \sqrt{V_{L\max}^2 + V_{site-to-site}^2 + V_{data}^2 + V_{IM}^2 + V_{DF}^2} \tag{37}$$

$V_{site\text{-}to\text{-}site}$ is obtained by comparing the $\overline{L}_{\max}$ values from different WIM sites within the state. For the data shown in Table 6, $V_{site\text{-}to\text{-}site}$ is observed to vary from about 12% to up to 21% depending on the span length, the load effect and the ADTT. On the other hand, the within a site variability of L_{max} is observed to remain below V_{Lmax}=8% with the higher values corresponding to the ADTT=100. Additional uncertainties are associated with L_{max} due to the limited number of data points used in the projections and the confidence levels associated with the number of sample points. Using the +/-95% confidence limits, Sivakumar, Ghosn & Moses (2008) estimated that the COV associated with the use of one year worth of WIM data is on the order of V_{data}=3% for the two-lane case and 2% for the one-lane case.

Nowak (1999) observed that the dynamic amplification factors augmented the static load effect, L_{max}, by an average of 10% for side-by-side trucks. This indicates that the mean dynamic amplification factor in Eq. (31) is $\overline{IM} = 1.10$. The dynamic amplification also resulted in a COV of V_{IM}=5.5% on the two-lane effect. For single lanes of traffic, the dynamic mean and COV are $\overline{IM} = 1.13$ and V_{IM}=9%.

As seen in Table 7 and other data assembled by Puckett et al (2007), the uncertainties in estimating the lane distribution factor is associated with a COV as high V_{DF}=11.5% for the two-lane loading case on steel multi-girder bridges. Even higher COV's are reported for different types of configurations.

The use of Equations (31) and (32) along with the approach followed in this report to find $\overline{L}_{\max}$ based on the implementation of Eq. (14), assume that the truck traffic within the bridge service life will not experience any growth in volume or changes in the truck types, configurations or weight spectra during the service life period. This is the same assumption made by Nowak (1999) and Moses (2001). The projection of changes in truck volume and type require a detailed forecasting analysis of economic growth and economic changes and its effect on the transportation of goods and the modes of transportation throughout the U.S. that is beyond the scope of this study.

Approach for Reliability Calibration of Live Load Factors for Legal Loads

The statistical model for the resistance, dead loads, and live loads as described in the previous pages will be used in the next step of this project to find the reliability index, β, that is currently implied in the current AASHTO Load Factor (LFR) load rating method. The approach will consist of the following steps:

- Using the AASHTO LFR live load factors and assuming a Rating Factor R.F. =1.0, use Equation (19) to find the nominal resistance, R_n, for each of the bridge configurations assembled (see for example Table 1). The live load and resistance and load factors used should reflect the reliability index in the code being investigated. When checking the reliability level implied in the LFR Legal Load ratings, we use the AASHTO Legal loads along with the resistance and load factors as well as impact factor and lane distribution factors of the AASHTO Standard Specifications.
- Use Eq. (20) and (21) to find the mean, $\overline{R}$, COV, V_R, and standard deviation, σ_R, for each member resistance in the assembled bridge population.
- Use Eq. 15, 16, 17 and 18 to assemble the mean, $\overline{DL}$, COV V_{DL}, and standard deviation, σ_{DL}, for the dead load applied on each member of the assembled bridge population.
- Use Eq. 31, 32 and 37 along with the results in Tables 5 and 6 to assemble the mean, $\overline{LL}$, COV V_{LL}, and standard deviation, σ_{LL}, for the live load applied on each member of the assembled bridge population.
- Set up the failure function of Eq. (3) as:

$$Z = R - S = R - DL - LL \qquad (38)$$

- Use Eq. (38) to find the reliability index, β, for each member of the bridge population assembled.
- Study the reliability index in current LFR ratings to find its average value and how it varies with span length, beam spacing or bridge cross section and material type.
- The AASHTO LRFR was calibrated to provide a uniform reliability index β_{target}=2.5 based on the live load model obtained from the Ontario truck weight database. Review if the current U.S. National WIM database would lead to different reliability levels and interact with the Project Panel to decide whether the target reliability level should be changed based on the findings of this task. For example, work performed by this team for the NYSDOT LRFR recalibration has shown that the higher truck weights observed on New York State bridges led to a lower reliability index on the order of β=1.7 for LFR. Accordingly, a target reliability index β_{target}=2.0 was set for the LRFR recalibration effort.
- If necessary adjust the AASHTO LRFR live load factors and the AASHTO Legal Trucks to uniformly match the new target reliability level.

A similar approach to that described above will be followed for calibrating the Permit Load Factors as will be described in the next section.

L_{max} Values for Random Truck Loading

The method developed for NCHRP 12-76 was used to find the maximum expected live load over a 5-year evaluation period for single lane loading and side-by-side loading. The results from the 12 directional WIM sites identified for this study are averaged and the normalized maximum live load for a 5-year return period, L_{max}, are obtained by dividing the maximum load by the effect of the HL-93 vehicle. The calculations are performed for sites with ADTT=5000, ADTT=1000 and ADTT=100. For the two-lane loading, the side-by-side probabilities P_{sxs}=2%, 1.25% and 0.5% are used for average daily truck traffic ADTT=5000, ADTT=1000 and ADTT=100 respectively. Table 8 shows how the results obtained form this WIM data set compare to the values obtained in the AASHTO LRFD calibration study. The results show that the side-by-side cases are reasonably close even though the results from the current WIM values are slightly higher. On the other hand, for the one-lane loadings, current WIM data show considerably increased loads as compared to the data used in the AASHTO LRFD. This could imply that in many cases, the one-lane loading may govern the loading on multi-lane bridges. At this stage and following the procedure of the AASHTO LRFD and LRFR calibrations, the reliability analysis is executed for random trucks on two-lane bridges assuming that the side-by-side event is the governing loading scenario. The calibration of the new AASHTO LRFR live load factors will require checking if one-lane loadings may govern the loading of multi-lane bridges.

Table 8. Comparison of Lmax values from this study to the AASHTO LRFD values

	Average of Recent WIM Data		AASHTO LRFD		AASHTO LRFD	
ADTT= 5000	5-year max		5-year max		75-year max	
Span	one lane	two lanes	one lane	two lanes	one lane	two lanes
20	1.78	2.48	1.21	2.06	1.30	2.21
40	1.78	2.41	1.28	2.18	1.35	2.30
60	1.64	2.20	1.24	2.11	1.32	2.24
80	1.60	2.17	1.25	2.13	1.32	2.24
100	1.61	2.20	1.24	2.11	1.31	2.23
120	1.63	2.21	1.22	2.07	1.29	2.19
160	1.61	2.16	1.17	1.99	1.24	2.11
200	1.55	2.06	1.16	1.97	1.23	2.09

The results obtained in this study for all the span lengths and the ADTT's for both moment and shear under one-lane and two-lane loadings are provided in Tables 9, 10 and 11 for sites with ADTT=5000, 1000 and 100. The Tables also express the uncertainties associated with determining L_{max} as an average COV for L_{max} which accounts for variations within a site, and the site-to-site COV. The tables show that uncertainties in estimating L_{max} for a given site with its own set of WIM data are relatively low with COV on the order of 4% for ADTT=5000 increasing to a maximum COV=8% for sites with ADTT=100. On the other hand, the site-to-site variability show high COV values which may reach as high as 22% for short span bridges. The site-to-site variability reflects the large variation in the types of trucks that travel on different bridges and the truck weight statistics for the different sites as well as the level of truck weight enforcement and regulations applied by different states.

These L_{max} values and corresponding COV's are used to find the reliability index associated with current AASHTO LFR methodology as will be explained next.

Table 9. Summary of average L_{max} for a five year return period and COV for sites with ADTT=5000

ADTT 5000	AVERAGE OF ALL WIM SITES											
	SIMPLE MOMENT						SIMPLE SHEAR					
	1-LANE			MULTI LANE			1-LANE			MULTI LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.78	0.04	0.22	2.48	0.04	0.21	1.80	0.04	0.21	2.50	0.04	0.20
40	1.78	0.04	0.18	2.41	0.04	0.19	1.71	0.04	0.16	2.32	0.04	0.17
60	1.64	0.04	0.14	2.20	0.04	0.17	1.72	0.04	0.16	2.34	0.04	0.18
80	1.60	0.04	0.14	2.17	0.04	0.17	1.76	0.04	0.16	2.40	0.04	0.18
100	1.61	0.04	0.15	2.20	0.04	0.18	1.79	0.04	0.15	2.41	0.04	0.17
120	1.63	0.04	0.15	2.21	0.04	0.18	1.79	0.04	0.13	2.38	0.04	0.17
160	1.61	0.04	0.13	2.16	0.04	0.18	1.72	0.04	0.12	2.28	0.04	0.16
200	1.55	0.04	0.12	2.06	0.04	0.18	1.64	0.04	0.12	2.16	0.04	0.16

Table 10. Summary of average L_{max} for a five year return period and COV for sites with ADTT=1000

ADTT 1000	AVERAGE OF ALL WIM SITES											
	SIMPLE MOMENT						SIMPLE SHEAR					
	1-LANE			MULTI LANE			1-LANE			MULTI LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.70	0.04	0.22	2.32	0.04	0.21	1.71	0.04	0.21	2.33	0.04	0.20
40	1.69	0.04	0.18	2.24	0.05	0.19	1.62	0.04	0.16	2.17	0.05	0.18
60	1.56	0.04	0.14	2.04	0.05	0.17	1.63	0.04	0.16	2.18	0.05	0.18
80	1.52	0.04	0.14	2.01	0.05	0.18	1.67	0.04	0.17	2.24	0.05	0.18
100	1.53	0.04	0.15	2.05	0.05	0.19	1.69	0.04	0.15	2.24	0.05	0.18
120	1.54	0.04	0.15	2.05	0.05	0.19	1.69	0.04	0.13	2.21	0.05	0.17
160	1.52	0.04	0.14	2.00	0.05	0.18	1.63	0.05	0.13	2.11	0.05	0.17
200	1.47	0.05	0.13	1.90	0.05	0.18	1.55	0.05	0.12	1.99	0.05	0.17

Table 11. Summary of average L_{max} for a five year return period and COV for sites with ADTT=100

ADTT 100	AVERAGE OF ALL WIM SITES											
	SIMPLE MOMENT						SIMPLE SHEAR					
	1-LANE			MULTI LANE			1-LANE			MULTI LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.57	0.05	0.22	2.03	0.06	0.21	1.58	0.05	0.21	2.03	0.06	0.20
40	1.55	0.05	0.18	1.94	0.07	0.20	1.49	0.05	0.16	1.89	0.06	0.19
60	1.42	0.05	0.15	1.76	0.07	0.19	1.50	0.05	0.17	1.90	0.06	0.20
80	1.39	0.05	0.15	1.73	0.07	0.20	1.54	0.05	0.17	1.94	0.07	0.20
100	1.41	0.05	0.16	1.77	0.07	0.20	1.55	0.05	0.16	1.94	0.07	0.19
120	1.41	0.05	0.16	1.77	0.07	0.20	1.55	0.05	0.14	1.90	0.07	0.19
160	1.39	0.05	0.15	1.71	0.07	0.20	1.49	0.05	0.14	1.80	0.07	0.19
200	1.34	0.06	0.14	1.62	0.07	0.20	1.41	0.06	0.13	1.70	0.08	0.19

RELIABILITY ANALYSIS FOR PERMIT LOADS

The calibration of the permit load factors must account for the uncertainties associated with estimating the load effects of the permit truck as well as the random trucks that may cross the bridge simultaneously with the permit truck. The latter (random truck) will depend on the actual truck weight histograms as well as the multiple presence statistics for typical U.S. bridges as collected from the WIM database. The object of this section is then to describe the approach to evaluate the reliability indices in current permit practices and to calibrate an appropriate set of Permit load factors for application in the AASHTO LRFR equation as presented in Eq. (19).

Permit Load Classification

There are several Permit classifications depending on the permit loading type and number of trips allowed. For the purposes of this recalibration study, the permit loading types are classified into two categories: Routine Permits (Annual/Blanket Permits) and Special Permits (Trip Permits). Special Permits are assumed to be controlled so that the truck weights are known to be equal to the permit weight. Routine permits, which are normally issued for a year's period, are less easily controlled and some of these have been observed to be overloaded by exceeding the permit weight limits. In terms of trip categories, the permits in this report will be divided into single-trip and multi-trip permits.

The recalibration of the live load factors for short to intermediate span bridges will consider the following four cases:

V. Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.
VI. Unlimited crossings of multiple trip permits where two Permit trucks could cross a bridge simultaneously side-by-side.
VII. Unlimited crossings where a Permit truck mixes with other random vehicles.
VIII. Single Permit trips where the Permit truck could mix with other random vehicles.

Case I is not affected by the WIM data. Case II depends on the probability of having two Permit trucks side-by-side and this in turn will depend on the number of permits that may cross a bridge within the five year rating period and the probability of side-by-side events for that number of permit crossings. Following the approach taken by Moses (2001), we will assume up to 100 Permits per day on one bridge as an upper limit. The probability of having two side-by-side permits will then be equal to 0.5% based on the WIM data collected on New York state sites on low truck traffic volume days. For cases III and IV, the reliability analysis should account for the number of random vehicles. Following the AASHTO LRFR classifications we will consider sites with ADTT=5000, 1000 or 100. The percentage of side-by-side vehicles will be P_{sxs}=2% for sites with ADTT=5000, P_{sxs}=1.25% for sites with ADTT=1000 and P_{sxs}=0.5% for sites with ADTT=100. These P_{sxs} values are upper bound values obtained from the headway data collected at ten New York State WIM sites as reported by Sivakumar et al (2008).

Variability in Routine Permit Weights

Permit trucks carrying divisible loads have been observed to be often overloaded. For example, the data collected by Sivakumar et al (2008) as part of NCHRP 12-76 have shown that New York State Type 6 permit trucks (6 axle dump trucks) that are normally limited to 120 kips often carry higher loads than the permit weight limits. Figure 14 shows the gross weight histogram of the Type 6 trucks collected from the east bound lane of New York WIM site.8280 on I-84. As illustrated in Figure 15, an exponential fit of the data for the trucks with gross weights

exceeding 120 kips shows that the data can be reasonably well represented by a shifted Exponential distribution with a parameter λ=0.151 for the eastbound data and λ=0.145 for the westbound data. A typical value of λ=0.15 will be used in this report. This would result in a mean value for the overloaded permit trucks equal to:

$$\overline{P} = P + \frac{1}{\lambda} \tag{39}$$

Where in this case, $\overline{P}$ is the mean weight of the overloaded permits, P is the permit weight and λ is the exponential distribution parameter. The standard deviation of the overloads is given as:

$$\sigma_P = \frac{1}{\lambda} \tag{40}$$

Since no other data is available on overloaded routine permits, in this report we will assume that all routine permit trucks may be overloaded in such a way that the trucks follow an exponential probability distribution function with the same parameter λ=0.15. Special permits (typically non-divisible loads) are assumed to have weights exactly equal to the permitted load.

As an example, the results indicate that when Type 6 Permit trucks with a Permit gross weight limit of 120 kips are provided with multiple crossing permits, the actual average weight of the trucks would be $\overline{P}$ =127 kips (120+1./0.15) with a standard deviation σ_P=6.67 kips.

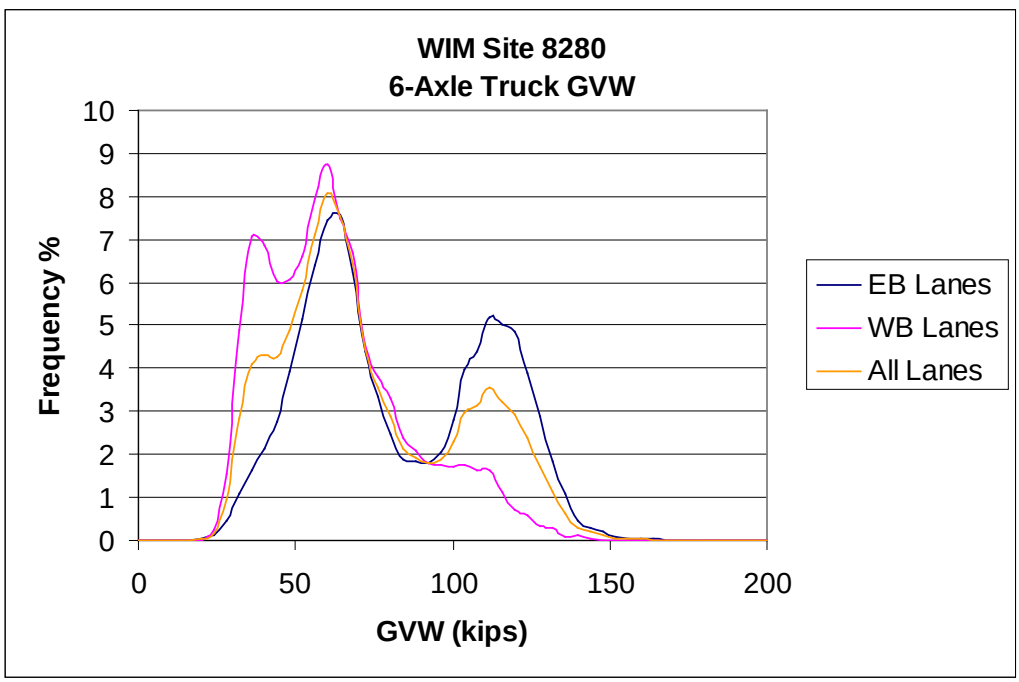

Figure 14. Gross weight histogram for NY Type 6 permit trucks.

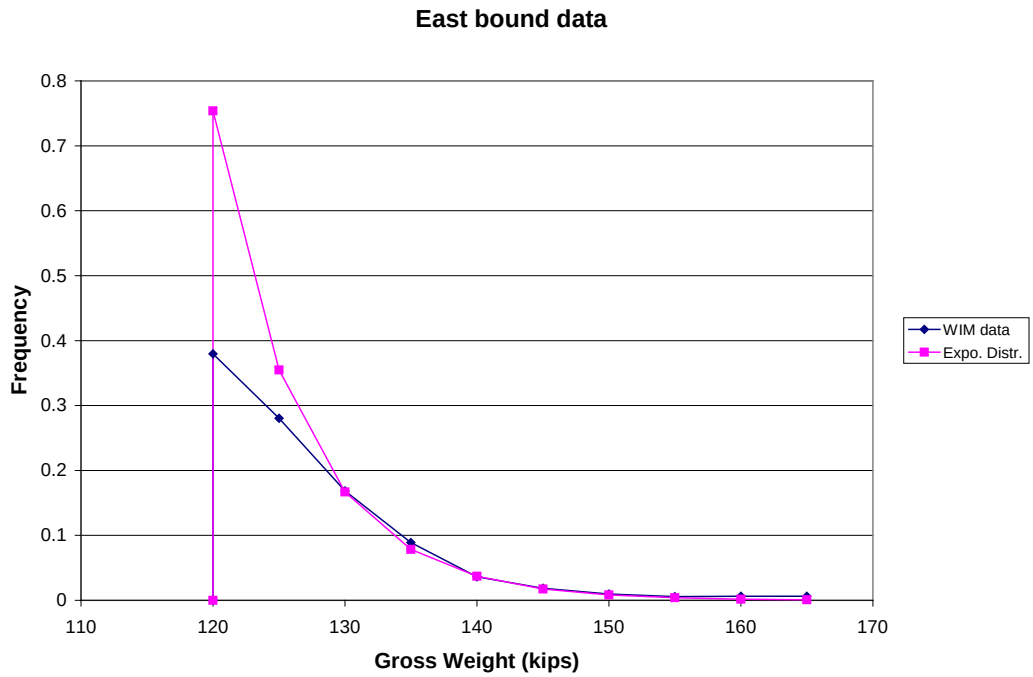

Figure 15. Exponential Distribution fit to WIM data for NY Routine Permits

Reliability Analysis for Case I – Permit Vehicle Alone

Deterministic Special Permits

In the case where a special permit truck is alone on the bridge, we can assume that the axle configuration and axle weights of the permit truck are perfectly known so that the total maximum static live load effect on the bridge P is a deterministic value. However, this does not imply that the total live load effect on a bridge member is deterministic due to the uncertainties in estimating the dynamic effect represented by the dynamic amplification factor, IM, and the uncertainties in the structural analysis process that allocates the fraction of the total load to the most critical member. For multi-girder bridges, the structural analysis is represented by the load distribution factor, D.F. The equations for the D.F. of multi-girder bridges loaded by a single lane given in the AASHTO LRFD specifications already include a multiple presence factor MP=1.2. Nowak (1999) assumed that the equations provided in the AASHTO LRFD for calculating D.F. give on the average good approximations to the actual distribution factor. This assumption will be revisited in the course of this study to find the mean DF value and its COV V_{DF}. In any case, the expression for estimating the mean value of the maximum load effect on the most critical beam when a single vehicle is on the bridge can be calculated from:

$$\overline{LL} = \overline{P} \times \overline{IM} \times \overline{D.F.}/1.2 \qquad (41)$$

Where P is the effect of the permit truck, IM is the dynamic amplification factor and D.F. is the load distribution factor. Dividing D.F. of one lane by 1.2 is done to remove the AASHTO LRFD multiple presence factor.

Assuming that the weight and axle configuration of the permit vehicle are exactly known, the Coefficient of Variation (COV) of the maximum beam live load effect is obtained from the COV of IM, V_{IM}, and the COV of DF, V_{DF}:

$$V_{LL} = \sqrt{V_{IM}^2 + V_{DF}^2} \tag{42}$$

For the random variables considered in Eq. (39) we will use the data of Nowak (1999) who observed that the actual dynamic amplification factor augmented the load effect of a single truck by an average of $\overline{IM} = 1.13$ or an additional 13% for one lane of traffic and was associated with a COV V_{IM}=9%.

The reliability will be executed for a set of typical permit vehicles.

The analysis of the permit loads will be performed for typical bridges having the dead load data assembled in this study for concrete T-beam, prestressed concrete and steel I-girder bridges. Table 1 gives examples of the dead weight data for simple span composite steel bridges.

The steps that will be followed to perform the recalibration of the Permit Load Factors for Special Permit loads will follow similar steps to those outlined earlier as follows:

- Using the AASHTO LFR live load factors and assuming a Rating Factor R.F. =1.0, use Equation (19) to find the nominal resistance, R_n, for the permit vehicles for each of the bridge configurations assembled (see for example Table 1).
- Use Eq. (20) and (21) to find the mean, $\overline{R}$, COV, V_R, and standard deviation, σ_R, for each member resistance in the assembled bridge population.
- Use Eq. 15, 16, 17 and 18 to assemble the mean, $\overline{DL}$, COV V_{DL}, and standard deviation, σ_{DL}, for the dead load applied on each member of the assembled bridge population.
- Use Eq. 41 and 42 along where P is the load effect of the trucks to assemble the mean, $\overline{LL}$, COV V_{LL}, and standard deviation, σ_{LL}, for the live load applied on each member of the assembled bridge population.
- Set up the failure function of Eq. (3) as given earlier in Eq. (38) which is repeated below:

  $$Z = R - S = R - DL - LL \tag{38}$$
- Use Eq. (38) to find the reliability index, β, for each member of the bridge population assembled and for each of the Permit trucks
- Study the reliability index to find its average value and how it varies with span length, beam spacing or bridge cross section and material type.
- If necessary adjust the AASHTO LRFR live load factors to uniformly match the new target reliability level.
- Repeat the steps for different Permit Load factors until you find the Permit factor that will produce an average reliability index as close as possible to β_{target} for all the bridge configurations and for all the Permit configurations.

Random Weight Routine Permits

As mentioned earlier, in the case where we have multi-crossings of routine permits, it is possible to find that some of the permitted trucks may have gross weights exceeding the permit

limit. In these cases, the Permit truck load effect, P, in Equation 39 cannot be assumed to be deterministic. As observed from the data collected at WIM site 8280, the permit truck gross weight may be assumed to follow an exponential distribution where P is the effect of a shifted gross vehicle weight distribution with parameter λ=0.15. The shift corresponds to the weight of the permit truck.

In this case, the reliability index calculations will be repeated for the typical Routine Permits having the configurations presented in Table 23. The table provides a representative sample of typical Routine Permits in the U.S. U.S. Typical permit vehicles from several jurisdictions, including the various geographic regions of the country were assembled. Permits data from past LRFR projects performed by the PI was accessed for this purpose. Data on permits were also obtained from the NCHRP 12-78 project.

The steps followed are the same as those outlined for the deterministic Special permits. However, the mean permit load will be taken from Equation (39) and the COV of the Permit load from Eq. (40). The COV for the live Load LL will be obtained from:

$$V_{LL} = \sqrt{V_P^2 + V_{IM}^2 + V_{DF}^2} \tag{43}$$

The steps that will be followed to perform the recalibration of the Permit Load Factors for divisible load Routine Permits will be similar to those outlined earlier as follows:

- Using the AASHTO LFR live load factors for Operating Rating and assuming a Rating Factor R.F. =1.0, use Equation (19) to find the nominal resistance, R_n, for the permit vehicles of Table 23 for each of the bridge configurations assembled (see for example Table 1).
- Use Eq. (20) and (21) to find the mean, $\overline{R}$, COV, V_R, and standard deviation, σ_R, for each member resistance in the assembled bridge population.
- Use Eq. 15, 16, 17 and 18 to assemble the mean, $\overline{DL}$, COV V_{DL}, and standard deviation, σ_{DL}, for the dead load applied on each member of the assembled bridge population.
- Use Eq. 41 and 42 where $\overline{P}$ is the mean load effect of the trucks in Figure 14 obtained by using Eq. 39 and assemble the mean, $\overline{LL}$, COV V_{LL}, using Eq. 43 and find standard deviation, σ_{LL}, for the live load applied on each member of the assembled bridge population.
- Set up the failure function of Eq. (3) as given earlier in Eq. (38) which is repeated below:

  $$Z = R - S = R - DL - LL \tag{38}$$
- Use Eq. (38) to find the reliability index, β, for each member of the bridge population assembled and for each of the Permit trucks
- Study the reliability index to find its average value and how it varies with span length, beam spacing or bridge cross section and material type.
- If necessary adjust the AASHTO LRFR live load factors to uniformly match the new target reliability level.
- Repeat the steps for different Permit Load factors until you find the Permit factor that will produce an average reliability index as close as possible to β_{target} for all the bridge configurations and for all the Permit configurations.

Reliability Analysis for Case II – Two Routine Permits side-by-side

The crossing of Routine/Annual permits may cause a situation where two permits may be side-by-side on the bridge. For the purpose of defining the probability that such event will actually take place, we will assume that there will be up to 100 crossings of a Permit truck in one day over one particular bridge. According to the New York WIM data, this will mean that the probability of having two side-by-side Permits is 0.50%.

For two lanes loaded by the same permit truck, the mean value of the live load effect on one member can be given as:

$$\overline{LL} = \overline{P} \times \overline{IM} \times \overline{D.F.} \tag{44}$$

According to Nowak (1999), the dynamic amplification augments the truck load effect by an average of 9% for heavy side-by-side trucks or $\overline{IM}$ =1.09. The dynamic amplification also produces a Coefficient of Variation COV equal to V_{IM}=5.5%. The statistics of the distribution factor may be obtained based on the data from Puckett et al (2007). The COV of the live load LL can then be obtained from Eq. (43).

The reliability index conditional on the arrival of two side-by-side permits on the bridge can then be calculated using the FORM algorithm and the failure function given in Eq. (3). The reliability index calculated from the FORM algorithm where the live load is described by Eq. (44) is designated as β_c which is defined as the reliability index conditional on having two side-by-side trucks. The probability that a bridge member would fail given that two permit vehicles are side-by-side can be calculated from:

$$P_{f|side-by-side} = \Phi(-\beta_C) \tag{45}$$

where $\Phi(...)$ is the cumulative standard Normal distribution function. The final unconditional probability of failure will depend on the conditional probability of failure given two side-by-side permit events, $P_{f|side-by-side}$ and the probability of having a situation with side-by-side permits, P_{SxS}. Thus:

$$P_f = P_{f|side-by-side} \times P_{S \times S} \tag{46}$$

where the probability of having two-side-by-side permits P_{SxS} is related to the total number of permit crossings. The probability of two permits side-by-side is not affected by the ADTT of the site but it rather depends on the number of permits per day. In these calculations we will assume P_{SxS}=0.5% for up to 100 independent crossings of Permits in one day at one bridge site as obtained as an upper limit from Sivakumar et al (2008).

The final unconditional reliability index, β, is obtained from:

$$\beta = -\Phi^{-1}(P_f) \tag{47}$$

The steps that will be followed to perform the recalibration of the Permit Load Factors for side-by-side divisible load Routine Permits will be similar to those outlined earlier as follows:

- Using the AASHTO LFR live load factors for Operating Rating and assuming a Rating Factor R.F. =1.0, use Equation (19) to find the nominal resistance, R_n, for the permit vehicles of Table 23 for each of the bridge configurations assembled (see for example Table 1).
- Use Eq. (20) and (21) to find the mean, $\overline{R}$, COV, V_R, and standard deviation, σ_R, for each member resistance in the assembled bridge population.
- Use Eq. 15, 16, 17 and 18 to assemble the mean, $\overline{DL}$, COV V_{DL}, and standard deviation, σ_{DL}, for the dead load applied on each member of the assembled bridge population.
- Use Eq. 44 and 42 where $\overline{P}$ is the mean load effect of the trucks in Table 23 obtained by using Eq. 39 and assemble the mean, $\overline{LL}$, COV V_{LL}, using Eq. 43 and find the standard deviation, σ_{LL}, for the live load applied on each member of the assembled bridge population.
- Set up the failure function of Eq. (3) as given earlier in Eq. (38) which is repeated below:

$$Z = R - S = R - DL - LL \qquad (38)$$

- Use Eq. (38) to find the conditional reliability index, β_c, for each member of the bridge population assembled and for each of the Permit trucks
- Use Equations, 45, 46 and 47 to find the unconditional reliability index, β.
- Study the reliability index to find its average value and how it varies with span length, beam spacing or bridge cross section and material type.
- If necessary adjust the AASHTO LRFR Permit load factors to uniformly match the new target reliability level.
- Repeat the steps for different Permit Load factors until you find the Permit factor that will produce an average reliability index as close as possible to β_{target} for all the bridge configurations and for all the Permit configurations.
- It is expected to find that side-by-side permits will not govern the safety of the typical bridges because of the low probability of having side-by-side permits.

Reliability Analysis for Case III – Permit Truck alongside a Random Truck for unlimited crossings of Permits.

Load Modeling

For this case, we need to analyze the maximum live load effect that is due to the permit truck alongside the maximum truck expected to occur simultaneously in the other lane. The maximum total load effect is random and it depends on the number of side-by-side events expected within the return period.

To determine the number of side-by-side permit-random truck events that would occur within the five-year rating period, we will assume that the number of side-by-side events involving one random truck will depend on the ADTT. Based on the upper envelopes of the New York State WIM data, sites with ADTT=100 will have 0.5% of the loading events formed by side-by-side trucks. Sites with ADTT=1000 and 5000 will have 1.25% and 2% probability of side-by-side trucks respectively.

In this report, the estimation of the load effect of the random trucks that will cross a bridge alongside a Permit truck will be calculated using the actual live load effects of the trucks recorded

by WIM. The calculations of the random truck load effect, L_{max}, will follow the same steps described in Equations (22) through (26). However the number of events N must correspond to the number of permits and the probability of having a random truck alongside the permit truck. The number of events N is obtained from the ADTT and the WIM headway data at a site.

We define N_p as the number of events where a Permit will cross the bridge alongside a random truck within the five-year rating period, T. Given a five-year return period and 100 crossings of Permits per day, N_p=182,500. The percentage of side-by-side events involving a random truck is P_{sxs}. Thus, within a return period, T, there will N_R crossings of Permits alongside a random truck:

$$N_R = P_{S\times S} \times N_P \tag{48}$$

where P_{SxS} is the percentage of side-by-side events which depends on the ADTT and N_p is the number of permits within the return period of interest. For example, for a bridge site with ADTT=5000 and 100 permits per day and given a probability of side-by-side events P_{sxs}=2%, the number of events when the permit truck will be alongside a random truck within a 5-year rating period is N_R=3650 (100 truck per day x 365 days per year x 5 years x 2%).

The application of Eq. (22) through (26) will be executed for the WIM data from all WIM sites to obtain the average L_{max}, V_{Lmax} and $V_{site\text{-}to\text{-}site}$.

The COV data can be used to find the level of uncertainty associated with estimating the effect of the random truck that may cross the bridge alongside the Permit. Given that the analysis of the static load effect for one beam is associated with a COV V_{DF}, the overall coefficient of variation for the static random truck load effect $L^*_{\max} = L_{\max N_R} \times HL93 \times DF_R$ without the dynamic effect can then be estimated as:

$$V_{L\max^*} = \sqrt{V^2_{L\max} + V^2_{site\ to\ site} + V^2_{sample\ size} + V^2_{DF}} \tag{49}$$

This COV is for the effect of the random truck only..

Given N_R events where the permit truck will be alongside a random truck, the maximum load will occur when the permit truck will be alongside the one truck out of the N_R which produces the maximum load effect. The mean normalized live load effect of this truck is labeled $L_{\max N_R}$ and its actual effect is $L_{\max N_R} \times HL93$. The effect of the permit truck is P. The maximum total live load effect on a member is obtained from:

$$\overline{LL} = \left(\overline{P} \times \overline{DF}_P + \overline{L}_{\max N_R} \times HL93 \times \overline{DF}_R\right)\overline{IM} \tag{50}$$

where P is the load effect of the permit truck, DF_P is the load distribution factor for the load P, $L_{max\ N_R}$ is the maximum load effect of N_R random trucks, DF_R is the distribution factor for the random load, and IM is the impact factor for side-by-side events. $\lambda_{site\ to\ site}$ is introduced to reflect the variability in L_{max} from site to site with a mean value of 1.0 and a COV $V_{site\ to\ site}$ as per Table 6. λ_{data} is introduced to reflect the variability in L_{max} due to the WIM data sample size, it is associated with a mean value of 1.0 and a COV $V_{sample\ size}$=2% as per NCHRP 12-76. The Coefficient of variation of the live load can also be calculated from the COV of each of the parameters in Eq. (50). That is by taking the COV of L_{max*} as given in Eq. (49) and the COV of P as given in Eq. (40) and including the COV of DF and IM.

The steps to perform the recalibration of the live load factors for the cases where a permit truck can travel alongside a random truck, will follow similar steps to those outlined earlier as follows:

- Using the AASHTO LFR live load factors for Operating Rating and assuming a Rating Factor R.F. =1.0, use Equation (19) to find the nominal resistance, R_n, for the permit vehicles of Table 23 for each of the bridge configurations assembled (see for example Table 1).
- Use Eq. (20) and (21) to find the mean, $\overline{R}$, COV, V_R, and standard deviation, σ_R, for each member resistance in the assembled bridge population.
- Use Eq. 15, 16, 17 and 18 to assemble the mean, $\overline{DL}$, COV V_{DL}, and standard deviation, σ_{DL}, for the dead load applied on each member of the assembled bridge population.
- Use Equations (22) through (26) with N obtained from Eq. (48) to find $\overline{L}_{max}$ and V_{Lmax} from the WIM data.
- Use Eq. 44 and 42 where $\overline{P}$ is the mean load effect of the trucks in Table 23 obtained by using Eq. 39 and assemble the mean, $\overline{LL}$, COV V_{LL}, using Eq. 50 and find the standard deviation, σ_{LL}, for the live load applied on each member of the assembled bridge population.
- Set up the failure function of Eq. (3) as given earlier in Eq. (38) which is repeated below:

$$Z = R - S = R - DL - LL \qquad (38)$$

- Use Eq. (38) to find the reliability index, β, for each member of the bridge population assembled and for each of the Permit trucks
- Study the reliability index to find its average value and how it varies with span length, beam spacing or bridge cross section and material type.
- If necessary adjust the AASHTO LRFR Permit load factors to uniformly match the new target reliability level.
- Repeat the steps for different Permit Load factors until you find the Permit factor that will produce an average reliability index as close as possible to β_{target} for all the bridge configurations and for all the Permit configurations.

Reliability Analysis for Case IV – Permit Truck alongside a Random Truck for Single Crossings of Permits.

In this case the same logic followed for Case III applies. However the number of permit crossings is only N_p=1 which according to Eq. (48) will lead to a side-by-side crossing rate less than 1 and Eq. (22) through (26) are not valid. For this reason, in this set of reliability calculations we need to use the Monte Carlo simulation to extract possible values for the random truck that could cross a bridge alongside a Permit truck directly from the load effect histograms obtained from the WIM data.

The Monte-Carlo simulation requires the performance of an analysis a large number of times and then assembling the results of the analysis into a histogram that will describe the scatter in the final results. The process can be executed for the situation where we have a permit truck side-by-side with a random truck as depicted in Figure 16 which gives a schematic representation of the Monte Carlo Simulation to execute the probability that the safety margin of Eq. (38) along with the data from Eq. (50) is less than 0. This will produce the conditional probability of failure

$P_{f|side-by-side}$ assuming an actual occurrence of side-by-side permit and random truck. The unconditional probability of failure will be obtained from:

$$P_f = P_{f|side-by-side} \times P_{S \times S} \tag{51}$$

The final unconditional reliability index, β, is obtained from

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{52}$$

The reliability index will then be used as outlined for Case III to find the Permit Load Factor.

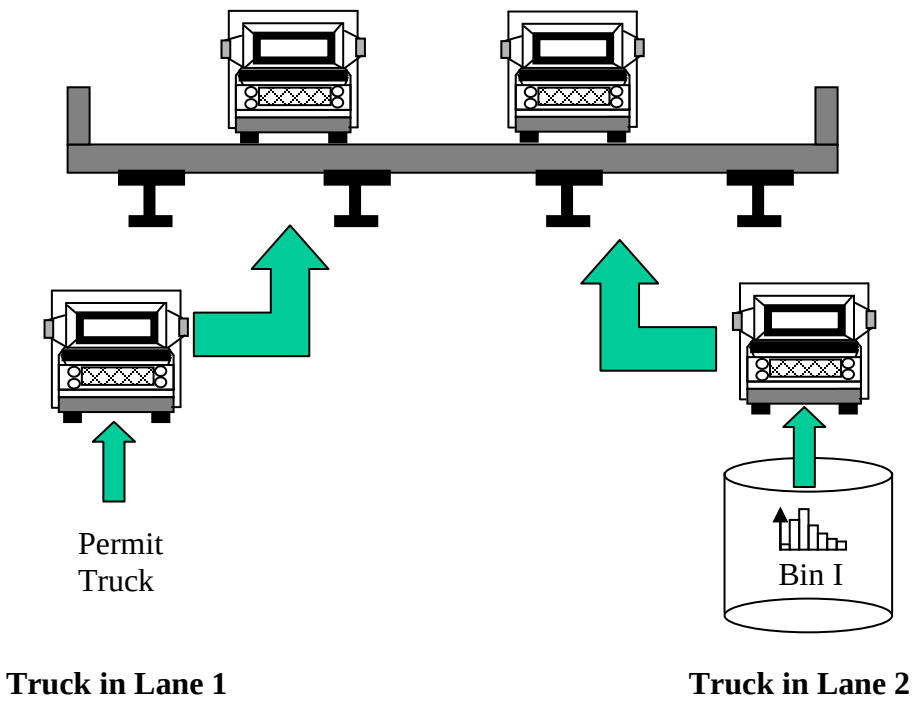

Figure 16. Schematic illustration of Monte Carlo simulation procedure.

CHAPTER 3

FINDINGS AND APPLICATIONS

CURRENT RELIABILITY LEVELS FOR LFR LEGAL LOAD RATINGS

Two-Lane Bridges that Satisfy AASHTO LFR Criteria

To find the reliability levels implied when a bridge is rated using the current AASHTO LFR criteria, the set of bridge configurations previously used during the AASHTO LRFD calibration is used. The bridge member resistances were calculated assuming that each bridge produces an LFR Operating Rating Factor exactly equal to R.F.=1.0 when rated for the AASHTO Legal Loads with a live load factor γ_L=1.30. In this case, all the bridges are assumed to be two-lane bridges.

The reliability analysis was performed for the typical bridge configurations consisting of T-beam bridges with spans varying between 20-ft and 120-ft with beams spaced from 4-ft to 12-ft center to center. Also, prestressed concrete, noncomposite and composite steel bridges with spans varying between 20-ft and 200-ft are analyzed for the same beam spacings varying between 4-ft and 12-ft. The analysis was performed for the midpsan moment and maximum shear.

The reliability analysis compares the resistance to the applied loads while accounting for the uncertainties associated with estimating these variables. To find the mean value and the COV of the applied live loads, the analysis assumed three different ADTT volumes ranging from ADTT=5000, 1000 and 100 with probability of side-by-side P_{sxs}= 2%, 1.25% and 0.5% respectively. The analysis assumed that two lanes of loading govern. It is noted that the L_{max} values representing the applied live load moments and shear forces are considerably higher than the moments and shear forces of the AASHTO legal trucks. This indicates that the AASHTO legal trucks do not provide an adequate envelope of the actual loads on highway bridges leading to lower reliability levels than might be expected.

Besides the expected increase in the reliability index with the lower ADTT, the results showed large variability in the reliability indexes depending on the span length, beam spacing and material type. Extremely low reliability index values were observed for very short spans for the bending of T-beam, prestressed concrete beams and composite I-beam bridges. Table 12 provides a summary of the average values for each bridge type and ADTT for both moment and shear. The overall average is on the order of 1.35 which is considerably lower than the 2.25 value obtained in previous studies. This lower value is due to the higher loads observed on many of the WIM sites which were used to extract the L_{max} values for this calibration effort. Also, the large variability in the observed L_{max} values form site to site contributed to lowering the reliability index. The reliability indices are particularly low for short span bridges (< 40 ft) which are typically governed by heavy axle groups.

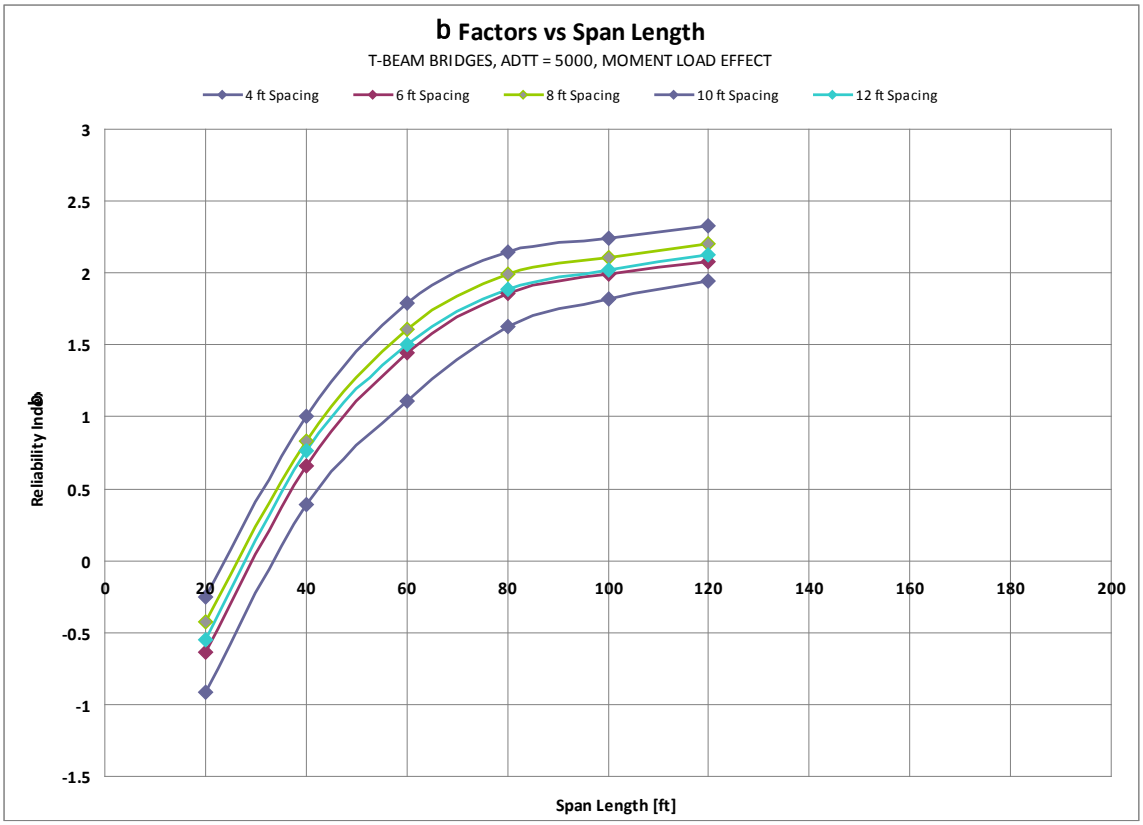

Figure 17. Reliability Index Values for T-beam bridge members in bending (ADTT=5000)

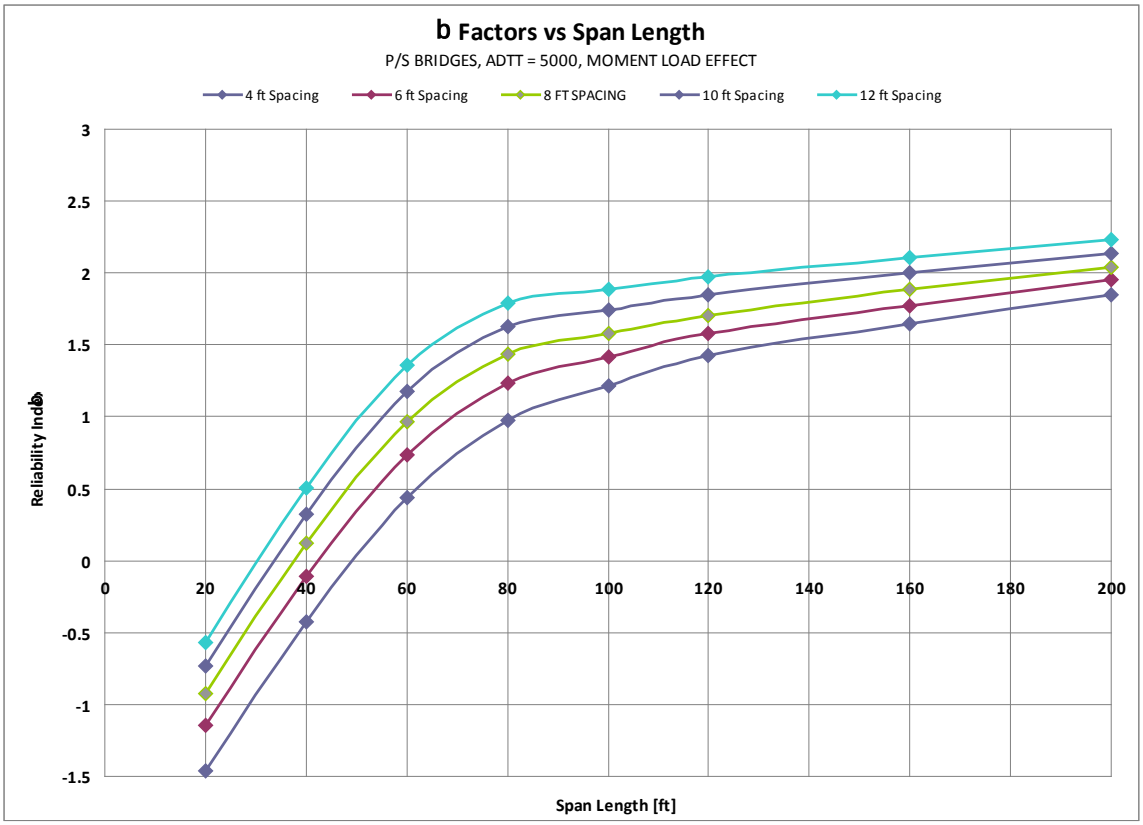

Figure 18. Reliability Index Values for P/S bridge members in bending (ADTT=5000)

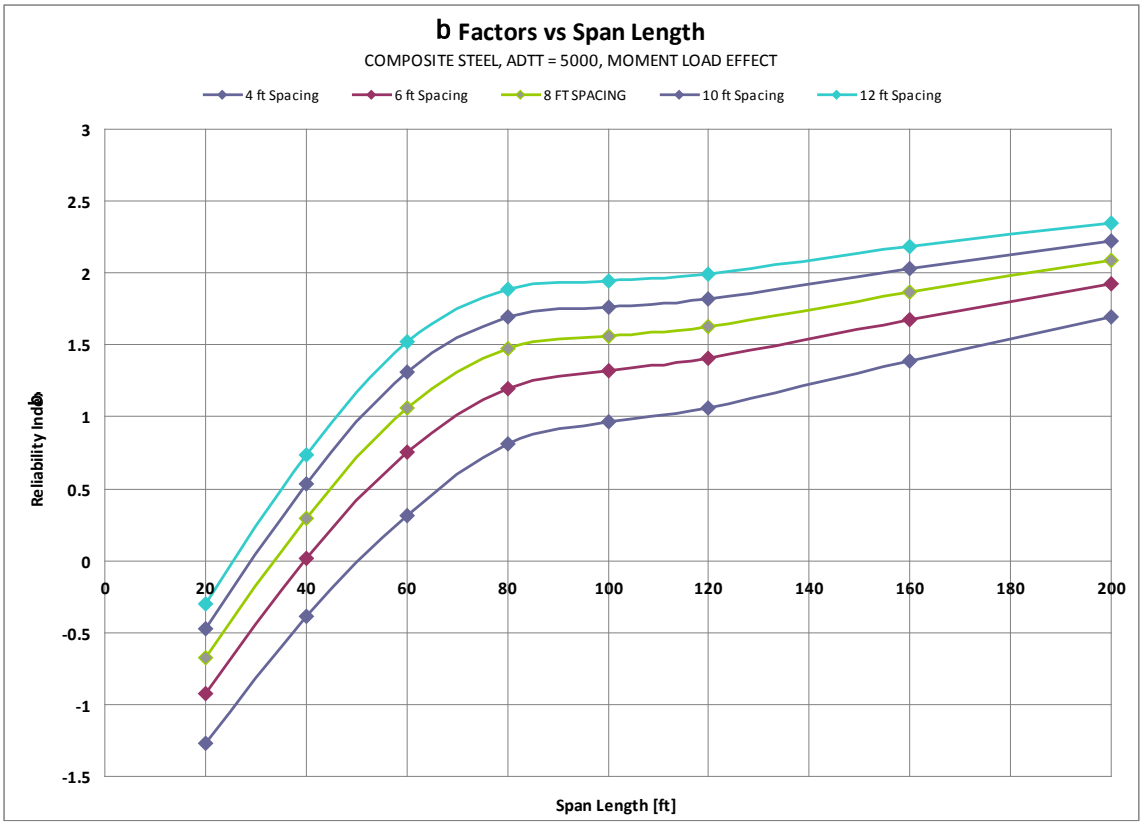

Figure 19. Reliability Index Values for Composite Steel bridge members in bending (ADTT=5000)

Table 12. Average Reliability Index Values for Random trucks for different Bridge Type and ADTT

	ADTT=5000		ADTT=1000		ADTT=100	
	Moment	Shear	Moment	Shear	Moment	Shear
T-BEAM	1.291	0.978	1.855	1.180	1.855	1.558
P/S	1.083	0.995	1.329	1.204	1.783	1.595
NC STEEL	1.124	0.809	1.361	1.067	1.799	1.547
Comp. Steel	1.113	0.793	1.351	1.052	1.790	1.533

CURRENT RELIABILITY LEVELS FOR LFR ROUTINE PERMIT RATINGS

Two-Lane Bridges that Satisfy AASHTO LFR Criteria

The reliability analysis was performed for the same typical bridge configurations studied above assuming that AASHTO LFR Operating Rating Factor for each permit truck of the set of typical Permit trucks is exactly equal to R.F.=1.0. The member resistances that would be required to allow a permit truck to cross each bridge are calculated assuming that the bridges are being evaluated using the current AASHTO LFR Operating rating criteria with a live load factor applied on the Routine Permit equal to γ_L=1.30 and using the two lane distribution factor.

The reliability analysis compares the member resistance of each bridge to the applied loads accounting for the uncertainties associated with estimating the resistance and the loads. To find the applied live load, the analysis assumed that each permit type may travel 100 times over the same bridge in one given day. To find the probability of having a permit truck alongside a random truck, the analysis assumed three different ADTT volumes ranging from ADTT=5000, 1000 and 100 with probability of side-by-side P_{sxs}= 2%, 1.25% and 0.5% respectively. The statistics for the load effects of the random truck are provided in Tables 13, 14 and 15. The weight of the permit truck itself is also assumed to be random and that it may possibly exceed the permitted weight. Therefore, the Permit weight is assumed to be on the average 7 kips above the permitted weight and that it follows an Exponential probability distribution. This assumption is made based on a limited set of observations made in New York State.

Besides the expected increase in the reliability index with the lower ADTT, the results showed large variability in the reliability indexes depending on the span length, beam spacing, material type and permit truck. As an example, Figure 20 depicts the range of reliability index values observed for composite steel bridges with beams at 8-ft center to center and ADTT=5000. The solid line gives the average reliability index for all the 32 Permits analyzed while the dashed lines give the upper and lower bounds. Figures 21, 22 and 23 show plots of the average values for different beam spacing for the T-beam bridges, P/S, and composite steel bridge members in bending. The results are summarized in Table 16. In this case the <u>overall average reliability index is on the order of 2.94</u>. This average value is considerably higher than that observed for the legal load rating (random trucks on span). This is because the rating process uses the actual permit load to find the rating and it also assumes that the random truck alongside the permit is also of equal weight. The difference between the high reliability index values obtained for permits as compared to random truck crossings is also due to the fact that the permit trucks weights are much better known and are associated with lower Coefficient of Variation values than the random trucks. Although the permit truck may still cross the bridge with a random truck, the chances of having a permit truck alongside a truck of equal or higher weight are relatively low. These factors lead to significantly higher reliability index values for the permit trucks than in the cases of random loading when the same AASHTO LFR live load factor γ_L=1.30 is specified.

Table 13. Lmax values for Random Truck alongside Permit for ADTT=5000

ADTT 5000	AVERAGE OF ALL WIM SITES					
	SIMPLE MOMENT			SIMPLE SHEAR		
	1-LANE			1-LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.32	0.07	0.23	1.32	0.07	0.21
40	1.28	0.07	0.20	1.24	0.07	0.18
60	1.17	0.08	0.18	1.24	0.07	0.19
80	1.15	0.08	0.18	1.27	0.07	0.19
100	1.16	0.08	0.19	1.28	0.08	0.19
120	1.16	0.08	0.20	1.26	0.08	0.18
160	1.14	0.08	0.19	1.21	0.08	0.17
200	1.08	0.09	0.18	1.14	0.09	0.17

Table 14. Lmax values for Random Truck alongside Permit for ADTT=1000

ADTT 1000	AVERAGE OF ALL WIM SITES					
	SIMPLE MOMENT			SIMPLE SHEAR		
	1-LANE			1-LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.28	0.07	0.23	1.29	0.07	0.22
40	1.25	0.08	0.20	1.20	0.08	0.19
60	1.13	0.08	0.18	1.21	0.08	0.19
80	1.11	0.08	0.19	1.24	0.08	0.20
100	1.13	0.08	0.20	1.24	0.08	0.19
120	1.13	0.08	0.20	1.23	0.09	0.19
160	1.10	0.09	0.20	1.17	0.09	0.18
200	1.05	0.09	0.19	1.10	0.09	0.18

Table 15. Lmax values for Random Truck alongside Permit for ADTT=100

ADTT 100	AVERAGE OF ALL WIM SITES					
	SIMPLE MOMENT			SIMPLE SHEAR		
	1-LANE			1-LANE		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.21	0.08	0.23	1.22	0.08	0.22
40	1.17	0.09	0.21	1.13	0.09	0.20
60	1.06	0.10	0.20	1.14	0.09	0.20
80	1.04	0.10	0.20	1.16	0.09	0.21
100	1.06	0.09	0.21	1.17	0.09	0.20
120	1.06	0.10	0.22	1.15	0.10	0.20
160	1.03	0.10	0.21	1.09	0.10	0.20
200	0.98	0.11	0.21	1.02	0.11	0.20

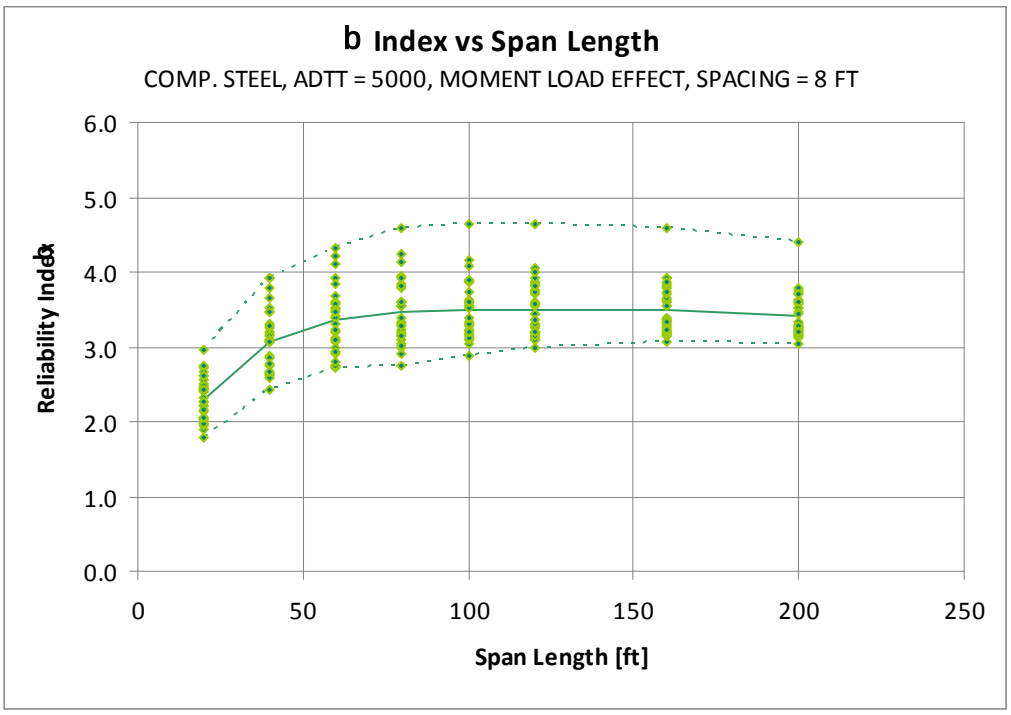

Figure 20. Range of Reliability Index values for various Permit Trucks on Composite steel bridge members at 8-ft spacing in bending (ADTT=5000).

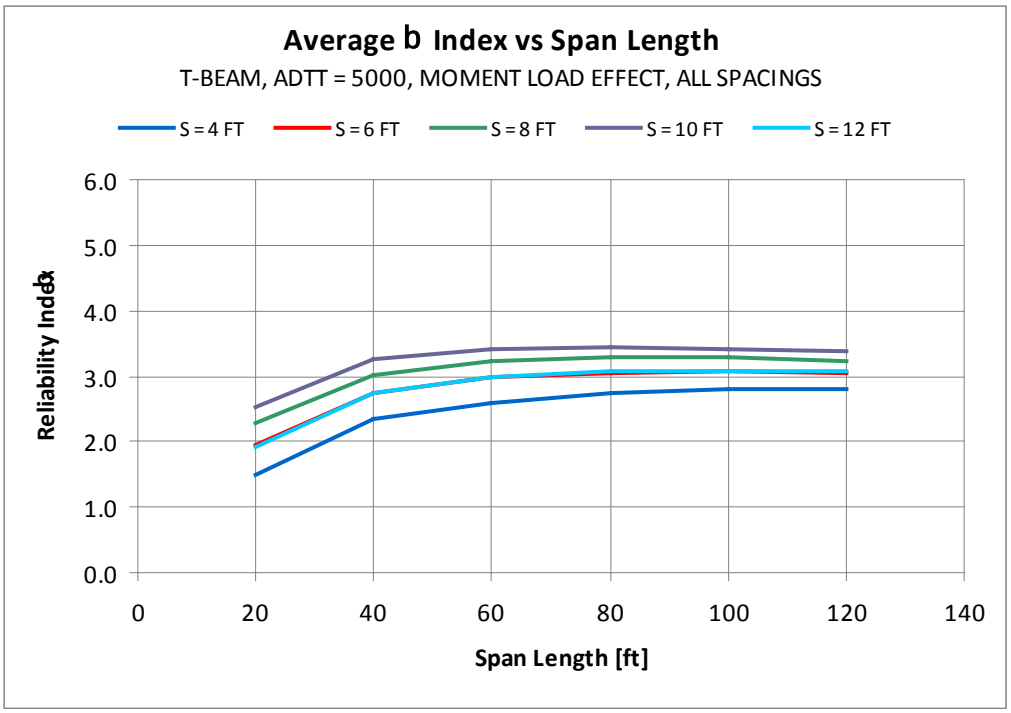

Figure 21. Average Reliability Index values for Permit Trucks on T-beam bridge members in bending (ADTT=5000).

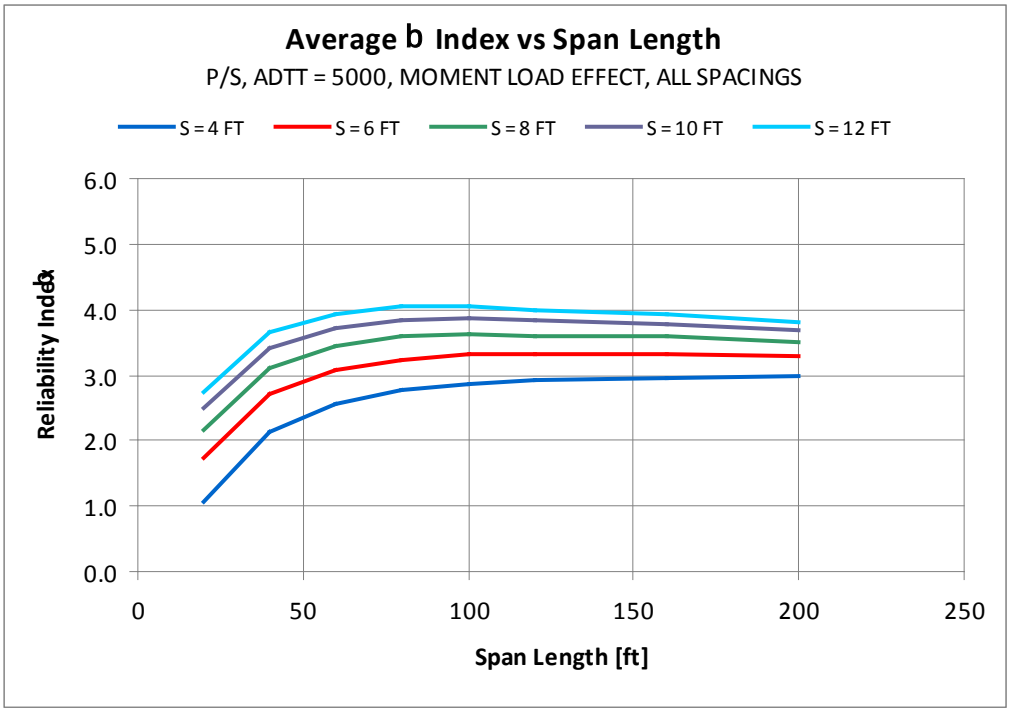

Figure 22. Average Reliability Index values for Permit Trucks on P/S bridge members in bending (ADTT=5000).

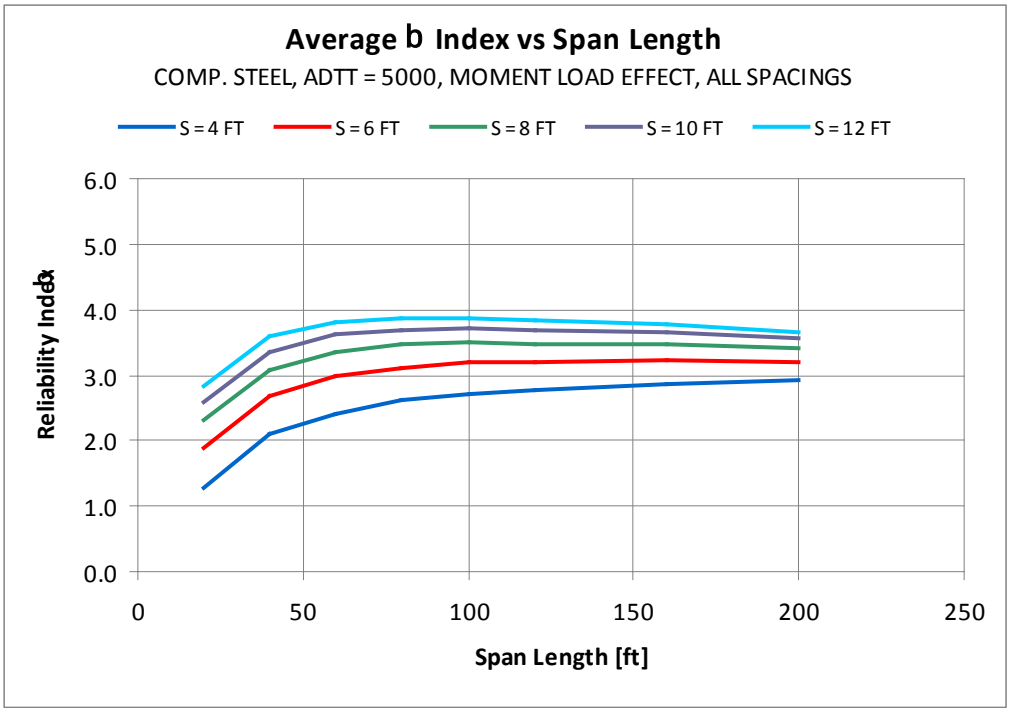

Figure 23. Average Reliability Index values for Permit Trucks on composite steel bridge members in bending (ADTT=5000).

Table 16. Average Reliability Index Values for Permit trucks on two-lane bridges for different Bridge Type and ADTT

	ADTT=5000		ADTT=1000		ADTT=100	
	Moment	Shear	Moment	Shear	Moment	Shear
T-BEAM	2.71	2.58	2.75	2.61	2.83	2.68
P/S	2.99	2.66	3.05	2.69	3.25	2.76
NC STEEL	3.01	3.10	2.99	3.14	3.10	3.23
Comp. Steel	2.93	3.09	2.98	3.13	3.10	3.23

Single-Lane Bridges that Satisfy AASHTO LFR Criteria

The reliability analysis was again repeated for the same typical bridge configurations studied above assuming that AASHTO LFR Rating Factor for each permit truck of the set of typical permit trucks is exactly equal to R.F.=1.0. For this set of calculations, the analysis is performed for single-lane bridges. The member resistances that would be required to allow a permit truck to cross each bridge are calculated assuming that the bridges are being evaluated using the current AASHTO LFR Operating rating criteria with a live load factor applied on the Permit equal to γ_L=1.30 and using the one lane distribution factor.

The reliability analysis compares the member resistance of each bridge to the applied loads accounting for the uncertainties associated with estimating the resistance and the loads. To find the applied live load, the analysis assumed that the weights of each Permit truck are random and may possibly exceed the permitted weight. Therefore, the Permit weight is assumed to be on the average 7 kips above the permitted weight and that it follows an Exponential probability distribution. This assumption is made based on a limited set of observations made in New York State.

The results in this case are presented in Table 17. The average reliability index for all the cases considered is on the order of 3.62 which is even higher than that obtained for the Permits crossing over two-lane bridges. This difference is largely due to the fact that on one-lane bridges, the permit is alone on the bridge which makes it that the total applied load is better known than in the case where the truck may cross alongside a random truck which would govern the analysis of permit crossing two-lane bridges.

Table 17. Average Reliability Index Values for Permit trucks on single lane bridges

	All ADTT's	
	Moment	Shear
T-BEAM	3.60	3.39
P/S	4.01	3.04
NC STEEL	3.91	3.56
Comp. Steel	3.91	3.55

The analysis performed above assumed that routine Permits may run on the average at a weight equal to 7-kips above the permitted weight and that the distribution of the Permit weight is exponential with a minimum value equal to the permit weight. To verify the sensitivity of the results to this assumption which was made based on very limited data on one site in New York, the same calculations are repeated assuming that the average weight is 15% higher than the permitted weighted and that the weight will also follow an exponential distribution. The results show a drop in the calculated reliability index so that the average reliability index for the Permit trucks crossing the bridge configurations considered drops from the originally calculated β=3.62 to β=3.12. A summary of the results calculated for the typical bridge types analyzed is provided in Table 18.

Table 18. Average Reliability Index Values for Permit trucks on single lane bridges assuming an average weight 15% higher than the permit weight

	All ADTT's	
	Moment	Shear
T-BEAM	3.47	2.96
P/S	3.33	2.96
NC STEEL	3.30	2.83
Comp. Steel	3.30	2.82

Reliability Analysis for Special Permit Loads

In the case of Special Permits we can assume that the axle configuration and axle weights of the permit truck are perfectly known so that the total maximum static live load effect on the bridge P is a deterministic value.

Case I; Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.

Case IV: Single Permit trips where the Permit truck could mix with other random vehicles.

Case I is not affected by the WIM data. Special permits will have no uncertainties. So, if they are running alone they will have higher betas than routine permits. For Case IV, the reliability analysis should account for the number of random vehicles.

The calculations of the reliability index values for Permit trucks performed in this report are for Routine Permits which are assumed to run at about a rate of 100 trucks per day on each bridge analyzed and their weights are assumed to have some level of uncertainty in the sense that they may actually exceed the permitted weight. For special permits, previous analyses have shown that the reliability index values will be higher than those of the routine permits. This is because, unlike routine permits, special permits will have: a) no uncertainties in their weights and b) will run only once on a bridge reducing the probability of running alongside a heavy random truck. Therefore, if Special Permits are running alone on one lane bridges they will have higher betas than routine permits because their weights are better known. Also, because the special permits will run only one at a time, when they are running alongside a random truck on multi-lane bridges there is a lower probability that the random truck is very heavy.

TARGET RELIABILITY INDICES FOR LRFR RECALIBRATION

The reliability index values obtained using the reliability calculation process is summarized below for random loads and for permit loads:

Table 19. Average Reliability Index Values for Random Trucks (non-permit)

	ADTT=5000		ADTT=1000		ADTT=100	
	Moment	Shear	Moment	Shear	Moment	Shear
T-BEAM	1.291	0.978	1.855	1.180	1.855	1.558
P/S	1.083	0.995	1.329	1.204	1.783	1.595
NC STEEL	1.124	0.809	1.361	1.067	1.799	1.547
Comp. Steel	1.113	0.793	1.351	1.052	1.790	1.533

Table 20. Average Reliability Index Values for Permit Trucks on Two-Lane Bridges

	ADTT=5000		ADTT=1000		ADTT=100	
	Moment	Shear	Moment	Shear	Moment	Shear
T-BEAM	2.71	2.58	2.75	2.61	2.83	2.68
P/S	2.99	2.66	3.05	2.69	3.25	2.76
NC STEEL	3.01	3.10	2.99	3.14	3.10	3.23
Comp. Steel	2.93	3.09	2.98	3.13	3.10	3.23

Table 21. Average Reliability Index Values for Permit Trucks on Single Lane Bridges

	All ADTT's	
	Moment	Shear
T-BEAM	3.60	3.39
P/S	4.01	3.04
NC STEEL	3.91	3.56
Comp. Steel	3.91	3.55

The reliability analysis was performed for typical bridge configurations assuming a member resistance such that AASHTO LFR Rating Factor for each loading is exactly equal to R.F. =1.0 at the Operating level.

The L_{max} values representing the applied live load moments and shear forces from actual traffic representing non-permit loads are considerably higher than the moments and shear forces of the AASHTO Legal Trucks. This indicates that the AASHTO Legal Trucks do not provide an adequate envelope of the actual loads on highway bridges leading to lower reliability levels than might be expected. The results showed large variability in the reliability indexes depending on the span length, beam spacing and material type. Extremely low reliability index values were observed for very short spans. Also, the large variability in the observed L_{max} values form site to site contributed to lowering the reliability index. The beta values range from 0.81 to 1.86. The overall average is on the order of 1.35 for legal loads.

The overall average reliability index for Routine Permits is on the order of 2.94 for two lane bridges and even higher for one lane bridges. The beta values range from 2.58 to 3.25 for two lane bridges. The analysis considered the possibility that some of the Routine Permit trucks may have gross weights exceeding the permit limit. This average value is considerably higher than that observed for the random non-permit trucks.

In the case of Special Permits we can assume that the axle configuration and axle weights of the permit truck are accurately known so they will have higher betas than Routine permits. Authorized. Also, for a single trip or a limited number of crossings permit there is a lower probability that the random truck running alongside is heavy, compared with Routine Permits.

The reliability index values obtained using the reliability calculation process, should not be taken as actuarial values but should only be used as a means to compare different design and rating options. This means that having a reliability index β=2.0 does not necessarily imply that 2.3% of the bridges are actually failing. This is because the reliability analysis process ignores many of the modeling uncertainties that make our bridges much safer than implied by these numbers. As an example the AASHTO LRFD code calibration process, which is followed in this report, assumes that the LRFD distribution factors give on the average an accurate evaluation of the load distribution in actual bridges. However, as demonstrated by Puckett et al (2007), the AASHTO LRFD load distribution tables are conservative implying a hidden safety factor that has not been taken into consideration. There are many other such hidden factors that are not included in the reliability analysis. These factors are usually lumped under the term modeling uncertainties. Other modeling uncertainties include the differences between the shear equations used in the AASHTO LFR codes and those of the LRFR code when compared to results of actual tests and other similar factors.

Because of the fact that the reliability index calculations do not normally account for all the modeling uncertainties, traditionally, the target reliability level that a new code is calibrated to produce is extracted from the reliability of bridges that have shown good performance in the past. If on the average, bridge engineering experts are satisfied with the old code, the average reliability index obtained from a representative sample of bridges that satisfy the old code is used as the target that the new code should satisfy. In general the old code shows large variations in the reliability index values based on the span length, beam spacing, material type and other parameters. The new code calibration process proposes a nominal load, as well as a set of resistance and live load factors that will produce uniform reliability index values for the whole range of applicable designs and provide a narrow range of reliability index values The object is to have a network with as little variation in the safety level between the bridges of the network as possible.

The analysis performed in this NCHRP 20-07 study has shown large variations in the reliability index values of bridges that satisfy the current AASHTO LFR method. In particular, it is noted that short span bridges with narrow beam spacing produce very low reliability levels which increase as the span length and the beam spacing is increased. The legal load ratings show considerably lower reliability levels than those for the permit trucks. These variations are due to many factors including the fact that the AASHTO LFR uses load distribution factors that are known to be highly inaccurate with various levels of inaccuracies based on the span length and

beam spacing. Also, the AASHTO LFR does not account for the uncertainties in estimating the various loads including the dead load, the permit load and the random truck load.

Figures 17, 18, 19, and 20 through 23 as well as many similar plots obtained in these calculations show that bridges with short spans and narrow beam spacing have considerably lower reliability index values than longer spans and bridges with larger beam spacing. The figures also show that the LFR load permit load rating is much more conservative than LFR legal load rating method. For the legal load rating, the analysis for shear produces lower reliability levels than those for the moments. For spans of 80-ft and higher, the legal load rating produces reliability index values that roughly vary between 1.0 and 2.5 for sites with ADTT=5000. The average value for all the sites, ADTT's, material types, span lengths and beam spacing is on the order of 1.35. But, this value does not take into consideration the actual number of bridges of each category and type. Also, there is evidence to suggest that truck weights and volumes are increasing over time.

Reliability analyses of legal loads have shown that the average reliability index for existing bridges under random loads is less than 2.0. This is considerably lower than the reliability indices for LFR permit evaluations. The overall average reliability index for Routine Permits is on the order of 2.94. The current rating methods that apply the same load factor for permits and random trucks may be imposing unnecessary restrictions on truck permits. The fact that so many of the random trucks (non-permit trucks) are running at considerably higher loads than the Legal limits does not seem to justify imposing high live load factors on permit trucks.

RECALIBRATION OF LRFR LEGAL LOAD FACTORS

Reliability analyses of LFR legal load ratings have shown that the average beta value is on the order of 1.35. Using an average beta value for national calibration of legal load factors would not be a recommended approach due to the fact that there is great disparity in random non-permit traffic as seen by the calculated Lmax (5-year maximum load effects) values given in the Table 22 below. If an average beta is used for national calibration many states such as CA, IN will be burdened with high load factors on account of the heavy traffic seen in some states such as NY and FL. States with effective control of weight limits should not be penalized for lax enforcement in other states. Evaluation approaches should be geared to traffic within the state, not the worst case national loading.

Table 22 Lmax Simple Moment 2-Lanes

State		Span Length [ft]							
		20	**40**	**60**	**80**	**100**	**120**	**160**	**200**
Direction 1 & 2	**CA**	1.671	1.611	1.599	1.628	1.667	1.695	1.739	1.705
	FL	2.499	2.421	2.112	2.012	1.994	1.980	1.870	1.745
	IN	1.748	1.794	1.702	1.681	1.709	1.703	1.658	1.589
	MS	2.244	2.211	2.035	2.019	2.048	2.087	2.185	2.137
	NY	2.872	2.649	2.337	2.286	2.346	2.398	2.317	2.180
	TX	1.889	2.012	1.837	1.750	1.744	1.716	1.664	1.585

For an optimal approach, it is recommended that legal load calibrations should be performed based on state-specific traffic loads and reliability targets. Some states load rate their bridges for heavier grandfathered loads which could lead to higher beta indices than that obtained for load ratings based on AASHTO legal loads. Also, recent WIM data has shown the significant differences that exist in traffic conditions from state to state. The presence of grandfathered vehicles, trucks with divisible loads permits, and the general compliance with truck weight limits

are all contributing factors to the differences in Lmax values. These studies have shown that one national recalibration of legal load factors using average traffic statistics and a single reliability target is not justifiable nor economical. Furthermore, national adoption of these load factors that are likely to be somewhat higher than current values is considered unlikely.

Other approaches to adjustment of legal load factors can be considered. Some states have embarked on their own state specific recalibrations of LRFR as the most optimal approach to achieving load factors that are geared to their traffic environment and operational needs. Differences in truck traffic exist between the Interstates and non-Interstates within a state. Such variabilities can be better explored in a state specific calibration of legal loads. Conditions can be identified in the AASHTO MBE with limits within which a state must have its Lmax values (based on state WIM data) in order for the current AASHTO LRFR calibrations to be valid. If the WIM data is more severe than used in the LRFR calibrations, and the Lmax is higher, then the state should undertake their own LRFR recalibrations using state specific WIM data, as was recently done by NYSDOT. Currently the MBE provides guidance on using site-specific WIM data and calculation of Lmax values. This guidance can be enhanced to cover state-specific recalibrations for legal loads.

RECALIBRATION OF LRFR PERMIT LOAD FACTORS

Reliability Targets for Permit Load Recalibrations

It is recommended that average target reliability index values similar to legal loads be used for permit truck calibrations. In order to account for known uncertainties in permit loads without considerably increasing the future costs to our bridge infrastructure system, it is herein recommended that a target reliability index set for the calibration is β_{target}=2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50. The 2.5 beta represents the higher end of safety indices inherent in current ratings for random traffic and should be entirely appropriate for permit traffic that has much lower uncertainties.

Permit Load Classifications

The AASHTO LRFR uses several different Permit classifications depending on the permit type and the number of trips allowed. The categories are divided as follows:

a) Routine Permits: These are trucks that are issued annual permits for unlimited number of trips and can mix with regular truck traffic such that a random heavily loaded truck may cross the bridge simultaneously with the Permit;
b) Special Multiple-trip Permits: In this case, the same truck is expected to make less than 100 crossings of a bridge within the permit validity period;
c) Single-trip Special Permits: The permit truck is allowed to cross a bridge one time only but can mix with regular truck traffic; and
d) Escorted Single-trip Special Permits: In this case, the permit truck crosses the bridge with no other vehicles allowed on the bridge.

Special Permits are assumed to be controlled so that the truck weights are known to be equal to the permit weight. Routine Permits, which are issued for a year's period, are less easily controlled and some of these have been observed to be overloaded by exceeding the permit weight limits.

To account for all possible load combinations, the recalibration of the Permit live load factors for the AASHTO LRFR for short to intermediate span bridges will consider the following four cases:

I. Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.
II. Unlimited crossings of multiple trip permits where two Permit trucks could cross a bridge simultaneously side-by-side.
III. Unlimited crossings where a Permit truck mixes with other random vehicles.
IV. Limited number of trips (less than one hundred total) where the Permit truck can mix with other random trucks. The single Permit trips will be considered as part of this case.

Cases I, II and III are valid for Routine Permits where the actual truck weight may sometimes exceed the weight limit. Cases I, II, and IV are valid for Special Permits where the weights are assumed to be fully controlled and are not expected to exceed the permit weight allowed. For Routine Permits, it is Case III that is expected to govern, while Case IV should govern for special Permits. Case I will be analyzed to check the safety of escorted permit trip. The analysis of Case II will be performed to verify that it will be overshadowed by Case II and IV.

Case I is not affected by the WIM data for the random trucks. Case II depends on the probability of having two Permit trucks side-by-side and this in turn will depend on the number of permits that may cross a bridge within the five year rating period and the probability of side-by-side events for that number of permit crossings. Following Moses (2001), we will assume up to 100 Permits per day as an upper limit for the number of trips that a truck may take. The probability of having two side-by-side permits will then be equal to 0.5% based on the WIM data collected on New York state sites on low truck traffic volume days (Sivakumar et al, 2008).

For cases III and IV, the reliability analysis should account for the number of random vehicles that may cross the bridge simultaneously with the Permit truck. Following the AASHTO LRFR classifications we will consider sites with ADTT=5000, 1000 or 100. The percentage of side-by-side vehicles is taken as P_{sxs}=2% for sites with ADTT=5000, P_{sxs}=1.25% for sites with ADTT=1000 and P_{sxs}=0.5% for sites with ADTT=100. These P_{sxs} values are upper bounds obtained from the headway data collected at ten New York State WIM sites as reported by Sivakumar et al (2008).

Deterministic Special Permits

A representative sample of Special permits is provided in Table 24. The table shows special permits with GVW exceeding 300 kips. The weights of trucks that have been issued special permits are assumed to exactly match the permitted weights. Thus, special permits are assumed to be deterministic values. The vehicles shown in Table 24 are used for the calibration of the Special Permit live load factors performed in this study.

Table 23 – Representative Sample of Routine Permits

Representative Routine Permits & Legal Loads			Axles (KIP)										Spacings (FT)										
Truck Designation	**State**	**Truck Identifier**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	No Axles	Wt (Kips)
CT-04	CT	CT 6-120	11	23	23	21	21	21					11	4	24	4	4					6	120
CT-05		CT 7-140	12	24	24	20	20	20	20				15.2	4.5	42.3	4.3	4.3	4.3				7	140
CT-07		CT 8-160	12	22	22	22	20.5	20.5	20.5	20.5			11.8	4.5	4.6	42.3	4.3	4.3	4.3			8	160
MI-65	MI	MI 3-126	42.0	42.0	42.0								12	4								3	126
MI-66		MI 3-108	36.0	36.0	36.0								12	4								3	108
MI-67		MI 3-90	30.0	30.0	30.0								12	4								3	90
OK-03	OK	OK 5-95	15	20	20	20	20						15	4.25	20	4.25						5	95
OK-05		OK 6-115	15	20	20	20	20	20					15	4.25	20	4.25	4.25					6	115
OK-12		OK 7-135	15	20	20	20	20	20	20				12	4.25	4.25	20	4.25	4.25				7	135
OK-19		OK 8-130	15	20	20	15	15	15	15	15			15	4.25	20	4.25	4.25	4.25	4.25			8	130
OK-25		OK 8-140	15	20	20	20	16.25	16.25	16.25	16.25			15	4.25	4.25	20	4.25	4.25	4.25			8	140
OK-41		OK 9-150	15	20	20	20	15	15	15	15	15		15	4.25	4.25	20	4.25	4.25	4.25	4.25		9	150
WA-01	WA	WA 5-96	10	21.5	21.5	21.5	21.5						10	4	12	4						5	96
NJ OL1	NJ	NJ 4-80	12.5	22.5	22.5	22.5							16	4.5	4.5							4	80
NJ OL2		NJ 5-100	12	22	22	22	22						13	4.5	32	4.5						5	100
NJ OL3		NJ 6-112	12	20	20	20	20	20					12	4	18.5	4	4					6	112
NJ OL4		NJ 6-125	12.5	22.5	22.5	22.5	22.5	22.5					13	4.5	4.5	4.5	4.5					6	125

Table 23. Representative Sample of Routine Permits (cont.)

NY Type 6A	NY	NY 6-120	11	20.2	20.2	23	23	23					13	4.6	12	4.2	4.2					6	120.4
NY Type 1A		NY 6-102	9.2	15.4	15.4	20.7	20.7	20.7					15.4	4.3	27	4.1	4.1					6	102.1
NY Type 1		NY 4-77	17.9	17.9	20.6	20.6							19.6	4.2	4.2							4	77
LA Type 8	LA	LA 6-88	6	18.5	18.5	15	15	15					11	4	16	4	4					6	88
LA OFRD		LA 10-209	17	17	17	26	26	26	26	18	18	18	5.4	10.5	5.4	7.2	5.4	5.4	17.1	4	4	10	209
VA BP90	VA	VA 5-90	12.5	22	22	16.75	16.75						8	4	28	4						5	90
VA BP 115		VA 7-115	12	17.8	17.8	17.8	16.5	16.5	16.5				8	4	4	40	4	4				7	114.9
OR CTP 2A	OR	OR 8-105A	10.5	16.5	16.5	14	14	14	10	10			17	4.5	29	4.5	4.5	18	4.5			8	105.5
OR CTP 2B		OR 8-105B	11	14.5	14.5	14.5	12.75	12.75	12.75	12.75			15	5	5	37	4.5	4.5	4.5			8	105.5
OR CTP 3		OR 5-98	12	21.5	21.5	21.5	21.5						11	4	24	4						5	98
MS BP1	MS	MS 5-92	10	20	20	21	21						10	4	30	4						5	92
MS BP2		MS 6-96	10	20.5	20.5	15	15	15					15	4	30	4	4					6	96
MS BP3		MS 7-101	10	20.5	20.5	12.5	12.5	12.5	12.5				12	4	30	4	4	4				7	101
WI 1	WI	WI 6-98A	18	16	16	16	16	16					17	4.2	4.2	10	15.8					6	98
WI 2		WI 6-98B	12	17.2	17.2	17.2	17.2	17.2					13.2	4.2	25.3	4.2	4.2					6	98

Table 24. Representative Sample of Special Permits

Truck Identifier	AXLE WEIGHTS [kips]													AXLE SPACINGS [ft]												No Axles	GVW [kips]
	1	2	3	4	5	6	7	8	9	10	11	12	13	1	2	3	4	5	6	7	8	9	10	11	12		
BP-3	10.0	20.5	20.5	12.5	12.5	12.5	12.5							12.0	4.0	30.0	4.0	4.0	4.0							7	101.0
BP-4	10.0	14.7	14.7	14.7	12.5	12.5	12.5	12.5						12.0	4.0	4.0	30.0	4.0	4.0	4.0						8	104.1
SL-10-198	13.5	20.5	20.5	20.5	20.5	20.5	20.5	20.5	20.5	20.5				16.0	4.5	4.5	27.0	4.0	4.0	4.0	4.0	4.0				10	198.0
SL-13-280	13.0	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	22.3	16.0	4.5	4.5	14.5	5.0	5.0	49.5	5.0	5.0	14.5	5.0	5.0	13	280.0
OT-7-151	11.0	26.0	26.0	22.0	22.0	22.0	22.0							16.0	4.5	30.0	4.5	4.5	4.5							7	151.0
OT-8-162	16.0	22.0	22.0	22.0	20.0	20.0	20.0							18.0	4.5	4.5	34.0	4.5	4.5	4.5						7	142.0
C-7-150	12.0	23.0	23.0	23.0	23.0	23.0	23.0							12.0	4.0	4.5	4.5	4.5	4.5							7	150.0
C-8-150	16.0	22.0	22.0	22.0	17.0	17.0	17.0	17.0						12.0	5.0	5.0	23.0	4.0	4.0	4.0						8	150.0
NYP1	26.0	26.0	27.3	27.3	27.3	27.3								4.9	11.3	4.9	4.9	4.9								6	161.0
NYP2	23.8	23.8	15.9	15.9	15.9	15.9	19.0	19.0	22.6	23.9	23.9			4.9	4.9	9.7	4.9	4.9	4.9	4.9	23.8	4.3	4.3			11	219.4
NYP3	13.5	24.7	24.7	24.7	24.7	24.7	24.7	24.7	24.7	24.7	24.7	24.7	24.7	15.0	4.5	4.5	14.0	5.0	5.0	48.0	5.0	5.0	14.8	5.0	5.0	13	309.9
NYP4	25.6	25.8	27.7	26.6										5.4	6.3	5.8										4	105.7
NYP5	27.3	27.1	29.5	29.5	29.5									8.0	5.4	8.0	5.4									5	142.9
OVLD-5	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0					13.0	5.0	5.0	37.5	4.5	4.5	4.5	4.5					9	180.0
OVLD-6	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	13.1	4.6	4.6	17.0	4.5	4.5	36.4	4.5	4.5	14.1	4.5	4.5	13	260.0
OVLD-7	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0		30.1	4.6	4.6	16.1	4.5	4.5	30.1	4.5	4.5	14.0	4.5		12	240.0
OR-STP-4C	13.0	15.0	21.5	21.5	21.5	21.5	21.5	15.0						11.5	5.5	4.5	30.0	5.0	5.0	12.0						8	150.5
OR-STP-4D	12.0	21.5	21.5	21.5	21.5	21.5	21.5	21.5						10.0	4.0	21.0	4.0	6.0	16.0	4.0						8	162.5
OR-STP-4E	18.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	12.0	5.5	4.5	15.0	5.0	5.0	43.0	5.0	5.0	16.0	5.0	5.0	13	258.0
OR-STP-5BW	12.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0					18.0	4.5	14.0	5.0	32.0	5.0	16.0	4.5					9	204.0
HP1	24.0	24.0	24.0	24.0	24.0									7.7	5.4	6.6	5.4									5	120.0
HP2	12.0	21.7	21.7	21.7	40.0	40.0								11.5	4.0	4.0	28.0	4.0								6	157.1
HP3	11.1	20.2	20.2	16.6	16.6	16.6	18.1	18.1	18.1	18.1	18.1	18.1		16.5	4.6	14.8	4.4	4.4	38.8	4.5	4.5	13.7	4.5	4.5		12	209.9
RI-OP2	12.0	18.0	25.0	25.0	25.0	25.0								8.3	4.3	33.5	4.5	4.5								6	130.0
RI-OP3	13.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0						15.7	4.3	4.4	35.0	4.3	4.3	4.3						8	160.0
RI-OP4	9.0	15.0	17.0	17.0	17.0	17.0	14.0	20.0	20.0	20.0	20.0	20.0	20.0	11.8	4.3	4.5	15.0	4.0	5.6	36.0	4.3	4.3	10.5	4.3	4.3	13	226.0
VA-1	21.3	21.3	22.5	22.5										4.5	14.8	4.7										4	87.5
VA-2	22.0	21.4	25.2	29.2	29.6									8.8	5.9	6.7	5.9									5	127.4
VA-3	15.0	23.3	23.3	23.3	23.3	23.3	23.3	22.5	22.5					17.0	4.5	4.5	36.0	4.5	4.5	14.1	4.5					9	199.8
VA-4	14.6	20.0	20.0	20.0	25.0	25.0	20.0	20.0	20.0	25.0				12.8	4.4	4.4	14.0	4.4	36.0	4.2	4.2	13.5				10	209.6
VA-5	20.0	20.0	20.0	20.0	22.0	22.0	20.0	20.0	20.0	22.0	22.0			16.3	4.6	4.5	12.8	5.0	35.7	5.0	5.0	14.4	5.0			11	228.0
VA-6	12.0	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	10.0	4.5	4.5	14.1	5.0	5.0	43.5	5.0	5.0	14.1	5.0	5.0	13	292.8

Variability in Load Distribution Factors

Background

The analysis of the live load effects on multi-beam bridges have traditionally relied on the load distribution factors, D.F., provided in the AASHTO specifications. The equations for the D.F. of multi-girder bridges are for single lane and multi-lane bridges. The equations for single lanes given in the AASHTO LRFD specifications already include a multiple presence factor MP=1.2 to account for the lower probability of having two-side-by-side heavy trucks as compared to a single heavy truck. Nowak (1999) as well as Moses (2001) assumed that the equations provided in the AASHTO LRFD for calculating D.F. give on the average good approximations to the actual distribution factor and they both assumed that mean value of the distribution factor is equal to the nominal value specified in the AASHTO LRFD such $\overline{\boldsymbol{DF}} = \boldsymbol{DF}$. However, in an extensive analysis performed as part of NCHRP 592, Puckett et al (2007) found that the LRFD load distribution factors can be significantly more conservative when compared to those obtained from refined finite element and grillage analysis models.

Table 25 shows the ratios of the LRFD distribution factors over the refined analysis results obtained by Puckett et al (2007) for the different bridge types labeled as “a” through “k” classified by the AASHTO LRFD specifications. Specifically, bridge Type “a” represents steel I-girder bridges. Bridge Type “e” is for concrete T-beam bridges and Type “k” is for prestressed concrete I-beam bridges. Table 25 shows that for bridges of type “a”, “k”, and “e”, the LRFD equations for the moment for interior beams under two lanes of loading give the closest results compared to those of the refined analysis but are still about 11% more conservative with a COV equal to $V_{DF\ LRFD}$=13%. The other cases, particularly for the shear of exterior beams under two lanes of loading, show that the AASHTO LRFD may be up to 44% more conservative than observed from the refined analysis with a COV that may reach up to 25%. We can see from these results that the conservatism of the AASHTO LRFD load distribution factors which may lead to higher reliability index values than those calculated during the calibration of the LRFD and LRFR live load factors may be offset by higher COV values which would tend to lower the estimated reliability indexes.

Another issue that arises when checking the safety of bridges is the accuracy of the estimated distribution factors obtained from a refined analysis compared to those actually experienced by the bridge itself. Researchers have recommended various Finite Element and grillage modeling techniques that resulted in good matches between refined analysis results and those from field measurements. These modeling techniques are usually calibrated based on data collected from very few bridges. Because each bridge is a unique structural system with its own characteristics, it is usually difficult to verify how the different recommended models would compare if they are applied to bridges which were not used in the model calibration process. For example, due to corrosion of the bearings as well as their different types, locking of expansion joints and other factors such as detailing of connections, some bridges may exhibit end restraint conditions when none was originally intended or they may show unexpected flexibility at structural connections. Also, the effects of barriers, curbs, secondary members and other non-structural elements and the deterioration of the bridge components may change the stiffness of some members leading to alterations in the load distribution factors. Other factors such as unintended composite action or beam corrosion would influence the strains and may also affect the load distribution factors particularly if they do not act uniformly on all the main members. End restraints may break at high loads and must be carefully studied when executing a safety analysis of the bridge.

On the other hand, Schwarz and Laman (2001) found that the results from grillage analyses using uni-dimensional beam elements compared very well to field results. Barr, Eberhard and Stanton (2001) have also found that a Finite Element (FE) model combining frame and shell elements would lead to good matches with field data. Bae and Oliva (2009) have also compared the results from different FE models. They found that models using shell and frame elements as well as models that used only shell elements gave good matches to field results.

Similar observations were made by other researchers. For example in a MS Thesis at the University of Cincinnati, Jamboktar (2006) assembled field data from about 13 bridges and compared the field measured load distribution factor to those from "tuned" Finite Element models. The tuned models were optimized models which were calibrated to minimize the difference between the results of dynamic modal tests and the Finite Element predictions. If we compare the load distribution factors of the "tuned" finite element analysis to bridge field tests, we observe that the average ratio is close to 1.0 with a COV close to 8% for interior girders. However, for exterior girders the COV for the ratio of the test over the finite element results is on the order of 18%. This higher COV may be due to the variability in how the secondary components as well as barriers and parapets contribute to the stiffness and consequently to the load distribution of the exterior girders. In Table 25 the count gives the number of cases that were used to obtain the mean and the COV value for each category.

Table 25 - Comparison of AASHTO LRFD distribution factors to refined analysis

	Values are ratio of AASHTO LRFD / Rigorous Methods		MOMENT				SHEAR			
			INTERIOR		EXTERIOR		INTERIOR		EXTERIOR	
			1 LANE	2 LANES	1 LANE	2 LANES	1 LANE	2 LANES	1 LANE	2 LANES
BRIDGE TYPE	A	AVERAGE	1.294	1.113	1.369	1.191	1.388	1.216	1.358	1.436
		ST DEV	0.094	0.128	0.179	0.142	0.159	0.197	0.180	0.283
		COV	0.072	0.115	0.131	0.119	0.115	0.162	0.132	0.197
		COUNT	107	112	108	109	115	114	99	103
	B	AVERAGE	1.609	1.259	2.181	1.395	1.308	1.073	1.516	1.236
		ST DEV	0.615	0.301	0.405	0.262	0.086	0.072	0.147	0.097
		COV	0.382	0.239	0.186	0.188	0.066	0.067	0.097	0.079
		COUNT	60	59	40	40	55	54	41	40
	D	AVERAGE	1.605	1.299	2.660	2.086	1.290	1.015	1.425	1.348
		ST DEV	0.209	0.118	0.291	0.237	0.147	0.064	0.109	0.193
		COV	0.130	0.091	0.109	0.114	0.114	0.063	0.076	0.143
		COUNT	86	92	38	92	272	104	272	104
	E	AVERAGE	1.264	1.074	1.415	1.241	1.312	1.134	1.415	1.337
		ST DEV	0.168	0.135	0.207	0.198	0.119	0.178	0.289	0.334
		COV	0.133	0.125	0.147	0.160	0.091	0.157	0.204	0.250
		COUNT	49	47	48	46	50	48	49	47
	F and g	AVERAGE	1.390	1.899	2.689	2.414	1.337	1.079	1.481	1.482
		ST DEV	0.483	0.609	0.195	0.356	0.164	0.122	0.381	0.459
		COV	0.347	0.321	0.072	0.148	0.123	0.113	0.257	0.310
		COUNT	50	138	16	122	137	133	118	116
	k	AVERAGE	1.292	1.109	1.348	1.208	1.296	1.134	1.305	1.307
		ST DEV	0.108	0.115	0.178	0.267	0.094	0.154	0.172	0.312
		COV	0.083	0.104	0.132	0.221	0.072	0.136	0.132	0.239
		COUNT	73	73	73	73	73	73	69	69

Model for Refined Analysis

For the purposes of this study and based on the observations made by Shwarz & Laman (2001), Barr et al (2001) as well as Jamboktar (2006), it will be assumed that the refined analysis that will be used for permit load checking has been carefully calibrated and "tuned" so that the distribution factor obtained from the analysis is on the average similar to the one actually experienced by the bridge with a COV equal to $V_{an=}$8%. It will also be assumed that the load distribution factor found from the refined analysis will follow a Normal probability distribution.

AASHTO LRFD Load Distribution Factors

The COV's obtained by Puckett et al (2007) reflecting the variation between the load distribution factors of the AASHTO LRFD as compared to those of the refined analysis $V_{DF\ LRFD}$ which are provided in Table 25 do not account for the variability between the results of the refined analysis and those of field measurements. To account for the overall uncertainties between the LRFD values and the actual measured values, the COV's of Table 25 will have to be increased to also account for the variations between the refined analysis and field measured distribution such that the final COV for the load distribution factor is expressed as:

$$V_{DF} = \sqrt{V_{DF\ LRFD}^2 + V_{an}^2} \tag{53}$$

Where $V_{DF\ LRFD}$ is the Coefficient of Variation of the ratio of the refined analysis over the AASHTO LRFD load distribution and V_{an} is the Coefficient of Variation of the ratio of the measured over the refined analysis load distribution so that V_{DF} is the COV of the measured over the AASHTO LRFD value.

The final values of V_{DF} that will be used in this calibration of permit live load factors when using the AASHTO LRFD equations for checking the permit ratings of interior beams are presented in Table 26 along with the average ratios of the LRFD distribution factors compared to actual factors. The actual distribution factor will also be assumed to follow a Normal probability distribution.

Table 26 - Comparison of AASHTO LRFD distribution factors to those from field measurements for interior beams

	Values are for the bias λ_{DF} = Mean AASHTO LRFD / Field results		MOMENT		SHEAR	
			1 LANE	2 LANES	1 LANE	2 LANES
BRIDGE TYPE	A	AVERAGE	1.29	1.11	1.39	1.22
		COV	11%	14%	14%	18%
	E	AVERAGE	1.26	1.07	1.31	1.13
		COV	16%	15%	12%	18%
	K	AVERAGE	1.29	1.11	1.30	1.13
		COV	12%	13%	11%	16%

Variability in Dynamic Amplification Factors

The AASHTO LRFD specifies that a nominal dynamic amplification IM=1.33 be used on the truck load effect to account for the increased stresses due to the vibrations of the bridge under moving loads. However, it has been well established that this value is a conservative upper bound. On the average, Nowak (1999) indicates that heavily loaded trucks usually produce lower values than the nominal value with a mean value $\overline{IM} = 1.13$ and a COV of V_{IM}=9% for individual truck crossings. For simultaneous crossings in multi-lanes the average is $\overline{IM} = 1.09$ and the COV is V_{IM}=5.5%. In this Report we will assume that the dynamic amplification factor follows a Normal probability distribution.

Reliability Analyses for Permit Loadings

Reliability Analysis for Case I – Special Permit Vehicle Alone on the Bridge

The expression for estimating the maximum load effect on the most critical beam when a single vehicle is on the bridge can be given as:

$$\overline{LL} = P \times IM \times D.F./1.2 \tag{54}$$

Where for special permits for which the load effects are known without any uncertainty the mean value of the load effect is the same as that obtained from the issued permit so that $\overline{P} = P$ with no COV or V_p=0, IM and D.F. are random variable having the biases and COV's listed in the previous sections. Dividing the one lane D.F. by 1.2 is done to remove the multiple presence factor which is embedded in the AASHTO LRFD equations for single lanes.

Assuming that the weight and axle configuration of the special permit vehicle are exactly known, the Coefficient of Variation (COV) of the maximum beam live load effect is obtained from the COV of IM, V_{IM}, and the COV of DF, V_{DF}:

$$V_{LL} = \sqrt{V_{IM}^2 + V_{DF}^2} \tag{55}$$

During the calibration of the AASHTO LRFD, Nowak (1999) assumed that the AASHTO LRFD distribution factors give values that are very close to the mean of the actual distribution factors. However, as shown in Table 25, the AASHTO LRFD load distribution factors are found to be higher than those obtained from a refined analysis. Also, as mentioned earlier, the refined analysis itself is associated with certain level of uncertainty. As observed above, the uncertainty in the results of the refined analysis may be represented by a COV V_{an}=8%. This V_{an}=8% must be combined with that reflecting the variation of the AASHTO LRFD from the values obtained from the refined analysis and presented in Table 26 as expressed in Eq. (53). Table 26 provides the estimated ratio between the LRFD load distribution factors and the field measured results along with the corresponding estimated COV. The bias λ_{DF} and the COV V_{DF} implied in Table 26 will be used to find the mean of the distribution factor and its variability.

The reliability analysis is executed in this section using a FORM algorithm for the set of typical Special permit vehicles having the configurations shown in Table 24. The analysis of the permit loads is performed for typical simple span concrete T-beam, prestressed concrete I-beam, composite and non-composite steel I-beam bridges with spans varying between 20-ft up to 200-ft

in length and having beams spaced from 4-ft to 12-ft center on center. The dead load data for these bridges was obtained by interpolating the data provided by Nowak (1999).

To execute the calculation of the reliability index implicit in using different values of Permit load factor γ_L the following failure function is defined:

$$Z = R - DL - LL \tag{56}$$

Where R is the member resistance, DL is the dead load effect and LL is the live load effect. All three variables R, DL and LL are random.

$$R.F. = \frac{\varphi_n R_n - \gamma_{DC}\left(D_{C1} + D_{c2}\right) - \gamma_{DW} D_W}{\gamma_L L_L} \tag{57}$$

Equation (57) is used to find the nominal resistance, R_n, required to carry each of the Special Permit trucks of Table 24 for different permit load factors, such that the Rating Factor is always equal to R.F.=1.0. R_n, is found by assuming that the nominal resistance is obtained using a live load factor γ_L applied on the permit truck effect with the distribution factor for one lane after removing the multiple presence factor MP=1.2 following the procedure outlined in the current AASHTO LRFR.

Given the nominal values, the mean and the standard deviations for the dead loads are obtained assuming that the dead load follows a Normal distribution.

Given the nominal resistance, the mean and the COV are obtained assuming that the resistance follows a lognormal distribution.

The live load model is obtained using Eq. (54) where P of special permits is deterministic but DF and IM are Normal random variables having the statistical data associated with a single truck.

To verify the results of the FORM algorithm, a simple case is first analyzed in the next section using closed form expressions and some simplifying assumptions.

Simplified Example for Analysis of Reliability for Cases when Permit Check Uses LRFD Load Distribution Factors

A simplified example is provided to illustrate the relationship between the different random variables. In this simplified example some assumptions are made on the random variables to permit the application of a closed-form expression for the reliability index, β. The final results presented further below use the FORM algorithm along with the probability distribution types for each random variable as previously presented.

The example involves the reliability analysis of a 100-ft simple span T-beam bridge with beams at 6-ft center to center. The dead loads on the bridge produce a moment effect for the precast members equal to D_{c1}=0 while the cast in place material produces a nominal dead load moment D_{c2}=2186.5 kip-ft along with a moment D_w=202.6 kip-ft for the wearing surface. The bridge is expected to carry a Special Permit truck labeled SL-10-198 in Table 24 having a GVW=198 kip. The moment that this truck produces on a 100-ft span is 3096 kip-ft.

According to current AASHTO LRFR criteria, this Special Permit truck would be allowed to cross the bridge alone when escorted if the moment capacity of the T-beams satisfies the LRFR rating with a rating factor R.F.=1.0 with a live load factor γ_L=1.15. The impact factor is IM=1.33 and the AASHTO LRFD load distribution factor including the multiple presence factor MP=1.2 is D.F.=0.366. Accordingly, using a resistance factor φ_n =0.9 and dead load

factors γ_{DC}=1.25 on the cast in place moments and γ_W=1.50 for the wearing surface, the minimum required nominal moment capacity R_n is obtained from Eq. (57) by setting R.F.=1.0 leading to:

$$0.9 \times R_n = 1.25 \times (2186.5) + 1.50 \times (202.6) + 1.15 \times (3096 \times 1.33 \times 0.366/1.2) = 4979 \text{ kip-ft}$$

Using a resistance bias B_R=1.14 for the moment of T-beams, the mean resistance is obtained as $\overline{R}$ =5676 kip-ft and the COV is V_R=13%.

The dead load biases are 1.05 for the cast in place concrete and the COV is 10%. Thus, the mean load effect is 2296 kip-ft and the standard deviation is 230 kip-ft. For the wearing surface, the bias is 1.0 and the COV is 25% leading to a mean 203 kip-ft and a standard deviation 51 kip-ft.

The mean live load combines the moment of the special permit to the mean impact and the mean load distribution factor as shown in Eq. (54). For the case of the special permit this becomes:

$$\overline{LL} = 3096 \times 1.13 \times \frac{(0.366/1.2)}{1.26} = 847\ kip - ft$$

The 1.26 in the denominator represents the implicit safety factor that the AASHTO LRFD distribution factors include when compared to the results of a refined analysis. Given that the Special Permit is assumed to have known axle weights and configuration, the COV for the live load will only include the COV of the dynamic amplification and that of the distribution factor, leading to:

$$V_{LL} = \sqrt{V_{DF}^2 + V_{IM}^2} = \sqrt{(16\%)^2 + (9\%)^2} = 18.4\%$$

The standard deviation of the live load is obtained as σ_{LL}=156 kip-ft.

The total load combining the effects of the cast in place dead weight, the wearing surface weight and the live load will then have a mean value $\overline{S} = 847 + 2296 + 203 = 3346\ kip - ft$ and a standard deviation $\sigma_S = \sqrt{230^2 + 51^2 + 156^2} = 283\ kip - ft$ or a COV V_S=8.4%

An approximate value for the reliability index, β, can be obtained from the Lognormal model using the equation:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{5676}{3346}\right)}{\sqrt{(13\%)^2 + (8.4\%)^2}} = 3.41$$

The use of a more rigorous FORM algorithm where the resistance is assumed to be lognormal, the dead load is normal and the live load being the product of P times the IM and DF that are both normal with the Permit's load effect that is deterministic would lead to a reliability index β=3.43 which is very close to the 3.41 value obtained from the simplified Lognormal equation.

Similar analyses are subsequently performed using the FORM algorithm for a range of span lengths, beam spacings, beam types and Special Permit configurations. The analysis also uses different live load factors varying between γ_L=1.0 to γ_L=1.15. Two cases are considered: a) the live load effect is calculated using the AASHTO LRFD load distribution factors and b) the live load effect is calculated using the load distribution factor obtained from a refined analysis. The results are presented and discussed in the next sections.

Reliability Indexes for Special Permit Alone on the Bridge when Permit Check Uses LRFD Load Distribution Factors

As an example, the reliability index values obtained from the FORM algorithm when the AASHTO LRFR γ_L=1.15 set for escorted trips with no other vehicles on the bridge is used along with the AASHTO LRFD load distribution factors to determine whether a Permit should be allowed to cross a bridge are plotted in Figure 24 for all the span lengths, beam spacings and Special Permit trucks considered for simple span composite steel beams with different span lengths and beam spacing. The results in the figure show that there is no significant difference due to beam spacing. This is because we are assuming that the same conservative factor implied in the LRFD Load distribution factors is applicable to all beam spacings. There is some drop in the reliability index with the span length. This may be attributed to the contribution of the dead load whose effect increases with the span length. The overall average reliability index is about 3.71 with a range shown in the figure that varies between 3.25 and 4.11. The additional scatter due to the effect of various Permit truck configurations would lead to a spread between a reliability index β=3.04 and 4.13. Such a range of reliability index values indicates that the live load factor γ_L=1.15 is conservative and could be lowered to produce lower reliability index averages and ranges. A summary of the reliability index values obtained for different live load factors is provided in Table 27.

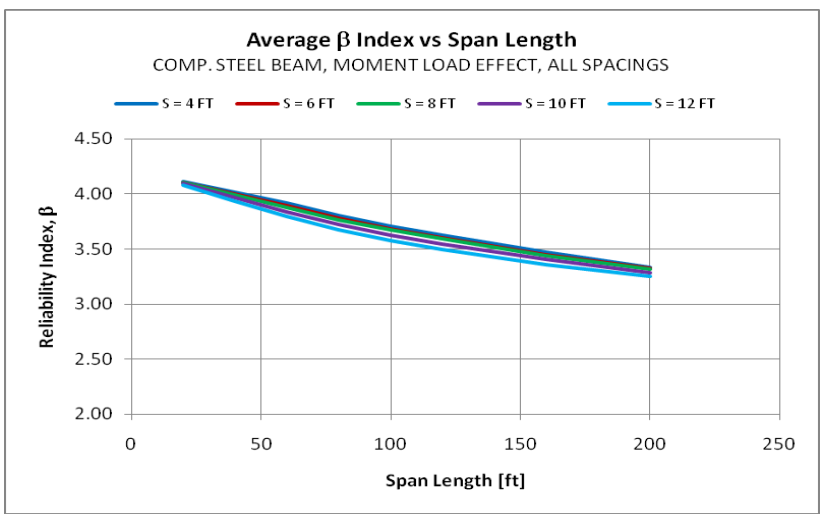

Figure 24. Average Reliability Indexes for composite steel bridges evaluated using γ_L=1.15 and using the AASHTO LRFD Load distribution factors

The results of the reliability index calculations for different values of the live load factors γ_L are presented in Table 27. The results show that even a live load factor γ_L=1.00 would still lead to an overall average reliability index value β_{ave}=3.25 with a minimum value of β_{min}=2.45

and a maximum value of β_{max}=3.58. The lowest observed values for the reliability index are those for Prestressed concrete bridges in shear. This is because the bias in the shear predictive models for prestressed concrete bridge members as provided by Nowak (1999) and used during the calibration of the AASHTO LRFD is relatively low with a relatively high COV. It should be noted however, that this bias and COV were set based on shear analysis and design models applicable for the AASHTO LFD. It is not known how these values would change with the new shear design methods adopted in the AASHTO LRFD.

As shown in Table 27 a live load factor γ_L=1.10 would still lead to an overall average reliability index value β_{ave}=3.56 with a minimum value of β_{min}=2.56 and a maximum value of β_{max}=4.04. These values are observed for the case when the Permit checking is executed using the AASHTO LRFD load distribution equations which are conservative containing an implicit level of safety. Although a lower live load factor would still lead to an average reliability index higher than 2.5, a minimum live load factors of 1.10 is recommended for all possible Permit categories.

The next section performs the same analysis assuming that the Permit check is based on load distribution factors obtained from a refined analysis of the bridge system.

Table 27. Reliability Indexes for Special permits alone on the bridge using the AASHTO LRFD load distribution factors.

SPECIAL PERMIT AASHTO LRFD	γ_L=1.00			γ_L=1.05			γ_L=1.10			γ_L=1.15		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.
T-beam bending	2.912	3.304	3.175	2.960	3.481	3.295	3.007	3.660	3.413	3.054	3.836	3.528
T-beam shear	3.022	3.500	3.323	3.085	3.702	3.460	3.148	3.901	3.594	3.211	4.093	3.725
Ps/C bending	2.840	3.369	3.220	2.887	3.600	3.383	2.933	3.847	3.544	2.979	4.097	3.701
Ps/C shear	2.446	2.910	2.749	2.502	3.130	2.889	2.558	3.346	3.025	2.614	3.555	3.158
Noncomp. Bending	2.911	3.385	3.245	2.955	3.627	3.401	2.999	3.882	3.553	3.042	4.134	3.703
Noncomp. Shear	3.160	3.577	3.514	3.233	3.801	3.701	3.305	4.036	3.883	3.380	4.273	4.064
Comp. bending	2.911	3.384	3.245	2.956	3.627	3.402	3.000	3.882	3.555	3.043	4.134	3.706
Comp. shear	3.161	3.576	3.515	3.235	3.801	3.702	3.308	4.036	3.885	3.308	4.036	3.885
ALL BRIDGE TYPES	2.446	3.577	3.248	2.502	3.801	3.404	2.558	4.036	3.557	2.614	4.273	3.684

Simplified Example for Analysis of Reliability for Cases when Permit Check Uses Refined Analysis

A simplified example is provided to illustrate the reliability analysis followed for the case when the Permit check uses the results of refined analysis when the truck is expected to run alone on the bridge. In this example we use the same 100-ft simple span T-beam bridge with beams at 6-ft center to center analyzed above. The dead loads on the bridge produce a moment effect for the precast members equal to D_{c1}=0 while the cast in place material produces a nominal dead load moment D_{c2}=2186.5 kip-ft along with a moment D_w=202.6 kip-ft for the wearing surface. The bridge is expected to carry a Special Permit truck labeled SL-10-198 in Table 24 having a GVW=198 kips. The moment that this truck produces on a 100-ft span is 3096 kip-ft.

According to current AASHTO LRFR criteria, this Special Permit truck would be allowed to cross the bridge alone when escorted if the moment capacity of the T-beams satisfies the LRFR rating with a rating factor R.F.=1.0 with a live load factor γ_L=1.15. The impact factor is IM=1.33. In this case, we assume that the refined analysis led to a load distribution factor D.F.=0.242, which in this example is selected to be equal to the LRFD value (0.366/1.2) divided by the 1.26 conservative factor. Accordingly, using a resistance factor φ_n =0.9 and dead load factors γ_{DC}=1.25 on the cast in place moments and γ_W=1.50 for the wearing surface, the minimum required nominal moment capacity R_n needed to allow the crossing of this Permit is obtained from Eq. (57) by setting R.F.=1.0 leading to:

$$0.9 \times R_n = 1.25 \times (2186.5) + 1.50 \times (202.6) + 1.15 \times (3096 \times 1.33 \times 0.242) = 4648 \text{ kip-ft}$$

Using a resistance bias B_R=1.14 for the moment of T-beams, the mean resistance is obtained as $\overline{R}$ =5299 kip-ft and the COV is V_R=13%.

The dead load biases are 1.05 for the cast in place concrete and the COV is 10%. Thus, the mean load effect is 2296 kip-ft and the standard deviation is 230 kip-ft. For the wearing surface, the bias is 1.0 and the COV is 25% leading to a mean 203 kip-ft and a standard deviation 51 kip-ft.

The mean live load combines the moment of the special permit to the mean impact and the mean load distribution factor as shown in Eq. (54). For the case of the special permit this becomes:

$$\overline{LL} = 3096 \times 1.13 \times 0.242 = 847 \; kip - ft$$

In the case where the refined analysis is used, we will assume that our estimate of the live load effect on the bridge member being analyzed is improved and will be associated with a lower COV such that V_{DF}=8%. Given that the Special Permit is assumed to have known axle weights and configuration, the COV for the live load will only include the COV of the dynamic amplification and that of the distribution factor, leading to:

$$V_{LL} = \sqrt{(8\%)^2 + (9\%)^2} = 12\%$$

The standard deviation of the live load is obtained as σ_{LL}=102 kip-ft.

The total load combining the effects of the cast in place dead weight, the wearing surface weight and the live load will then have a mean value $\overline{S} = 847 + 2296 + 203 = 3346\ kip - ft$ and a standard deviation $\sigma_S = \sqrt{230^2 + 51^2 + 102^2} = 257\ kip - ft$ or a COV V_S=7.7%

An approximate value for the reliability index, β, can be obtained from the Lognormal model using the equation:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{5299}{3346}\right)}{\sqrt{(13\%)^2 + (7.7\%)^2}} = 3.04$$

The use of a more rigorous FORM algorithm where the resistance is assumed to be lognormal, the dead load is normal and the live load being the product of the IM and DF that are both normal with the Permit's load effect that is deterministic would lead to a reliability index β=3.05 which is very close to the 3.04 value obtained from the Lognormal equation.

Similar analyses are subsequently performed using the FORM algorithm for a range of span lengths, beam spacings, beam types and Special Permit configurations. The analysis also uses different live load factors varying between γ_L=1.0 to γ_L=1.15. The results are presented and discussed in the next section.

Reliability Indexes for Special Permit Alone on the Bridge when Permit Check Uses a Refined Analysis

The reliability index values obtained when the AASHTO LRFR γ_L=1.15 set for escorted trips with no other vehicles on the bridge is used along with distribution factors obtained from refined analyses to determine whether a Permit should be allowed to cross a bridge are plotted in Figure 25 for all the span lengths, beam spacings and Special Permit trucks considered for simple span composite steel beams with different span lengths and beam spacing. The results in Figure 25 show that there is no significant change in β due to beam spacing or span length. The overall average reliability index is about 2.97 with a range shown in the figure that varies between 2.89 and 3.01. The additional scatter due to the effect of various Permit truck configurations would lead to a spread between a reliability index β=2.82 and 3.03. The effect of the dead load is less important than what was observed for the case when the check uses the LRFD load distributions as the reliability index is reduced. Although the reliability indexes drop when compared to the values obtained if the Permit checking uses the AASHTO LRFD load distribution factors, the observed range of reliability index values indicates that the live load factor γ_L=1.15 is still conservative when the Permit check is performed using the results of refined analysis. The live load factor proposed in the AASHTO LRFR can still be lowered to some extent.

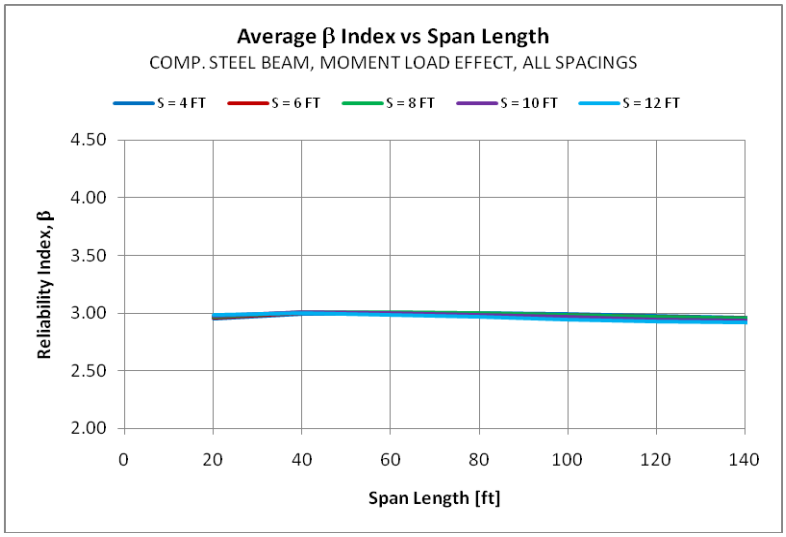

Figure 25. Average Reliability Indexes for composite steel bridges evaluated using γ_L=1.15 and refined analysis

The results of the reliability index calculations for different values of the live load factors γ_L are presented in Table 28. The results show that even a live load factor γ_L=1.00 would still lead to an overall average reliability index value β_{ave}=2.54 with a minimum value of β_{min}=1.84 and a maximum value of β_{max}=2.86. For prestressed concrete members in shear, the average reliability index is β_{ave}=2.11 with a minimum value of β_{min}=1.84 and a maximum value of β_{max}=2.22.

Using a live load factor γ_L=1.05 would ensure that the overall average reliability index value remains above 2.50 with β_{ave}=2.68 while the minimum value at β_{min}=2.06 remains above 2.0.

As shown in Table 28 a live load factor γ_L=1.10 would still lead to an overall average reliability index value β_{ave}=2.82 with a minimum value of β_{min}=2.27 and a maximum value of β_{max}=3.13. As explained earlier, a minimum live factor γ_L=1.10 is recommended for all Permit categories even if it provides reliability levels higher than β_{target}=2.50.

Table 28. Reliability Indexes for Different Live Load Factors used along with a refined analysis for checking the Rating of Special Permits alone on the bridge

SPECIAL PERMIT RIGOROUS	γ_L=1.00			γ_L=1.05			γ_L=1.10			γ_L=1.15		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.
T-beam bending	2.571	2.858	2.782	2.765	2.967	2.893	2.804	3.129	3.002	2.843	3.303	3.109
T-beam shear	2.496	2.756	2.688	2.707	2.861	2.811	2.805	3.009	2.930	2.865	3.165	3.048
Ps/C bending	1.993	2.690	2.537	2.294	2.774	2.682	2.586	2.902	2.825	2.744	3.063	2.966
Ps/C shear	1.842	2.217	2.100	2.061	2.268	2.223	2.272	2.366	2.344	2.319	2.507	2.462
Noncomp. Bending	2.126	2.731	2.560	2.397	2.798	2.698	2.658	2.886	2.833	2.819	3.029	2.966
Noncomp. Shear	2.145	2.770	2.544	2.417	2.840	2.713	2.681	2.949	2.879	2.926	3.085	3.041
Comp. bending	2.124	2.728	2.556	2.395	2.796	2.695	2.657	2.884	2.831	2.818	3.028	2.965
Comp. shear	2.141	2.768	2.540	2.415	2.838	2.710	2.679	2.948	2.877	2.926	3.084	3.040
ALL BRIDGE TYPES	1.842	2.858	2.538	2.061	2.967	2.678	2.272	3.129	2.815	2.319	3.303	2.950

Summary and Recommendation

Case I -- Special Permit Loads Alone on the Bridge

The calculations executed in this section, demonstrate that the reliability index remains on the average higher than the target β_{target}=2.50 even when a live load factor γ_L=1.0 is used for the case when a single permit truck is alone on the bridge assuming that the permit truck weights are exactly known. However, it is not deemed reasonable to not use any live load factor as that may be perceived as implying that no uncertainties are involved in estimating the load effect of a permit truck. For this reason, it is herein recommended that the single crossing of a non-divisible load Permit should be associated with a minimum live load factor γ_L=1.10. This live load factor would be used for single lane bridges when the truck has acquired a non-divisible load permit and the weight is controlled to avoid any changes from the permitted weights. Also, this same γ_L=1.10 should be used as a minimum value for permits on multi-lane bridges. This minimum live load factor may have to be exceeded if the case of a permit truck alongside random trucks may govern as will be examined in the remainder of this report. For the case where an escorted Permit truck travels at crawl speed, it is recommended to apply a Dynamic Amplification Factor IM =1.05 when performing a rigorous analysis in order to ensure that the reliability index does not fall below a minimum value β_{min}=1.50.

Reliability Analysis for Case II – Two Permit Trucks side-by-side.

In this case, we start by assuming that the axle weights and axle configurations of the two permit trucks are the same so that the total maximum static live load effect on the bridge is equal to 2P. To have two Permits side-by-side implies that a large number of Permits are allowed on the bridge simultaneously. This can only happen with sufficiently level of probability for Routine Permits. Thus, for this case we assume that the two side-by-side Routine Permits have a certain level of uncertainty in their load effects. We will also assume that the two weights are correlated to provide a conservative envelope to the results. In addition, to the uncertainty in the truck weight, the load effect must account for the uncertainties in estimating the dynamic vibration represented by the dynamic amplification factor, IM, and the uncertainties in the structural analysis process that allocates the fraction of the total load to the most critical member as represented by the load distribution factor, D.F.

The probability of having two permits simultaneously on the bridge depends on the number of such permits that will be allowed on a particular bridge within the evaluation period. We will assume that there will be up to 100 crossings of a particular type of Permit trucks in one day over each analyzed bridge. According to the NCHRP 12-76 headway data, this will mean that the probability of having two side-by-side Permits is 0.50%.

For two lanes loaded by the same permit truck, the mean value of the live load effect on one member can be given as:

$$\overline{\boldsymbol{LL}} = \overline{\boldsymbol{P}} \times \overline{\boldsymbol{IM}} \times \overline{\boldsymbol{D.F}}. \tag{58}$$

In the case of Routine loads, P is assumed to follow a shifted Exponential distribution.

According to Nowak (1999), the dynamic amplification augments the truck load effect by an average of 9% for heavy side-by-side trucks or $\overline{IM}$ =1.10. The dynamic amplification also produces a Coefficient of Variation COV equal to V_{IM}=5.5%.

The data of Table 26 are used to represent the mean value and the COV for the lane distribution factors.

The reliability index conditional on the arrival of two side-by-side permits on the bridge can then be calculated using the FORM algorithm and the failure function given in Eq. (56) and the live load model in Eq. 58. The reliability index calculated from the FORM algorithm where the live load is described is designated as β_c which is defined as the reliability index conditional on having two side-by-side trucks.

The probability that a bridge member would exceed the limit state of Eq. 56 conditional on having two permit vehicles side-by-side can be calculated from:

$$P_{f|side-by-side} = \Phi(-\beta_C) \tag{59}$$

where $\Phi(...)$ is the cumulative standard Normal distribution function. The final unconditional probability of failure will depend on the conditional probability of limit state exceedance given two side-by-side permit events, $P_{f|side-by-side}$ and the probability of having a situation with side-by-side permits, P_{SxS}. Thus:

$$P_f = P_{f|side-by-side} \times P_{S\times S} \tag{60}$$

where the probability of having two-side-by-side permits P_{SxS} is related to the total number of permit crossings. The probability of two permits side-by-side is not affected by the ADTT of the

site but it depends on the number of permits per day. In these calculations we will assume P_{SxS}=0.5% for up to 100 independent crossings of Permits in one day at one bridge site as obtained as an upper limit from Sivakumar et al (2008).

The final unconditional reliability index, β, is obtained from:

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{61}$$

The reliability calculations are performed using the FORM algorithm assuming that a Permit load factor γ_L =1.00 is used for finding the nominal resistance R_n. The conditional reliability index values are shown in Table 29. The final unconditional reliability index values for all the span lengths and beam spacing of the composite steel bridges are provided in Table 30 for the bridge configurations analyzed in this example.

Table 29. Summary of Conditional Reliability Indexes for Two Permits Side-by-Side

PERMIT ALONGSIDE SAME PERMIT	γ_L=1.00		
	Min	Max	Ave.
T-beam bending	2.310	2.910	2.766
T-beam shear	2.422	2.877	2.755
Ps/C bending	1.972	2.895	2.646
Ps/C shear	1.949	2.368	2.248
Noncomp. Bending	2.086	2.883	2.646
Noncomp. Shear	2.348	3.054	2.819
Comp. bending	2.084	2.880	2.643
Comp. shear	2.346	3.053	2.816
ALL BRIDGE TYPES	1.949	3.054	2.667

Table 30. Summary of Unconditional Reliability Indexes for Two Permits Side-by-Side

PERMIT ALONGSIDE SAME PERMIT	γ_L=1.00		
	Min	Max	Ave.
T-beam bending	3.880	4.287	4.186
T-beam shear	3.953	4.264	4.178
Ps/C bending	3.669	4.277	4.103
Ps/C shear	3.656	3.918	3.841
Noncomp. Bending	3.739	4.268	4.104
Noncomp. Shear	3.905	4.391	4.223
Comp. bending	3.738	4.266	4.102
Comp. shear	3.903	4.390	4.221
ALL BRIDGE TYPES	3.656	4.391	4.120

Table 29 shows that even when a live load factor γ_L=1.0 is used, the average conditional reliability index is $\beta_{average}$=2.67 with a minimum value of 1.95 and a maximum value of 3.05.

The average unconditional reliability index is 4.12 with a minimum value of 3.66 and a maximum value of 4.39 as shown in Table 30. The higher reliability levels observed for the unconditional case are clearly due to the low probability of having two Permits side-by-side when 100 Permits are independently crossing the same bridge each day.

The results of the calculations performed in this section illustrate the following points:

- The difference in the average reliability index for the most unconservative live load factor of γ_L=1.00 between the unconditional and conditional situation is significant with $\beta_{average}$ increasing from about 2.67 to 4.12. This increase is due to the very low probability of having two side-side permits in situations where up to 100 permits independently cross the same bridge in one day. The assumptions made are that the crossings of the permit trucks over a given bridge are totally random and no clumping of permit trucks is expected but that the weights of the permits are deterministic.
- Even if two permits happen to cross the bridge side-by-side, the average conditional reliability index of $\beta_{average}$=2.67 is significantly higher than the target β_{target}=2.50 set in AAHTO LRFR for the load rating of Permit trucks. This indicates that the lowest live load factor γ_L=1.10 is conservative for side-by-side permits.

According to the observations made in the previous bullets, the case of two side-by-side permits of the same type is not expected to control the safety of a bridge system. However, it is possible that the case of a permit alongside a heavy random truck may control as will be discussed in the next section.

Summary and Recommendation

Case II – Two Permit Trucks side-by-side

In summary the use of live load factor of γ_L=1.10 would lead to relatively high reliability levels that exceed the AASHTO LRFR target β_{target}=2.5 for the cases when side-by-side permits

could occur even if the two permit weights are assumed to be random to account for the possibility of having overloaded permit trucks.

Reliability Analysis for Case III – Permit Truck alongside Random Truck for Unlimited Permit Crossings.

Load Modeling

For this case, we need to analyze the maximum live load effect that is due to the permit truck alongside the maximum truck expected to occur simultaneously in the other lane. The maximum total load effect is random and it depends on the number of side-by-side events expected within the return period. As mentioned earlier, a five-year return period is used in this study following the recommendation of Moses (2001).

To determine the number of side-by-side permit-random truck events that would occur within the five-year rating period, we will assume that the number of side-by-side events involving one random truck will depend on the ADTT. Based on the upper envelopes of headway data collected at New York sites in NCHRP 12-76, sites with ADTT=100 will have 0.5% of the loading events formed by side-by-side trucks. Sites with ADTT=1000 will have of 1.25% of the trucks side-by-side and sites with ADTT=5000 will have 2% probability of side-by-side trucks.

In this report, the estimation of the load effect of the random trucks that will cross a bridge alongside a Permit truck are calculated using the actual live load effects of the trucks recorded by WIM from several sites throughout the U.S. An example histogram of the moment load effects of trucks moving in one lane of a 100-ft simple span bridge is provided in Figure 26 based on the data collected at WIM site 0199. The load effects are normalized by dividing the actual moments by the moment of the HL-93 live load. The tail end consisting of the upper 5% of the data is fitted by a Normal probability distribution producing the histogram shown in red.

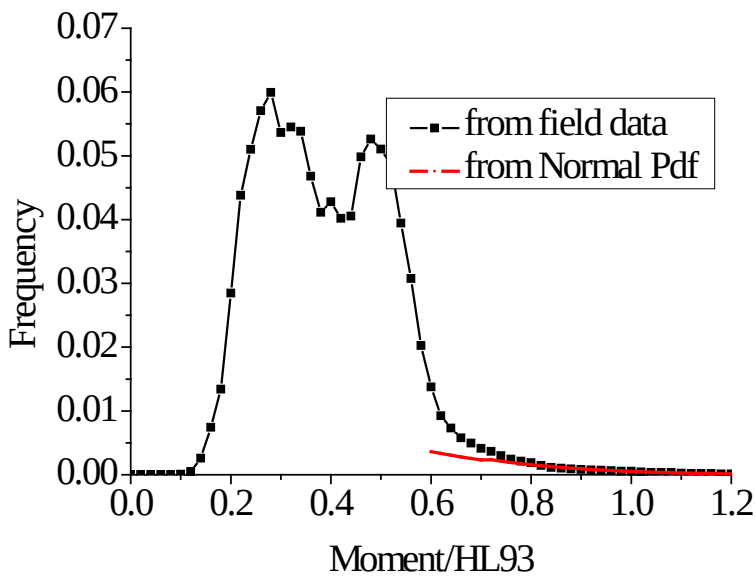

Figure 26 - Histogram of normalized moment effect for a single lane events

As explained in Tasks 1 and 2 findings, the histogram of the load effect in a single lane and its cumulative distribution can be used to find the cumulative distribution of the maximum load effect in any given rating period. To find the cumulative distribution for the maximum loading event in a return period of time T, we have to start by estimating the number of loading events, N, that occur during this period of time T. The number of events N is obtained from the ADTT and the WIM headway data at a site.

We define N_p as the number of events where a Permit will cross the bridge alongside a random truck within the five-year rating period, T. The percentage of side-by-side events involving a random truck is P_{sxs}. Thus, within a return period, T, there will N_R crossings of Permits alongside a random truck:

$$N_R = P_{S \times S} \times N_P \tag{62}$$

The percentage of side-by-side events, P_{SxS}, depends on the ADTT. As an example, for a bridge site with ADTT=5000 and 100 permits per day the number of permit crossings will be N_p= 182,500 (100 truck per day x 365 days per year x 5 years). Given a probability of side-by-side events P_{sxs}=2% for sites with ADTT=5000, the number of events when the permit truck will be alongside a random truck within a 5-year rating period is N_R=3650 (182,500 x 2%).

For low values of N_R, the cumulative distribution of the maximum load effect for the random truck that may cross alongside a Permit may be obtained directly by raising the cumulative distribution of the one event to the power N_R. As N_R increases, it is possible to use extreme value probability theory to obtain the maximum load effect. Figure 27 shows examples of cumulative probability distributions for different values of N_R. The plot shows how the cumulative distribution shifts to the right as N_R increases resulting in higher mean values $\overline{L}_{max}$ for the random truck that will cross alongside the Permit.

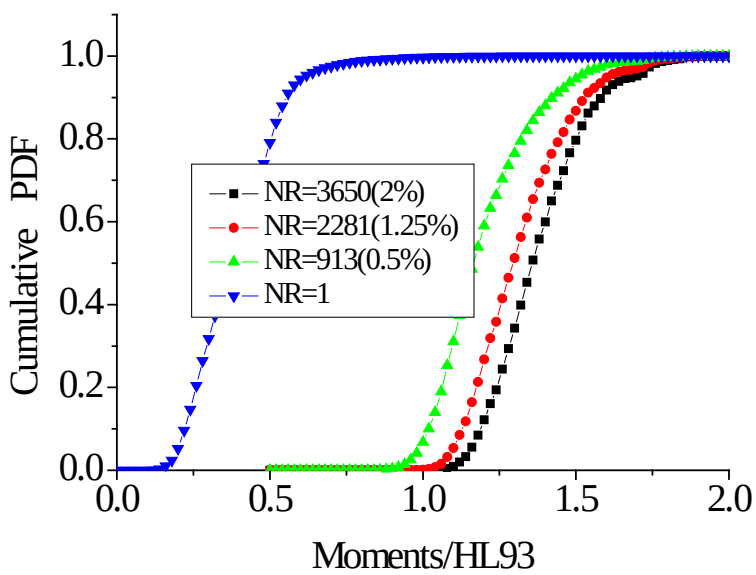

Figure 27. Plot of cumulative distribution for different number of events.

The cumulative distributions or the extreme value probabilities can be used to calculate the mean value and the COV of the maximum expected live load effect for different N_R for the WIM data collected from different U.S. sites for different span lengths. Table 31 gives a summary of the results for each of the ADTT considered. The summary gives the overall average of all the values represented by $\overline{L}_{max}$, the average of the COV's of each L_{max} given as V_{Lmax}, the variability of $\overline{L}_{max}$ from site to site expressed in terms of $V_{site\text{-}to\text{-}site}$ which is the COV of all the $\overline{L}_{max}$ values obtained from the twelve WIM sites analyzed in this study. These are provided for the cases where the random truck traffic volume is considered to be ADTT=5000 with 2% probability of side-by-side events as well as sites with ADTT=1000 with P_{sxs}=1.25% and ADTT=100 with P_{sxs}=100. Both the shear load and the bending moment are tabulated for span lengths varying between 20-ft and 200-ft. For example, $\overline{L}_{max}$ varies between 1.32 to 1.08 depending on the span length for the sites with ADTT=5000 when analyzing the bending effects of the trucks that could possibly run alongside a Permit truck. The COV for variations in L_{max} within a site varies between 7% and 9% while the site-to-site variability in L_{max} is between 23 and 18%.

Table 31. Summary of L_{max} statistics for random trucks crossing alongside Routine Permits

ADTT 5000						
	SIMPLE MOMENT			SIMPLE SHEAR		
SPAN (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.32	0.07	0.23	1.32	0.07	0.21
40	1.28	0.07	0.20	1.24	0.07	0.18
60	1.17	0.08	0.18	1.24	0.07	0.19
80	1.15	0.08	0.18	1.27	0.07	0.19
100	1.16	0.08	0.19	1.28	0.08	0.19
120	1.16	0.08	0.20	1.26	0.08	0.18
160	1.14	0.08	0.19	1.21	0.08	0.17
200	1.08	0.09	0.18	1.14	0.09	0.17

ADTT 1000						
	SIMPLE MOMENT			SIMPLE SHEAR		
SPAN (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.28	0.07	0.23	1.29	0.07	0.22
40	1.25	0.08	0.20	1.20	0.08	0.19
60	1.13	0.08	0.18	1.21	0.08	0.19
80	1.11	0.08	0.19	1.24	0.08	0.20
100	1.13	0.08	0.20	1.24	0.08	0.19
120	1.13	0.08	0.20	1.23	0.09	0.19
160	1.10	0.09	0.20	1.17	0.09	0.18
200	1.05	0.09	0.19	1.10	0.09	0.18

ADTT 100						
	SIMPLE MOMENT			SIMPLE SHEAR		
SPAN (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	1.21	0.08	0.23	1.22	0.08	0.22
40	1.17	0.09	0.21	1.13	0.09	0.20
60	1.06	0.10	0.20	1.14	0.09	0.20
80	1.04	0.10	0.20	1.16	0.09	0.21
100	1.06	0.09	0.21	1.17	0.09	0.20
120	1.06	0.10	0.22	1.15	0.10	0.20
160	1.03	0.10	0.21	1.09	0.10	0.20
200	0.98	0.11	0.21	1.02	0.11	0.20

It is further noted that the analysis of the data performed in NCHRP 12-76 demonstrates that the WIM data sample size will result in a variability in the estimated value of L_{max} for each site that may be expressed by a coefficient of variation $V_{sample\ size}$=2%.

Given N_R events where the permit truck will be alongside a random truck, the maximum load will occur when the permit truck will be alongside the one truck out of the N_R which produces the maximum load effect. The mean normalized live load effect of this truck is labeled $\overline{\boldsymbol{L}}_{\mathbf{max}}$ and its actual effect is $\overline{\boldsymbol{L}}_{\mathbf{max}} \times \boldsymbol{HL93}$. The maximum total live load effect on a member is obtained from:

$$LL = \left(P \times DF_P + \lambda_{site\ to\ site} \times \lambda_{sample\ size} \times L_{\max\ N_R} \times HL93 \times DF_R\right) IM \qquad (63)$$

Where P is the load effect of the permit truck, DF_P is the load distribution factor for the Permit load P, $\boldsymbol{L}_{\mathbf{max}}$ is the maximum load effect of N_R random trucks, DF_R is the distribution factor for the random load, and IM is the impact factor for side-by-side events. $\lambda_{site\ to\ site}$ is introduced to reflect the variability in L_{max} from site to site with a mean value of 1.0 and a COV $V_{site\ to\ site}$ obtained from Table 31 for the corresponding span length and ADTT. $\lambda_{sample\ size}$ is introduced to reflect the variability in L_{max} due to the WIM data sample size, it is associated with a mean value of 1.0 and a COV $V_{sample\ size}$=2% as per NCHRP 12-76.

The COV data provided in Table 31 is used to find the level of uncertainty associated with estimating the effect of the random truck that may cross the bridge alongside the Permit. Given that the analysis of the static load effect for one beam is associated with a COV V_{DF}, the overall coefficient of variation for the static random truck load effect $L^*_{\max} = L_{\max\ N_R} \times HL93 \times DF_R$ without the dynamic effect can then be estimated from:

$$\boldsymbol{V}_{\boldsymbol{L}\,\mathbf{max}^*} = \sqrt{\boldsymbol{V}^2_{\boldsymbol{L}\,\mathbf{max}} + \boldsymbol{V}^2_{site\ to\ site} + \boldsymbol{V}^2_{sample\ size} + \boldsymbol{V}^2_{DF}} \qquad (64)$$

If one assumes that some of the Routine Permit trucks may be overloaded, then the load effect P of Eq. (63) must be considered to be a random variable. In this set of calculations we assume that the Gross Vehicular weight associated with an overloaded permit follows an Exponential probability distribution with a parameter λ=0.15.

The tables for the load distribution factors, D.F., provided in the AASHTO LRFD for two lanes assume that the two side-by-side trucks are of equal weight. The same assumption is made in NCHRP 592 which compared the results of refined analysis to the LRFD load distribution factor tables. This assumption is clearly not accurate and cannot be used in the case where a Permit is alongside a random truck. Therefore, following the procedure provided in Eq. 4.6.2.2.4-1 of the LRFD, DF_p will be obtained based on the one-lane distribution factor, while DF_R is obtained from the difference between the DF of two lanes and that of a single lane. DF_R and DF_p are assumed to be correlated and their coefficients of variation are obtained as V_{DF} from Table 26.

For the side-by-side trucks, the mean value of IM is given as $\overline{\boldsymbol{IM}} = \mathbf{1.10}$ and the COV is V_{IM}=5.5%.

Simplified Analysis for Routine Permit alongside Random Truck

A reliability simplified analysis is performed for the case when the Routine Permit check is performed using a live load factor γ_L=1.80. The Routine Permit truck which weighs 80 kips produces a moment of 1368 kip-ft on an 80-ft composite steel bridge with beams at 8-ft center to center. The dead loads are given as D_{C1}=192.1 kip-ft, D_{C2}=736.3 kip-ft and D_w=172.8 kip-ft.

Given a distribution factor DF=0.644, the nominal resistance that would be needed to allow the crossing of the Permit is obtained as:

$$R_n = 1.25(192.1 + 736.3) + 1.50(172.8) + 1.80(1370.6 \times 0.644 \times 1.33) = 3533\ kip - ft$$

The mean resistance is obtained as $\overline{R} = 1.12 \times 3533 = 3957\ kip - ft$. The COV is V_R=10%.

The mean dead load is obtained as $\overline{D} = 1144\ kip - ft$ and the standard deviation is σ_D=90 kip-ft.

The mean weight of the Permit truck is obtained as 80+1/0.15=86.7 kips and the standard deviation is 6.7 or the COV is V_P=7.7%. The mean moment is thus obtained as 1485 kip-ft.

Given the bias values in Table 25 for the distribution factors, the unbiased two-lane distribution factor is 0.644/1.11=0.58. The unbiased one lane distribution factor after removing the multiple presence factor MP=1.2 becomes (0.461/1.2)/1.29=0.30. Accordingly, the load distribution factors for the Permit and the Random trucks respectively are DF_P=0.30 and DF_R=0.58-0.30=0.28

The mean static effect of the Permit truck is obtained from $\overline{P}^* = \overline{P} \times \overline{DF}_P = 1485 \times 0.30 = 446\ kip - ft$ and the COV is $V_P^* = \sqrt{V_P^2 + V_{DF}^2} = \sqrt{(7.7\%)^2 + (14\%)^2} = 16\%$ and the standard deviation is $\sigma_P^* = 71\ kip - ft$.

For sites with ADTT=5000, the mean static effect of the Random Truck is: $\overline{L}^* = \overline{L}_{\max} \times HL_{93} \times \overline{DF}_R = 1.15 \times 1672 \times 0.28 = 538\ kip - ft$. The COV is $V_{L*} = \sqrt{(8\%)^2 + (18\%)^2 + (2\%)^2 + (14\%)^2} = 24\%$ or the standard deviation is $\sigma_L^* = 130\ kip - ft$

Assuming uncorrelated variables, the standard deviation of the total static effect is $\sigma_{LL*} = \sqrt{130^2 + 71^2} = 148\ kip - ft$, the mean is 446+538=984 kip-ft and the COV is 15%.

By including the dynamic effect the final live load effect has a mean value $\overline{LL} = 984 \times 1.09 = 1073\ kip - ft$ and the COV is $V_{LL} = \sqrt{(15\%)^2 + (5.5\%)^2} = 16\%$ or the standard deviation is σ_{LL}=172 kip-ft.

The mean final total load including live and dead loads is $\overline{S} = 1073 + 1144 = 2217\ kip - ft$ and the final standard deviation is $\sigma_S = \sqrt{172^2 + 90^2} = 194\ kip - ft$ or a COV V_S=9%.

The approximate reliability index is obtained as: $\beta = \dfrac{\ln\left(\dfrac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \dfrac{\ln\left(\dfrac{3957}{2217}\right)}{\sqrt{(10\%)^2 + (9\%)^2}} = 4.31$

The application of the FORM algorithm assuming that the load distribution factors DF_R and DF_P are fully correlated leads to a reliability index for this example equal to β=4.14 which is quite close to the value 4.31 obtained from the hand calculation example with the various simplifying assumptions on the correlation and probability distribution type.

Reliability Indexes for Routine Permit Mixing with Random Truck Traffic

The calculation of the reliability index values is executed using the FORM algorithm with the failure function expressed as shown in Eq. (56) with the expression for the live load effect LL of Eq. (63). The analysis assumes that R is Lognormal, DL is Normal, and L_{max} is Gumbel. The effect of the Routine Permit truck follows an Exponential distribution. The rest of the random variables are assumed to follow Normal distributions and the load distribution factors DF_R and DF_P are assumed to be fully correlated having the COV V_{DF} for two lanes of traffic as given in Table 26.

For Routine Permits, the AASHTO LRFR table gives a range of live load factors between γ_L=1.10 and γ_L=1.80 depending on the ADTT and the Permit weight. The Trucks are divided into three gross weight categories: a) Truck with Gross Vehicles GVW up to 100 kips, b) Trucks with 100 kip $\leq$ GVW $<$ 150 kips, and c) Trucks with GVW $\geq$ 150 kip. For the case when GVW < 100 kips, the live load factors are selected to be equal to those of the Legal trucks. This is based on the assumption that the probability of a permit truck would exceed the permit limit is similar to that of a random truck exceeding the legal limits. This assumption was made due to the lack of actual data. Although it is true that Permit truck often exceed the permit limits the levels by which these limits are exceeded is considerably less than that of the number of illegal random trucks. Also, lower live load factors are specified in the AASHTO LRFR for the cases where the GVW exceed 100 kips to account for the lower probability of having a random truck alongside the permit with equal or higher load as the Permit truck weight increases.

Also, due to the lack of data, the categorization of the trucks by GVW groups as done in the AASHTO LRFR assumes that all random and permit trucks have the same semi-trailer configurations. This categorization does not take into consideration differences in the axle configurations. In order, to better reflect the effect of the different truck types, it is herein proposed to categorize the trucks based on a combination of their gross vehicle weights (GVW) in kips and their first to rear axle lengths (AL) in feet. Thus, three different truck group categories are established: a) trucks with GVW/AL < 2.0 (kip/ft), b) 2.0 $\leq$ GVW/AL $<$ 3.0, and c) Trucks with GVW/AL $\geq$ 3.0.

The results for various values of the live load factor γ_L applied to check the Permit load safety are presented in Table 32 for the different truck weight categories. The following observations are then made:

- For Permit weight category (a) allowed on sites with ADTT=5000, a live load factor γ_L=1.35 would lead to an average reliability index β_{ave}=2.78 which is higher than the target β_{target}=2.50 while the minimum reliability remains above β_{min}=1.50. However, to remain consistent with other weight groups and ADTT categories, it is recommended to use a live load factor γ_L=1.40 which produces an average reliability index β_{ave}=2.89.
- For Permit weight category (b) allowed on sites with ADTT=5000, a live load factor γ_L=1.30 would lead to an average reliability index β_{ave}=2.94 which is higher than the target β_{target}=2.50 while the minimum reliability remains above β_{min}=1.50.
- For Permit weight category (c) allowed on sites with ADTT=5000, a live load factor γ_L=1.25 would lead to an average reliability index β_{ave}=3.18 which is

higher than the target β_{target}=2.50 while the minimum reliability remains above β_{min}=1.50.

- For Permit weight category (a) allowed on sites with ADTT=1000, a live load factor γ_L=1.35 would lead to an average reliability index β_{ave}=2.82 which is higher than the target β_{target}=2.50 while the minimum reliability of 1.62 remains above β_{min}=1.50.
- For Permit weight category (b) allowed on sites with ADTT=1000, a live load factor γ_L=1.25would lead to an average reliability index β_{ave}=2.86 which is higher than the target β_{target}=2.50 while the minimum reliability is equal to the minimum value β_{min}=1.50.
- For Permit weight category (c) allowed on sites with ADTT=1000, a live load factor γ_L=1.20 would lead to an average reliability index β_{ave}=3.08 which is higher than the target β_{target}=2.50 while the minimum reliability remains slightly above β_{min}=1.50.
- For Permit weight category (a) allowed on sites with ADTT=100, a live load factor γ_L=1.30 would lead to an average reliability index β_{ave}=2.80 which is higher than the target β_{target}=2.50 while the minimum reliability remains above β_{min}=1.50.
- For Permit weight category (b) allowed on sites with ADTT=100, a live load factor γ_L=1.20 would lead to an average reliability index β_{ave}=2.81 which is higher than the target β_{target}=2.50 while the minimum reliability at β=1.47 is just a little lower than β_{min}=1.50.
- For Permit weight category (c) allowed on sites with ADTT=100, a live load factor γ_L=1.15 would lead to an average reliability index β_{ave}=3.01 which is higher than the target β_{target}=2.50 while the minimum reliability is at β_{min}=1.50.

Table 32. Reliability index values for routine permits mixed with random trucks.

ROUTINE PERMIT WLG1 ADTT = 5000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.22	2.75	2.25	1.37	2.89	2.36	1.52	3.04	2.47	1.67	3.19	2.57	1.81	3.34	2.68	1.94	3.48	2.78	2.08	3.62	2.88
T-beam shear	1.51	2.90	2.40	1.65	3.00	2.51	1.79	3.12	2.62	1.92	3.26	2.73	2.04	3.40	2.84	2.17	3.54	2.94	2.29	3.67	3.05
PS/C bending	0.68	2.69	1.95	0.86	2.78	2.09	1.04	2.92	2.23	1.21	3.10	2.36	1.38	3.30	2.49	1.54	3.49	2.62	1.70	3.67	2.75
PS/C shear	1.01	2.40	1.91	1.15	2.51	2.02	1.29	2.64	2.14	1.43	2.79	2.25	1.56	2.95	2.36	1.69	3.09	2.46	1.81	3.23	2.57
Noncomp. Bending	0.93	2.71	1.99	1.10	2.80	2.13	1.26	2.90	2.26	1.42	3.06	2.38	1.57	3.24	2.51	1.72	3.42	2.63	1.87	3.59	2.75
Noncomp. Shear	1.33	3.11	2.37	1.49	3.24	2.51	1.65	3.37	2.66	1.80	3.52	2.80	1.95	3.66	2.94	2.09	3.81	3.07	2.23	3.96	3.20
Comp. bending	0.93	2.70	1.99	1.10	2.80	2.12	1.26	2.90	2.25	1.42	3.06	2.38	1.57	3.24	2.50	1.72	3.42	2.63	1.87	3.59	2.75
Comp. shear	1.33	3.11	2.36	1.49	3.24	2.51	1.65	3.37	2.65	1.80	3.52	2.80	1.95	3.66	2.93	2.09	3.81	3.07	2.23	3.96	3.20
ALL BRIDGE TYPES	0.68	3.11	2.15	0.86	3.24	2.28	1.04	3.37	2.41	1.21	3.52	2.53	1.38	3.66	2.66	1.54	3.81	2.78	1.70	3.96	2.89

ROUTINE PERMIT WLG1 ADTT = 1000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.39	2.93	2.47	1.55	3.04	2.59	1.70	3.16	2.71	1.85	3.28	2.83	1.99	3.40	2.94	2.14	3.54	3.05	2.27	3.68	3.16
T-beam shear	1.68	2.99	2.60	1.83	3.11	2.72	1.97	3.23	2.84	2.10	3.35	2.96	2.23	3.48	3.08	2.36	3.61	3.19	2.48	3.74	3.30
PS/C bending	0.88	2.88	2.24	1.07	3.01	2.39	1.25	3.16	2.54	1.43	3.33	2.69	1.60	3.49	2.83	1.77	3.65	2.97	1.93	3.81	3.11
PS/C shear	1.18	2.49	2.11	1.33	2.62	2.23	1.47	2.75	2.35	1.62	2.89	2.47	1.75	3.02	2.59	1.89	3.16	2.70	2.02	3.29	2.82
Noncomp. Bending	1.12	2.89	2.27	1.29	2.99	2.42	1.45	3.12	2.56	1.62	3.25	2.70	1.78	3.41	2.83	1.93	3.56	2.96	2.08	3.72	3.09
Noncomp. Shear	1.53	3.22	2.62	1.70	3.36	2.77	1.86	3.51	2.93	2.01	3.65	3.08	2.17	3.81	3.23	2.32	3.97	3.37	2.46	4.13	3.51
Comp. bending	1.11	2.88	2.27	1.29	2.99	2.41	1.45	3.11	2.55	1.62	3.24	2.69	1.77	3.40	2.83	1.93	3.56	2.96	2.08	3.71	3.09
Comp. shear	1.53	3.22	2.61	1.69	3.36	2.77	1.86	3.51	2.93	2.01	3.66	3.08	2.17	3.81	3.22	2.31	3.97	3.37	2.46	4.13	3.51
ALL BRIDGE TYPES	0.88	3.22	2.40	1.07	3.36	2.54	1.25	3.51	2.68	1.43	3.66	2.81	1.60	3.81	2.94	1.77	3.97	3.07	1.93	4.13	3.20

ROUTINE PERMIT WLG1 ADTT = 100	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.57	3.53	2.86	1.73	3.67	2.99	1.89	3.82	3.13	2.04	3.96	3.26	2.19	4.10	3.38	2.33	4.23	3.51	2.47	4.37	3.63
T-beam shear	2.02	3.35	2.86	2.17	3.51	2.99	2.32	3.67	3.12	2.46	3.82	3.24	2.60	3.97	3.37	2.73	4.12	3.49	2.86	4.26	3.60
PS/C bending	1.09	3.62	2.68	1.28	3.83	2.85	1.47	4.04	3.02	1.65	4.24	3.18	1.83	4.44	3.34	2.01	4.63	3.50	2.17	4.82	3.65
PS/C shear	1.53	2.88	2.33	1.69	3.06	2.46	1.84	3.24	2.59	1.99	3.41	2.72	2.13	3.57	2.84	2.24	3.73	2.97	2.31	3.88	3.08
Noncomp. Bending	1.31	3.53	2.68	1.49	3.72	2.84	1.66	3.91	3.00	1.83	4.10	3.15	1.99	4.28	3.30	2.15	4.46	3.44	2.30	4.63	3.59
Noncomp. Shear	1.92	3.54	2.88	2.10	3.71	3.05	2.27	3.90	3.21	2.43	4.10	3.37	2.60	4.28	3.53	2.75	4.47	3.68	2.91	4.64	3.83
Comp. bending	1.31	3.53	2.68	1.48	3.72	2.84	1.66	3.91	3.00	1.82	4.10	3.15	1.99	4.28	3.30	2.15	4.46	3.44	2.30	4.63	3.59
Comp. shear	1.92	3.54	2.88	2.10	3.71	3.05	2.27	3.90	3.21	2.43	4.10	3.37	2.60	4.28	3.53	2.75	4.47	3.68	2.91	4.64	3.83
ALL BRIDGE TYPES	1.09	3.62	2.73	1.28	3.83	2.88	1.47	4.04	3.03	1.65	4.24	3.18	1.83	4.44	3.32	2.01	4.63	3.46	2.17	4.82	3.60

ROUTINE PERMIT WLG2 ADTT = 5000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.28	2.80	2.29	1.43	2.94	2.40	1.58	3.09	2.51	1.73	3.24	2.62	1.87	3.38	2.72	2.01	3.52	2.82	2.14	3.66	2.92
T-beam shear	1.57	2.92	2.43	1.71	3.02	2.55	1.84	3.15	2.66	1.97	3.29	2.77	2.10	3.43	2.88	2.22	3.57	2.98	2.34	3.70	3.08
PS/C bending	0.76	2.72	2.01	0.94	2.82	2.15	1.11	2.98	2.28	1.29	3.16	2.42	1.45	3.35	2.55	1.62	3.54	2.68	1.77	3.73	2.80
PS/C shear	1.06	2.42	1.95	1.21	2.53	2.06	1.35	2.67	2.17	1.48	2.83	2.29	1.62	2.98	2.39	1.75	3.12	2.50	1.87	3.27	2.61
Noncomp. Bending	1.00	2.74	2.05	1.17	2.84	2.18	1.33	2.94	2.31	1.49	3.11	2.44	1.64	3.29	2.56	1.79	3.47	2.68	1.93	3.64	2.80
Noncomp. Shear	1.40	3.13	2.41	1.56	3.26	2.56	1.71	3.41	2.70	1.87	3.55	2.84	2.01	3.70	2.98	2.16	3.85	3.12	2.30	4.00	3.25
Comp. bending	1.00	2.74	2.04	1.16	2.84	2.17	1.33	2.93	2.30	1.48	3.11	2.43	1.64	3.29	2.56	1.79	3.47	2.68	1.93	3.64	2.80
Comp. shear	1.39	3.13	2.41	1.56	3.26	2.56	1.71	3.40	2.70	1.86	3.55	2.84	2.01	3.70	2.98	2.16	3.85	3.12	2.30	4.00	3.25
ALL BRIDGE TYPES	0.76	3.13	2.20	0.94	3.26	2.33	1.11	3.41	2.46	1.29	3.55	2.58	1.45	3.70	2.70	1.62	3.85	2.82	1.77	4.00	2.94

ROUTINE PERMIT WLG2 ADTT = 1000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.45	2.96	2.52	1.61	3.08	2.64	1.76	3.20	2.76	1.91	3.32	2.87	2.06	3.44	2.98	2.20	3.58	3.09	2.33	3.72	3.20
T-beam shear	1.74	3.01	2.64	1.88	3.13	2.76	2.02	3.25	2.88	2.16	3.38	3.00	2.29	3.51	3.11	2.41	3.64	3.23	2.54	3.76	3.34
PS/C bending	0.95	2.92	2.30	1.14	3.05	2.45	1.32	3.21	2.60	1.50	3.38	2.74	1.67	3.54	2.89	1.84	3.70	3.03	2.01	3.86	3.17
PS/C shear	1.24	2.51	2.14	1.39	2.64	2.27	1.53	2.78	2.39	1.67	2.91	2.51	1.81	3.05	2.63	1.94	3.19	2.74	2.07	3.32	2.85
Noncomp. Bending	1.18	2.92	2.32	1.35	3.03	2.47	1.52	3.16	2.61	1.68	3.29	2.75	1.84	3.45	2.88	2.00	3.61	3.01	2.15	3.76	3.14
Noncomp. Shear	1.59	3.25	2.66	1.76	3.38	2.82	1.92	3.54	2.97	2.08	3.68	3.12	2.23	3.84	3.27	2.38	4.00	3.41	2.52	4.16	3.55
Comp. bending	1.18	2.92	2.32	1.35	3.03	2.46	1.52	3.15	2.61	1.68	3.29	2.74	1.84	3.45	2.88	2.00	3.61	3.01	2.15	3.76	3.14
Comp. shear	1.59	3.25	2.66	1.76	3.38	2.82	1.92	3.54	2.97	2.08	3.69	3.12	2.23	3.84	3.27	2.38	4.00	3.41	2.52	4.16	3.55
ALL BRIDGE TYPES	0.95	3.25	2.44	1.14	3.38	2.58	1.32	3.54	2.72	1.50	3.69	2.86	1.67	3.84	2.99	1.84	4.00	3.12	2.01	4.16	3.24

ROUTINE PERMIT WLG2 ADTT = 100	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.62	3.56	2.90	1.79	3.71	3.03	1.95	3.85	3.17	2.10	3.99	3.30	2.25	4.13	3.42	2.39	4.27	3.55	2.53	4.41	3.67
T-beam shear	2.07	3.39	2.89	2.22	3.55	3.02	2.37	3.71	3.15	2.51	3.86	3.28	2.65	4.01	3.40	2.78	4.15	3.52	2.91	4.30	3.64
PS/C bending	1.16	3.66	2.73	1.36	3.87	2.90	1.54	4.08	3.07	1.73	4.28	3.23	1.90	4.48	3.39	2.08	4.68	3.54	2.25	4.86	3.70
PS/C shear	1.58	2.92	2.36	1.74	3.10	2.50	1.89	3.28	2.63	2.04	3.45	2.75	2.19	3.61	2.88	2.28	3.77	3.00	2.34	3.92	3.12
Noncomp. Bending	1.37	3.57	2.73	1.55	3.76	2.89	1.72	3.95	3.05	1.89	4.14	3.20	2.05	4.32	3.35	2.21	4.50	3.49	2.37	4.67	3.63
Noncomp. Shear	1.98	3.58	2.92	2.16	3.75	3.09	2.33	3.94	3.25	2.49	4.14	3.41	2.66	4.33	3.57	2.81	4.51	3.72	2.97	4.69	3.87
Comp. bending	1.37	3.57	2.73	1.55	3.76	2.89	1.72	3.95	3.04	1.89	4.14	3.20	2.05	4.32	3.34	2.21	4.50	3.49	2.37	4.67	3.63
Comp. shear	1.98	3.58	2.92	2.16	3.75	3.09	2.33	3.94	3.25	2.49	4.14	3.41	2.66	4.33	3.57	2.81	4.51	3.72	2.97	4.69	3.87
ALL BRIDGE TYPES	1.16	3.66	2.77	1.36	3.87	2.93	1.54	4.08	3.08	1.73	4.28	3.22	1.90	4.48	3.36	2.08	4.68	3.50	2.25	4.86	3.64

ROUTINE PERMIT WLG3 ADTT = 5000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.41	2.88	2.38	1.56	3.03	2.49	1.71	3.18	2.60	1.86	3.33	2.70	2.00	3.47	2.81	2.14	3.62	2.91	2.27	3.75	3.01
T-beam shear	1.68	2.96	2.51	1.83	3.07	2.62	1.96	3.21	2.74	2.09	3.36	2.85	2.22	3.49	2.95	2.34	3.63	3.06	2.46	3.76	3.16
PS/C bending	0.91	2.80	2.12	1.09	2.91	2.26	1.27	3.09	2.39	1.44	3.27	2.53	1.60	3.47	2.66	1.77	3.65	2.78	1.93	3.83	2.91
PS/C shear	1.18	2.47	2.02	1.33	2.58	2.14	1.47	2.74	2.25	1.61	2.89	2.36	1.74	3.04	2.47	1.87	3.19	2.58	1.99	3.34	2.68
Noncomp. Bending	1.14	2.82	2.15	1.31	2.91	2.29	1.47	3.04	2.41	1.63	3.22	2.54	1.78	3.40	2.66	1.93	3.58	2.79	2.07	3.75	2.90
Noncomp. Shear	1.53	3.19	2.51	1.69	3.32	2.65	1.84	3.47	2.80	2.00	3.61	2.94	2.14	3.76	3.07	2.29	3.91	3.21	2.43	4.07	3.34
Comp. bending	1.14	2.81	2.15	1.30	2.91	2.28	1.47	3.04	2.41	1.62	3.22	2.54	1.78	3.40	2.66	1.93	3.58	2.78	2.07	3.74	2.90
Comp. shear	1.53	3.19	2.50	1.69	3.32	2.65	1.84	3.47	2.79	2.00	3.61	2.93	2.14	3.76	3.07	2.29	3.91	3.21	2.43	4.07	3.34
ALL BRIDGE TYPES	0.91	3.19	2.29	1.09	3.32	2.42	1.27	3.47	2.55	1.44	3.61	2.67	1.60	3.76	2.80	1.77	3.91	2.91	1.93	4.07	3.03

ROUTINE PERMIT WLG3 ADTT = 1000	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.57	3.04	2.60	1.73	3.16	2.72	1.89	3.28	2.84	2.04	3.40	2.96	2.18	3.53	3.07	2.32	3.67	3.18	2.46	3.81	3.29
T-beam shear	1.85	3.05	2.71	2.00	3.18	2.83	2.13	3.30	2.95	2.27	3.43	3.07	2.40	3.57	3.19	2.53	3.70	3.30	2.65	3.82	3.41
PS/C Bending	1.10	3.00	2.40	1.29	3.13	2.56	1.47	3.30	2.70	1.65	3.47	2.85	1.82	3.63	2.99	1.99	3.79	3.13	2.15	3.94	3.27
PS/C shear	1.35	2.56	2.22	1.50	2.70	2.34	1.65	2.83	2.46	1.79	2.97	2.58	1.93	3.11	2.70	2.06	3.25	2.82	2.19	3.40	2.93
Noncomp. Bending	1.32	2.99	2.43	1.49	3.11	2.57	1.66	3.23	2.71	1.82	3.39	2.85	1.98	3.54	2.98	2.13	3.70	3.11	2.28	3.85	3.24
Noncomp. Shear	1.72	3.30	2.75	1.89	3.44	2.91	2.05	3.60	3.06	2.21	3.74	3.21	2.36	3.91	3.36	2.51	4.07	3.50	2.65	4.23	3.64
Comp. bending	1.32	2.99	2.42	1.49	3.11	2.57	1.66	3.23	2.71	1.82	3.39	2.84	1.98	3.54	2.98	2.13	3.70	3.11	2.28	3.85	3.24
Comp. shear	1.72	3.30	2.75	1.88	3.44	2.91	2.05	3.60	3.06	2.20	3.75	3.21	2.36	3.91	3.36	2.51	4.07	3.50	2.65	4.23	3.64
ALL BRIDGE TYPES	1.10	3.30	2.53	1.29	3.44	2.68	1.47	3.60	2.81	1.65	3.75	2.95	1.82	3.91	3.08	1.99	4.07	3.21	2.15	4.23	3.33

ROUTINE PERMIT WLG3 ADTT = 100	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L=1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.75	3.63	2.98	1.91	3.78	3.11	2.07	3.92	3.25	2.22	4.07	3.38	2.37	4.20	3.50	2.52	4.34	3.63	2.66	4.50	3.75
T-beam shear	2.18	3.46	2.96	2.33	3.62	3.09	2.48	3.78	3.22	2.62	3.93	3.35	2.76	4.08	3.47	2.89	4.23	3.59	3.02	4.37	3.71
PS/C bending	1.31	3.75	2.83	1.50	3.96	3.00	1.69	4.17	3.17	1.87	4.37	3.33	2.05	4.56	3.48	2.22	4.75	3.64	2.39	4.93	3.79
PS/C shear	1.69	3.00	2.43	1.85	3.18	2.57	2.01	3.36	2.70	2.16	3.53	2.82	2.28	3.69	2.95	2.35	3.85	3.07	2.41	4.00	3.19
Noncomp. Bending	1.51	3.66	2.83	1.69	3.85	2.99	1.86	4.04	3.14	2.03	4.22	3.29	2.19	4.40	3.44	2.35	4.58	3.58	2.50	4.75	3.72
Noncomp. Shear	2.10	3.66	3.01	2.28	3.83	3.18	2.45	4.03	3.34	2.62	4.22	3.50	2.78	4.41	3.65	2.94	4.59	3.80	3.08	4.77	3.95
Comp. bending	1.51	3.66	2.82	1.68	3.85	2.98	1.86	4.04	3.14	2.03	4.22	3.29	2.19	4.40	3.44	2.35	4.58	3.58	2.50	4.75	3.72
Comp. shear	2.10	3.66	3.01	2.28	3.83	3.17	2.45	4.03	3.34	2.62	4.22	3.50	2.78	4.41	3.65	2.94	4.59	3.80	3.07	4.77	3.95
ALL BRIDGE TYPES	1.31	3.75	2.86	1.50	3.96	3.01	1.69	4.17	3.16	1.87	4.37	3.31	2.05	4.56	3.45	2.22	4.75	3.59	2.39	4.93	3.72

Summary and Recommendation

Case III – Permit Truck alongside Random Truck for Unlimited Permit Crossings.

In summary for the Routine Permits, the live load factors summarized in Table 33 are recommended. For the most heavy trucks with GVW/AL>3.0 the live load factors are reasonably similar to those in the current AASHTO LRFR. For the lower truck weight category, the live load factors are lower than those in the AASHTO LRFR reflecting the lower level of uncertainty associated with the estimation of the Permit truck load effects used in this study. These observations are made although the overall COV for the random truck load effects (due to the higher site to site variability in the WIM data and the higher variability in the load distribution factors) are higher than those used in the AASHTO LRFR calibration. However, this increase in the COV seems to be offset by the conservative bias observed with the AASHTO LRFD load distribution equations which were not accounted for during the AASHTO LRFR calibration.

Table 33. Recommended Live Load Factors for Routine Permits

		Truck Volume	GVW/AL<2.0	2.0<GVW/AL<3.0	GVW/AL>3.0
Unlimited crossings of Routine Permits	AASHTO LRFD two or more lane D.F.	ADTT=5000	γ_L=1.40	γ_L=1.35	γ_L=1.30
		ADTT=1000	γ_L=1.35	γ_L=1.25	γ_L=1.20
		ADTT=100	γ_L=1.30	γ_L=1.20	γ_L=1.15

Reliability Analysis for Case IV – Permit Truck alongside a Random Truck for a limited number of Permit Crossings.

The reliability analysis is executed assuming that there are a total of 100 crossings of a Special permit over the rating Period. Initially, we assume that the Permit will always cross the bridge alongside a Random truck. Thus, the number of side-by-side events is assumed to be N_R=100. The maximum effect of the random truck that would cross alongside the Permit L_{max} will be associated with the statistical values given in Table 34. The reliability analysis conditional on having a random truck alongside the Permit uses the same steps followed when analyzing Case III but with the L_{max} data of Table 34 rather than those of Table 31. Table 34 shows a considerable reduction in the L_{max} values compared to the values shown in Table 31 for the unlimited number of Permit Crossings. On the other hand, the COV of L_{max} shown in Table 34 is considerably higher. The COV reflecting the site-to-site variability remains on the same order with a small reduction in $V_{site\text{-}to\text{-}site}$ for the short spans.

Table 34. Maximum live load effect L_{max} for N_R=100

N=100	AVERAGE OF ALL STATES			AVERAGE OF ALL STATES		
	SIMPLE MOMENT			SIMPLE SHEAR		
Span (ft)	Average Lmax	Average COV	site to site COV	Average Lmax	Average COV	site to site COV
20	0.80	0.17	0.19	0.81	0.18	0.19
40	0.77	0.19	0.18	0.75	0.19	0.17
60	0.69	0.20	0.17	0.75	0.19	0.18
80	0.67	0.20	0.17	0.76	0.20	0.20
100	0.68	0.20	0.18	0.75	0.20	0.19
120	0.67	0.21	0.19	0.73	0.22	0.19
160	0.64	0.22	0.18	0.68	0.24	0.20
200	0.60	0.24	0.18	0.63	0.25	0.19

The maximum total live load effect on a member conditional on having a random truck alongside the Permit is obtained from Eq. (63) that is repeated below for convenience:

$$LL = \left(P \times DF_P + \lambda_{site\ to\ site} \times \lambda_{sample\ size} \times L_{\max} \times HL93 \times DF_R\right) IM \qquad (63)$$

where P is the load effect of the permit truck, DF_P is the load distribution factor for the load P, in this case $L_{\max}$ is the maximum load effect of N_R=100 random trucks, DF_R is the distribution factor for the random load, and IM is the impact factor for side-by-side events. $\lambda_{site\ to\ site}$ is introduced to reflect the variability in L_{max} from site to site with a mean value of 1.0 and a COV $V_{site\ to\ site}$ obtained from Table 34 for the corresponding span length and ADTT. $\lambda_{sample\ size}$ is introduced to reflect the variability in L_{max} due to the WIM data sample size, it is associated with a mean value of 1.0 and a COV $V_{sample\ size}$=2% as per NCHRP 12-76.

If one assumes that the Special Permit trucks that have been issued the Limited number of Permit trips, then the load effect P of Eq. (63) are known, then P of Eq. (63) is assumed to be a deterministic variable for this Case.

The tables for the load distribution factors, D.F., provided in the AASHTO LRFD for two lanes assume that the two side-by-side trucks are of equal weight. The same assumption is made in NCHRP 592 which compared the results of refined analysis to the LRFD load distribution factor tables. This assumption is clearly not accurate and cannot be used in the case where a

Permit is alongside a random truck. Therefore, following the procedure provided in Eq. 4.6.2.2.4-1 of the LRFD, DF_p will be obtained based on the one-lane distribution factor, while DF_R is obtained from the difference between the DF of two lanes and that of a single lane. DF_R and DF_p are assumed to be correlated and their coefficients of variation is obtained as V_{DF} for the two lanes from Table 26.

For the side-by-side trucks, the mean value of IM is given as $\overline{IM} = 1.10$ and the COV is VIM=5.5%.

The calculation of the reliability index values conditional on having the 100 permits cross the bridge simultaneously with a Random truck is executed using the FORM algorithm with the failure function expressed as shown in Eq. (56) with LL of Eq. (63). R is Lognormal, DL is Normal, and L_{max} is Gumbel. The effect of the Permit truck is known. The rest of the random variables are assumed to follow Normal distributions. The conditional reliability index is designated as β_c.

The probability that a bridge member would exceed the limit state of Eq. 56 conditional on having two permit vehicles side-by-side can be calculated from:

$$P_{f|side-by-side} = \Phi(-\beta_C) \tag{65}$$

where $\Phi(...)$ is the cumulative standard Normal distribution function. The final unconditional probability of failure will depend on the conditional probability of limit state exceedance given two side-by-side permit events, $P_{f|side-by-side}$ and the probability of having a situation with side-by-side permits, P_{SxS}. Thus:

$$P_f = P_{f|side-by-side} \times P_{S\times S} \tag{66}$$

where the probability of having two-side-by-side a random truck along side a permit P_{SxS} is related to the ADTT The probability of having a permit alongside a random truck depends on the probability of side-by-side events P_{sxs}. For sites with ADTT =5000 the side-by-side probability is P_{SxS}=2%. For sites with ADTT=1000 the probability is P_{sxs}=1.25% and for ADTT=100 P_{sxs}=0.5%. The final unconditional reliability index, β, is obtained from:

$$\beta = -\Phi^{-1}(P_f) \tag{67}$$

Simplified Example for Permit Checks with AASHTO LRFD Load Distribution Factors

The AASHTO LRFR recommends to use the single lane load distribution factor after removing the multiple presence factor MP=1.2 when checking if a Permit truck should be allowed to cross a bridge for a limited number of multiple trips. A simplified reliability calculation analysis is first provided for one example to study the interaction between the different parameters. The case considered in this example is for a composite steel bridge member in bending where the span length is 100-ft and the beam spacing is 8-ft. We assume a live load factor γ_L=1.10. The dead load moments are given as D_{C1}=386 kip-ft, D_{C2}=1150 kip-ft and D_w=270 kip-ft. The Special Permit truck that is being checked has a moment effect P=2380.5 kip-ft. The AASHTO LRFD load distribution factor for a single lane is calculated to be DF_1=0.435. The Permit truck would be allowed on the bridge if the nominal resistance capacity of the bridge member is:

$$R_n = 1.25(386+1150)+1.50(270)+1.10(2380.5\times 1.33\times 0.435/1.2) = 3588\ kip-ft$$

This indicates that the mean member resistance is $\overline{R} = 1.12\times 3588 = 4018\ kip-ft$ the COV for the resistance is V_R=10%.

The mean dead load is obtained as: $\overline{D} = 1.03D_{C1} + 1.05D_{C2} + 1.00D_W = 1875\ kip-ft$, the corresponding standard deviation is σ_D=142 kip-ft.

The two-lane AASHTO LRFD distribution factor is DF_2=0.619. After removing the bias given in Table 26, the expected distribution factor is $\overline{DF}_2$=0.619/1.11=0.558. The one lane AASHTO distribution factor without the multiple presence is DF_1=0.435/1.2=0.363. After removing the bias, the mean value is $\overline{DF}_1$=0.363/1.29=0.281. The distribution factor applied on the Permit truck is thus assumed to be $\overline{DF}_P$=0.281 while the mean value of the distribution factor applied on the random truck that may cross the bridge alongside the Permit is $\overline{DF}_R = \overline{DF}_2 - \overline{DF}_P$=0.558-0.281=0.277.

The mean value of static Permit load effect is $P\times DF_P$=2381$\times$0.281=669 kip-ft. The standard deviation is 669$\times$14%=94 kip-ft.

The mean value of the static random truck effect, $\lambda_{site\ to\ site}\times\lambda_{sample\ size}\times L_{\max N_R}\times HL93\times DF_R$ is given as $1.0\times 1.0\times 0.68\times 2320\times 0.277 = 437\ kip-ft$.

The COV is V*=$\sqrt{(20\%)^2+(2\%)^2+(18\%)^2+(14\%)^2} = 30\%$ leading to a standard deviation = 437$\times$30%=132 kip-ft.

The mean and standard deviation of the total static live load effect $\left(P\times DF_P + \lambda_{site\ to\ site}\times\lambda_{sample\ size}\times L_{\max N_R}\times HL93\times DF_R\right)$ are obtained as 1106 kip-ft and $\sigma_L^* = \sqrt{94^2+132^2} = 162\ kip-ft$. The COV being $V_L^* = 162/1106 = 15\%$.

Given a mean impact for side-by-side trucks $\overline{IM}$ =1.09 and V_{IM}=5.5%, the mean and COV of the total live load, LL, are given as:

$\overline{LL} = 1106\times 1.09 = 1205\ kip-ft$, $V_{LL} = \sqrt{(15\%)^2+(5.5\%)^2} = 16\%$ and the standard deviation is $\sigma_{LL} = 193\ kip-ft$

The mean of the total load including live and dead loads is $\overline{S} = \overline{DL} + \overline{LL} = 1875 + 1205 = 3080\ kip-ft$. The standard deviation is $\sigma_S = \sqrt{193^2+142^2} = 240\ kip-ft$ with a COV V_S=240/3080=8%.

An approximation for the unconditional reliability index for this case is given as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{4018}{3080}\right)}{\sqrt{(10\%)^2 + (8\%)^2}} = 2.08$$

The above calculations assumed that DF_P and DF_R are independent. Also, it is assumed that a random truck will always cross the bridge alongside the Permit vehicle. Using the FORM algorithm with the appropriate probability distribution function for each of the random variable and assuming that DF_P and DF_R are correlated, the unconditional reliability index is calculated to be β=2.03 which is only slightly lower than the 2.08 value obtained from the simplified calculations.

Reliability Analysis for Permit Checks with AASHTO LRFD Load Distribution Factors

The FORM algorithm is subsequently used to find the conditional reliability for a range of bridge configurations and for different live load factor γ_L values ranging between 1.10 and 1.40. The results are summarized in Table 35 which gives the conditional reliability index values in the cases where the Permit check is executed using the AASHTO LRFD load distribution factors. The unconditional reliability indexes are provided in Table 36.

Table 35. Conditional Reliability Indexes for Limited Crossings of Permits Checked using the AASHTO LRFD Distribution Factors

SPECIAL PERMIT AASHTO LRFD 1 LANE	γ_L=1.10			γ_L=1.15			γ_L=1.20			γ_L=1.25			γ_L=1.30			γ_L=1.35			γ_L==1.40		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave	Min	Max	Ave
T-beam bending	1.18	2.74	2.28	1.33	2.86	2.39	1.47	2.99	2.49	1.60	3.13	2.60	1.73	3.26	2.70	1.86	3.39	2.80	1.98	3.53	2.89
T-beam shear	1.36	3.11	2.55	1.49	3.26	2.66	1.61	3.41	2.78	1.73	3.56	2.89	1.84	3.71	3.00	1.96	3.85	3.11	2.07	4.00	3.22
Ps/C bending	0.56	2.67	1.98	0.73	2.76	2.11	0.91	2.88	2.25	1.07	3.05	2.38	1.23	3.23	2.50	1.39	3.41	2.63	1.54	3.58	2.75
Ps/C shear	0.82	2.64	2.06	0.95	2.76	2.18	1.08	2.93	2.30	1.21	3.10	2.41	1.33	3.26	2.53	1.45	3.42	2.64	1.57	3.58	2.75
Noncomp. Bending	0.83	2.69	2.02	0.99	2.79	2.15	1.14	2.88	2.28	1.29	2.98	2.40	1.44	3.14	2.52	1.58	3.31	2.64	1.72	3.47	2.75
Noncomp. Shear	1.11	3.43	2.56	1.26	3.59	2.71	1.41	3.75	2.87	1.55	3.90	3.01	1.68	4.05	3.16	1.81	4.20	3.30	1.94	4.38	3.43
Comp. bending	0.82	2.68	2.01	0.99	2.78	2.14	1.14	2.88	2.27	1.29	2.98	2.39	1.44	3.14	2.52	1.58	3.31	2.64	1.72	3.47	2.75
Comp. shear	1.11	3.43	2.56	1.26	3.59	2.71	1.40	3.75	2.86	1.54	3.91	3.01	1.68	4.06	3.16	1.81	4.21	3.30	1.94	4.38	3.43
ALL BRIDGE TYPES	0.56	3.43	2.25	0.73	3.59	2.38	0.91	3.75	2.51	1.07	3.91	2.64	1.23	4.06	2.76	1.39	4.21	2.88	1.54	4.38	3.00

Table 36. Unconditional Reliability Indexes for Limited Crossings of Permits Checked using the AASHTO LRFD Distribution Factors

SPECIAL PERMIT AASHTO LRFD 1 LANE	γ_L=1.10			γ_L=1.15			γ_L=1.20		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.
T-beam bending	2.82	3.84	3.51	2.90	3.93	3.58	2.98	4.03	3.66
T-beam shear	2.92	4.12	3.70	3.00	4.24	3.78	3.07	4.36	3.87
Ps/C bending	2.53	3.78	3.30	2.60	3.86	3.39	2.68	3.94	3.48
Ps/C shear	2.64	3.76	3.35	2.71	3.86	3.44	2.77	3.98	3.52
Noncomp. Bending	2.64	3.80	3.33	2.72	3.87	3.42	2.80	3.95	3.50
Noncomp. Shear	2.79	4.38	3.71	2.87	4.51	3.82	2.95	4.63	3.94
Comp. bending	2.64	3.80	3.33	2.72	3.87	3.41	2.80	3.95	3.50
Comp. shear	2.79	4.38	3.70	2.87	4.51	3.82	2.95	4.64	3.93
ALL BRIDGE TYPES	2.53	4.38	3.49	2.60	4.51	3.58	2.68	4.64	3.68

SPECIAL PERMIT AASHTO LRFD 1 LANE	γ_L=1.10			γ_L=1.15			γ_L=1.20		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.
T-beam bending	2.97	3.95	3.63	3.05	4.04	3.71	3.13	4.14	3.78
T-beam shear	3.07	4.23	3.81	3.14	4.34	3.90	3.21	4.46	3.98
Ps/C bending	2.69	3.90	3.43	2.76	3.97	3.52	2.84	4.06	3.61
Ps/C shear	2.80	3.88	3.48	2.86	3.97	3.56	2.92	4.09	3.64
Noncomp. Bending	2.80	3.92	3.46	2.87	3.99	3.54	2.95	4.06	3.63
Noncomp. Shear	2.94	4.48	3.82	3.01	4.60	3.94	3.09	4.73	4.05
Comp. bending	2.80	3.91	3.45	2.87	3.98	3.54	2.95	4.06	3.62
Comp. shear	2.94	4.48	3.82	3.01	4.61	3.93	3.09	4.73	4.04
ALL BRIDGE TYPES	2.69	4.48	3.62	2.76	4.61	3.70	2.84	4.73	3.79

SPECIAL PERMIT AASHTO LRFD 1 LANE	γ_L=1.10			γ_L=1.15			γ_L=1.20		
	Min	Max	Ave.	Min	Max	Ave.	Min	Max	Ave.
T-beam bending	3.24	4.17	3.86	3.31	4.25	3.93	3.39	4.35	4.00
T-beam shear	3.33	4.43	4.04	3.40	4.54	4.12	3.46	4.65	4.20
Ps/C bending	2.98	4.12	3.67	3.05	4.18	3.76	3.12	4.26	3.84
Ps/C shear	3.08	4.10	3.72	3.14	4.18	3.80	3.20	4.30	3.87
Noncomp. Bending	3.08	4.13	3.70	3.15	4.20	3.78	3.22	4.27	3.86
Noncomp. Shear	3.21	4.67	4.04	3.28	4.79	4.15	3.35	4.91	4.26
Comp. bending	3.08	4.13	3.69	3.15	4.20	3.77	3.22	4.27	3.85
Comp. shear	3.21	4.67	4.04	3.28	4.79	4.15	3.35	4.92	4.25
ALL BRIDGE TYPES	2.98	4.67	3.85	3.05	4.79	3.93	3.12	4.92	4.02

Table 35 shows that a live load factor γ_L=1.40 leads to an average conditional reliability index is $\beta_{average}$=3.00 with a minimum value of 1.54 and a maximum value of 4.38. Due to the low probability of having a random truck cross a bridge alongside the special permit, the actual unconditional reliability index will be significantly higher.

Table 36 shows that even a live load factor γ_L=1.10 would still lead to an average reliability index of 3.49 with a minimum value of 2.55 and a maximum value of 4.38 for the sites with ADTT=5000. Higher reliability levels are observed for sites with lower ADTT.

Reliability for Permit Checks with Rigorous Analysis

If the Permit check is to be executed using a rigorous analysis, it is important for the Rating engineer to know what live load factor, γ_{LP}, to apply on the Permit truck and what truck load, L_n, and what live load factor γ_{LL}, to apply in the adjacent lane. In this case, the permit will be allowed to cross the bridge if the nominal resistance R_n satisfies the following equation:

$$\phi R_n = \gamma_{DC1} DC_1 + \gamma_{DC2} DC_2 + \gamma_W DW + \left(\gamma_{LP} P \times DF_P + \gamma_{LL} L_n \times DF_L\right) IM \qquad (68)$$

Where in addition to the variables identified along with Eq. 57, γ_{LP} is defined as the live load factor applied on the Permit, P is the permit load effect, DF_P represents the distribution factor for the Permit (or the percentage of the Permit truck effect that will be acting on the member being analyzed), γ_{LL} is the live load factor applied on the Legal truck placed in the adjacent lane, L_n is the load effect of the Legal truck, DF_L represents the distribution factor for the Legal truck (or the percentage of the Permit truck effect that will be acting on the member being analyzed).

To remain consistent with the current AASHTO LRFR and LFR, it is proposed to use the most critical of the three AASHTO Legal trucks to obtain L_n.

The conditional reliability analysis is executed using the limit state function of Eq. (56) and the unconditional reliability is calculated from Eq. (65) through (67).

Simplified Example

A simplified reliability calculation analysis is first provided for one example to study the interaction between the different parameters. The case considered in this example is for a composite steel bridge member in shear where the span length is 60-ft and the beam spacing is 8-ft.

We assume a live load factor γ_P=1.00 and a live load factor on the legal truck γ_L=1.20. The dead load shears are given as D_{C1}=4.7 kip, D_{C2}=27.6 kip and D_W=6.5 kip. The Special Permit truck that is being checked has a shear effect P=111 kip. The controlling AASHTO Legal truck is the 3-S2 with a shear load equal to L_n=54.7 kips. The AASHTO LRFD load distribution factor for a single lane is calculated to be DF_1=0.68. After we remove the multiple presence factor MP=1.2 and the bias implicit in the LRFD distribution factors, the estimated mean value of the distribution factor is 0.408 (=0.68/1.2/1.39). The two-lane LRFD shear distribution factor is 0.84. After removing the bias implied in the LRFD, the estimated distribution factor for the truck alongside the permit is given as 0.280 (=0.84/1.22-0.408)

The Permit truck would be allowed on the bridge if the nominal resistance capacity of the bridge member is:

$$R_n = 1.25(4.7 + 27.6) + 1.50(6.5) + 1.0(111 \times 1.33 \times 0.408) + 1.20(54.7 \times 1.33 \times 0.28) = 134.9\ kip$$

This indicates that the mean member resistance in shear is $\overline{R} = 1.14 \times 134.9 = 153.8\ kip$ the COV for the resistance is V_R=10.5%.

The mean dead load is obtained as: $\overline{D} = 1.03 D_{C1} + 1.05 D_{C2} + 1.00 D_W = 40.3\ kip$, the corresponding standard deviation is σ_D=3.35 kip.

After removing the bias and the multiple presence factor, the mean value of the distribution factor applied on the Permit truck is thus assumed to be $\overline{DF}_P$ =0.408 while the distribution factor applied on the random truck that may cross the bridge alongside the Permit is $\overline{DF}_R = \overline{DF}_2 - \overline{DF}_P$ =0.28.

The mean value for the static effect of the Permit $P \times DF_P$ is obtained as 111×0.408=45.29 kip. Because we are performing a rigorous analysis which presumably provides closer results to the actual distribution factor, a COV=8% is applied and the standard deviation for the static Permit load effect is $45.29 \times 8\%$=3.62 kip.

The mean value of static effect of the Random truck $\lambda_{site\ to\ site} \times \lambda_{sample\ size} \times L_{\max N_R} \times HL93 \times DF_R$ is given as $1.0 \times 1.0 \times 0.75 \times 79.9 \times 0.28 = 16.78\ kip$.

The COV is V*=$\sqrt{(19\%)^2 + (2\%)^2 + (18\%)^2 + (8\%)^2} = 27\%$ leading to a standard deviation = $16.78 \times 27\%$=4.60 kip.

The mean and standard deviation of the total static live load effect $\left(P \times DF_P + \lambda_{site\ to\ site} \times \lambda_{sample\ size} \times L_{\max N_R} \times HL93 \times DF_R\right)$ are obtained as 62.07 kip and $\sigma_L^* = \sqrt{3.6^2 + 4.6^2} = 5.85\ kip$. The COV becomes $V_L^* = 5.85/62.07 = 9.4\%$.

Given a mean impact for side-by-side trucks $\overline{IM}$ =1.09 and V_{IM}=5.5%, the mean and COV of the total live load effect LL are given as:

$\overline{LL} = 62.07 \times 1.09 = 67.7\ kip$, $V_{LL} = \sqrt{(9.4\%)^2 + (5.5\%)^2} = 11\%$ and the standard deviation is $\sigma_{LL} = 7.4\ kip$

The mean of the total load including live and dead loads is $\overline{S} = \overline{DL} + \overline{LL} = 40.3 + 67.7 = 108\ kip$.

The standard deviation is $\sigma_S = \sqrt{3.35^2 + 7.4^2} = 8.1\ kip$ with a COV V_S=8.1/108=7.5%.

An approximation for the conditional reliability index for this case is given as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{153.8}{108}\right)}{\sqrt{(10.5\%)^2 + (7.5\%)^2}} = 2.74$$

The above calculations assumed that a random truck will always cross the bridge alongside the Permit vehicle. Using the FORM algorithm with the appropriate probability distribution function for each of the random variables, the conditional reliability index is calculated to be β=2.73 which is very close to the 2.74 value obtained from the simplified calculations.

Reliability Analysis

The FORM algorithm is subsequently used to find the reliability for a range of bridge configurations giving the results summarized in Table 37 which gives the conditional reliability index values in the cases where the Permit check is executed using a rigorous analysis where the Permit truck is associated with a live load factor γ_P=1.0 while the truck alongside is the governing AASHTO Legal truck associated with a live load factor γ_L=1.10. The unconditional reliability levels are shown in Table 38.

Table 37 shows that the average conditional reliability index is $\beta_{average}$=2.51 with a minimum value of 1.60 and a maximum value of 2.86. The average unconditional reliability index is 3.06 with a minimum value of 3.68 and a maximum value of 3.93 for the sites with ADTT=5000 as shown in Table 38. Even higher reliability levels are observed for the cases with lower ADTT. The higher reliability levels observed for the unconditional case are clearly due to the low probability of having two Permits side-by-side when 100 Permits are independently crossing the same bridge each day.

Table 37. Conditional Reliability Indexes for γ_P=1.0 and γ_L=1.1 when using a refined analysis.

PERMIT ALONGSIDE RANDOM PERMIT (RIG)	γ_L=1.10		
	Min	Max	Ave.
T-beam bending	2.221	2.862	2.712
T-beam shear	2.376	2.834	2.741
Ps/C bending	1.602	2.703	2.406
Ps/C shear	1.777	2.234	2.151
Noncomp. Bending	1.815	2.736	2.451
Noncomp. Shear	2.041	2.802	2.601
Comp. bending	1.813	2.732	2.447
Comp. shear	2.039	2.800	2.598
ALL BRIDGE TYPES	1.602	2.862	2.513

Table 38. Unconditional Reliability Indexes for γ_P=1.0 and γ_L=1.1 when using a refined analysis.

PERMIT ALONGSIDE RANDOM PERMIT (RIG) ADTT=5000	γ_L=1.10		
	Min	Max	Ave.
T-beam bending	3.47	3.93	3.82
T-beam shear	3.58	3.91	3.84
Ps/C bending	3.06	3.81	3.60
Ps/C shear	3.17	3.48	3.42
Noncomp. Bending	3.20	3.84	3.63
Noncomp. Shear	3.34	3.89	3.74
Comp. bending	3.20	3.83	3.63
Comp. shear	3.34	3.89	3.73
ALL BRIDGE TYPES	3.06	3.93	3.68

PERMIT ALONGSIDE RANDOM PERMIT (RIG) ADTT=1000	γ_L=1.10		
	Min	Max	Ave.
T-beam bending	3.59	4.04	3.93
T-beam shear	3.70	4.02	3.95
Ps/C bending	3.20	3.93	3.72
Ps/C shear	3.31	3.60	3.54
Noncomp. Bending	3.33	3.95	3.75
Noncomp. Shear	3.47	4.00	3.85
Comp. bending	3.33	3.95	3.75
Comp. shear	3.47	4.00	3.85
ALL BRIDGE TYPES	3.20	4.04	3.79

PERMIT ALONGSIDE RANDOM PERMIT (RIG) ADTT=100	γ_L=1.10		
	Min	Max	Ave.
T-beam bending	3.82	4.25	4.15
T-beam shear	3.92	4.23	4.17
Ps/C bending	3.46	4.14	3.94
Ps/C shear	3.56	3.83	3.78
Noncomp. Bending	3.58	4.17	3.97
Noncomp. Shear	3.71	4.21	4.07
Comp. bending	3.58	4.16	3.97
Comp. shear	3.71	4.21	4.07
ALL BRIDGE TYPES	3.46	4.25	4.02

CHAPTER 4

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The Object of this NCHRP 20-07 Task 285 project is to use recent national WIM data to recalibrate the LRFR live load factors for Permit Load Ratings given in the AASHTO MBE and to adjust these live load factors based on the method of analysis used during the load rating process. This study has taken into consideration current WIM data to verify that the AASHTO LRFR produce acceptable and uniform levels of reliability for typical U.S. bridges under current loading conditions.

To achieve this goal, it was critical to use the most representative statistical information on truck weights, truck configurations, and multiple presence data. For the purposes of this study, data collected by Sivakumar et al (2008) in each direction of six WIM U.S. sites as part of NCHRP project 12-76 is analyzed to obtain projections for the maximum bridge load effects. The WIM six sites are located in: New York, Mississippi, Indiana, Florida, California, and Texas. The WIM data from each direction of these sites contained the number of axles for each truck, the axle spacings and the axle weights. Multiple presence probabilities were assembled form a representative site in New York. The protocols established in NCHRP Project 12-76 were adapted to the application here of bridge rating recalibration.

The recalibration of the live load factors for permits considered the following four cases:

I. Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.
II. Unlimited crossings of multiple trip permits where two Permit trucks could cross a bridge simultaneously side-by-side.
III. Unlimited crossings where a Permit truck mixes with other random vehicles.
IV. Single Permit trips where the Permit truck could mix with other random vehicles.

This report presents the results of the reliability calibration of the Permit live load factors for use in proposed revised version of the AASHTO LRFR. The target reliability index set for the calibration is β_{target}=2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50.

The calculations performed in this report demonstrate that using live load factors γ_L=1.10 for escorted Special permit loads will provide average reliability index values greater than the target β_{target}=2.5 when the single lane AASHTO LRFD load distribution factors are used to check whether the Permit truck can be allowed to cross a bridge. When performing a refined analysis of the bridge, it is recommended to use the same γ_L=1.1 for escorted Special Permits. Special permits travelling over bridges at crawl speed should still be checked with a dynamic allowance factor of 1.05 to satisfy the minimum value of β_{min}=1.50.

For the case when a rigorous analysis is performed for special permits that may mix with traffic, the target reliability is also exceeded when a live load factor γ_P=1.0 is applied on the permit truck while a live load factor γ_L=1.10 is applied on the governing AASHTO legal truck placed in the adjacent lane.

For the cases of routine permits, where data shows that permit loads may exceed the permit weight limits, having live load factors varying from γ_L=1.40 for sites with ADTT=5000, γ_L=1.35 for sites with ADTT=1000, and γ_L=1.30 for sites with ADTT=100 will increase the reliability index values so that the minimum value remains above β=1.50. These checks should be performed with the two-lane AASHTO LRFD load distribution factors.

The above live load factors for routine permits can be reduced for the cases where the Permit truck's Gross vehicle weight is high to reflect the lower probability of having a random truck of equal or higher weight crossing alongside the Permit truck. Specifically, it is recommended that trucks with Gross Vehicle Weight (GVW) over front axle to rear axle length (AL) ratios (GVW/AL) < 2.0 use the above mentioned factors γ_L=1.40 for sites with ADTT=5000, γ_L=1.35 for sites with ADTT=1000, and γ_L=1.30 for sites with ADTT=100. For trucks with GVW/AL between 2.0 and 3.0 the recommended live load factors are γ_L=1.35 for sites with ADTT=5000, γ_L=1.25 for sites with ADTT=1000, and γ_L=1.20 for sites with ADTT=100. For trucks with GVW/AL above 3.0, the live load factors γ_L=1.30 for sites with ADTT=5000, γ_L=1.20 for sites with ADTT=1000, and γ_L=1.15 for sites with ADTT=100.

The analysis performed in this report covered the bending and shear loading effects of bridges with simple span lengths having parallel T-beams, prestressed I-beams, composite and non-composite steel I-beams.

SUGGESTED RESEARCH

Calibrated live load factors for permits that achieve more optimum reliability levels consistent with current practice and also allow the use of refined methods of analysis have been recommended based on the findings of this research. Suggested research on this topic for the future are as follows:

1. Develop a reliability based methodology to perform LRFR ratings using measured strain data. Perform rating based on bridge response data would provide even more realistic load capacity assessments than using refined analysis.
2. The condition factors given in the MBE are based on expert opinion. Research should be initiated to calibrate these factors in a manner consistent with reliability methods. Accounting for regional differences in material deterioration with age and with differences in construction practices should also be addressed.
3. Additional live load studies using recent WIM data should be conducted on calibrating the LRFR live load model and load factors for continuous spans.
4. Additional live load studies using recent WIM data should be conducted on calibrating the LRFR live load model and load factors for spans greater than 200 ft..

Recommended Revisions to the MBE LRFR Permit Load Rating Provisions

6A.4.5—Permit Load Rating

6A.4.5.1—Background

Bridge Owners usually have established procedures and regulations which allow the passage of vehicles above the legally established weight limitations on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicle and/or its load and, in most jurisdictions, which specifies the allowable route or routes of travel.

Permits are issued by States on a single trip, multiple trip, or annual basis. Routine or annual permits are usually valid for unlimited trips over a period of time, not to exceed one year, for vehicles of a given configuration within specified gross and axle weight limits. Special permits are usually valid for a single trip only, for a limited number of trips, or for a vehicle of specified configuration, axle weights, and gross weight. Special permit vehicles are usually heavier than those vehicles issued annual permits. Depending upon the authorization, these permit vehicles may be allowed to mix with normal traffic or may be required to be escorted in a manner which controls their speed, lane position, the presence of other vehicles on the bridge, or some combination thereof.

C6A.4.5.1

To assure that permit restrictions and conditions are met and to warn the other traffic, special escort vehicles may be needed or required by State law. Traffic safety needs should always be considered.

6A.4.5.2—Purpose

Article 6A.4.5 provides procedures for checking bridges to determine the load effects induced by the overweight permit loads and their capacity to safely carry these overloads. Permit load rating should be used only if the bridge has a rating factor greater than 1.0 when evaluated for AASHTO legal loads.

C6A.4.5.2

Permit vehicles should be rated by using load-rating procedures given in Article 6A.4.5, with load factors selected based on the permit type, loading condition, and site traffic data. The live load to be used in the load-rating equation for permit decisions shall be the actual permit vehicle weight and axle configuration.

The factors recommended for evaluating permit loads are calibrated with the assumptions that the bridge, as a minimum, can safely carry AASHTO legal loads, as indicated by the evaluation procedures given in Article 6A.4.4. This requirement is especially evident when using reduced live load factors for permits based on a small likelihood that there will be multiple presence of more than one heavy vehicle on the span at one time. Such multiple presence situations are considered in the calibration of the checking equations of both the *AASHTO LRFD Bridge Design Specifications* and the evaluation procedures given in this Manual.

6A.4.5.3—Permit Types

6A.4.5.3.1—Routine (Annual) Permits

Routine permits are usually valid for unlimited trips over a period of time, not to exceed one year. The permit vehicles may mix in the traffic stream and move at normal speeds without any movement restrictions. Some

permits may be restricted to specified routes.

6A.4.5.3.2—Special (Limited Crossing) Permits

Special permits are usually valid for a single trip only or for a limited number of trips. These permit vehicles are usually heavier than those vehicles issued routine permits.

Single-trip permits are good for only one trip during a specified period of time (typically 3–5 days). Multiple-trip permits grant permission to transport overweight shipments during a 30–90 day period.

Single-trip permits for excessively heavy loads may have certain conditions and restrictions imposed to reduce the load effect, including, but not limited to:

- Requiring the use of escorts to restrict all other traffic from the bridge being crossed.
- Requiring the permit vehicle to be in a certain position on the bridge (e.g., in the center or to one side) to reduce the loading on critical components.
- Requiring crossing at crawl speed (<10 mph) to reduce dynamic load allowance.

C6A.4.5.3.2

Upper limit of 100 special permit crossings was used for calibration purposes in this Manual. Permits operating at a higher frequency should be evaluated as routine permits.

6A.4.5.4—Live Load and Load Factors

6A.4.5.4.1—Live Load

The live load to be used in the evaluation for permit decisions shall be the actual permit truck or the vehicle producing the highest load effect in a class of permit vehicles operating under a single permit. The loading shall consider the truck weight, its axle configuration and distribution of loads to the axles, designated lane position, and any speed restrictions associated with the issuance of the permit.

For spans up to 200 ft, only the permit vehicle shall be considered present in the lane. For spans between 200 and 300 ft, and when checking negative moments in continuous span bridges, an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane. The lane load may be superimposed on top of the permit vehicle (for ease of analysis) and is applied to those portions of the span(s) where the loading effects add to the permit load effects.

C6A.4.5.4.1

Service limit states that are relevant to permit load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

6A.4.5.4.2—Load Factors

Table 6A.4.5.4.2a-1 specifies live load factors for permit load rating that are calibrated to provide a uniform and acceptable level of reliability. Load factors are defined based on the permit type, loading condition, and site traffic data.

Permit load factors given in Table 6A.4.5.4.2a-1 for the Strength II limit state are intended for spans having a rating factor greater than 1.0 when evaluated for AASHTO legal loads. Permit load factors are not intended for use in load-rating bridges for legal loads.

C6A.4.5.4.2

The target reliability index set for the calibration of permit load factors is β_{target}=2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50 and keeping a minimum live load factor of γ_L=1.1. The minimum reliability index always governed the recalibration, which raised the average beta values to be mostly greater than 2.5.

Reliability index values for permit checking, multiple presence probabilities and estimation of the load

effect of the random trucks that will cross a bridge alongside a permit truck are calculated using the actual live load effects of trucks recorded by WIM from several sites throughout the U.S. Results of this study may be found in the Final Report for NCHRP 20-07 Task 285.

6A.4.5.4.2a—Routine (Annual) Permits

The live load factors given in Table 1 for evaluating routine permits shall be applied to a given permit vehicle or to the maximum load effects of all permit vehicles allowed to operate under a single-routine permit. A multi-lane loaded distribution factor shall be used to account for the likelihood of the permits being present alongside other heavy vehicles while crossing a bridge.

C6A.4.5.4.2a

In order, to better reflect the load effects from the different truck types, the routine permits are categorized based on a combination of their gross vehicle weights (in kip) and their first to rear axle lengths (in ft). The load factors for routine permits can be reduced for the cases where the permit truck's gross vehicle weight and load effect is high to reflect the lower probability of having a random truck of equal or higher weight and load effect crossing alongside the permit truck as given in Table 1. These checks should be performed with the multi-lane AASHTO LRFD load distribution factors.

Deleted: The target reliability level for routine permit crossings is established as the same level as for legal loads given in Article 6A.4.4, namely, consistent with traditional AASHTO Operating ratings. ¶
The live load factors for routine permits given in Table 1 depend on both the *ADTT* of the site and the magnitude of the permit load. In the case of routine permits, the expected number of such permit-crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.

Deleted: Table 6A.4.5.4.2a-1—Permit Load Factors: γ_L¶
Permit Type ...

Table 6A.4.5.4.2a-1—Permit Load Factors: γ_L

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Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight Ratio[b]		
					GVW / AL < 2.0 (kip/ft)	2.0 < GVW/AL < 3.0 (kip/ft)	GVW/AL > 3.0 (kip/ft)
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.40	1.35	1.30
				=1000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
					All Weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.10		
	Single Trip	Mix with traffic (other vehicles may be on the bridge)	One Lane	All ADTTs	1.20		
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.40		

[a] *DF* = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] Permit Weight Ratio = GVW/AL;. GVW = Gross Vehicle Weight; AL = Front axle to rear axle length; Use only axles on the bridge.

The live load distribution analysis for routine permits is done using LRFD two-lane distribution factors which assume the simultaneous side-by-side presence of two equally heavy vehicles in each lane. This condition is too conservative for permit load analysis. The live load factors herein were derived to account for the possibility of simultaneous presence of nonpermit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher *ADTT*s and lower for heavier permits. The live load factors in Table 1 for routine permits must be applied together with the upper limit of permit weights operating under a single permit and the corresponding two-lane distribution factor.

The live load factors in Table 1 should be used for interpolation between various *ADTT*s.

6A.4.5.4.2b—Special (Limited-Crossing) Permits

Special permits shall be evaluated using the live load factors given in Table 6A.4.5.4.2a-1. These factors shall be applied to the load effects induced by a permit load of magnitude and dimensions specified in the permit application. The live load factors given in Article 6A.4.5.4 for special permits shall only be used for spans having a rating factor of 1.0 or higher for AASHTO legal loads or the design load.

A one-lane distribution factor shall be used for special permit review. Such a distribution factor shall be based on tabulated LRFD-distribution factors without including any built-in, multiple presence factor, statistical methods where applicable, or refined analysis.

C6A.4.5.4.2b

For special permits that are valid for a limited number of trips (below 100 crossings), the probability of simultaneous presence of heavy vehicles alongside the permit vehicle is small. The calibration of these live load factors reflects some contribution from vehicles in adjacent lanes.

If the agency expects that the special permit will be used with a frequency greater than 100 crossings, then the permit shall be treated as a routine permit.

The live load distribution shall be based on only a single-lane loaded condition. If tabulated LRFD one-lane distribution factors are used, any built-in multiple presence factor (such as a value of 1.2) should be divided out.

Using a live load factors γ_L=1.10 for escorted Special permit loads will provide average reliability index values greater than the target β_{target}=2.5 when the single lane AASHTO LRFD load distribution factors are used after removing the multiple presence factor MP=1.2.

For Special permits mixed with traffic, a live load factor γ_L=1.40 leads to an average conditional reliability index of $\beta_{average}$=3.00 with a minimum value of 1.50. The conditional reliability analysis assumed that a heavy random truck will always cross alongside the permit truck.

6A.4.5.4.2c—Permit Checks Using Refined Analysis

When routine permit checks are evaluated using a refined analysis, the load factors as given in Table 6A.4.5.4.2a-1 shall be increased (by adding) 0.10

C6A.4.5.4.2c

If the permit check with no traffic restriction is to be executed using a rigorous analysis, it is important for the rating engineer to know what live load factor to apply on

Deleted: For situations where the routine permit is below 100 kips, the live load factors are the same as those given for evaluating legal loads. This requirement reflects the fact that in a traffic stream, the presence of random, heavy, overloaded vehicles may control the extreme loading case when compared to permit weights, which are close to the limit of 80 kips. When the routine permit weight is above 100 kips, then the live load factors are reduced as shown in Table 1. This reduction reflects the lower probability of two simultaneously heavy vehicles equal to the permit weight crossing the span at the same instant (LRFD two-lane distribution factor assumes that an identical vehicle is simultaneously present in each lane). The calibration of these live load factors for routine permits uses the same traffic statistics used in calibrating the *AASHTO LRFD Bridge Design Specifications* as well as the evaluation factors in Article 6A.4.2 of this Manual, but the traffic stream is supplemented by the addition of the permit vehicles being checked.¶

Deleted: and weight limits

Deleted: For single and multiple-trip special permits that are allowed to mix with traffic (no restrictions on other traffic), the live load factors were explicitly derived to provide a higher level of reliability consistent with AASHTO inventory ratings and LRFD-design level reliability. The higher target reliability is justified as a very heavy special permit or superload may represent the largest loading effect that a bridge has yet experienced in its lifetime. The increased risk of structural damage and associated benefit/cost considerations leads to higher safety requirements for very heavy special permit vehicles than for other classes of trucks.¶
The live load factors for single-trip escorted permits that are required to cross bridges with no other vehicles present have been calibrated to reliability levels consistent with traditional AASHTO operating ratings. A target reliability at the operating level is allowed because of the reduced consequences associated with allowing only the escorted permit vehicle alone to cross the bridge. If an agency elects to check escorted permits at the higher Design- or Inventory-level reliability, then the 1.15 value for the permit load factor for the escorted case shown in Table 6A.4.5.4.2a-1 should be increased to 1.35. Further discussion of these issues and more refined live load factors suitable for specific permitting situations not covered by Table 6A.4.5.4.2a-1 may be found in NCHRP Report 454, *Calibration of Load Factors for LRFR Bridge Evaluation*.

and applied on the two permit trucks placed in adjacent lanes .

When escorted Special Permits with no other vehicles on the bridge are evaluated using a refined analysis, it is recommended to use . γ_{L} =1.1 on the escorted vehicle

When special permits mixed with traffic are evaluated using a refined analysis, a live load factor γ_{L} =1.0 is applied on the permit truck while a γ_{L} =1.10 is applied on the governing AASHTO legal truck placed in the adjacent lane.

the permit truck and what truck load and what live load factor to apply to a random truck in the adjacent lane.

In the case of routine permits, the expected number of such permit-crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.

Escorted special permits travelling over bridges at crawl speed should still be checked with a dynamic allowance factor of 1.05 to satisfy the minimum value of β_{min}=1.50.

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The calibration of load factors for refined analysis for multi-girder bridges accounts for the conservatism and the variability of the AASHTO LRFD load distribution factor compared to those obtained from refined structural analyses.

6A.4.5.5—Dynamic Load Allowance: *IM*

The dynamic load allowance to be applied for permit load rating shall be as specified in Article 6A.4.4.3 for legal loads, except that for slow moving (≤10 mph) permit vehicles the dynamic load allowance may be reduced to 1.05.

6A.4.5.6—Exterior Beams

Permit load factors given in Table 6A.4.5.4.2a-1 are applicable to both interior and exterior beam ratings. Distribution of live load to exterior beams as defined in LRFD Design Article 4.6.2.2.2d shall apply with the following modifications:

- For special permits, use a one-lane loaded condition only. Where a one-lane loaded condition is assumed, the LRFD multiple presence factor need not be applied (the built-in multiple presence factor in the LRFD one-lane distribution factor should be divided out).
- For routine permits, a multi-lane loaded condition shall be assumed. Permit trucks of equal weights shall be assumed to be present in each lane in determining the governing distribution factor.

C6A.4.5.6

In LRFD, live load distribution to the exterior beams for bridges with diaphragms or cross-frames must be checked by an additional investigation that assumes rigid body behavior of the section, per LRFD Design Article 4.6.2.2.2d.

6A.4.5.7—Continuous Spans

Closely spaced heavy axles can cause uplift in end spans of continuous bridges. During permit reviews, uplift in continuous span bridges and its effect upon bearings should be considered.

C6A.4.5.7

When the upward *LL* reaction reduces the total reaction to less than ten percent of normal downward *DL* reaction, uplift may be considered to occur. Unless the uplift is counteracted (by weights or tie-downs), the vehicle should not be permitted on the bridge.

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APPENDIX A

PLOTS OF RELIABILITY INDICES

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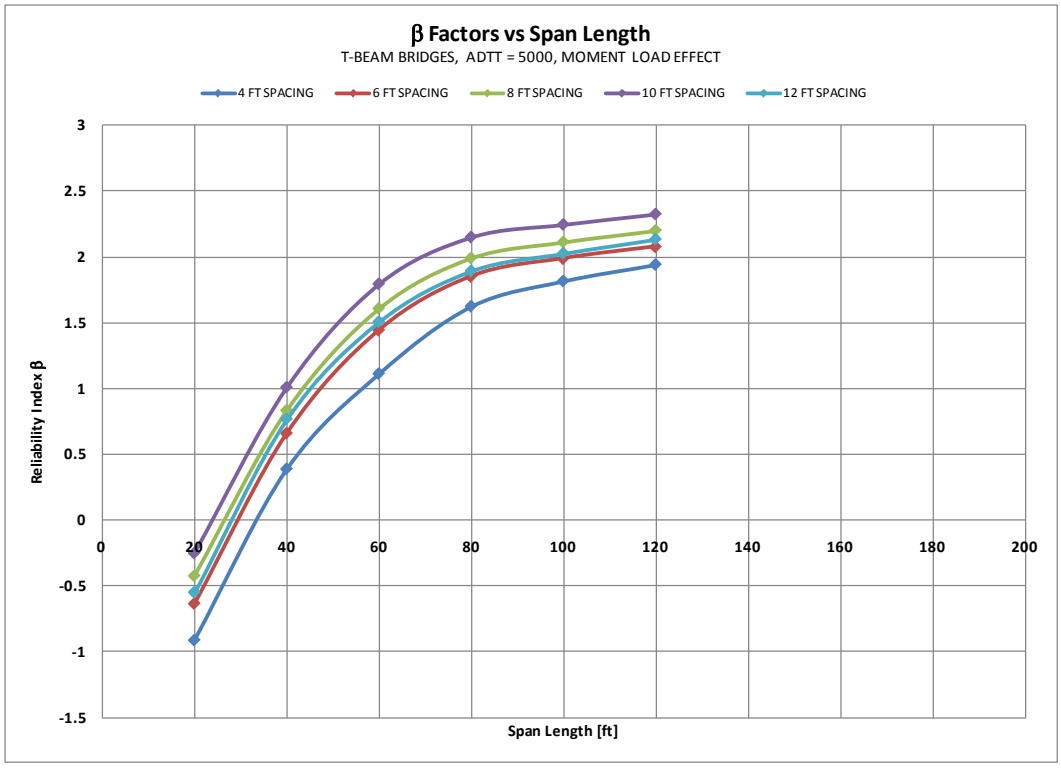
REINFORCED CONCRETE T- BEAM

LEGAL LOAD PLOTS (ALL WIM SITES)


β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
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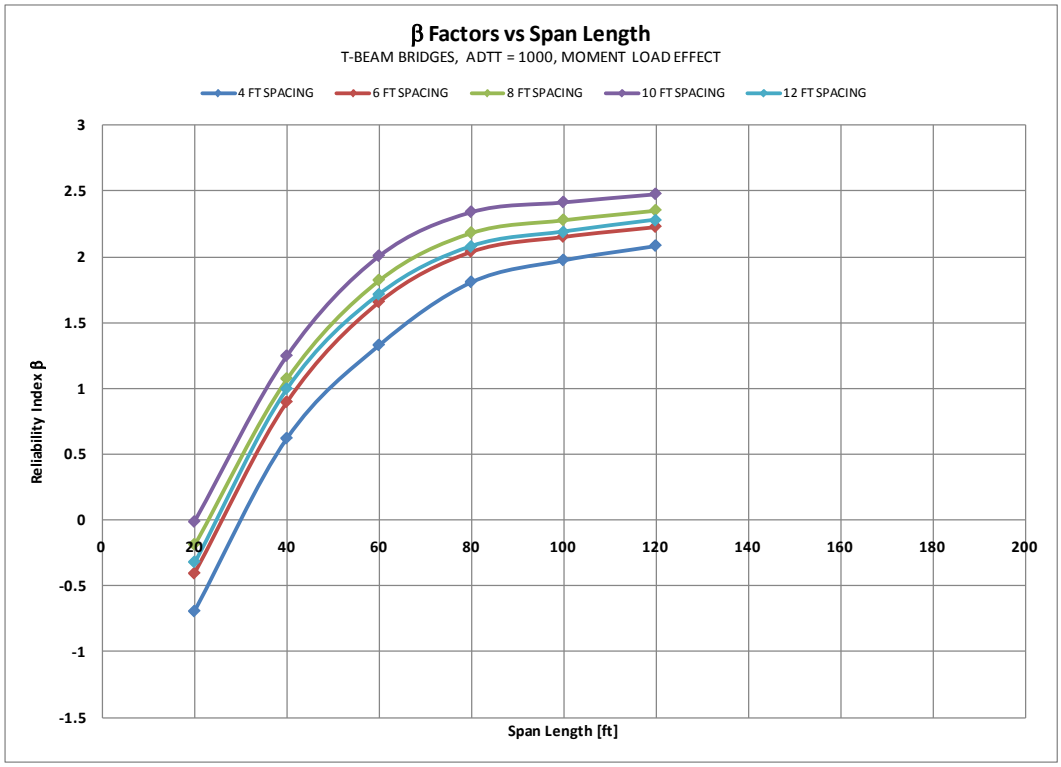
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12 FT SPACING
Reliability Index β
Span Length [ft]
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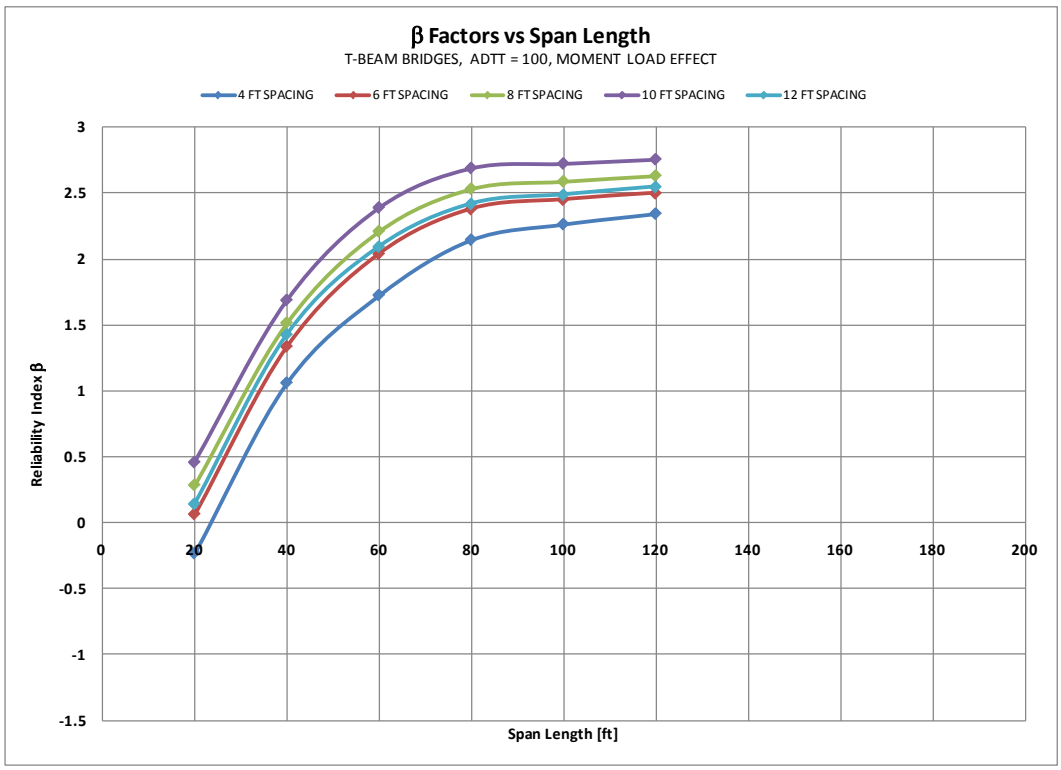
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Reliability Index β
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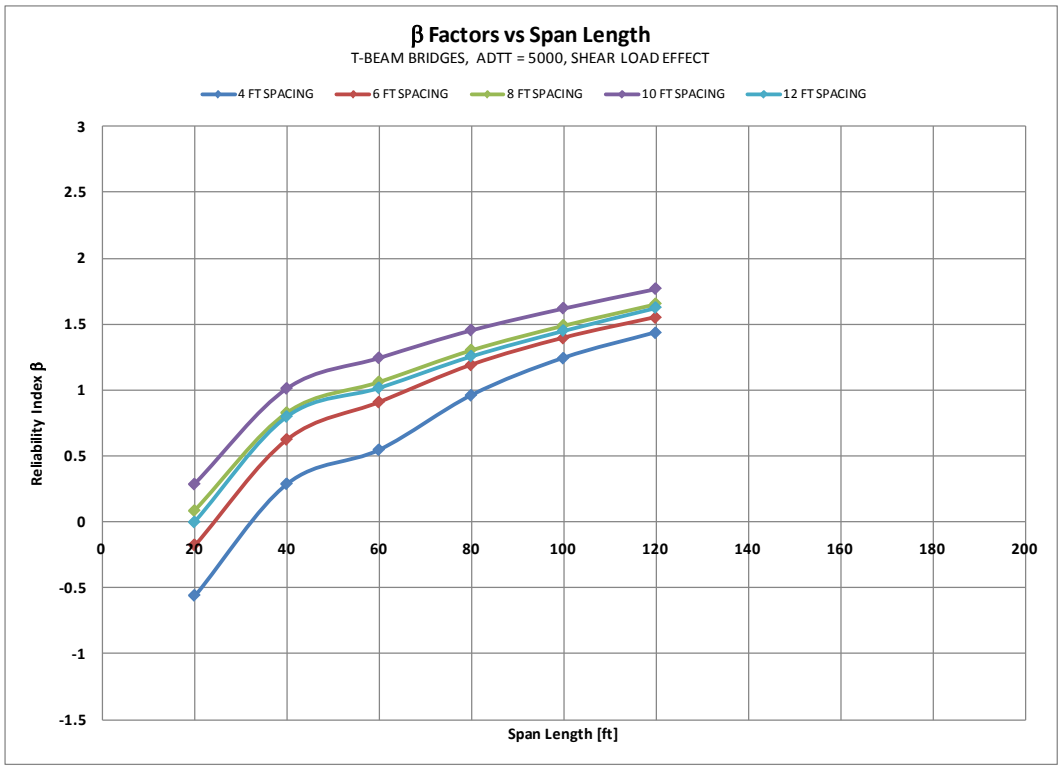
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Reliability Index β
Span Length [ft]
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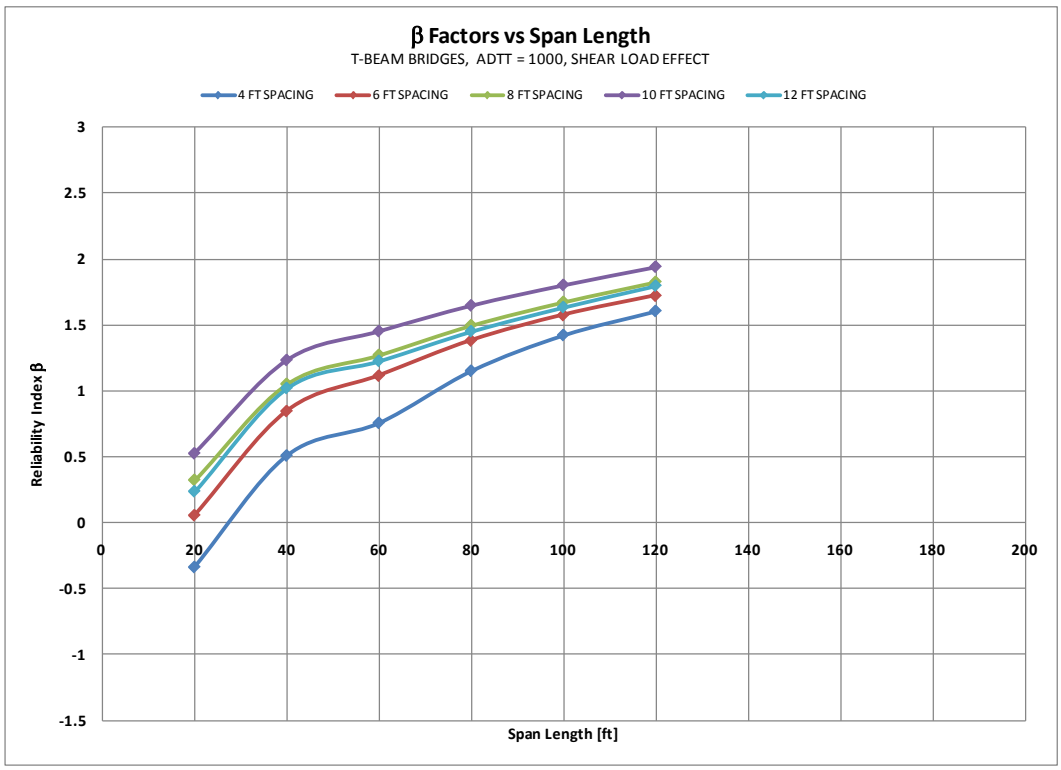
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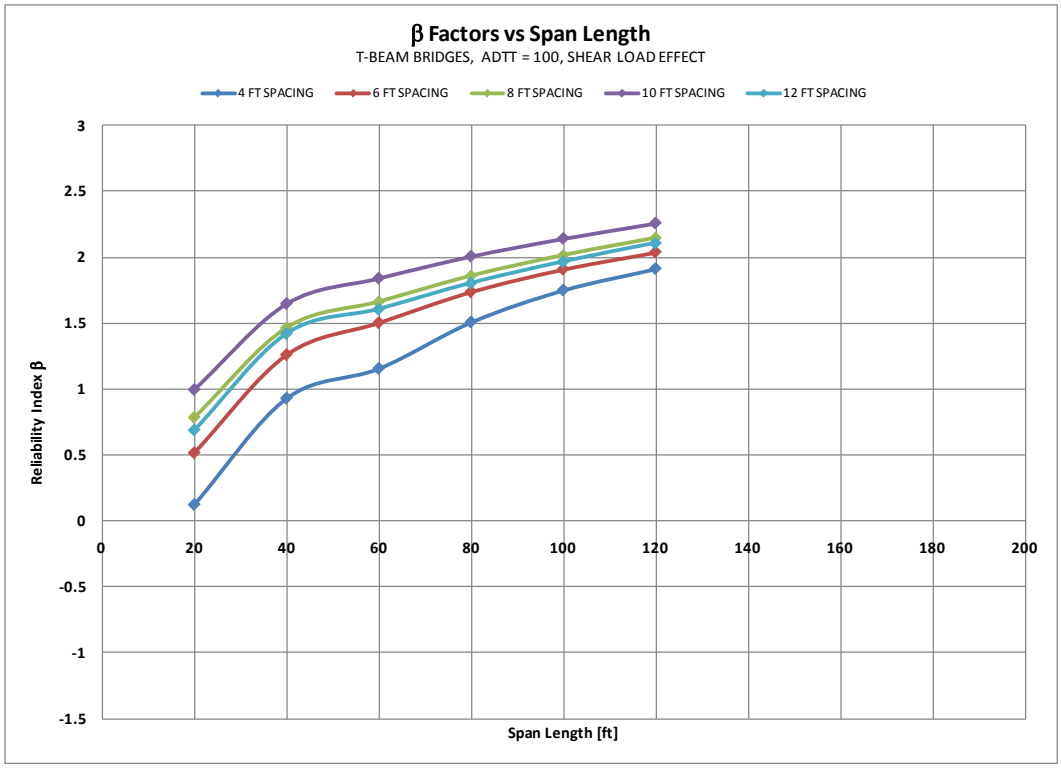
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Reliability Index β
Span Length [ft]
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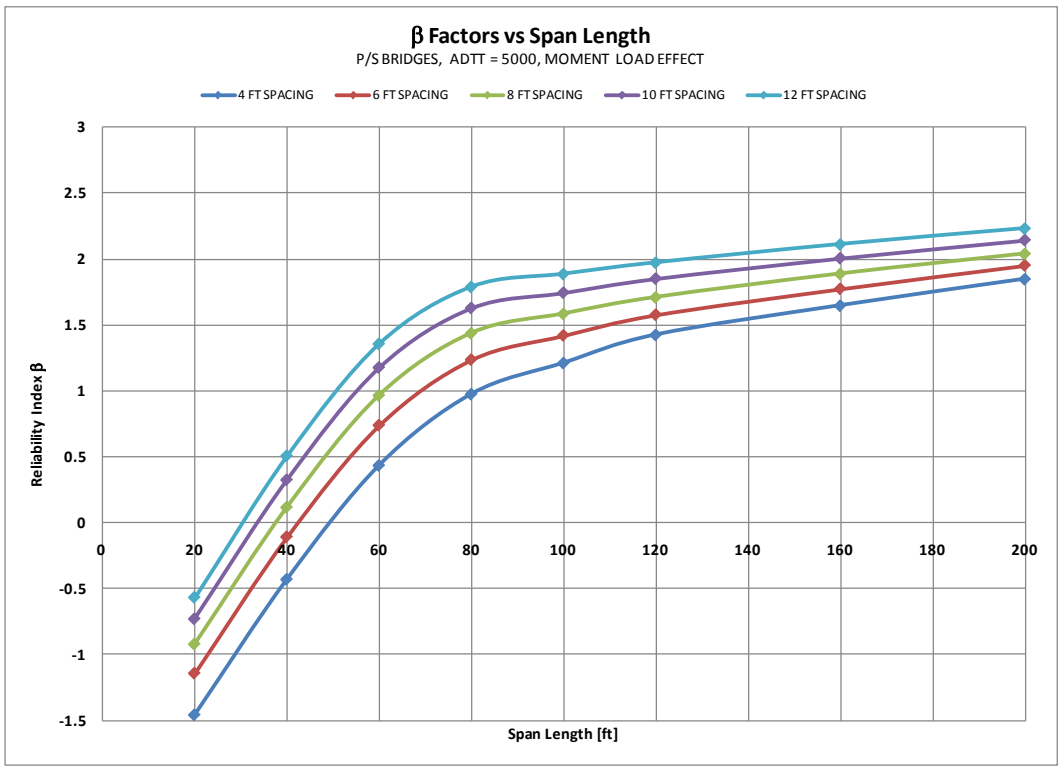
PRESTRESSED CONCRETE BEAM

LEGAL LOAD PLOTS (ALL WIM SITES)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Reliability Index β
Span Length [ft]
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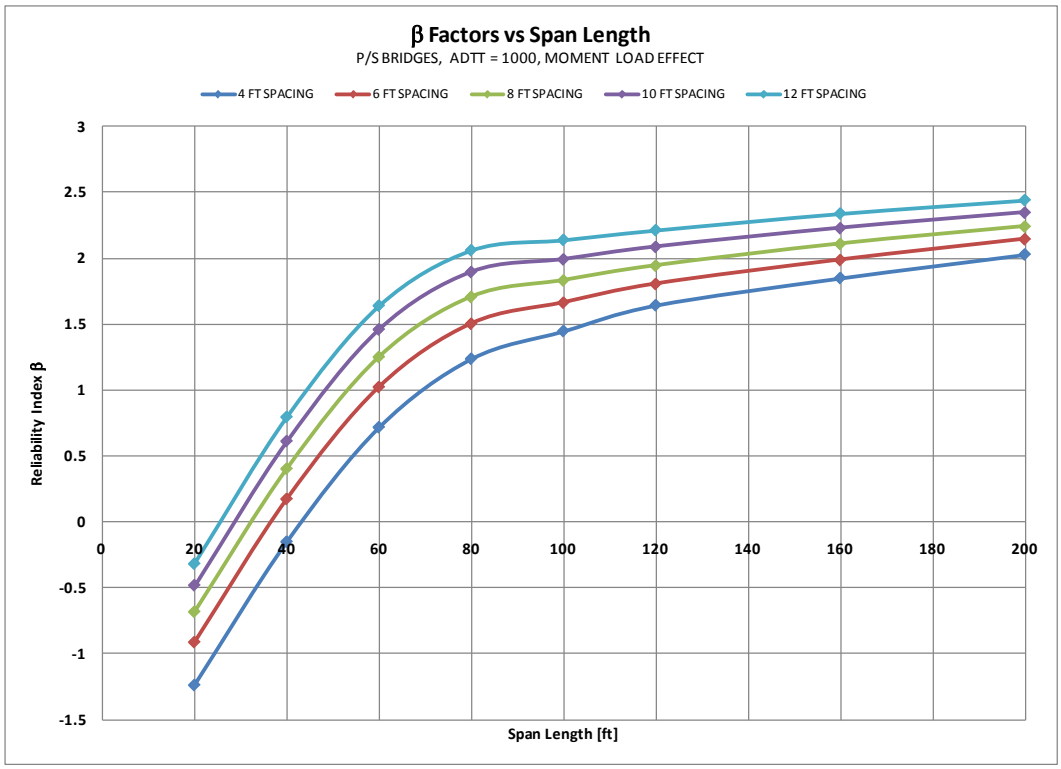
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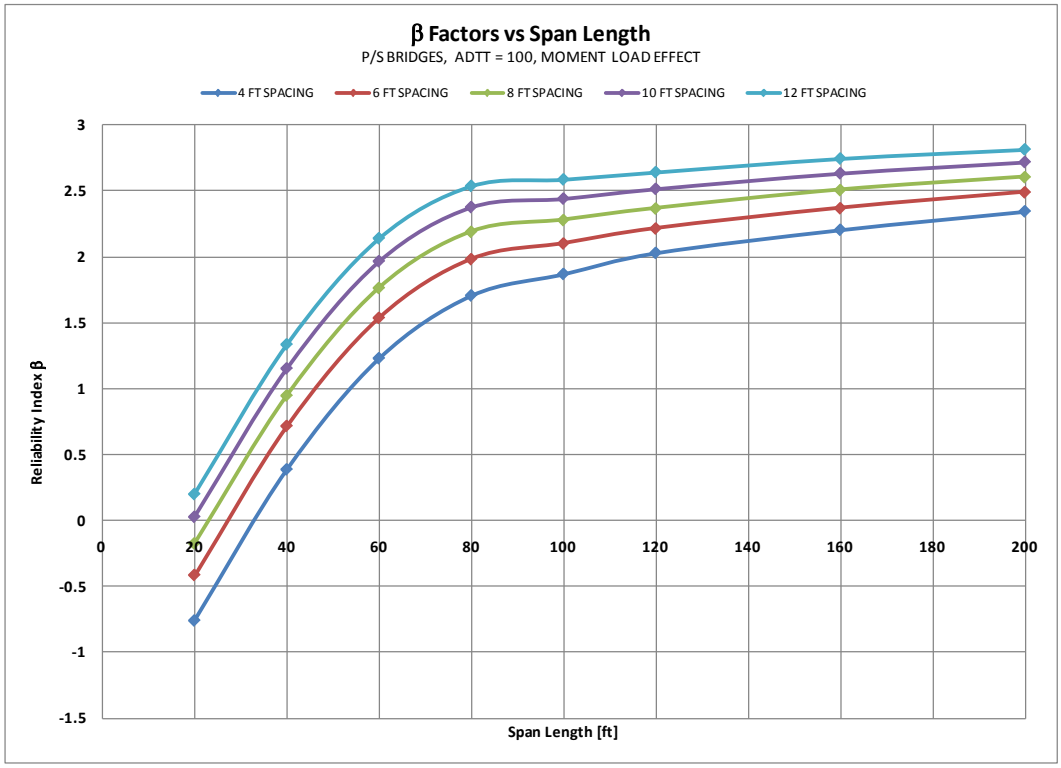
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Reliability Index β
Span Length [ft]
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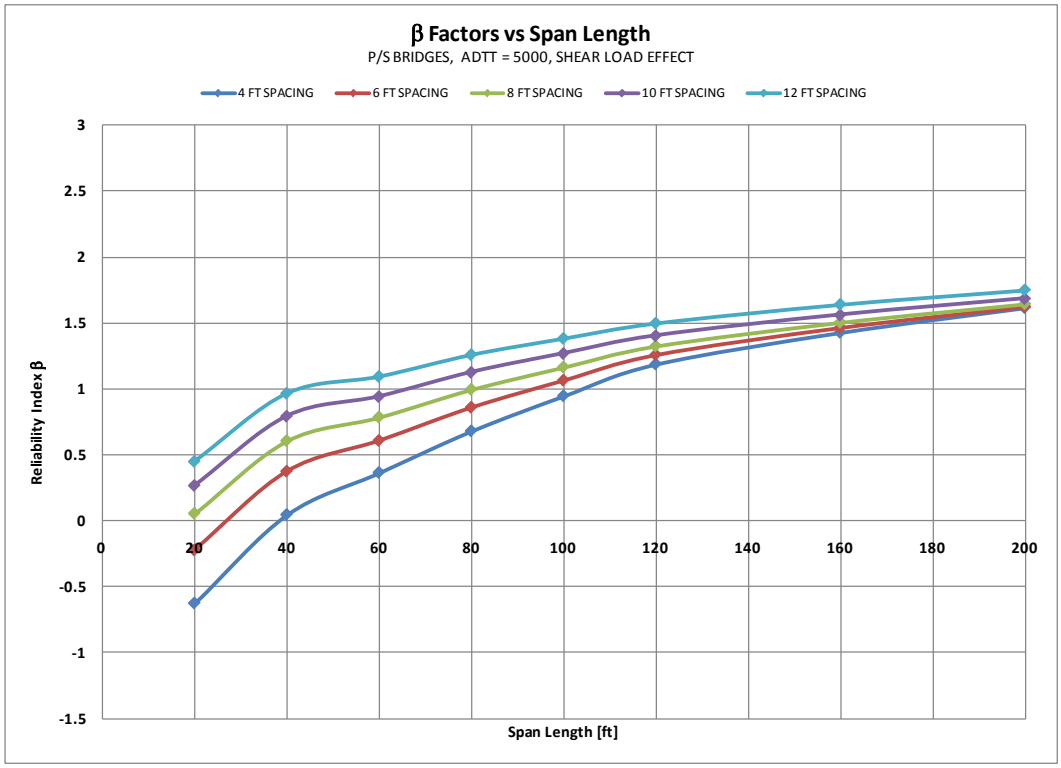
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Reliability Index β
Span Length [ft]
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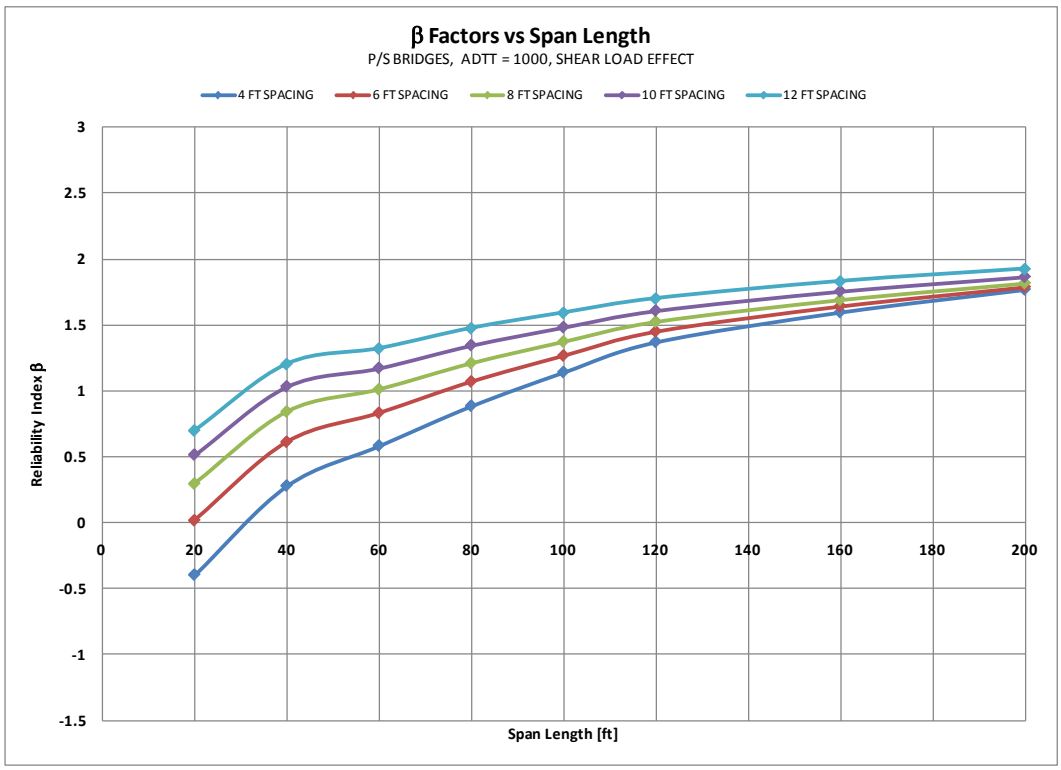
β Factors vs Span Length
P/S BRIDGES, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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-0.5
-1
-1.5
0
20
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60
80
100
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140
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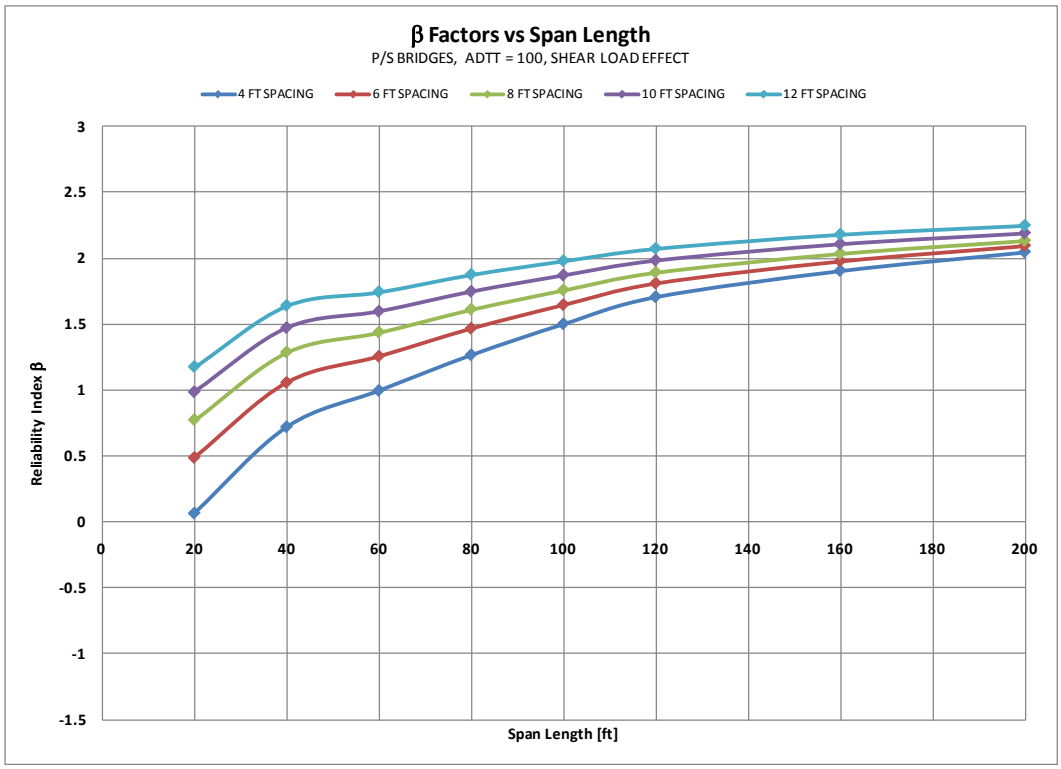
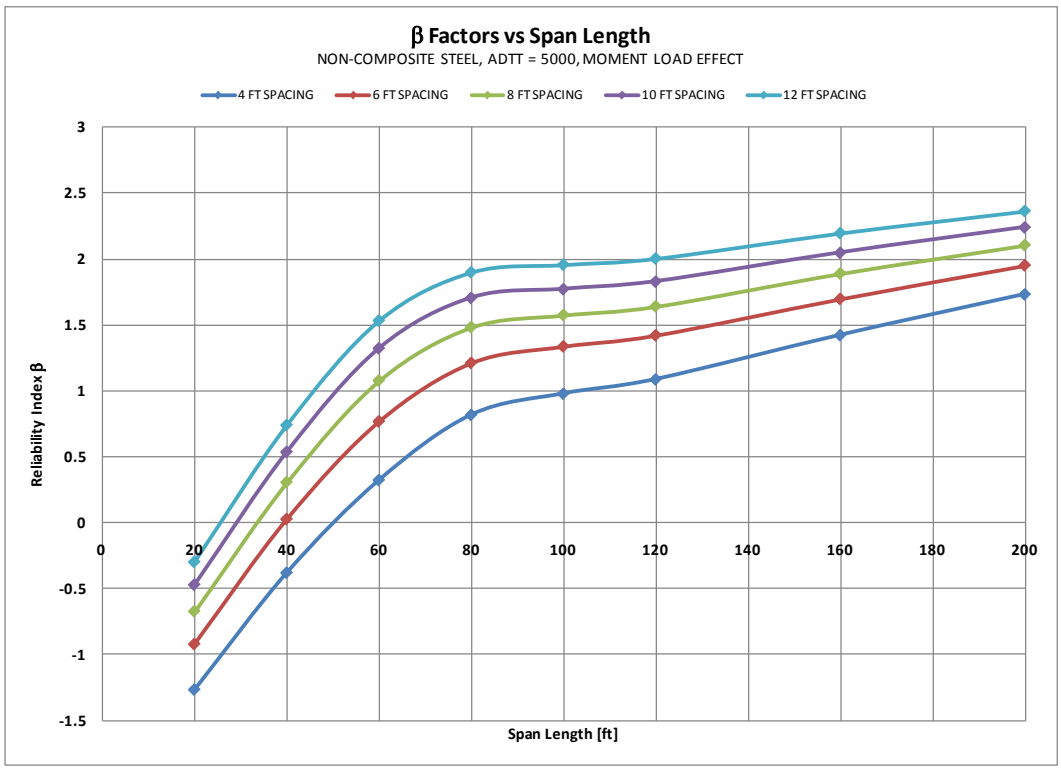

β Factors vs Span Length
P/S BRIDGES, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

STEEL NON-COMPOSITE BEAM

LEGAL LOAD PLOTS (ALL WIM SITES)


β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
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60
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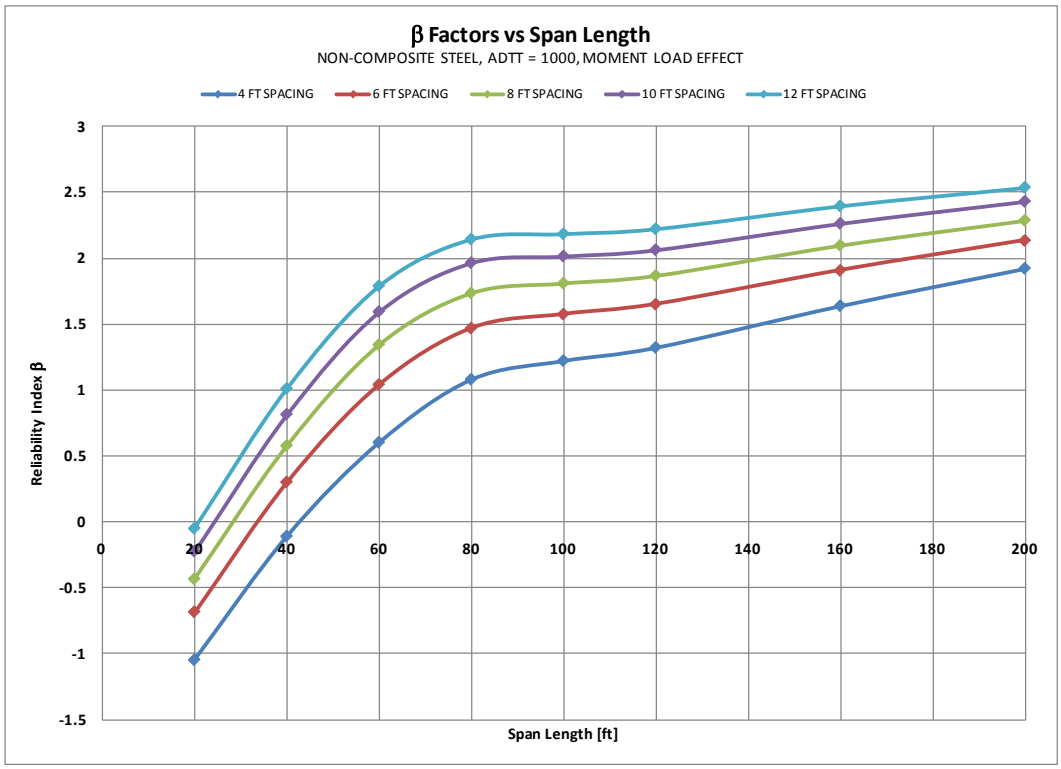
β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
-1.5
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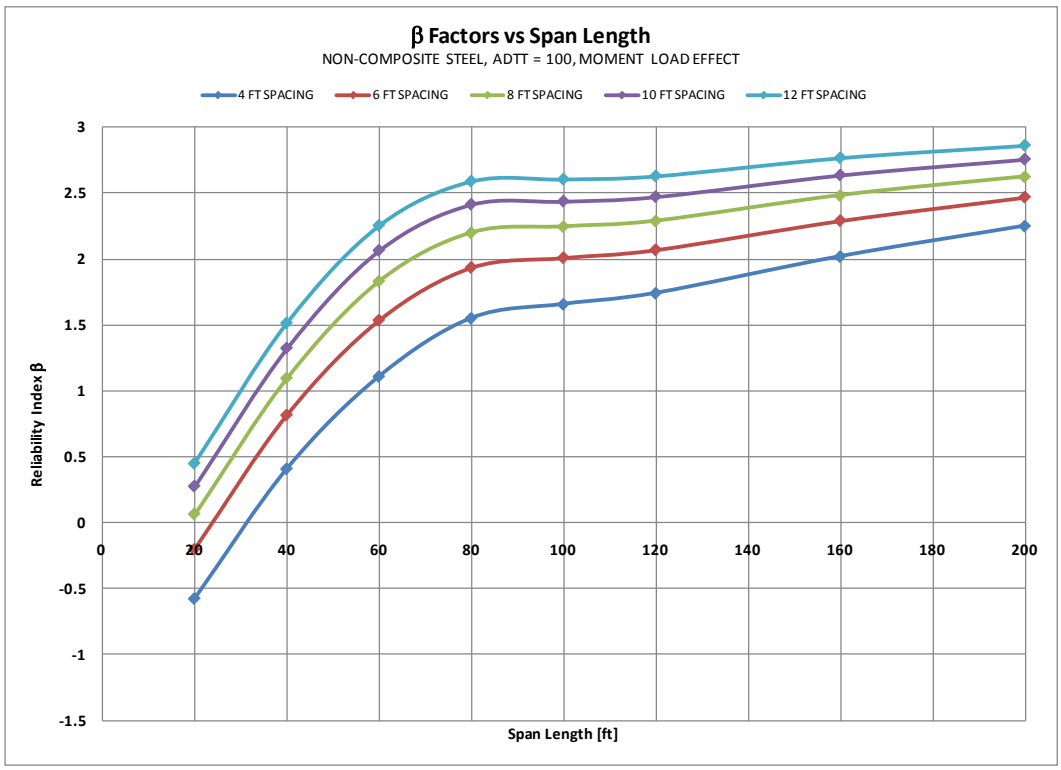
β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
-1.5
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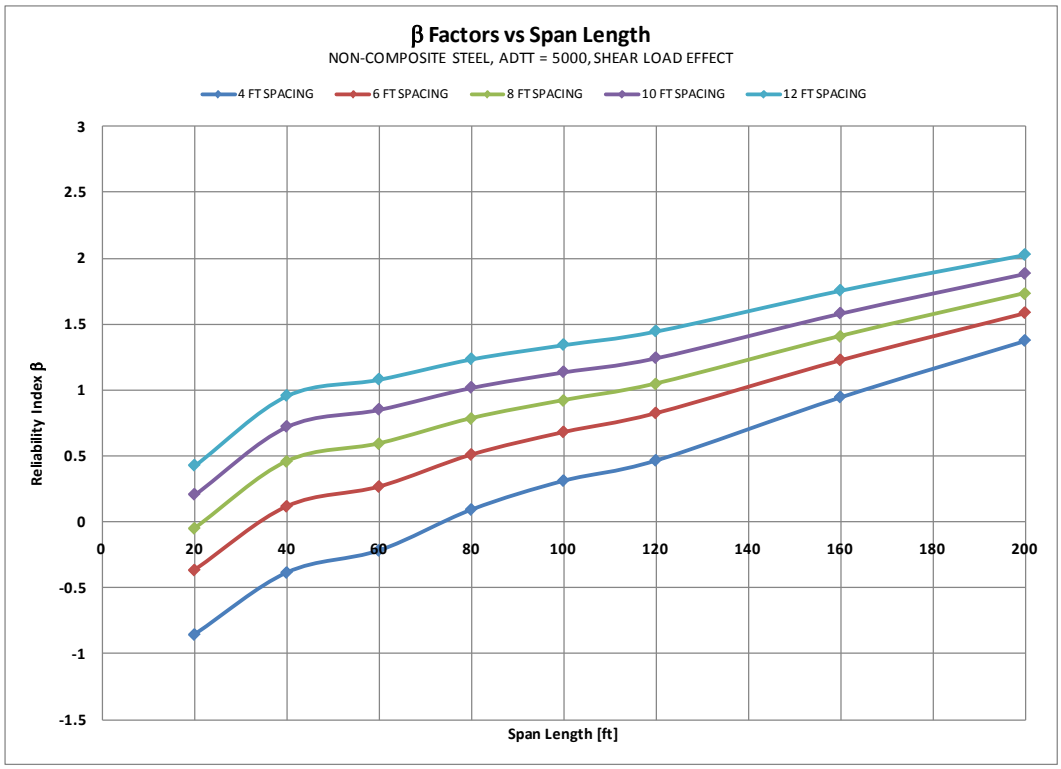
β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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-0.5
-1
-1.5
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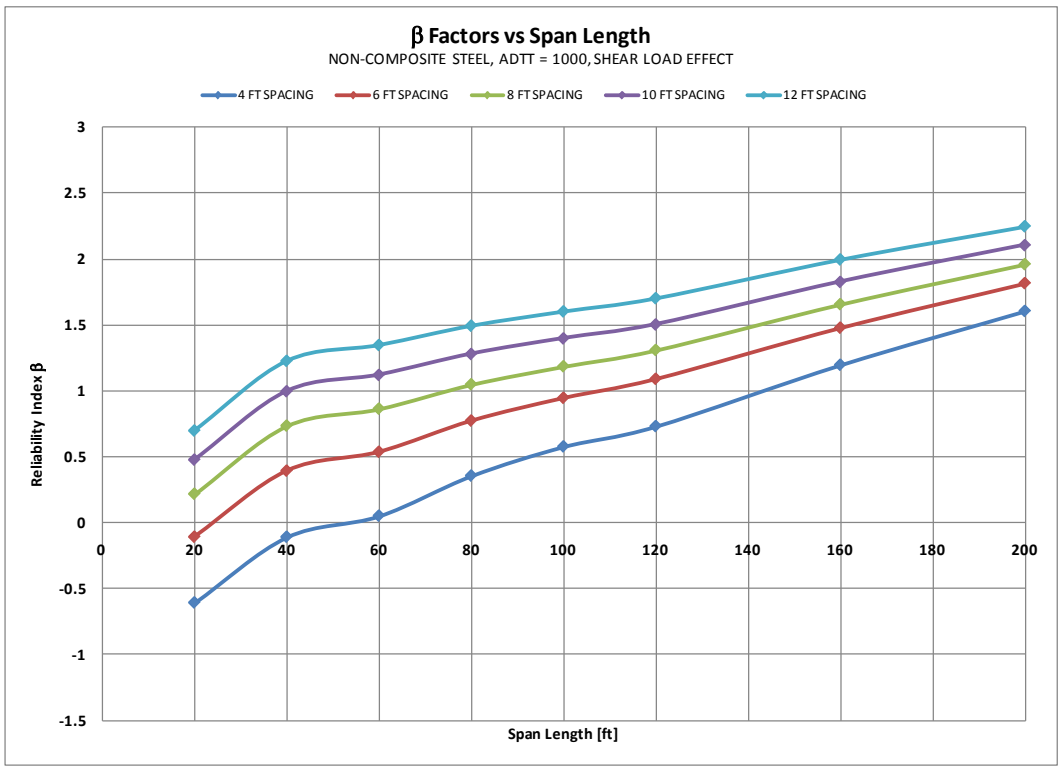
β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
-1.5
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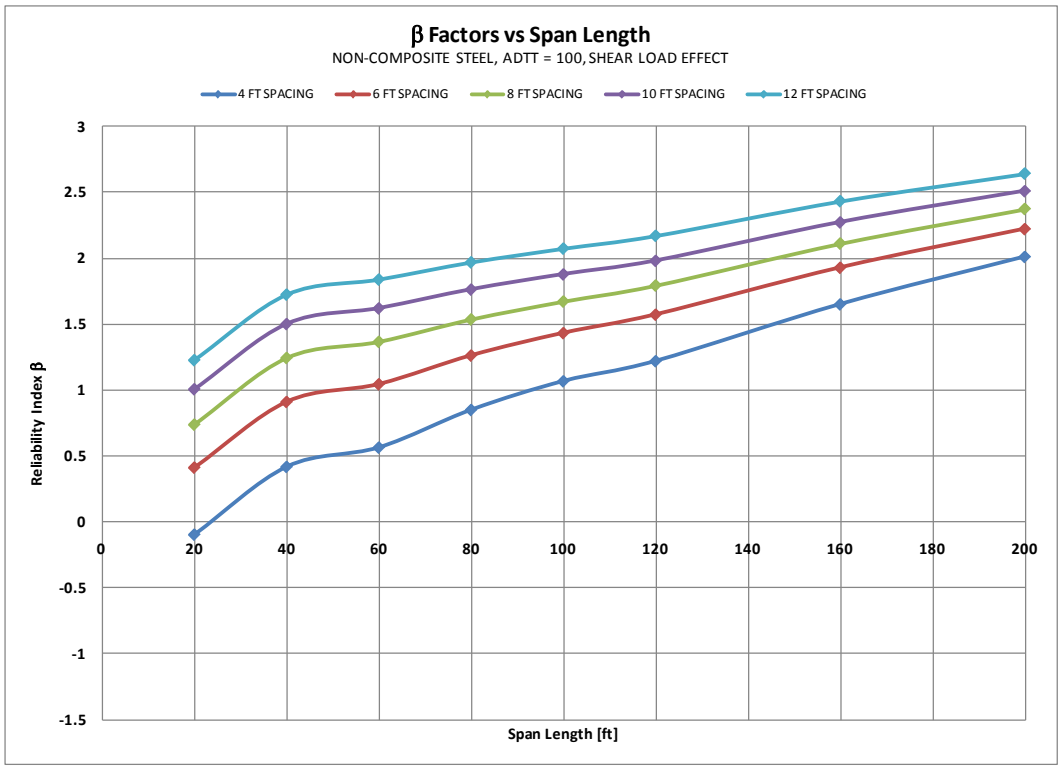
β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
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20
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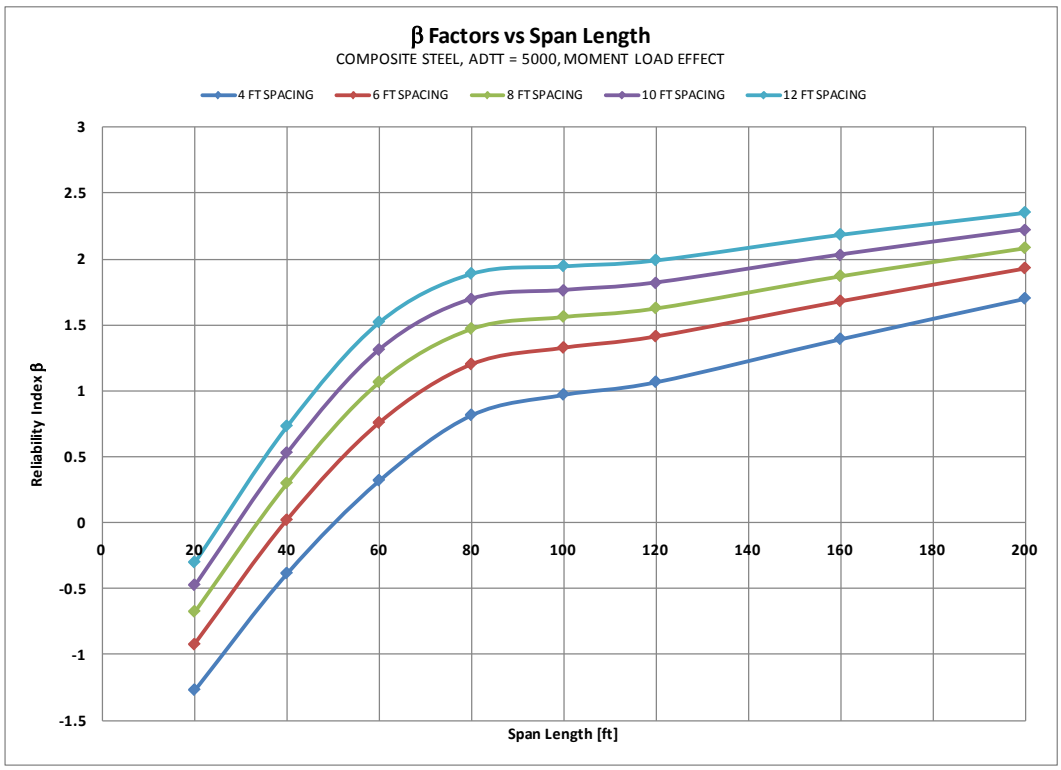
100
120
140
160
180
200

STEEL COMPOSITE BEAM

LEGAL LOAD PLOTS (ALL WIM SITES)


β Factors vs Span Length
COMPOSITE STEEL, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
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60
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100
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140
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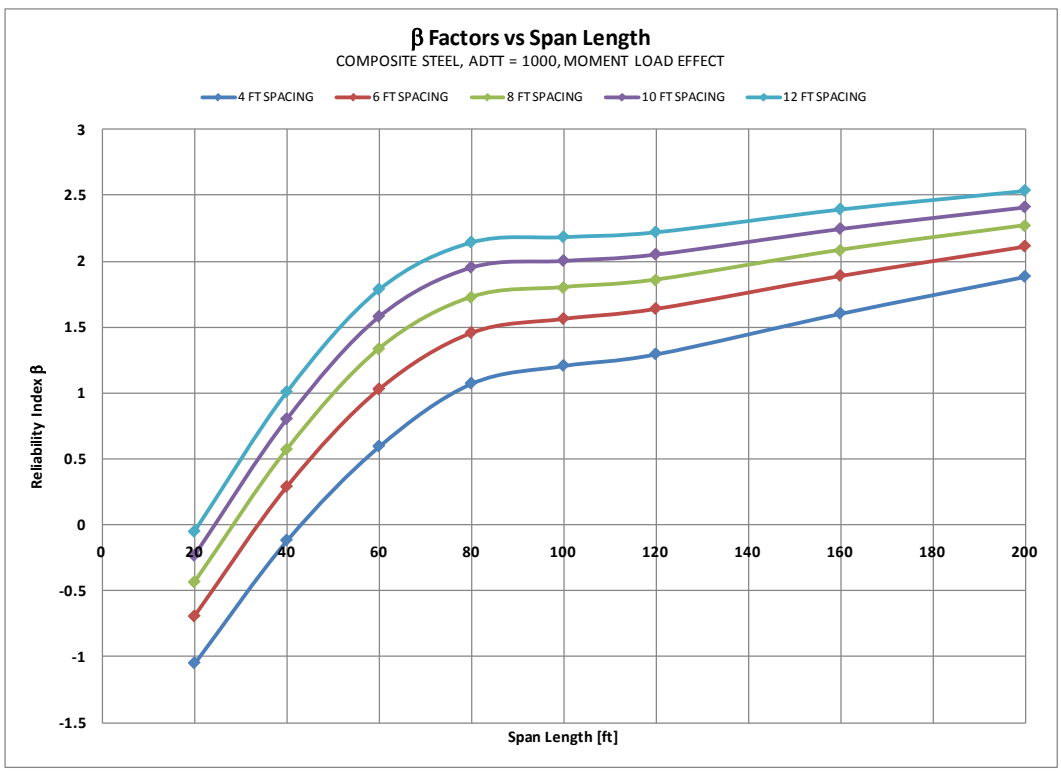
180
200


β Factors vs Span Length
COMPOSITE STEEL, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
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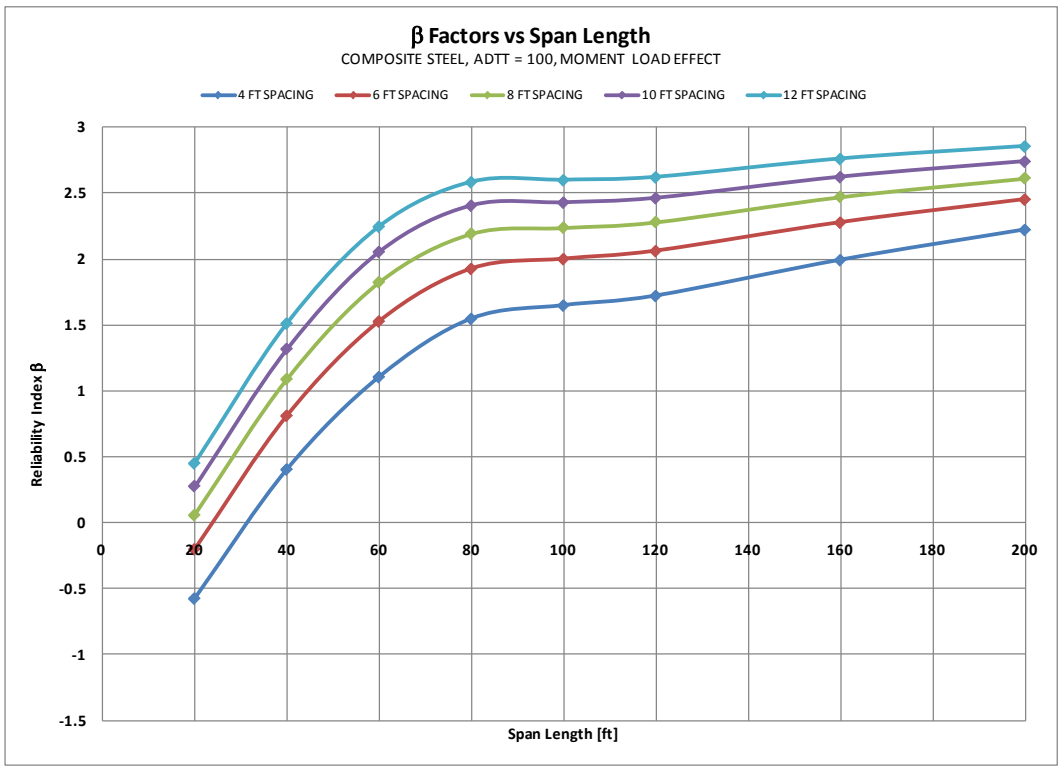
β Factors vs Span Length
COMPOSITE STEEL, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
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1.5
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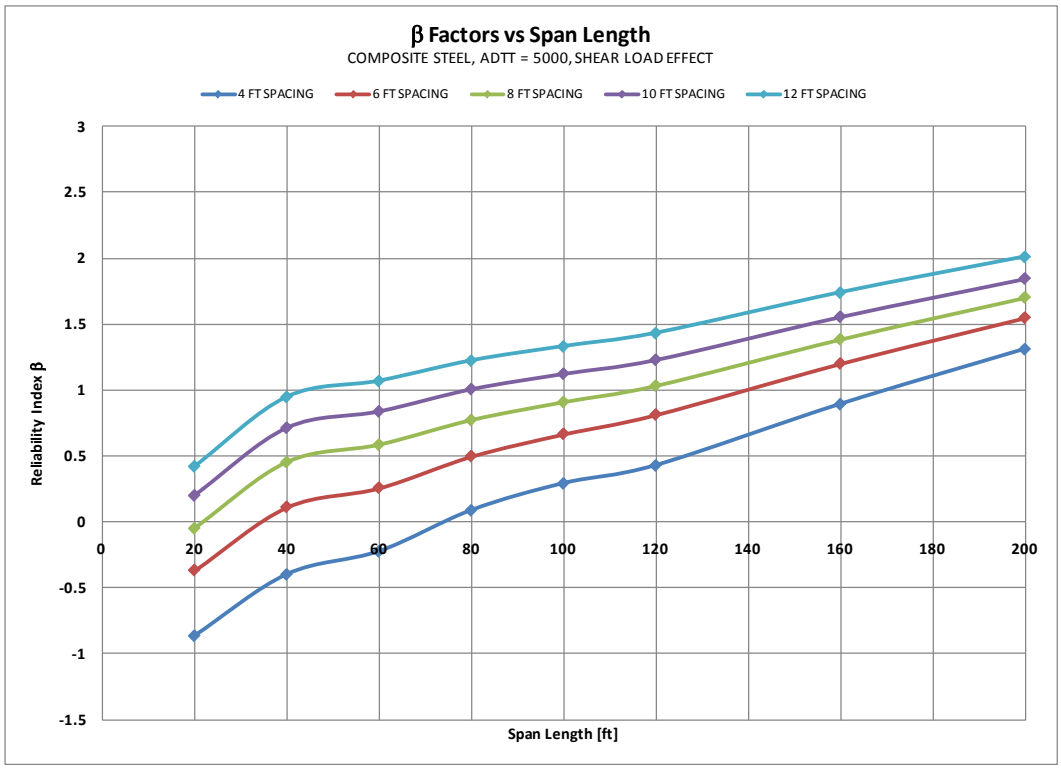
β Factors vs Span Length
COMPOSITE STEEL, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
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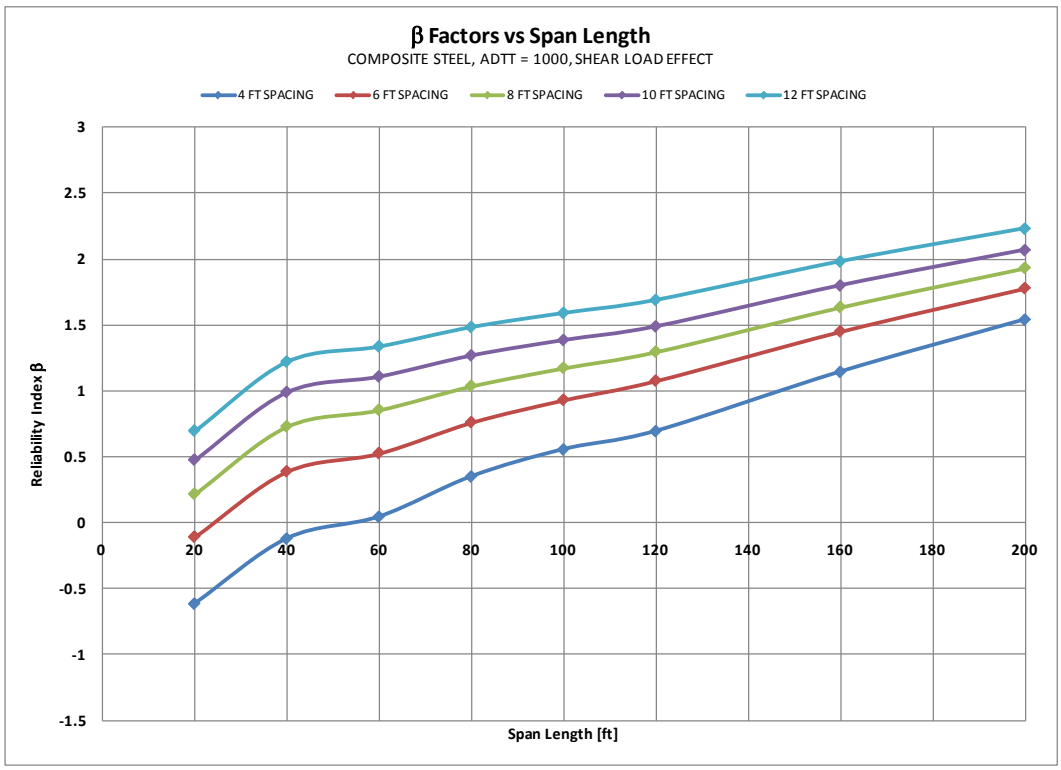
β Factors vs Span Length
COMPOSITE STEEL, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
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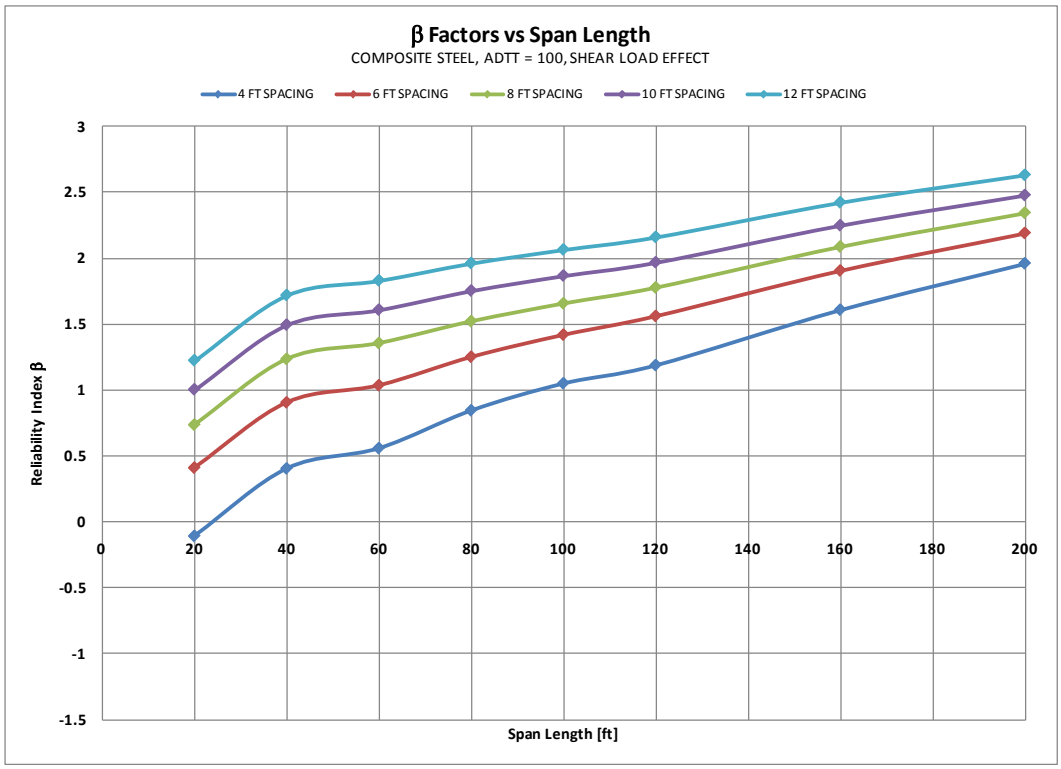
β Factors vs Span Length
COMPOSITE STEEL, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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-0.5
-1
-1.5
0
20
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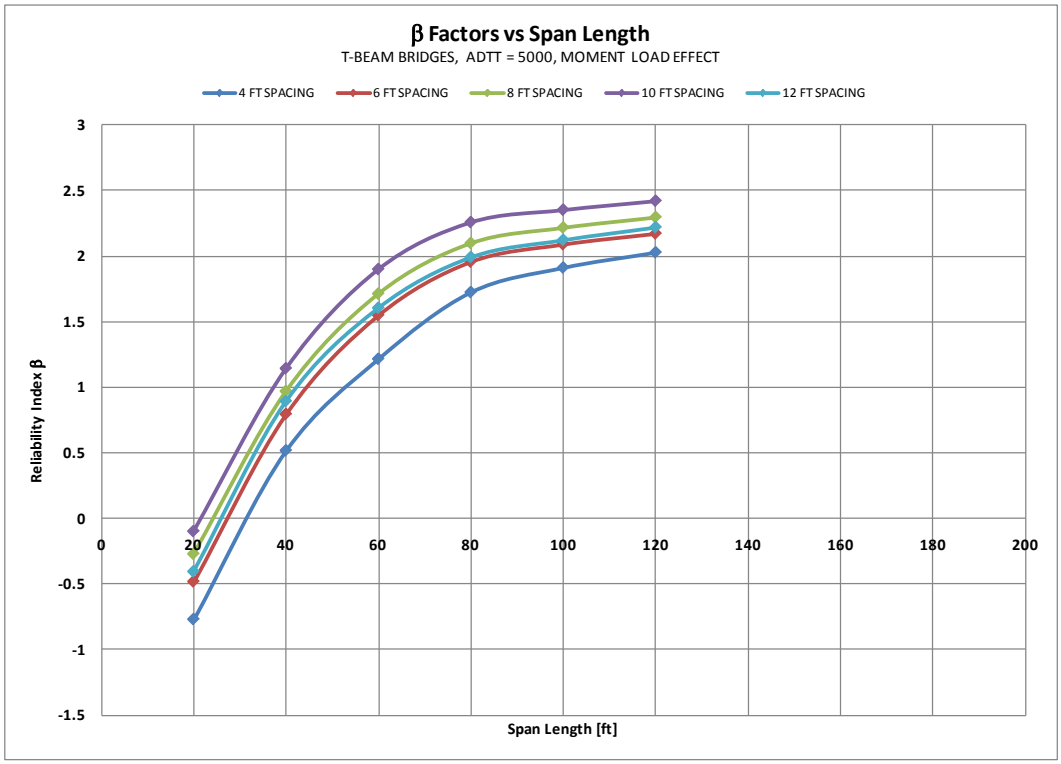
140
160
180
200

REINFORCED CONCRETE T- BEAM

LEGAL LOAD PLOTS (NY WIM SITES NOT INCLUDED)


β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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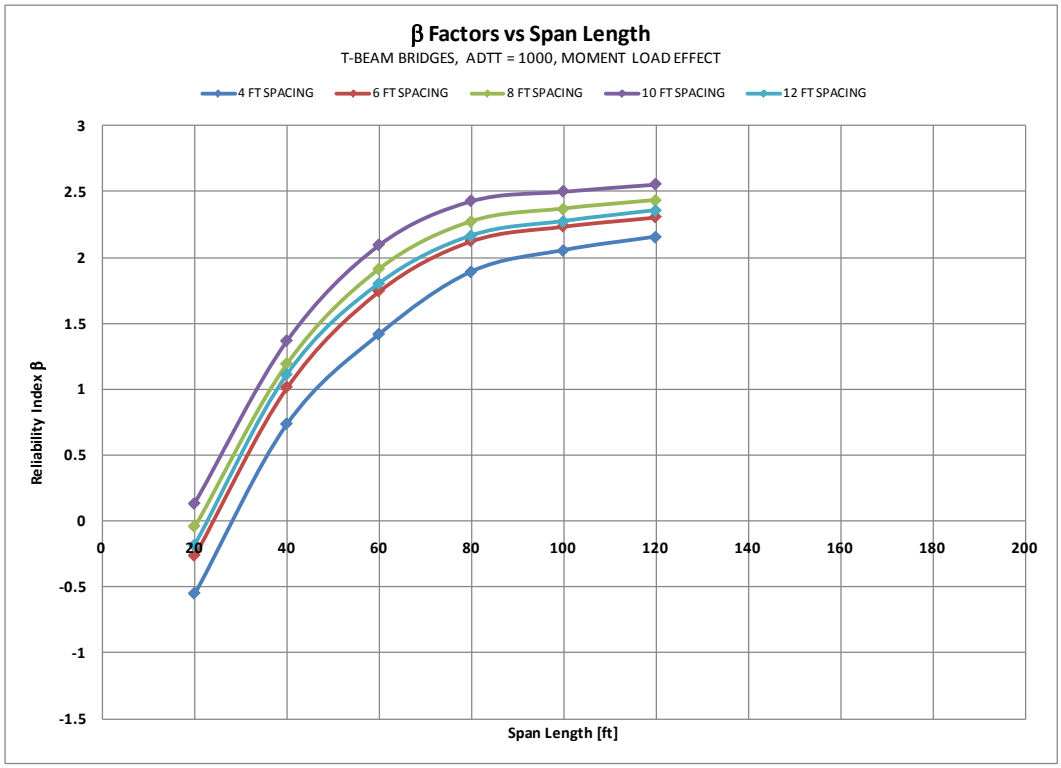
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200


β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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-0.5
-1
-1.5
0
20
40
60
80
100
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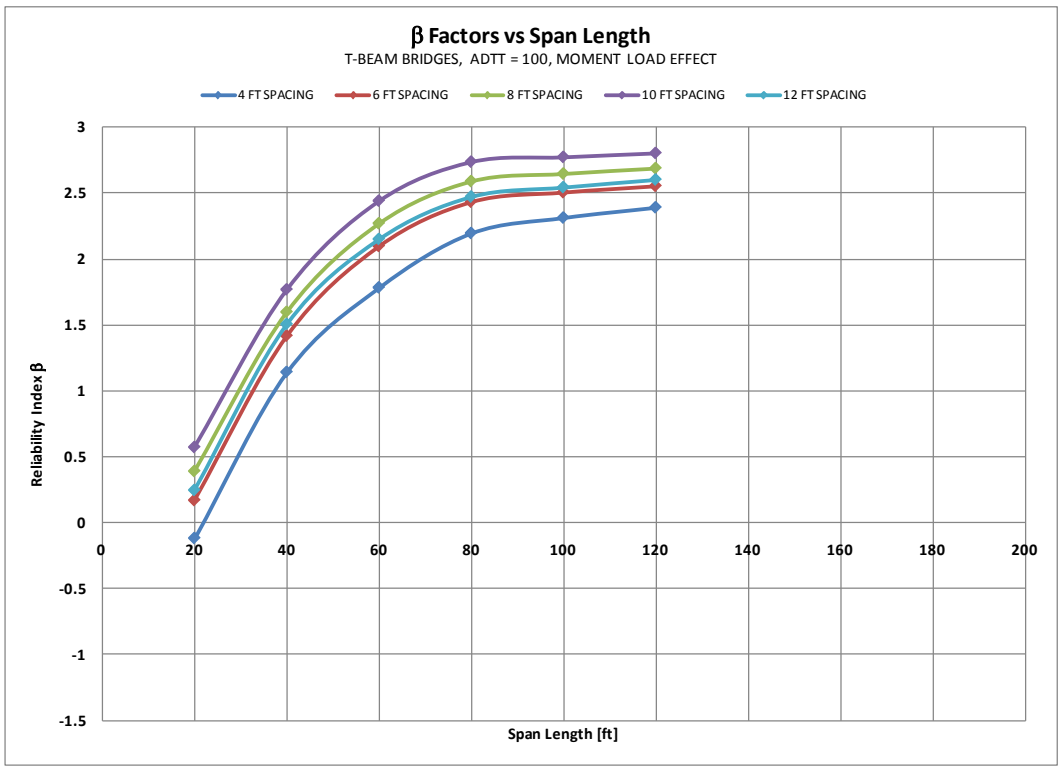
140
160
180
200


β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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0.5
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-0.5
-1
-1.5
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20
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60
80
100
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140
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180
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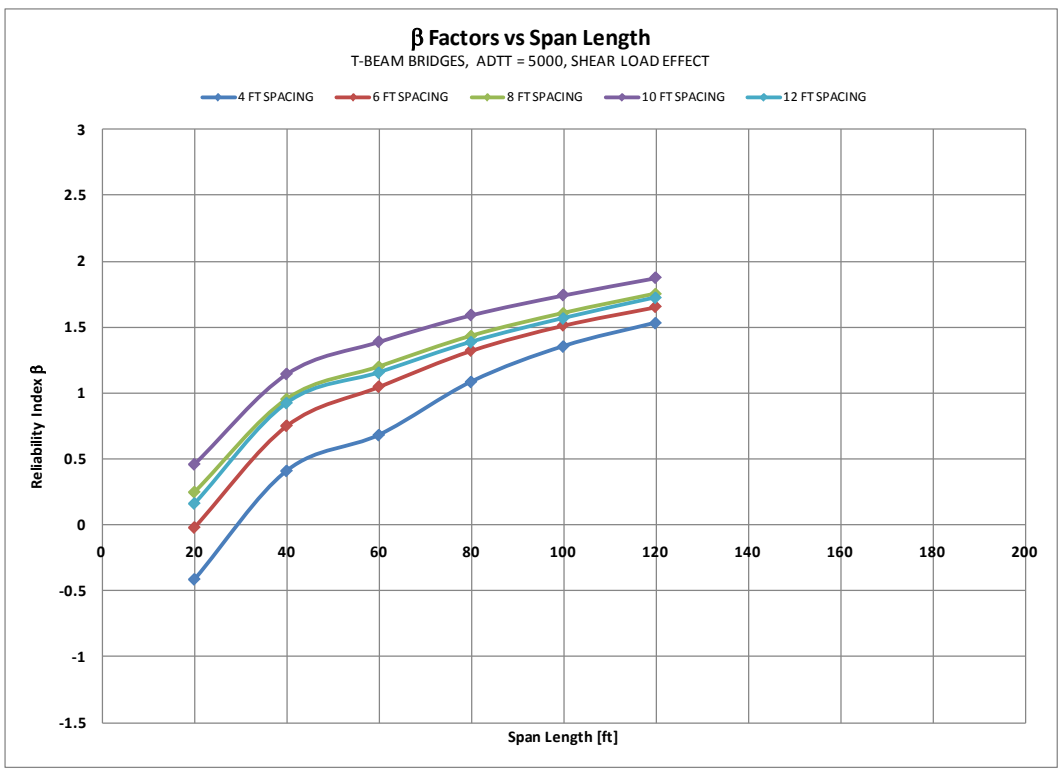
β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
0
20
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60
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180
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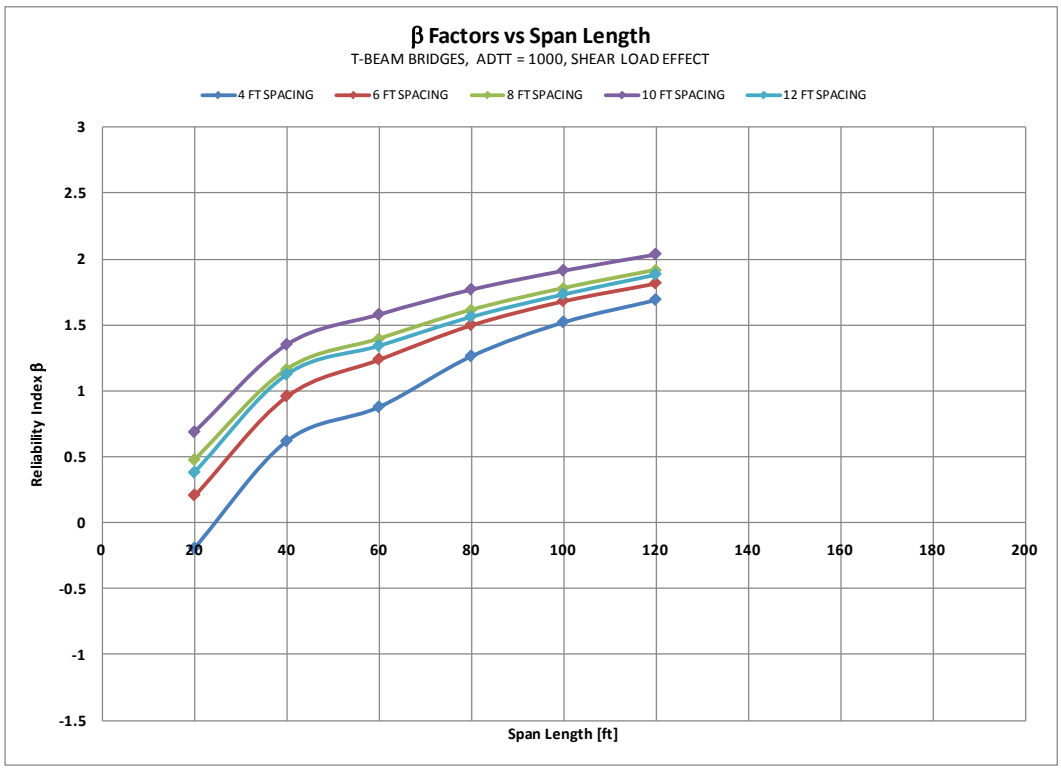
β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
-1.5
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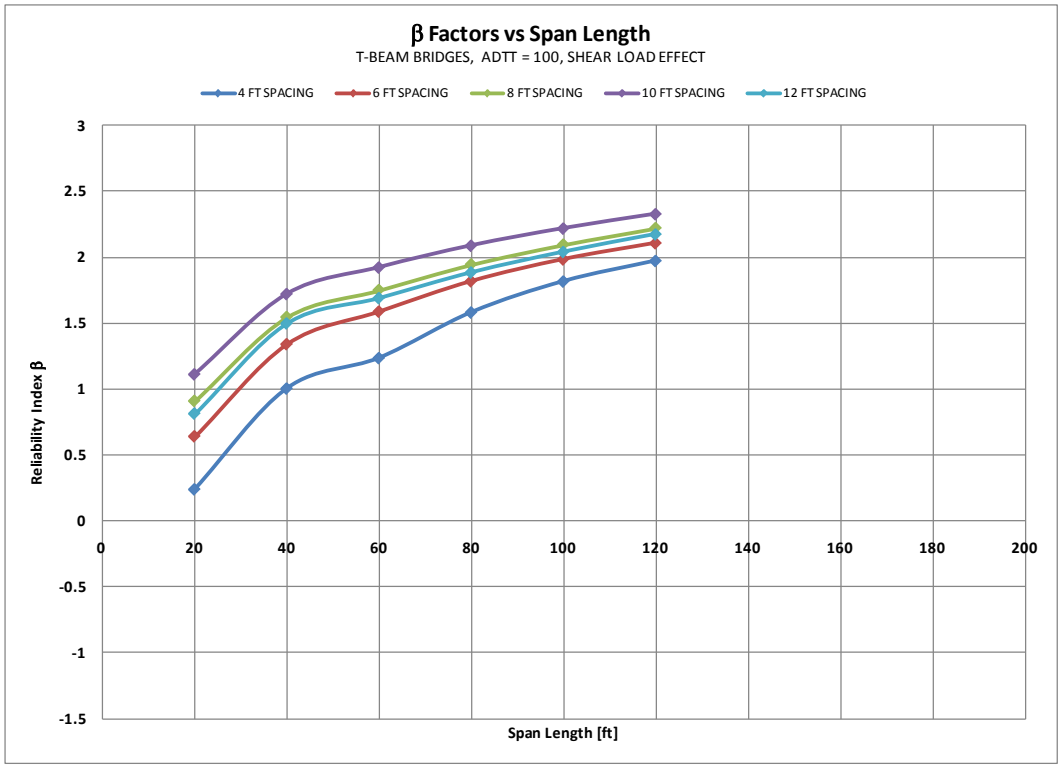
β Factors vs Span Length
T-BEAM BRIDGES, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
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60
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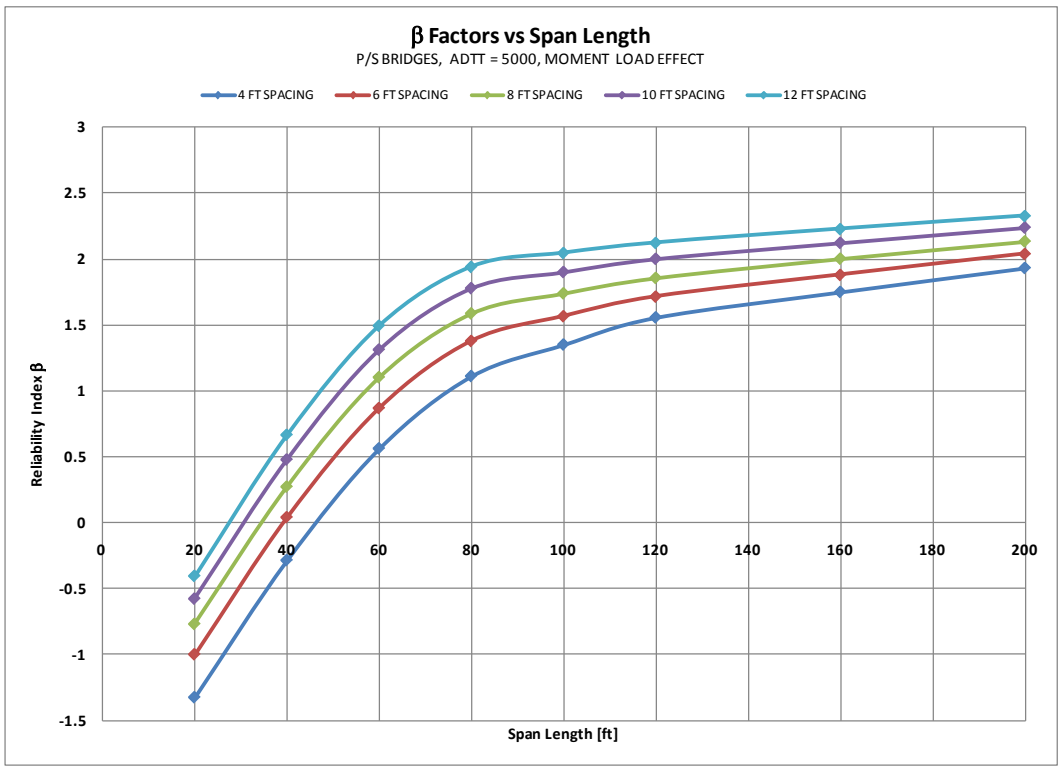
100
120
140
160
180
200

PRESTRESSED CONCRETE BEAM

LEGAL LOAD PLOTS (NY WIM SITES NOT INCLUDED)


β Factors vs Span Length
P/S BRIDGES, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
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-0.5
-1
-1.5
0
20
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60
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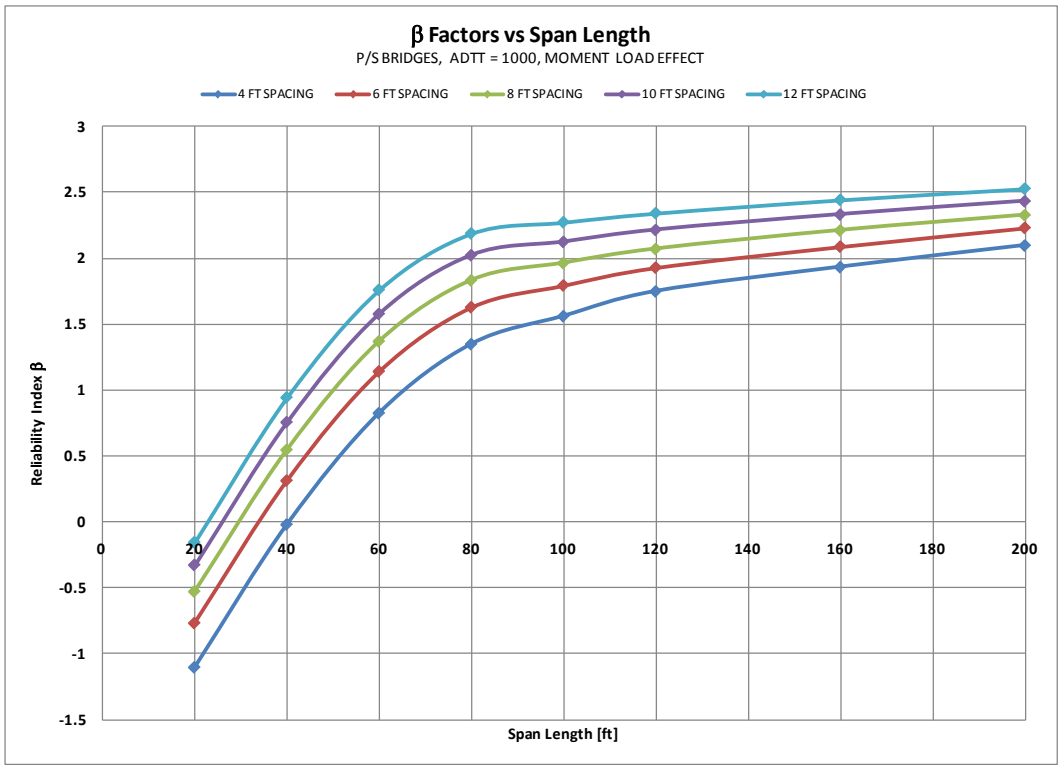
β Factors vs Span Length
P/S BRIDGES, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
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60
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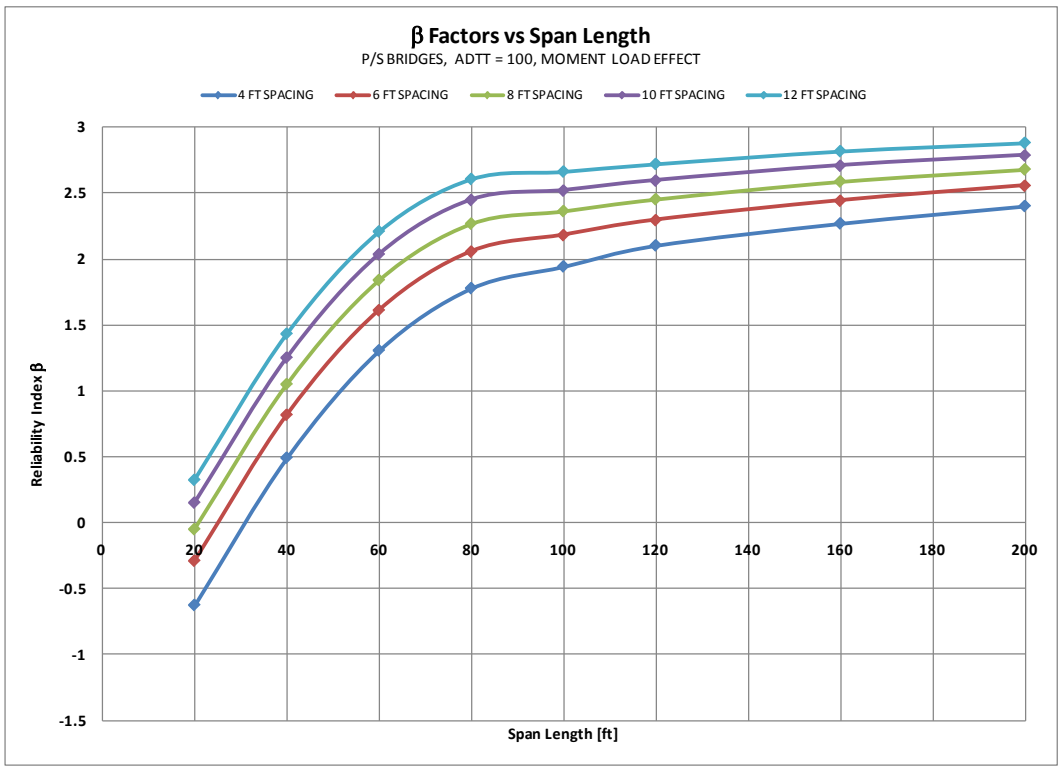
β Factors vs Span Length
P/S BRIDGES, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
-1.5
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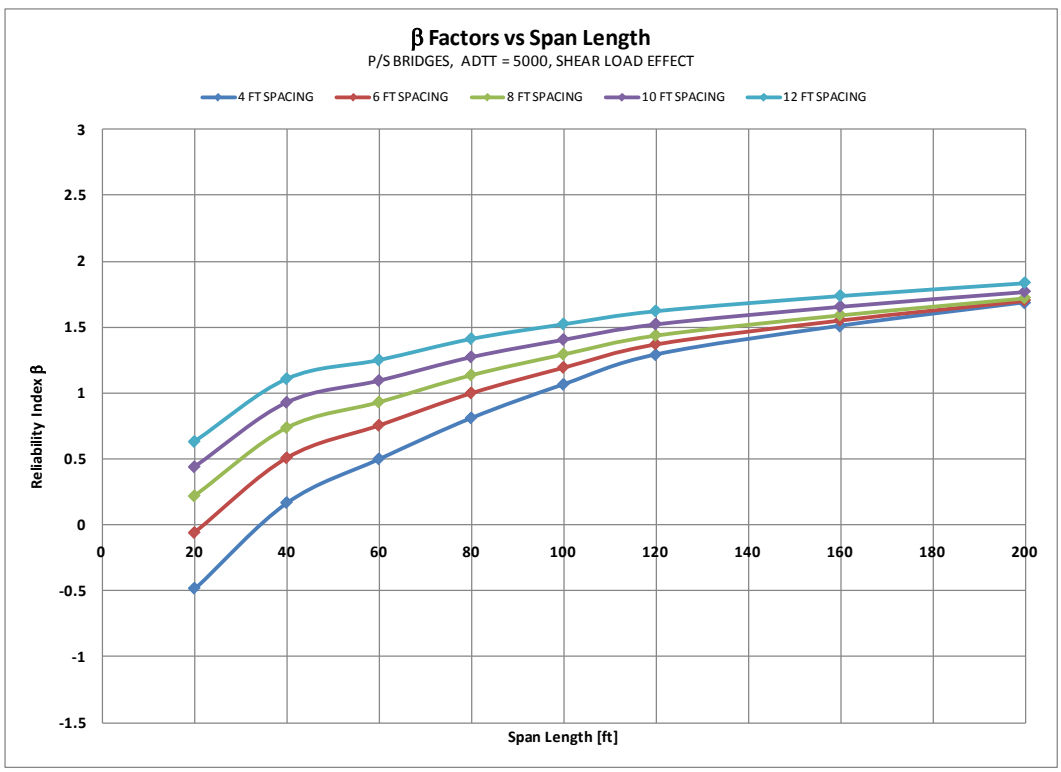
β Factors vs Span Length
P/S BRIDGES, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-1
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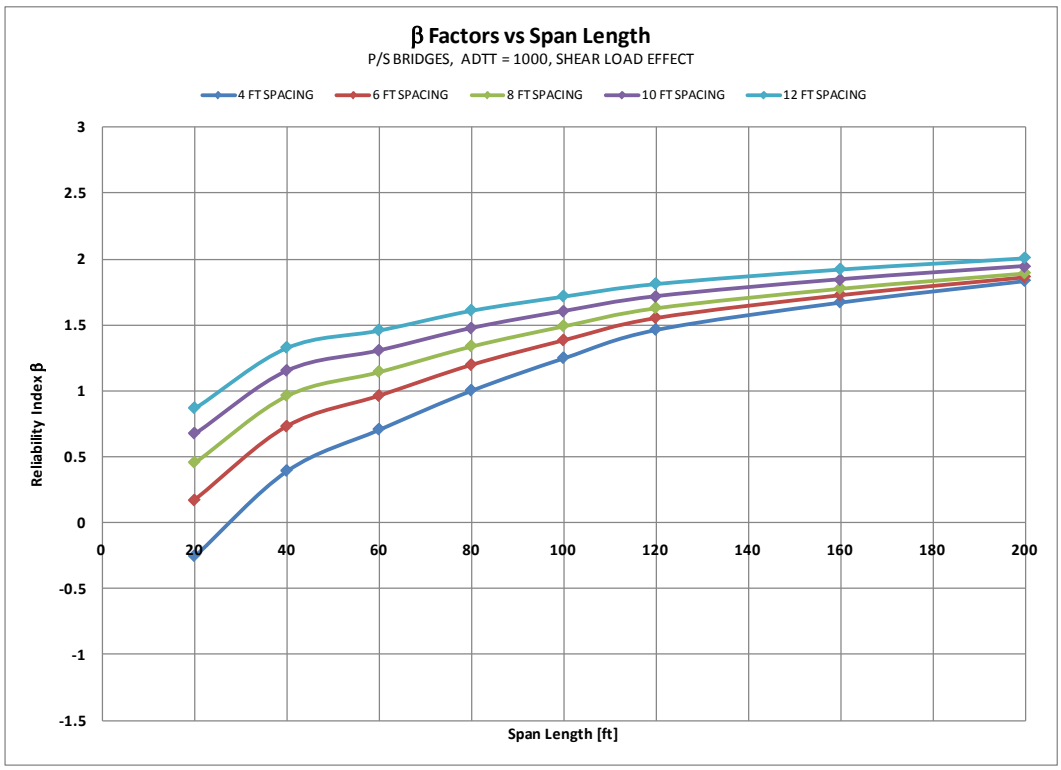
β Factors vs Span Length
P/S BRIDGES, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
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1.5
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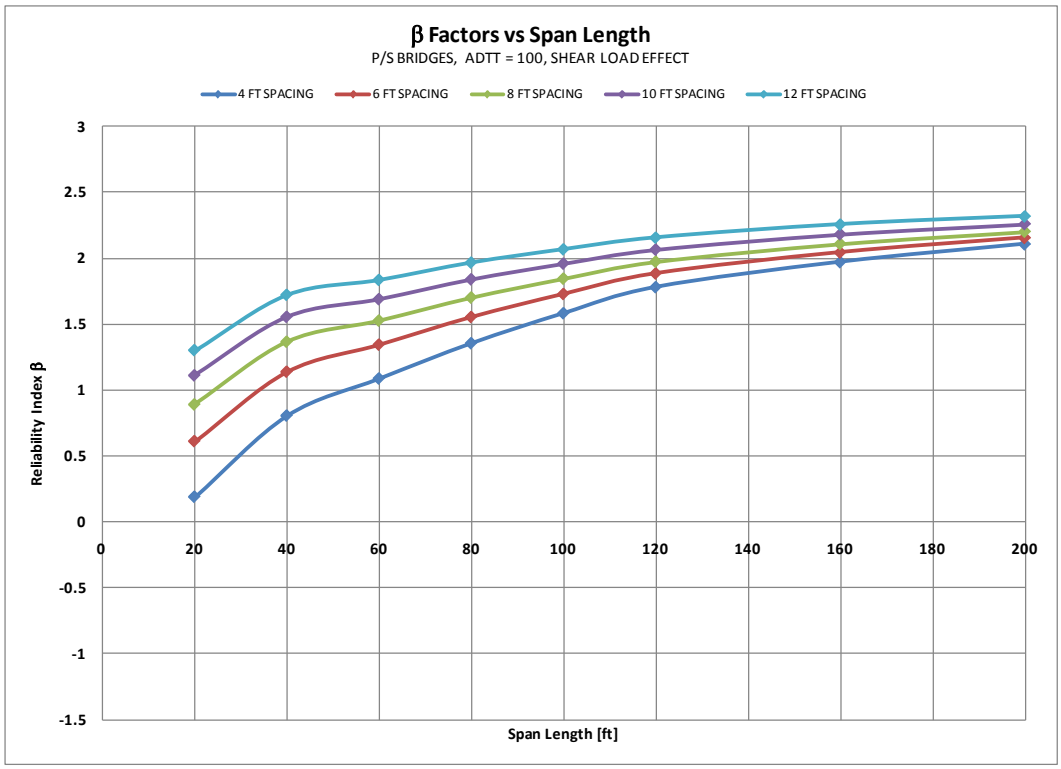
β Factors vs Span Length
P/S BRIDGES, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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-0.5
-1
-1.5
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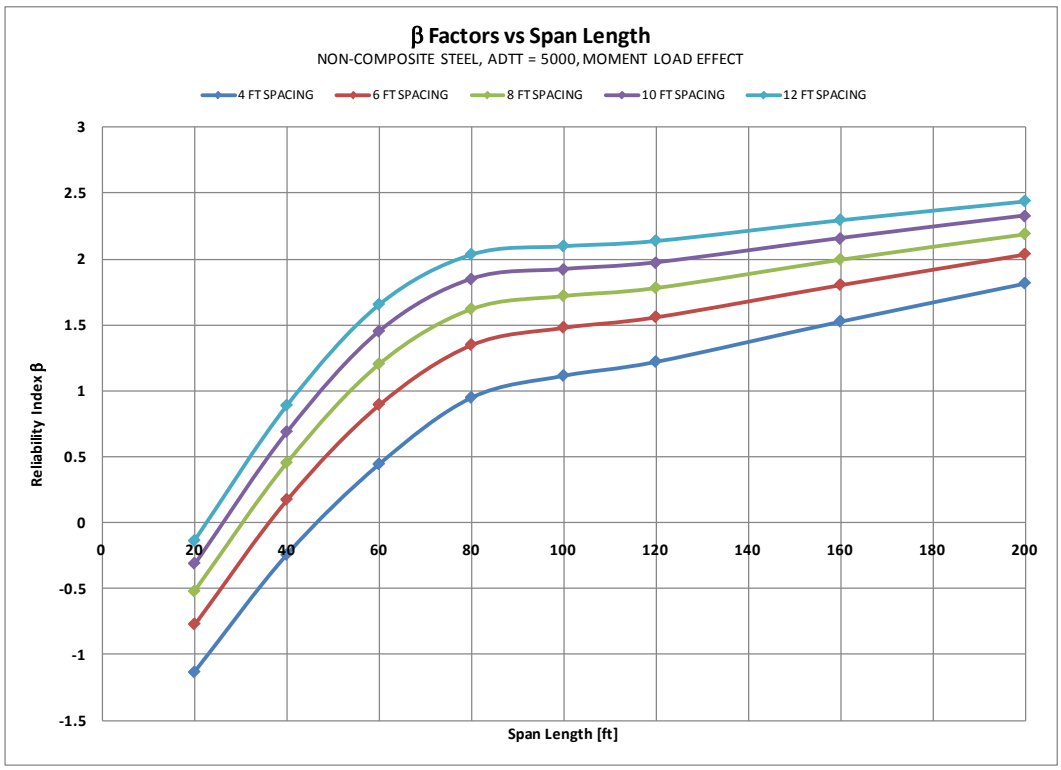
STEEL NON-COMPOSITE BEAM

LEGAL LOAD PLOTS (NY WIM SITES NOT INCLUDED)


β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
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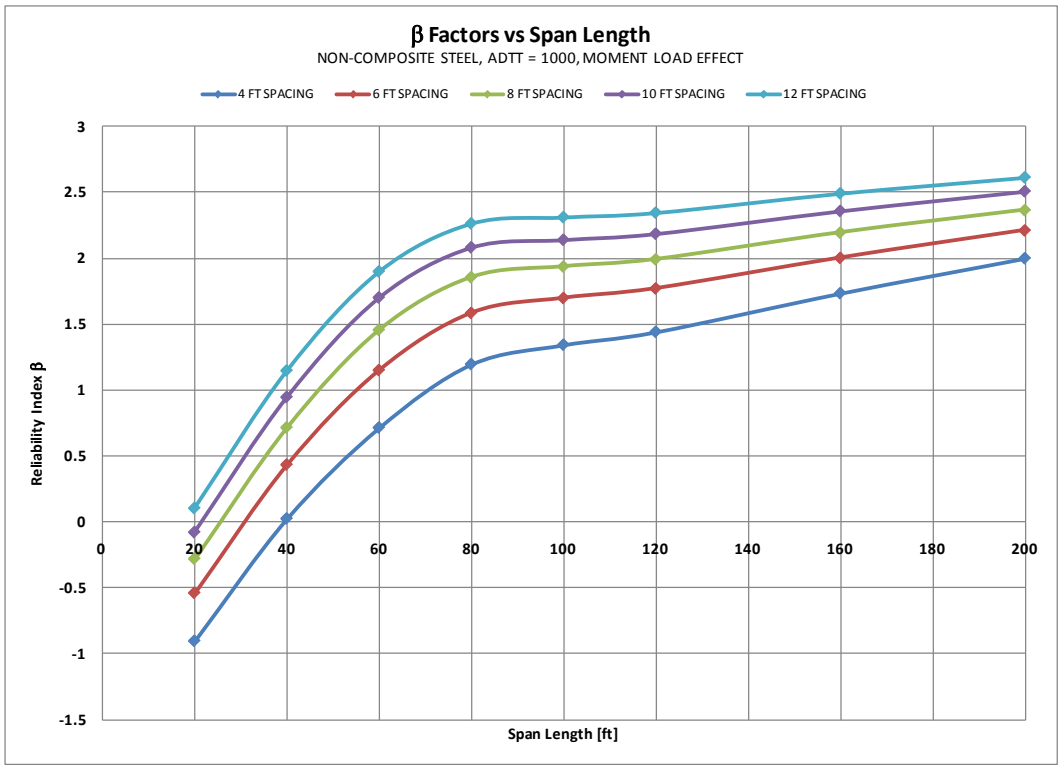
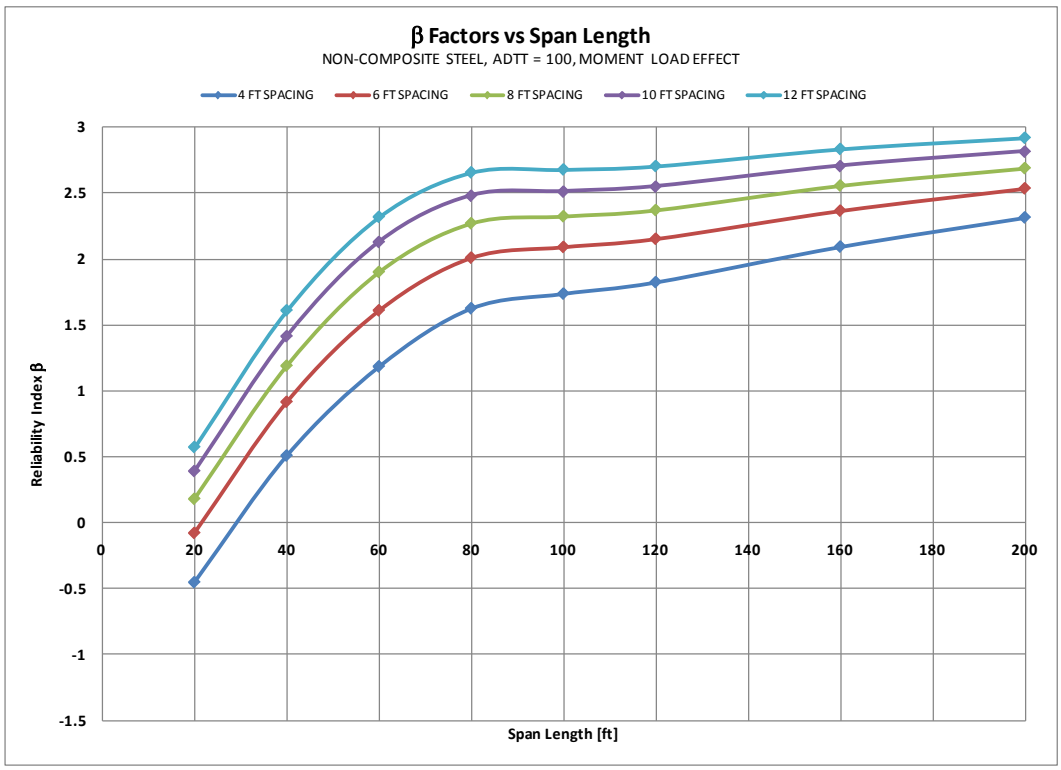
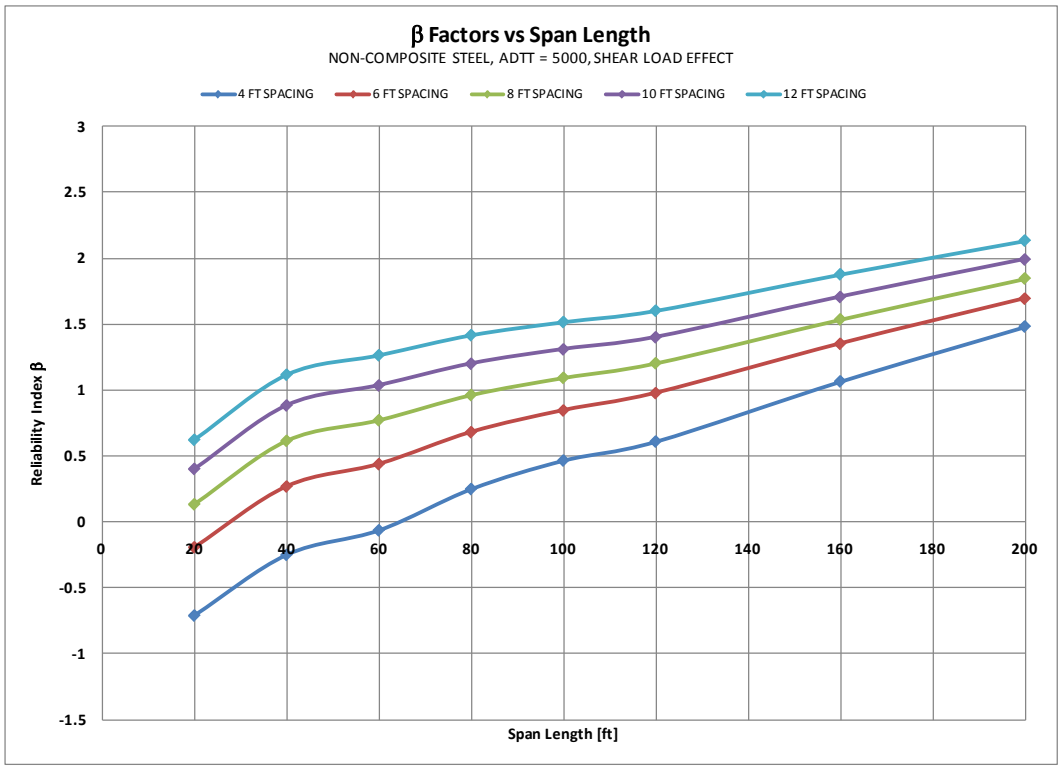
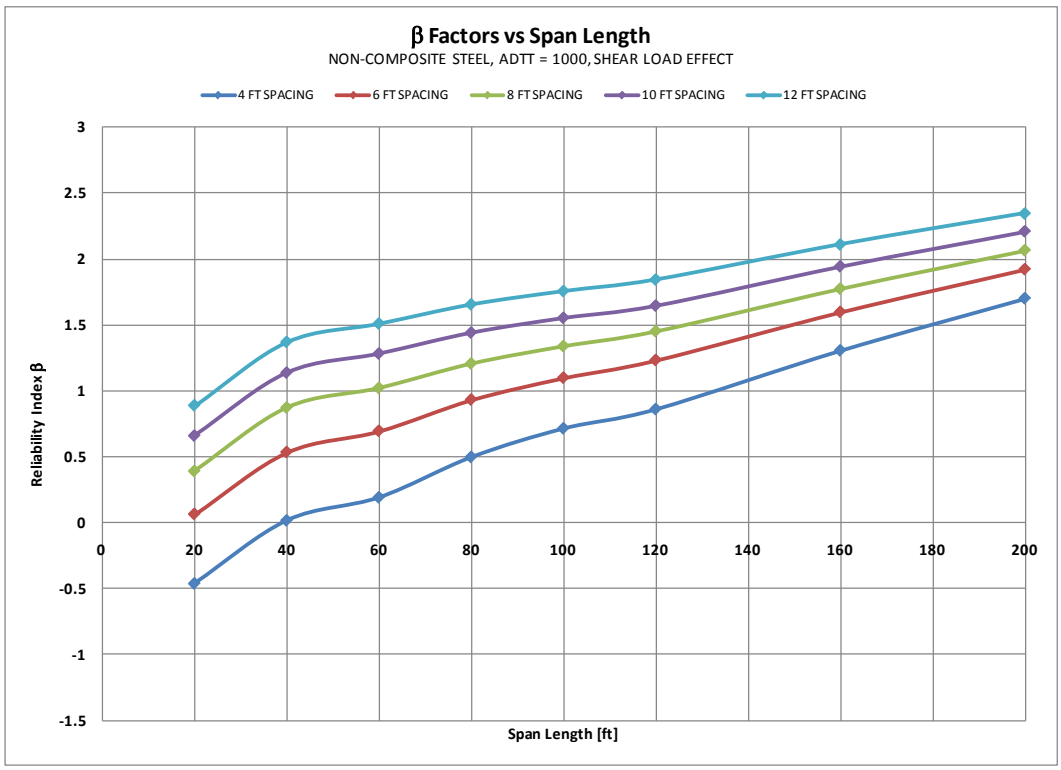
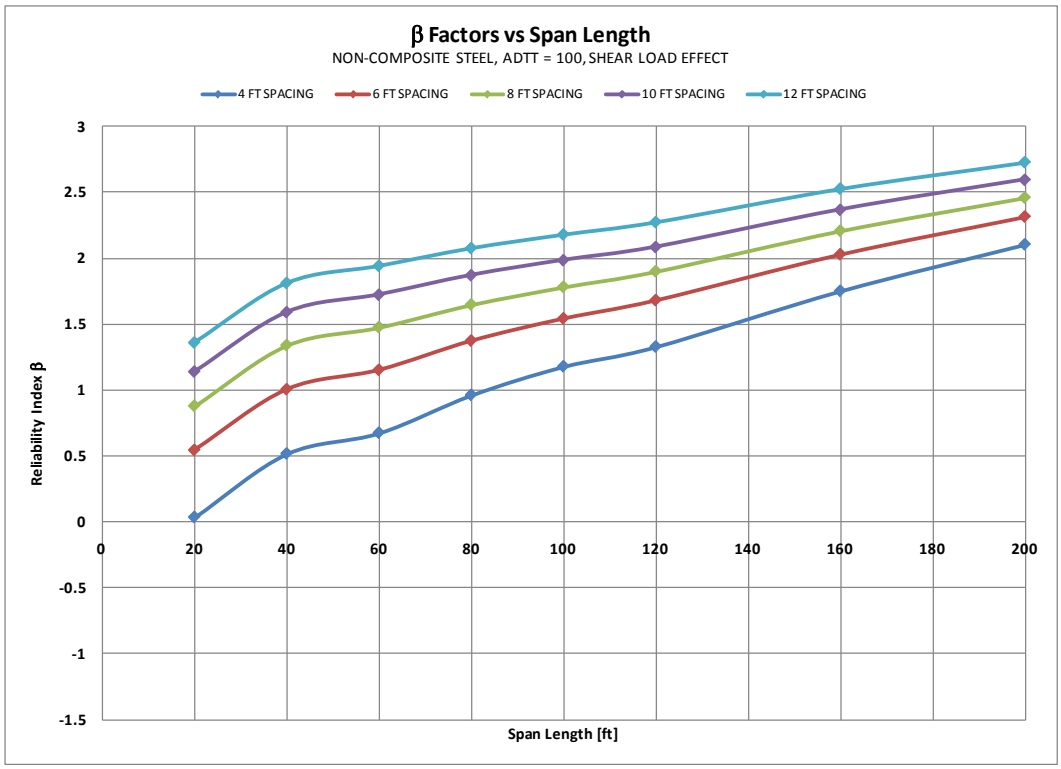
180
200


β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200


β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

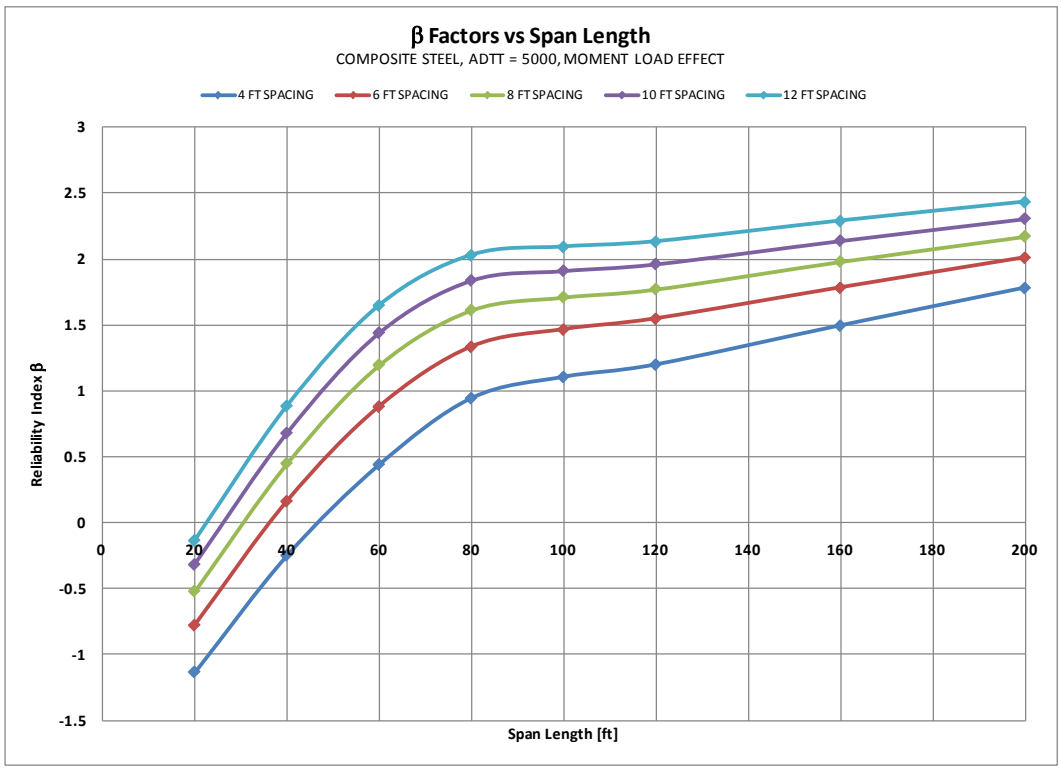

β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

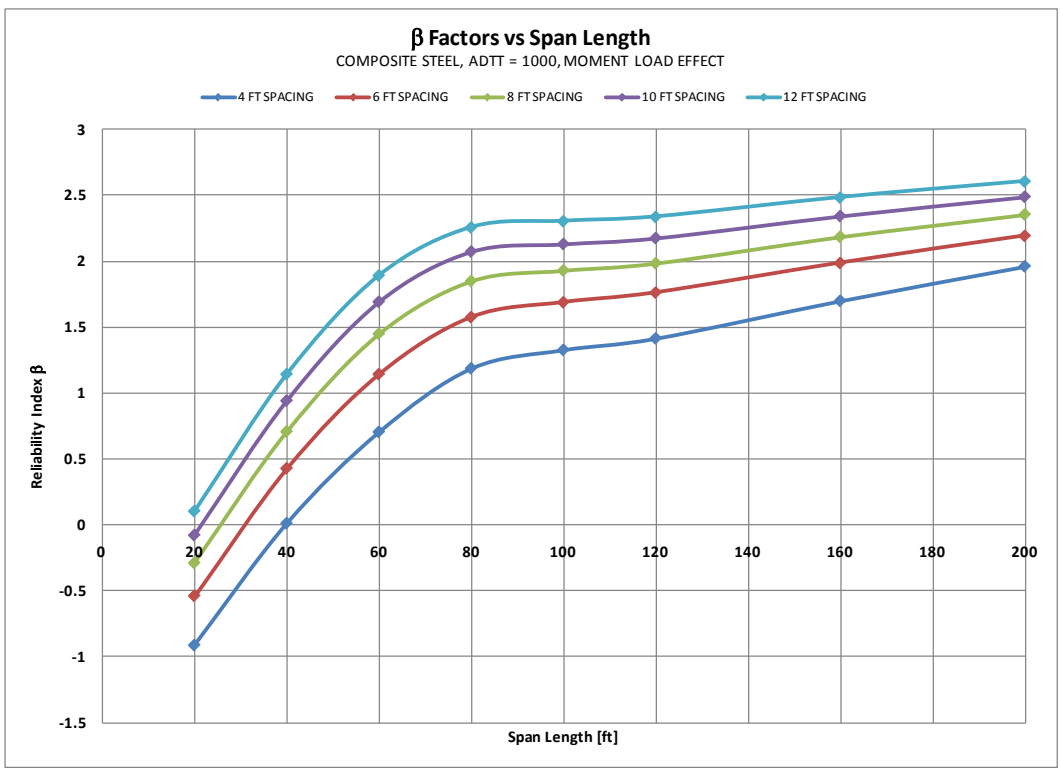

β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

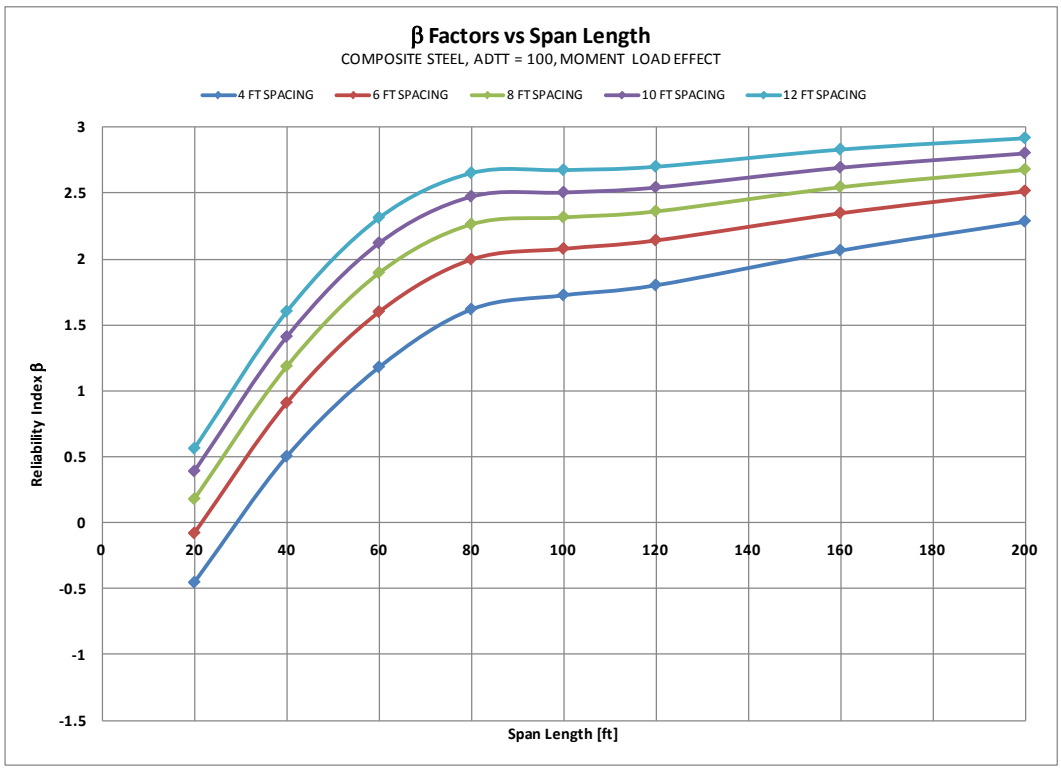

β Factors vs Span Length
NON-COMPOSITE STEEL, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

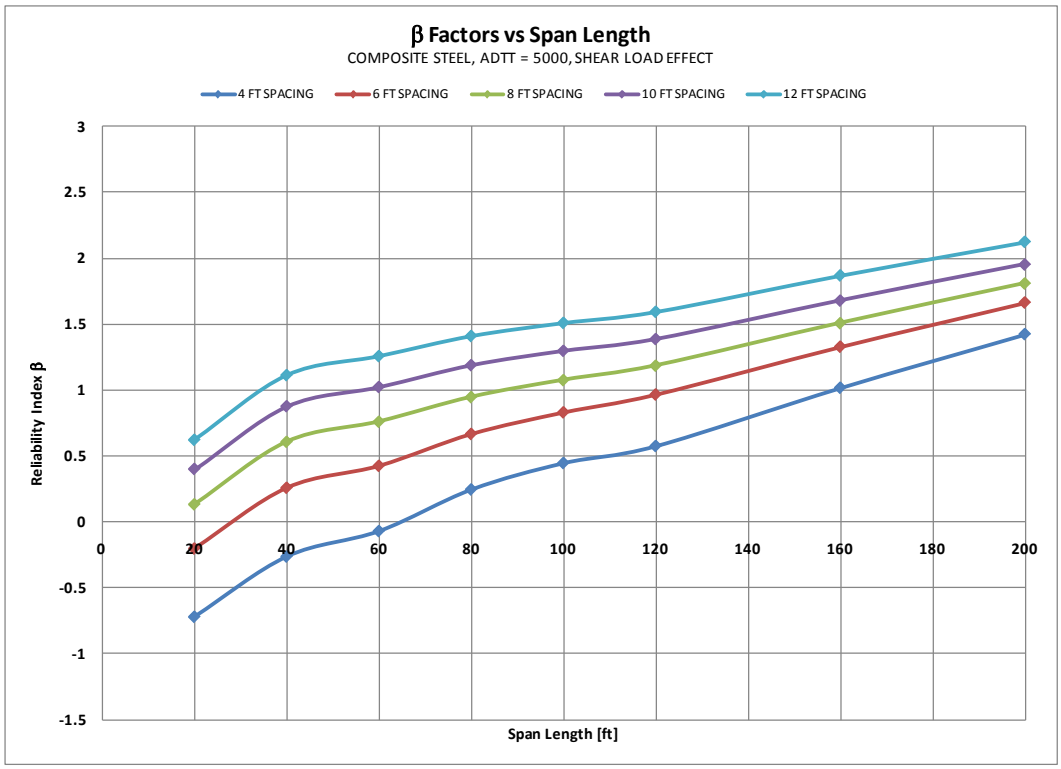
STEEL COMPOSITE BEAM

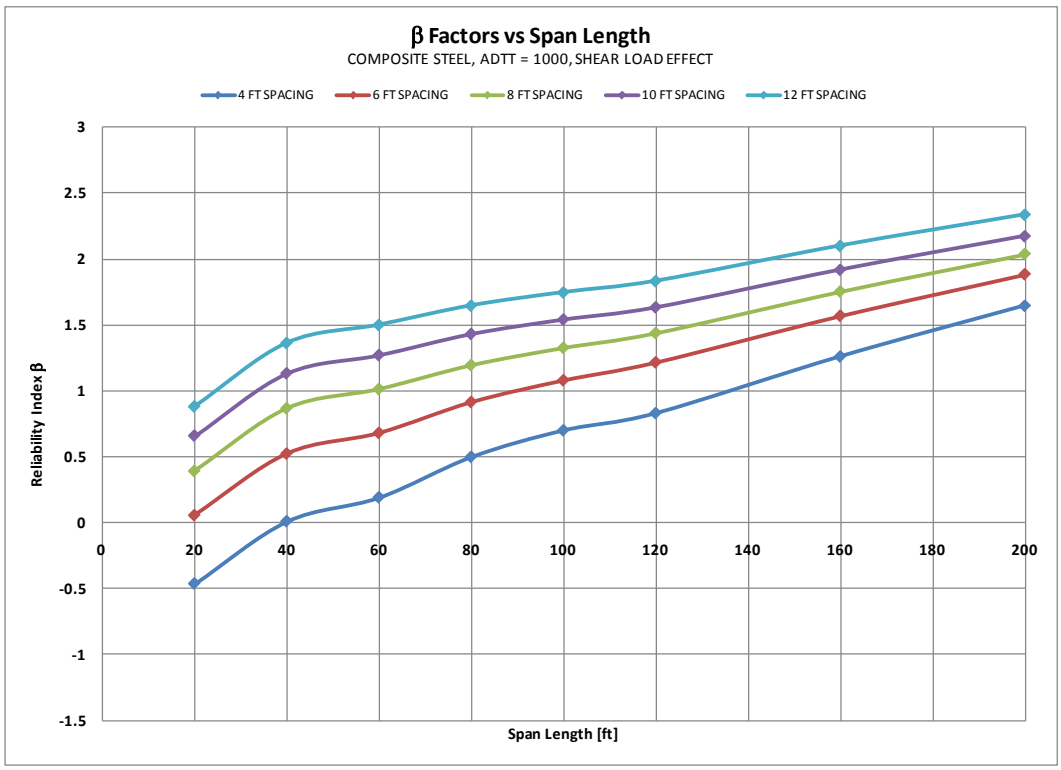
LEGAL LOAD PLOTS (NY WIM SITES NOT INCLUDED)

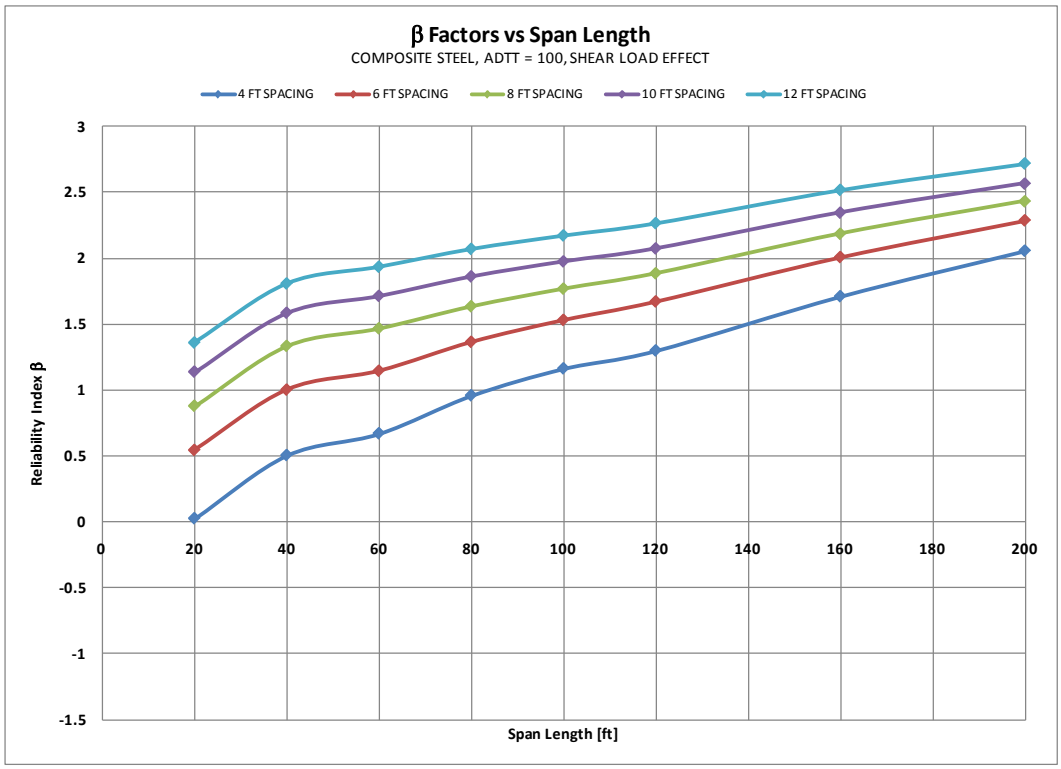

β Factors vs Span Length
COMPOSITE STEEL, ADTT = 5000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200


β Factors vs Span Length
COMPOSITE STEEL, ADTT = 1000, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200


β Factors vs Span Length
COMPOSITE STEEL, ADTT = 100, MOMENT LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200


β Factors vs Span Length
COMPOSITE STEEL, ADTT = 5000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

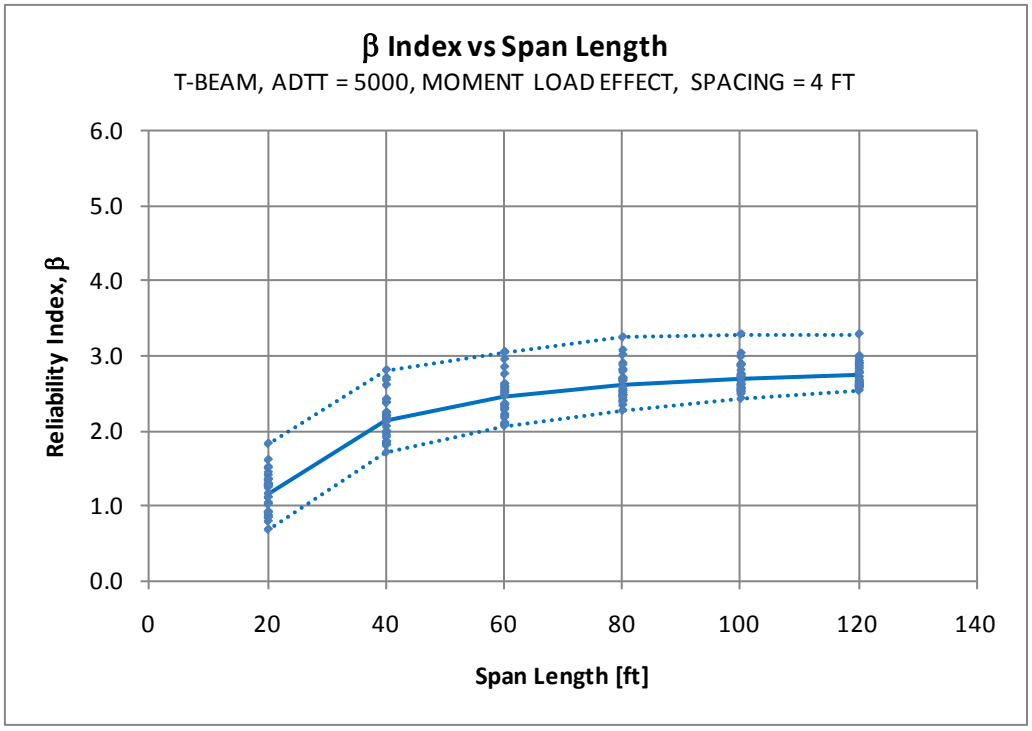

β Factors vs Span Length
COMPOSITE STEEL, ADTT = 1000, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

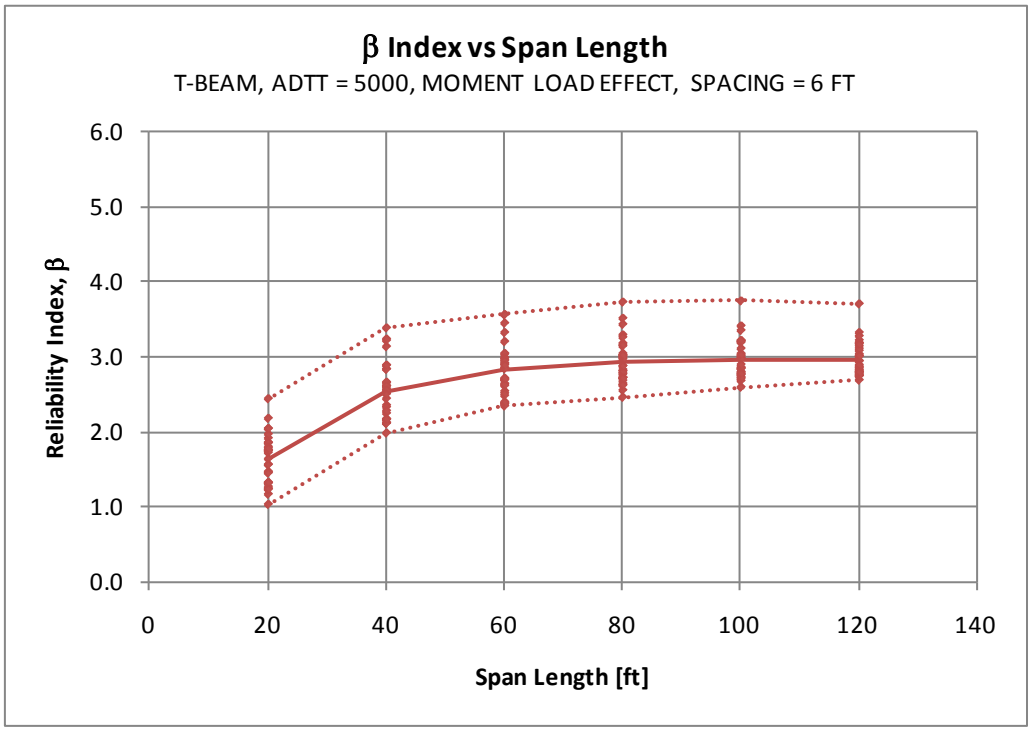

β Factors vs Span Length
COMPOSITE STEEL, ADTT = 100, SHEAR LOAD EFFECT
4 FT SPACING
6 FT SPACING
8 FT SPACING
10 FT SPACING
12 FT SPACING
Reliability Index β
Span Length [ft]
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
20
40
60
80
100
120
140
160
180
200

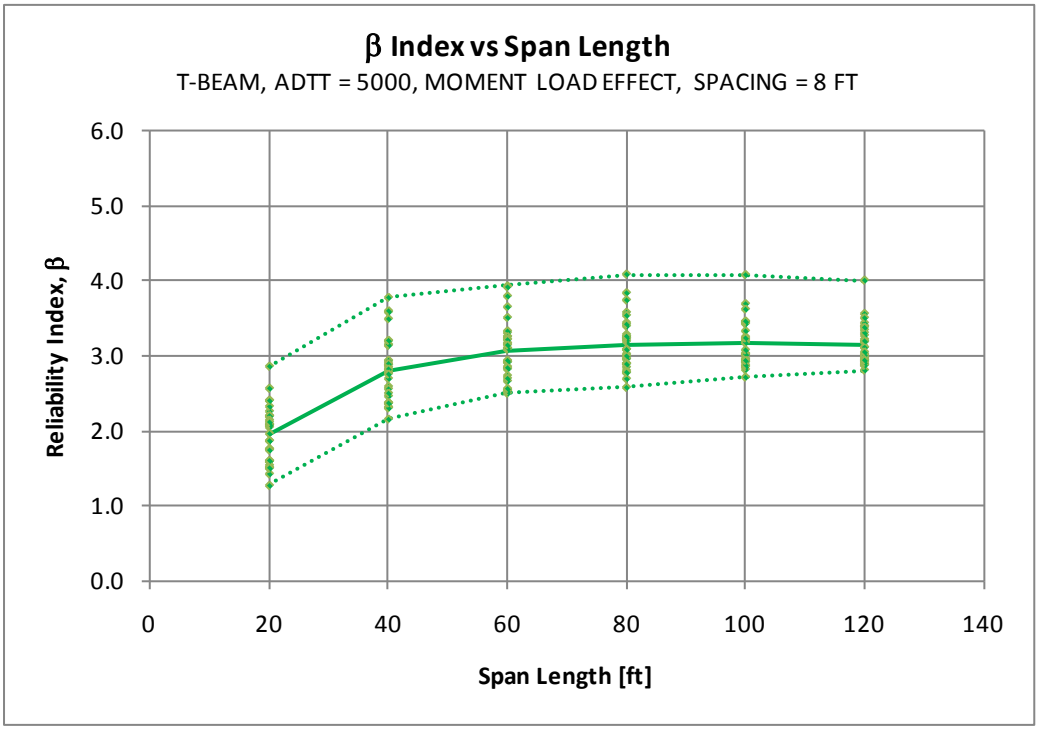
REINFORCED CONCRETE T- BEAM

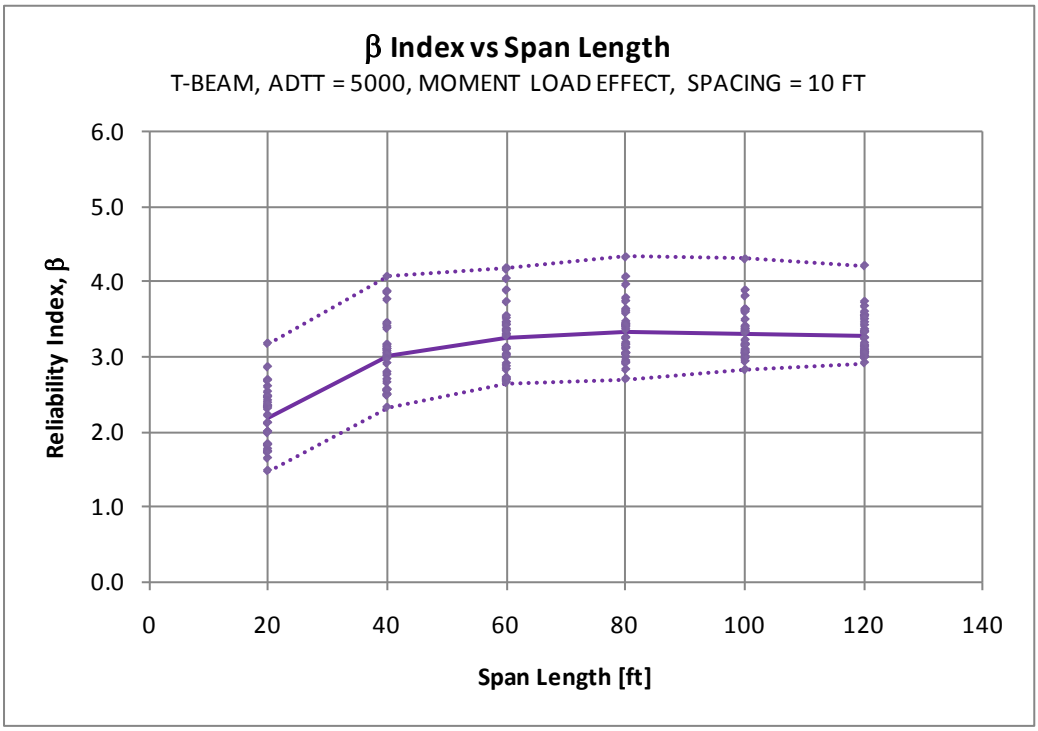
PERMIT PLOTS

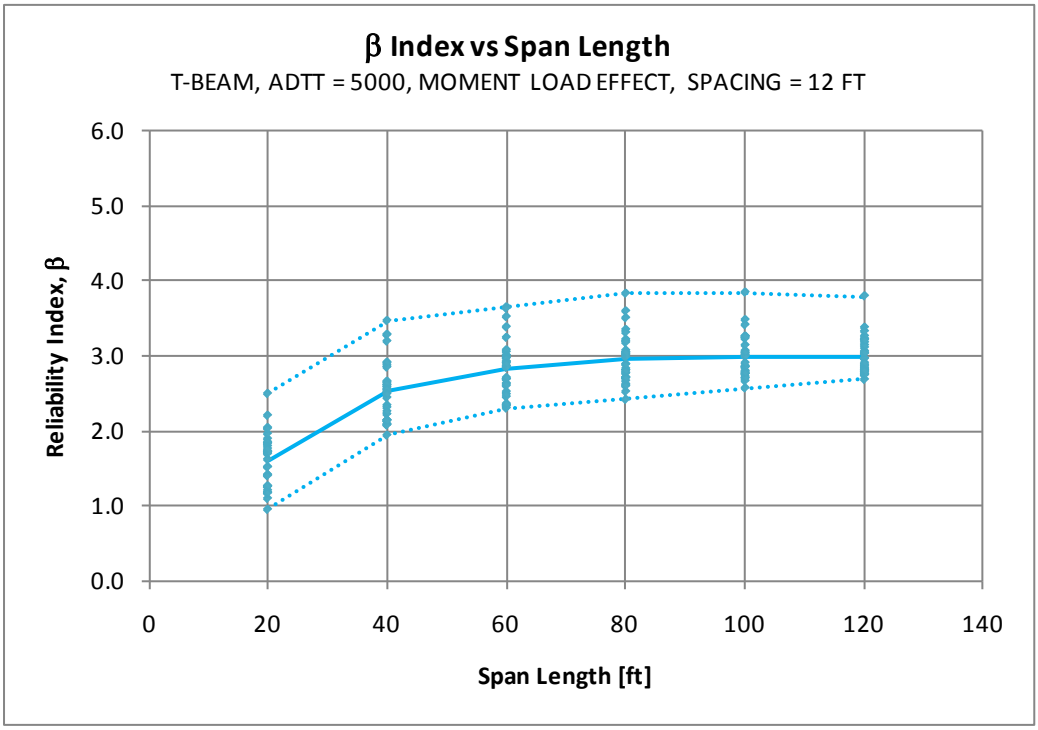
(TWO LANES)

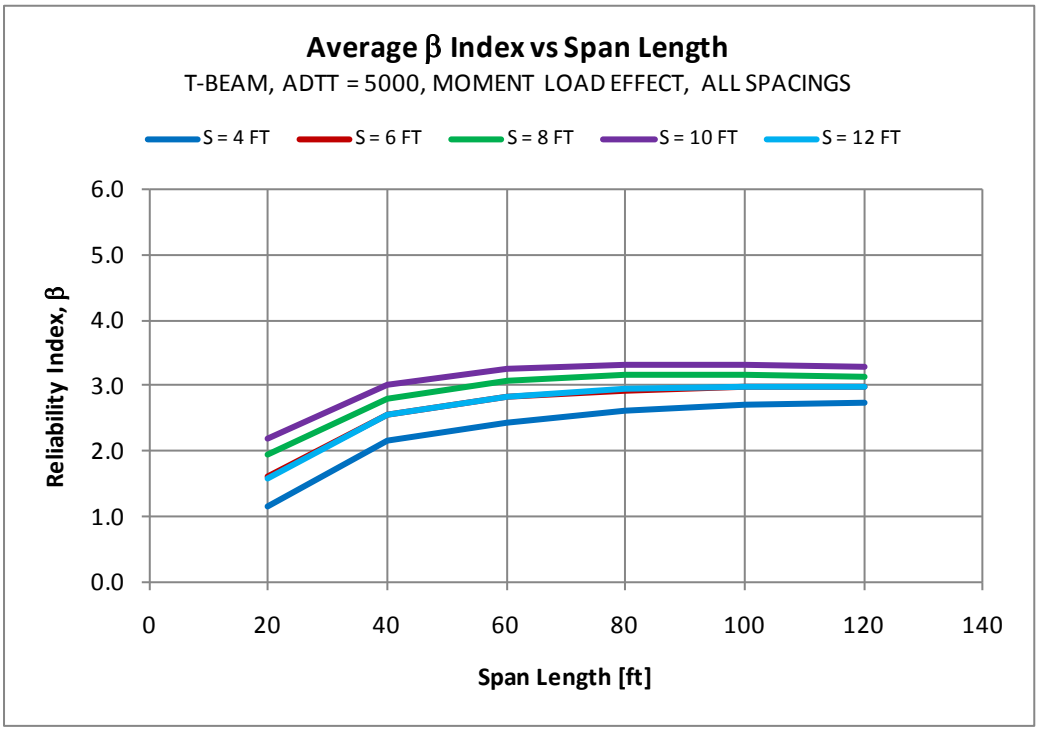

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

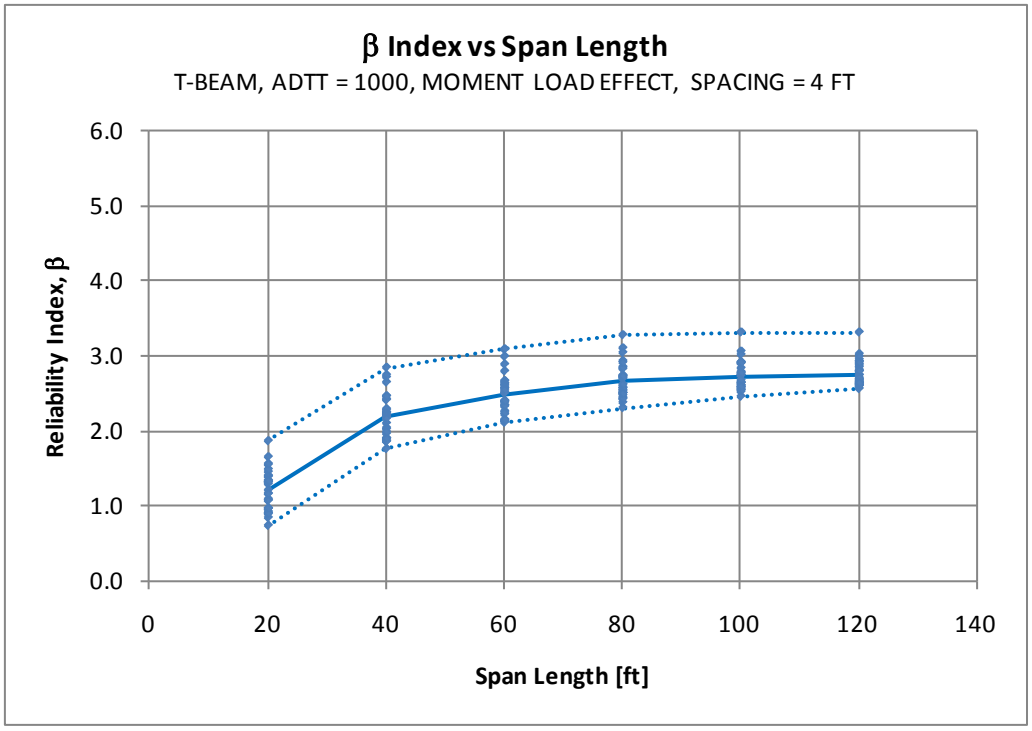

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

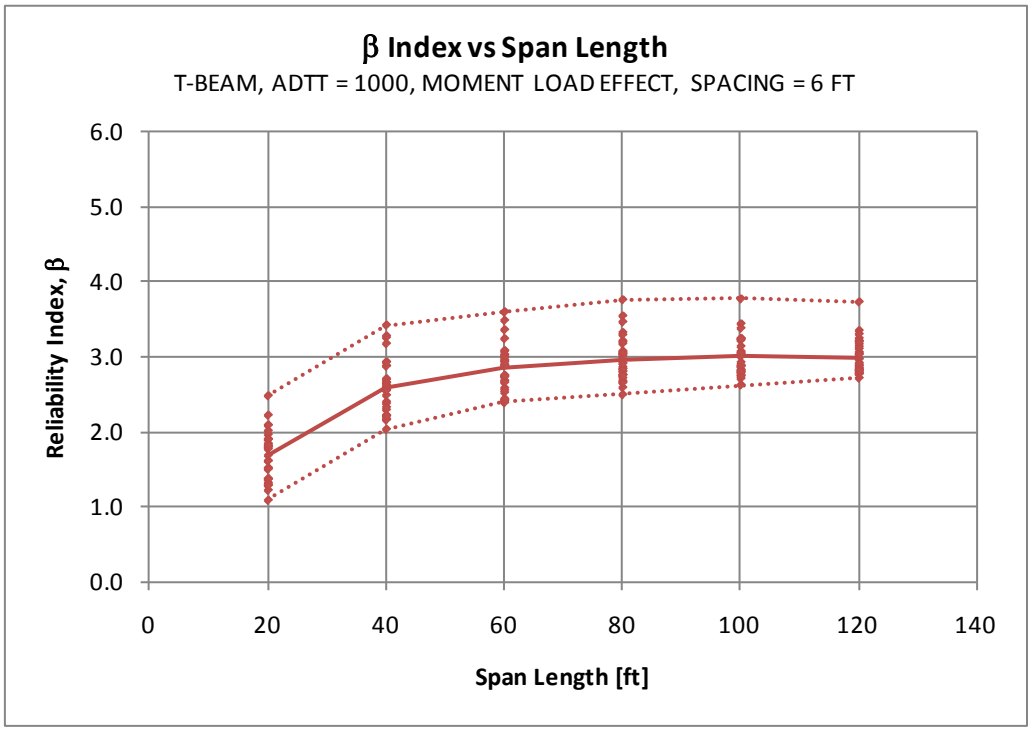

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

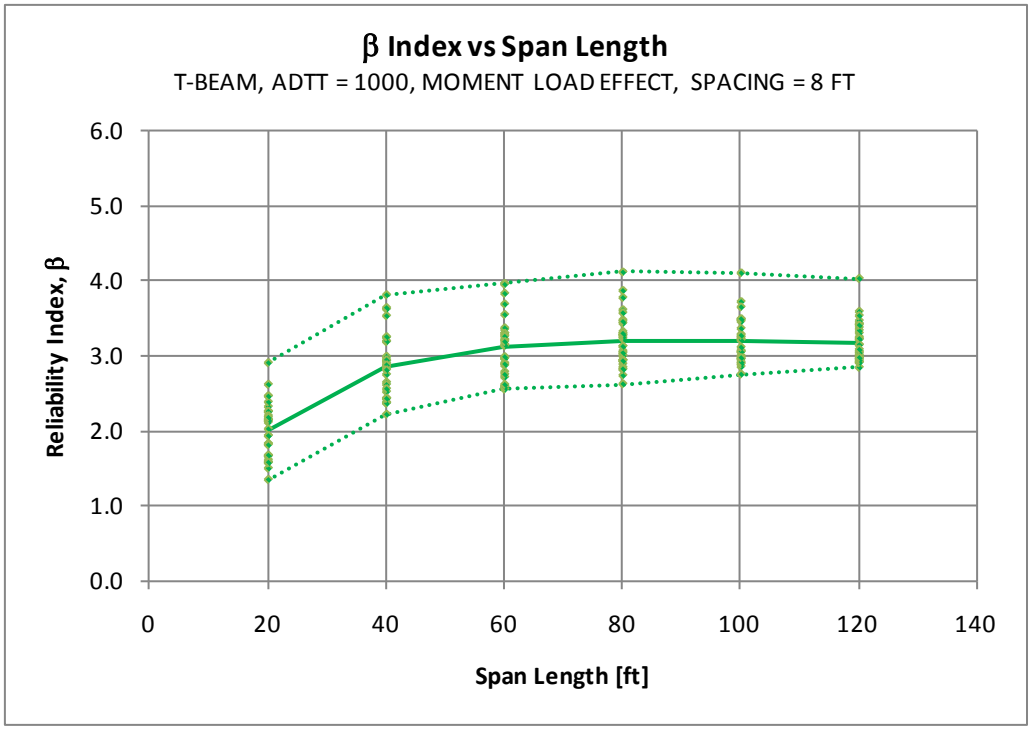

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

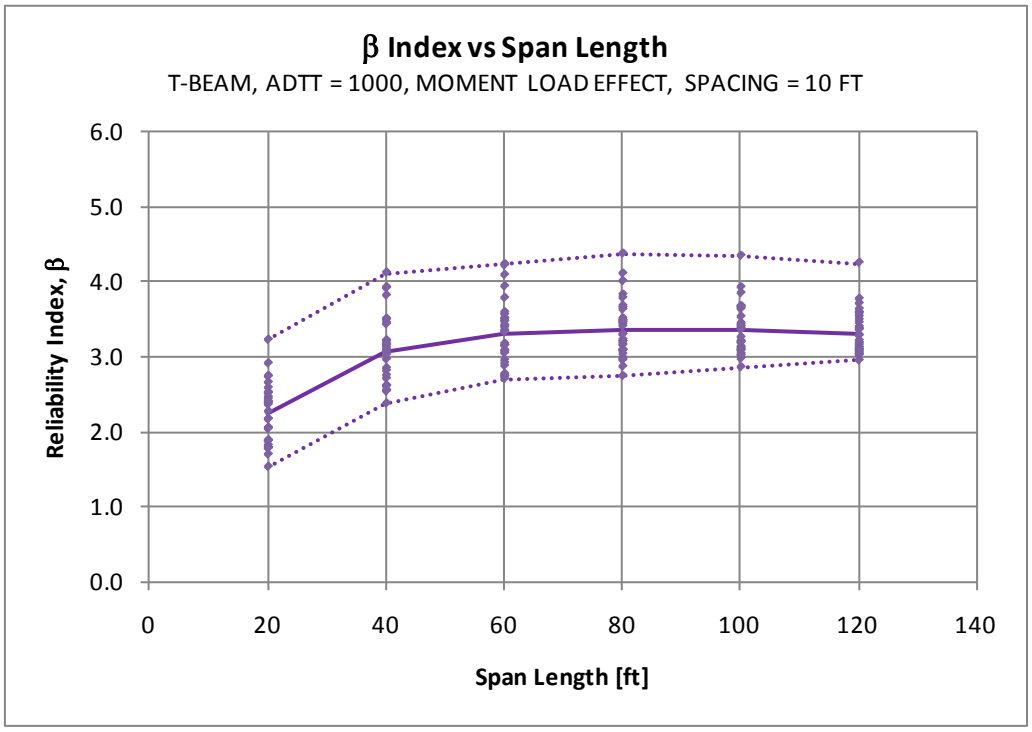

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

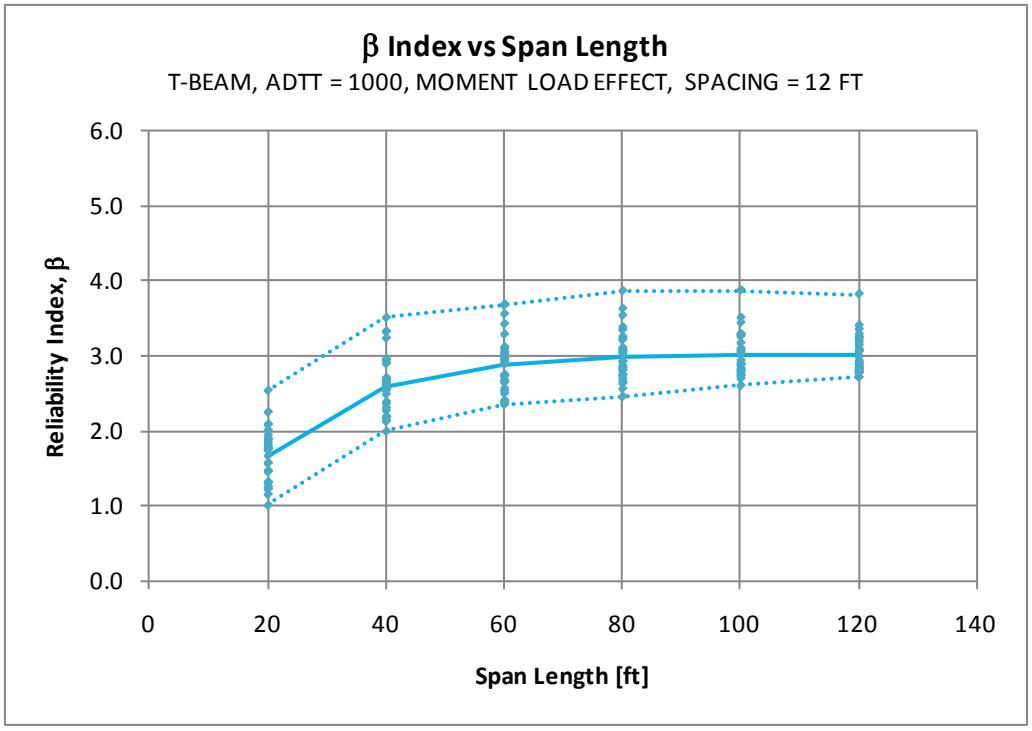

Average β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

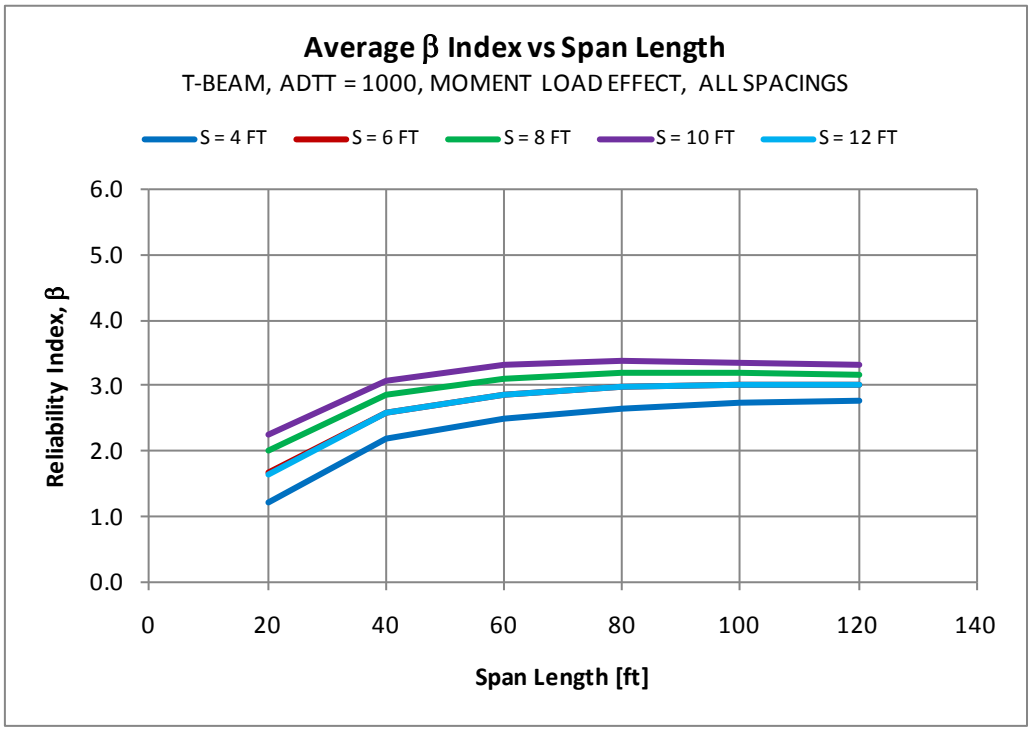

β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

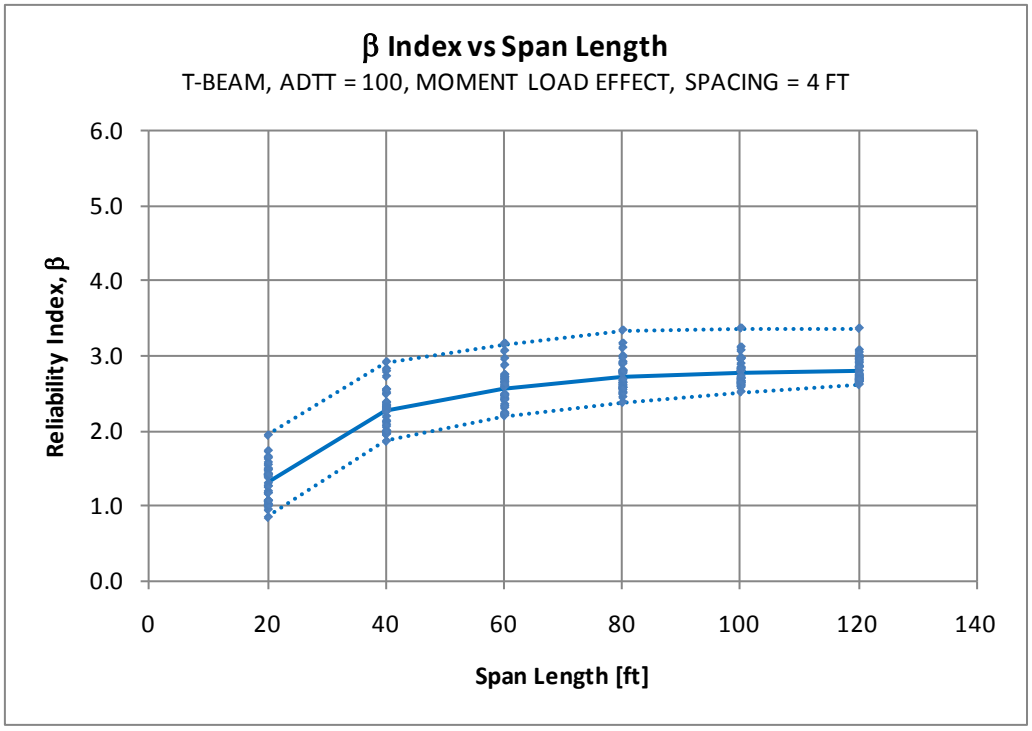

β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

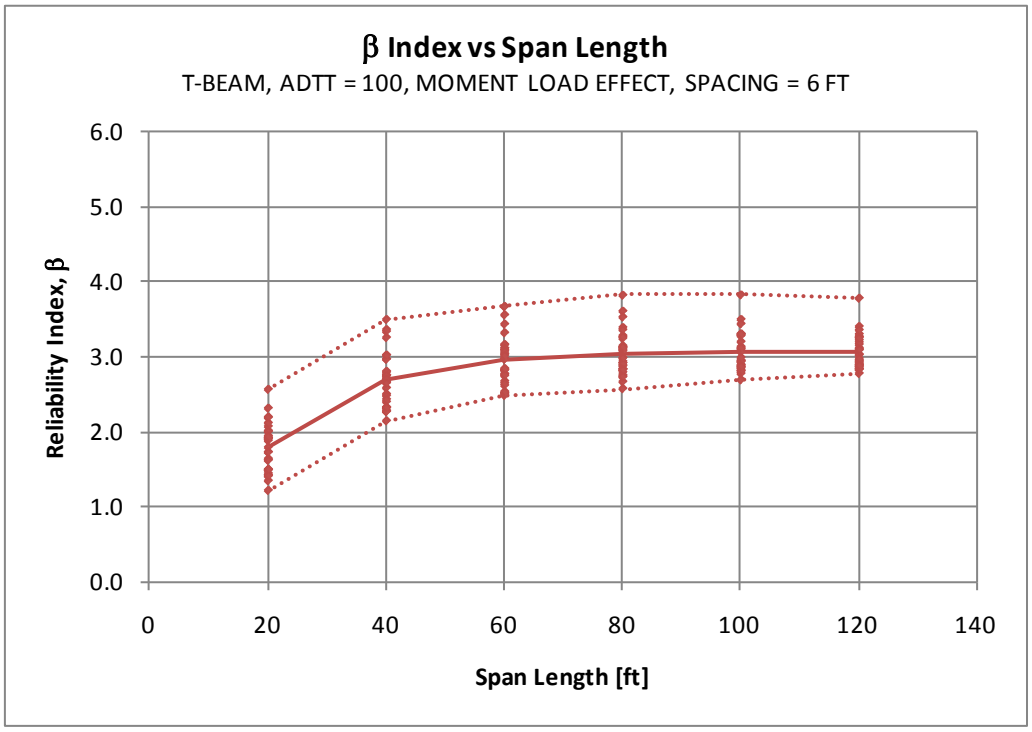

β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

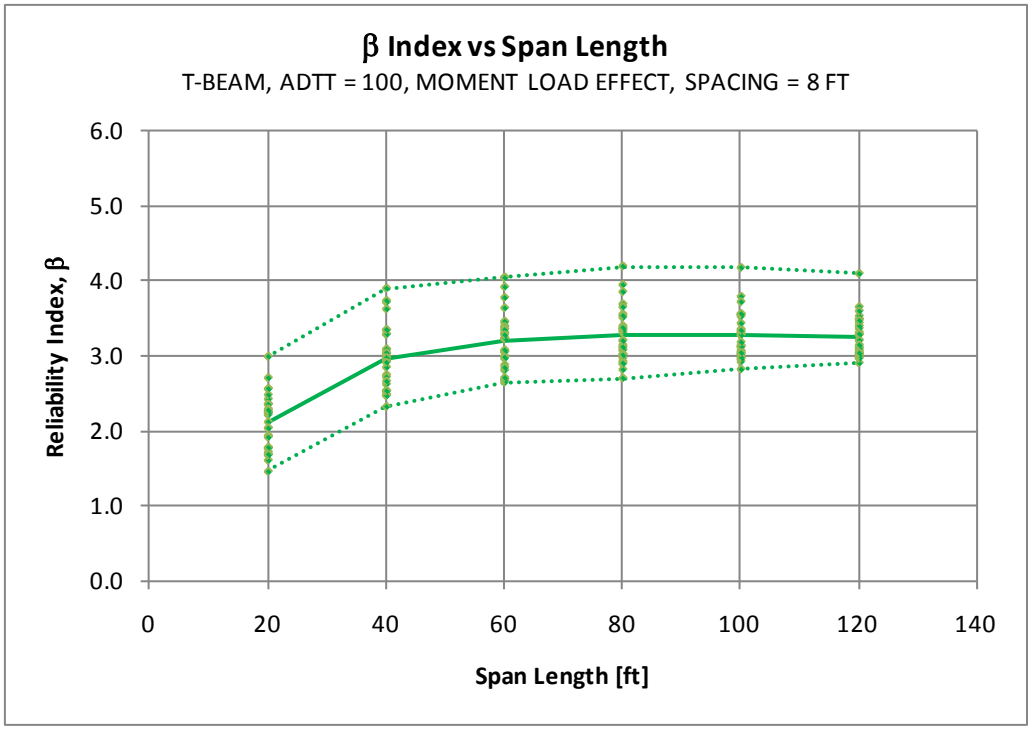

β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

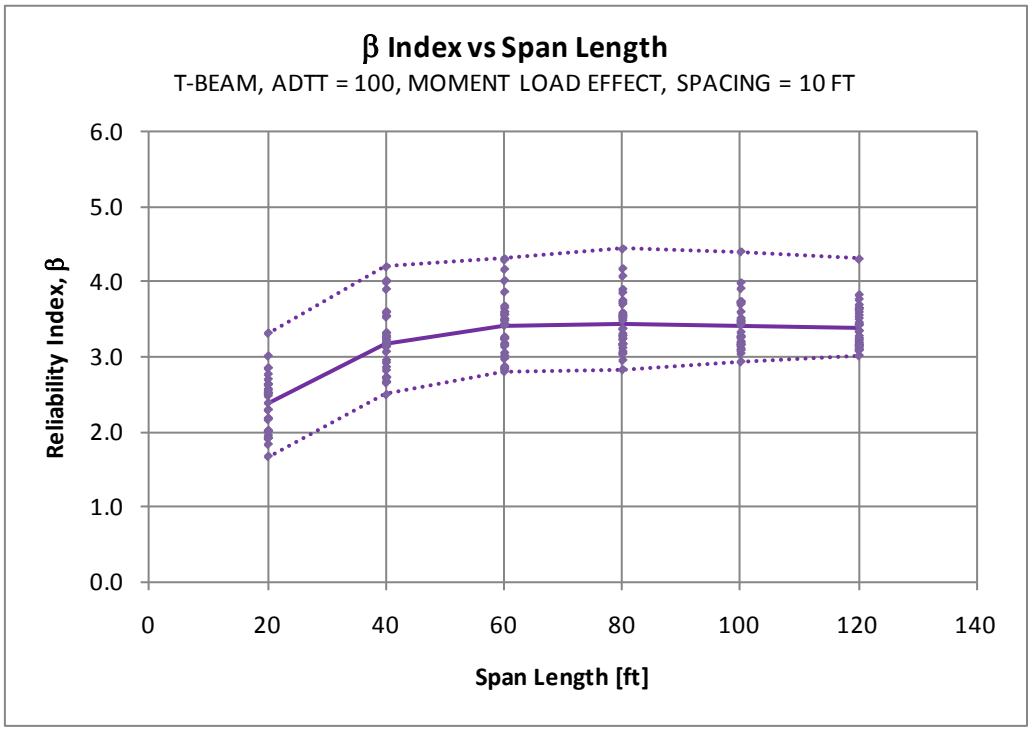

β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

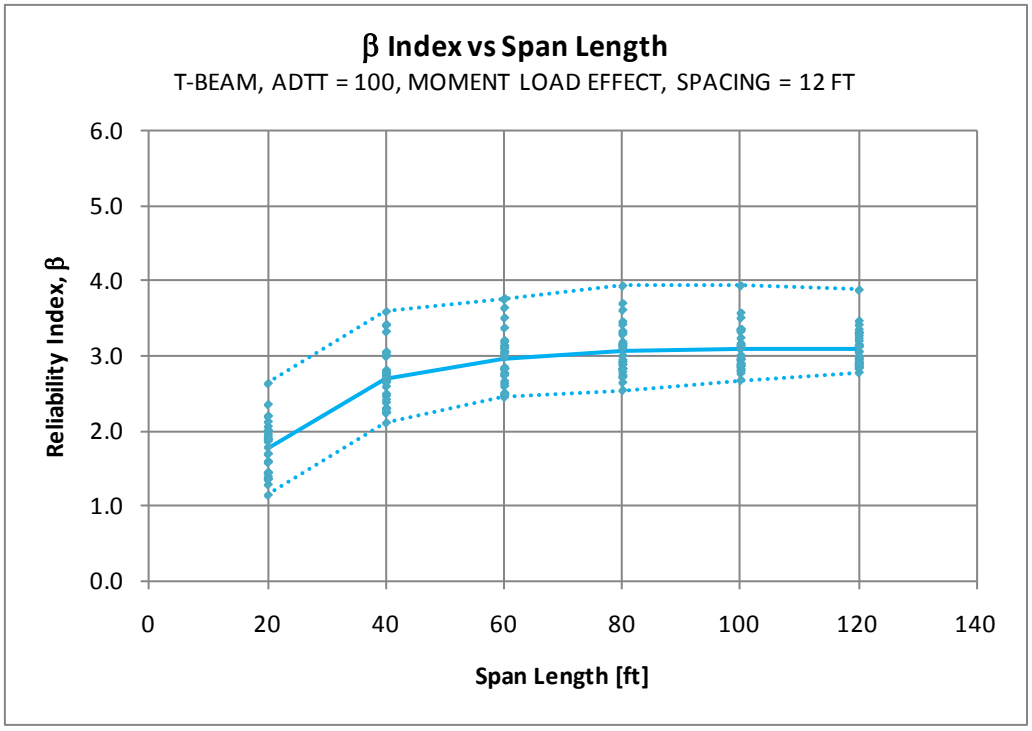

Average β Index vs Span Length
T-BEAM, ADTT = 1000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

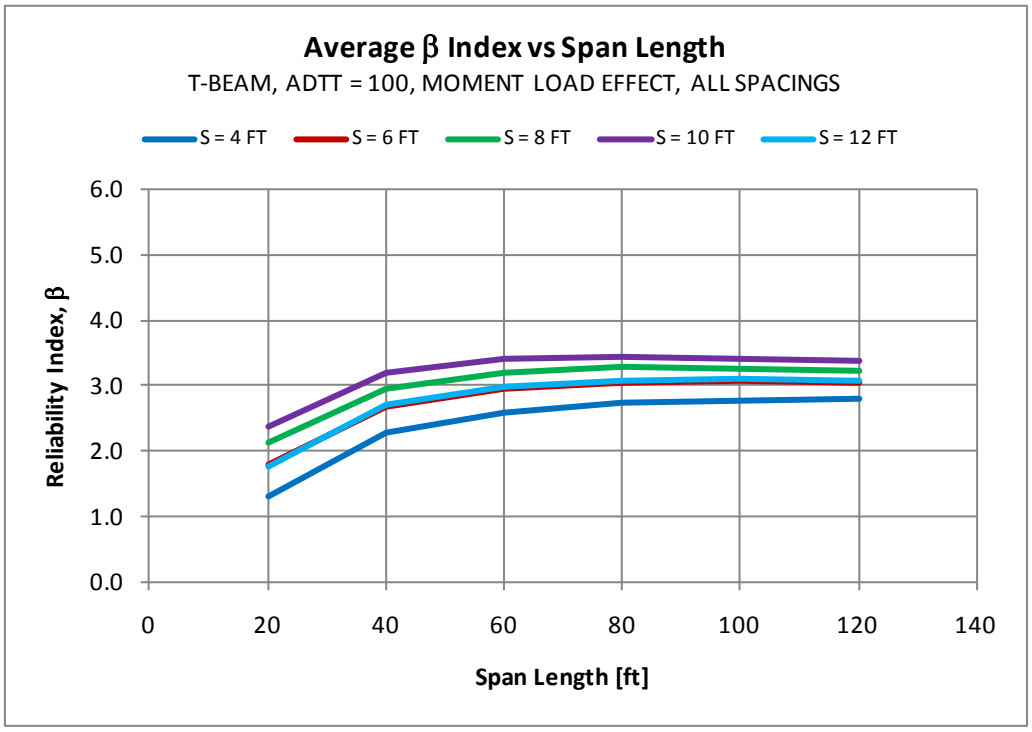

β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

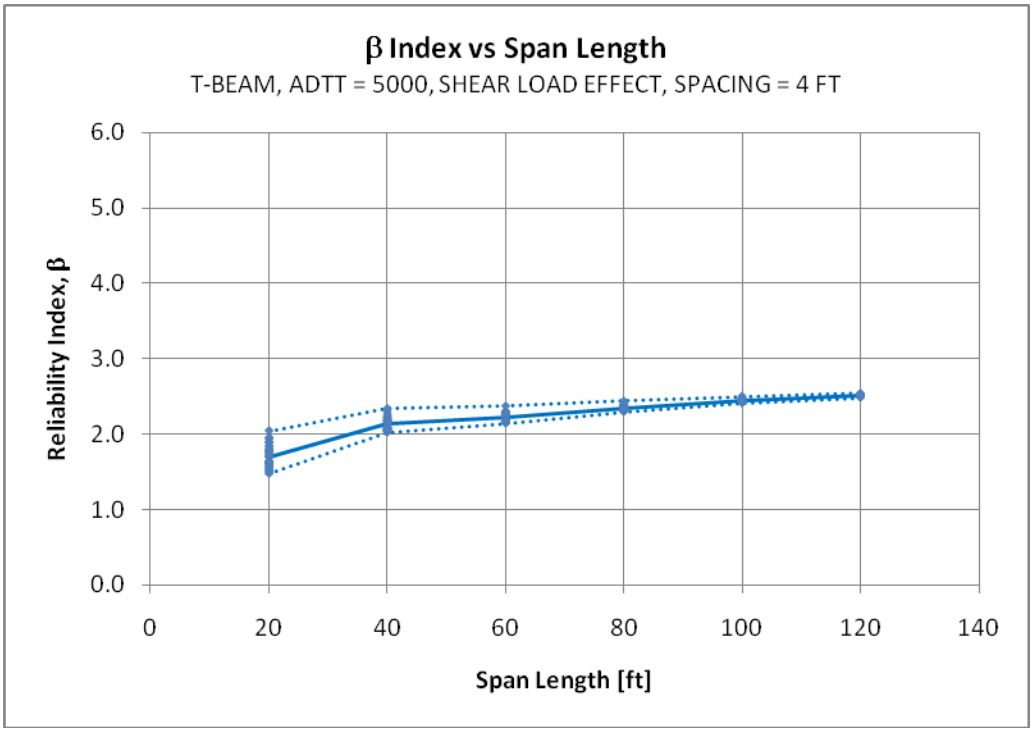

β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

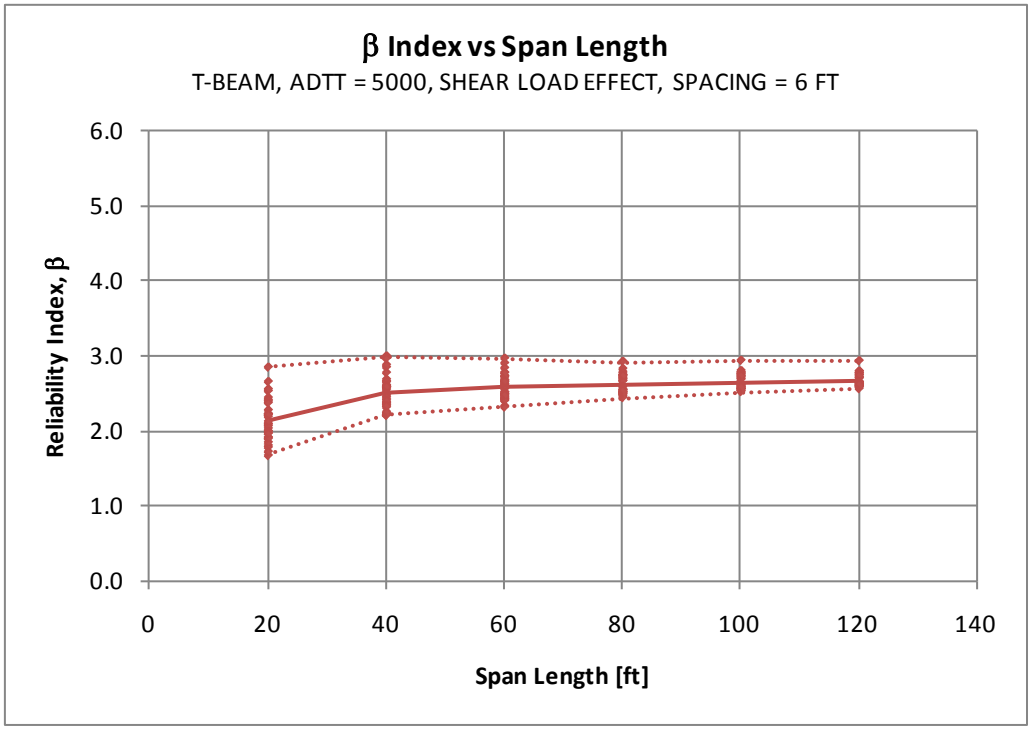

β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

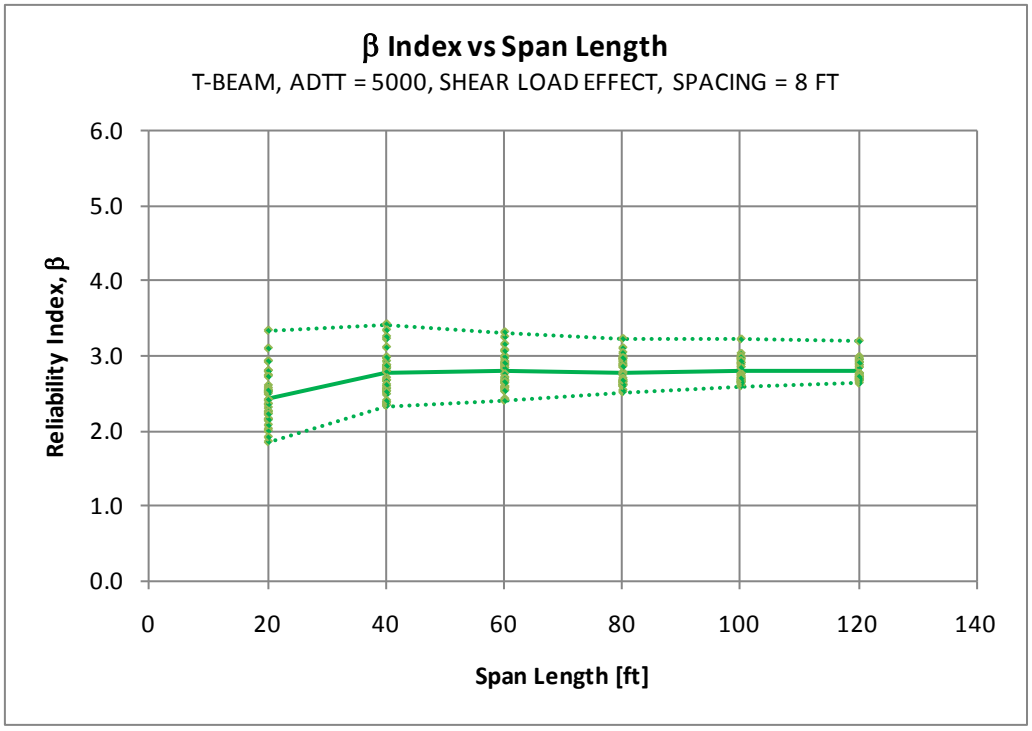

β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

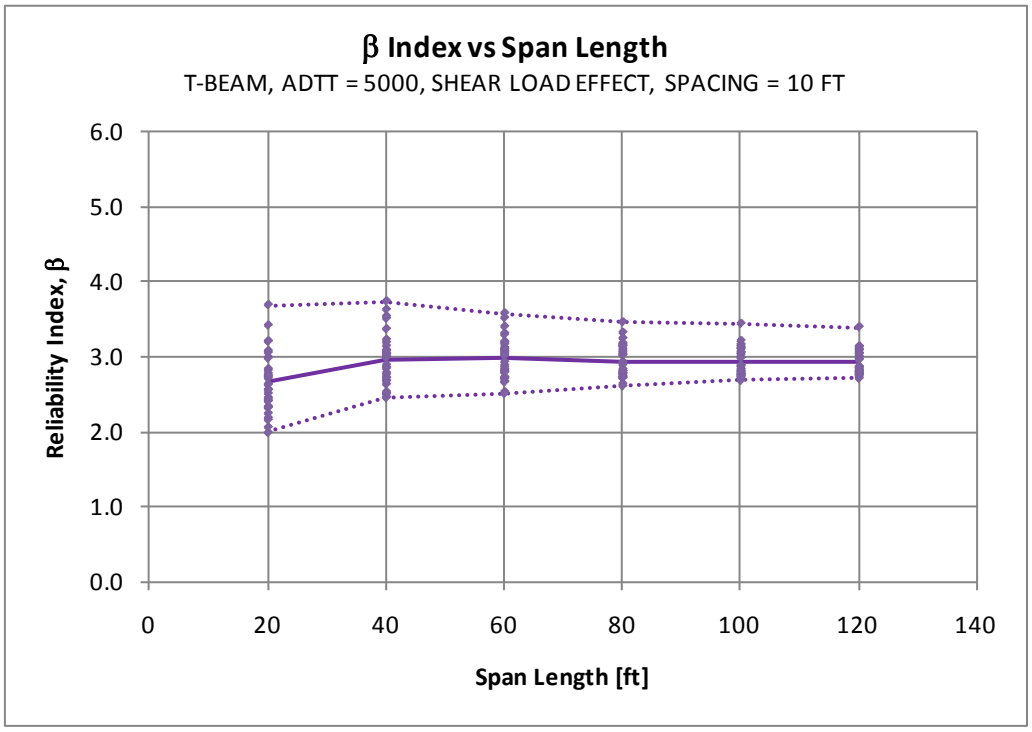

β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

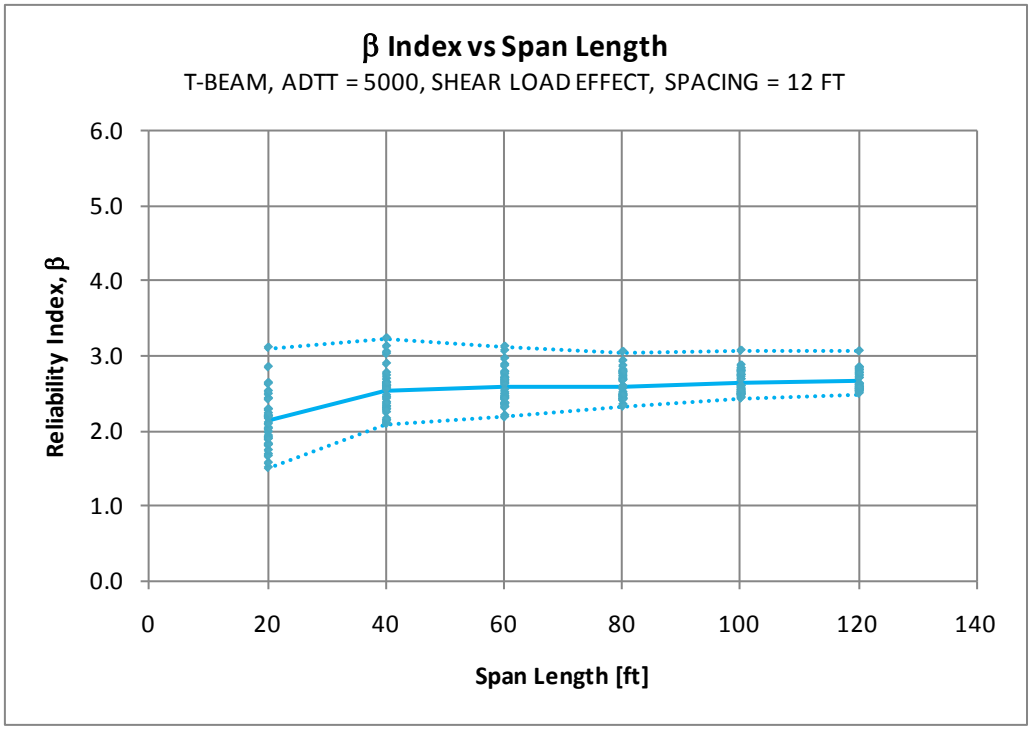

Average β Index vs Span Length
T-BEAM, ADTT = 100, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

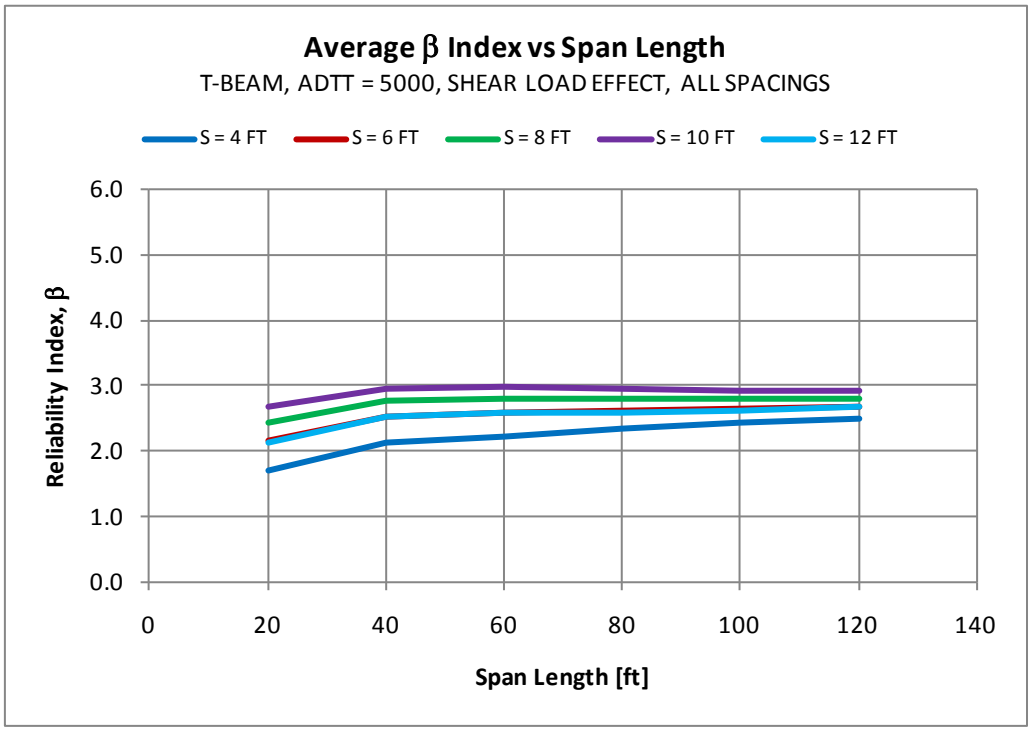

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

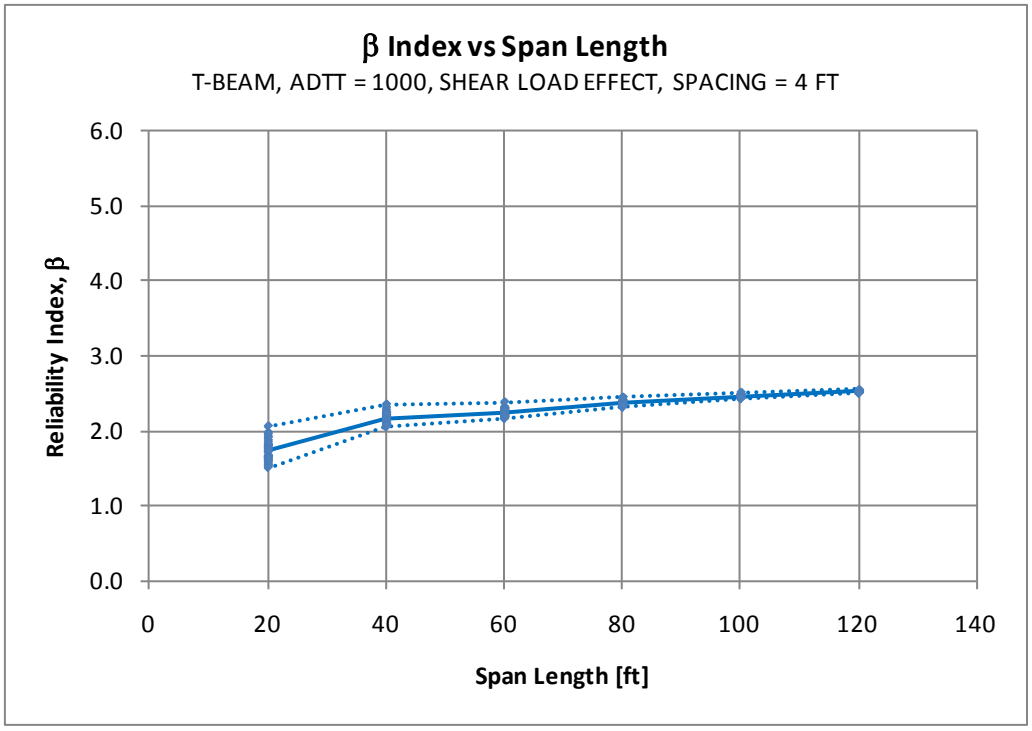

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

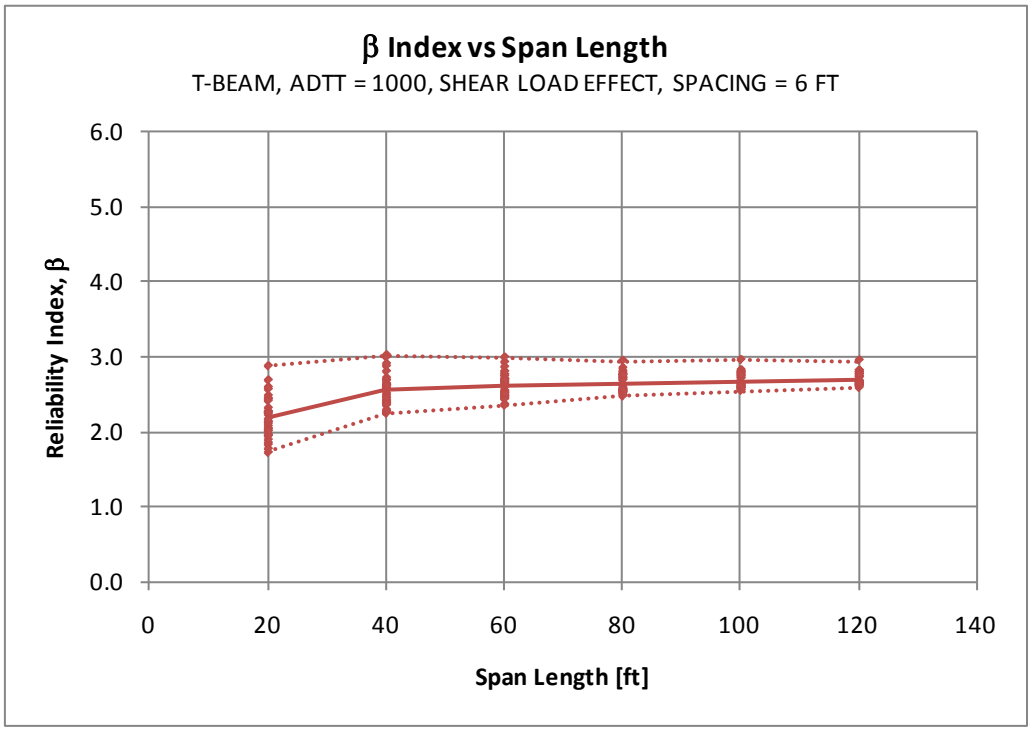

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

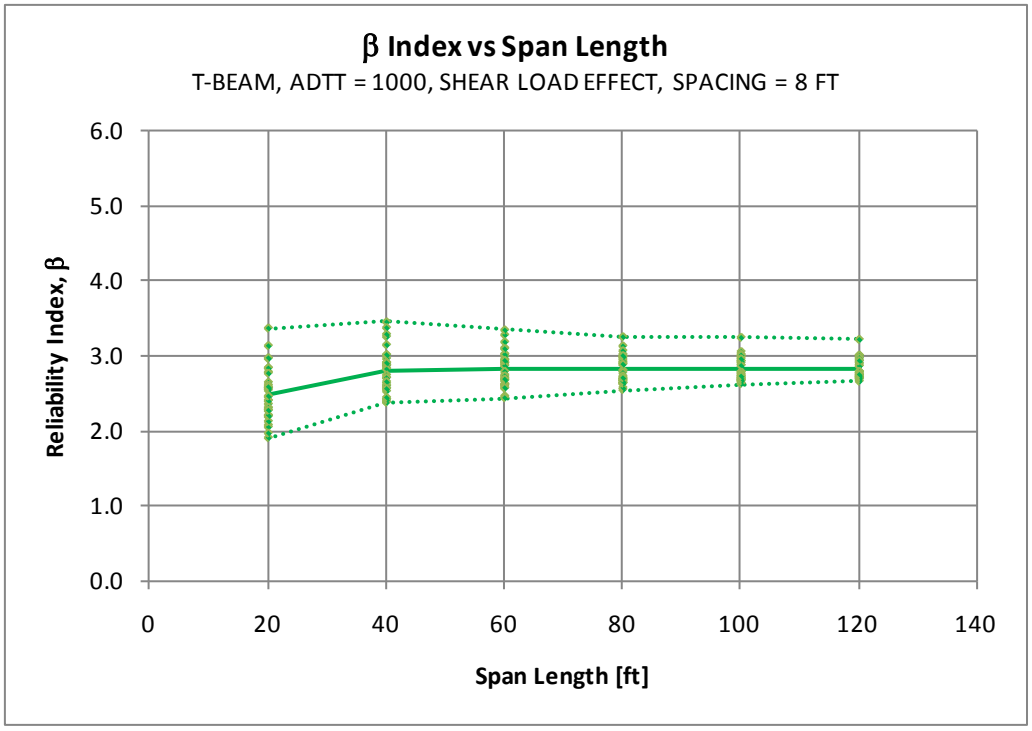

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

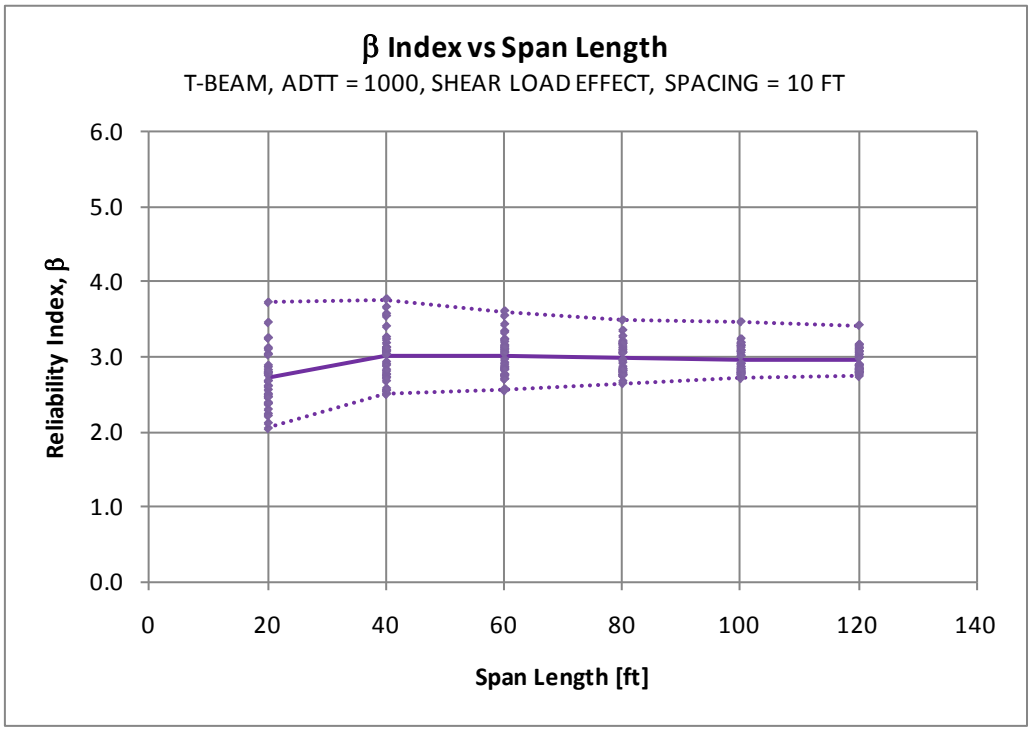

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

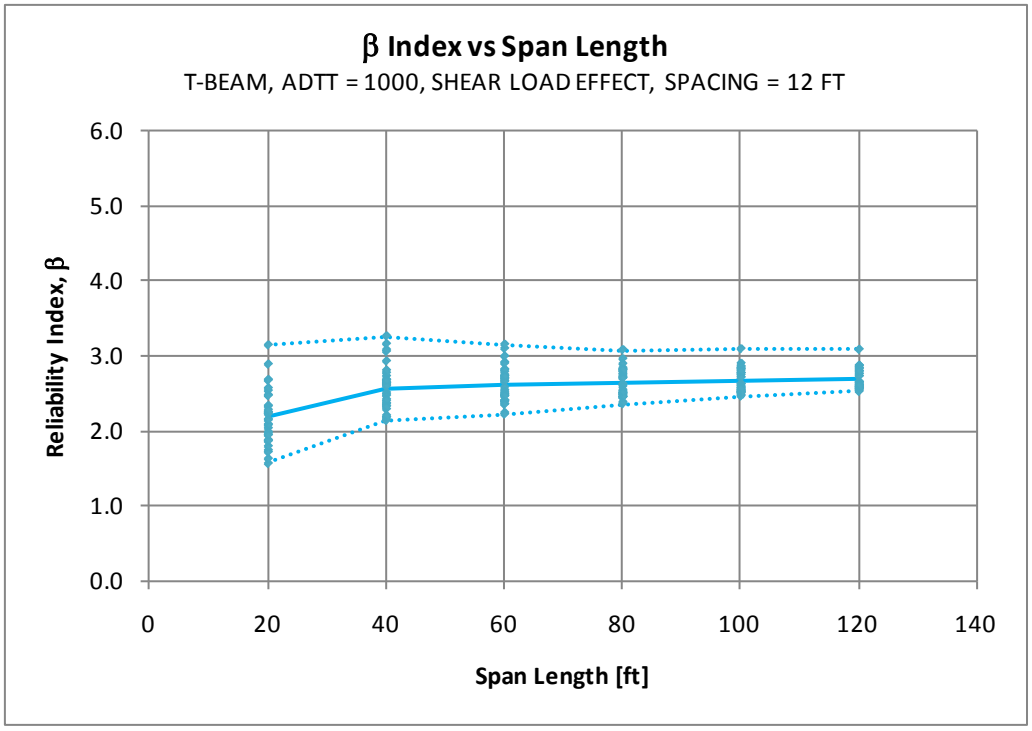

Average β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

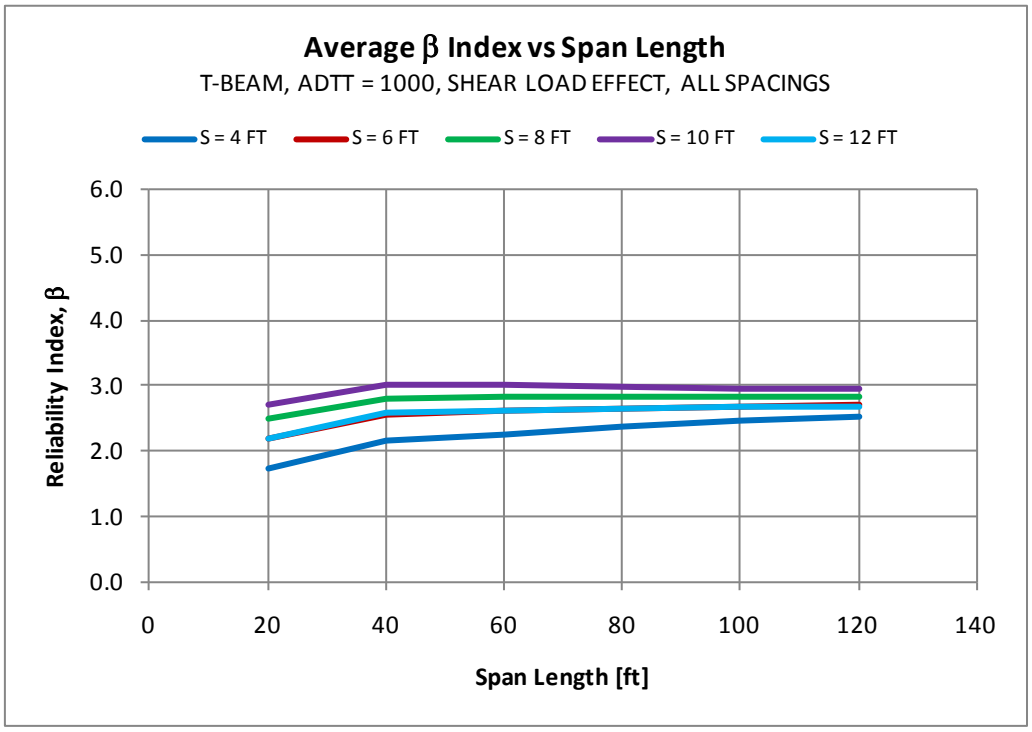

β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

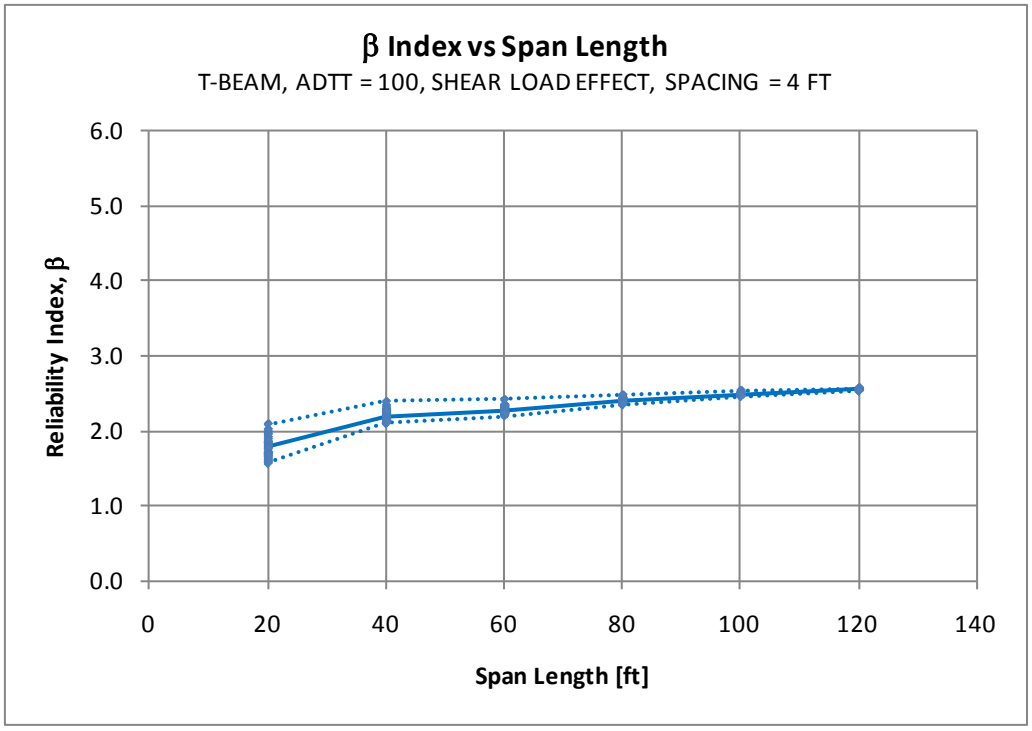

β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

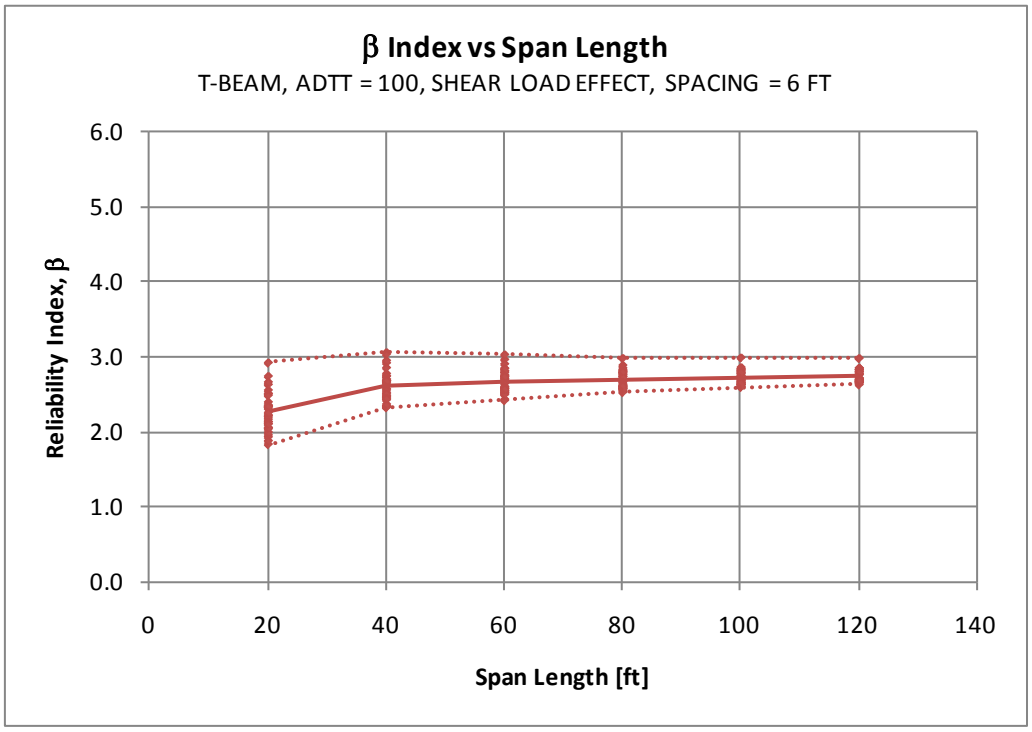

β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

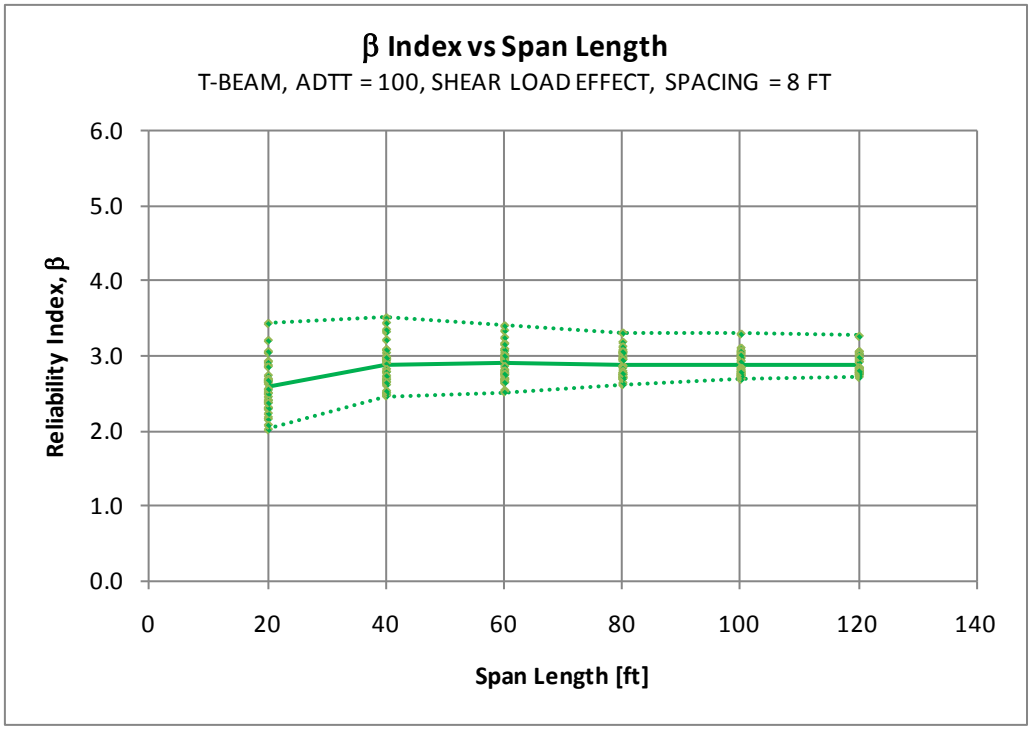

β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

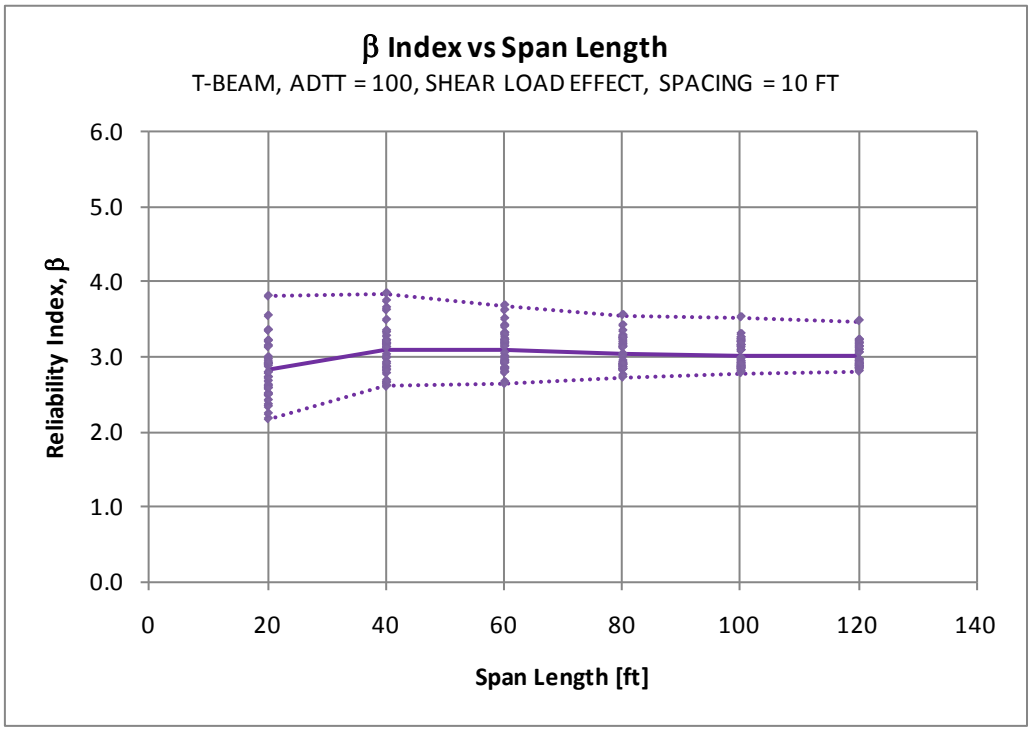

β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

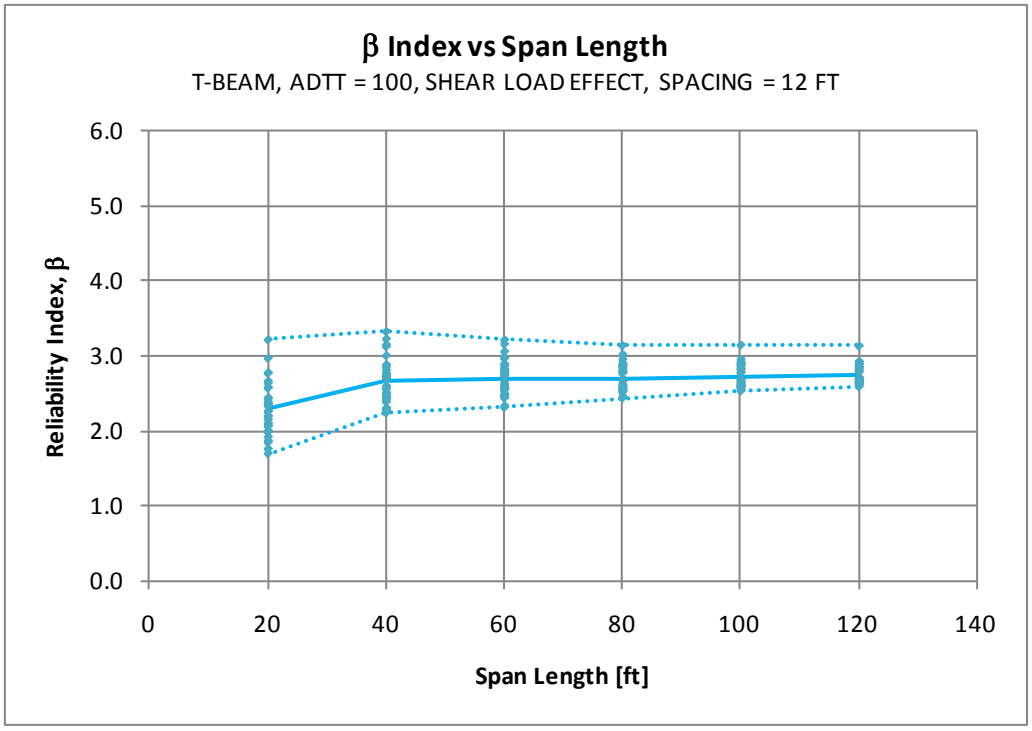

Average β Index vs Span Length
T-BEAM, ADTT = 1000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

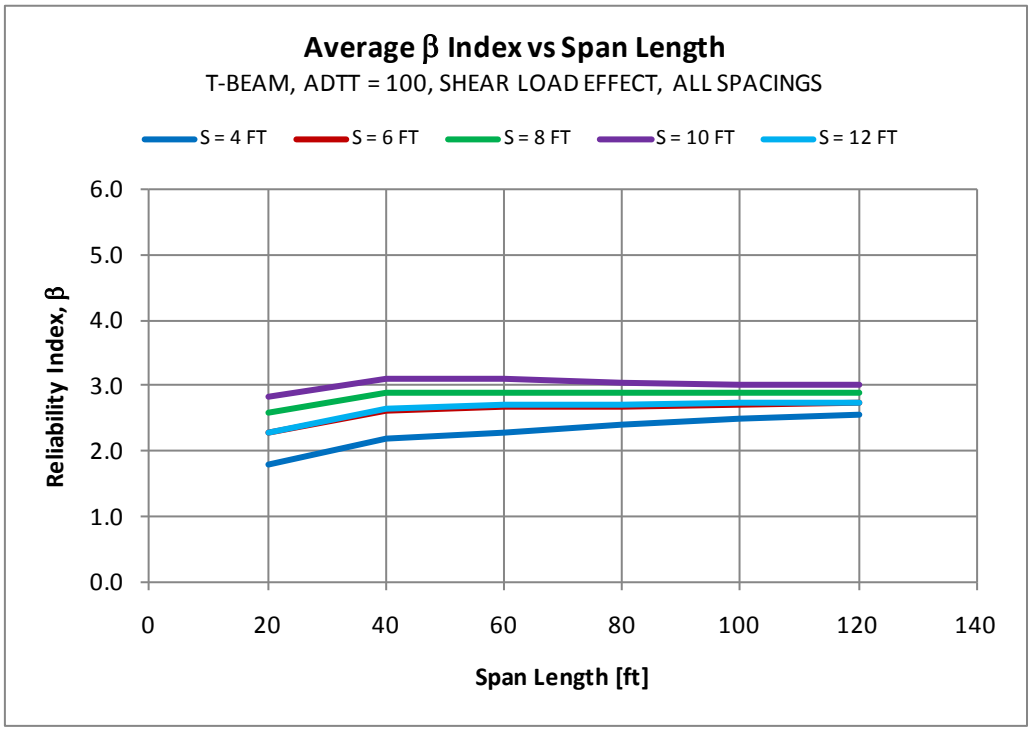

β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

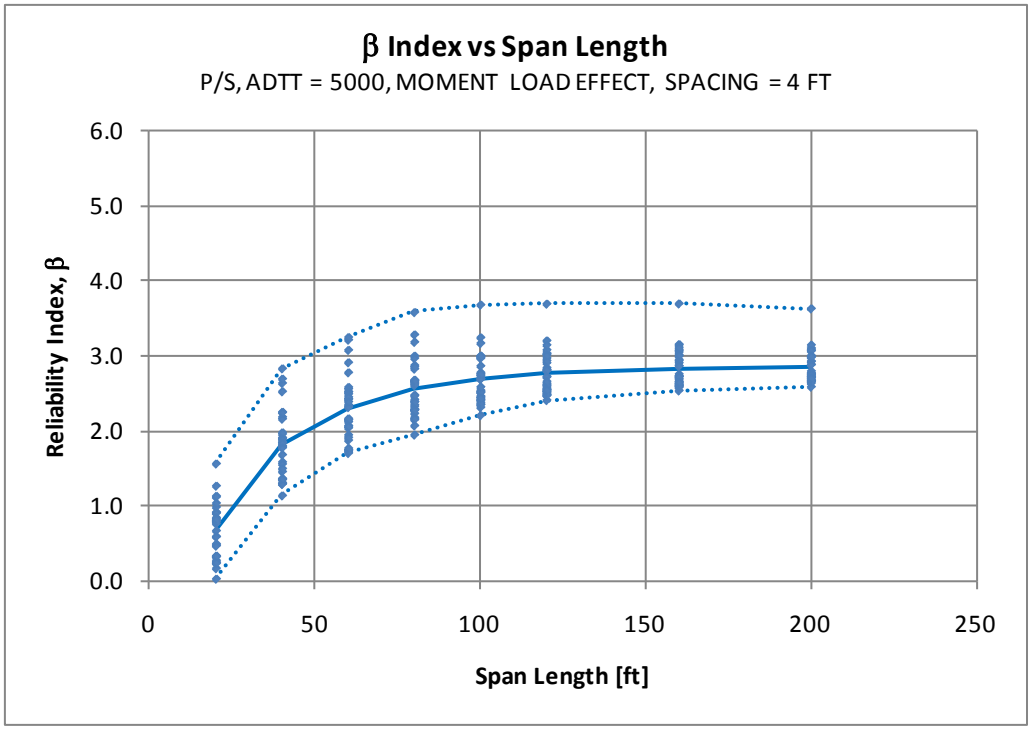

β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

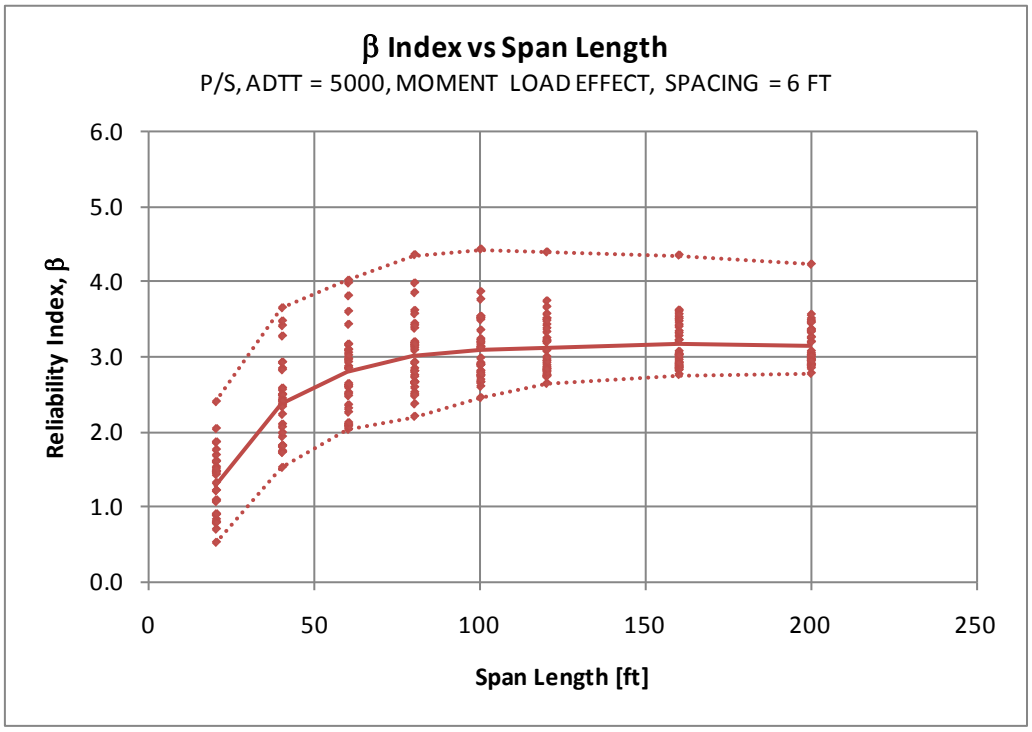

Average β Index vs Span Length
T-BEAM, ADTT = 100, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

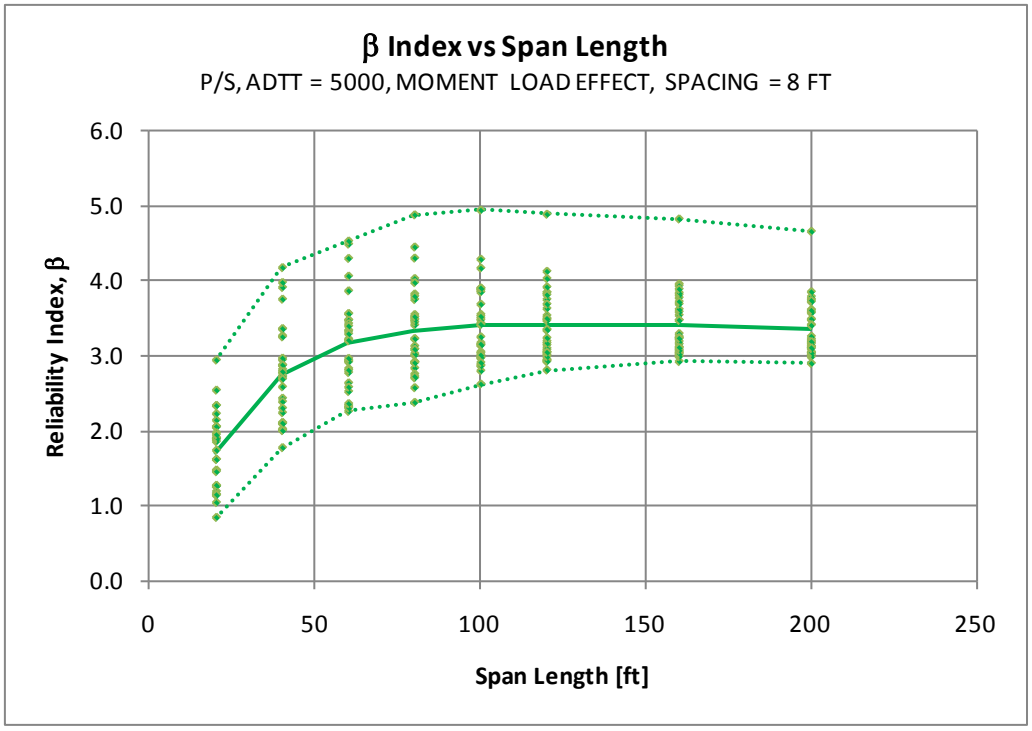
PRESTRESSED CONCRETE BEAM

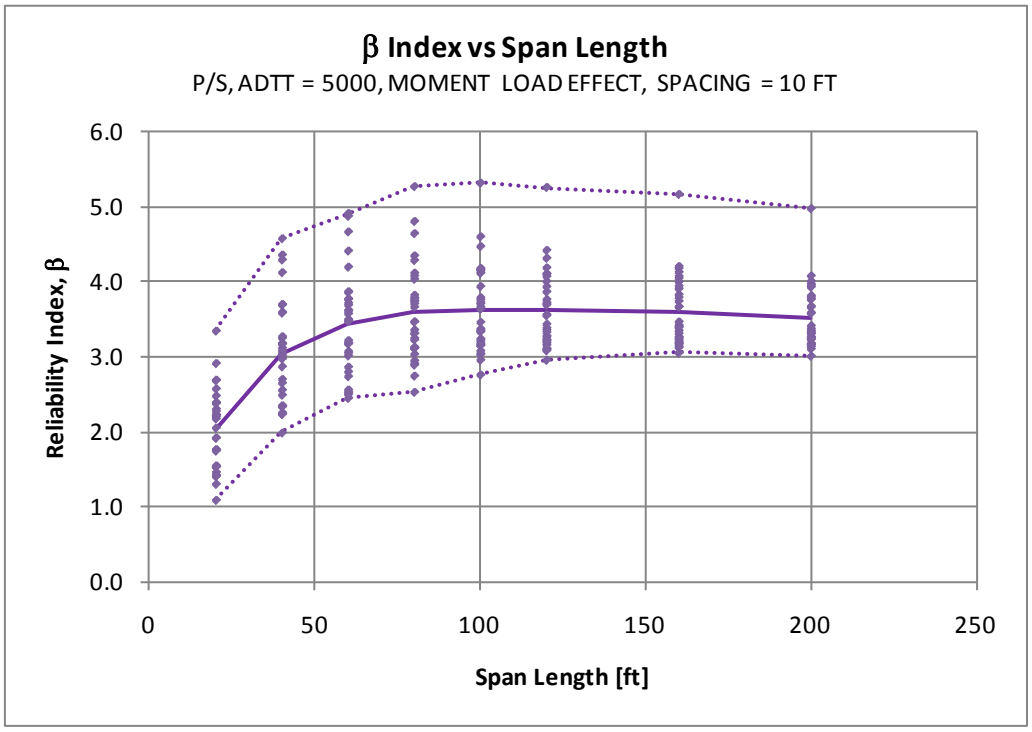
PERMIT PLOTS

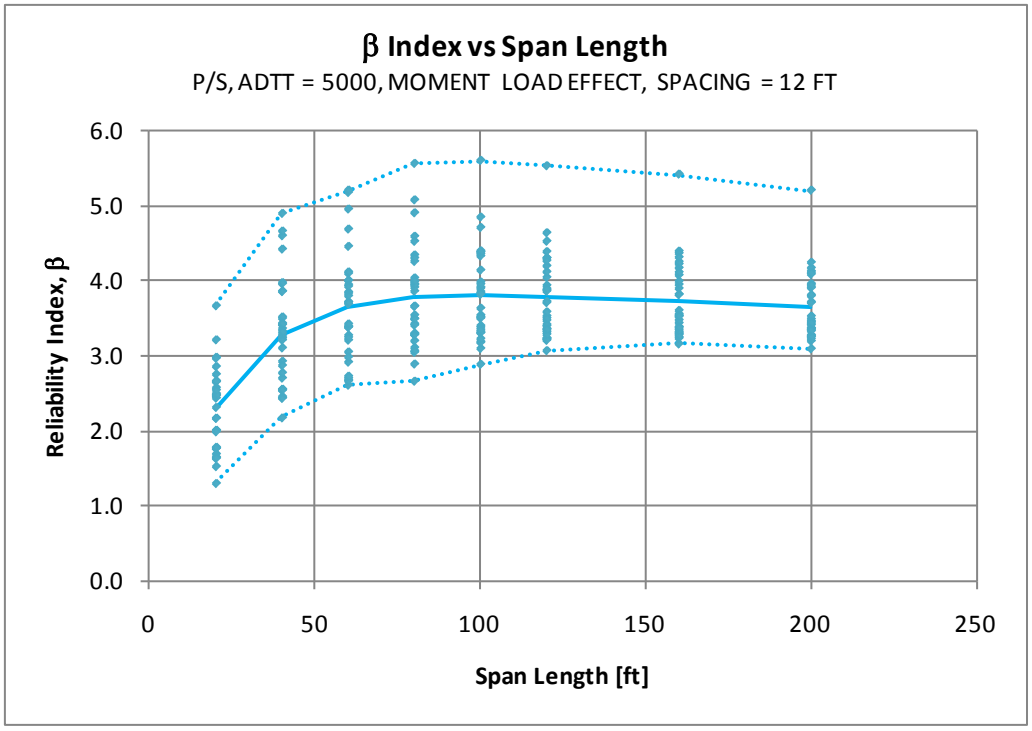
(TWO LANES)

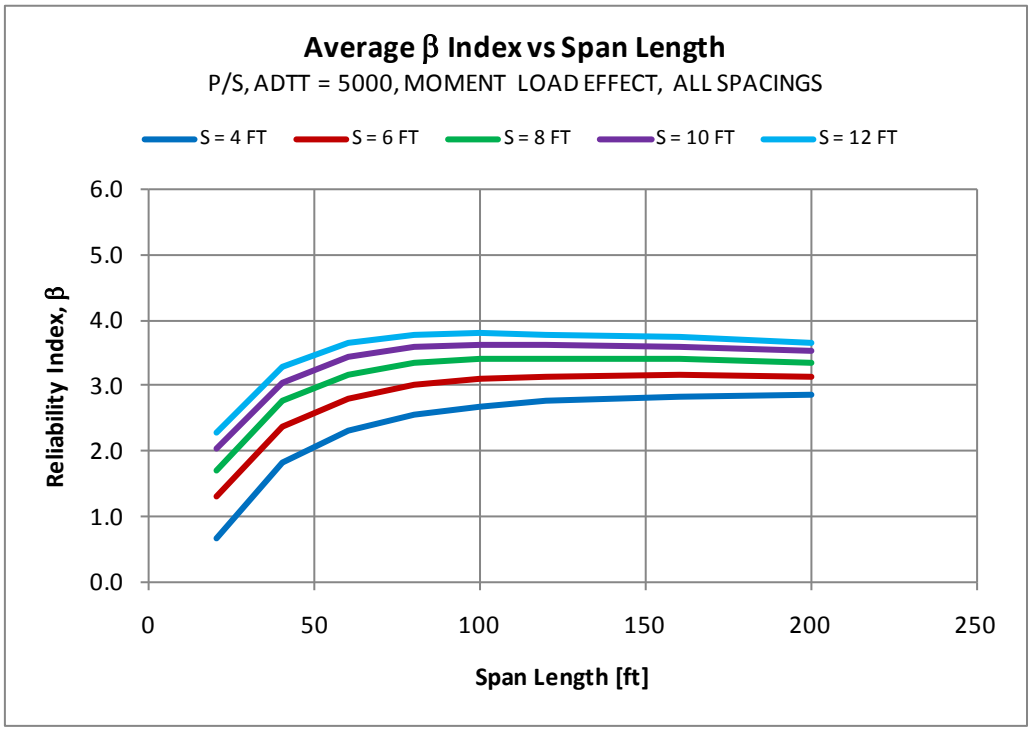

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

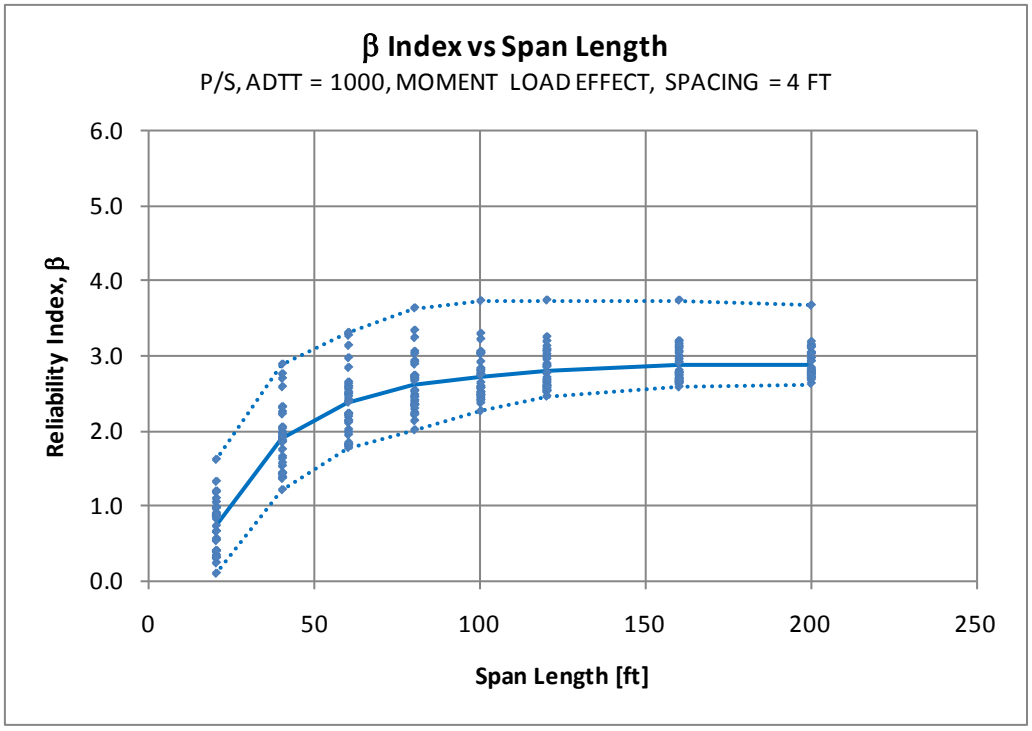

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

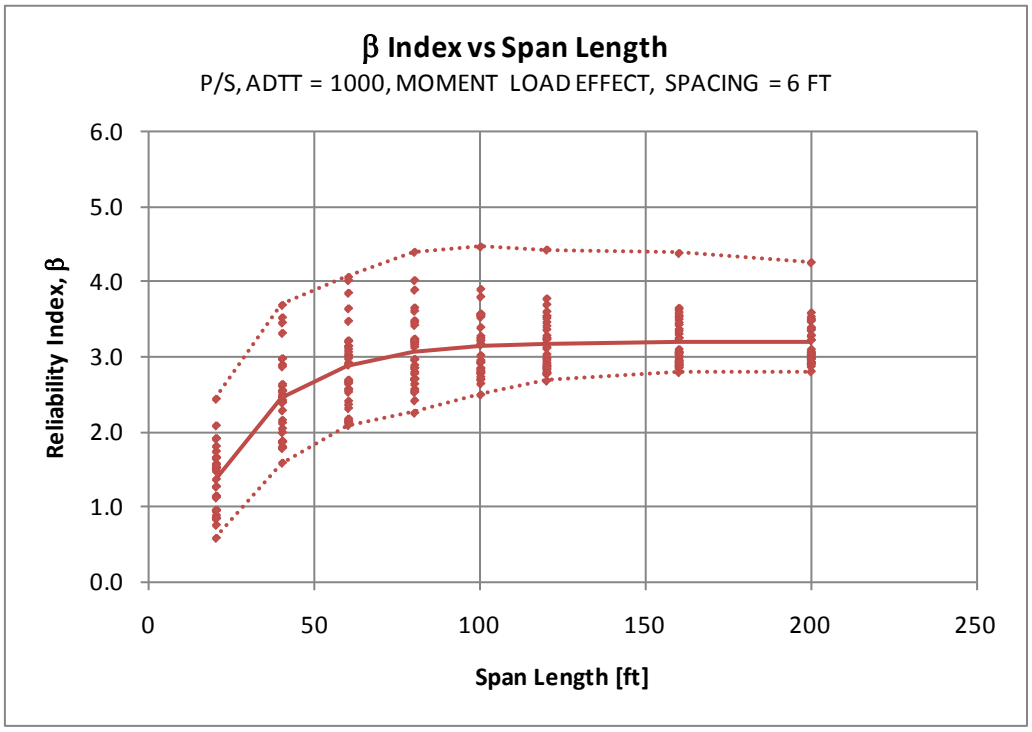

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

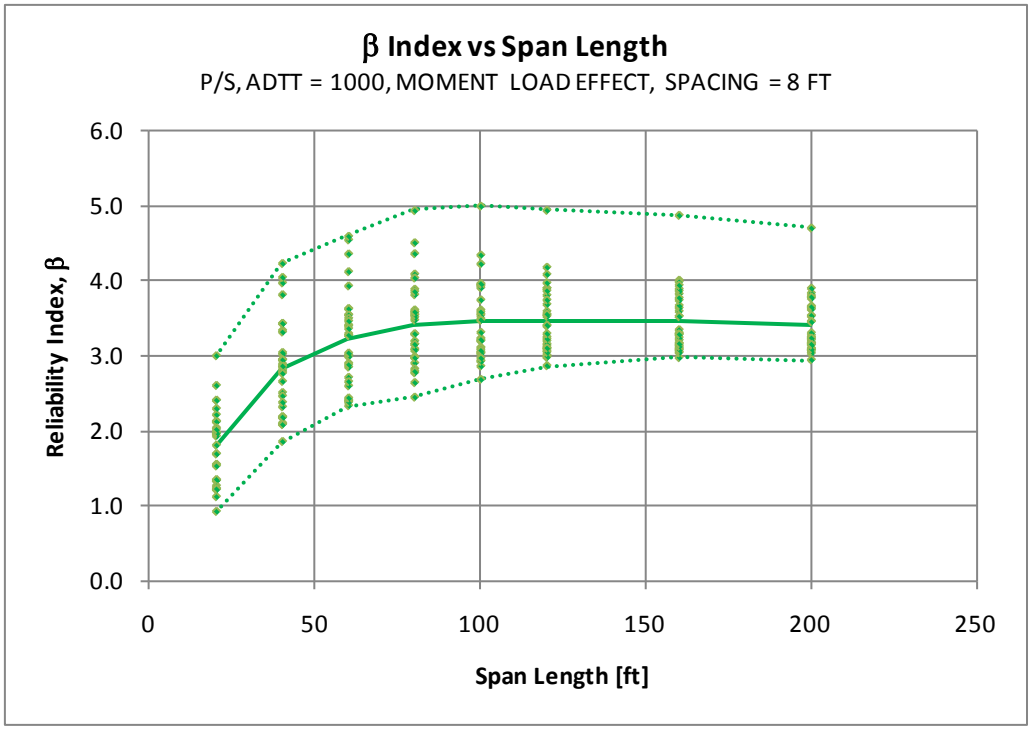

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

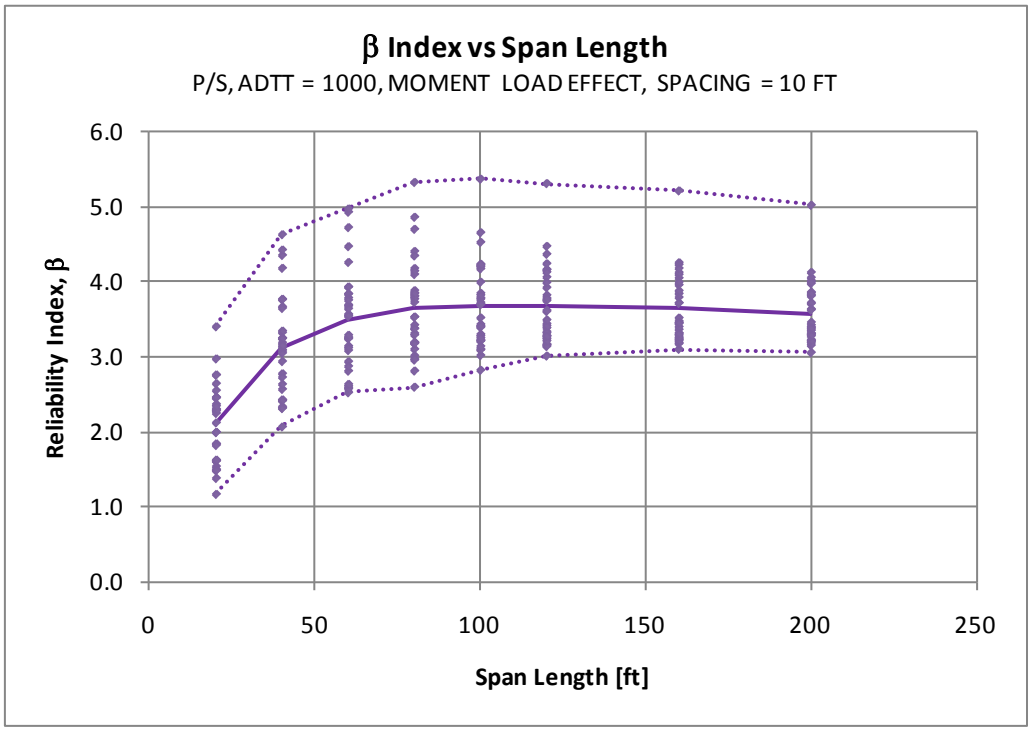

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

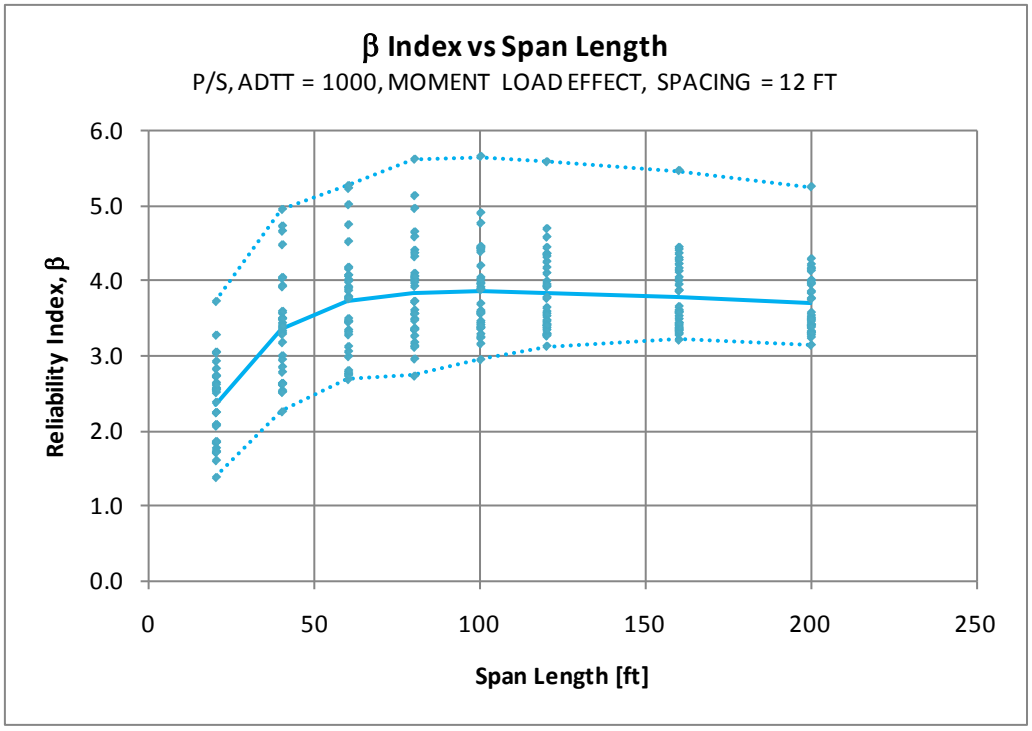

Average β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

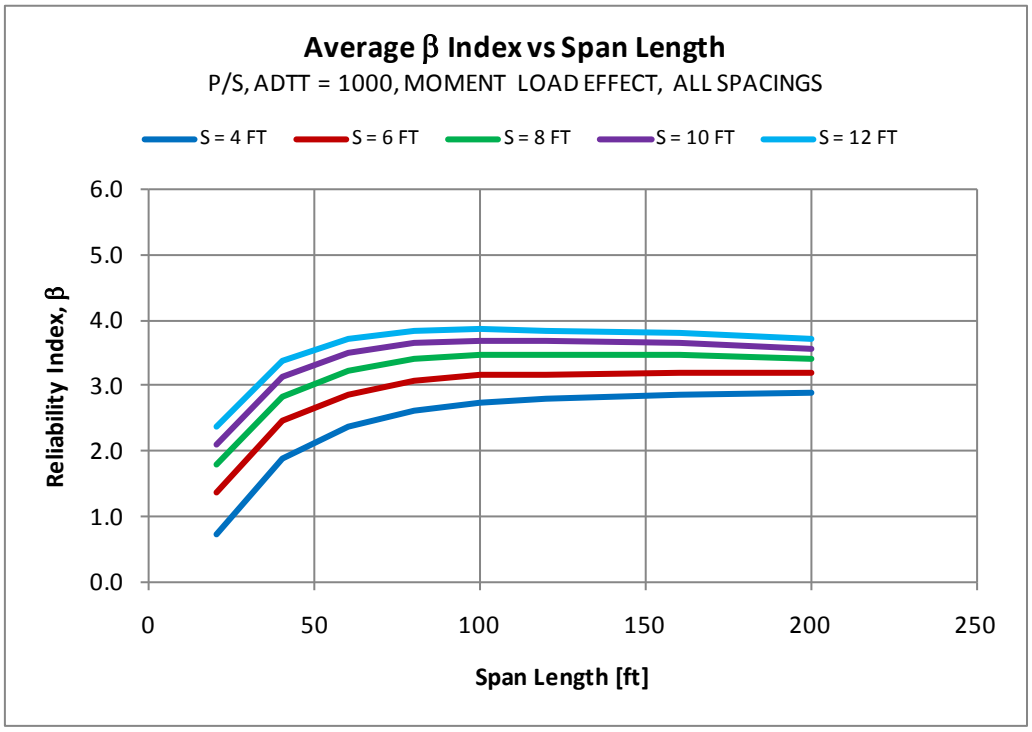

β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

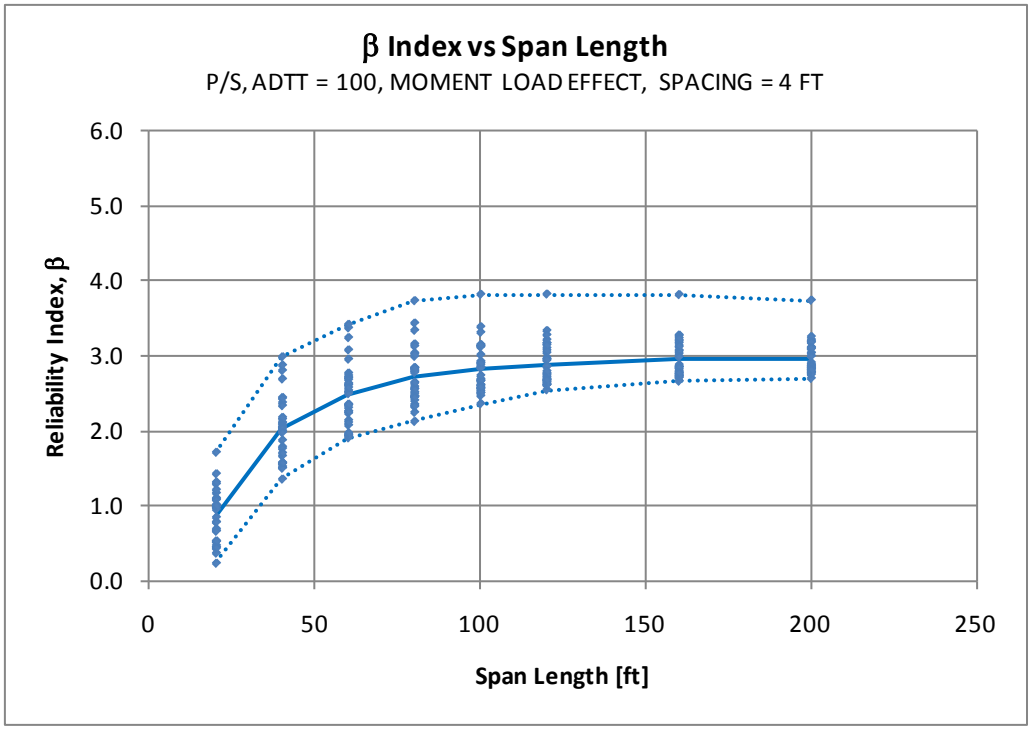

β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

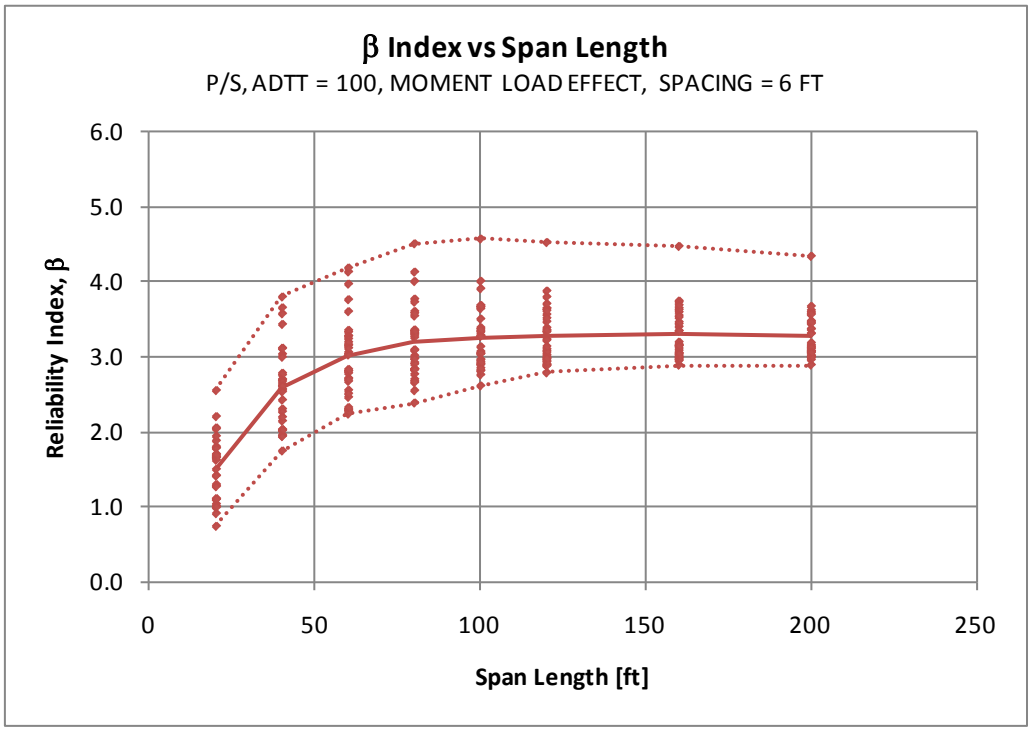

β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

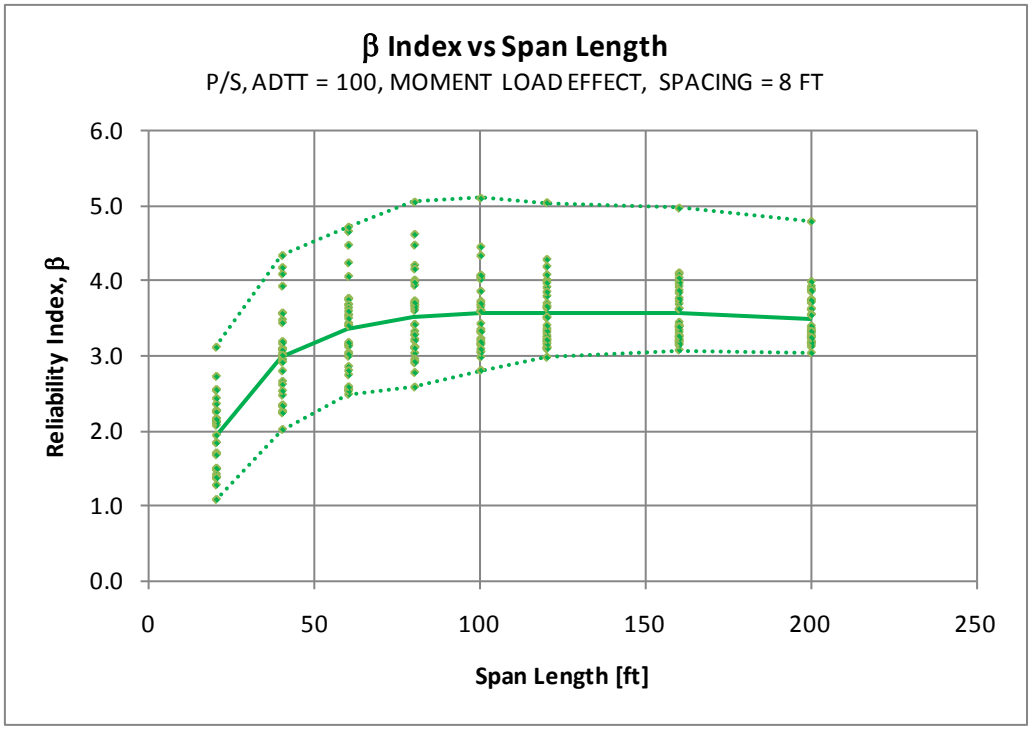

β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

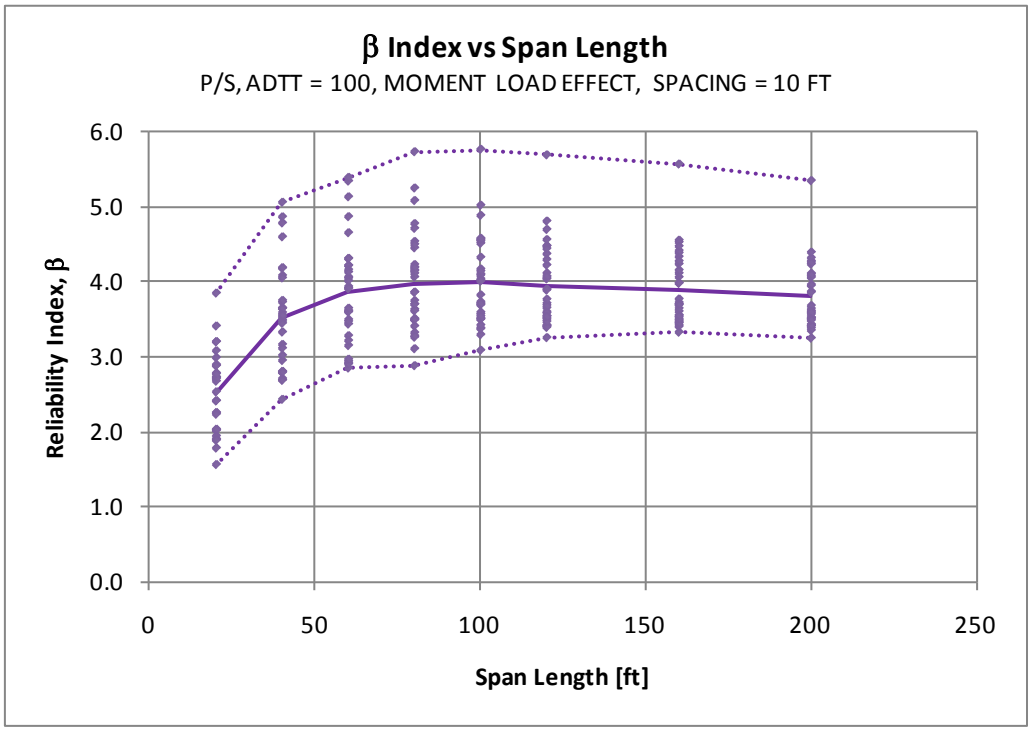

β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

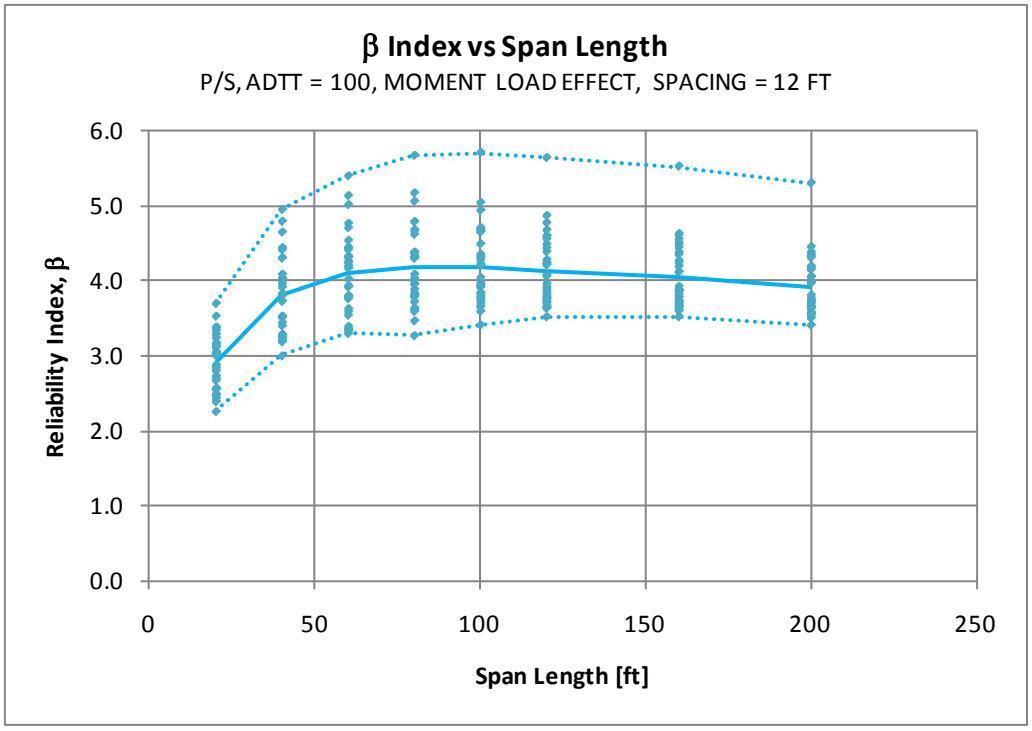

Average β Index vs Span Length
P/S, ADTT = 1000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

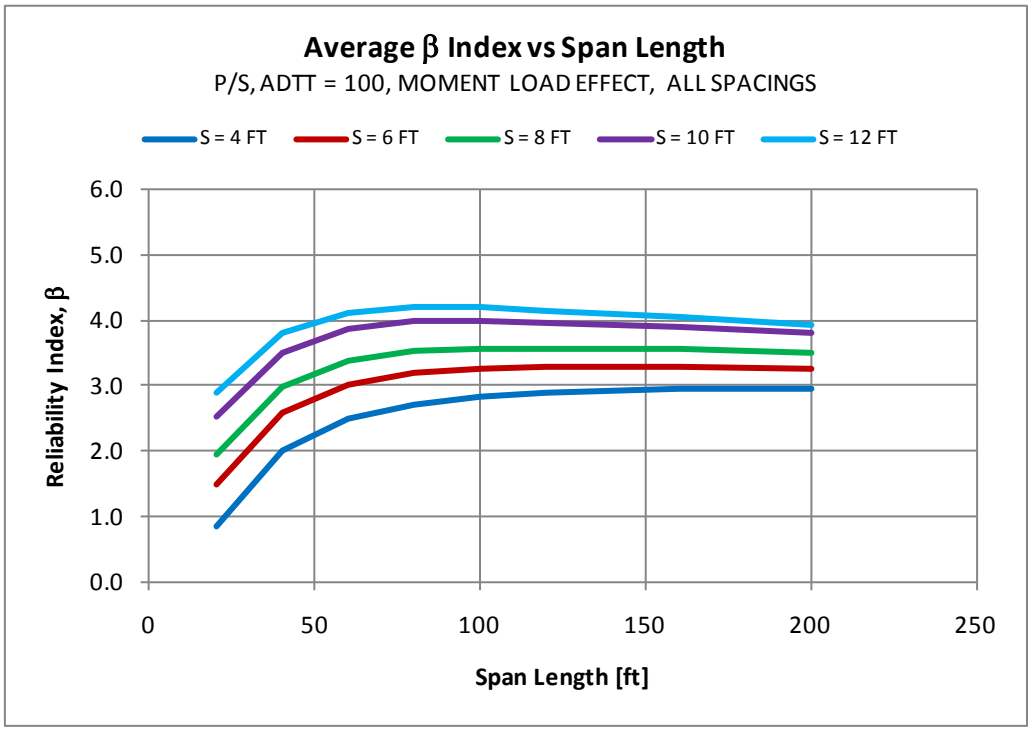

β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

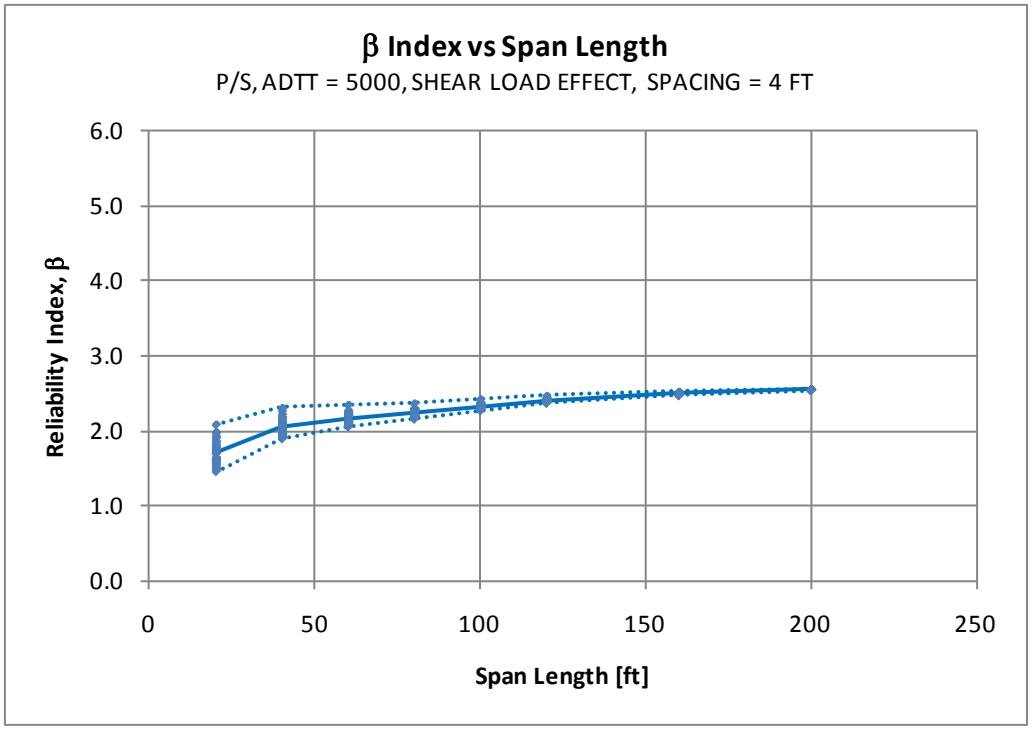

β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

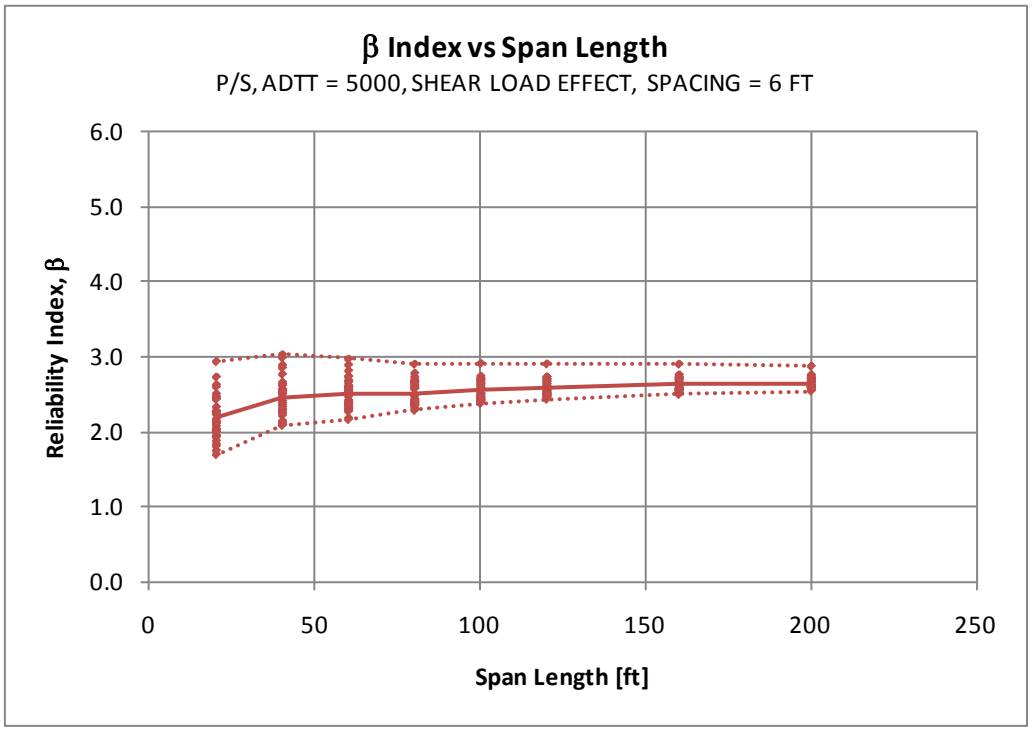

β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

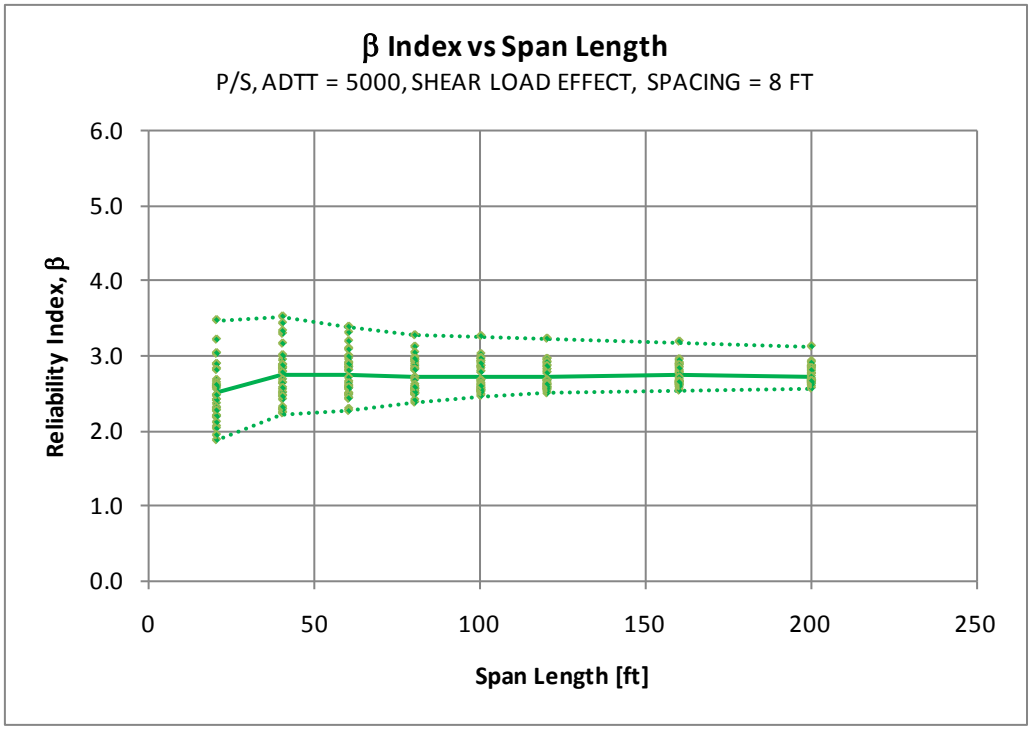

β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

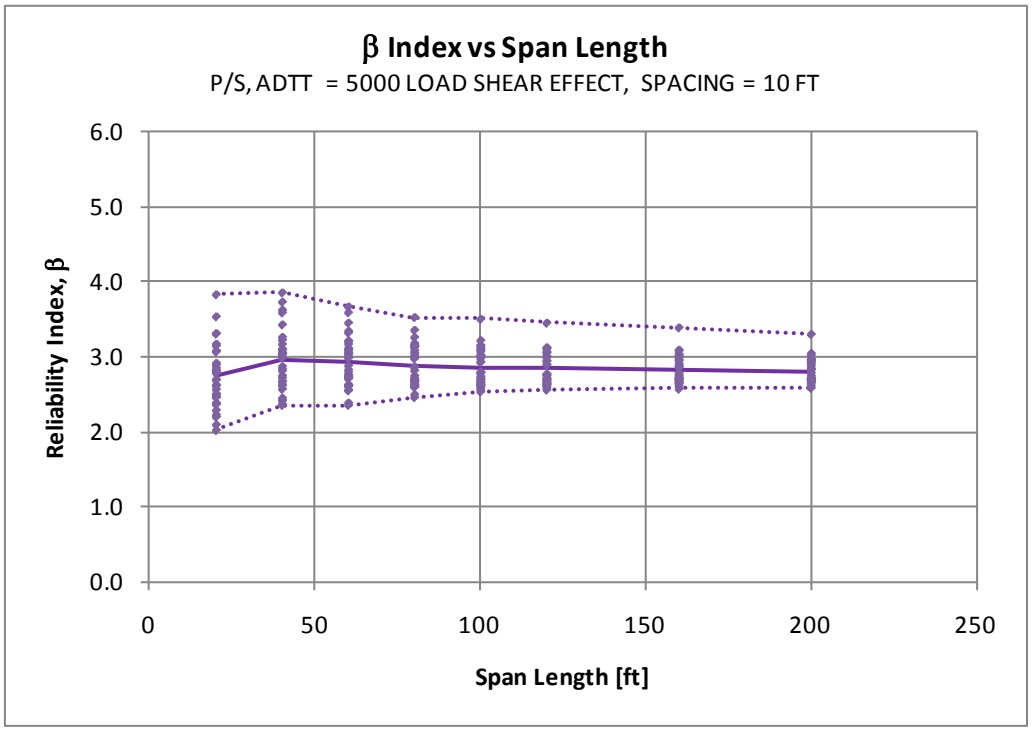

β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

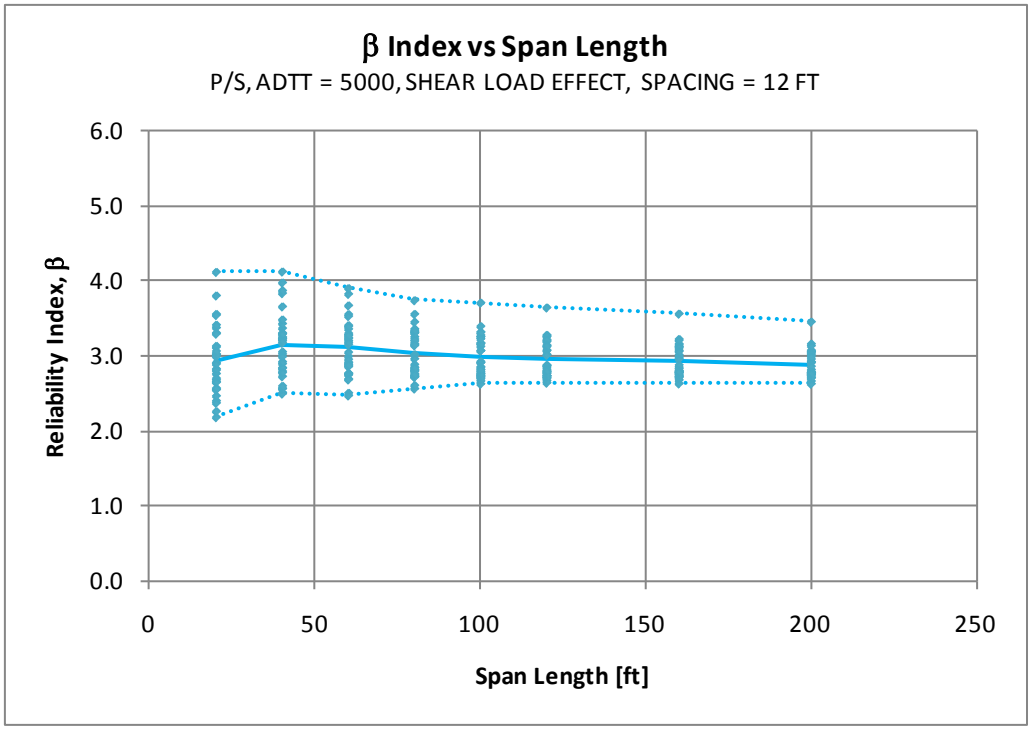

Average β Index vs Span Length
P/S, ADTT = 100, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

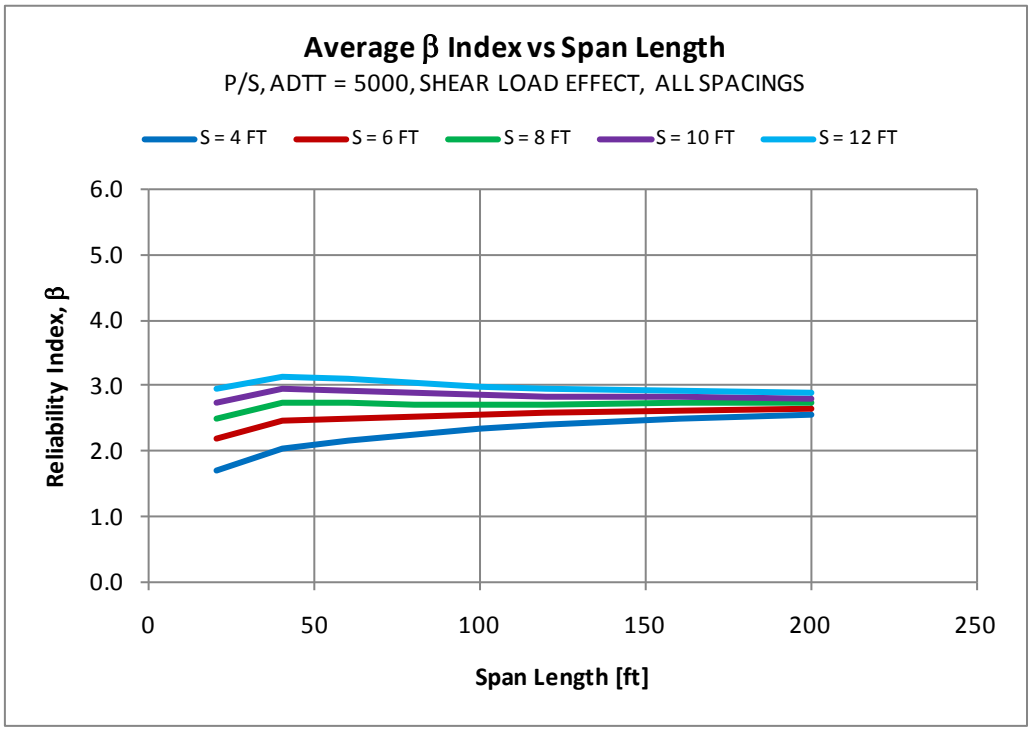

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

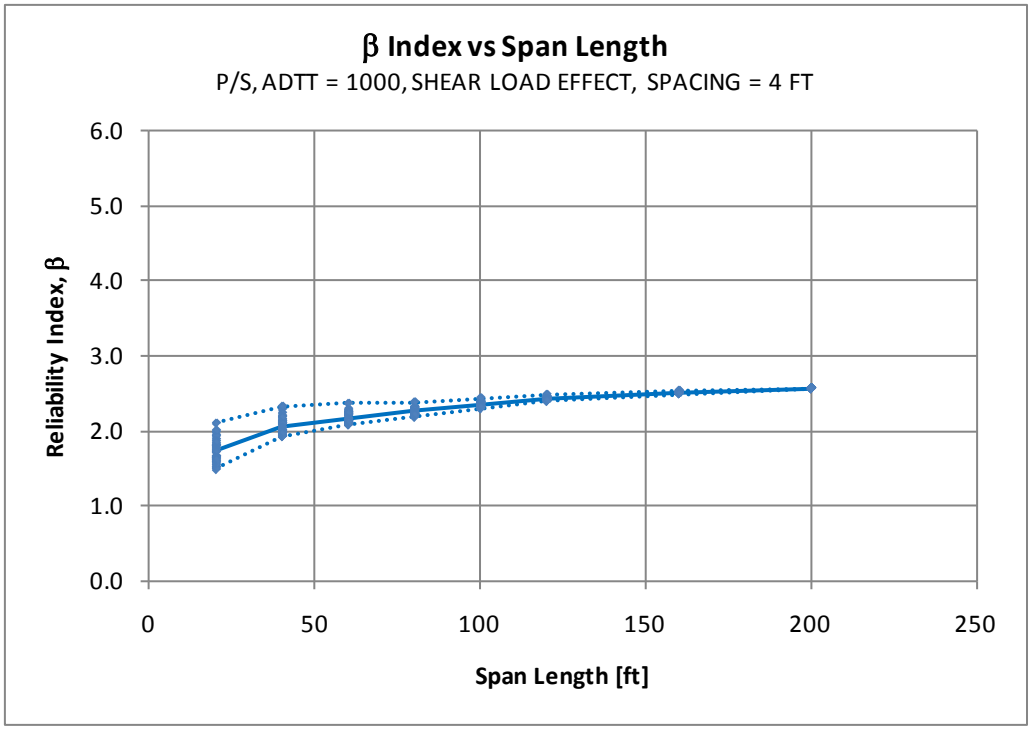

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

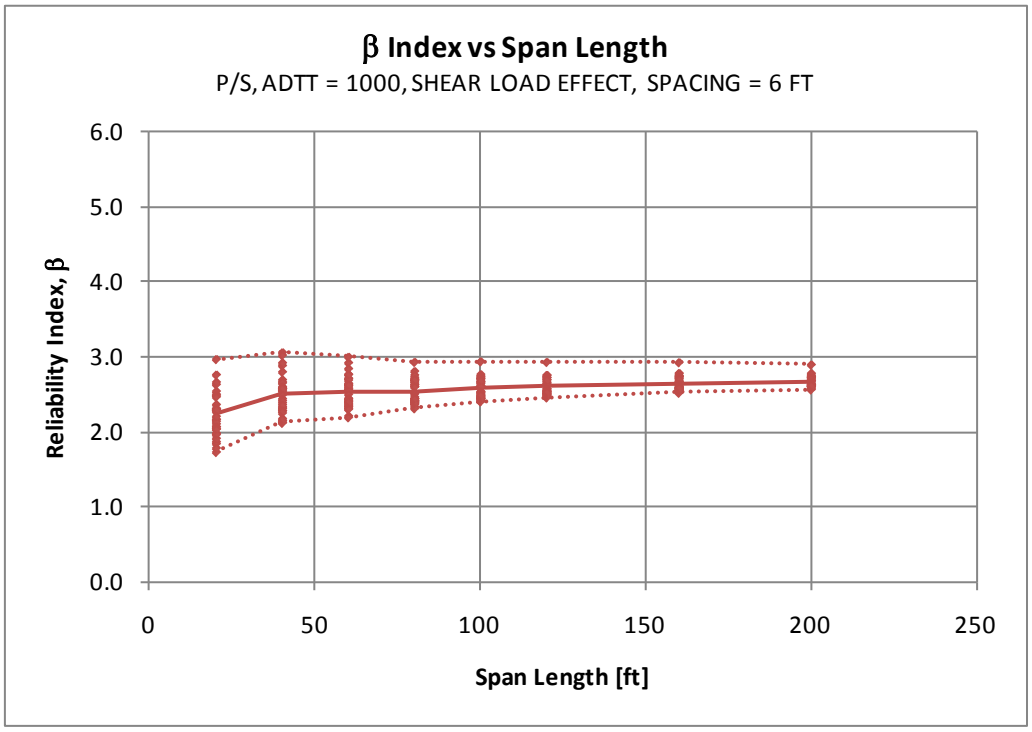

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

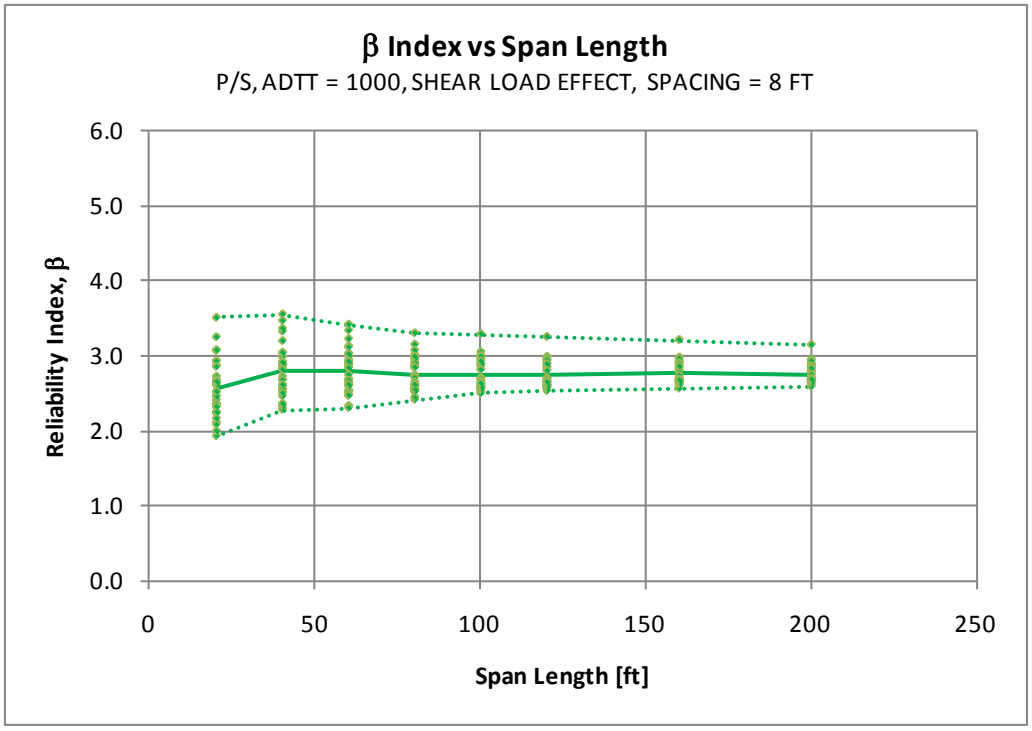

β Index vs Span Length
P/S, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

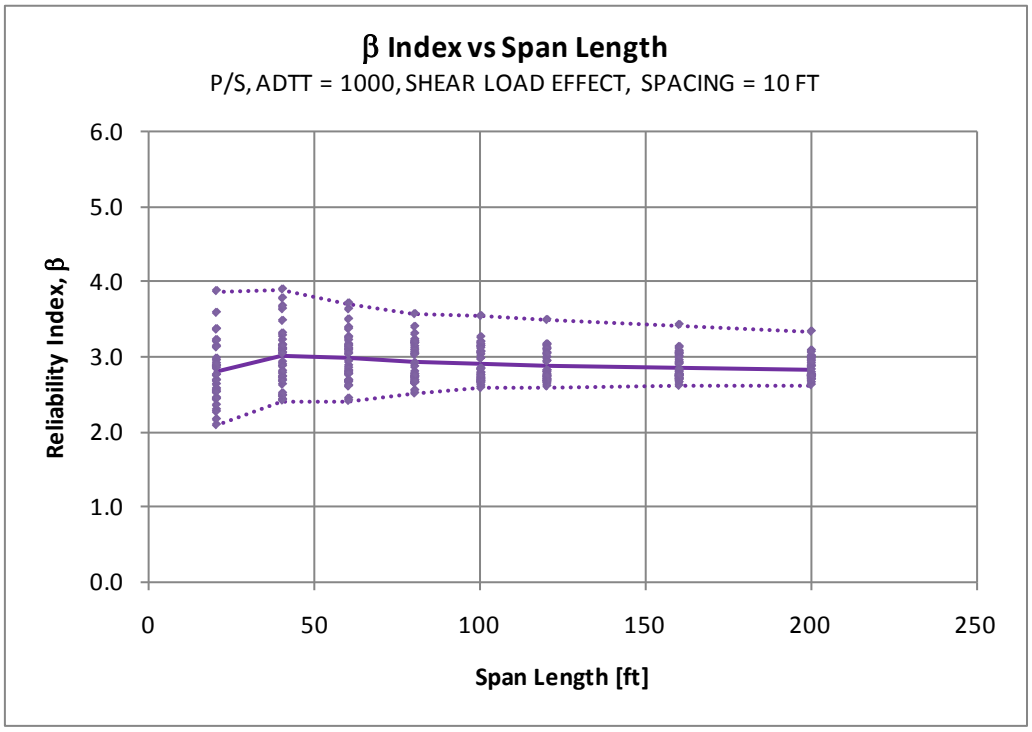

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

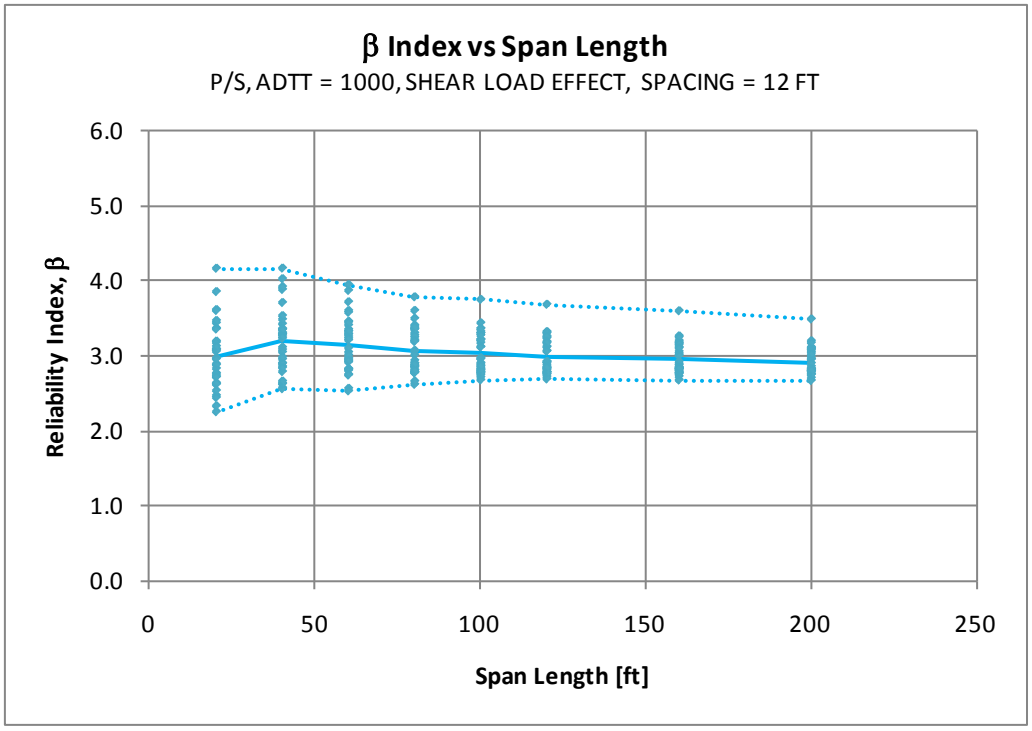

Average β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

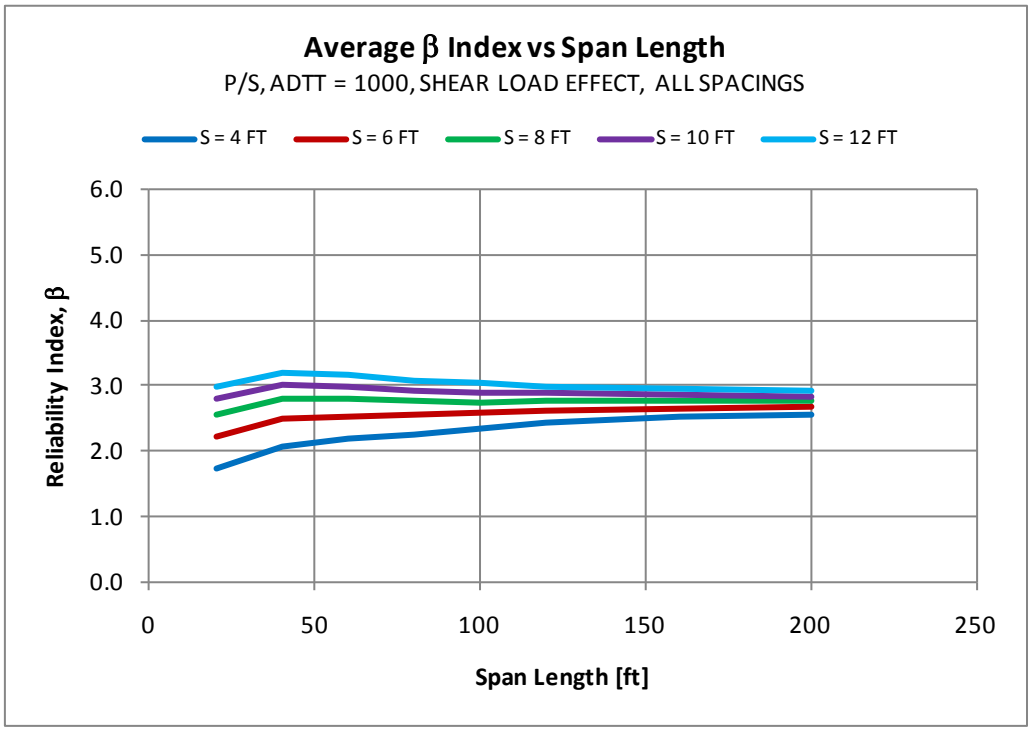

β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

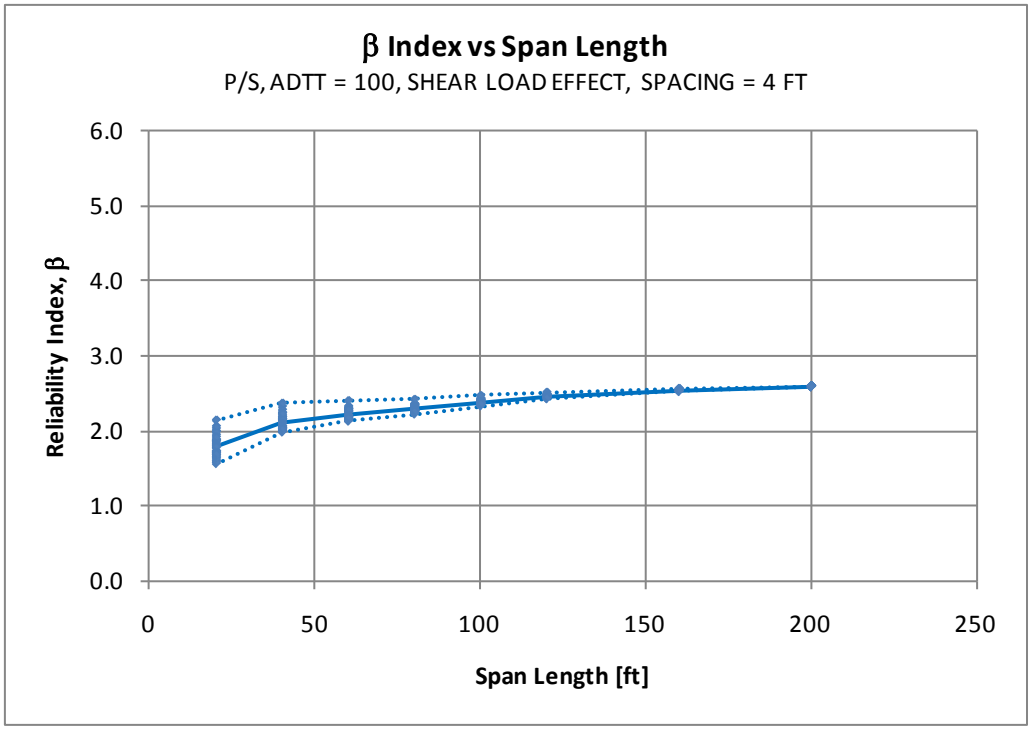

β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

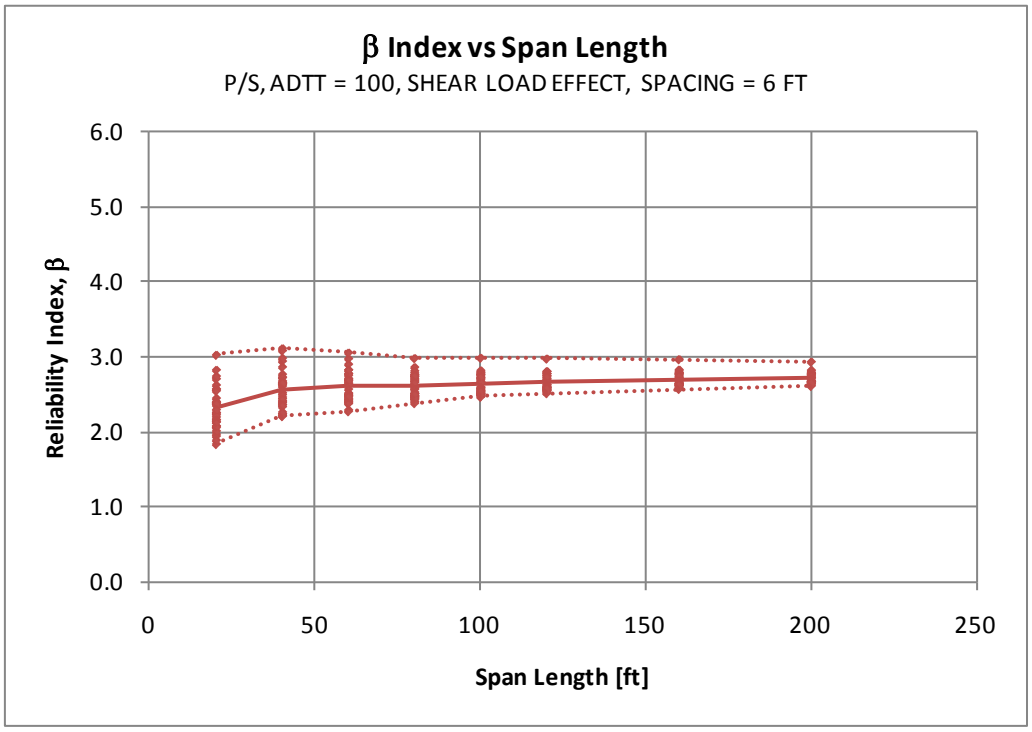

β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
P/S, ADTT = 1000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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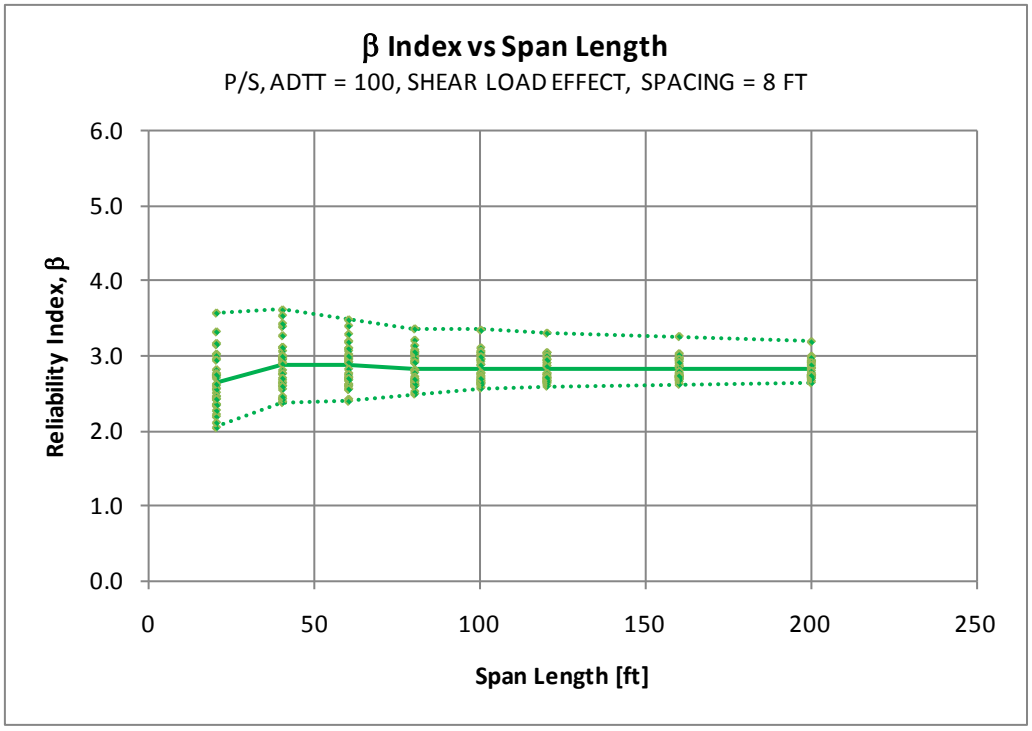
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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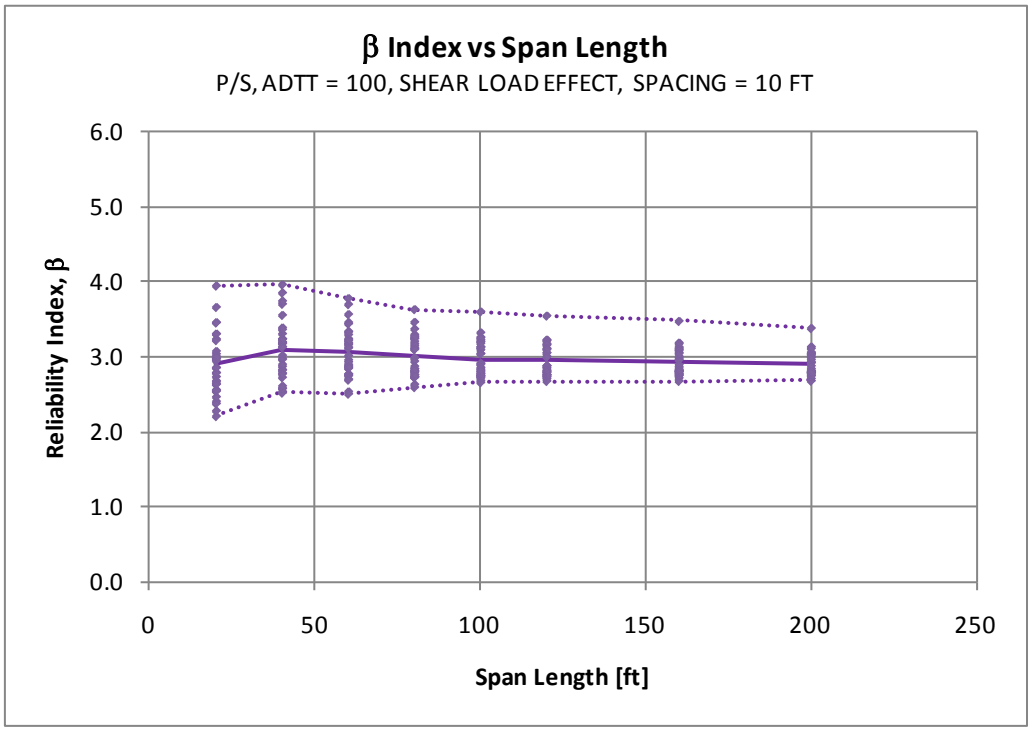
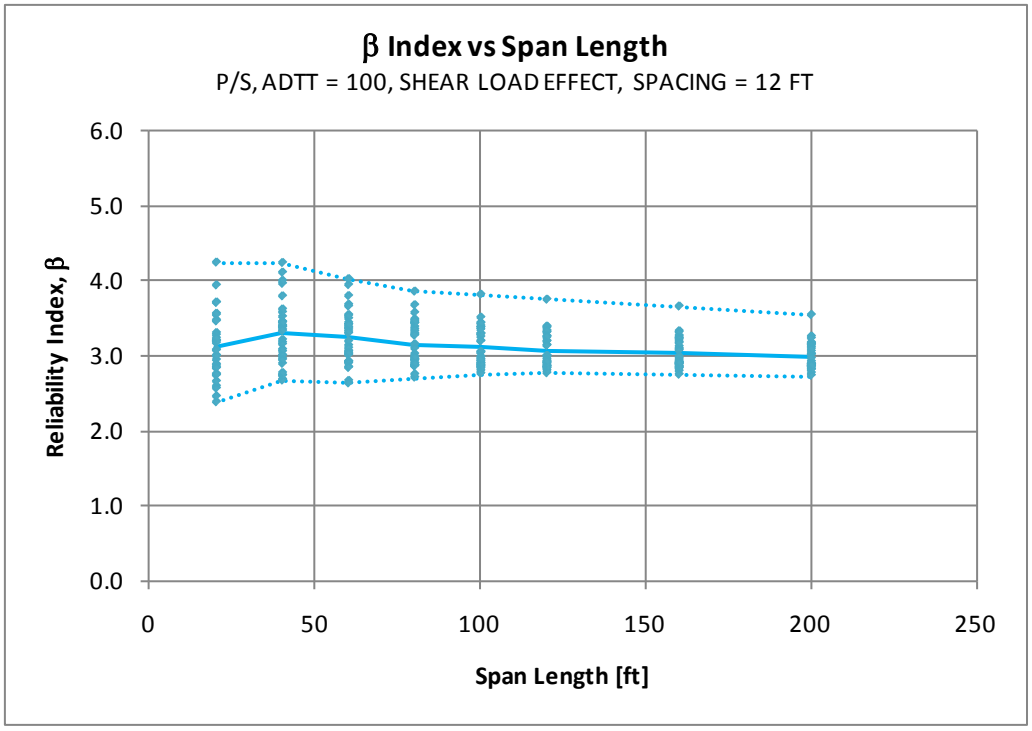
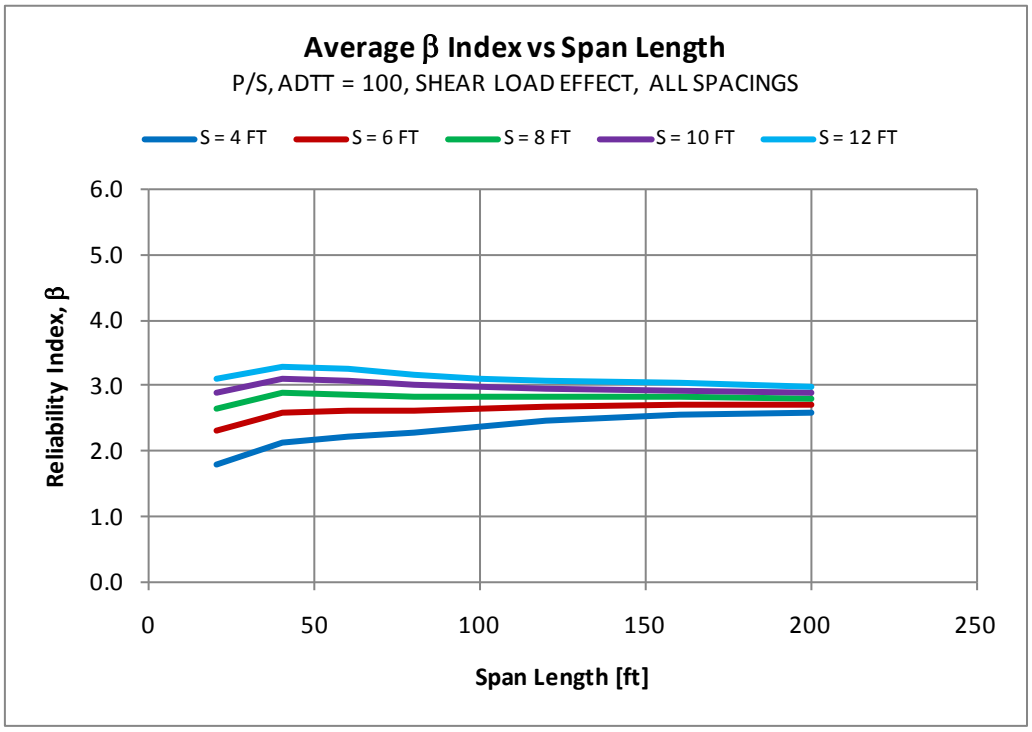
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

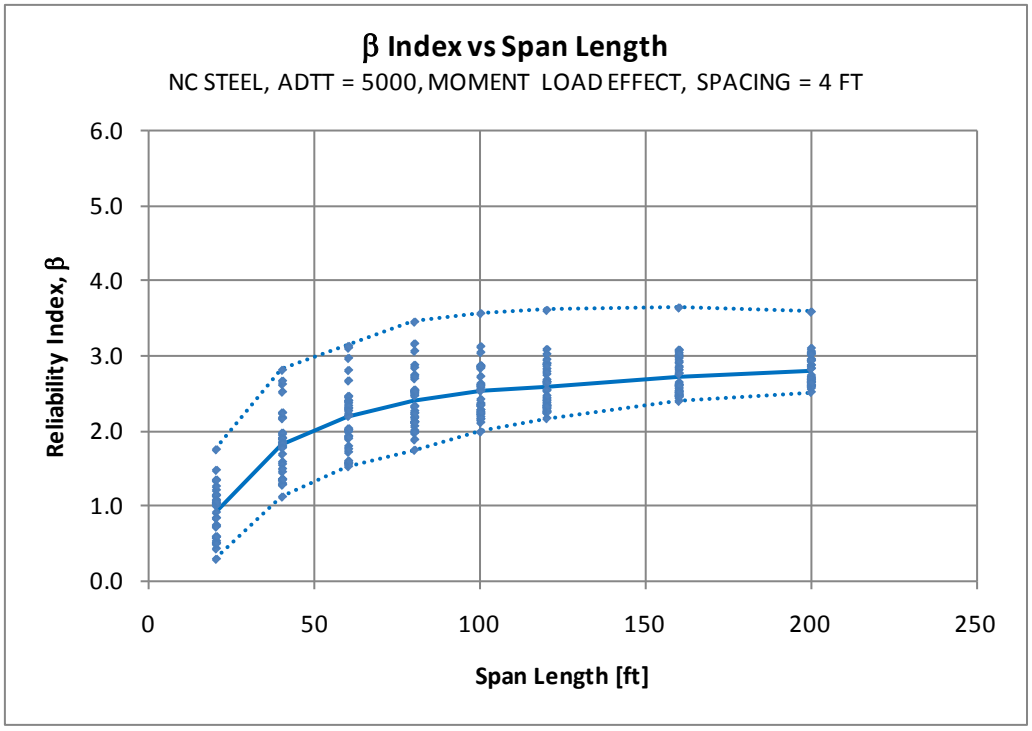

β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

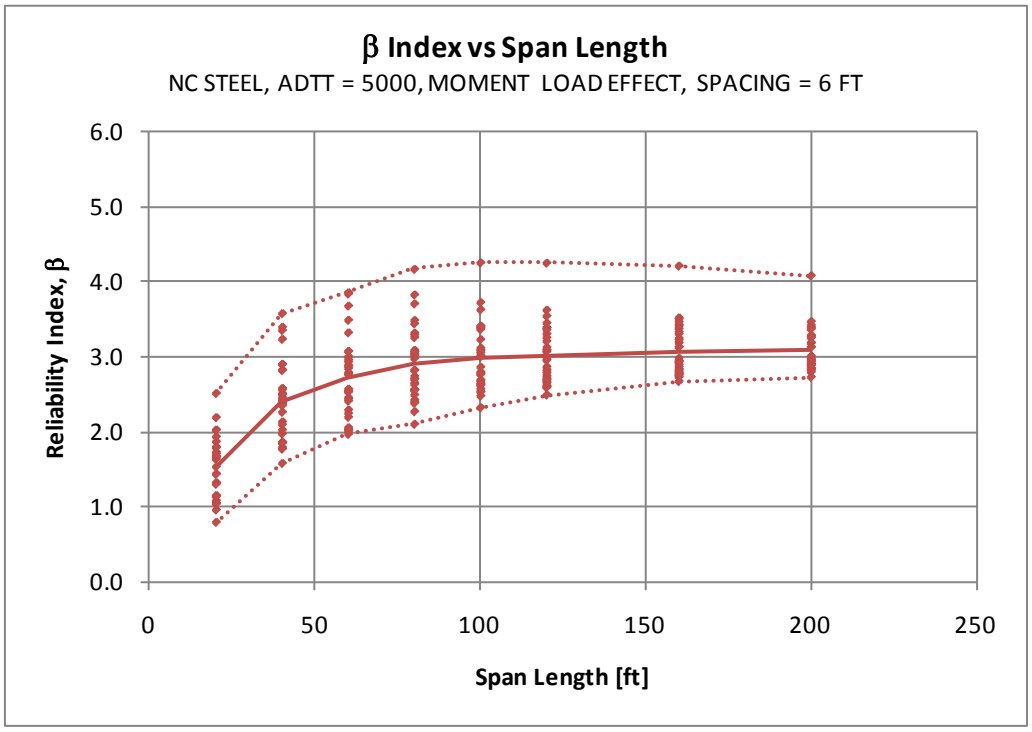

Average β Index vs Span Length
P/S, ADTT = 100, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

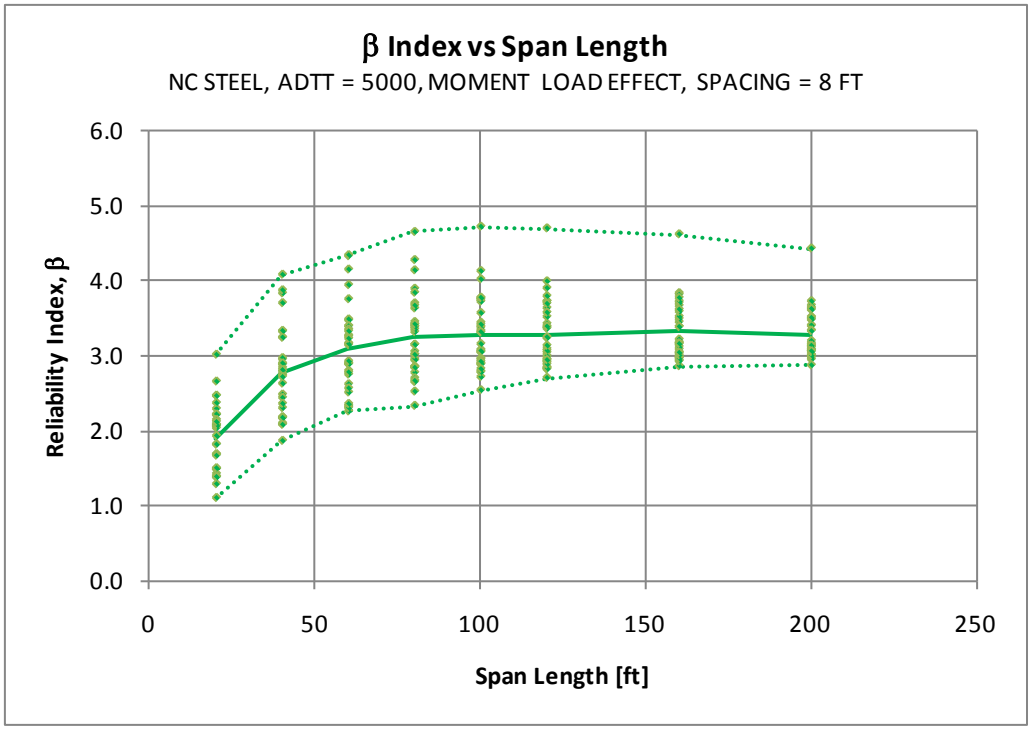
NON-COMPOSITE STEEL BEAM

PERMIT PLOTS

(TWO LANES)


β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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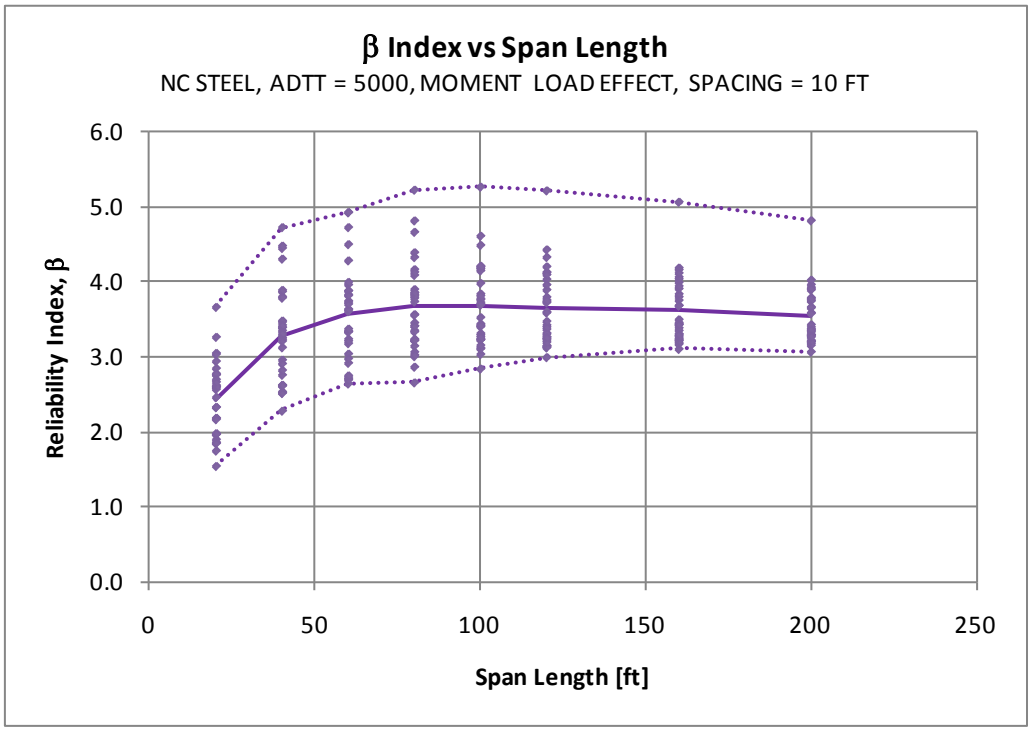
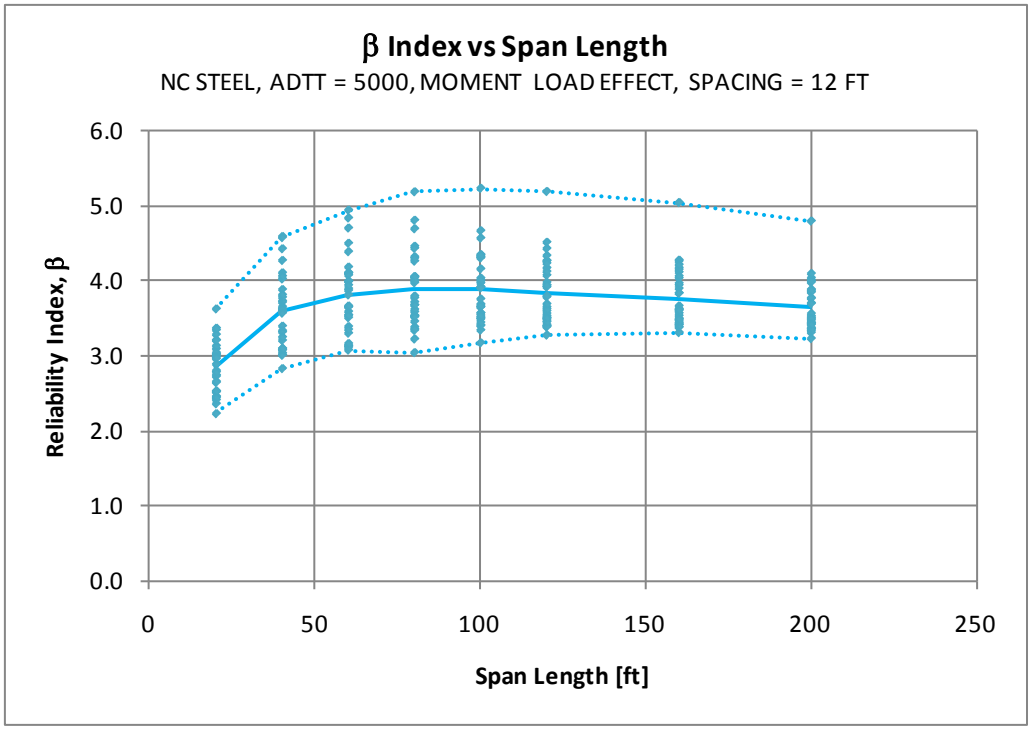
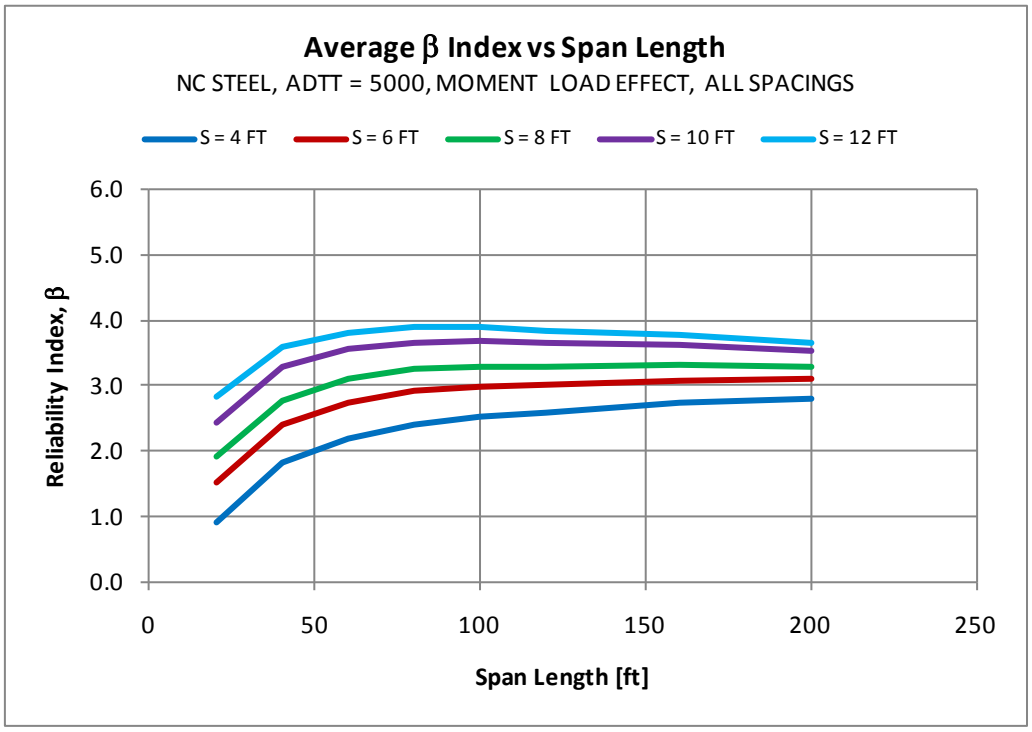
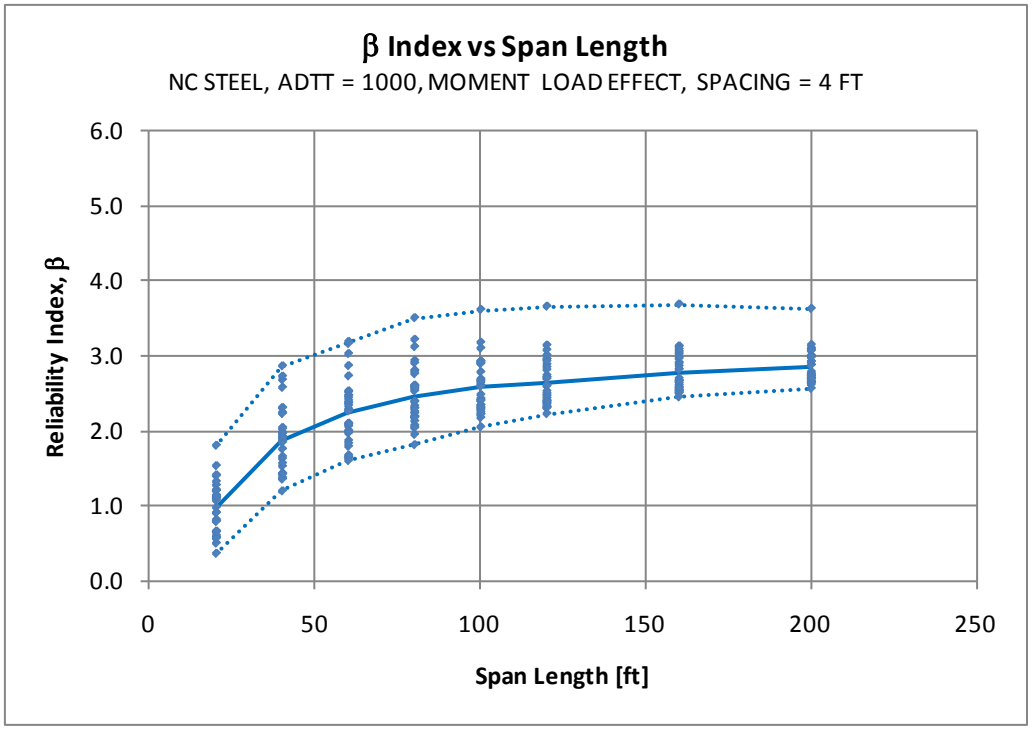
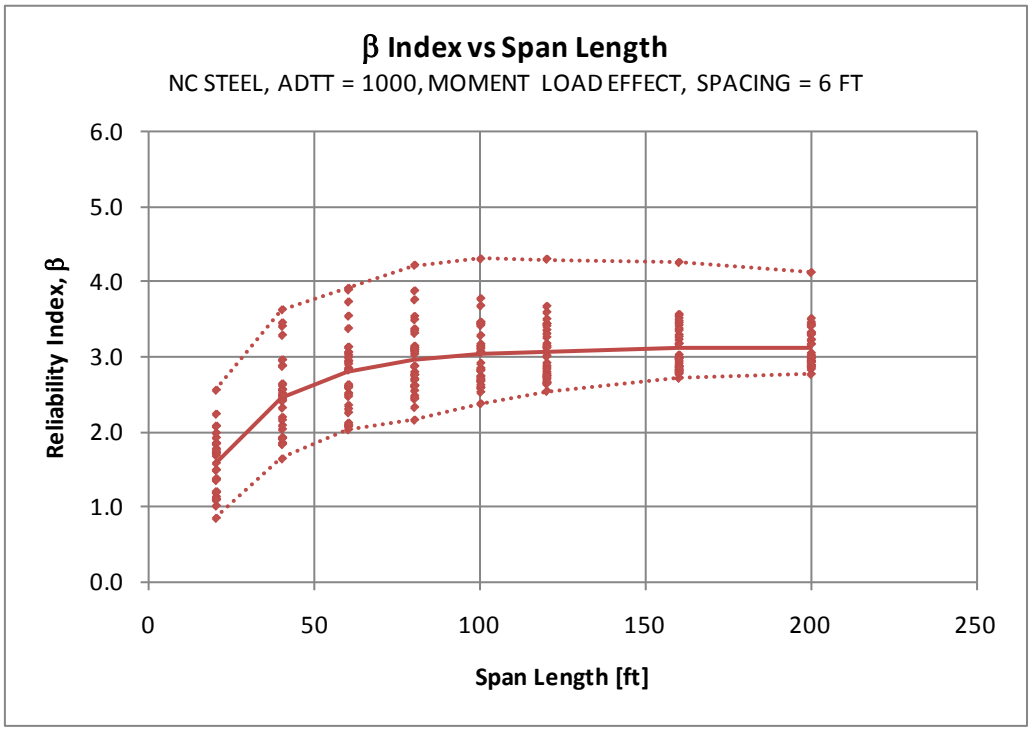
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

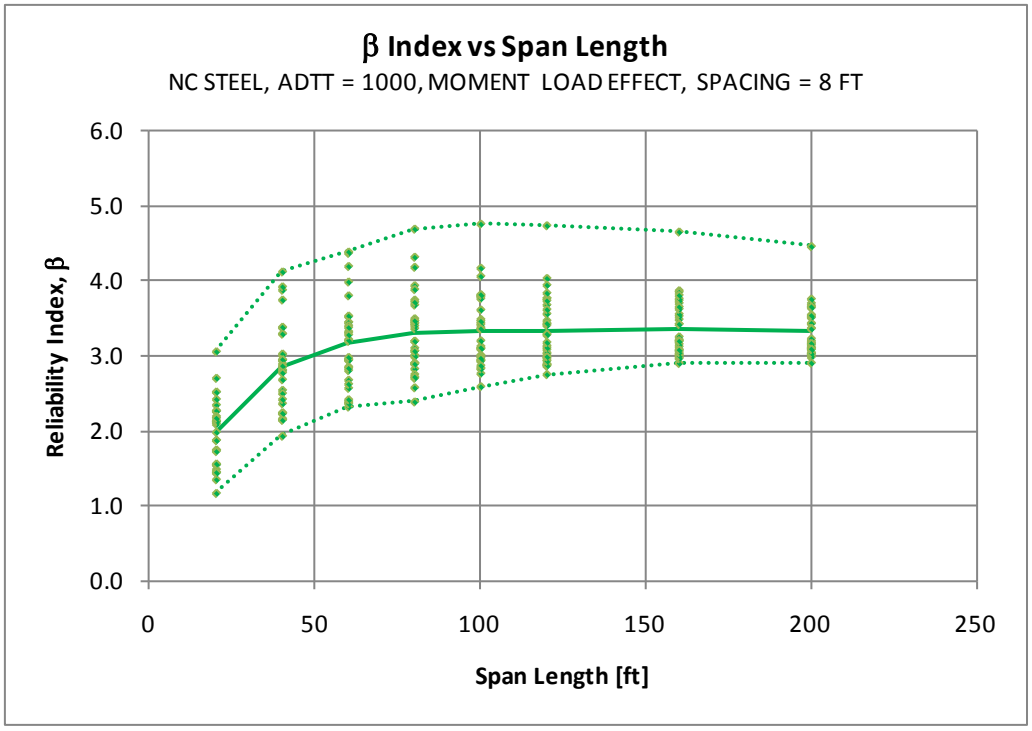

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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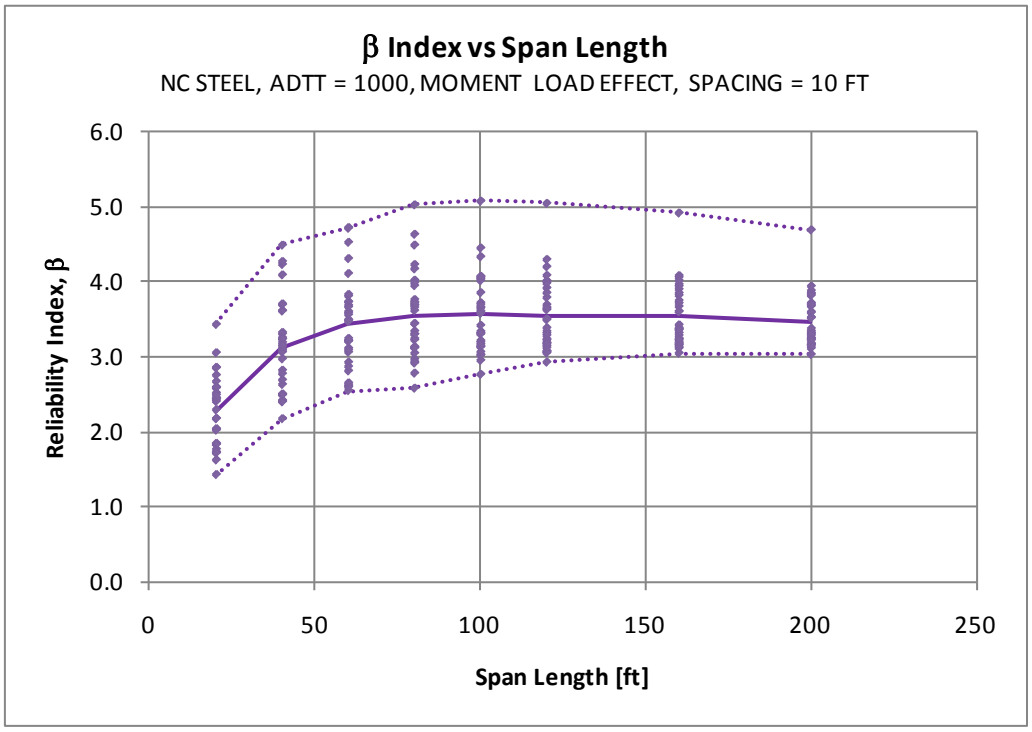
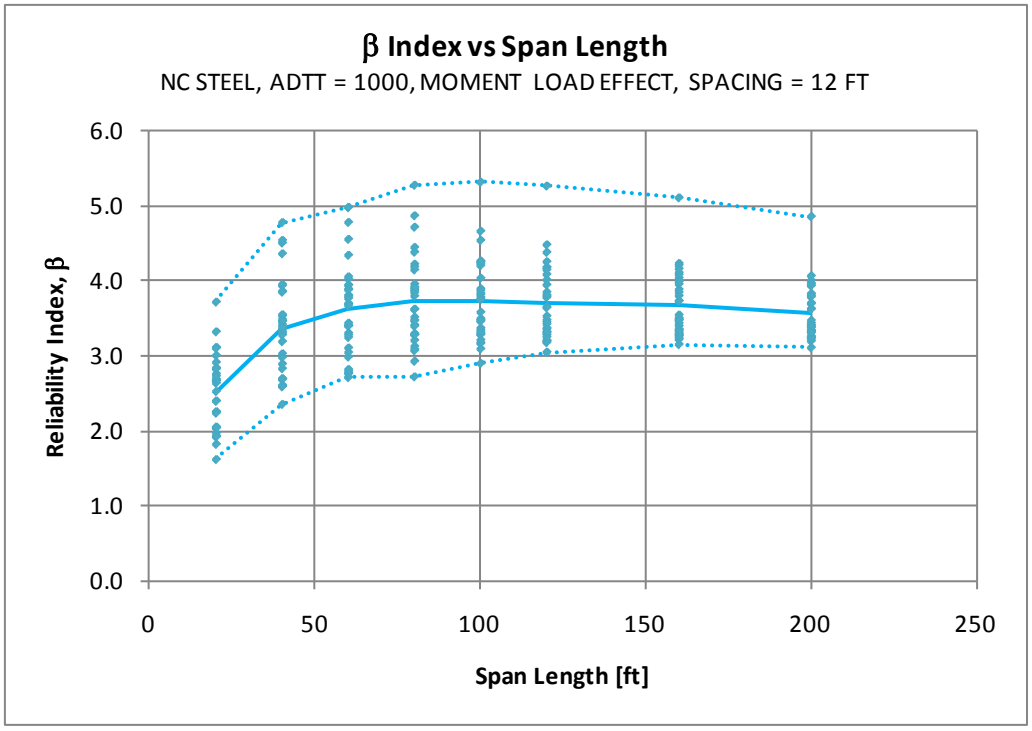
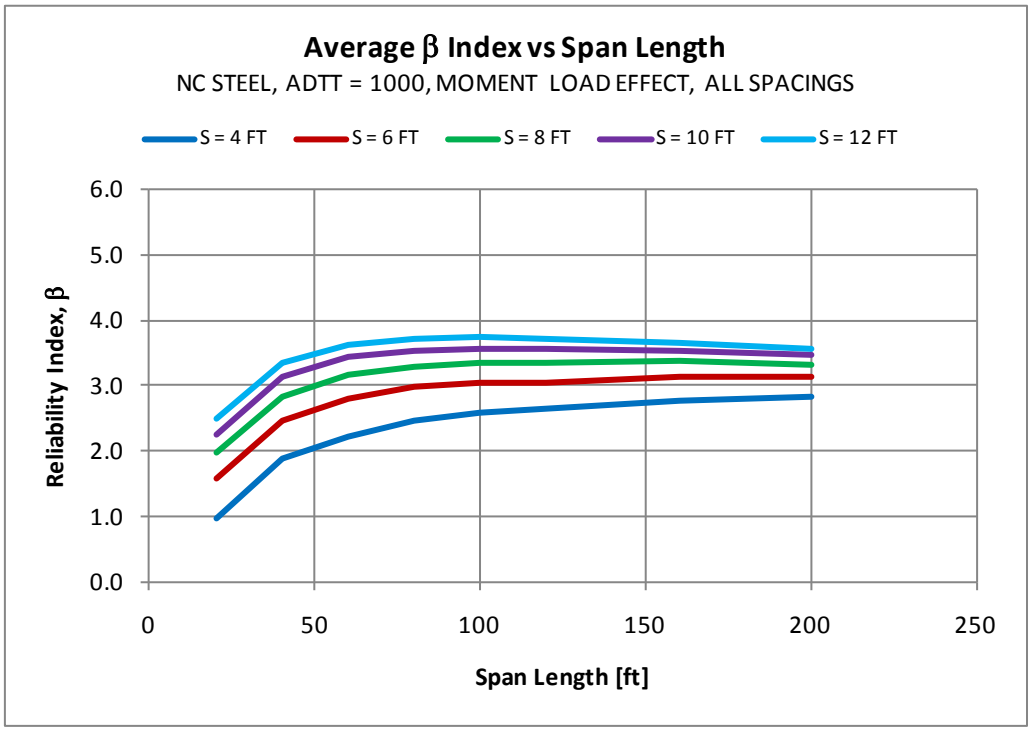
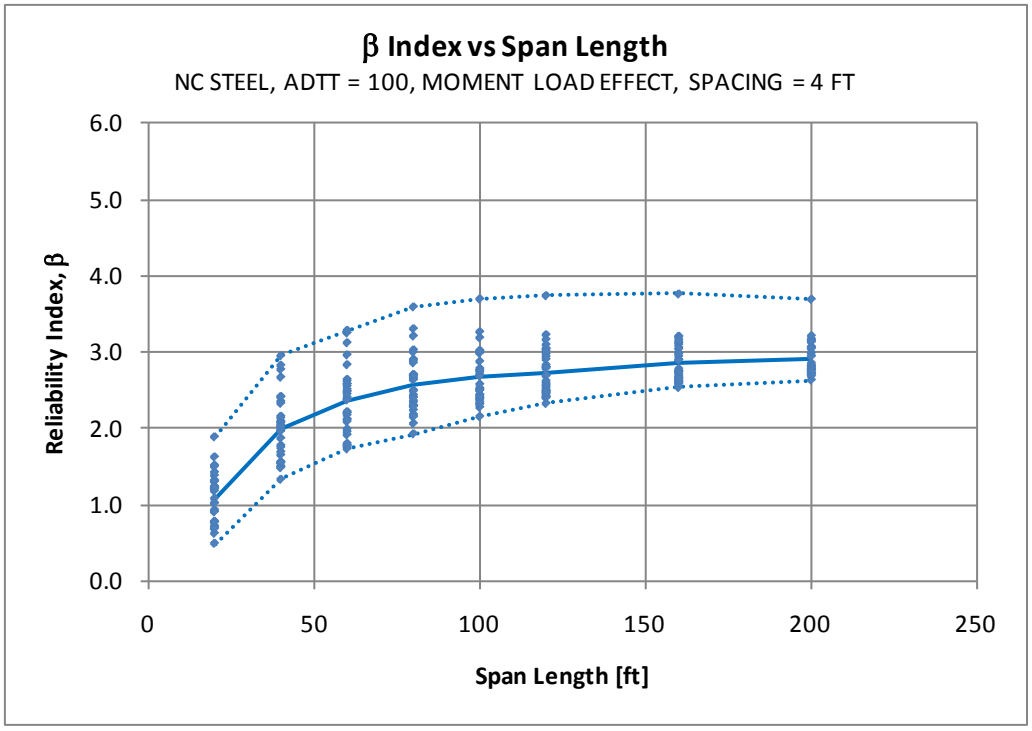
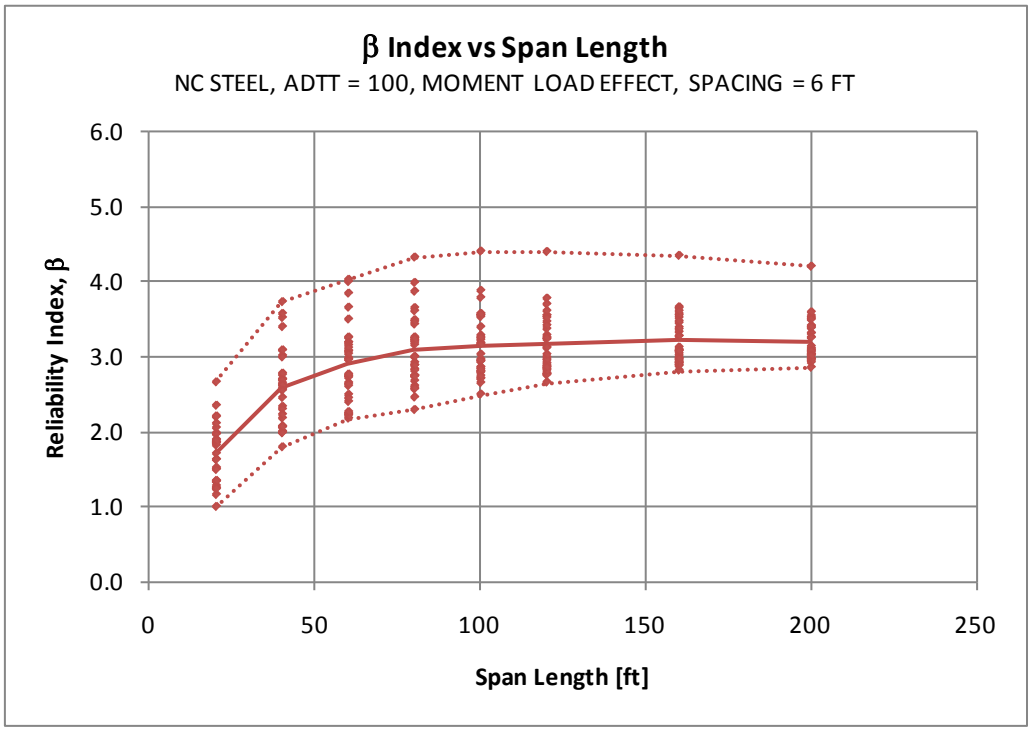
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 1000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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5.0
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3.0
2.0
1.0
0.0
0
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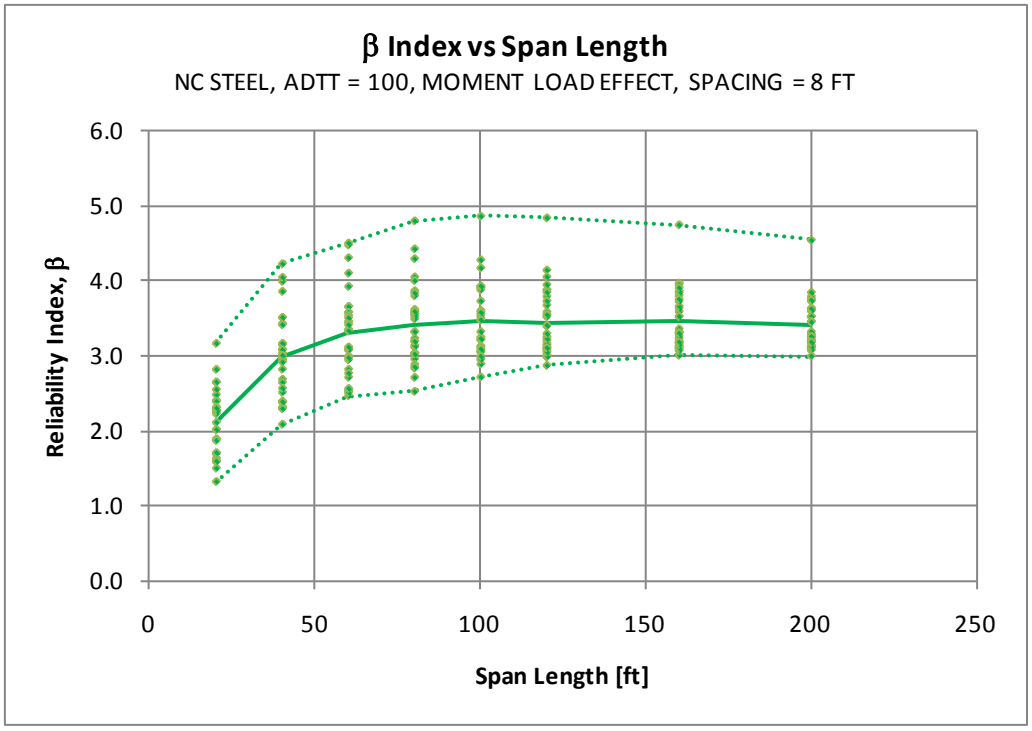
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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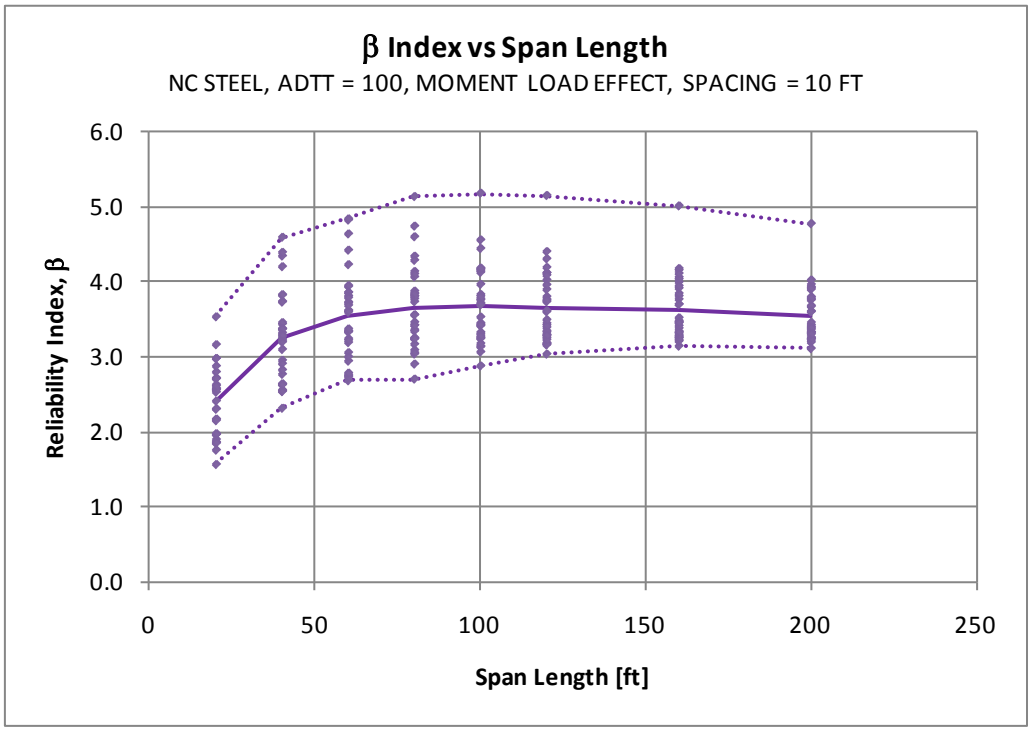
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
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3.0
2.0
1.0
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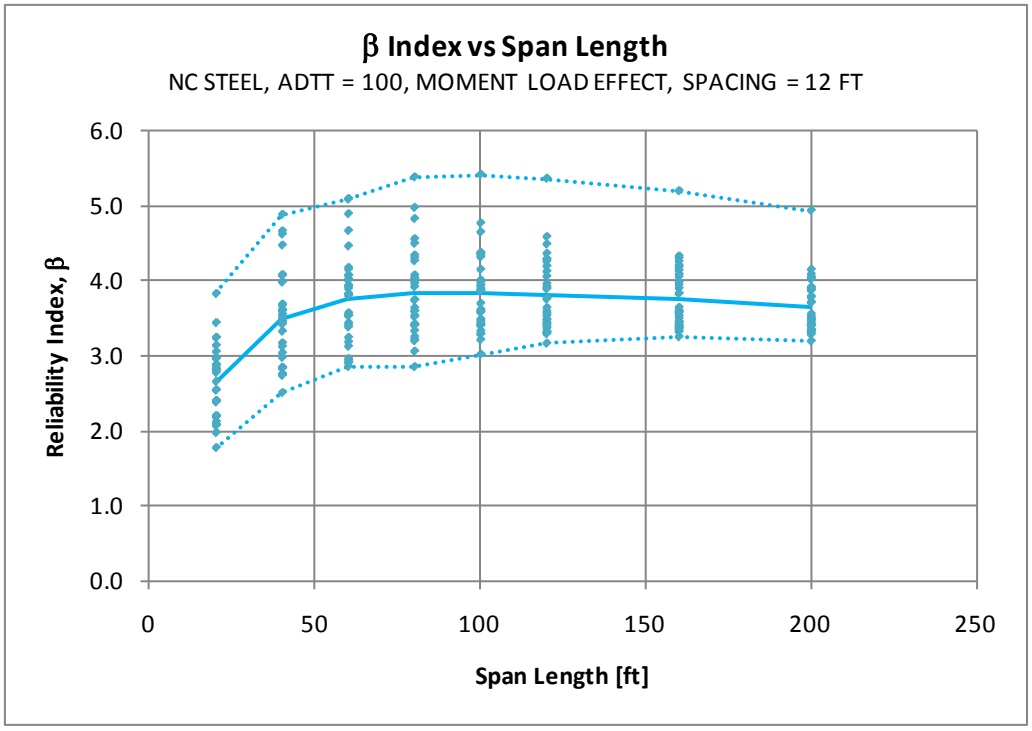
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
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3.0
2.0
1.0
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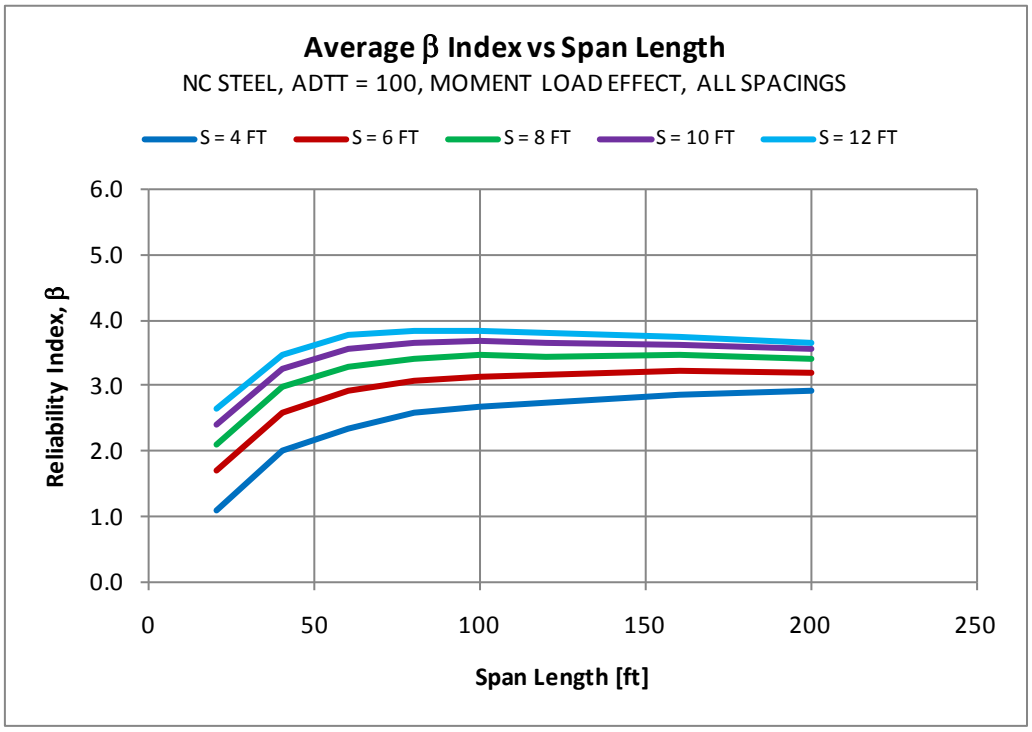
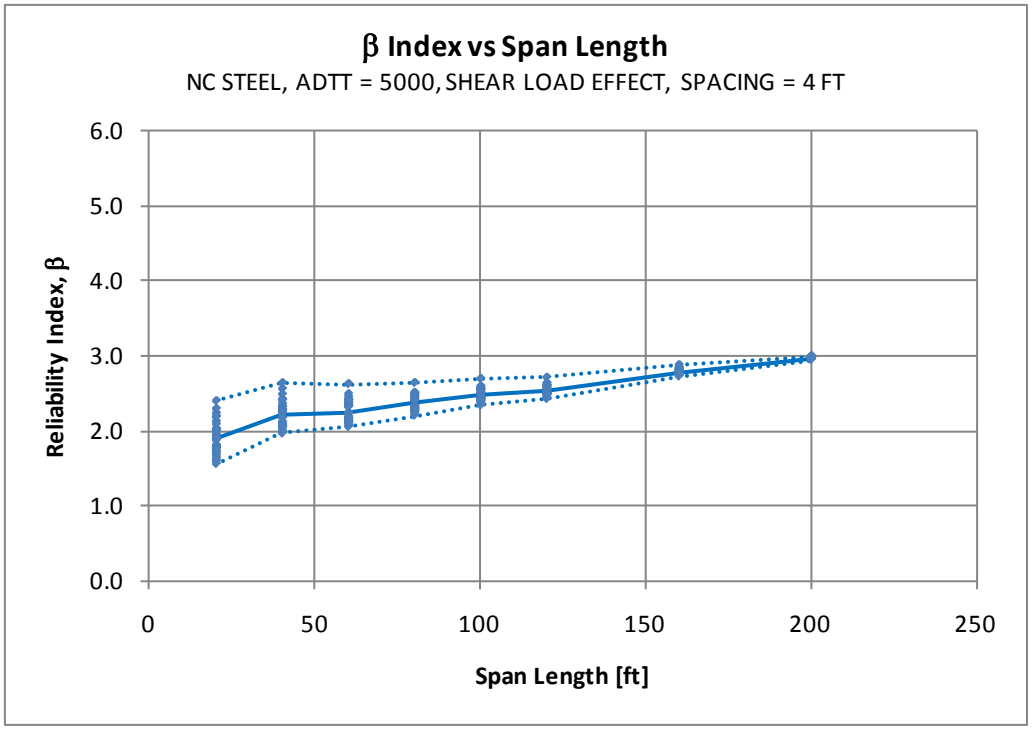
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 100, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
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5.0
4.0
3.0
2.0
1.0
0.0
0
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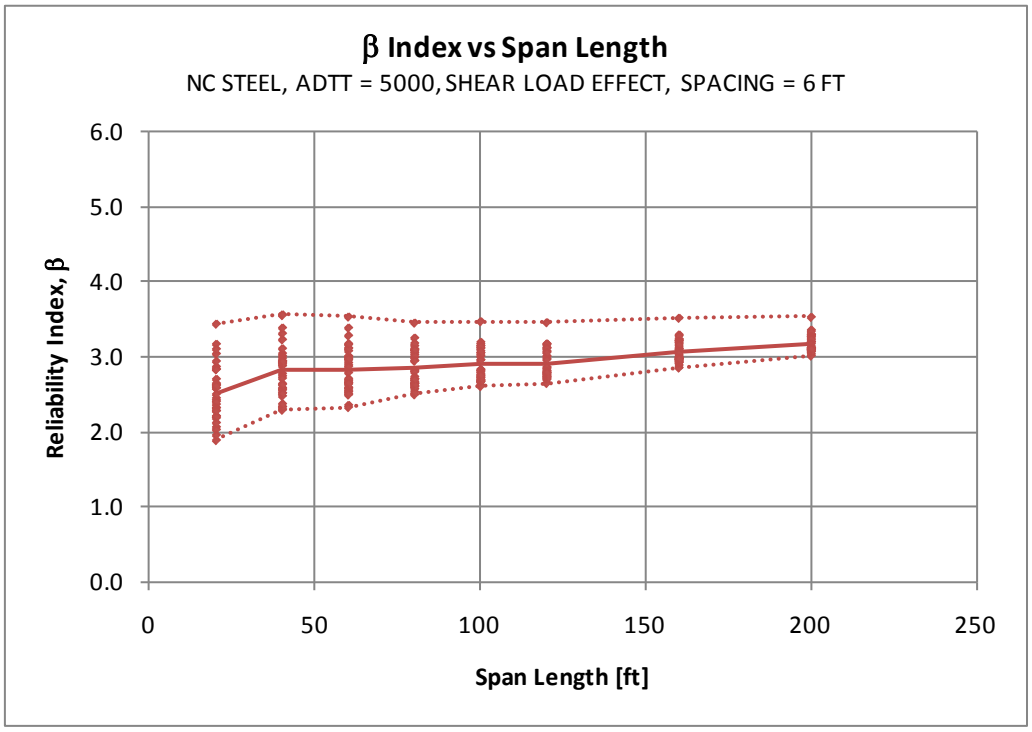
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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4.0
3.0
2.0
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0
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200
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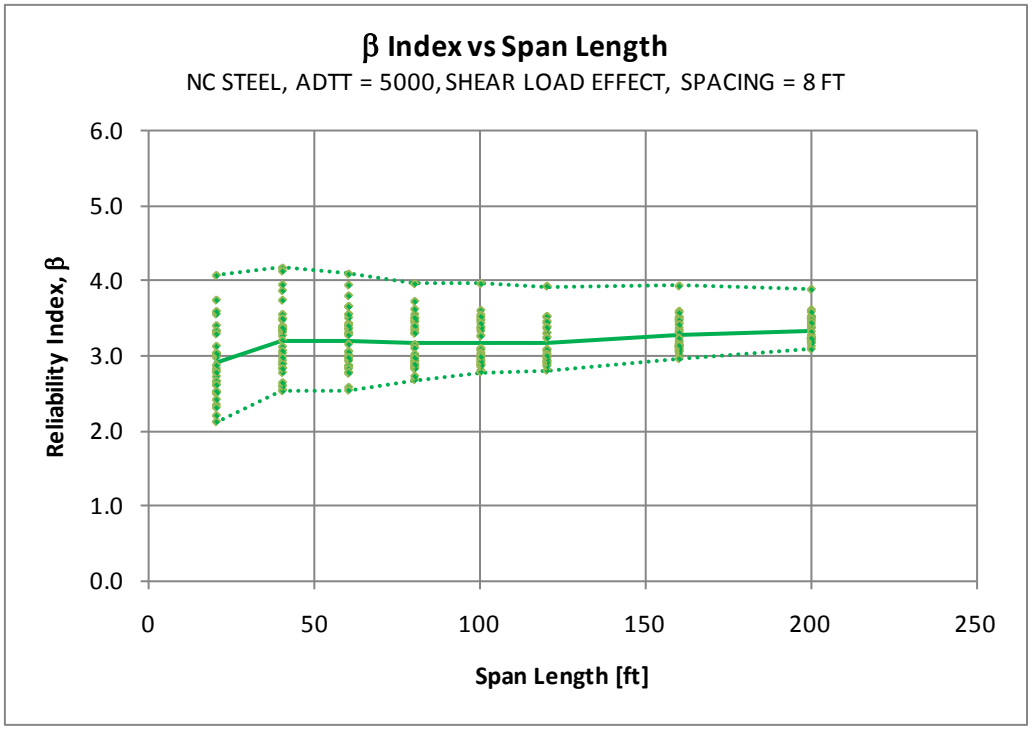
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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3.0
2.0
1.0
0.0
0
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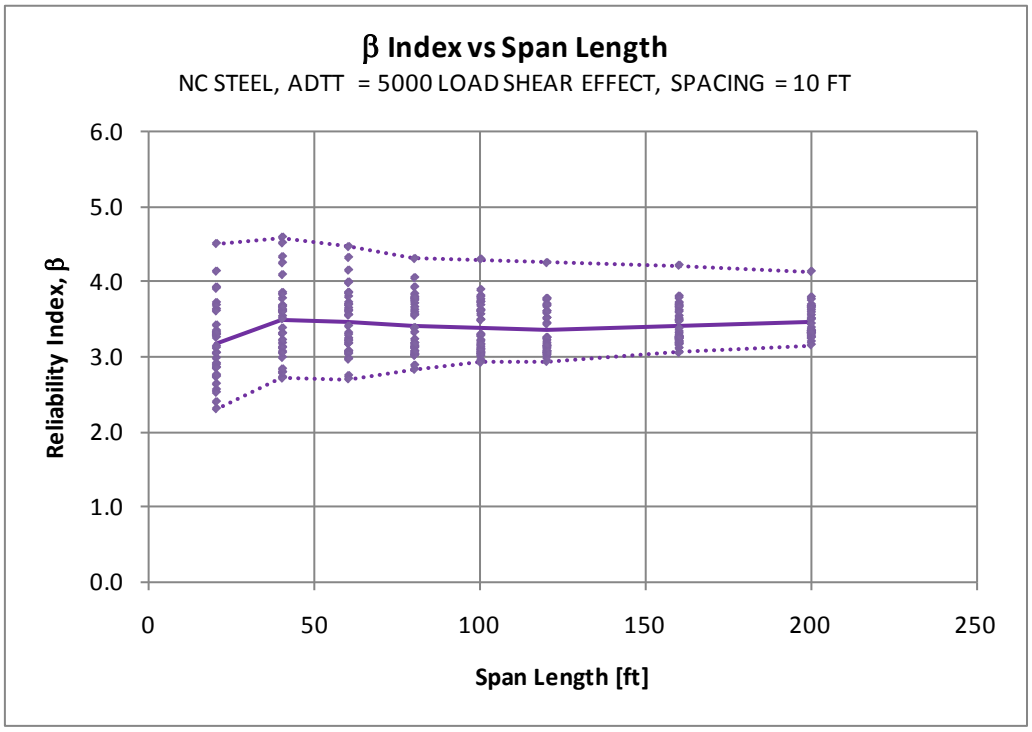
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
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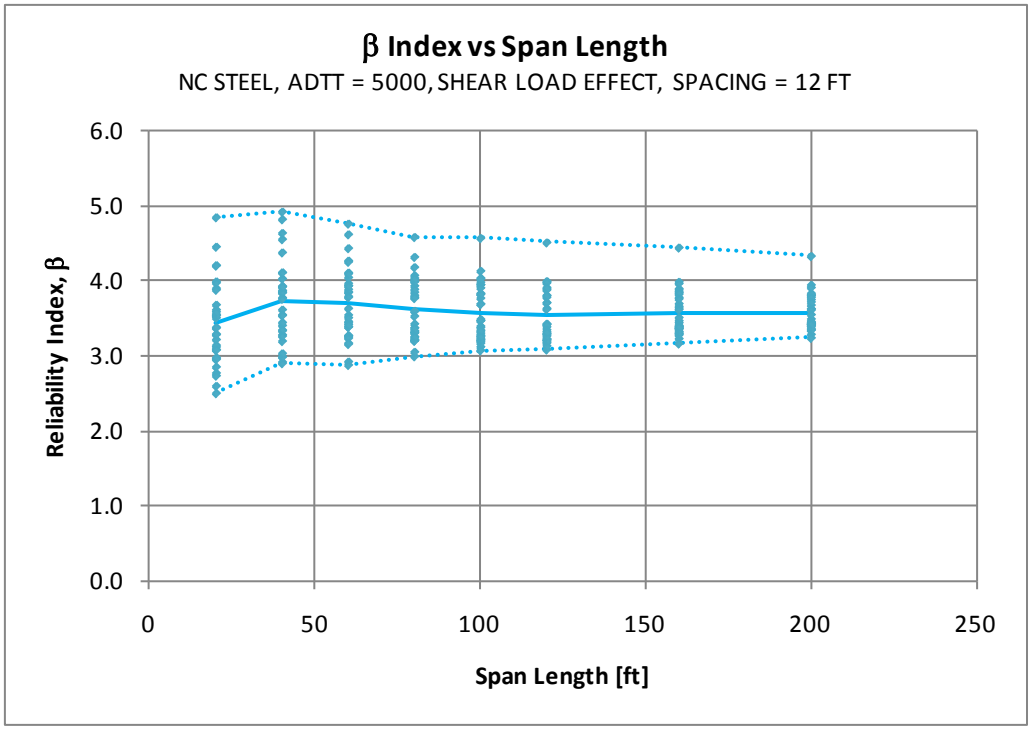
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
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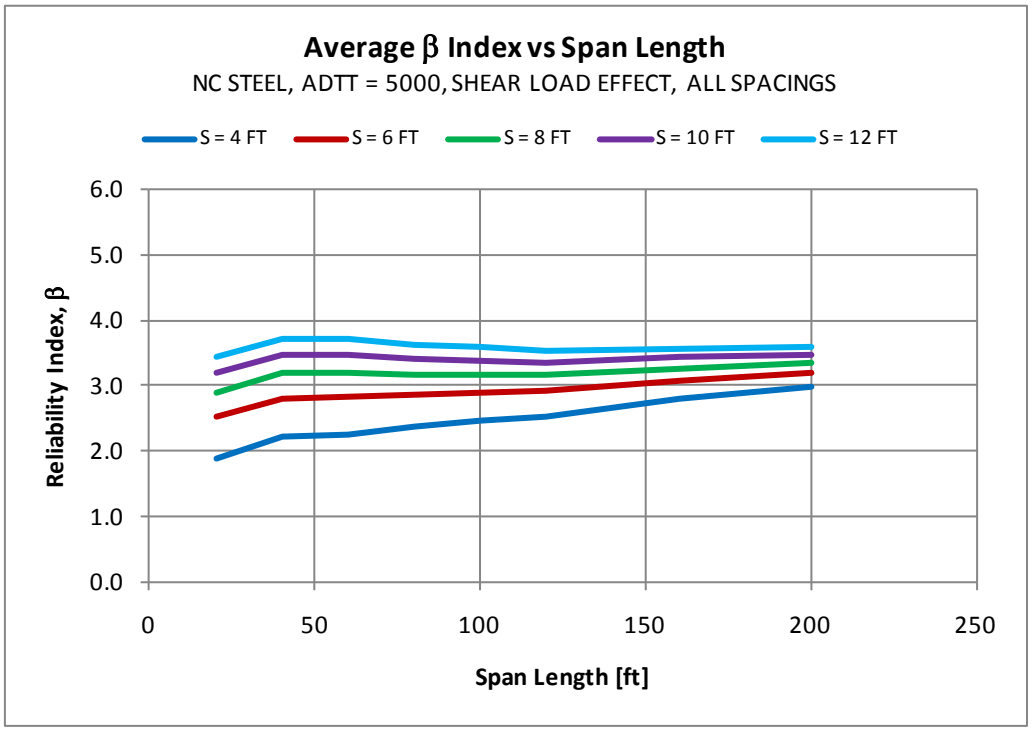
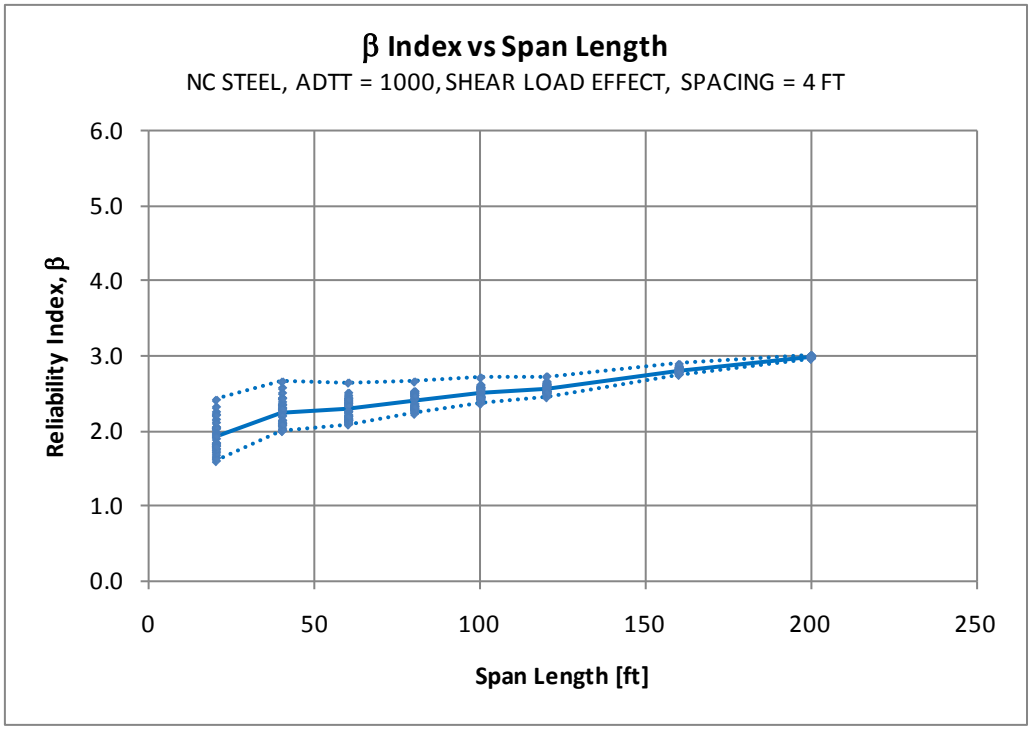
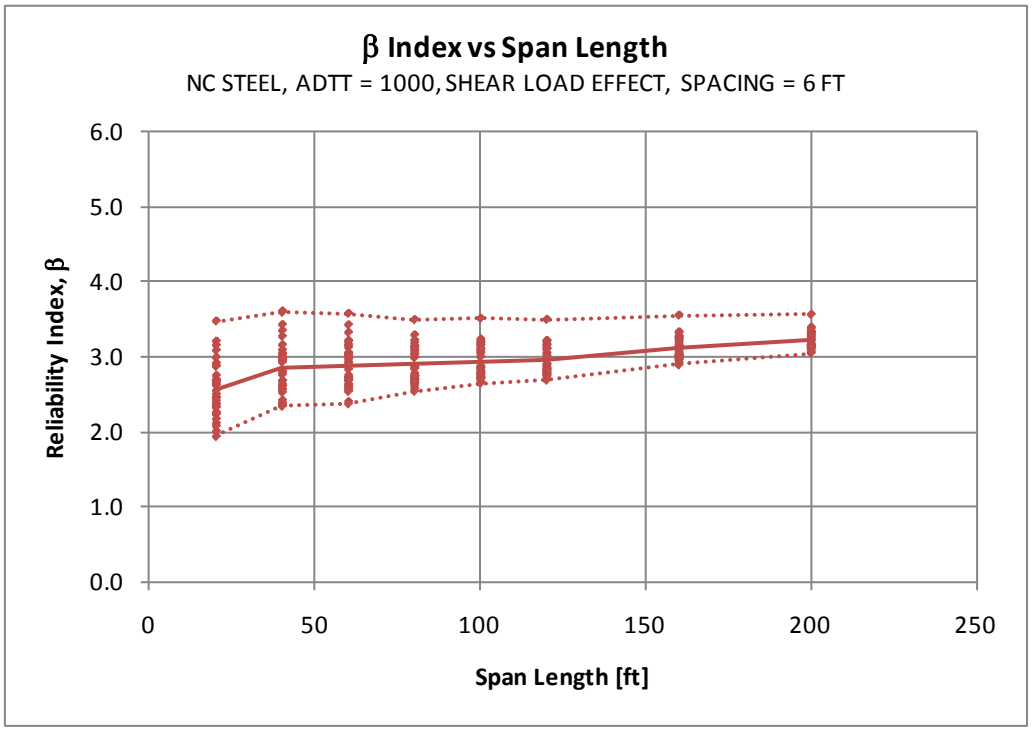
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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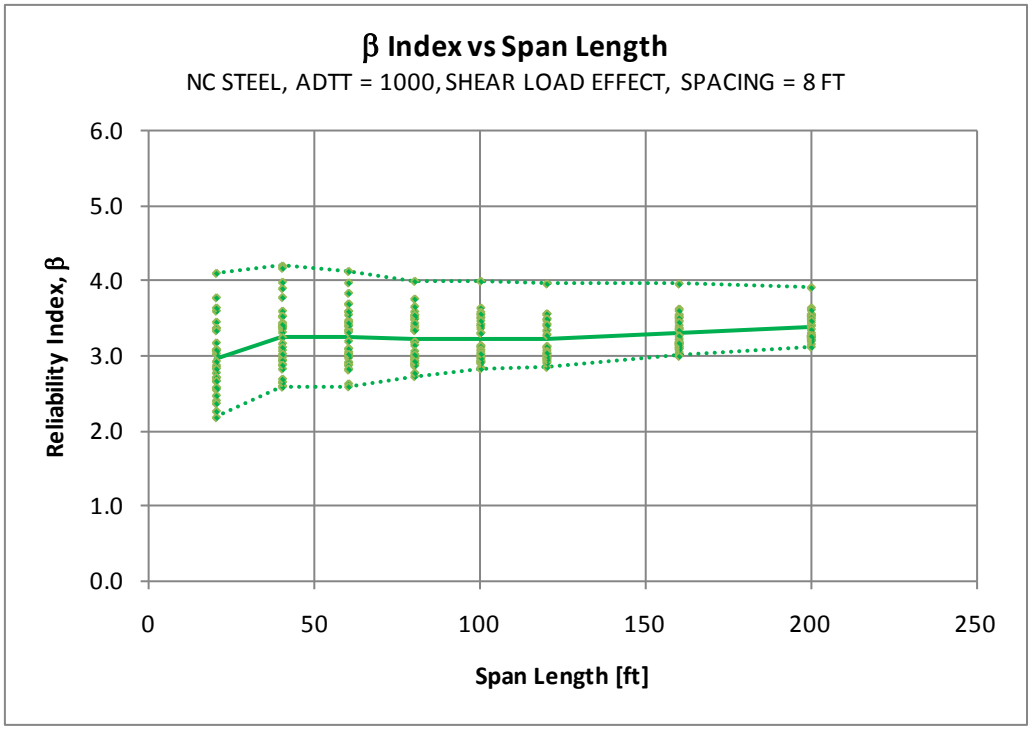
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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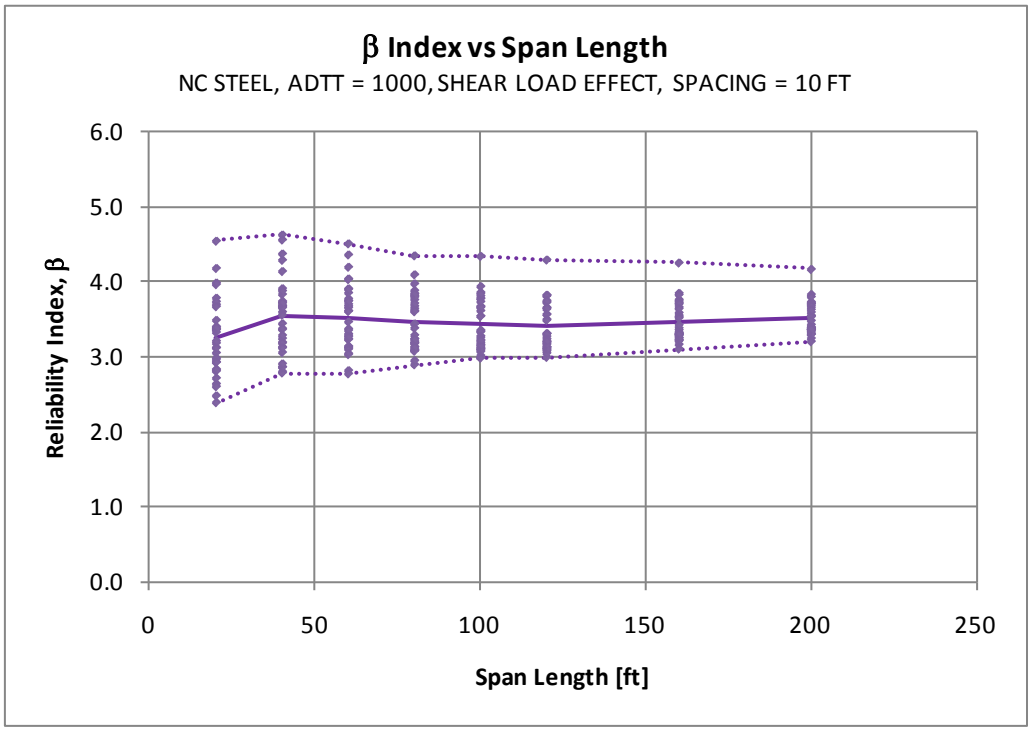
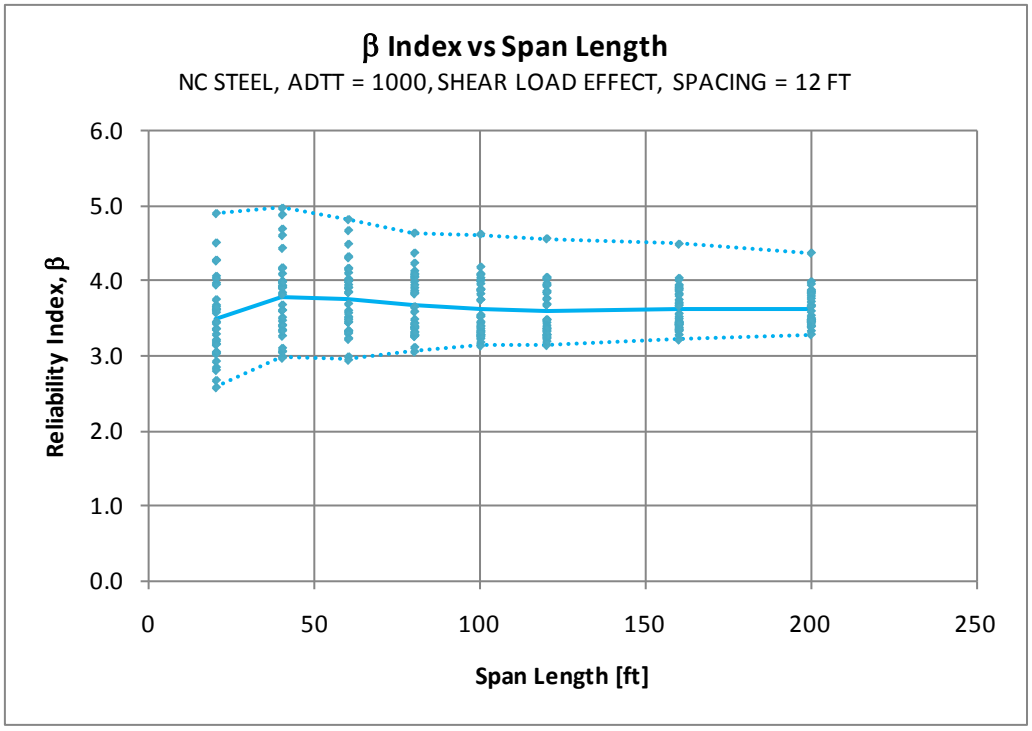
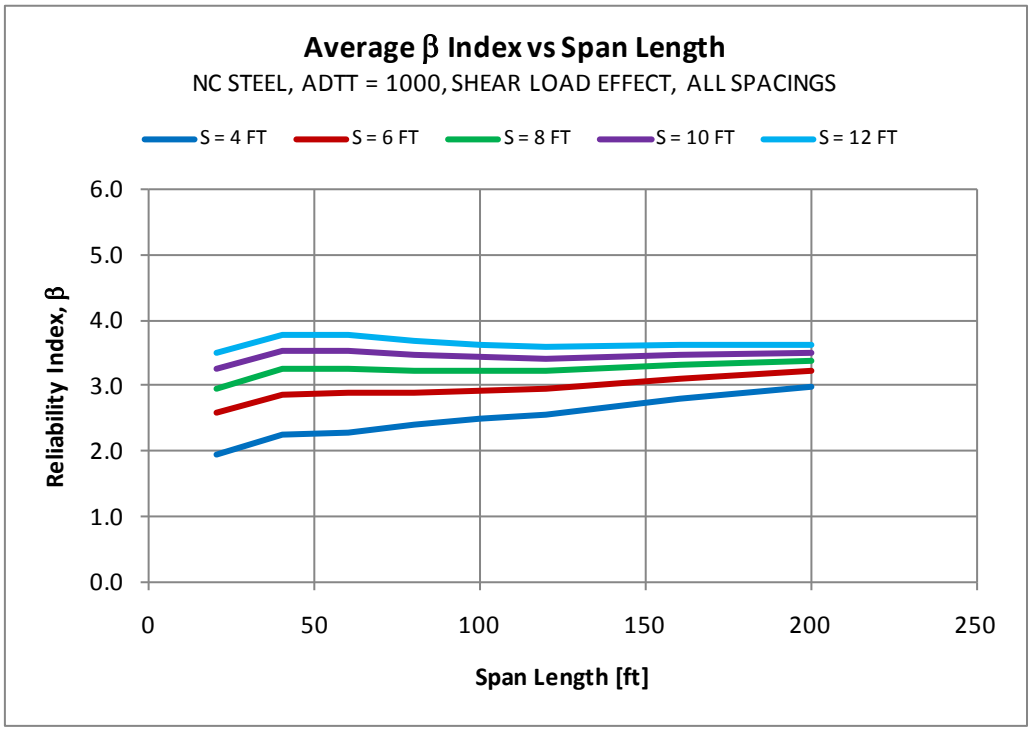
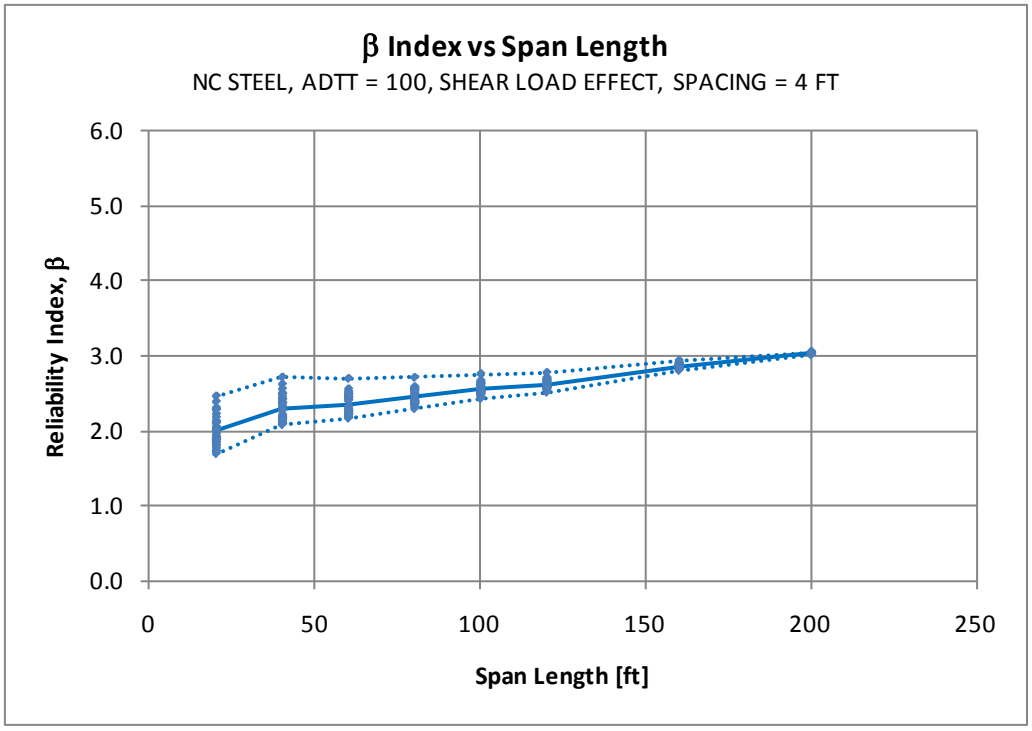
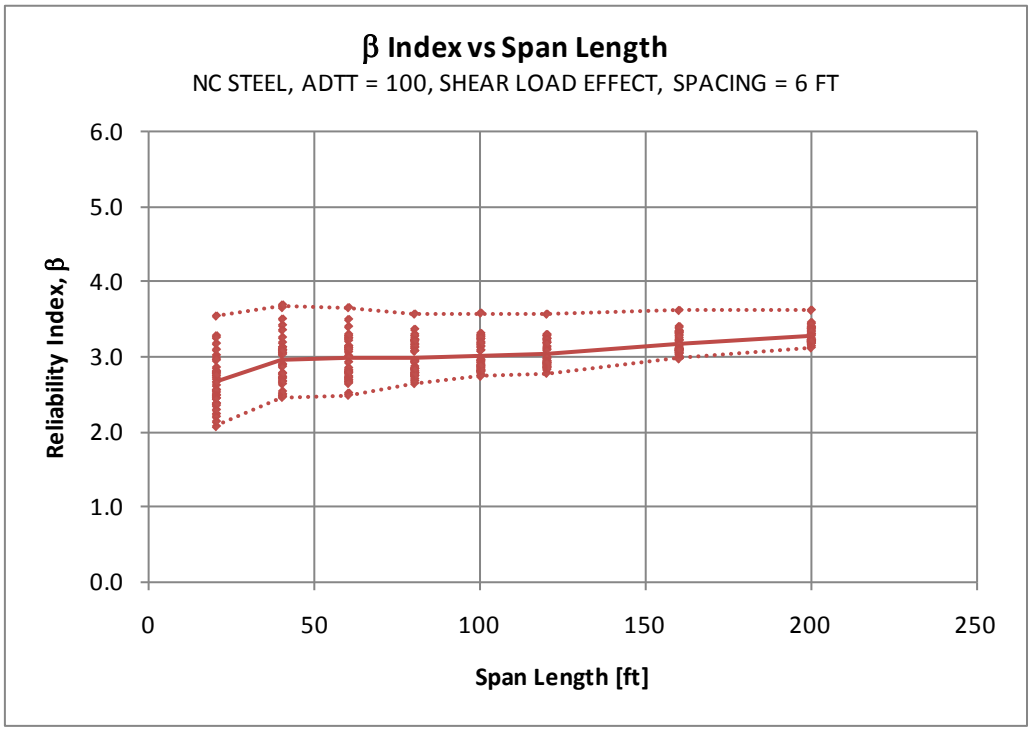
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 1000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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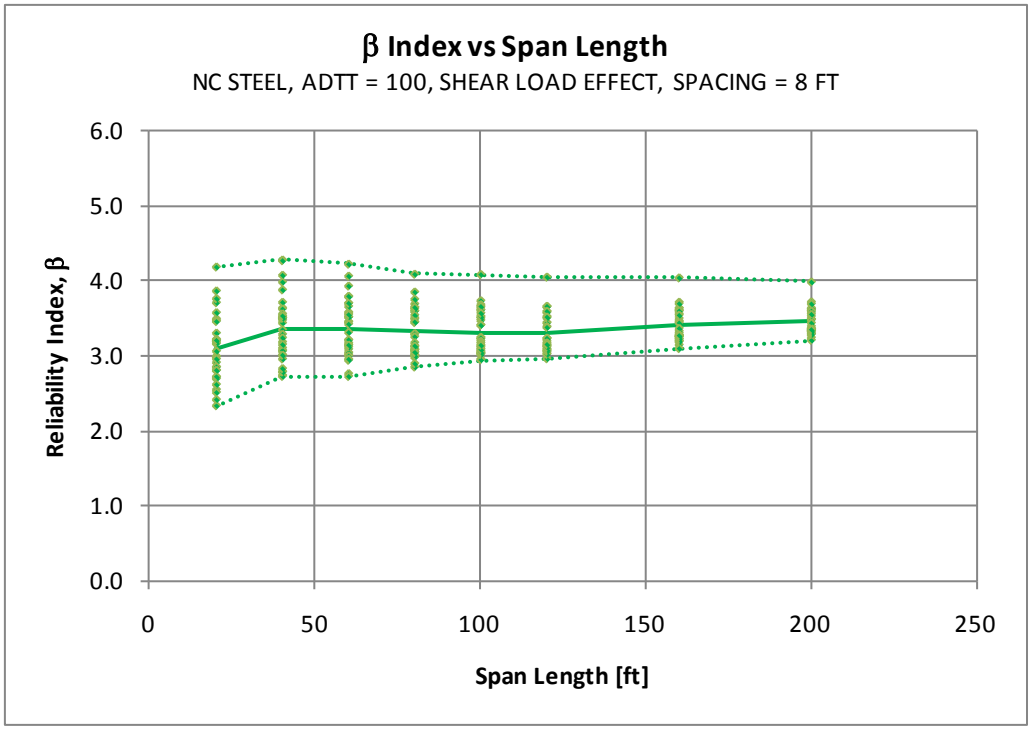
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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4.0
3.0
2.0
1.0
0.0
0
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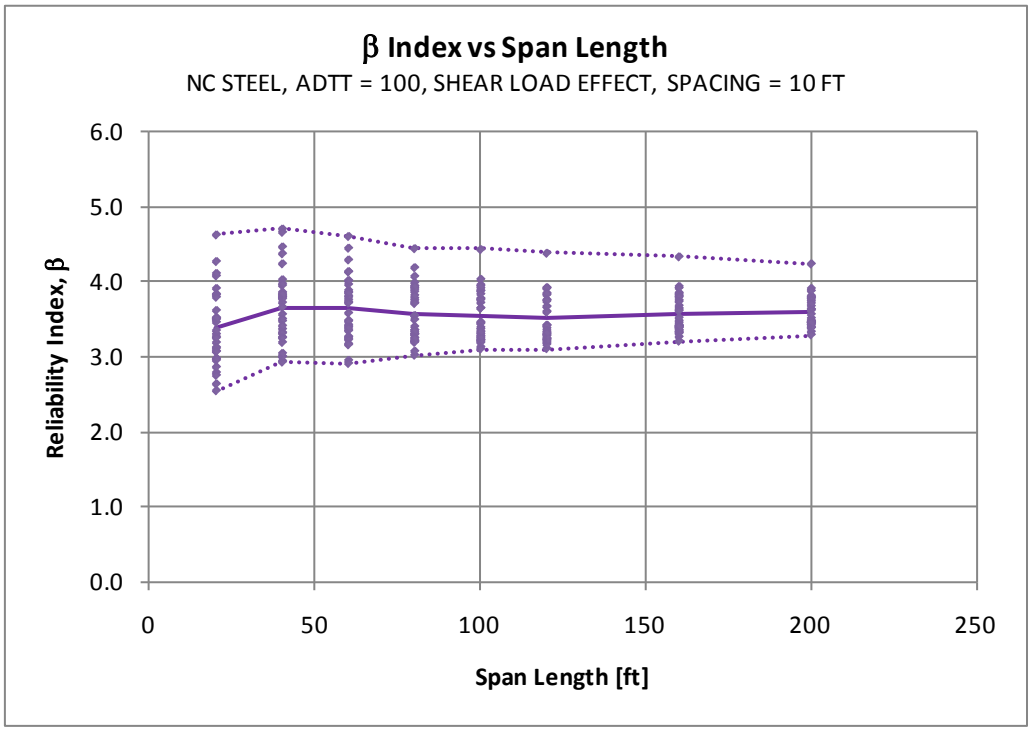
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
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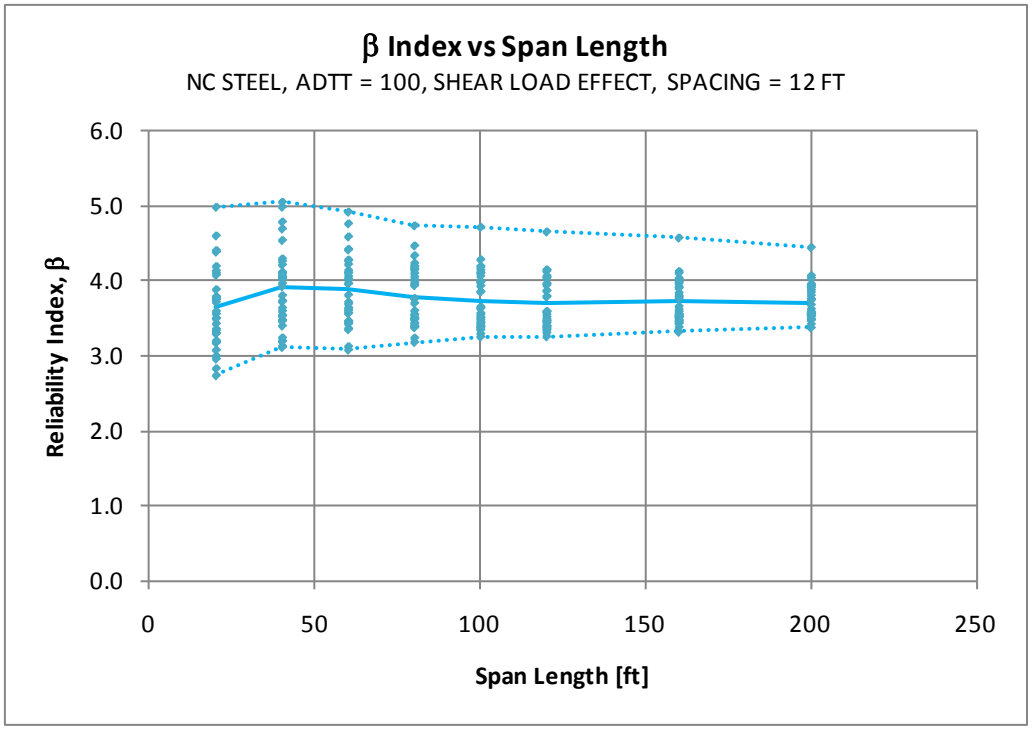
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
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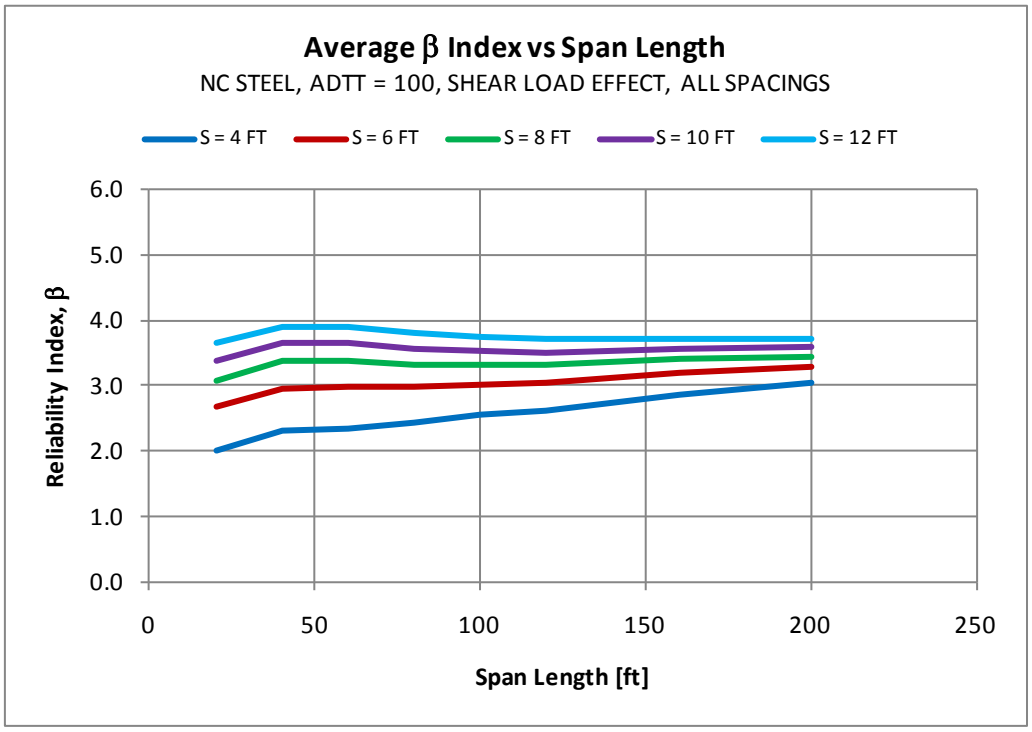
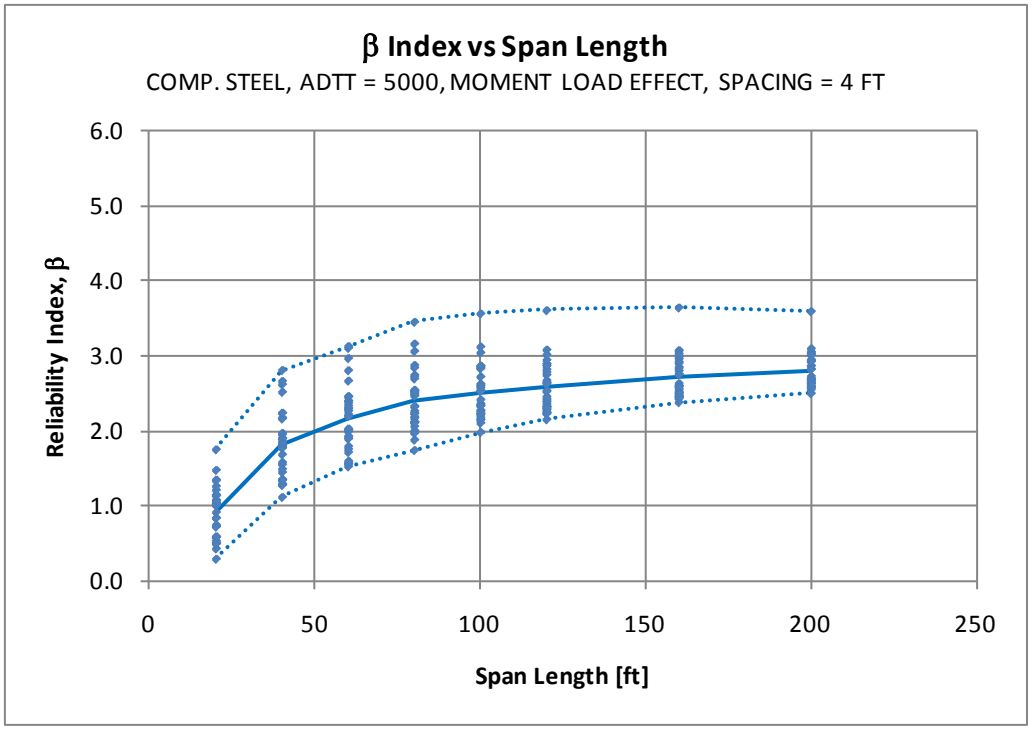
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
NC STEEL, ADTT = 100, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

COMPOSITE STEEL BEAM

PERMIT PLOTS

(TWO LANES)


β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
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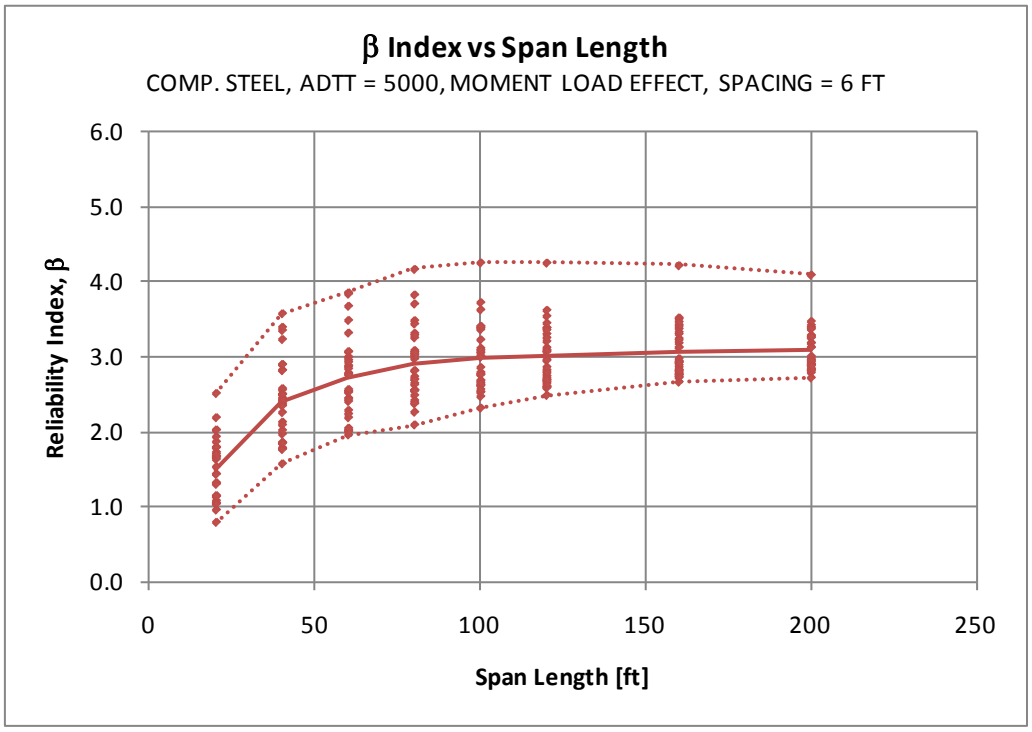
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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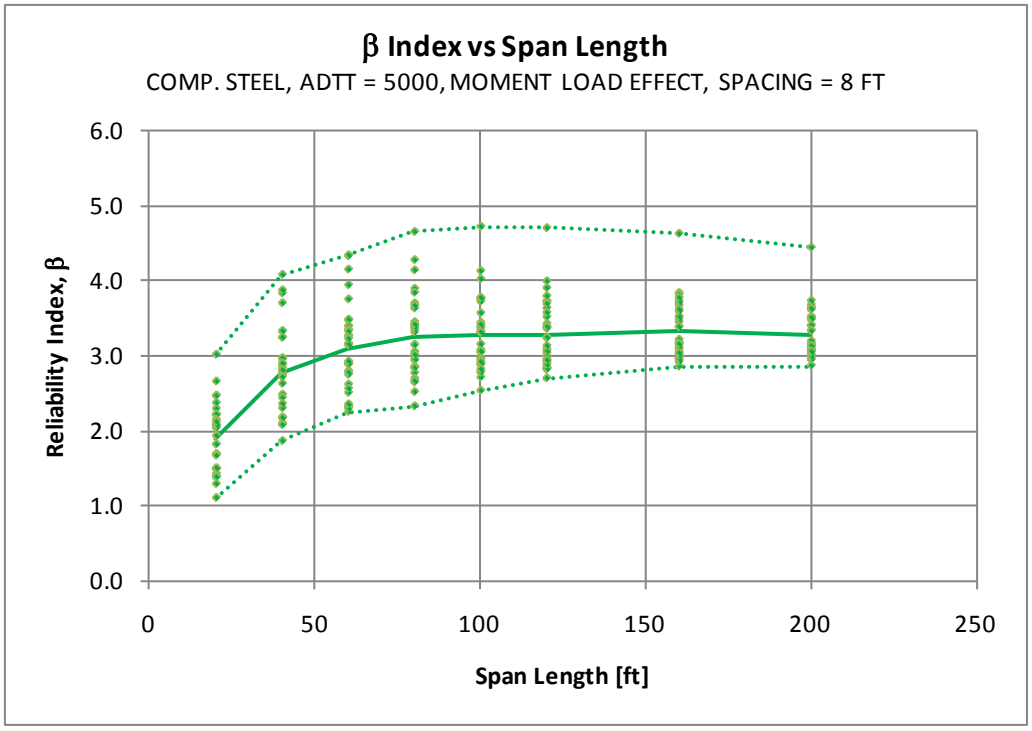
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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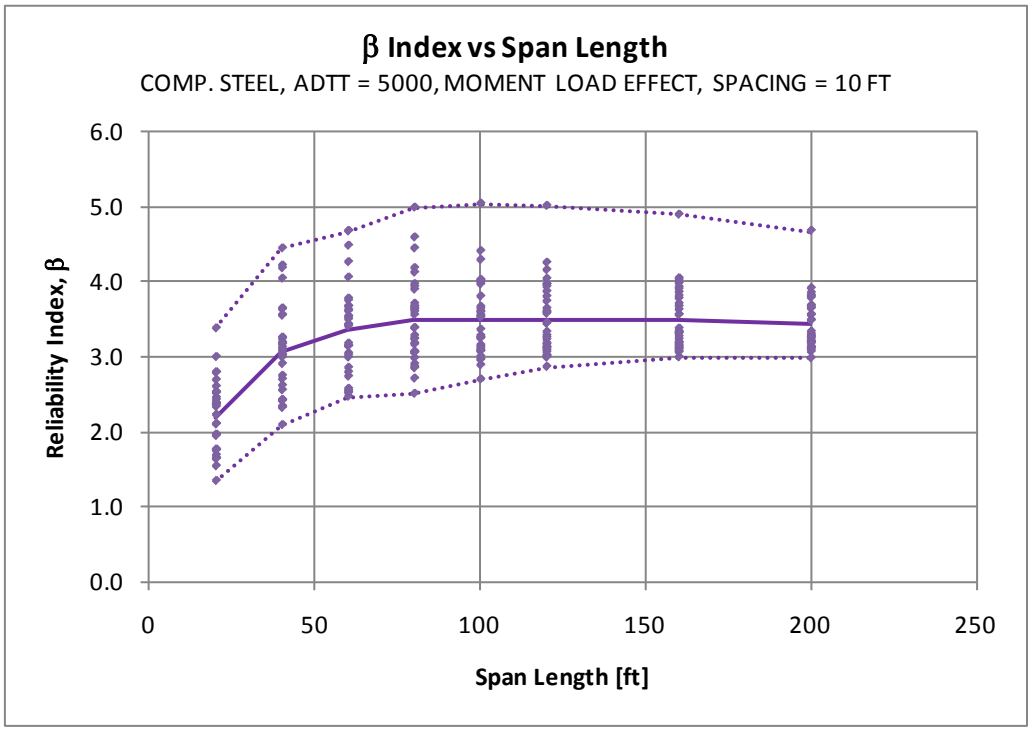
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
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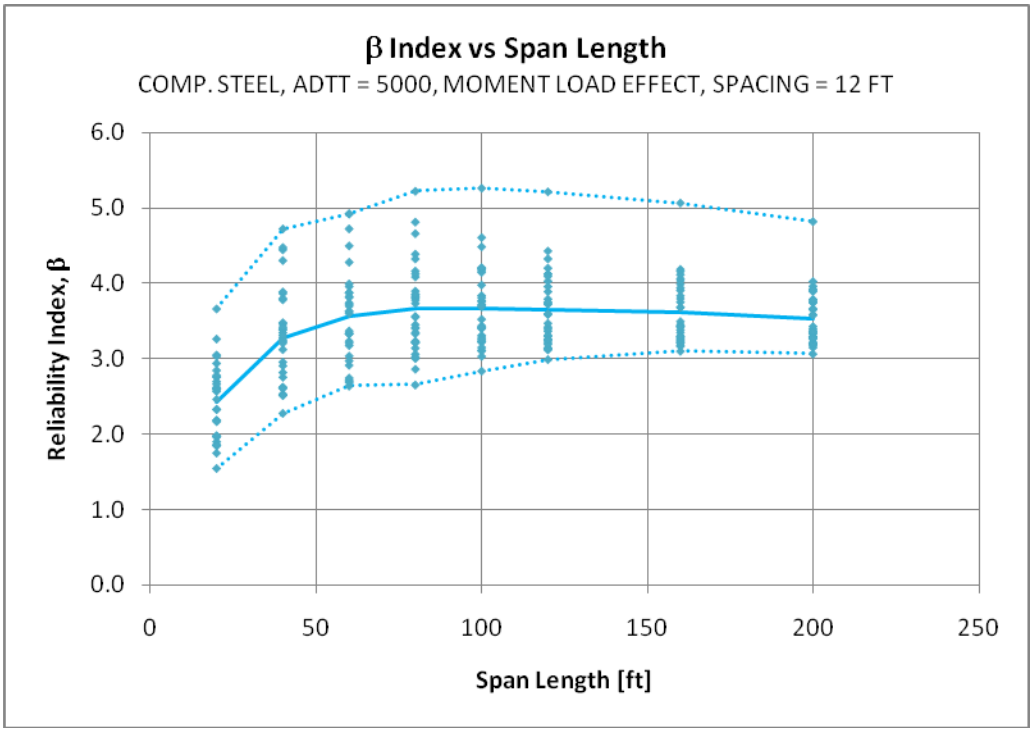
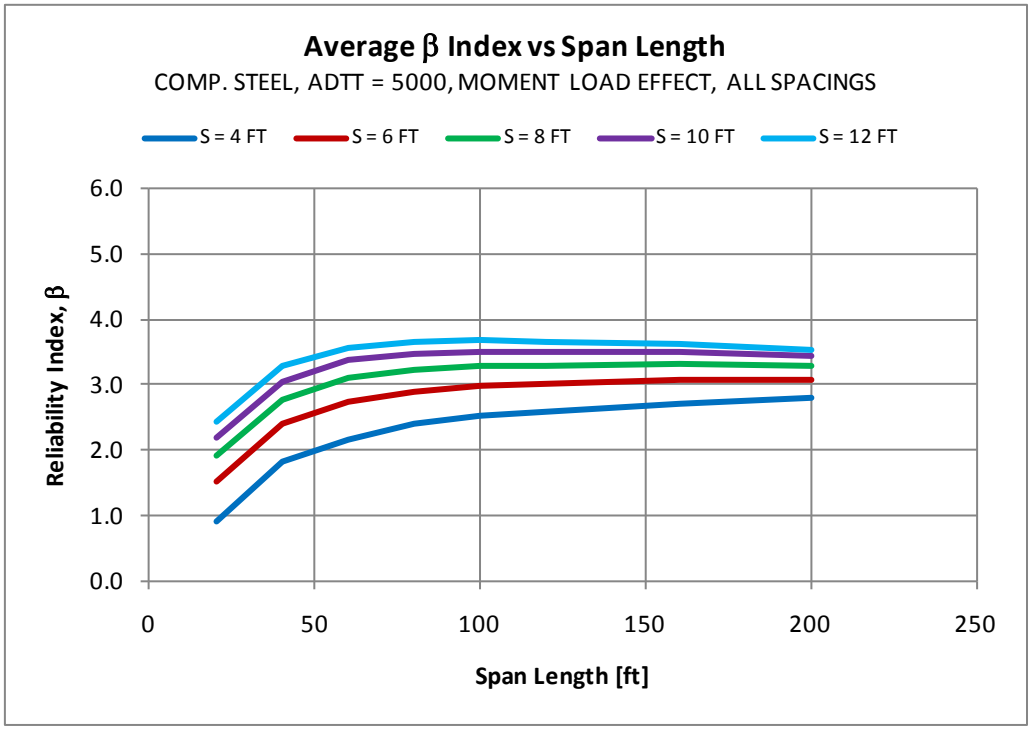
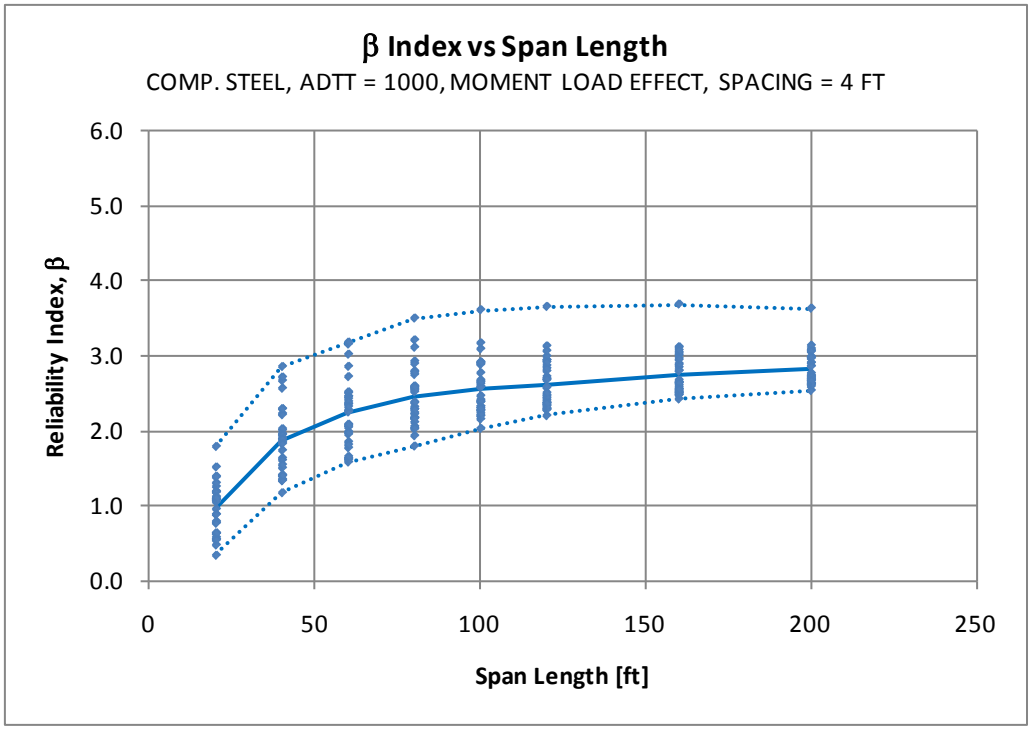
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
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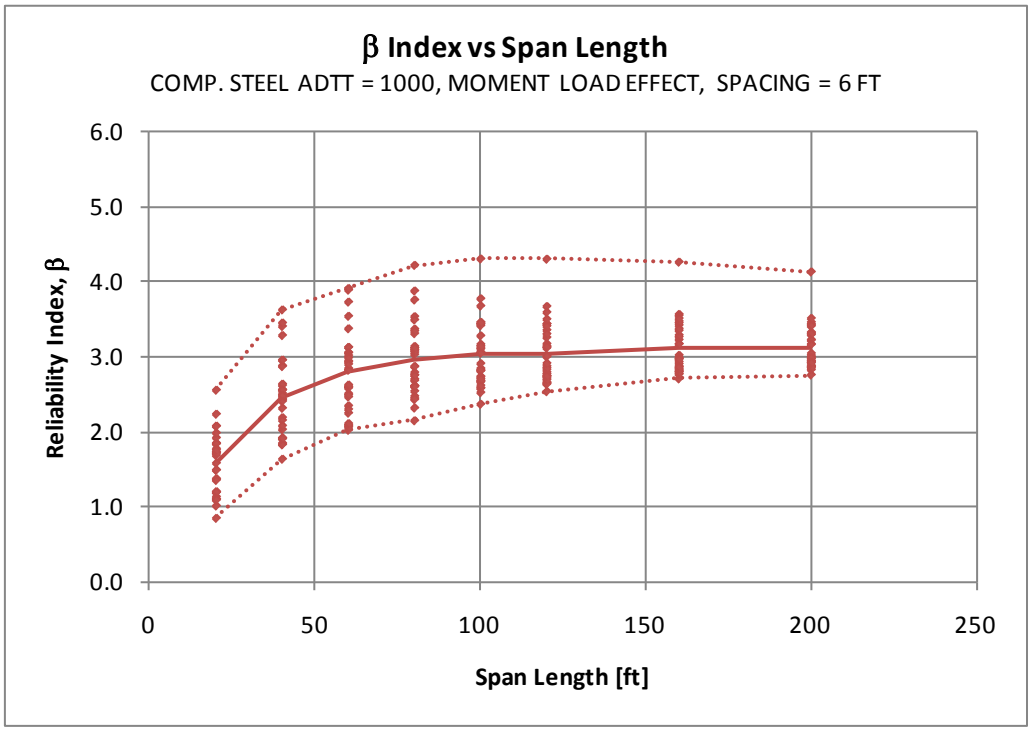
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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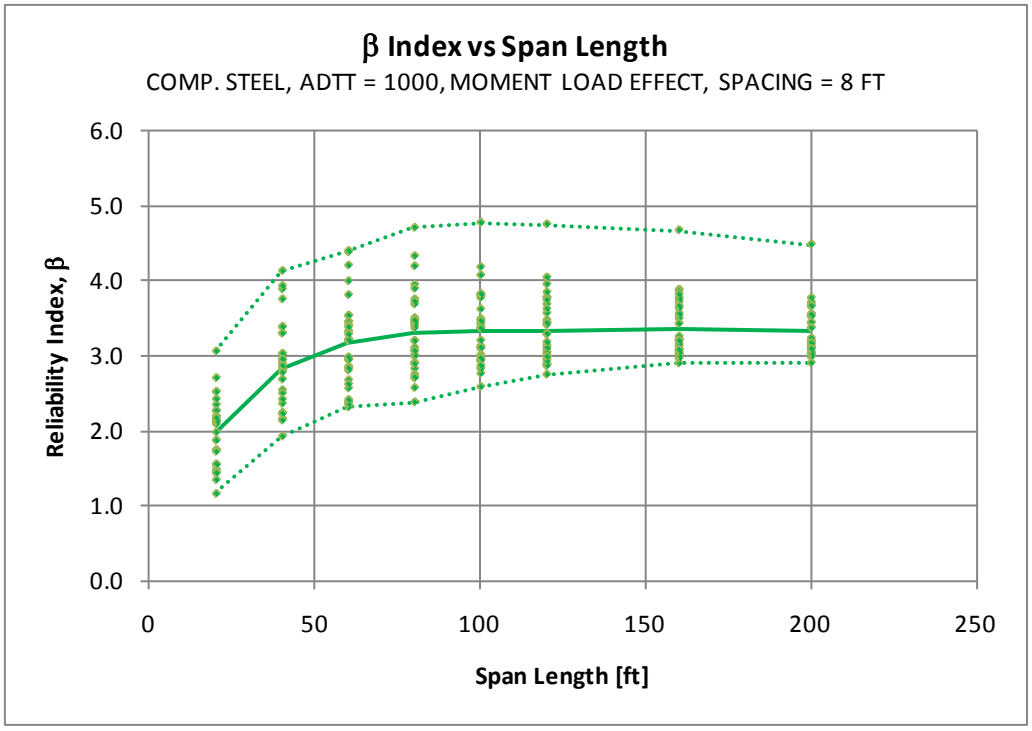
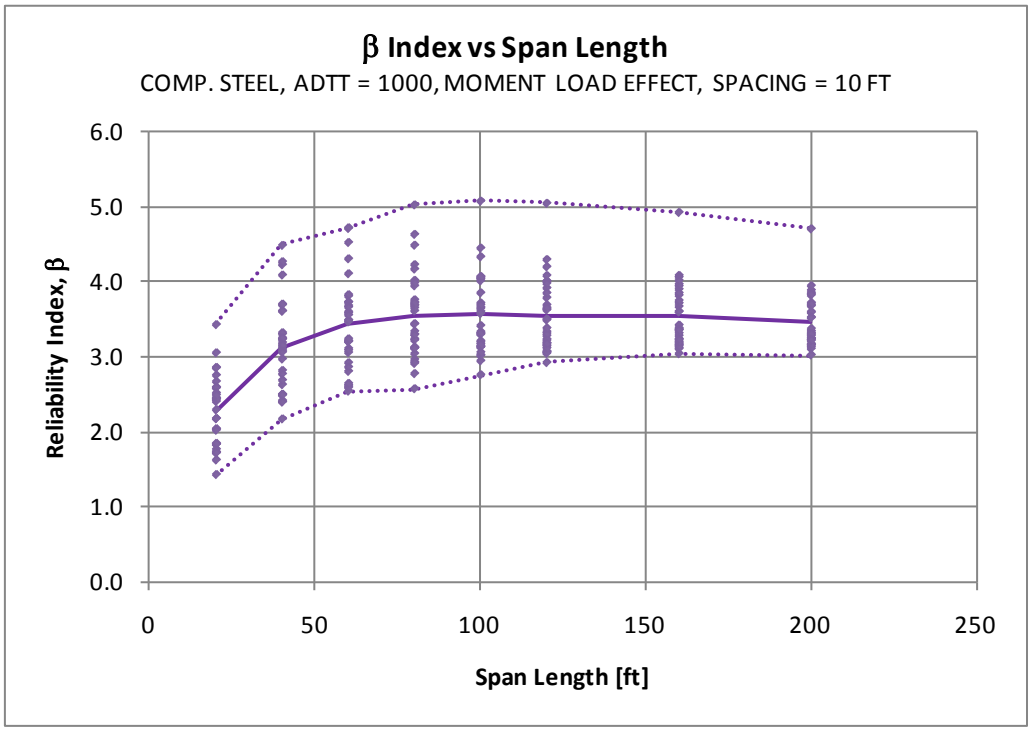
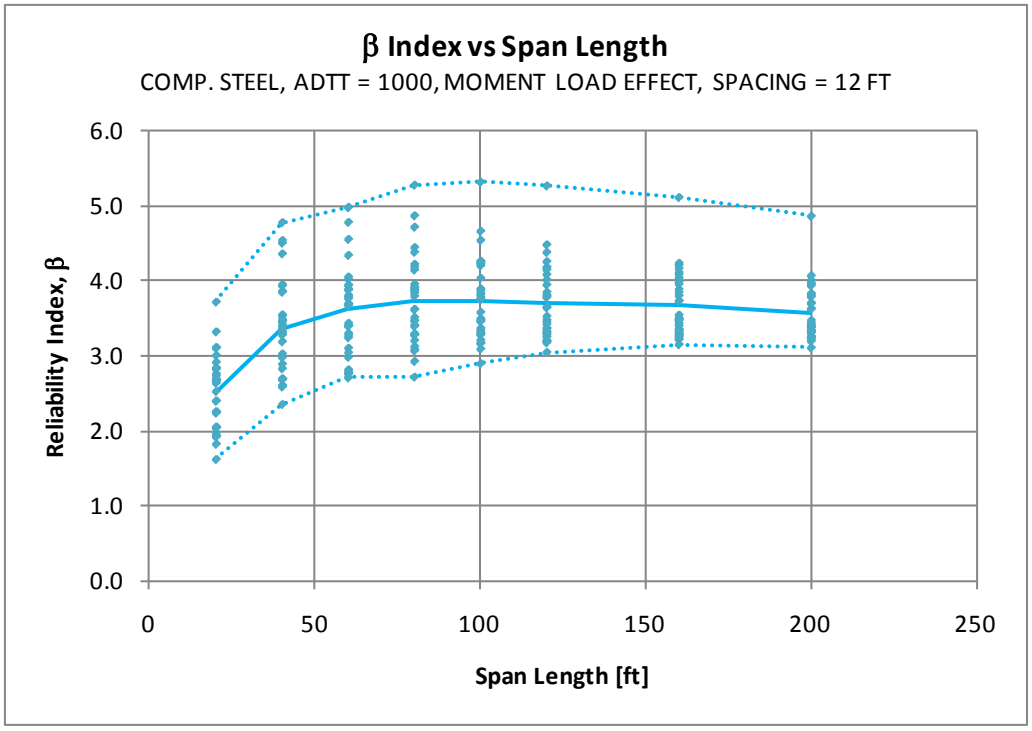
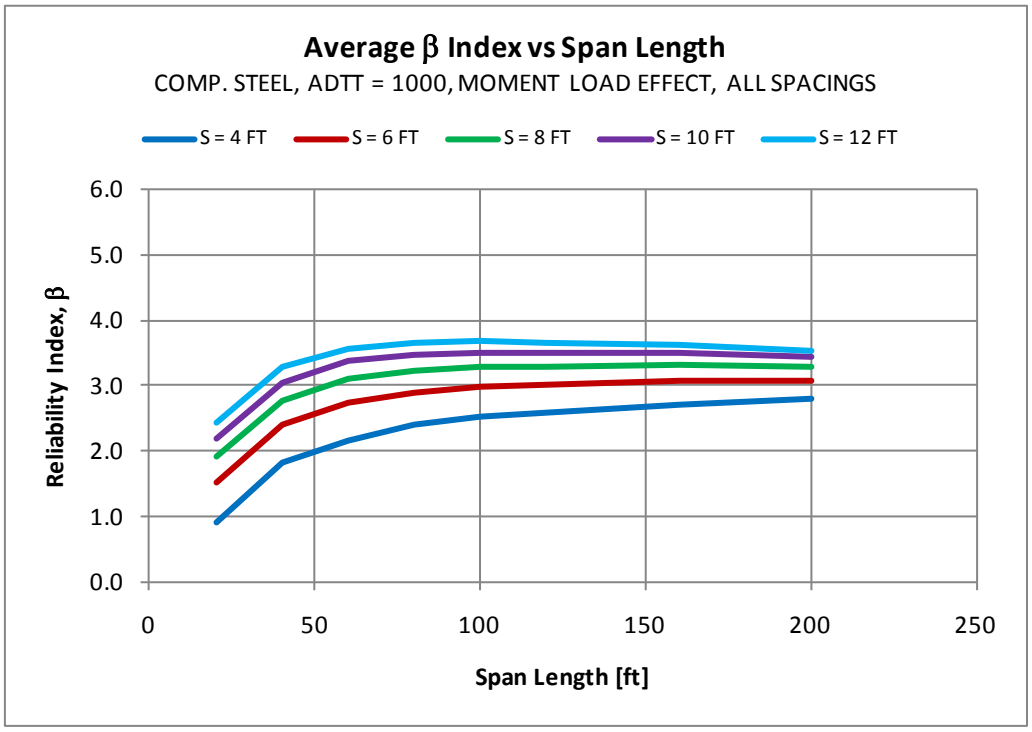
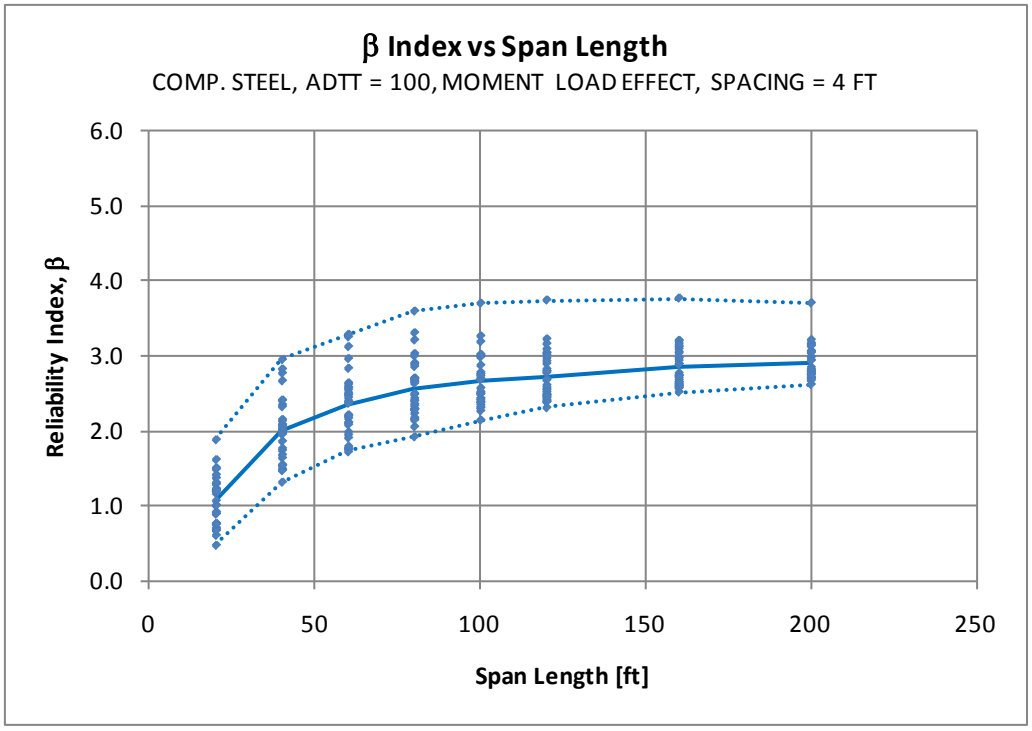
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

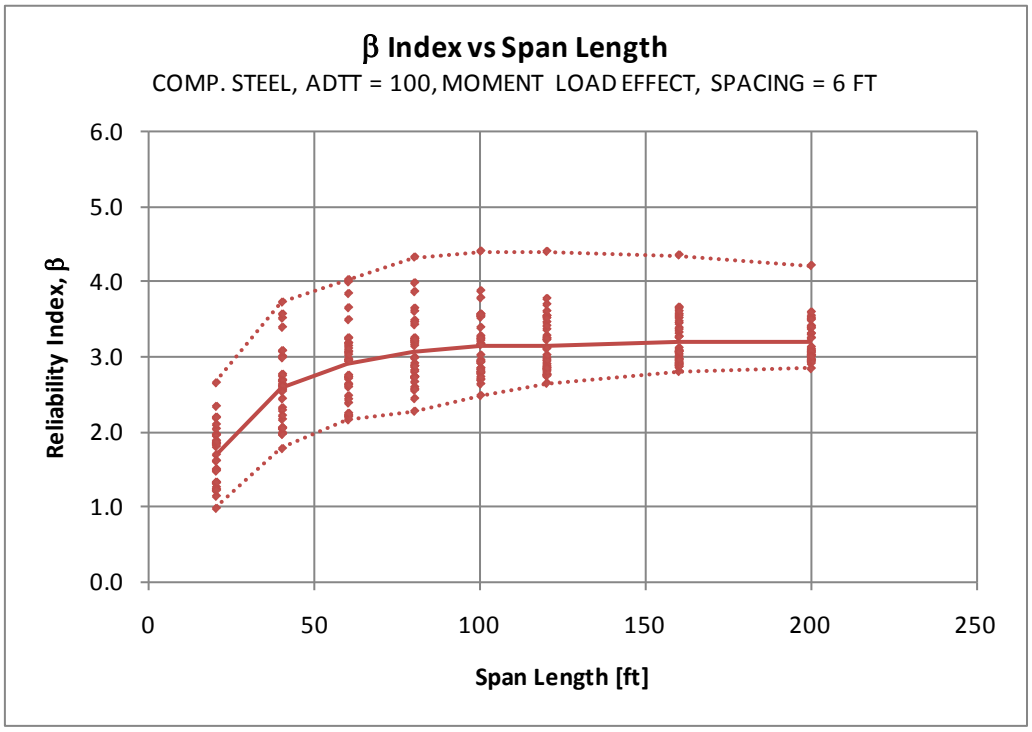

β Index vs Span Length
COMP. STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

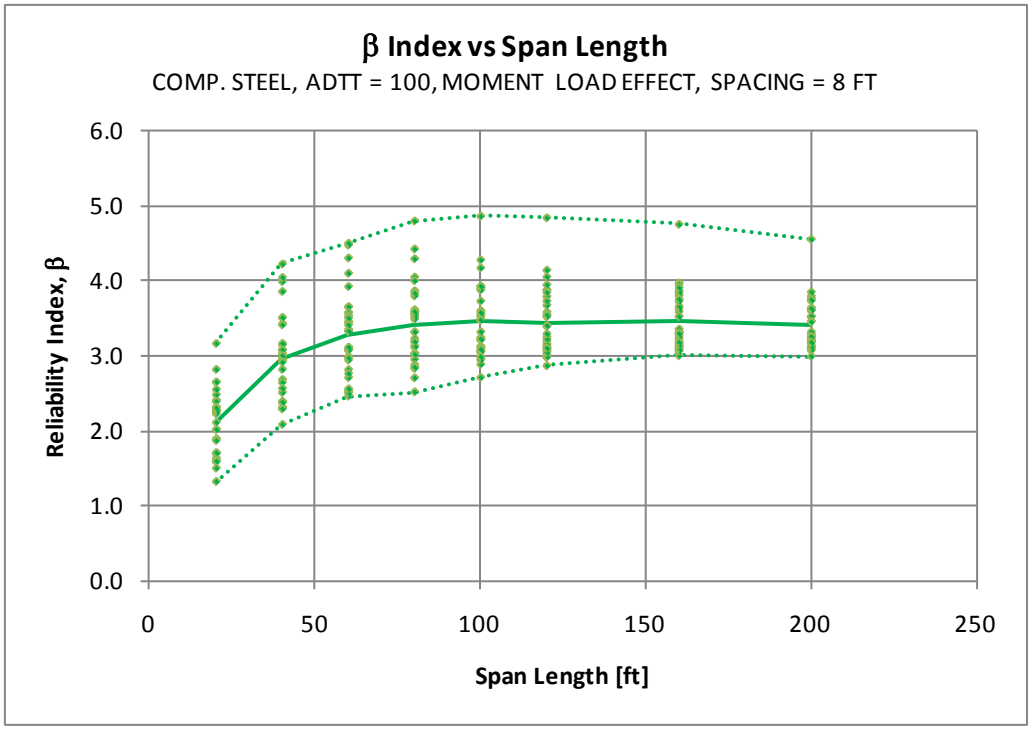

β Index vs Span Length
COMP. STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

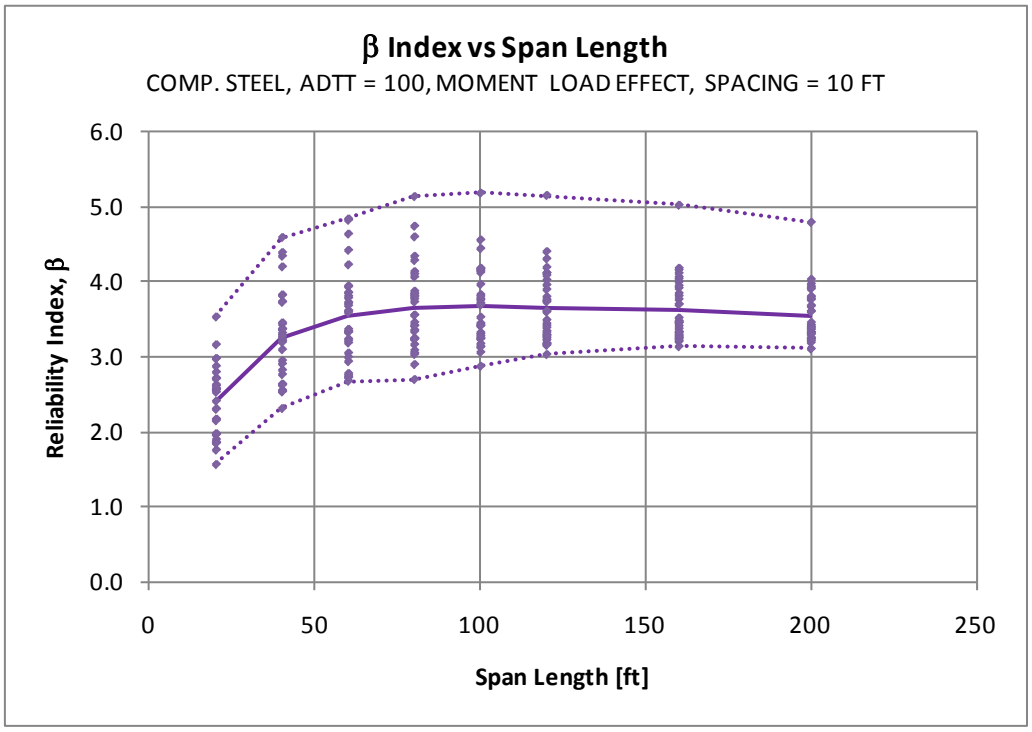

β Index vs Span Length
COMP. STEEL, ADTT = 1000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

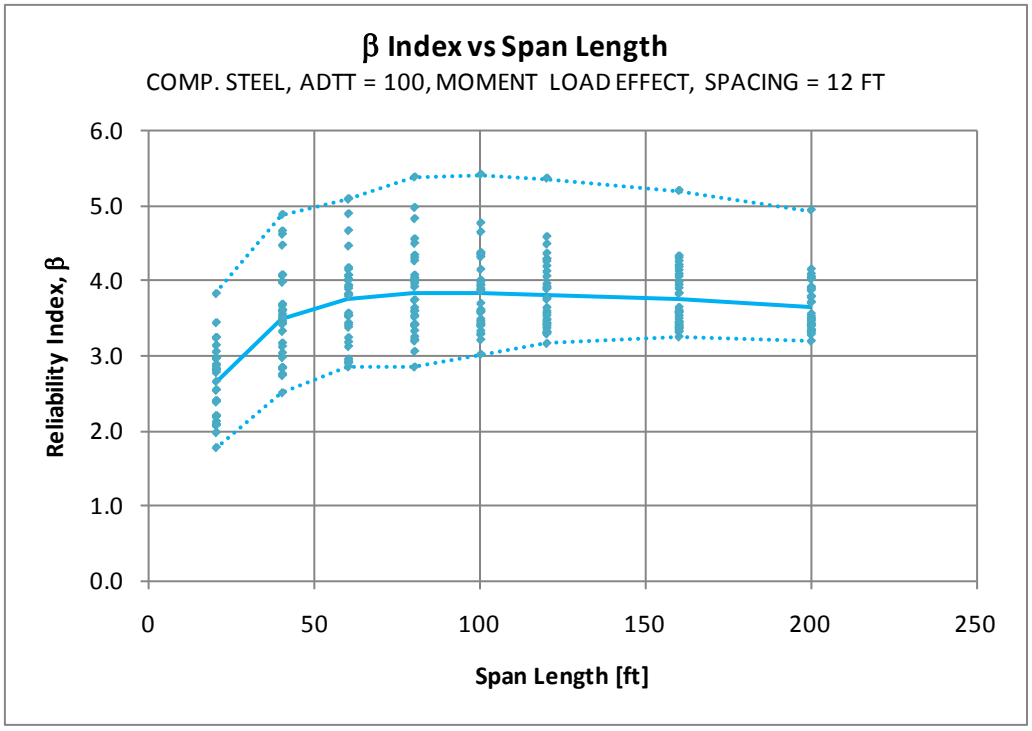

Average β Index vs Span Length
COMP. STEEL, ADTT = 1000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

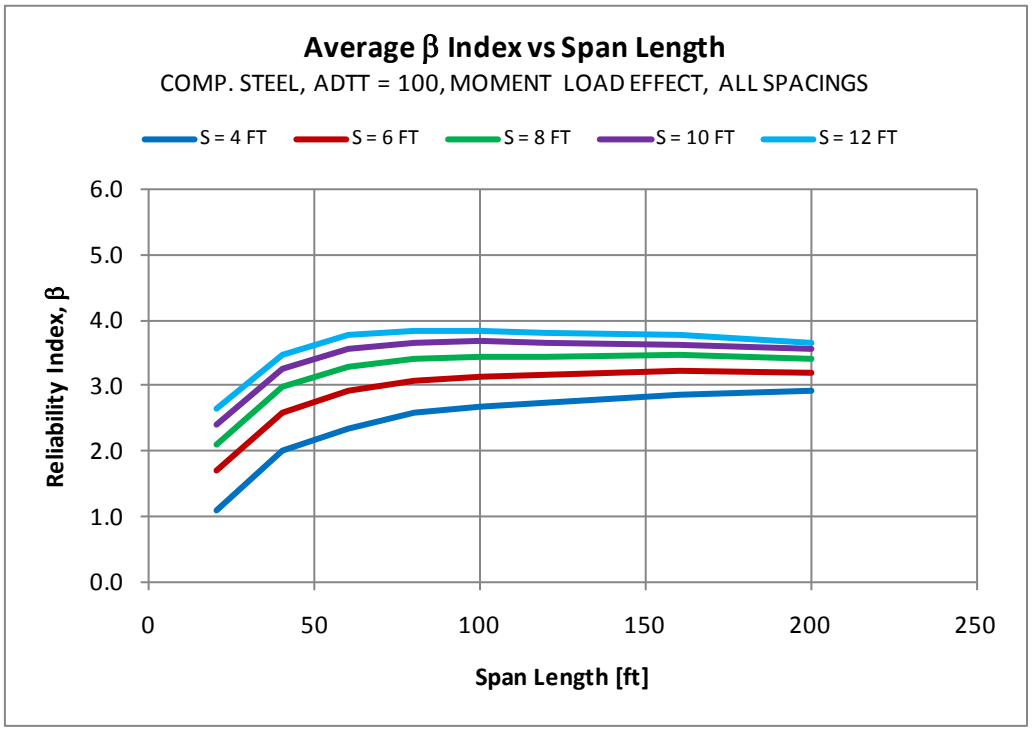

β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

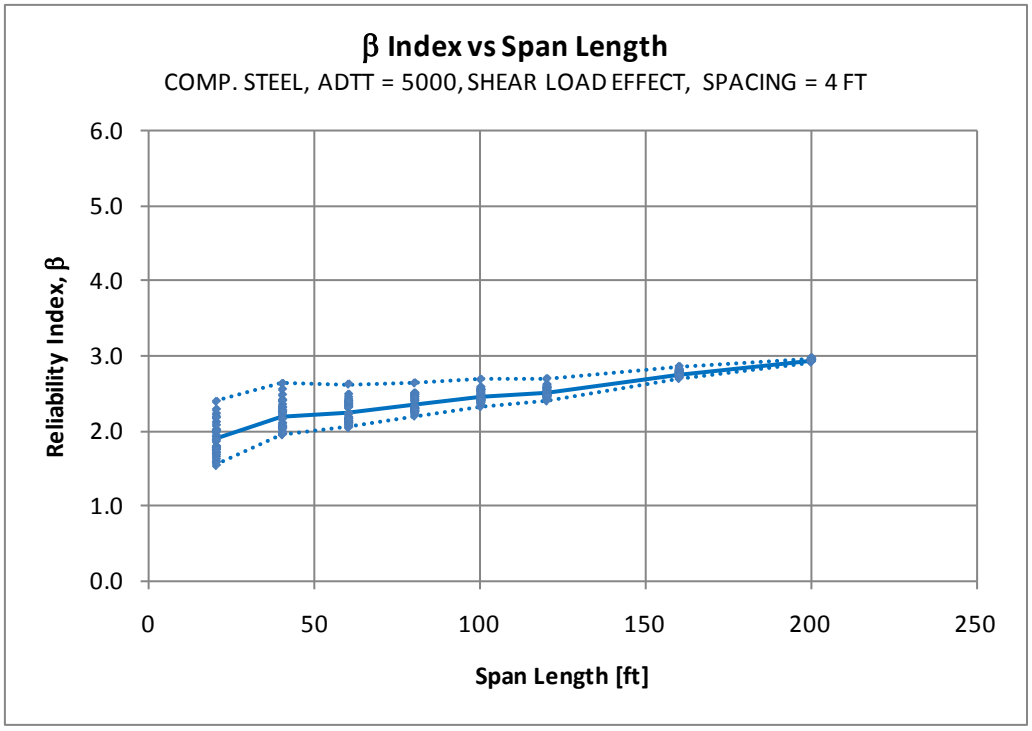

β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

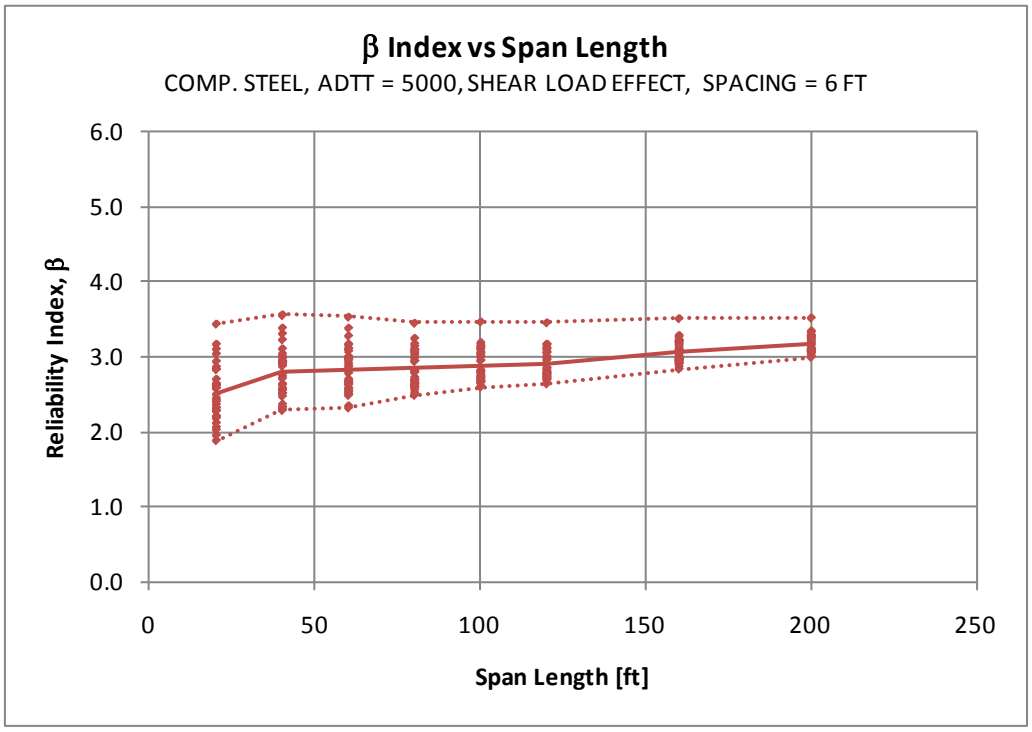

β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

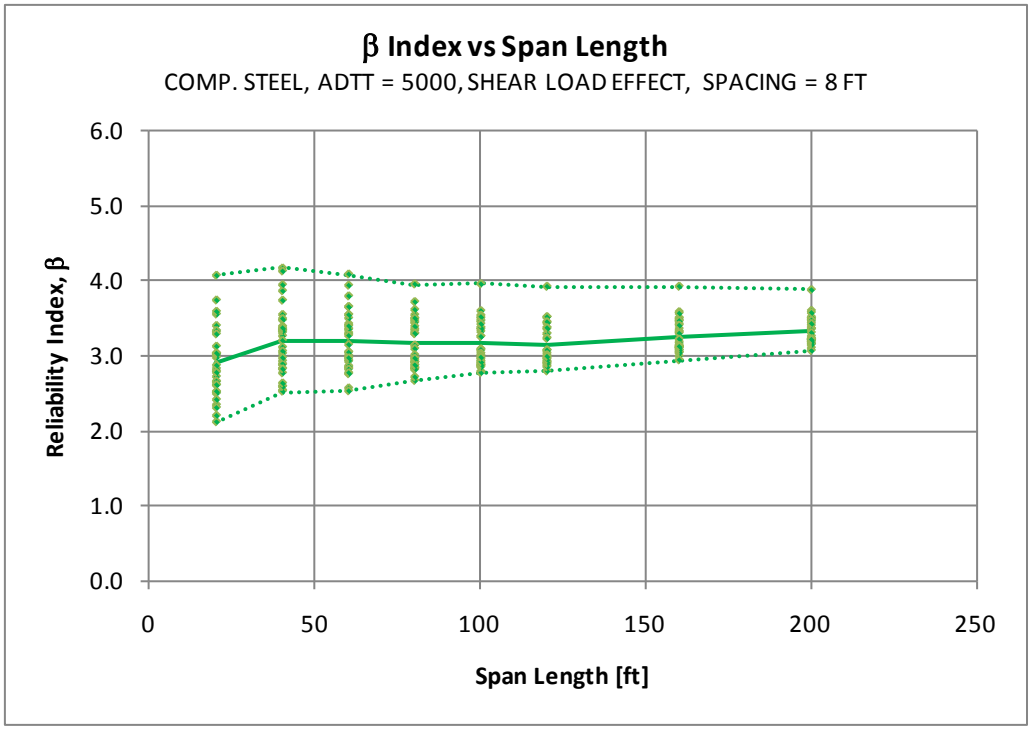

β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
COMP. STEEL, ADTT = 100, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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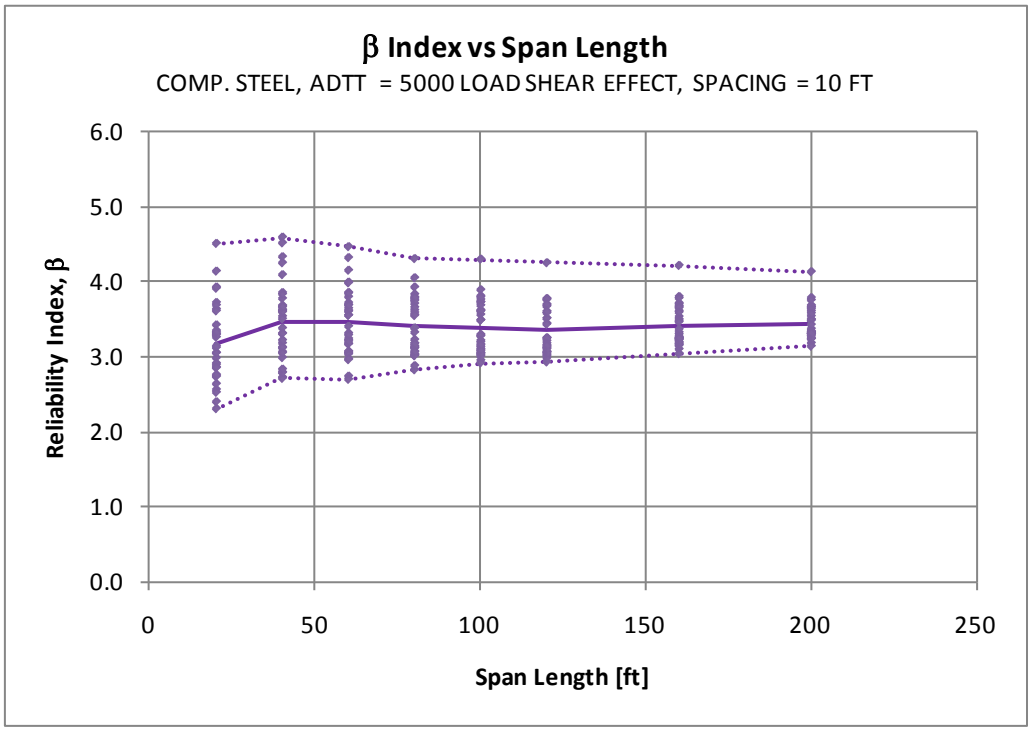
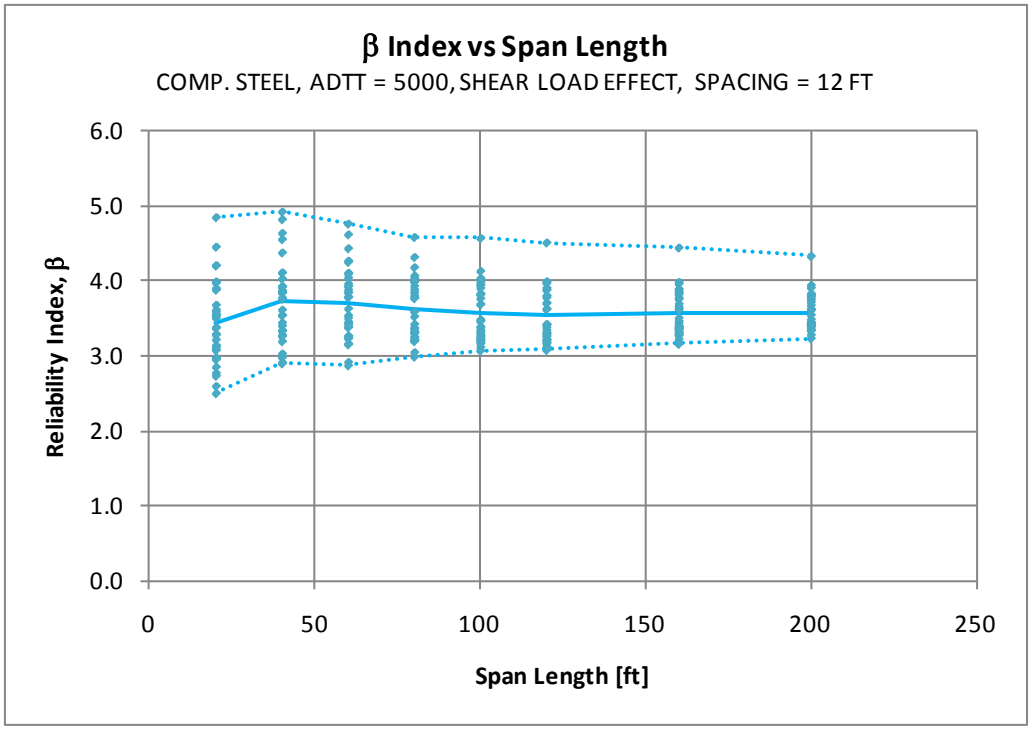
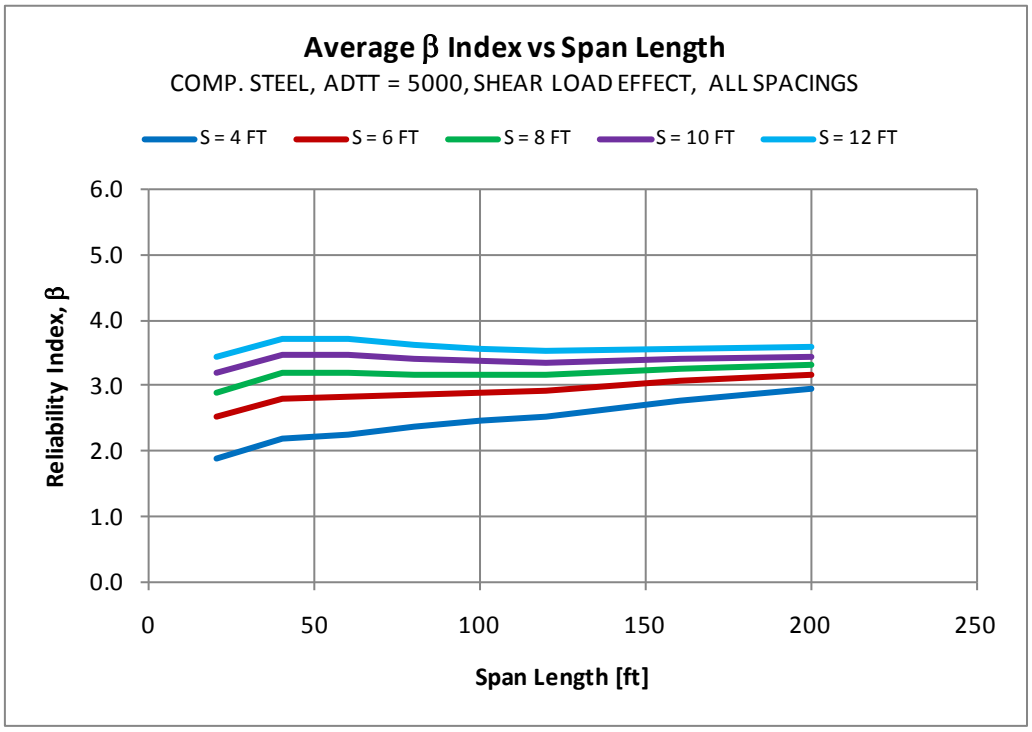
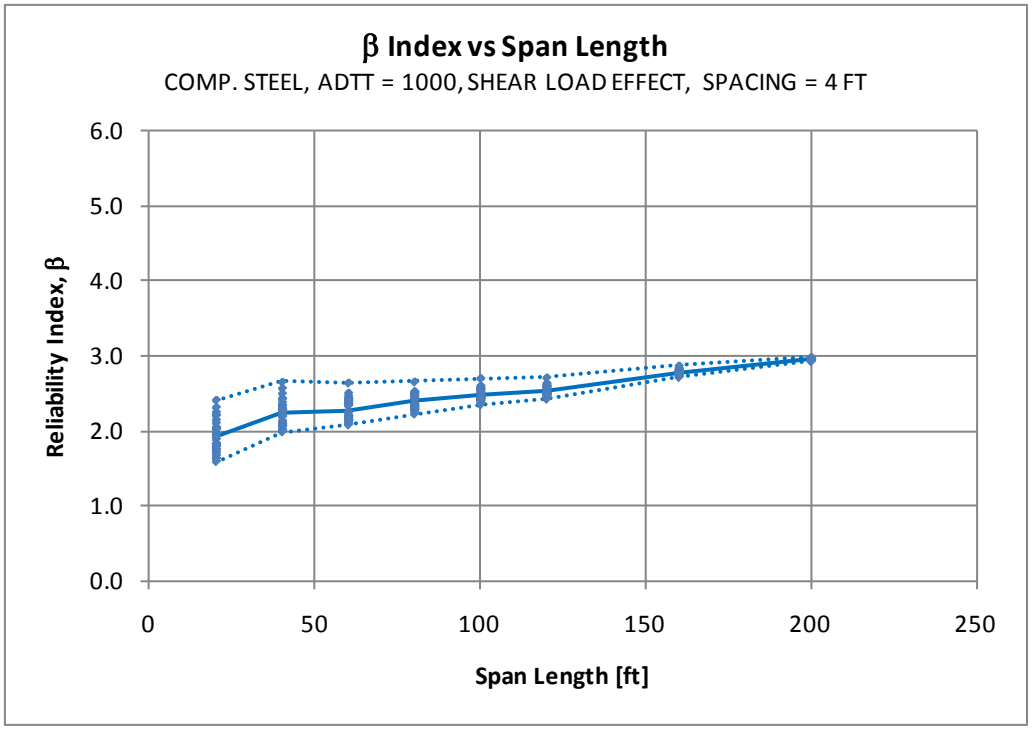
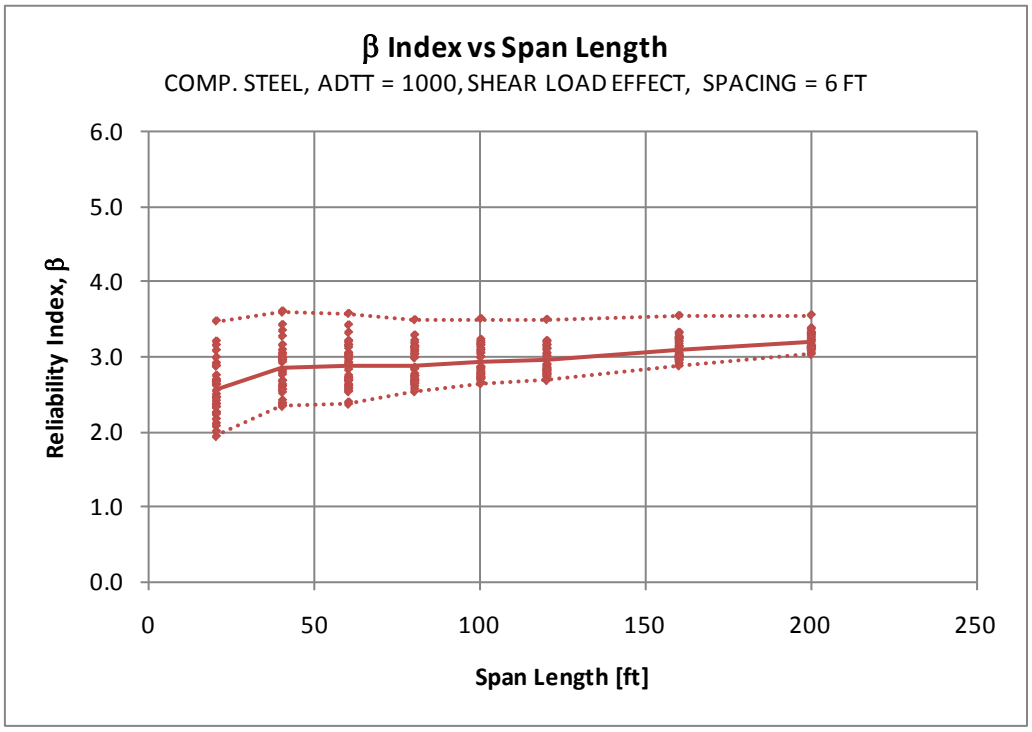
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
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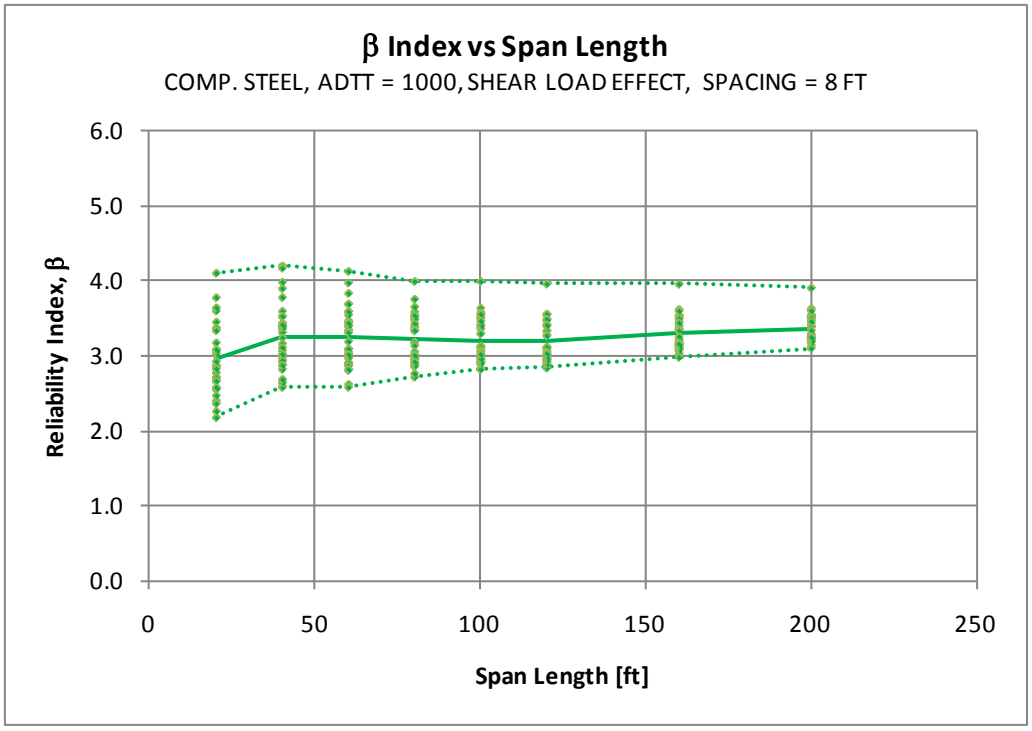
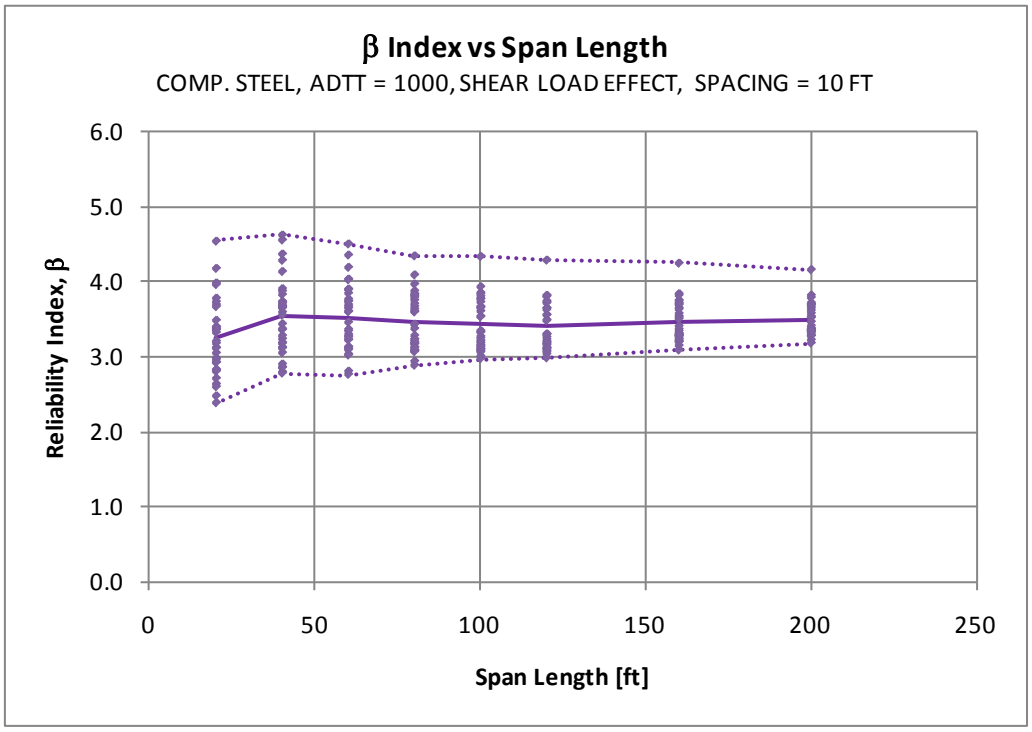
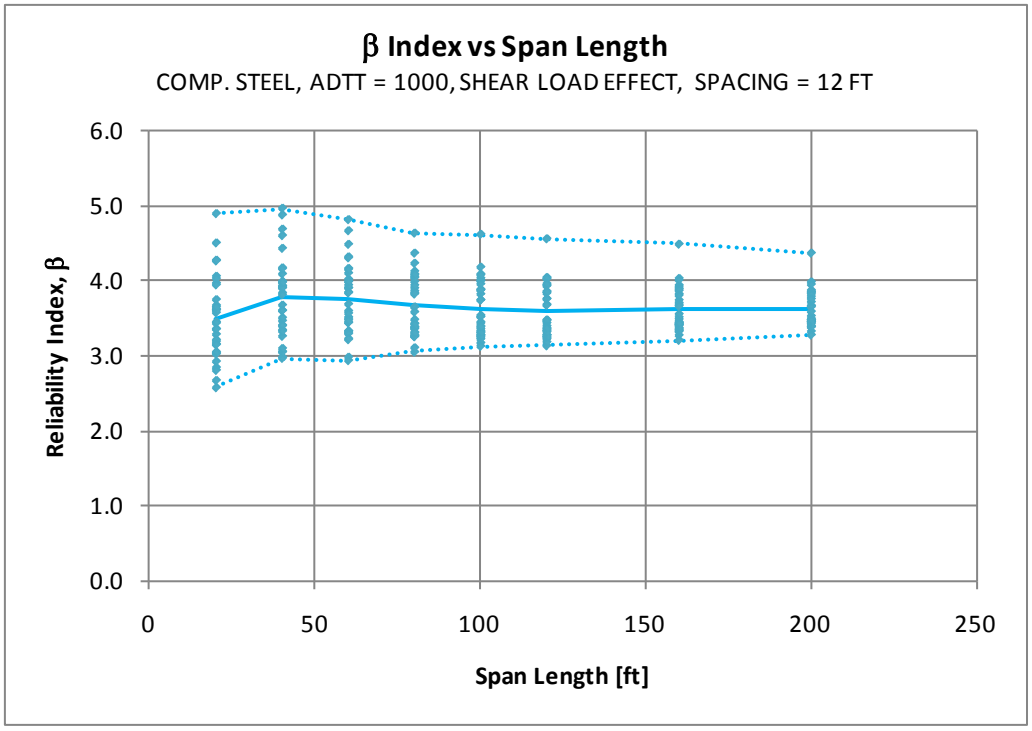
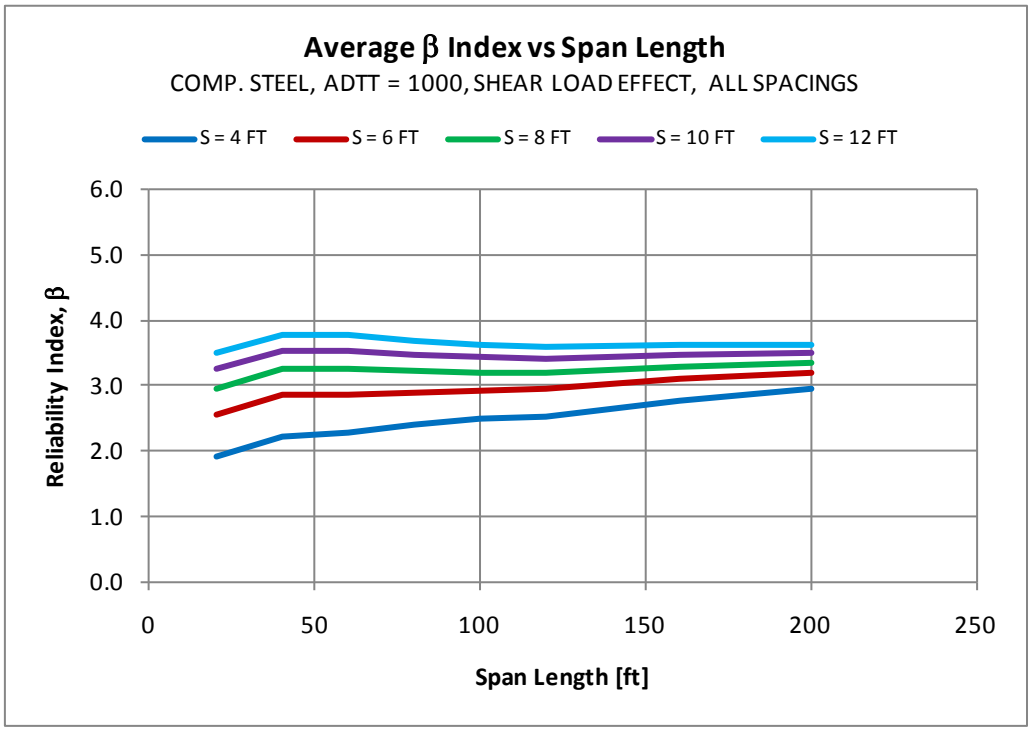
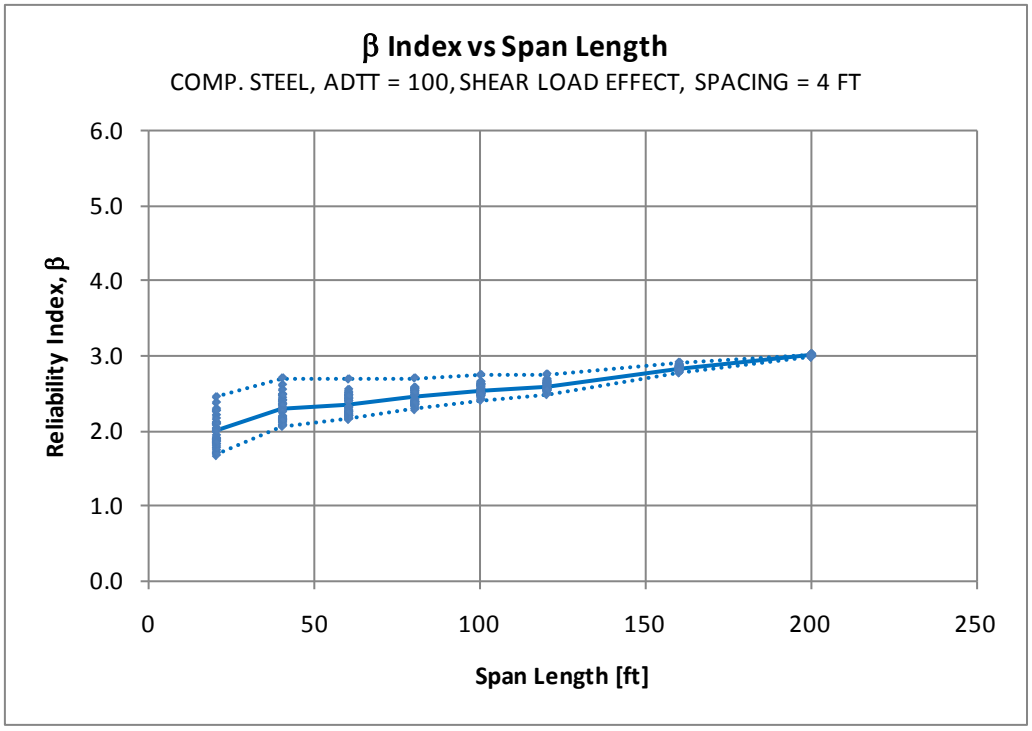
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

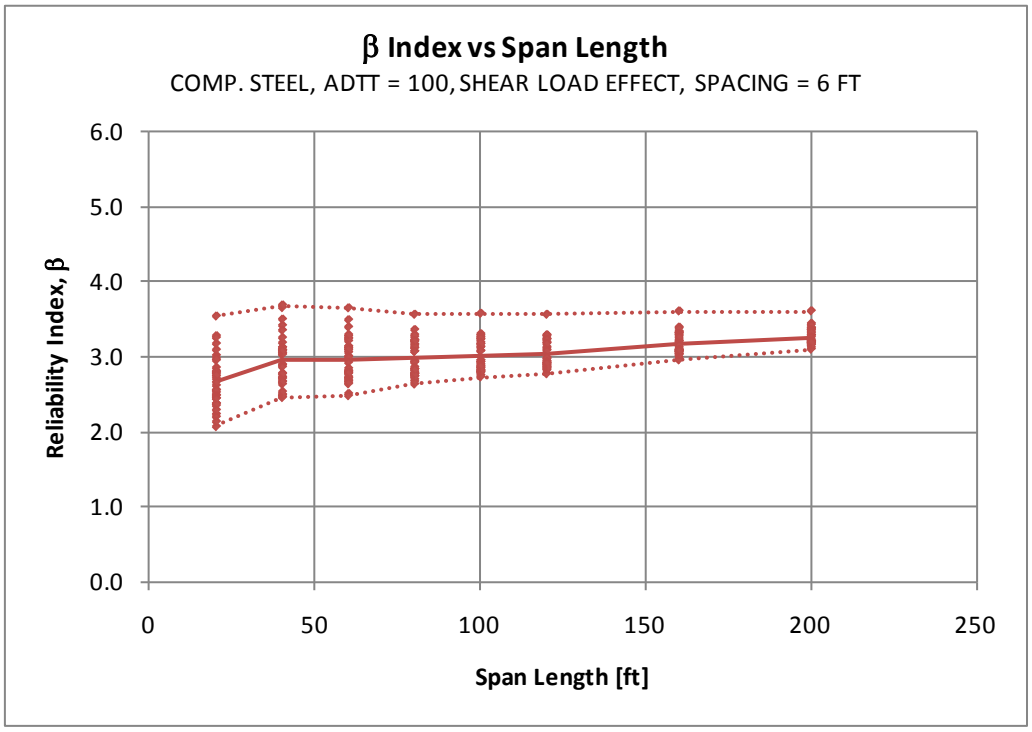

β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

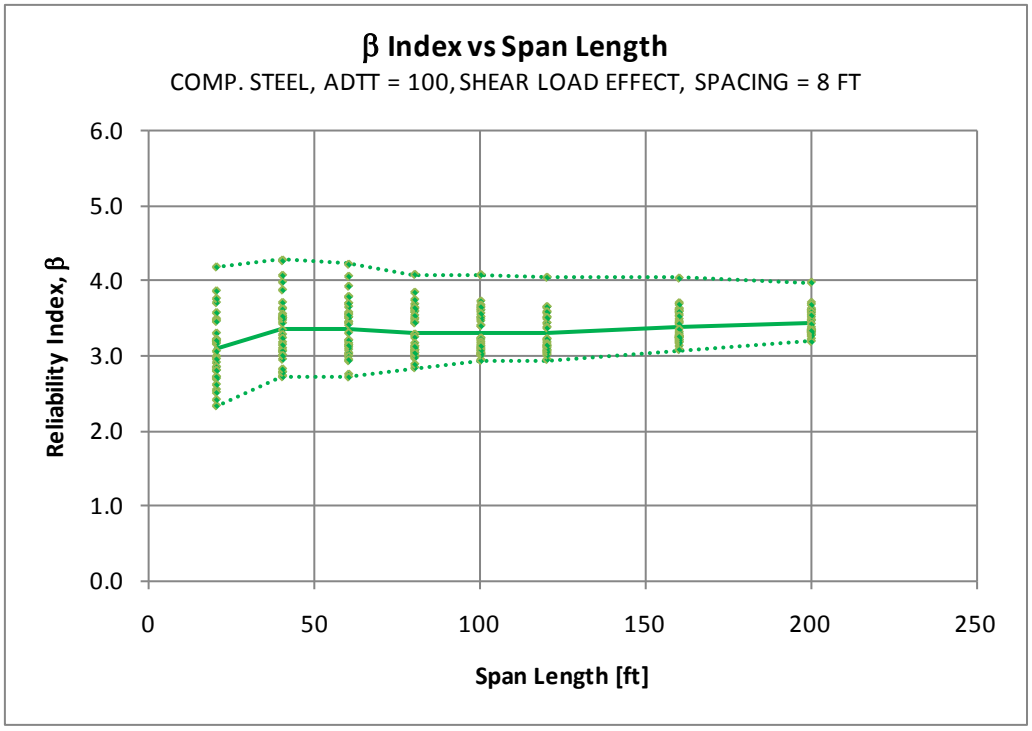

β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


Average β Index vs Span Length
COMP. STEEL, ADTT = 1000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
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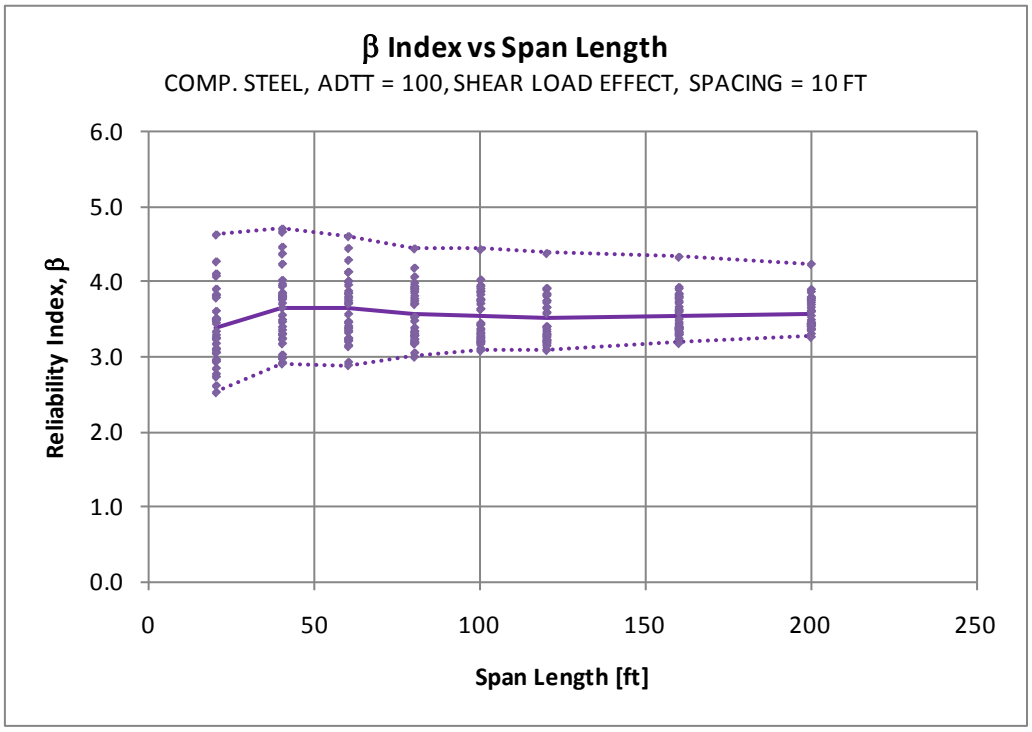
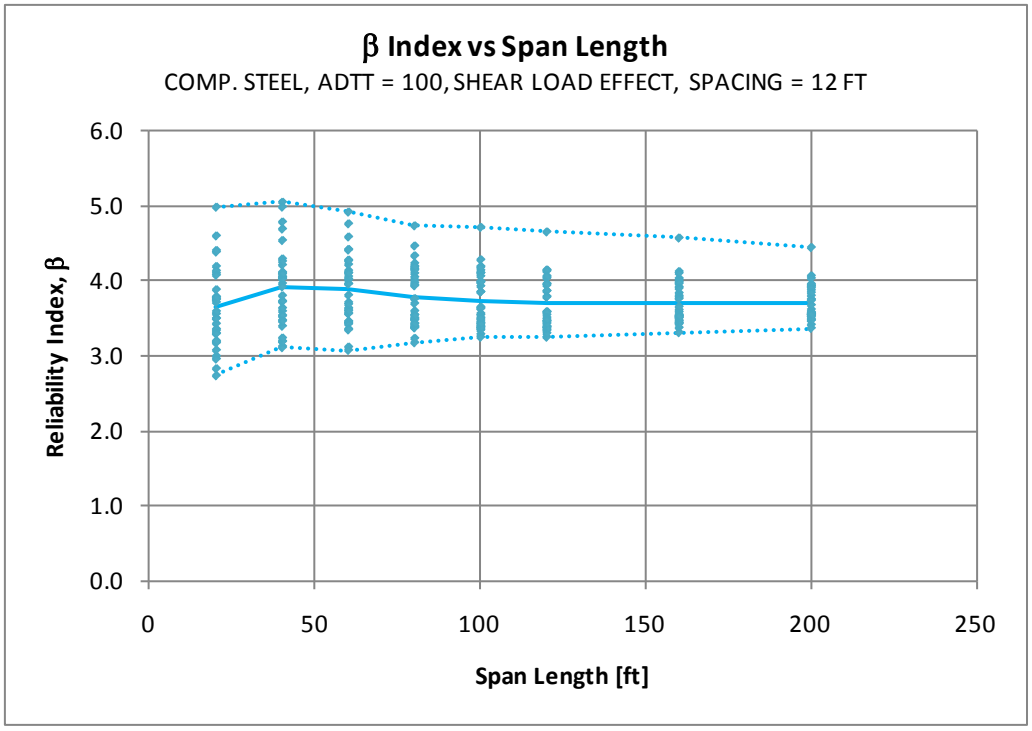
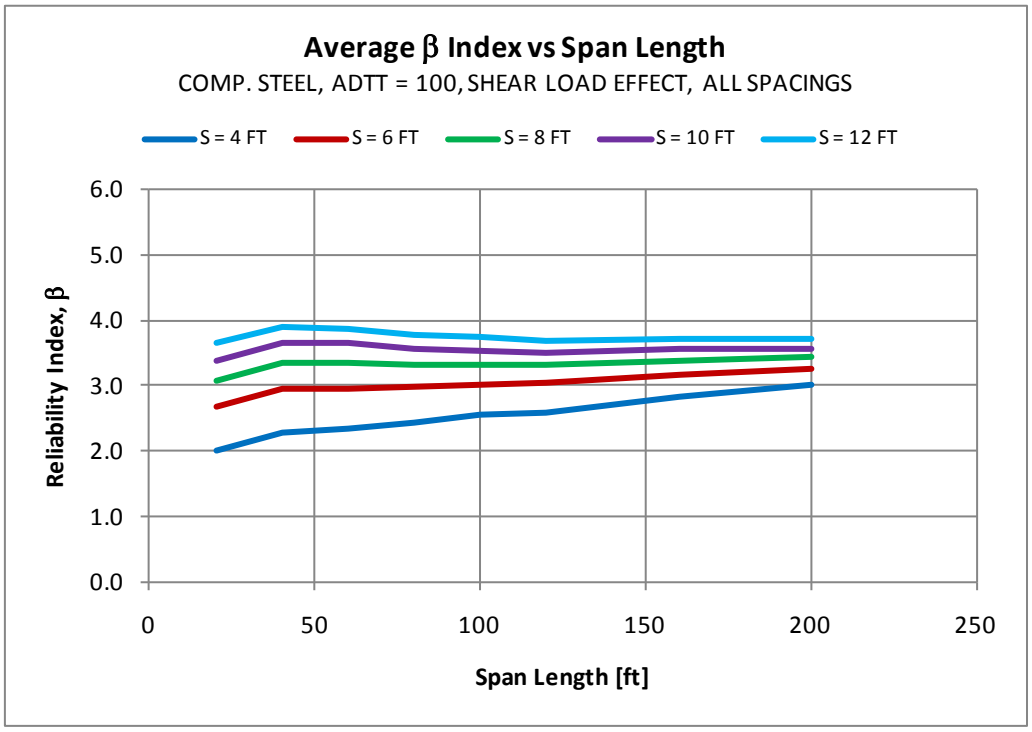
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

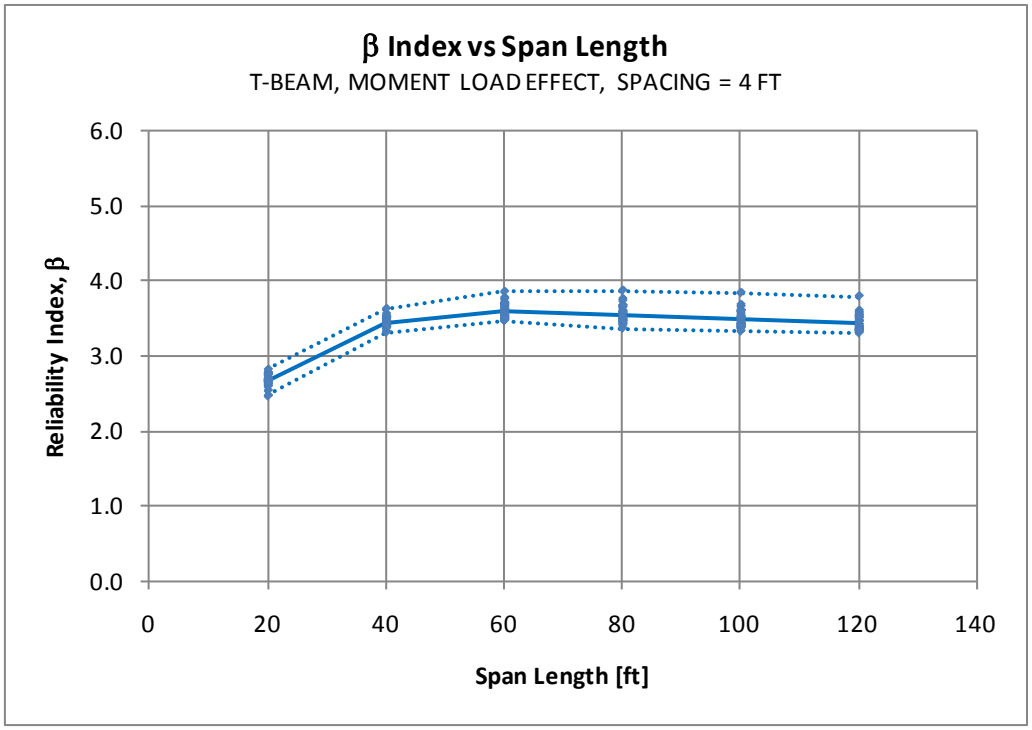

β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

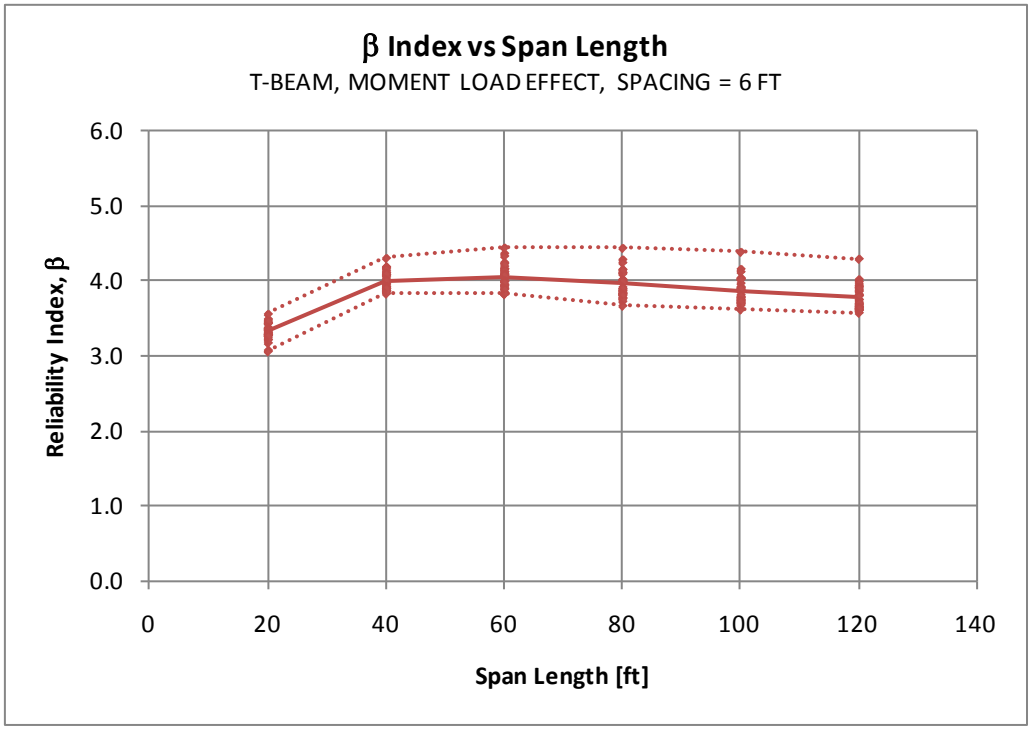

Average β Index vs Span Length
COMP. STEEL, ADTT = 100, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

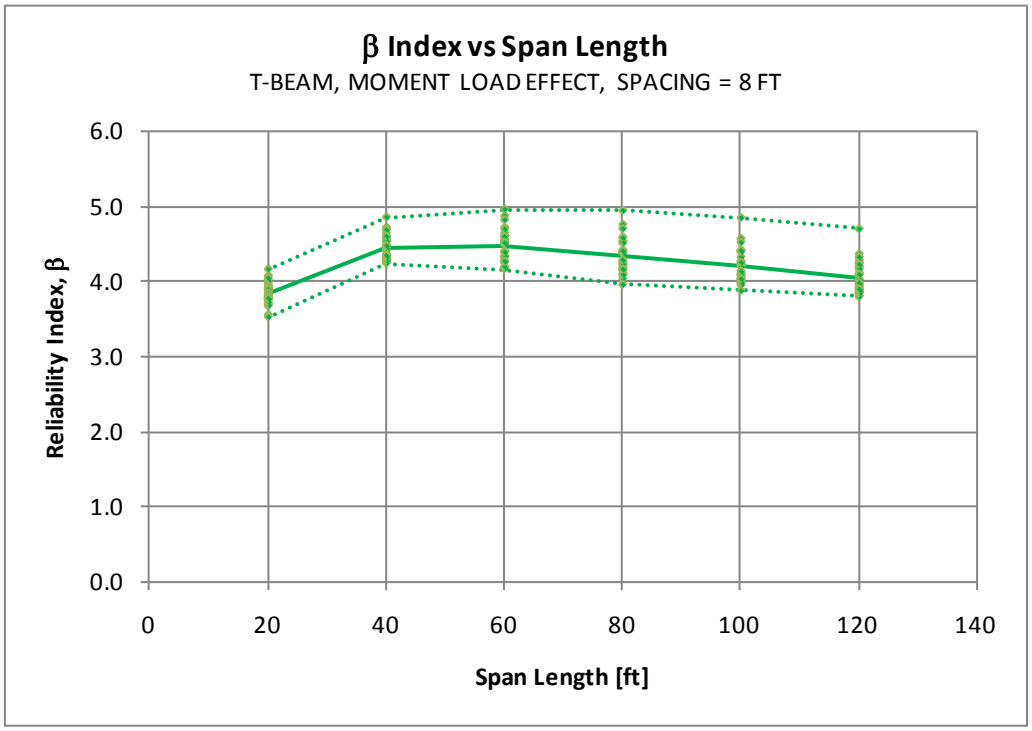
REINFORCED CONCRETE T- BEAM

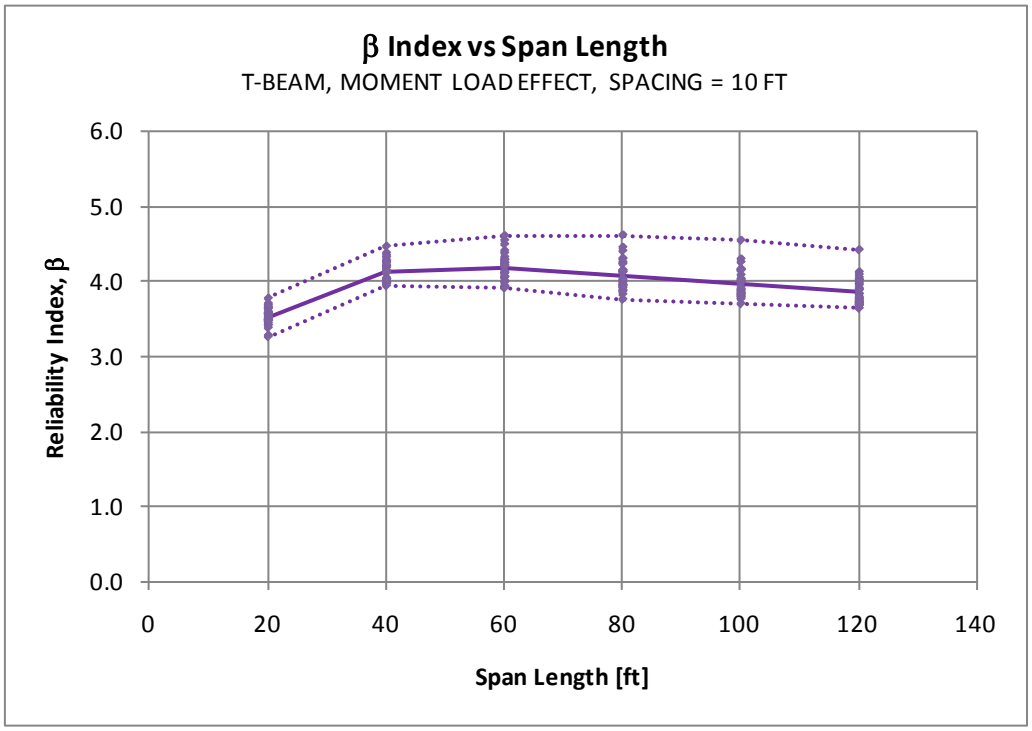
PERMIT PLOTS

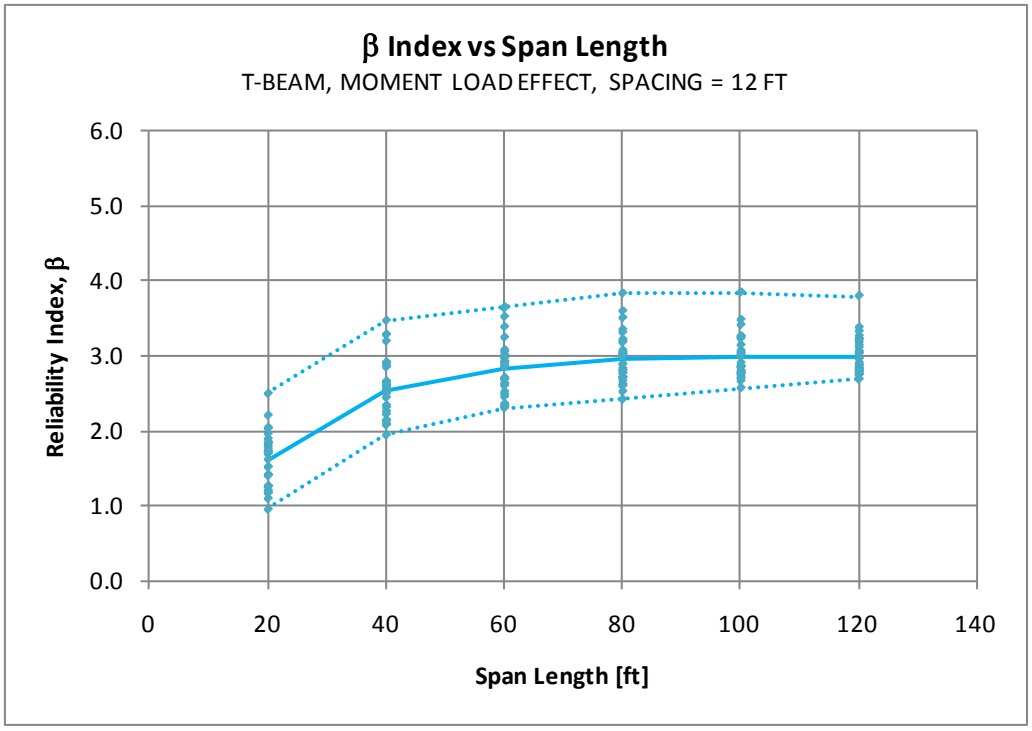
(ONE LANE)

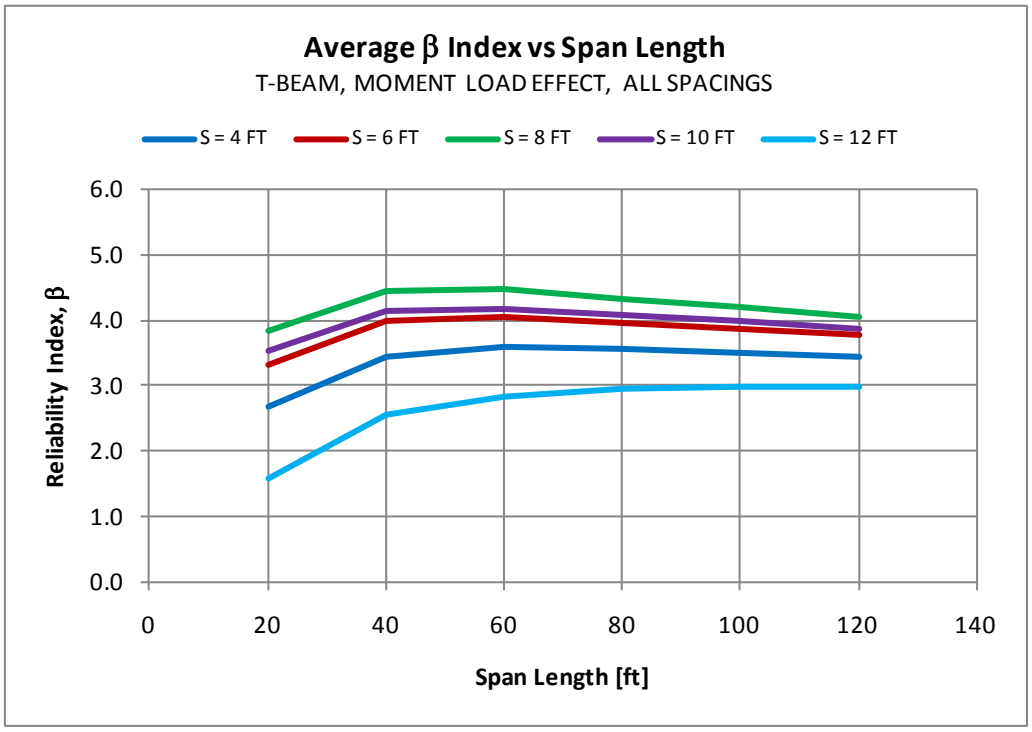

β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

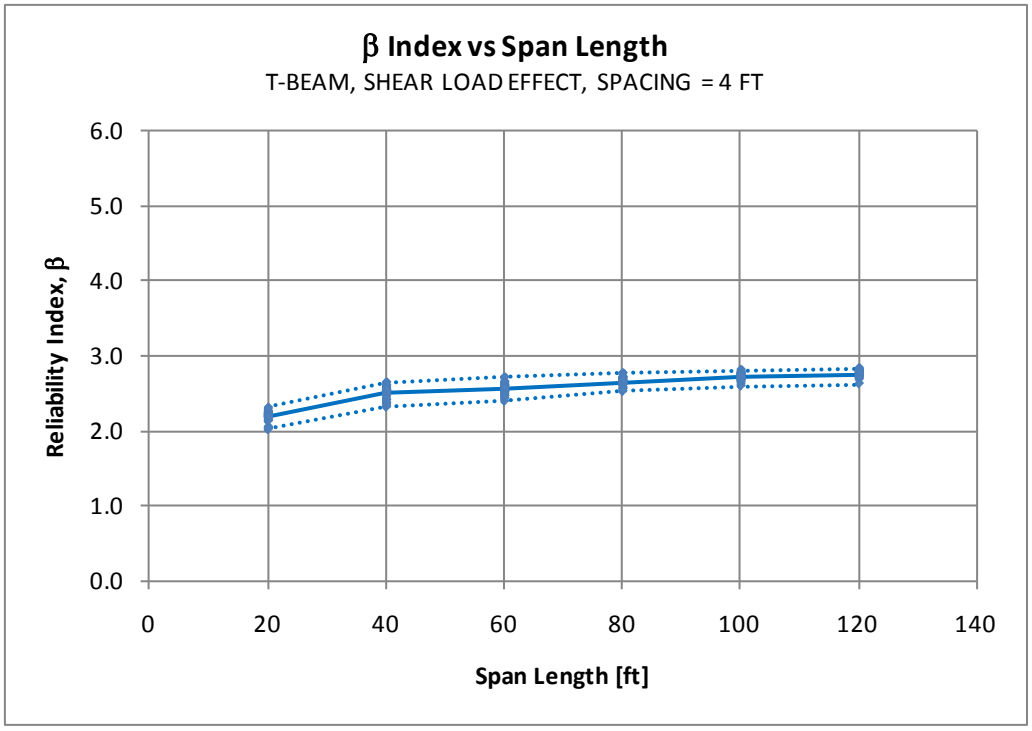

β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

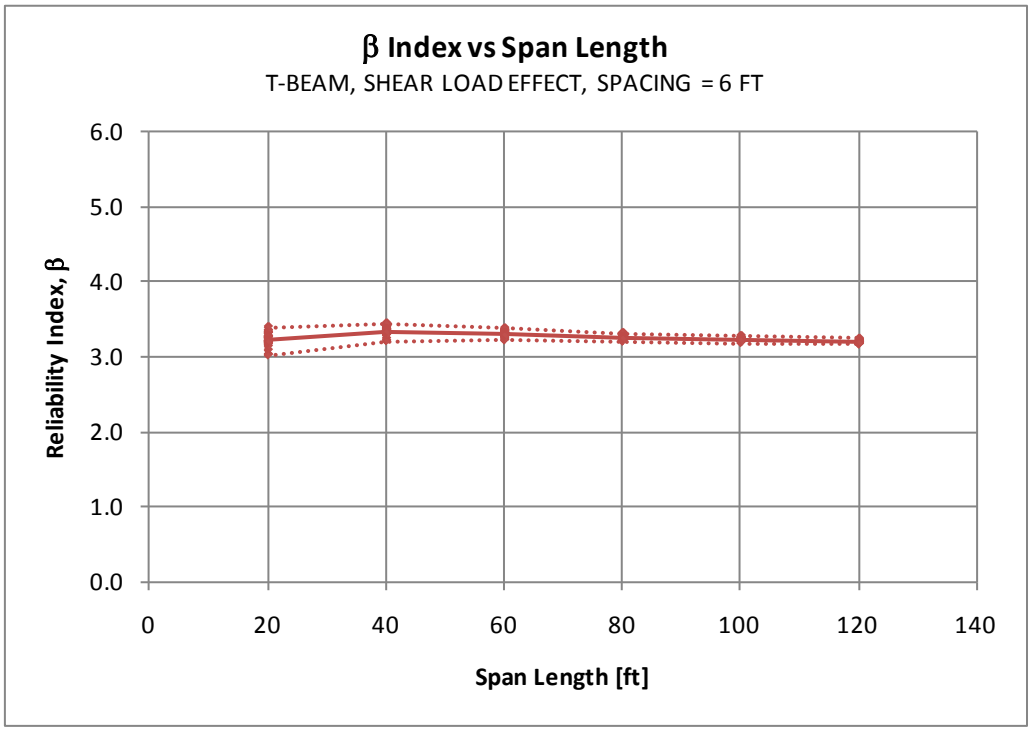

β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

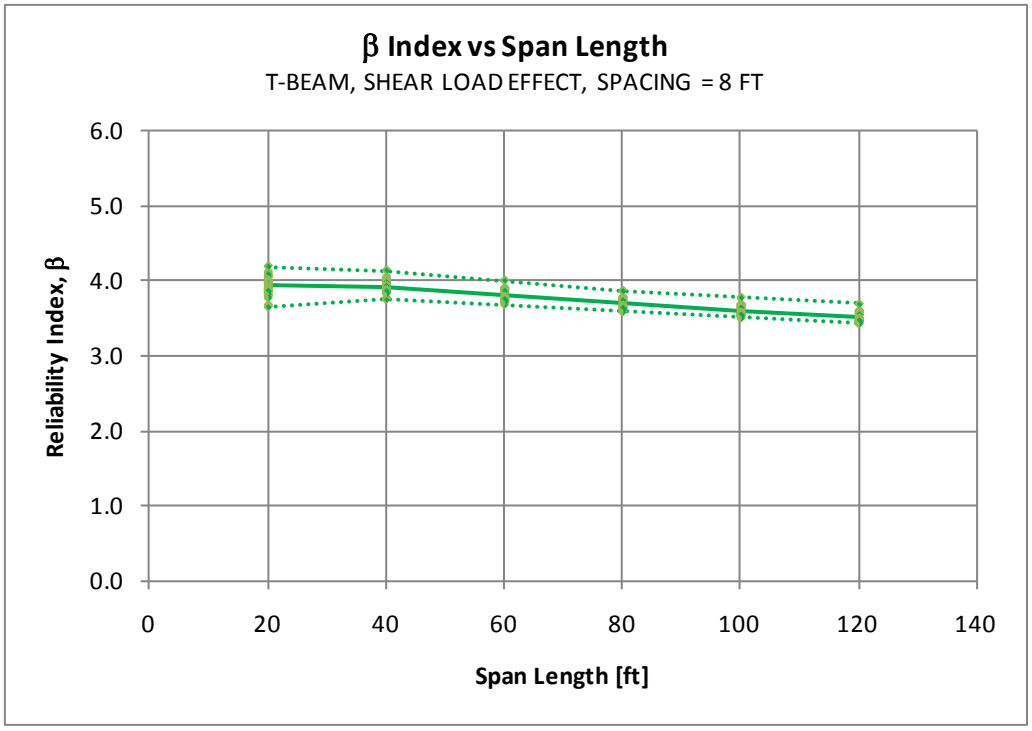

β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

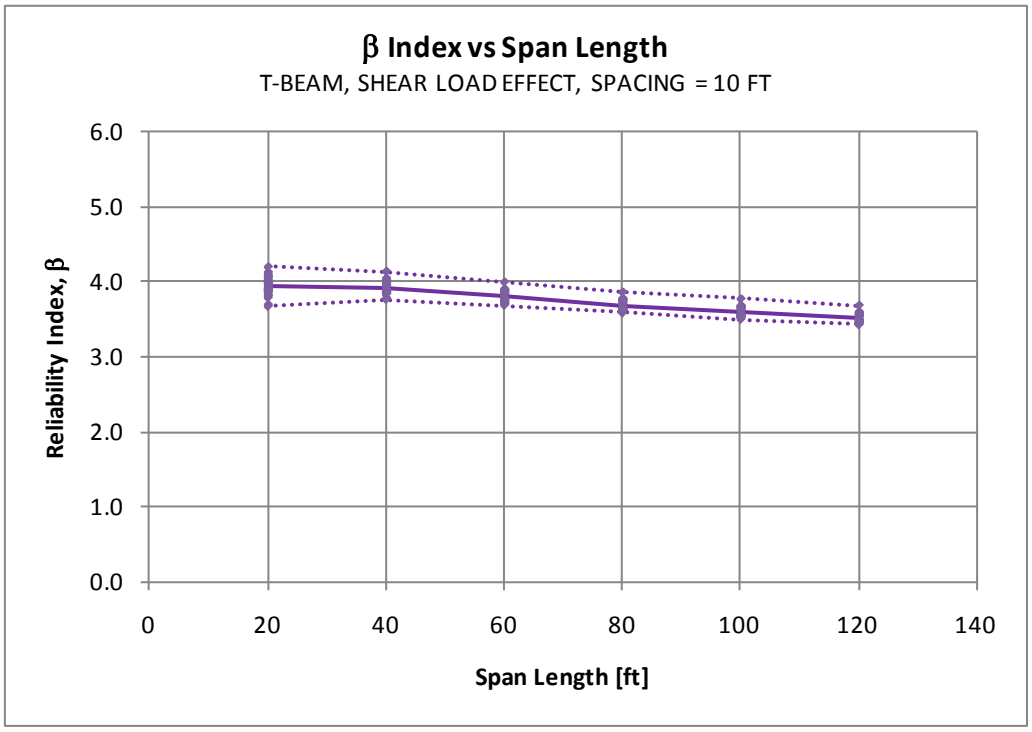

β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

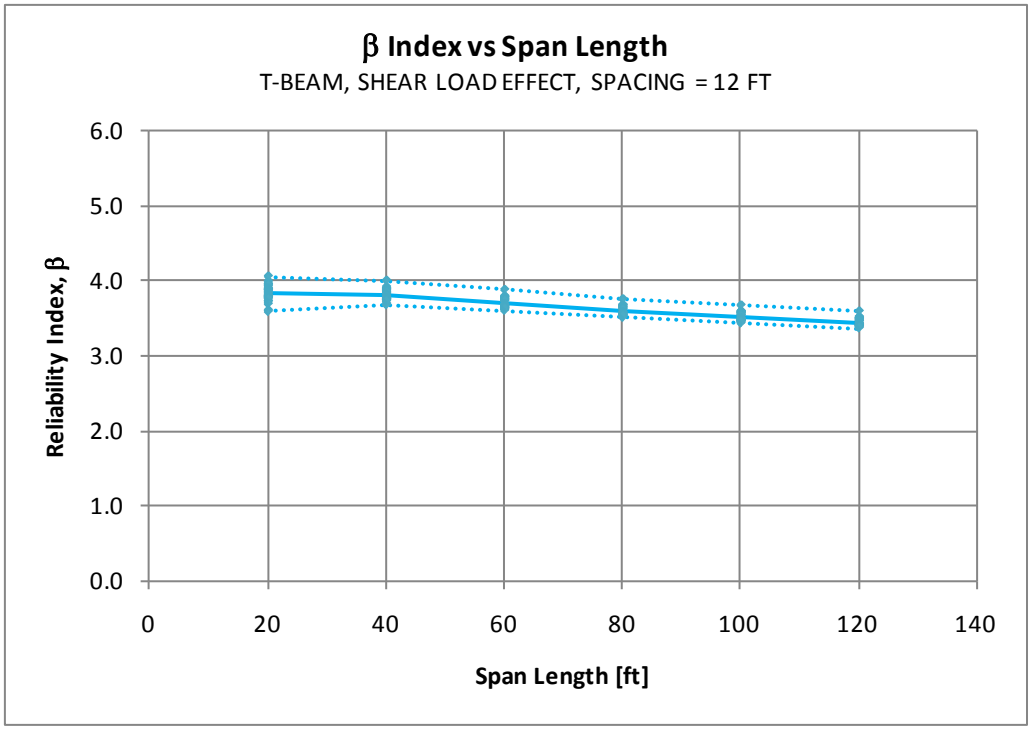

Average β Index vs Span Length
T-BEAM, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

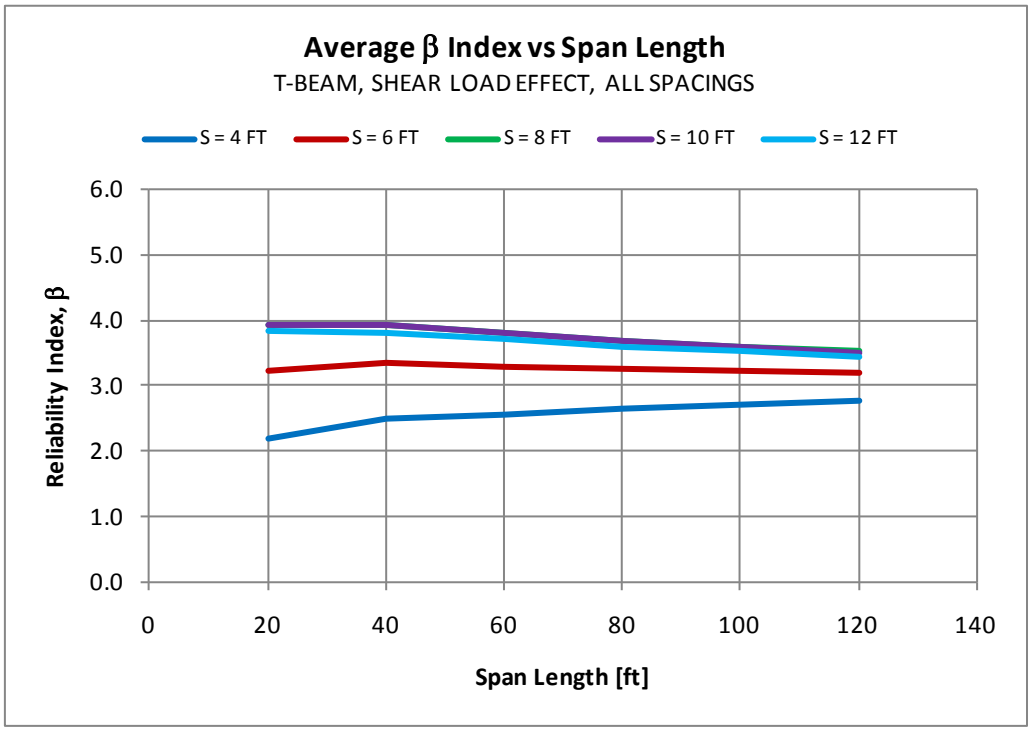

β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

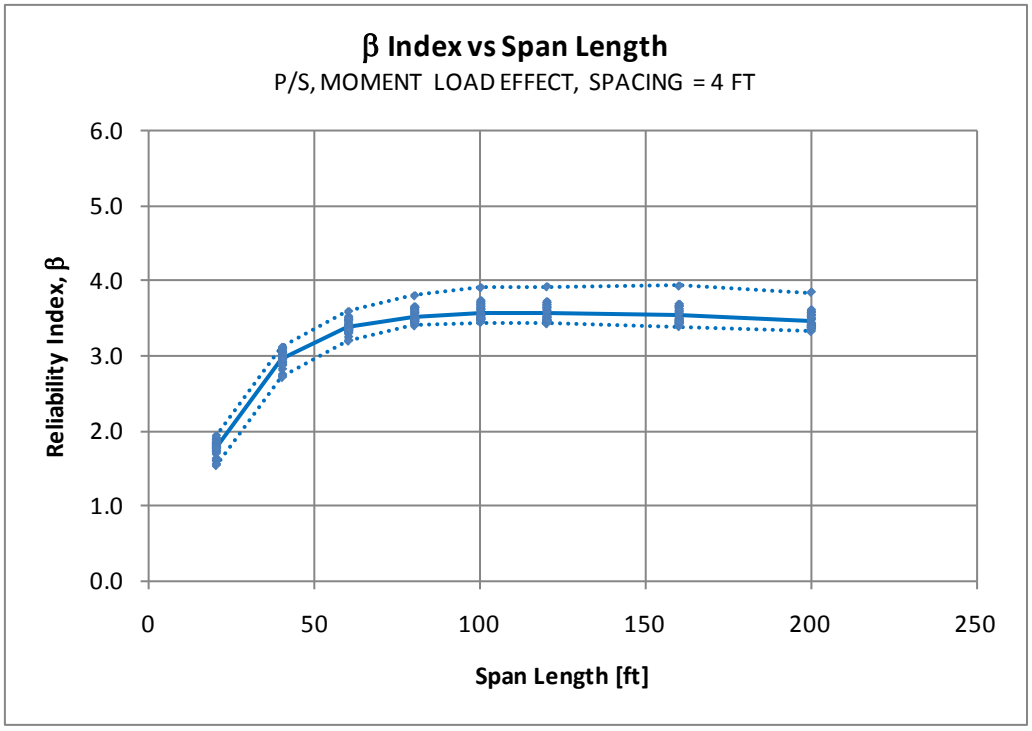

β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

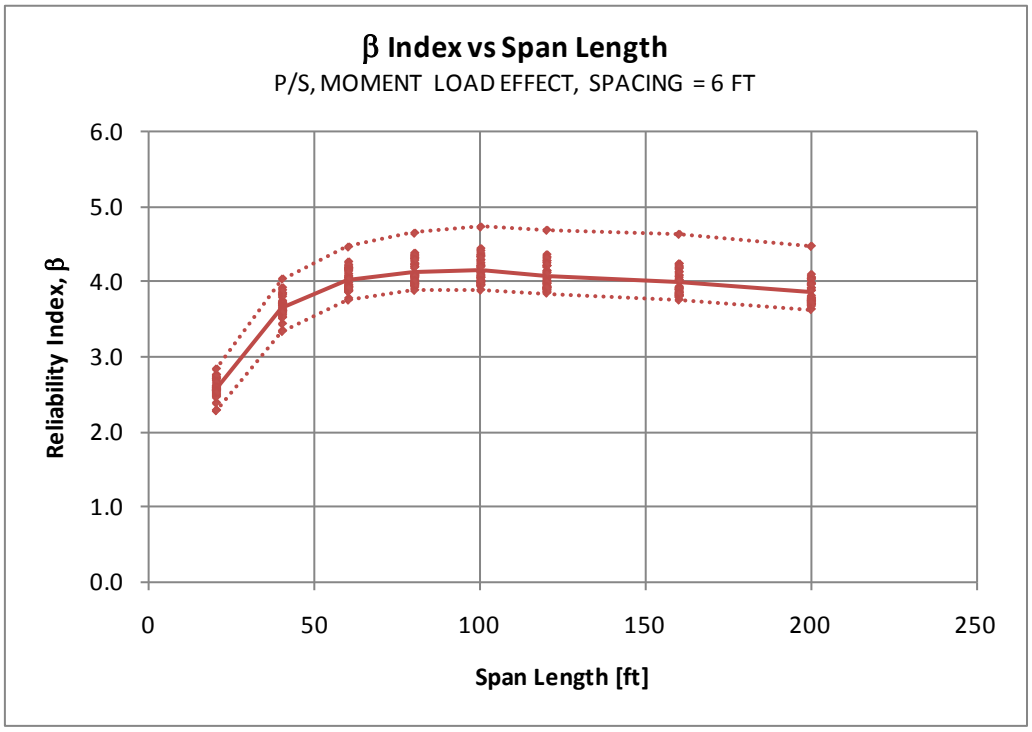

Average β Index vs Span Length
T-BEAM, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

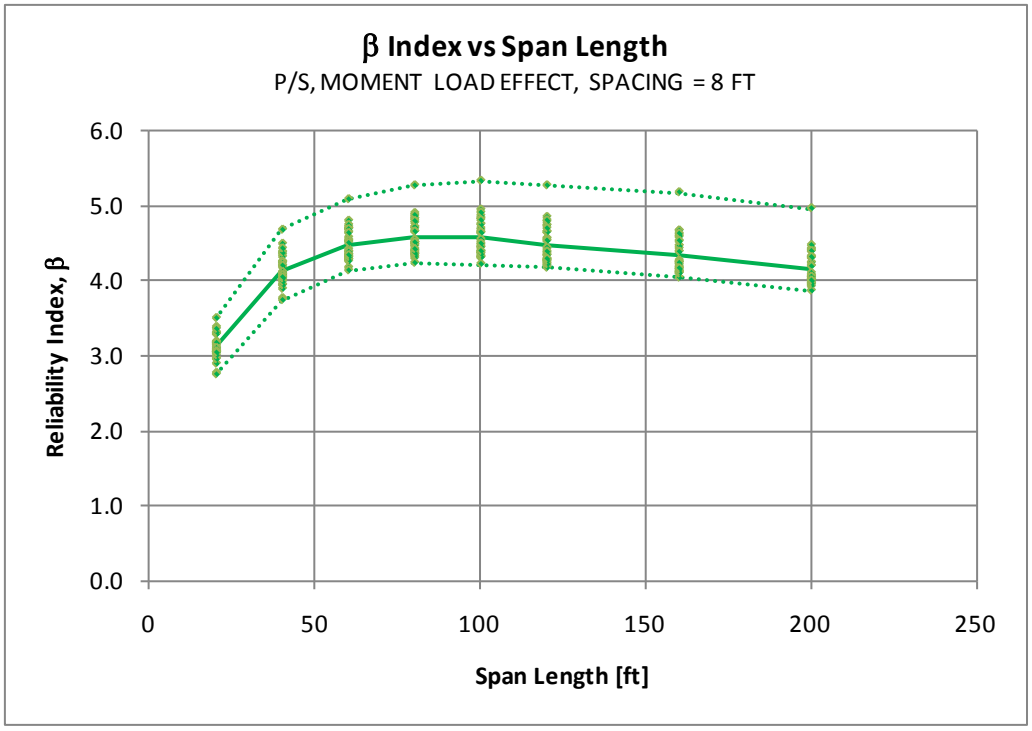
PRESTRESSED CONCRETE BEAM

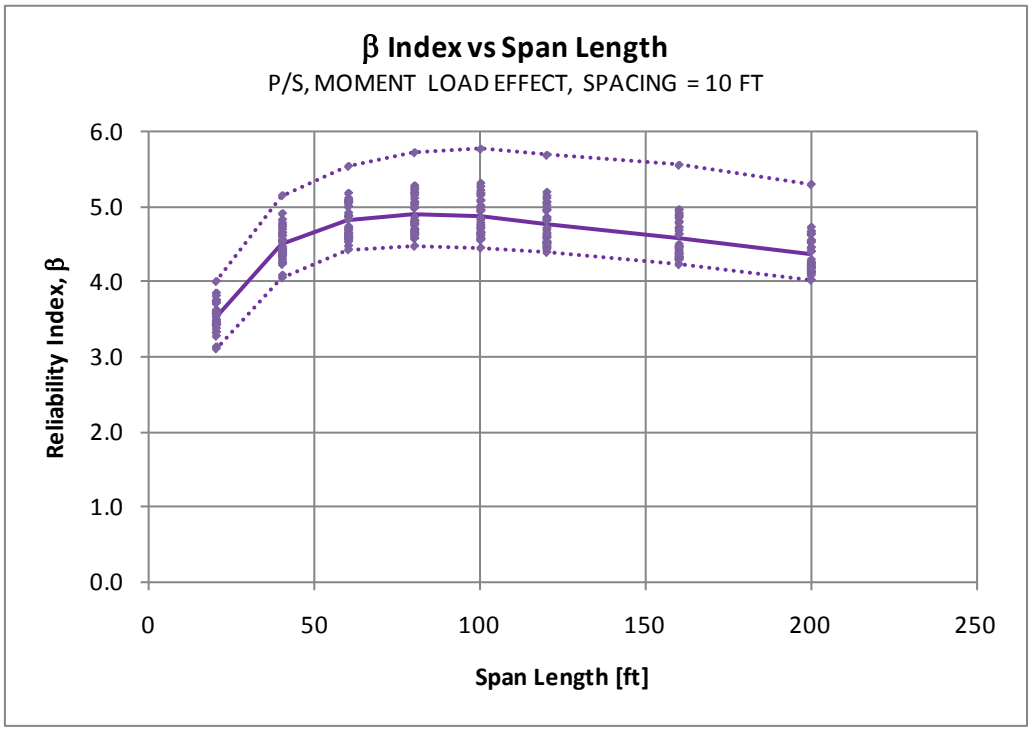
PERMIT PLOTS

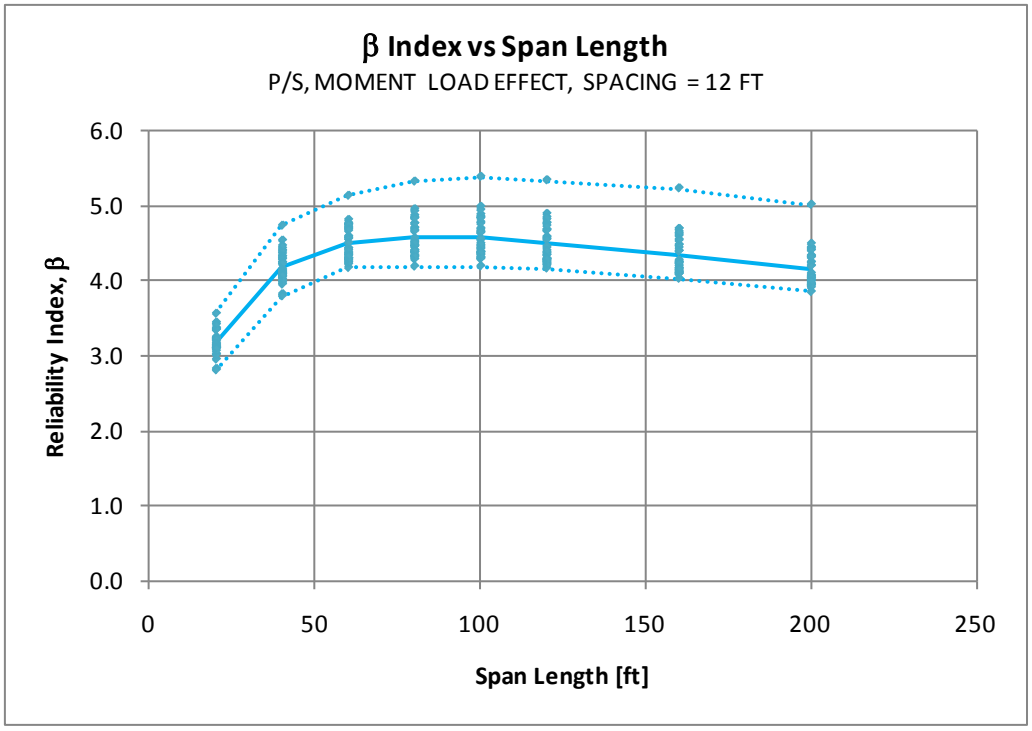
(ONE LANE)

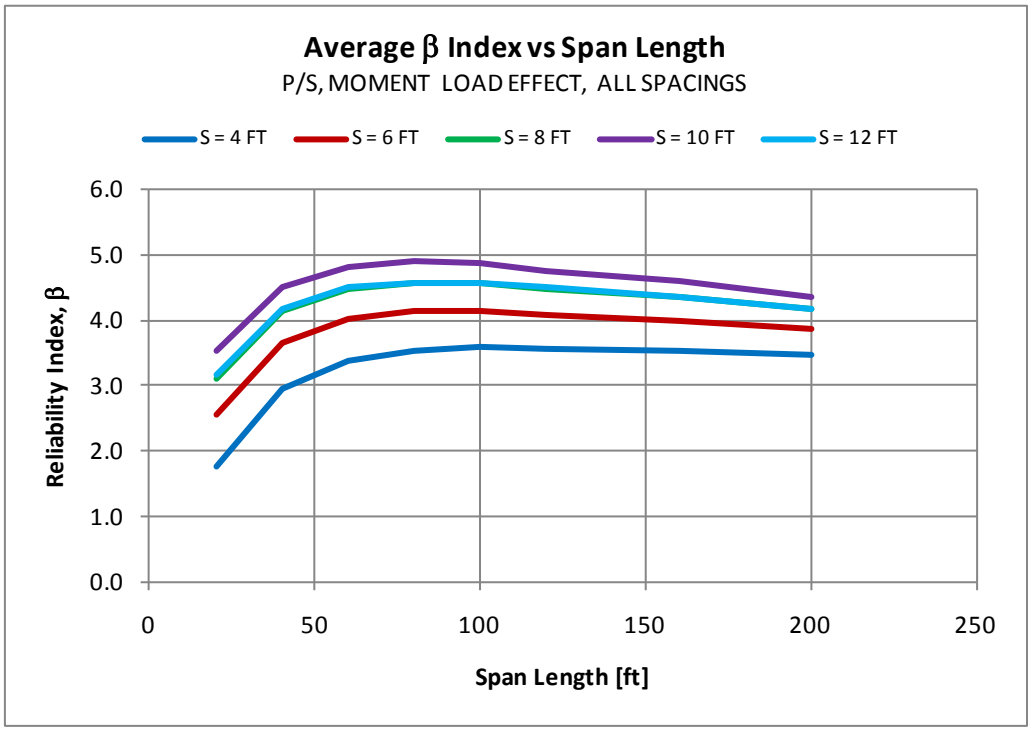

β Index vs Span Length
P/S, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

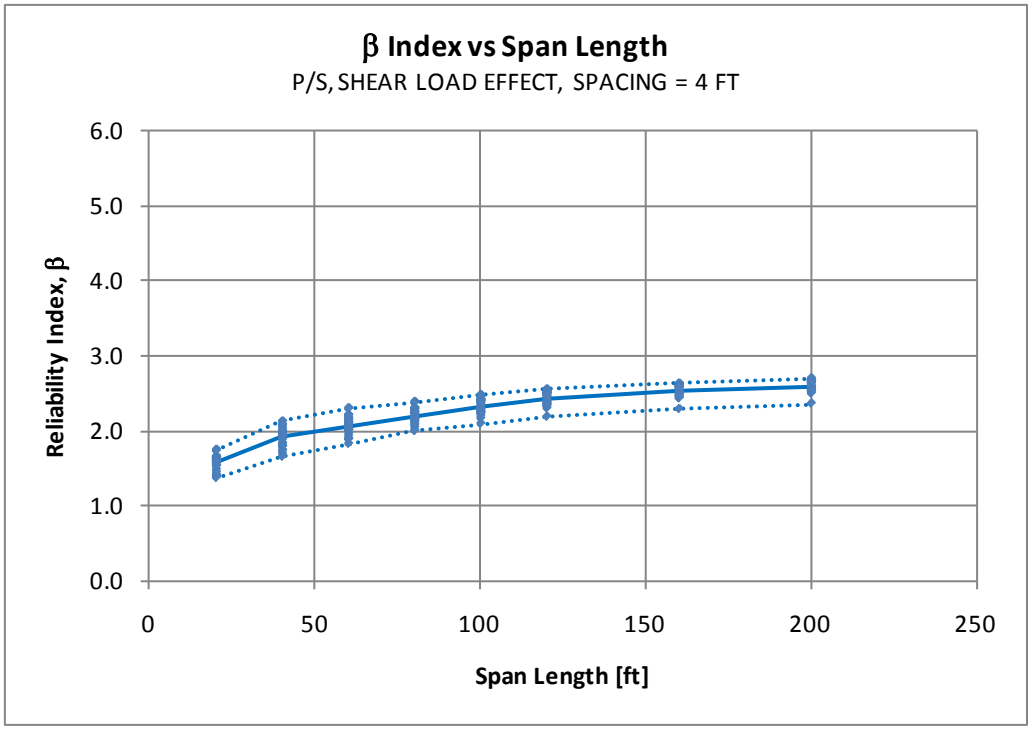

β Index vs Span Length
P/S, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

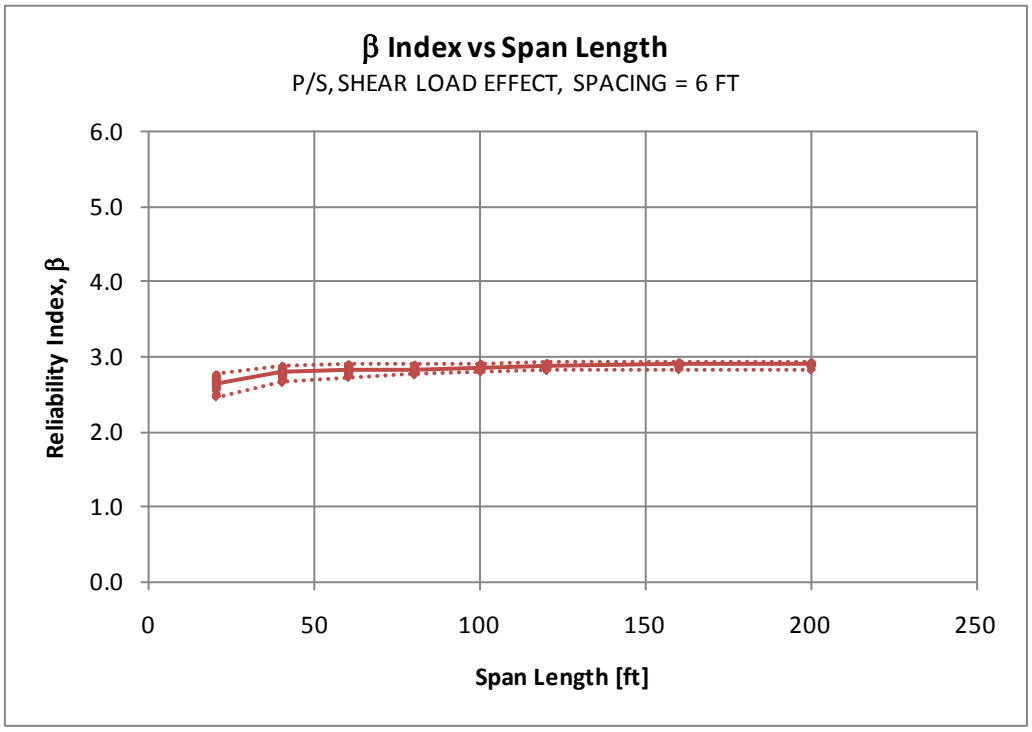

β Index vs Span Length
P/S, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

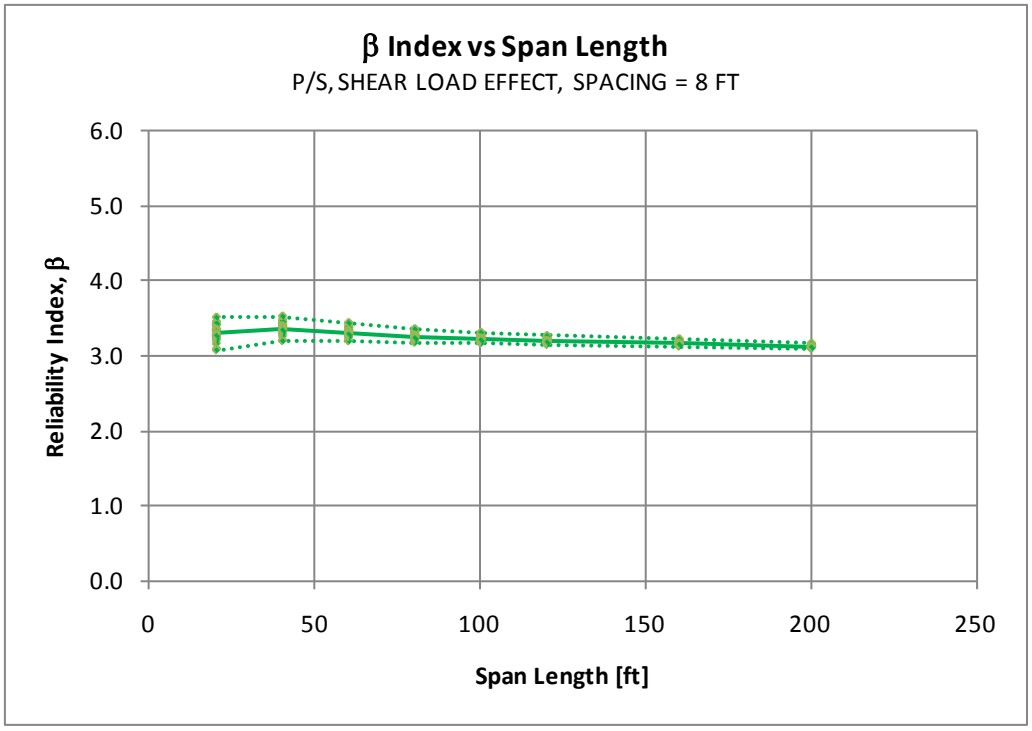

β Index vs Span Length
P/S, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

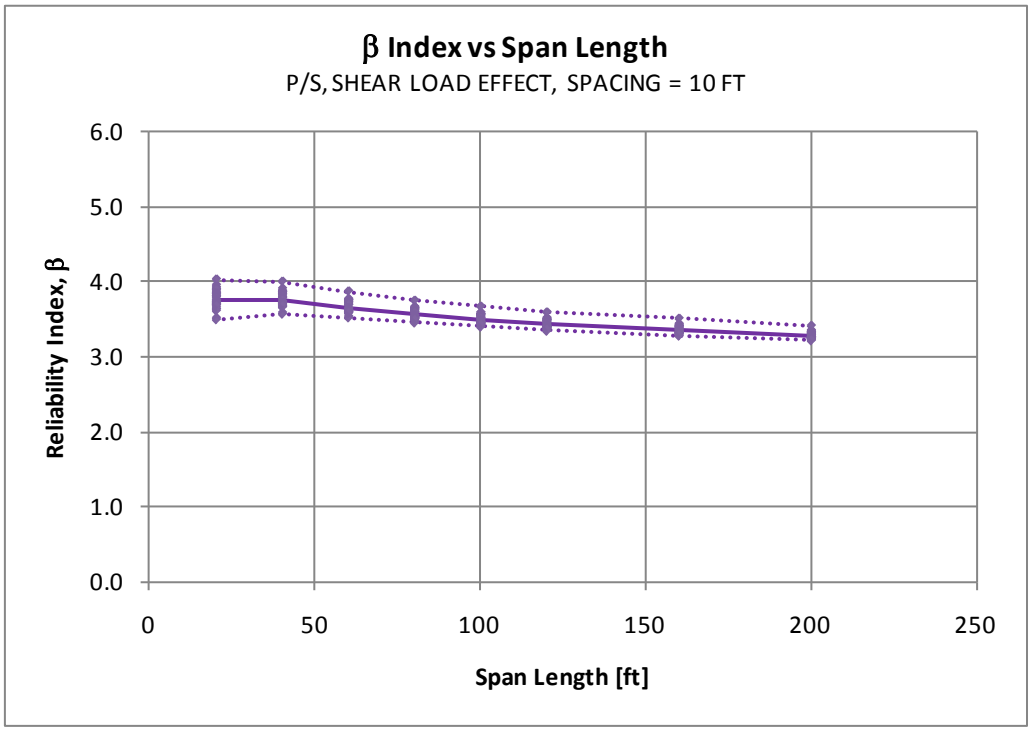

β Index vs Span Length
P/S, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

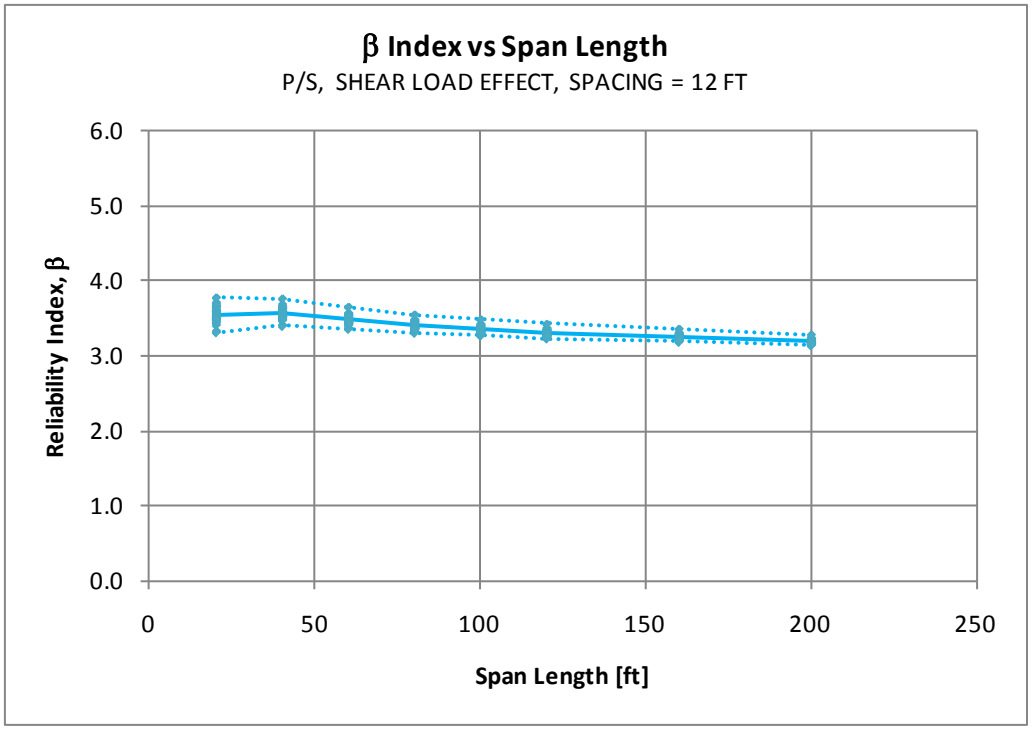

Average β Index vs Span Length
P/S, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

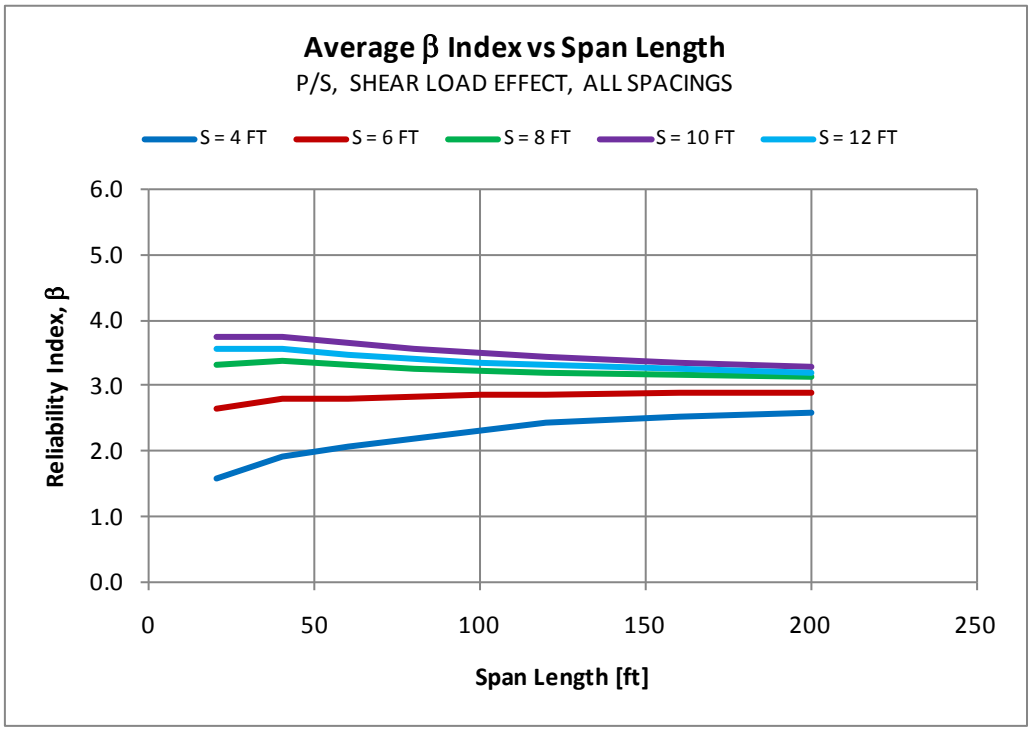

β Index vs Span Length
P/S, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

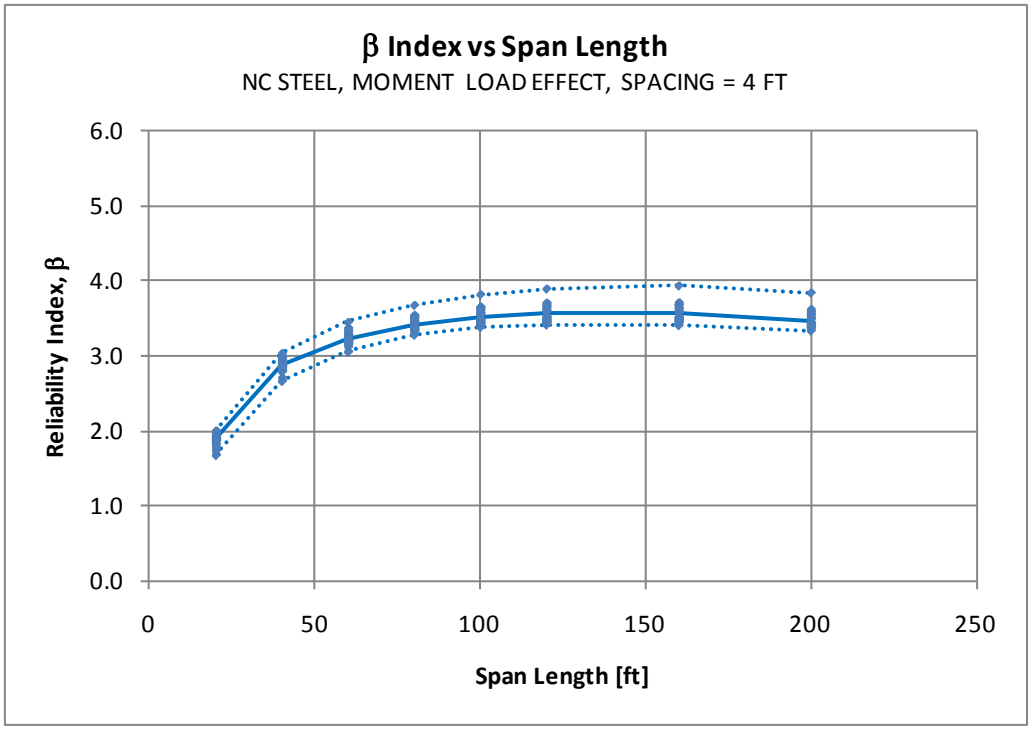

β Index vs Span Length
P/S, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

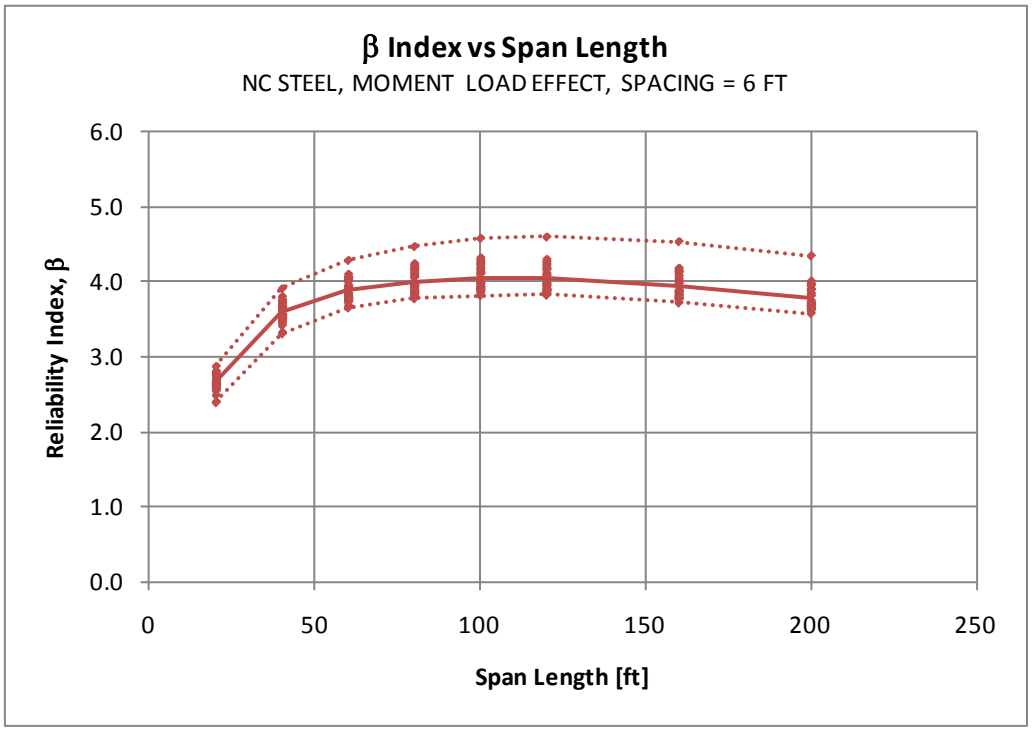

Average β Index vs Span Length
P/S, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

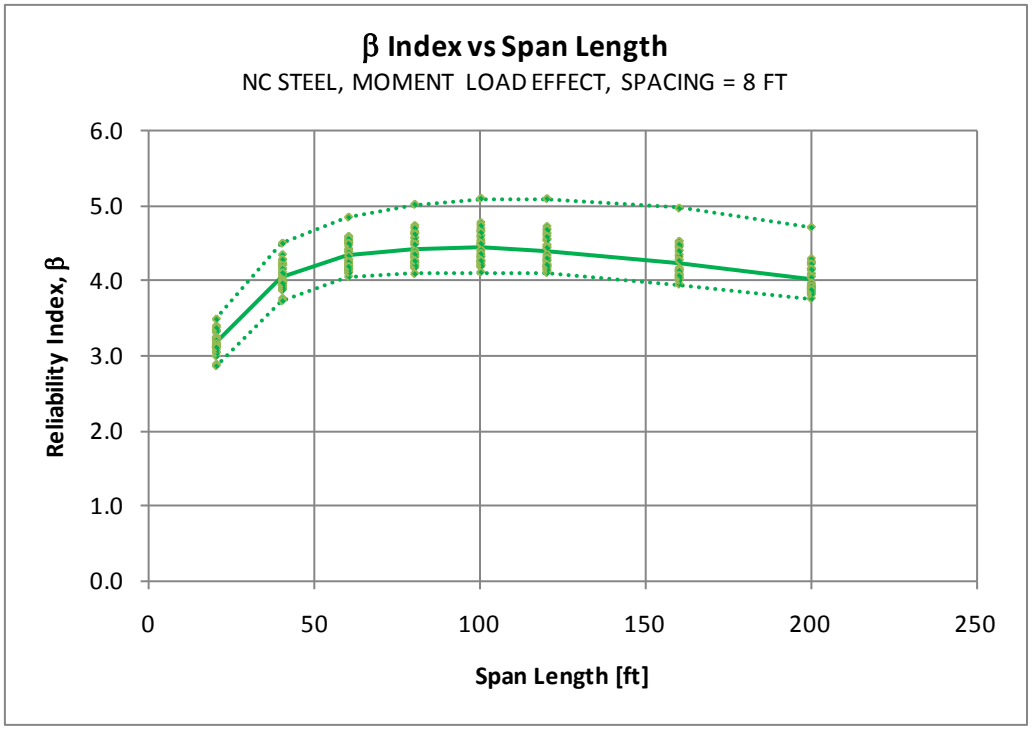
NON-COMPOSITE STEEL BEAM

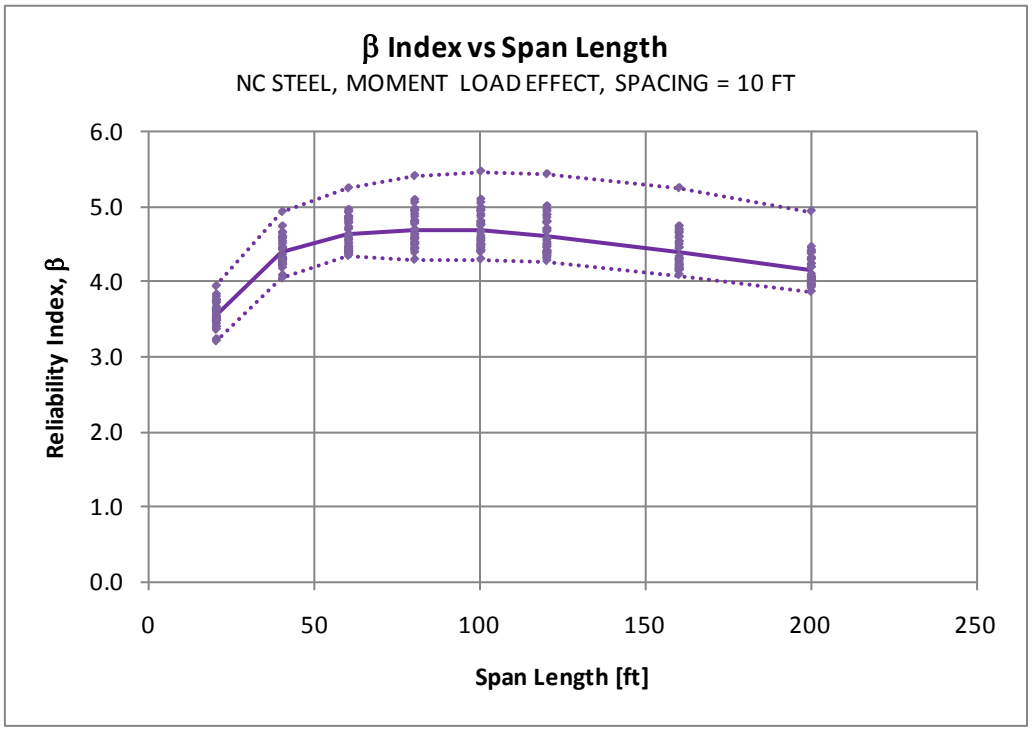
PERMIT PLOTS

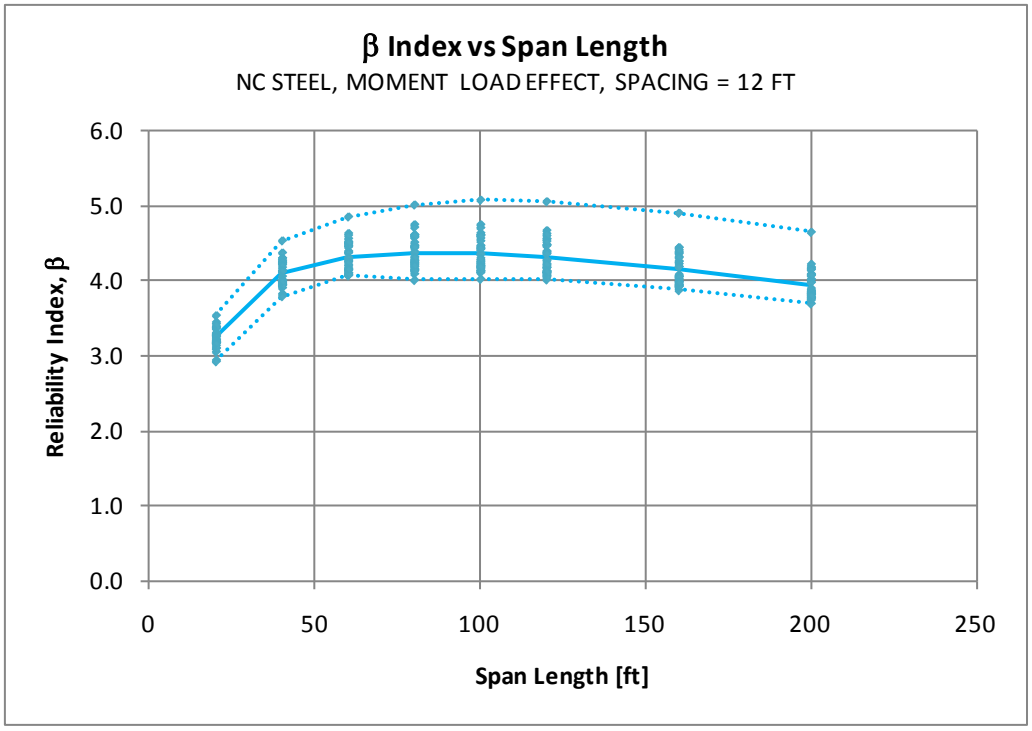
(ONE LANE)

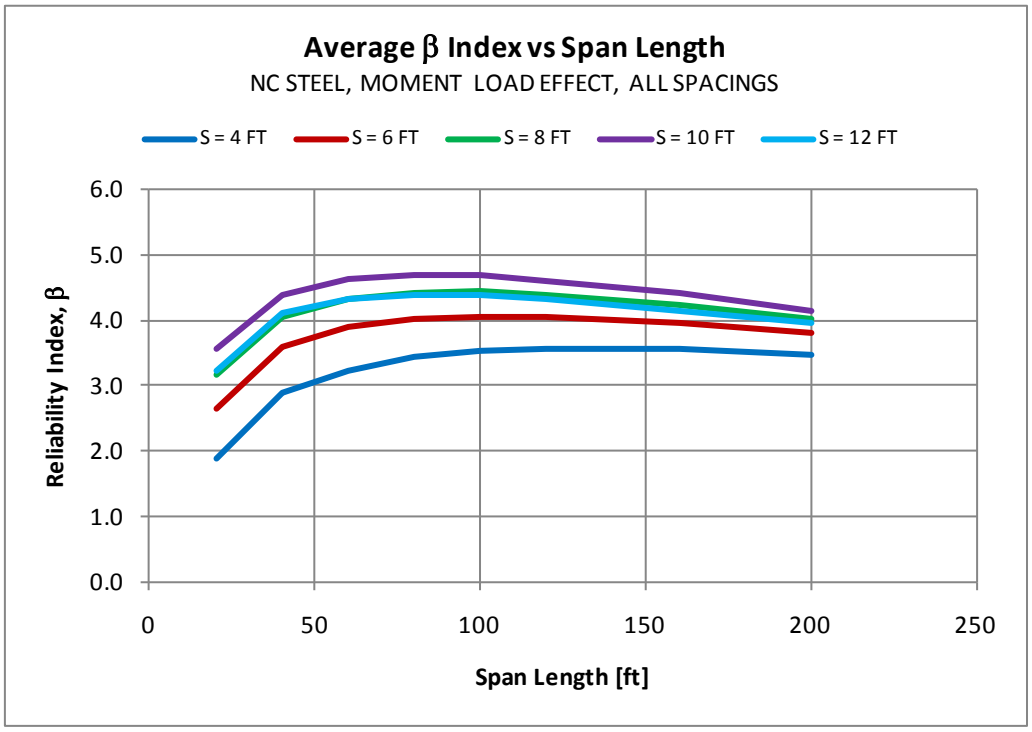

β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

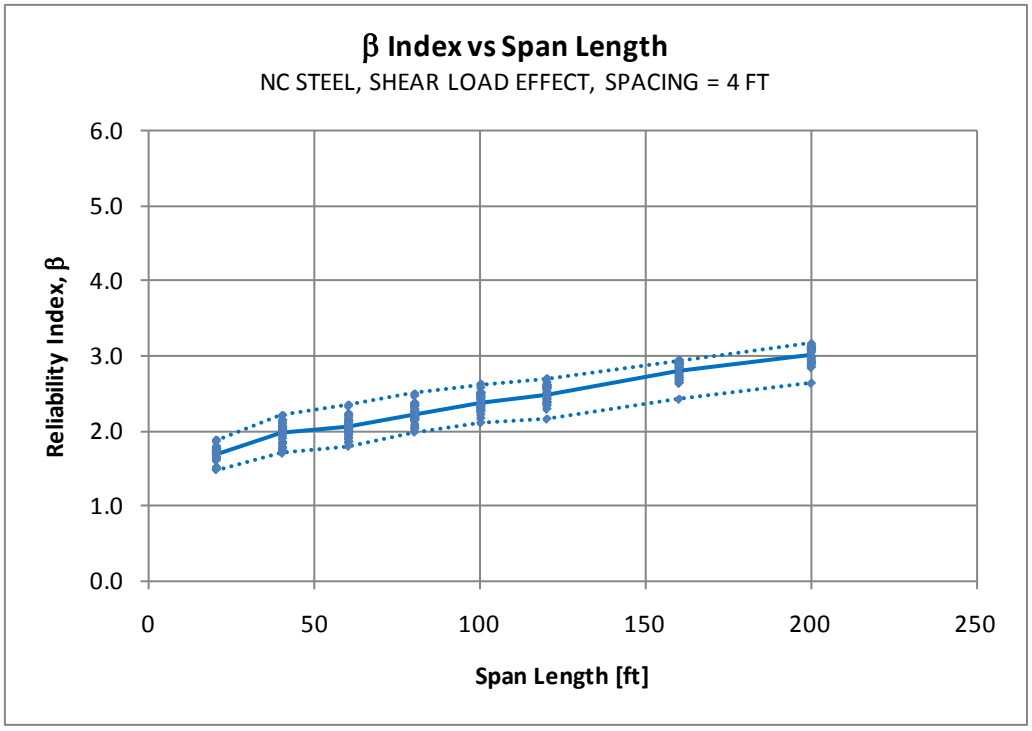

β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

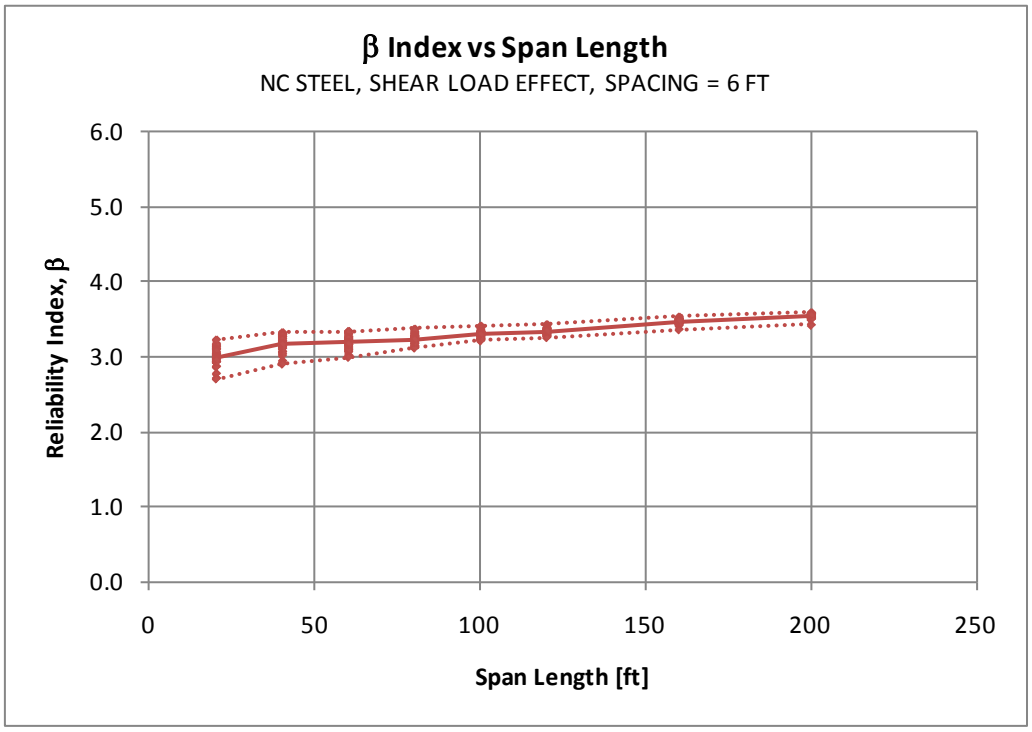

β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

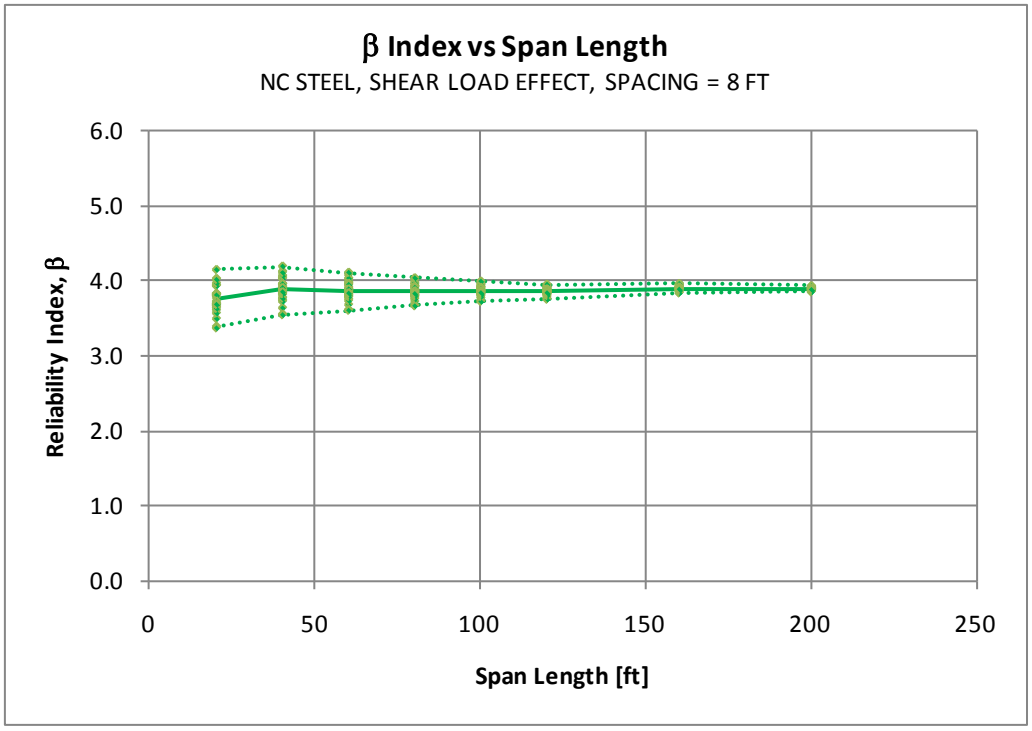

β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

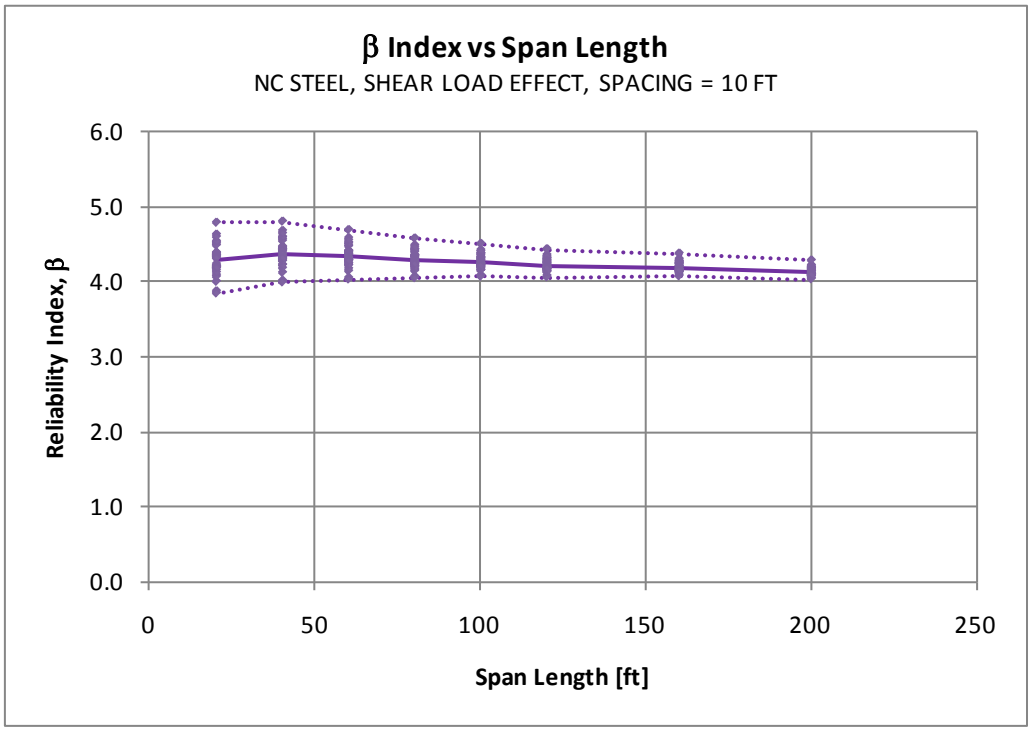

β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

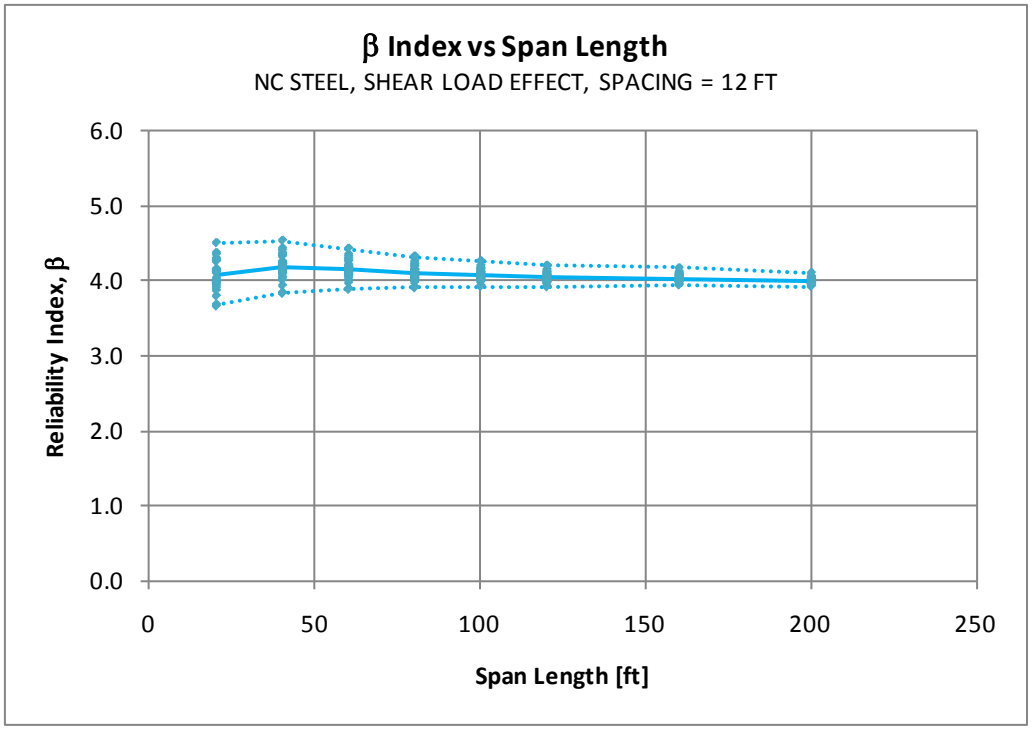

Average β Index vs Span Length
NC STEEL, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

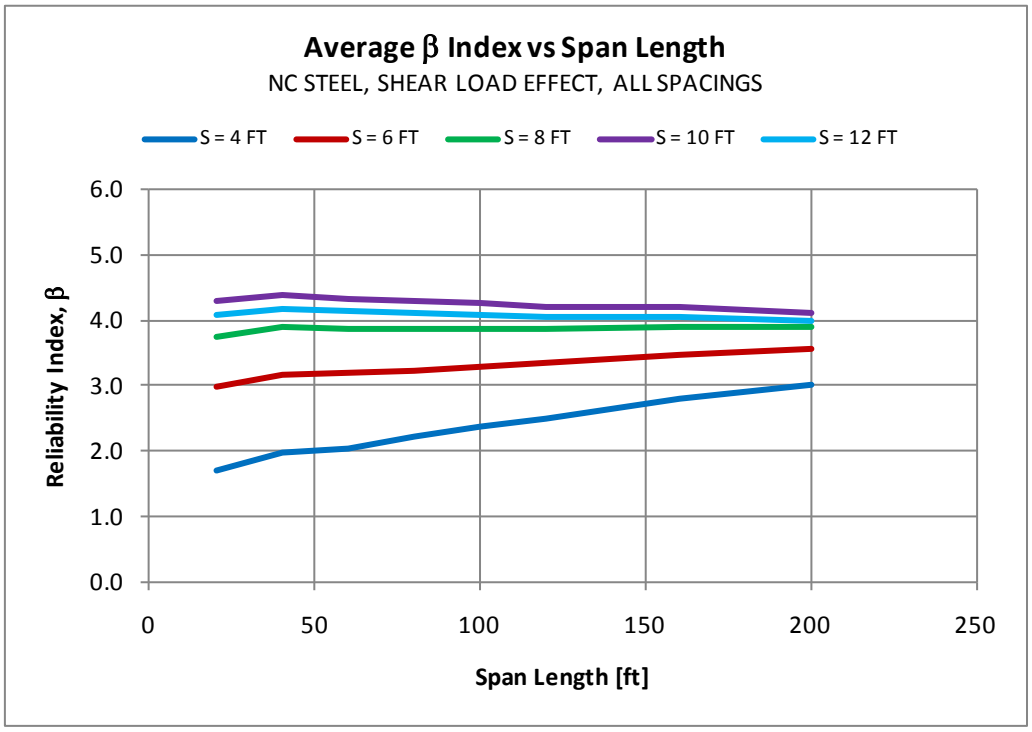

β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

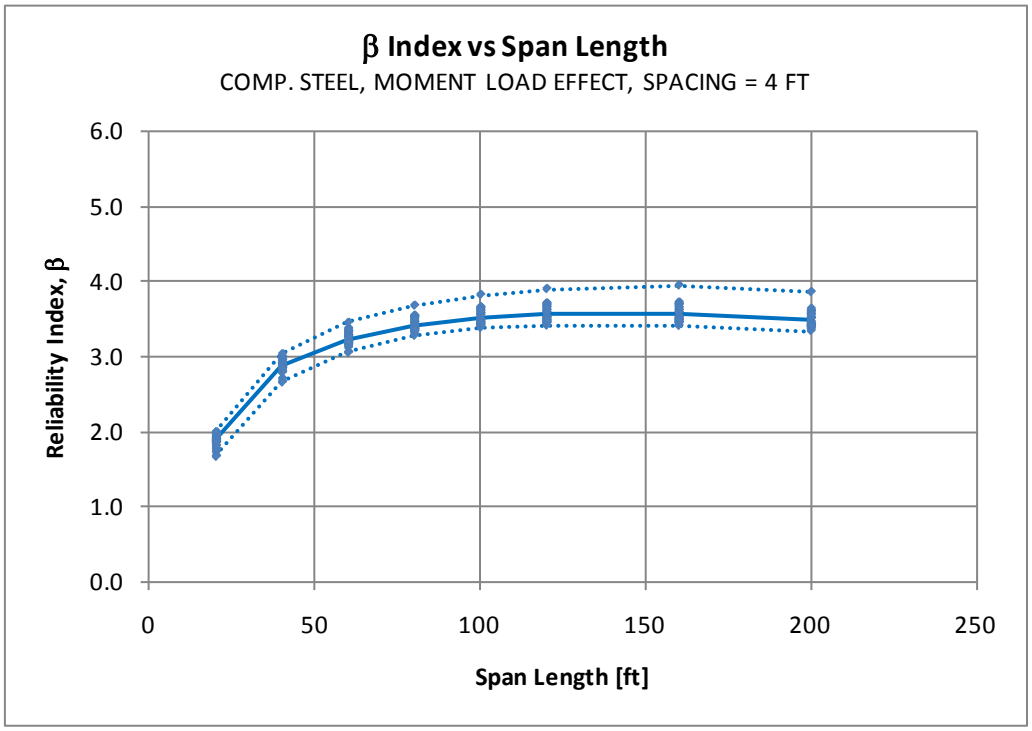

β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

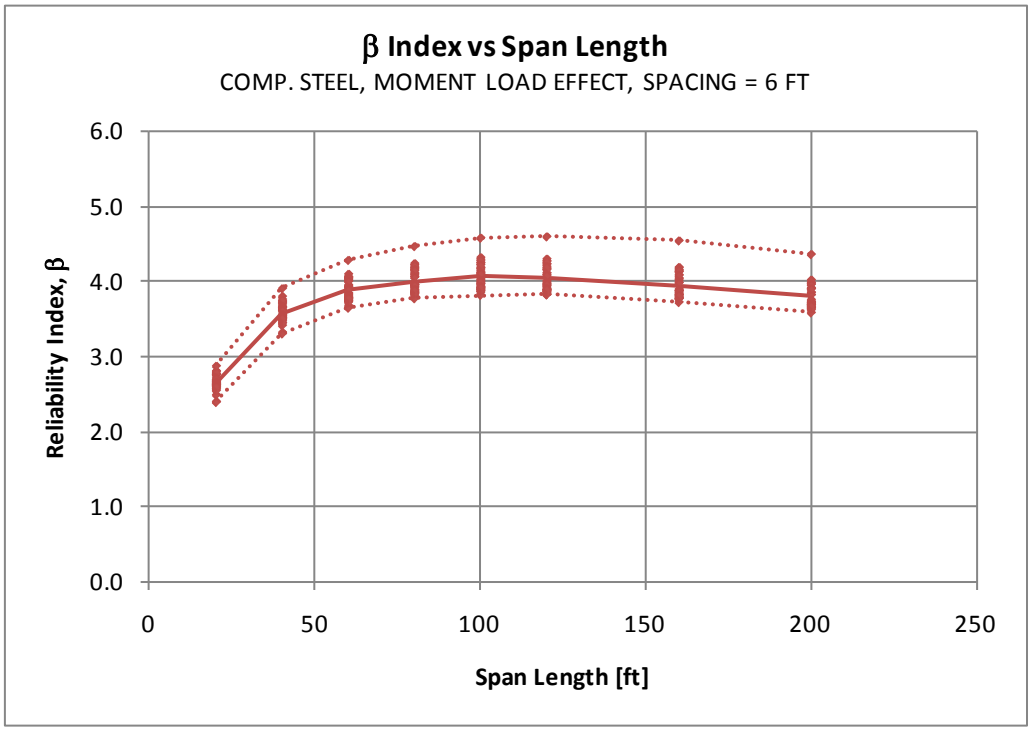

Average β Index vs Span Length
NC STEEL, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

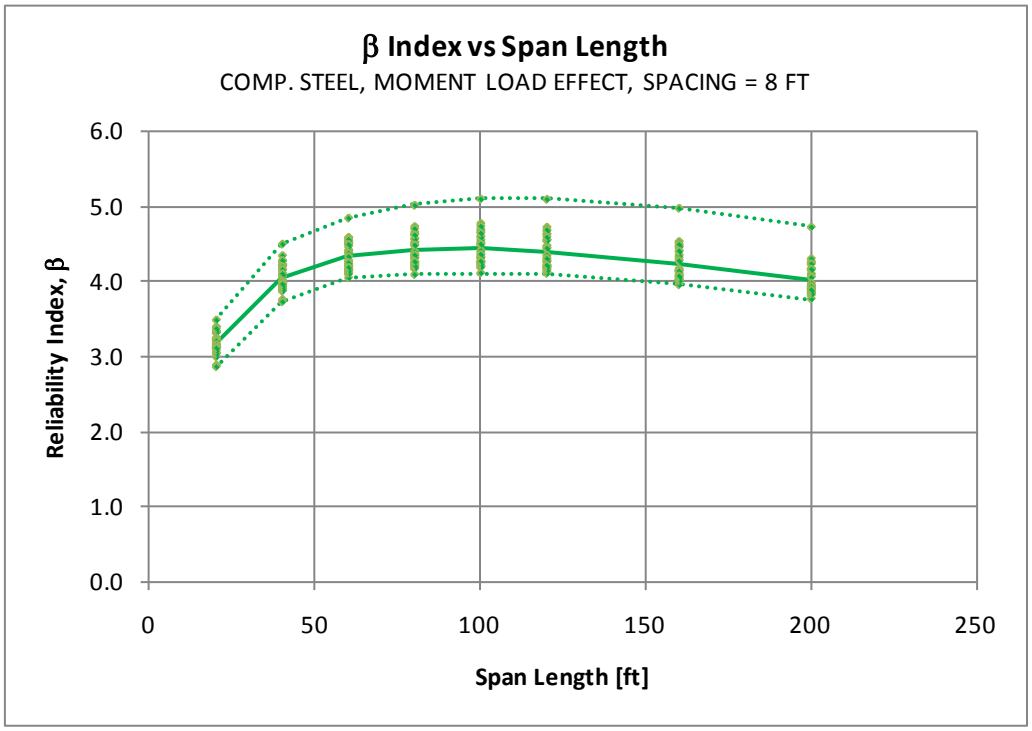
COMPOSITE STEEL BEAM

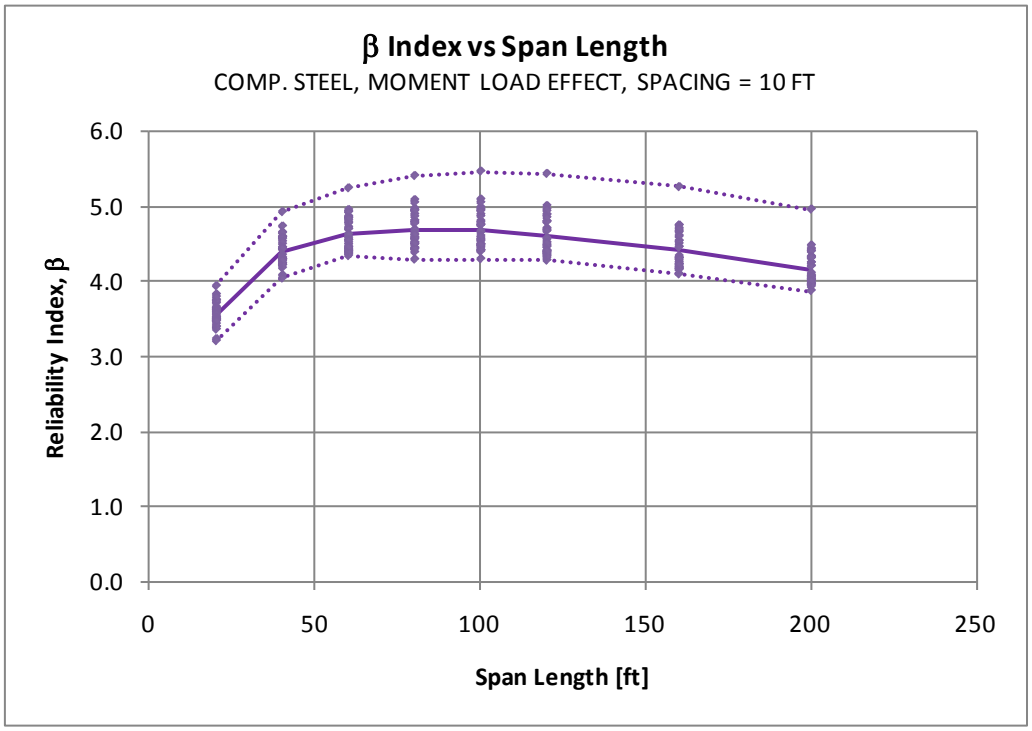
PERMIT PLOTS

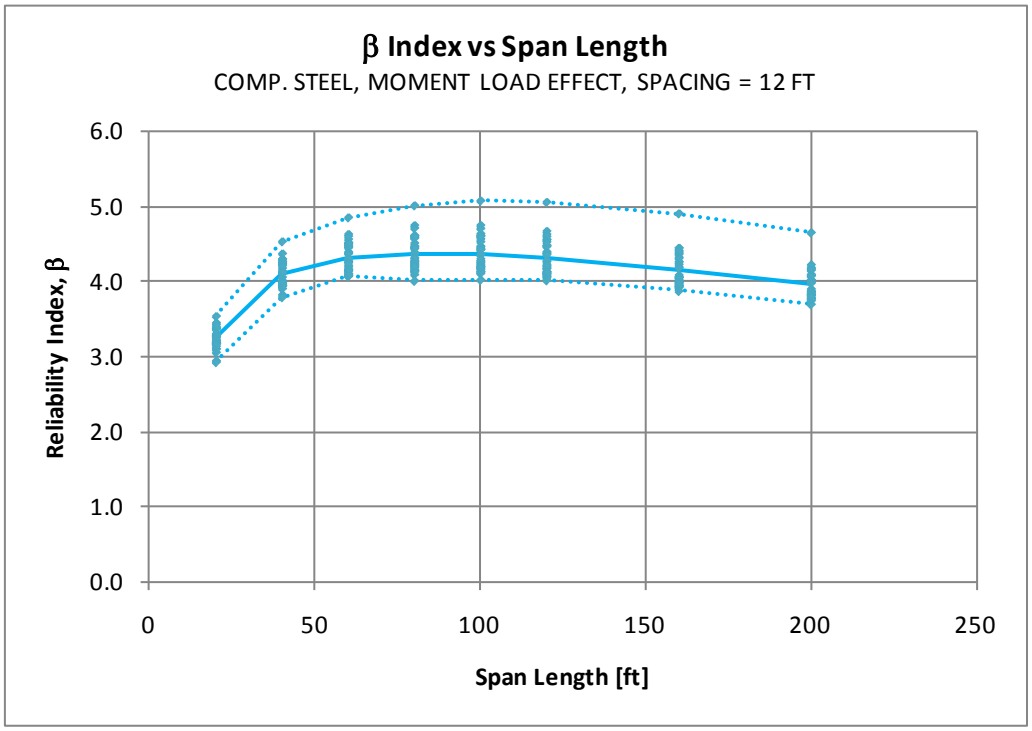
(ONE LANE)

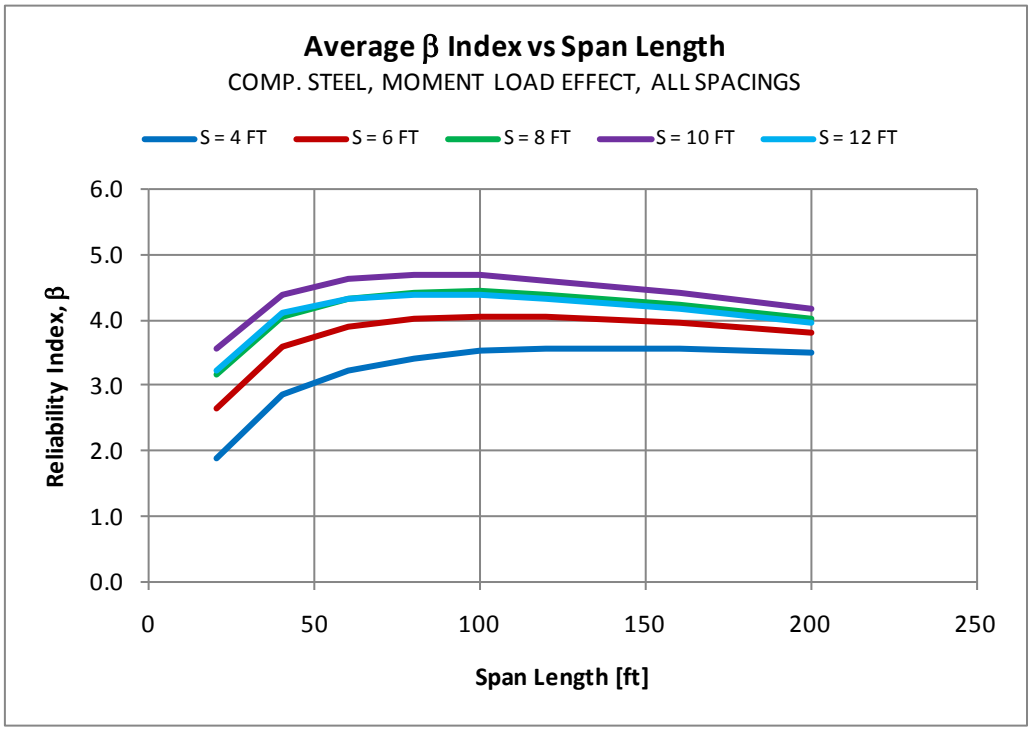

β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

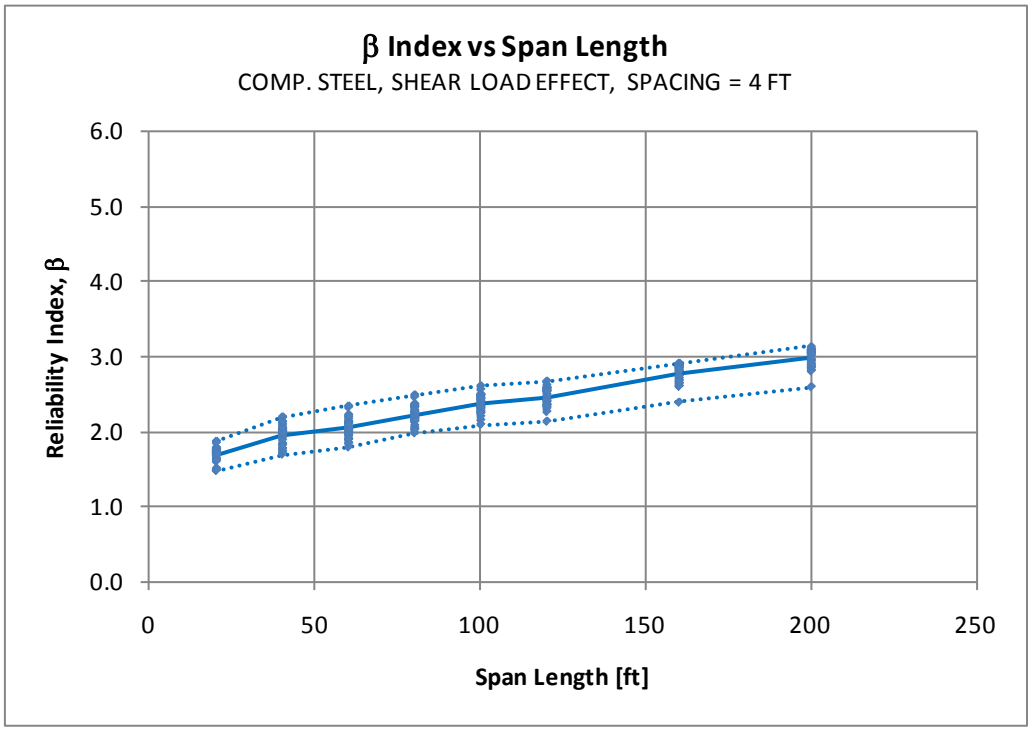

β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

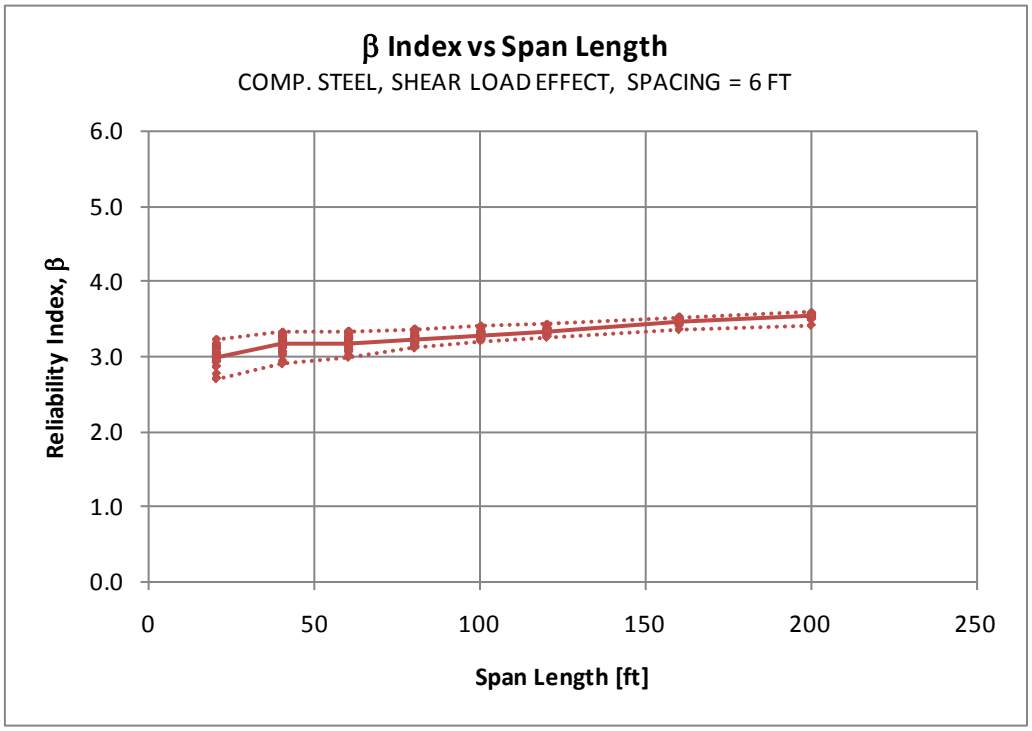

β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

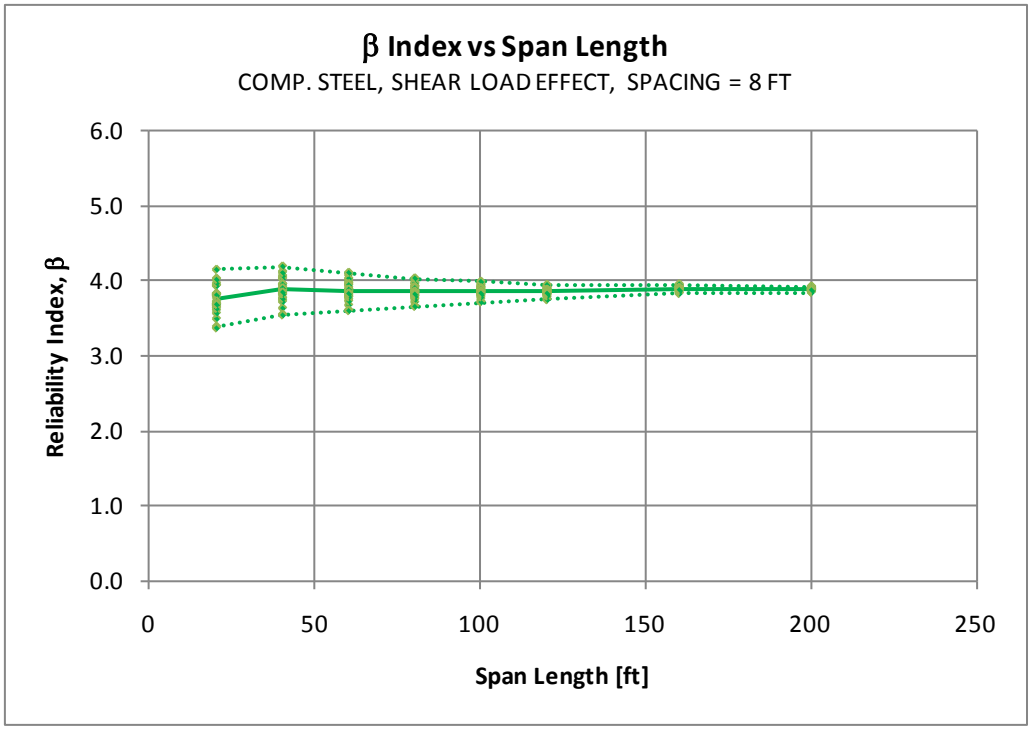

β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

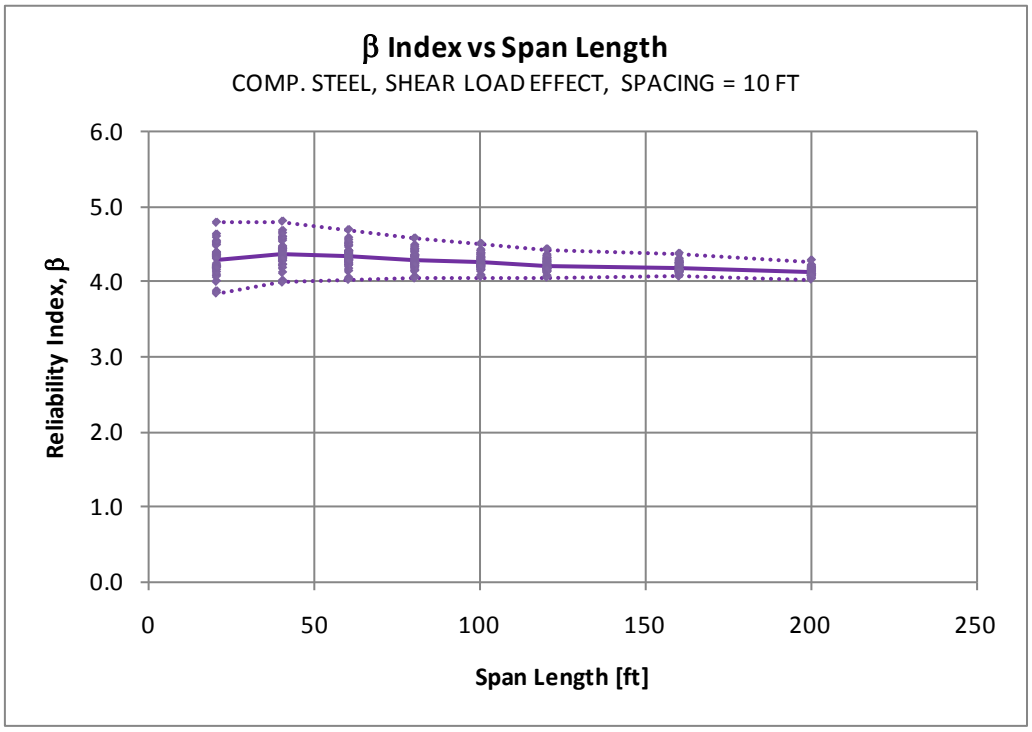

β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

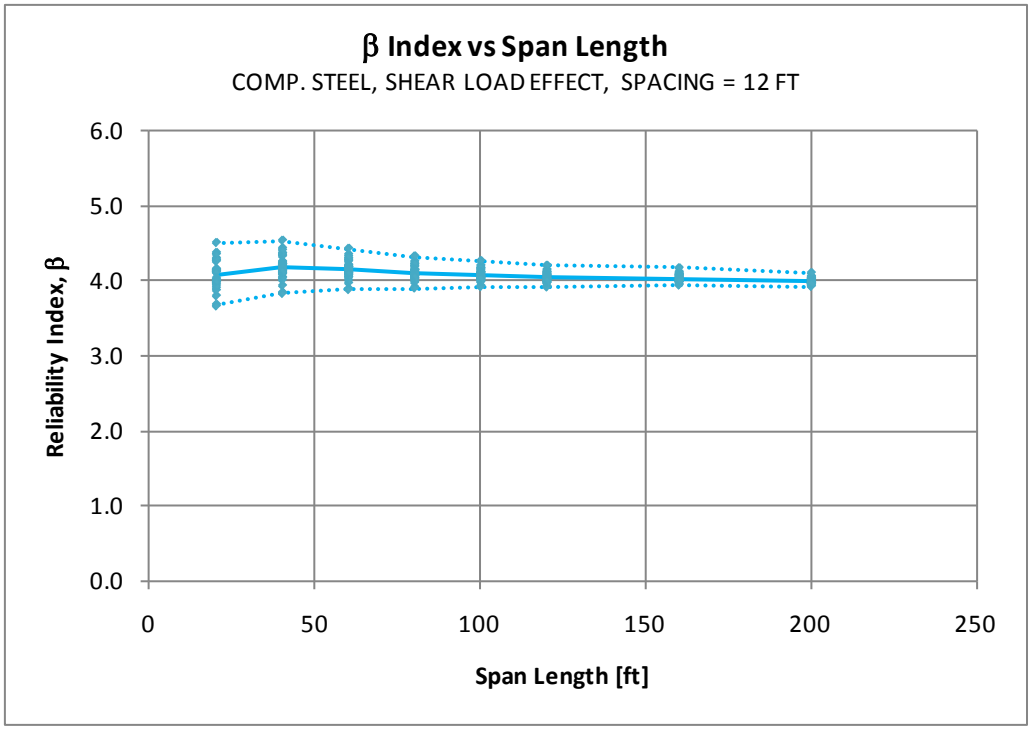

Average β Index vs Span Length
COMP. STEEL, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

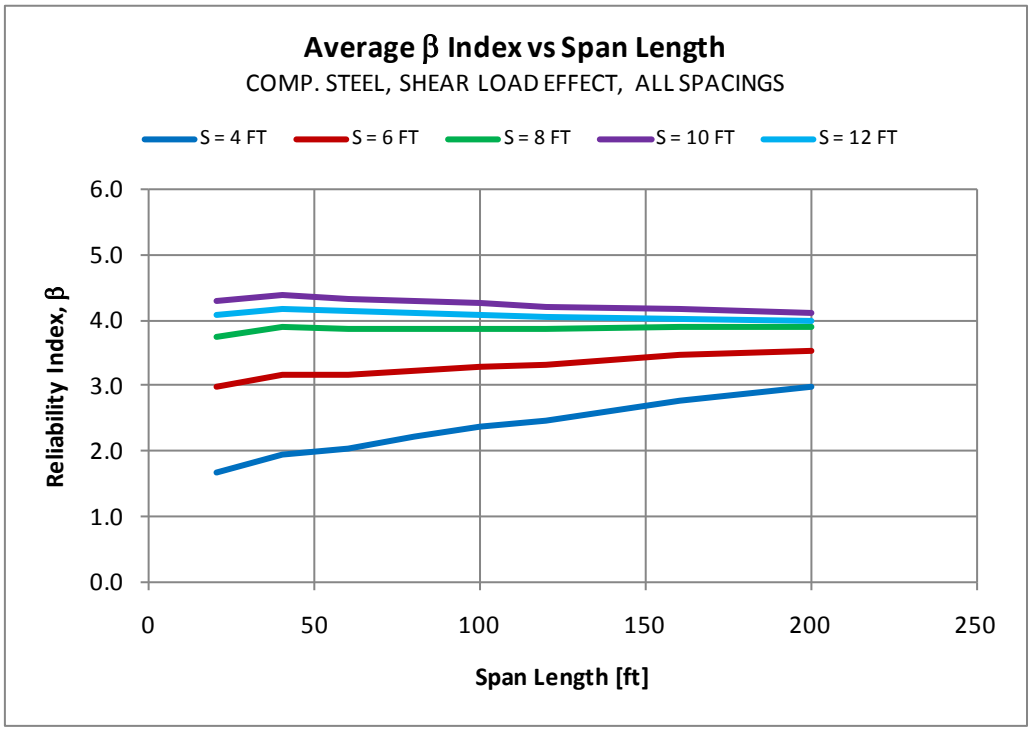

β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

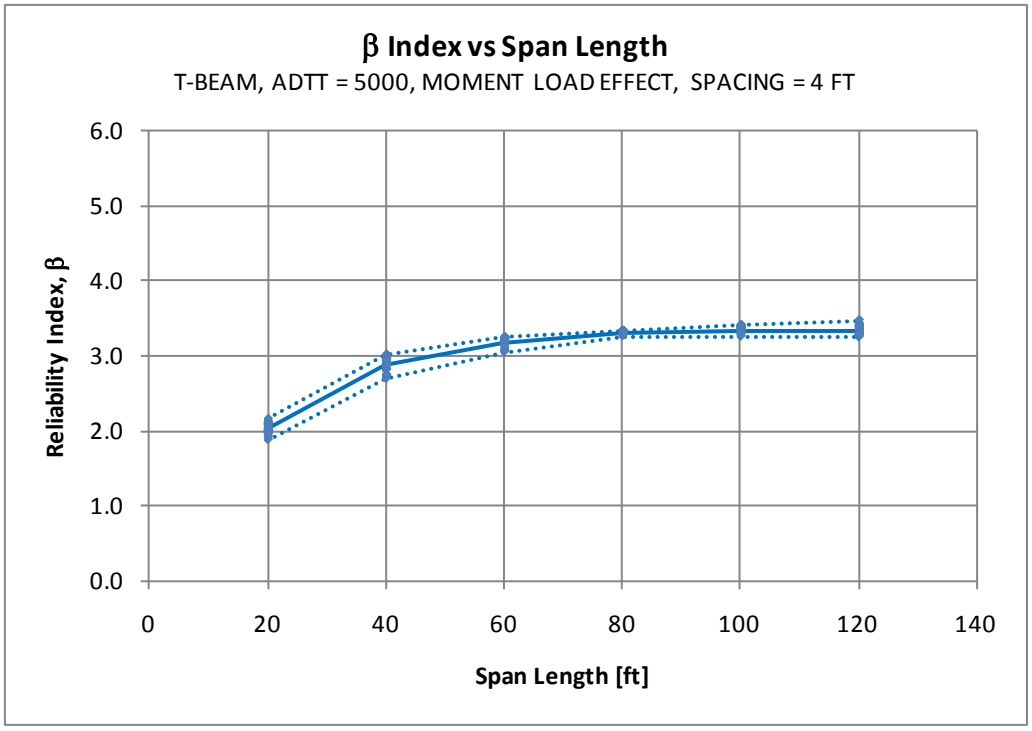

β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

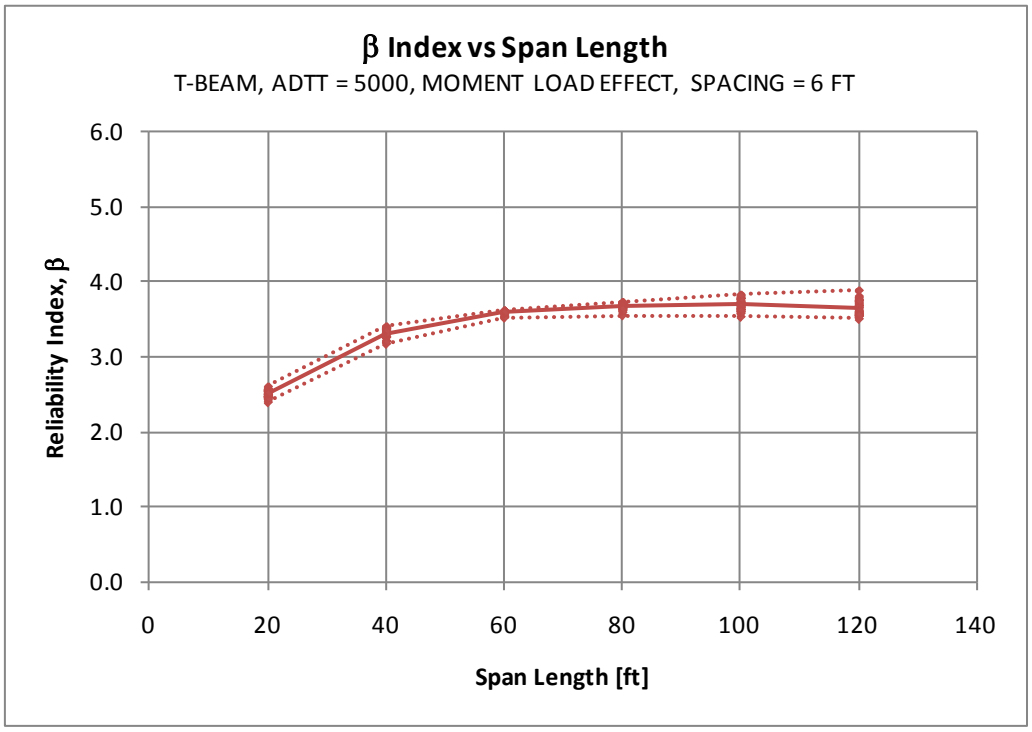

Average β Index vs Span Length
COMP. STEEL, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

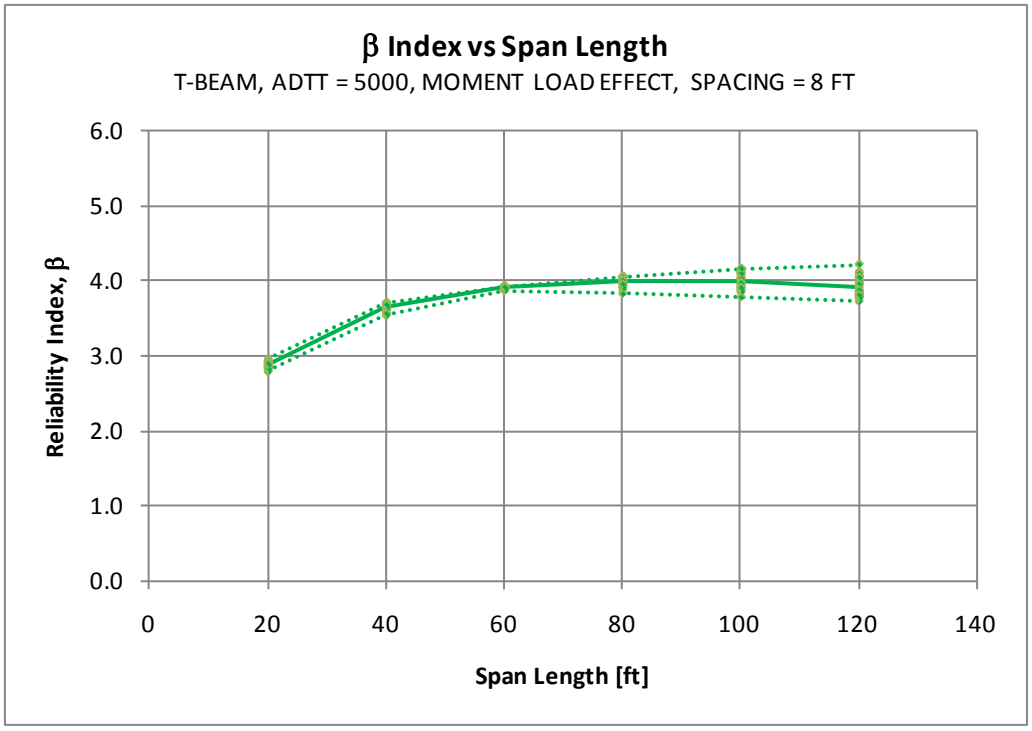
REINFORCED CONCRETE T- BEAM

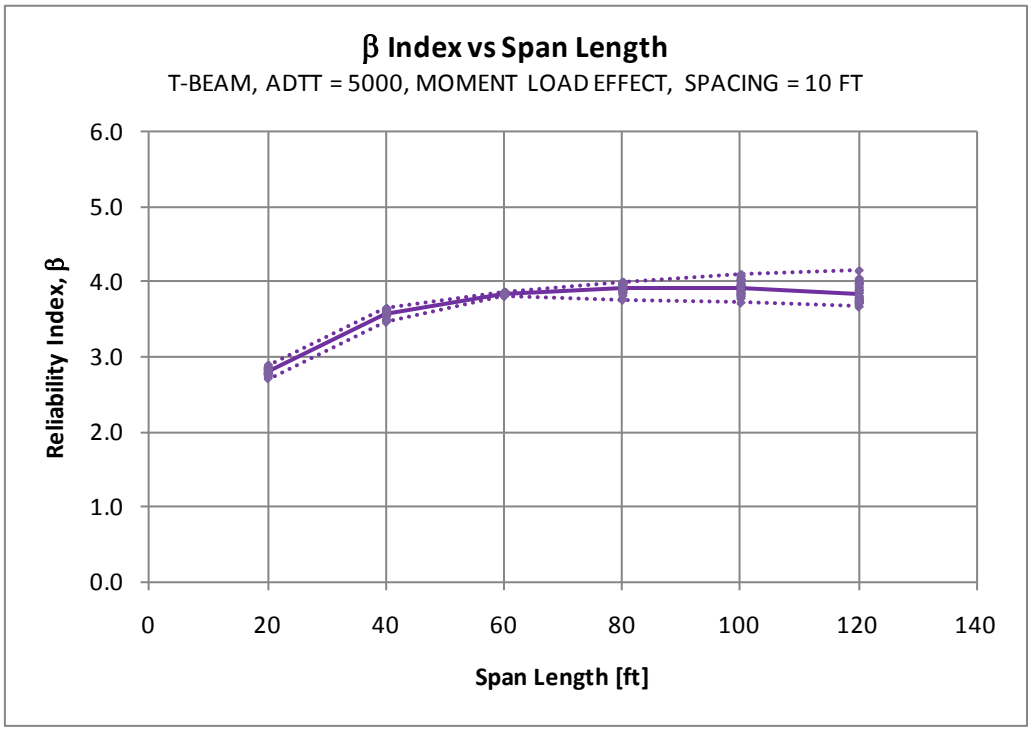
15% OVERLOAD PERMIT PLOTS

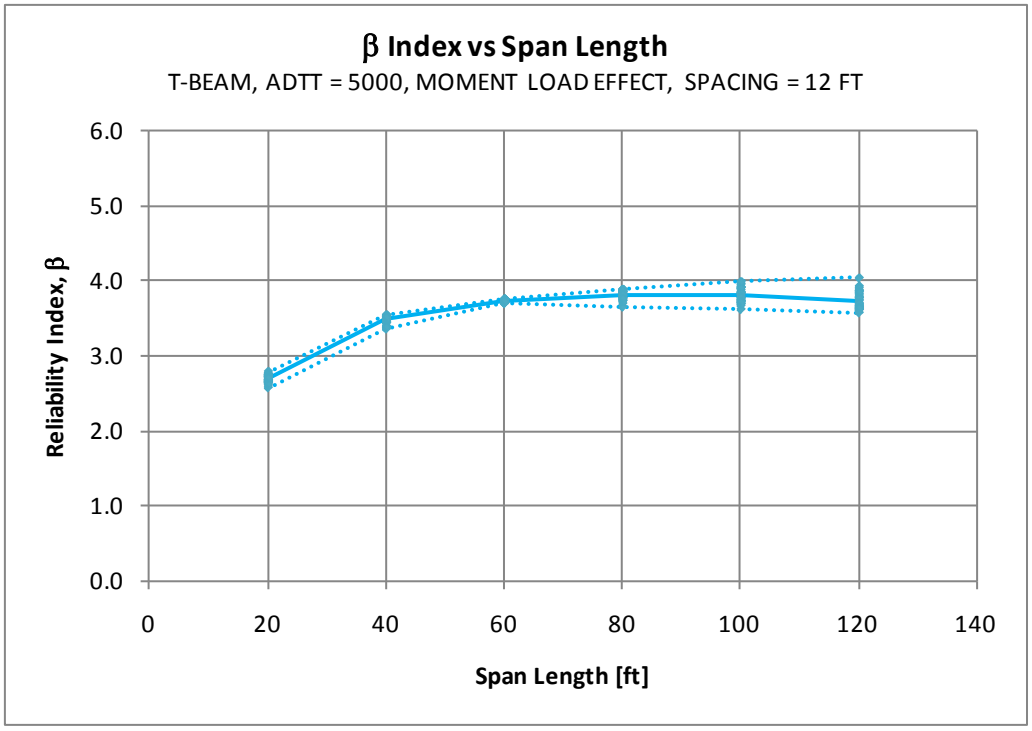
(ONE LANE)

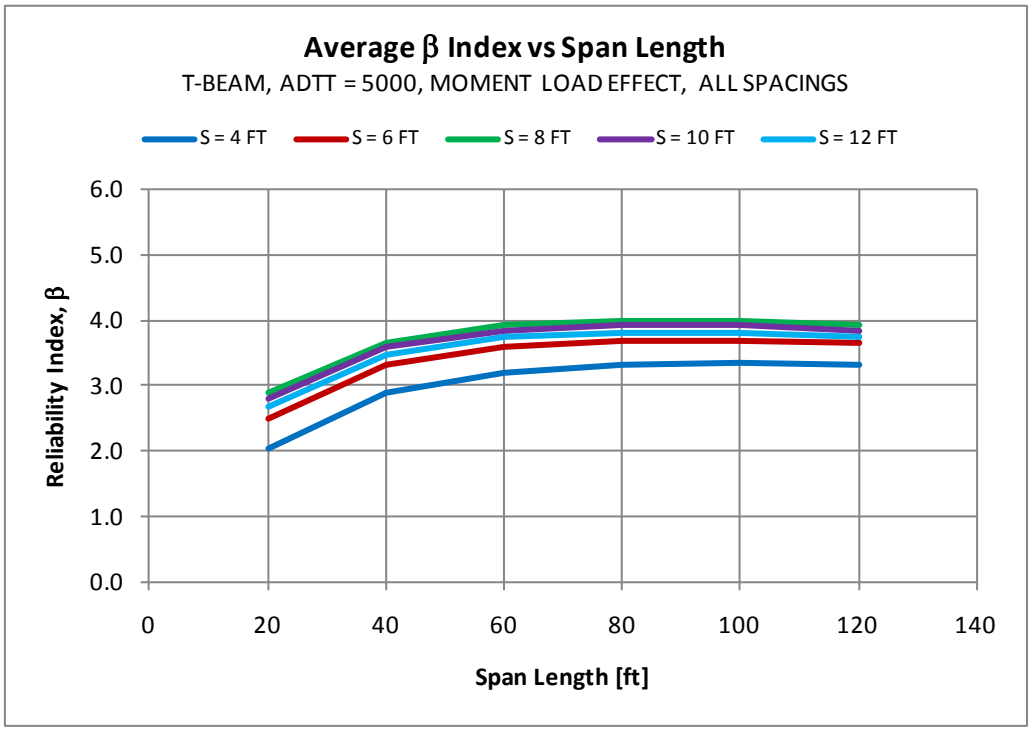

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

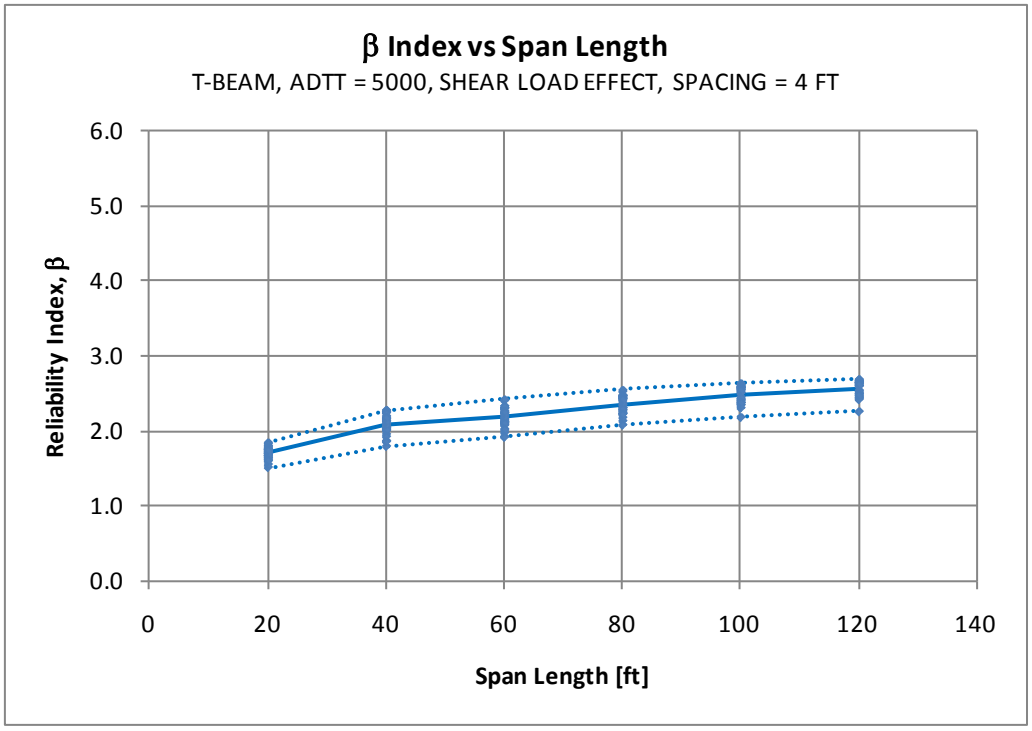

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

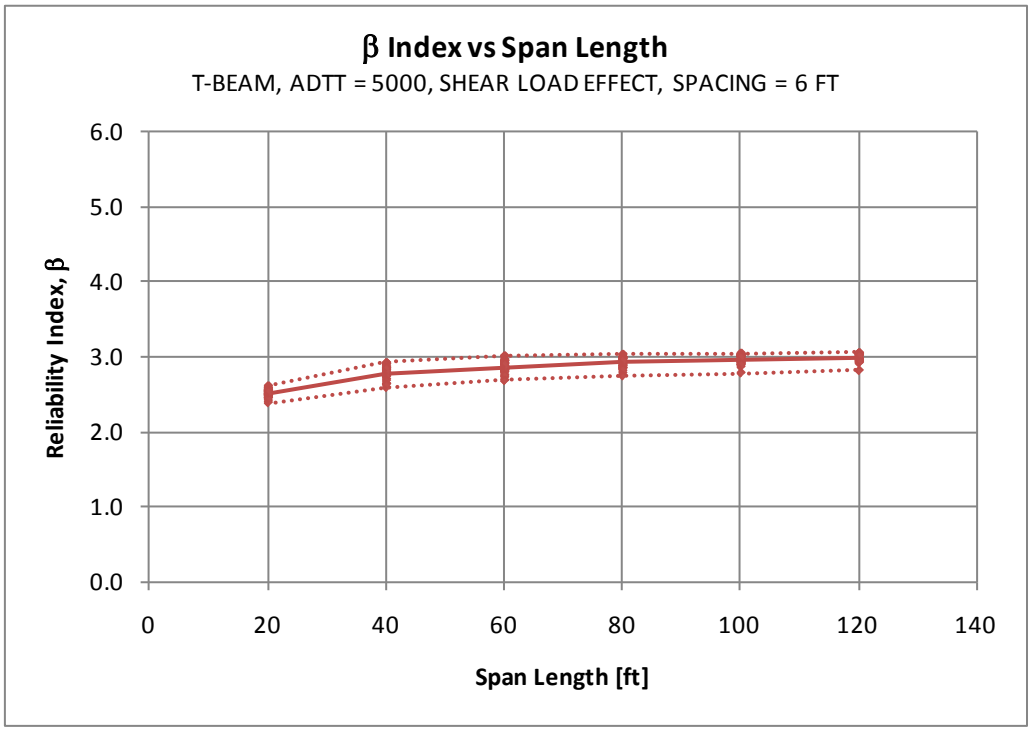

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

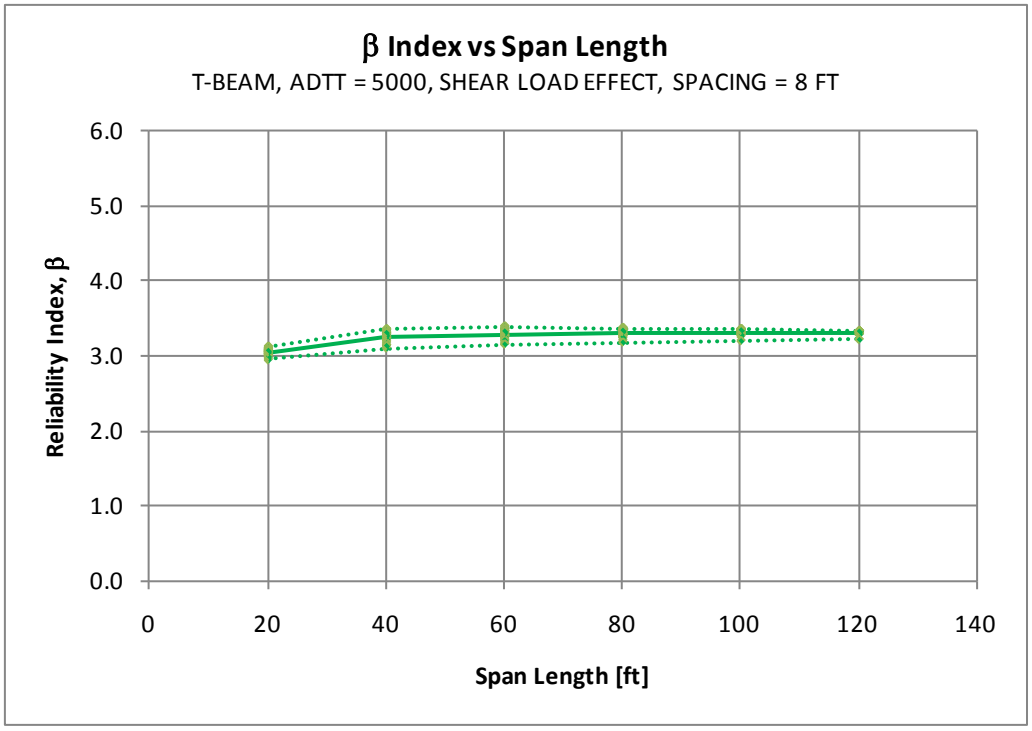

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

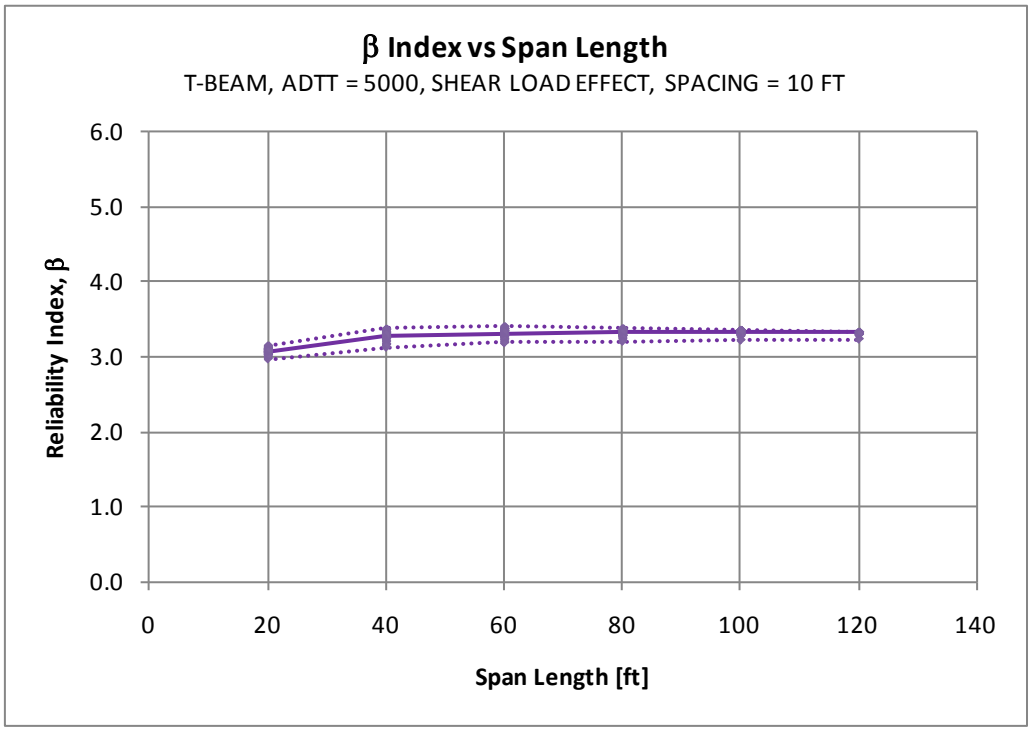

β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

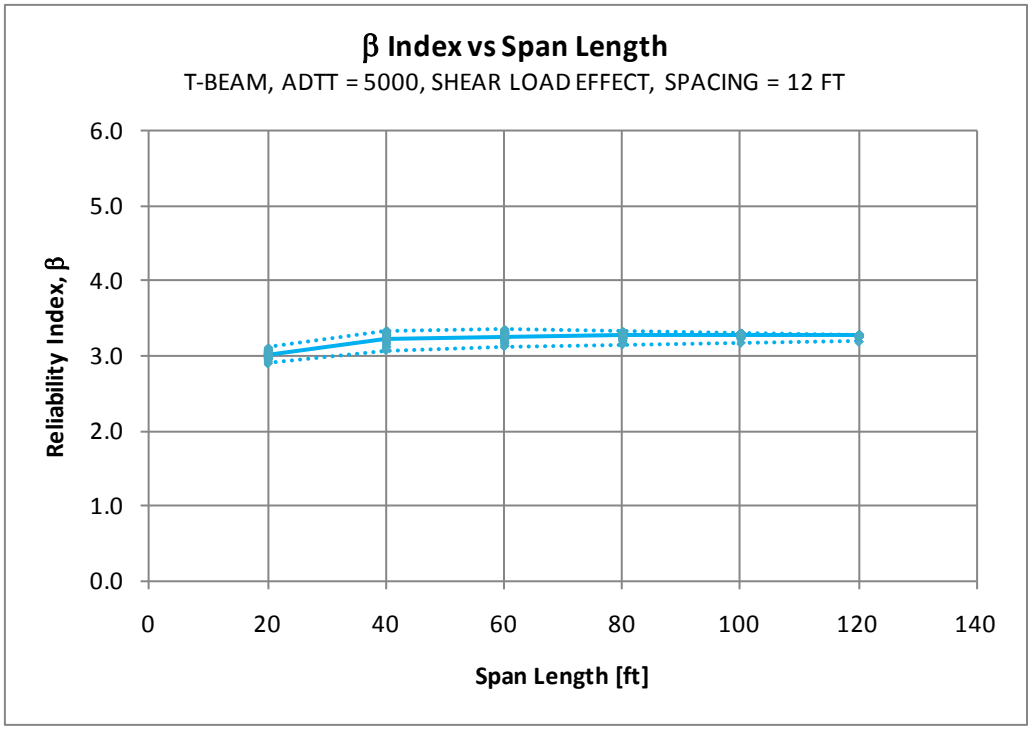

Average β Index vs Span Length
T-BEAM, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

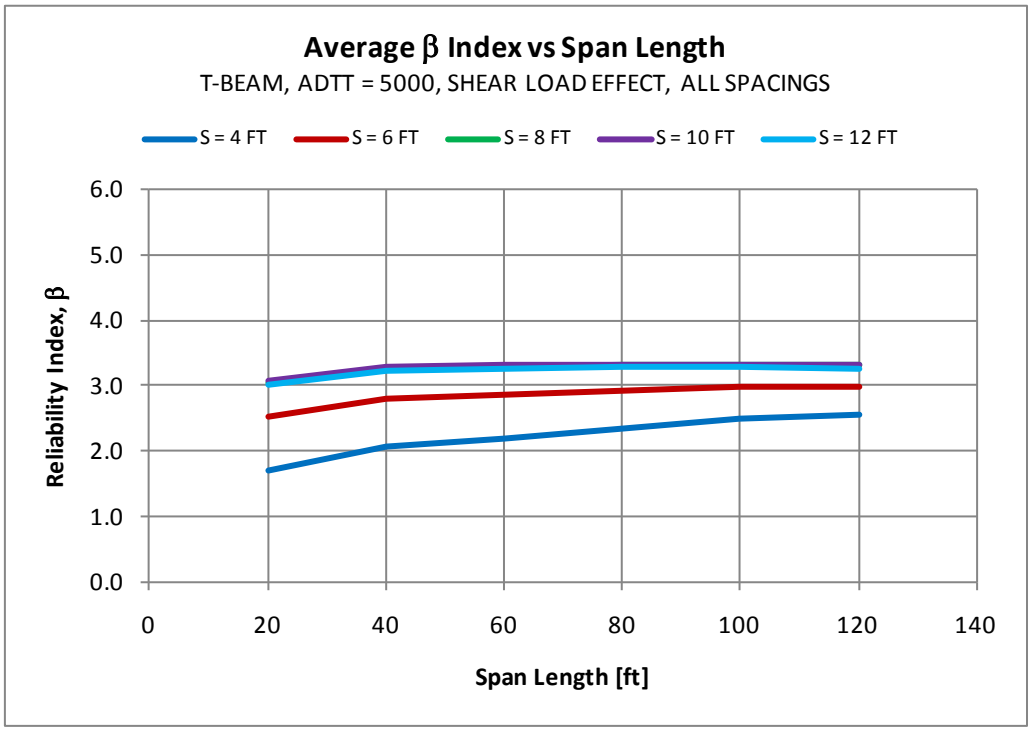

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]


β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

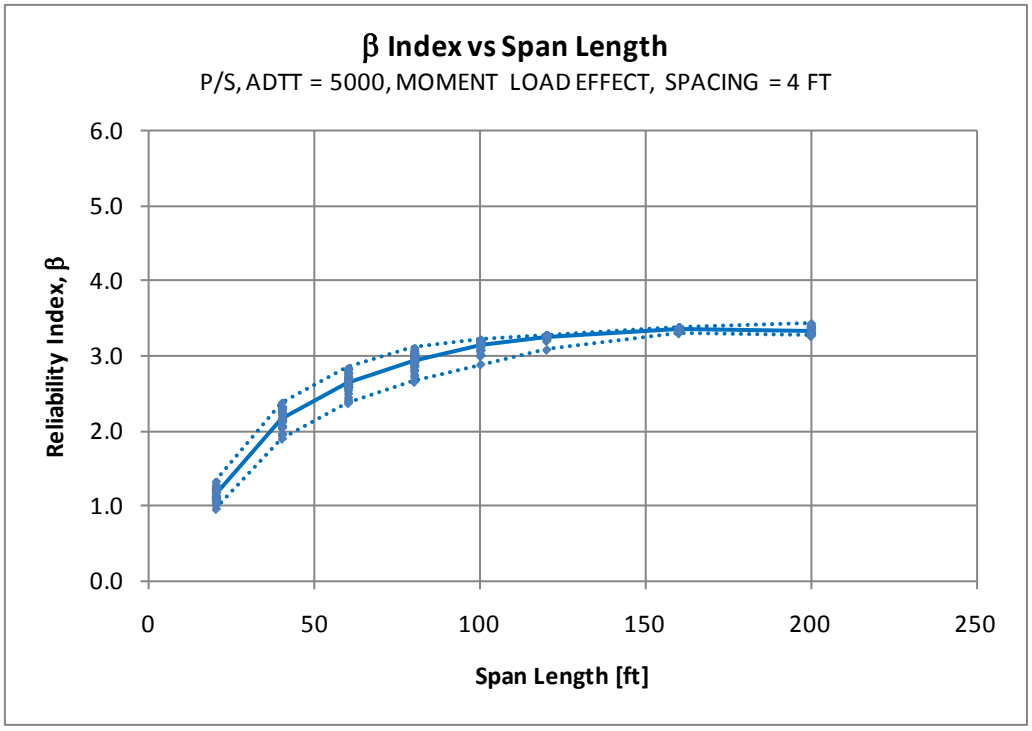

β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

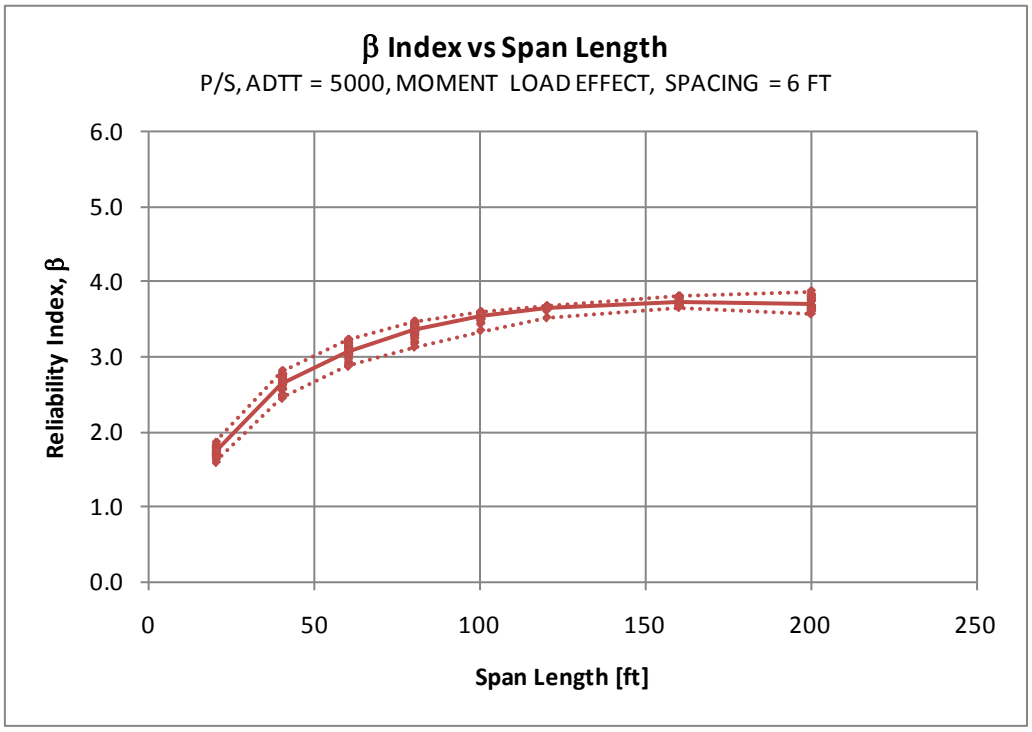

Average β Index vs Span Length
T-BEAM, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
60
80
100
120
140
Span Length [ft]

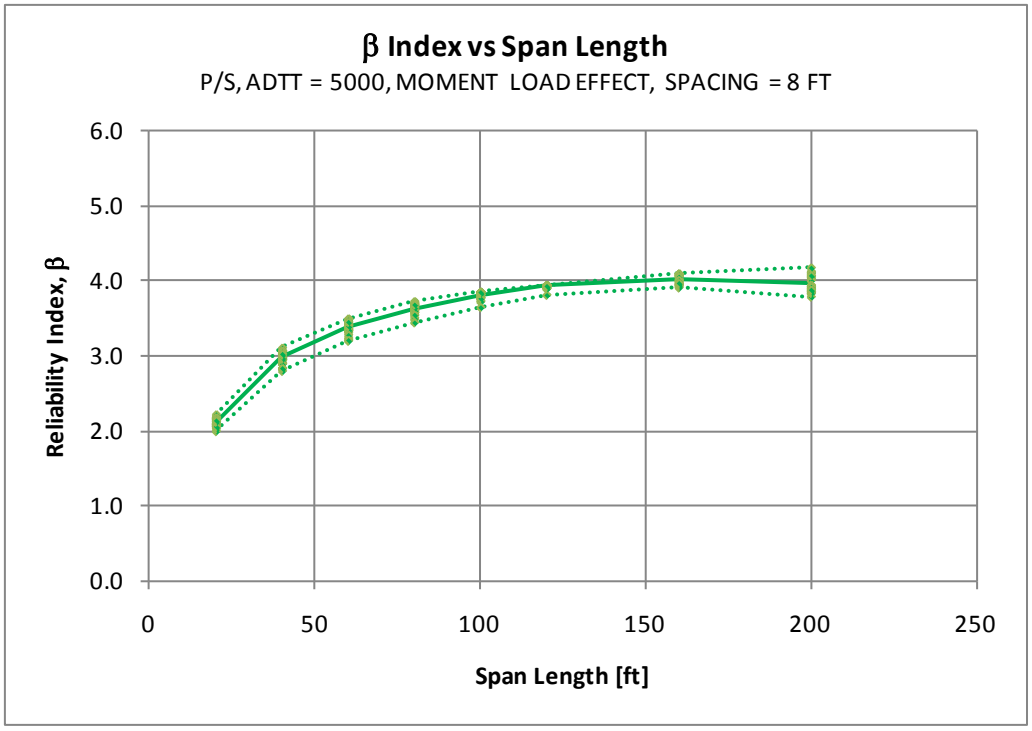
PRESTRESSED CONCRETE BEAM

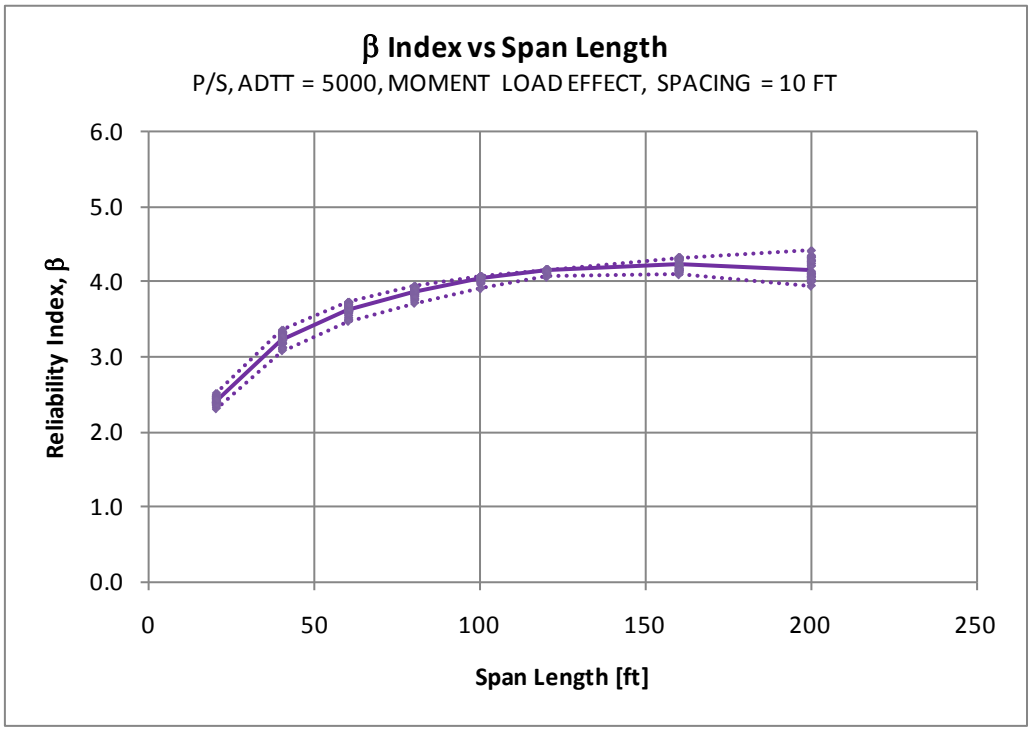
15% OVERLOAD PERMIT PLOTS

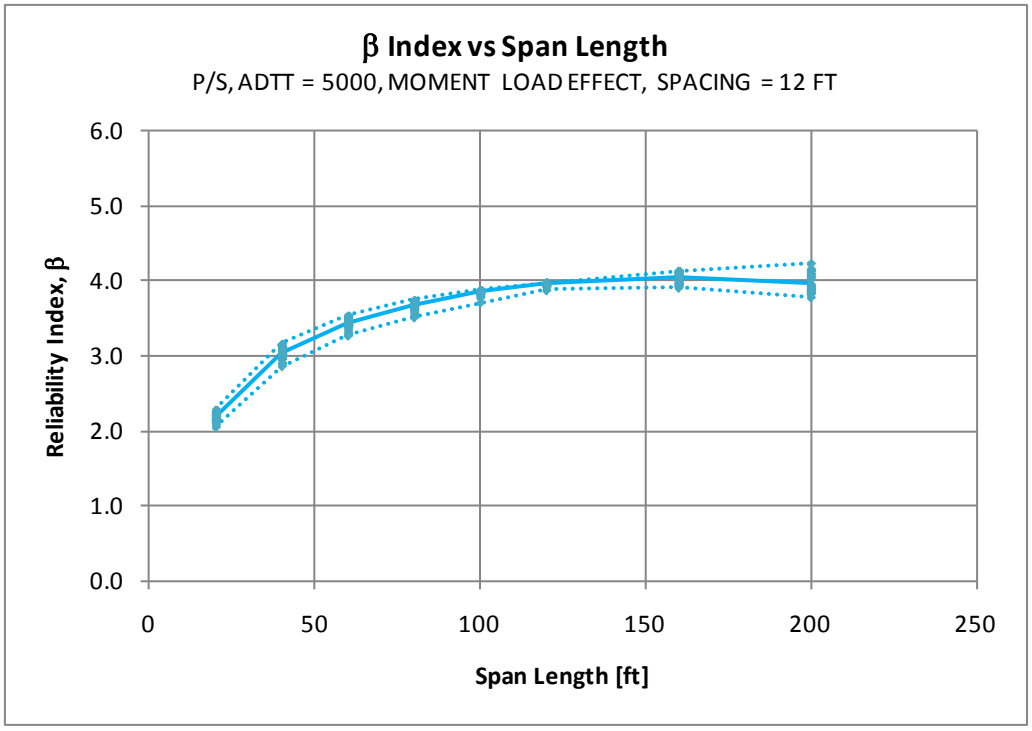
(ONE LANE)

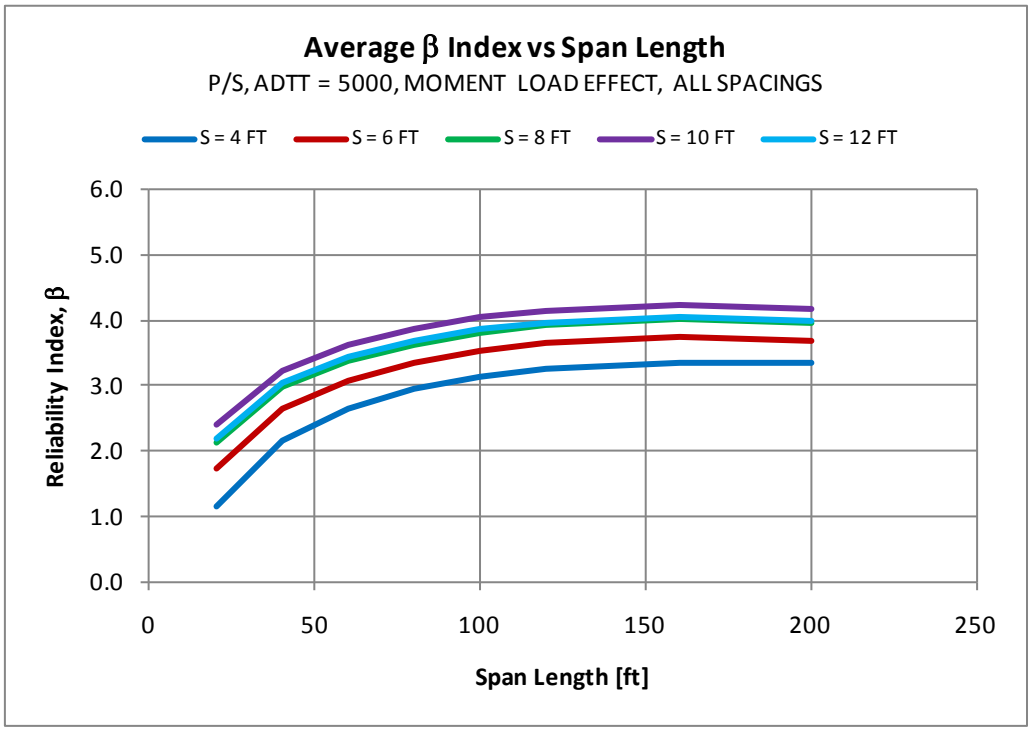

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

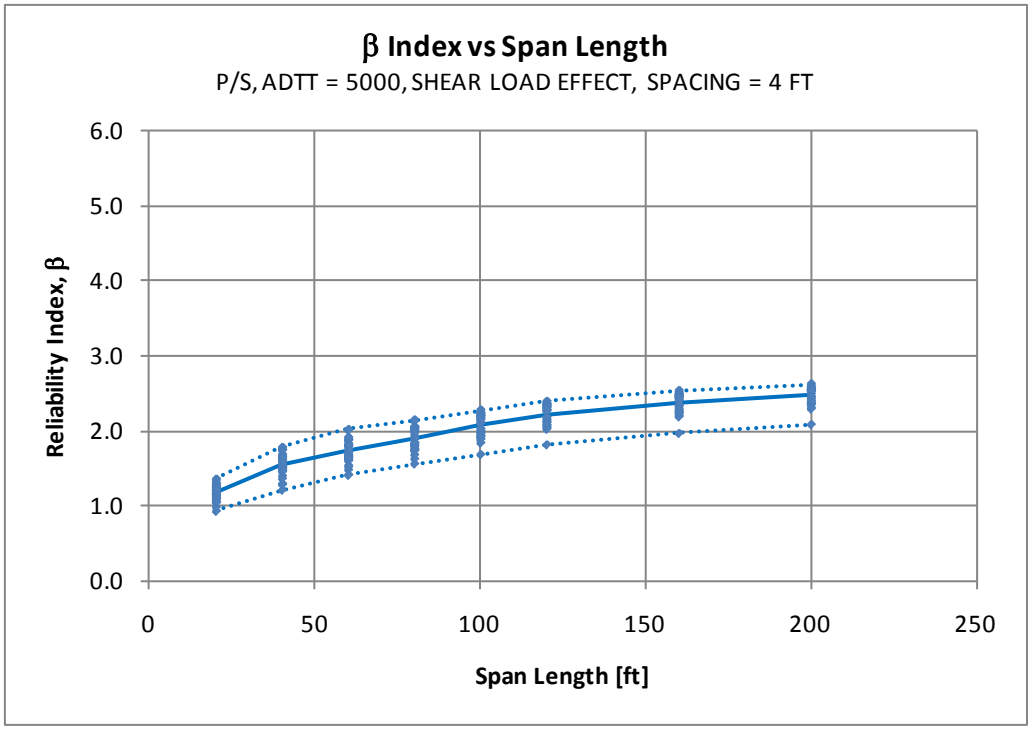

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

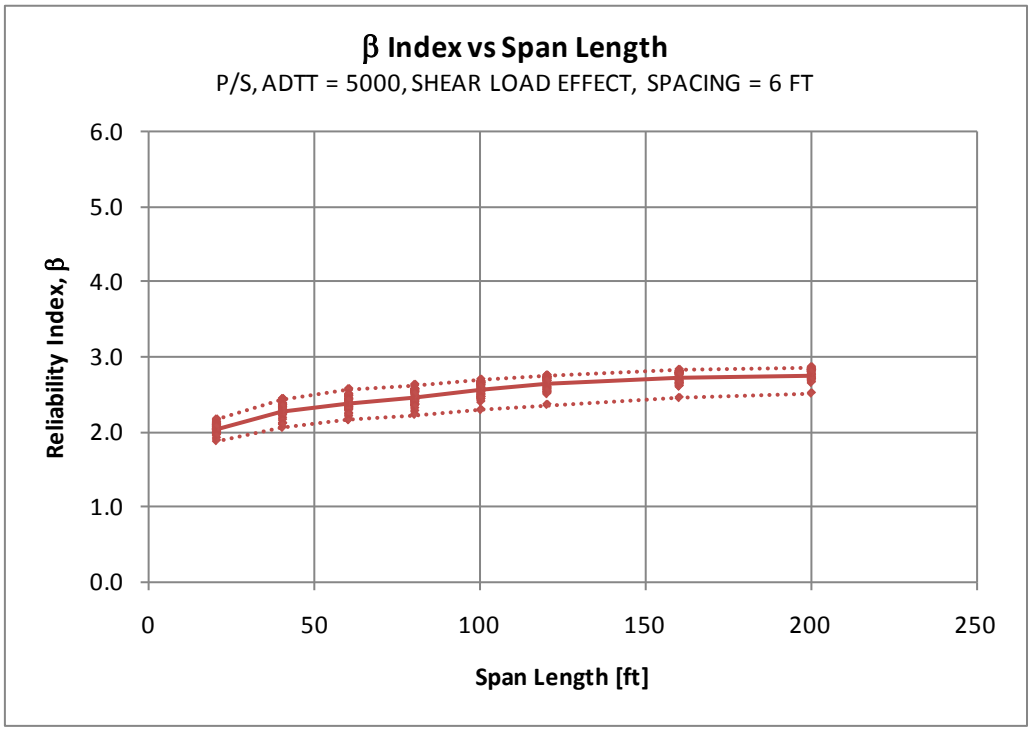

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

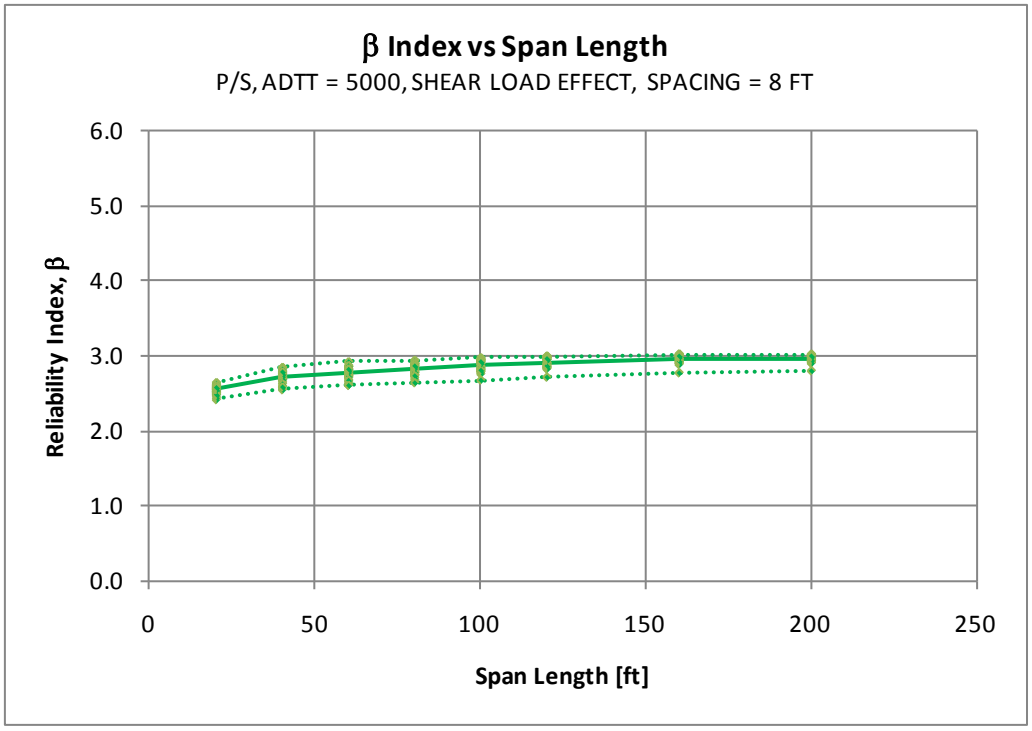

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

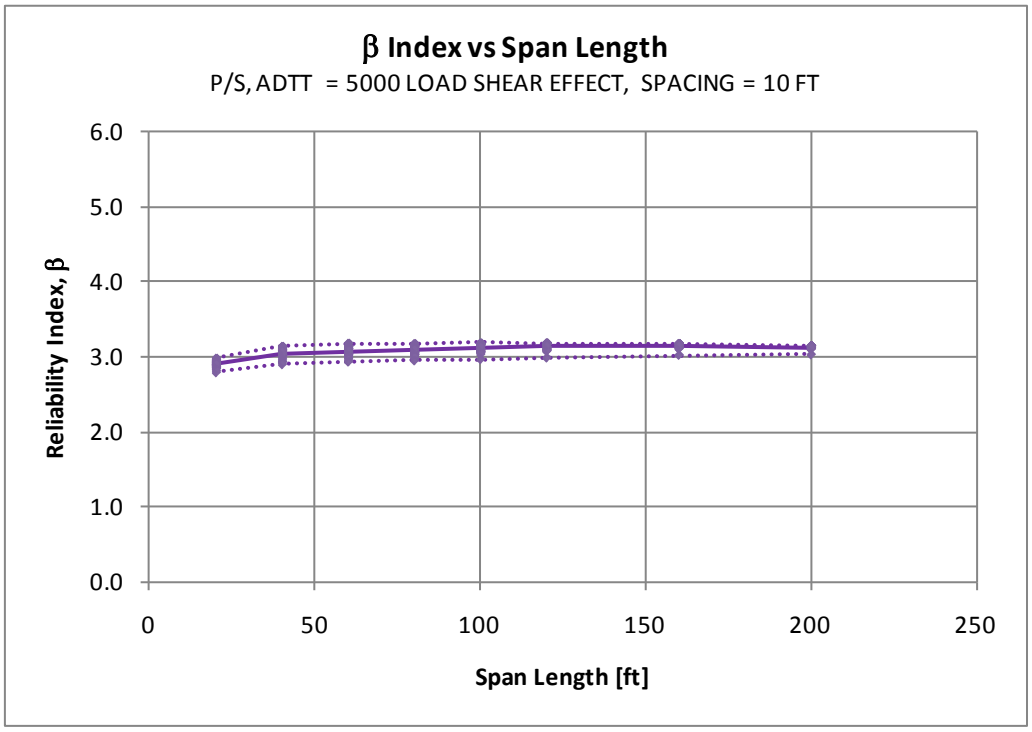

β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

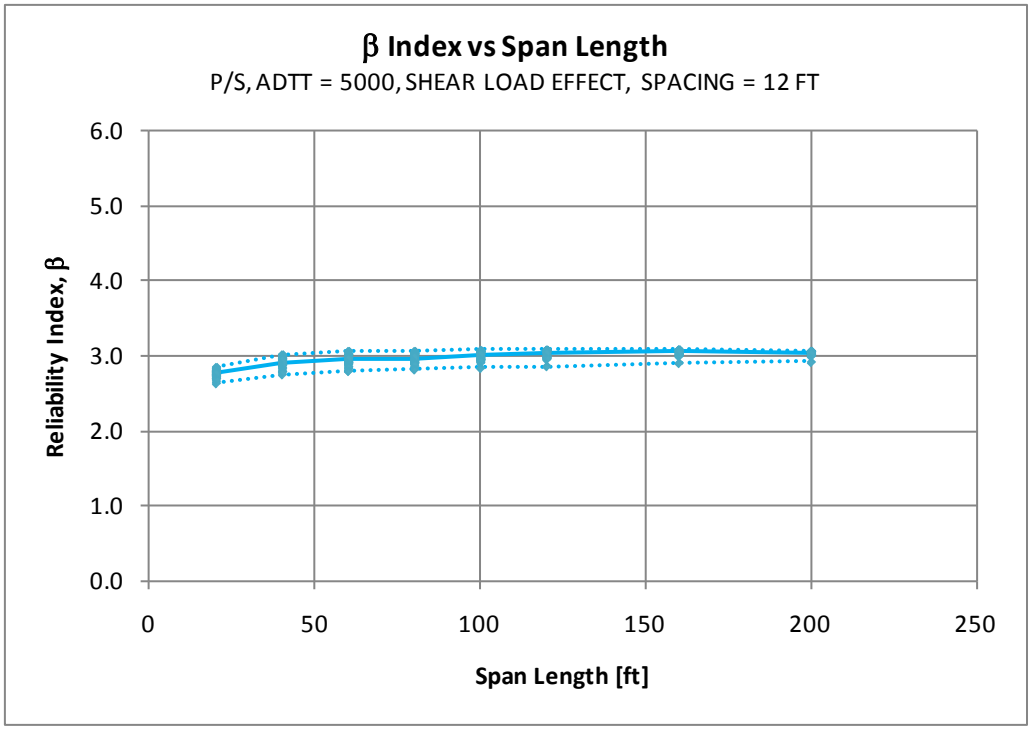

Average β Index vs Span Length
P/S, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

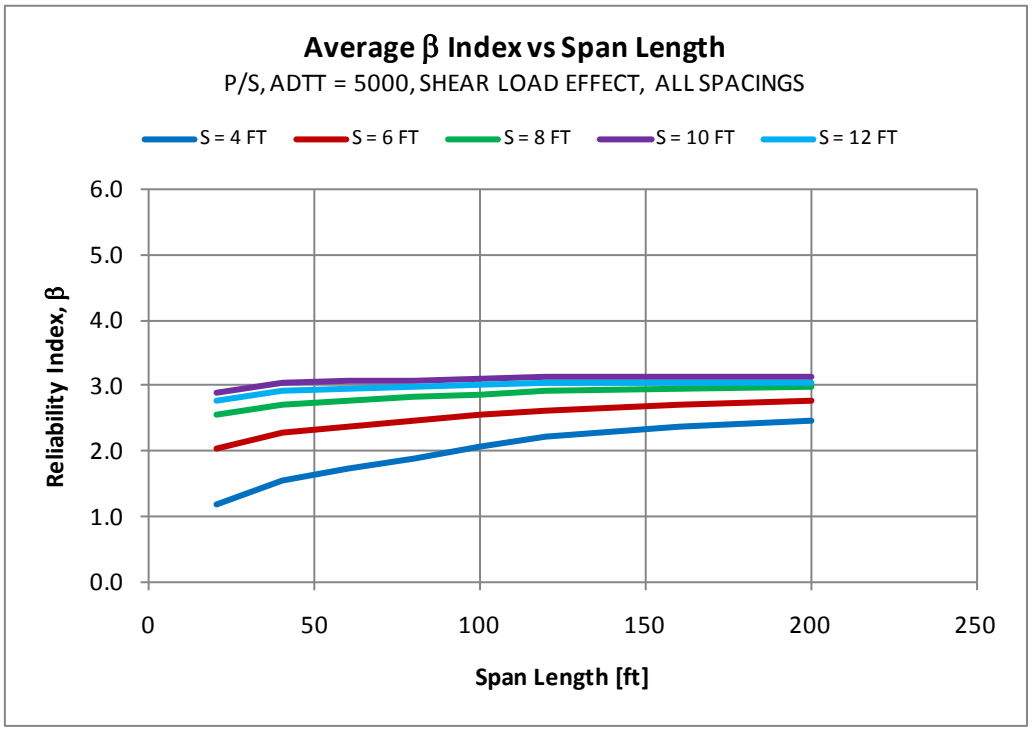

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
P/S, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

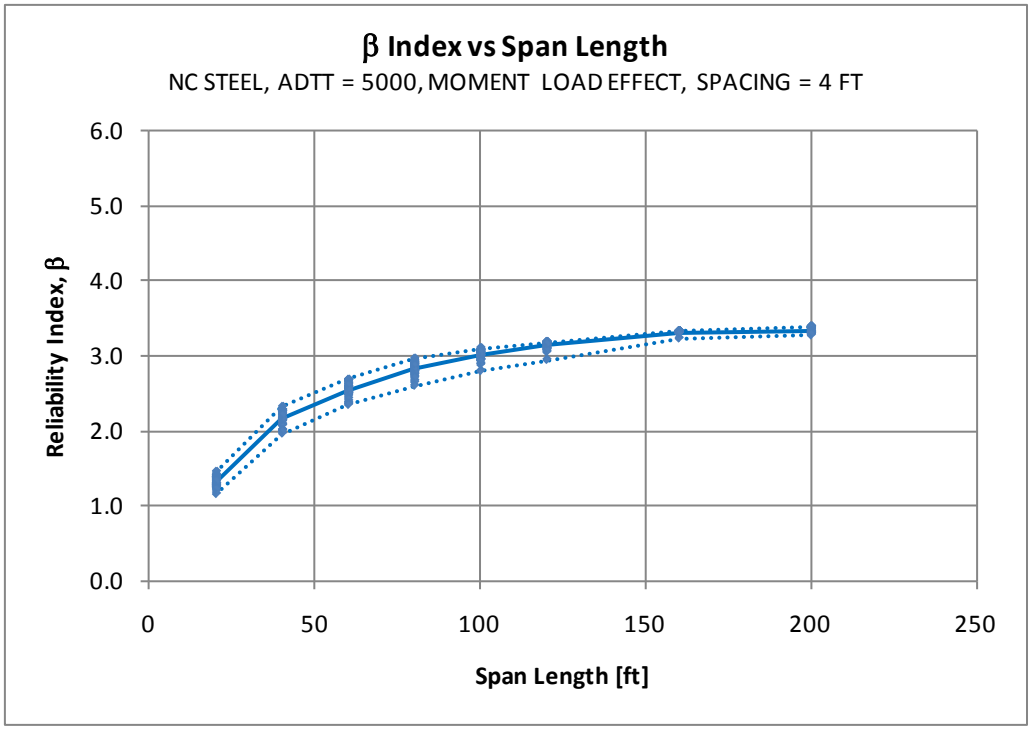

β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

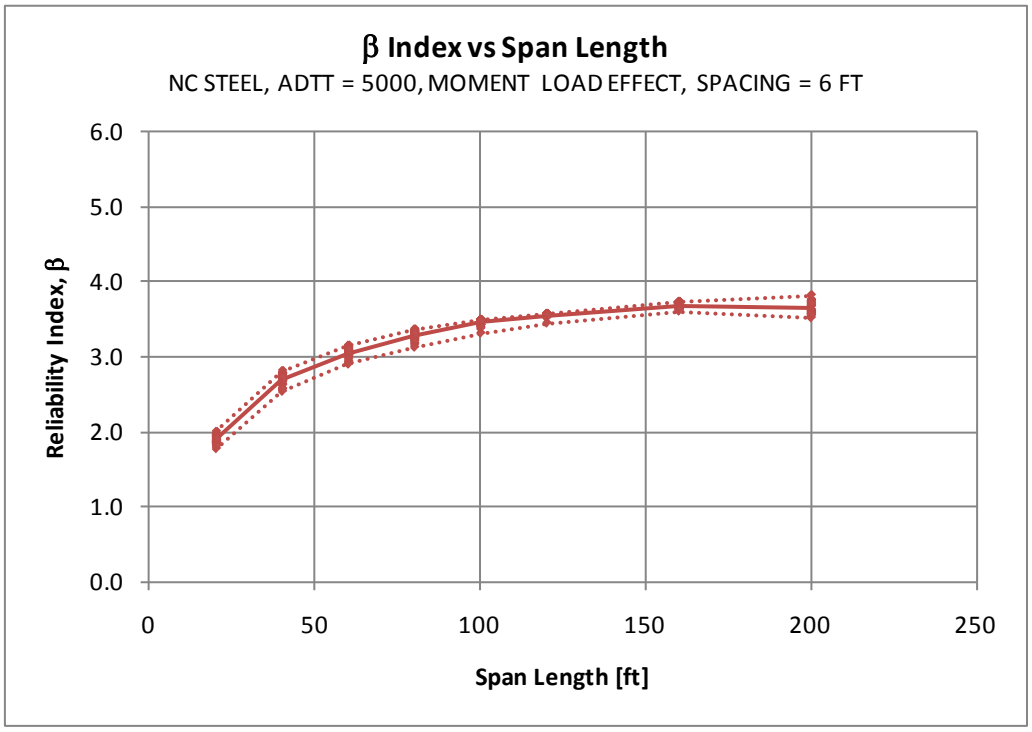

Average β Index vs Span Length
P/S, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

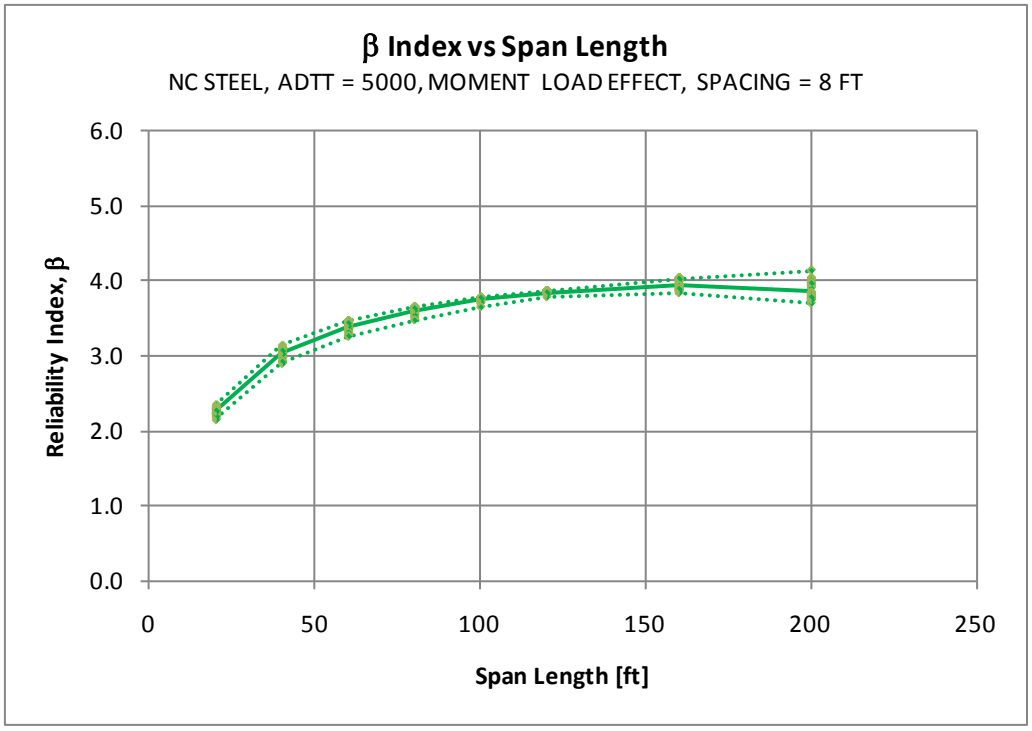
NON-COMPOSITE STEEL BEAM

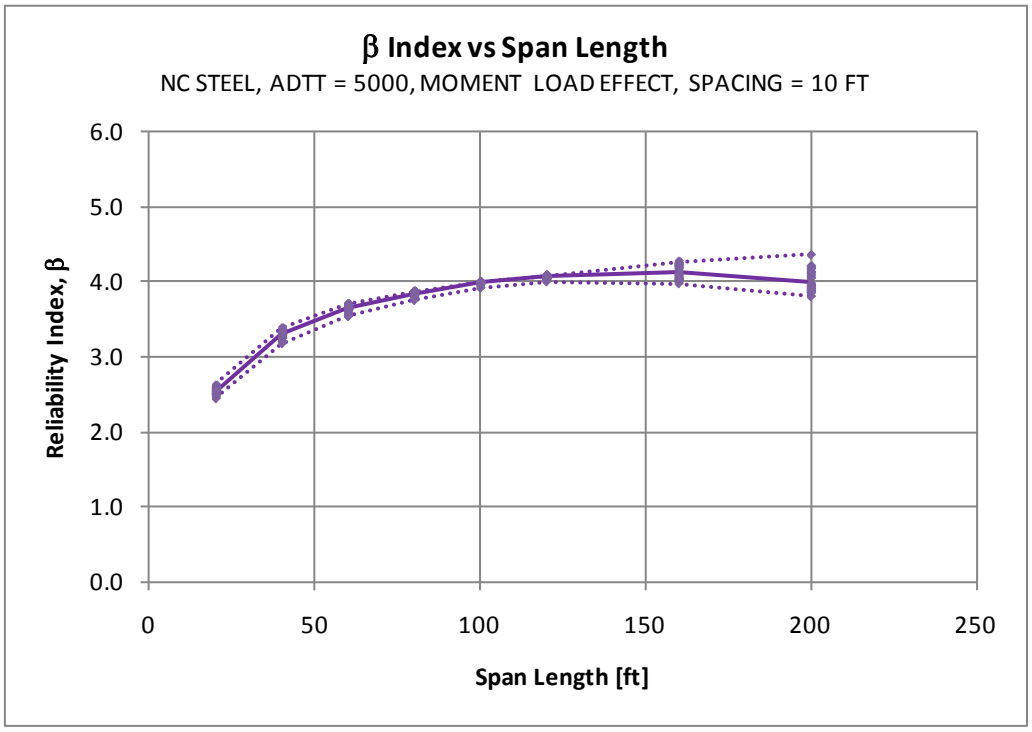
15% OVERLOAD PERMIT PLOTS

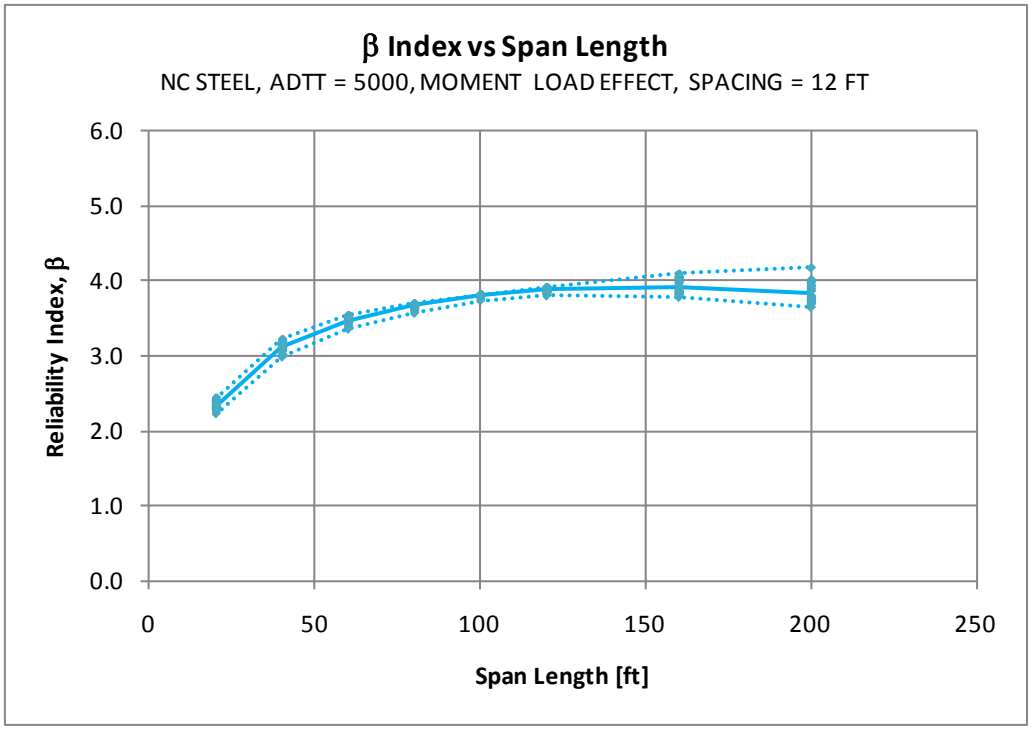
(ONE LANE)

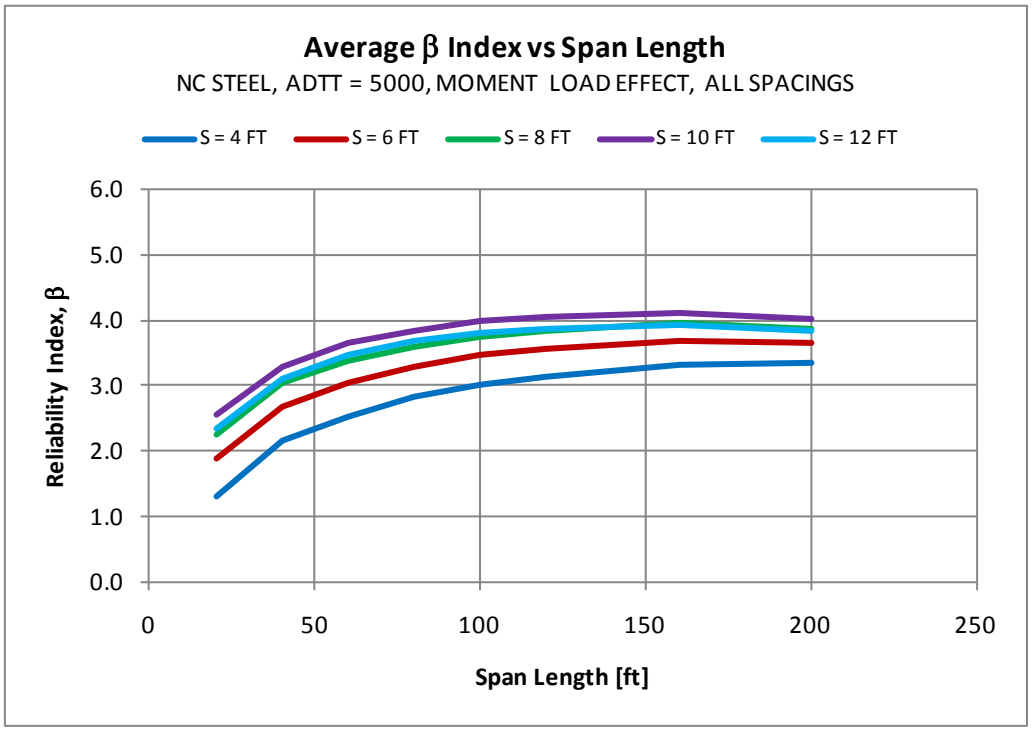

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

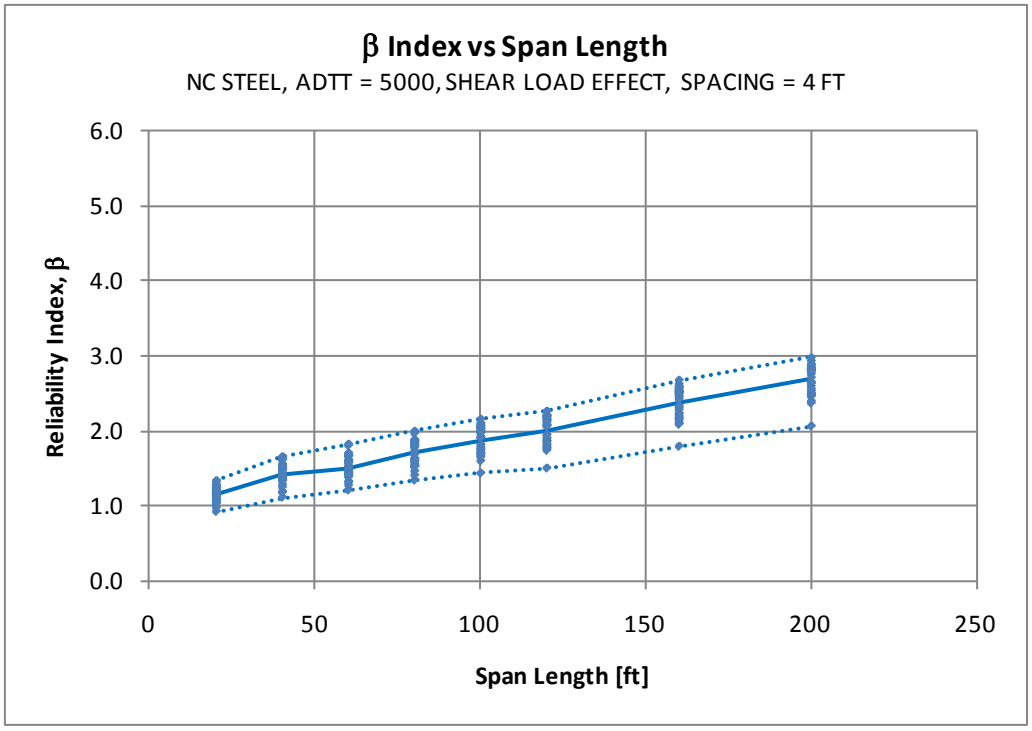

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

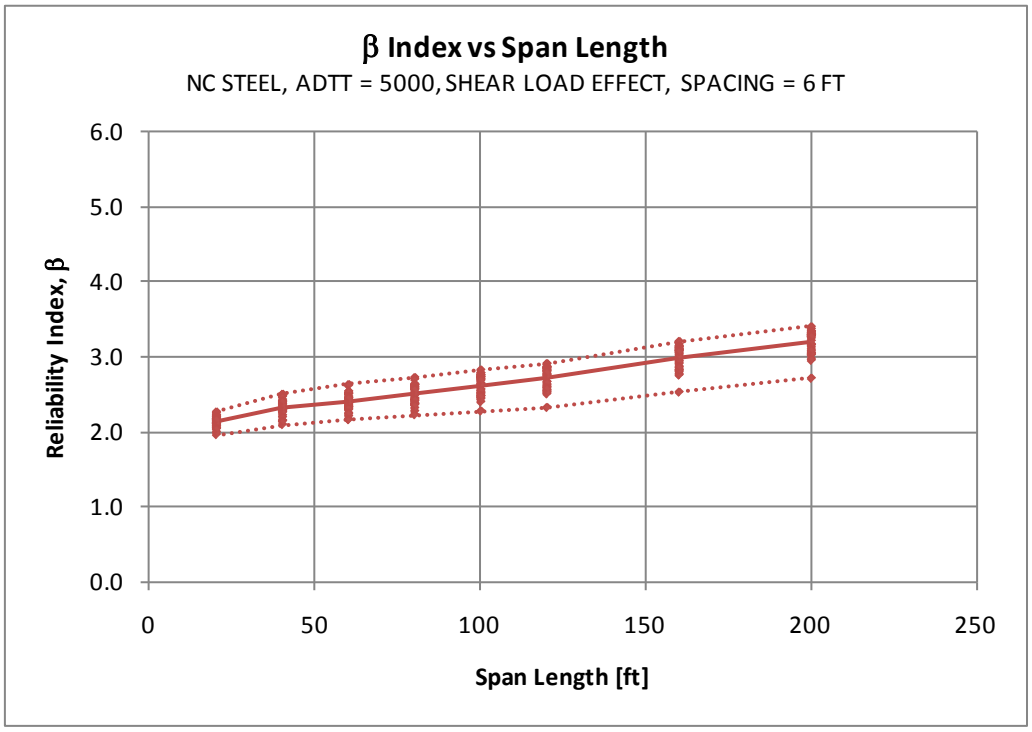

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

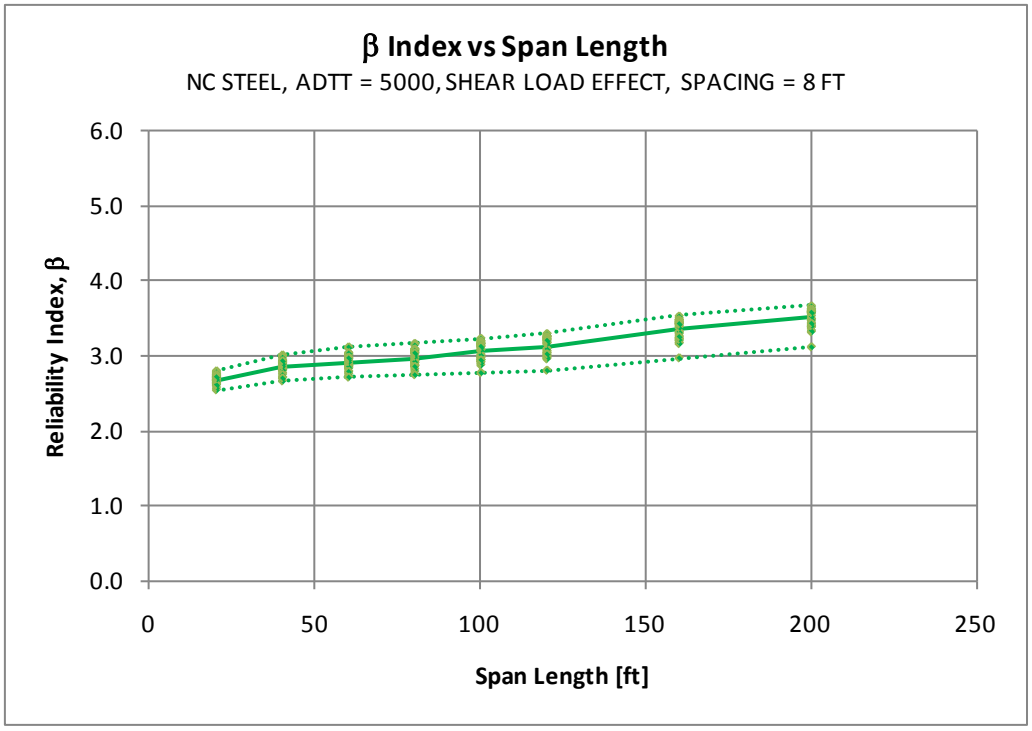

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

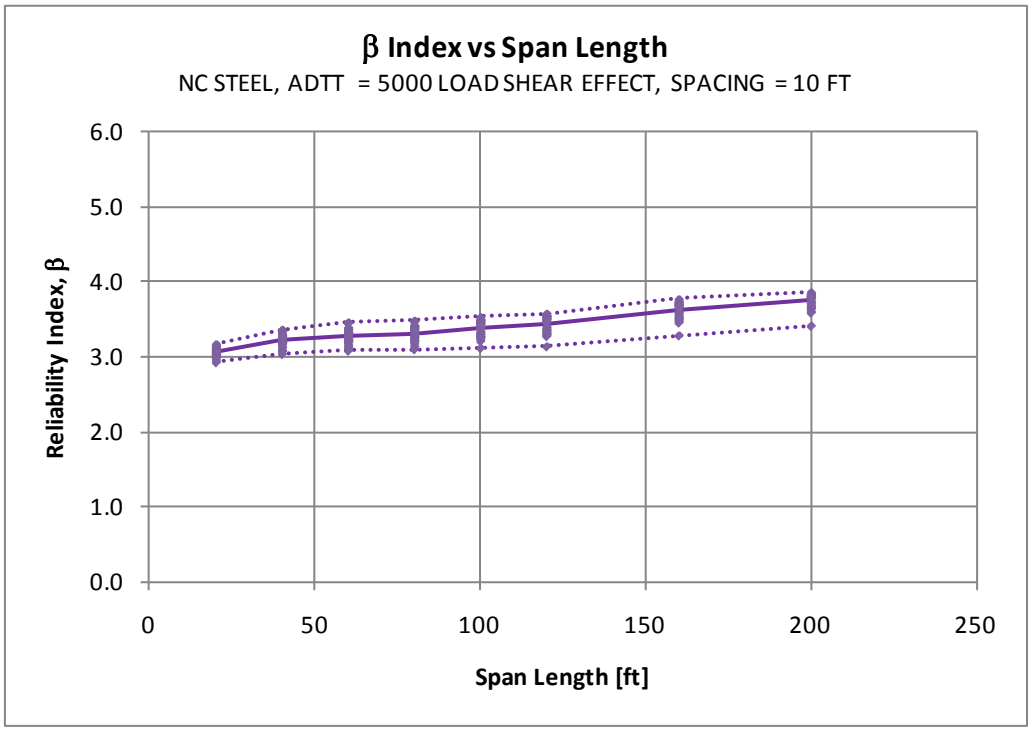

β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

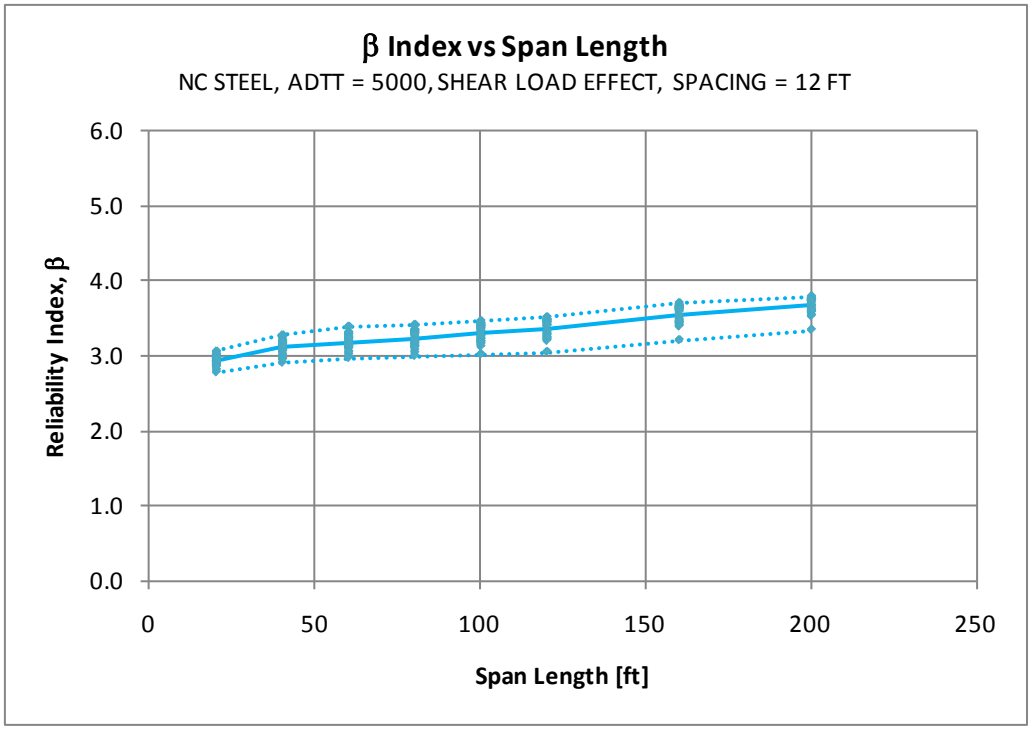

Average β Index vs Span Length
NC STEEL, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

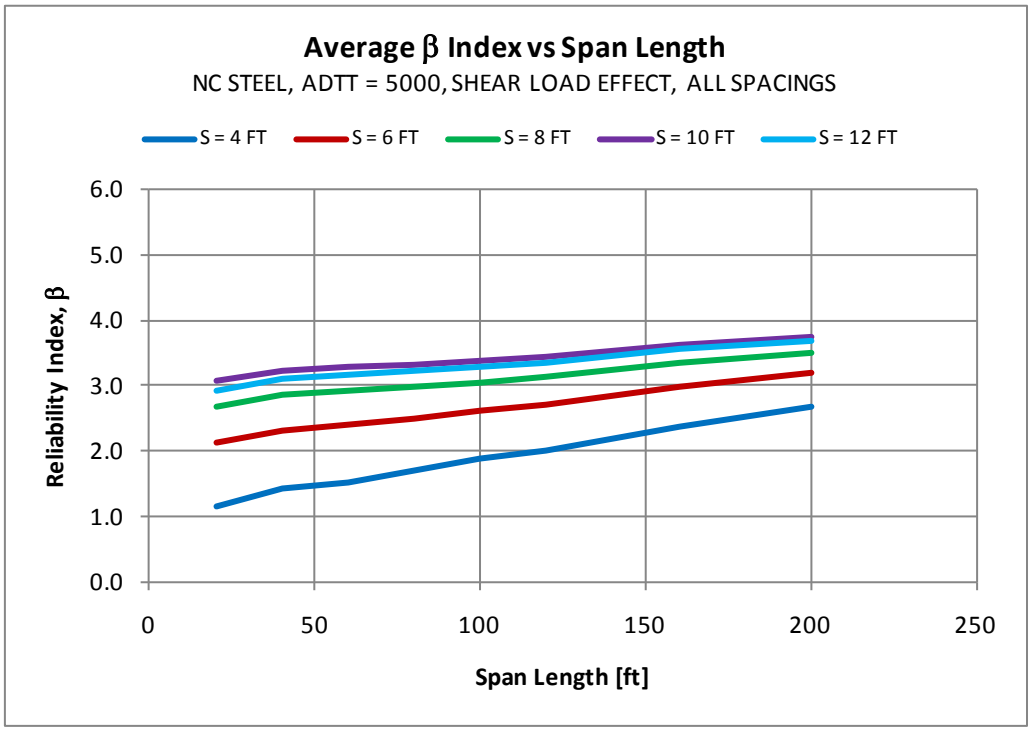

β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
NC STEEL, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

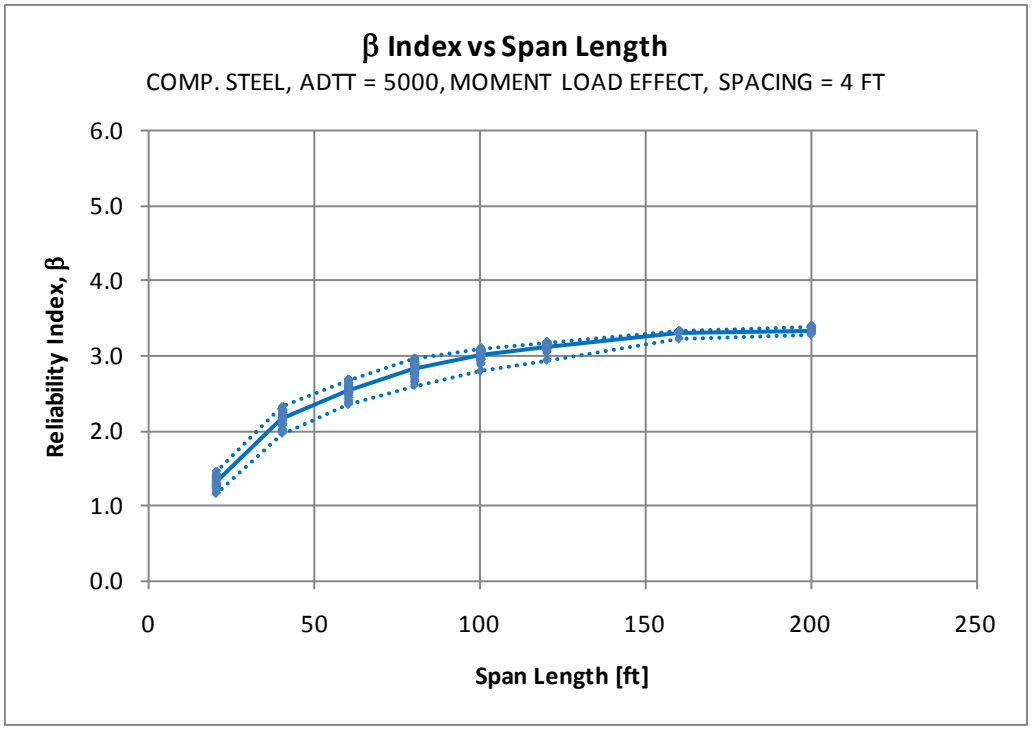

β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

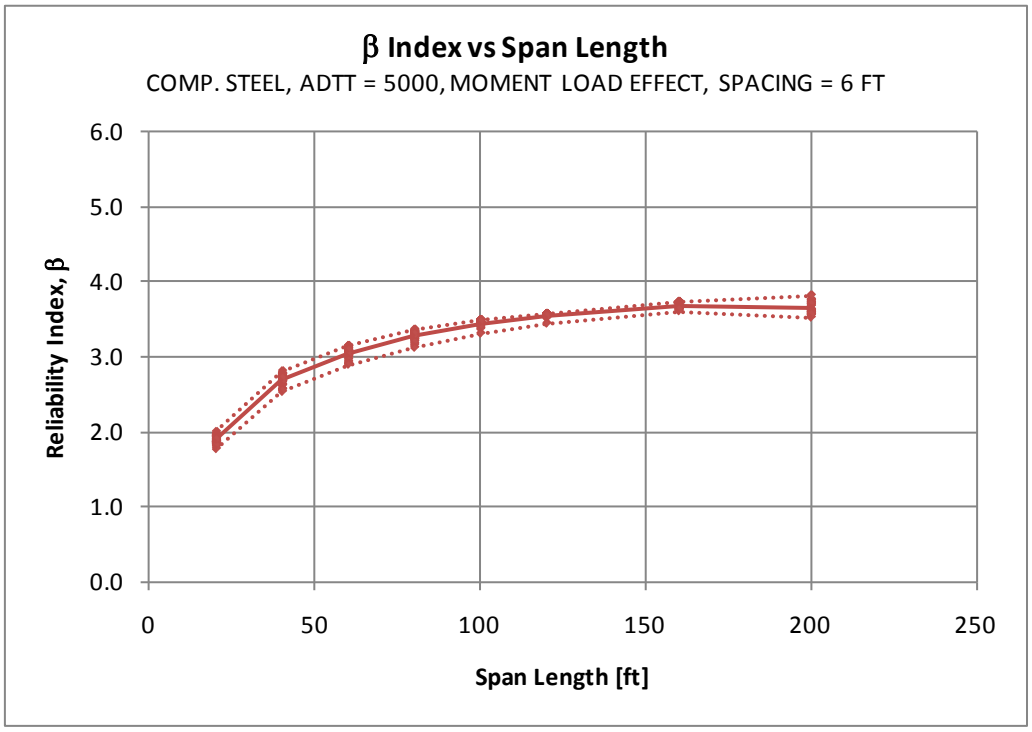

Average β Index vs Span Length
NC STEEL, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

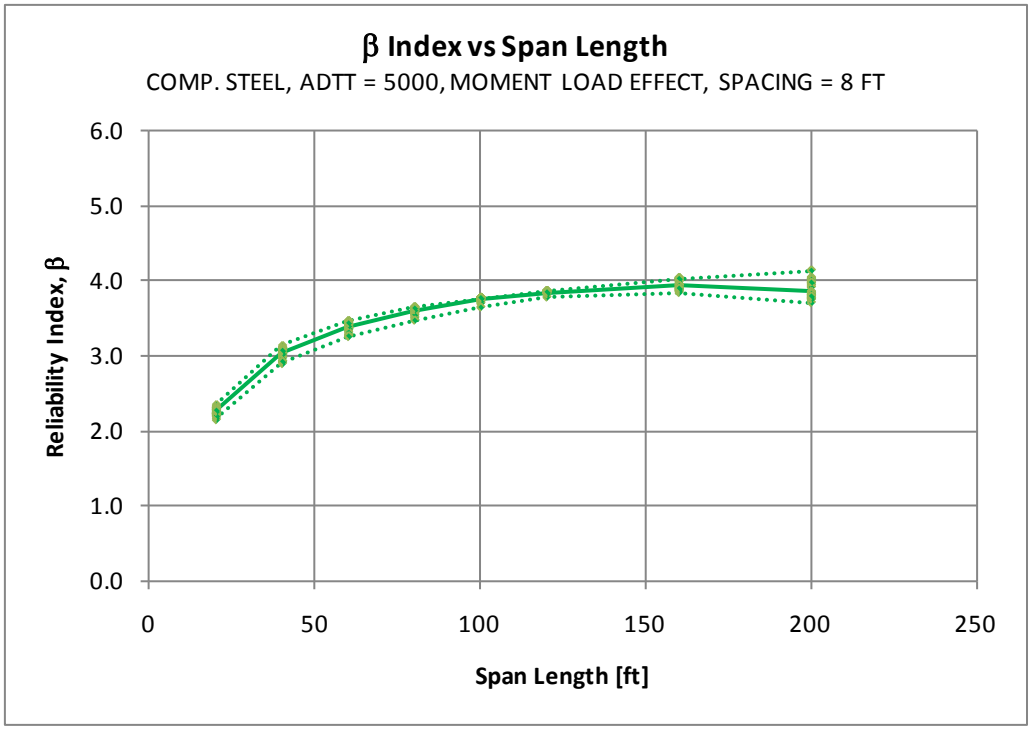
COMPOSITE STEEL BEAM

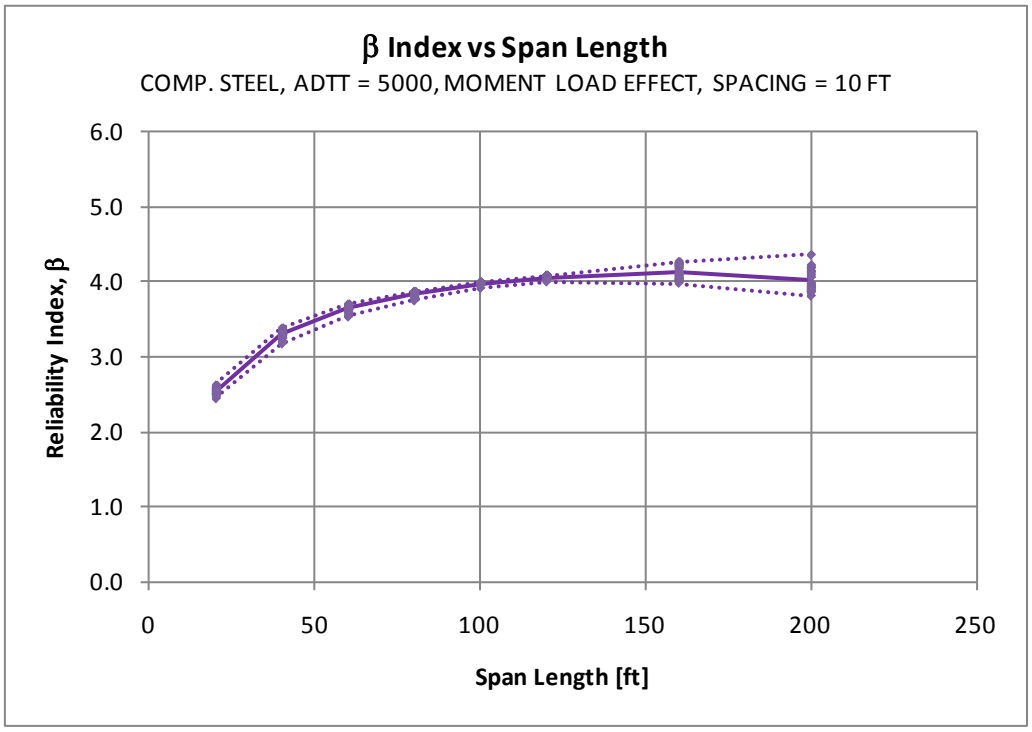
15% OVERLOADED PERMIT PLOTS

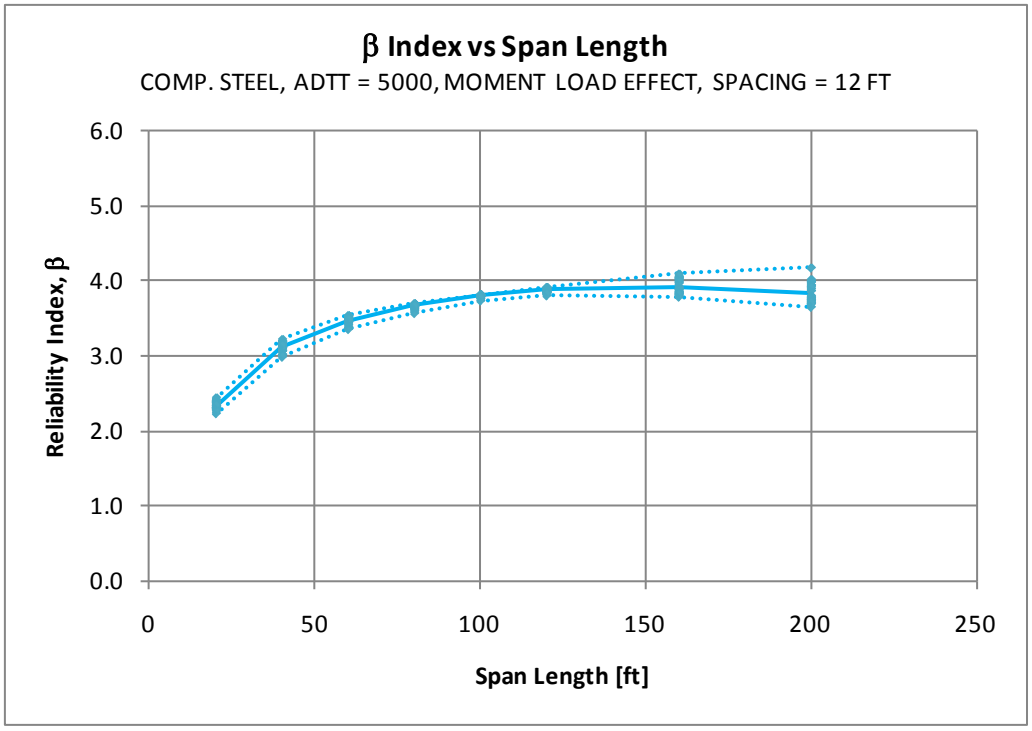
(ONE LANE)

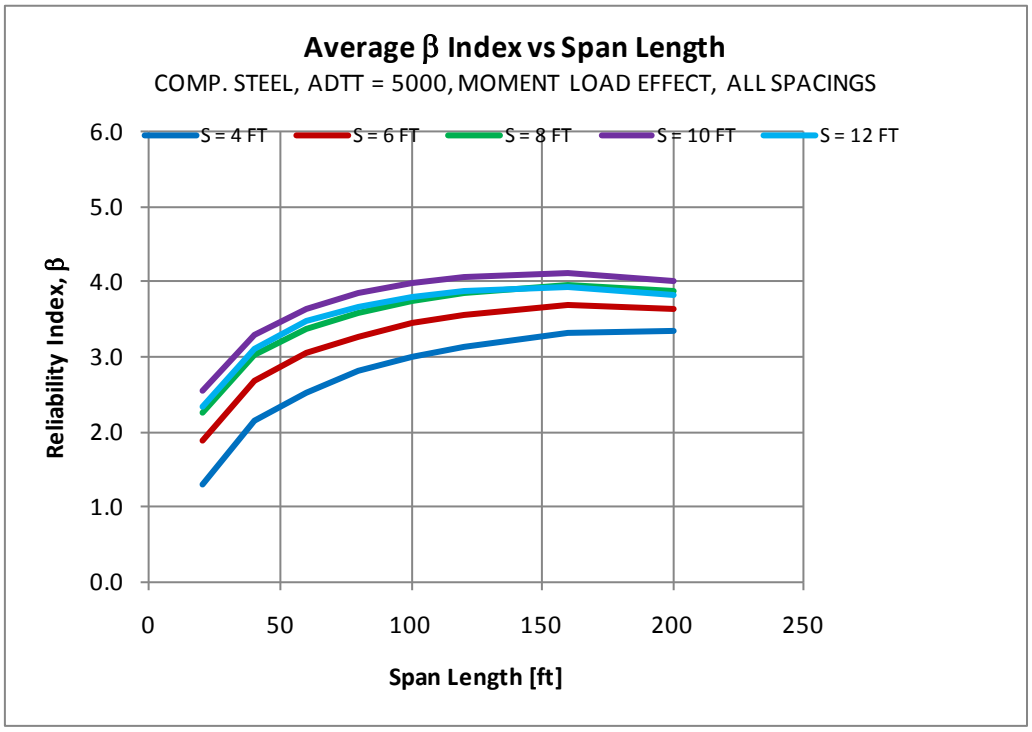

β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

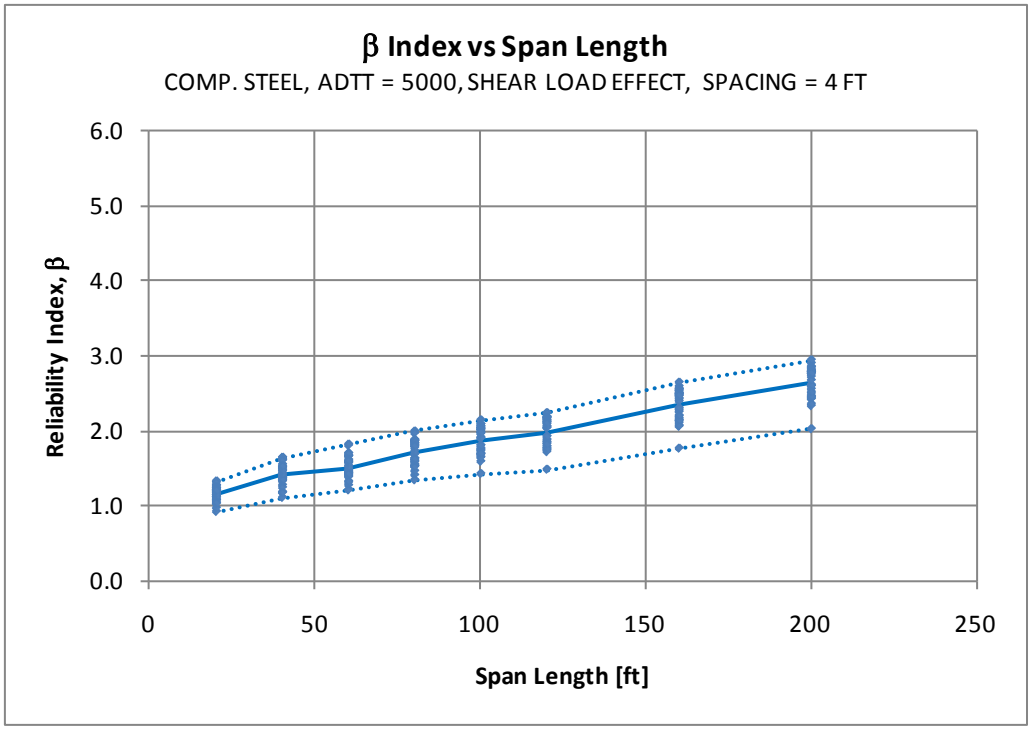

β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

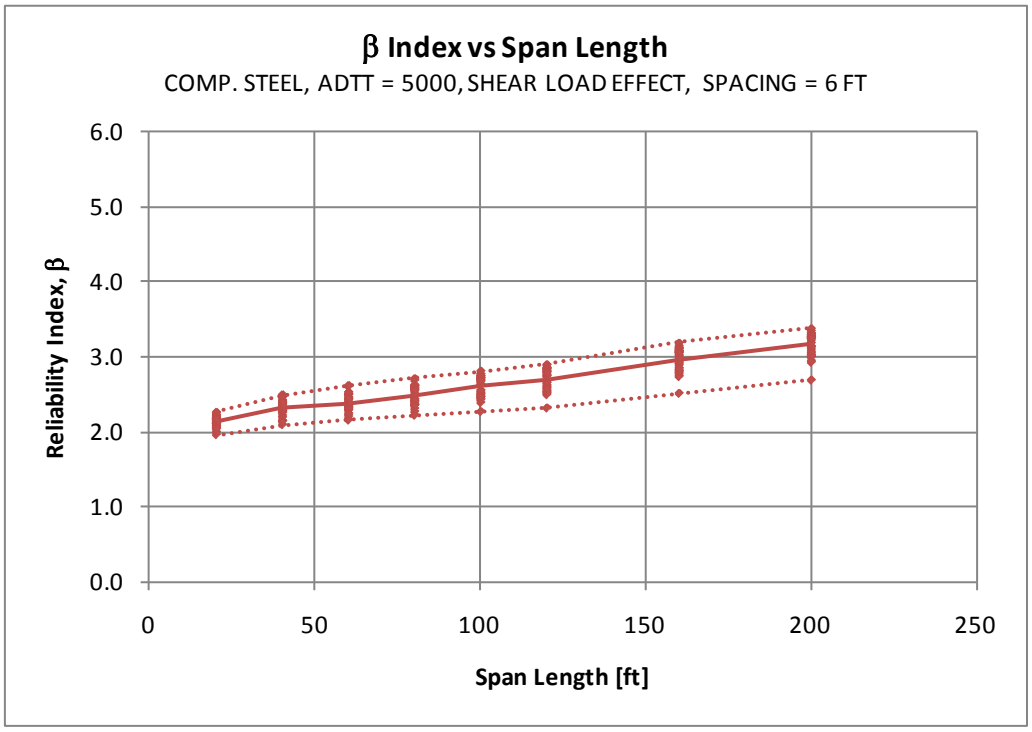

β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

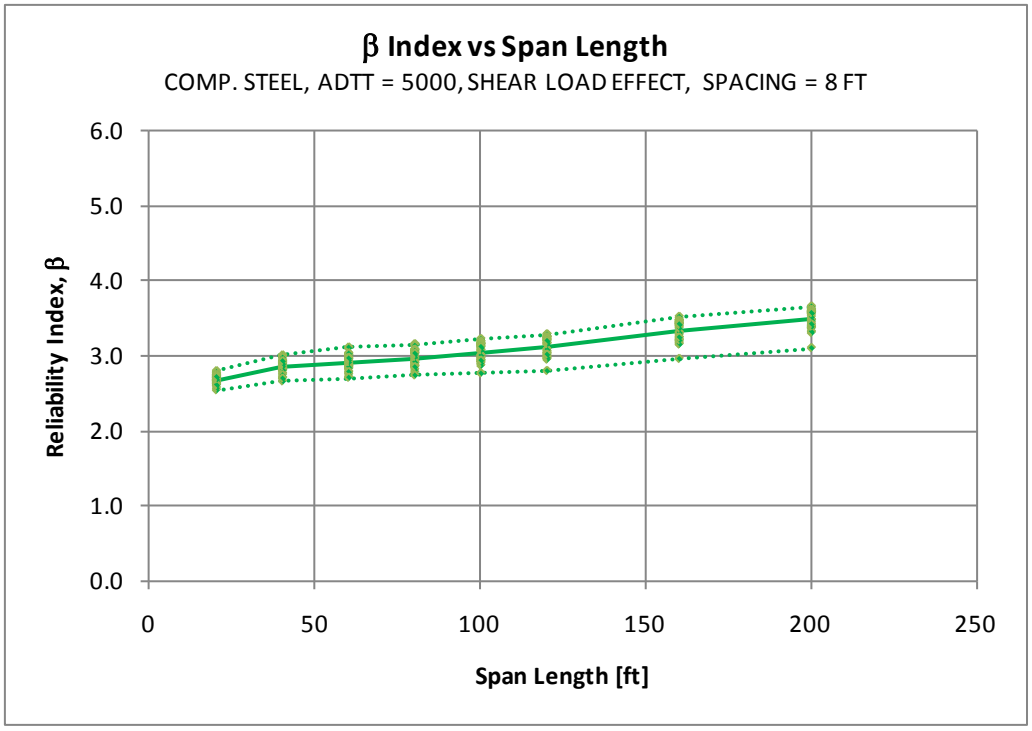

β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

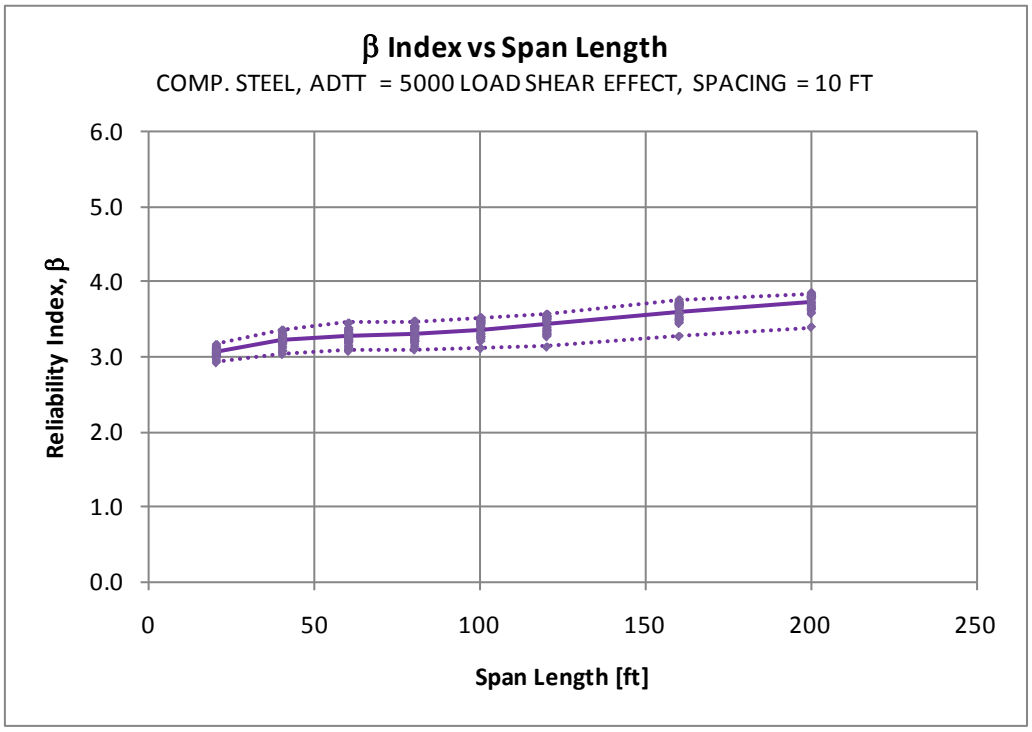

β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

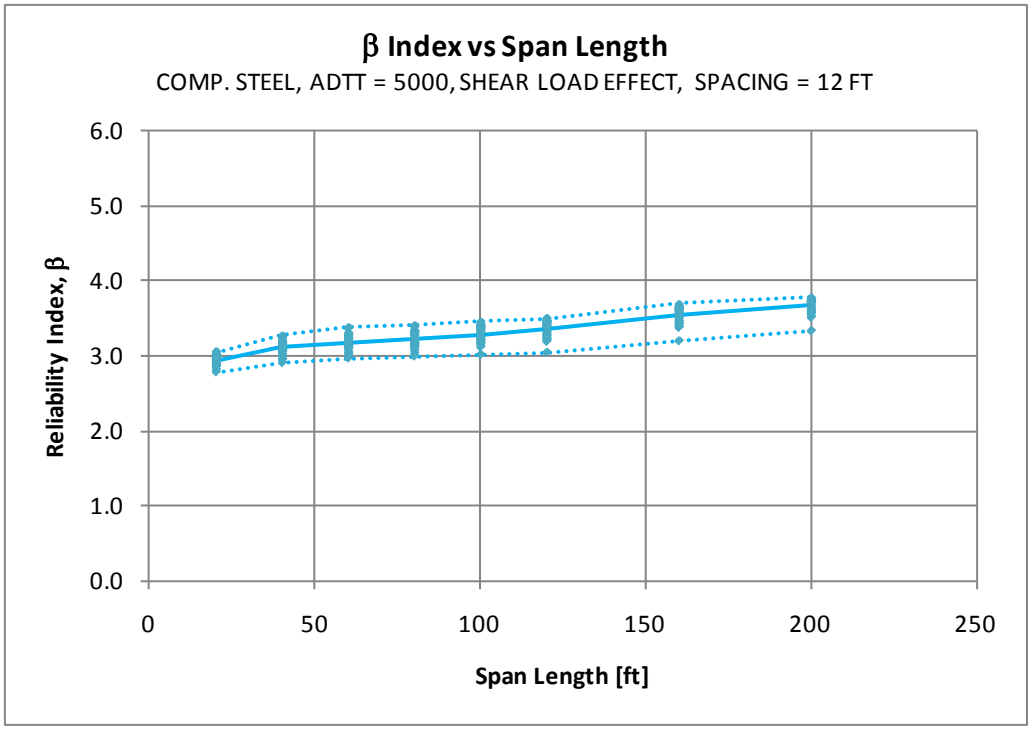

Average β Index vs Span Length
COMP. STEEL, ADTT = 5000, MOMENT LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

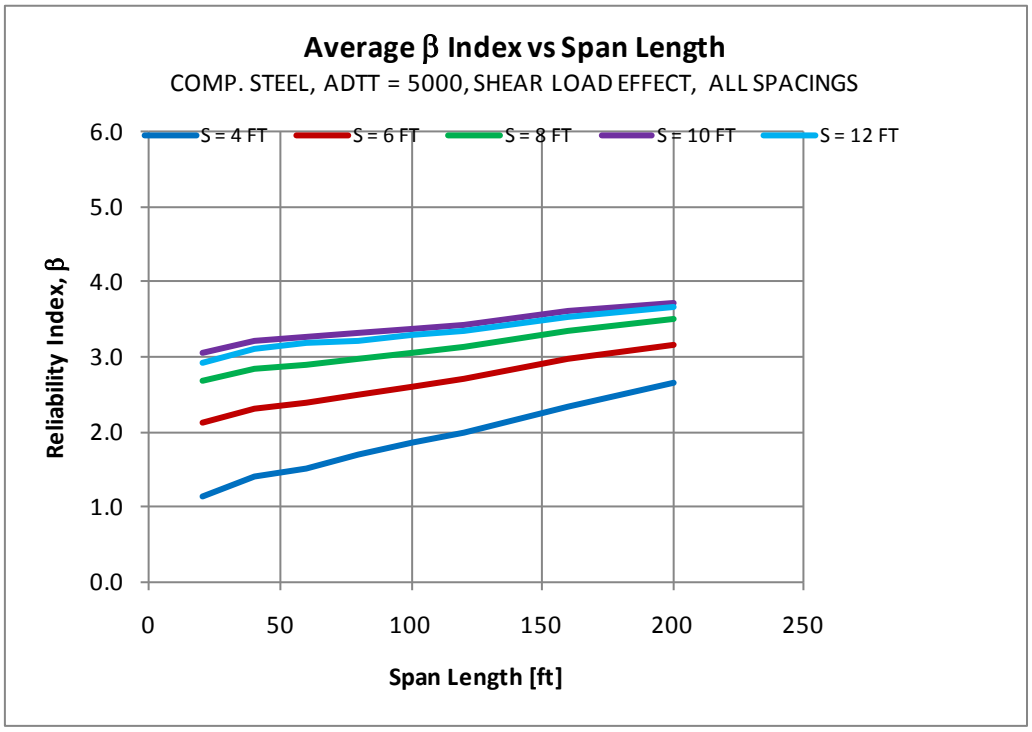

β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 4 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 6 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]


β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 8 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

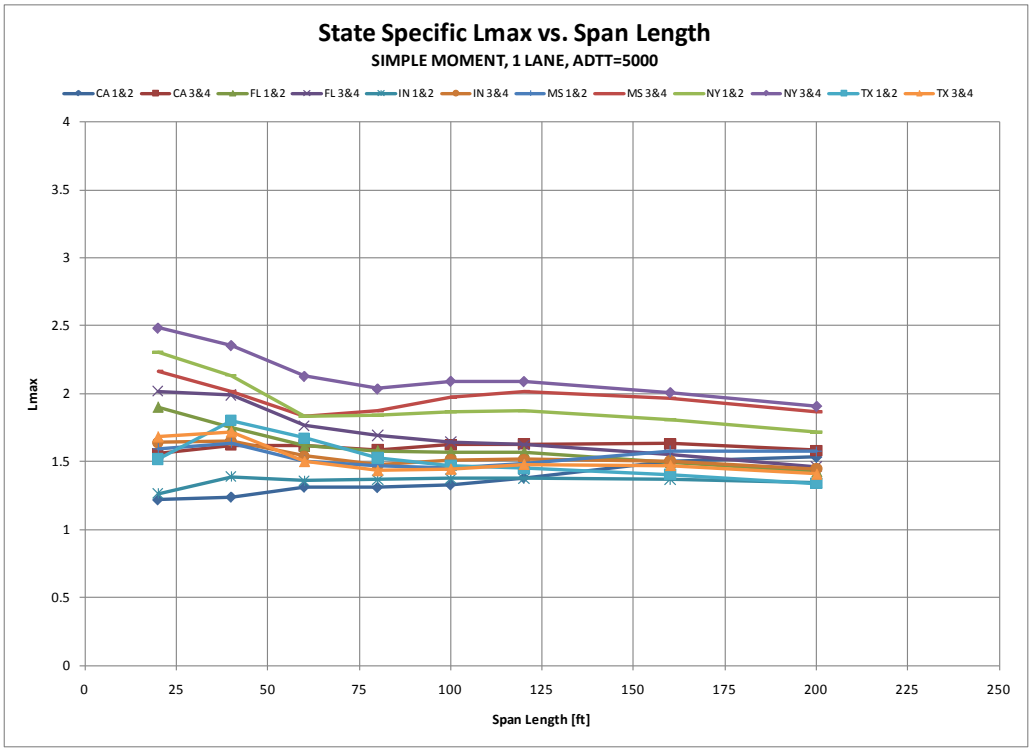

β Index vs Span Length
COMP. STEEL, ADTT = 5000 LOAD SHEAR EFFECT, SPACING = 10 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

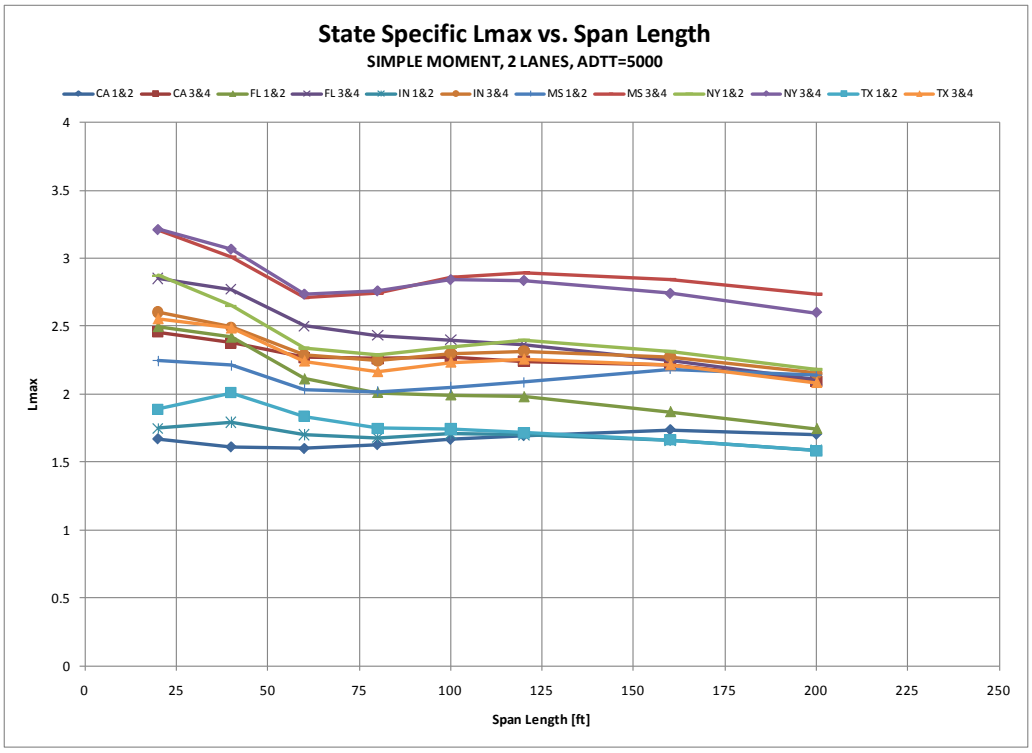

β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, SPACING = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

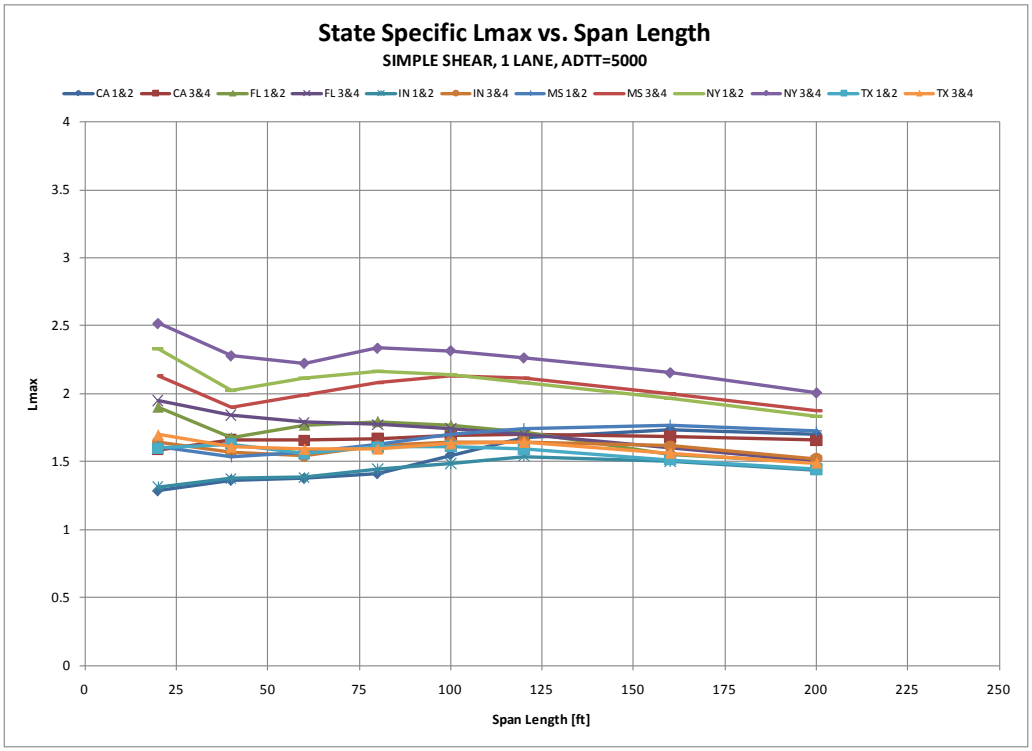

Average β Index vs Span Length
COMP. STEEL, ADTT = 5000, SHEAR LOAD EFFECT, ALL SPACINGS
S = 4 FT
S = 6 FT
S = 8 FT
S = 10 FT
S = 12 FT
Reliability Index, β
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
50
100
150
200
250
Span Length [ft]

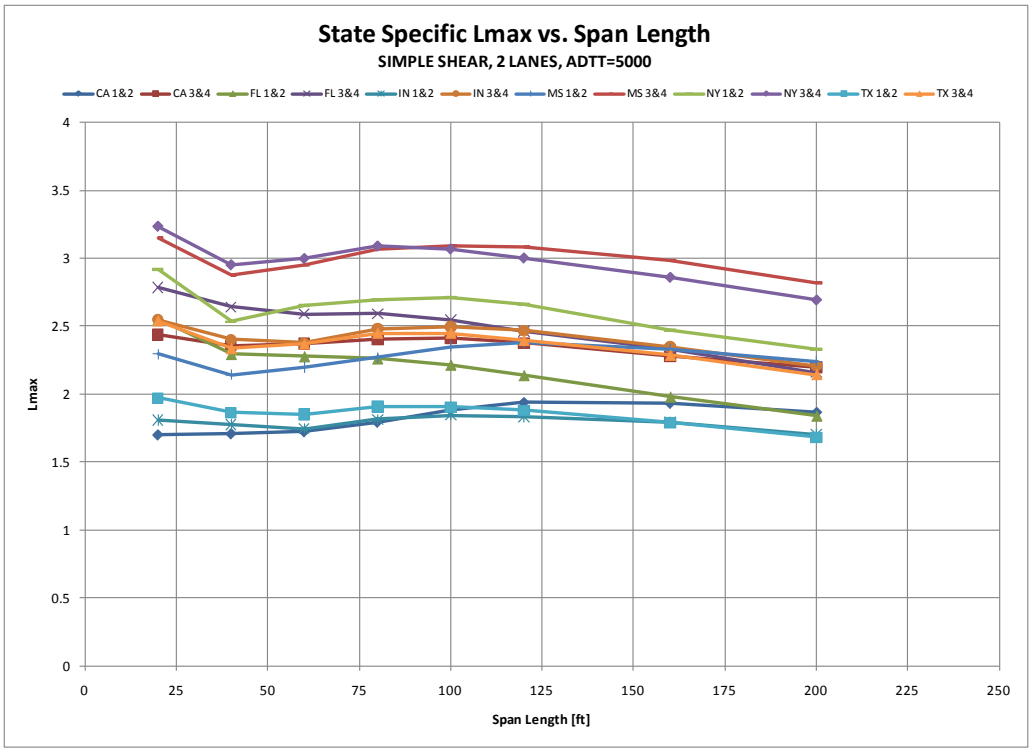
STATE SPECIFIC Lmax vs. SPAN LENGTH PLOTS

SIMPLE MOMENT & SHEAR (ALL STATES)


State Specific Lmax vs. Span Length
SIMPLE MOMENT, 1 LANE, ADTT=5000
CA 1&2
CA 3&4
FL 1&2
FL 3&4
IN 1&2
IN 3&4
MS 1&2
MS 3&4
NY 1&2
NY 3&4
TX 1&2
TX 3&4
Lmax
Span Length [ft]


State Specific Lmax vs. Span Length
SIMPLE MOMENT, 2 LANES, ADTT=5000
CA 1&2
CA 3&4
FL 1&2
FL 3&4
IN 1&2
IN 3&4
MS 1&2
MS 3&4
NY 1&2
NY 3&4
TX 1&2
TX 3&4
Lmax
Span Length [ft]


State Specific Lmax vs. Span Length
SIMPLE SHEAR, 1 LANE, ADTT=5000
CA 1&2
CA 3&4
FL 1&2
FL 3&4
IN 1&2
IN 3&4
MS 1&2
MS 3&4
NY 1&2
NY 3&4
TX 1&2
TX 3&4
Lmax
Span Length [ft]

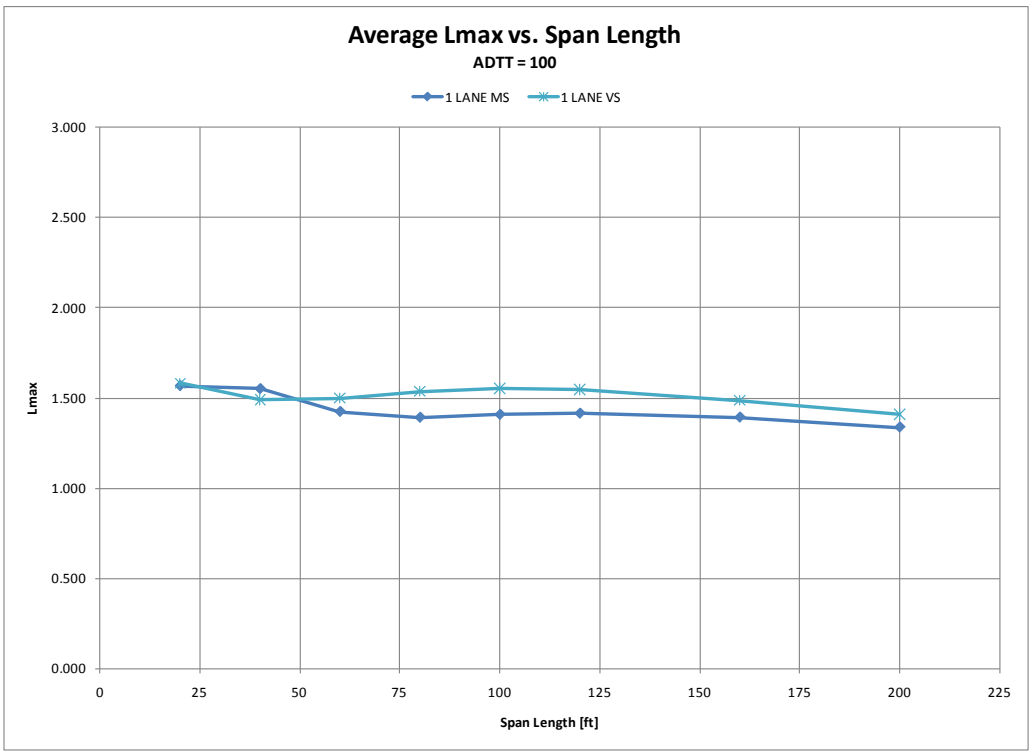

State Specific Lmax vs. Span Length
SIMPLE SHEAR, 2 LANES, ADTT=5000
CA 1&2
CA 3&4
FL 1&2
FL 3&4
IN 1&2
IN 3&4
MS 1&2
MS 3&4
NY 1&2
NY 3&4
TX 1&2
TX 3&4
Lmax
Span Length [ft]

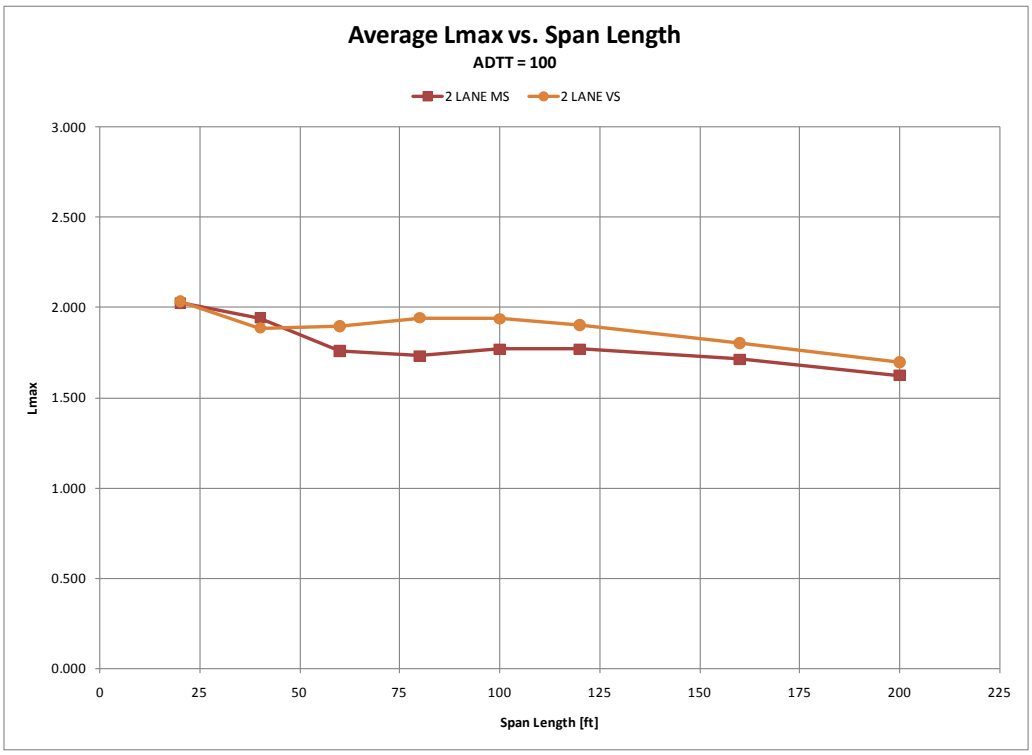
AVERAGE Lmax vs. SPAN LENGTH

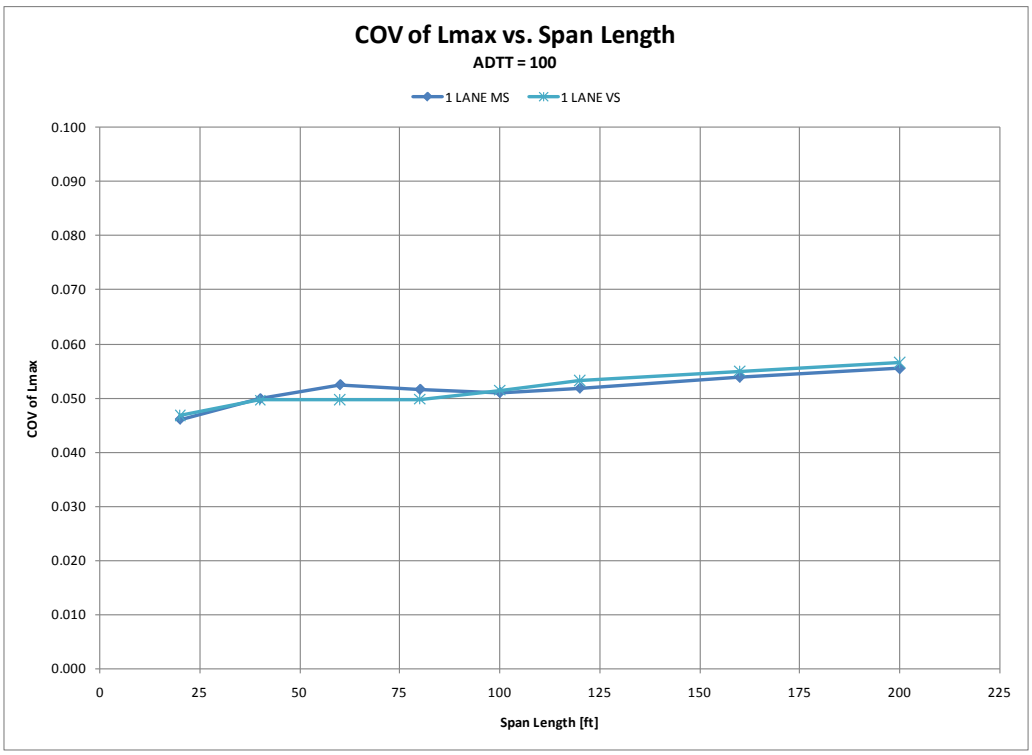
&

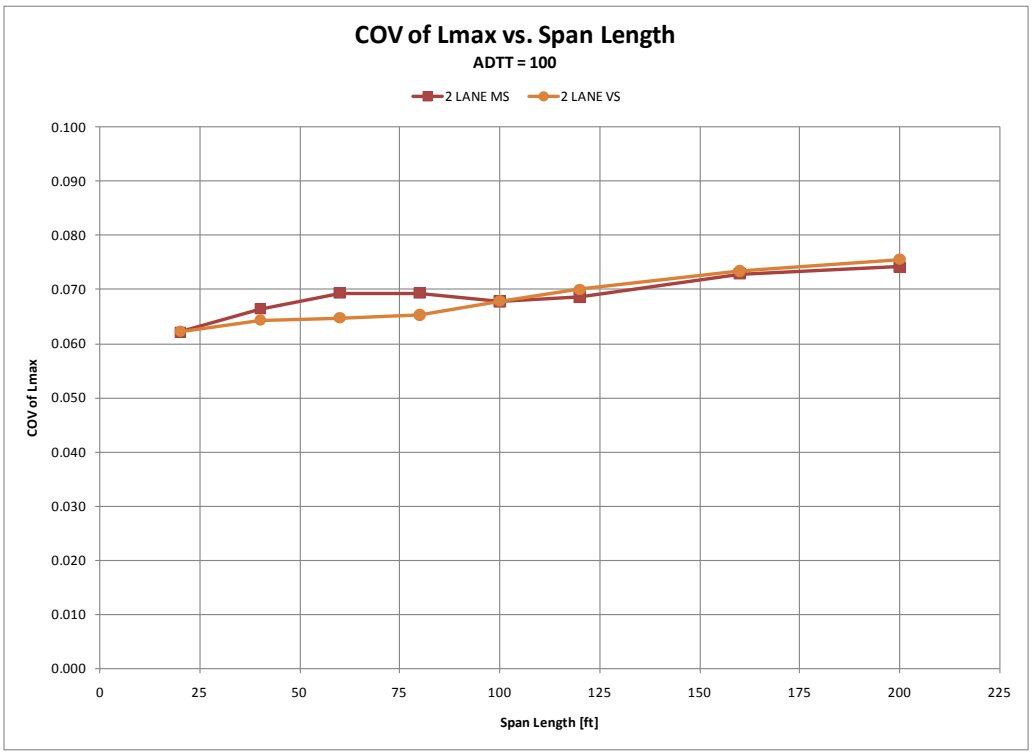
COV of Lmax vs. SPAN LENGTH PLOTS

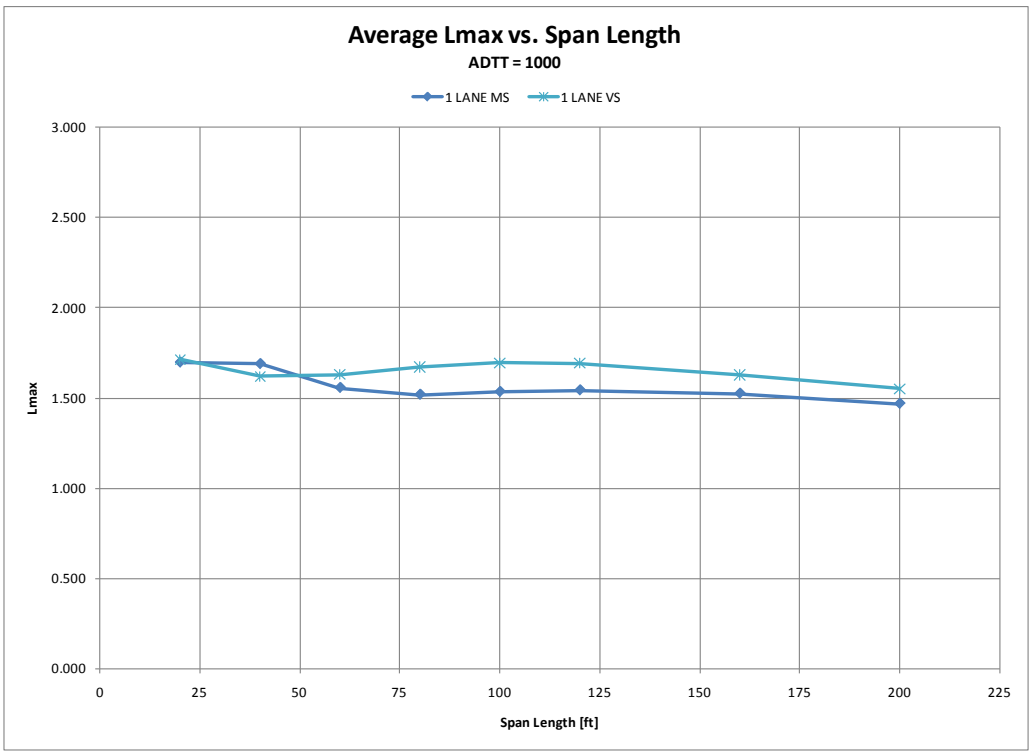
SIMPLE MOMENT & SHEAR

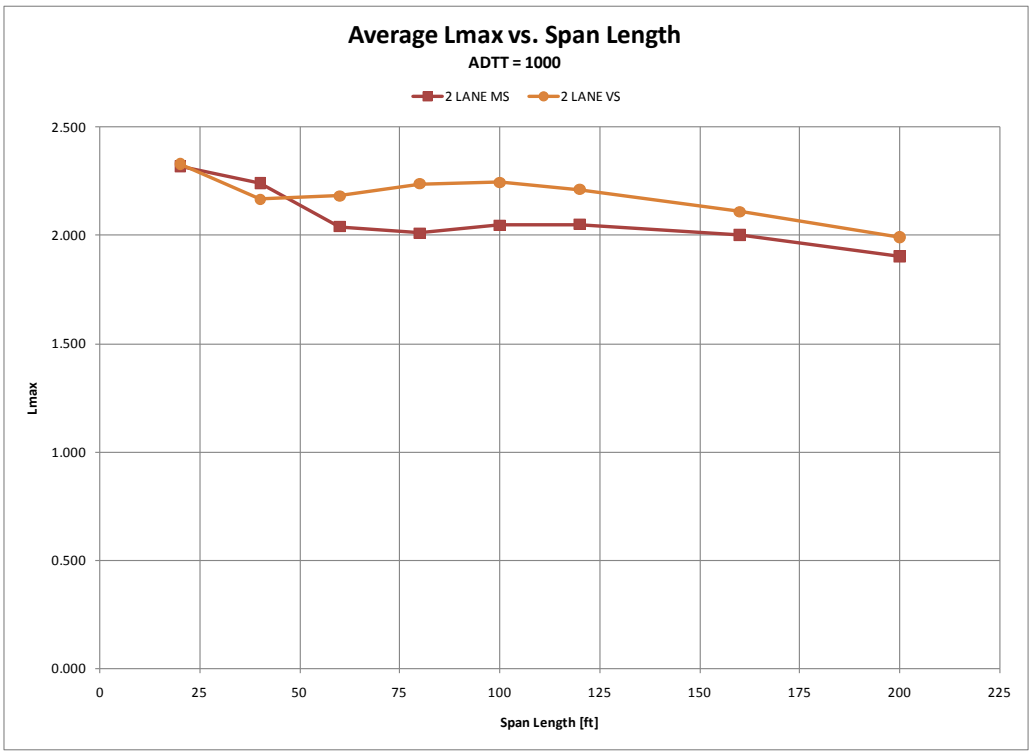

Average Lmax vs. Span Length
ADTT = 100
1 LANE MS
1 LANE VS
Lmax
3.000
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

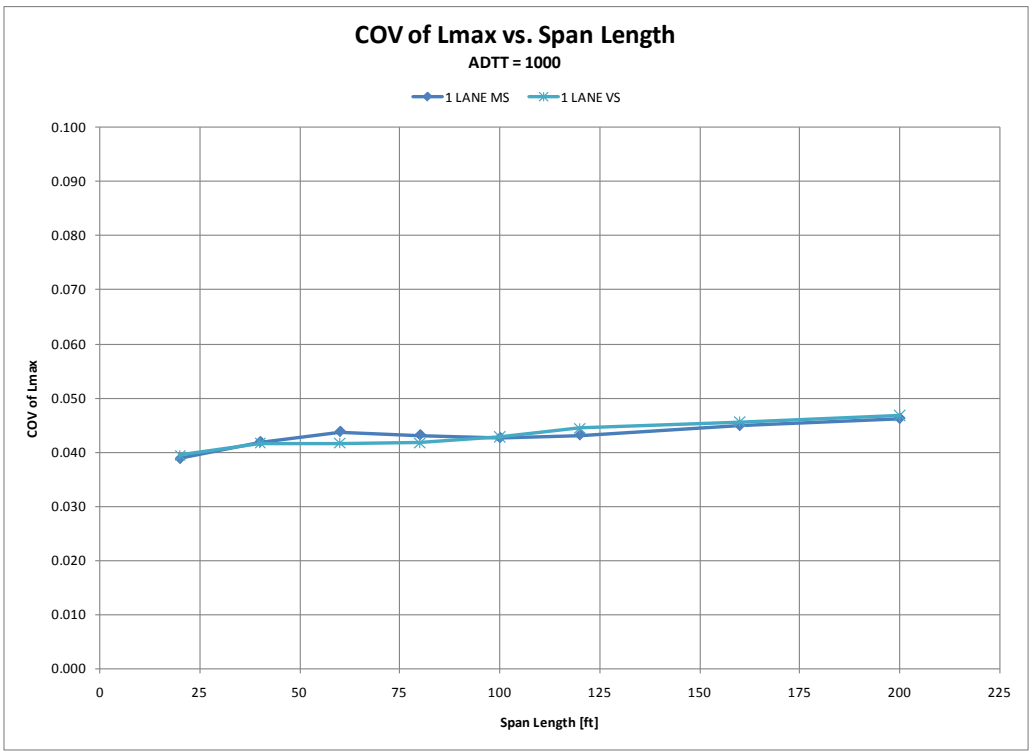

Average Lmax vs. Span Length
ADTT = 100
2 LANE MS
2 LANE VS
Lmax
3.000
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

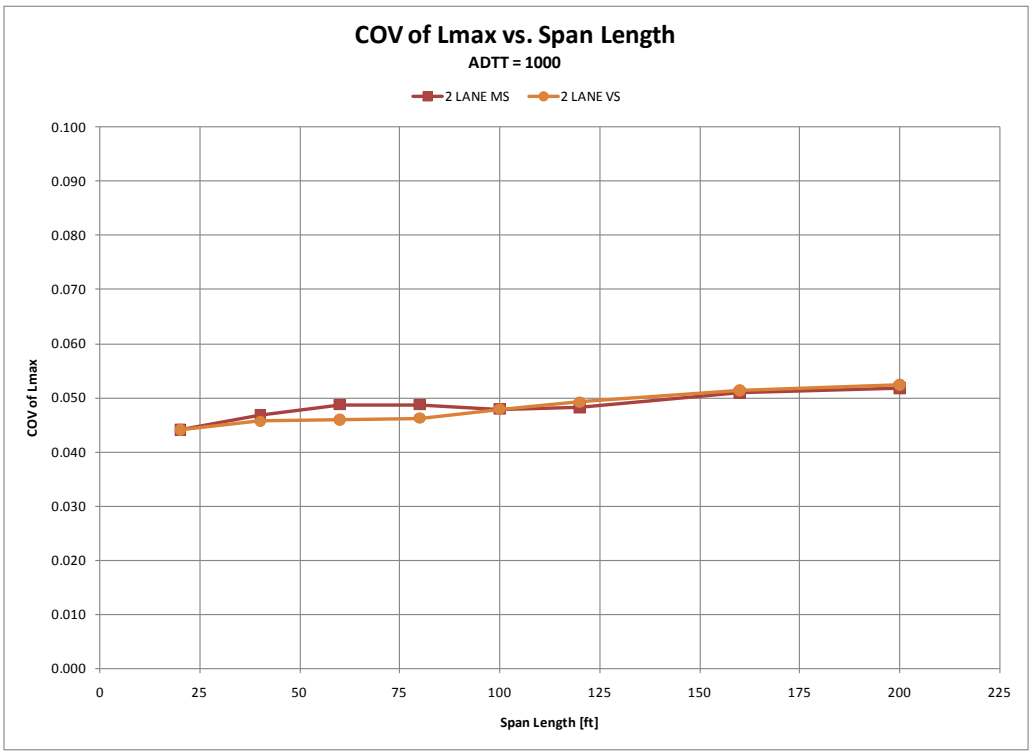

COV of Lmax vs. Span Length
ADTT = 100
1 LANE MS
1 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

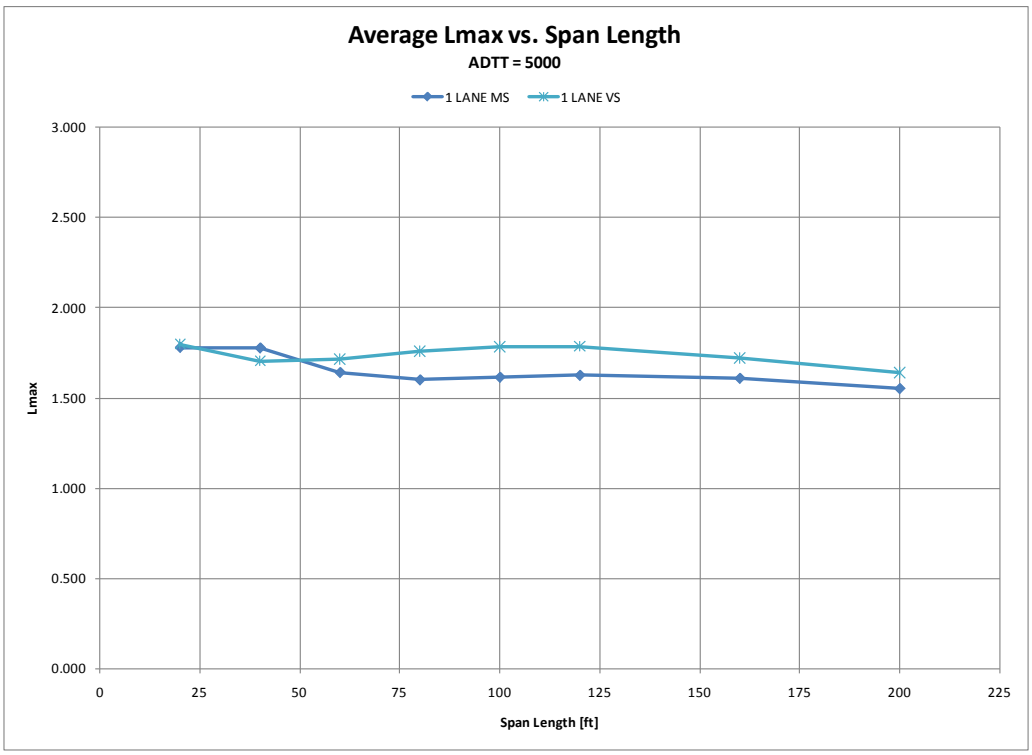

COV of Lmax vs. Span Length
ADTT = 100
2 LANE MS
2 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

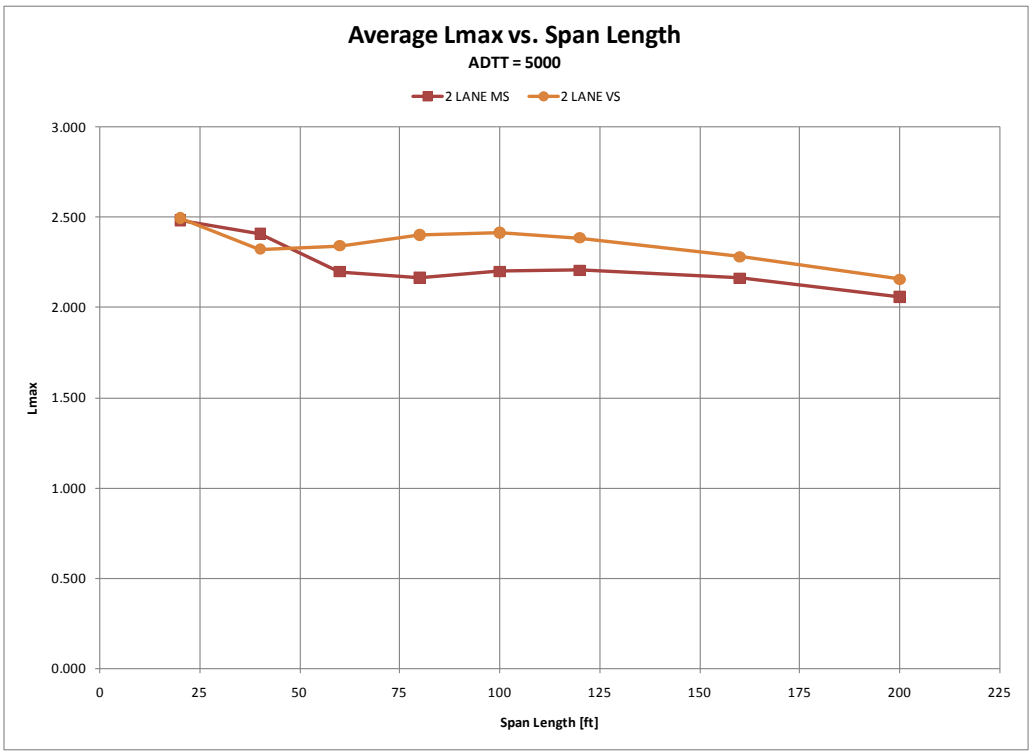

Average Lmax vs. Span Length
ADTT = 1000
1 LANE MS
1 LANE VS
Lmax
3.000
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

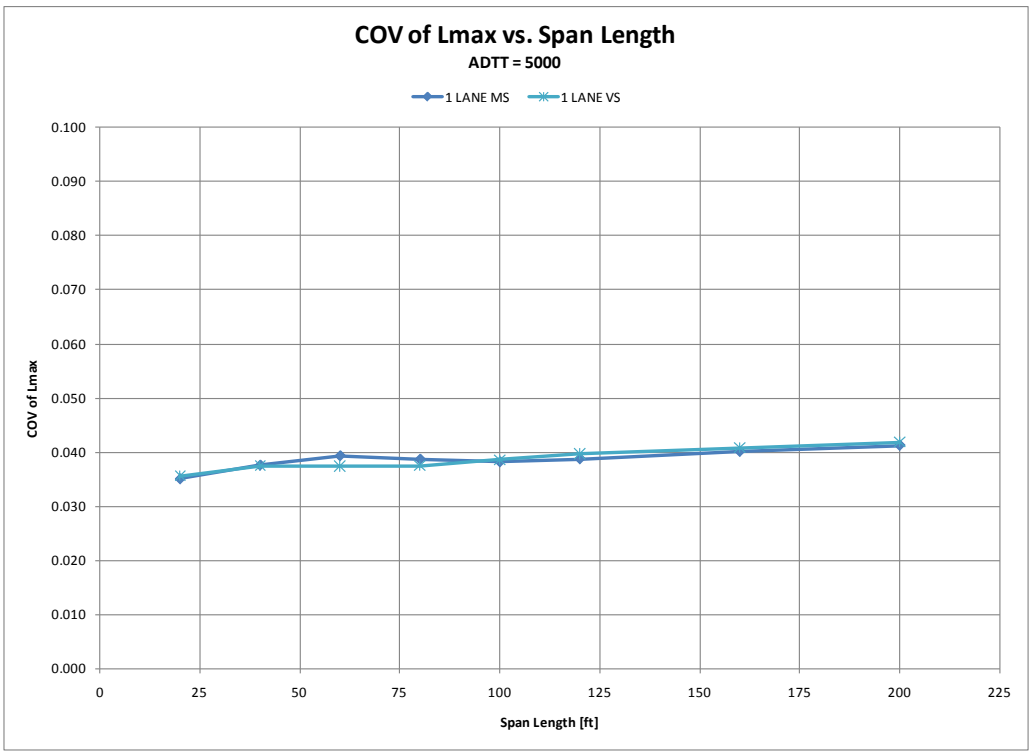

Average Lmax vs. Span Length
ADTT = 1000
2 LANE MS
2 LANE VS
Lmax
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]

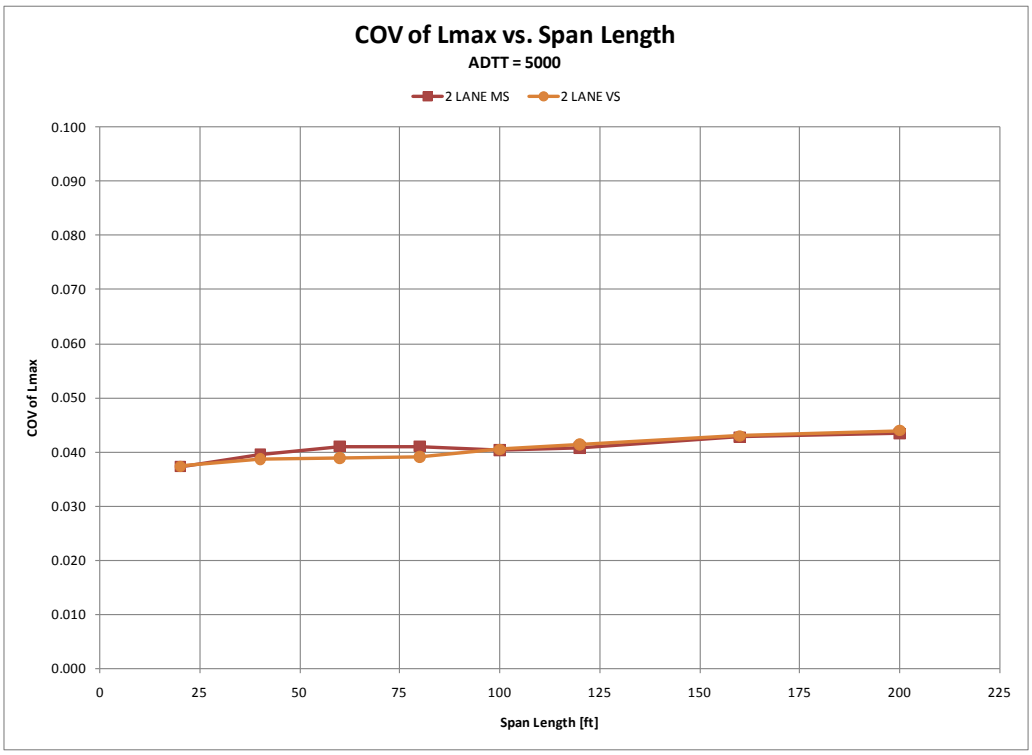

COV of Lmax vs. Span Length
ADTT = 1000
1 LANE MS
1 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]


COV of Lmax vs. Span Length
ADTT = 1000
2 LANE MS
2 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]


Average Lmax vs. Span Length
ADTT = 5000
1 LANE MS
1 LANE VS
Lmax
3.000
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]


Average Lmax vs. Span Length
ADTT = 5000
2 LANE MS
2 LANE VS
Lmax
3.000
2.500
2.000
1.500
1.000
0.500
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]


COV of Lmax vs. Span Length
ADTT = 5000
1 LANE MS
1 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]


COV of Lmax vs. Span Length
ADTT = 5000
2 LANE MS
2 LANE VS
COV of Lmax
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
25
50
75
100
125
150
175
200
225
Span Length [ft]