APPENDIX C

CALIBRATION OF LRFD DESIGN SPECIFICATIONS FOR STEEL CURVED GIRDER BRIDGES

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APPENDIX C

CALIBRATION OF LRFD DESIGN SPECIFICATIONS FOR STEEL CURVED GIRDER BRIDGES

C1 INTRODUCTION

The objective of this report is to document the calibration of the design code for steel curved girder bridges, consistent with the AASHTO LRFD Code. The calibration means here calculation of load and resistance factors such that the reliability of bridges designed using the code is at the target level. It has been assumed that load factors are the same as for straight girders and this project is focused on the resistance factors only. The reliability is measured in terms of the reliability index, β. There are various methods of calculation of β, as shown in textbooks, for example by Nowak and Collins (2000). For consistency with calibration performed for straight girders, the reliability index for curved girders is calculated using the same formula as given in NCHRP Report 368 (Nowak 1999).

The relationship between the resistance factor, ϕ, and reliability index, β, is a complex function that includes nominal (design) values of load and resistance, and statistical parameters of load and resistance such as bias factors, λ, and coefficients of variation, V. The bias factor is defined as the ratio of mean-to-nominal value, and coefficient of variation is the ratio of standard deviation-to-mean value. The statistical parameters were derived for straight girders (Nowak 1999). An important part of this calibration is to determine values of these parameters for curved girders.

The statistical load model developed for straight bridges (Nowak 1999) includes the maximum expected effects of dead load and live load and dynamic load. The maximum truck weights corresponding to various periods of time up to 75-years were determined by extrapolation of truck survey results. The multiple truck presence in lane and in adjacent lanes was considered based on field observations and by Monte Carlo simulations. The statistical parameters of truck weights, including extrapolations for longer periods of time, do not depend on bridge curvature.

However, the live load effect in a girder (moment and shear) depends on load distribution. In particular, girder distribution factor (GDF) represents the fraction of the lane moment (or shear) per girder. In calibration of the code for straight girders it was assumed that the bias factor for GDF is 1.0. This means, that on average, the code specified GDF is equal to the actual GDF. Because of geometry, the load distribution strongly depends on the degree of curvature. Therefore, an important task in this study is calculation of the bias factors and coefficients of variation for the load distribution method used in the design. It is assumed that the design analysis is performed using the commercial program developed by BSDI. To determine the statistical parameters for load distribution, the results of design analysis are compared with field measurements and results of an advanced finite element analysis.

The bridge resistance model depends on the statistical parameters of materials and geometry. Therefore, the latest available material test data is reviewed and used in derivation of the bias factors and coefficients of variation for moment and shear capacity of curved girders.

The statistical parameters of load, load distribution and resistance are derived for selected structures. The structural and reliability analysis is performed for three representative structures:

- Bridge A: Minnesota Bridge No. 27998
- Bridge B: Minnesota Bridge No. 62705
- Bridge C: Fore River Bridge, Portland Maine

This final report includes results and conclusion from analysis of these bridges. Analysis of Bridge A is the most comprehensive and the results are compared to experimental data. The details of computations for Bridge A are presented in the report. Analyses of Bridge B and Bridge C are performed to support conclusions for Bridge A by additional computations.

The basic parameters for the bridges considered in this study are as follows:

- **Bridge A:**
  - Location: Minnesota
  - Length: 295 ft
  - Number of spans: 2 – Continuous
  - Radius of curvature: 285 feet
  - Number of girders: 4 spaced at 9’
  - Roadway width: 30 feet (two lanes)
- **Bridge B:**
  - Location: Minnesota
  - Length: ~ 105 feet
  - Number of spans: 1 – simply supported
  - Radius of curvature: 106 feet
  - Number of girders: 4 spaced at 8’4”
  - Roadway width: 28 feet (two lanes)
- **Bridge C:**
  Location : Portland, Maine
  - Name : Fore River Bridge No. 2
  - Length : 273 feet
  - Number of spans : 3 – Continuous
  - Radius of Curvature : 175 feet
  - Number of Girders : 4 (spaced @ 8’)
  - Roadway width : 28 feet (2 lanes)

Basic assumptions in the calibration analysis:

1. Linear behavior of the structure as a system; the load distribution is not affected by non-linear properties of materials.

2. Load model is the same as for straight girders; the bias factors and coefficient of variation for truck loads, parameters of multiple presence (probability of occurrence of two trucks side-by-side and/or in the same lane).
3. Resistance model is based on the finite element analysis performed at the University of Michigan, and for Bridge A also field testing carried out by the University of Minnesota.
4. Design (nominal) values of load are obtained from the analysis performed by Modjeski and Masters (M&M) using the program developed by BSDI.
5. Load factors are assumed to be the same as for straight girders.
6. Resistance factors are rounded to the nearest 0.05.
7. The design of curved girders can be governed by the construction loads. Therefore, load and resistance factors are also considered for various construction stages.

The calibration work involved the development of load and resistance models, reliability analysis procedure, selection of the target reliability index, and calculation of load and resistance factors. The analytical boundary conditions in the finite element method (FEM) analysis were calibrated using the actual field test data for Bridge A. The objective of FEM analysis for Bridges B and C was to validate of the statistical model developed for Bridge A.

C2 CALIBRATION PROCEDURE

The calibration procedure was developed for the development of AASHTO LRFD Code, as described by Nowak (1995; and 1999). For the calibration of the code provisions for curved steel girders, the major steps include:

1. Selection of representative structures. Various State DOT's were asked to provide drawings and other data for recently constructed or planned structures. The parameters such as span, curvature, number of girders, spacing between the girders, were considered. From the population of curved girder steel bridges, three representative structures were selected to be used as a reference in this study. Bridge A tested by the University of Minnesota was also included in this set, and that provided an opportunity to compare analytical and experimental (test) results.
2. Identification of the load and resistance parameters, and formulation of the limit state functions. The load parameters include dead load, live load, dynamic load, and also load effects such as bending, torsion, and shear, and their combinations. It is important to determine the absolute value of load effects individually, and in various combinations. The behavior of a girder was based on the results of a study by White et al. (2001).
3. Development of load and resistance models. This step involved gathering of the available statistical data, calculation of missing and/or additional parameters by simulations. For Bridge A, the work on resistance models included the analysis of test results (University of Minnesota) and advanced FEM computations to develop a reference for comparisons. For Bridges B and C, the FEM analysis was performed by the University of Michigan, and the results were compared with the analysis carried out by Modjeski and Masters using BSDI program.
4. Selection of the reliability analysis procedure. The numerical procedure selected for this project is similar to that used for straight bridges. The reliability analysis is performed to

determine the reliability indices. As in the original calibration, the procedure was performed for 75-year economic life and it involved extrapolations.

5. Reliability analysis for the selected representative structures. The reliability indices were calculated for the limit state functions identified in Step 2. The reliability index spectrum was reviewed to identify the trends and discrepancies.
6. Selection of the target reliability index. The selected target reliability index is consistent with the LRFD Code provisions for straight bridges, as calculated in the Calibration Report (Nowak, 1999).
7. Calculation of resistance factors. It is assumed that load factors remain the same as in the LRFD Code. However, the load factors for some of the combinations that are specific for curved girders may require some special load combination factors. The resistance factors are determined by trial-and-error approach. Various possible resistance factors were tried (each rounded to 0.05); for each set of factors the reliability indices were calculated, and the optimum resistance factors correspond to the closest fit to the target reliability index.
8. Final selection of the resistance factors. This step involves the verification of the calculated factors by additional reliability analysis, check of special cases (e.g. combinations with dominating dead load), and selection of load and resistance factors consistent with the rest of the LRFD Code. Simplicity of the Code is an important consideration.

C3 LOAD MODELS

C3.1 LOAD COMPONENTS

The major load components of highway bridges are dead load, live load (static and dynamic), environmental loads (temperature, wind, earthquake) and other loads (collision, emergency braking). Load components are random variables. Their variation is described by the cumulative distribution function (CDF), and/or parameters such as the mean value, bias factor (mean-to-nominal ratio) and coefficient of variation. The relationship among various load parameters is described in terms of the coefficients of correlation.

The basic load combination for highway bridges is a simultaneous occurrence of dead load, live load and dynamic load. Therefore, these three load components are considered in the present study. It is assumed that the economic life time for newly designed bridges is 75 years. The extreme values of load are extrapolated from the available data base. Nominal (design) values of load components are calculated according to AASHTO Standard (1996) and AASHTO LRFD Code (1998).

C3.2 DEAD LOAD

Dead load is the gravity load due to self weight of the structural and non-structural elements permanently connected to the Bridge. Because of different degrees of variation, it is convenient to consider three components of dead load: weight of factory made elements (steel, precast concrete members), weight of cast-in-place concrete members, and weight of the wearing surface (asphalt). All components of dead load are treated as normal random variables. The statistical parameters were derived in conjunction with the development of the Ontario Highway Bridge Design Code (OHBDC 1979, 1983 and 1991) and AASHTO LRFD Code (1994 and

1998), and they are listed in Table C-1. The bias factors are taken as used in the previous bridge code calibration work; however, the coefficients of variation are increased to include human error as recommended by Ellingwood, Galambos, MacGregor and Cornell (1980).

In the case of steel curved girder bridges, the critical load combination can occur during the construction, prior to composite action with the concrete deck slab. The parameters of dead load during construction are also taken as given in Table C-1.

The calculation of dead load effects (e.g. moments and shear forces) for curved girders involves a considerable degree of variation. In this study, the statistical parameters were determined based on the field measurements performed by the University of Minnesota (UMinn test), finite element analysis performed by the University of Michigan (UMich FEM), and calculations performed by the University of Minnesota (UMinn) using grillage model for Bridge A. The ratio of the stresses obtained by UMich FEM and UMinn is plotted on the normal probability paper in Figure C-1. The average value is 0.95, and the coefficient of variation is 0.12.

Therefore, in this study, the bias factor for dead load effect are based on field measurements UMinn test with the bias factor, λ equal to 1.00, and the coefficient of variation, V equal to 0.15. The normal cumulative distribution function (CDF) corresponding to these parameters is also shown in Figure C-1. The dead load is a time-invariant load component; therefore, the parameters are the same for different time periods.

C3.3 LIVE LOAD

Live load covers a range of forces produced by vehicles moving on the bridge. The effect of live load depends on many parameters including the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). The effect of these parameters is considered separately. The live load model was developed in conjunction with AASHTO LRFD Code (1994 and 1998), as presented by Nowak and Hong (1991) and Nowak (1993 and 1999).

For curved girder bridges, the spans are mostly 60 to 150 ft (18 to 45m). For this span range, the bias factor is 1.25 to 1.35, with the larger value corresponding to shorter spans. The mean value is the 75-year maximum mid-span moment, and nominal value is the AASHTO LRFD specified HL-93 load effect per one lane.

For two lanes, the maximum 75-year live load is the effect of two trucks side-by-side, each being the maximum two-month truck (Nowak 1993 and 1999). The ratio of the mean maximum two-month moment and the mean maximum 75-year moment is 0.85. Therefore, the bias factor for two lanes is a product of 1.25 to 1.35 and 0.85, resulting in 1.05 to 1.15.

The design is based on consideration of load per girder. The girder distribution factors (GDF) were determined for straight girders and they are included in the AASHTO LRFD Code (1994 and 1998). In this study, the bias factor for GDF for curved girders was determined based

on UMinn test and UMich FEM analysis. The nominal value of GDF is the result of analysis carried out using the BSDI program. The cumulative distribution function (CDF) of the ratio of stresses UMich FEM/UMinn test, is shown in Figure C-2, on the normal probability paper.

The statistical parameters of the stress ratio (UMich FEM/UMinn test) can be determined from Figure C-2, as shown in the textbooks (e.g. Nowak and Collins 2000). The mean stress ratio corresponds to value of zero on the vertical scale, and it is 0.75. The standard deviation is determined from the slope of the CDF, and it is 0.09. Therefore, the coefficient of variation of the stress ratio is 0.09/0.75 equal to 0.12.

The overall bias factor for live load moment in a curved girder is the product of the bias factor for two lane live load (1.05 to 1.15) and stress ratio, 0.75, resulting in 0.80 to 0.85. The coefficient of variation of live load (including dynamic load) in a curved girder is

$$V = (0.18^2 + 0.12^2)^{0.5} = 0.215$$

where 0.18 is the coefficient of variation of live load in a straight girder (Nowak 1999).

C3.4 DYNAMIC LOAD

The dynamic load model was developed by Hwang and Nowak (1991), and it was verified by field measurements by Nassif and Nowak (1995) and Kim and Nowak (1997). Dynamic load is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). It was observed that dynamic strain and deflection are almost constant and they do not depend on truck weight. Therefore, the dynamic load, as a fraction of live load, decreases for heavier trucks.

For the maximum 75-year values, the corresponding dynamic load factor (DLF) does not exceed 0.15 of live load for a single truck and 0.10 of live load for two trucks side-by-side. Therefore, in this study the mean value of DLF is taken as 0.10. The coefficient of variation of dynamic load is about 0.80. The results of the simulations indicate that DLF values are almost equally dependent on road surface roughness, bridge dynamics and vehicle dynamics. The actual contribution of these three parameters varies from site to site and it is very difficult to predict. Therefore, it is recommended to specify DLF as a constant percentage of live load.

C3.5 LOAD RATIOS

In the reliability analysis, the absolute values of load components are not important. However, the relative values of load components affect the statistical parameters of the total load effect. Therefore, load components are expressed in terms of relative values (load ratios). The ratios of load components are determined for the selected bridges. These load ratios for Bridge A are listed in Table C-2. For example, D1 equal to 4 and D2 equal to 9.5 means that the ratio of D1/D2 equal to 4/9.5. The load ratios are different during construction, and they are also shown in Table C-2. Similarly, load ratios are calculated for Bridges B and C. These bridges are considered as representative for the current trends in curved girder bridge design.

C4 RESISTANCE MODELS

For straight girders, the resistance (load carrying capacity), R, is considered as a product of three factors, M, F and P,

$$R = M\,F\,P \tag{1}$$

where M represents the material properties (strength), F represents the dimensions (area, section modulus, moment of inertia), and P represents the professional factor (analysis). In curved girder bridges, there is an additional factor, S, representing system behavior. The statistical parameters of S are based on field tests and finite element method analysis.

C4.1 MATERIALS

The basic materials considered in this study include structural steel, concrete and reinforcing steel. Curved girders are made of plates; therefore, the parameters are different than for hot-rolled sections. The statistical parameters used in this calibration are λ equal to 1.06 and V equal to 0.06 (see Attachment A).

C4.2 FABRICATION

The statistical parameters for dimensions are λ equal to 1.00 and V equal to 0.05 (see Attachment A).

C4.3 PROFESSIONAL FACTOR

The statistical parameters of the professional factor are λ equal to 1.10 and V equal to 0.05 (see Attachment A).

C4.4 RESISTANCE

The nominal (design) load carrying capacity (resistance) required by the code (AASHTO LRFD 1998), R_n, is

$$R_n = [1.25\ \mathrm{D} + 1.75\ (\mathrm{LL} + \mathrm{IL})]/\phi \tag{2}$$

where ϕ equals 1.0. For the considered design cases, values of D, LL and IL are presented in Table C-2.

The statistical parameters of resistance are calculated as follows. The mean of R is

$$m_R = \lambda_R\, R_n \tag{3}$$

where

$$\lambda_R = \lambda_M\, \lambda_F\, \lambda_P = (1.06)(1.00)(1.10) = 1.165 \tag{4}$$

and the coefficient of variation of R is

$$V_R = [(V_M)^2 + (V_F)^2 + (V_P)^2]^{0.5} = [(0.06)^2 + (0.05)^2 + (0.06)^2]^{0.5} = 0.095 \quad (5)$$

C5 ANALYSIS AND TESTS OF BRIDGE A

C5.1 INTRODUCTION

Bridge A is a two-span continuous curved structure, with a composite slab on steel girders (cast-in-place reinforced concrete deck). The basic information about Bridge A is:

- Location: Minnesota
- Length: 295 ft
- Number of spans: 2 – Continuous
- Radius of curvature: 285 feet
- Number of girders: 4 spaced at 9'
- Roadway width: 30 feet (two lanes)

The distribution of load (load per component) is determined by analytical methods. It is assumed that the design analysis is performed using the computer program developed by the BSDI. For the bridges considered in this study, the calculations were carried out by Modjeski and Masters, using the BSDI program.

For Bridge A, the analysis was also performed by UMinn using a grillage model analogy (Huang 1996; Galambos et al. 1996; Hajjar et al. 1997). The calculated design values of stresses (provided by Modjeski and Masters) were compared to the UMinn test results and UMinn analytical results. In addition, analysis was also performed by the team at the University of Michigan using the advanced finite element method (ABAQUS), UMich FEM.

Therefore, for the Bridge A, the results are compared from four sources:

(a) Field tests by the University of Minnesota (UMinn test)
(b) University of Minnesota analysis (UMinn)
(c) Modjeski and Masters analysis using BSDI program (M&M)
(d) University of Michigan analysis using ABAQUS software (UMich FEM)

For Bridge A, the resulting stresses from the four sources listed above, are shown in Tables C-6 through C-19. The stress ratios (bias factors) and statistical parameters of these bias factors are also presented in these tables.

The ranking of the analytical models according to the degree of sophistication is as follows: UMich FEM, UMinn, and M&M. The ABAQUS solver allows for a relatively close fit to the field test stresses which is the actual structural behavior.

It is assumed that the nominal (design) values are represented by M&M, and the actual behavior is represented by UMich FEM. This is a conservative assumption because, in general, it is observed that analytical results are more conservative than field test results. Therefore, the bias factors for load distribution (GDF) for the reliability analysis are determined as ratio of

stresses calculated using ABAQUS and design stress (BSDI). Use of a computer program different than that developed by the BSDI, can result in different bias factors.

C5.2 ANALYSIS AND TESTS OF A TWO-SPAN CONTINUOUS STRUCTURE

The analysis was performed for the bridge structure tested and analyzed by the University of Minnesota. The bridge ID according to the Minnesota DOT is 27998. The objective of this analysis was to determine stress due to dead load and live load. The calculations by the University of Michigan are performed using ABAQUS Standard, the finite element method based software. For comparison, the obtained values are compared with the results of computations by the University of Minnesota (grillage model), and field tests also by the University of Minnesota. Additional analysis was carried out by Modjeski and Masters team, using BSDI program.

C5.2.1 Finite Element Method (FEM) Model

The superstructure was modeled and analyzed using ABAQUS, a program based on finite element method (FEM). In the 3-dimensional FEM model of the bridge the steel girders were modeled by shell elements, cross-frames by beam elements and slab by continuum 3-dimensional elements. The superstructure is composed of 22,000 finite elements.

Three types of boundary conditions were considered in this study:

(a) simply supported,
(b) partial constraint of displacements in longitudinal direction of each girder,
(c) full constraint of displacements in longitudinal direction of each girder.

The boundary conditions were calibrated using field test results. The spring elements were introduced to model the existing boundary conditions. Stiffness of the spring supports was established during calibration.

Structural steel was modeled as an elastic-plastic material, concrete as an elastic-plastic material with tension stiffening. The reinforcing steel was also modeled as an elastic-plastic material.

In the FEM analysis it was assumed that each truck wheel load was applied to the nodes of the upper layer of continuum elements of the slab. The stress calculations were performed at the same points where the strain gages were located in the tests performed by the University of Minnesota (UMinn test). Three strain gage lines were considered: at midspan of the girders (strain gage line A), close to the end supports of the girders (strain gage line B) and midspan of the cross-frames (strain gage line C), as shown in Figure C-3. The cross section of the bridge with notation used for girders and cross frames is presented in Figure C-4. The location of points within the cross section on the girders and for the cross frame are also shown in Figure C-4.

C5.2.2 Other Analytical and Field Test Models

The results of the FEM analysis carried out by the team at the University of Michigan (denoted by UMich) were compared with

(a) analysis by the University of Minnesota using the grillage method (Galambos et al. 1996), denoted by UMinn;
(b) analysis Modjeski and Masters using BSDI program, denoted by M&M;
(c) field tests by the University of Minnesota (Galambos et al. 1996), denoted by Test (truck used in test is presented in Figure C-5).

C5.2.3 Structural Analysis and Detailed Results

Three load cases were considered:

(a) Non-composite steel structure loaded by dead load (steel and concrete) and construction loads (forms),
(b) Composite structure loaded with two test trucks side-by-side
(c) Composite structure loaded with two test trucks one in each span.

Cases (b) and (c) correspond to load cases No. 3 and 7 in the report by Galambos et al. (1996), as it is presented in Figures C-6 through C-8.

In general, it was observed that the measured stresses (UMinn test) are smaller than the analytical results. In most cases, stresses from UMich FEM analysis are between the measured stresses and those obtained in the grillage method analysis (UMinn). Results are presented in Tables C-3 through C-5 and Figures C-9 through C-20.

C5.3 STATISTICAL ANALYSIS OF STRESS RATIOS (BIAS RATIOS)

C5.3.1 Statistical Analysis

Two cases are analyzed (a) non-composite steel structure loaded by dead load and (b) composite structure loaded with two test trucks side-by-side on one span and with two test trucks one in each span.

The statistical analysis was performed separately for both cases. The objective of this analysis was to determine bias factors for different types of analysis due to dead load and live load.

(a) Non-composite steel structure loaded by dead load

The statistical analysis was performed combined for the top and bottom flanges of the steel girders. The results for strain gage line A are shown in Table C-3 and for strain gage line B in Table C-4. At the bottom of each table, there are also shown values of the mean, standard deviation and coefficient of variation. The data from strain gage lines A and B in Tables C-3 and C-4 is combined together and the results from combined statistical analysis are presented in Table C-6.

Some of the field test values were not consistent with other results. There can be various reasons including malfunctioning testing equipment and/or reading/recording error. A more detailed analysis of these seemingly inconsistent results was not possible. Therefore, these data points were considered as outliers and they were eliminated from the data base. Table C-6 includes results for the data without the outliers. The statistical parameters (mean, standard deviation and coefficient of variation) shown in Table C-6 were used in the reliability analysis.

(b) Composite structure loaded with two test trucks side-by-side (Case 3) in one span and with two test trucks one in each span (Case 7).

The analysis was performed separately for load Case 3 and 7. The results are shown in Table C-7 and 8 for strain gage lines A and B, respectively, and in Table C-7 for load Case 3 and in Table C-8 for load Case 7. The combined statistical results for Cases 3 and 7 are shown in Table C-9.

As in the case of the dead load, the outliers were identified and eliminated from the data base. The combined results presented in Table C-9 for load case 3 and 7 are without outliers. The statistical parameters listed in Table C-9 were used in the reliability analysis.

C5.3.2 Summary of Analytical and Test Results

The ratios of stresses obtained from the four considered sources (as described in Section 5.1) are calculated, and the results are presented. These ratios are referred to as the bias factors. Two cases are considered, a non-composite steel structure under dead load only (during construction), and a composite steel and concrete structure under dead load and live load.

Case (a)

Non-composite steel structure loaded by dead load (steel and concrete) and construction loads (formwork). The design values are not available for this case (BSDI program was used to calculate live load effects only). Therefore, the bias factors are determined by comparison of results from source UMich FEM and UMinn, rather than M&M BSDI. The statistical analysis was performed separately for the each gage lines. The objective of this analysis was to determine bias factors for different types of analysis due to dead load. The bias factors for the stress ratio of UMich FEM and UMinn stresses are presented in Table C-10, combined for both strain gage lines. They are also shown separately for each strain gage line in Figures C-20 and C-21, and combined together in Figure C-22. The gage line A is at the midspan, and gage line B is at the support. The spread is larger for midspan stress.

Case (b)

Composite structure loaded with two test trucks side-by-side on one span and with two test trucks one in each span. In this case, the stress ratios are calculated for results from UMich FEM and M&M BSDI analysis. The statistical analysis was performed for each gage line. The resulting stress ratios have a smaller degree of variation compared with dead load only case. The

bias factors for the ratio of UMich FEM and M&M BSDI stresses are presented in Table C-11 and shown in Figure C-23.

C5.4 RELIABILITY ANALYSIS FOR BRIDGE A

The procedure is as in the previous calibration (Nowak, 1999). The reliability is measured in terms of the reliability index. Load is treated as a normal random variable, and resistance as a lognormal variable. The relative values of load components are used in the reliability analysis with the values from Table C-2. Statistical parameters of load and resistance are taken as determined in Section 3 and 4. The mean values of load and resistance are calculated by multiplying the design values and corresponding bias factors. The reliability index is calculated using the following formula,

$$\beta = \{m_R (A) [1 - \ln (A)] - m_Q\} / [(m_R V_R A)^2 + \sigma_Q^2]^{0.5} \quad (6)$$

where A is equal to $(1 - k V_R)$, and k is about 2.

For the considered bridge, in normal operation (as opposed to construction), the nominal resistance is calculated using Equation 2 and nominal (design) load values from Table C-2,

$$R_n = [1.25 (4 + 9.5) + 1.75 (4.5 + 0.75)] / 1.00 = 26.06$$

and the mean value of resistance is calculated using Equation 3

$$m_R = (1.165)(26.06) = 30.37$$

the mean total load effect is calculated

$$m_Q = (1.00)(4 + 9.5) + (0.85) (4.5) (1.1) = 17.71$$

and

$$A = 1 - k V_R = 1 - (2)(0.095) = 0.81$$

The standard deviation of dead load is calculated using parameters from Section 3.2 (bias factor equal to 1.00 and coefficient of variation equal to 0.15)

$$\sigma_D = \{[(1.00)(4)(0.15)]^2 + [(1.00)(9.5)(0.15)]^2\}^{0.5} = 1.545$$

and for live load from Section 3.3 (bias factor equal to 0.85 and coefficient of variation equal to 0.215), with dynamic load equal to 0.10 of static live load (factor equal to 1.1)

$$\sigma_L = (0.85) (4.5) (1.1)(0.215) = 0.905$$

So the standard deviation for the total load effect is

$$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 1.79$$

And the reliability index is

$\beta = 4.08$

For the bridge during construction, the reliability index is calculated using a similar procedure and formulas,

$R_n = [1.25\ (4 + 7.5) + 1.75\ (0.25)]\ /\ 1.00 = 14.81$

or $R_n = [1.5\ (4 + 7.5)]\ /\ 1.00 = 17.25$ (governs)

$m_R = (1.165)(17.25) = 20.09$

$m_Q = (1.00)(4 + 7.5) + (0.25)\ (1.1) = 11.7$

$A = 1 - (2)(0.095) = 0.81$

$\sigma_D = \{[(1.00)(4)(0.15)]^2 + [(1.00)(7.5)(0.15)]^2\}^{0.5} = 1.275$

$\sigma_L = (0.25)\ (1.1)(0.215) = 0.06$

$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 1.275$

$\beta = 3.93$

The target reliability index is assumed the same as for straight bridges (β_T equal to 3.5).

C5.5 CONCLUSIONS FOR BRIDGE A

The objective of the task was to perform full analysis for a representative, horizontally curved steel girder Bridge A and analyze results in order to develop some specific and general conclusions and recommendations for the design of curved I-girder structures. Conclusions and recommendations for Bridge A are summarized, after supporting them with analyses of Bridge B and Bridge C, at the end of the Report.

C6 ANALYSIS OF BRIDGE B

C6.1 INTRODUCTION

Bridge B is a one span, simply supported structure, with curved composite slab on steel girders (cast-in-place reinforced concrete deck). The basic information about the bridge considered in this study is:

- Location : Minnesota
- Length : ~ 105 feet
- Number of spans : 1 – simply supported
- Radius of Curvature : 106 feet
- Number of Girders : 4 spaced at 8’4”
- Roadway width : 28 feet (two lanes)

For Bridge B, the calculations were carried out by Modjeski and Masters (M&M), using the BSDI program. The results from this source include values of normal in steel girders determined in selected points of interest. These calculated values of stresses were compared to the analytical results obtained by the team at the University of Michigan, using the advanced finite element method analysis in ABAQUS software (UMich FEM). Similarly as for Bridge A,

the design values of load effect are represented by M&M BSDI analysis and the actual behavior is represented by UMich FEM.

The statistical parameters of the bias factor – ratio between stresses obtained from the FEM analysis and the BSDI analysis - due to dead loads and live load with dynamic load allowance were obtained for Bridge B. They were obtained separately for constructional and operational loading stages. For the constructional loading stage (non-composite dead load of steel structure, i.e. girders plus diaphragms and stiffeners) the mean of bias factor is 0.79, with the coefficient of variation equal to 10.5%. Respectively, for operational loading stage (due to live load with dynamic load allowance) the bias factor is equal to 0.90, with coefficient of variation 22.5%.

The calculated reliability indices for Bridge B are close to the target reliability index β_T equal to 3.5, as for Bridge A. In summary, the analysis for Bridge B confirms the results presented for Bridge A.

C6.2 ANALYSIS OF A 1-SPAN SIMPLY SUPPORTED BRIDGE

The static analysis of the bridge is performed by using two methods: computer program developed by BSDI (calculations were carried out by Modjeski and Masters, M&M), and finite element method computations using ABAQUS by the University of Michigan.

The output from this BSDI analysis includes values of normal and warping stresses in steel girders obtained in selected points of interest. The calculated design values of stress were compared to the analytical results obtained by the team at the University of Michigan (UMich).

Thus, for the considered bridge, the results from two sources are compared:

(a) Modjeski and Masters analytical analysis using BSDI software (M&M),
(b) University of Michigan analytical analysis using ABAQUS software (UMich).

The results include the tables with values of normal stresses in steel girders obtained from both (a) and (b) analyses; comparison of obtained stresses (stress ratios expressed as bias factors); statistical analysis of these ratios; and statistical parameters of the bias factors.

It is assumed that the nominal (design) values of loads are represented by results from source (a), and the actual behavior is represented by results from source (b). This is a conservative assumption because, in general, it is observed that analytical results, obtained using the FEM program, are more conservative than field test results. Therefore, the bias factors for loads distribution (girder distribution factors) for the reliability analysis are determined as ratios of stress calculated using ABAQUS and design stress calculated using BSDI program.

C6.3 FINITE ELEMENT METHOD (FEM) ANALYSIS

The finite element method (UMich FEM) analysis was performed by the University of Michigan.. The analysis and modeling is limited to the superstructure part of the bridge. Substructure is not considered in this task.

C6.3.1 University of Michigan finite element model (FEM)

The superstructure was modeled and analyzed using ABAQUS. The bridge was modeled as a 3-dimensional FEM model. The steel girders were modeled by shell elements, the cross-frames by beam elements and the slab by continuum solid 3-dimensional elements. The entire superstructure was approximately composed of 100,000 finite elements. Figure C-24 shows plan view of the considered bridge.

In the FEM analysis, it was assumed that live load due to truck wheel load was applied to the node of the upper layer of continuum element of the slab. The uniformly distributed lane load was assumed as a uniformly distributed pressure applied to the upper layer of continuum elements of the slab.

The stress calculations were performed in one selected section for each girder, as shown in Figure C-24. The cross section of the bridge is presented in Figure C-25. Locations of analyzed points within the cross section of the girder are also shown in Figure C-25.

C6.3.2 Finite Element Analysis (FEM) of Bridge B

The results of the UMich FEM analysis were compared to M&M analysis performed using the BSDI program.

Two load cases were considered:

- Stg1 - DL: Non-composite steel girders loaded by dead load (construction stage, loading: weight of steel girders and structural attachments such as: cross-frames, stiffeners, etc…),
- Stg2 - LL+IM: Composite steel girders with a concrete deck structure loaded with live load (operational stage, loading: load combination of live load with dynamic load allowance; two test trucks side-by-side for maximum positive moments in span; plus uniformly distributed lane load in both cases), dead load is not included.

In general, the stresses obtained from FEM analysis are smaller than those obtained from the BSDI analysis; it confirms the results obtained in previous analysis for another bridge. From the UMich FEM results, it is also seen that the distribution of loads for girders are different than in the BSDI analysis. It refers especially to load cases Stg1-DL and these different distributions are possibly due to differences in the lateral stiffness of the non-composite steel structure (provided by the steel cross-frames composed of stiffeners and diaphragms) in the FEM and the BSDI analysis. In the load case Stg2-LL+IM these differences decrease, because most of the lateral stiffness is provided by the very stiff concrete deck, which is similarly modeled in both the M&M BSDI and the UMich FEM analyses.

Summary of obtained results from structural analysis is presented in Tables C-12 and C-13. In these tables results from both sources are expressed in terms of normal stresses in steel girders.

The following system of notation has been used in Tables C-12 and C-13:

- A: cross section, see Figure C-25
- I through IV: girder's number, see Figure C-24
- 1 through 6: points where stresses were calculated, see Figure C-25

C6.4 STATISTICAL ANALYSIS OF STRESS RATIOS (BIAS FACTOR)

C6.4.1 Statistical Analysis

The objective of this analysis was to determine bias factor for different types of analysis due to dead loads and live load. The statistical analysis concerns on the comparison between the results of the static analyses from two analytical methods M&M BSDI and UMich FEM. Comparison is expressed in terms of stress ratios UMich FEM/M&M. These bias factors are usually below 1.0; and as it was observed analytical results, obtained using BSDI program, are more conservative than FEM analysis.

The ratios between stresses (bias factors) obtained from the two considered sources are calculated, and the results are presented in Tables C-14 through C-17. They are also presented in Figure C-28 and Figure C-29. This statistical analysis was performed separately for constructional stages (Stg1-DL) and operational loading stages (Stg2-LL+IM). Results from bottom and top flanges of steel girders are combined together in statistical analysis. Figures C-26 and C-27 contain graphs with relationship between the bias factor (UMich FEM /M&M) and the absolute values of stresses obtained from the UMich FEM.

The statistical parameters of the bias factors are calculated after omitting some of the values of stresses from both types of analysis. However, the omitted values of the stresses are not representative because these are points of relatively small stresses. In these points, even small differences in location of the neutral axis, cause large differences in ratio of stresses obtained from the UMich FEM and from the M&M BSDI analysis.

C6.4.2 Summary of Results for Bridge B

The statistical parameters of the bias factor – ratio between stresses obtained from the FEM analysis and the BSDI analysis - due to dead loads and live load with dynamic load allowance are shown in the tables below. They are presented separately for constructional and operational loading stages.

C6.5 RELIABILITY ANALYSIS FOR BRIDGE B

The procedure is as applied in the previous calibration (Nowak, 1999), the same as presented and used for Bridge A. In practice, we are interested only in a normal operation of the bridge and the bridge during construction; therefore, only for those two cases the reliability analysis is performed.

The ratios of load components are determined for the selected bridges. The load ratios are different during construction, and they are also shown in Table C-20.

The reliability index for the considered bridge, in normal operation (as opposed to construction) loading is calculated as follows (for the details of the procedure see Section 5.4):

$R_n = [1.25\ (2 + 4.5) + 1.75\ (5.0 + 0.5)] / 1.00 = 17.75$
$m_R = (1.165)(17.75) = 20.67$
$m_Q = (1.00)(2 + 4.5) + (0.85)\ (5.0)\ (1.1) = 11.17$
$A = 1 - (2)(0.095) = 0.81$
$\sigma_D = \{[(1.00)(2)(0.15)]^2 + [(1.00)(4.5)(0.15)]^2\}^{0.5} = 0.738$
$\sigma_L = (0.85)\ (5.0)\ (1.1)(0.215) = 1.005$
$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 1.247$
$\beta = 4.51$

The reliability index for the considered bridge during construction (as opposed to normal operation) loading is calculated as follows:

$R_n = [1.25\ (2 + 3.7) + 1.75\ (0.3)] / 1.00 = 7.65$
or $R_n = [1.5\ (2 + 3.7) / 1.00 = 8.55$ (governs)
$m_R = (1.165)(8.55) = 9.96$
$m_Q = (1.00)(2 + 3.7) + (0.3)\ (1.1) = 6.03$
$A = 1 - (2)(0.095) = 0.81$
$\sigma_D = \{[(1.00)(2)(0.15)]^2 + [(1.00)(3.7)(0.15)]^2\}^{0.5} = 0.630$
$\sigma_L = (0.3)\ (1.1)(0.215) = 0.071$
$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 0.634$
$\beta = 3.70$

The target reliability index is assumed the same as straight bridges (β_T equal to 3.5).

C6.6 CONCLUSIONS FOR BRIDGE B

The objective of this task was to compare two different types of static analysis of a curved bridge under dead load and live load. The BSDI program uses a grillage analogy method and UMich FEM analysis is based on an advanced finite element method using ABAQUS software. The comparison of two methods is expressed in terms of the bias factor, which is the ratio of normal stresses obtained from the UMich FEM analysis and the M&M BSDI analysis. Two load cases are considered:

- Stg1 - DL : Non-composite steel girders under dead load (steel only),
- Stg2 - LL+IM: Composite steel girders with concrete deck structure, under live load only (dynamic load allowance included), dead load not included.

The web line of the section was considered in the evaluation of the normal stresses. The results from the comparison are presented in form of the bias factors. Values of the bias factor, if both analyses would be equally accurate and give the same values of stress, should be equal to 1.0. From the analysis, the mean values of the bias factor for the non-composite structure, load case Stg1-DL, is 0.79, the coefficient of variation is about 10.5%, and the standard deviation is about 0.094. The bias factor for the composite structure, load case Stg2 LL+IM, is 0.90, the coefficient of variation is 22.5%, and the standard deviation is 0.254.

In general, the absolute values of stress obtained from the UMich FEM analysis are smaller than those obtained from the M&M BSDI analysis. It can be explained as follows. In the FEM analysis, 3-dimensional structural behavior is considered without any geometrical simplifications and without mechanical simplifications (material properties). The BSDI software simplifies the model of the structure (grillage analogy method). It is possible that the load effect and the structural response are increased by some factors that take into account uncertainties arising from the method of analysis.

Overall, the analysis for Bridge B supports the results presented for Bridge A, which is presented in the main part of the report, meaning that the bias factors obtained in analyses have similar values, such that the mean is around 0.90 and with the larger coefficient of variation is around 10 to 23%.

C7 ANALYSIS OF BRIDGE C

C7.1 INTRODUCTION

Bridge C is a three span continuous structure, with curved composite slab on steel girders bridge (cast-in-place reinforced concrete deck). The basic information about the bridge considered in this study is:

- Location : Portland, Maine
- Name : Fore River Bridge
- Length : 273 feet
- Number of spans : 3 – Continuous
- Radius of Curvature : 175’
- Number of Girders : 4 (spaced @ 8’)
- Roadway width : 28’ (2 lanes)

For Bridge C, the calculations were carried out by Modjeski and Masters (M&M), using the BSDI program. The results include values of normal stresses in steel girders obtained in selected points of interest. The calculated values of stresses were compared to the analytical results obtained by the team at the University of Michigan (UMich), using the advanced finite element method analysis in ABAQUS software (UMich FEM). Similarly as for Bridges A and

B, the design values of load effects are represented by M&M BSDI analysis and the actual behavior is represented by UMich FEM analysis.

The statistical parameters of the bias factor (ratio between stresses obtained from the FEM analysis and the BSDI analysis) due to dead loads and live load with dynamic load allowance were calculated for Bridge C. They were obtained separately for constructional and operational loading stages. For the constructional loading stages (non-composite dead load of steel structure with girders plus diaphragms and stiffeners, and non-composite dead load of fresh concrete deck) the mean of bias factor is 0.91, with the coefficient of variation equal of 12.5%. Respectively, for the operational loading stages (due to live load with dynamic load allowance and due to loading of composite concrete deck components) the mean is equal to 0.89, with the coefficient of variation of 26.5%.

Calculated reliability indices for Bridge C showed that, the target reliability indices are close to β_T equal to 3.5. In summary, the analysis for Bridge C supports results presented for Bridge A.

C7.2 ANALYSIS OF 3-SPAN CONTINUOUS BRIDGE

The static analysis of the bridge is performed by analytical methods using FEM and BSDI programs. The distribution of loads (load per component) is also determined using them. The calculated design values of stresses (M&M BSDI) were compared to the analytical results obtained using the advanced finite element method analysis (UMich FEM).

Thus, for the considered bridge, the results from two sources are compared:

(a) Modjeski and Masters analytical analysis using BSDI software (M&M),
(b) University of Michigan analytical analysis using ABAQUS software (UMich).

The results include the tables with absolute values of normal stresses in steel girder obtained from both (a) and (b) analyses; the tables with comparison between stresses obtained in analyses (stresses ratios expressed as the bias factors); the tables with statistical analysis of these ratios; and statistical parameters of these bias factors.

Based on a comparative analysis of the field tests performed for Bridge A, it is assumed that the calibrated FEM results obtained using UMich program are representative for the actual behavior. As mentioned above, the nominal (design) values of load effects are represented by results from M&M and the actual behavior is represented by results from UMich FEM. This is a conservative assumption because, in general, it is observed that analytical results, obtained using the BSDI program, are more conservative than field test results or the FEM analysis. Therefore, the bias factors for loads distribution (GDF) for the reliability analysis are determined as ratios of stresses calculated using ABAQUS and design stresses calculated using BSDI program.

C7.3 FINITE ELEMENT METHOD (FEM) ANALYSIS

The finite element method (UMich FEM) analysis was performed by the University of Michigan. The analysis was done using computer software ABAQUS. The analysis and modeling is limited to superstructure part of the bridge. Substructure is not considered in this task.

C7.3.1 University of Michigan finite element model (FEM)

The superstructure was modeled and analyzed using ABAQUS, software based on the finite element method (FEM). The bridge was modeled as the 3-dimensional FEM model. The steel girders were modeled by shell elements, the cross-frames by beam elements and the slab by continuum solid 3-dimensional elements. The entire superstructure was approximately composed of 300,000 finite elements. Figure C-28 shows plan view of the considered bridge.

In the FEM analysis, it was assumed that live load due to truck wheel load was applied to the node of the upper layer of continuum element of the slab. The uniformly distributed lane load was assumed as a uniformly distributed pressure applied to the upper layer of continuum elements of the slab.

The stress calculations were performed in three selected sections for each girder, as shown in Figure C-29. The considered sections were located as follows: in span 1 (section 1), over the support #2 (section 2), and over the support #3 (section 3). The cross section of the bridge is presented in Figure C-29. Locations of analyzed points within the cross section of the girder are also shown in Figure C-29.

C7.4 FINITE ELEMENT METHOD (FEM) ANALYSIS

The results of the UMich FEM analysis were compared to M&M analysis performed using the BSDI program.

Four load cases were considered:

- Stg1-DL: Non-composite steel girders loaded by dead load (construction stage, loading: weight of steel girders and structural attachments such as: cross-frames, stiffeners, etc…),
- Stg6-DL: Non-composite steel girders loaded by dead load (construction stage, loading: weight of the fresh concrete deck)
- Stg7-DL: Composite steel girders with a concrete deck structure (operational stage, loading: weigh of composite elements of the deck, such as parapets, barriers, etc).,
- Stg-LL+IM: Composite steel girders with a concrete deck structure loaded with live load (operational stage, loading: load combination of live load with dynamic load allowance; two test trucks side-by-side for maximum positive moments in span; or four trucks, two on each lane for maximum negative moment over supports #2 and #3; plus uniformly distributed lane load in both cases), dead load is not included.

In general, the stresses obtained from UMich FEM analysis are smaller than those obtained from the M&M BSDI analysis; it confirms the results obtained in previous analysis for another bridge. From the UMich FEM results, it is also seen that the distribution of loads for girders are different than in the BSDI analysis. It refers especially to load cases Stg1-DL and Stg6-DL; and these different distributions are possibly due to differences in the lateral stiffness of the non-composite steel structure (provided by the steel cross-frames composed of stiffeners and diaphragms) in the FEM and the BSDI analysis. In the load cases Stg7-DL and Stg-LL+IM these differences decrease, because most of the lateral stiffness is provided by the very stiff concrete deck, which is similarly modeled in both the BSDI and the FEM analyses.

The results from structural analyses from both sources were compared in few selected, representative sections in the structures. All points were located in steel section of the girders. The results include stresses obtained in 4 points of interest (POI). These POI's are:

- POI1: Span 1, Girder 3, at 38.05 ft. from the support (i.e. midspan, section 1),
- POI2: Span 1, Girder 4, at 39.69 ft. from the support (i.e. midspan, section 1),
- POI3: Span 3, Girder 3, at 0 ft. from the support (i.e. support 3, section 3),
- POI4: Span 2, Girder 4, 0 ft. from support (i.e. support 2, section 2).

Summary of obtained results from structural analysis is presented in Tables C-21 and C-22. In these tables results from both UMich FEM and M&M are expressed in terms of normal stresses in steel girders.

C7.5 STATISTICAL ANALYSIS OF STRESS RATIOS (BIAS FACTOR)

C7.5.1 Statistical Analysis

The objective of this analysis was to determine bias factor for different types of analysis due to dead loads and live load. The results of static analysis are compared for two methods M&M BSDI and UMich FEM. These bias factors are usually below 1.0; and as it was observed analytical results, obtained using BSDI program, are more conservative than FEM analysis.

The ratios between stresses (bias factors) obtained from the two considered sources are calculated, and the results are presented in Tables C-21 and C-22. They are also presented in Figures C-30 through C-33. This statistical analysis was performed separately for constructional stages (Stg1-DL and Stg6-DL) and operational loading stages (Stg7-DL and Stg-LL+IM). Results from bottom and top flanges of steel girders are combined together in statistical analysis. Figures C-30 through C-33 contain graphs with relationship between the bias factor (UMich FEM / M&M) and the absolute values of stresses obtained from the UMich FEM.

The values of the bias factor should be equal to 1.0, if both analyses would be equally accurate and give the same values of stresses. Statistical analysis was performed graphically and the data is plotted on the normal probability paper in Figures C-34 and C-35. In these figures CDF's of the bias factors are plotted and linear approximation (with the equation of the linear function) is presented. The vertical axis shows the number of standard deviations from the mean value for the standard normal distribution. Statistical parameters of obtained distributions can be determined from the graphs. For non-composite girders and load cases Stg1-DL and Stg6-DL,

the mean value of the bias factor, is 0.91, and the coefficient of variation is about 12.5%, with the standard deviations about 0.11. For composite girders and load cases Stg7-DL and Stg-LL+IM, the mean of bias factor is 0.89, and the coefficient of variation is 26.5%, with the standard deviation about 0.24.

The statistical parameters of the bias factors are calculated after omitting some of the values of stresses from both types of analysis. However, the omitted values of the stresses are not representative because these are points of relatively small stresses. In these points, even small differences in location of the neutral axis, cause large differences in ratio of stresses obtained from the UMich FEM and from the M&M BSDI analysis.

C7.6 SUMMARY OF RESULTS FOR BRIDGE C

The statistical parameters of the bias factor – ratio between stresses obtained from the UMich FEM analysis and the M&M BSDI analysis - due to dead loads and live load with dynamic load allowance are shown in the Tables C-23 and C-24. They are presented separately for constructional and operational loading stages.

C7.7 RELIABILITY ANALYSIS FOR BRIDGE C

The procedure is as applied in the previous calibration (Nowak, 1999), the same as presented and used for Bridge A. In practice, we are interested only in a normal operation of the bridge and the bridge during construction; therefore, only for those two cases the reliability analysis is performed.

The ratios of load components are determined for Bridge C. The load ratios are different during construction, and they are also shown in Table C-25.

For the considered bridge, in normal operation (as opposed to construction) (for the details of the procedure see Section 5.4):

$R_n = [1.25\ (4 + 18.9) + 1.75\ (5.6 + 1.8)] / 1.00 = 41.58$
$m_R = (1.165)(41.58) = 48.44$
$m_Q = (1.00)(4 + 18.9) + (0.85)\ (5.6)\ (1.1) = 28.14$
$A = 1 - (2)(0.095) = 0.81$
$\sigma_D = \{[(1.00)(4)(0.15)]^2 + [(1.00)(18.9)(0.15)]^2\}^{0.5} = 2.90$
$\sigma_L = (0.85)\ (5.6)\ (1.1)(0.215) = 1.06$
$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 3.09$
$\beta = 4.00$

For the bridge during construction:

$R_n = [1.25\ (4 + 13.6) + 1.75\ (0.4)] / 1.00 = 22.7$
or $R_n = [1.5\ (4 + 13.6)] / 1.00 = 26.4$ (governs)
$m_R = (1.165)(26.4) = 30.76$
$m_Q = (1.00)(4 + 13.9) + (0.4)\ (1.1) = 18.34$
$A = 1 - (2)(0.095) = 0.81$

$\sigma_D = \{[(1.00)(4)(0.15)]^2 + [(1.00)(13.6)(0.15)]^2\}^{0.5} = 2.12$
$\sigma_L = (0.34)\ (1.1)(0.215) = 0.08$
$\sigma_Q = (\sigma_D^2 + \sigma_L^2)^{0.5} = 2.12$
$\beta = 3.82$

The target reliability index is assumed the same as straight bridges (β_T equal to 3.5).

C7.8 CONCLUSIONS FOR BRIDGE C

The objective of this task was to compare two different types of static analysis of a curved bridge under dead load and live load. The BSDI program uses a grillage analogy method and the FEM analysis is based on an advanced finite element method using ABAQUS Software. The comparison of two methods is expressed in terms of the bias factor, which is the ratio of normal stresses obtained from the UMich FEM analysis and the M&M BSDI analysis. Four load cases are considered:

- Stg1-DL : Non-composite steel girders under dead load (steel only),
- Stg6-DL : Non-composite steel girders with the fresh concrete deck under dead load (concrete of the deck only),
- Stg7-DL : Composite steel girders with concrete deck structure, dead load of composite elements of the concrete deck.
- Stg-LL+IM : Composite steel girders with concrete deck structure, under live load only (dynamic load allowance included), dead load not included.

Four sections were considered for each girder to evaluate normal stresses: the critical section in the first span, the critical section in the third span (middle span), and the section over the supports between the first and second span, and second and third.

The results from the comparison are presented in form of the bias factors. Values of the bias factor, if both analyses would be equally accurate and give the same values of stress, should be equal to 1.0. From the analysis, the mean values of the bias factor for the non-composite structure, load cases Stg1-DL and Stg2-DL, is 0.91, the coefficient of variation are about 12.5%, and the standard deviations are about 0.11. The bias factor for the composite structure load cases Stg7-DL and Stg-LL+IM, is 0.89, the coefficient of variation is 26.5%, and the standard deviation is 0.24.

In general, the absolute values of stress obtained from the UMich FEM analysis are smaller than those obtained from the M&M BSDI analysis. It can be explained as follows. In FEM analysis, 3-dimensional structural behavior is considered without geometrical simplifications and mechanical simplifications (material properties). The BSDI software simplifies the model of the structure (2-D grillage analogy model). It is possible that the load effect and the structural response are increased by some factors that take into account uncertainties arising from the method of analysis.

In summary, the analysis performed for Bridge C supports the results obtained for Bridge A, presented in the main body of the report. The bias factor is around 0.90 and the larger than in

general coefficient of variation is around 12 to 27%. The large coefficient of variation for Bridge C for live load response may be explained by the small number of samples which are used in analysis.

C8 LOAD AND RESISTANCE FACTORS

It is assumed that load factors remain the same as in the AASHTO LRFD (1998). The objective of this study is to determine the optimum value of resistance factor for curved girder bridges. The major difference between a straight bridge and a curved bridge is in the girder distribution factors. This is the only major factor that can affect the reliability.

The analysis performed for Bridge A, supported by the results of field tests, indicates that girder distribution factors for curved girders are subjected to a higher degree of variation (compared with straight girders). However, the bias factor (ratio of mean to nominal value) is lower for curved girders, and this more than compensates the negative effect of increased coefficient of variation.

The reliability analysis was performed to establish the relationship between the resistance factor and reliability index for the three considered bridges. The results are shown in Figure C-36, Figure C-37 and Figure C-38 for Bridges A, B and C, respectively. They are also summarized in Table C-26. It is clear that the resistance factor ϕ equal to 1.0 provides the reliability indices exceeding the target value β_T equal to 3.5 (Nowak 1999).

Additional calculations, performed for Bridge B and Bridge C, are consistent with the results obtained for Bridge A. The obtained reliability indices corresponding to ϕ equal to 1.0 all exceed the target reliability index of 3.5, therefore, it is recommended to use the same resistance factors for straight and curved girders.

C9 CONCLUSIONS AND RECOMMENDATIONS

The conclusions are based on the analysis of three curved steel girder bridges and test results for one of them. The results indicate that the resistance factors for curved steel girders can be the same as for straight girders. The observations can be summarized as follows.

1. Construction stage is very important for curved girders. There is a considerable degree of variation in stress values, confirmed by the discrepancy between the analysis and test results.

2. Typical load ratios for construction are steel 30%, concrete slab 70%, with less than 5% for forms and other loads.

3. Typical load ratios for composite structure are steel 20%, concrete 50%, live load 25-30%.

4. The results of the FEM analysis carried out by the team at the University of Michigan were compared with analysis by Modjeski and Masters using the BSDI program. For Bridge A, in addition, the results were also compared with the analysis by the University of Minnesota using the grillage method (Galambos et al. 1996), and field tests by the University of Minnesota

(Galambos et al. 1996). In general, it was observed that the measured stresses are smaller than the analytical results. In most cases, stresses obtained from FEM analysis are between the measured stresses and those obtained by the grillage method. For Bridges B and C, it is assumed that the ratio of actual and FEM stresses are similar as for Bridge A.

5. The test results are very important. They provide information about the actual behavior of the structure. However, the results can be affected by inaccuracies in the installation of equipment, and more importantly, by local conditions. The measured strains can reflect the component-specific effect of non-structural elements, rigidity (partial fixity) of connections, irregular (structure-specific) distribution load, and so on. It was observed that some readings were inconsistent, in particular, some measured strains due to dead load were very low, while others were closer to expected values. This could be explained by the local effects. Therefore, the test results were not treated as an absolute reference in calculation of bias factors.

6. It is assumed that the designer will used BSDI program and, therefore, it is considered as a reference. Use of ABAQUS requires advanced knowledge of FEM and it may be difficult to expect to it be used by consulting offices. The University of Minnesota grillage method is reliable, but ABAQUS analysis performed by the University of Michigan is more accurate because of a fine mesh and handling of the boundary conditions. The finite element method (FEM) is mathematically correct, and with a fine mesh, it can provide very accurate results. However, the accuracy of computations depends on the accuracy of input data, in particular boundary conditions. Therefore, the results of measurements served as a basis for verification of the boundary conditions. If the results of measurements were inconsistent, e.g. three gages were positioned so that the expected readings would be about the same, but one of them was much lower than the other two, it was assumed that this is due to local conditions. On the other hand, previous practice in computations using FEM showed that calculated strain can be unrealistically high in the area of concentrated force application (support), and therefore, the analysis can require a special approach.

7. Bias factor for dead load is considered as a product of the bias factor for the load itself (1.03) and dead load analysis (0.95), therefore 1.0 is used.

8. The statistical model for live load includes the live load itself (weight of trucks) and live load effect (analysis, distribution of tuck load to girders). The weight of trucks is the same as for straight girders, and the bias factor is 1.25 to 1.35 (assuming HL 93 is the design load). Similarly, the probability of a simultaneous occurrence in a lane or even side-by-side in two adjacent lanes is considered as not affected by the curvature of the bridge, therefore, the ratio of the mean maximum two-month truck and mean maximum 75-year truck is 0.85. The difference between straight and curved bridges is in the distribution of live load to girders. It is assumed, that in curved girder bridges, the live load effect per girder is determined by analysis performed using BSDI. GDF used in the report is the fraction of the lane load calculated using BSDI program. The overall bias factor for live load is the product of three factors: (a) bias of lane load (1.25 to 1.35), (b) ratio of two-month truck and 75-year truck (0.85), and (c) bias factor for analysis using BSDI (0.75).

9. The basic difference between the straight and curved girders is the girder distribution factor, that involved a higher degree of variation for curved girders, but the bias factor is lower for curved girders. The bias factor is the ratio of mean to nominal, and nominal is what is obtained from M&M (using BSDI program).

10. The effect of a higher coefficient of variation of the girder distribution factor for curved girders is practically neutralized by a lower bias factor. Therefore, the resistance factors derived for straight girders are adequate for curved girders also.

C10 FIGURES

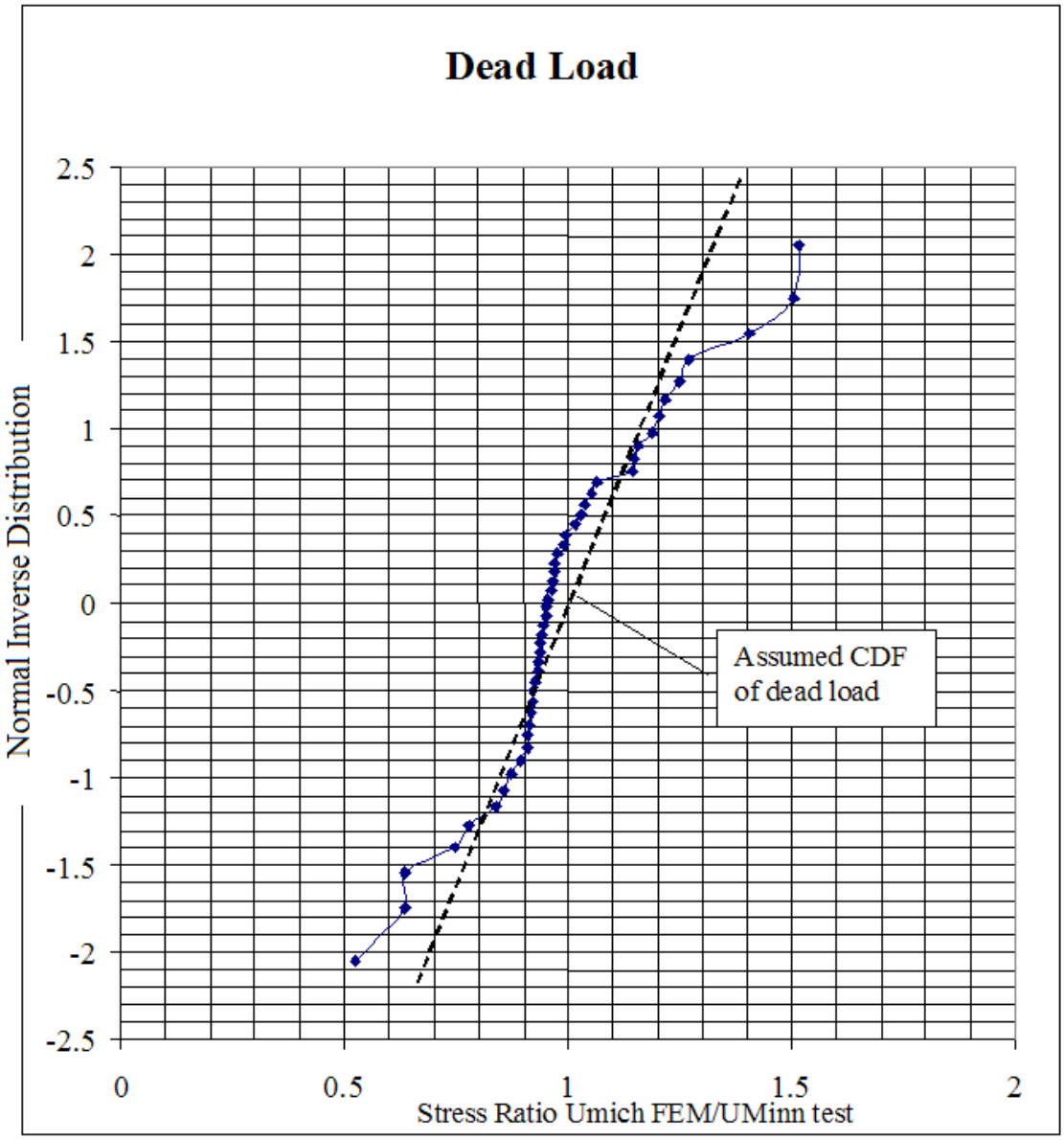


Figure C-1. Cumulative Distribution Function of the Stress Ratio, UMich FEM / UMinn test, Due to Dead Load determined for Bridge A.

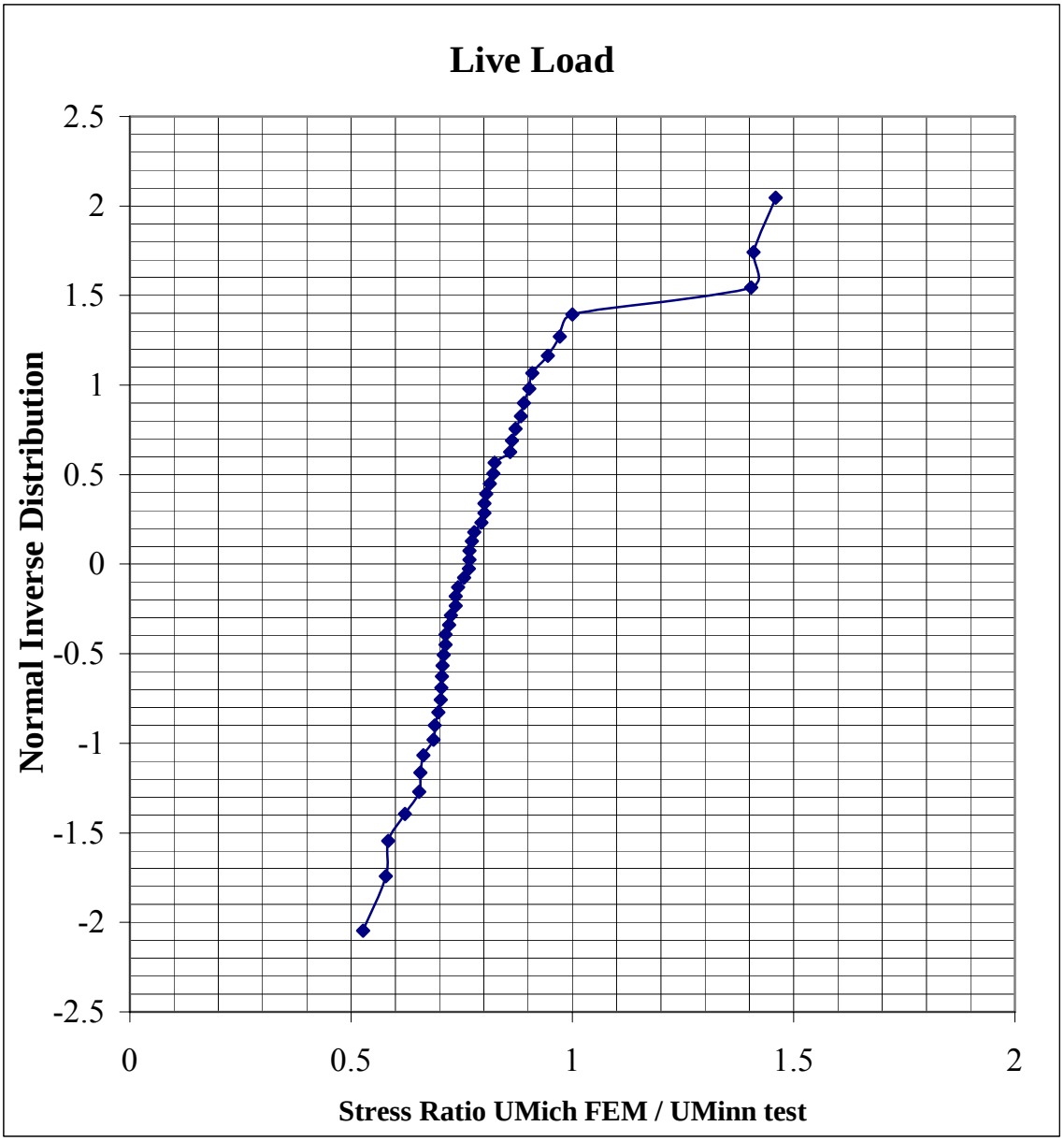


Figure C-2. Cumulative Distribution Function of the Stress Ratio, UMich FEM/ UMinn test, Due to Live Load determined for Bridge A.

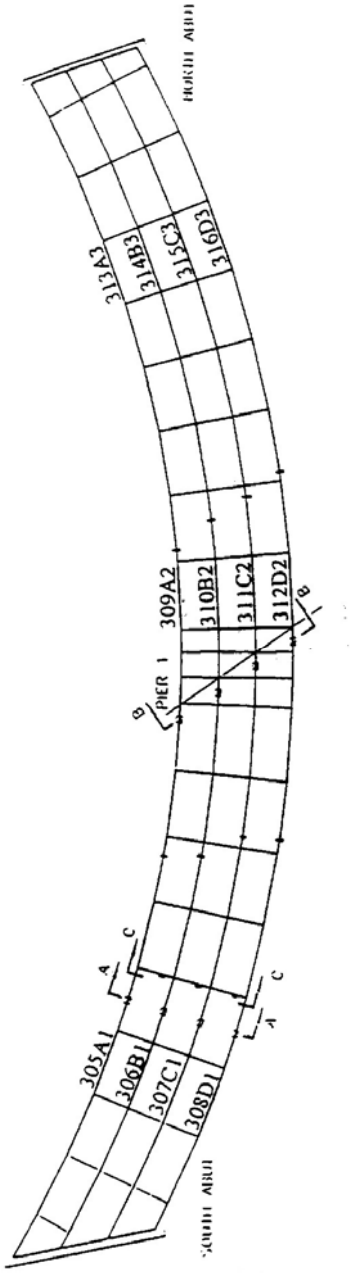


Figure C-3. Plan of Bridge A

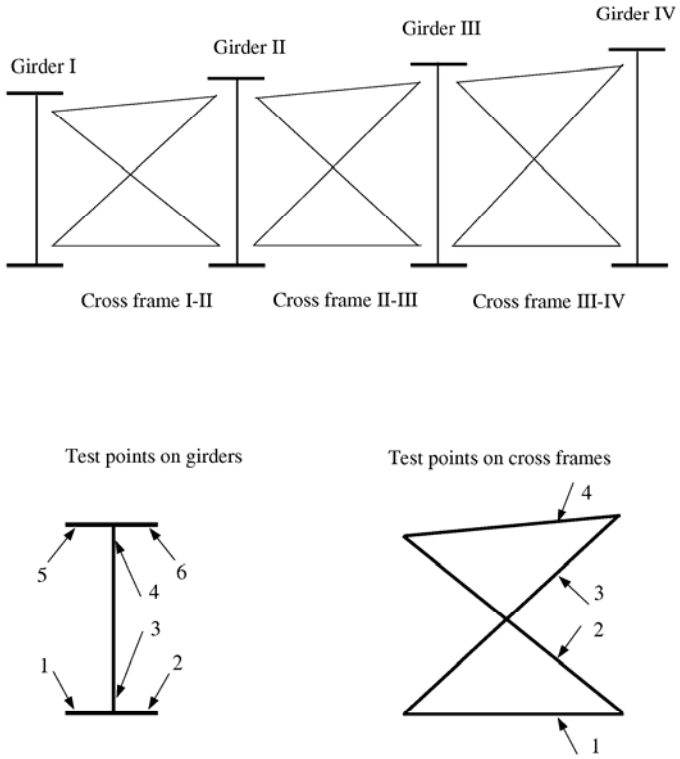
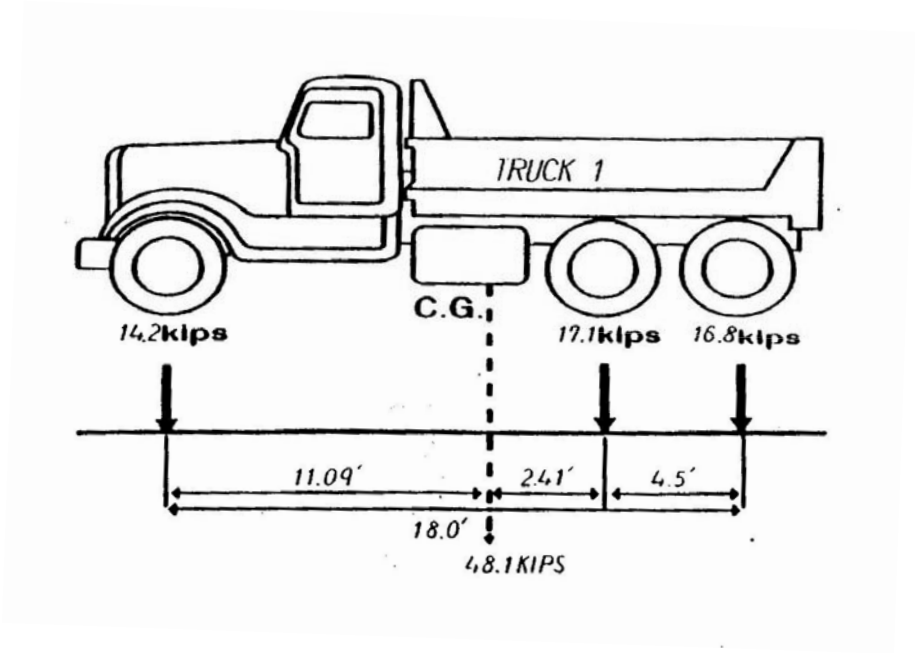


Figure C-4. Cross Section of Bridge A and Location of Points for Stress Analysis



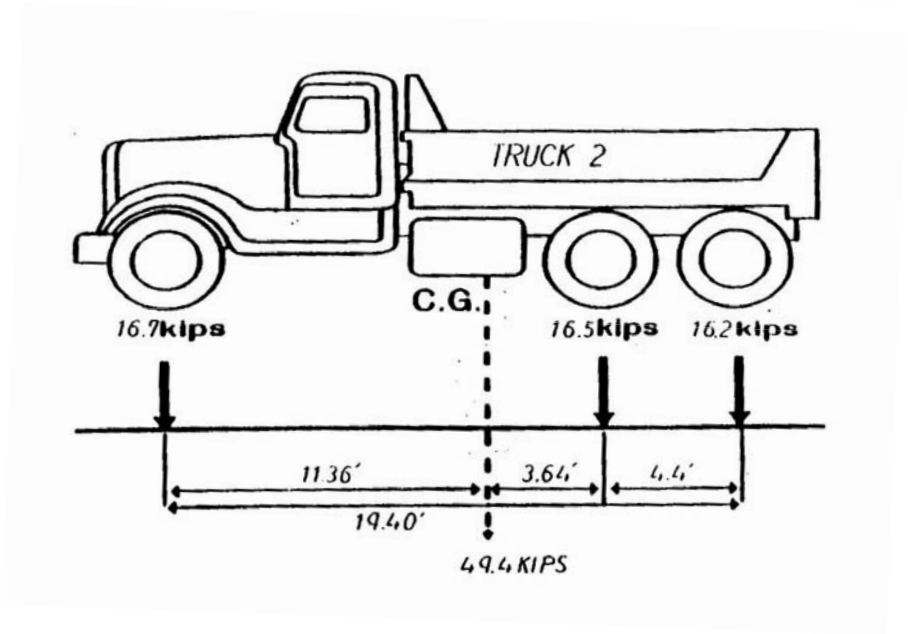


Figure C-5. Test Trucks used In Field Tests of Bridge A

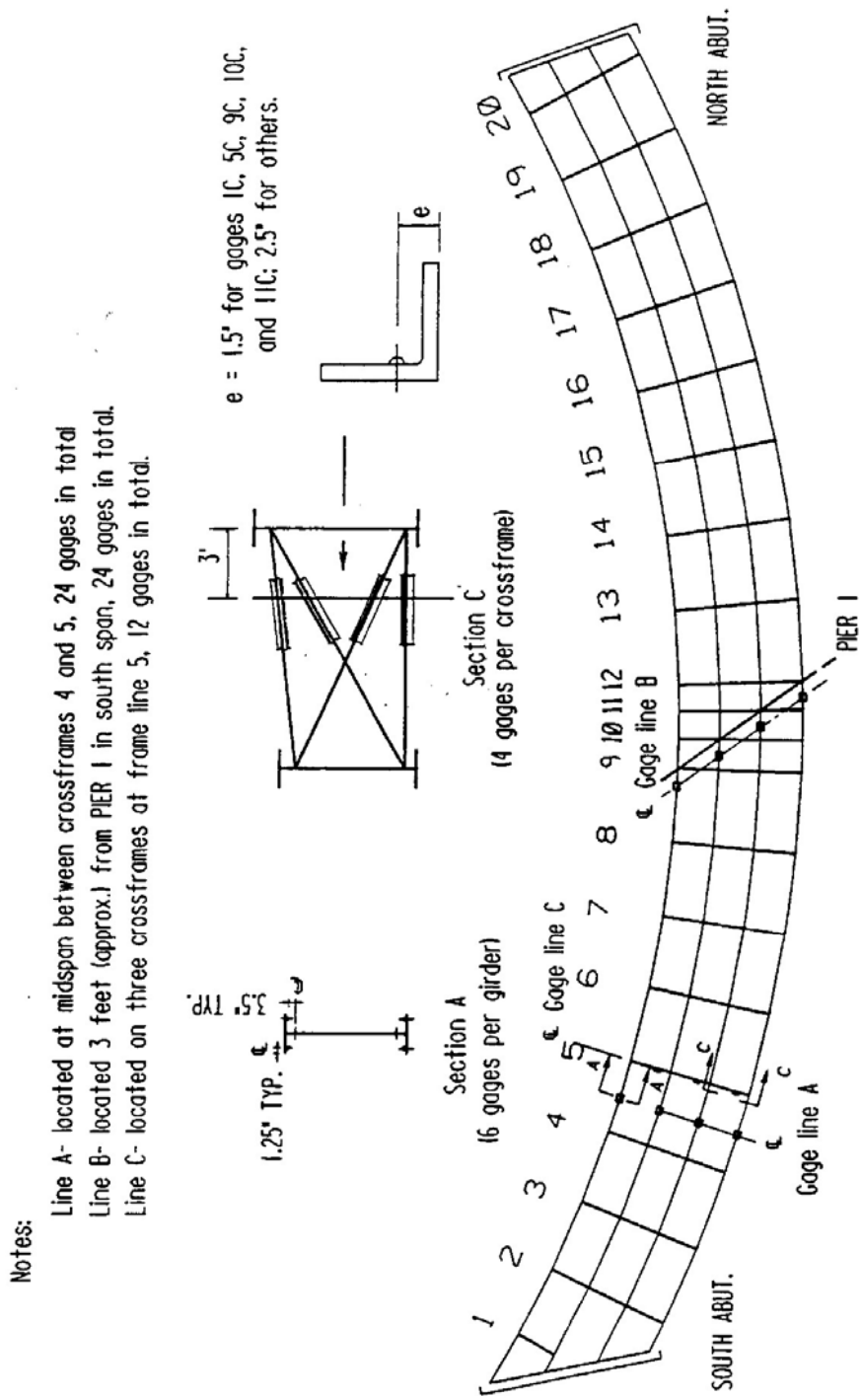


Figure C-6. Plan view of Bridge A – Location of Gage Lines

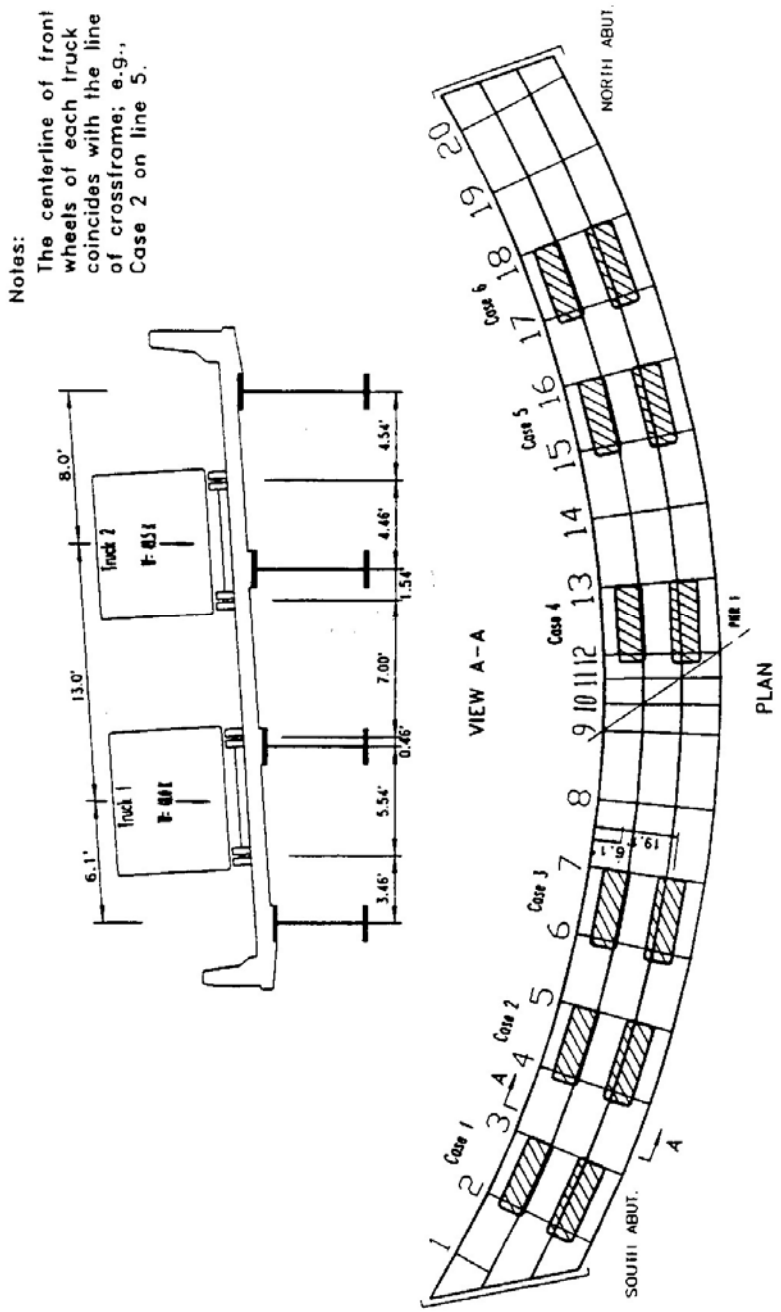


Figure C-7. Bridge A: Live Load Cases 1-6, Cross Frame III-IV

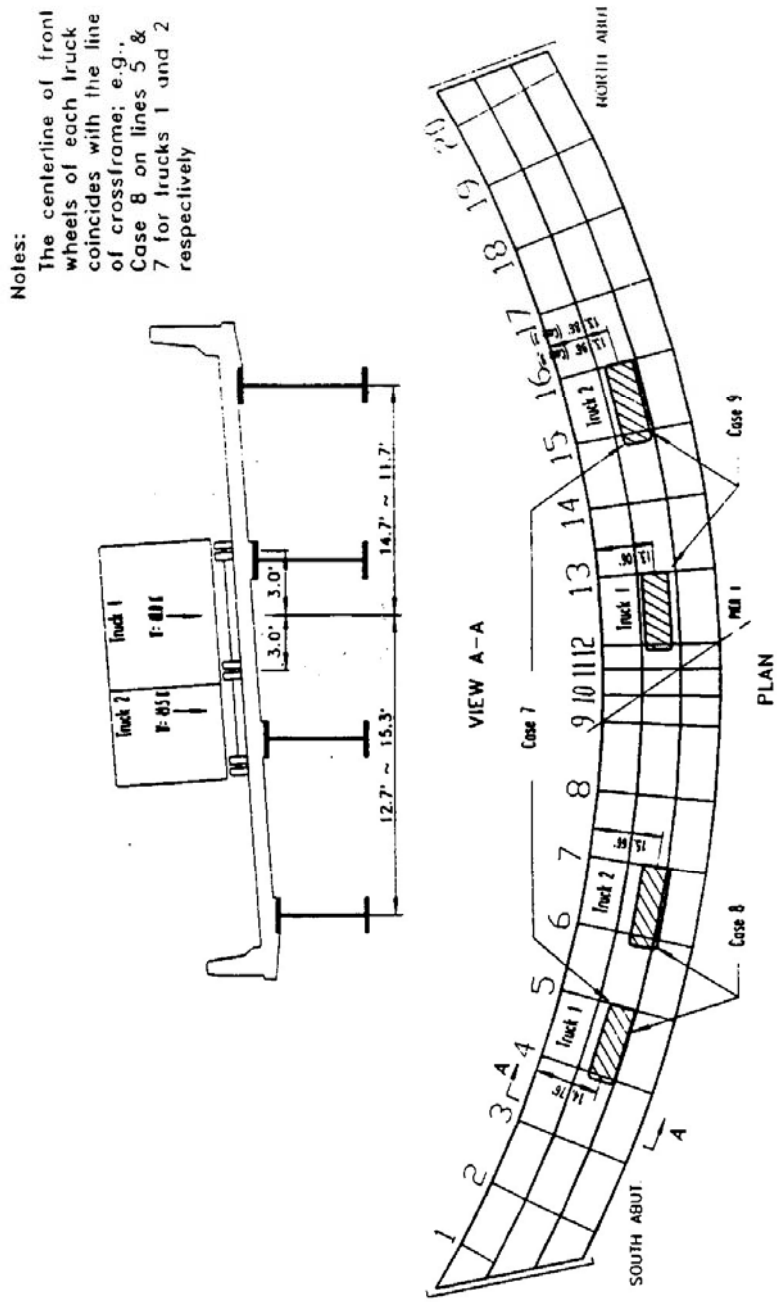


Figure C-8. Bridge A: Live Load Cases 7-9, Cross Frame III-IV

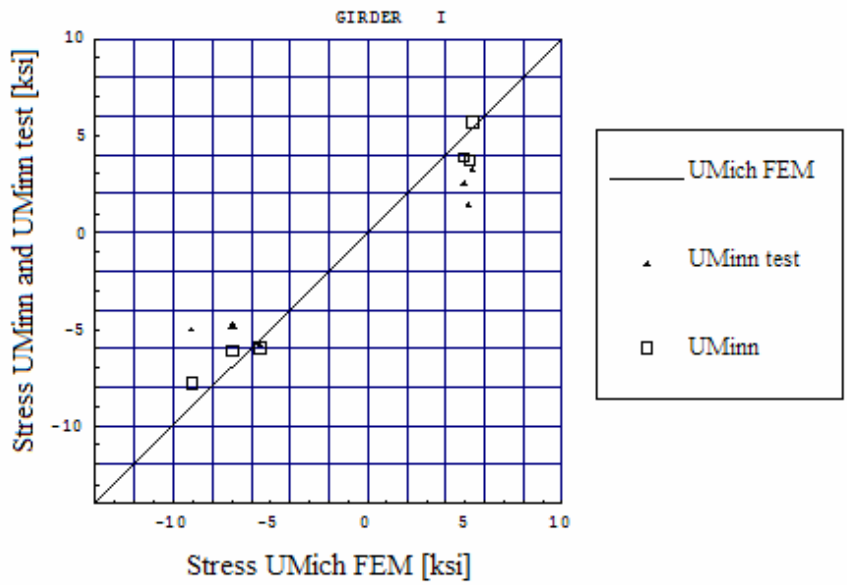


Figure C-9. Bridge A: Stress due to Dead Load, Gage Line A, Girder I

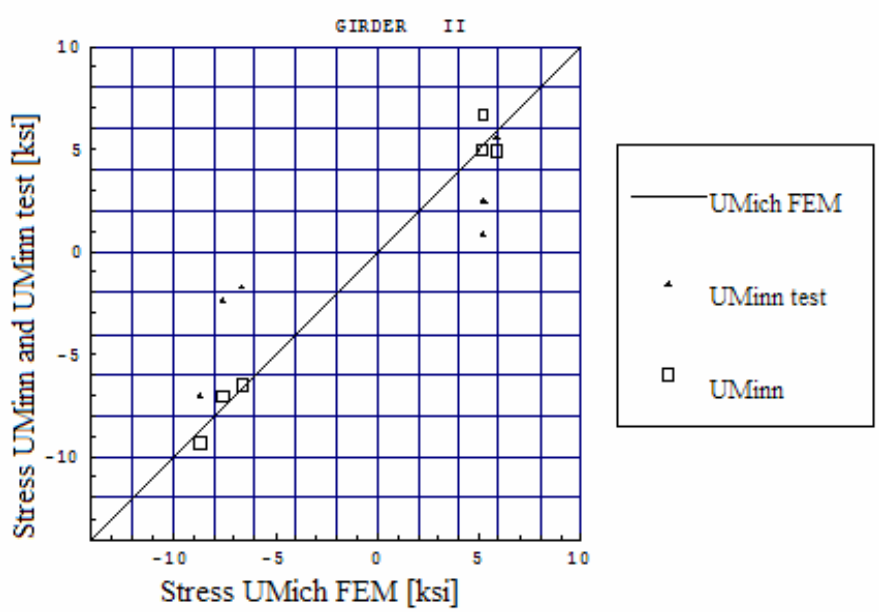


Figure C-10. Bridge A: Stress due to Dead Load, Gage Line A, Girder II

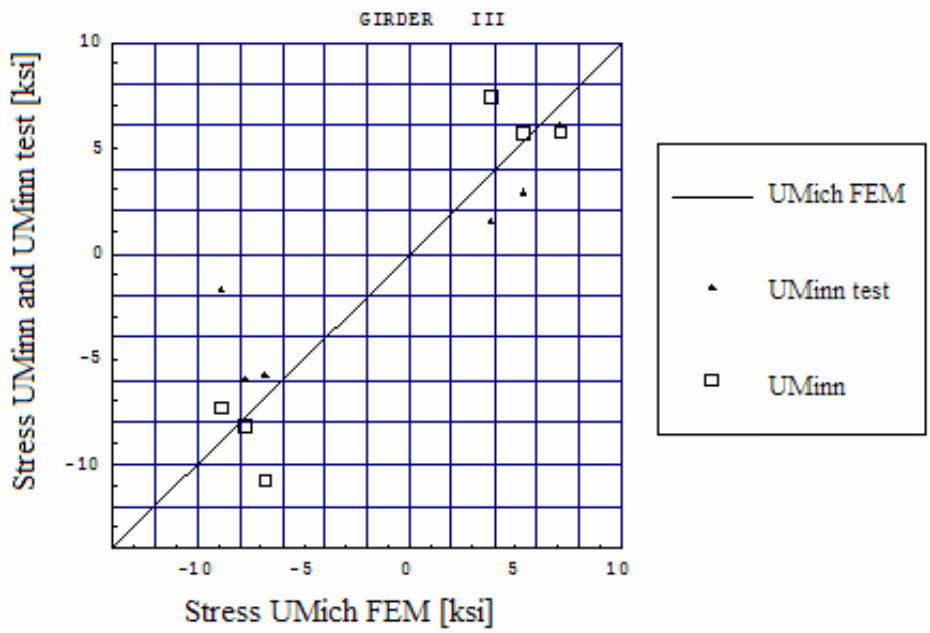


Figure C-11. Bridge A: Stress due to Dead Load, Gage Line A, Girder III

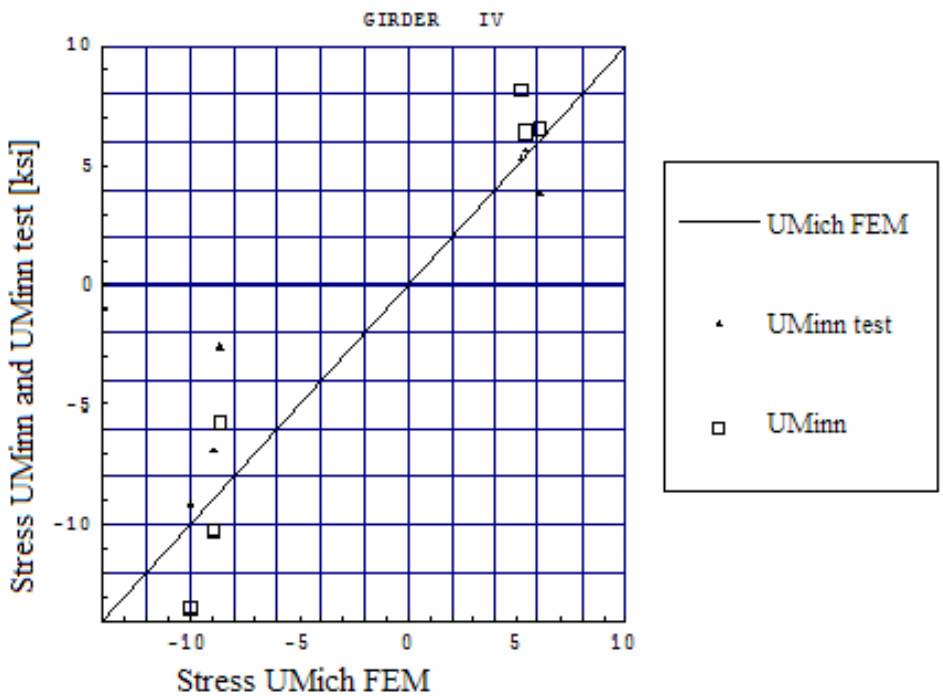


Figure C-12. Bridge A: Stress due to Dead Load, Gage Line A, Girder IV

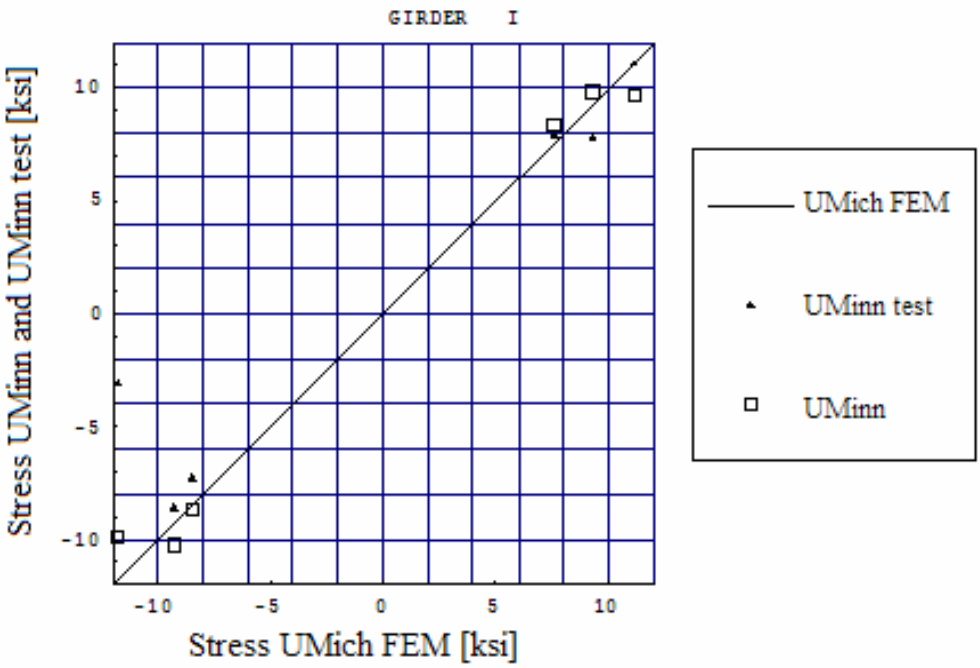


Figure C-13. Bridge A: Stress due to Dead Load, Strain Gage Line B, Girder I

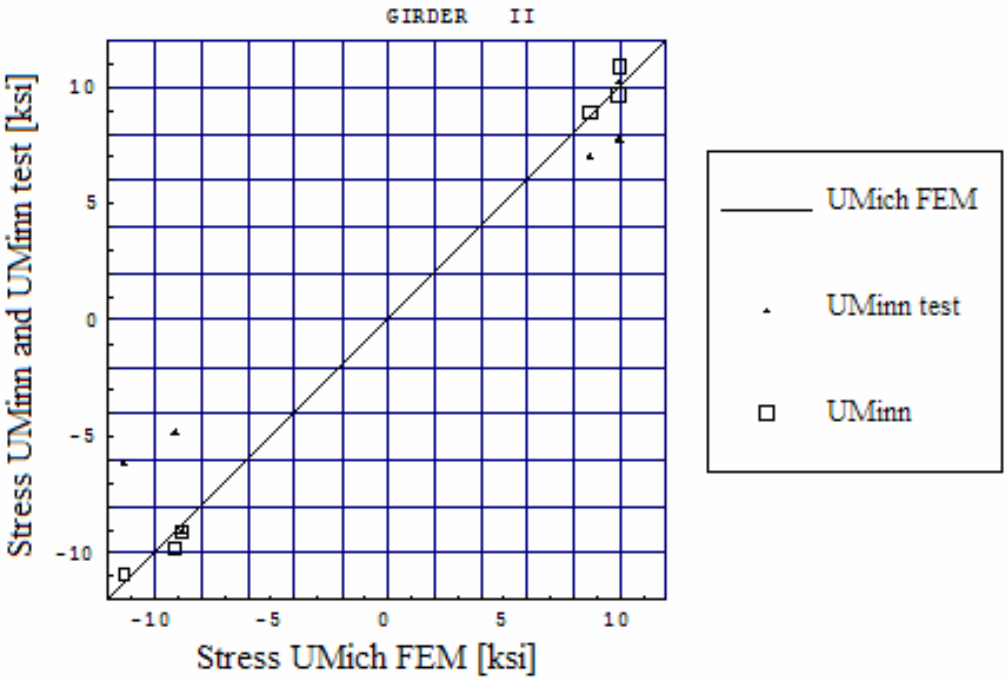


Figure C-14. Bridge A: Stress due to Dead Load, Strain Gage Line B, Girder II

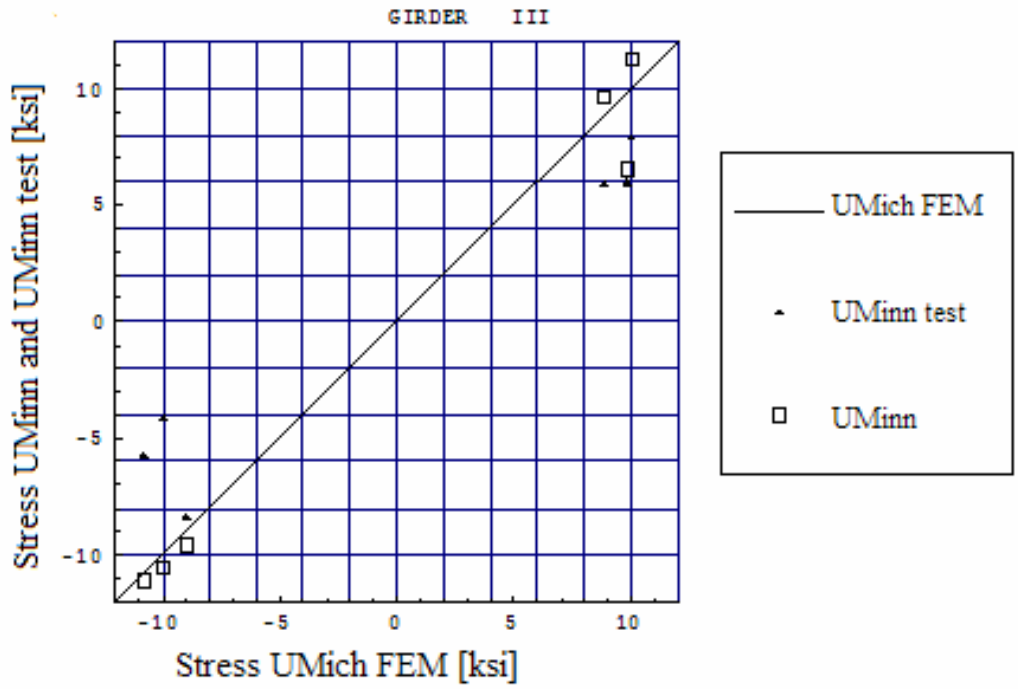


Figure C-15. Bridge A: Stress due to Dead Load, Strain Gage Line B, Girder III

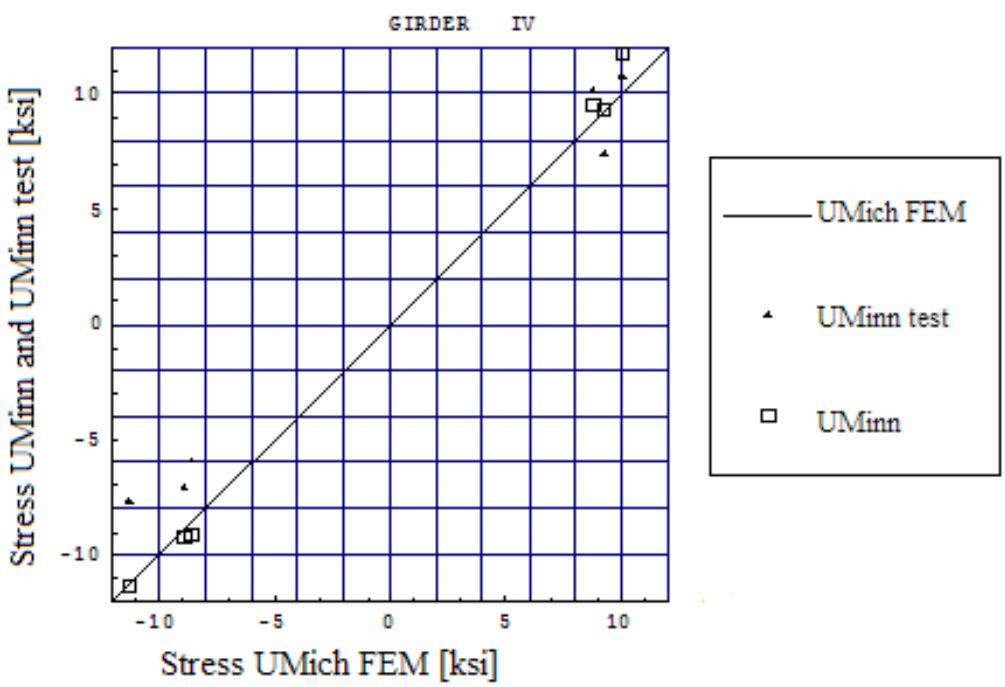


Figure C-16. Bridge A: Stress due to Dead Load, Strain Gage Line B, Girder IV

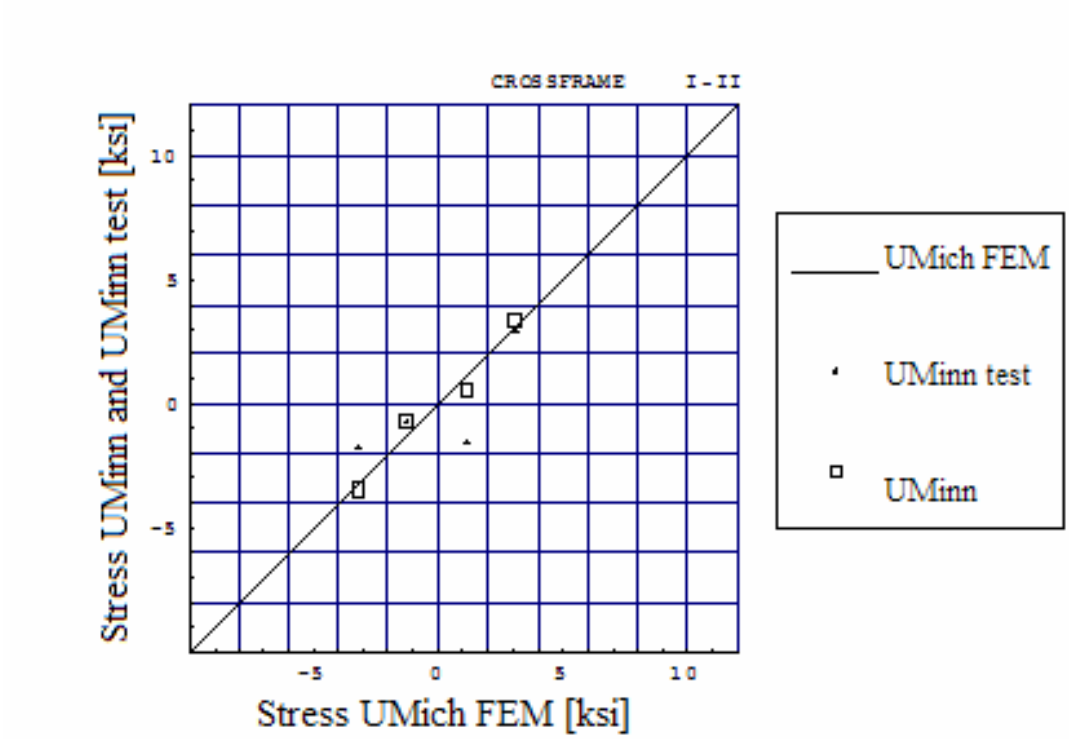


Figure C-17. Bridge A: Stress due to Dead Load, Strain Gage Line C, Cross Frame I-II

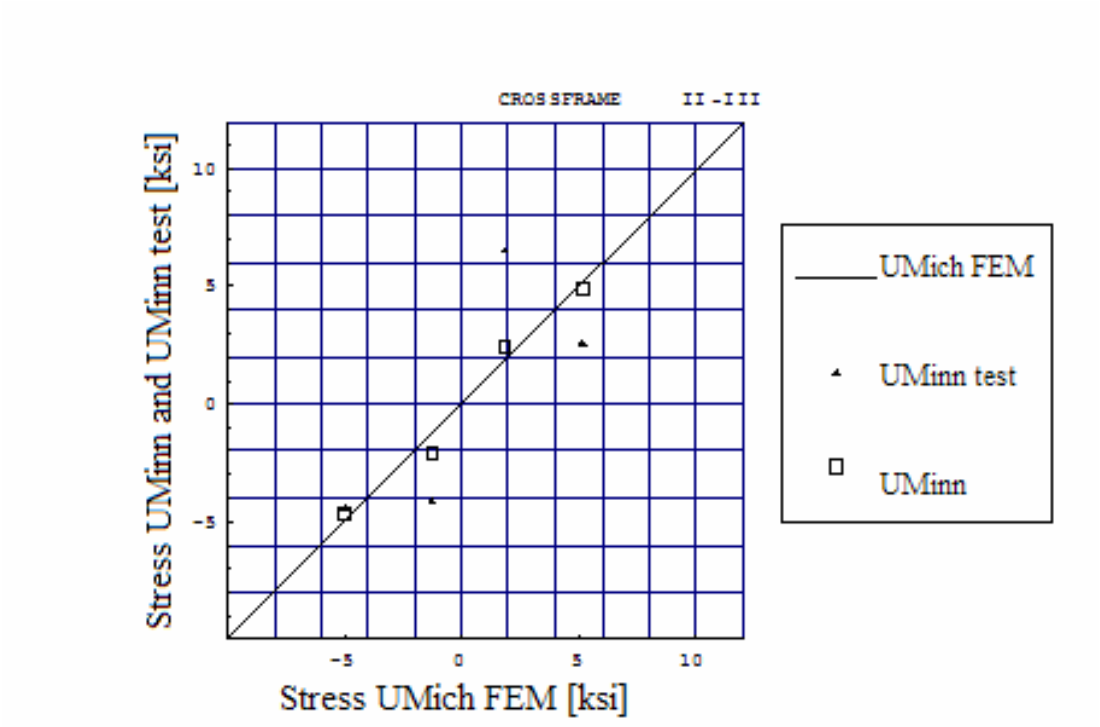


Figure C-18. Bridge A: Stress due to Dead Load, Strain Gage Line C, Cross Frame II-III

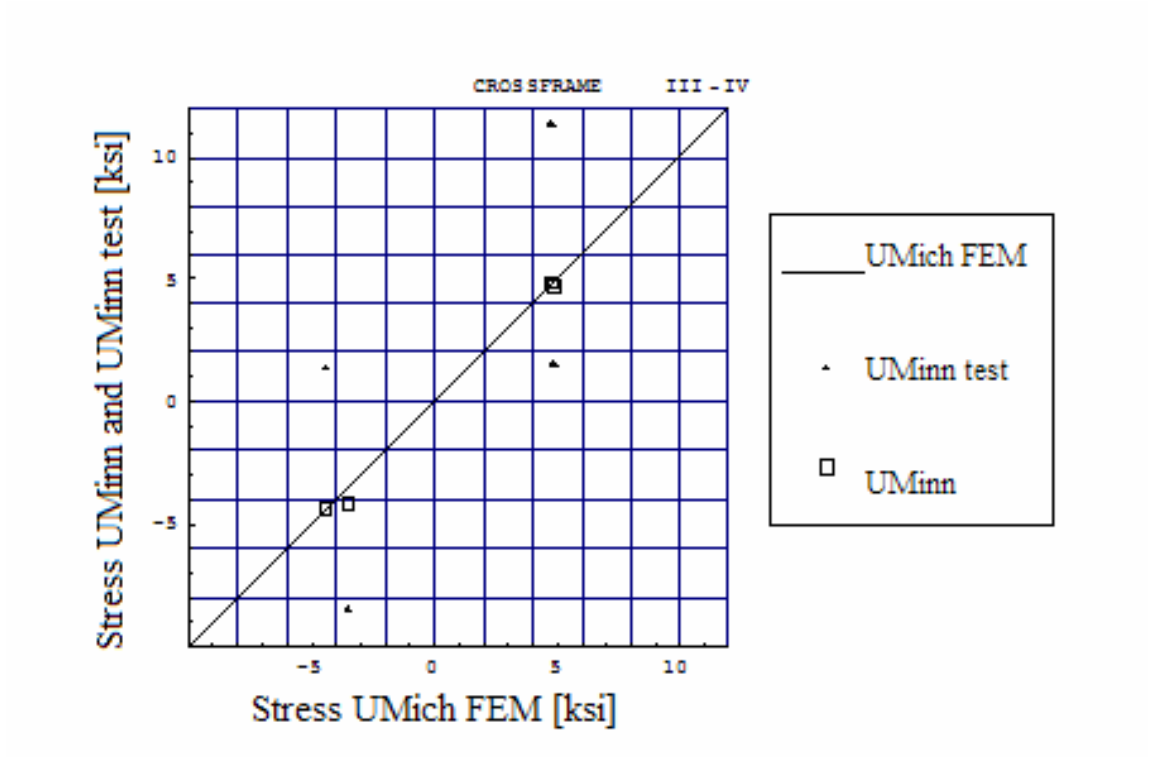


Figure C-19. Bridge A: Stress due to Dead Load, Strain Gage Line C, Cross Frame III-IV

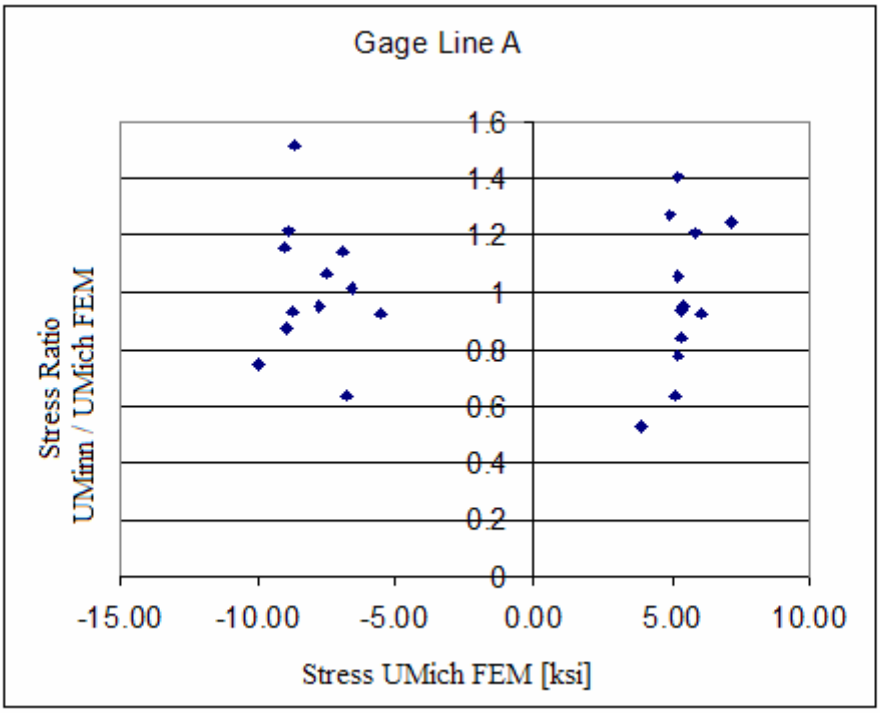


Figure C-20. Bridge A: Bias Factors for Normal Stresses in Girders in Gage Line A

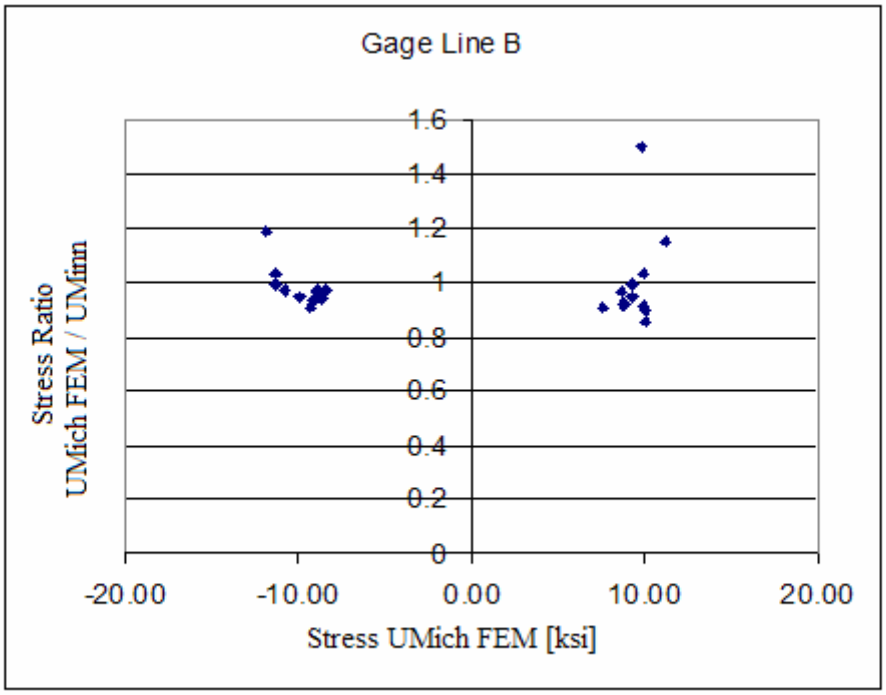


Figure C-21. Bridge A: Bias Factors for Normal Stresses in Girders in Gage Line B

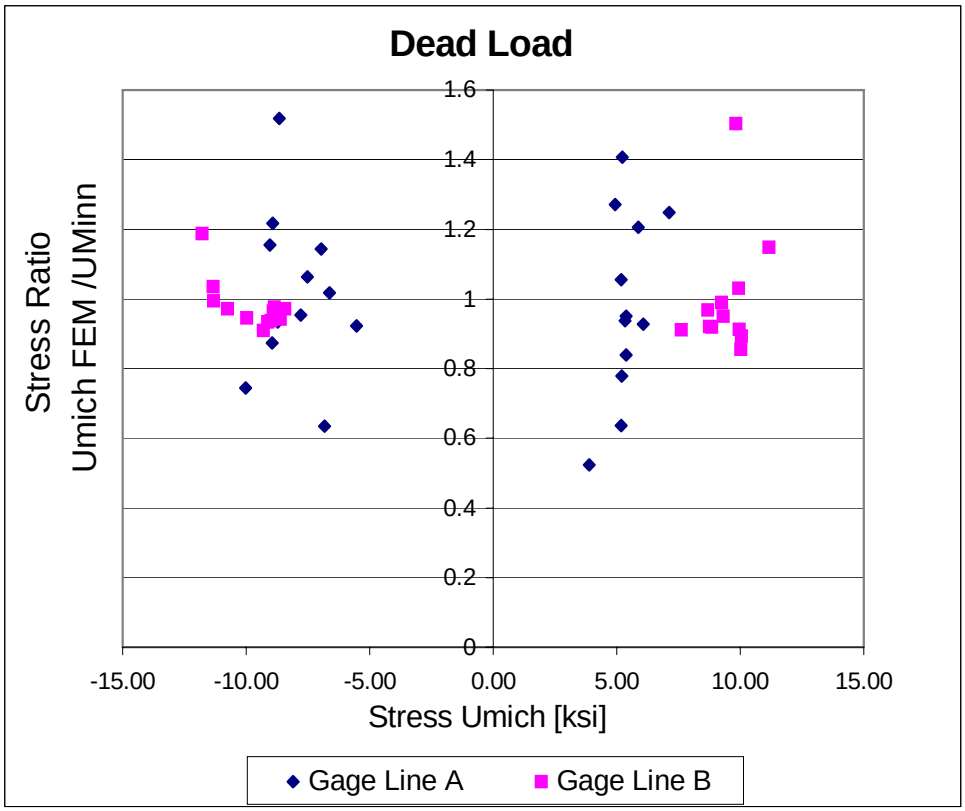


Figure C-22. Bridge A: Bias Factors for Normal Stresses in Girders Due to Dead Load

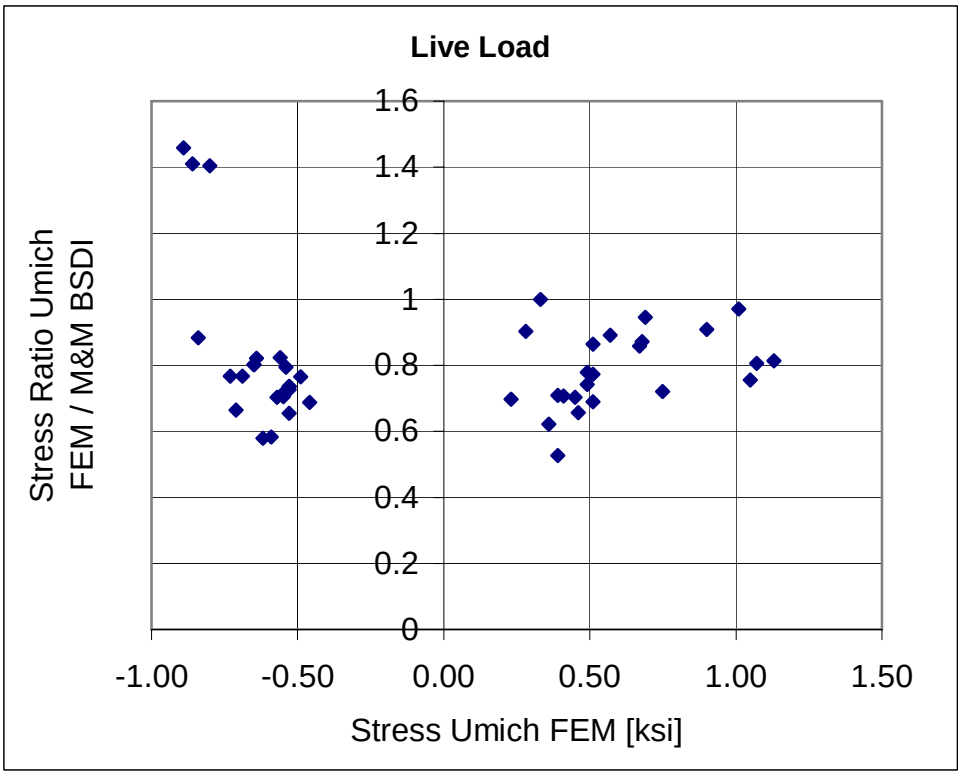


Figure C-23. Bias Factors for Stresses due to Live Load.

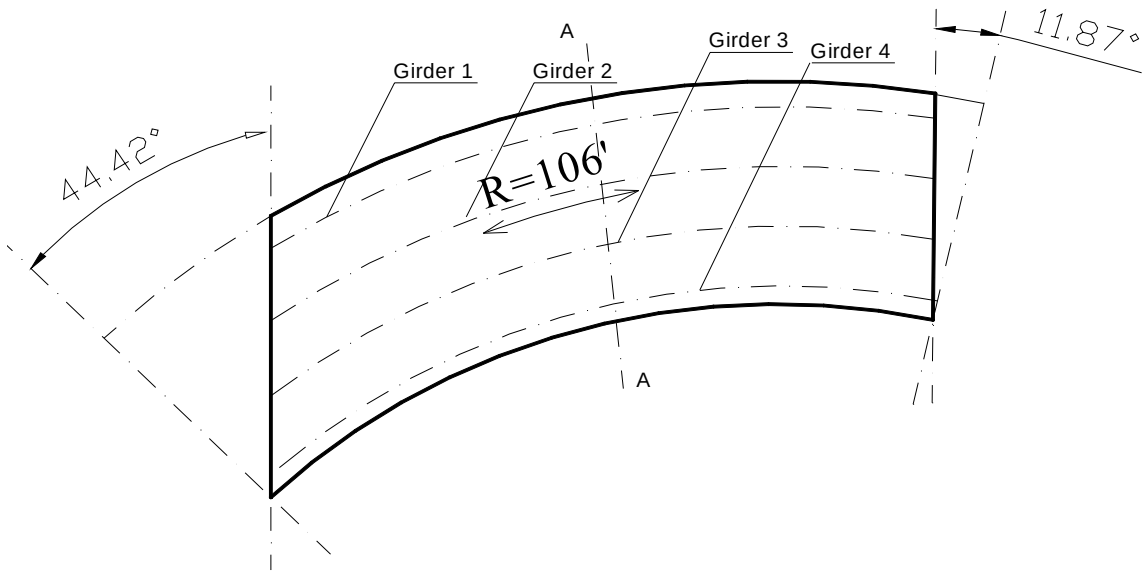


Figure C-24. Plan of Bridge B

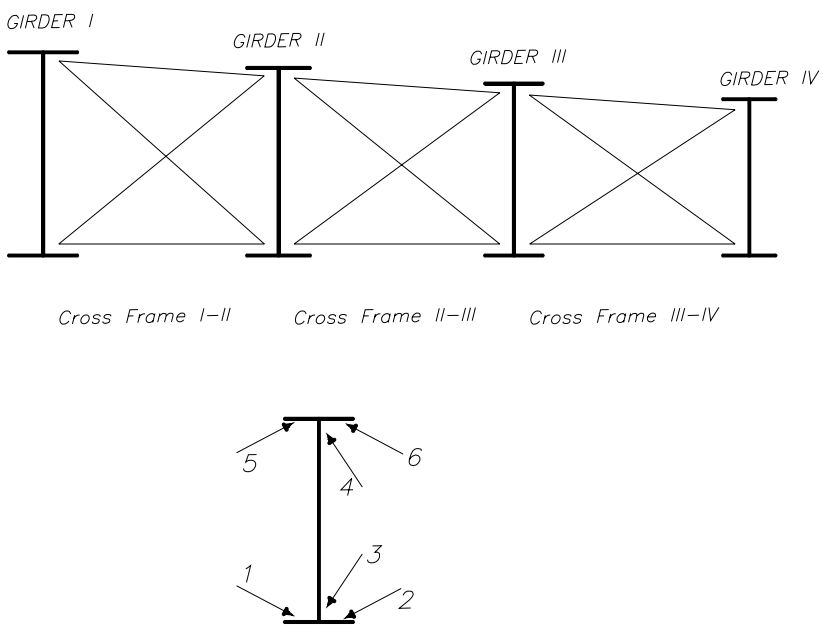


Figure C-25. Cross Section of Bridge B and Location of Points of Interest for Stress Analysis

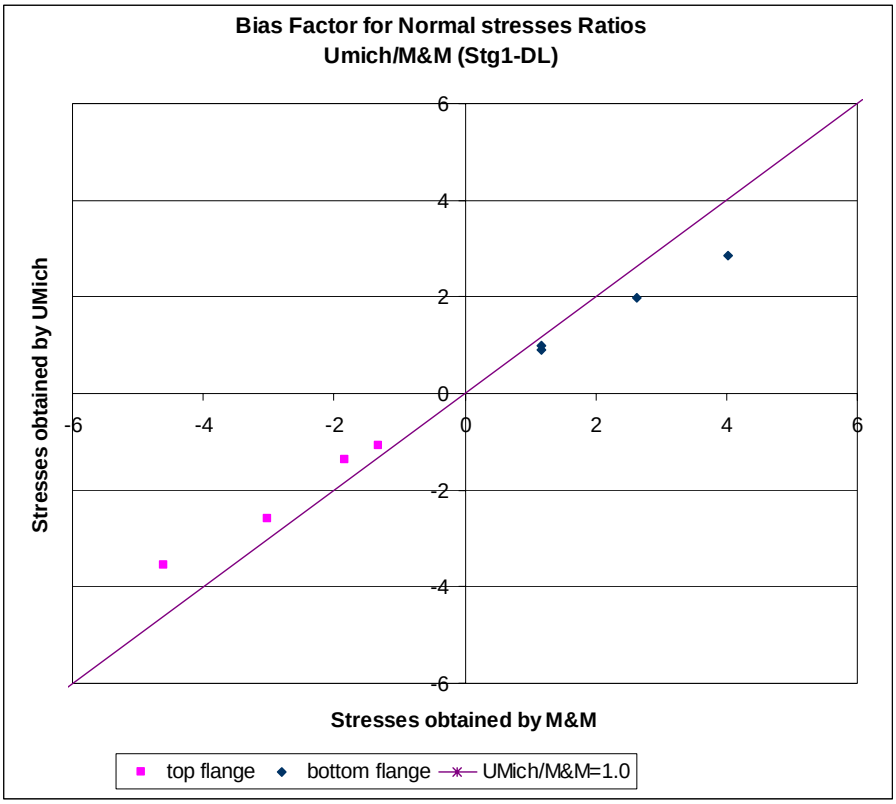


Figure C-26. Bridge B: Ratios of Stresses obtained by UMich FEM and by M&M (Stg1 DL)

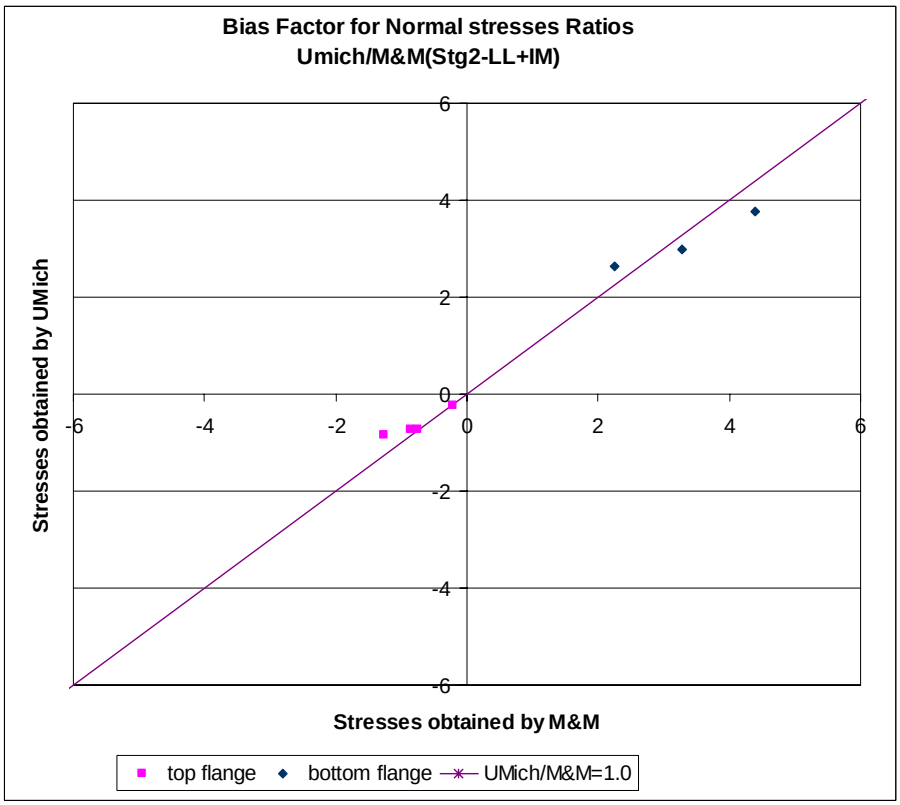


Figure C-27. Bridge B: Ratios of Stresses obtained by UMich and by M&M (Stg2 LL+IM)

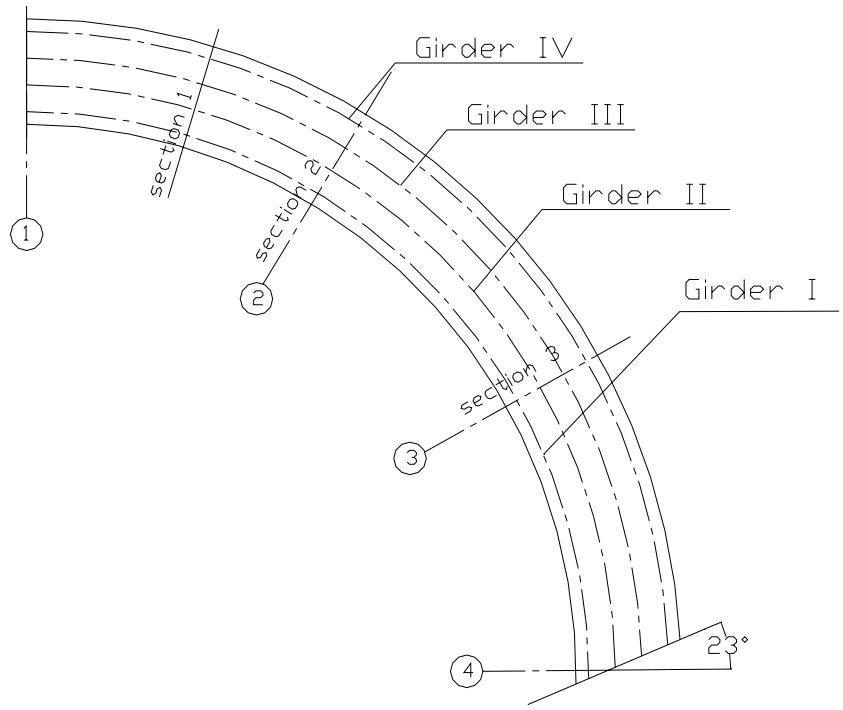


Figure C-28. Plan of Bridge C

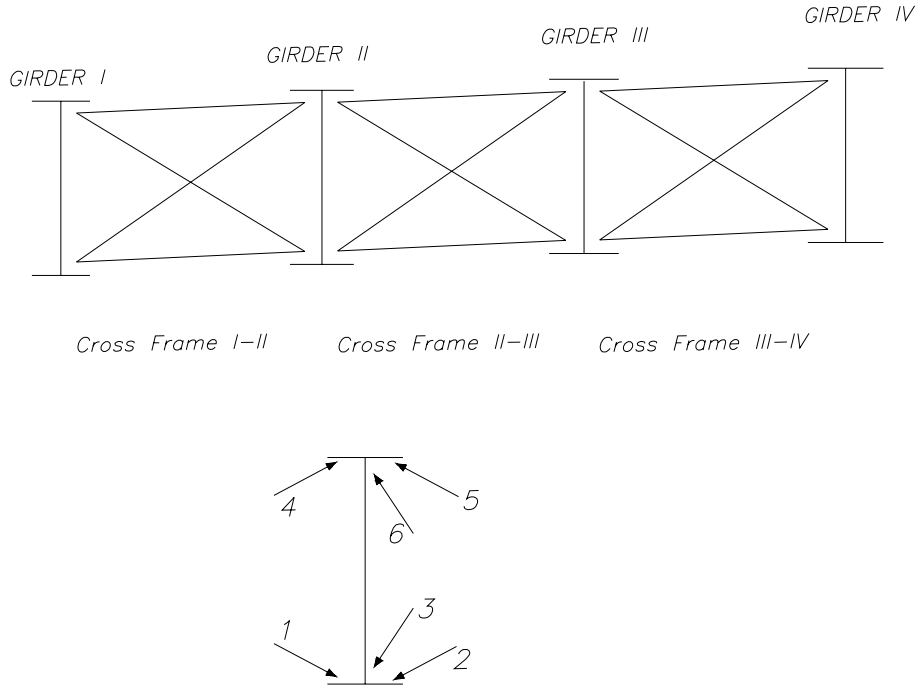


Figure C-29. Cross Section of Bridge C and Location of Points of Interest for Stress Analysis

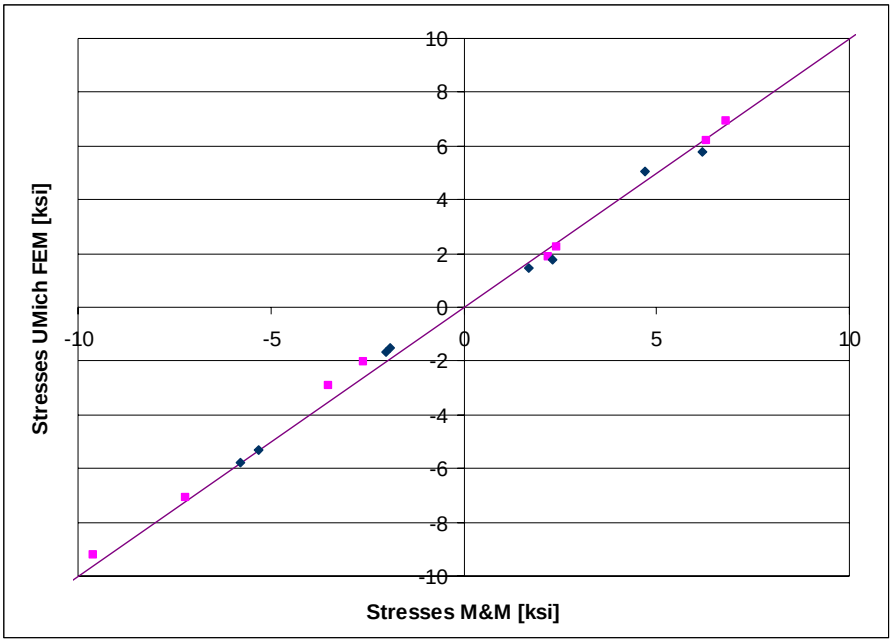


Figure C-30. Bridge C: Ratios of Stresses obtained by UMich FEM and by M&M (Stg1-DL and Stg6-DL)

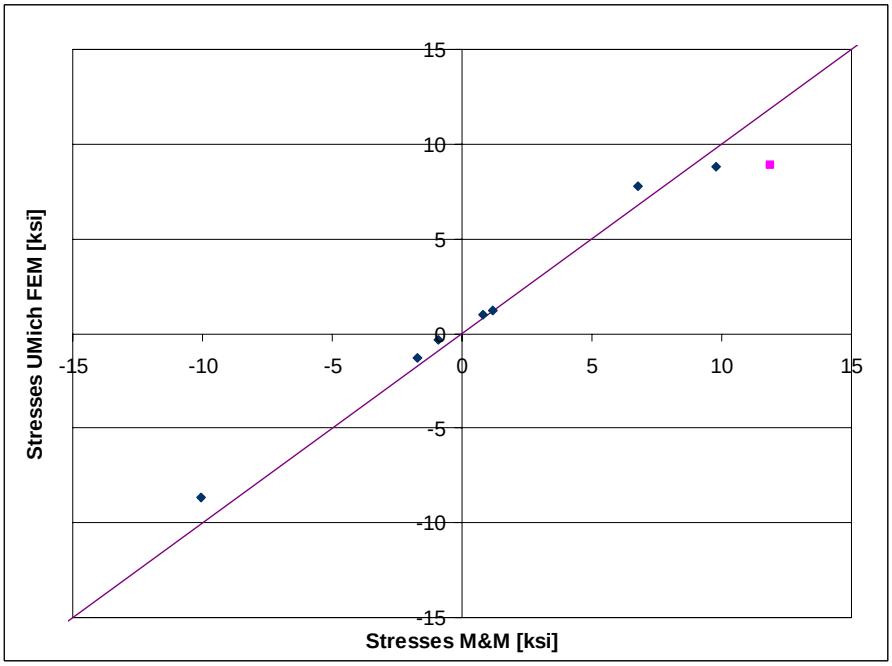


Figure C-31. Bridge C: Ratios of Stresses obtained by UMich FEM and by M&M (Stg7-DL and Stg-LL+IM)

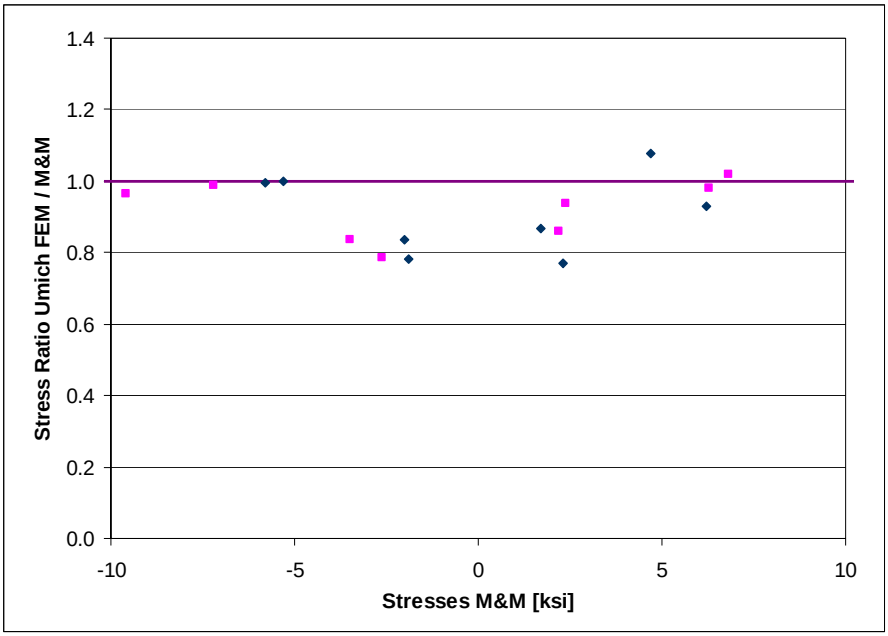


Figure C-32. Bridge C: Bias Factor the Ratio of Stresses obtained by UMich FEM / M&M plotted vs. M&M Stresses (Stg1-DL and Stg6-DL)

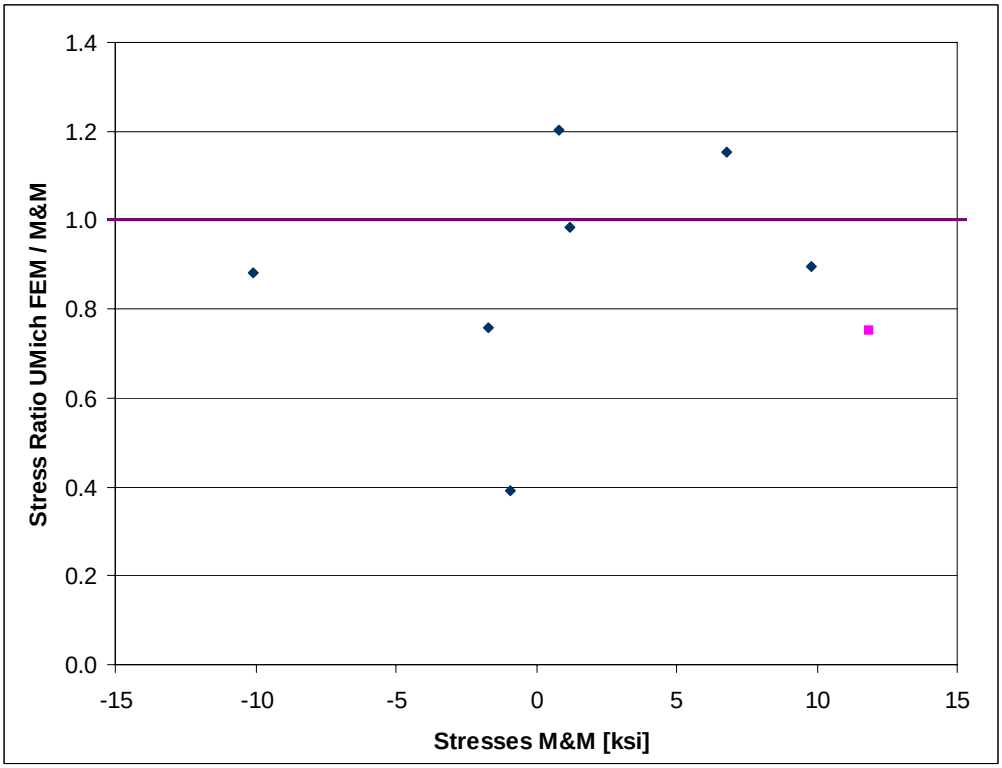


Figure C-33. Bridge C: Bias Factor the Ratio of Stresses obtained by UMich FEM / M&M plotted vs. M&M Stresses (Stg7-DL and Stg-LL+IM)

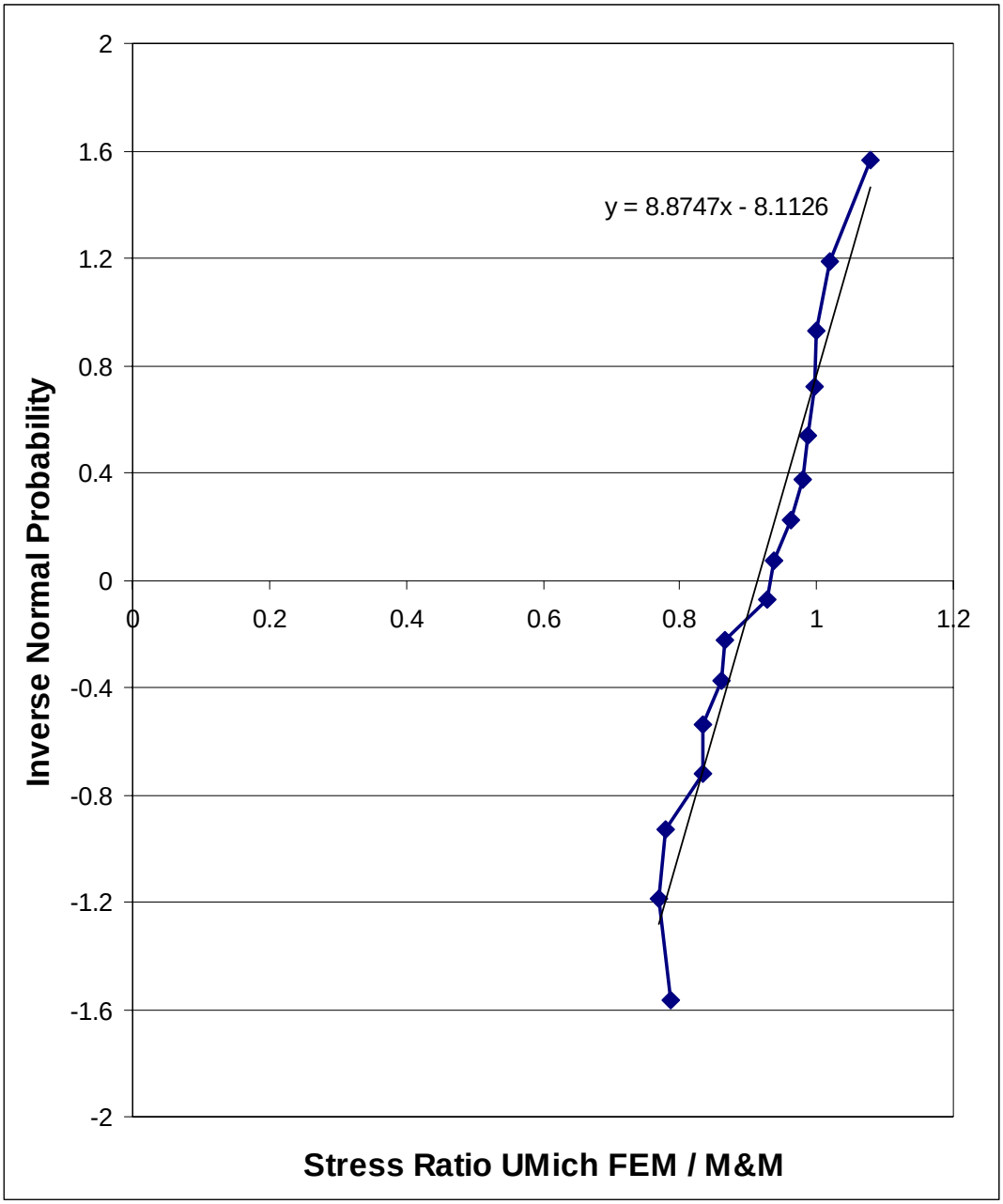


Figure C-34. Bridge C: Normal Distribution Estimation of the CDF of Bias Factor of Normal Stresses Ratio of UMich FEM / M&M (Stg1-DL and Stg6-DL)

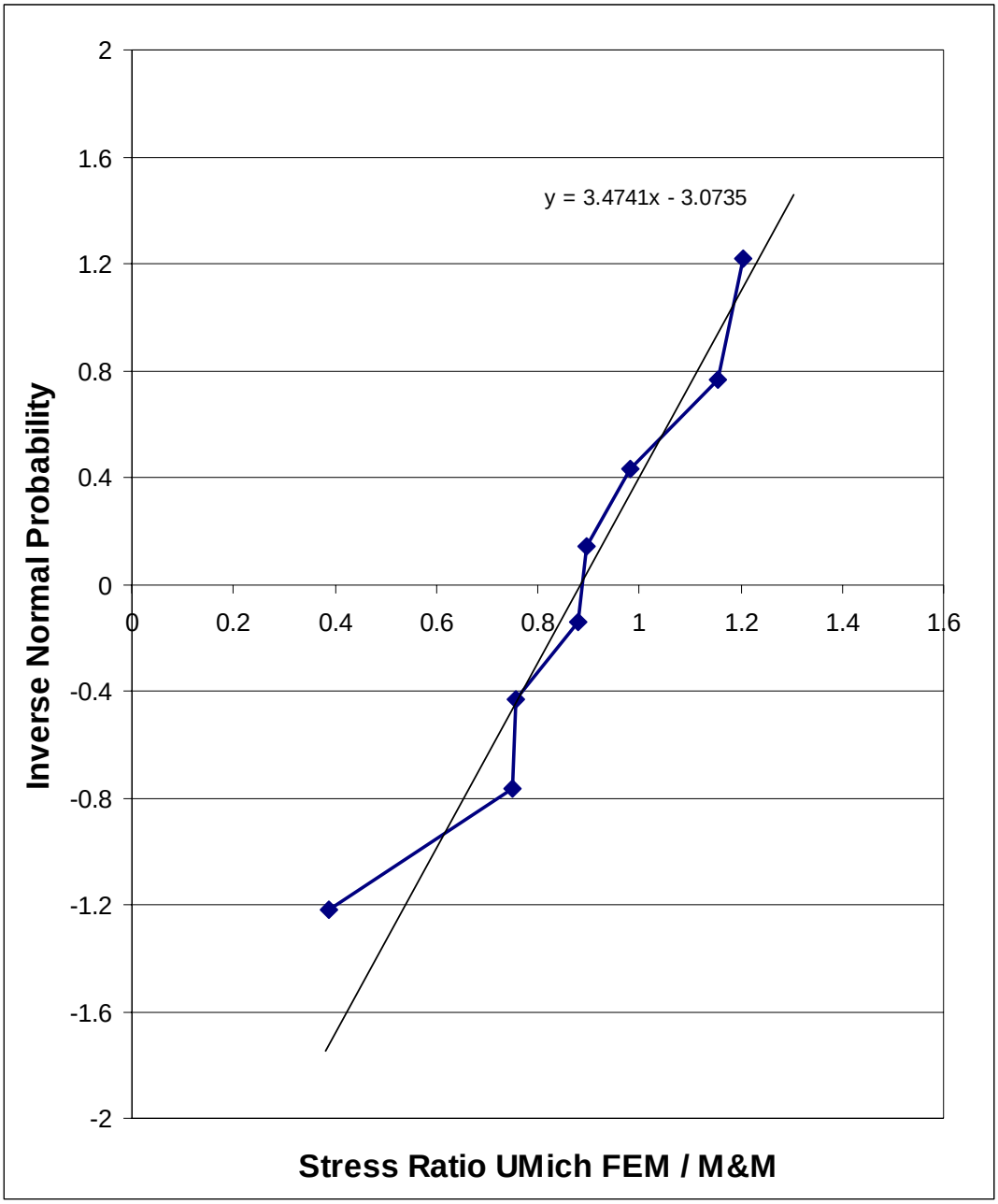


Figure C-35. Bridge C: Normal Distribution Estimation of the CDF of Bias Factor of the Normal Stresses Ratio UMich FEM / M&M (Stg7-DL and Stg-LL+IM)

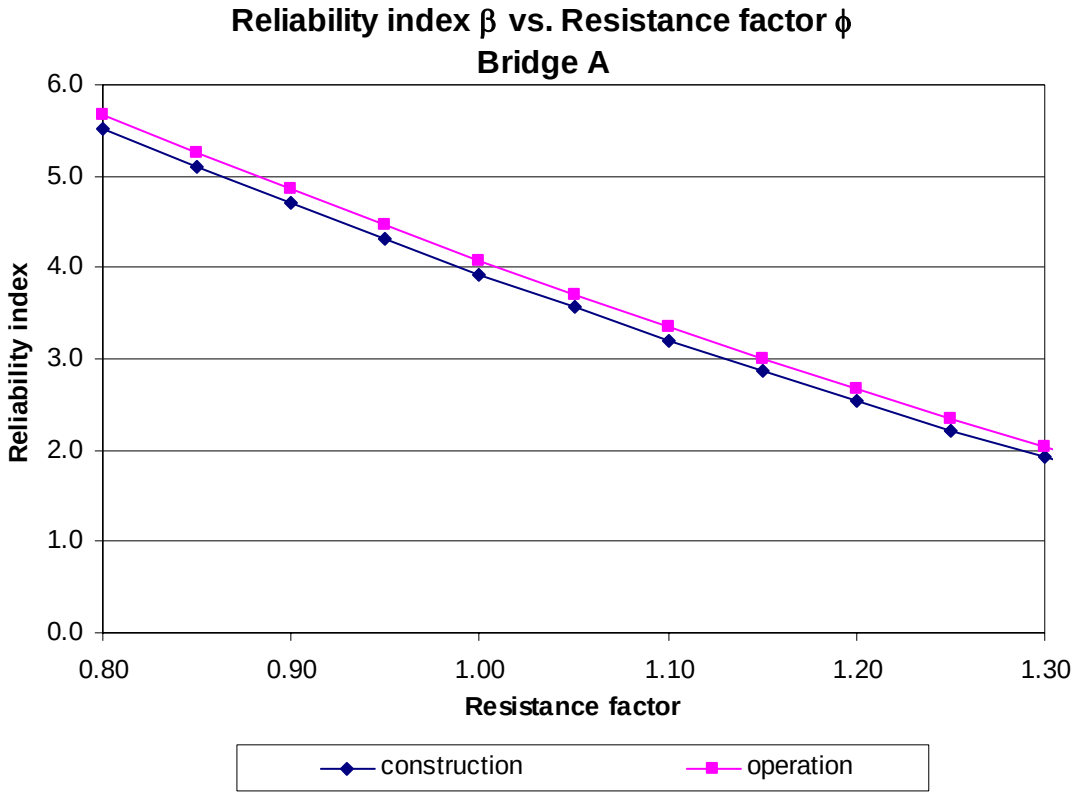


Figure C-36. Reliability Index as a Function of Resistance Factor for Bridge A.

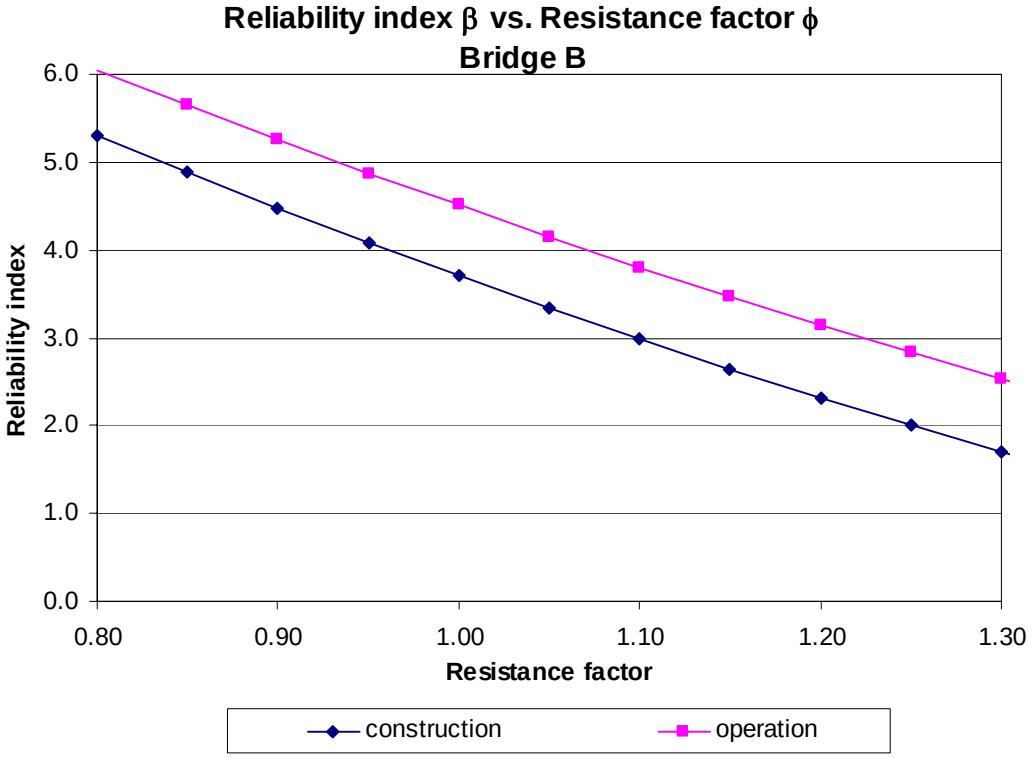


Figure C-37. Reliability Index as a Function of Resistance Factor for Bridge B.

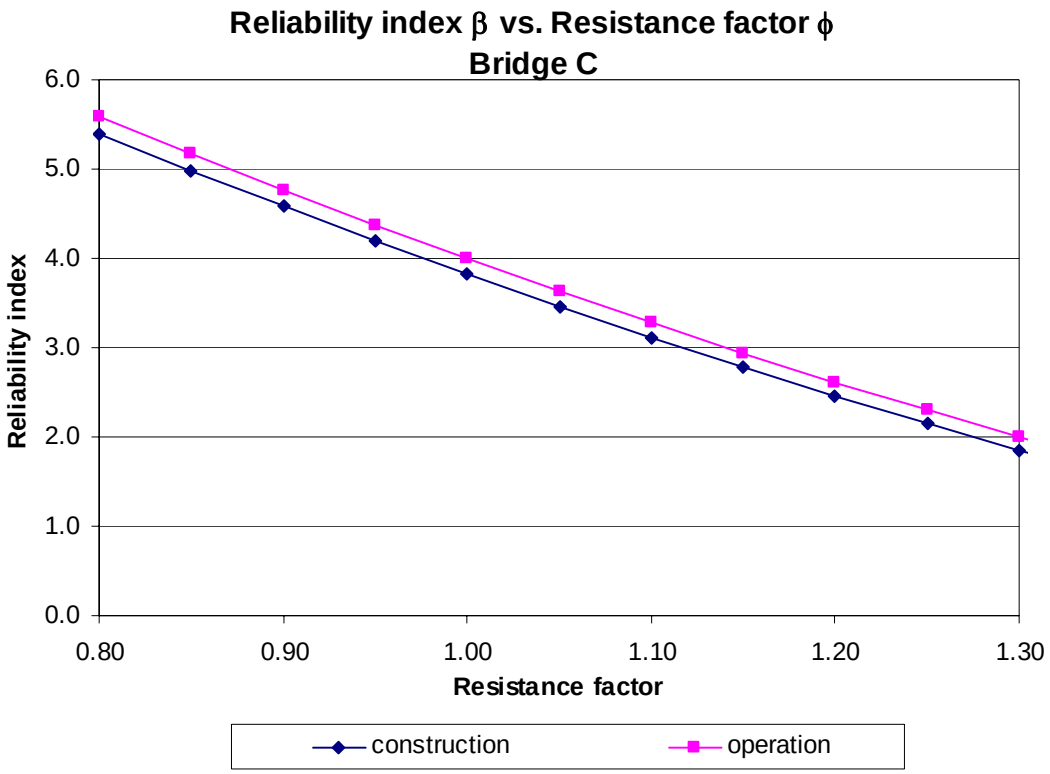


Figure C-38. Reliability Index as a Function of Resistance Factor for Bridge C.

C11 TABLES

Table C-1. Statistical Parameters of Dead Load

Category of component	Bias factor	Coefficient of variation
Factory-made (precast)	1.03	0.08
Cast-in-place	1.05	0.10
Asphalt surface	1.00*	0.25

* mean thickness equal to 3.5 in (90 mm)

Table C-2. Load Ratios Considered in this Study for Bridge A.

Stage.	Spans	D_1	D_2	D_3	LL	IL
operation	150 ft	4	9.5	0	4.5	0.75
construction	150 ft	4	7.5	0	0.25	0

Table C-3. Bridge A: Comparison of Stresses due to Dead Load: Strain Gage Line A, Girders I, II, III, and IV

Point	**UMinn** [ksi]	**UMich** [ksi]	**Test** [ksi]	**Test / UMinn**	**Test / UMich**	**UMich / UMinn**
AI-1	5.67	5.39	3.20	0.564	0.594	0.951
AI-2	3.71	5.22	1.39	0.375	0.266	1.407
AI-3	3.88	4.93	2.49	0.642	0.505	1.271
AI-4	-6.10	-6.97	-4.88	0.800	0.700	1.143
AI-5	-7.82	-9.03	-5.07	0.648	0.561	1.155
AI-6	-6.00	-5.54	-5.83	0.972	1.052	0.923
AII-1	6.66	5.19	2.44	0.366	0.470	0.779
AII-2	4.86	5.86	5.50	1.132	0.939	1.206
AII-3	4.91	5.18	0.81	0.165	0.156	1.055
AII-4	-7.07	-7.52	-2.47	0.349	0.328	1.064
AII-5	-9.34	-8.72	-7.06	0.756	0.810	0.934
AII-6	-6.51	-6.63	-1.79	0.275	0.270	1.018
AIII-1	7.42	3.89	1.42	0.191	0.365	0.524
AIII-2	5.71	7.13	6.07	1.063	0.851	1.249
AIII-3	5.68	5.33	2.82	0.496	0.529	0.938
AIII-4	-8.18	-7.80	-6.02	0.736	0.772	0.954
AIII-5	-10.79	-6.84	-5.83	0.540	0.852	0.634
AIII-6	-7.33	-8.92	-1.79	0.244	0.201	1.217
AIV-1	8.12	5.17	5.27	0.649	1.019	0.637
AIV-2	6.55	6.07	3.81	0.582	0.628	0.927
AIV-3	6.41	5.38	5.57	0.869	1.035	0.839
AIV-4	-10.26	-8.96	-7.00	0.682	0.781	0.873
AIV-5	-13.48	-10.03	-9.23	0.685	0.920	0.744
AIV-6	-5.71	-8.67	-2.64	0.462	0.304	1.518
			Mean=	**0.594**	**0.621**	**0.998**
			StDev=	**0.264**	**0.283**	**0.245**
			V=	**0.445**	**0.456**	**0.246**

Table C-4. Bridge A: Comparison of Stress, Dead Load: Strain Gage Line B, Girders I, II, III, and IV

Point	**UMinn** [ksi]	**UMich** [ksi]	**Test** [ksi]	**Test / UMinn**	**Test / UMich**	**UMich / UMinn**
BI-1	-10.23	-9.31	-8.63	0.844	0.927	0.910
BI-2	-9.92	-11.78	-3.08	0.310	0.261	1.188
BI-3	-8.68	-8.44	-7.31	0.842	0.866	0.972
BI-4	8.36	7.62	7.82	0.935	1.026	0.911
BI-5	9.70	11.15	11.01	1.135	0.987	1.149
BI-6	9.80	9.31	7.75	0.791	0.832	0.950
BII-1	-9.79	-9.14	-4.86	0.496	0.532	0.934
BII-2	-10.97	-11.35	-6.19	0.564	0.545	1.035
BII-3	-9.08	-8.86	-9.10	1.002	1.027	0.976
BII-4	8.97	8.69	7.02	0.783	0.808	0.969
BII-5	9.64	9.93	10.19	1.057	1.026	1.030
BII-6	10.90	9.95	7.77	0.713	0.781	0.913
BIII-1	-10.55	-9.97	-4.18	0.396	0.419	0.945
BIII-2	-11.08	-10.77	-5.78	0.522	0.537	0.972
BIII-3	-9.60	-9.00	-8.42	0.877	0.936	0.938
BIII-4	9.62	8.85	5.88	0.611	0.664	0.920
BIII-5	6.53	9.82	5.93	0.908	0.604	1.504
BIII-6	11.24	10.04	7.87	0.700	0.784	0.893
BIV-1	-9.14	-8.62	-5.95	0.651	0.690	0.943
BIV-2	-11.38	-11.31	-7.70	0.677	0.681	0.994
BIV-3	-9.23	-8.91	-7.11	0.770	0.798	0.965
BIV-4	9.50	8.75	10.12	1.065	1.157	0.921
BIV-5	9.34	9.25	7.38	0.790	0.798	0.990
BIV-6	11.73	10.03	10.70	0.912	1.067	0.855
			Mean=	**0.765**	**0.781**	**0.991**
			StDev=	**0.212**	**0.223**	**0.132**
			V=	**0.277**	**0.285**	**0.133**

Table C-5. Bridge A: Comparison of Stresses due to Dead Load, Span 1, Strain Gage Line C, Cross Frames

Point	UMinn [ksi]	UMich [ksi]	Test [ksi]
C-1	0.56	1.14	-1.59
C-2	-3.46	-3.24	-1.75
C-3	3.38	3.09	2.95
C-4	-0.64	-1.28	-0.73
C-5	-2.11	-1.27	-4.14
C-6	-4.65	-5.02	-4.47
C-7	4.87	5.16	2.54
C-8	2.40	1.83	6.44
C-9	-4.21	-3.52	-8.56
C-10	-4.33	-4.45	1.35
C-11	4.74	4.85	1.50
C-12	4.88	4.77	11.33

Table C-6. Bridge A: Comparison of Stresses due to Dead Load: Strain Gage Lines A and B – without Outliers

Point	UMinn [ksi]	UMich [ksi]	Test [ksi]	$\frac{\text{Test}}{\text{UMinn}}$	$\frac{\text{Test}}{\text{UMich}}$	$\frac{\text{UMich}}{\text{UMinn}}$
			Mean=	0.702	0.720	0.988
			StDev=	0.248	0.256	0.121
			V=	0.353	0.356	0.122

* Outliers are: AI-2, AIII-1, AIII-5, AIV-1, AIV-6, and BIII-5

Table C-7. Bridge A: Comparison of Stresses due to Live Load: Case 3, Strain Gage Lines A and B

Point	UMich [ksi]	Test [ksi]	M&M [ksi]	Test / UMich	Test / M & M	UMich / M & M
AI-1	0.57	0.65	0.64	1.140	1.016	0.891
AI-2	0.45	0.40	0.64	0.889	0.625	0.703
AI-3	0.51	0.61	0.59	1.196	1.034	0.864
AII-1	0.67	0.60	0.78	0.896	0.769	0.859
AII-2	0.68	0.84	0.78	1.235	1.077	0.872
AII-3	0.69	0.69	0.73	1.000	0.945	0.945
AIII-1	0.75	0.85	1.04	1.133	0.817	0.721
AIII-2	1.01	0.79	1.04	0.782	0.760	0.971
AIII-3	0.90	0.85	0.99	0.944	0.859	0.909
AIV-1	1.13	1.13	1.39	1.000	0.813	0.813
AIV-2	1.05	0.96	1.39	0.914	0.691	0.755
AIV-3	1.07	0.86	1.33	0.804	0.647	0.805
BI-1	-0.53	-0.70	-0.72	1.321	0.972	0.736
BI-2	-0.53	-0.39	-0.72	0.736	0.542	0.736
BI-3	-0.46	-0.55	-0.67	1.196	0.821	0.687
BII-1	-0.71	-0.53	-1.07	0.746	0.495	0.664
BII-2	-0.62	-0.65	-1.07	1.048	0.607	0.579
BII-3	-0.59	-0.66	-1.01	1.119	0.653	0.584
BIII-1	-0.84	-0.62	-0.95	0.738	0.653	0.884
BIII-2	-0.73	-0.56	-0.95	0.767	0.589	0.768
BIII-3	-0.69	-0.56	-0.90	0.812	0.622	0.767
BIV-1	-0.89	-0.44	-0.61	0.494	0.721	1.459
BIV-2	-0.86	-0.66	-0.61	0.767	1.082	1.410
BIV-3	-0.80	-0.47	-0.57	0.588	0.825	1.404
			Mean=	0.928	0.776	0.866
			StDev=	0.214	0.173	0.239
			V=	0.231	0.223	0.276

Table C-8. Bridge A: Comparison of Stresses due to Live Load: Case 7, Strain Gage Lines A and B

Point	UMich [ksi]	Test [ksi]	M&M [ksi]	Test / UMich	Test / M & M	UMich / M & M
AI-1	0.33	0.37	0.33	1.121	1.121	1.000
AI-2	0.23	0.17	0.33	0.739	0.515	0.697
AI-3	0.28	0.23	0.31	0.821	0.742	0.903
AII-1	0.41	0.43	0.58	1.049	0.741	0.707
AII-2	0.36	0.50	0.58	1.389	0.862	0.621
AII-3	0.39	0.50	0.55	1.282	0.909	0.709
AIII-1	0.39	0.63	0.74	1.615	0.851	0.527
AIII-2	0.51	0.60	0.74	1.176	0.811	0.689
AIII-3	0.46	0.65	0.70	1.413	0.929	0.657
AIV-1	0.51	0.53	0.66	1.039	0.803	0.773
AIV-2	0.49	0.56	0.66	1.143	0.848	0.742
AIV-3	0.49	0.38	0.63	0.776	0.603	0.778
BI-1	-0.54	-0.61	-0.68	1.130	0.897	0.794
BI-2	-0.56	-0.50	-0.68	0.893	0.735	0.824
BI-3	-0.49	-0.60	-0.64	1.224	0.938	0.766
BII-1	-0.55	-0.38	-0.78	0.691	0.487	0.705
BII-2	-0.64	-0.58	-0.78	0.906	0.744	0.821
BII-3	-0.53	-0.56	-0.73	1.057	0.767	0.726
BIII-1	-0.57	-0.55	-0.81	0.965	0.679	0.704
BIII-2	-0.65	-0.50	-0.81	0.769	0.617	0.802
BIII-3	-0.55	-0.53	-0.77	0.964	0.688	0.714
BIV-1	-0.53	-0.56	-0.81	1.057	0.691	0.654
BIV-2	-0.65	-0.75	-0.81	1.154	0.926	0.802
BIV-3	-0.55	-0.59	-0.77	1.073	0.766	0.714
			Mean=	1.060	0.778	0.743
			StDev=	0.227	0.144	0.095
			V=	0.214	0.186	0.127

Table C-9. Bridge A: Comparison of Stresses due to Live Load: Cases 3 and 7, Strain Gage Lines A and B – without Outliers

Point	UMich [ksi]	Test [ksi]	M&M [ksi]	Test / UMich	Test / M & M	UMich / M & M
			Mean=	1.002	0.775	0.777
			StDev=	0.196	0.158	0.091
			V=	0.196	0.204	0.117

* Outliers are: AIII-1, BII-2, and BII-3

Table C-10. Bridge A, Statistical Parameters of the Bias Factor Dead Load Non-composite Loading Stage

	UMinn test / UMich FEM	UMinn test / UMinn	UMich FEM / UMinn
Mean=	0.70	0.72	0.99
StDev=	0.25	0.26	0.12
V=	0.35	0.36	0.12

Table C-11. Bridge A, Statistical Parameters of the Bias Factor Live Load Composite Loading Stage

	UMinnTest / UMich FEM	UMinn Test / M & M	UMich FEM / M & M
Mean=	1.00	0.78	0.78
StDev=	0.20	0.16	0.09
V=	0.20	0.20	0.12

Table C-12. Bridge B: Stresses – Stg1 – DL, Girders (self-weight of girders plus stiffeners and diaphragms)

Point of interest	UMich FEM [ksi]	M&M BSDI [ksi]
AI-1	2.85	-
AI-2	2.54	-
AI-3	3.18	4.04
mean	2.86	4.04
AI-4	-3.87	-
AI-5	-3.60	-
AI-6	-3.20	-4.60
mean	-3.56	-4.60
AII-1	1.95	-
AII-2	1.74	-
AII-3	2.25	2.64
mean	1.98	2.64
AII-4	-2.85	-
AII-5	-2.60	-
AII-6	-2.35	-3.01
mean	-2.60	-3.01
AIII-1	0.95	-
AIII-2	0.90	-
AIII-3	1.15	1.17
mean	1.00	1.17
AIII-4	-1.35	-
AIII-5	-1.80	-
AIII-6	-0.99	-1.83
mean	-1.38	-1.83
AVI-1	0.85	-
AIV-2	0.76	-
AIV-3	1.10	1.16
mean	0.90	1.16
AIV-4	-1.02	-
AIV-5	-1.25	-
AIV-6	-0.95	-1.31
mean	-1.07	-1.31

Table C-13. Bridge B: Stresses – Stg2 – LL+IM, Composite structure loaded with two test trucks side-by-side

Point of interest	UMich FEM [ksi]	M&M BSDI [ksi]
AI-1	3.90	-
AI-2	4.59	-
AI-3	5.08	6.45
mean	4.52	6.45
AI-4	-0.77	-
AI-5	-0.71	-
AI-6	-0.71	-0.86
mean	-0.73	-0.86
AII-1	3.27	-
AII-2	3.75	-
AII-3	4.25	4.39
mean	3.76	4.39
AII-4	-0.98	-
AII-5	-0.79	-
AII-6	-0.75	-1.26
mean	-0.84	-1.26
AIII-1	2.67	-
AIII-2	3.25	-
AIII-3	3.03	3.27
mean	2.98	3.27
AIII-4	-0.75	-
AIII-5	-0.73	-
AIII-6	-0.69	-0.75
mean	-0.72	-0.75
AVI-1	2.11	2.25
AIV-2	2.98	2.25
AIV-3	2.80	2.25
mean	2.63	2.25
AIV-4	-0.22	-
AIV-5	-0.26	-
AIV-6	-0.20	-0.22
mean	-0.23	-0.22

Table C-14. Bridge B: Stg1-DL: bottom flange

Point of interest	UMich FEM [ksi]	M&M [ksi]	UMich FEM / M&M
AI-1-2-3	2.86	4.035	0.709
AII-1-2-3	1.98	2.637	0.751
AIII-1-2-3	1.00	1.166	0.858
AVI-1-2-3	0.90	1.162	0.775
		Mean=	**0.773**
		StDev=	**0.071**
		***V*=**	**0.092**

Table C-15. Bridge B: Stg1-DL: top flange

Point of interest	UMich FEM [ksi]	M&M [ksi]	UMich FEM / M&M
AI-4-5-6	-3.56	-4.6	0.774
AII-4-5-6	-2.60	-3.007	0.865
AIII-4-5-6	-1.38	-1.833	0.753
AIV-4-5-6	-1.07	-1.313	0.815
		Mean=	**0.802**
		StDev=	**0.094**
		***V*=**	**0.118**

Table C-16. Bridge B: Stg2-LL+IM: bottom flange

Point of interest	UMich FEM [ksi]	M&M [ksi]	UMich FEM / M&M
AI-1-2-3	4.52	6.45	0.701
AII-1-2-3	3.76	4.39	0.856
AIII-1-2-3	2.98	3.27	0.911
AVI-1-2-3	2.63	2.25	1.196
		Mean=	**0.909**
		StDev=	**0.341**
		***V*=**	**0.375**

Table C-17. Bridge B: Stg2-LL+IM: top flange

Point of interest	UMich FEM [ksi]	M&M [ksi]	UMich FEM / M&M
AI-4-5-6	-0.73	-0.86	0.849
AII-4-5-6	-0.84	-1.26	0.667
AIII-4-5-6	-0.72	-0.75	0.960
AIV-4-5-6	-0.23	-0.22	1.045
		Mean=	**0.880**
		StDev=	**0.346**
		***V*=**	**0.393**

Table C-18. Bridge B: Statistical Parameters of the Bias Factor for Constructional Loading Stage (Stg1 – DL)

	UMich / M&M
Mean	0.790
StDev	0.094
V	0.105

Table C-19. Bridge B: Statistical Parameters of the Bias Factor for Operational Loading Stage (Stg2 – LL+IM)

	UMich / M&M
Mean	0.895
StDev	0.251
V	0.225

Table C-20. Load Ratios Considered in this Study for bridge B

Stage	D_1	D_2	D_3	LL	IL
DL	2	3.7	0	0.3	0
LL+IM	2	4.5	0	5	0.5

Table C-21. The comparison between the M&M and the UMich FEM results from structural analysis expressed in terms of average normal stresses ratios (loading stages Stg1-DL and Stg6-DL).

Location	Top Flange						Bottom Flange					
Point of interest	Stg1-DL			Stg6-DL			Stg1-DL			Stg6-DL		
	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M
	(ksi)	(ksi)		(ksi)	(ksi)		(ksi)	(ksi)		(ksi)	(ksi)	
POI 1	-	-2.58		-	-8.82		-	1.79		-	6.17	
	-	-1.65		-	-5.75		-	1.2		-	4.11	
	-2.6	-1.91		-7.2	-6.74		1.7	1.43		4.7	4.94	
Mean:	-2.6	-2.05	0.787	-7.2	-7.10	0.987	1.7	1.47	0.867	4.7	5.07	1.079
POI 2	-	-3.65		-	-11.08		-	2.09		-	6.8	
	-	-2.4		-	-7.83		-	1.48		-	4.84	
	-3.5	-2.72		-9.6	-8.82		2.3	1.74		6.2	5.64	
Mean:	-3.5	-2.92	0.835	-9.6	-9.24	0.963	2.3	1.77	0.770	6.2	5.76	0.929
POI 3	2.2	1.9	0.86	6.3	6.2	0.98	-1.9	-1.49	0.78	-5.3	-5.31	1.00
POI 4	-	1.79		-	5.84		-	-1.46		-	-5.03	
	-	2.71		-	7.21		-	-1.65		-	-5.66	
	2.4	2.25		6.8	7.7		-2.0	-1.9		-5.8	-6.64	
Mean:	2.4	2.25	0.938	6.8	6.92	1.02	-2.0	-1.67	0.835	-5.8	-5.78	0.996
		Mean	**0.855**		**Mean**	**0.988**		**Mean**	**0.813**		**Mean**	**1.001**

*stresses sign convention: plus sign in tension

Table C-22. The comparison between the M&M and the UMich FEM results from structural analysis expressed in terms of average normal stresses ratios (loading stages Stg7-DL and Stg-LL+IM)

Location	Top Flange						Bottom Flange					
Point of interest	Stg7-DL			Stg-LL+IM			Stg7-DL			Stg-LL+IM		
	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M	M&M	UMich FEM	UMich FEM/M&M
	(ksi)	(ksi)		(ksi)	(ksi)		(ksi)	(ksi)		(ksi)	(ksi)	
POI 1	-	0.06		-2.03	0.2		-	0.78		-	6.4	
	-	0.04		-2.03	0.3		-	0.96		-	7.8	
	-0.6	0.08		-2.03	0.4		0.8	1.15		6.76	9.2	
Mean:			N/A			N/A	0.8	0.96	1.204	6.76	7.80	1.154
POI 2	-	0.05		-2.73	0.7		-	1		-	7.6	0.77
	-	0.09		-2.73	0.7		-	1.2		-	8.8	0.9
	-0.8	0.13		-2.73	0.7		1.2	1.34		9.81	10	1.02
Mean:			N/A			N/A	1.2	1.18	0.983	9.81	8.80	0.897
POI 3	-	0		-	-	-	-	-0.4		-	-	-
	-	-0.07		-	-	-	-	-0.35		-	-	-
	1.1	-0.1		9.57	-	-	-0.9	-0.3		-8.11	-	-
Mean:	-		N/A	9.57	-	-	-0.9	-0.35	0.39	-8.11	-	-
POI 4		-0.05		-	9.3	0.78	-	-1.36		-	-9.2	0.91
	-	-0.1		-	8.9	0.75	-	-1.3		-	-8.9	0.88
	2.0	-0.15		11.87	8.5	0.72	-1.7	-1.2		-10.07	-8.5	0.84
Mean:			N/A	11.87	8.9	0.75	-1.7	-1.29	0.757	-10.07	-8.67	0.881
		Mean	**-**		**Mean**	**0.75**		**Mean**	**0.834**		**Mean**	**0.977**

*stresses sign convention: plus sign in tension

Table C-23. Bridge C: Statistical Parameters of the Bias Factor for Constructional Loading Stages (Stg1-DL and Stg6-DL)

	UMich FEM / M&M
Mean	0.91
StDev	0.11
V	0.125

Table C-24. Bridge C: Statistical Parameters of the Bias Factor for Operational Loading Stages (DL-Stg7 and Stg-LL+IM)

	UMichFEM / M&M
Mean	0.89
StDev	0.24
V	0.265

Table C-25. Load Ratios Considered in this Study for Bridge C

Stage.	Spans	D_1	D_2	D_3	LL	IL
operation	92 ft	4	18.9	0	5.6	1.8
construction	92 ft	4	13.6	0	0.4	0

Table C-26. Reliability Indices for Various Values of ϕ for the Considered Bridges

Resistance factor, ϕ	**Reliability index, β**					
	Construction			**Operation**		
	Bridge A	**Bridge B**	**Bridge C**	**Bridge A**	**Bridge B**	**Bridge C**
0.80	5.52	5.31	5.40	5.67	6.04	5.59
0.85	5.10	4.89	4.98	5.26	5.64	5.17
0.90	4.70	4.48	4.58	4.85	5.25	4.77
0.95	4.31	4.08	4.19	4.46	4.87	4.38
1.00	3.93	3.70	3.82	4.08	4.51	4.00
1.05	3.56	3.34	3.46	3.71	4.15	3.64
1.10	3.21	2.98	3.11	3.35	3.80	3.28
1.15	2.87	2.65	2.78	3.00	3.47	2.94
1.20	2.54	2.32	2.46	2.67	3.15	2.61
1.25	2.22	2.01	2.15	2.35	2.83	2.30
1.30	1.92	1.71	1.86	2.03	2.53	1.99
1.35	1.63	1.42	1.58	1.73	2.24	1.70
1.40	1.35	1.14	1.31	1.44	1.96	1.42
1.45	1.08	0.88	1.05	1.17	1.69	1.15
1.50	0.82	0.63	0.80	0.90	1.43	0.89

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ATTACHMENT A

NOTES ON CURVED GIRDER BRIDGE CALIBRATION LOAD EFFECT CONSIDERATION

Prepared by Theodore V. Galambos, University of Minnesota

RE-EXAMINATION OF THE RESISTANCE FACTOR ϕ

Assume: Point of calibration: $L/D = 3.0$, as in AISC LRFD

1. Simple approaches.

a) Load statistics

Dead load: $\bar{D} = 1.05\, D_n$; $V_D = 0.10$
mean COV

Live load: $\bar{L} = L_n$; $V_L = 0.25$

b) Yield stress

Mill test results: $\bar{F}_{ym} = 56.47$ ksi
$V_{F_{ym}} = 0.0656$
$n = 28,318$

Note: some of these data were from coupons taken from flanges, and some are from webs.

This needs to be cleared up | all from flanges |

Adjustments for static F_y: 4 ksi reduction
(TVG, ASCE ST9, Sep. 78)

Note: is this true for the modern steels?

Our testing program will need to look into this effect

$F_{yn} = 50$ ksi; $\dfrac{F_{ym} - 4}{F_{yn}} = \dfrac{56.5 - 4}{50} = 1.05 = \bar{M}$

$V_{F_y} = 0.066 = V_M$

c) Fabrication factor:

$\bar{F} = 1.0$ and $V_F = 0.05$, as previously

d) Professional factor:

Statically determinate beams under uniform moment:

$\bar{P} = 1.02$; $V_P = 0.06$ Table 2 (ASCE ST9, '78, Yura)

e) Resistance statistics:

$\bar{R} = R_n \bar{M}\bar{F}\bar{P}$

$R_n = Z_n F_{yn}$; nominal capacity

$\bar{R} = R_n (1.05 \times 1.00 \times 1.02) = 1.07 R_n$; $V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = \sqrt{0.06^2 + 0.05^2 + 0.0}$

$V_R = 0.10$

f) Nominal resistance

$L_n = 3D_n$; $\frac{L_n}{D_n} = 3.0$

$\phi = 0.9$

$\phi R_n = 1.2D_n + 1.6L_n = D_n\left[1.2 + 1.6\frac{L}{D}\right] = D_n\left[1.2 + 1.6 \times 3\right] = 6D_n$

$R_n = \frac{6D_n}{\phi}$ ← required by AISC-LRFD

g) Load effect

$\bar{Q} = \bar{D} + \bar{L} = 1.05D_n + L_n = D_n\left[1.05 + \frac{L_n}{D_n}\right] = D_n\left[1.05 + 3\right] = 4.05D_n$

$\sigma_Q^2 = \sigma_D^2 + \sigma_L^2 = (\bar{D}V_D)^2 + (\bar{L}V_L)^2 = (1.05D_n \times 0.1)^2 + (L_n \times 0.25)^2$

$\sigma_Q^2 = D_n^2\left[(1.05 \times 0.1)^2 + (3 \times 0.25)^2\right]$; $\sigma_Q = 0.757 D_n$

$V_Q = \frac{0.757D_n}{4.05D_n} = 0.19$

h) Assume normal distribution:

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

$$\bar{R} = 1.07 R_n = \frac{1.07 \times 6 D_n}{\phi} = \frac{6.42 D_n}{\phi}$$

$$\sigma_R = \bar{R} V_R = \frac{0.1 \times 6.42 D_n}{\phi} = \frac{0.642 D_n}{\phi}$$

$$\bar{Q} = 4.05 D_n$$

$$\sigma_Q = 0.757 D_n$$

D_n cancels out:

$$\beta = \frac{\frac{6.42}{\phi} - 4.05}{\sqrt{\left(\frac{0.642}{\phi}\right)^2 + 0.757^2}}$$

Solve for β, given $\phi = 0.9$

$\beta = 2.96$

Solve for ϕ, given $\beta = 2.6$ (As in AISC-LRFD spec.)

$\phi = 0.96$

j) Assume lognormal distribution

$$\beta = \frac{\ln \bar{R}/\bar{Q}}{\sqrt{V_R^2 + V_Q^2}} = \frac{\ln\left(\frac{6.42}{4.05\phi}\right)}{\sqrt{0.10^2 + 0.19^2}} ;$$

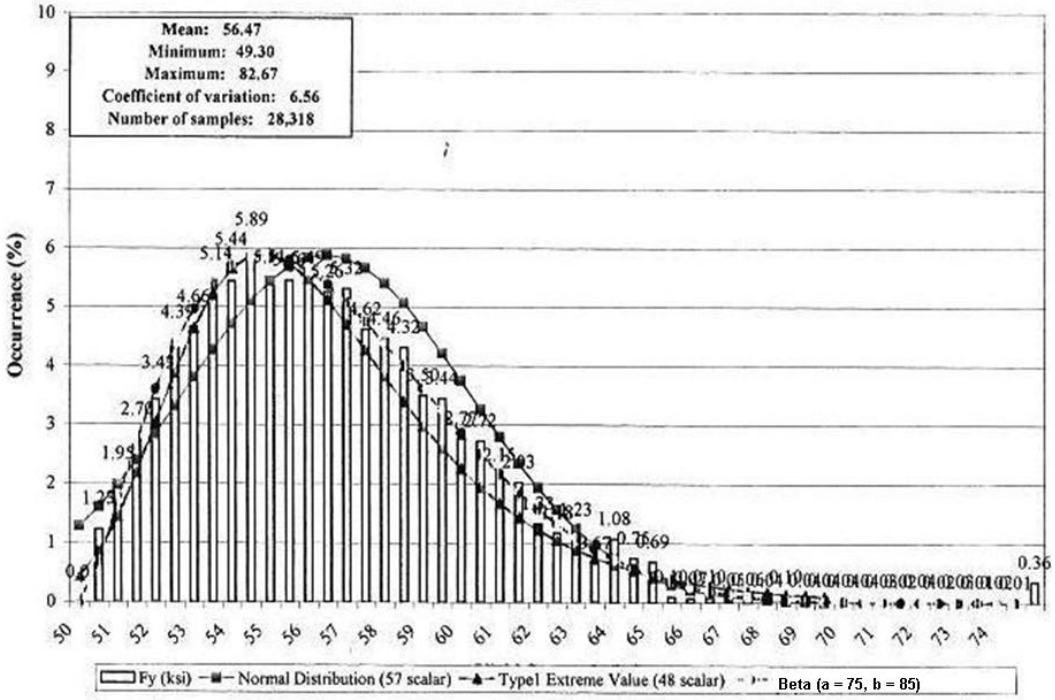
Solve for β, given $\phi = 0.9$: $\beta = 2.64$

Solve for ϕ, given $\beta = 2.6$ $\phi = 0.91$

Reference:
Yura, J.A., Galambos, T.V., and Ravindra, M.K. (1978), "The Bending Resistance of Steel Beams," ASCE J. Struct. Div., Vol.104, No. ST9, pp.1356-1370.

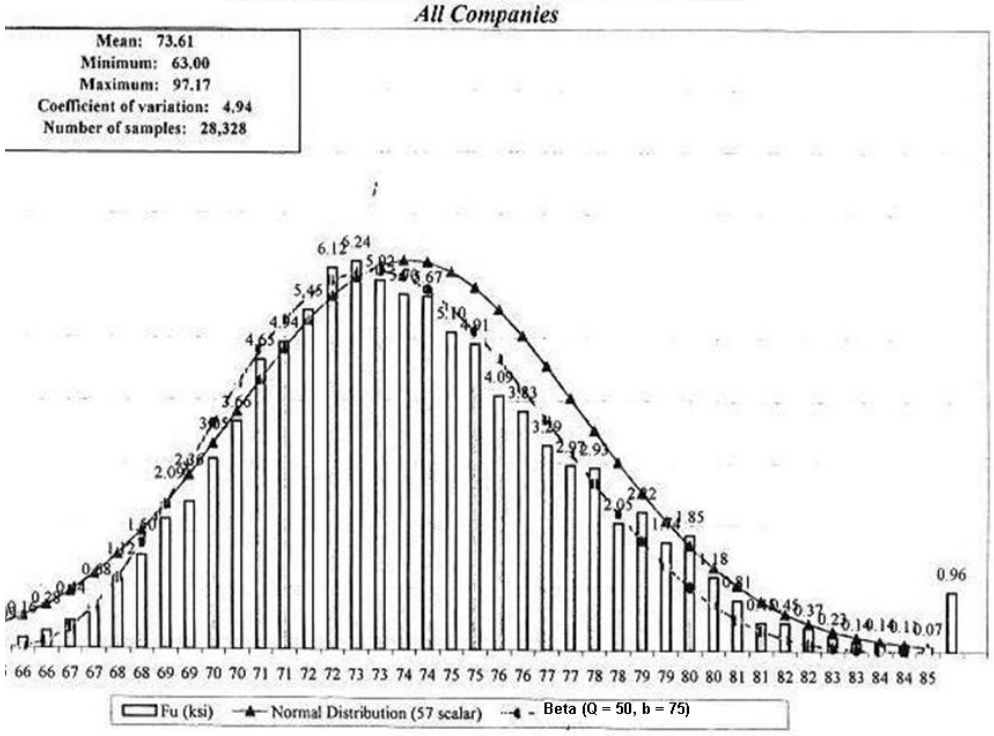
Yield Strength for Specified Grades

All Companies



Ultimate Strength for Specified Grades

All Companies



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ATTACHMENT B

CALIBRATION OF A DESIGN CODE FOR CURVED GIRDER BRIDGES

Prepared by Andrzej S. Nowak, University of Nebraska-Lincoln

INTRODUCTION

This report is prepared to clarify the code calibration procedure that has been used for steel curved girder design in NCHRP Project 12-52. The procedure is consistent with the one recently applied to a new generation of design codes, including AASHTO LRFD (1994, 1998, 2004), ACI 318 (2005), OHBDC (1979, 1983, 1991), and CHBDC (2000).

The older generation of design codes was based on allowable stresses. The design was acceptable if the calculated stress was less than the specified allowable value. In practice, the safety reserve was concentrated in the conservatively low value of the allowable stress. The allowable stress was much lower than the critical/ultimate value; therefore, the linear/elastic analysis was sufficient.

The new generation of codes is based on consideration of limit states. The designer determines the ultimate load carrying capacity of structural components, and the design is acceptable if the factored load does not exceed the factored resistance. In the reliability-based calibration of design codes, the acceptability criterion is closeness to the target reliability level. Therefore, the calibration involves:

- development of reliability analysis procedure (how do we measure reliability?)
- development of limit state functions (what is the limit of acceptable performance formulated in terms of mathematical equations?), this requires the identification of the parameters that determine the structural performance
- development of load and resistance models (what are the statistical parameters of variables that represent load and resistance?)
- selection of the target reliability level
- selection of load and resistance factors that result in structural components with reliability close to the target values.

OBJECTIVE OF CODE CALIBRATION IN NCHRP 12-52

In the AASHTO LRFD Code, the basic design formula is given as:

$$\gamma_D \text{ DL} + \gamma_{DA} \text{ DA} + \gamma_L \text{ (LL + IL)} \leq \phi R \qquad (1)$$

where γ_D is the dead load factor equal to 1.25, γ_{DA} is the dead load factor for the wearing surface equal to 1.5, γ_L is the live load factor equal to 1.75 live load factor, and ϕ is the resistance factor and is equal to 1.0 for steel straight girders.

In NCHRP 12-52, it was assumed that load factors for curved girders are to be the same as for straight girders. Therefore, the objective of the project is to determine the resistance factor for steel curved girders, so that the reliability of these girders would be close to the target level.

CALCULATION OF RELIABILITY

Reliability of structural components can be measured in terms of the reliability index, β. The reliability index can be related to the probability of failure, P_F. There are several procedures for calculation of the reliability index; they are described in text books (e.g. Nowak and Collins 2000). They differ with regard to accuracy, required input data, and computational effort. For consistency, in this study, β is calculated using the same procedure as described in NCHRP Report 368 (Nowak 1999).

The limit state function is a mathematical representation of the acceptability criterion. In the basic design case (Eq. 1), the limit state function g is defined as

$$g = R - (\mathrm{DL} + \mathrm{LL} + \mathrm{IL}) = 0 \tag{2}$$

where R, DL, LL and IL are random variables representing resistance and load components. This is very different from Eq. 1, where these symbols represent nominal values. If g greater than or equal to 0, the structural component is safe, and if g less than 0, the component fails. The boundary between safe and unsafe domain is represented by g equal to 0. This corresponds to the case of total load being equal to resistance.

Probability of failure for the considered component, P_F, is equal to:

$$P_F = \mathrm{P}(g < 0) \tag{3}$$

In this calibration, the probability of failure is calculated indirectly, as a function of the reliability index, β,

$$P_F = \Phi(-\beta) \tag{4}$$

where Φ is the standard normal cumulative distribution function, as shown in Table 1.

The input data are statistical parameters for each load component and resistance:

- Mean value, and bias factor, λ equal to the ratio of the mean-to-nominal
- Coefficient of variation, V equal to the ratio of standard deviation and the mean value
- Type of cumulative distribution function

The load and resistance models are based on the available literature available and additional studies performed especially for this calibration. The total load effect is represented by a single random variable, Q, so that the limit state function is:

$$g = R - Q = 0 \tag{5}$$

where Q is equal to the sum of DL, LL, and IM.

The reliability is calculated in terms of the reliability index, β. The loads are represented by a normal random variable, Q, and resistance is represented by a lognormal random variable, R. The limit state function is given by Eq. 5. The reliability analysis is performed using the Rackwitz-Fiessler procedure (Nowak and Collins 2000), based on approximation of non-normal distributions with normal distributions at the so-called design point with coordinates (R^*, Q^*). The design point satisfies the following limit state equation.

$$R^* - Q^* = 0 \tag{6}$$

so R^* is equal to Q^*, and it is located between the mean values of R and Q. The design point is found by iterations.

The procedure is shown graphically in Figure 1. The cumulative distribution functions (CDF) of resistance, R, and load, Q, are plotted on the normal probability paper. Because Q is a normal random variable, its CDF is represented by a straight line. The horizontal line corresponding to zero on the vertical scale intersects with CDF at the mean value of Q. R is a lognormal random variable, therefore, its CDF is not a straight line. However, because of relatively low value of the coefficient of variation ($V << 0.2$), the curvature of CDF is also low.

The analytical procedure involves iterations, starting with guessing the initial value of the design point, shown in Figure 1. Then, a lognormal CDF of R is approximated by a normal CDF at the design point. The approximating normal CDF is a straight line that is tangent to the lognormal CDF, shown in Figure 1. The mean and standard deviation of the approximating normal CDF can be read directly from the graph. The intermediate value of the reliability index can be calculated using the following formula,

$$\beta = \frac{m'_R - m_Q}{\sqrt{\sigma'^{\,2}_R + \sigma_Q^{\,2}}} \tag{7}$$

where m'_R is the mean of the approximating normal CDF of R, and σ'_R is the standard deviation of the approximating normal CDF of R. The new design point is found,

$$R^* = Q^* = m'_R - \frac{\sigma'^{\,2}_R \, \beta}{\sqrt{\sigma'^{\,2}_R + \sigma_Q^{\,2}}} \tag{8}$$

and iterations are continued until β does not change any more. The design point can be defined as

$$R^* = m_R - k\, \sigma_R \tag{9}$$

where k is a parameter to be determined by iterations. The mean and standard deviation can be replaced using the nominal value, R_n, bias factor, λ_R, and coefficient of variation, V_R,

$$R^* = R_n\, \lambda_R\, (1 - k\, V_R) \tag{10}$$

and the intermediate value of the reliability index is

$$\beta = \frac{R_n \cdot \lambda_R \cdot (1 - k \cdot V_R) \cdot [1 - \ln(1 - k \cdot V_R)] - m_Q}{\sqrt{[R_n \cdot V_R \cdot \lambda_R \cdot (1 - k \cdot V_R)]^2 + \sigma_Q{}^2}} \tag{11}$$

For the design cases considered in calibration of the AASHTO LRFD Code, parameter k is about 1.8 to 2.1. Furthermore, the reliability index is not sensitive with regard to the exact position of design point, represented by parameter k. Therefore, in this calibration, the reliability indices are calculated for k equal to 2.

IMPORTANCE OF LOAD AND RESISTANCE FACTORS

Load and resistance are random variables due to natural and man-made uncertainties. A random variable can be described by the CDF, the probability density function (PDF), or, at a minimum, the statistical parameters such as the mean value, standard deviation or coefficient of variation.

Examples of PDF are shown in Figure 2 for load and Figure 3 for resistance. In general, the nominal (design) values of load and resistance are different than the mean values. Safety reserve is included partly in the factored load and partly in the factored resistance. To provide an adequate safety level, the design values of load and resistance have to be conservative: loads are overestimated (Figure 2) and load carrying capacity (resistance) is underestimated (Figure 3). Therefore, load and resistance factors represent partial safety margins.

The higher the load factors, the higher are the safety reserve and reliability. However, an increase of the resistance factors results in a decrease of reliability. Therefore, the required reliability can be achieved by adjustment of load and resistance factors.

In this calibration, we considered only resistance factor. Three representative steel curved girder bridges were selected. For each considered bridge, the reliability analysis was performed using several values of ϕ (only multiples of 0.05 were used). The calculated reliability indices, β, were compared to the target value, β_T. The final ϕ factors recommended for the design code provided the best fit to β_T.

It is assumed that the target β for steel curved girders is the same as for steel straight girders, β_T equal to 3.5.

STRAIGHT AND CURVED GIRDER CALIBRATION DIFFERENCES

The basic difference is in the statistical parameters of load and resistance. Most of the other items that affect resistance factors are the same, including the format of design formula, load factors, reliability analysis procedure and equation for the reliability index, Eq.11. Therefore, the major effort is focused on the development and/or verification of the bias factors

and coefficients of variation for resistance, and dead load and live load effects during construction and normal operation.

The resistance parameters are determined by three factors representing uncertainties in material properties, dimensions and analytical model. The actual values of bias factors and coefficients of variation are based on the information provided by Professor Ted Galambos, the leading national authority in this area. In calibration of the AASHTO LRFD Code for straight steel girders, the information on statistical parameters was also provided by Professor Galambos.

The statistical parameters of load effects were based on the results of advanced analysis for three selected bridges performed at the University of Michigan using the finite element analysis software ABAQUS, and field measurements for one of the considered bridges (provided by the University of Minnesota). It was assumed that the statistical parameters of truck loads (including truck weight, axle configuration, multiple presence in-lane and side-by-side) are the same as for straight bridges (Nowak 1999). Therefore, the difference between load parameters for straight and curved girders is practically due to load distribution factors.

The nominal (design) values of load effects were provided by Modjeski and Masters, Inc. using a grid structural analysis program developed by Bridge Software Development International Ltd (BSDI). The field measurements and FEM results were compared with nominal values to obtain bias factors. The most important conclusion of this comparison is that the bias factors for dead load and live load for curved girder bridges are lower than for straight girder bridges.

EXAMPLES OF RELIABILITY ANALYSIS

The reliability index is calculated for two bridges; one with straight girders and the other one for curved girders, both with the same span and girder spacing.

Reliability Analysis for a Straight Girder

For the reliability analysis of a straight girder bridge, the statistical parameters are defined by Nowak (1999). The input data includes the parameters for dead load components such as factory-made members, D_1, cast-in-place members, D_2, live load components (live load, LL, and dynamic load, IL), girder distribution factor and resistance.

The considered bridge dimensions:
Span, L = 120 ft
Girder spacing, S = 8 ft

The nominal (design) loads:
D_1 (structural steel, per girder) = 650 k-ft
D_2 (concrete slab, per girder) = 1656 k-ft
LL (HL-93 per lane) = 3032 k-ft
IL (per lane) = 621 k-ft

Statistical parameters of dead load:
$\lambda_{D1} = 1.03$, $V_{D1} = 0.08$
$\lambda_{D2} = 1.05$, $V_{D1} = 0.10$

Statistical parameters of live load:
$\lambda_{LL} = 1.29$, $V_{LL} = 0.18$ (including IL)
Mean live load is increased by 2.5% for average daily truck traffic (ADTT) = 5000
Bias factor of the girder distribution factor = 1.00
$m_{IL} = 0.1\ m_{LL}$
Multiple presence factor = 0.85 (two lanes loaded)
Girder distribution factor = $0.75 + (S/9.5)^{0.6}(S/L)^{0.2} = 0.075 + (8/9.5)^{0.6}(8/120)^{0.2} = 0.38$

Mean total load per girder, m_Q:
m_Q (mean total load, per girder) = (1.03)(650) + (1.05)(1656) +
$+ 1.29(1.025)(3032)(1.1)(0.85)(1.0)[0.075 + (8/9.5)^{0.6}(8/120)^{0.2}] = 4660$ k-ft

Variance of total load per girder, σ_Q^2:
σ_Q^2 (variance of total load, per girder) = $[(1.03)(650)(0.08)]^2 + [(1.05)(1656)(0.1)]^2 + [1.29(1.025)(3032)(1.1)(0.85)(1.0)[0.075 + (8/9.5)^{0.6}(8/120)^{0.2}](0.18)]^2 = 444^2$

The required nominal (design) resistance, R_n, is calculated according to AASHTO LRFD (2004). The resistance factor, ϕ, for a composite steel girder bridge 1.0, and the nominal resistance, R_n, is calculated as follows:
$R_n = [1.25\ (D_1 + D_2) + 1.75\ (LL + IL)]/\phi =$
$= [1.25\ (650 + 1656) + 1.75\ (3032 + 621)\ [0.075 + (8/9.5)^{0.6}(8/120)^{0.2}]\ ]\ /1.0 = 6716$ k-ft

Statistical parameters of resistance:
$\lambda_R = 1.12$
$V_R = 0.10$
$m_R = 1.12\ (6716) = 7522$ k-ft

Reliability index, β, is calculated using Eq. 11,

$$\beta = \frac{6716 \cdot 1.12 \cdot (1 - 2 \cdot 0.10) \cdot [1 - \ln(1 - 2 \cdot 0.10)] - 4660}{\sqrt{[6716 \cdot 0.10 \cdot 1.12 \cdot (1 - 2 \cdot 0.10)]^2 + 444^2}}$$

$\beta = 3.61$

Reliability Analysis for a Curved Girder

The reliability analysis is performed for a curved bridge with identical dimensions to those in the previous example. The statistical parameters for dead load components, D_1 and D_2, and girder distribution factor are taken from this report. The statistical parameters for live load, LL, dynamic load, IL, multiple presence factor and statistical parameters of resistance, λ_R and V_R, are taken from Nowak (1999).

The considered bridge dimensions:
Span, L = 120 ft
Girder spacing, S = 8 ft
Radius of curvature = 285 ft

The nominal (design) loads:
D_1 (structural steel, per girder) = 650 k-ft
D_2 (concrete slab, per girder) = 1656 k-ft
LL (HL-93 per lane) = 3032 k-ft
IL (per lane) = 621 k-ft

Statistical parameters of dead load:
$\lambda_{D1} = 1.0$, $V_{D1} = 0.15$
$\lambda_{D2} = 1.0$, $V_{D1} = 0.15$

Statistical parameters of live load:
$\lambda_{LL} = 1.29$, $V_{LL} = 0.215$ (including IL)
Mean live load is increased by 2.5% for ADTT = 5000
Bias factor of the girder distribution factor = 0.75
$m_{IL} = 0.1\ m_{LL}$
Multiple presence factor = 0.85 (two lanes loaded)
Girder distribution factor = $0.075 + (8/9.5)^{0.6}(8/120)^{0.2} = 0.38$

Mean total load per girder, m_Q:
m_Q (mean total load, per girder) =
$= (1.0)(650 + 1656) + 1.29(1.025)(3032)(1.1)(0.85)(0.75)[0.075 + (8/9.5)^{0.6}(8/120)^{0.2}] =$
= 3995 k-ft

Variance of total load per girder, σ_Q^2:
σ_Q^2 (variance of total load, per girder) = $[(1.0)(650 + 1656)(0.15)]^2 +$
$+ [1.29(1.025)(3032)(1.1)(0.85)(0.75)[0.075 + (8/9.5)^{0.6}(8/120)^{0.2}](0.215)]^2 = (501)^2$

The required nominal (design) resistance, R_n, is calculated according to AASHTO LRFD (2004). The resistance factor, ϕ, for a composite steel girder bridge 1.0, and the nominal resistance, R_n, is calculated as follows:
$R_n = [1.25\ (D_1 + D_2) + 1.75\ (LL + IL)]/\phi =$
$= [1.25\ (650 + 1656) + 1.75\ (3032 + 621)\ [0.075 + (8/9.5)^{0.6}(8/120)^{0.2}]\ ]\ /1.0 = 6716$ k-ft

Statistical parameters of resistance:
$\lambda_R = 1.165$
$V_R = 0.095$
$m_R = 1.165\ (6716) = 7824$ k-ft

Reliability index, β, is calculated using Eq. 11,

$$\beta = \frac{6716 \cdot 1.165 \cdot (1 - 2 \cdot 0.095) \cdot [1 - \ln(1 - 2 \cdot 0.095)] - 3995}{\sqrt{[6716 \cdot 0.095 \cdot 1.165 \cdot (1 - 2 \cdot 0.095)]^2 + 501^2}}$$

$\beta =$ 4.69

FINAL CALIBRATION RESULTS

The reliability index is calculated for straight girders as well as for curved girders. The statistical parameters of load and resistance for steel curved girders are either more conservative or the same as for steel straight girders. Therefore, using the same resistance factors for curved and straight girders (equal to 1.0), results in reliability indices for curved girders that are not lower than for straight girders.

FIGURES AND TABLES

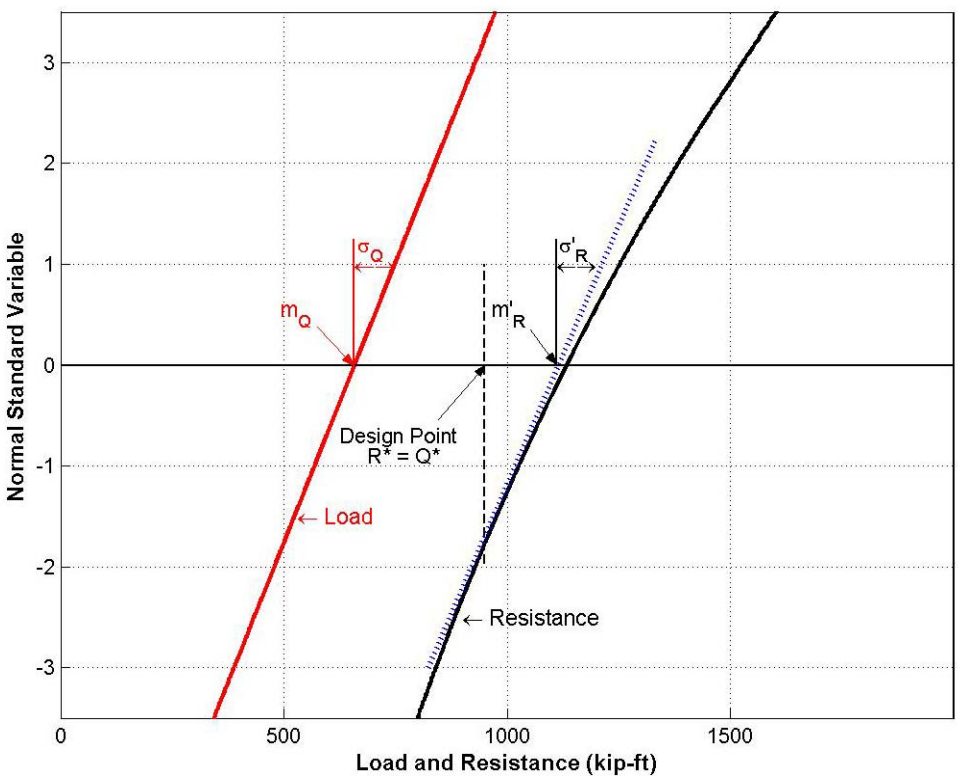

Figure 1. Graphical Procedure for Calculation of the Reliability Index.

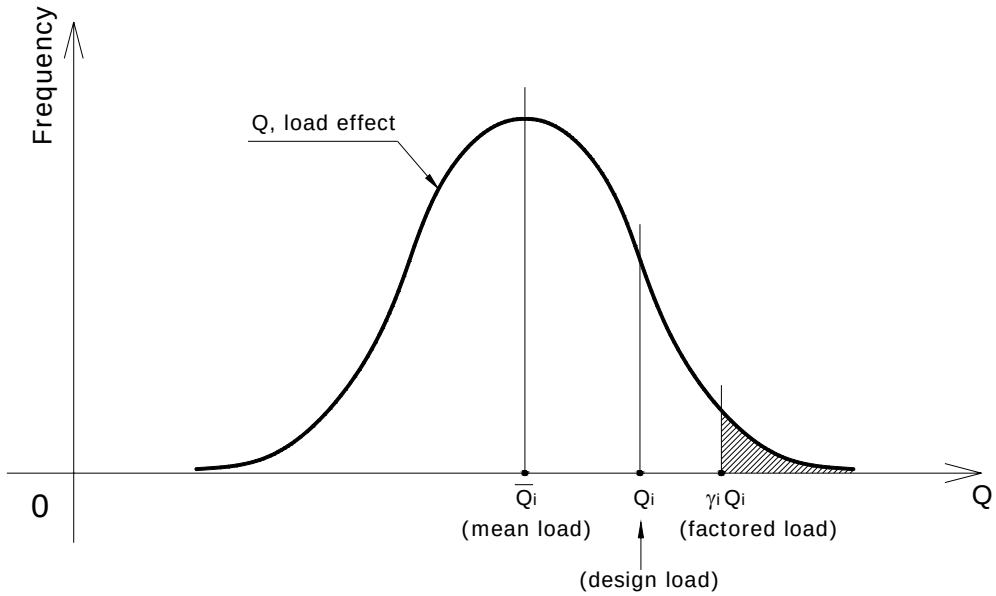

Figure 2. Mean Load, Nominal (Design) Load and Factored Load.

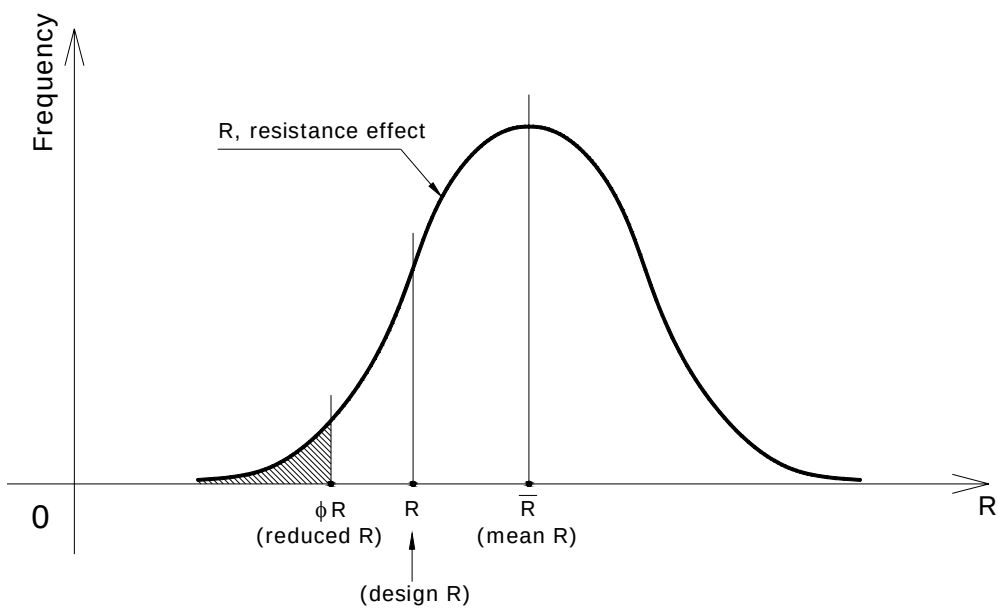

Figure 3. Mean Resistance, Nominal (Design) Resistance and Factored Resistance.

Table 1. Probability of Failure and Reliability Index.

P_F	β
0.5	0
10^{-1}	1.28
10^{-2}	2.33
10^{-3}	3.09
10^{-4}	3.71
10^{-5}	4.26
10^{-6}	4.75
10^{-7}	5.19
10^{-8}	5.62
10^{-9}	5.99