Quarterly Progress Report

to the

NCHRP

on Project NCHRP 12-103

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Drexel University

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NCHRP 12-103:
BRIDGE SUPERSTRUCTURE TOLERANCE TO TOTAL AND DIFFERENTIAL FOUNDATION MOVEMENTS

Quarterly Progress Report – Q3 of Phase II, July 2015

Overview of Project Motivation and Goals

Currently, there is little rational and practical guidance available to bridge designers as to what constitutes tolerable total and differential support movements for various bridge types/configurations. The semi-empirical and anecdotal evidence available from past research suggests that large relative movements may be tolerable from a structural safety perspective, but designating such large movements as "tolerable" runs contrary to many designers' sensibilities. While current practice can vary significantly from state to state, it is not uncommon for arbitrarily small differential support movements to be specified by bridge designers. Depending on the specific case, this excessive conservatism can lead to a significant increase in cost (e.g. through precluding the use of certain foundation types) with no clear evidence that it produces a commensurate improvement in long-term bridge performance.

Motivated by this knowledge gap, the overarching aim of this research is to develop a more sound and comprehensive understanding of the levels of differential support movements that bridges may tolerate without performance problems. More specifically, this research was devised to satisfy the following two objectives:

1) Develop procedures to determine the acceptable levels of bridge foundation movements based upon superstructure tolerance to total and differential movements considering service and strength limit states
2) Propose revisions to the AASHTO LRFD Bridge Design Specifications related to foundation movement limits that shall include vertical, horizontal, and rotational movements.

This Quarterly Progress Report (QPR) is for the third Quarter of Phase II NCHRP 12-103, and summarizes responses to Panel Comments on the previous QPR, the progress and results to date, problems encountered in the past quarter, and the work planned for Q4.

Responses to Panel Comments from Previous QPR

The Research Team is grateful to the Project Panel for the important comments they provided on the 2nd Quarterly Report, which was submitted to NCHRP in early April 2015. Table 1 provides responses to each comment received.

Table 1 – Response to comments from the Project Panel on the 2nd Quarterly Report

Comment	Comment	Response
1.1	Why is phi sub s referred to as "allowable" rather than "tolerable" settlement factor? Allowable is an ASD term.	All references to "allowable" settlement have been revised to "tolerable" settlement.
1.2	If I understand the figure (p. 16, Figure 7) it shows that while the trend of allowable settlement versus span length is similar to Moulton's work, the results from this project show that the bridges analyzed have a tolerable movement at least twice the limit recommended by Moulton. If bridge designers are reluctant to accept Moulton's criteria, especially for long-span bridge, how will bridge designers come to accept the results of this study?	Moulton's expression was intended to be a lower bound on tolerable support movement. The preliminary results presented are consistent with Moulton's work in this regard. The authors certainly appreciate the reviewers comments related to designer sensibilities and will keep this in mind throughout the remainder of the project (especially in generating recommendations related to bridge design practice).
2.1	The effect of redistribution of superstructure moments caused by settlement has not been well discussed. For prismatic steel girders when negative moment region governs design, applying settlement will reduce negative moment and increase positive moment. Therefore, the control region may switch from negative to positive.	During the second quarter, only settlements of the abutment were considered, which increase flexure demands in the negative moment region. However, the overall study will also impose interior support movements, which will cause the governing section to switch to the positive moment region.
2.2	It looks that any presumed settlement (like 1 in.) is differential, as absolute settlement is not directly an important parameter for structural checks. Please clarify.	This is correct, from a structural response perspective only relative support movements are of interest. However, when recommendations are developed based on this work, the research team will also consider absolute support movements (e.g. clearance, ride-quality, etc.).
2.3	The results shown in Appendix D, mostly do not show any specific trend. Is there any method to classify and maybe filter the data to reach an acceptable conclusion? For example, why some bridges show zero tolerable settlement? Please discuss and explain your strategy on how to make a reasonable conclusion. Will this strategy be a probabilistic one?	The preliminary results shown in Appendix D of the 2nd QPR were erroneous due to an error in the algorithm used to size the negative moment region of the girder. This error was identified and noted in the 2nd QPR, and has been corrected for the results shown in the 3rd QPR. Based on these results, the use of a probabilistic approach to estimating tolerable support movements does not appear necessary (i.e., the lower bound approach of Moulton appears to be a practical and useful approach).

3.1	The QPR did not show the progress on the project tasks overall and the budget status.	Please refer to Appendix C.
3.2	The QPR focused on the Strength I limit state; the research team will also include the other applicable limit states in their future work, correct?	This is correct. Strength I, Service II, and Service A limit states were all considered this quarter. Accompanying results are included in the report.
3.3	The proportions of some girder cross-sections in the appendix are unrealistic (I can't envision a girder flange that is 9 inch wide, ½ inch thick); Fatigue I/II design would bump that up (resulting in a stiffer element). So for these models with very slim proportions, are we truly being unconservative when the suite is based on a strength design optimized to have rating factors of 1.0/1.3 at the inventory/operating levels?	The research team agrees that the inclusion of the Fatigue Limit State is important for this study, and it was added to the member-sizing software this quarter. The impact of this change was very minor in the case of abutment support movements (since the negative moment region governs). Obviously the impact of this change related to interior support movements will be far more significant.
4	Are strength and stiffness linearly related for shear forces? A superstructure that has been proportioned based merely on strength requirements is going to be more flexible than one that meets all design limit states (including Fatigue I or II). Then any settlement of the stiffer structure will result in larger settlement forces. For moment I can see the linear relation between strength and stiffness, but I'm not sure it holds up for shear; that is I can double the girder cross-section inertia without changing the girder depth, so the does the one-to-one relationship between stiffness and strength hold up for shear?	This is a very interesting point and one that will be directly investigated in the following quarter when shear-related limit states are examined.
5	The research team proposes to define the allowable settlement factor to be the net capacity divided by settlement. The net capacity is defined based on Strength I load factors. However, foundation movements are calculated based on the service limit state. There is an apparent compatibility issue with the propose formulation. Please address this issue.	This quarter, the definition of tolerable support movement was expanded to include Strength I, Service II, and Service A Limit States. Please refer to the Work Performed section of this report. In the next quarter, additional limit states will be considered (as outlined in the Phase I report).
6.1	Work Performed, T2.1.3 mentions "common details of secondary elements (e.g. barrier and sidewalk sizes, diaphragm type/spacing, deck thickness, etc. defined using heuristics)." Is there a list of all of the secondary elements and which ones are, or otherwise might affect any, continuous or discrete parameters?	Please refer to Appendix A.2 for data on bridge configuration, member-sizing, member capacity, live load distribution factors, and secondary members.

6.2	Work Performed, T2.1.3 discusses a case where “one of the sample models appeared to be over-designed” due to finding “a solution that resides in a local minimum” as opposed to the desired global minimum. This is an issue that often occurs in minimization algorithms, and is not always easy to overcome. But this is also an issue that can skew the final results. Therefore, it is important that this problem be overcome. Simply discarding samples that are over-designed because of the problem may not solve the issue because if it tends to occur more with a particular part of continuous or discrete parameter, then that missing part or portion of the particular parameter would then skew the final results. When the single-line rating ends up much more conservative than the desired rating of 1.0 as a result of this issue, then additional iterations around the local minimum can sometimes get the minimization algorithm unstuck. There may be an easier way to get around this issue, I’m not sure; but it is an issue that needs to be addressed.	In response to comment 6.2, sample designs that appear to be "stuck" in a local minimum will no longer be discarded. For those samples and samples with no solution, additional iterations will be conducted to obtain an acceptable solution (i.e. one with a SLG rating of 1.0).

Summary of Progress and Results to Date

During the first two quarters the project investigators: (a) coded and validated the sampling methodology that was proposed in Phase I (T2.1.1), (b) expanded the capabilities of the automated member sizing software to include the AASHTO LRFD Specifications for steel multi-girder continuous and simply-supported bridges (T2.1.2), (c) validated the member sizing approach through comparison with manual designs (T2.1.2), and (d) developed and validated an automated simulation tool to conduct support movement, dead load, and live load analyses and extract key results (T2.1.3). Using these tools, a preliminary study was carried to estimate the tolerable support movement for 100 notional bridges at the end of the second quarter. The results of this study were presented in the previous quarterly report (T2.2).

Based on this preliminary study, three modifications to the underlying design assumptions were proposed in the 2nd Quarterly Progress Report (2 QPR) and implemented during Q3. These included (1) the elimination of the requirement that the interior and exterior girders have the same geometric properties, (2) the inclusion of an iterative procedure to account for the non-constant girder cross-sections within the single-line girder (SLG) analysis used within the member sizing process, and (3) the removal of the rounding rules proposed as a means to estimate current practice. Changes (1) and (3) resulted in the removal of additional conservatism, which may in fact reflect current practice, but is not explicitly required by the AASHTO LRFD Specifications. As a result, the decision was made to make these changes to provide member sizes without any arbitrary additional conservative.

In addition to implementing these changes, the Research Team (RT) modified the member sizing algorithm to include the Fatigue I limit state (as suggested by the Project Panel in response to QPR 2). This addition had a marked effect on the girder sizing in the positive moment region, but little influence (at this stage) on the computed tolerable support movements, as this is typically governed by the negative moment region over interior supports. As the project begins to look at shear-related limit states and differential support movements (along a support) it is expected that the additional stiffness that results from the inclusion of the Fatigue I limit state will likely prove more influential.

Using the updated procedure, a 400 model suite of notional bridges was generated and analyzed to compute their tolerable support movements. This study generated 100 samples for the continuous parameters (e.g. span length, skew, girder spacing, etc.) and, for each sample, considered the bounding combinations of barrier stiffness (on/ off) and rotational fixity at supports (fixed against rotation/ free to rotate). These two-span continuous models were analyzed for tolerable vertical support movements at the abutments only. Figure 1 below shows the relation between span length and tolerable support movement for the Strength I Limit State. From this plot two observations can be drawn. First, the influence of barrier stiffness appears to be negligible. This is attributed to the consideration of only flexural-related limit states at this point. It is expected that this parameter will prove more influential for shear-related limit states.

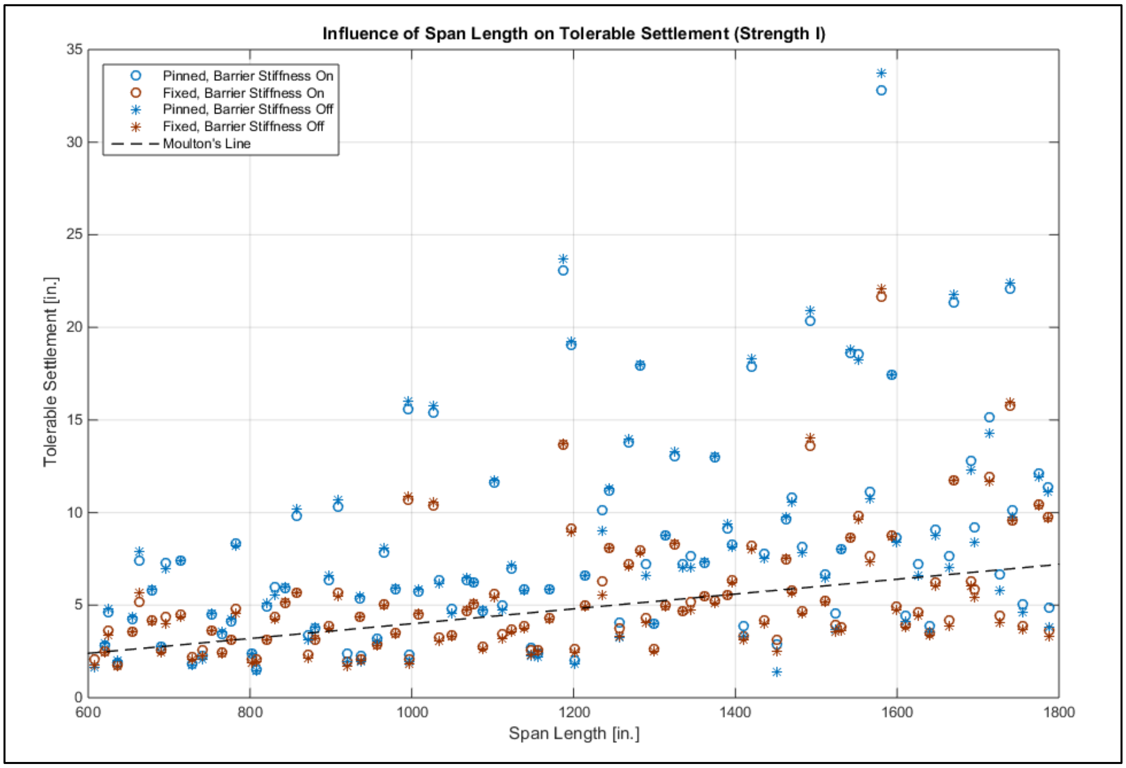


Figure 1 - Influence of Span Length on absolute tolerable settlement for the Strength I Limit State

The second observation is that several of the notional bridges have tolerable support movements that are significantly (up to 50% to 75%) less than Moulton's expression suggests. Through a series of single-variate analyses of the data, it was observed that this apparent lack of conservatism was associated with bridges that had skews of greater than 20°. Figure 2 below shows the same results (i.e. Strength I) with bridges with skew angles larger than 20° faded. As apparent from this figure, it appears that Moulton's expression indeed provides a fairly accurate lower bound on tolerable support movement for the Strength I Limit State for bridges with skew angles less than 20°. Given the time of Moulton's study in the early-1980s, it is likely that heavily skewed bridges were not a large portion of the population he examined. As a point of reference, the current AASHTO LRFD Specifications does use a skew of 20° in several places to delineate the appropriateness of certain procedures, as higher skew bridges are known to have more complex responses and require more refined approaches for their analysis and design.

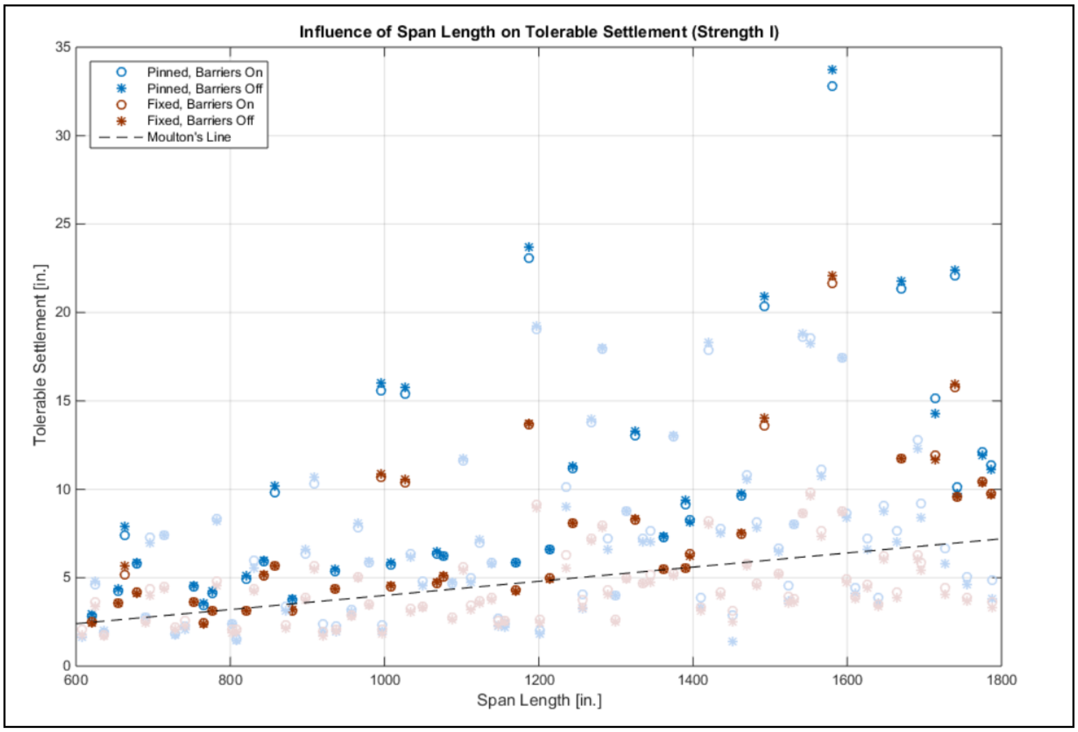


Figure 2 - Influence of Span Length on absolute tolerable settlement for the Strength I Limit State (tolerable support movements from bridges with skew angles greater than 20° are faded in this plot).

The tolerable support movement for the Service II Limit State for this same population was also computed and is shown in Figure 3. These results indicate that Moulton's expression represents a lower bound on tolerable support movement for the Service II Limit State (regardless of skew angle). Further, for the bridges examined, the Strength I Limit State governs the level of tolerable support movement over the Service II Limit State.

The final set of results computed were for a "proposed" limit state (denoted Service A) that was associated with the cracking of the concrete deck in the negative moment region. This limit state is not included within the AASHTO LRFD Specifications and so the loading employed load and resistance factors of 1.0, with the criteria that stress in the concrete deck (above interior supports) remain below the tensile strength of concrete. Figure 4 shows the results for these analyses and indicates that a large portion of the bridges examined have no tolerance to support movement based on this criteria (all negative values were set to zero in Figure 4). Since this limit state is not currently included within the design specifications, and since numerous bridges would violate this limit state even without support movement, it is proposed that this limit state no longer be considered by this research as it will have wide ranging implications on bridge design beyond those related to support movement.

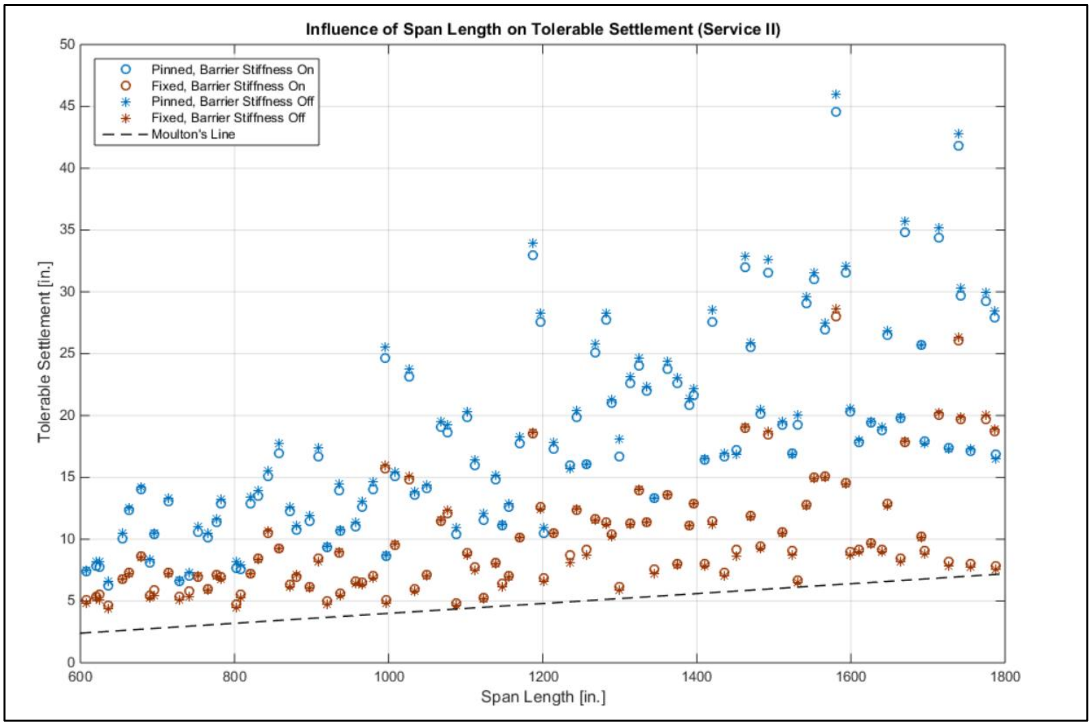


Figure 3 - Influence of Span Length on absolute tolerable settlement for the Service II Limit State

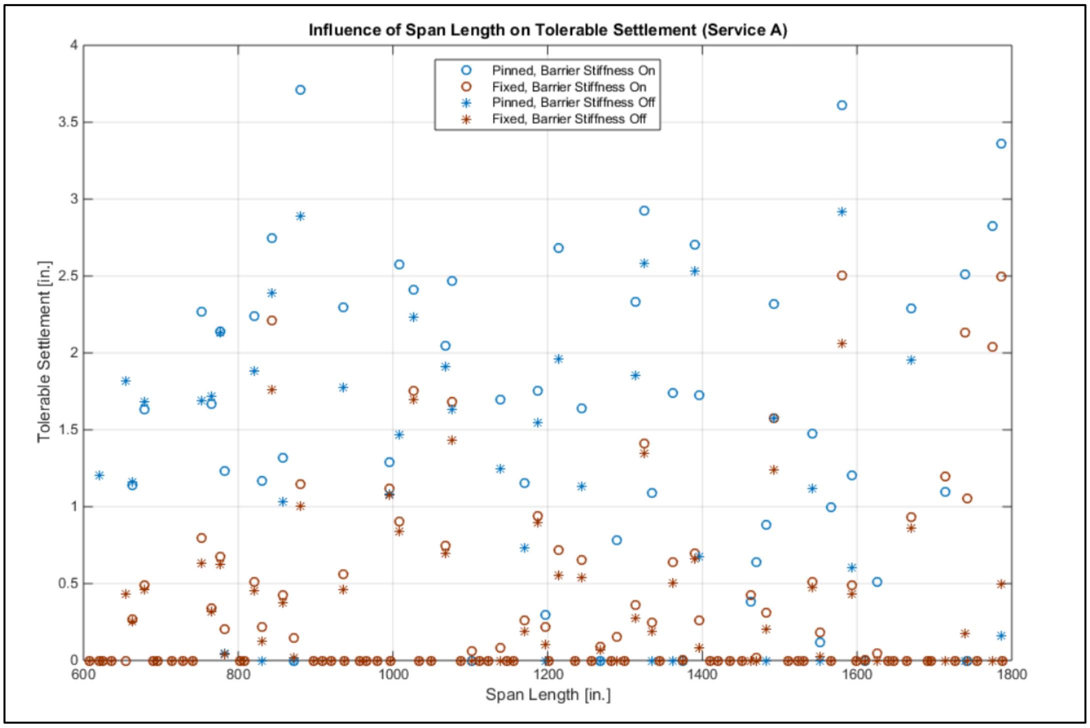


Figure 4 - Influence of Span Length on absolute tolerable settlement for the Service A Limit State

Summary of Work Planned for Q4

Work during Q4 will focus on two primary areas. First, the RT hopes to complete the analyses for multi-girder steel bridges. To do this, the following sub-tasks will be pursued:

- Expansion of the suite of notional bridges to include three-span continuous bridges
- Inclusion of the shear-related limit states within the computation of tolerable support movements
- Inclusion of support movements for interior piers
- Sampling and analyses of a second, independent suite of notional bridges to assess convergence

The second area of focus during Q4 will be the expansion of the member sizing algorithms to select appropriate prestressed concrete girders and the computation of the level of prestressing and eccentricity required to satisfy the AASHTO LRFD Limit States.

Work Performed in Q3 of Phase II

Task 2.1 – Definition and Construction of 3D FE Bridge Models

Sub-tasks of Task 2.1 continued during Q3. Due to the change of some design assumptions and the necessity to re-code portions of the member sizing algorithm, the RT team has fallen approximately one month behind the original schedule.

T2.1.1 – Sampling of Parameters to Define a Bridge Suite

Task Completion: 90%

In Q3, a sample space of continuous parameters was generated for 100 steel multi-girder bridge samples using the Matlab function written in the previous quarter. Initial development of the full notional bridge suite consisted of varying support conditions and barrier stiffness for the 100 models. Support conditions included pinned and fixed (for rotation about the transverse axis) for the abutment only. Barrier stiffness was varied on and off for each of the support conditions producing a total of 400 (see Appendix A.1 for additional details on the sampling procedure). No additional development of the sampling algorithms was required; therefore, no further work was needed to develop the required samples in Q3.

T2.1.2 – Automated LRFD Design of Bridge Suite

The goal of this task is to properly develop member sizes (e.g. steel girder dimensions, pre-stressing force/eccentricity, etc.) for each bridge configuration produced in Task 2.1.1. The approach was to develop an automated member sizing algorithm (further detailed in Appendix A.2) that paralleled current practice to the fullest possible extent (while neglecting conservative practices that are not codified or explicitly required by the AASHTO LRFD Specifications). In Q3, the software was updated to reflect the findings of previous validation studies as well as to address panel comments from the last quarterly report.

Early validation studies identified a shortcoming of the initial methodology that implicitly assumed a constant cross-section for the analysis used in the design (i.e. the single-line girder model) even though the software sized the positive and negative moment regions independently. The result of this inconsistency was the systematic under-design of the negative moment regions and, in turn, erroneous estimates of tolerable support movements. This quarter, the single-line girder analyses used within the member sizing process was updated to account for the differential stiffness between positive and negative moment regions in an iterate manner. More specifically, the updated process begins by sizing the member using approximate moment demands obtained under the assumption of a constant cross-section. When a solution is found, the single-line girder model is updated to reflect this initial design and a new set of demands are computed. The member designs are then updated based on the new demands and the process iterates until convergence is achieved (defined as the new demands being within 5% of the previous iteration). The flow chart shown in Figure 5 illustrates this process.

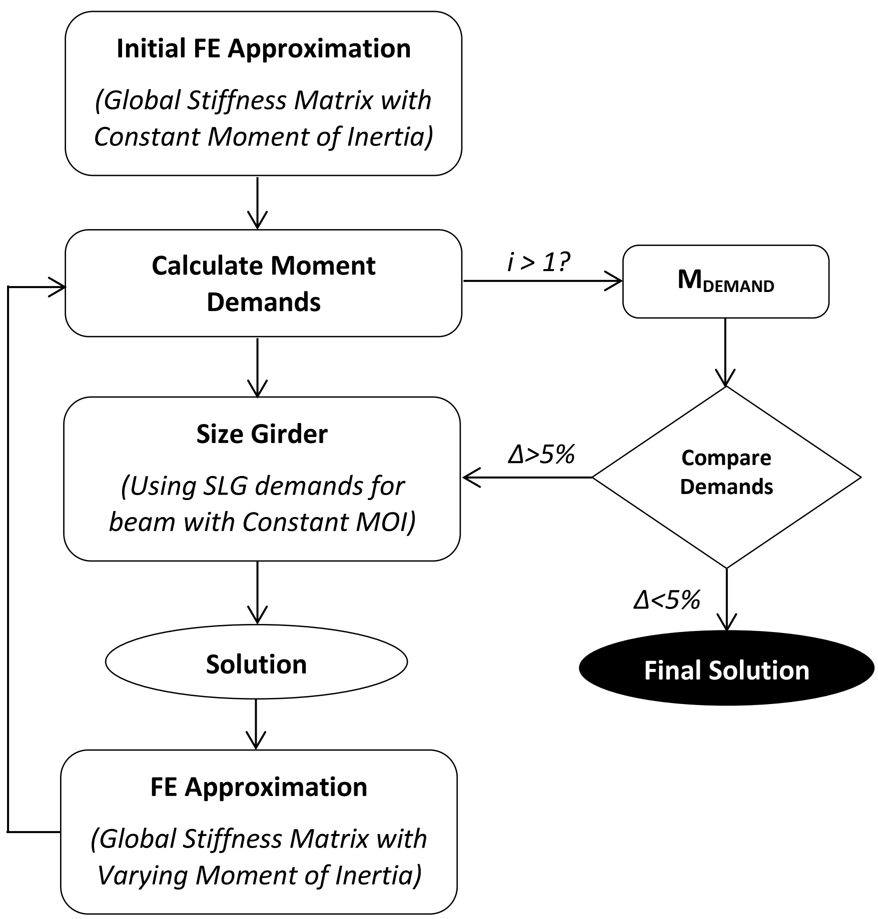


Figure 5 - Updated member-sizing process showing additional layer of iteration.

The panel comments on the previous QPR suggested that the Fatigue I Limit state should be considered within the member sizing process. The RT is grateful to the panel for this comment as it points out an oversight within the proposed research plan. Since support movements are not cyclic in nature, the RT originally proposed to ignore this limit state. While this is appropriate in the calculation of tolerable support movements, this limit state still needs to be considered during the design of the notional bridges as it can have an impact on member sizes. As a result, the Fatigue I Limit State was added to the member sizing algorithm and it had a significant influence over the flange area within the positive moment region of the members (as illustrated by Figure 6).

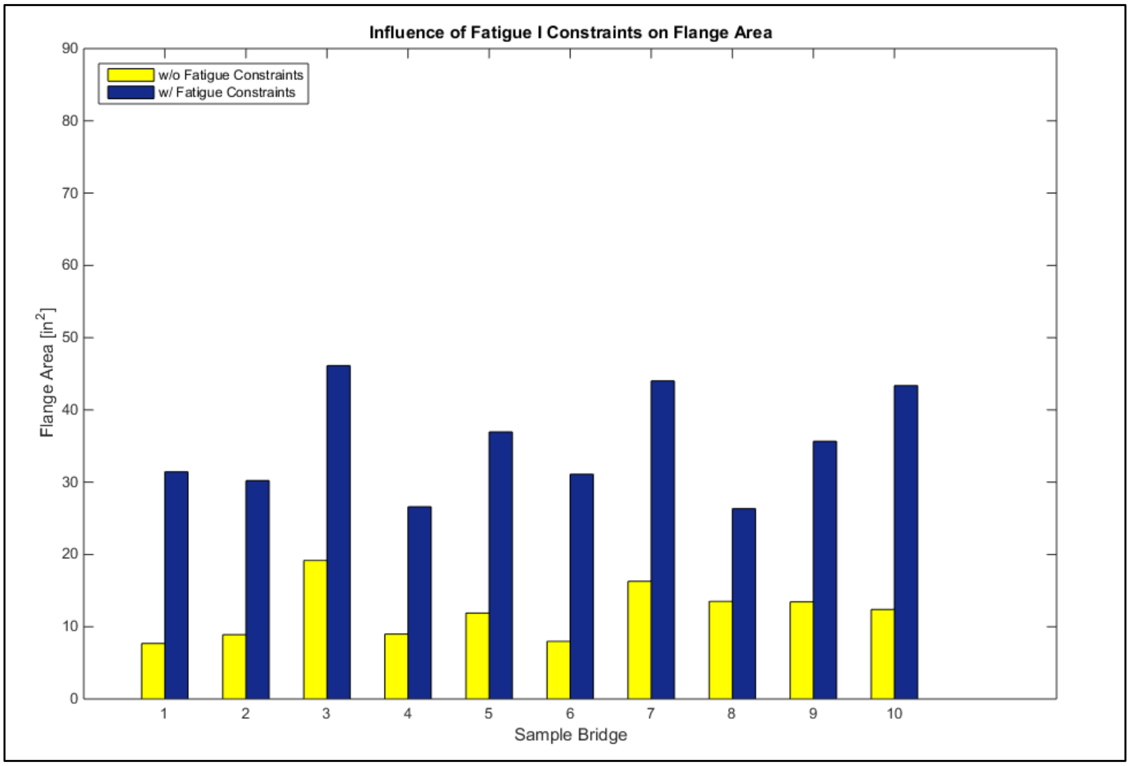


Figure 6 – Example influence of Fatigue I constraints on flange area (for exterior girder).

The effect of considering the Fatigue I Limit State is that it reduces the size of the cover plates required (or the need for a cover plate altogether) within the negative moment region, since it acts to increase the size of the girder in the positive moment region rather significantly. For many bridge configurations, Fatigue I controls the overall design of the girders and therefore additional capacity is introduced for Strength and Service Limit States. This results in larger single-line girder ratings than previous reported; however, these ratings still reflect the ***least conservative*** section possible considering all the AASHTO LRFD requirements summarized in Table 2.

Table 2 - Member sizing constraints and respective AAHSTO references

Sizing Constraints	AASHTO LRFD Section
Ductility	6.10.7.3
Flange Proportioning Limits	6.10.2.2
Web Proportioning Limits	6.10.2.1
Strength I Flexure	6.10.6 for Positive & 6.10.8 for Negative
Service II Flexure	6.10.4
Shear	6.10.9
Fatigue I	6.6.1.2

The refined automated member-sizing software is now capable of building steel multi-girder simply-supported and multiple-span continuous models that meet the AASHTO LRFD Design Specifications. As suggested by a panel comment, a more detailed accounting of the member sizing process employed is provided in Appendix A.2.

Task Completion: 80% for Steel Multi-girder

Automated member-sizing that follows the latest AASHTO LRFD Bridge Design Specifications for steel multi-girder bridges is currently functional. In Q4, work will be initiated on the development and validation of automated member-sizing for pre-stressed concrete multi-girder bridges. It is expected that this automation will build upon the experience gained thus far, and therefore will take less time to develop.

T2.1.3 – Automated 3D FE Modeling of Bridge Suite

During Q3, 400 FE models of two-span continuous, steel multi-girder models were constructed. These models were based on the sampling of bridge parameters carried out in T2.1.1 and the member sizing carried out in T2.1.2. Utilizing the API capabilities of Strand7, the models were constructed, simulated under relevant demands (including the live load and dead load demands as per the AASHTO LRFD Specifications and support movements), and key responses were extracted. During Q3 only vertical support movements of the abutments were considered. In the following quarter the results will be expanded to include support movements of interior piers.

Task Completion: 80% for Steel Multi-Girder

In Q3, development of the full suite of notional bridges was initiated for steel multi-girder bridges. Four hundred samples were obtained using the automated model-building software and analyzed for tolerable support movement for the abutment only. In Q4, the suite of notional multi-girder steel bridges will be expanded to its full size through the addition of three-span continuous and simply-supported bridges (1200 bridges total). In addition, tolerable support movements for interior piers and shear-related limit states will be computed.

Task 2.2 – Estimation of Maximum Tolerable Support Movements

Utilizing the results obtain in T2.1.3, the tolerable support movements for the notional bridge suite sampled in T2.1.1 and designed in T2.1.2 were estimated. Tolerable support movements (ϕ_s) were found for Strength I, Service II, and Service A Limit States using dead load (DL), live load (LL) and support movement demands (for moment only). The tolerable support movements for these three limit states were calculated using the equations below. For Strength I and Service II, the composite moment demands were obtained from the moment and axial forces within the beam elements of the FE model. For Service A (cracking in the deck) the strain at the top of the steel girder was projected to the top of the deck and converted to stress using the elastic modulus of concrete.

Equation 1 - Formula for calculating tolerable settlement factor for Strength I.

$$\varphi_{s\_St1} = \frac{Capacity - 1.25 * DL - 1.75 * LL * (1.0 + IM)}{Demand\ of\ Unit\ Support\ Movement}$$

Equation 2 - Formula for calculating tolerable settlement factor for Service II.

$$\varphi_{s\_Sv2} = \frac{0.95 * F_y - 1.0 * DL - 1.3 * LL * (1.0 + IM)}{Demand\ from\ Unit\ Support\ Movement}$$

Equation 3 - Formula for calculating tolerable settlement factor for Service A.

$$\varphi_{s\_SvA} = \frac{7.5 * \sqrt{f'_c} - 1.0 * DL - 1.0 * LL * (1.0 + IM)}{Demand\ from\ Unit\ Support\ Movement}$$

As described in the "Summary of Progress and Results to Date" section, the tolerable support movements computed for the notional bridge suite support the following three preliminary conclusions:

(1) Moulton's expression appears to be a lower bound for vertical support movement (only considering moment actions) for bridges with skew angles less than 20^{o}.
(2) The Strength I Limit State is generally more stringent than the Service II Limit State for the bridges examined, and thus tends to govern the magnitude of tolerable support movement.
(3) Bridges designed as per the AASHTO LRFD Specifications may violate the proposed Service A Limit State without any support movement.

Given (3) above, it is recommended that the Service A Limit State no longer be included within this study as it represents a rather significant departure from current bridge design practice.

Appendix B provides plots of the results obtained during Q3.

Task Completion: 25% for steel

In Q3, a suite of 400 notional bridges were generated and analyzed for tolerable support movement (considering only moment-related limit states) for a vertical settlement at the abutment. In Q4, the tolerable support movements for the full suite of bridges (1200 in total) will be estimated for both moment- and shear-related limit states, under abutment and pier support movements.

Task 2.3 – Identification of Parameters that influence Support Movement-Driven Performance Problems

While initial results have been gathered, only preliminary work on this task has been carried out, which consisted of a series of single-variate analyses of the results obtained thus far. Preliminary conclusions from this study indicate that span length, skew angle, girder spacing, and boundary conditions are influential (see Appendix A.3).

Task 2.4 – Spot Checking Additional Bridge Types

No work was performed on this task during this quarter.

Task 2.5 – Reporting

No work was performed on this task during this quarter.

Problems Encountered in Q3 of Phase II

Task 2.1 – Definition and Construction of 3D FE Bridge Models

Problems encountered with each sub-task are described in the following.

T2.1.1 – Sampling of Parameters to Define a Bridge Suite (Moon)

No problems were encountered with parameter sampling in this quarter.

T2.1.2 – Automated LRFD Design of Bridge Suite (Moon, Mertz)

The RT has encountered three primary problems associated with this sub-task:

- The assumption of a constant cross-section within the SLG model resulted in the systematic under-designing of the negative moment region of the girder, and was thus leading to erroneous estimates of tolerable support movements.
- The assumption of consistent interior and exterior girders, and the use of rounding rules, were introducing additional levels of conservatism within the member-sizing process (not explicitly required by the AASHTO LRFD Specifications), and thus were resulting in potentially unconservative estimates of tolerable support movements.
- The proposed member sizing approach that ignored the Fatigue I Limit State was systematically under-sizing the girder in the positive moment region, and this may potentially result in unconservative estimates of tolerable support movements when shear-related limit states are considered.

To date, each of these problems has been addressed and corrected as detailed in the previous section. Due to this unforeseen work, the project is now approximately one month behind schedule.

T2.1.3 – Automated 3D FE Modeling of Bridge Suite (Moon)

No problems were encountered with automated modeling in this quarter.

Task 2.2 – Estimation of Maximum Tolerable Support Movements (Moon)

See task T.2.1.2

Work Planned for Q4 of Phase II

Task 2.1 – Definition and Construction of 3D FE Bridge Models

For steel multi-girder bridges, Task 2.1 is nearly complete. Work planned for next quarter for Task 2.1 will be in regards to development and validation of automated member-sizing for pre-stressed concrete multi-girder bridges.

T2.1.1 – Sampling of Parameters to Define a Bridge Suite (Moon)

In Q4, the *Bridge Suite* will continue to be expanded for both two-span and 3-span continuous bridges using additional combinations of discrete parameters.

T2.1.2 – Automated LRFD Design of Bridge Suite

In Q4, the majority of the work will focus on code development and validation for the design of multi-girder prestressed concrete bridges. Once the software is fully developed, it will be used to generate multi-girder prestressed concrete designs for the suite of notional bridges identified in T2.1.1.

T2.1.3 – Automated 3D FE Modeling of Bridge Suite

Once the bridge suite has been fully defined (through sampling in Task 2.1.1 and automated design in Task 2.1.2) for both steel and pre-stressed concrete members, FE models for each bridge configuration will be constructed using the automated modeling software. Since the automated modeling software is currently functional for multi-girder steel bridges, it is expected that this sub-task will be completed with little additional effort for multi-girder prestressed concrete bridges.

Task 2.2 – Estimation of Maximum Tolerable Support Movements

In Q4, differential and vertical support movements will be examined for two and three-span continuous steel multi-girder bridges at the abutment and at the pier. In addition, the computed tolerable support movements will consider both flexure- and shear-related limit states.

References

1. Moulton, L. K., GangaRao, H. V. S. and G. T. Halvorsen (1985). *Tolerable Movement Criteria for Highway Bridges,* Report No. FHWA RD-85-107, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.
2. Risk Assessment Forum. 1997. Guiding Principles for Monte Carlo Analysis, EPA/630/R-97/001. Washington DC: U.S. Environmental Protection Agency.

Appendix A – Task 2.1: Definition and Construction of 3D Finite Element Bridge Models

Appendix A.1 – Sampling of Parameters to Define a Bridge Suite

In order to carry out the parametric study, it is necessary to sample the parameters of interest to develop a representative "bridge suite" (or sample of bridges) to allow the impact of total and differential support movements to be fully examined. The goal of this sampling is to effectively and efficiently cover the parameter space. To satisfy this objective a hybrid sampling method was proposed and refined in Phase I that utilizes a statistical sampling approach known as Latin Hypercube Sampling (LHS) for a set of continuous parameters, and a Design of Experiments (DoE) approach for different set of discrete parameters. The U.S Environmental Protection Agency Risk Assessment Forum (EPARAF) published a document stating that "Latin Hypercube sampling is considered to be more efficient than random sampling, that is, it requires fewer simulations to produce the same level of precision."(3) LHS divides each parameter distribution into *n* number of bins with equal probability density. In the case of this study, each parameter was assumed to have a uniform probability distribution and all bins were of equal size. From within each of these bins, the parameter is randomly sampled. Once the parameter from the bin is sampled, that bin can no longer be used. This ensures a more efficient sampling of the entire uniform distribution. The hybrid sampling methodology uncouples the discrete parameters from the continuous parameters to allow for finer sampling of the later, since the discrete parameters assume such a small number of values and a full-factorial approach (i.e. every possible combination of these parameters) is feasible. The continuous parameters are then coupled with each of the discrete parameters in order to make up the entire sample space. Figure A.1 depicts the LHS/DoE method.

A Matlab function was written to develop the sample space used for the model validation and support settlement studies. This function uses LHS to sample five continuous parameters: span length, skew, exterior-to-exterior girder width, girder spacing, and span-to-depth ratio. Each of the parameters is given a range of values to sample based on predetermined upper and lower bounds (as identified in Phase I). One modification from Phase I was made: girder spacing was added as a continuous parameter, removing it as a discrete parameter, which cuts the number of DOE combinations of the discrete parameters in half.

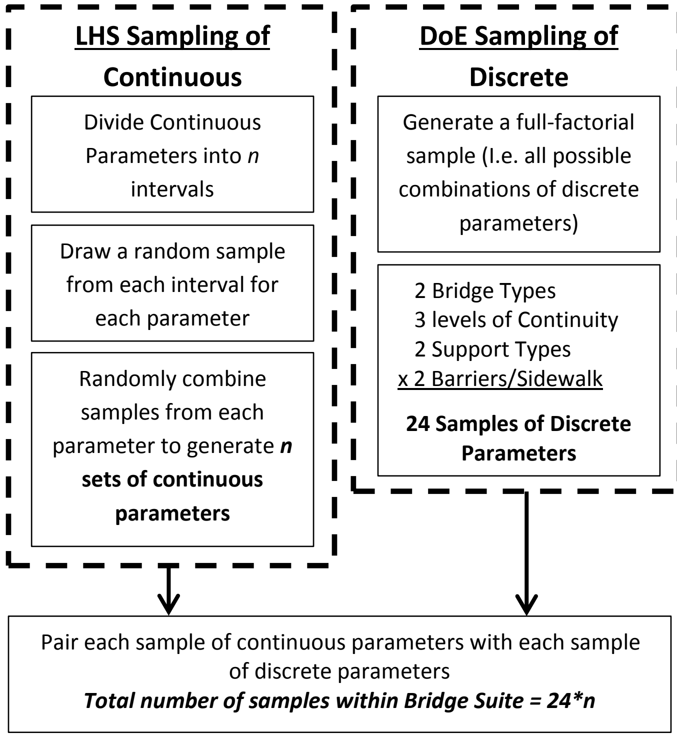


Figure A.1 - Sampling Methodology

While the addition of girder spacing to the continuous parameters will provide better information on the influence of this parameter, it presents a challenge in that it is related to the bridge width parameter. To overcome this challenge two assumptions were made. First, it was assumed that the deck overhangs (i.e., the portion of the deck that extends beyond the fascia girder) is constant at 36 in. Second, when combining the random sample of both width and girder spacing one value must be "corrected" as the number of girders must be an integer value. Given its relative insensitivity, it was decided to adjust the exterior-to-exterior girder width to the closest multiple of the girder spacing. Specifically, the exterior-to-exterior width value obtained using LHS is adjusted by first dividing this value by the girder spacing and rounding to the nearest whole number. This number gives the total number of girder spaces, which is then multiplied by the actual girder spacing to obtain the adjusted exterior-to-exterior girder width. An example calculation for the adjusted exterior-to-exterior girder width as well as the entire sample space of parameters is provided below.

The following is an excerpt of Matlab code detailing how adjusted exterior-to-exterior width is obtained.

```
% NumGirder (based on number of girder spaces that fit in road width)
Spaces = round(ExtWidth/GirderSpacing);
NumGirder = Spaces + 1;

% Adjusted Road width
AdjustedExtWidth = Spaces*GirderSpacing;

% Add 36" overhang to either side
Overhang = 36;
Width = AdjustedExtWidth + 2*Overhang; % Out-to-out
```

Figure A.2 below presents a matrix plot of the sample space for all continuous parameters for the suite of notional bridges developed during Q3. Table A.1 provides the specific parameters of the sampled bridges.

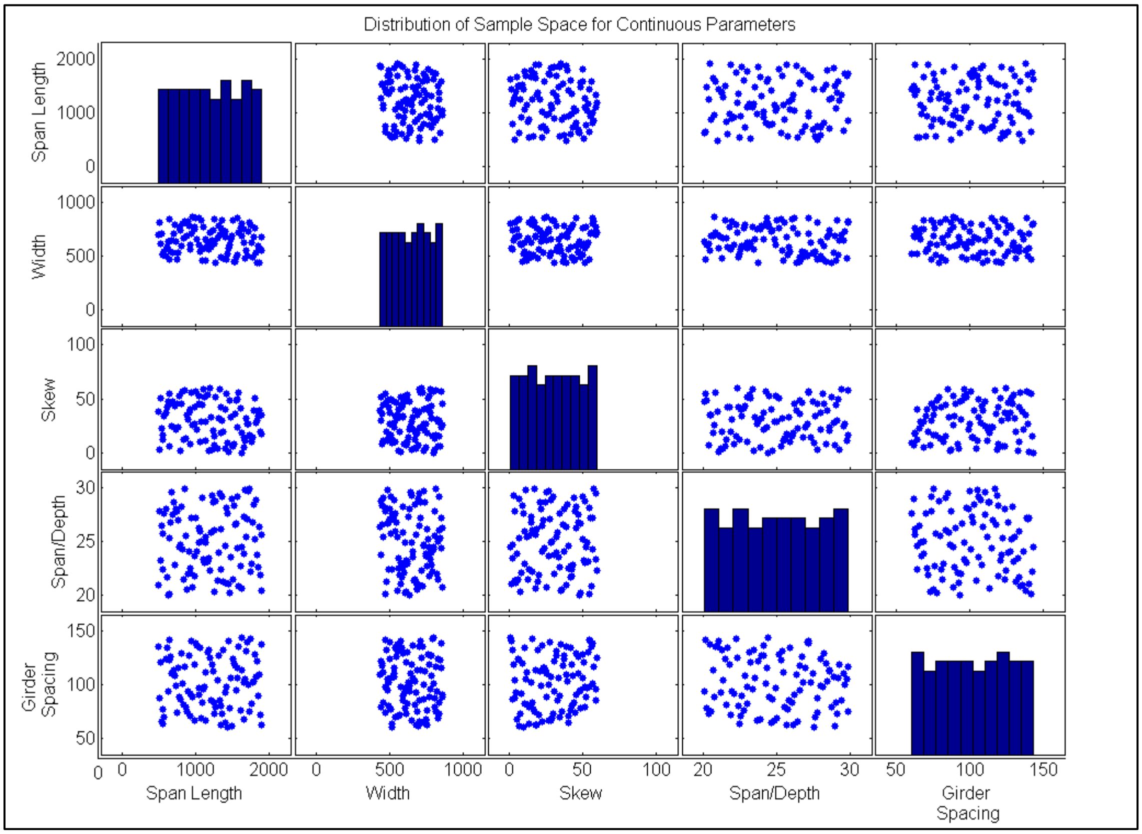


Figure A.2 - Distribution of sample space for continuous parameters.

Table A.1 - Bridge configuration data

Model Name	Length	Width	Skew	Span/Depth	Girder Spacing	No. Girders	Adjusted Width	Out-to-Out
Bridge_1	1123.11	739.95	57.47	21.45	74.00	11	739.95	799.95
Bridge_2	1324.79	661.27	7.83	20.93	82.66	9	661.27	721.27
Bridge_3	1743.02	585.24	5.36	28.04	117.05	6	585.24	645.24
Bridge_4	776.19	795.91	12.05	22.78	113.70	8	795.91	855.91
Bridge_5	1313.61	573.04	23.80	24.30	95.51	7	573.04	633.04
Bridge_6	858.13	492.41	16.83	23.37	70.34	8	492.41	552.41
Bridge_7	1714.10	584.76	10.71	26.76	97.46	7	584.76	644.76
Bridge_8	1788.32	485.94	35.84	20.76	121.48	5	485.94	545.94
Bridge_9	1049.28	664.37	29.22	25.24	132.87	6	664.37	724.37
Bridge_10	1551.68	469.44	24.20	26.18	78.24	7	469.44	529.44
Bridge_11	1463.11	729.46	5.88	29.12	81.05	10	729.46	789.46
Bridge_12	1361.58	740.30	14.50	25.70	105.76	8	740.30	800.30
Bridge_13	995.00	689.07	3.12	28.22	76.56	10	689.07	749.07
Bridge_14	1436.42	539.48	53.70	20.45	77.07	8	539.48	599.48
Bridge_15	752.54	520.06	15.98	24.52	130.01	5	520.06	580.06

Bridge_16	1481.98	561.04	27.04	21.23	112.21	6	561.04	621.04
Bridge_17	1566.44	495.24	21.20	29.84	123.81	5	495.24	555.24
Bridge_18	741.36	625.13	41.87	25.83	125.03	6	625.13	685.13
Bridge_19	1599.59	807.79	48.71	26.31	80.78	11	807.79	867.79
Bridge_20	1111.84	648.74	48.02	28.17	92.68	8	648.74	708.74
Bridge_21	935.52	564.21	9.91	26.57	141.05	5	564.21	624.21
Bridge_22	1640.18	853.37	45.73	24.45	106.67	9	853.37	913.37
Bridge_23	1267.99	798.49	25.38	28.68	88.72	10	798.49	858.49
Bridge_24	1374.21	661.47	33.02	20.57	60.13	12	661.47	721.47
Bridge_25	1257.30	597.83	38.89	28.53	119.57	6	597.83	657.83
Bridge_26	802.28	586.93	59.52	22.37	83.85	8	586.93	646.93
Bridge_27	1235.62	435.73	55.16	27.22	108.93	5	435.73	495.73
Bridge_28	1026.25	708.72	1.15	22.15	64.43	12	708.72	768.72
Bridge_29	713.97	554.72	28.17	26.46	69.34	9	554.72	614.72
Bridge_30	1147.89	571.22	49.64	23.41	142.80	5	571.22	631.22
Bridge_31	1419.54	857.40	43.15	28.87	65.95	14	857.40	917.40
Bridge_32	1200.92	885.33	59.02	25.03	98.37	10	885.33	945.33
Bridge_33	1451.72	448.29	51.54	27.35	112.07	5	448.29	508.29
Bridge_34	844.07	813.78	0.38	26.64	116.25	8	813.78	873.78
Bridge_35	1170.18	611.12	18.56	24.70	122.22	6	611.12	671.12
Bridge_36	1087.39	647.19	56.64	20.21	71.91	10	647.19	707.19
Bridge_37	766.09	777.87	19.81	21.10	111.12	8	777.87	837.87
Bridge_38	979.23	788.97	42.34	25.70	87.66	10	788.97	848.97
Bridge_39	1492.52	459.36	6.49	21.68	65.62	8	459.36	519.36
Bridge_40	1102.70	538.37	31.89	22.51	67.30	9	538.37	598.37
Bridge_41	965.33	891.90	41.26	21.98	99.10	10	891.90	951.90
Bridge_42	1155.89	790.07	44.16	25.55	131.68	7	790.07	850.07
Bridge_43	636.21	830.57	52.89	23.02	75.51	12	830.57	890.57
Bridge_44	1609.73	712.55	40.73	22.21	118.76	7	712.55	772.55
Bridge_45	1664.78	524.39	50.27	23.85	74.91	8	524.39	584.39
Bridge_46	1187.35	676.76	8.97	29.09	67.68	11	676.76	736.76
Bridge_47	1523.42	536.97	43.29	24.93	134.24	5	536.97	596.97
Bridge_48	808.66	803.91	56.19	27.78	114.84	8	803.91	863.91
Bridge_49	728.27	759.86	50.66	27.44	126.64	7	759.86	819.86
Bridge_50	607.91	456.63	36.89	26.83	114.16	5	456.63	516.63
Bridge_51	1008.72	774.25	7.69	23.94	129.04	7	774.25	834.25
Bridge_52	654.50	631.27	13.56	29.29	126.25	6	631.27	691.27
Bridge_53	1067.30	813.12	9.36	24.39	101.64	9	813.12	873.12
Bridge_54	690.20	812.87	34.35	29.52	135.48	7	812.87	872.87
Bridge_55	1299.32	821.37	58.13	27.93	91.26	10	821.37	881.37
Bridge_56	696.45	479.28	47.27	28.74	79.88	7	479.28	539.28
Bridge_57	1396.10	556.96	16.27	28.30	139.24	5	556.96	616.96
Bridge_58	1727.16	441.98	55.53	21.06	88.40	6	441.98	501.98
Bridge_59	1695.23	484.86	45.50	22.08	121.21	5	484.86	544.86
Bridge_60	1541.73	695.82	23.14	21.35	63.26	12	695.82	755.82

Bridge_61	830.18	586.48	20.61	28.45	117.30	6	586.48	646.48
Bridge_62	1625.49	496.97	30.60	21.56	124.24	5	496.97	556.97
Bridge_63	1511.50	863.14	32.40	27.88	143.86	7	863.14	923.14
Bridge_64	1410.62	786.97	30.07	24.71	131.16	7	786.97	846.97
Bridge_65	1690.90	584.50	38.33	23.19	73.06	9	584.50	644.50
Bridge_66	663.61	855.55	19.06	25.91	85.55	11	855.55	915.55
Bridge_67	880.40	549.64	7.06	20.02	137.41	5	549.64	609.64
Bridge_68	1244.53	691.73	11.04	27.02	138.35	6	691.73	751.73
Bridge_69	1670.75	639.39	13.88	23.56	71.04	10	639.39	699.39
Bridge_70	1592.73	621.12	21.73	23.64	69.01	10	621.12	681.12
Bridge_71	678.63	862.97	11.43	29.62	86.30	11	862.97	922.97
Bridge_72	1138.81	565.99	25.92	24.18	141.50	5	565.99	625.99
Bridge_73	1344.45	515.27	58.77	22.98	128.82	5	515.27	575.27
Bridge_74	920.43	543.94	40.17	21.72	135.98	5	543.94	603.94
Bridge_75	1739.56	746.73	4.21	29.48	82.97	10	746.73	806.73
Bridge_76	1076.88	514.86	3.76	23.30	102.97	6	514.86	574.86
Bridge_77	1033.46	557.44	39.24	22.87	92.91	7	557.44	617.44
Bridge_78	957.65	820.92	46.54	29.00	136.82	7	820.92	880.92
Bridge_79	1581.21	560.17	1.72	27.57	62.24	10	560.17	620.17
Bridge_80	1335.53	646.77	22.73	25.35	107.79	7	646.77	706.77
Bridge_81	820.22	507.98	15.36	20.12	84.66	7	507.98	567.98
Bridge_82	1282.49	739.96	26.61	27.69	61.66	13	739.96	799.96
Bridge_83	621.18	700.81	19.66	26.94	140.16	6	700.81	760.81
Bridge_84	1774.78	479.64	2.51	26.05	119.91	5	479.64	539.64
Bridge_85	898.15	831.35	44.96	24.04	103.92	9	831.35	891.35
Bridge_86	1787.05	768.57	2.26	22.45	109.80	8	768.57	828.57
Bridge_87	871.48	448.90	31.76	20.81	89.78	6	448.90	508.90
Bridge_88	624.26	661.52	54.15	29.93	94.50	8	661.52	721.52
Bridge_89	1390.63	804.35	17.73	20.67	100.54	9	804.35	864.35
Bridge_90	1647.52	838.42	36.14	29.73	104.80	9	838.42	898.42
Bridge_91	1289.97	399.03	34.09	24.83	99.76	5	399.03	459.03
Bridge_92	909.05	709.82	35.23	29.31	78.87	10	709.82	769.82
Bridge_93	997.50	542.45	47.71	20.36	108.49	6	542.45	602.45
Bridge_94	782.91	387.68	29.96	25.15	96.92	5	387.68	447.68
Bridge_95	1470.48	565.77	25.05	26.23	94.30	7	565.77	625.77
Bridge_96	1197.17	513.18	28.60	27.15	64.15	9	513.18	573.18
Bridge_97	936.73	766.50	37.32	25.49	127.75	7	766.50	826.50
Bridge_98	1755.13	816.02	51.85	22.70	102.00	9	816.02	876.02
Bridge_99	1213.82	534.94	12.89	23.79	133.73	5	534.94	594.94
Bridge_100	1530.99	816.82	52.64	21.80	90.76	10	816.82	876.82

Appendix A.2 – Automated LRFD Design of Bridge Suite

Member sizing is based on the single-line girder (SLG) method of structural analysis as defined by the AASHTO LRFD Bridge Design Specifications.

Single Line Girder Approximation of Demands

For dead load demands, member actions of the single-line girder are obtained by first applying a unit distributed load and then calculating the resulting member actions (moments and shears). Using the principle of superposition, the dead load demand is obtained by scaling those actions by the actual distributed dead load calculated for the structure. For live load, single-lane member actions are obtained by stepping point loads (representing the axle loads of the design trucks) across the entire length of the bridge together with distributed lane loads (when applicable) as per the AASHTO LRFD Specifications. Single-lane member actions (moments and shears) are then calculated for each combination and scaled by the dynamic impact factor. All of the single-lane member actions are combined based on the applicable load combinations and the resulting envelopes are scaled using the applicable distribution factors. The formulas used in the calculation of live load distribution factors are given in Tables A.2 and A.3 below.

Table A.2 - Distribution factors for exterior girder for one and two lanes loaded (from Table 4.6.2.2.2b-1).

One Design Lane Loaded	Two Design Lanes Loaded
$DF_1 = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12\ L\ t_s^3}\right)^{0.1}$	$DF_2 = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12\ L\ t_s^3}\right)^{0.1}$

Table A.3 - Distribution factors for exterior girder for one and two lanes loaded (from Table 4.6.2.2.2d-1).

One Design Lane Loaded	Two Design Lanes Loaded
$DF_1 = Lever\ Rule$	$DF_2 = e * DF_{interior}$

Member Sizing via the F_{mincon} Minimization Algorithm

The software is capable of building single span models with a rolled or plate girder section, as well as continuous models of two or more spans with a plate girder section. Rolled sections are used for single spans when the section meets all AASHTO LRFD Specifications. If a rolled section does not satisfy all requirements for the single span bridge, or if the bridge is multiple-span continuous, the software builds the model with a plate girder section. Plate girder sections are obtained using a built-in optimization algorithm in Matlab called *fmincon*. Matlab documentation notes that the *fmincon* algorithm "attempts to find a constrained minimum of a scalar function of several variables starting at an initial estimate."[(1)] The approach of finding a constrained minimum is referred to as non-linear optimization. For this project, the variables of the function are the flange width, flange thickness, web thickness, and for multiple span continuous bridges, the thickness of the cover plate in the negative moment region.

The scalar, or "objective" function, within the member-sizing algorithm is the area of the steel section. In the same manner that a typical designer may attempt to find the most economical section that still passes all constraints set by AASHTO LRFD Specifications, the *fmincon* algorithm attempts to find the combination of variables—plate girder dimensions—that pass all constraints while minimizing the area (minimizing the objective function). Using *fmincon* and the proper sizing constraints, the steel girder cross-sections can be sized such that it is the ***least conservative*** section possible that still passes all the requirements of the AAHSTO LRFD Specifications. That is, the section is sized right on the margin of capacity and demands (considering all of the applicable limit states). Using a section designed at this margin, the results of the population study for total and differential support movements will provide the most conservative estimate of tolerable settlement. This approach is a slight change from the one originally proposed that included rounding up of plate sizes based on current practice. The reason this rounding was eliminated is that it provided an additional conservativism (that while common in practice) was not explicitly required by the AASHTO LRFD Specifications. Since it is possible (if economically undesirable) to forego this rounding and thus not provide this additional conservatism, it was decided to simply design the girders precisely at the safety margin.

Hierarchy of Member Sizing Process

The process of the automated member-sizing is outlined in Figure A.3 for single span and Figure A.4 for multiple-span continuous. The overall hierarchy for member-sizing and model building is outlined in Figures A.5 and A.6. The specifications constraining the member-sizing are detailed in Tables A.4 and A.5.

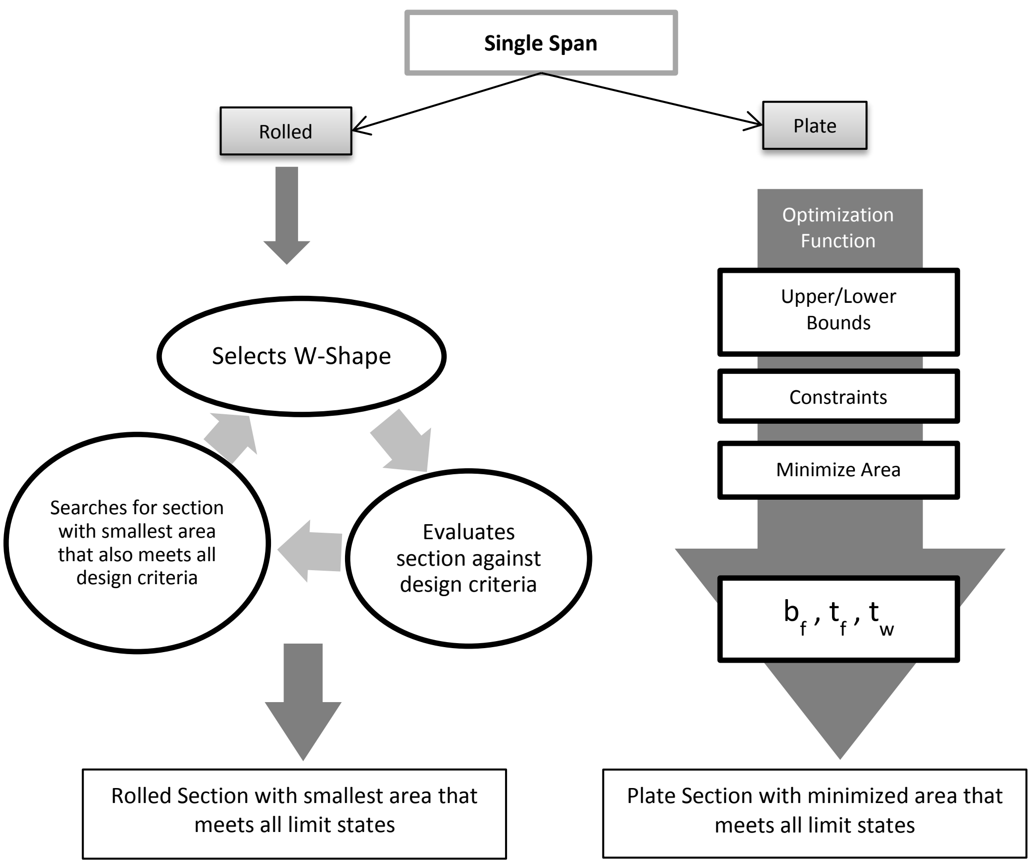


Figure A.3 - Automated member sixing for single span.

Table A.4 - AAHSTO LRFD Specifications constraining member-sizing for all regions of a single span bridge and for the positive moment region of multiple span continuous bridges.

1) **Depth Criteria (2.5.2.6.3)** a) $L * 0.033 \leq D_{beam}$ b) $L * 0.040 \leq D_{section}$ 2) **Ductility (6.10.7.3)** a) $D_p \leq 0.42 * D_{section}$ 3) **Web Thickness (6.7.3)** a) $t_w \geq 0.3125"$ 4) **Section Proportions (6.10.2)** a) $\frac{D_w}{t_w} \leq 150$ b) $\frac{b_f}{2t_f} \leq 12$ c) $b_f \geq \frac{D_w}{6}$ d) $t_f \geq 1.1 * t_w$ $0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$	5) **Service Limit (6.10.4)** a) For Compact: i. $f_{c,t} \leq 0.95 * F_y$ b) For Non-Compact: i. Does not control. (C6.10.4.2.2) 6) **Strength Limit (6.10.6)** a) For Compact: i. $\frac{2D_c}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$ ii. $M_u \leq M_n$ iii. $V_u \leq V_n$ b) For Non-compact: i. $\frac{2D_c}{t_w} \geq 3.76\sqrt{\frac{E}{F_y}}$ ii. $f_{c,t} \leq F_n$ iii. $V_u \leq V_n$

Table A.5 - Additional Fatigue I Limit State constraints.

7) **Fatigue Limit (6.6.1)**	For Load-Induced Fatigue: $\gamma(\Delta f) \leq (\Delta F)_{TH}$

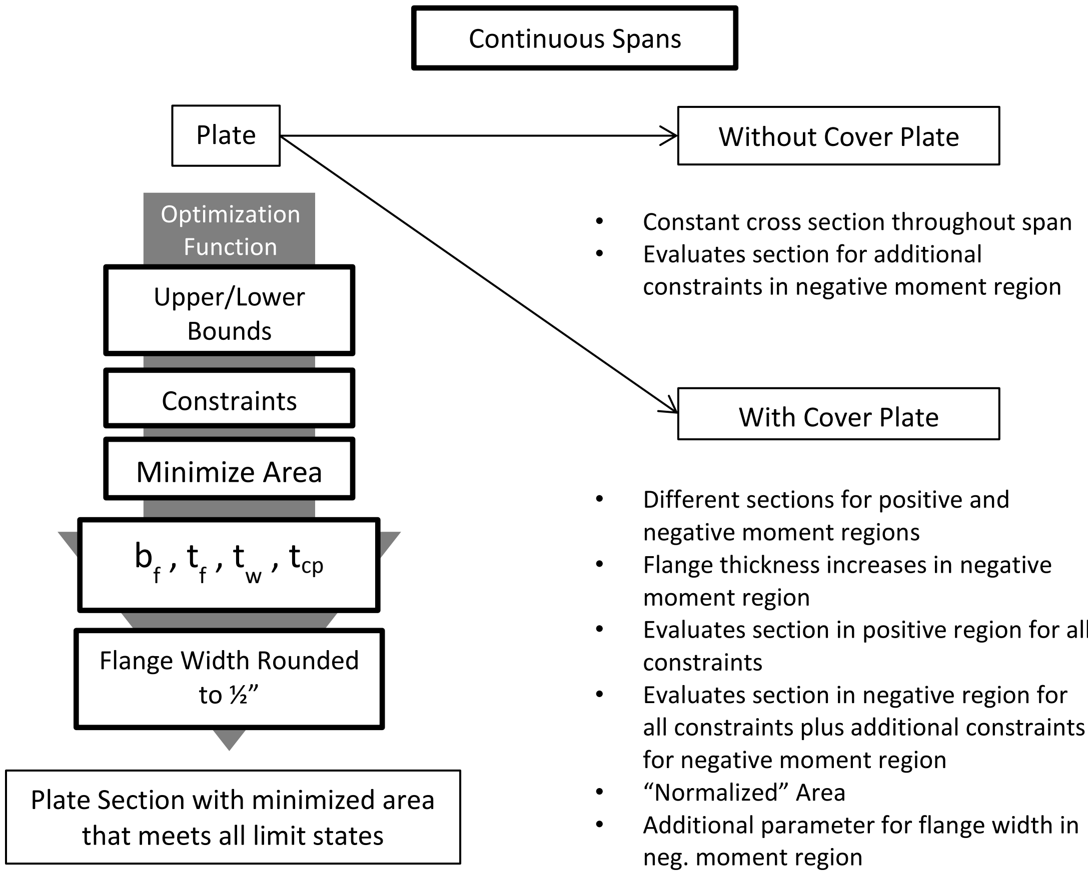


Figure A.4 - Automated member sizing for multiple span continuous.

Input Geometry & Material Properties (from Task 2.1.1)

Assignments:

- Bridge Type (Steel, Pre-stressed)
- Number of Spans
- Span Length
- Road Width
- New/Far Skew
- Deck Thickness
- Sidewalk Height
- Sidewalk Widths
- Barrier Height
- Barrier Width
- Concrete Deck Strength
- Concrete Barrier Strength
- Concrete Sidewalk Strength
- Steel Strength

Diaphragm Configuration (Manual or Automated)

- Select Diaphragm Configuration: Parallel to Skew, Normal to Girder, Staggered
- Select Diaphragm Type manually or choose "Automatically Assign"
- Choose number of interior diaphragm lines based on bridge span length

Assignments :

- Number of Diaphragms
- Diaphragm Spacing
- Diaphragm Dimensions
- Diaphragm Type

Girder Sizing

- Choose Design Method and Design Truck (Default = LRFD and HL-93).
- Assign girder criteria: Span-to-Depth, Number of Girders, Girder Spacing
- Choose levels of composite action
- Assign wearing surface thickness
- For negative moment regions within multi-girder steel bridges, select whether or not to build section with cover plate
- Input dimensions manually of run sizing algorithm to size and evaluate plate girder section against AASHTO design constraints

Assignments (Input or Outputs):

- Number of Girders
- Girder Spacing
- Span-to-Depth
- Composite action
- Wearing surface thickness
- Cover plate length (optional)
- Flange Width
- Flange Thickness
- Web Thickness
- Rolled Section dimensions

Figure 2 - Initial steps in automated member-sizing process.

Perform Single Line Girder Analysis

- Given current design iteration dimensions, computes actions/responses to load cases for single line girder with dimensions input manually or found using the optimization algorithm

Assignments (Outputs):

- Min/Max Moment for Dead Load
- Min/Max Shear for Dead Load
- Max live load displacement
- Min/Max Moment for Live Load
- Min/Max Shear for Live Load

↓

Compute Section Properties and Forces

- Computes section properties such as Moment of Inertia, Elastic Section Modulus, moment arm, etc.
- Computes tension and compression flange stress under load cases for positive and negative (if applicable)moment regions

↓

Compute Section Resistance

- Given current design iteration dimensions, computes flexural and shear strength of the girder following the AASHTO Specifications.
- Flexural strength checks include: local buckling resistance, lateral torsional buckling resistance, and tension flange yielding resistance

Assignments (Outputs):

- Section Capacity for positive moment region
- Section capacity for negative moment region (if applicable)
- Shear Capacity

↓

Return to girder sizing algorithm and iterate until design converges

Figure A.6 - Steps in automated member-sizing continued.

Cover Plates in the Negative Moment Region

While the process is similar between simple and multiple-span continuous bridges, one key consideration with continuous bridges is the negative moment region over the pier(s). In the negative moment region, the cross-section is considered non-composite with the deck. As this region generally governs the design for multiple-span continuous bridges, the automated member sizing software is capable of including cover plates to reinforce this region or to design a plate girder that remains constant throughout all spans. Sizing with constant cross-section girder introduces built-in conservatism in the positive moment region for multiple-span continuous models due to the increased material needed in the negative moment region in order to satisfy the AASHTO LRFD Specifications. For this reason, the least conservative path was chosen, to size the cross section with a cover plate in the negative moment region, allowing both the positive moment region and negative moment region cross-sections to be sized without any arbitrary, additional conservatism. The refined software allows a cover plate to be included over the negative moment region if needed. The associated requirements for a cross-section in the negative moment region are presented in Table A.6.

Table A.6 - Additional AASHTO LRFD Specifications constraining member-sizing for the negative moment region(s) of multiple span continuous bridges

5) **Service Limit (6.10.4)**	6) **Strength Limit (6.10.8)**
a) $f_{c,t} \leq 0.95 * F_y$ b) $f_c \leq F_{crw}$	a) $\frac{2D_c}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$ b) $f_c \leq F_n$ c) $f_t \leq F_y$

In development of the software, an issue arose due to the decision to move to non-uniform cross-sections for continuous bridges. Initial approximation of demand for continuous single-line girder models implicitly assumed a constant cross-section. When the girder section over the pier is stiffened with a cover plate, the increased stiffness acts to attract more moment, and this additional moment was not originally being considered in the sizing of the negative moment region. Modifications to the software introduced a additional layer of iteration to ensure that the elastic distribution of forces is consistent with the selected member sizes.

Sizing Separate Interior and Exterior Sections

Another important consideration in member-sizing is the location of the girder (interior vs exterior). Two strategies were considered for member-sizing in regards to interior and exterior girders. The first strategy was to use a single cross-section for both interior and exterior girders. This strategy was ultimately discarded for the purposes of this project, since—like the constant cross-section approach to continuous bridges— it will provide additional conservatism that some designers may elect to forego. It is possible to have an exterior girder with different effective width and distribution factor than an interior girder. Since effective width and live load distribution factor influence member sizing, this introduces the possibility of having different cross-sections for interior and exterior girders. Forcing both interior and exterior girders to have the same cross-sections will necessarily provide excess capacity to the smaller of the two cross-sections. This does not provide a model with the least conservative member-sizing. For this reason, it was concluded that interior and exterior members would be sized separately.

Validation of the Software

Validation of the software was a critical task of the project. In order to draw accurate conclusions about tolerable support movements, the software must be capable of producing member sizes that are consistent with all the requirements AASHTO LRFD Bridge Design Specifications without excess conservatism. In an effort to validate the member-sizing software, researchers from the University of Delaware acted as independent partners to provide a "peer review" of the model design philosophy and assumptions utilized in the development of the software. A "one-to-one" approach was used in this validation effort. Several designs were conducted by hand, and key parameters of these designs (see Table A.7) were compared to the models created with the software.

Using Microsoft Excel, the research personnel from the University of Delaware developed a design spreadsheet based on the single line girder design method. The spreadsheet requires cross section dimensions, load cases, and bridge orientation information as inputs, and calculates all relevant section properties as well as the flexural and shear strength of the girder following the AASHTO LRFD Specifications. Flexural strength checks include: local buckling resistance, lateral torsional buckling resistance, and tension flange yielding resistance for both composite (positive moment region) and non-composite sections (negative moment region). The shear strength calculations only include the resistance of an unstiffened web. This was done in an attempt to parallel the approach of the automated modeling software, which does not include transverse or longitudinal shear stiffeners. In addition to the strength criteria, all proportional limits were checked. To size the girders using the spreadsheet, calculated moment and shear capacities were checked against the factored load cases to determine the most efficient girder cross section. In this case, efficiency is determined by minimizing the cross sectional area. Several design examples were manually generated utilizing the design spreadsheet in order to compare results and validate the automated member sizing approach.

Table A.7 - "One-to-One" Checks made by University of Delaware Personnel

"One-to-One" Validation Checks	
• Flange Area • Web Area • Girder Depth • Girder Moment of Inertia • Girder Section Modulus • Girder Capacity (Lateral Torsional, Local Buckling Resistance Calculations)	• Composite Moment of Inertia • Composite Section Modulus • Composite Section Capacity (Plastic Neutral Axis and Plastic Moment Calculations) • Single Line Dead Load Computations • Single Line Live Load Computations

The "one-to-one" validation approach was very effective, but in order to validate all parts of the possible design paths a line by line analysis was also performed. This analysis was used to ensure that there were no typos and all appropriate equations were being calculated properly. There were two errors that had a significant effect on the automated sizing. The first was associated with a process that was originally used to merge interior and exterior girder designs, which was ultimately abandoned for reasons described above. The second error was a constraint that forced one of the girder designs to be greater than (instead of less than) the non-compact slenderness limit defined by AASHTO LRFD Eqn. 6.10.6.2.2-1. This caused the software to incorrectly distinguish compact from non-compact cross-sections in the negative moment regions. All errors have since been corrected.

To further validate the process, the flow of the software was compared to the flow charts provided in the AASHTO LRFD Specifications (Section C.6). Results from the 10-model validation study after the Fatigue I Limit State was added are provided in the figures below. The plots compare flange area and Strength I rating factors both with and without the Fatigue I limit state constraints.

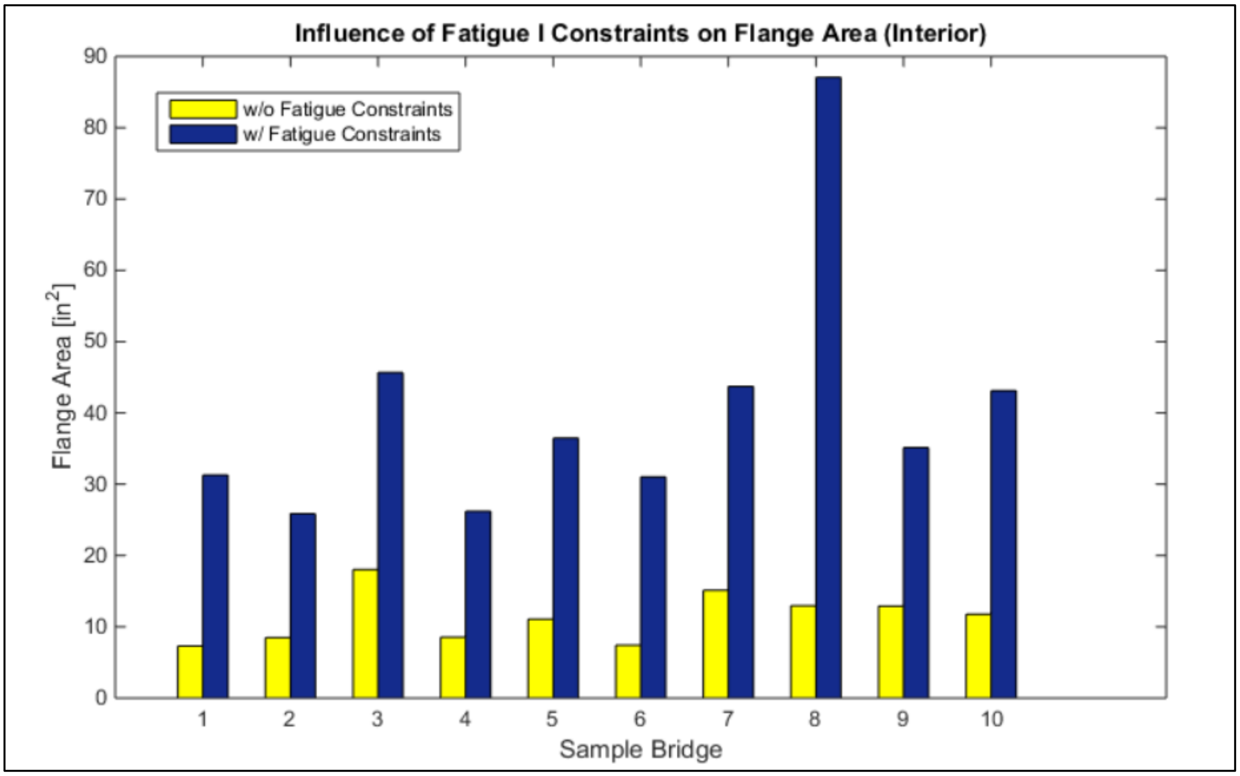


Figure 37 - Influence of Fatigue I constraints on flange area in the positive moment region (for interior girder).

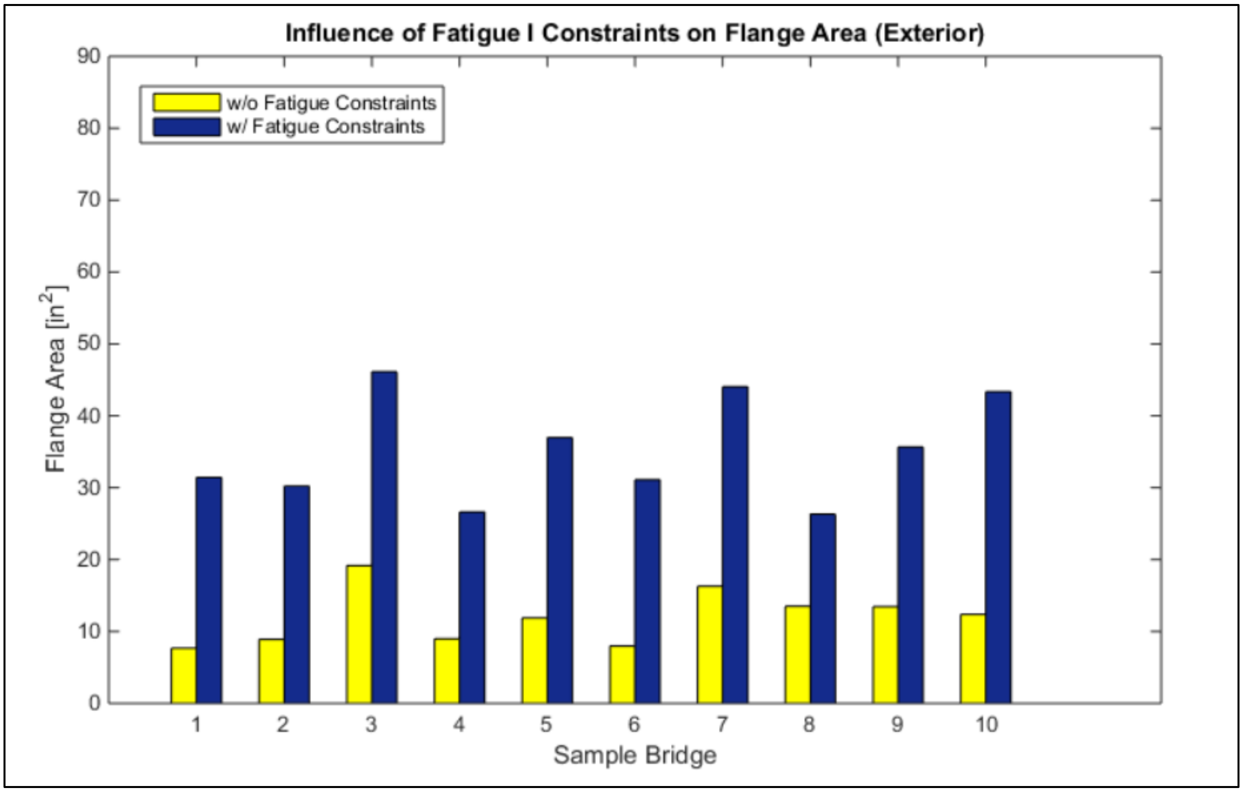


Figure 4 - Influence of Fatigue I constraints on flange area in the positive moment region (for exterior girder).

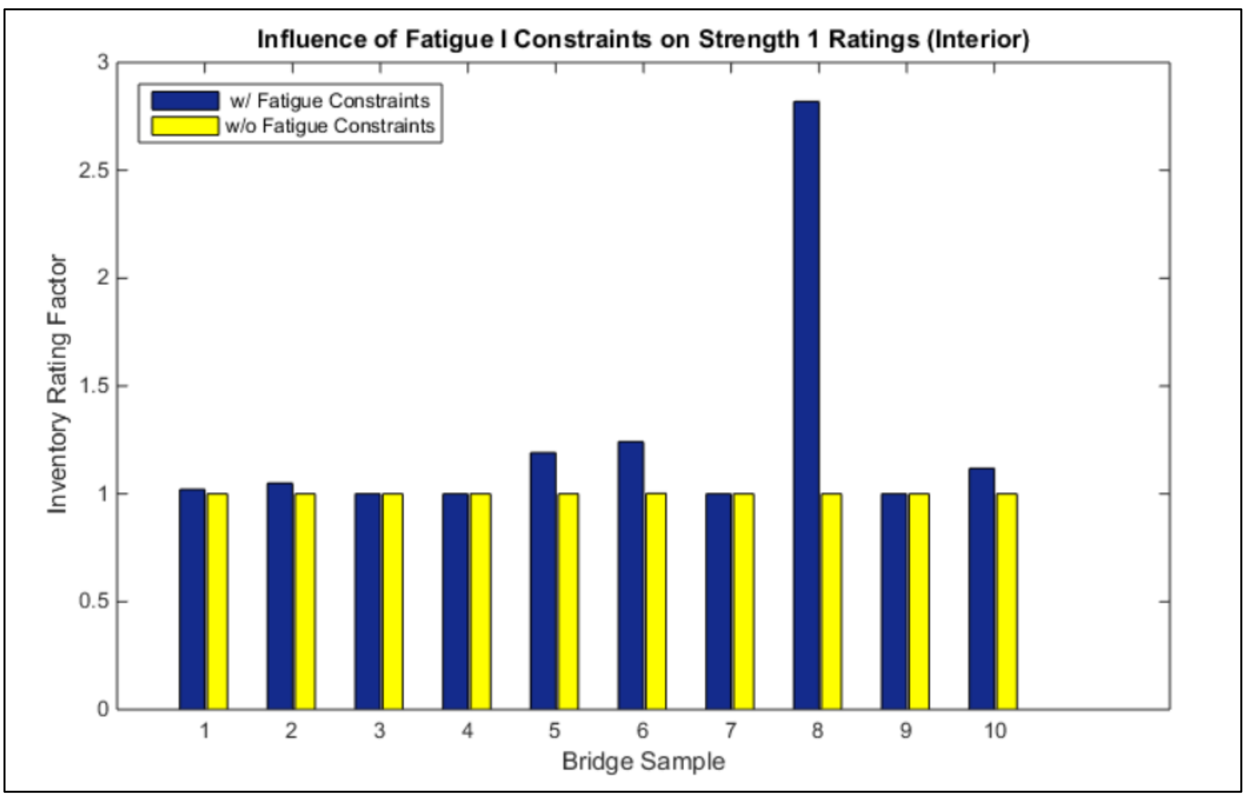


Figure 5 - Influence of Fatigue I constraints on Strength I Inventory Ratings Factor (for interior girder).

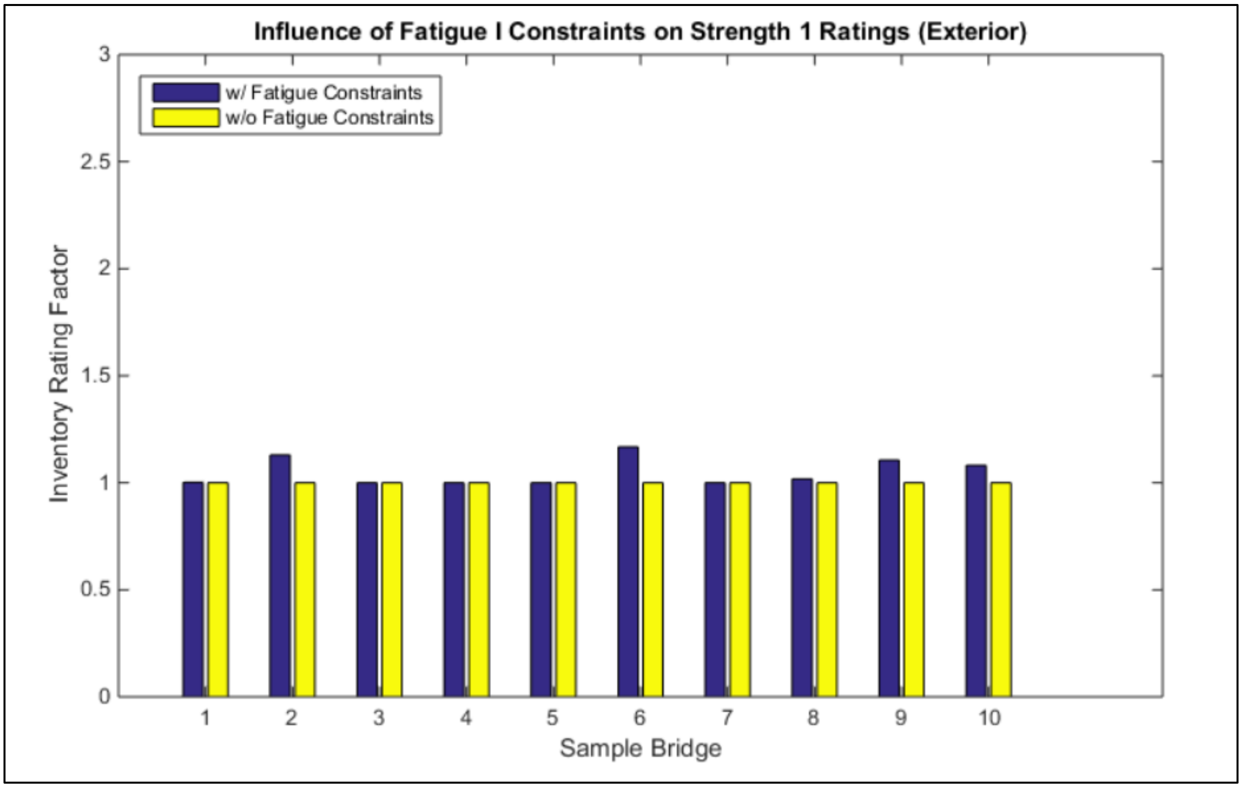


Figure 6 - Influence of Fatigue I constraints on Strength I Inventory Ratings (for exterior girder).

Appendix A.3 – Automated 3D FE Modeling of Bridge Suite

The unique aspect of the research approach is the use of an automated tool to create 3D FE models of multi-girder bridges. This approach makes use of commercially-available FE simulation software (www.strand7.com) that provides a seamless interface with Matlab through an Application Programming Interface (API). The API allows Matlab to drive the model construction and results extraction activities that are normally done through tedious manual interaction. With this approach, all manual interaction with the simulation software is eliminated and replaced by an automated interaction via Matlab. This allows for rapid creation of models with several different design configurations which will be needed in developing the large bridge suite previously described.

To generate a 3D FE model of a bridge, the software requires that a number of parameters and common practices be explicitly defined. Specifically, the required information can be grouped into three categories based on how it will be obtained:

1) Bridge configuration (achieved through sampling the parameters to be carried out under T2.1.1).
2) Member sizes (achieved through automation of the bridge design specifications to be carried out under T2.1.2)
3) Common details of secondary elements (e.g. barrier and sidewalk sizes, diaphragm type and spacing, deck thickness, etc. defined using heuristics)

Once defined, the automation of the FE model construction and results extraction process proceeds in systematic manner as shown in Figure A.11.

Step 1 – Initialization of the API (opens communication between Strand7 and Matlab)

Step 2 – Definition of global geometric grid, mesh size, and nodal locations

Step 3 – Definition of member geometry and nodal connectivity

Step 4 – Definition of continuity and boundary conditions (use of links, offsets, releases as appropriate)

Step 5 – Definition of element geometric and material properties

Step 6 – Definition of loads and load cases (both magnitude and spatial locations)

Step 7 – Execution of analysis cases and extraction of key responses (element actions/stresses, global displacements)

Figure A.11- Summary of automated FE modeling procedure.

Table A.8 provides bridge configuration data for the sample models developed using the automated member-sizing software.

Table A.8 - Member sizing data.

Model Name	Interior				Exterior			
	Flange Width	Flange Thickness	Web Thickness	CP Thickness	Flange Width	Flange Thickness	Web Thickness	CP Thickness
Bridge_1	14.09	1.69	0.93	0.0098	8.67	3.65	0.40	0.0000
Bridge_2	10.50	2.88	0.47	0.0000	10.50	2.91	0.48	0.0000
Bridge_3	13.94	3.33	0.54	0.0000	19.01	2.46	0.56	0.0000
Bridge_4	7.83	3.37	0.35	0.0000	8.12	3.29	0.35	0.0000
Bridge_5	9.00	4.12	0.45	0.0000	13.75	2.70	0.47	0.0000
Bridge_6	8.85	3.52	0.33	0.0000	8.88	3.52	0.33	0.0000
Bridge_7	22.84	1.92	0.53	0.0000	11.07	4.04	0.53	0.0000
Bridge_8	17.02	1.56	0.62	0.0000	18.27	1.48	0.63	0.0000
Bridge_9	11.74	3.02	0.42	0.0000	12.11	2.97	0.43	0.0000
Bridge_10	13.91	2.69	0.89	0.0477	11.15	3.91	0.47	0.0000
Bridge_11	9.23	3.32	2.03	0.4873	18.73	2.33	0.95	0.0311
Bridge_12	9.70	4.22	0.46	0.0000	10.54	3.92	0.47	0.0000
Bridge_13	10.62	3.83	0.34	0.0000	10.68	3.83	0.34	0.0000
Bridge_14	11.67	2.56	0.49	0.0000	11.67	2.58	0.49	0.0000
Bridge_15	10.02	2.79	0.36	0.0000	10.15	2.81	0.36	0.0000
Bridge_16	11.50	2.62	0.54	0.0000	19.42	1.55	0.55	0.0000
Bridge_17	14.79	3.40	0.50	0.0000	20.30	2.50	0.52	0.0000
Bridge_18	9.00	3.23	0.34	0.0000	9.26	3.20	0.35	0.0000

Bridge_19	12.31	3.07	0.92	0.4427	11.06	4.04	0.48	0.0000
Bridge_20	9.26	4.63	0.37	0.0000	9.33	4.63	0.37	0.0000
Bridge_21	14.01	2.50	0.40	0.0000	13.95	2.56	0.40	0.0035
Bridge_22	11.17	3.45	0.53	0.0000	16.42	2.06	0.81	0.0017
Bridge_23	11.90	3.80	0.49	0.0021	12.92	3.46	0.58	0.0021
Bridge_24	11.54	2.54	0.60	0.0629	11.00	2.85	0.47	0.0000
Bridge_25	10.65	4.18	0.43	0.0000	11.48	3.92	0.44	0.0000
Bridge_26	6.64	4.19	0.31	0.0000	6.69	4.19	0.32	0.0000
Bridge_27	18.80	2.25	0.43	0.0003	9.12	4.70	0.42	0.0000
Bridge_28	9.94	3.22	0.37	0.0000	9.93	3.22	0.38	0.0000
Bridge_29	9.61	3.08	0.31	0.0005	8.52	3.47	0.31	0.0000
Bridge_30	11.75	2.81	0.46	0.0000	11.63	2.89	0.46	0.0000
Bridge_31	14.75	3.30	0.41	0.0000	14.75	3.30	0.42	0.0000
Bridge_32	7.89	4.92	0.40	0.0000	7.96	4.92	0.41	0.0000
Bridge_33	9.60	4.61	0.47	0.0000	20.51	2.16	0.49	0.0000
Bridge_34	9.81	3.39	0.36	0.0000	10.38	3.25	0.36	0.0000
Bridge_35	11.11	3.29	0.45	0.0000	11.74	3.16	0.46	0.0000
Bridge_36	10.48	2.10	0.90	0.0107	8.83	3.30	0.40	0.0000
Bridge_37	7.19	3.33	0.36	0.0000	7.42	3.27	0.36	0.0000
Bridge_38	8.25	4.38	0.35	0.0000	8.31	4.38	0.36	0.0000
Bridge_39	14.69	2.10	0.63	0.0002	11.33	2.96	0.49	0.0000
Bridge_40	10.07	3.36	0.39	0.0000	11.44	2.96	0.40	0.0000
Bridge_41	17.57	1.68	0.40	0.0000	7.17	4.20	0.38	0.0000
Bridge_42	11.23	3.32	0.44	0.0000	16.78	2.15	0.67	0.0394
Bridge_43	6.12	3.78	0.31	0.0000	6.17	3.78	0.31	0.0000
Bridge_44	12.27	2.62	0.56	0.0000	15.48	2.10	0.57	0.0000
Bridge_45	11.50	3.40	0.50	0.0000	12.66	3.09	0.51	0.0001
Bridge_46	13.90	3.38	0.37	0.0000	13.91	3.38	0.38	0.0000
Bridge_47	13.61	2.81	0.53	0.0000	14.27	2.72	0.54	0.0000
Bridge_48	8.31	3.94	0.33	0.0000	8.72	3.82	0.34	0.0000
Bridge_49	9.18	3.27	0.35	0.0000	16.90	2.42	0.35	0.0225
Bridge_50	7.75	3.24	0.38	0.0000	8.01	3.20	0.40	0.0000
Bridge_51	10.47	3.10	0.42	0.0031	10.80	3.05	0.42	0.0000
Bridge_52	11.54	2.43	0.41	0.0000	11.87	2.42	0.42	0.0000
Bridge_53	8.14	4.37	0.40	0.0000	13.19	2.71	0.41	0.0000
Bridge_54	12.84	2.34	0.40	0.0000	12.92	2.38	0.41	0.0000
Bridge_55	9.86	4.56	0.41	0.0000	12.91	2.88	1.15	0.0147
Bridge_56	15.62	1.97	0.31	0.0000	7.55	3.99	0.32	0.0000
Bridge_57	16.42	2.72	0.49	0.0000	17.31	2.62	0.50	0.0000
Bridge_58	13.50	2.27	0.55	0.0000	13.50	2.29	0.56	0.0000
Bridge_59	22.00	1.42	0.58	0.0000	13.66	2.35	0.59	0.0000
Bridge_60	12.00	2.70	0.49	0.0000	15.54	2.06	0.50	0.0000
Bridge_61	10.47	3.30	0.36	0.0000	11.12	3.16	0.36	0.0000
Bridge_62	14.64	2.03	0.58	0.0000	25.37	1.18	0.60	0.2035
Bridge_63	17.19	2.59	0.52	0.0000	17.87	2.53	0.53	0.0000

Bridge_64	13.41	2.81	0.51	0.0000	14.08	2.71	0.52	0.0000
Bridge_65	12.00	3.12	0.51	0.0000	12.00	3.13	0.52	0.0000
Bridge_66	15.12	1.73	0.36	0.0066	6.78	3.86	0.31	0.0000
Bridge_67	11.09	2.16	0.42	0.0000	10.75	2.28	0.42	0.0000
Bridge_68	20.09	2.03	0.48	0.0000	16.07	2.49	0.68	0.0542
Bridge_69	13.58	2.62	0.67	0.1267	17.67	2.06	0.61	0.0033
Bridge_70	11.27	3.10	0.69	0.1320	14.48	2.48	0.64	0.1058
Bridge_71	8.08	3.79	0.37	0.0000	8.42	3.68	0.39	0.0000
Bridge_72	13.83	2.47	0.47	0.0000	13.79	2.52	0.47	0.0000
Bridge_73	19.29	1.74	0.50	0.0000	13.19	2.61	0.50	0.0000
Bridge_74	10.47	2.64	0.41	0.0000	10.36	2.72	0.41	0.0000
Bridge_75	12.36	4.15	0.49	0.0000	15.82	3.25	0.50	0.0001
Bridge_76	8.12	4.11	0.41	0.0000	8.59	3.91	0.42	0.0000
Bridge_77	10.28	3.16	0.40	0.0015	9.57	2.45	1.17	0.0864
Bridge_78	12.78	3.06	0.38	0.0000	13.07	3.04	0.39	0.0000
Bridge_79	15.33	3.11	0.45	0.0000	15.28	3.11	0.46	0.0000
Bridge_80	9.69	4.10	0.46	0.0000	18.65	1.75	1.06	0.0033
Bridge_81	6.67	3.66	0.35	0.0000	6.67	3.69	0.35	0.0000
Bridge_82	12.87	3.07	0.95	0.0126	14.71	3.08	0.41	0.0000
Bridge_83	10.70	2.30	0.40	0.0000	10.43	2.42	0.40	0.0000
Bridge_84	15.13	2.74	0.57	0.0000	17.11	2.45	0.58	0.0000
Bridge_85	12.73	2.48	0.37	0.0004	8.05	3.97	0.36	0.0000
Bridge_86	13.17	2.48	0.59	0.0000	14.58	2.27	0.60	0.0000
Bridge_87	6.83	3.95	0.36	0.0000	6.83	3.98	0.36	0.0000
Bridge_88	11.81	2.46	0.33	0.0000	7.62	3.72	0.42	0.0001
Bridge_89	11.17	2.61	0.51	0.0000	14.40	2.03	0.52	0.0001
Bridge_90	13.24	3.83	0.49	0.0000	16.04	3.19	0.51	0.0000
Bridge_91	8.50	4.57	0.44	0.0000	14.41	2.49	0.76	0.3600
Bridge_92	12.36	3.22	0.32	0.0000	14.15	2.84	0.33	0.0000
Bridge_93	8.00	3.41	0.41	0.0000	8.00	3.45	0.42	0.0000
Bridge_94	12.02	2.46	0.33	0.0004	7.56	3.92	0.33	0.0000
Bridge_95	10.80	3.13	1.16	0.1939	17.61	2.42	0.49	0.0000
Bridge_96	13.42	3.21	0.38	0.0000	13.41	3.21	0.39	0.0000
Bridge_97	10.80	3.18	0.38	0.0000	11.16	3.12	0.38	0.0000
Bridge_98	12.83	2.63	0.56	0.0000	12.83	2.66	0.57	0.0000
Bridge_99	11.79	2.89	0.48	0.0000	12.12	2.86	0.48	0.0000
Bridge_100	11.69	3.03	0.87	0.0227	11.67	2.82	0.52	0.0000

Table A.9 provides data on member capacity and live load distribution factor for the sample models developed using the automated member-sizing software.

Table A.9 - Member capacity and live load distribution factor.

Model Name	Interior			Exterior		
	M_n (+)	F_n (-)	DF	M_n (+)	F_n (-)	DF
Bridge_1	1.223E+08	4.352E+04	0.462	1.057E+08	3.775E+04	0.480
Bridge_2	1.427E+08	4.031E+04	0.567	1.410E+08	4.031E+04	0.590
Bridge_3	1.965E+08	4.400E+04	0.693	1.955E+08	4.703E+04	0.769
Bridge_4	6.423E+07	3.946E+04	0.720	5.984E+07	4.002E+04	0.757
Bridge_5	1.322E+08	3.837E+04	0.618	1.327E+08	4.422E+04	0.672
Bridge_6	6.615E+07	3.988E+04	0.512	6.528E+07	3.992E+04	0.549
Bridge_7	1.970E+08	4.853E+04	0.614	1.866E+08	4.125E+04	0.683
Bridge_8	2.280E+08	4.493E+04	0.696	2.291E+08	4.565E+04	0.747
Bridge_9	9.976E+07	4.338E+04	0.788	9.416E+07	4.375E+04	0.821
Bridge_10	1.773E+08	4.343E+04	0.533	1.594E+08	4.121E+04	0.556
Bridge_11	1.438E+08	3.698E+04	0.546	1.569E+08	4.720E+04	0.578
Bridge_12	1.372E+08	3.911E+04	0.658	1.337E+08	4.039E+04	0.723
Bridge_13	7.826E+07	4.298E+04	0.531	7.678E+07	4.305E+04	0.553
Bridge_14	1.638E+08	4.051E+04	0.483	1.627E+08	4.051E+04	0.495
Bridge_15	6.174E+07	4.337E+04	0.788	5.622E+07	4.353E+04	0.812
Bridge_16	1.743E+08	4.168E+04	0.697	1.739E+08	4.726E+04	0.751
Bridge_17	1.662E+08	4.469E+04	0.722	1.641E+08	4.759E+04	0.793
Bridge_18	5.798E+07	4.228E+04	0.707	5.276E+07	4.264E+04	0.739
Bridge_19	1.827E+08	4.157E+04	0.502	1.657E+08	4.064E+04	0.533
Bridge_20	9.518E+07	3.943E+04	0.553	9.189E+07	3.956E+04	0.603
Bridge_21	8.513E+07	4.470E+04	0.826	7.893E+07	4.465E+04	0.843
Bridge_22	1.887E+08	4.011E+04	0.610	2.002E+08	4.451E+04	0.672
Bridge_23	1.180E+08	4.314E+04	0.580	1.175E+08	4.403E+04	0.632
Bridge_24	1.560E+08	4.083E+04	0.444	1.490E+08	4.046E+04	0.524
Bridge_25	1.212E+08	4.186E+04	0.676	1.162E+08	4.283E+04	0.735
Bridge_26	6.129E+07	3.604E+04	0.487	5.914E+07	3.618E+04	0.504
Bridge_27	1.213E+08	4.784E+04	0.589	1.117E+08	3.984E+04	0.644
Bridge_28	9.069E+07	4.147E+04	0.482	9.045E+07	4.143E+04	0.544
Bridge_29	4.445E+07	4.064E+04	0.503	4.321E+07	3.877E+04	0.548
Bridge_30	1.206E+08	4.207E+04	0.739	1.141E+08	4.194E+04	0.750
Bridge_31	1.350E+08	4.436E+04	0.447	1.347E+08	4.435E+04	0.518
Bridge_32	1.125E+08	3.788E+04	0.533	1.085E+08	3.802E+04	0.582
Bridge_33	1.489E+08	4.009E+04	0.610	1.512E+08	4.821E+04	0.672
Bridge_34	6.749E+07	4.168E+04	0.718	6.326E+07	4.243E+04	0.766
Bridge_35	1.175E+08	4.114E+04	0.741	1.126E+08	4.191E+04	0.788
Bridge_36	1.173E+08	3.981E+04	0.458	1.031E+08	3.854E+04	0.481
Bridge_37	6.413E+07	3.817E+04	0.713	5.994E+07	3.870E+04	0.746
Bridge_38	8.120E+07	3.959E+04	0.552	7.853E+07	3.971E+04	0.587
Bridge_39	1.735E+08	4.447E+04	0.481	1.645E+08	4.161E+04	0.545
Bridge_40	1.003E+08	4.060E+04	0.478	1.007E+08	4.235E+04	0.530
Bridge_41	8.847E+07	4.796E+04	0.610	8.224E+07	3.723E+04	0.648

Bridge_42	1.145E+08	4.151E+04	0.716	1.163E+08	4.591E+04	0.750
Bridge_43	4.039E+07	3.385E+04	0.484	3.913E+07	3.434E+04	0.495
Bridge_44	1.950E+08	4.142E+04	0.674	1.928E+08	4.430E+04	0.727
Bridge_45	1.873E+08	4.037E+04	0.475	1.876E+08	4.173E+04	0.508
Bridge_46	1.016E+08	4.405E+04	0.483	1.011E+08	4.406E+04	0.547
Bridge_47	1.744E+08	4.380E+04	0.722	1.689E+08	4.434E+04	0.763
Bridge_48	6.168E+07	3.994E+04	0.607	5.735E+07	4.066E+04	0.650
Bridge_49	5.544E+07	4.279E+04	0.678	6.107E+07	4.859E+04	0.703
Bridge_50	4.150E+07	3.979E+04	0.678	3.749E+07	4.028E+04	0.718
Bridge_51	9.619E+07	4.239E+04	0.778	9.087E+07	4.280E+04	0.809
Bridge_52	4.655E+07	4.412E+04	0.764	4.152E+07	4.443E+04	0.801
Bridge_53	9.699E+07	3.779E+04	0.651	9.622E+07	4.451E+04	0.704
Bridge_54	5.160E+07	4.471E+04	0.763	4.587E+07	4.477E+04	0.787
Bridge_55	1.203E+08	4.030E+04	0.510	1.322E+08	4.307E+04	0.558
Bridge_56	4.451E+07	4.657E+04	0.512	4.069E+07	3.714E+04	0.526
Bridge_57	1.470E+08	4.556E+04	0.796	1.412E+08	4.606E+04	0.838
Bridge_58	2.163E+08	4.310E+04	0.515	2.151E+08	4.309E+04	0.554
Bridge_59	2.139E+08	4.801E+04	0.670	2.078E+08	4.353E+04	0.721
Bridge_60	1.751E+08	4.186E+04	0.468	1.780E+08	4.491E+04	0.543
Bridge_61	6.480E+07	4.279E+04	0.720	6.055E+07	4.353E+04	0.770
Bridge_62	2.011E+08	4.339E+04	0.718	2.036E+08	4.855E+04	0.765
Bridge_63	1.672E+08	4.643E+04	0.778	1.610E+08	4.678E+04	0.814
Bridge_64	1.569E+08	4.301E+04	0.744	1.516E+08	4.360E+04	0.786
Bridge_65	1.946E+08	4.230E+04	0.487	1.940E+08	4.229E+04	0.527
Bridge_66	4.336E+07	4.664E+04	0.586	3.963E+07	3.595E+04	0.611
Bridge_67	8.455E+07	4.233E+04	0.832	7.915E+07	4.193E+04	0.833
Bridge_68	1.287E+08	4.824E+04	0.803	1.273E+08	4.630E+04	0.836
Bridge_69	1.989E+08	4.228E+04	0.500	1.972E+08	4.537E+04	0.549
Bridge_70	1.838E+08	4.033E+04	0.492	1.843E+08	4.377E+04	0.548
Bridge_71	4.087E+07	3.876E+04	0.579	3.888E+07	3.943E+04	0.616
Bridge_72	1.176E+08	4.420E+04	0.827	1.116E+08	4.417E+04	0.844
Bridge_73	1.503E+08	4.714E+04	0.633	1.441E+08	4.333E+04	0.666
Bridge_74	8.910E+07	4.106E+04	0.761	8.295E+07	4.092E+04	0.770
Bridge_75	1.827E+08	4.271E+04	0.542	1.847E+08	4.548E+04	0.593
Bridge_76	1.017E+08	3.747E+04	0.661	9.806E+07	3.845E+04	0.711
Bridge_77	9.627E+07	4.181E+04	0.585	1.042E+08	3.925E+04	0.621
Bridge_78	8.420E+07	4.345E+04	0.726	7.834E+07	4.372E+04	0.755
Bridge_79	1.627E+08	4.495E+04	0.452	1.629E+08	4.490E+04	0.542
Bridge_80	1.353E+08	3.938E+04	0.669	1.519E+08	4.645E+04	0.732
Bridge_81	6.690E+07	3.543E+04	0.589	6.478E+07	3.544E+04	0.605
Bridge_82	1.253E+08	4.345E+04	0.456	1.177E+08	4.541E+04	0.542
Bridge_83	4.514E+07	4.380E+04	0.833	4.155E+07	4.351E+04	0.840
Bridge_84	2.107E+08	4.453E+04	0.710	2.078E+08	4.578E+04	0.780
Bridge_85	7.729E+07	4.406E+04	0.618	7.203E+07	3.749E+04	0.659
Bridge_86	2.255E+08	4.238E+04	0.675	2.237E+08	4.363E+04	0.741

Bridge_87	7.430E+07	3.414E+04	0.588	7.162E+07	3.415E+04	0.615
Bridge_88	3.776E+07	4.495E+04	0.547	3.424E+07	3.915E+04	0.589
Bridge_89	1.596E+08	4.030E+04	0.649	1.585E+08	4.367E+04	0.699
Bridge_90	1.728E+08	4.281E+04	0.612	1.715E+08	4.499E+04	0.689
Bridge_91	1.264E+08	3.781E+04	0.609	1.359E+08	4.474E+04	0.666
Bridge_92	6.815E+07	4.374E+04	0.522	6.742E+07	4.521E+04	0.539
Bridge_93	9.619E+07	3.835E+04	0.630	9.214E+07	3.835E+04	0.666
Bridge_94	5.969E+07	4.500E+04	0.639	5.537E+07	3.888E+04	0.680
Bridge_95	1.721E+08	4.044E+04	0.610	1.550E+08	4.688E+04	0.666
Bridge_96	1.067E+08	4.348E+04	0.469	1.066E+08	4.346E+04	0.544
Bridge_97	8.315E+07	4.149E+04	0.726	7.796E+07	4.194E+04	0.760
Bridge_98	2.156E+08	4.242E+04	0.575	2.131E+08	4.241E+04	0.633
Bridge_99	1.278E+08	4.333E+04	0.792	1.221E+08	4.367E+04	0.823
Bridge_100	2.198E+08	4.105E+04	0.542	1.755E+08	4.156E+04	0.577

Table A.10 provides details of the secondary members for each bridge sample.

Table A.10 - Secondary member data.

Model Name	Diaphragm Type	Diaphragm Section
Bridge_1	'Cross'	'L4X3X1/4'
Bridge_2	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_3	'Chevron'	'L3X2X3/16'
Bridge_4	'Chevron'	'L3X2X3/16'
Bridge_5	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_6	'Chevron'	'L2X2X1/8'
Bridge_7	'Chevron'	'L3X2X3/16'
Bridge_8	'Cross'	'L5X3X1/4'
Bridge_9	'Chevron'	'L5X3X1/4'
Bridge_10	'Cross'	'L3X2X3/16'
Bridge_11	'Chevron'	'L2-1/2X1-1/2X3/16'
Bridge_12	'Chevron'	'L3X2X3/16'
Bridge_13	'Chevron'	'L2X2X1/8'
Bridge_14	'Cross'	'L4X3X1/4'
Bridge_15	'Beam'	'C15X33.9'
Bridge_16	'Chevron'	'L4X3X1/4'
Bridge_17	'Chevron'	'L4X3X1/4'
Bridge_18	'Beam'	'C15X33.9'
Bridge_19	'Cross'	'L4X3X1/4'
Bridge_20	'Chevron'	'L4X3X1/4'
Bridge_21	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_22	'Chevron'	'L5X3X1/4'
Bridge_23	'Chevron'	'L3X2X3/16'
Bridge_24	'Cross'	'L3X2X3/16'

Bridge_25	'Chevron'	'L5X3X1/4'
Bridge_26	'Chevron'	'L5X3X1/4'
Bridge_27	'Chevron'	'L6X3-1/2X5/16'
Bridge_28	'Cross'	'L2X2X1/8'
Bridge_29	'Beam'	'C15X33.9'
Bridge_30	'Chevron'	'L6X3-1/2X5/16'
Bridge_31	'Cross'	'L3X2X3/16'
Bridge_32	'Chevron'	'L6X3-1/2X5/16'
Bridge_33	'Chevron'	'L5X3X1/4'
Bridge_34	'Chevron'	'L3X2X3/16'
Bridge_35	'Chevron'	'L3X2X3/16'
Bridge_36	'Cross'	'L4X3X1/4'
Bridge_37	'Chevron'	'L3X2X3/16'
Bridge_38	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_39	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_40	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_41	'Chevron'	'L4X3X1/4'
Bridge_42	'Chevron'	'L5X3X1/4'
Bridge_43	'Beam'	'C15X33.9'
Bridge_44	'Chevron'	'L5X3X1/4'
Bridge_45	'Cross'	'L4X3X1/4'
Bridge_46	'Chevron'	'L2X2X1/8'
Bridge_47	'Chevron'	'L5X3X1/4'
Bridge_48	'Beam'	'C15X33.9'
Bridge_49	'Beam'	'C15X33.9'
Bridge_50	'Beam'	'C12X20.7'
Bridge_51	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_52	'Beam'	'C12X20.7'
Bridge_53	'Chevron'	'L2-1/2X1-1/2X3/16'
Bridge_54	'Beam'	'C12X20.7'
Bridge_55	'Chevron'	'L5X3X1/4'
Bridge_56	'Beam'	'C12X20.7'
Bridge_57	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_58	'Cross'	'L5X3X1/4'
Bridge_59	'Chevron'	'L5X3X1/4'
Bridge_60	'Cross'	'L3X2X3/16'
Bridge_61	'Beam'	'C15X33.9'
Bridge_62	'Chevron'	'L5X3X1/4'
Bridge_63	'Chevron'	'L5X3X1/4'
Bridge_64	'Chevron'	'L5X3X1/4'
Bridge_65	'Cross'	'L3-1/2X2-1/2X1/4'
Bridge_66	'Beam'	'C15X33.9'
Bridge_67	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_68	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_69	'Cross'	'L2-1/2X1-1/2X3/16'

Bridge_70	'Cross'	'L3X2X3/16'
Bridge_71	'Beam'	'C12X20.7'
Bridge_72	'Chevron'	'L5X3X1/4'
Bridge_73	'Chevron'	'L7X4X3/8'
Bridge_74	'Chevron'	'L5X3X1/4'
Bridge_75	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_76	'Chevron'	'L3X2X3/16'
Bridge_77	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_78	'Chevron'	'L6X3-1/2X5/16'
Bridge_79	'Cross'	'L2X2X1/8'
Bridge_80	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_81	'Chevron'	'L2-1/2X1-1/2X3/16'
Bridge_82	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_83	'Beam'	'C12X20.7'
Bridge_84	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_85	'Chevron'	'L4X3X1/4'
Bridge_86	'Cross'	'L3-1/2X2-1/2X1/4'
Bridge_87	'Chevron'	'L3X2X3/16'
Bridge_88	'Beam'	'C10X15.3'
Bridge_89	'Chevron'	'L3X2X3/16'
Bridge_90	'Chevron'	'L4X3X1/4'
Bridge_91	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_92	'Chevron'	'L3X2X3/16'
Bridge_93	'Chevron'	'L5X3X1/4'
Bridge_94	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_95	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_96	'Cross'	'L2-1/2X1-1/2X3/16'
Bridge_97	'Chevron'	'L5X3X1/4'
Bridge_98	'Cross'	'L5X3X1/4'
Bridge_99	'Chevron'	'L3-1/2X2-1/2X1/4'
Bridge_100	'Cross'	'L5X3X1/4'

Appendix B – Task 2.2: Estimation of Maximum Tolerable Support Movements

Figures B.1 through B.20 provide results on superstructure response to vertical support movement of the abutment for the five continuously sampled parameters (span length, skew, span/depth, girder spacing) combined with the four discrete parameters (support condition, barrier stiffness contribution).

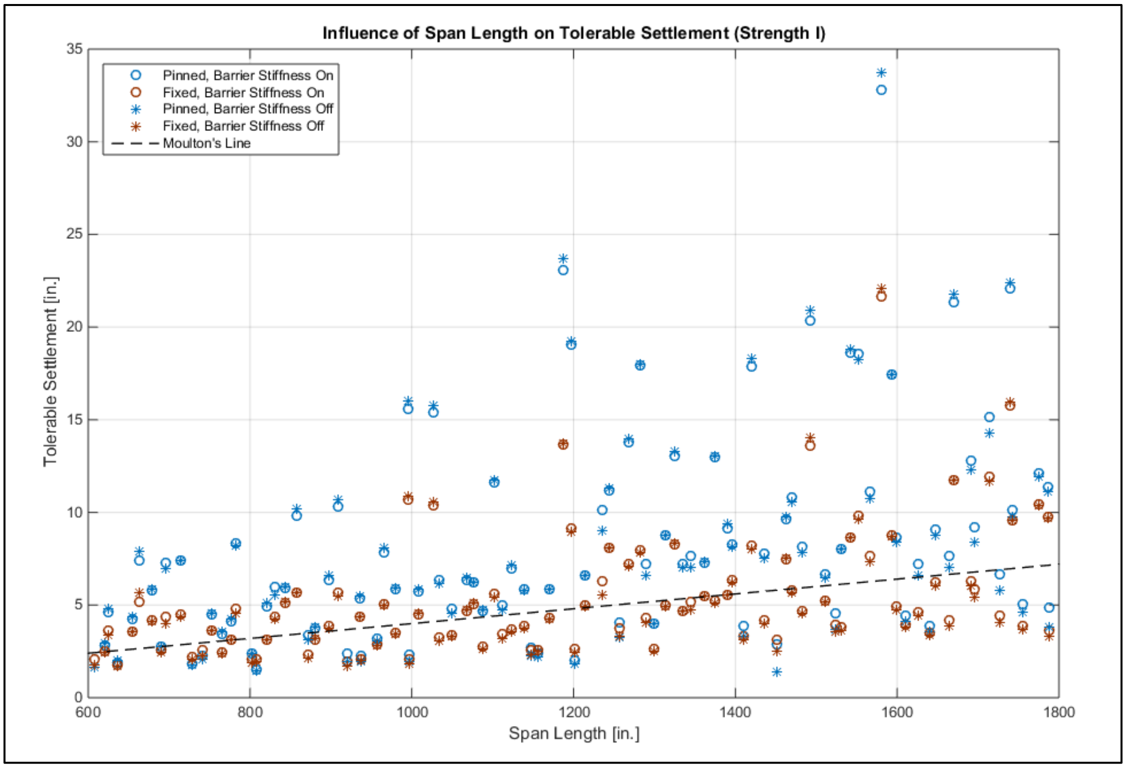


Figure B.1 - Influence of span length on tolerable support movement (Strength I Limit State).

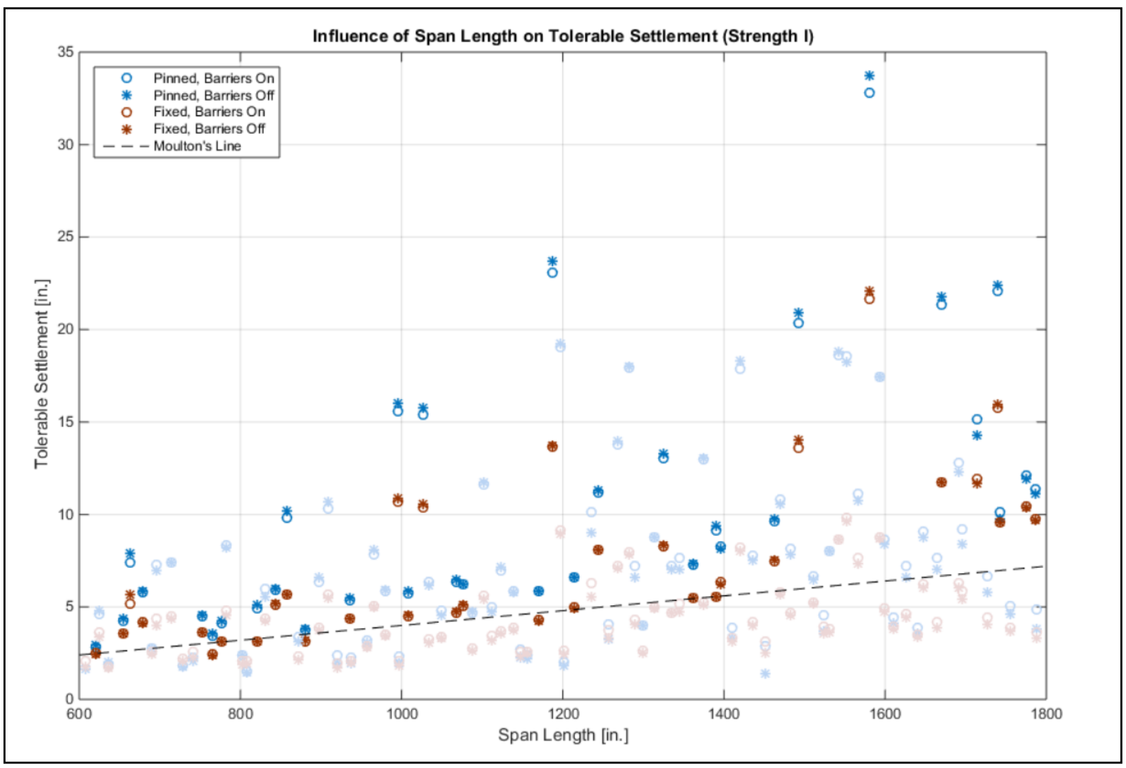


Figure B.2 - Influence of span length on tolerable support movement (bridges with skew angles greater than 20° are faded) (Strength I Limit State)

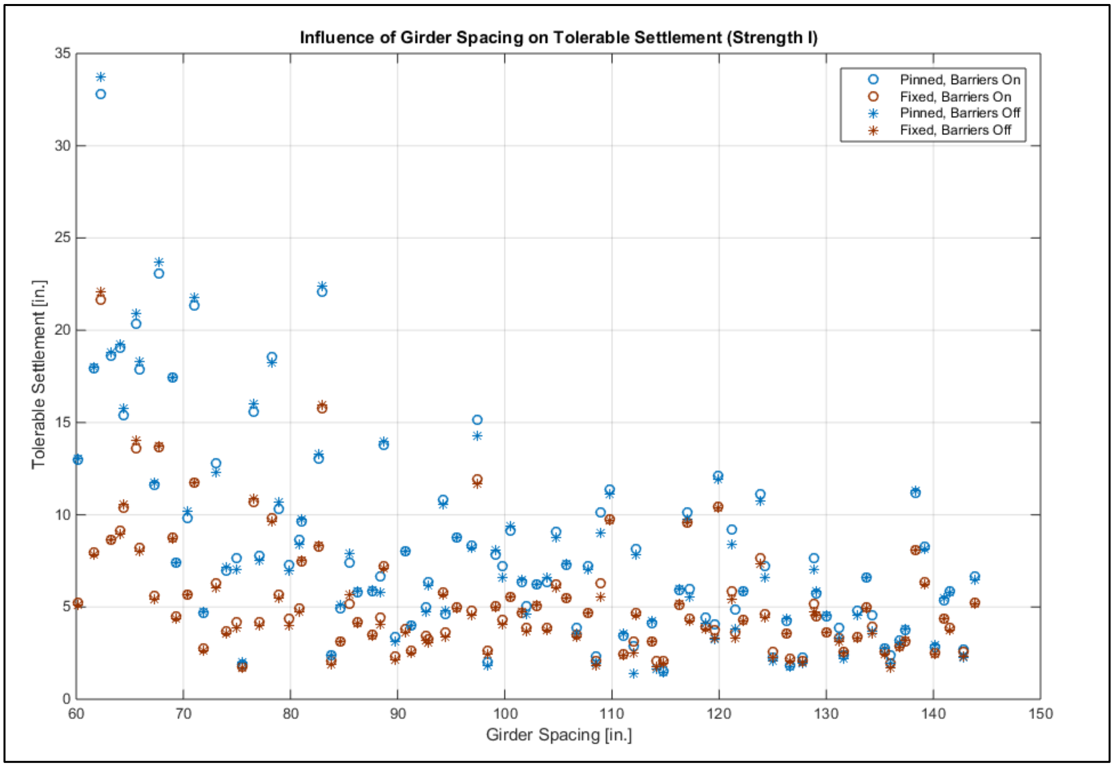


Figure B.3 - Influence of girder spacing on tolerable support settlement (Strength I Limit State).

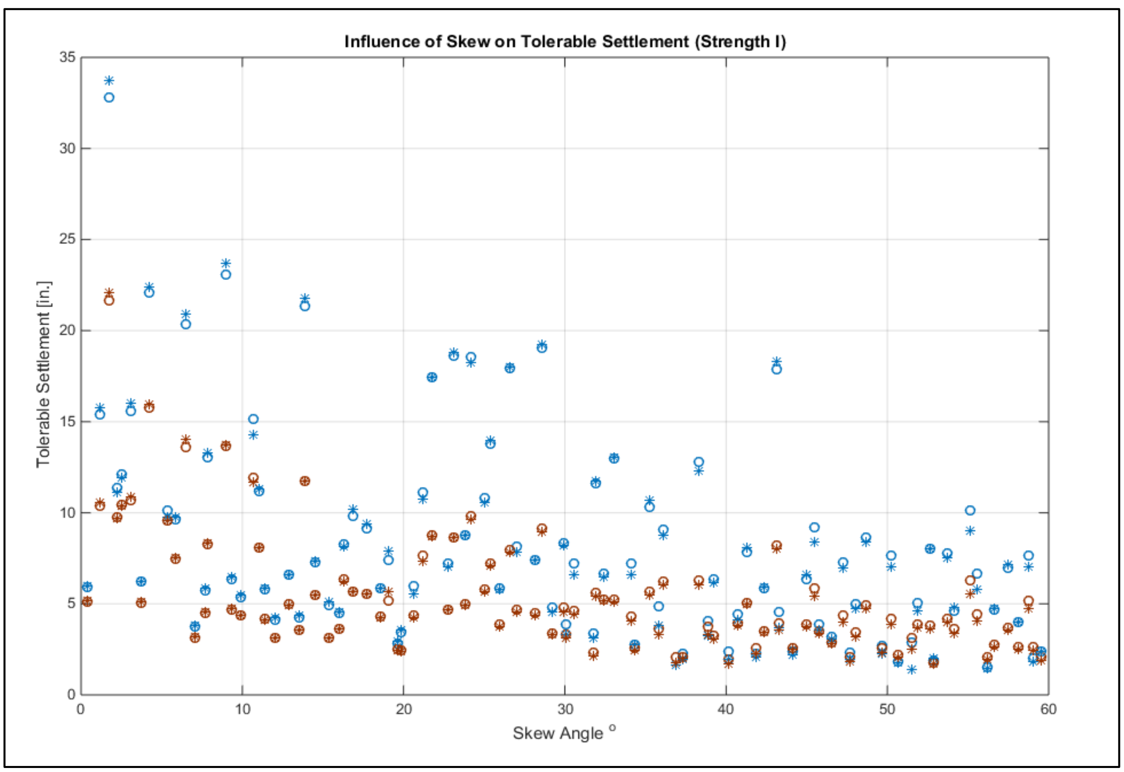


Figure 7 - Influence of skew on tolerable support movement (Strength I Limit State).

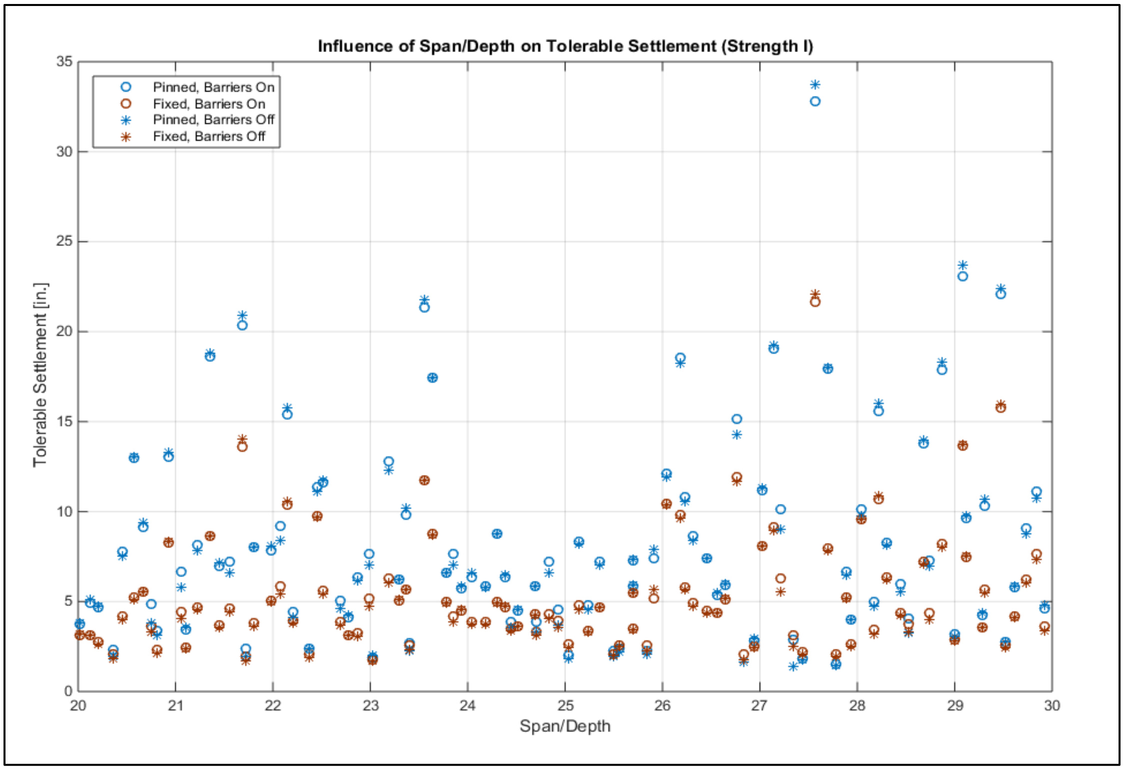


Figure B.5 - Influence of span/depth on tolerable support movement (Strength I Limit State).

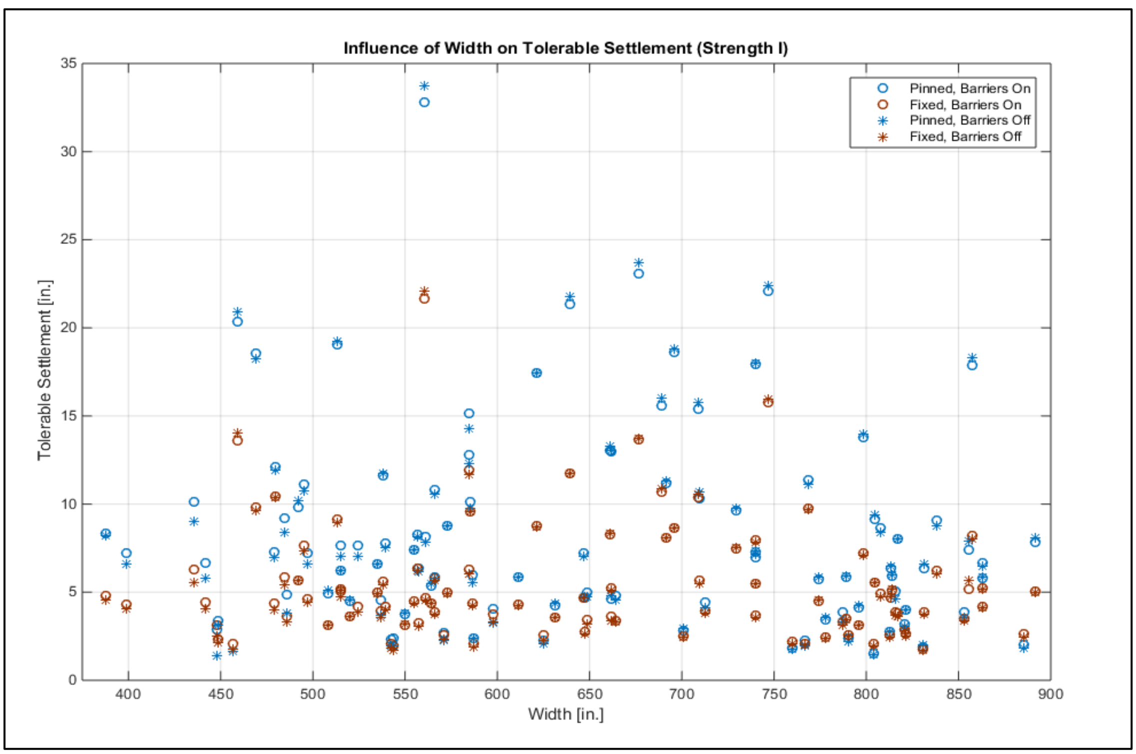


Figure 8 - Influence of width on tolerable support movement (Strength I Limit State).

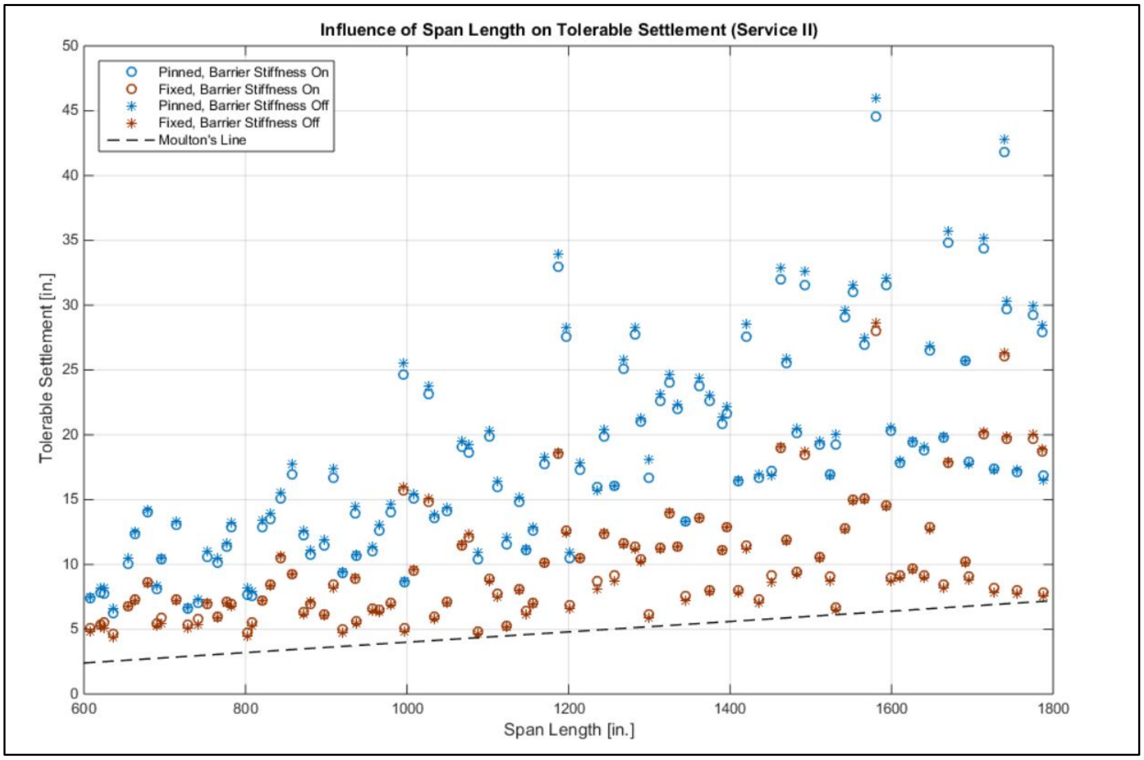


Figure B.7 - Influence of span length on tolerable support movement (Service II Limit State).

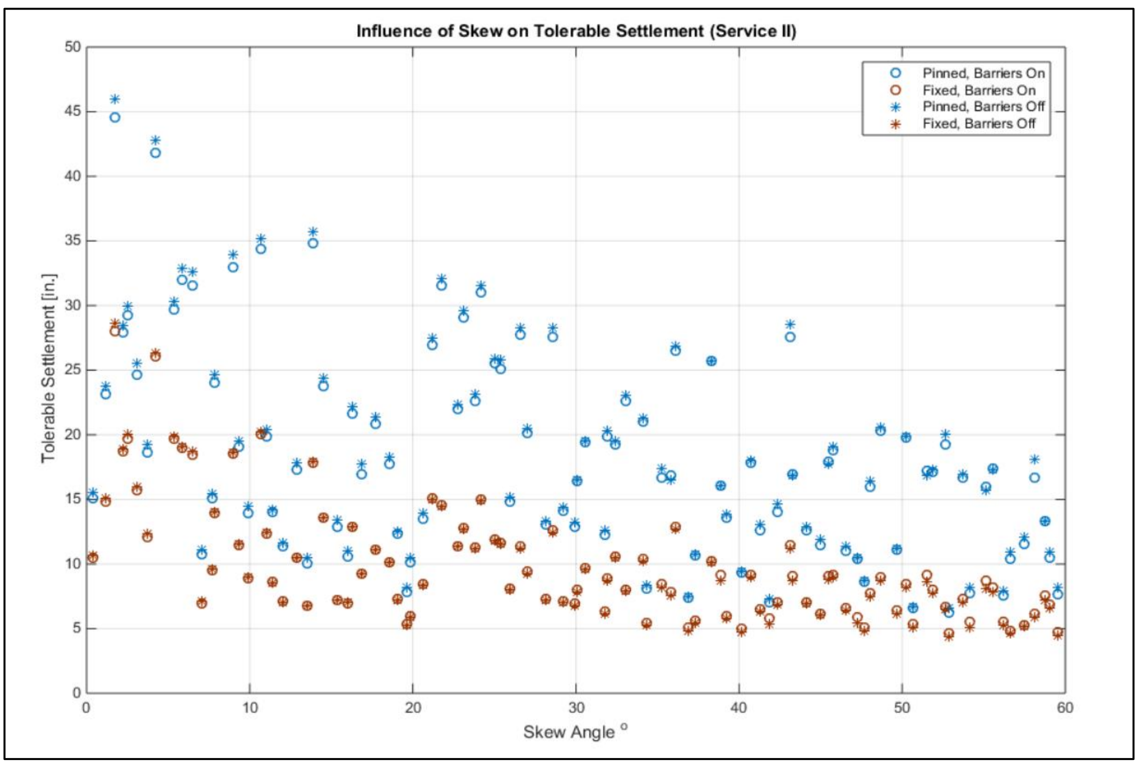


Figure 9 - Influence of skew on tolerable support movement (Service II Limit State).

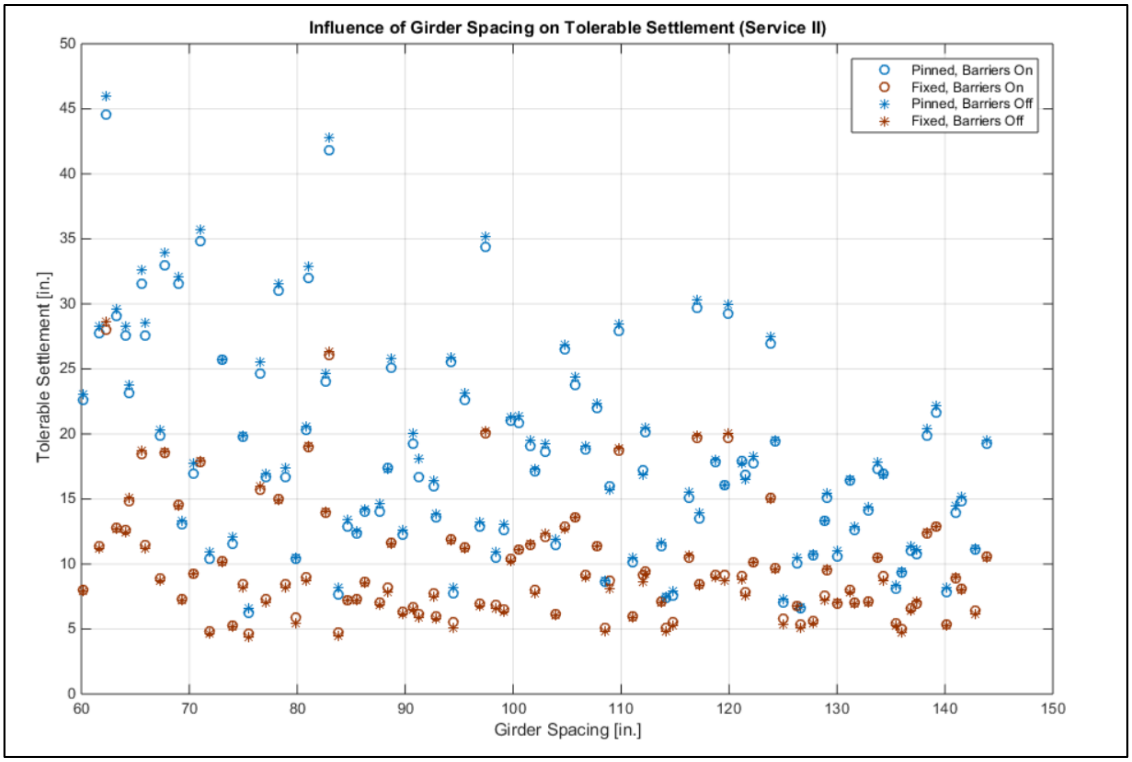


Figure 10 - Influence of girder spacing on tolerable support movement (Service II Limit State).

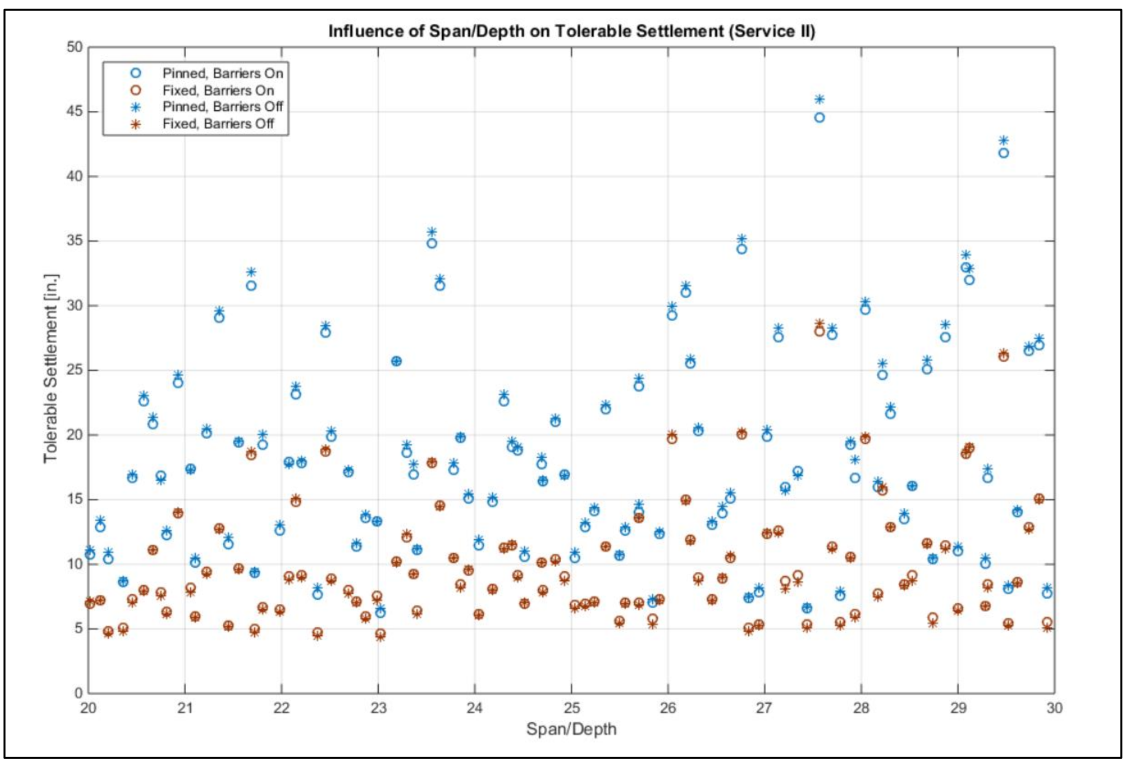


Figure 110 - Influence of Span/Depth on tolerable support movement (Service II Limit State).

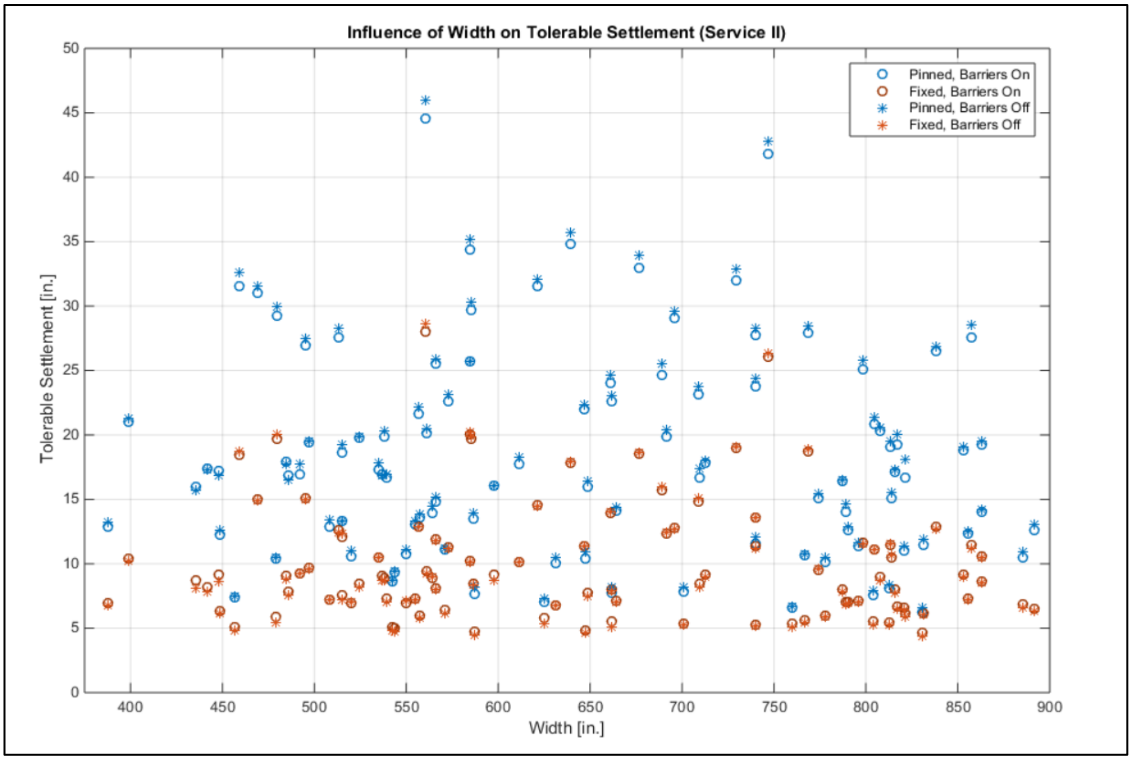


Figure 12 - Influence of width on tolerable support movement (Service II Limit State).

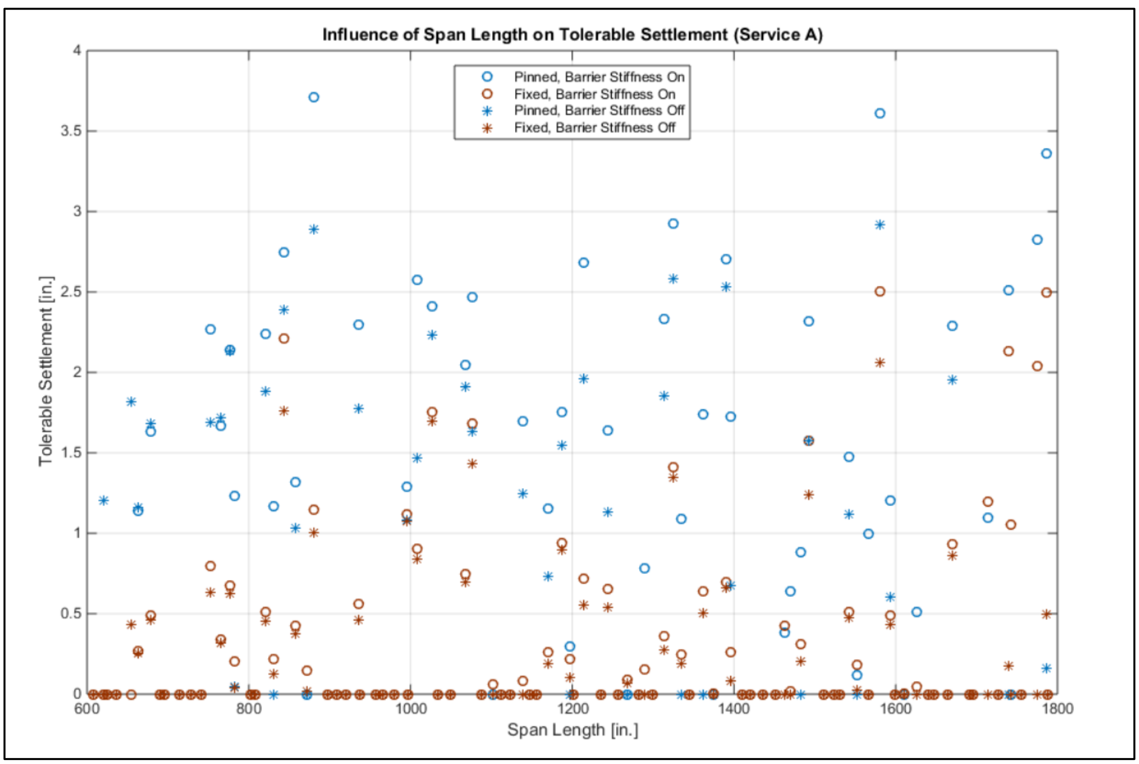


Figure 13 - Influence of span length on tolerable support movement (Service A Limit State).

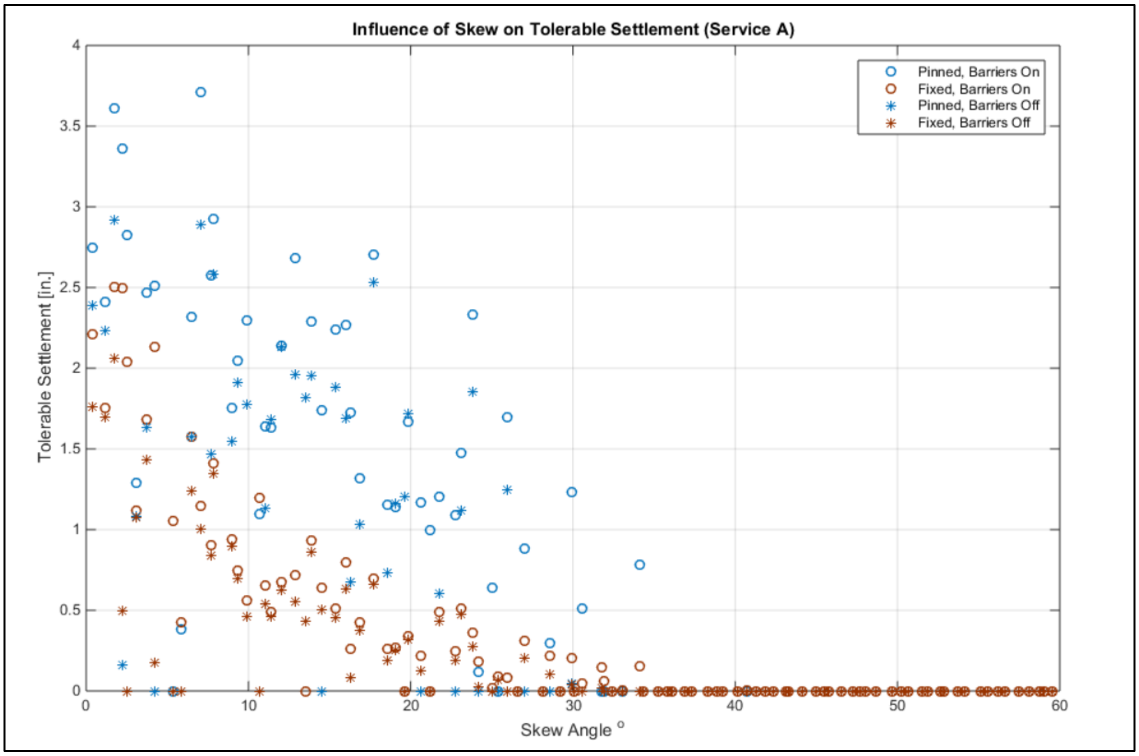


Figure B.13 - Influence of skew on tolerable support movement (Service A Limit State).

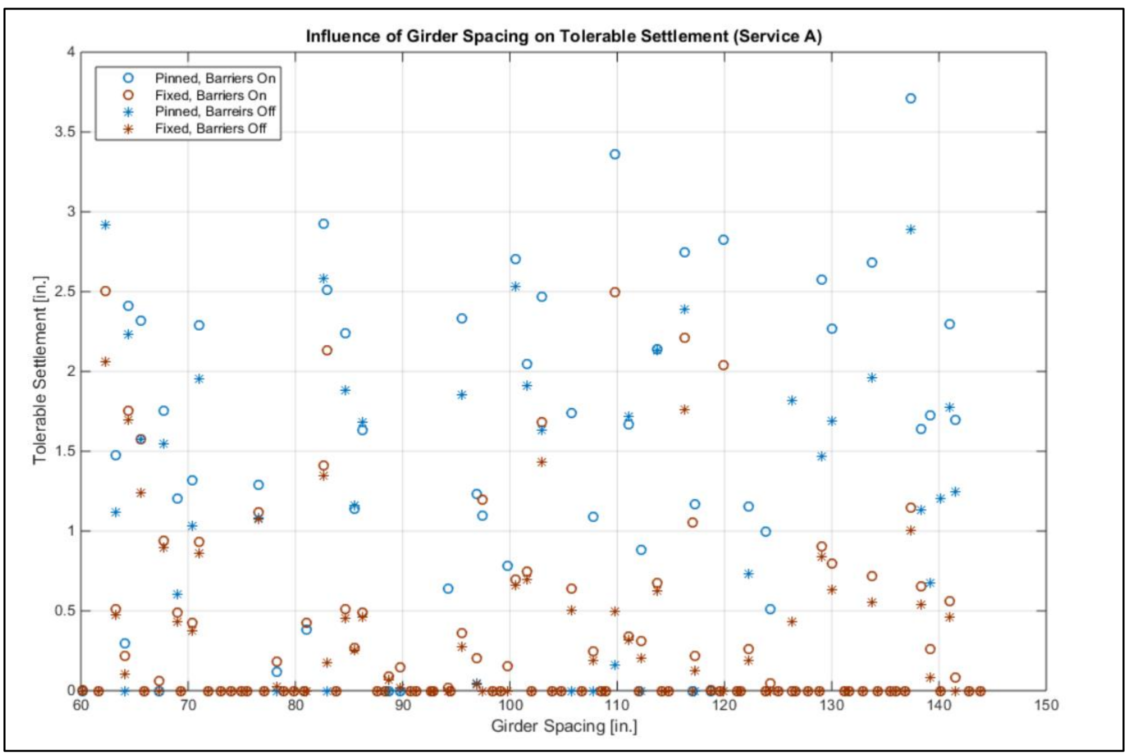


Figure B.14 - Influence of girder spacing on tolerable support movement (Service A Limit State).

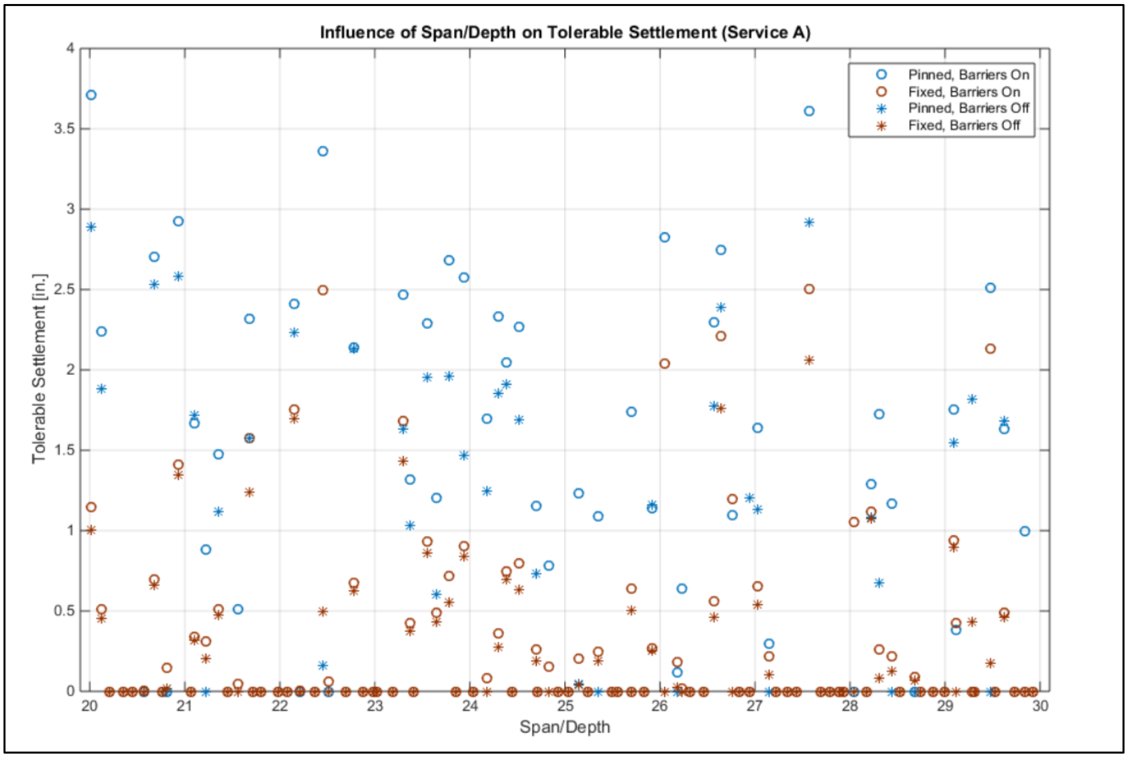


Figure B.15 - Influence of Span/Depth on tolerable support movement (Service A Limit State).

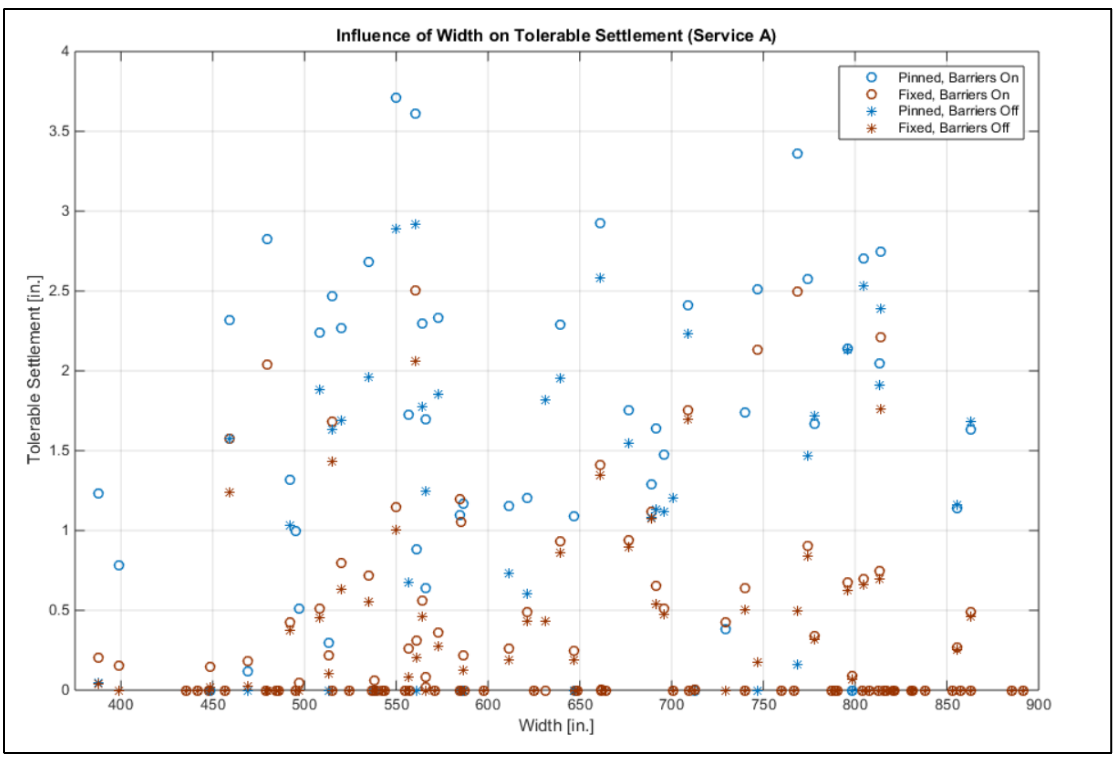


Figure B.16 - Influence of width on tolerable support movement (Service A Limit State).

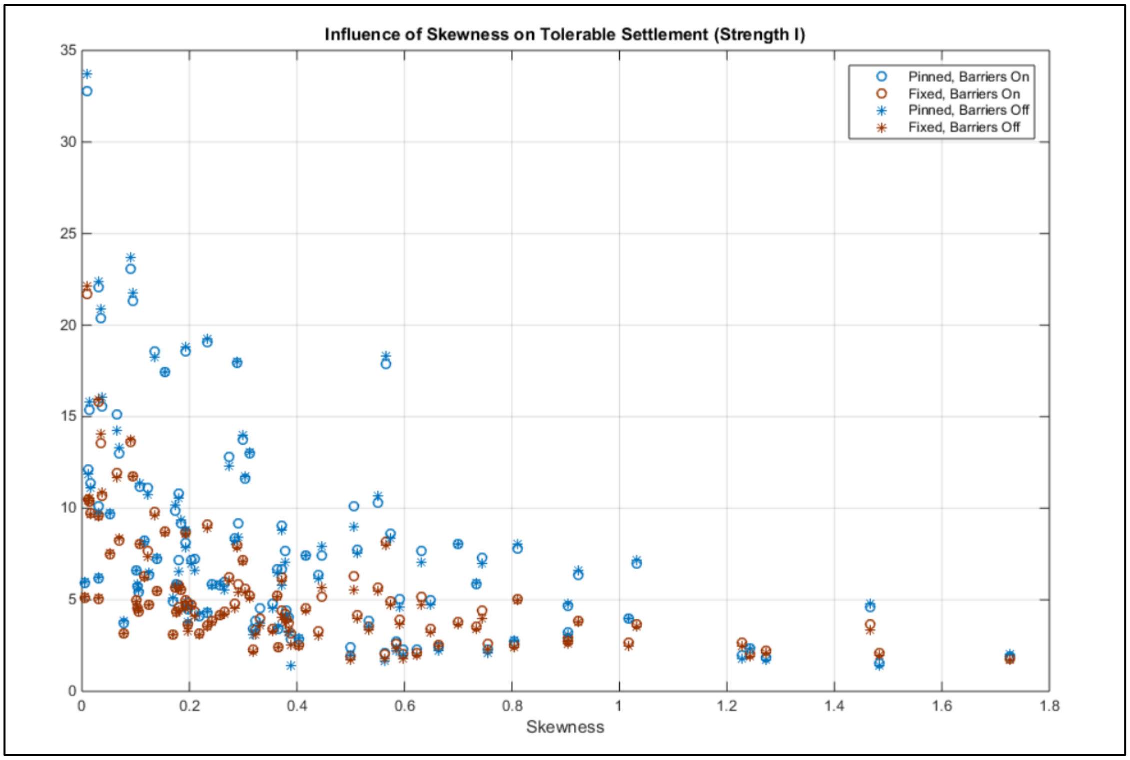


Figure B.17 - Skewness Influence on tolerable support movement (Strength I Limit State).

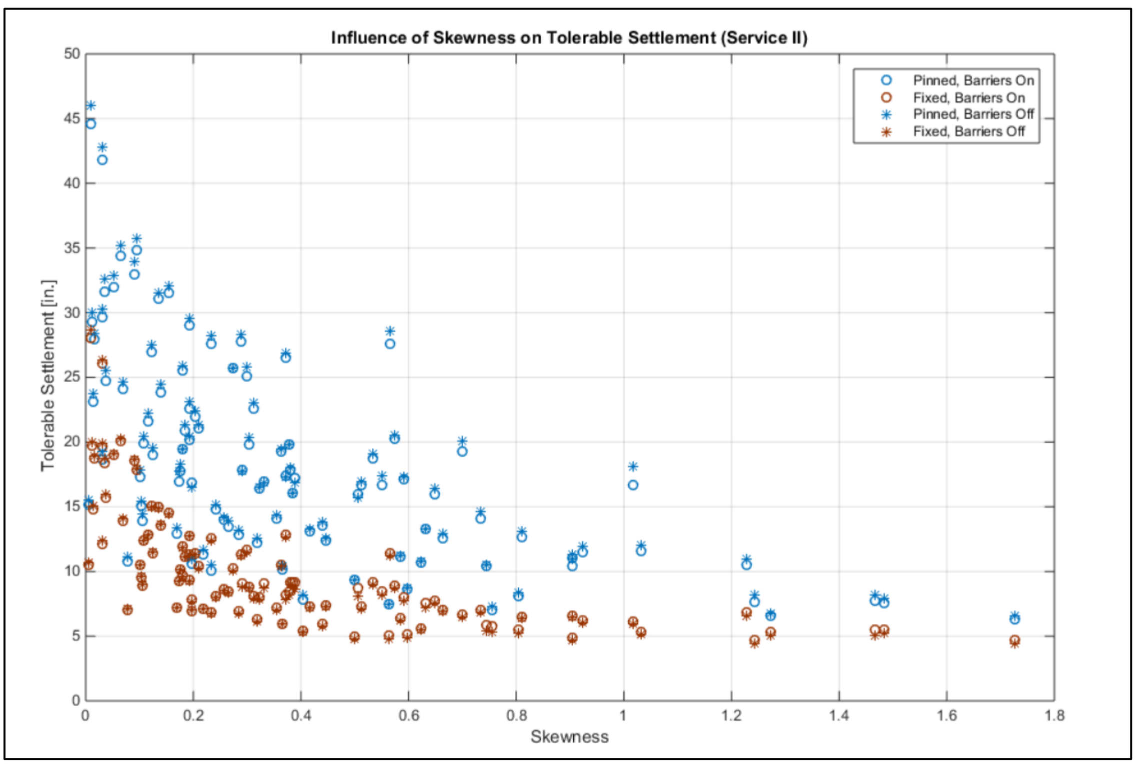


Figure B.18 - Skewness influence on tolerable support movement (Service II Limit State).

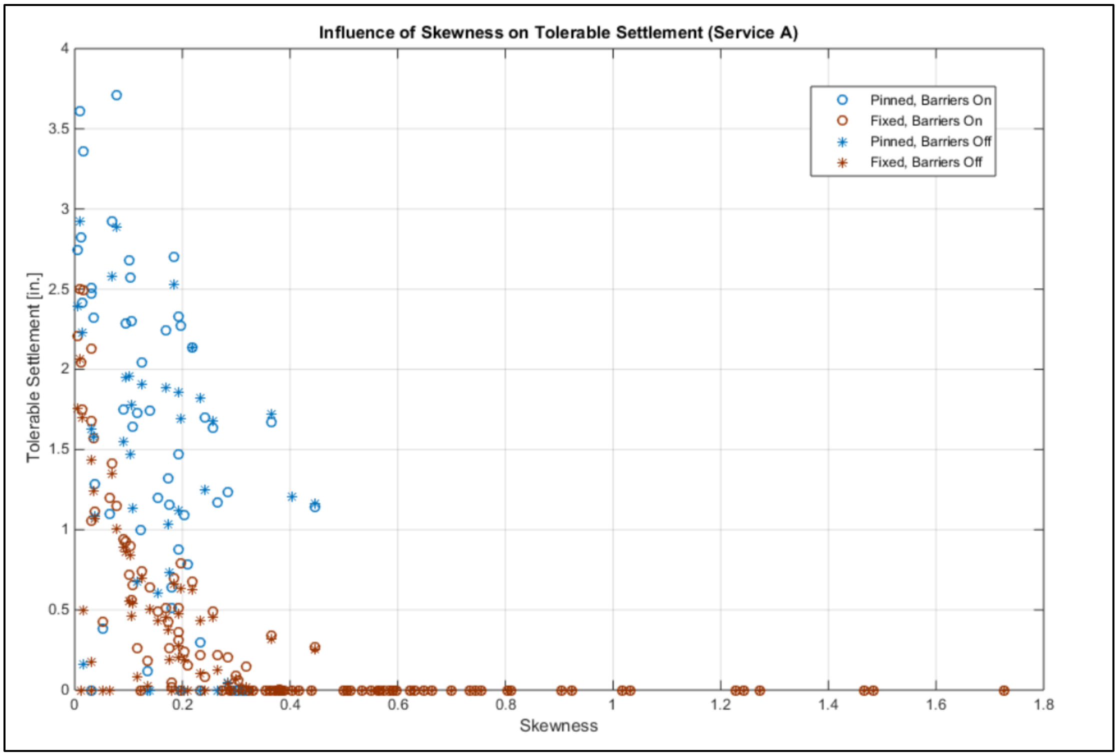


Figure B.19 - Skewness influence on tolerable support movement (Service A Limit State).

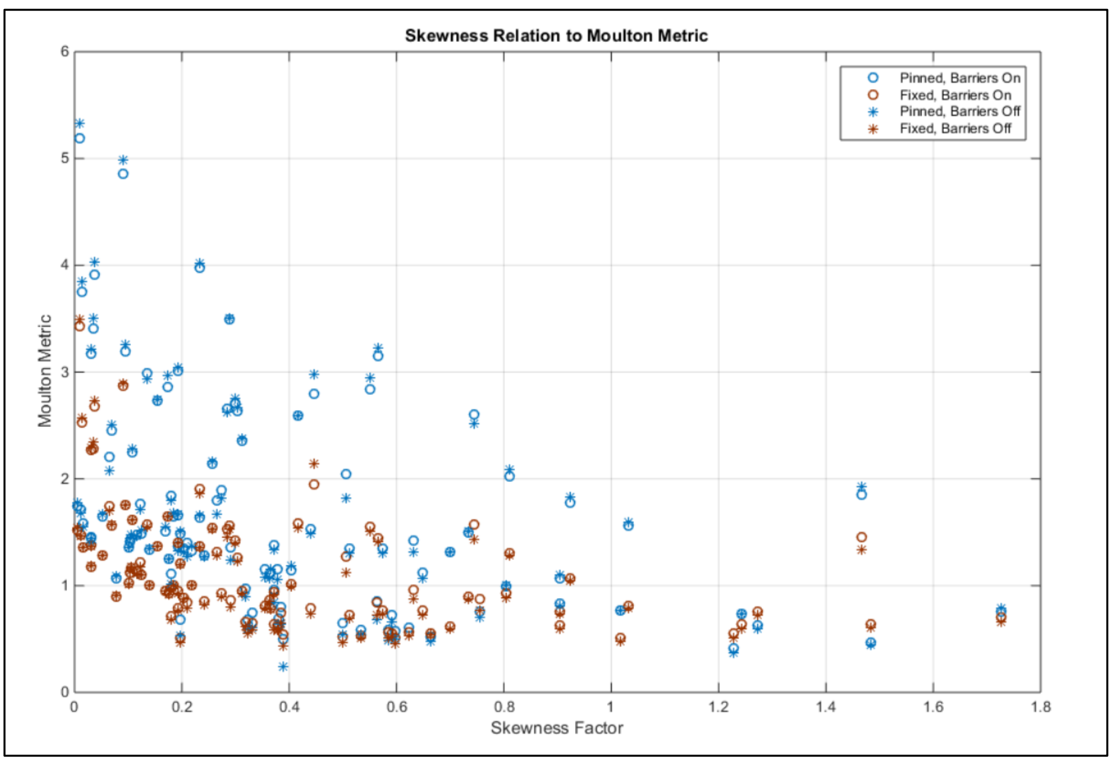


Figure B.20 - Skewness vs Moulton Metric (Strength I Limit State)

Appendix C – Project Schedule and Budget

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

PROGRESS SCHEDULE

NCHRP Project No. 12-103 FY 2015 Interim Report No 07
Research Agency Drexel University
Principal Investigator Franklin Moon

	RESEARCH TASK	2014								2015												2016										ESTIMATED % COMPLETION
		J	F	M	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	
Phase I	T1.1 - Fact Finding, Literature and Standards Review	50	50																													100
		50	50																													
	T1.2 - Selection of Critical Bridge Parameters	40	40	20																												100
		40	40	20																												
	T1.3 - Modeling Study for Non-Multi-Girder Bridges	40	40	20																												100
		40	40	20																												
	T1.4 - Revision of Phase II Work Plan		50	50																												100
			50	50																												
	T1.5 - ID Design Specifications that may Require Revisions		50	50																												100
			50	50																												
	T1.6 - Reporting		50	50																												100
			50	50																												
	NCHRP Review & Meeting #1				R																											
Phase II	T2.1 - Def and Const of 3D FE Bridge Models					5	10	10	10	10	5	5	5	0																		
						10	20	20	20	20	10																					
	T2.2 - Estimation of Maximum Tolerable Support Movements										15	15	15	0																		
											25	25	25	25																		
	T2.3 - ID of Parameters that influence Tol. Sup. Mov.												10	0	0																	
													30	30	40																	
	T2.4 - Spot Checking Additional Bridge Types																															
															30	30	40															
	T2.5 - Reporting																															
																30	30	30	10													
	NCHRP Review & Meeting #2																			R												
Phase III	T3.1 - Development of Recommendations																															
																					50	50										
	T3.2 - ID the Influence of Proposed Ballot Items																															
																						50	50									
	T3.3 - Reporting																															
																							40	40	30							
	NCHRP Review																									R						
IV	T4.1 - Submission of Final Deliverables																															
																											20	20	20	20	20	
	NCHRP Review																													Final		
	Planned % Complete	2	5	10	13	16	19	22	25	30	35	40	45	50	56	62	68	74	80	82	84	86	88	90	92	94	96	97	98	99	100	
	Overall % Completion	1	3	10	13	16	19	22	25	30	35	40	45	45	45																	
Deliverables	Monthly Progress Reports	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	31	32	33	34	
	Quaterly Progress Reports			1		3			4			5			6			7			8			9			10			11		
	Interim/Final Reports			IR											IR						IR								FRD		FR	
	Meeting with Project Panel				M											M																

FIG. A -- OVERALL PROJECT SCHEDULE

FIG. B -- CONTRACT FUNDS

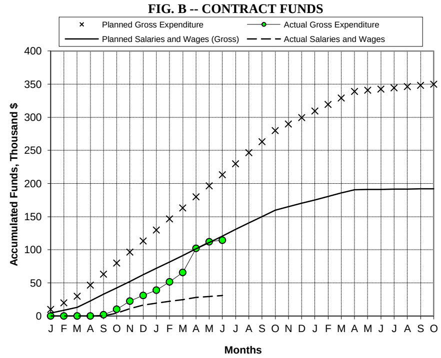
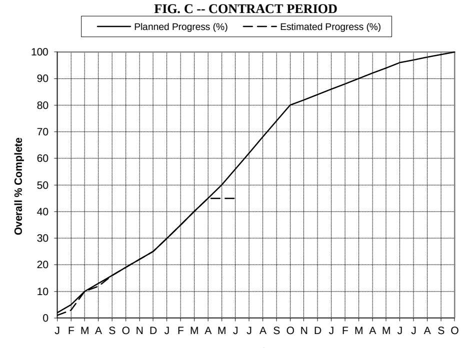


FIG. C -- CONTRACT PERIOD



Funds Expended	% 38.7	Time Expended	% 45
Contract Amount	$ 290000	Starting Date	1/1/2014
Expended This Month	$ 10023	Completion Date	6/30/2016
Total Exp. to Date	$ 112150.05		
Balance	$ 177849.95		

Salaries and Wages Estimated This Mo	$ 9780
Salaries and Wages Spent This Month	$ 1500
Accumulated Salaries and Wages To Da	$ 29500.01