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Load Rating Mobility As A Framework For Bridge Owners

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LOAD RATING MOBILITY AS A FRAMEWORK FOR BRIDGE OWNERS

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Abstract

In this time of financial economy, bridge owners are faced with the important decision of where to focus financial assets while attempting to determine which structures are in most need. Bridge Engineers and researchers alike agree that a large portion of the United States bridge population are reaching the end of their design life. Researchers are currently investigating various means and methods of lengthening the amount of usefulness of these key pieces of infrastructure. This paper introduces the novel idea of bridge Load Rating Mobility and how it can be leveraged by stake holders to assist in making those financial decisions. The paper covers a basic explanation of current bridge load rating methodology and an introduction to the innovative concept of Load Rating Mobility. Added, is a single bridge case study and discussion of a steel multi-girder highway bridge and how it pertains to this new concept, Load Rating Mobility. This research concludes by presenting a collection of all influence variables in a sensitivity analysis of steel multi-girder bridge to those variables and its positive load rating variability. This paper provides a methodology by which Load Rating Mobility may be used to avoid the diminishing returns inherent in what will most certainly become large scale bridge re-evaluations.

Contents

List of Tables

List of Figures

1. Introduction

1.1. Problem Statement

With the consensus being that the majority of United States bridges are reaching the end of their design life, researchers have posited various means and methods of extending the life of these transportation critical structures in order to meet the accumulating demand for products through commerce (Chajes, Mertz, & Commander, 1997, Gheitasi & Harris, 2015). This is occurring in an ever-changing environment where legislatures can increase the legal standards for truck loads, shipping companies can change their transportation habits and routes, and the damaging effects of nature are always present. A bridge in a cold wet climate will be subjected to many freeze-thaw cycles as well as deleterious de-icing salts. When bridge components corrode, their geometric properties change which reduces the structure's capacity to resist load. Shipping companies ship goods as efficiently as possible to reduce any wasted investments, which equates to meticulous planning and coordination to, whenever possible, load trucks to their maximum capacity. Our federal and state legislatures however regulate these weights to ensure that there is a standard by which bridges are designed and therefore a standard probability of failure for these transportation structures. Trucks are growing in size and weight and increasing their capability of carrying larger loads. Bridges and the environments surrounding them must be constantly and consistently managed to ensure the longevity of this system of bridges.

1.2. Purpose of the Study

The Federal Highway Administration (FHWA) and state Department of Transportations (DOTs) have required implementation of Bridge Management Systems which seek to improve decision making about structures through condition data analysis, cost data analysis, and optimization

(AASHTO LRFD, 2011, AASHTO MBE, 2011). Included in these are deterioration models and the National Bridge Inventory database, which includes inspection information as well as bridge load ratings. Regulations state that when the load rating of a bridge decreases to a certain point the bridge must be posted for the allowable loads that it can carry (AASHTO MBE 2011). If not properly maintained, all bridges will reach a point where they must be posted and eventually closed. This research explores the question of what can be done when a bridge load rating passes the point where it is posted for load or in danger of being posted.

The load rating of bridges is an exercise born out of the need for standards and regulations of highway and road traffic. It begins with the idea that vehicle loads, specifically those of heavy transportation trucks, must be regulated to ensure the structural stability and serviceability of bridges for the safety of the populace. Modern and contemporary bridges are designed to withstand substantial loads but technological advancements in truck design have led to the increasing weight of cargo trucks which may one day outpace the capacity of the US bridge inventory. As a result, federal, state, and local governments in 1970 established legal loads, or weight limits on the amount of weight a truck can legally carry on each standard or tandem axel (ASCE Load Rating Seminar 2014). Legal loads not only pertain to the weight of each axle, but also to the Gross Vehicle Weight (GVW) and whether or not an axle is a single or tandem axle (ASCE Load Rating Seminar 2014). With well-established weight limits and coinciding axle configurations for trucks, bridge designers have been able to analyze structures with known load cases. Furthermore, "states can evaluate bridges for their capacity to carry legal vehicles," (Hearn 2014) but these evaluation methods vary.

At the point when a bridge load rating falls past the level where it must then be posted most bridge owners will have the bridge re-evaluated. The objective of re-evaluations is to obtain a better

assessment of individual bridges in the population to increase their load ratings and with it their useful life. This goal is soundly based on the assumption that current methods of design and evaluation are inherently conservative (M. Chajes, Mertz, and Commander 1997) & (Cai and Shahawy 2003). Adding to the conservative nature of bridge design and evaluation is the fact that the composite action of decking and peripheral structures that increase the stiffness and durability of a bridge are often not considered initially. What is not being addressed is the question of whether or not a bridge owner will see any benefit at all should he or she take steps to re-analyze a structure. A sample bridge may have a load rating that no amount of analytical change will increase. Furthermore, a bridge may be sound enough that re-analysis of the load rating via different methods may not be necessary, because the bridge is not in danger of being posted.

This research seeks to distinguish between those two situations by briefly looking into the characteristics that define these populations and then explores how much valuable change exists within a load rating method at each variable within its process. This study then presents a framework to identify the useful difference in load rating methods that bridge owners can safely call up on to increase bridge longevity. That framework is hereafter defined as Load Rating Mobility and is presented within this paper as a useful approach in the decision-making process of bridge owners nation-wide.

1.3. Thesis Organization

Chapter 2 covers how bridge load rating fundamentals, descriptions of how bridge load postings are executed, and includes basics on Bridge Management Systems (BMS). The third chapter introduces the innovative idea of bridge Load Rating Mobility (LRM) and the central thesis of the paper. Chapter 4 presents a case study in load rating mobility, introducing the Schuster Overpass

Bridge and the multiple methods that were used to analyze and determine Rating Factors. Here and in-depth analysis of the case study bridge illustrates how the novel concept of LRM can be implemented. Chapter 5 provides the results of the case study and a discussion of those results. Chapter 6 provides conclusions and several recommendations for future research and development of the LRM concept.

1.4. Notation

A_1 = Dead Load factor

A_2 = Live Load factor

C = Bridge Capacity

DC = Dead Load Effect Structural

DL = Dead Load

DW = Dead Load Effect Wearing Surfaces

IM = Dynamic Load Allowance (Impact Factor)

LL = Live Load Effect

P = Safe Posting Load

R = Resistance

RF = Rating Factor

RT = Bridge member load rating in metric tons

W = GVW of rating vehicle

γ_{DC} = Structural Components Dead Load Factor

γ_{DW} = Wearing Surface & Utilities Dead Load Factor

γ_{LL} = Live Load Factor

φ = LRFD Resistance Factor

φ_s = System Factor

φ_c = Condition Factor

2. Load Rating of Bridges

2.1. Basic Principles

The various methods of bridge evaluation and their differences aside, they all have at their core the same basic equation. That equation states that the capacity of the bridge less the self-weight or dead load should be able to carry the rest of the live load, and that relationship is expressed as a ratio called the Rating Factor (RF) (MBE 6A4.2).

$$RF = \frac{C - DL}{LL}$$

C = Bridge Capacity

DL = Dead Load

LL = Live Load

RF = Rating Factor

RT = Bridge member load rating in metric tons

With this expression one can see that when a bridge has enough capacity to support itself and the live load, it will have a RF of at least 1.0, while a bridge that cannot withstand the weight of the live and dead load will have a RF less than 1.0. This may be misleading once one analyzes the National Bridge Inventory (NBI), because the NBI lists all reported bridge load ratings in terms of metric tons as compared to an HL-93 standard truck load (LRFD Bridge Design Spec Figure 3.6.1.2.2-1) seen below. While bridge owners are directed to report bridge capacity in terms of a

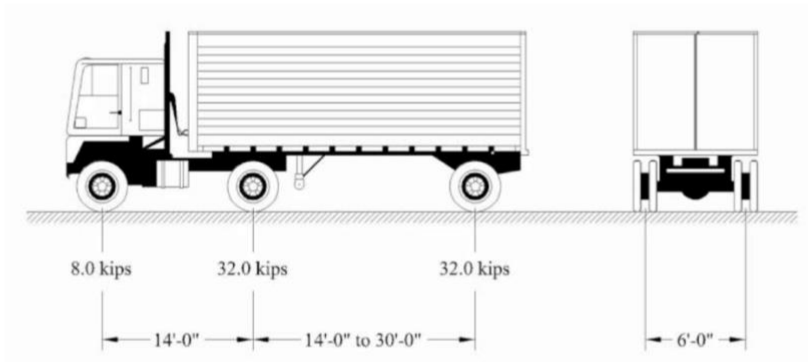

Figure 1: AASHTO LRFD Bridge Design Spec 3-24

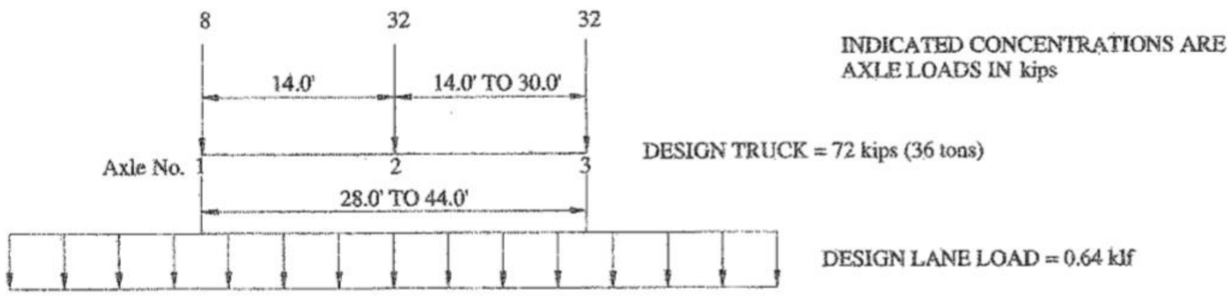

Figure 2: AASHTO MBE LRFD DESIGN LIVE LOAD (HL93)

Rating Factor, the NBI instead catalogs the Load Ratings, which is the maximum load that the bridge can withstand in terms of the design truck configuration. That is to say, the load rating is the capacity of the weakest member of the structure. That conversion depicted below is a simple one between Rating Factor (RF) and the bridge load rating (RT in metric tons) (MBE 6A4.4.4):

$$RT = (RF) * W$$

Where:

RT = Bridge member load rating in metric tons

W = GVW of the rating vehicle load in metric tons

Once any analysis of a critical structural member of a bridge is determined to be incapable of carrying the live and dead loads for any legal vehicle and any load case the bridge owner should

place a posting sign for the load that the bridge will support (MBE Ch 6, 6A.4.4.1). The bridge is then considered to be "posted."

2.2. Allowable Stress Rating

There are analytical approximate methods as well as refined load rating methods that are used to calculate bridge load ratings. The analytical approximate methods can be broken down into three separate approaches, and they derive from the methods of design used historically for structure design. Those three methods are the Allowable Stress Rating (ASR), Load Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR) methods, with the first of these being the oldest technique developed and the latter being the most recent. The ASR method is the oldest and also the most simplistic in that it analyzes the chosen stress type and determines whether or not the member has the capacity to withstand the desired load and the resultant stresses given a factor of safety. While reliable, the engineering community went away from this method because the factor of safety was arbitrary and subjective albeit based on the historical experience of that same community. This methodology, while steady and fairly reliable, was changed to accurately take into consideration the likelihood of differing load combinations acting simultaneously on a structure. That said, there are still thousands of bridges that were designed using ASD that are still functioning in the national bridge population today.

Knowing this, bridge owners can re-analyze them using ASR, or another more advanced method. Regardless of the method chosen there are two rating types that must be calculated for each member under inspection, and they are the inventory and operating rating. The general equation for a member load rating using ASR and LFR are as follows:

$$RF = \frac{C - A_1 D}{A_2 L (1 + I)}$$

Where:

C = Member Capacity

D = Dead Load effect for axial, bending, or shear forces

L = Live Load effect

I = Impact factor

A_1 = Dead Load factor

A_2 = Live Load factor

The two methods of ASR and LFR use the same equation but use different factors for the dead and live load factors differ. Allowable Stress Rating methods use 1.0 as the factor for both the dead and live load factors for both the inventory and operating level of analysis, while LFR method uses different ordinates for those factors covered below. For ASR the bridge designer must use empirical data specific to the given member geometry and material in order to determine the factor of safety (FS) that will be applied to the yield strength of the design member. If that data is not available, then one should reference the MBE for the material information.

2.3. Inventory Rating

Inventory rating is a factor of the bridge capacity that the bridge should be able to withstand indefinitely or for the entire lifespan of the bridge. In ASR the FS for an inventory rating is 2.12. A bridge inventory rating should be calculated but a bridge posting is generally based off of the bridge rating that has been calculated for the operating rating. The bridge load ratings that are calculated using any of the approximate methods may be conservative, and the Allowable Stress Method is no different given the empirical nature of its factors of safety. Hodson writes that “code-based procedures are typically based on conservative assumptions and frequently result in lower

inventory and operating load ratings in comparison with the actual in-service values" (Hodson et al. 2013).

2.4. Operating Rating

The operating rating is a level of the bridge capacity which the bridge should be able to withstand, however only for controlled situations. It is more closely comparable to the maximum capacity of the bridge where repeated loading of the bridge at this magnitude would be detrimental to the structure. One must keep in mind that this analysis is conducted on all applicable members resulting in a controlling member that has the lowest rating. In Allowable Stress Ratings the factor of safety that yields the operating rating is 1.7.

Table 1: Minimum Mechanical Properties of Structural Steel by Year of Construction

Year of Construction	Minimum Yield Point of Minimum Yield Strength, F_y, ksi	Minimum Tensile Strength, F_u, ksi
Prior to 1905	26	52
1905 to 1936	30	60
1936 to 1963	33	66
After 1963	36	66

2.5. As-Built Plans

Much of the information regarding a structure is difficult to determine without material testing and field measurements. As-built plans, or plans provided by the contractor who constructed a bridge can provide valuable measurements and information on materials used during construction. The bridge in the following case study has as-built plans that were provided by TxDOT, however it must be noted that without plans or material sampling one must consult the MBE for standard material properties that coincide with the materials that were most commonly used at the time of the bridges' construction as seen in Table 1.

2.6. Load Factor Rating

The basic ASR/LFR load rating equation does not change when switching from ASR to LFR. Nor does the fact that a thorough bridge inspection should be conducted to determine if any deterioration has occurred that would have an effect on the geometry and capacity of the member under inspection. What does change between ASR and LFR are the load factors for the corresponding rating method. Additionally, ASR uses a safety factor while the Load Factor Rating method uses different factors that are based on the uncertainty inherent in the calculation of loads. When material properties are unknown a material test or the tabulated values in the MBE must be referenced to best estimate the material properties.

2.6.1. Inventory Rating

The definition of an inventory rating as it relates to a bridge's lifespan remains the same. The dead and live load factors, A_1 and A_2, for an inventory rating of an LFR bridge rating are 1.3 and 2.17 respectively.

2.6.2. Operating Rating

Likewise, the definition of the operating rating for a steel girder bridge does not change. The dead and live load factors for an operating rating of an LFR bridge rating are both 1.3. In all cases the effect of vehicles moving at high velocities across the bridge must be accounted for in the terms of a dynamic load allowance or impact factor, IM.

2.7. Load and Resistance Factor Rating

The LRFR or Load Resistance Factor Rating is the most complex of the three methods covered here. The reason for that complexity is based on the fact that the factors used are applied to both the loads and the structural resistance or capacity of a bridge. The factors that we apply increase

the loads and reduce the amount of capacity of our structure, in essence adding a factor of safety similar to the ASR method. That is where the similarity ends, because while the factor of safety in the allowable stress method is empirical in nature, the factors for LRFR method are more scientific. They were created based on reliability and the probabilistic likelihood of specific load combinations as applicable to specific limit states. These factors are calibrated across an entire database of structures using an overlapping distribution of load and resistance cases. Given that in any situation where the live load is larger than the remaining resistance of a member results in failure, these distributions, with their corresponding standard deviations, were overlaid to settle on a target safety index, β, of 2.5 for rating and 3.5 for design. According to (Ghosn et al. 2013) what resulted were new live load models, load distribution, impact, and load factors, as well as different multi-presence factors. This, the newest method for structural design, yields a new equation for the rating factor which follows:

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

Where:

φ_s = System Factor

φ_c = Condition Factor

φ = LRFD Resistance Factor

R = Resistance

γ_{DC} = Structural Components Dead Load Factor

γ_{DW} = Wearing Surface & Utilities Dead Load Factor

γ_{LL} = Live Load Factor

DC = Dead Load Effect Structural

DW = Dead Load Effect Wearing Surfaces

LL = Live Load Effect

IM = Dynamic Load Allowance (Impact Factor)

Ultimately this, the newest of the methods, promotes an increased level of confidence as a result of the uniform probability of failure. Designers and evaluators for bridge rating factors have accepted this method and it, as a result, has been the standard for bridge design and evaluation since October 2010.

2.7.1. Inventory Rating, LRFR

The inventory and operating rating definitions for LRFR are no different than for ASR and LFR, and the procedure is the same for both inventory and operating evaluations with the only change being that the live load factor γ_{LL} is 1.75, and 1.35, respectively.

2.7.2. Operating Rating, LRFR

The procedure need not be recalculated for operating rating either, as specified in the AASHTO MBE commentary, C6A.1.1. One need only multiply the calculated inventory rating by the ratio of the inventory and operating ratings, to produce the operating rating.

2.8. Single Line-Girder Analysis

Which approach a state DOT chooses to use is, to one extent, controlled by USDOT regulations and to another controlled by the prerogative of the state. As previously mentioned, USDOT has dictated through regulation that after October 1st, 2010 all new bridges or replacement bridges shall be designed, and load rated using the LRFR method. Bridges built prior may be analyzed with

LRFR or LFR and bridges that were designed with the ASR method may be rated using ASR, LFR, or LRFR methods. An extension of these methods are computer programs that utilize the two-dimensional single line girder analysis approach to achieve the same load ratings. Examples of these programs are VIRTIS, AASHTOWare Bridge, and BRASS (Hearn 2014). The Bridge Rating and Analysis Structural System, or BRASS, which was developed by Wyoming Highway Department uses the single line girder method of analysis. AASHTOWare Bridge uses the same method, which is a simplified approach that analyses each girder as a member separate from the rest of the bridge structure. Using empirical methods of calculating the load distribution, the VIRTIS and BRASS software produce a rating for each individual member with the lowest load rated member acting as the control for the bridge. The single line girder method of analysis does not always accurately account for the in-situ three-dimensional (3D) system behavior of a multi-girder bridge where loads may be transferred more transversely than the empirical calculations allow. In fact, for many different lengths and shapes of multi-girder bridges, the empirical formulas within the single line method may distribute loads orders of magnitude above what a refined approach might determine.

2.9. Refined Rating Approaches

More refined approaches use computing software to analyze an entire bridge structure as one interconnected system, instead of as individual members. While in some states like Louisiana bridge owners are prevented from using refined methods as a rating approach without state permission, other states like Massachusetts use both girder line software as well as refined techniques and modeling software, like STAAD for bridge load ratings (Hearn 2014).

2.9.1. Load testing

A separate method of attaining load ratings can be found in load testing, of which there are two types. Load testing consists of proof load testing and diagnostic load testing. The MBE specifically states that there is no preferred method of load rating, however every method has benefits and drawbacks, and load testing is no different. Load testing can overcome the conservative results from analytical load ratings but costs a significant more in terms of equipment and man hours as well as productivity lost due to the fact that the majority of load testing methods require the temporary closing of a bridge. Bridge owners must conduct a cost benefit analysis to determine if a load test would yield a better load rating. By uncovering any additional capacity of a bridge, owners can avoid costly rehabilitation that would be used on a structure previously thought to have had a lower rating.

2.9.2. Bridge Load Rating through Proof Load Testing

In proof loading a bridge owner gradually loads a bridge until it reaches a target load, thus proving the capacity of the structure. There is a risk of causing permanent damage to the structure, so this method should include proper safety checks and observations for any non-linear behavior (Casas and Gómez 2013). When complete a lower load level is chosen as the safe loading level based a safety margin that should have be incorporated initially into the target load.

2.9.3. Bridge Load Rating through Diagnostic Load Testing

Diagnostic load ratings are another method of measuring and calculating load ratings. There are many different methods that are in use and are still being experimented with. The commonality of them all is that they use sensors to measure the response behavior of the bridge when loaded which allows for an extrapolation of the bridge capacity. It is a safer method than the proof loading, as the total load need not come close to reaching the maximum bridge capacity. However,

the diagnostic load test is mainly used to verify the results of an analytical load rating, which based on the bridge behavior will allow the engineer to adjust some of his/her prior assumptions. According to the MBE 2014, a diagnostic test can be used as long as the bridge does not display plastic behavior in any of its critical members. Even within this path, methods vary as some engineers experiment with using dynamic bridge responses as opposed to static bridge response as seen in (Islam, Jaroo, and Li 2015), where bridge vibration responses were measured as it correlated to bridge stiffness.

2.9.4. Refined Distribution Factors from Computer Modeling

Using the power of computers for modeling in three dimensions is the last method that will be discussed for measuring and calculating load ratings. The benefits of using Computer Aided Design are ubiquitous, however one stands out. When determining the load rating for a structure, a 3D model can reveal system behavior in a way that the single line girder analysis can never truly replicate. While the line girder analysis takes into account this behavior by using distribution factors to replicate the path of the vehicle load onto the various girders, the 3D model analyzes the entire bridge as a unified structure with all of the members, deck, and bracing working together to distribute load. The 3D model simulates the system of the superstructure while the SLG analysis uses empirical calculations to estimate what the highest distribution factor will be. Gheitasi & Harris, (2015) state that the distribution factor can be calculated by taking the proportion of load effect in one girder to the summation of "all primary load-carrying member" effects. Once a model has been created along with the load cases that are chosen for generation, one takes the results of all the load effects along the bridge cross section at the point of maximum effect. With these values, following this method one can extract more accurate distribution factors that can be

substituted into the classic calculations to derive a more accurate rating factor (Gheitasi and Harris 2015; Provines, Connor, and Sherman 2014).

2.10. Load Posting

According to the AASHTO Manual for Bridge Evaluation (MBE) the standard formula for determining the appropriate posted load for a bridge is as follows.

$$P = \frac{W}{0.7} * [(RF) * 0.3]$$

Where:

P = Safe Posting Load

W = GVW of rating vehicle

Given this equation two question arises. Which RF do you input into the formula? Do states actually use this formula at all? The simple answer is that it depends on the state, because twenty-two states use the operating rating, five states use the inventory rating, while four use the LRFR posting equation above. Twelve states use some other combination of intermediate level to load post bridges (Hearn 2014). In most cases the method chosen depends on the condition rating of the bridge and the level of redundancy in the structure.

2.11. Bridge Management System (BMS)

As previously mentioned, state DOTs and local governments must submit bridge ratings along with all NBI data points to the FHWA, which then publishes that data in the NBI. Doing this annually ensures that on a national level there is consistency and familiarity around the methods of Bridge Management (Markow and Hyman 2016). In Bridge Management Systems for Transportation Agency Decision Making we learn that the FHWA may adjust the NBIS items

periodically to ensure that the data points that are collected include rising issues. The FHWA's foundation for identification of needy bridges and funding distribution is the NBI data. That data is also the basis for biennial reporting to Congress. The MBE tells us that assessing future needs based on current condition data is an essential component of BMS data Analysis.

2.12. Economic Factors & Decision Making

Consideration of agency or owner costs along with user costs over the lifetime of a structure is essential in BMS. The goal is to minimize functional deficiencies which consist of the effects to bridge users such as detour times and the associated accident costs. Beyond the costs associated with the actions taken to renovate or rebuild a bridge there exist the costs that come with the decision-making process. To assess a bridge, a bridge owner must hire an engineer to conduct an inspect the bridge, perform analysis, build a computer model, instrument, or to conduct a load test on the structure. All of these actions have different costs and outcomes, but they all in some ways should be considered as another economic factor.

3. Load Rating Mobility

Load rating mobility is a novel concept introduced here as a process that quantifies the potential variation, or analytical flexibility, in approaches to the calculation of bridge load ratings. While doctrine and many scholars express understanding of the problems that Load Rating Mobility solves, none of them provide the systematic approach of LRM and with it the consistent results it should yield for bridge owners. Most acknowledge that the typical path of analysis begins with the simplest methods and moves to the more difficult, which is prescribed in the MBE as seen in the Load and Resistance Factor Rating flowchart (AASHTO MBE 2011) seen in Figure 3. Likewise, there is a consensus that the more difficult the method, the more expensive and time consuming it will be (Bell, Lefebvre, and Sanayei 2013). The community also agrees that the more simpler the method the more conservative it will be (Bell, Lefebvre, and Sanayei 2013; M. J. Chajes and Shenton 2006). Some DOTs currently approve of methods for tailored load factors (Ghosn et al. 2013), however the ability to adjust a load rating through an analysis of latent bridge capacity isn't currently considered in the analytical method, in particular as it pertains to bridges on the verge of becoming load posted.

3.1. Tiered Approach

The fundamental question behind load rating mobility is whether the investment in more refined rating approaches is likely to yield the desired change, or movement, in the load rating value. This is the question that bridge managers grapple with, and load rating mobility will help aid in decision-making. Qualitatively, it makes sense to think of bridges in tiers which describe how their load ratings may change through variation in rating approaches. There are four tiers that

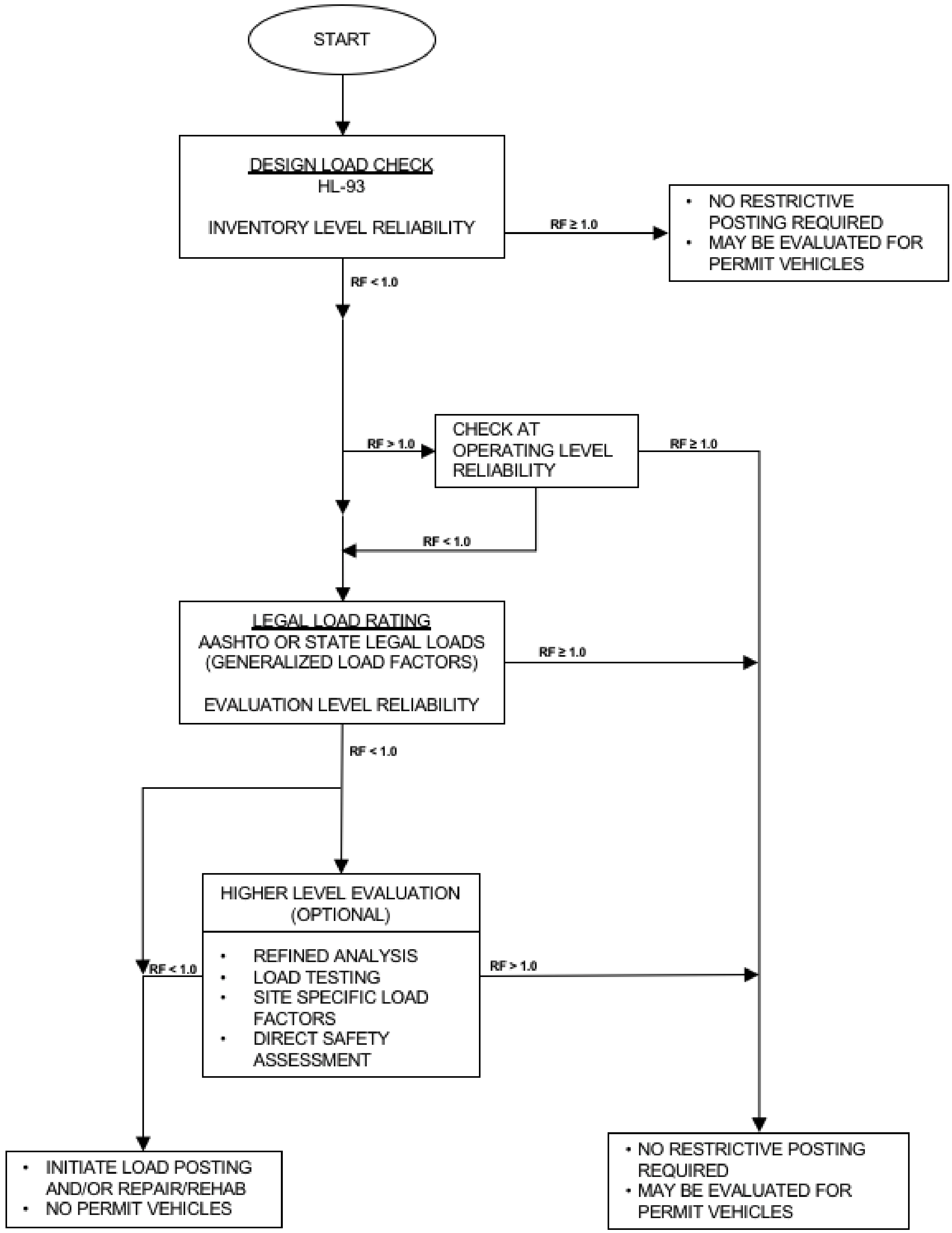

Figure 3:MBE Load and Resistance Factor Rating Flow Chart

logically emerge when considering the question of whether or not a bridge should be re-rated in order to obtain a better load rating.

3.1.1. Tier 1

This tier represents bridges that have been designed to withstand current or larger than legal truck loads and are in no near danger of their operating capacity dropping below the legal limits. Specifically, it is unlikely that increases in truck sizes and weights will drop the load rating factor below one. This subset of the bridge population retains characteristics, such as road width or span length that makes them resistant to the ever-increasing loads from changing legal truck configurations.

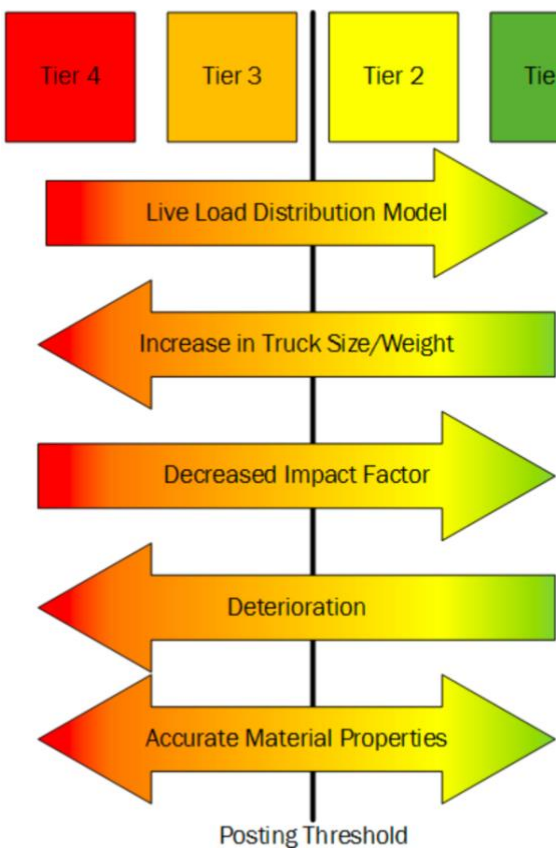

Figure 4: LRM Tiered Approach

3.1.2. Tier 2

The second tier, one of two critical to this research, represents the subset of the bridge population that currently retain enough capacity to withstand legal truck loads, but due to ever-changing legislation are on the cusp of becoming posted. These bridges are not currently posted for load, but a change in bridge condition rating, transportation routing, or an increase in legal truck loads could lead to these bridges being posted for load, resulting in a significant economic cost to bridge users who may need to then alter their routes. This subpopulation may reside in critical transportation corridors, either locally or nationally, which increases the likelihood of them falling into the rating-critical bridge subset.

3.1.3. Tier 3

The third tier of bridges are also rating critical structures but exist as a more well-defined subset of the bridge population. These bridges have been posted for load already but maintain sufficient reserve capacity to remove their posting due to the positive mobility potential of their load rating. That inherent mobility potential is something that load rating engineers can take advantage of through a shift in analysis or operational bridge management to re-open the bridge back to truck traffic carrying legal loads.

3.1.4. Tier 4

The fourth and final bridge tier is bridges that are posted and, due to their initial designs, do not maintain enough positive mobility potential to remove a load posting. Simply put, these bridges are damaged to the point where they cannot support current legal loads or were not designed to ever support current legal loads. This set of structures, while better defined than the second tier of bridges, may initially appear as a subset of Tier 3. It is crucial to distinguish Tier 4 bridges from

Tier 3, so as not to waste valuable resources on structures that will not overcome their deficiencies without exceeding any common-sense limits for diminished returns.

3.2. Variable Sensitivity Analysis

What follows is a discussion of the characteristic variables that factor into a load rating that are believed to be open to further analysis, inspection, or change in a way that can influence the Rating Factor of a bridge. These Influence Variables seen in Table 2 are all aspects of the load rating calculation that can be change due to engineering expertise or known changes to measurements, structural relationships, or force estimations. Some do not require any actions at the bridge location to change the behavior of traffic as it interacts with the bridge, while some involve changes to the physical bridge site. Presented in the table are the levels to which these variables were

Table 2: Influence Variable Variance

Steel Girder	Variation
Rating Approach ASR	—
Rating Approach LRFR	—
Girder Distribution Factors	**- 0.1**
Material Properties	**+ 4 ksi**
Material Properties	**+ 1 ksi**
Material Geometry - Flange Plate Width	**+ 1/16"**
Material Geometry - Flange Plate Depth	**+ 1/16"**
System Factor	**- 0.15**
Slab Depth	**+ 1/2 "**
Road Width Reduction	—
Resurface Approach Deck (IM)	**- 0.23**
Speed Enforcement (IM)	**- 0.13**
Encasement	—

altered in order to calculate the resulting influence each has on the bridge load rating. These variations are used in the following case study, while the methods by which these analytical values might differ from in-situ bridge values are described further in this section.

3.2.1. Dynamic Load Allowance (IM)

The first method to discuss is the impact factor. The impact factor is a means of taking into account the added force effects created by the vibration and bounce of a truck traveling across a bridge at speed. This factor is a function of truck suspension, weight, axle configuration, and speed. That speed is a function of the bridge approach angles, and surface roughness. A highway overpass in a city with traffic lights controlling both lanes, will not experience vehicle speeds of the same magnitude as an overpass servicing a major interstate highway. This factor is multiplied by the live load in the denominator of the load rating equation and can be reduced by various methods. Initially in the LRFR method of load ratings the MBE dictates that the impact factor, or dynamic load allowance, IM, should be 33%; 1.33 as it is applied to the live load, but even the MBE states that this magnitude of an impact factor is conservative, and considers many unknown conditions of the bridge surface and approach conditions. If one makes those conditions known by resurfacing the bridge deck and approach avenues to ensure smooth transitions at joints, it may be reduced to 20% or even 10% in accordance with MBE table C6A.4.4.3-1. Reducing the speed of trucks crossing the bridge in a manner that ensures that the speed can be controlled without regular speed violations will also allow for a reduction in IM. An example in an urban setting would be a stop light or stop sign right before the bridge. Serpentine barriers could also provide a means of reducing speed while ensuring no speed violations. These methods range in influence. For vehicles moving less than 10 mph the dynamic load allowance may be removed entirely.

3.2.2. Material Properties

Material properties directly affect stiffness and capacity, which are critical drivers of load ratings, however due to the age of a bridge and limited records this effect may fluctuate. For concrete structures the curing process begins immediately after it is poured, and the mixture begins to harden. After 28 days of curing and hardening a concrete member is generally considered to have reached its full strength. However, the reality is that the concrete continues to harden and gain strength years after the construction of a bridge.

Steel manufacturing techniques and procedures have changed in the century plus that engineers have used this material for construction. That said, without accurate records of a bridge, or as-built plans, the exact properties of a steel member may not be known. Going back far enough in history one will find bridges constructed of cast iron. Without plans one must make assumptions of the material density, tensile, and compressive strengths of the metal used.

In both cases, concrete or steel, we can test the material. This means cutting out a sample of the material which can then be tested. For concrete, a core sample can be cut out with a coring drill bit. On a steel structure a small sliver of steel can be cut away from a section of a steel member that, through structural analysis, is designated to be under low stress. This sample taken from a steel member is called a coupon. At that point, after measuring the density, and geometry of the sample, it can be subjected to a uniaxial compression test or a tensile strength test to determine the materials strength. For example, using table 6A.6.2.1-1 from the MBE and the assumption that a structure from 1925 has a yield strength of 30 KSI versus a coupon test of 36 KSI strength could yield almost 74% increase in flexural capacity and 28% increase in shear capacity for a steel girder bridge interior stringer.

3.2.3. Condition

The condition of a bridge as it pertains to the condition rating determined by a licensed bridge inspector may have several impacts on the load rating. Bridges are inspected bi-annually and in turn are given a bridge rating between zero and nine. Bridges are grouped in accordance with the Fixing America's Transportation Act and the performance-based guidelines set forth in the Moving Ahead for Progress in the 21st Century Act. Those groups classify bridges at or above a seven as "Good," and bridges that are at or below a four as "Poor." Bridges with a condition rating of five or six are classified as "Fair." These bridge condition ratings are documented along with the load ratings for every bridge. According to Hearn, (2014) different states allow the bridge condition ratings to control the frequency for when load ratings must be calculated for a structure. The bridge condition ratings indicate the level to which deterioration has affected any structural components of a bridge.

A distinction must be made at this point between how condition ratings are classified and used at the engineering and design versus policy making echelons. The NBI coding guide as well as the MBE categorize and group condition ratings differently and with more specificity. The NBI and MBE state that a level of six is deemed "Satisfactory Condition," however at this point slight corrosion has begun to present on structural members. Condition rating five is considered "Fair Condition" and a rating of four is considered "Poor" and at this point "advanced section loss, deterioration, spalling or scour" has affected a structural member according to the NBI (1995). These three condition factors are significant because of the preceding changes in the bridge members affected, and this is confirmed in the MBE, where Table 3 provides an approximate conversion table for the selection of the condition factor, φ_c, which is applied directly to the

resistance of a member in the LRFR RF formula. Table 3, seen below, makes the same previous statement, that ratings of 4, 5, and 6 are poor, fair and satisfactory, respectively.

Table 3: MBE Condition Factor and Selection

Condition Factor and Selection, φc		
Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition	φ_c
6 or Higher	Good or Satisfactory	1.00
5	Fair	0.95
4 or lower	Poor	0.85

With that information one enters Table 2 in order to factor the resistance down. The MBE, in section 6A.4.2.3, states however that use of the condition factors is not mandatory and wholly dependent on the organization conducting the load rating, and this makes sense. If one obtains accurate measurements of the member in question at the point of deterioration, then these measurements can be applied directly into the analysis to calculate an accurate member capacity. The condition factor, φ_c, reduces the member capacity due to the uncertainty of an affected member's resistance due to deterioration. Utilizing nondestructive testing to take an accurate cross-sectional measurement in the area affected by material loss may produce a better load rating than the conservative results that may come from using the condition factor. Determining whether or not the previous load rating was calculated using condition factors would be crucial in producing an increase to the load rating through analytical means.

3.2.4. System Behavior

The behavior of a bridge as a system can be analyzed and interpreted in different ways in order to see an increase in a bridge load rating. This characteristic begins the branch between basic analytical load rating calculations and the more advanced computer modeling of the entire bridge

as an interconnected system. Within the analytical method there are system factors applied to the capacity in the load rating calculations that take into account the redundancy of the main force

Table 4: MBE System Factor for Flexural and Axial Effects

Superstructure Type	φ_s
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eye-bar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floor beams with Spacing > 12 ft and Non-continuous Stringers	0.85
Redundant Stringer Subsystems between Floor beams	1.00

carrying members of a bridge superstructure. These system factors, according to the MBE, look at "internal redundancy and structural redundancy" and penalize bridges by reducing their factored member capacities in order to maintain a sufficient level of bridge safety. These system factors will differ depending on the type of superstructure system.

In Table 3 one sees that a welded member in a two-girder bridge will have its capacity reduced by a factor of 0.85, while a four-girder bridge with girder spacing less than four feet will only have its capacity multiplied by a factor of 0.95. It is important for an engineer completing a bridge load rating to consider the previous rating and whether or not the system behavior was considered as a drawback. As one example, "girder-type bridge superstructures tend to have a great amount of additional reserve capacity because of the inherent redundancy and system-level interaction" (Gheitasi and Harris 2015). There may be redundancies present that make a sudden bridge failure far less likely and may not have been taken into account. If this is the case, there is an argument to be made for increasing the system factor. That said, care must be taken to ensure that bridges

with few members that are equally loaded, and lack actual reserve capacity do not have their system factors elevated.

System behavior can more accurately be analyzed using computer modeling software to model the bridge superstructure as a whole, unlike the single-line girder analysis. Finite-Element Modeling (FEM) approaches examine how the load is distributed across the structure through diaphragms and girders in a way that cannot be seen in line girder analysis. Portions of the bridge structure that are not directly in any apparent load path can absorb load, adding capacity and effecting the load distribution. System behavior such as this is revealed through refined methods of 3D modeling, and with load distribution as the primary target for refined rating approaches, refined model utilization often bares an increased live load capacity and bridge load rating. What bridge owners encounter now are situations where a bridge can no longer carry legal loads based on results of an approximate analysis method, which as discussed earlier is viewed as inherently conservative by the bridge engineering community. In one case researchers concluded from analysis of several bridges in the state of Maine that a refined rating method centered on Finite Element methodology produced an average increase in operational load rating of 26% for short span flat slab bridges (Davids, Poulin, and Goslin 2013).

3.2.5. Live Load Analysis

There are multiple means of increasing the Rating Factor through manipulation of the live load computational analysis. Reverifying that that the correct girder distribution factors for a bridge were used in the calculations is crucial, as there are many different formulas that are based on various types of superstructure construction. Not realizing that corrugated steel deck was used in construction of the deck could have significant effects on the load rating of a structure because of the distribution factor that results. For this reason, it is most important to ensure that you have

accurate as-built plans, or some form of detailed inspection combined with non-destructive testing that can ensure proper load distribution factors are used. With that, every distribution factor listed in the Design Specification comes with two options. One distribution factor formula will be used in situation where there is a single lane loaded condition, and another distribution factor calculation must be used in situation where there are two or more lanes loaded. One option that is explored here, is the use of operational restrictions to a two-lane bridge that would restrict it to a single lane. This may have second and third order effects if this method causes the rerouting of traffic and customers who choose not to deal with this restriction. It could, however, keep the bridge open to legal truck traffic by allowing a different distribution factor to be used for the load rating analysis. A different distribution factor would be able to significantly increase the rating factor for the bridge.

Another option, albeit more time consuming and expensive, is to develop state specific live load factors. This method would not apply to states that suspected themselves to have heavier trucks utilizing their roadways, but for states that suspect the opposite, Live Load factors could theoretically be reduced. Researchers detail a method used to accomplish this task in New York State using Weigh-in-Motion (WIM) sensors and separate calibration of the live load factors aimed at a specific reliability index in order to account for site or state specific truck loads (Ghosn et al. 2013). This process was done for New York State where the live loads were suspected to be higher than those used to develop the live load factors in the specification in order to increase safety.

4. Bridge Case Study

For this research a local bridge was selected for a case study on Load Rating Mobility. A sample bridge from the population of bridges recorded on the National Bridge Inventory was selected using a specific selection process. As-built plans were obtained from TXDOT and a manual theoretical analysis of the LRFR rating factor was calculated for the inventory and operating rating. At that point a FE model was built in order to gain a better understanding of the system behavior. The next step was the composition of a program in MATLAB that allowed for rapid numerical single line girder analyses of the bridge in the format of the AAHSTO MBE rating factor calculations. Finally, the results from the analysis were compared to determine which variables have the most influence on the load rating with the entire process seen in Figure 5.

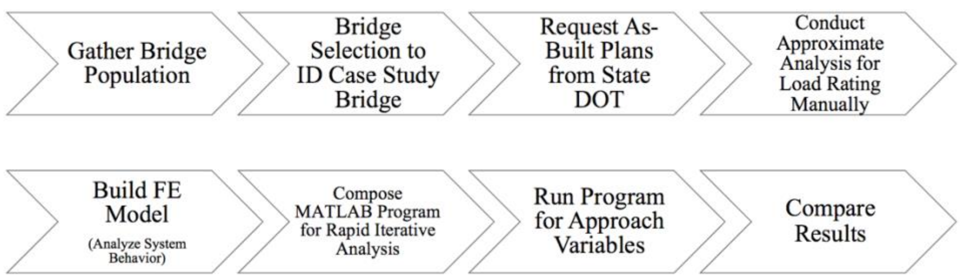

Figure 5: LRM Case Study Procedure

4.1. The Selection Process

Many factors went into the selection of the bridge that would be used for the study. With the desire that this process could be applied to a wide breath of bridge types, several bridges were initially selected. In order to establish a baseline with this case study, the skew on the bridge selection criteria was kept to a minimum. Bridge owner was an important criterion to consider, in so far as

Table 5: Initial Case Study Population

Bridge No	Priority	Owner	Open / Closed	Posted	OR (Metric Tons)	OR Method	IR (Metric Tons)	IR Method	ADT	Design Load	Type (Material)	Type (Method)	Year Built
240720212102279	1	TXDOT	Open	No	44.1	LF	32.4	LF	69550	MS18/HS20	Steel Continuous	Multi-Beam	1968
240720212102278	2	TXDOT	Open	No	44.1	NA	32.4	NA	77705	MS18/HS20	Steel Continuous	Multi-Beam	1968
240720000212107	3	TXDOT	Open	No	44.1	LF	30.6	LF	18400	M18/H20	Steel	Multi-Beam	1963
240720B35760002	4	City	Open	Yes	44.1	ASR	26.1	ASR	4470	M18/H20	Steel	Multi-Beam	1952
240720B60640003	5	City	Open	Yes	44.1	ASR	26.1	ASR	12710	M18/H20	Steel	Multi-Beam	1952
240720212107244	6	TXDOT	Open	No	44.1	LF	32.4	LF	123456	MS18/HS20	Prestressed Concrete	Box Beam Girder - Multi	1990
240720016701249	7	TXDOT	Open	No	44.1	NA	32.4	NA	93370	MS18/HS20	Prestressed Concrete	Multi-Beam	1978
240720067402005	8	TXDOT	Open	Yes	39.6	LF	19.8	LF	1990	M13.5	Concrete	Slab	1916

to ensure that as-built plans were available for any bridges chosen. Having a mix between completely open and posted bridges will be crucial in order to assist in homing in on the second and third tier bridges where this research will be most helpful. The last two criteria for consideration were Average Daily Truck Traffic (ADTT) and the year of construction. Bridges that were nearing or past the 50-year mark for their date of construction were more desirable for identifying Tier 2 & 3 bridges. Newer bridges were assumed to be more likely to be in good condition and closer to Tier 1. Finally, the location of the bridge was taken into account for ease of access and distance. A bridge that was accessible to the campus and nearby El Paso was preferable for the ease of travel to and from the bridge location. The population was narrowed down to the list of structures seen in Table 5 and of those a single bridge was selected.

4.2. Schuster Street Overpass

As a demonstration case study, TXDOT bridge number 212102279, which was built in 1968 on

interstate highway I10 in EL Paso, Texas, was chosen. The Schuster St. overpass, as shown in Figure 1: AASHTO LRFD Bridge Design Spec 3-24 is comprised of two near-identical bridges laid side-by-side measuring 46.34 m (152 ft) long and 42.68 m (140 ft) wide total. Each individual bridge is 21.54 m (70.7 ft) and 21.11 m (60.25 ft) wide respectively with a one-inch open joint separating the two. For the continued purpose of this research one 152 x 70 ft span will be referenced henceforth as the bridge under analysis. The Schuster St Bridge has ten identical steel girders with the measurements seen in Figure 6 and Figure 7 representing a twelve-foot span at the center of the girder where it rests on the support pier. These measurements follow, as the maximum moments calculated for this structure all occur in the negative moment region, where the bridge rests on the pier at center span.

As seen in Figure 8, girders are evenly spaced, with nine spaces at 7'-2" for a total bridge width of 64'-6", and an overhang of 3'-7" at the edges. The deck is a seven-inch-thick reinforced concrete slab, with 10" x 1' high parapets incorporated only on the outer portion of the bridge, away from the one inch open joint. There are three types of steel diaphragms that are laid out in-line transversely and spaced 18'-10' 3/16" longitudinally, with the Type A diaphragms at the

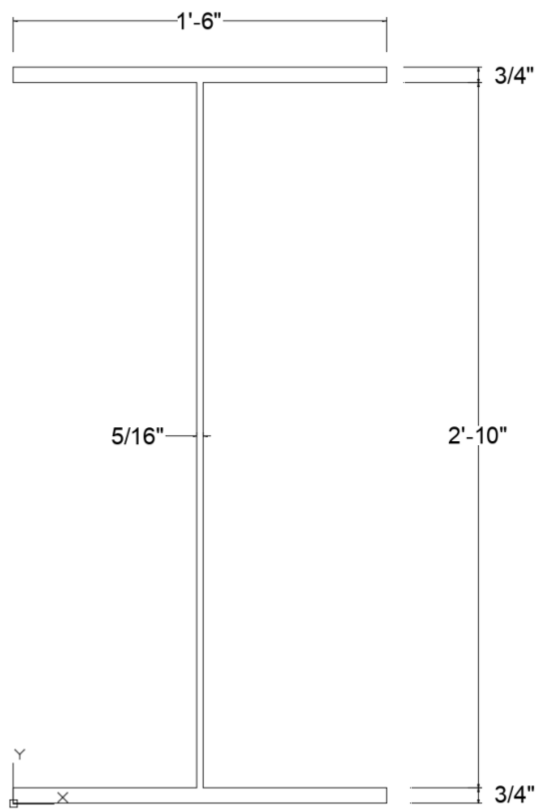

Figure 6: Typical Girder Cross Section_End

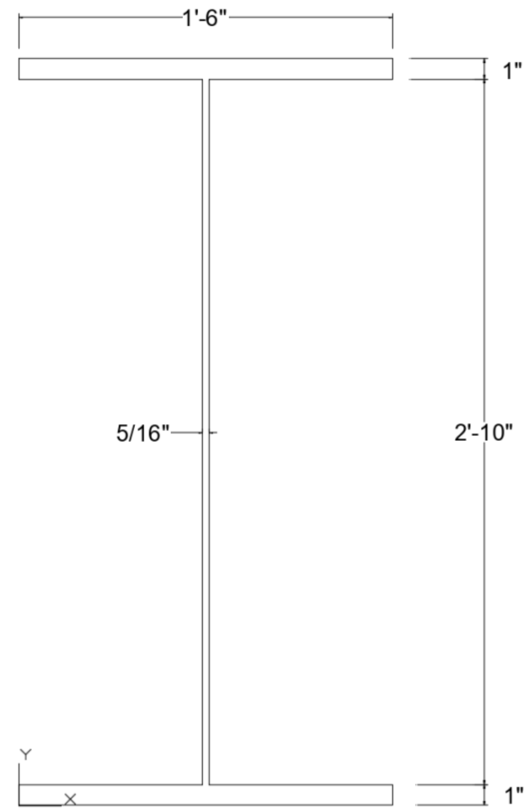

Figure 7: Typical Girder Cross Section_Mid

bridges ends (Figure 9) before the armored joint. Next are the Type B and C diaphragms, seen in Figure 10 and Figure 11, which alternate by type with each transverse line, starting with the Type B diaphragm.

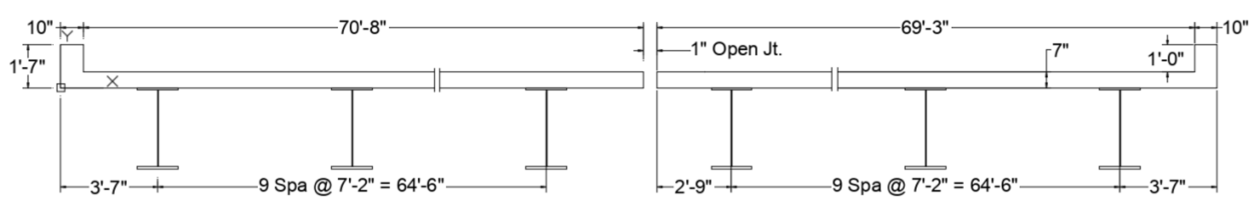

Figure 8: Schuster Street Overpass Typical Section

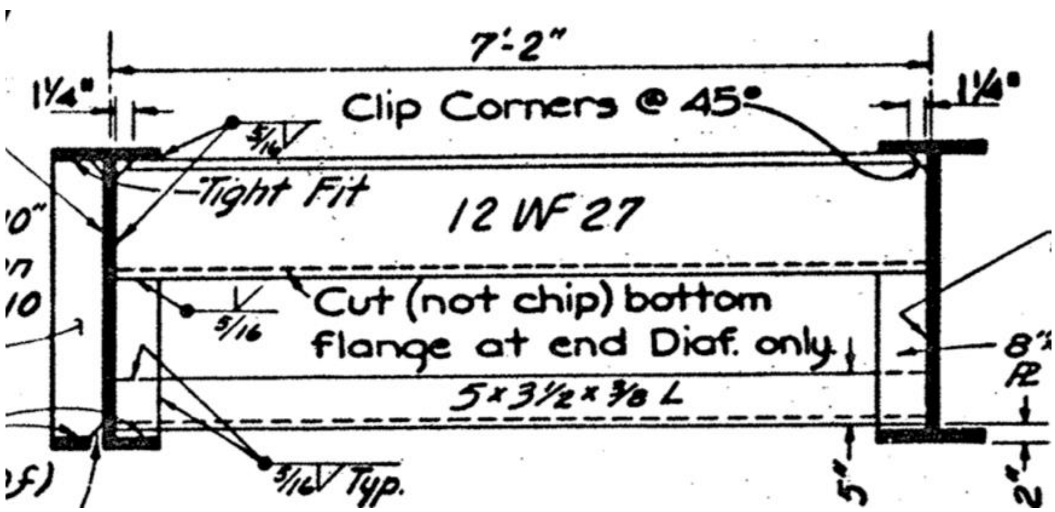

Figure 9: Type A Diaphragm

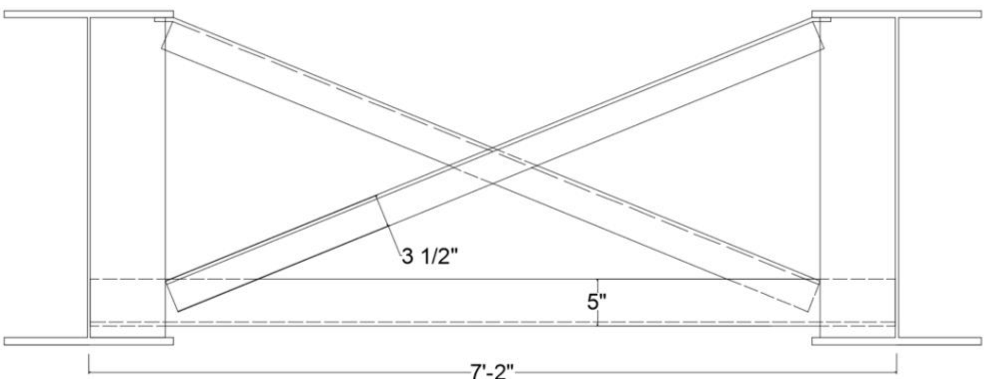

Figure 10: Type B Diaphragm

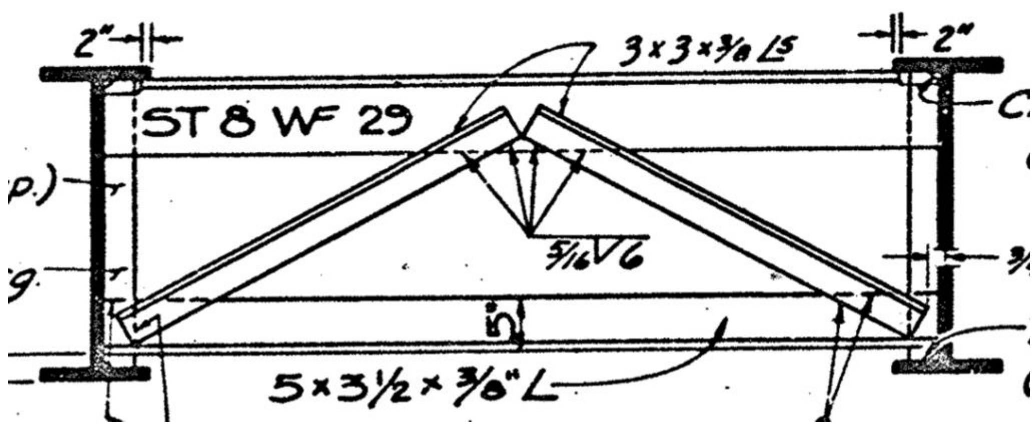

Figure 11: Type C Diaphragm

The bridge has zero skew, an Average Daily Traffic (ADT) of 69550, a condition rating of the superstructure of 7, and a deck condition rating of 7 reported in the 2016 National Bridge Inspection (NBI) standard. The NBI also has this bridge reported within an inventory Load Rating of 32.4 metric tons (71.43 kips) and an operating load rating of 44.1 metric tons (97.22 kips) with both having been calculated using the Load Factor method for an HS-20 design truck load.

4.3. National Bridge Inspection Standards

As referenced above, one of the major references for all bridge rating discussion is the National Bridge Inventory Database, or NBI, and the accompanying Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation' s Bridges, hereby referred to as the NBI Guide. The NBI is a massive database, promulgated by the US Department of Transportation and the Federal Highway Administration, and comprised of 135 different categories of information on all the nations bridges. The information is collected by the FHWA and State DOTs, and built for use by the states, relevant agencies, and researchers, as well by Congress (AASHTO NBI, 1995). More than anything it specifies the criteria by which states across the country maintain "consistent bridge condition evaluation practices" (Bell, Lefebvre, and Sanayei 2013).

States are required by the federal government to inspect, rate, and post state-owned structures while also reporting the status of all bridges in regards to posting (Hearn, 2014). The NBI along with the coding guide, were the starting point for selection of the bridges that have been and will be analyzed in this continuing process. And while the information contained within is not enough to determine the size of the tier 2 and 3 population, that data combined with Graphical Information Systems (GIS) can provide a valuable assessment that will allow bridge owners to rule out many tier 1 and 2 bridges.

4.4. Theoretical Approximate Analysis Load Rating

Once the bridge selection was complete an AASHTO theoretical approximate analysis was performed to determine the load rating. TxDOT provided as built plans for the bridge which were consulted to create a MathCAD worksheet that performed all of the calculations in accordance with Appendix A of the AASHTO MBE. These calculation cover those of a simply supported steel multi-girder bridge using LRFR. Once the basic worksheet was created and checked against the illustrative examples, it was modified in order to account for the specifics of the Schuster overpass bridge, specifically that the example bridge is a two-span continuous bridge. The results showed that the calculations could be performed fairly simply using a math program, but this program did not provide the flexibility to analyze the influence of each variable that would be analyzed during this study. Another method was required.

4.5. Load Rating Program

Multiple programs were then written in MATLAB in accordance with the MBE calculations. The first program was simple and similar to the manual calculations performed earlier where a single load rating was produced for a single given bridge and load case. As with the manual load rating, the as-built plans were meticulously reviewed to create a spreadsheet covering all characteristics of the bridge's construction. This spreadsheet was created in the ubiquitous Comma Separated Variable (CSV) format. The code begins by reading that CSV file and converting quantitative and qualitative values into computing variables. This initial program required some interaction with the user who would interface in order to provide some specific information. Some examples were prompts that required the user to answer whether or not the analysis was to be performed for an interior or exterior girder, or whether or not the girders in question were standard shapes found in the Manual of Steel Construction or if they were unique built up shapes. The program then

produced a simple output, within MATLAB, with the operating and inventory rating factors for shear and moment. The program also responded with the location of the plastic neutral axis within the composite profile. The program flow can be seen here in Figure 12.

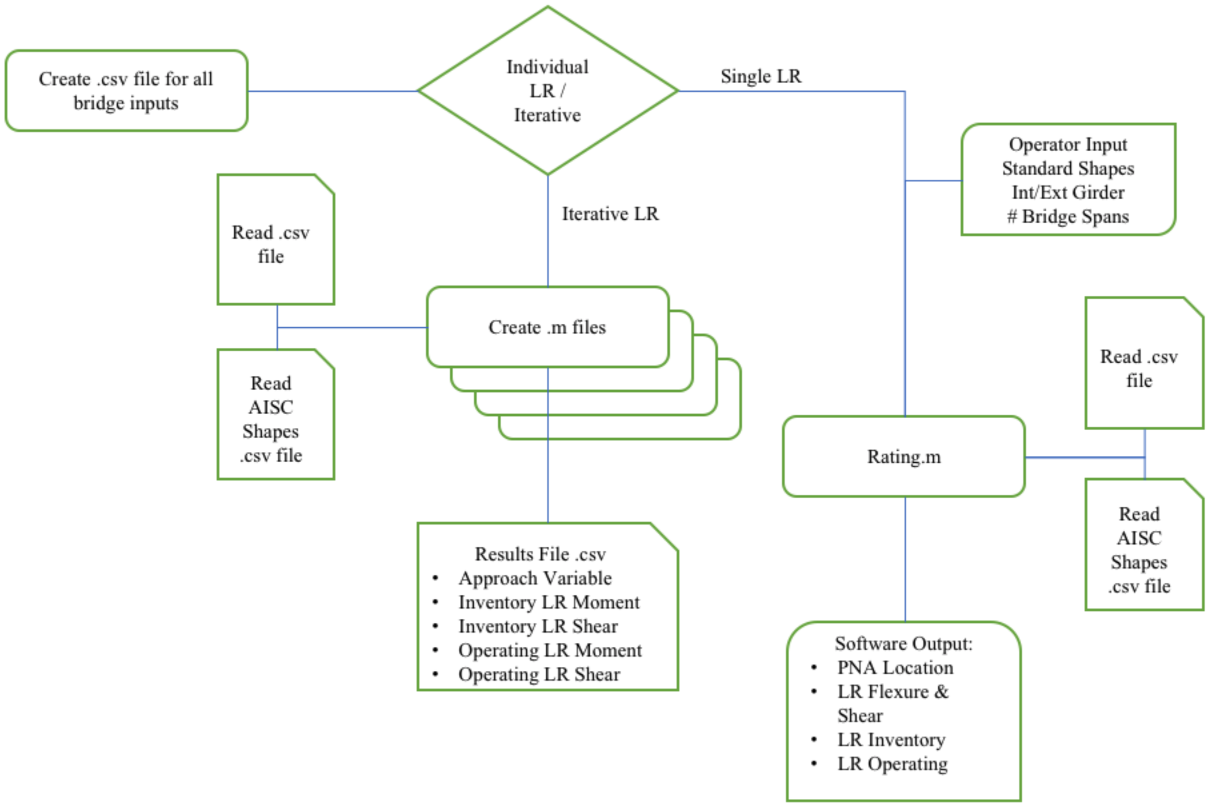

Figure 12: MATLAB Program Flow

The next step spawned a group of programs, each one focused on varying a specific influence variable of the variable sensitivity analysis. Using a looping structure, the programs iterate a chosen number of times while randomly varying the specified variable between a chosen range, each time calculating the rating factors for inventory and operating loads. Each program then tabulates the results and exports them to another csv file along with the chosen independent variable. These files can later be opened in any spreadsheet program for further manipulation and

graphing. Through this process results were produced for deltas in values for steel yield strength, concrete compressive strength, minute changes to all aspects of the girder and deck thickness as well as changes to distribution factors for moment and shear.

4.6. Refined Rating: Finite Element (FE) Model

At this point a model was created to gain a better understanding of the system behavior of the Schuster overpass bridge. A complete model of the superstructure was built and then evaluated in comparison to the single line girder theoretical calculations for rating factor.

4.6.1. Model Creation

After thoroughly studying the as-built plans for the Schuster St overpass structural details a model was created, with particular attention paid to all aspects of the member and slab geometries. Of note was the flange thickness of the stringers in the negative moment region above the pier at midspan. This is where the moment was largest. At this location the flange thickness increased from .75 to one inch for six feet in either longitudinal direction from the pier. A basic wire frame for the structure was built first using AutoCAD. There the dimensions and spacing could easily be generated using the programs graphical tools while ensuring that the coordinate axis was oriented according to common bridge design practice. Not orienting the bridge so that direction of travel is along the ‘z’ axis would leave the model mis-oriented in the model space after it’s imported into STAAD causing errors.

Creating nodes along each member so that they would line up with the plates yet to be created in the modeling software was critical. Two-foot spacings between the nodes were adhered to as much as possible while maintaining symmetry. The last step in AutoCAD was critical because in the that step the wire frame of the bridge was imported into STAAD-Pro V8, the program chosen for

all further model construction and analysis. At this point all of the steel elements of the bridge were present in the computer model. Dimensions of each element were included, and boundary conditions were applied to the appropriate nodes. The next step was the creation of the slab, which required detailed manipulation of the FEM mesh with two-foot wide plates while ensuring that the nodes of the plates were aligned with the nodes from the girders and supports. The final step in the creation of the model structure was an overview to ensure that all elements were offset to the appropriate levels to prevent overlapping materials in three-dimensional space.

Once the mesh discretization was complete for all plates and all the offsets had been input, multiple checks of the geometry were run for collinear members, improperly connected plates, improper beam plate connectivity, zero length members and orphan nodes. There are also checks for duplicate members and nodes. Any one of these issues could prevent the computer from completing the analysis in the future, so making sure that there were zero errors was essential in the process. Material properties were applied next, ensuring that they reflected any coupon or core testing results, if taken. For these purposes, the as-built plans included material properties that were incorporated here. Once the mesh discretization was complete for all plates and all checks were completed with no errors, the load cases are built. With the modeling system in STAAD-Pro custom truck loads with designer chosen distinct axle loads or standard truck loads can be used. For this case, an 8-kip axle load on the front axle, with two 32-kip axle loads for the two rear axles with all axles spaced at 14 feet. Each axle has the two contact points for the wheels spaced 6 feet from each other, in accordance with the MBE 6B.6.2-2. The load cases in STAAD-PRO allowed for movement of the load across a desired truck path. The program then tabulates the data for each step along the path, displaying an envelope complete with the results for the load case generated that produced the maximum effects for shear and moment in every global direction. The load

generation in STAAD moves the load along the deck in steps, and those step sizes were chosen to align with the nodes from the finite element mesh established earlier in the model construction.

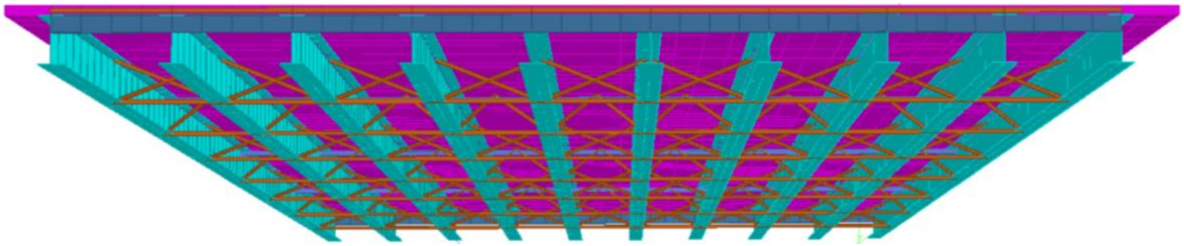

Figure 13: Schuster Overpass FE Model, STAAD Pro

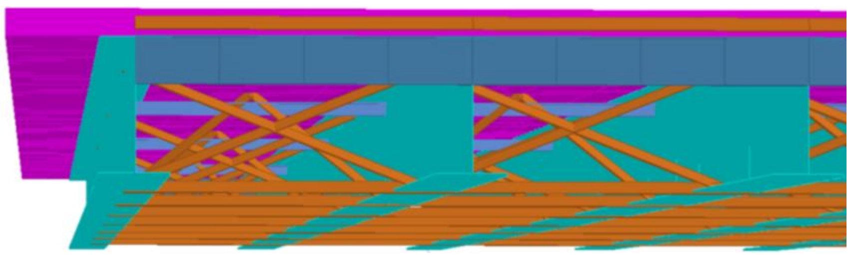

Figure 14: FE Model Diaphragm

Particular to the Schuster over pass is its combination of length and configuration as a two-span continuous bridge with a pier located at the midpoint. This arrangement meant that the load case that would cause the greatest effect would be combination of two trucks on each side of the pier

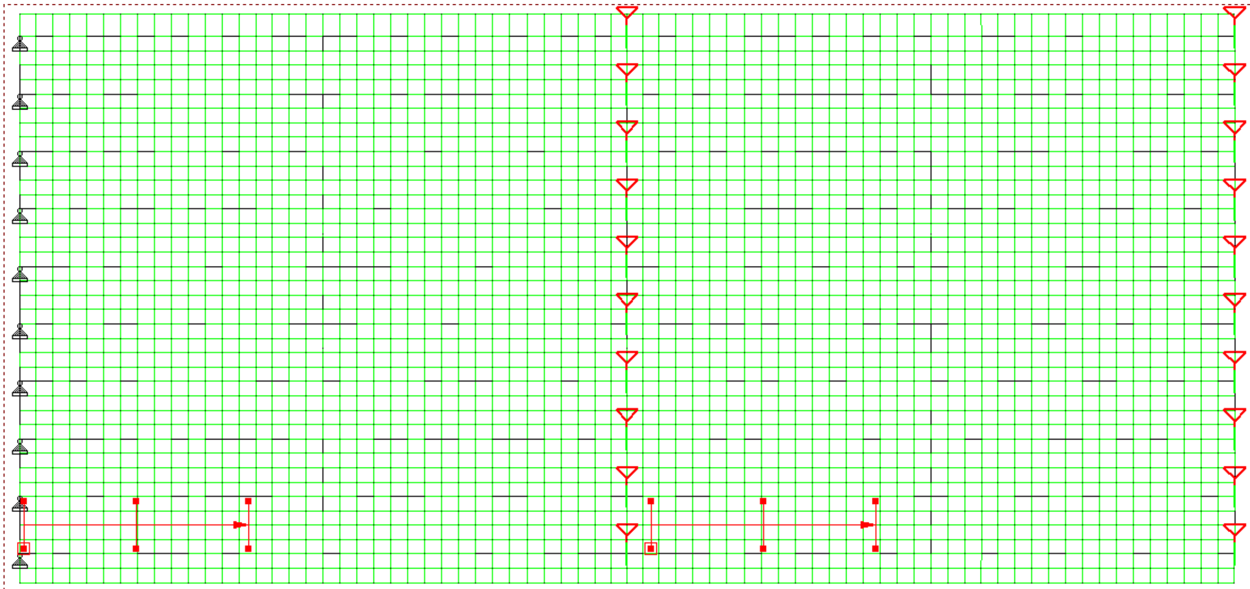

Figure 15: Additional Truck Load Case

with a large negative moment resulting above the pier. From this a final load case was created as seen in Figure 15 which was used in accordance with the MBE load case shown in Figure 16. The same design truck loads are used, only reduced 90%, to include the 640 pound per linear foot design lane load. It was found that this case did in fact generate the largest member effects and was used for all further analysis and comparison of distribution factors.

The spacing of the plates generated to represent the bridge deck are not two feet in measure laterally, which prevents one from reproducing a ten-foot-wide design lane load whose edges are aligned with the nodes of the plate elements of the deck. In all, there are 2808 plate elements representing the bridge deck and 790 elements representing the girder elements. STAAD-Pro allows one to create a load case and assign it to the desired plates as the modeler wishes, but that load must be applied to the entire plate or not at all. This results in a load case that is not precisely ten feet in width as the LRFD or MBE requires. In the initial load case, the width of the lane load may be 8.96 or 10.75 feet in width, depending on whether or not one applies the load case to 5 or

6 plates, respectively. In order to remain at least as conservative as the MBE and LRFD the 10.75-foot-wide load case was considered, and the design lane load of 640 kip/ft over the standard 10-foot-wide lane or 64 kips/ft/ft of width was simply applied to the 10.75-foot-wide modeling lane.

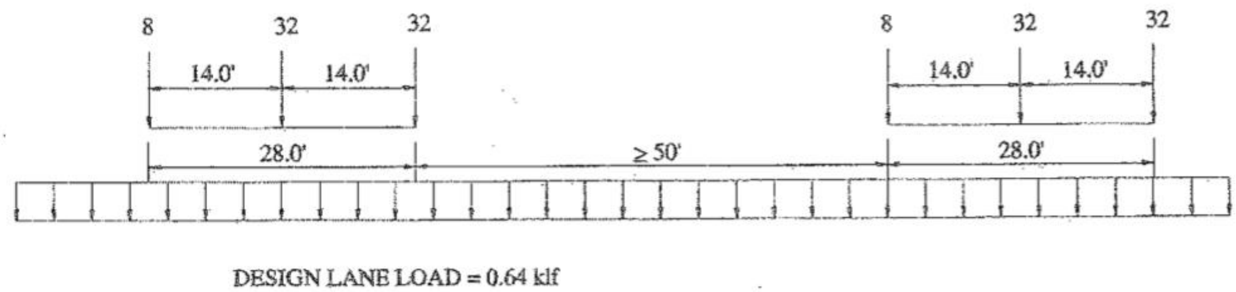

Figure 16: Additional Load Model for Negative Moment

4.6.2. FE Model Validation

The commercial STAAD-Pro software was utilized for the Schuster Overpass Bridge case study in order to model bridge's system behavior and more specifically to evaluate the load distribution. Upon researching it was found that after creation of a Finite Element Model or computer aided design, it is common place for the next step to be the validation or calibration of that model using real world diagnostics and load tests (Bell et al., 2013; M. J. Chajes & Shenton, 2006; Gheitasi & Harris, 2015; Provines et al., 2014). Without the ability to shut down traffic on a major national thoroughfare, an alternate means had to be used to validate and establish confidence in the STAAD-Pro Model. One means of accomplishing this was the reference of the information included in the AASHTO LRFD Bridge Design Specification Standards in regard to the maximum allowable deflection of a bridge by length. Using this method, while not as accurate as physical diagnostic load test of the in-situ bridge behavior, did provide an acceptable level of confidence because the loaded finite element model produced deflections less than the maximum deflection

of a span length. The Design Spec standard of L/800 for "decks with no pedestrian traffic" for the Schuster Bridge yields a maximum deflection of 1.14 inches while the model produced a deflection of .844 inches at the maximum. This is well within limits, but also realistic as a result.

4.6.3. Model Force Evaluation

Figure 15 shows the typical composite cross-section that is incorporated into the calculations for model force evaluations. Each model analysis of the separate truck paths generated a maximum moment at one of the beam nodes, which is then expanded into this form seen in Figure 17. In this focused view one can observe that each node is associated with one beam element and four deck elements, making up the tributary area for that node. The resultant axial forces

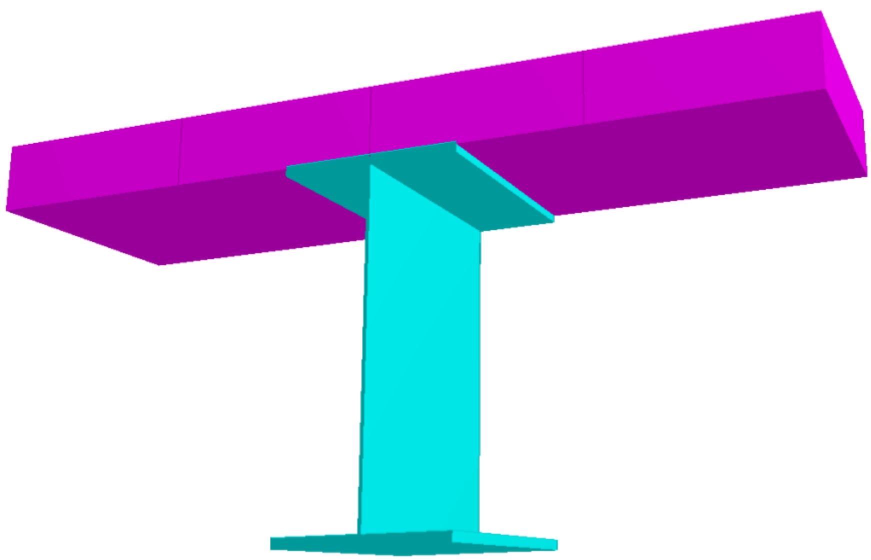

Figure 17: Typical 5-Element Deck & Girder Composite Section

and moment effects were then combined for the lane load effects and design truck effects. What results is are multiple calculations for each plate. Crucial to this work was the knowledge of the local coordinate system for each plate to ensure that the force effects were collected for the two nodes of the four-node plate that were adjacent to the plane cutting along the beam node under the

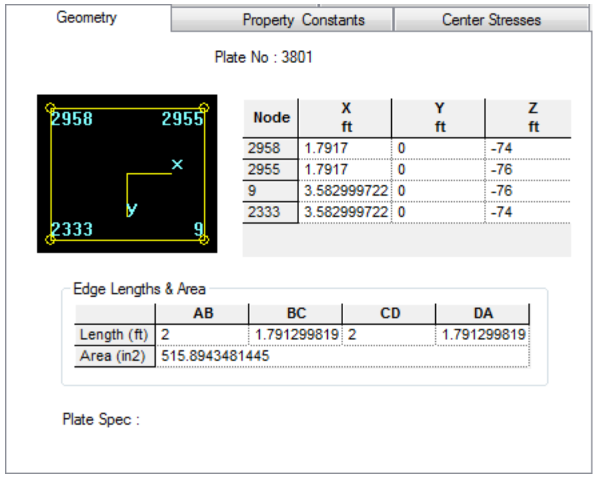

Figure 18: Sample Plate Local Coordinates

largest moment or shear. Not all plate coordinate axis matched the global coordinate axis for the model. A sample plate, taken from STAAD-PRO is shown in Figure 17. Using this technique, the load path generating the largest moment and shear effects were recorded from the FE model. It was determined that the maximum moment occurred during the 12th load generation and along the second truck path, four feet from the edge of the bridge, with a total moment of 1032.23 kip-ft.

5. Results and Discussion

5.1. Influence Variable Sensitivity Analysis

Each of the influence variables were used in the MATLAB program through a normal distribution with one-thousand iterations of rating factor calculations in accordance with Table 2. This analysis provides the answers to the bridge owner who asks where in the bridge variables can one extract improvements to the load rating, and which variables will provide the most positive change.

5.1.1. Deck Thickness

The design deck thickness was seven inches. At this value the inventory rating factor for shear is calculated to be 1 for inventory level and 1.294 at operational level. The rating factor decreases with the increases in the deck thickness maintaining this inverse relationship nearly linearly, indicating that the deck thickness only serves to add dead load to the bridge, which reduces its shear capacity for live load. The deck thickness does however work in composite with the steel girders to add flexural strength in a near linear fashion. At seven inches of depth the inventory and operational level flexural rating factors are 1.446 and 1.874, respectively.

The output data from the rating factor program, when graphed showed that the influence of additional, unaccounted for slab depth could produce an additional 3% to the inventory rating factor for moment, and 1.7% for shear. Compared to other influence factors, increases to deck thickness would be lower on the list of priority. One can see that in Table 6 the slab depth is the lowest in rank of influence for increasing rating factors for shear or moment.

5.1.2. Flange/Flange Plate Depth

There was a high level of confidence in the data plots for the bottom plate thickness. The member rating factor falls below tolerance in shear, just as the thickness of the bottom plate becomes

significant, however the capacity of the member for flexure begins to increase. The amount of loss in shear capacity is negligible compared to the gains in flexural strength. It follows from an understanding of beam behavior in bending that the self-weight gained by thickening the flange plate has a larger effect on the shear demand than the shear capacity added from that additional thickness. For the same reasons, it makes sense that the bridge designers included the thicker flange only in a 12’ range around the peak moment. If the entire length of the girder incorporated the larger flange, the added demand in shear would far outweigh the gains from the added material, most likely resulting in a completely redesigned girder.

That additional depth, implemented specifically at the location of the peak moment, does have a significant effect on the rating factor for flexure which increases 5.2% for inventory moment but not at all for shear. The depth was varied plus or minus one eighth of an inch from the design width.

5.1.3. Flange/Flange Plate Width

Much of the same observations for the flange and flange plate depth and their effect on the load rating can be made of the flange and flange plate widths and their effect on the load rating and rating factor. There is very little change in rating factor for shear as the plate width is increased. Again, the additional mass from this change in geometry has more of an effect on the dead load of the structure than it does on the capacity of the structure.

5.1.4. Distribution Factor for Moment

The next influence factor that engineers should analyze is the moment distribution factor. In the rating factor program for moment the factors were allowed to generate rating factors resulting from a change in the distribution factor range of 0.5. Distribution factors plus .25 and minus .25 of the

original rating factor were generated, and the result was a 0.1 decrease in the distribution factor for moment generated a 23.6% increase in the rating factor.

Using this influence factor has the most effect on the load ratings for bridges, and the bridge engineering community has researched this aspect in depth. The discussion of whether or not the amount of potential gain in rating factor from this analysis is worth the cost put into the model generation and analysis. This influence factor is a complement with the refined modeling approach, whose primary goal is to produce more accurate distribution factors. With an idea of how much of an increase can be generated bridge owners can determine if the cost-benefit ratio is feasible.

5.1.5. Distribution Factor for Shear

The same observations made for the distribution factor for moment can be made for the effect that the shear distribution factor has on the rating factor. Distribution factors plus .25 and minus .25 of the original rating factors were also generated, and like the plots for the moment distribution factors the rating factor at the 0.1 mark were noted. A 0.1 decrease in this distribution factor generated a 11.7% increase in the rating factor, which is not as large an increase as was observed for moment yet is still significant.

This influence factor is second on the influence factor chart seen in table 4 but should be considered to be studied in conjunction with the analysis of the moment distribution factors. The process is much of the same. An engineer would not create a finite element model for a structure only to extract only the distribution factors for one part of the load effects. The generation of an FE model will benefit both influence factors. Generating the distribution factors for shear after having generated them for moment is a few more steps in an already lengthy process and the cost benefit

is substantive. As the rankings of these influence factors go, the increase in rating factor from a more accurate shear distribution factor would be ranked number one with the moment DF influence variable.

5.1.6. Compression Strength of Concrete (f′c)

The rating factor for shear controls and has an ordinate of 1.0 for every value of concrete compressive strength. For this test f′c was varied uniformly between 2.4 and 6 ksi for a thousand iterations. The rating factor for inventory shear stays constant at 1.0 while the rating factory for operational shear remains constant at 1.3.

The rating factors for inventory/operational flexure increase linearly with the increase in concrete compressive strength from 2.4 to 3.3 ksi. At that point (3.3 ksi) the rating factor jumps down from 1.48 (1.93 operational), to a RF of 1.37 (1.79 operational). It then begins to increase again, now parabolically to a RF of 1.66 (2.15 operational) at a concrete compressive strength of 6. The step in the function was revealed to be a change in the empirical formulas used in the analytical approach that occurs when the neutral axis moves through the composite beam cross-section from the steel to the concrete.

The concrete compressive strength as an influence factor comes fourth on the rankings of moment influence and doesn’t rank for shear. This is directly a result of the fact that the rating factors for shear see no change. That said, engineering judgment should be used in determining which influence factor to pursue. If the rating factors for shear control a tier 2 or 3 bridge, starting with core samples from the concrete deck to do strength testing on those samples would most likely not produce a significant increase in rating factor for the number of man-hours spent on the inherent field and lab work.

5.1.7. Yield Strength of Steel (Fy)

The yield strength of the steel in the case study bridge was the next influence factor under examination. For this factor, a lower bound of 30 ksi and an upper bound of 40 ksi were used. The Schuster bridge was built with 36 ksi steel and the data demonstrates that when the strength of steel surpasses 36 ksi the inventory rating for shear simultaneously surpasses a RF of one. The plot shows a positive relationship between yield strength and rating factor for all cases shown, but also shows a step in the trend line around 33 ksi, which, like the concrete analysis, was a result of movement of the plastic neutral axis upward through different elements of the composite cross-section.

When the percentage of change to the rating factor was calculated it was found to be the most impactful of all the chosen influence factors. An increase of four ksi produced a 23.6% increase to the inventory level rating factor for moment and a 6.8% increase to the inventory level rating factor for shear. If there is reason to believe the strength is higher than what has been previously estimated, performing further strength testing may produce significant benefits in rating factor calculations. Caution and engineering judgment are required as with most aspects, since coupon testing could reveal a lower strength steel in use. The results were linear with minor exceptions, so the losses in rating factor would be equal and opposite.

5.1.8. Dynamic Load Allowance (Impact Factor)

The results for the data gathered when manipulating the dynamic load allowance follows when viewing the plots for the effect of IM vs RF. Increasing IM causes a decrease in the rating factor with the RF for inventory shear crossing a value of 1 at .33, which is a standard value of IM that the bridge was designed for. More importantly it shows that if IM can be lowered to .2 an increase in RF of 7.5% is obtained. The rating factors for flexure are well above 1, with the RF for flexure

displaying a value of 1.88 at .33IM and over 2 at an IM for .10.

The question for bridge owners is how they wish to affect the dynamic load allowance. This influence factor falls into the operational category, meaning that something physically must be

Table 6: Influence Factor Results

Steel Girder		Moment	Shear
Analytical	Rating Approach ASR	-	-
	Rating Approach LRFR	1.45	1.00
	Girder Distribution Factors (minus 0.1)	23%	11.7%
	Material Properties (Fy plus 4 ksi)	23.6%	6.8%
	Material Properties (f'c plus 1 ksi)	14.8%	0.0%
	Material Geometry - Flange Plate Width (plus 1/16")	0.0%	0.0%
	Material Geometry - Flange Plate Depth (plus 1/16")	5.2%	0.0%
	System Factor (minus .15)	20.8%	20.4%
	Slab Depth (plus .5 inches)	3.0%	1.7%
Operational	Road Width Reduction	-	-
	Resurface Approach Deck	18.8%	14.5%
	Speed Enforcement	9.8%	7.7%
	Encasement	-	-

altered on the bridge site to reduce the effects of vehicles traveling across the bridge at high speed. The first means of accomplishing this is to reduce the dynamic load effects from moving vehicles due to an uneven approach to the bridge. Resurfacing the road on the approach to the bridge would cause a smooth transition for trucks driving onto the bridge, lessening vertical oscillations as they move across the bridge which normally causes greater force effect than that of a static load.

The second means is speed enforcement, which is a difficult bar to achieve. According to the

MBE if speed is reduced to lower levels and is enforceable then the dynamic load allowance may be reduced. This action will have consequences on traffic and businesses that may take detours to avoid the speed enforcement methods and possible traffic jams that these means may cause. There are 2nd order effects to bridges on detour routes that may see an increase in their Average Daily Truck Traffic (ADTT). Bridges on those routes may now be subject to loading that will expedite their deterioration. But key in this factor reduction is the idea that this speed reduction must be enforced if it is to be viable.

5.1.9. Deck Thickness

Shear is the controlling factor for this bridge, and the RF intersects the value of 1.0 at the level that it was designed for, a deck thickness of 7.0 inches. However, when plotted the data shows that a lower deck thickness could be used which would allow higher RF for shear while maintaining a flexural capacity that keeps the RF well above one as well. At six inches the RF would be 3% higher and the RF for inventory flexure is well above 1.3. The supposition, keeping in mind that this analysis takes place at the negative moment zone, is that increased deck thickness adds to bridge life, as there is no additional wearing surface in the as built designs of this bridge. Standards of design and building practice that require proper amounts of bottom and top cover for the steel reinforcement are considered the deck thickness as well. Specifically, with the concrete in the center region above the pier is in tension with the reinforcing steel there supporting the tension zone of the superstructure.

Ultimately, engineering judgment and experience will be the determining factor for exercise of this influence factor, as the effect provides a 3% increase in the inventory moment rating factor with a corresponding increase of deck thickness of .5 inches. That however will cause a drop in the shear rating factor by 1.7%. Moving left on the plot one observes that a decrease in the deck

thickness by .5 yields an increase of 1.8% in the shear rating factor for inventory levels, and a corresponding drop of 3.1% in the moment rating factor also for inventory levels.

5.2. Distribution Factor Analysis

The distribution factors that were calculated using the analytical approach from the MBE needed to be compared to the distribution factors that were calculated from FE model. The model doesn't use distribution factors; however, they can be calculated. From there they are input back into the analytical approach, overriding those calculation and producing a new refined load rating.

Table 7: Distribution Factor Calculations

Beam	Load Case (Uniform load / Vehicle Load)	Maximum moment (kip - in)	Combined moment (kip - in)	Total moment	Distribution Factor
7	LOAD CASE 1	1395.889	6736.992	16086	0.419
	LOAD GEN, LOAD #12	5341.103			
17	LOAD CASE 1	1017.381	4795.777		0.298
	LOAD GEN, LOAD #12	3778.396			
27	LOAD CASE 1	536.104	2756.838		0.171
	LOAD GEN, LOAD #12	2220.734			
37	LOAD CASE 1	251.761	1324.932		0.082
	LOAD GEN, LOAD #12	1073.171			
47	LOAD CASE 1	89.835	471.461		0.029
	LOAD GEN, LOAD #12	381.626			
57	LOAD CASE 1	5.646	28.247	-884.05	-0.032
	LOAD GEN, LOAD #12	22.601			
67	LOAD CASE 1	-30.333	-160.603		0.182
	LOAD GEN, LOAD #12	-130.27			
77	LOAD CASE 1	-42.88	-226.189		0.256
	LOAD GEN, LOAD #12	-183.309			
87	LOAD CASE 1	-47.22	-249.151		0.282
	LOAD GEN, LOAD #12,	-201.931			
97	LOAD CASE 1	-52.202	-276.354		0.313
	LOAD GEN, LOAD #12	-224.152			

To do this the model would be run for every truck path, varying the spacing of each truck path by the same discretization of the model deck plates. Once the situation that produces the maximum moment is found that load case and truck path become the path for all future examination. With that specific load case and truck path being held constant a cross-section of the bridge is cut at the

point of the highest ordinate of the moment. That beam element, approximately two feet in length, is identified and each similar girder element along the path of the cross-sectional cut is also identified.

The output for each of these ten-beam sections is collected and analyzed together, summing all of the moments in order to find the percentage that the stringer with the maximum moment withstands from the total. This is the distribution factor. For the case of the Schuster Bridge the largest moment occurred as a negative moment directly above the pier near on the exterior girder closest to the truck paths. This followed logically from the as-built plans, where this section of stringers was stiffer due to a quarter inch thicker top and bottom flange. For twelve feet of the negative moment region the bridge girders maintain thicker flanges. The calculations mirror the calculations described by Gheitasi et al (2015) where the distribution factor is "determined by taking the ratio of response in a given member to the summation of all primary load-carrying member responses."

5.3. LRM Approach

Acknowledging that accurate calculations can be made for every influence factor for the majority of bridge types, one can theorize that these factors may be referenced in the decision-making process on future actions for Tier 2 and 3 bridges. With a fully develop influence factor list, beginning with this research, and the knowledge of which aspect of a bridge is the controlling factor placing it amongst the Tier 2 & 3 populations, one need only standard engineering judgment to help bridge owners make better decisions. Figuring out where to focus is assisted with the following formula:

$$RF_{initial} \times IF \geq RF_{acceptable} - RF_{initial}$$

Where:

IF= Influence Factor

$RF_{acceptable}$ = Acceptable Rating Factor

$RF_{initial}$ = Current Rating Factor

The left side of the equation is the amount of increase that a particular influence factor can be expected to exert on the current rating factor, while the right side shows the delta required to bring the current rating factor to within standards.

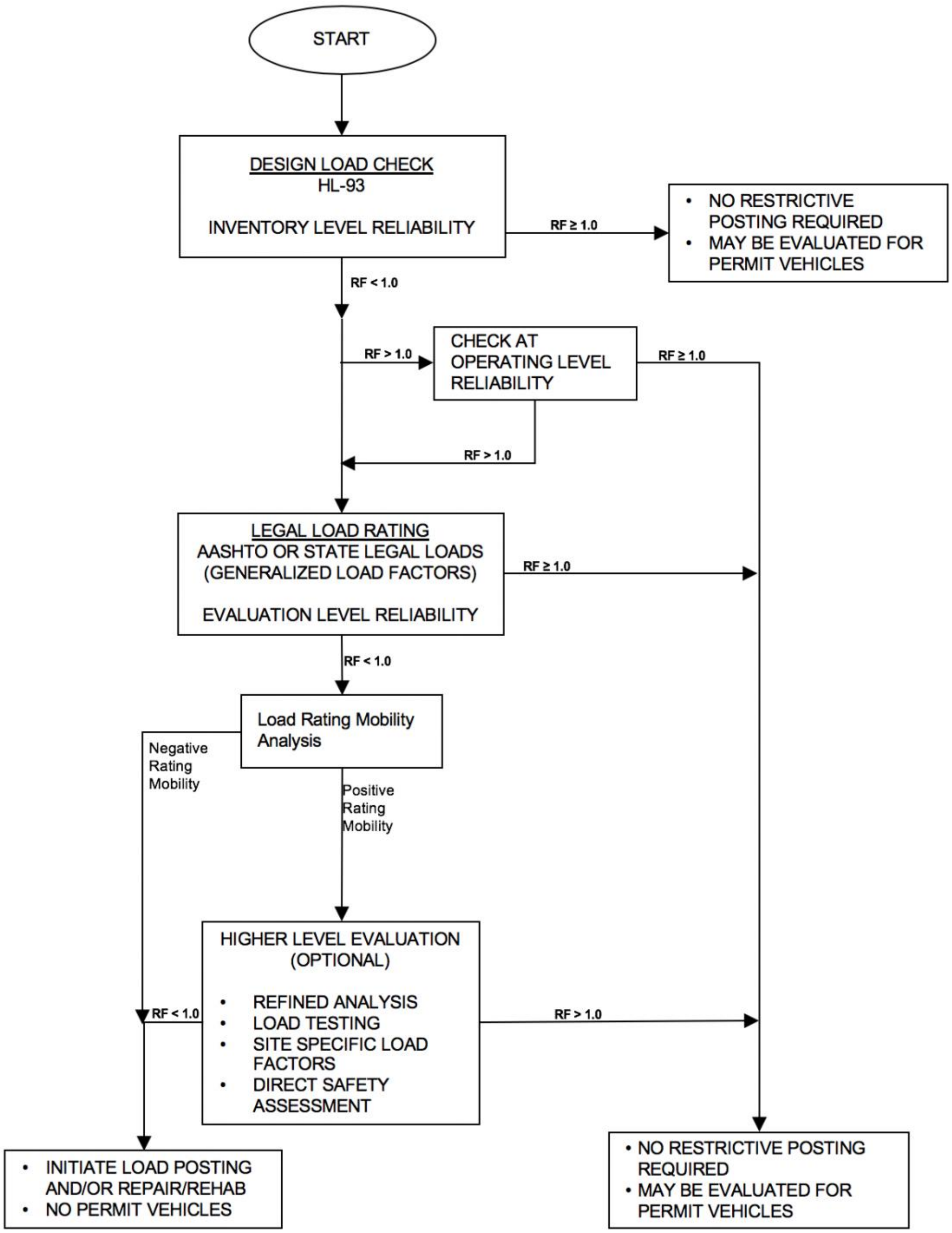

Figure 19: Revised Load Rating Flow Chart (LRM Analysis Included)

Determining which inputs mattered will still be an engineering judgement but this method, once fully developed, will create an additional check that will produce results for bridge owners in the number of man-hours not wasted on evaluations that will most likely not yield desired results.

Load Rating Mobility doesn't uncover the flexibility in the system of load ratings, which the community already concurs leaves a lot of available load capacity untapped. LRM provides a systematic approach which can be used to focus efforts on a specific bridge population that can be maintained in the un-posted category while simultaneously disregarding bridges that are beyond the benefits.

Load Rating Mobility, if adopted could spark discussion as an additional argument for the changing appropriation of moneys meant for infrastructure rehabilitation. As a means to more efficiently rate and rank the possible options for bridges in danger of being posted or having just been posted, Load Rating Mobility could be a means of shifting funding priorities away from wasteful projects and towards projects that will provide benefits to the current transportation network.

6. Conclusions and Recommendation

6.1. Conclusion

Load Rating Mobility is a novel approach to the decision-making process surrounding bridge load rating and rehabilitation. That initial process is one that bridge owners must go through trying to decide whether or not to post a bridge or enact repairs. Based on the premise that some bridges are beyond repair, some are completely functional, and many are between, LRM uses a tiered structure to classify these populations. This paper presented the tiers of bridges, the process used to select bridges in these tiers, and a case study on one bridge to introduce and demonstrate the various influence factors and how they can be used to reveal a more accurate and favorable load rating value. Several programs were written using the AASHTO theoretical calculations for load ratings, which allowed testing and manipulation of the various inputs in order to record the effects of the rating factor's sensitivity to their change. A validated Finite Element Model was built from as-built plans provided by TxDOT. That model provided information on the system behavior of the case study bridge as well as refined distribution factors that were substituted for the empirical distribution factors calculated in the standard numerical approach. This was another influence factor, and a very common factor among bridge engineering researchers addressing rating factors.

This research, if continued, should have a long-lasting effect in the bridge engineering and load rating community as well as with Bridge Owners that must balance bridge maintenance with funding shortfalls. There are never enough funds for all of the projects required to keep this vital aspect of the infrastructure functioning to the standards required by the public. The method introduced in this research, if practiced would go a long way to ensure that resources flow to the projects that will see the most benefit from those means.

6.2. Recommendations for Future Work

This study was successful as a starting point for a larger more influential process that should see more research and analysis done to make Load Rating Mobility an integral process in the overall methodology of bridge load rating. Many additional tasks exist for a full-fledged Load Rating Mobility system.

Table 8: RF Influence Factor Results

		RF Percent Change from Factor (Inventory)							
		Slab		Concrete Multi-Girder		Steel Girder		Other Bridge Types	
		M	V	M	V	M	V	M	V
Analytical	Rating Approach ASR	-	-	-	-	-	-	-	-
	Rating Approach LRFR	-	-	-	-	1.45	1.00	-	-
	Girder Distribution Factors (minus 0.1)	-	-	-	-	23%	11.7%	-	-
	Material Properties (Fy plus 4 ksi)	-	-	-	-	23.6%	6.8%	-	-
	Material Properties (f'c plus 1 ksi)	-	-	-	-	14.8%	0.0%	-	-
	Material Geometry - Flange Plate Width (plus 1/16")	-	-	-	-	0.0%	0.0%	-	-
	Material Geometry - Flange Plate Depth (plus 1/16")	-	-	-	-	5.2%	0.0%	-	-
	System Factor (minus .15)	-	-	-	-	20.8%	20.4%	-	-
	Slab Depth (plus .5 inches)	-	-	-	-	3.0%	1.7%	-	-
Operational	Road Width Reduction	-	-	-	-	-	-	-	-
	ADTT	-	-	-	-	-	-	-	-
	Resurface Approach Deck	-	-	-	-	7.8%	78.8%	-	-
	Speed Enforcement	-	-	-	-	7.8%	7.3%	-	-
	Encasement	-	-	-	-	-	-	-	-

More work must be done to conduct a Load Rating Mobility analysis on all types of bridges in a

similar manner as the case study documented within this study. The information in Table 8, when completed, would go a long way to furthering this research. Each bridge type should have multiple bridges evaluated for each including the load rating program that address the particular influence variables of each bridge type in order to increase the level of confidence in this method. This includes the creation of a finite element model of each bridge type to insure incorporation of accurate distribution factors, which was the main driver for lowering the rating factor. The more bridges researched in this manner and brought under the awning of the LRM study the higher the confidence factor for the process going further.

Better coordination in the future for all bridge research brought under this program will help with validation of 3-D Finite Element Models. Field work that is incorporated specifically for validating future models with diagnostic load testing will also go a long way to building confidence in the results of this research and the processes within.

Finally, execution of a full Load Rating Mobility analysis on a local level bridge without plans should be included in future work. Utilizing field measurements, coupon testing and core sampling from a bridge without as-built plans would assist in bringing more structure types under the exploration of this study and provide valuable data for Tier 2 and Tier 3 bridges that may not exist under Federal and State ownership.

This research has introduced an innovative approach to decision making in regard to load rating and the posting of bridges failing to meet federal and state capacity requirements. Continuing research along this line of study should act as a foundation for a more efficient system of load rating and resource management for bridge owners in the future. A fully formed Load Rating Mobility procedure, nested in the AASHTO load rating methodology would insure that valuable

resources are not squandered on Tier 4 bridges while reducing the number of load posted bridges across the entire US bridge population.

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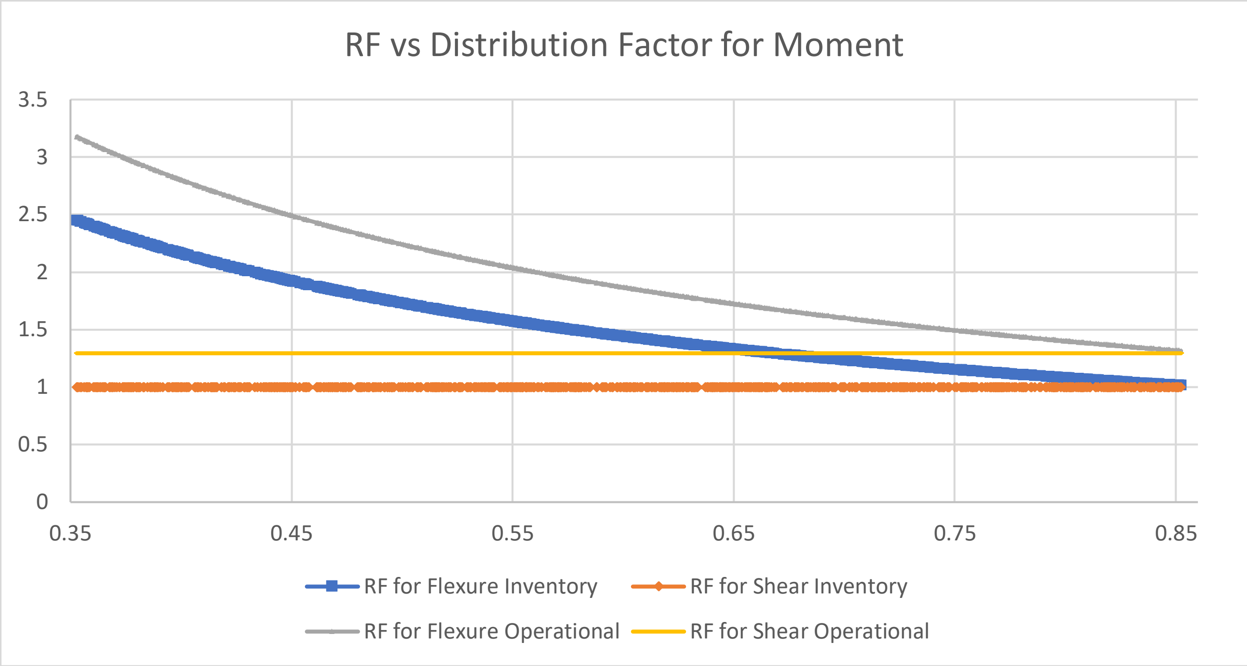
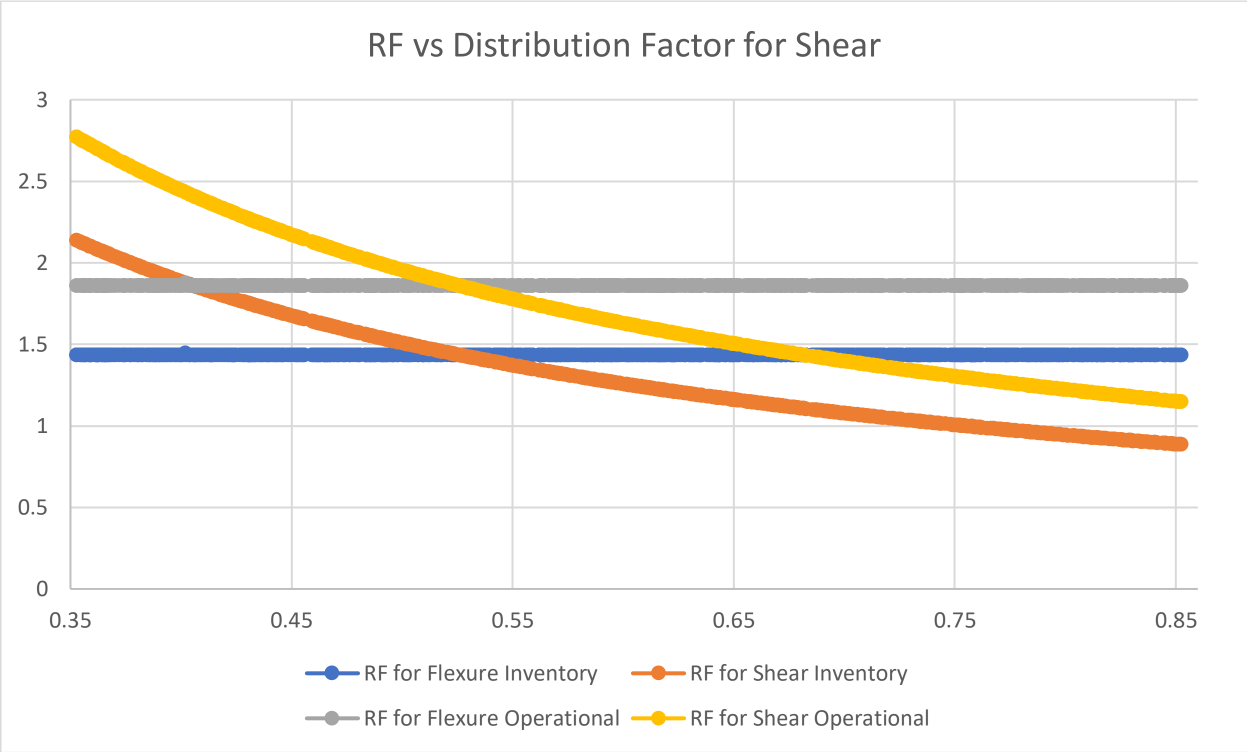
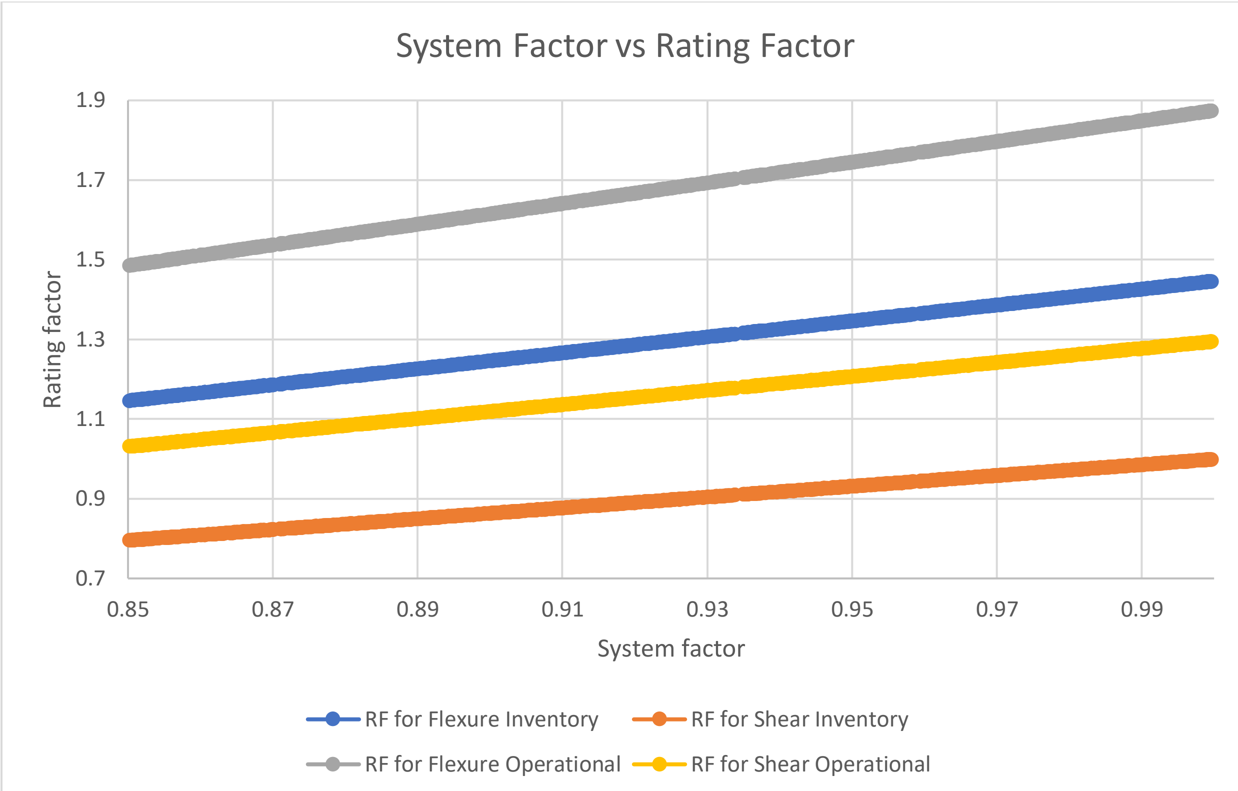
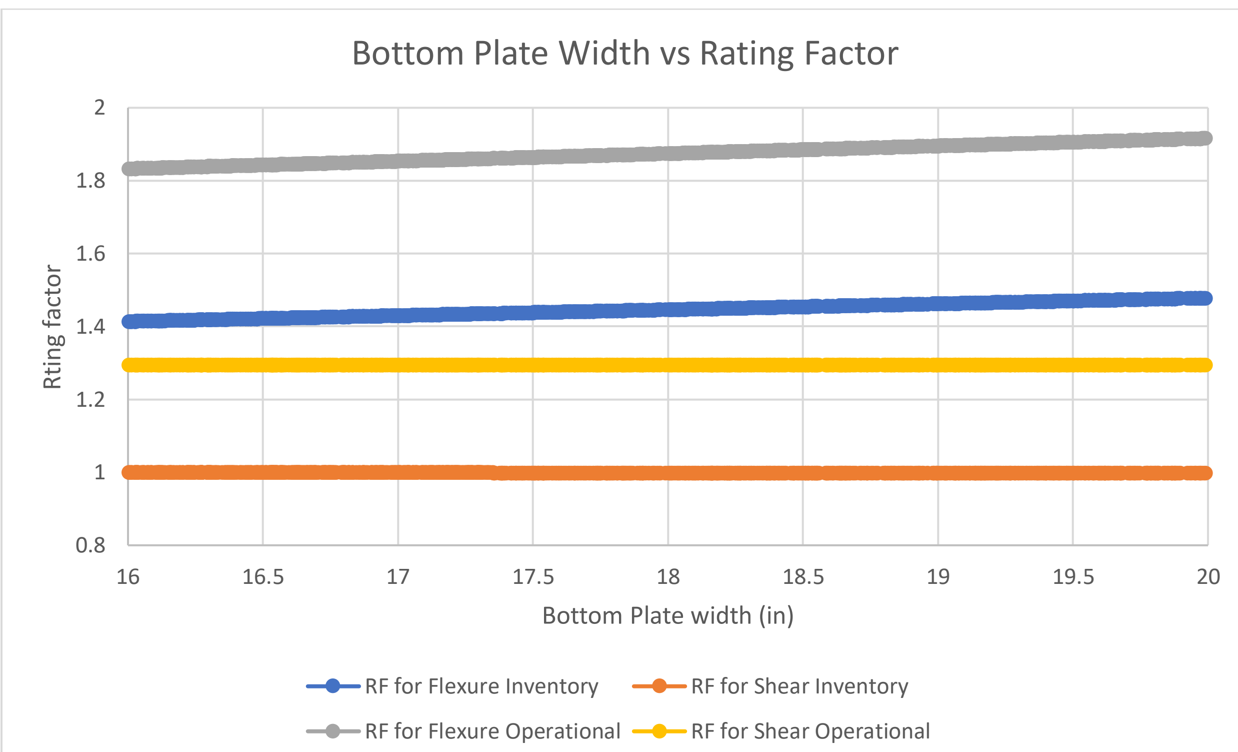
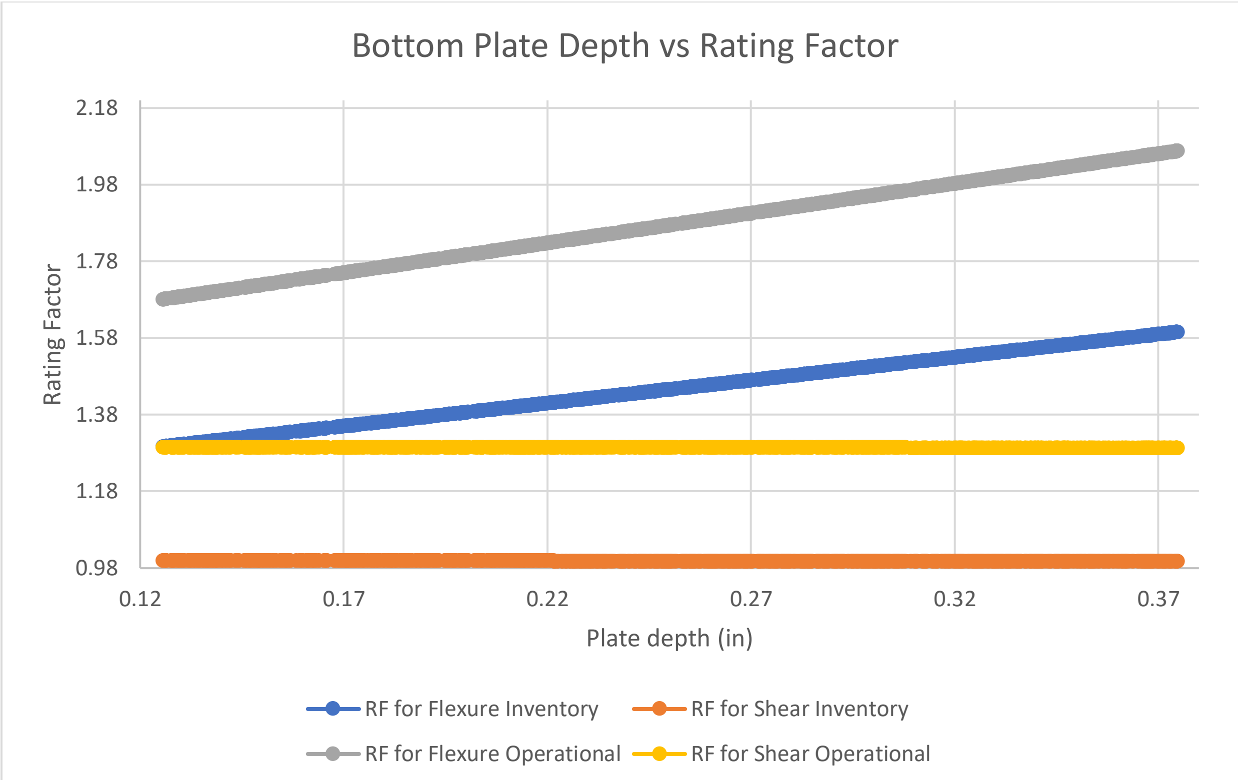
8. Appendices



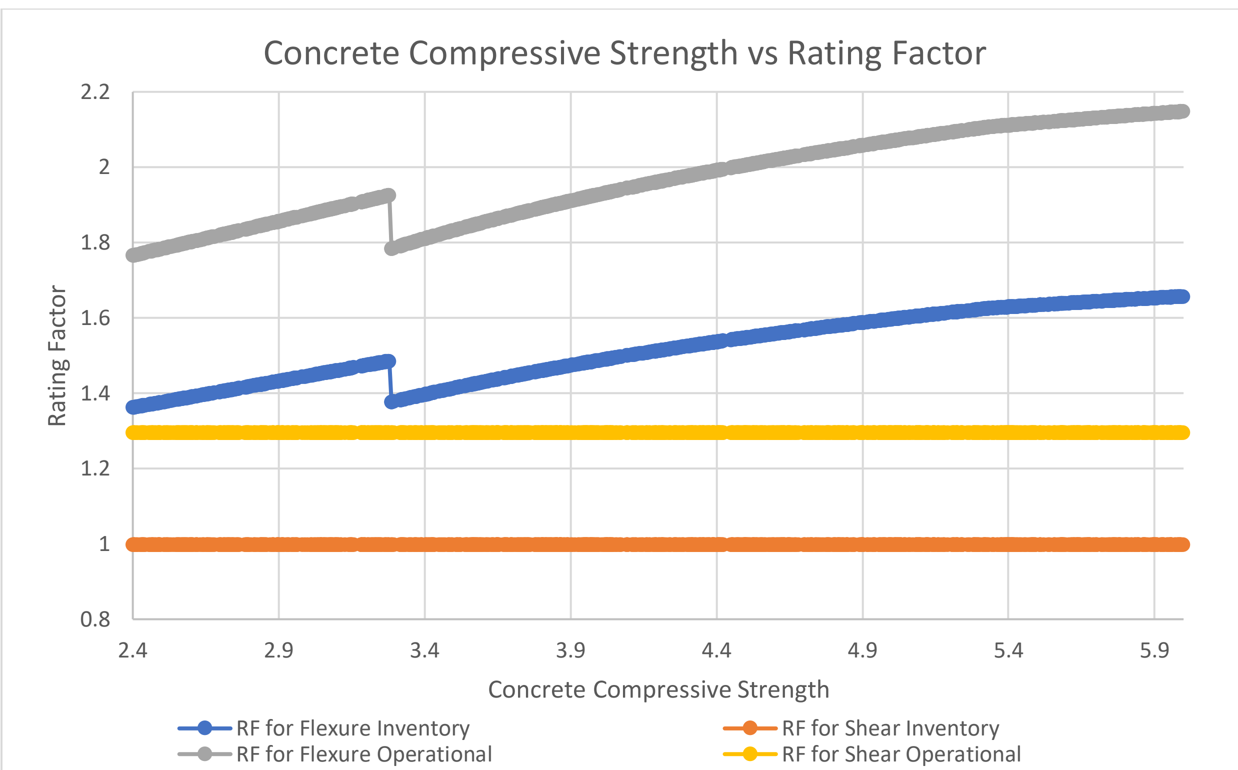



System Factor vs Rating Factor
Rating factor
1.9
1.7
1.5
1.3
1.1
0.9
0.7
0.85
0.87
0.89
0.91
0.93
0.95
0.97
0.99
System factor
RF for Flexure Inventory
RF for Shear Inventory
RF for Flexure Operational
RF for Shear Operational

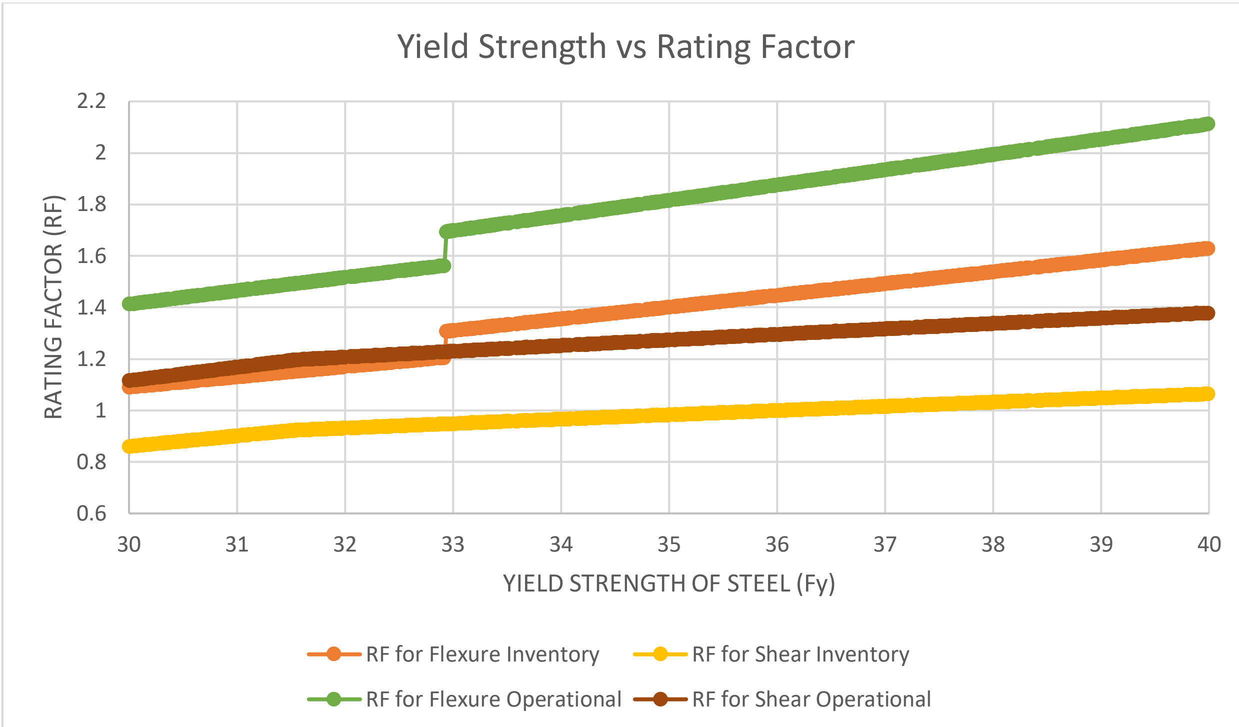

Bottom Plate Width vs Rating Factor
Rting factor
2
1.8
1.6
1.4
1.2
1
0.8
16
16.5
17
17.5
18
18.5
19
19.5
20
Bottom Plate width (in)
RF for Flexure Inventory
RF for Shear Inventory
RF for Flexure Operational
RF for Shear Operational



Bottom Plate Depth vs Rating Factor
Rating Factor
2.18
1.98
1.78
1.58
1.38
1.18
0.98
0.12
0.17
0.22
0.27
0.32
0.37
Plate depth (in)
RF for Flexure Inventory
RF for Shear Inventory
RF for Flexure Operational
RF for Shear Operational



Concrete Compressive Strength vs Rating Factor
Rating Factor
2.2
2
1.8
1.6
1.4
1.2
1
0.8
2.4
2.9
3.4
3.9
4.4
4.9
5.4
5.9
Concrete Compressive Strength
RF for Flexure Inventory
RF for Shear Inventory
RF for Flexure Operational
RF for Shear Operational


Yield Strength vs Rating Factor
RATING FACTOR (RF)
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
30
31
32
33
34
35
36
37
38
39
40
YIELD STRENGTH OF STEEL (Fy)
RF for Flexure Inventory
RF for Shear Inventory
RF for Flexure Operational
RF for Shear Operational

9. Vita

Bryan Stephens was born in 1980 in the Republic of Panama to Ricardo and Celeste Stephens. The son of an Army officer he decided to earn his Bachelor of Science in Civil Engineering from The United States Military Academy at Westpoint, New York in 2004. Bryan then served in the United States Army for twelve years as an Army Artillery officer before separating from the army at the rank of Captain. Through that time, he deployed to Iraq, commanded an Artillery Battery, taught safety and range operations at the National Training Center, at Fort Irwin, California and married his wife of 6 years. Following his service Bryan attended The University of Texas at El Paso where he worked as a research and teaching assistant and is currently pursuing a Master of Science in Civil Engineering. Following completion Bryan seeks to enter the Civil Engineering industry in the area of bridge design.