SUPERSTRUCTURE BRIDGE SELECTION BASED ON BRIDGE LIFE-CYCLE COST ANALYSIS

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To my loved ones, you made this possible.

To Mom who keeps smiling from wherever you are.

To Endrina, the love of my life.

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ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AB	Prestressed Concrete AASHTO Beams
ABC	Accelerated Bridge Construction
ABD	Prestressed Concrete AASHTO Beams with Diaphragms
ACI	American Concrete Institute
A-D	Anderson-Darling Test
ADTT	Average Daily Truck Traffic
AFSD	Almost First Stochastic Dominance
ASSD	Almost Second Stochastic Dominance
ASTM	American Society for Testing and Materials
BCR	Benefit-Cost Ratio
BD	Superstructure Removal Cost
BLCCA	Bridge Life-Cycle Cost Analysis
BLS	Bureau of Labor Statistics
BR	Bearing Replacement Cost
BT	Prestressed Concrete Bulb Tee
BTD	Prestressed Concrete Bulb Tee with Diaphragms
CB	Prestressed Concrete Box Beams
CC	Construction Costs
CDF	Cumulative Density Function
CLT	Central Limit Theorem
CP	Full-Depth Concrete Patching Cost

CPI	Constumer Price Index
CRF	Capital Recovery Factor
DC	Design Costs
DR	Concrete Deck Reconstruction Cost
DR	Discount Rate
ECDF	Empirical Cumulative Density Function
ER	Cost-Effectiveness Ratio
ES	Efficient Set
EUAC	Equivalent Uniform Annual Cost
EUAR	Equivalent Uniform Annual Return
FP	Full Painting Cost
FPG	Folded Plate Girder Bridge System
FRC	Fiber-Reinforced Concrete
FS	Feasible Set
FSD	First Stochastic Dominance
IBMS	Indiana Bridge Management System
INDOT	Indiana Department of Transportation
IQR	Interquartile Range
IRR	Internal Rate of Return
IS	Inefficient Set
K-S	Kormogorov-Smirnov Test
LCC	Life-Cycle Cost
LCCA	Life-Cycle Cost Analysis
LCCAP	Life-Cycle Cost Analysis in Perpetuity
LCCP	Life-Cycle Cost Profile
LFT	Linear Foot
LRFD	Load and Resistance Factor Design
MCS	Monte Carlo Simulation
ML	Most Likely Value

NBI	National Bridge Inventory
NCHRP	National Cooperative Highway Research Program
NDOR	Nebraska Deparment of Roads
NHS	National Highway System
NPV	Net Present Value
O	Concrete Overlays Cost
PCI	Precast Concrete Institute
PDF	Probability Density Function
PERT	Program Evaluation and Review Technique
PMS	Pavement Management Systems
PWC	Present Worth of Cost
RC	Rehabilitation Costs
SBXG	Structural Steel Beam Galvanized
SBXP	Structural Steel Beam Painted
SB	Slab Bridge
SC	Sealing of the Deck Surface Cost
SD	Stochastic Dominance
SDCL	Simply Supported Span for Dead Load and Continuous for Live Load Steel Beams
SFDF	Sinking Fund Deposit Factor
SFT	Square Feet
SL	Service Life
SP	Spot Painting Cost
SPACF	Single Payment Compound Amount Factor
SPGXG	Structural Steel Plate Girder Galvanized
SPGXP	Structural Steel Plate Girder Painted
SPPWF	Single Payment Present Worth Factor
SR	Structural Steel Recycle Cost
SSD	Second Stochastic Dominance

SSSBA	Short Span Steel Bridge Alliance
SV	Salvage Cost
TTC	Travel Time Cost
UC	User Costs
USCAF	Uniform Series Compound Amount Factor
USPWF	Uniform Series Present Worth Factor
VOC	Vehicle Operation Cost
WC	Washing and Cleaning of Deck Surface Cost

ABSTRACT

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Life cycle cost analysis (LCCA) has been defined as a method to assess the total cost of a project. It is a simple tool to use when a single project has different alternatives that fulfill the original requirements. Different alternatives could differ in initial investment, operational and maintenance costs among other long term costs. The cost involved in building a bridge depends upon many different factors. Moreover, long-term cost needs to be considered to estimate the true overall cost of the project and determine its life-cycle cost. Without watchful consideration of the long-term costs and full life cycle costing, current investment decisions that look attractive could result in a waste of economic resources in the future. This research is focused on short and medium span bridges (between 30-ft and 130-ft) which represents 65% of the NBI INDIANA bridge inventory.

Bridges are categorized in three different groups of span ranges. Different superstructure types are considered for both concrete and steel options. Types considered include: bulb tees, AASHTO prestressed beams, slab bridges, prestressed concrete box beams, steel beams, steel girders, folded plate girders and simply supported steel beams for dead load and continuous for live load (SDCL). A design plan composed of simply supported bridges and continuous spans arrangements was carried out. Analysis for short and medium span bridges in Indiana based on LCCA is presented for different span ranges and span configurations.

Deterministic and stochastic analysis were done for all the span ranges considered. Monte Carlo Simulations (MCS) were used and the categorization of the different superstructure alternatives was done based on stochastic dominance. First, second, almost first and almost second stochastic dominance rules were used to determined the efficient set for each span length and all span configurations. Cost-effective life cycle cost profiles for each superstructure type were proposed. Additionally, the top three cost-effective alternatives for superstructure types depending on the span length are presented as well as the optimum superstructure types set for both simply supported and continuous beams. Results will help designers to consider the most cost-effective bridge solution for new projects, resulting in cost savings for agencies involved.

1. INTRODUCTION

Life cycle cost analysis (LCCA) is a method used to assess the total cost of a project. LCCA is particularly useful when a single project has different alternatives that fulfill the original requirements. Different alternatives could vary in initial investment or cost, operational costs, maintenance costs or other long term costs. This kind of analysis, when applied to bridge infrastructure projects is called Bridge Life-cycle Cost Analysis (BLCCA). According to NCHRP Report 483 [1]: Several recent legislative and regulatory requirements recognized the potential benefits of life-cycle cost analysis and call for consideration of such analyses for infrastructure investments, including investments in highway bridge programs. This contemporary tendency has been the main driving force for the research and use of BLCCA throughout the country. The current study is focused on efforts to identify the best approach to incorporate BLCCA in new bridge construction in Indiana.

The true cost of a bridge structure is the cost to build, inspect and maintain the bridge over the entire lifespan of the bridge. Typically, decisions regarding selection of the superstructure type when a new or replacement bridge is needed are based solely upon the initial construction cost, rather than the life-cycle cost. There are very few data or prior published studies regarding the life-cycle cost of entire bridge structures in Indiana that utilize different materials. A study to evaluate these costs would be useful for efficient and cost-effective future planning.

This research is focused on short to medium span bridges (less than 130-ft) which represents 65% of the NBI Indiana bridge inventory. Bridges are categorized in three different groups of span ranges. Different superstructure types are considered for both concrete and steel options. Types considered include: bulb tees, AASHTO prestressed beams, slab bridges, prestressed concrete box beams, steel beams, steel girders, folded plate girders and simply supported steel beams for dead load and continuous for live load (SDCL). A design plan composed of simply supported bridges and continuous spans arrangements was carried out. Analysis for short and medium span bridges in Indiana based on LCCA is presented for different span ranges and span configurations. Findings will help designers to consider the most cost-effective bridge solution for new projects, resulting in cost savings for agencies involved.

The cost involved in building a bridge depends upon different factors. Features such as the number of substructure elements needed, the right-of-way and earthwork required to develop the height of the approach due to the depth of the bridge structure type, the typical deck span and thickness for the superstructure, the span length, the material properties, the distance for shipping from the precast plant or fabrication shop to the bridge site, and the familiarity of the contractors with the type of bridge construction play a role in the first cost to some extent. However, long-term cost needs to be considered to estimate the overall cost of the project and determine its LCCA.

Long-term cost includes, but are not limited to, the following costs: repair or rehabilitation of the bridge deck, repair of collision-damaged concrete or steel girders, re-painting a steel bridge, removal of the deck for a pre-stressed bulb-tee without damaging the girder, routine maintenance, the cost of inspection for fracture-critical steel bridges, inspection to identify and repair duct voids in grouted post-tensioned concrete bridges, and miscellaneous minor repairs such as spot painting or concrete patching.

Without watchful consideration of the long-term costs and full life cycle costing, current investment decisions that look attractive could result in a waste of economic resources in the future. The design decision at the beginning of the project can create less than optimal requirements in future years. According to American Society of Civil Engineers and Eno Center of Transportation [2]. “An examination of the full life cycle costs can help an agency in determining the appropriate investment in an asset given current and future constraints.”

1.1 Objective

The purpose of the proposed research is to examine the life-cycle costs associated with steel and concrete bridge structures of comparable types and sizes. The bridge study will be limited to bridges that have an overall length in the range of 30-ft to 130-ft. The study will examine various bridges for a given site condition such as a particular span length and optimal configuration for the overall bridge length considering structural continuity, etc to determine the life-cycle costs of the bridges. The final result of the study will then be a set of guideline recommendations that a designer may use to achieve the greatest long-term cost efficiency.

The research objectives of this project are as follows:

1. Evaluate different design solutions for different span arrangements in terms of its cost-effectiveness using Bridge Life Cycle Cost Analysis.

2. Categorize the most effective bridge solutions in different span ranges.

3. Propose life-cycle profiles for different superstructure types.

4. Identify the most cost-effective maintenance and major work actions for each design option from the LCCA stand point.

1.2 Organization

A literature review is shown in Chapter 2, including: topics as cost effective alternatives for short and medium bridges, deterioration rates used for prediction of service lives of different bridge structures, and a summary of bridge life-cycle cost assessment.

Chapter 3 presents all the considerations made to establish the bridge design plan used for this research. Topics such as superstructure type selection, span configurations and span range selection are covered. Finally, a final bridge design matrix is presented along with the common design assumptions made for all the designs developed.

Cost allocation is summarized in Chapter 4. Description of agency and user costs are presented. Specifics on values and database usage for every pay item identified are shown. In addition to common statistic indicators for every pay item, probability distribution fitting and probability distribution parameterization is done and presented.

Chapter 5 shows the deterioration models used for different superstructure types, NBI data and existing deterioration models proposed by different authors are presented and used.

A literature review on different working actions is presented in Chapter 6. Based on the deterioration models obtained before, different life-cycle profiles for different superstructure types are proposed. Finally, the most cost-effective life-cycle profiles are summarized for different superstructure types.

Chapter 7 presents both deterministic and stochastic approaches used for computing the life-cycle cost analysis. Deterministic analysis compared not only the life-cycle cost but also the initial cost for different superstructure types. Additionally, Monte Carlo simulations are used for the stochastic analysis. Conclusions on both methods are presented as well as the most cost effective alternatives depending on the bridge span length.

Finally, Chapter 8 offers a summary of study along with concluding remarks and suggestions for practitioners.

2. LITERATURE REVIEW

This section presents a literature review on innovative cost effective solutions for short span bridges. Also, a literature review on deterioration curves is included. In addition, current approaches taken to conduct a Bridge Life Cycle Cost Assessment are summarized.

2.1 Bridge Superstructure Types

Multiple design solutions have been investigated and used throughout the years with the objective not only of proposing a structural solution for bridges but also to provide a cost-effective option for owners and agencies. These two have been the motivating force of numerous advances in the steel and concrete bridge industries. Structural systems such as reinforced concrete slab bridges, prestressed concrete bulb tees, prestressed concrete box beams, prestressed concrete AASHTO beams, steel beams, steel plate girders and steel box girders have been commonly used across the country. Nonetheless, the options discussed herein correspond to new technologies or, in some cases, recent approaches to standard systems that could provide a great design solution with competitive costs.

2.1.1 Steel Bridges

Folded plate girder (FPG) bridge system

This design approach utilizes U-type shapes built from, cold-bending flat steel plates into tub sections using a press-brake. According to the Short Span Steel Bridge Alliance (SSSBA) a maximum span of 60-ft is able to take advantage of this system [1]. Folds are uniform but thicknesses and dimensions vary depending on project conditions. Concrete is typically cast in the shop to connect the folded plates to the deck as part of a prefabricated section. Two different options have been considered in recent years. One is a folded plate that is closed at the top by the concrete deck which is connected by shear studs placed in top flanges disposed at each side of the beam (See Figure 2.1). For further references this option will be called the folded plate bridge system. In contrast, the second option uses the folded plate upside down, which means that the deck will be connected throughout the back of the folded plate by shear studs. This second option implies that the bottom of the bridge is open (see Figure 2.1). For further references this option will be called the inverse folded plate bridge system.

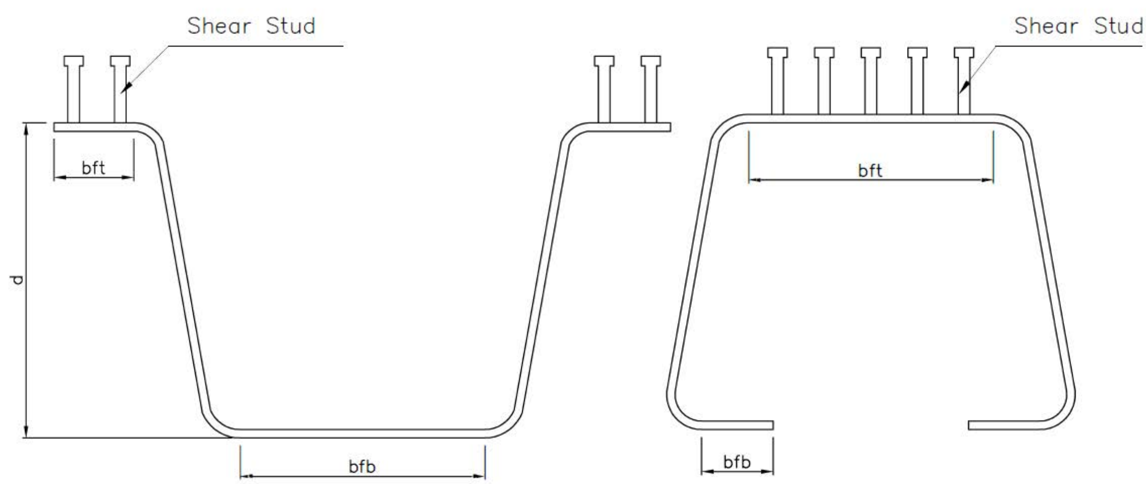


Fig. 2.1. Folded plate girder.

Since late 1970s the idea of prefabricated press-formed steel T-Box girder bridge system has been of special concern of the structural research community. Taly and Gangaro [3] proposed this system as a feasible option for highway bridges. Top-

[1]Short Span Steel Bridge Alliance,"Press-Brake-Formed Tub Girders",American Iron and Steel Institute.www.shortspansteelbridges.org/steel-solutions/press-brake-tub-girders (accessed January 24, 2019).

ics treated includes design basics, fabrication solutions, feasibility study, erection considerations, bearing types, end joints solutions, curb, parapet and railing types, maintenance aspects and alternative design procedures.

The investigation developed by Barth et al. [4] describes the procedure to develop the FPG bridge system. Methodology of the design proposed, along with experimental validation for the composite girders flexural capacity are presented. Results show that AASHTO specifications used to compute composite girders ultimate capacity are conservative. Finally, a more accurate proposal to compute the flexural capacity is proposed.

Inverse folded bridge system described by Burner [5] is cold bent out of a single sheet of steel. Six specimens containing closure regions were subjected to both positive and negative moment loading to investigate their behavior and failure modes under ultimate load. Fatigue resistance along with hooked construction joints were studied (in comparison with the headed bars construction joints). Conclusions of the research indicates that this bridge system can withstand the equivalent 75 years of the physical maximum traffic without significant loss of stiffness. Additionally, headed bars and hooked bars for the construction joint provided sufficient strength and ductility for both positive and negative moments, however, hooked joints are preferred due to its low-cost fabrication and ease in detailing and fabrication.

A project that used inverse folded plate girders as an ABC solution was monitored by Civjan et al. [6]. This study was sponsored by the Massachusetts Department of Transportation, and focused on monitoring a single-span integral-abutment bridge. Results indicated that the neutral axis is located above the one assumed from section properties. However, stresses in concrete and steel components are within values expected not only during construction, but also during long term data collection and truck load testing.

A report presented to the Michigan Department of Transportation by Pavlich and Burgueo [7] had the objective to evaluate through numerical simulations the feasibility of creating an entirely prefabricated composite box girder bridge system and employing such system for highway bridges. Topics such as composite girder/deck joints, vibration characteristics, longitudinal joint of girder/deck units, transversally posttensioned joints and others were studied. Different longitudinal joint connections are reviewed including: grouted shear keys, reinforced shear keys, post tensioned grouted shear keys, welded plate grouted shear key blocks, reinforced grouted moment key blocks and posttensioned grouted moment keys. Cost, structural performance, constructability, design ease and other topics were analyzed for spans under 100-ft. There is not a conclusive selection of joints based on performance or strength. However, it is concluded that according to the parametric study the performance of all the different joints considered were adequate for spans ranging from 50-ft to 100-ft..

Other researches like the one published by Nakamura [8] describes a new type of steel and concrete composite bridge with steel U-shape girders. From the economical point of view, lack of welding in comparison with regular I-shape girders is an advantage for this system and therefore very cost-effective. Testing of folded plate girders replicating loads due to construction without using prefabricated beams were carried out at the University of Nebraska [9]. Two different plate girder specimens were tested. To consider proper behavior simulating construction stages, the behavior of the girder alone was evaluated and no concrete slab was cast in any specimen. The objective of the test was to estimate not only the overall behavior but the girder components performance. Load levels to cause failure were included, also modes of failure were reported. Results prove that the folded plate girder provides adequate strength and stability resistance during construction.

Simply supported span for dead load and continuous for live load (SDCL)

Simple span steel members are utilized at the early construction stages (dead load only), and then modified by adding the required continuity tension and compression details during construction to create a continuous structural system. This structural system eliminates field splices when spans are shorter than transportation limitations. According to the SSSBA normal detailing includes various combinations of anchor bolts, sole plates and often expensive bearing types. The SDCL method is considered as a special construction process rather than an application of special bridge elements.

Azizinamini et al. [10] in conjunction with the Nebraska Department of Roads (NDOR) and the University of Nebraska Lincoln examined a new steel bridge system which considers simply supported beams for dead load and continuous spans for live loads. Two full-scale specimens were constructed and tested in order to determine their structural behavior. Ultimate load tests were conducted to investigate the failure mechanism. As a result, design equations were developed and verified through finite element analysis.

Independent design professionals have been proposing SDCL systems as a cost-effective solution for the bridge industry according to Henkle [11]. For Instance, Hoorpah et al. (2015) presents the experience with Colville Deverell bridge located in Mauritius Island. The SDCL system is presented as an economic and fast construction technology for developing countries. Zanon et al. [12] presented an example of the use of an SDCL project as part of a new express road construction in Gdansk, Poland. Some of the points highlighted by this project are mainly focused on the advantage of prefabrication cost and effective procedures for medium span bridges, especially for the span range between 80-ft and115-ft.

Finally, a cost-benefit analysis was conducted by Azizinamini et al. [10] for two different structures, a steel box girder superstructure and a steel I-girder superstructure. It is shown that girders are slightly heavier using the SDCL system in comparison with the conventional continuous bridge system. However, the elimination of field splices reduced the total cost of the structural elements by 7% in both cases.

2.1.2 Concrete Bridges

A paper summarizing the Japanese state of the art was published by Yamane et al. [13] on short to medium span (16-ft to 130-ft) precast pre-stressed concrete bridges. Topics such as construction techniques, design procedures and overall costs for bridges in Japan and the United States were reviewed. This document presents a summary of basic geometrical considerations for different bridge types including typical span ranges.

Bulb tee and hybrid bulb tee beams

Bridges using bulb tee beams consist of a horizontal slab supported by beams, which are supported either by abutments at both ends or at interior points for continuous beams. The cross section of the beam is designed to have optimal material and structural resistance, commonly fabricated in I shapes (see Figure 2.2). Due to the maximized moment of inertia obtained with the cross section, long spans can be considered for this type of bridge. Industry has standardized heights and general dimensions.

A precast bulb tee pre-stressed concrete girders system is being used as a bridge rapid construction option. Due to construction procedures, load transfer between adjacent girders is provided by the composite concrete deck. Bardow et al. [14] discussed the advantages of the approach through the examination of the New England bulb-tee precast girder proposed by New England Precast Concrete Institute (PCI) committee. Reasons such as limitations in the range of applicability from the previous standardized American Association of State Highway and Transportation Officials (AASTHO) I girders and successful experiences of other states using more efficient precast girder shapes influenced the committee to propose bulb tee girders as an option in bridge design. A summary is provided on the girder depth limitation, as well as shipping and erection issues. Also, reviews of the new standardized sections completed by University of Nebraska and PCI are mentioned. Parallel to this proposal, the bridge

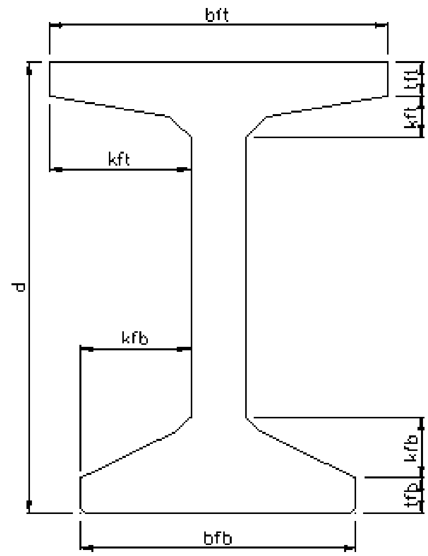


Fig. 2.2. Typical bulb tee girder.

portion of the Boston central artery project was designed using the new bulb tees suggested by the committee. As a result of this cooperation, a standardized bulb tee sections were adopted, and have been used in numerous projects since then.

2.1.3 Deterioration Factors

Deterioration models for bridges were introduced into the life cycle cost assessment during the 1980s as a result of the rising interest in predicting the future condition of infrastructure assets [15]. Nonetheless, those models have been researched prior to the 80s for pavement management systems (PMS). Difference between these two approaches focus mainly on the importance of safety, construction materials used and structural functionality. Even knowing the differences between them, the approaches used to deal with the deterioration of infrastructure assets (no matter its origin) are based on the same principles. “By definition, a bridge deterioration model is a link between a measure of bridge condition that assesses the extent and severity of damages, and a vector of explanatory variables that represent the factors affecting bridge deterioration such as age, material properties, applied loads, environmental conditions, etc." [16].

Deterioration curves have been understood as a model intended to describe the process and mechanisms by which assets deteriorate and even fail through its service life. Probabilistic and statistical methods are usually used to accomplish this goal, leading to a graphical representation of the deterioration of the structure (see example in Figure 2.3 based on the deterioration curves given by Moomen et al. [17]).

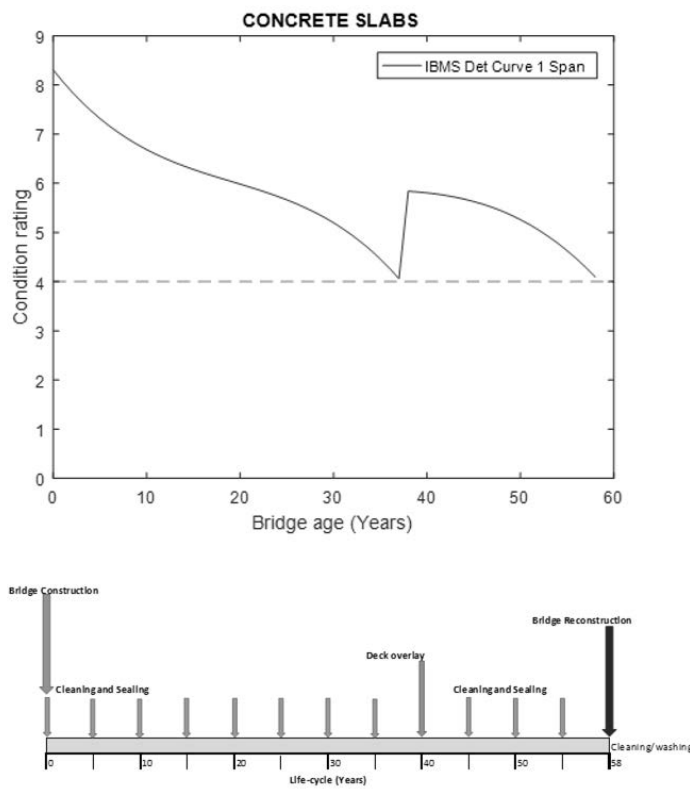


Fig. 2.3. Typical life cycle condition with repairs and renewals.

There are some key components that must be determined to develop a deterioration model of a structure. The most important of them are the following:

- The anticipated deterioration rate of the element. Known as the pace at which an asset degrades over time under operating conditions. This must be taken into account from the beginning of the life of the structure.

- The thresholds that define the start and the end of the maintenance stages.

- Actions to take into account at different points and during sequential stages. The jumps in the deterioration curves are intended to extend the service life of the asset or to accomplish the overall life cycle objective of the structure.

The basic data used to develop a deterioration prediction is based on the condition ratings. Condition ratings reflect the deterioration or damage of the structure but not design deficiencies. To address these scenarios, the National Bridge Inventory (NBI) classifies them as Structurally Deficient or Functionally Obsolete. Based on field inspections the condition ratings are considered more like snapshots in time rather than prediction of future conditions or behavior of the structure.

As a rule, the NBI regulated the condition ratings as a numerical coding from 0 to 9, in which 9 reflects "excellent condition" and 0 represents the "failed condition" - see Table 2.1. For further details, see the official NBI condition ratings document.

Using condition ratings, it is possible to develop a model that predicts the future condition of the structure analyzed. The basic representation of this analysis takes the current condition of the asset and predicts how the condition rating will change in future years if no maintenance is performed. Some of the options found in the literature for the predictive modeling include deterministic analysis and stochastic analysis.

Deterministic analysis

Deterministic analysis models contain no random variables (no probabilities involved) and no degree of randomness. It is dependent on a mathematical formula for the relationship between the factors affecting the bridge deterioration and the measure of the condition of the asset. The output obtained is commonly expressed by deterministic values that represent the average predicted condition. This type of model can be developed using extrapolations, regressions or curve-fitting techniques [15].

Table 2.1.
General description of bride elements condition ratings

State	Description
N	Not applicable
9	Excellent Condition
8	Very Good Condition - No problem noted
7	Good Condition - Some minor problems
6	Satisfactory Condition
5	Fair Condition
4	Poor Condition
3	Serious Condition
2	Critical Condition
1	“Imminent” Failure Condition
0	Failed Condition

The Nebraska Department of Transportation sponsored a research project to develop specific models for Nebraskas bridges [18]. This project was focused on the application of both deterministic and stochastic analysis in bridge decks. Some key conclusions were made including the significant impact of the traffic volume (AADT and ADTT) on the deck deterioration. Also, the importance of environmental and climate changes throughout the state were addressed. It was found that higher traffic volumes increase the deterioration rate for bridge decks. In addition, in the detailed report on bridge decks, Morcous [15] also analyzed superstructures and substructures. Data suggest that prestressed concrete superstructures have similar performance to steel structures up to condition 6 for Nebraska bridges. Below condition 6 no adequate condition data for prestressed concrete superstructure were found.

Indiana sponsored a recent project focused on updating bridge deterioration models though its Department of Transportation [17]. The final report identifies independent variables such as bridge age, features to cross beneath the bridge, ADTT among others. This document presents different deterioration curves divided in different groups depending on the material and design types. Curves for decks, different superstructure types and substructures are summarized. Also, it presents the different significant explanatory variables used for each probabilistic model. Finally, deterministic and probabilistic case examples are presented using the outcome of the curves presented. Findings identified trends in the deterioration rates linked to the independent variables used. Data show that the road classification influences highway bridge deterioration due to the related ADTT. Higher ADTT values result in higher deterioration rates. In addition, bridges located over waterways tend to deteriorate faster than bridges traversing other features.

Stochastic analysis: Markov Chains

A stochastic model traces the projection of variables that can change randomly with certain probabilities. In this specific case, deterioration progression is set as one or more stochastic variables that capture the uncertainty of the process. Two different approximations could be made in this kind of model: state-based and time-based approximation [19]. State-based models predict the probability that an asset will undergo a change in condition state at a given time. One of the most known examples of this model are the Markov chains and the semi-Markov processes. On the other hand, time-based models predict the probability distribution of the time taken by an asset to change its condition state. This type of approximation has been used more frequently in pavement deterioration modeling. However, the two modeling approaches can be related. It is possible to use one modeling approach to predict the dependent variable of the other.

A stochastic process can be considered as Markovian if the future behavior depends only on the present condition but not on the past. In other words, if the state is known at any given time, no more information is needed in order to predict the future state of the asset [20].

The most important step when a Markov chain method is used is the computation of the matrix that contains the transition probabilities, which represents the probability of an element to remain or change from one rating to the other. Transition probabilities can be obtained either from accumulated condition data or by using an expert judgment elicitation procedure [15]. Different methods can be used to generate transition probabilities. However, there are two which have been used to solve this problem using the condition data available: regression based optimization and percentage prediction method. The first one solves the non-linear optimization problem minimizing the sum of the absolute differences between the regression curve that best fits the condition data and the predictions using the Markov chains. This method can be greatly influenced by maintenance that are not reported to the database used. This means that any change in the data base will have a significant impact in the outcome. The second approach relates the number of transitions from one state to another within a given time span with the number of structures in the original state.

Markovians biggest disadvantage is the inherent assumption of the future condition as independent of the historical condition of the asset. The Markov process assumes, in theory, a programmed and fixed inspection interval for bridges occurs, but in practice, bridges can be inspected less or more frequently than programmed for reasons such as financial limitations and technical challenges. The Markov chain has its merits, such as accounting for the stochastic nature of deterioration, facilitation of the condition characterization of large bridge networks and its computational efficiency and simplicity [17].

2.1.4 Bridge Life-Cycle Cost Analysis (BLCCA)

For projects related with infrastructure, decision makers often have constrained budgets. Consequently, decision makers and elected officials often only consider short term cost (a.k.a. initial cost), rather than the long term costs. However, failure to consider long term costs could lead to decisions that are costlier over the service life of the structure.

According to the American Society of Civil Engineers and Eno Center of Transportation [2] bridge life cycle cost analysis (BLCCA) is defined as “a data-driven tool that provides a detailed account of the total cost of a project over its expected life”. In addition, “BLCCA has been proven to create short-term savings for transportation agencies and infrastructure owners by helping decision-makers identify the most beneficial and cost effective projects and alternatives.” Numerous transportation agencies throughout the country have been using BLCCA as a tool for policymakers. BLCCA has several applications, including:

- Calculating the most cost-effective approaches to project implementation.
- Evaluating a design requirement within a specific project, such as material type in bridge construction.
- Comparing overall costs between different types of projects to help prioritize limited funding in an agency-wide program.

Even though BLCCA is presented as a precise tool to allocate budgets, the approximation itself has different limitations that the agency using it must consider. The most notorious constraint is the reliability of the prediction of future costs. Determination of such predictions are subjected to a substantial estimating risk that can radically modify the outcome. A second limitation is based on the time horizons of the analysis. Setting different time horizons can have a dramatic effect on the analysis results. However, the most important issue is attributed to the lack of transparency and full knowledge of how BLCCA works and how it can be implemented. It is important to understand that BLCCA must not be considered as an infallible tool to predict future costs. Nevertheless, it is a helpful instrument to provide better information to decision-makers.

BLCCA is based upon a series of factors that need to be quantified and investigated. First, there is a need to identify alternatives, not only of the structural type or material but also bridge maintenance and improvement that may vary with the locations depending on weather conditions and contractors experience. Second, agency costs need to be addressed. These are (but not limited to) maintenance, rehabilitation and replacement costs. "Most routine maintenance activities are performed by an agencys own workforce. Rehabilitation works consist of minor and major repair activities that may require the assistance of design engineers and contractors for construction. Most rehabilitation work is deck related. A major rehabilitation activity may involve deck replacement. The term bridge replacement" is, on the other hand, reserved for a complete replacement of the entire bridge structure [1].

An accurate estimation and prediction of such prices is a difficult task since they tend to fluctuate. Moreover, those prices are connected with the length and type of bridge work programed in each of the alternatives. Finally, user costs that are the value of time lost by the user due to delays, detours and road work. There are other costs such as salvage costs, staffing, tax implications, downtime and so forth, that would be present in the BLCCA depending on the government dispositions.

General models for BLCCA are summarized as the sum of nonrecurring cost and recurring costs. The final cost is the result of adding the construction costs, maintenance costs and rehabilitation costs among others. Those cost must include not only appropriate agency costs but also user costs. Specifically, the model for bridges is presented in Equation 2.1 [1].

$$LCC = DC + CC + MC + RC + UC + SV \tag{2.1}$$

were:

LCC: Life-Cycle Cost

DC: Design Cost

CC: Construction Cost

MC: Maintenance Cost

RC: Rehabilitation Cost

UC: User Cost

SV: Salvage Cost

Measurements commonly used for alternative selection are: net present value (NPV), equivalent uniform annual cost (EUAC) and incremental rate of return.

Life-cycle cost profiles (LCCP)

Life-cycle profiles were conceived as graphical representation of all the costs involved during the service life of a given structure. Those include not only the major correcting actions (e.g. reconstruction of an element, construction of overlays, bridge replacement) but also routine working actions characteristic of the bridge life. The combination of different maintenance, preventive or major corrective actions creates a unique profile that can be considered. Accurate estimation of service lives for all the working actions is a combination of agency experience, research efforts and engineering judgment.

Bridges typically involve three different elements that could have different working actions to consider: deck, superstructure, and substructure. It is true that a combination of all of them results in a complete LCCA. However, this research is only focused on the deck and the superstructure. Superstructure working actions often involve the full or partial intervention of the deck. Therefore, life-cycle profiles proposed here on are a combination of preventive / maintenance / repair / rehabilitation strategies of both elements.

The following are the crucial factors to consider when a life cycle profile is proposed: the service life of the structure, working actions considered, life-cycle of the treatments proposed, proposed schedule of major working actions and possible extensions of the structure service life due to preventive or corrective procedures.

The service life of the structures considered corresponds to the age at which the deterioration curve used reaches the limiting condition rating. According to Indiana experience, the limiting condition rating that triggers the scheduling of a working actions corresponds to Poor Condition (condition rating 4). It is true that this condition does not mean imminent failure or a collapse but it is considered a safe threshold to assure safety standards.

3. BRIDGE SUPERSTRUCTURE DESIGN ALTERNATIVES

3.1 Superstructure Type Selection

Information obtained from the National Bridge Inventory (NBI) was used to summarize the most common structures within the state and generate a bridge design matrix for the structures to analyze. The NBI database is an open source information that can be found in the National Bridge Inventory webpage and can be used freely.

The Indiana Department of Transportation (INDOT) has been collecting information on highway construction projects since 2011. This information has been organized and compiled in a single database that includes not only the total cost of different projects but also discretizes pay items involved. As can be seen in Figure 3.1, the INDOT database shows a predominant use of concrete that represents 72% of the bridge contracts built from 2011 to 2015. In contrast, structural steel was used only 28% of the time. This tendency can be seen at a network level also analyzing the NBI database. According to NBI data, approximately 67% of the structures are concrete or prestressed concrete bridges (distributed almost evenly) while 30% are structural steel. This trend may be driven by the first cost effectiveness of concrete in comparison with structural steel.

The designs selected for this study represent the most common structures found in Indiana (as shown in Figure 3.1), as well as other bridge design options. It should be noted, however, that design options for timber, masonry, aluminum or other materials are not considered. Bridge types used are the following:

- Slab bridge
- Prestressed concrete box beams

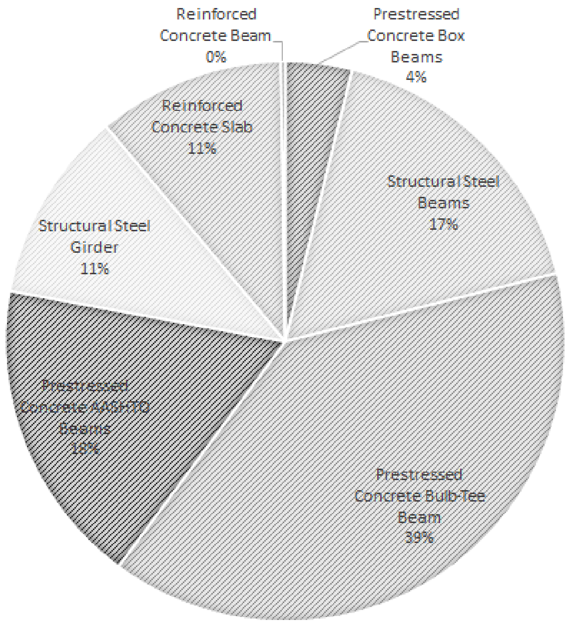


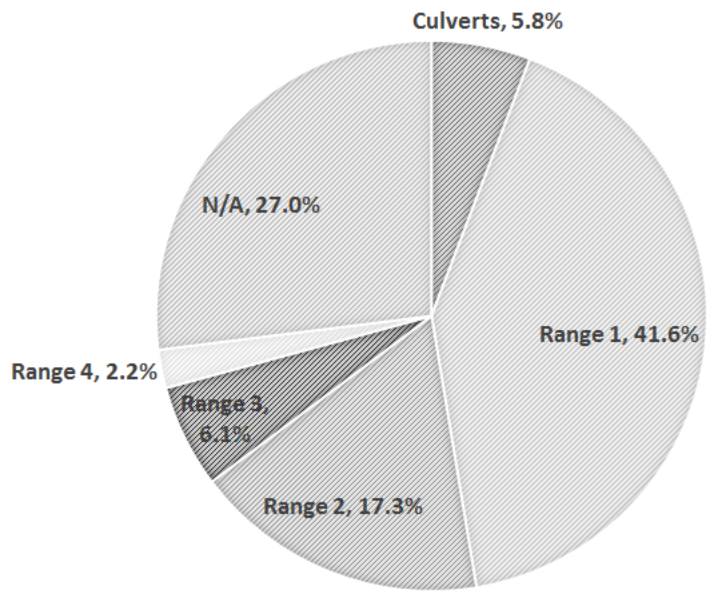
Fig. 3.1. INDOT database - Bridge structural type summary.

- Prestressed concrete AASHTO beams
- Prestressed concrete bulb tee and hybrid bulb tee
- Structural steel folded plate beams
- Structural steel beams
- SDCL steel beams
- Structural steel plate girders

3.1.1 Span Configuration and Span Ranges Selection

As shown in Figure 3.2, bridge spans between 30-ft and 130-ft represent 65% of the total Indiana bridge inventory. However, structures with spans shorter than 20-ft (5.8%) are considered "culverts" and are out of the scope of this research. In addition, bridges between 20-ft and 30-ft use predominantly slab and culvert superstructure types (82% of the time). Consequently, bridges between 30-ft and 130-ft were selected as the objective of this study.

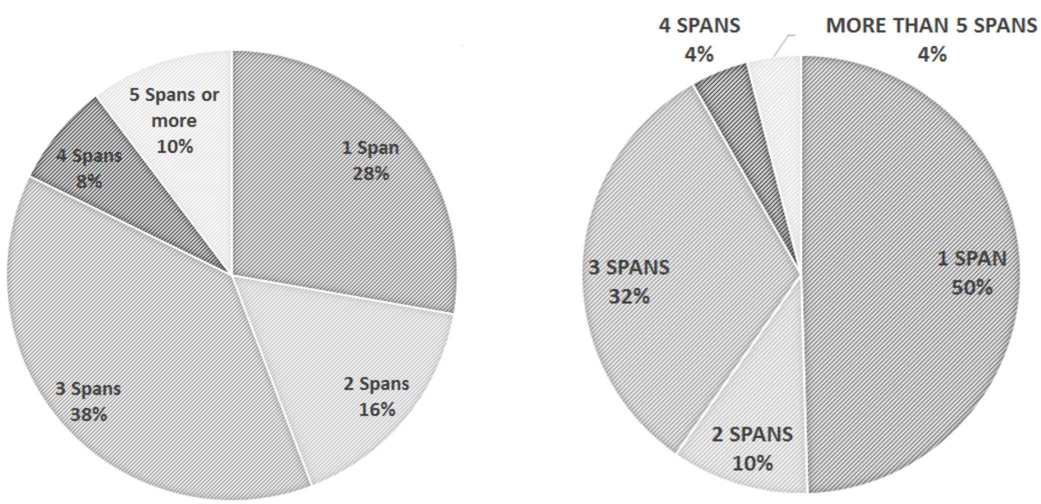
To categorize different design options, three different span ranges were established each with different ranges of maximum span lengths. Range 1 included bridges with spans within 30-ft and 60-ft, range 2 with span lengths between 60-ft and 90-ft, and range 3 spans with lengths in the range from 90-ft to 130-ft. Design types were selected depending on their cost-effectiveness potential for each of the span ranges.



Summary Ranges					
Ranges	ID	Span		%	Quantity
		Minimum	Maximum		
0	Culverts	LESS THAN 20 FT		5.8%	1113
1	Range 1	30	60	41.6%	7960
2	Range 2	60	90	17.3%	3315
3	Range 3	90	130	6.1%	1164
4	Range 4	130	200	2.2%	430
N/A	N/A	20	30	27.0%	5163
				100.00%	19145

Fig. 3.2. Span range summary based on NBI database.

Figure 3.3(a) shows the bridge span distribution within the state for bridges constructed in the last 6 years. It is clear that bridges with 4 or more spans are less common. Simple span (28%) and three-span arrangements (38%) are the most common structure found in Indiana. Nevertheless, the two-span configuration is also widely used (16%). Two spans are commonly used for longer bridges in highway crossroads. Moreover, Figure 3.3(b) shows that according to the NBI database, one and three span configurations comprised 82% of the concrete and steel bridges in Indiana. Conversely, by comparing span length and span ranges, it was found that one and three spans bridges are the most common configurations for span range 1 (94%) and span range 2 (65%), but for span range 3 the most commonly used option is the two-span arrangement (36%). Using this trend, the design plan utilized simple and three-span structures for span ranges 1 and 2, and simple and two-span structures for span range 3.



(a) Based on INDOT database.

(b) Based on NBI database.

Fig. 3.3. Span distribution summary.

Figure 3.4 shows the aspect ratio summary result of the INDOT database. As can be seen, the most common ratio between the longest span and the total span of the bridge are 0.50 and 0.30 for two and three span configurations, respectively. Therefore, two equal spans will be used for the two span configuration, while for three span configurations the design will use two external spans of 32% of the total length and a central span of 36% of the total span bridge length.

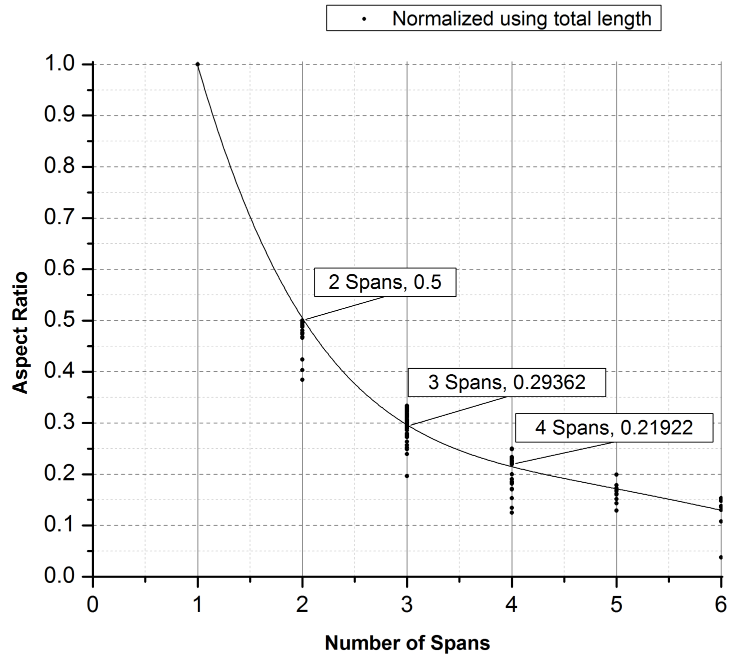


Fig. 3.4. Span aspect ratio summary based on INDOT database.

The final design plan includes bridge designs developed for extreme span ranges values and a single intermediate point along the range. Table 3 presents a summary of the designs developed for the simply supported configuration. As shown, different superstructure types are considered depending on its potential cost effectiveness for each span length. The same approach was used for the continuous span configuration design plan shown in Table 3.1. The span length shown in Table 3.1 corresponds to the maximum span length within the multiple spans and not the total length of the bridge.

3.1.2 Bridge Design

Bridge designs were then developed for the design plan. The seventh edition AASHTO LRFD specifications [21] and the Indiana Design Manual [22] were used for the designs. There are some simplifications and assumptions made that need to

Table 3.1.
Bridge Design Matrix

Superstructure Type	Span Length* (ft)		
	Simply Supported	Continuous	
		3 spans	2 Spans
(1)	(2)	(3)	(4)
Slab Bridge	30 , 45	30 , 45	-
Prestressed Concrete Box	30 , 45 , 60	30 , 45 , 60	-
Prestressed Concrete AASTHO Beam**	30 , 45 , 60 , 75 , 90	45 , 60 , 75 , 90	-
Structural Steel Beam (5 beams)***	30 , 45 , 60 , 75 , 90	45 , 60	-
Structural Steel Beam (4 beams)***	30 , 45 , 60 , 75 , 90	45 , 60	-
Folded Steel Plate	30 , 45 , 60	-	-
Structural Steel SDCL Beams***	-	60 , 75 , 90	-
Prestressed Concrete Bulb Tee Beams**	60 , 75 , 90 , 110 , 130	60 , 75 , 90	90 , 110 , 130
Structural Steel Plate Girders***	90 , 110 , 130	90	90 , 110 , 130

*For continuous arrangements corresponds to the maximum span length

**Design used for options with and without support diaphragms

***Design used for painted and galvanized options

be addressed. To simplify the design process some aspects are taken as constant for every option considered. These assumptions are as follows:

i . Two 12-ft lanes in opposite directions along with 8-ft shoulders on each side of the bridge. Total width of the bridge is 43-ft.

ii . Concrete bridge railing type FC was used per Indiana Design Manual and Standard Drawing No. E 706-BRSF-01.

iii . Skew: 0. INDOT database shows that most of the Indiana bridges have skew values less than 30 which in practical design terms will not significantly impact the final design.

iv . Moderate ADTT.

v . Concrete deck of 8-in, minimum longitudinal reinforcement of 5/8 and maximum rebar spacing of 8-in as the minimum required per the Indiana Design Manual.

vi . Structural steel ASTM A709 Grade 50. Modulus of Elasticity: 29,000ksi, Fy: 50ksi and Fu: 65ksi.

vii . Reinforcement steel AASHTO A615 Grade 60. Modulus of Elasticity: 29,000 ksi, Fy: 60ksi and Fu: 80ksi.

viii . Prestressing Strands: Low relaxation strands. Modulus of Elasticity: 28,500 ksi, Fy: 243ksi and Fu: 270ksi.

ix . Slab concrete fc: 4ksi, Modulus of Elasticity: 3,834ksi.

x . Concrete prestressed beams fc: 7ksi. Modulus of Elasticity: 5,072ksi. Conditions at transfer may vary.

The research described herein is focused on the superstructure only; the substructure was not designed for any of the bridges considered. Generalization of soil and foundation types throughout Indiana is not within the scope of this research.

Spread sheets that include applicable sections of the LRFD and the Indiana Design Manual specifications were created for every design option. As an input, live load envelopes were generated using a simple beam element model in SAP2000®. The models were also used to check deflection limits. Limit states checked are: service level, strength level, and fatigue and fracture. Different design examples were considered as a basis for the designs. Examples include those from Wassef [23], Florida Department of Transportation [24], Hartle et al. [25], Parsons Brikinckerhoff [26], Chavel and Carnahan [27], Grubb and Schmidt [28] and Wisconsin Departement of Transportation [29] were used.

As noted above, detailed bridge designs were developed for each of the options considered in the design plan. This involved the design of 64 bridges in total. Summary information from the designs can be found in the design drawings in Appendix A. Due to the length of each design, the detailed spread sheet designs for each bridge are available by request and not annexed to this document.

4. COST ALLOCATION

As noted earlier, the cost allocation model used herein is described in equation 2.1. Then, the final life-cycle cost for each alternative would be the sum of the agency costs, which includes design costs (DC), construction costs (CC), maintenance costs (MC), rehabilitation costs (RC) salvage costs (SC), and user costs (UC). Unless there is a reason to do otherwise, agency costs are typically assumed to be incurred at the end of the period in which expenditures actually will occur [1].

The most widely used basis to estimate those costs are the utilization of unit costs and bills of quantities. In the absence of this information, parametric cost estimating models may be used as a best-guess estimate [1]. This study is focused on the highway bridge system costs in Indiana. The Indiana Department of Transportation (INDOT) has been collecting information on highway construction projects since 2011. This information has been organized and compiled in a single database that includes not only the total cost of different projects but also discretizes pay items involved. Using this information, it is possible to identify the cost trend of basic pay items such as concrete, structural steel, structural elements among others.

In order to obtain the current price for each one of the values from the database, inflation rates need to be used. Inflation rates were calculated using the current consumer price index (CPI) published monthly by the Bureau of Labor Statistics (BLS). Values presented in Table 4.1 correspond to the average value throughout the year. Table 4.1 also presents the cumulative multiplier factor used to compute the net present value.

Table 4.1.
Inflation rates

YEAR	INFLATION RATE	
OF OCCURRENCE	Yearly Rate (%)	NPV Factor
2017	2.10	1.0210
2016	1.30	1.0343
2015	0.12	1.0355
2014	1.62	1.0523
2013	1.47	1.0678
2012	2.07	1.0899
2011	3.16	1.1243
2010	1.60	1.1423
2009	-0.40	1.1377
2008	3.80	1.1810
2007	2.80	1.2140
2006	3.20	1.2529
2005	3.40	1.2955
2004	2.70	1.3304
2003	2.30	1.3610
2002	1.60	1.3828
2001	2.80	1.4215
2000	3.40	1.4699
1999	2.20	1.5022

4.1 Outliers Identification

The definition of an outlier is at best a subjective idea. However, different investigators have been addressing this problem from different perspectives. One of the most accepted definitions of this term is presented by D'Agostino and Stephens [30]: "a discordant observation is one that appears surprising or discrepant to the investigator; a contaminant is one that does not come from the target population; an outlier is either a contaminant or a discordant observation." Once the outliers are identified there are different paths to treat the database shown as follows:

i . Omit the outliers and treat the reduced sample as a new database

ii . Omit the outliers and treat the reduced sample as a censored sample

iii . Replace the outliers with the value of the nearest good observation (Also called Winsorize the outliers)

iv . Take new observations to replace the outliers and,

v . Do two different analyses with and without outliers. If results are clearly different the conclusions need to be examined cautiously

Due to the source of the database used in this research the outliers will be identified and the reduced sample treated as a new database. There are multiple techniques to identify outliers in a sample which includes: Pierces criterion, modified Thompson Tau test, anomaly detention among others. Nevertheless, the method used for this sample was the implementation of the interquartile range (IQR) and the Tukeys fence approximation. The IQR is the difference between the first and the third quartile. The first Q_1 and third quartile Q_3 are the values in the database that holds 25% and 75% of the values below it respectively. According to the Tukeys fences method, outliers are values outside of the limits represented by 1.5 times the IQR below Q_1 and above Q_3. The generalization of the method is presented in Equations 4.1 to 4.3.

$$IQR = Q_3 - Q_1 \tag{4.1}$$

$$Lim_{Bot} = Q_1 - 1.5(IQR) \tag{4.2}$$

$$Lim_{Top} = Q_3 + 1.5(IQR) \tag{4.3}$$

Once the database is cleaned from outliers, a standard deviation and mean is computed for all the pay items involved. However, and in order to take into account the economics of size of the projects, a weighted average and standard deviation are chosen to use as an input in the BLCCA. The usage of a weighted average is based on the fact that larger projects would have a more significant impact on the computation of the mean than smaller projects, which could result in costlier unit prices. Weights are calculated based on the quantities for each one of the activities considered. Basic definition of weighted average (μ_w) and standard deviation (σ_w) is presented in Equations 4.4 and 4.5 where x_i represents a single value in the database and w_i is the weight associated to that specific value. Weights, as mentioned before, correspond to the ratio between the individual quantity of the data point and the total sum of quantities.

$$\mu_w = \frac{\sum_{i=1}^{n} w_i x_i}{\sum_{i=1}^{n} w_i} \tag{4.4}$$

$$\sigma_w = \sqrt{\frac{\sum_{i=1}^{n} w_i (x_i - \mu_w)^2}{\sum_{i=1}^{n} w_i}} \tag{4.5}$$

4.2 Design Costs (DC)

Includes all the engineering and regulatory studies, environmental and other reviews, and consultant contracts prior to the construction or major rehabilitation of an asset. It is a common practice to compute these values as a percentage of the construction cost when no data are available. However, these costs are not considered in the computation of the total LCCA for two reasons: Firstly, designs are made by the researchers and no cost is involved or considered due to such activities, however, in real projects this cost must be included. Secondly, since this research is not localizing the design structure in any specific location, environmental and other reviews along with consultant contracts are not needed.

4.3 Construction Costs (CC)

Includes all the activities made between the design and the operation of the asset. In a project, it may include bridge elements, ancillary facilities, and approach roads among others. In this study only major superstructure elements are considered. Substructure construction is neglected since this design is outside of the scope of the project. Barriers and other miscellaneous items are neglected also due to that all the alternatives share the same specifications, in other words, they will have the same elements in the same quantities. Pay items considered include: slab concrete, structural concrete elements, reinforcing steel and structural steel. Table 4.2 shows the summary of the construction cost for different superstructure elements. All pay items shown include all the activities needed until the elements are cast or erection of the element on site. No additional costs need to be considered due to erection of superstructure beams or provisional formwork for cast in place elements, since these costs are included in the pay item price.

Table 4.2.
Summary agency costs - construction costs

ITEM	**UNIT**	**UNIT PRICE ($/Unit)**					**Data**
		Min.	**Max.**	μ	μ_m	σ_m	
Concrete Type C superstructure	yd^3	354.25	898.76	589.04	564.03	109.61	354
Prestressed concrete bulb-T beam	LFT	188.86	419.06	294.98	298.99	54.86	132
Prestressed concrete box beam	SFT	139.03	320.99	241.37	241.51	54.86	13
Prestressed concrete AASHTO beam	LFT	107.53	346.43	221.07	219.21	66.93	55
Structural steel	lbs	0.64	3.00	1.94	1.72	0.44	63
Reinforcing steel	lbs	0.65	1.34	0.96	0.92	0.12	150
Epoxy reinforcing steel	lbs	0.74	1.40	1.05	1.02	0.13	324

A further analysis was done for the pay item related to the concrete of the superstructure. As a common practice it is assumed that concrete cost depends on the superstructure type used. As a general standardized exercise this cost is discretized depending on the superstructure material type. In other words, concrete superstructures are believed to have different concrete prices than steel superstructures. It is true that in past years the tendency was that steel superstructures resulted in costlier cast in place concrete slabs than the concrete superstructures as shown in Figure 9. Nonetheless, analyzing the historical data, the differences in prices between those two pay items has been reduced in the recent years. Therefore, concrete for superstructures pay item was taken as the same value independent of the material or superstructure type.

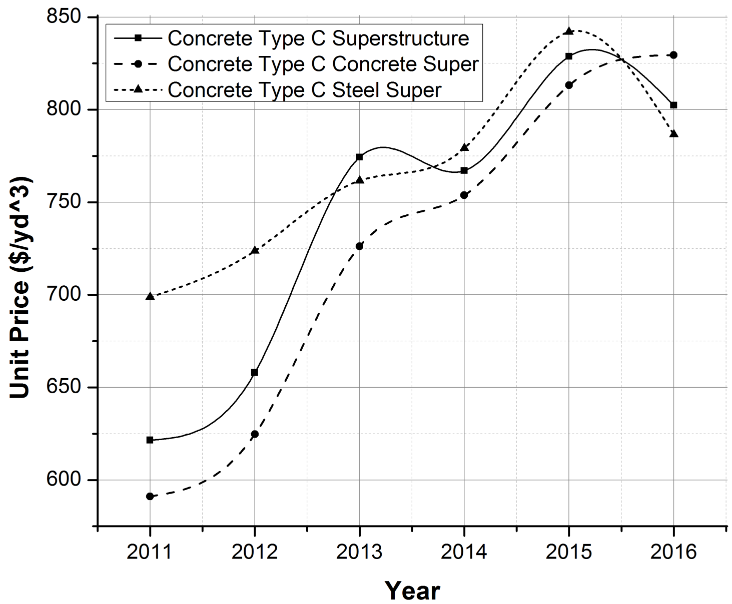


Fig. 4.1. Historical cost data Superstructure Concrete Pay Item.

In addition, the unit cost for concrete diaphragms and continuity concrete details for continuous spans needed to be determined. Since there is no discretization of any pay item in the database, it is not possible to determine this cost from historical data directly. However, a different approach was used that involved the average values for superstructure concrete and typical quantities of a continuous bridge.

Computation of the diaphragm cost is presented in Equations 4.6 and 4.7. The approximation proposed uses a weighted computation of the price since the value of the concrete is known for continuous spans (in this case 3 spans configuration: P_{Total}) and simply supported span (assumed as basically slab concrete: P_{Slab}), and also the relative percentage of concrete used for the slab (α_{Slab}) and the diaphragms (α_{Diaph}) of a typical bridge. To obtain the cost of the material used for continuity above the piers the procedure is as follows (P_{Diaph}):

$$P_{Total} = \frac{\sum_{i=1}^{n} w_i x_i}{\sum_{i=1}^{n} w_i} = \alpha_{Slab} P_{Slab} + \alpha_{Diaph} P_{Diaph} \tag{4.6}$$

$$\alpha_{Slab} = \frac{V_{Conc_{Slab}}}{V_{Conc_{Total}}} = 88\% \quad \text{and} \quad \alpha_{Diaph} = \frac{V_{Conc_{Diaph}}}{V_{Conc_{Total}}} = 1 - \alpha_{Slab} = 12\% \tag{4.7}$$

$$P_{Total} = \$600.59 \ / \ yd^3 \quad \text{and} \quad P_{Slab} = \$579.27 \ / \ yd^3$$

$$P_{Total} = \$600.59 \ / \ yd^3 = 88\%(\$579.27 \ / \ yd^3) + 12\% P_{Diaph}$$

then solving for P_{Diaph}:

$$P_{Diaph} = \$1123.60 \ / \ yd^3$$

As it can be seen in Table 4.2, unit cost for concrete superstructure elements like beams is given in dollars per linear foot independent of the beam type. This feature implies that the lack of data points of certain beam types (different bulb tees sections for instance) make the unit price for that specific section not accurate. To solve this problem this unit price can be converted to dollars per volume using the total area of the beam type. This additional step resulted in a general unit price for all beam types that can be converted into unique unit values for all different sections using again the net area. The same procedure was done for structural prestressed concrete box beams using the superficial area of all sections. A summary of unit cost for different prestressed concrete beam sections can be seen in Table 4.3.

Table 4.3.
Summary agency costs - Prestressed concrete elements costs

BEAM TYPE	**Area** (in^2)	**UNIT PRICE** ($/LFT)*	**BEAM TYPE**	**Area** (in^2)	**UNIT PRICE** ($/LFT)*
CB12x36	423	186.25	BT78x60	1102	323.23
CB17x36	471	207.38	BT84x48	1100	32.64
CB21x36	515	226.76	BT84x60	1144	335.55
CB27x36	581	255.82	HBT36x49	878.3	257.59
CB33x36	647	284.88	HBT36x61	932.4	273.48
CB42x36	746	328.47	HBT42x49	926.3	271.70
CB12x36	567	249.65	HBT42x61	980.4	287.56
CB17x36	603	265.50	HBT48x49	974.3	285.77
CB21x36	647	284.88	HBT48x61	1028.4	301.64
CB27x36	713	313.94	HBT54x49	1022.3	299.85
CB33x36	779	343.00	HBT54x61	1076.4	315.72
CB42x36	878	386.59	HBT60x49	1070.3	313.93
BT54x48	883	259.00	HBT60x61	1124.4	329.80
BT54x60	934	273.37	HBT66x49	1118.3	328.01
BT60x48	932	286.27	HBT66x61	1172.4	343.88
BT60x60	976	186.25	HBT72x49	1166.3	342.09
BT66x48	974	285.69	HBT72x61	1220.4	357.96
BT66x60	1018	298.59	A Type I	276	121.52
BT72x48	1016	298.01	A Type II	369	162.47
BT72x60	1060	310.91	A Type III	560	246.57
BT78x48	1058	310.32	A Type IV	789	347.40

CB: Prestressed concrete box. ($0.0367 / in^3)

BT: Prestressed concrete bulb tee. ($0.0244 / in^3)

HBT: Prestressed concrete hybrid bulb tee. ($0.0244 / in^3)

A: Prestressed concrete AASHTO beam. ($0.0367 / in^3)

*LFT: Linear foot

4.4 Maintenance Costs and Rehabilitation Costs (MC and RC)

Includes all the activities needed during the service life of the asset in order to maintain the current condition or improve it above acceptable criteria. These activities also cover all actions to repair or replace elements that threaten safe bridge operation. There are two types of maintenance activities: (a) a regularly scheduled operation such as deck flushing or deck cleaning, and (b) preventive or protective maintenance which are the response of an observed condition. Overlays, painting, patching among others generally are considered as part of the second type. As a general rule of thumb, the better approach to determine these costs and their service lives is by using agency experience in conjunction with historical cost data.

Rehabilitation costs may include full replacement of bridge elements or even the whole superstructure. Additionally, activities such as bridge widening or collision damage repairs are considered rehabilitations for most public agencies. This research is not considering any future contingencies such as change in specifications that involves widening, possible collisions during the service life of the asset, or repairs due to hazards.

Depending on the superstructure type, different activities could be considered. Concrete superstructures may require crack sealing or patching due to wearing. According to INDOT experience, prestressed superstructures tend to develop more beam end atypical deterioration when construction joints are used. On the other hand, steel superstructures could have fatigue cracking or excessive section loss due to corrosion. Actions needed to address such problems are considered as rehabilitation costs. However, these working actions are only triggered due to the operation of the asset and its prediction on new bridges is a complex task that needs historical data along with statistical and probabilistic methodologies. These problems could be avoided to some extent during the design process by using jointless bridges and adequate fatigue detailing. This research is based on this premises, which is the reason why those types of repairs and retrofitting activities are not considered in any of the cases analyzed. Determination of those costs then are not needed.

As described in more detail later in this document, working actions considered for the superstructure often involves deck maintenance and rehabilitation. These costs are obtained from the historical database mentioned earlier in this document. Table 4.4 presents the cost values used for different maintenance and rehabilitation activities done in Indiana.

As shown in the table, activities such as overlays and deck reconstruction involved more pay items that need to be considered in order to obtain the final cost. For instance, overlays as a maintenance activity also involves the removal of the wearing surface, a demolition activity alongside with the overlay material needed, in this case latex modified concrete as explained in chapter 6. Deck reconstruction on the other hand, involves the removal and replacement of the concrete deck structure. Those additional activities are summarized in Table 4.4.

4.5 User Costs(UC)

These costs are attributable to the functional deficiency of a bridge such as a load posting, clearance restriction, and closure [1]. Then, a proper way to address its estimation is to compute the cost of vehicle operation (VOC) and travel time (TTC) incurred due to detouring or traveling through narrow bridges or assets with poor deck surface conditions. According to Sinha et al. [20] Indiana resumed user costs due to three different deficiencies: load capacity limitation, vertical clearance limitation, and narrow bridge width. However, as related to the limitation, the final cost will be the sum of VOC and TTC. It is true that, as mentioned before, no contingencies other than regular deterioration of the bridge are considered, however, maintenance or rehabilitation activities may affect user costs mainly due to narrow lane traffic on and under the bridge. Nonetheless, and in order to compute those costs, a deep understanding of the traffic (quantities and type of vehicles), detour lengths,

Table 4.4.
Summary agency costs - Maintenance and rehabilitation costs

ITEM	UNIT	UNIT PRICE ($/Unit)					Data
		Min.	**Max.**	μ	μ_m	σ_m	
Overlay	*SFT*	29.27	56.29	40.65	39.65	5.92	-
Overlay	*SFT*	6.04	16.05	10.57	9.95	2.28	226
Overlay Remove	*SFT*	0.18	1.90	1.03	0.94	0.40	121
Hydrodemolition	*SFT*	1.98	15.57	7.13	6.83	2.72	212
Overlay Additional	*SFT*	21.07	22.76	21.92	21.92	0.51	263
Deck Patching - Partial Depth	*SFT*	5.35	133.74	53.41	37.97	56.77	276
Deck Patching - Full Depth	*SFT*	1.08	119.09	47.68	37.23	29.23	328
Bearing Elastomeric Assembly	*Unt*	214.00	2,275.40	966.72	930.16	658.17	31
Deck Reconstruction	*SFT*	25.37	88.55	48.67	47.41	15.25	-
Deck Reconstruction	*SFT*	14.06	39.63	25.72	25.01	5.97	65
Present Structure Remove	*SFT*	11.31	48.92	22.95	22.40	9.29	63
Painting	*SFT*	1.39	5.22	2.46	2.27	0.91	22
Deck Cleaning	*SFT*	[31], [32]			2.17	Data 1999	
Sealing of cracks	*SFT*	[31]			1.27	Data 2013	
Cleaning and Washing of Bearings	*Unt*	[33]			222.28	Data 2013	
Jacking Superstructure Elements	*Point*	[31], INDOT Personnel			2,552.50	Data 2013	
Spot Painting	*SFT*	[34]			2.19	Data 1999	
Bridge Removal	*SFT*	[33]			11.11	Data 2013	
Recycle Structural Steel	*lbs*	Current market price			0.08	Data 2018	

travel times and travel velocities is needed. As specified earlier in this document, all bridge designs have no specific location along any specific road. In other words, traffic, velocity and detour assumptions are not taken into account. Additionally, such assumptions are considered an oversimplification of the problem and could impact negatively the outcome of the LCC comparison. More information about user costs models can be found in Hawk [1] and Sinha et al. [20].

4.6 Probability Distribution Selection

In order to properly perform a Monte Carlo simulation, probability distributions for different pay items are needed. Basic concepts of probability needed to understand this process along with definitions, probability density functions and cumulative density functions of every distribution used herein are presented in Appendix B.

The usage of historical data as a mean to capture the inherent stochastic nature of different phenomena is a common practice. This can be accomplished by obtaining the probability distribution of such events. It is true that for mathematical simplicity simpler distributions like the *normal distribution* and the *lognormal distribution*, are preferable. Also, due to lack of available data other distributions like the *triangular distribution* and the *PERT distribution* could be useful, simplifying greatly the process [35].

However, when the shape of the distribution is important, especially when historical sample data are available, consideration of multiple distribution to fit the data is desirable. When two or more distributions appear to be plausible, certain statistical test can be used to discriminate the relative validity of those. These tests are known as *goodness-of-fit* tests for distributions. There are multiple techniques that can be used including: the *Akaike information criterion (AIC)*, the *Bayesian information criterion (BIC)*, the *Chi-squared test* (χ^2), the *Kolmogorov-Smirnov test (K-S)* and the *Anderson-Darling test (A-D)*. Among them, the first two are the more mathematically complex, while the last two are based on the comparison of the empirical cumulative density function (ECDF) of the sample and the cumulative density function (CDF) of the assumed distribution. The A-D test is particularly useful when the tails of a distribution are important [36]. The last three tests mentioned, as noted by Ang and Tang [36]: "should be used only to help verify the validity of a theoretical model that has been selected on the basis of other prior considerations, such as through the application of appropriate probability papers, or even visual inspection of an appropriate probability density function (PDF) with the available histogram". In light of this statement, the method used herein includes not only the use of the K-S and A-D test for PDF selection but also an adequate visual inspection.

4.6.1 Kormogorov-Smirnov (K-S) test for goodness of fit

The general purpose of the test is to compare the ECDF with the CDF of the tested theoretical distribution. The test acceptance is given by contrasting of the maximum discrepancy between the ECDF and the CDF (D_n, given by the equation 4.8 and represented in Figure 4.2) and the critical limit value (D_n^α) depending on the desired significance level α. The distribution is considered as acceptable if D_n is less than D_n^α.

$$D_n = \max_x \mid F(x) - F_n(x) \mid \tag{4.8}$$

The critical value D_n^α is defined for a particular significance level α by:

$$P(D_n \leq D_n^\alpha) = 1 - \alpha \tag{4.9}$$

Values for different significance levels can be consulted in different texts [36], however for a 5% significance level ($\alpha = 0.05$) and sets of more than 50 data points, the critical value can be computed as:

$$D_n^\alpha = \frac{1.36}{\sqrt{n}} \tag{4.10}$$

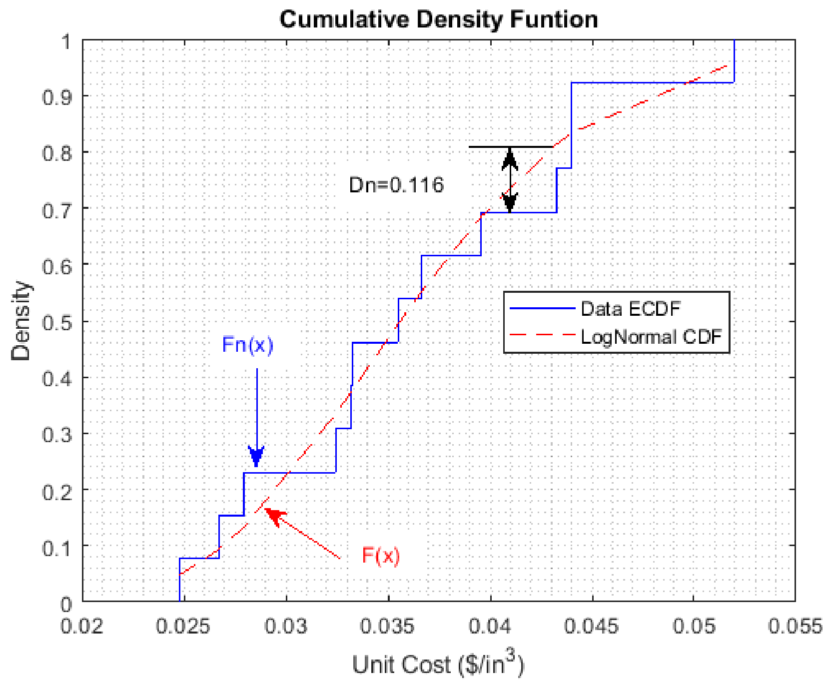


Fig. 4.2. ECDF ($F_n(x)$) vs CDF ($F(x)$), K-S test parameter D_n for Box Beams unit price

4.6.2 Anderson-Darling (A-D) test for goodness of fit

This test was first proposed by Anderson and Darling in 1954 [37] and is focused in placing more weight on the tails of the distribution. The calculation of the A-D statistic is as follows:

$$A^2 = -n - \frac{1}{n}\sum_{j=1}^{n}(2j-1)[\log u_j + \log(1-u_{n-j+1})] \tag{4.11}$$

then, an adjusted test statistic (A^*) and the critical value (c_α) corresponding to the desired significance level (in this case $\alpha = 0.05$) is computed. Values for such parameters depends on the type of distribution that is tested, for which tabulated values can be consulted [36]. Most software includes the estimation of this value within their routines. For this research Matlab®is used for both the K-S and the A-D tests.

4.6.3 PDF selection example

As described before, a proper PDF selections is based not only on the test statistic but also on adequate visual inspection. The pay item chosen for a general explanation of the process followed is the bridge concrete, specified as "Concrete Type C". There are 353 values found in the historic contractor database mentioned earlier in this document. The general process is the following:

i . The number of bins used for the histogram based on the Doane's formula, which is a variation of the Sturge's formula that attempsts to improve the performance with non-normal data, is computed as follows:

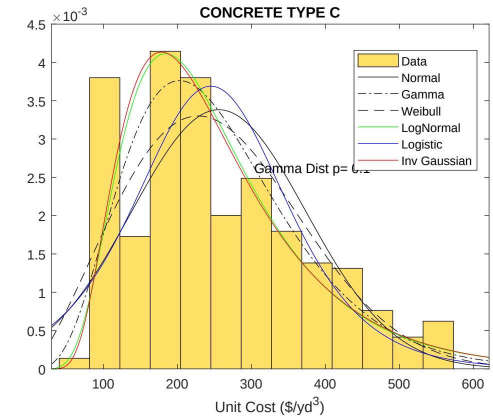
$$k = 1 + \log_2 n + \log_2 \left(1 + \frac{\mid g_1 \mid}{\sigma_{g_1}}\right) \tag{4.12}$$

$$\sigma_{g_1} = \sqrt{\frac{6(n-2)}{(n+1)(n+3)}} \tag{4.13}$$

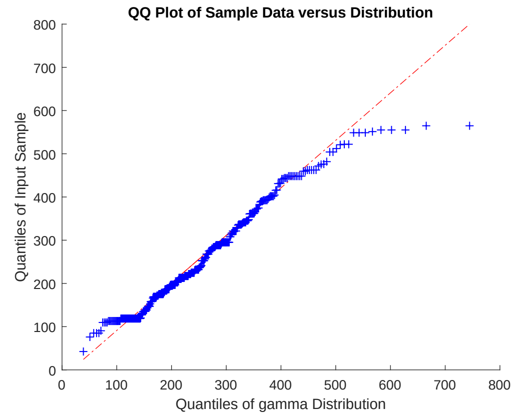
where k is the number of bins needed ($k = 13$), n is the number of values ($n = 353$) and g_1 is the skewness of the data ($g_1 = 0.71$).

ii . The histogram representing the data is plotted alongside with all the PDF's that are tested (see Figure 4.3(a)). The selection of the distributions to be tested is based solely on a visual inspection (i.e. based on the histogram shape, the distributions are selected among the wide variety available). In this case, and for all the pay items analyzed, the selected distributions are: Normal distribution, Lognormal distribution, Gamma distribution, Logistic distribution, Weibull distribution and Inverse Gaussian distribution. Appendix B presents the PDF's and the CDF's of all the distributions used in this document.

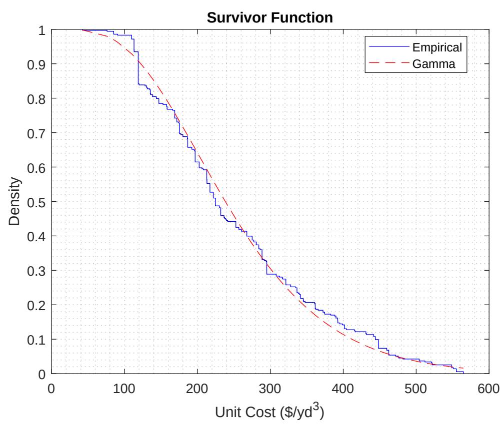
iii . K-S and A-D tests are used in order to rank the *goodness of fit* of each distribution. If more than one distribution is considered as a good fit for the historical data, the lowest value of the D_n statistic and the A^* statistic is selected. This process is presented in Table 4.5. The values of *Logical K-S* and *Logical A-D*



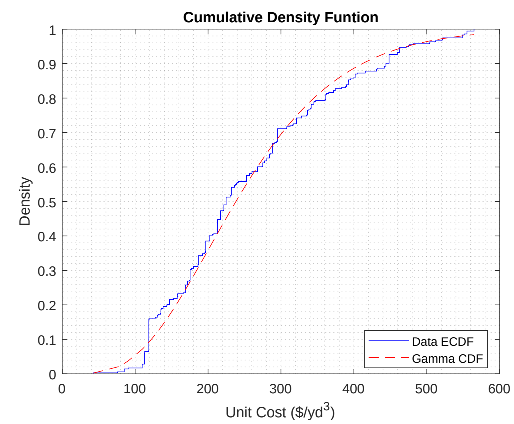
(a) Histogram and PDF's tested



(b) Q-Q plot - Selected PDF



(c) Survival Function - Selected PDF



(d) CDF - Selected PDF

Fig. 4.3. Graphical PDF Selection results - Pay item: Concrete Type C

represents the hypothesis test results as a logical value, where 0 indicates the failure to reject the null hypothesis (i.e. the data can be represented with the tested distribution), and 1 indicates the rejection of the null hypothesis. The table also presents the p-value and the limiting values for both the K-S and A-D tests, D_n^α and c_α respectively.

iv . In order to properly select the adequate distribution, both test are considered. Then, a weight is given to each distribution depending on its ranked position for each, the K-S and the A-D tests. Numeric values from 1 to 6 are given in

Table 4.5.
PDF Selection - Pay item: Concrete type C

Distribution	**K-S TEST**					**A-D TEST**					**Ordered**
	Log.	**ρ-value**	D_n	D_n^α	**Wg**	**Log.**	**ρ-value**	A^*	c_α	**Wg**	**Weight**
Gamma	0	0.079	0.067	0.071	6	0	0.060	2.335	2.492	4	0.100
Inverse Gauss	1	0.029	0.076	0.071	0	0	0.082	2.081	2.492	6	0.166
LogNormal	1	0.027	0.077	0.071	0	0	0.076	2.141	2.492	5	0.200
Weibull	1	0.004	0.092	0.071	0	1	0.014	3.572	2.492	0	-
Logistic	1	0.001	0.102	0.071	0	1	0.001	5.526	2.492	0	-
Normal	1	0.000	0.120	0.071	0	1	0.000	6.488	2.492	0	-

descending order from the best to the worst option. In case that the logical hypothesis result rejects the null hypothesis, a weight value of 0 is assigned. Then, an ordered weight is computed as the inverse of the sum of both weights. The distribution with the lowest ordered weight is then selected as the distribution that best fits the pay item historical data.

This procedure was used to select the PDF's for pay items that have multiple historical data records as specified in Table 4.4. However, pay items without historical records (e.g. deck surface washing) are not suitable to follow this process. Instead, those pay items will be modeled as a PERT distribution with most likely value (ML) equal to the unit price given in Table 4.4, a maximum value (max) of 1.25 times the ML, and a minimum value (min) equivalent to 0.75% of the ML. Table 4.6 shows the PDF's selected for each pay item with their respective distribution parameters.

Table 4.6.: Probability distribution functions for different pay items.

Item	Unit	Data	Min	Max	Distr.	Dist. Parameters	
						Parameter	Value
Concrete Type C	$\$/yd^3$	354	354.25	898.76	Gamma	Shape (κ)	4.81
						Scale (θ)	53.13
						Shift (ω)	334.0
P/S Concrete Bulb Tee	$\$/ft^3$	132	26.62	67.60	Logistic	Location (μ)	43.38
						Scale (σ)	4.23
P/S Concrete Box	$\$/in^3$	13	0.0248	0.0519	Lognormal	Mean (λ)	-3.33
						STD (ζ)	0.22
P/S Concrete Beam	$\$/ft^3$	55	40.22	125.26	Lognormal	Mean (λ)	4.18
						STD (ζ)	0.24
Structural Steel	$\$/lb$	63	0.64	3.00	Normal	Mean (μ)	1.94
						STD (σ)	0.56
Reinforcing Steel	$\$/lb$	150	0.65	1.34	Gamma	Shape (κ)	34.73
						Scale (θ)	0.03
Overlay							
Overlay	$\$/yd^2$	226	54.33	144.50	Gamma	Shape (κ)	21.17
						Scale (θ)	4.49
Overlay Removal	$\$/yd^2$	121	1.63	17.12	Weibull	Shape (κ)	2.83
						Scale (λ)	10.45
Hydrodemolition	$\$/yd^2$	212	17.84	140.16	Weibull	Shape (κ)	2.84
						Scale (λ)	71.32
Overlay Additional	$\$/ft^2$	263	16.44	27.40	PERT	Mean (ML)	21.92
Deck Patching	$\$/ft^2$	328	1.03	118.09	Gamma	Shape (κ)	2.45
						Scale (θ)	19.47

continued on next page

Table 4.6.: *continued*

Item	**Unit**	**Data**	**Min**	**Max**	**Distr.**	**Dist. Parameters**	
						Parameter	**Value**
Bearing Assembly	$\$/unt$	31	214.00	2,275	Lognormal	Mean (λ)	6.65
						STD (ζ)	0.72
Painting	$\$/ft^2$	22	1.39	5.22	Lognormal	Mean (λ)	0.83
						STD (ζ)	0.41
Deck Cleaning	$\$/ft^2$	-	1.63	2.71	PERT	Mean (ML)	2.17
Sealing of cracks	$\$/ft^2$	-	0.95	1.59	PERT	Mean (ML)	1.27
Jacking	$\$/unt$	-	1,914	3,190	PERT	Mean (ML)	2,553
Bridge Removal	$\$/ft^2$	-	8.33	13.89	PERT	Mean (ML)	11.11
Deck Reconstruction							
Deck Reconstruction	$\$/yd^3$	65	379.59	1,070	Inv. Gauss	Mean (μ)	695
						Shape (λ)	12,683
Structure Removal	$\$/ft^2$	63	1.39	5.22	Lognormal	Mean (λ)	3.05
						STD (ζ)	0.38
Spot Painting	$\$/ft^2$	-	1.64	2.74	PERT	Mean (ML)	2.19
Recycle Struc. Steel	$\$/lb$	-	0.06	0.10	PERT	Mean (ML)	0.08
Galvanizing	$\$/ft^2$	-	0.22	0.36	PERT	Mean (ML)	0.29
Discount rate	%	-	2.00	6.00	PERT	Mean (ML)	4.00

Finally, some PDF's have heavier tails than others, which means that when a simulation uses a random number generator those values could be easily chosen for a number of iterations. However, according to the historical data, extreme values are unlikely to happen. Consequently, for simulation purposes only, a range between 0.75 times the minimum value and 1.25 times the maximum value was used. Values outside these boundaries were taken as the 0.75 times the minimum and 1.25 times the maximum values, respectively.

5. DETERIORATION MODELS FOR INDIANA BRIDGES

Deterioration curves are critical for development of the BLCCA. Their accurate determination will lead to more precise answers and better recommendations to designers. The use of the NBI database to develop deterioration curves is the most commonly utilized practice. Since this study is focused only on the Indiana bridge system administrated by INDOT, deterioration curves will consider the Indiana NBI database. Accordingly, deterioration curves made for the Indiana state highway system by Moomen et al. [17], Sinha et al. [20] and Cha et al. [38] will be used.

In addition to the deterioration path for each material type, a limiting condition rating needs to be chosen in order to establish the lowest allowed bound of deterioration. This lower bound could vary depending on the budget allocation and availability. According to INDOT experience, the threshold for the state of Indiana is 4. Additionally, analyzing the historical NBI database it is clear that a condition rating of 4 is considered as the lowest deterioration limit before a major rehab or repair action is scheduled. Consequently, for this research a condition rating of 4 is established as the threshold before a major work action is needed.

Nonetheless, it is important to mention that there are some drawbacks related to the use of the NBI condition ratings. First, these are based on visual inspections which involves a variability between consecutive inspections. Since these inspections are inherently subjective, the usage of those needs to be done carefully. Second, and as it can be seen in Figure 5.1 there are condition ratings that are physically unlikely for the bridge age that is provided. For instance, there are records that indicates that a bridge could be a condition 9 (excellent condition) even for bridge ages older than 30 years, which is not likely. These abnormal records could be explained not only with the variability due to the visual inspection but also with the lack of historical records of maintenance, rehabilitation or reconstruction procedures for existing bridges that can be found in the database. As a result, the deterioration curves obtained from those records need to be treated carefully and with engineering judgment.

Deterioration rates vary depending on the database and method used to compute it. Nonetheless, it is clear that deterioration rate is time dependent. Focusing on steel structures only as shown in Figure 5.1, Moomen et al. [17] predicted that a steel bridge deteriorates to a replacement state in less than 50 years. In contrast, the constant deterioration rate used by Cha et al. [38] projected an age close to 90 years, while the deterioration curve used by Sinha et al. [20] for the Indiana Bridge Management System (IBMS) stated that this life value is in the vicinity of 80 years for the same threshold rating. Steel superstructure deterioration curves used in the IBMS appear to fit better the historical data.

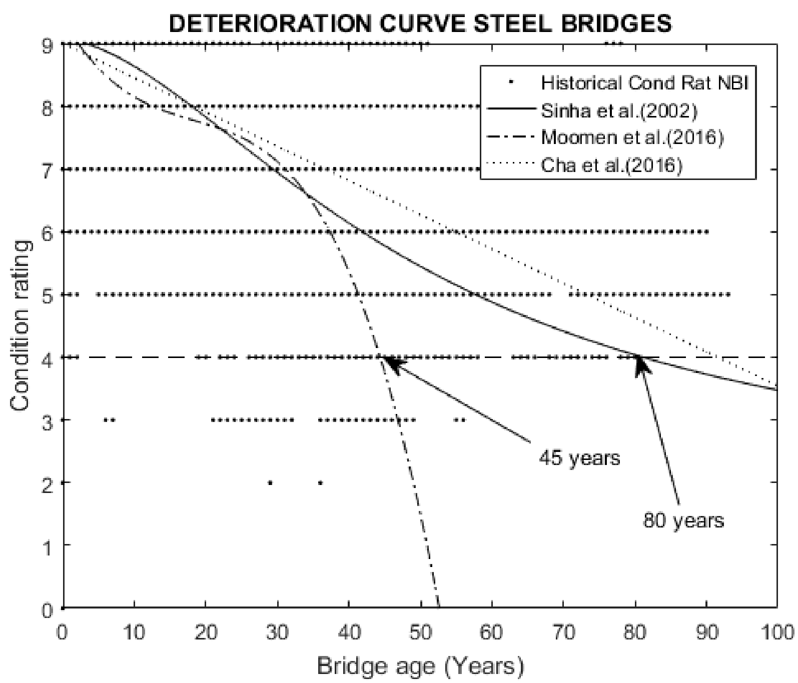


Fig. 5.1. Deterioration curves example for steel bridges.

Table 5.1.
Deterioration curves for cast in place concrete deck [17]

Structure	Location	Desc	Equation
Cast in Place Concrete Deck	Northen	NHS	$SUPCR = 9.5820 - 0.27195Age + 0.00874Age^2$ $-0.0000933Age^3 - 0.1991Int - 0.17981ServUnder$ $-0.71169FrzIndx$
		Non-NHS	$SUPCR = 8.85183 - 0.22032Age + 0.00598Age^2$ $-0.00005627Age^3 - 0.111229ADTT$
	Central	NHS	$SUPCR = EXP(2.10113 - 0.01135Age$ $-0.01968Int - 0.01845SpanNo)$
		Non-NHS	$SUPCR = EXP(2.13095 - 0.01255Age$ $-0.00027854Skew - 0.01169SpanNo$ $-0.0933ADTT)$
	Southern	NHS	$SUPCR = 8.1804 - 0.02287Age - 0.00058022Age^2$ $-0.06369SpanNo - 0.00942Length$ $-0.74059FrzIndx - 0.29919ADTT$
		Non-NHS	$SUPCR = 9.00 - 0.09891Age - 0.00108Age^2$ $-0.00000876Age^3 - 0.00458Skew - 0.11453SpanNo$ $-1,01643FrzIndx - 0.21873ADTT$

On the other hand, deck behavior appears to agree closely with the curve fitting approach (Table 5.1). Figure 5.2 shows the deterioration behavior of decks using curve fitting [17]. Additionally, the constant deterioration rate model and the IBMS deterioration curves both propose different deterioration paths depending on the superstructure material type. In contrast, curves used by Moomen et al. [17] indicates that superstructure material type is not a factor that affects the deterioration behavior. As shown in the figure, the service life proposed by this approach (service life when a condition rating of 4 is achieved) is close to 37 years. The likelihood of programing a deck replacement at a much greater service life is low according to actual data and INDOT experience, and it is often scheduled between 30 and 40 years. This means the deterministic method can be used reliably.

Deterioration curves for various concrete superstructures are presented in Figures 5.3 to 5.5. As explained in analyses for decks and steel structures, three different approaches are considered: Moomen et al. [17], Sinha et al. [20] and Cha et al. [38]. Moomen et al. [17] present different deterioration curves, each depending on the superstructure structural type. However, threshold rating age for different structural types lies between 55 and 65 years not only for the curve fitting approach but also for the constant deterioration rate method [38]. In contrast, IBMS deterioration curve reaches a condition rating of 4 at 80 years. INDOT experience indicates that is unlikely to have a concrete superstructure older than 70 years without any rehabilitation or repair. Deterioration models proposed by Moomen et al. [17], appear to better reflect the common practices in Indiana for concrete superstructures.

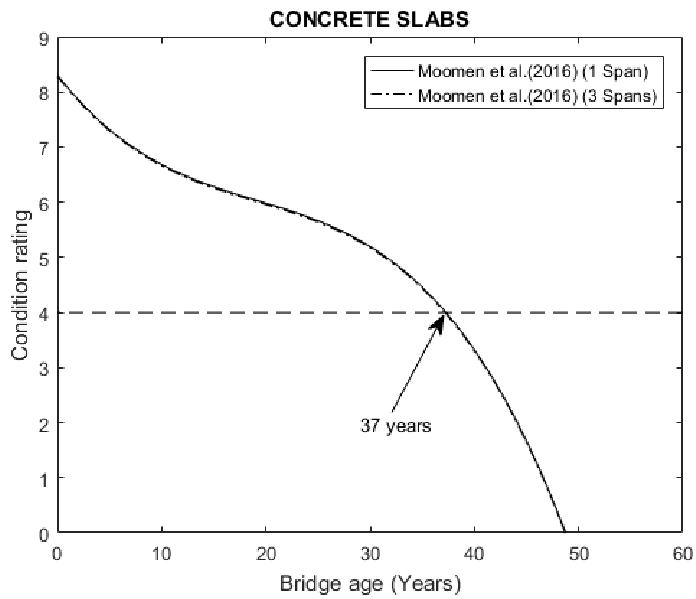


Fig. 5.2. Deck deterioration example.

Deterioration curves are used to predict maintenance, rehabilitation and reconstruction scheduling for each of the design options considered. For concrete structures, models proposed by Moomen et al. [17] were selected. Additionally, the model for steel structures corresponds to curves proposed by Sinha et al. [20]. Once an element reaches the threshold for each condition, a jump in the condition rating will be assumed and the deterioration afterwards will follow the correspondent curve (see Figure 2.3). Final deterioration profiles were used to allocate agency and user costs during the BLCCA process.

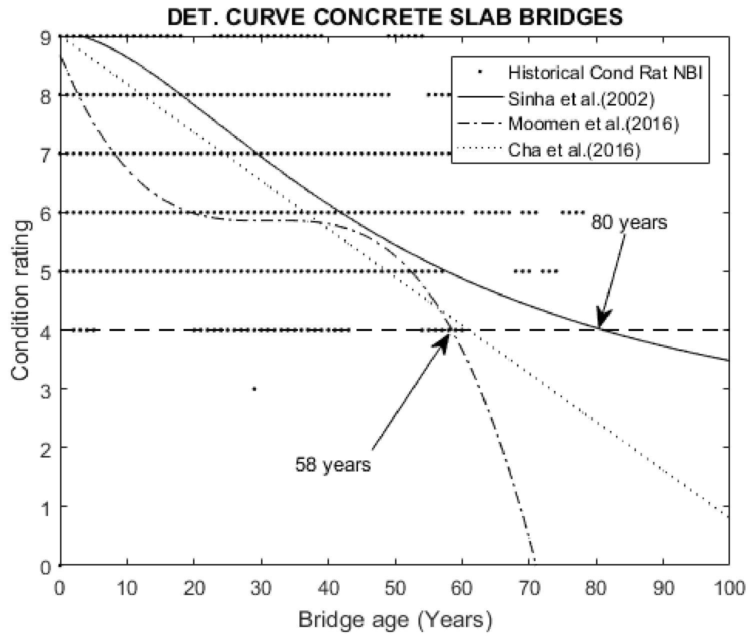


Fig. 5.3. Deterioration curves example for concrete slab bridges.

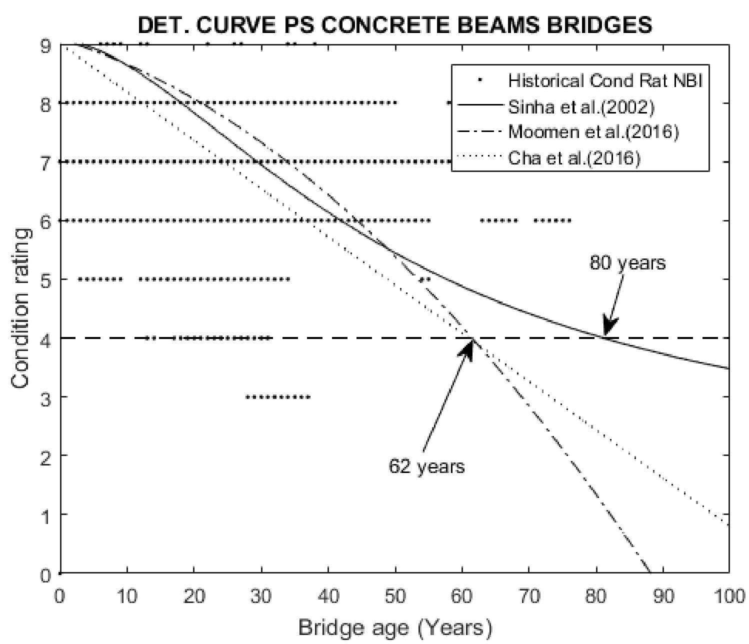


Fig. 5.4. Deterioration curves example for prestressed concrete beam bridges.

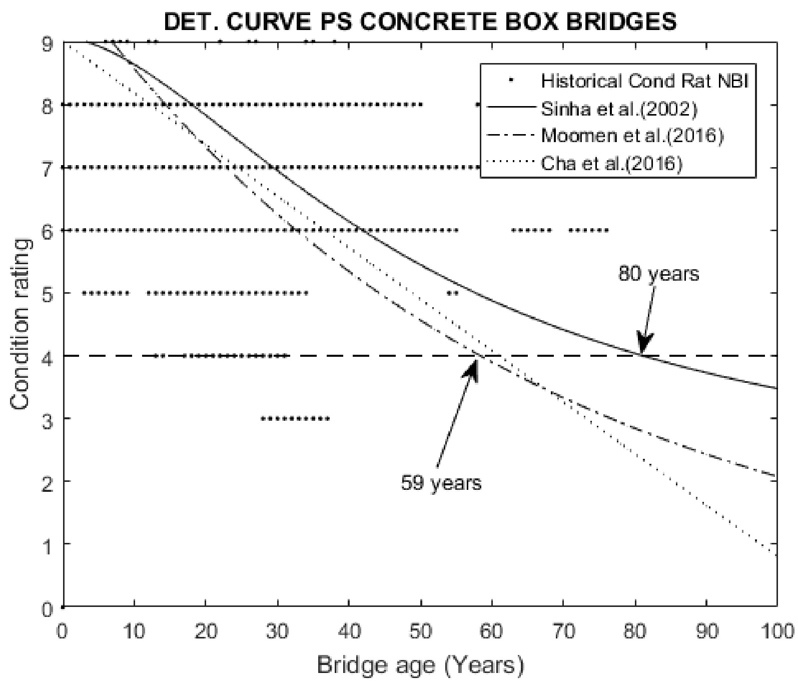


Fig. 5.5. Deterioration curves example for prestressed concrete box bridges.

6. LIFE-CYCLE COST PROFILES FOR INDIANA BRIDGES

For concrete structures, deterioration models proposed by Moomen et al. [17] are used. For concrete slabs, the model projected a service life of 59 years. Pre-stressed structures are divided into two structural types; pre-stressed concrete beams with a service life of 65 years and pre-stressed concrete boxes with a service life of 60 years. In contrast, the service life for steel structures is projected to be 80 years, according to the model proposed by Sinha et al. [20]. These expected lives limit the life-cycle of the structure and are the basis of profiles proposed.

As discussed before, working actions considered in the superstructure often involves deck interventions. For this reason, preventive and maintenance activities for decks must be considered in the life-cycle of the superstructure. Working actions recommended include cleaning and washing of the deck surface, deck and crack sealing, deck patching and deck overlays. In addition, joint maintenance needs to be addressed for bridge decks. However, this working action is not considered since all continuous bridges were designed jointless. Further information about costs, maintenance, scheduling and life cycle of different alternatives for joint replacement is discussed in the report by Bowman and Moran [31].

The research by Soltesz [39] concludes that a decrease of chloride content for decks is only appreciable if it is washed on a daily basis, which is not practical or cost-effective. However, ACI Committee 345 [40] recommends washing the exposed surfaces on a yearly basis in order to avoid extreme deterioration. Therefore, and following the recommendations made by Bowman and Moran [31] to INDOT, washing, and cleaning of the deck surface is considered on a yearly basis schedule.

Deck sealing has been proven to be beneficial to extend decks service life [41] [42]. However, INDOT regular bridge maintenance current practice only considers it during deck construction or reconstruction [31]. Soriano [43] and Mamaghani et al. [44] stated that the first sealing process should be done within 3 to 6 months after construction, with justification to consider it at year 0 or simultaneously with deck reconstructions. Regular use of sealants could extend the initial life of a deck up to 40 years according to Zemajtis and Weyers [45]. However, sealants depending on the fabricator, weather conditions, and traffic wearing have different service lives. Sealant service life expectancy varies from 5 to 10 years (based on studies made by Weyers et al. [46], Zemajtis and Weyers [45], Meggers [47], Soriano [43], Mamaghani et al. [44], Wenzlick [48] and ACI Commitie 345 [40]) and need to be replaced routinely. Both Bowman and Moran [31], and Frosch et al. [41] [42] recommended that Indiana bridge decks to be resealed every 5 years for high traffic roads. Consequently, profiles considering deck sealing every 5 years and a deck overlay after 40 years are considered.

Concrete deck patching involves the removal of contaminated concrete down to the level of the reinforcement steel in the affected area, followed by steel cleaning and replacement if necessary, and installation of the final patch with new high-quality concrete or mortar with low permeability [49]. There are some disadvantages using this method that are related mostly to the incomplete or insufficient removal of concrete in the affected area. In Indiana, some decks have experienced significant corrosion processes after only 7 years from the reparation according to Olek and Liu [49]. This repair action must be performed as early as possible in order to avoid accelerated corrosion problems. Bowman and Moran [31] proposed a 10 year life cycle for patching repairs for bridge decks areas with no more than 10% of the total deck surface repaired. Additionally, as considered by Weyers et al. [46] in their proposed life-cycle models, an increase in maintenance cost due to progressive deterioration needs to be considered.

Among the numerous deck protection systems that are available, overlays are considered as one of the most cost-effective options since the early 80's [50]. There are different types of overlays: portland cement overlays, polymer, and epoxy mortars or concretes and polymer impregnated concrete [51]. As noted by Frosch and Blackman

[52] Portland cement overlays include low-slump dense concrete (LSDC), polymer-modified concrete (also called latex-modified concrete) and fiber-reinforced concrete (FRC). Latex modified concrete overlays are the most common type found in Indiana. Polymer-impregnated concrete overlays will not be discussed in this report as they have not become generally effective, economical, or practical [51]. Asphaltic concrete overlays are relatively porous and, by themselves, do not provide an effective seal. This porosity entraps salt-laden moisture which, in the absence of an effective deck sealer, can promote deck deterioration [40]). The current INDOT policy considers the service life of the deck surface to be between 20 or 25 years, followed by a deck re-placement after 15 to 20 years [31]. This policy does not include deck maintenance activities. To conclude, latex modified concrete overlays after 25 years of bridge construction followed by deck reconstruction after 20 years is considered. The service life of over-lays after a bridge repair activity will be considered as 20 years as a lower bound.

Maintenance activities on the superstructure vary depending on the material type and in some cases on the structural type chosen. There are some activities that can be considered as common regarding those two characteristics. Bearing maintenance and replacement is one of them. Different bearing types are available such as elastomeric bearings, cotton duck pads, sliding bearing, manufactured high load multi-rotational bearings and mechanical steel bearings among others [53]. However, INDOT generally only uses two types of devices: for concrete members elastomeric pad devices, and for steel structures elastomeric and steel bearings [54]. This research only will consider elastomeric devices as a common bearing type for all structural designs. Preventive maintenance activities such as cleaning, washing, and flushing are commonly used for elastomeric bearings on a biannual basis [31].

The service life of elastomeric devices when they are well maintained, constructed and designed can last as long as the structure [55] [53]. However, in order to achieve a service life of 100-plus-years, more emphasis must be placed on manufacturing quality [53]. Aria and Akbari [56] proposed a service life between 30 to 50 years, while Azizinamini et al. [53] based on surveys across the United States report a service life of between 50 to 75 years. Case scenarios used in the BLCCA includes a bearing replacement after 60 years in conjunction with the appropriate preventive maintenance, and bearing replacement without maintenance every 40 to 55 years.

Additionally, steel structures could be subjected to preventive superstructure washing, spot painting or full beam recoating. However, superstructure washing is not considered in the LCCA profiles. Conversely, spot painting and recoating procedures need to be performed on a regular basis.

Protection against corrosion for steel structures includes painting, metalized coat, galvanization and weathering steel use. Among them, painting is the most common coating system to protect carbon steel bridges due to its relatively low initial cost and simplicity of application [31]. Fricker and Zayed [34] conducted an extensive evaluation of steel bridge maintenance practices using different types of painting procedures and coatings. Deterioration curves and LCCA were conducted. LCCA computation showed that the most cost-effective painting system is the three-coat painting system [57]. The service life of initial painting could vary from 30 to 50 years, however, repainting maintenance may not be as effective, and will generally last between 20 to 30 years as described by Soliman and Frangopol [58]. Internal communication with INDOT personnel indicates that for Indiana steel bridges the initial painting service life is assumed as 35 years and the repainting service life as 20 years.

Spot painting activities involve the treatment of a small damaged region of the painted area. Some researchers have studied the cost-effectiveness of the spot painting in comparison with the repainting alternative. Fricker and Zayed [34] proposed that the best re-habilitation scenario is to perform spot repairs every 15 years instead of replacing the coating with a total recoating option currently used by INDOT. Tam and Stiemer [59] performed an LCCA including spot painting, overcoat, and full recoat. They conclude that spot repair is the most cost-effective method for rehabilitating the corrosion resistance of a steel bridge. Bowman and Moran [31] proposed a maintenance practice that includes a two coat system (using a primer and a top coat) as part of spot painting that is performed every 10 years in areas not larger than 10% of the exposed area.

No extension in the service life of the different superstructures was assumed as a result of different maintenance and preventive working actions. There is not historical data to support such assumption into the analysis. The combination of different working actions as described before, and its application to a given structure, leads to a unique life-cycle profile. Different alternatives were considered for each of the superstructure types analyzed, leading to the optimal life-cycle profiles for each one of them based on lower present values computed using BLCCA. All the life-cycle profiles considered are presented in Appendix C. Several different profiles were chosen to compare cost effectiveness and are illustrated as follows:

- Slab bridges (see Figure 6.1). Cleaning and washing as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. A deck overlay at 40 years. Finally, a bridge superstructure replacement at the end of its service life (58 years).

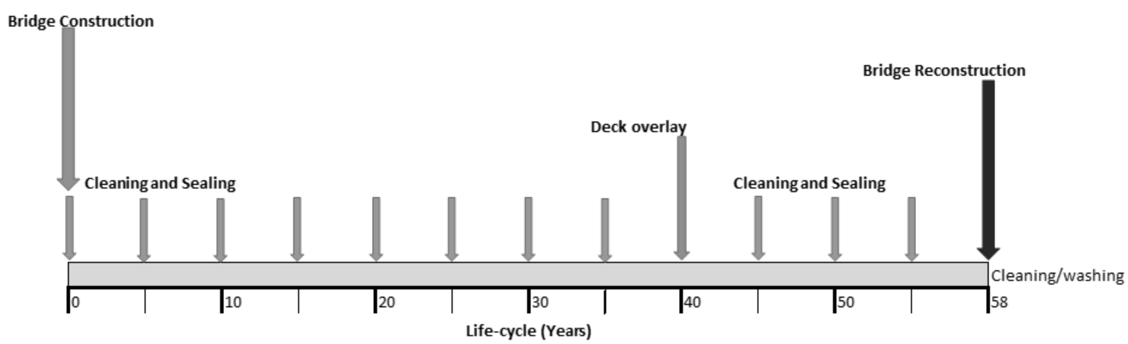


Fig. 6.1. Life-cycle profile for slab bridges.

- Prestressed concrete I beams with elastomeric bearings (see Figure 6.2). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. A full deck replacement at 40 years along with bearing replacements. Finally, a bridge superstructure replacement at the end of its service life (65 years).

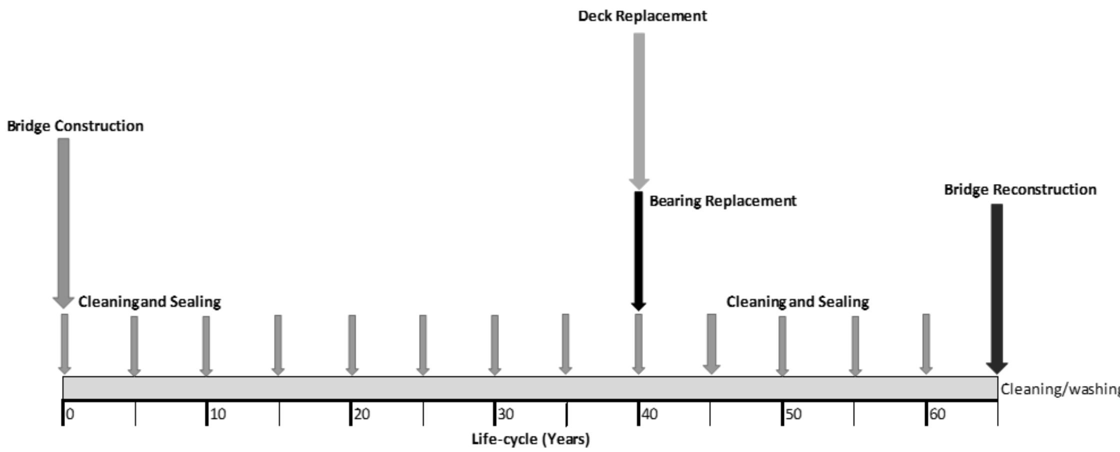


Fig. 6.2. Life-cycle profile for prestressed concrete I beams with elastomeric bearings.

- Pre-stressed concrete box beams (see Figure 6.3). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. A full deck replacement at 40 years along with bearings replacements. Finally, a bridge superstructure replacement at the end of its service life (60 years).

- Steel superstructures (see Figure 6.4). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. One bearings replacement at 40 years. A full deck replacement at 40 years. Spot painting every 10 years on less than 10% of the exposed beam area. Finally, a bridge superstructure replacement at the end of its service life (80 years).

Through discussion with INDOT personnel, it was noted that accelerated deterioration at beams ends is one of the main reasons of why prestressed elements show

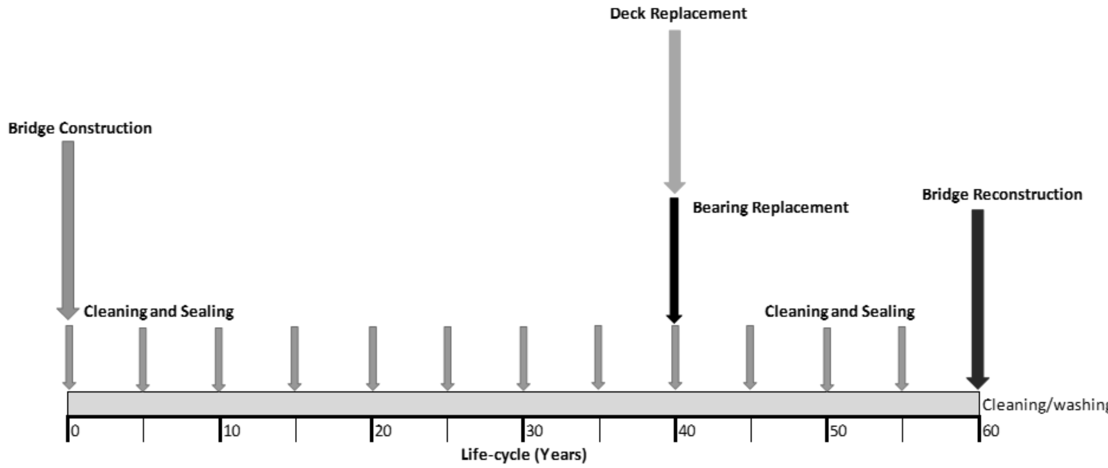


Fig. 6.3. Life-cycle profile for prestressed concrete box beams.

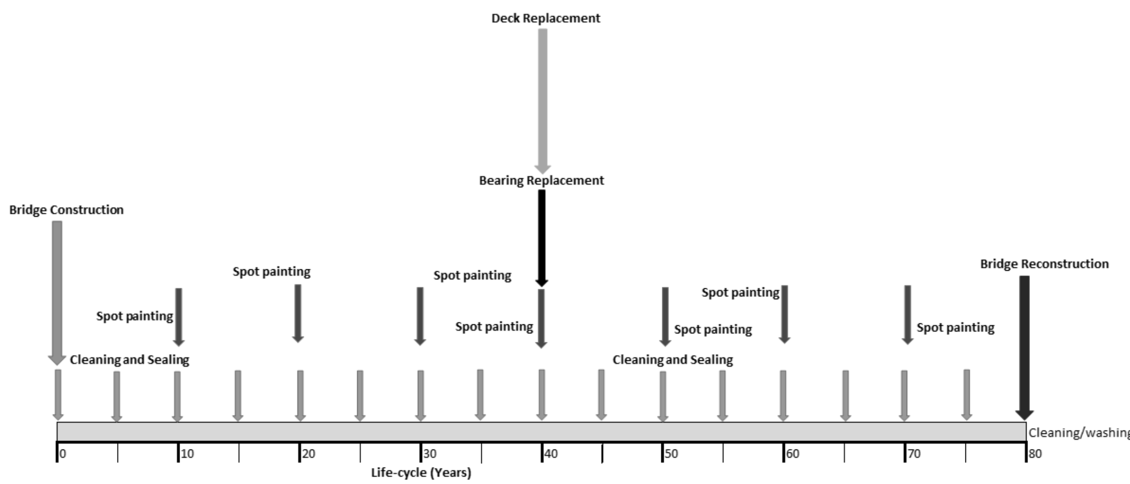


Fig. 6.4. Lifee-cycle profile for slab bridges.

shorter service lives compared with structural steel elements. One option to avoid this abnormal deterioration is to eliminate beam end joints and cast diaphragms over the piers and use integral end abutments. This alternative will undoubtedly extend the service life of prestressed structures. For the purpose of this study, it is assumed that this activity will extend the service life of these type of superstructures up to the same value as that used for structural steel elements, which is 80 years. That will effectively represent and is an extension of 15 years of the service life. Therefore, life cycle profiles including this improvement are also considered, adding the corresponding diaphragm initial cost to the alternative analyzed. In addition, SDCL system service life is also extended in the same proportion since the system itself is based on the same principle of integral abutments and intermediate pier diaphragms, making its service life 95 years. Consequently, profiles chosen to compare its cost effectiveness are the following:

- Steel superstructures SDCL (see Figure 6.5). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. A full deck replacement at 50 years. Spot painting every 10 years less than 10% of the exposed beam area. Finally, a bridge superstructure replacement at the end of its service life (95 years).

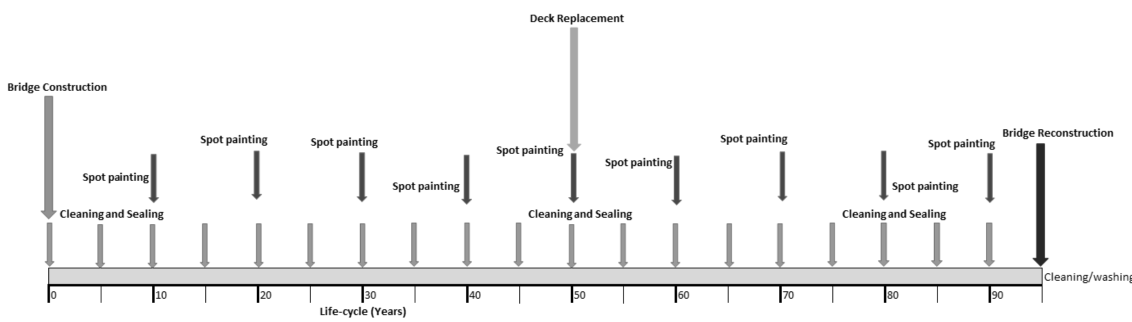


Fig. 6.5. Life-cycle profile for steel structures SDCL.

- Prestressed concrete I beams including diaphragms (see Figure 6.6). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. A full deck replacement at 40 years. Finally, a bridge superstructure replacement at the end of its service life (80 years).

Finally, section loss due to corrosion for steel superstructures is considered as one of the main reasons for deterioration. Therefore, corrosion protection is important

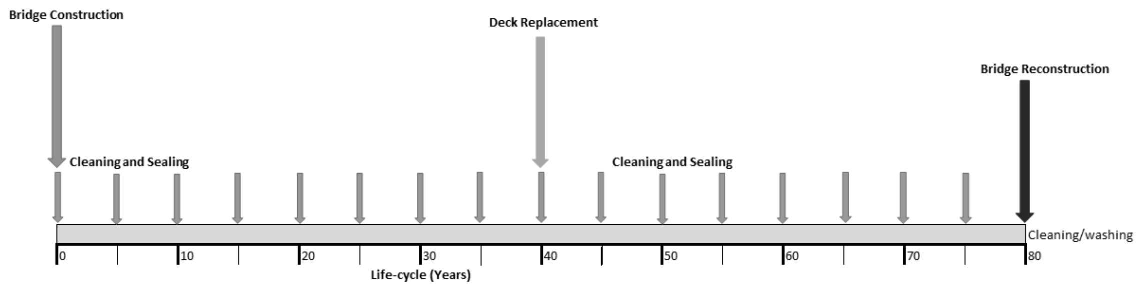


Fig. 6.6. Life-cycle profile for prestressed concrete I beams including diaphragms.

to enhance service lives in these type of superstructures. Different alternatives have been considered including painting, weathering steel, metallization and galvanization.

The life-cycle cost profile (LCCP) presented in Figure 6.4 only depicts the painted alternative. However, the use of other corrosion protection systems could increase the service life of steel elements significantly. According to the American Galvanizers Association [60], for suburban environments, a zinc average thickness of 4.0 mils or more could extend the service life of the initial coating up to 100 years or more, and it eliminates spot painting and represents an extension of the service life of 20 years compared with the painted elements. Accordingly, equivalent extension in the service life is considered for the SDCL galvanized option with integral end abutments, improving its service life to 115 years. Consequently, profiles chosen to compare its cost effectiveness are as follows:

- Steel superstructures - Galvanized (see Figure 6.7). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5 years following bridge construction. One bearings replacement at 50 years. A full deck replacement also at 50 years. Finally, a bridge superstructure replacement at the end of its service life (100 years).
- Steel superstructures SDCL Galvanized (see Figure 6.8). Cleaning and washing of the deck as a regular annual activity. Cleaning and deck sealing every 5

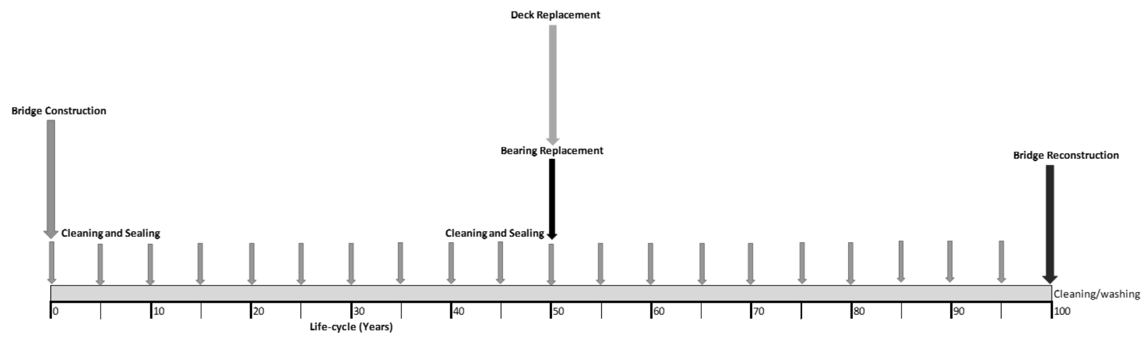


Fig. 6.7. Life-cycle profile for galvanized steel structures.

years following bridge construction. Full deck replacements at 40 and 80 years. Finally, a bridge superstructure replacement at the end of its service life (115 years).

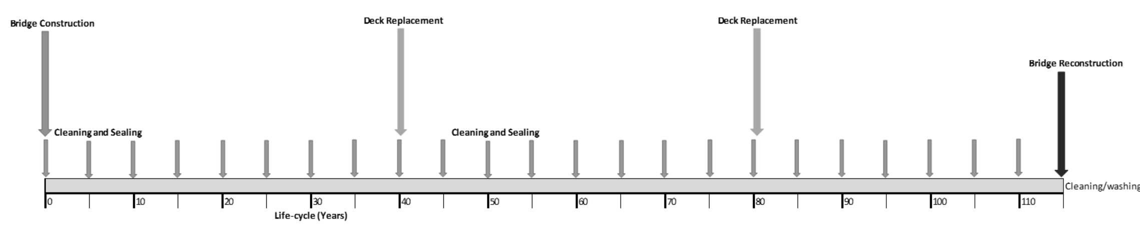


Fig. 6.8. Life-cycle profile for galvanized steel structures SDCL.

It is important to mention that continuous steel galvanized beam structures with integral end abutments are not considered in this study due to its cost-effectiveness. As it can be seen in Chapter 7 results for the case of SDCL, if galvanized and painted options are compared, the extension in service life due to galvanization involves an additional deck reconstruction, that impact negatively the cost effectiveness of this alternative. Following this trend, it is assumed that the extension in the service life due to the inclusion of integral end abutments for continuous steel galvanized structures will also require an additional deck reconstruction that will impact negatively the final outcome of this alternative.

7. LIFE-CYCLE COST ANALYSIS FOR INDIANA BRIDGES

Results of the bridge design, cost allocation, and deterioration curves were used to create the BLCCA for each design option. Those investigations will be the starting point for recommendations made to designers based on BLCCA.

Sinha et al. [20] developed a Life-Cycle Cost module for the Indiana bridge management system (IBMS) called LCCOST. The outcome of this module is the difference in expected life-cycle costs with or without the decision tree module recommendation (maintenance / rehabilitation / reconstruction). Nevertheless, LCCOST does not compare different alternatives for the same project in terms of life-cycle costs. This study can be understood as a complementary tool for agencies rather than an extension to the modules created for the IBMS.

Life cycle profiles indicate not only the possible location for each major and routine working actions, but they also indicate the length of the life cycle itself. Depending on the type of material, structural type and major work actions considered, the length of the life cycle could vary. In order to compare different options using BLCCA, there is a need to establish a comparable service life for all alternatives. If two alternatives with different service lives are to be compared, the least common multiple of the two estimated service lives of the two alternatives must be used according to Grant and Grant-Ireson [61]. However, it is assumed that in the case of highway assets with long service lives like bridges, it is likely to replace the structure in the same place over and over again rather than replace it in different locations each time. This factor implies that the life cycle is recurrent independent of the structure type used.

Consequently, it can be assumed that each alternative will be indefinitely replaced, in other words in perpetuity. Fwa [62] and Ford et al. [63] both describe methods to compute the present worth of life cycle cost in perpetuity. Equation 7.1 shows Ford‘s alternative, where P_p is the present worth of LCCA in perpetuity (LCCAP for further reference), P is the life cycle cost of a single service life at the beginning of the SL, i is the interest rate used and SL is the service life in years of each option. Using this equation, it is possible to compare different alternatives with different service lives in terms of life-cycle costs.

$$P_p = \frac{P(1+i)^{SL}}{(1+i)^{SL}-1} \tag{7.1}$$

It is important to clarify that all analyses and alternative cost considerations are made in constant dollars as is commonly done for economic analysis. Inflation rates will not be considered on the assumption that all costs and benefits of various alternatives are affected equally by inflation [64]. However, if it is considered that the inflation will affect the future costs differently of a given alternative, then such adjustment, need to be made according to the American Association of State Highway Transportation Officials [65].

7.1 Interest Rate, Inflation and Discount Rate

A generalized engineering economic principle states that all analyses that are based on the value of money is strictly related to the time during which the value is considered. In other words, a given amount of money does not have the same value in the present than it has in the past or the future due to the combination of the inflation and the opportunity cost that affects the value of money over time. On one hand, inflation (f) is the increase of prices of goods and services with time and is reflected by a decrease in the purchasing power of a given sum of money at a current period. On the other hand, opportunity cost is the income that is foregone at a later time by not investing a given sum of money at a current period [64].

Interest (i) is the value that represents the amount by which a given sum of money differs from its future value. In other words, it is the price of borrowing money or the time value of money. Additionally, the change of interest over a time (interest rate) used to compute the present value of a future sum or cash flow is known as discount rate (DR). By definition (see Equation 7.2), inflation has to be included when the discount rate needs to be determined. However, and as specified before, it is assumed that inflation will affect all costs the same, which is the reason why inflation is not considered or taken as 0%.

$$DR = \frac{i - f}{1 + f} \tag{7.2}$$

Discount rates differ depending on the economic activity analyzed. For instance, the discount rate used for social analyses is often different than that used for highway asset management. Some economist have suggested that the long-term true cost of money to be between 4% and 6% [50]. The value often used for highway bridge management according to the Indiana Department of Transportation is 4% [22] [31]. For the purposes of the stochastic simulation, the discount rate is modeled as a Pert distribution with a most probable value of 4%, a minimum value of 2% and a maximum value of 6%. Interest equations and equivalences for different cash flows can be consulted in Appendix D.

7.2 Life-Cycle Cost Analysis Comparison

There are several criteria used to assess the economic efficiency of a project. Some of them are listed as:

- Present worth of cost (PWC)
- Equivalent uniform annual cost (EUAC)
- Equivalent uniform annual return (EUAR)
- Net present value (NPV)
- Internal rate of return (IRR)
- Benefit-cost ratio (BCR)

The first two indicators of economic efficiency are applicable when all alternatives have a similar expected level of benefits and cost minimization is the main objective of the analysis. However, the alternatives analyzed in this document do not have the same level of benefits, as demonstrated by the salvage value for each superstructure type. The last two criteria require a solid estimation of the benefits resulting from the implementation of the alternatives analyzed. Therefore, a complete socio-economic analysis is needed. Such an analysis is outside of the scope of this project and requires a specific location for the alternative chosen. Additionally, these two alternatives are not used for comparing cost-differences in economic analyses. As a consequence, EUAR and NPV are the most common indicators used. However, only NPV is the approach used in this study because, for agency decision makers it is more useful to know the cost upfront than the equivalent uniform annual cost for this specific case.

7.2.1 Equivalent uniform annual return (EUAR)

The EUAR is the combination of all costs and benefits expected from a project expressed into a single annual value of return over the analysis period. This method is useful when all the alternatives have different level of cost or benefits, or when the analysis periods differ from one option to the other.

7.2.2 Net present value (NPV)

The NPV is understood as the difference between the present worth of benefits and the present worth of costs. Basically, this method represents the value of the project at the time of the base year of the analysis period or the year of the decision making. NPV is often considered as the most appropriate of all economic efficiency indicators because it provides a magnitude of net benefits in monetary terms [64]. Therefore, the alternative with the lowest NPV is considered the most economically efficient. For the case of this study, costs are treated as positive values and benefits as negative values. Consequently, the lowest value of NPV is desired.

7.3 Life-cycle cost analysis -Deterministc approach-

Initial cost comparison, as well as LCCA, were made for every superstructure type considered. Table 7.1 presents a summary of the life-cycle cost analysis for simply supported bridges with a simple span of 30-ft (detailed computation of this values can be found in the example given in Appendix E). The discount rate used for the life-cycle cost in perpetuity (LCCAP) is 4%. It presents the service life, total life cycle cost (LCCA), LCCAP and the cost-effectiveness-ratio between the initial cost and LCCAP of the different superstructure types ($ER_{InitialCost}$ and ER_{LCCAP}, respectively). Ratios shown correspond to the ratio between the option analyzed and the lowest price among all the alternatives for a given span length as shown in Equation 7.3.

$$ER_{cost} = \frac{C_{Alt}}{\min_i \left(C_{Alt_1}, C_{Alt_2}, \ldots, C_{Alt_i} \right)} \tag{7.3}$$

The results for the LCCA shown in Table 7.1 illustrate the evidence of considering all costs for various structural types. The cost-effectiveness ratio for initial cost, $ER_{InitialCost}$, clearly shows that slab bridges provide the best alternative, with most other systems costing an additional 15% or more. However, if the cost-effectiveness ratio in perpetuity is examined, ER_{LCCAP}, the results change notably. In this case (for a 30-ft span) the slab bridge is still the most cost-effective solution, but the cost differential - $ER_{InitialCost}$ versus ER_{LCCAP} - changes significantly, with other systems becoming more competitive. The four (4) beam and five (5) beam galvanized rolled beam system have notably closed the cost gap. Other structural systems have also improved in cost-effectiveness when all long-term costs are considered.

Figures 7.1 to 7.3 show the initial cost and LCCAP comparison for simply supported beams for all span ranges using the deterministic approach. As it can be seen in the figures, in general the inclusion of long term-costs using LCCA reduces the difference between all the alternatives for the same span length. Explicitly, for span range 1, it is shown that the slab bridge is the most cost-effective solution either

Table 7.1.
LCC summary example: simply supported beams -span length 30ft

Type	**S L (years)**	**Initial Cost ($)**	$ER_{InitialCost}$	**LCCA ($)**	**LCCAP ($)**	ER_{LCCAP}
Slab Bridge	58	51,438	1.00	133,591	148,900	1.00
Prestressed Concrete AASHTO Beams Bearings	65	59,747	1.16	157,199	170,522	1.15
Prestressed Concrete AASHTO Beams Diaphragms	80	73,639	1.43	164,639	172,106	1.16
Prestressed Concrete Concrete Box Beams	60	75,404	1.47	170,217	188,097	1.26
Structural Steel Beams Painted 4 beams	80	59,224	1.15	157,248	164,380	1.10
Structural Steel Beams Painted 5 beams	80	59,464	1.16	158,535	165,725	1.11
Structural Steel Beams Galvanized 4 beams	100	62,234	1.21	154,594	157,717	1.06
Structural Steel Beams Galvanized 5 beams	100	62,511	1.22	155,573	158,715	1.07
Structural Steel FPG Galvanized 4 beams	100	62,790	1.22	155,139	158,272	1.06
Structural Steel FPG Galvanized 6 beams	100	67,921	1.32	161,332	164,591	1.11

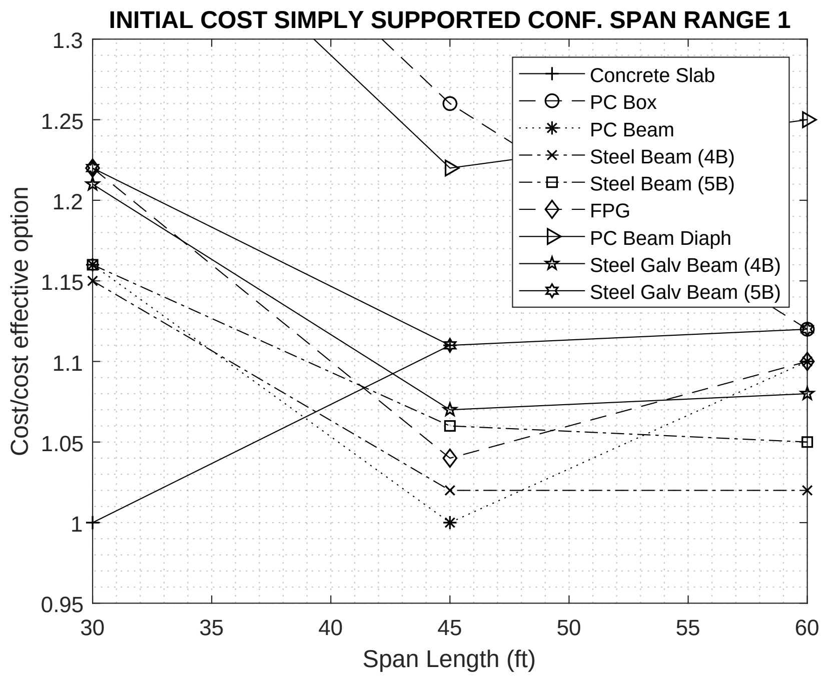
considering or not considering long-term costs for spans less than 45-ft. However, for spans longer than 45-ft, the inclusion of galvanized steel structures specifically the four (4) beam configuration is the most cost-effective alternative. In contrast, if only initial costs are considered, painted rolled beams and prestressed concrete AASHTO beams would be the preferable options. Additionally, it is important to mention that the FPG option is among the cost-effective solution for the second part of the span range; however, it is not the optimal selection.

For span range 2, 4 beam galvanized rolled beams are still cost-effective for spans shorter than 65-ft, while the prestressed concrete bulb tees became the optimal solution for longer spans. If only initial cost are considered, prestressed concrete bulb tees alone would be selected for this span range. This trend is attributed to the lower material and fabrication costs and resistance optimization achieved by the bulb tee system.

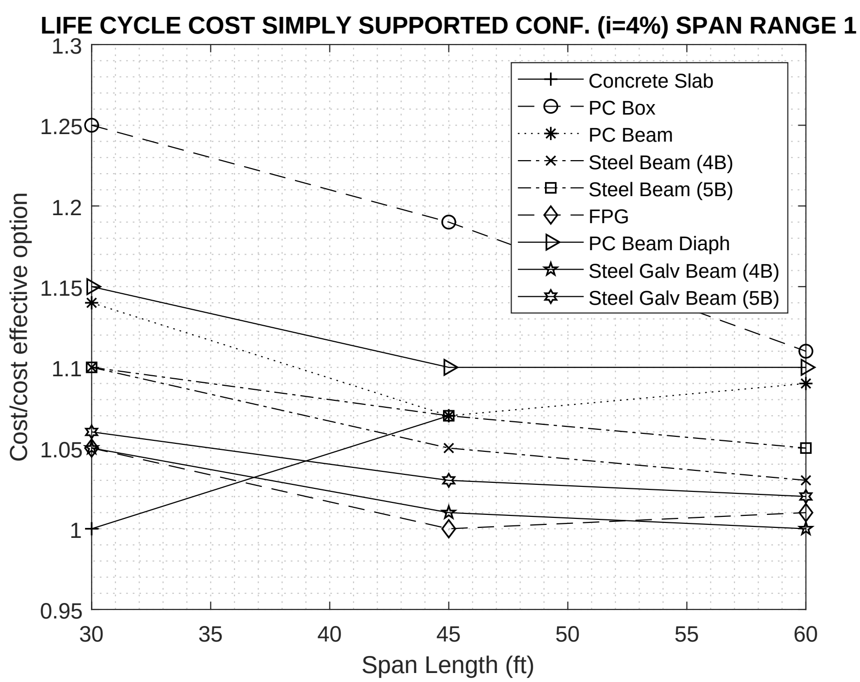
Span range 3 results show that including long-term costs suggests multiple cost-effective design solutions for spans up to 105-ft, with two optimal options being prestressed concrete bulb tees and galvanized steel plate girders. Beyond this point, bulb tees are the most cost effective solution. Again, if only first costs are considered, bulb tees would be the optimal solution for the entire span range.

Results for continuous beams are presented in Figures 7.4 to 7.6. For span range 1, several different outcomes were obtained considering and not considering long-term costs. Slab bridges and galvanized steel continuous beams are the most cost effective solutions for the two halves of the span range, respectively. However, prestressed concrete AASHTO beams are also a competitive option for spans between 45 and 60-ft. In contrast, span range 2 rejects the premise of the cost-effectiveness of the SDCL system for spans up to 90-ft. Additionally, it is noticeable that prestressed bulb tees become more attractive for longer spans. Finally, for span range 3, no variance in the cost-effectiveness of the bulb tee option is noticed between the initial cost comparison and the inclusion of long-term costs, although the cost differential is notably reduced.

It is important to underline the fact that results shown are not a precise measurement of cost-effectiveness. Rather, they are an approximation and the first approach to designers at the moment of bridge planning. This tool could clarify which superstructure options could be cost-effective during the planning process. However, final site conditions and project level cost estimations should represent accurately the best option for construction.

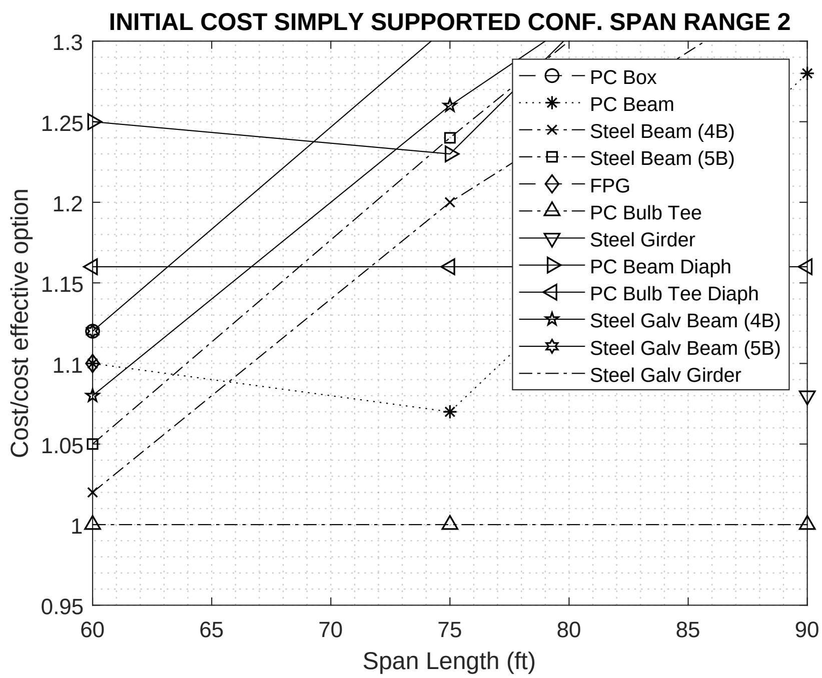


(a) Initial Cost.

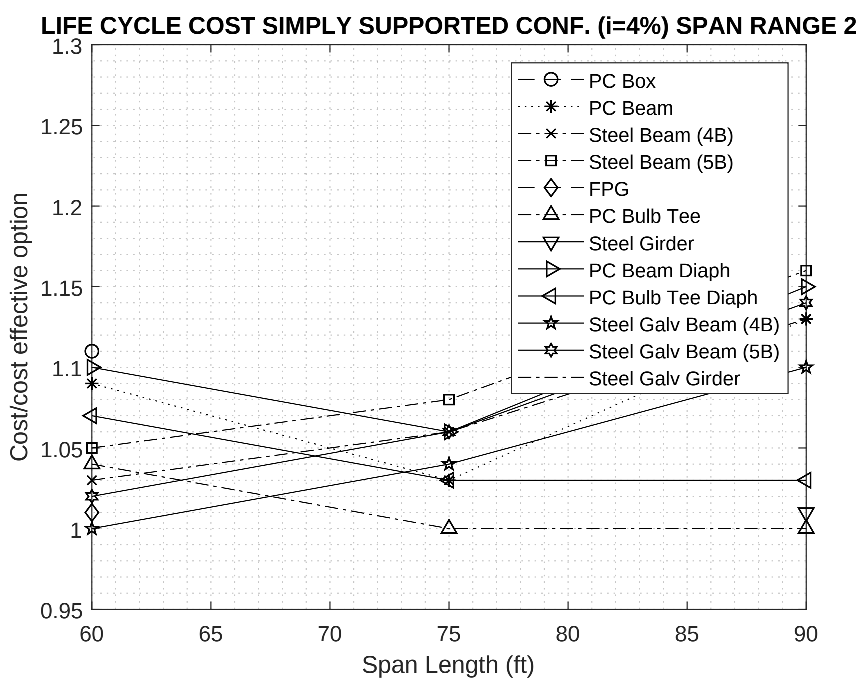


(b) LCCAP.

Fig. 7.1. Cost-effectiveness for simply supported beams -Span Range 1- Deterministc Approach.

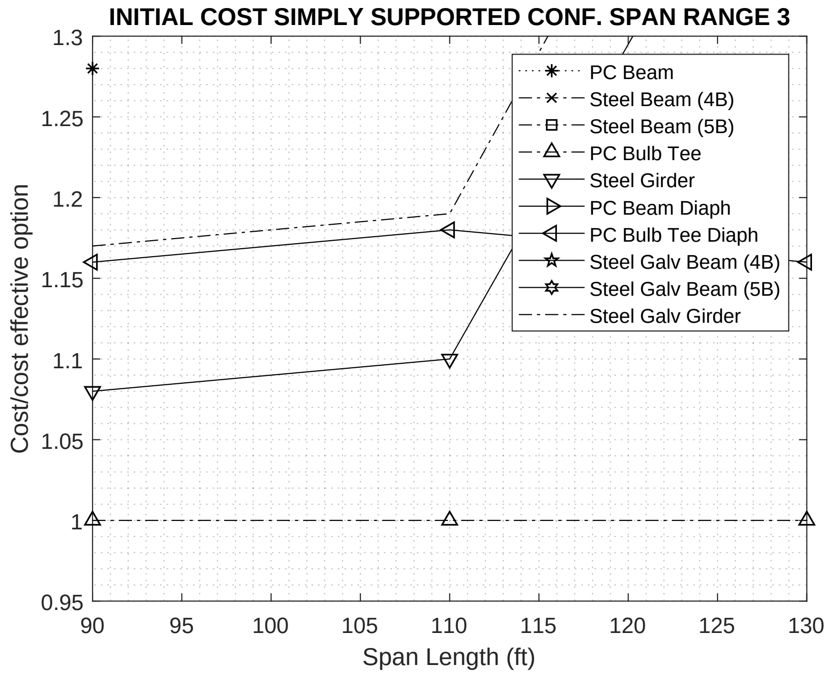


(a) Initial Cost.

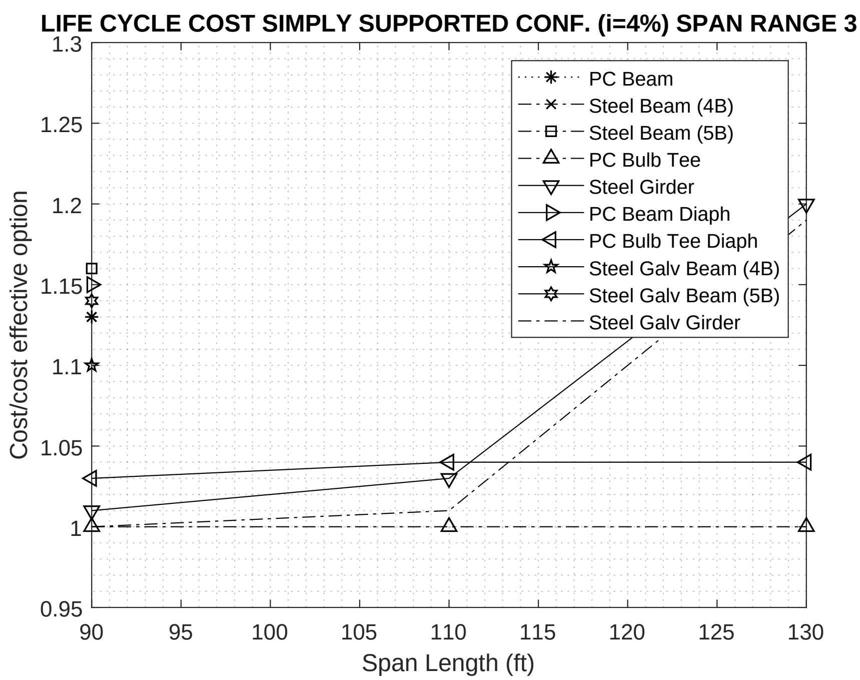


(b) LCCAP.

Fig. 7.2. Cost-effectiveness for simply supported beams -Span Range 2- Deterministc Approach.

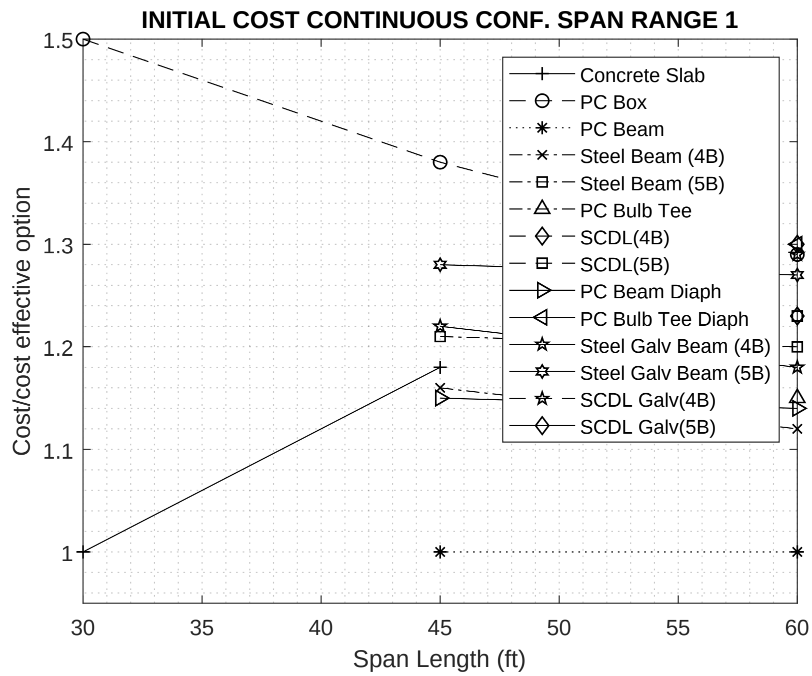


(a) Initial Cost.

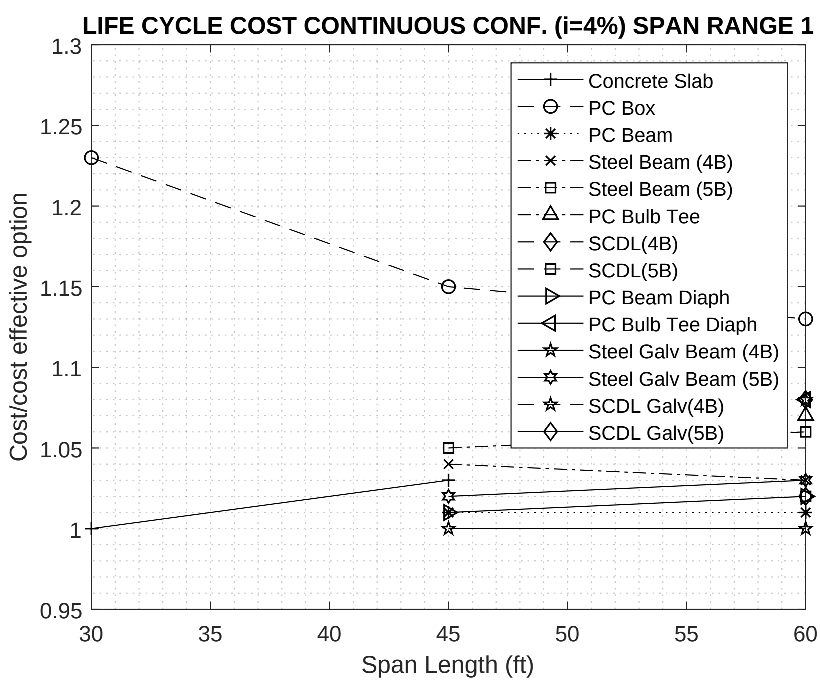


(b) LCCAP.

Fig. 7.3. Cost-effectiveness for simply supported beams -Span Range 3- Deterministc Approach.

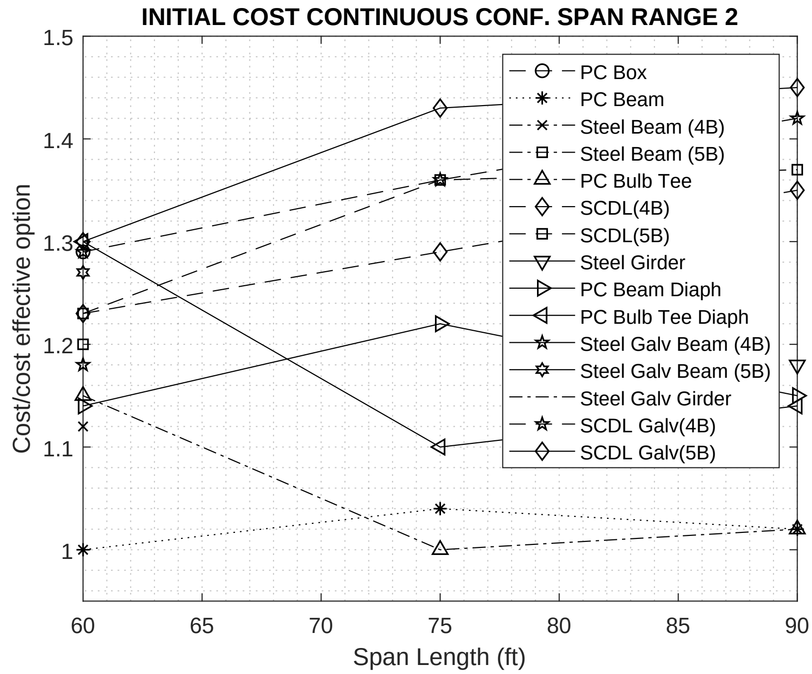


(a) Initial Cost.

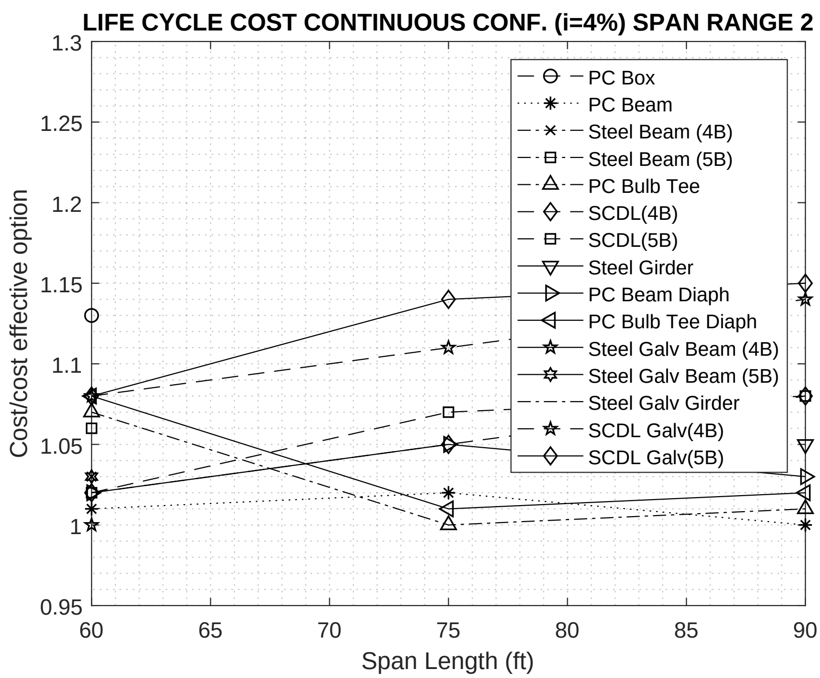


(b) LCCAP.

Fig. 7.4. Cost-effectiveness for continuous beams -Span Range 1- Deterministc Approach.

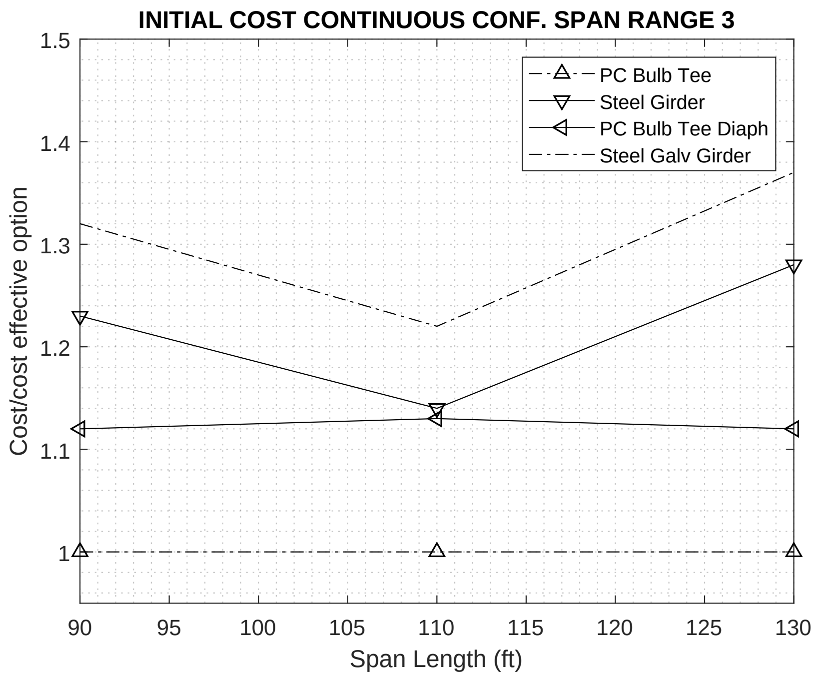


(a) Initial Cost.

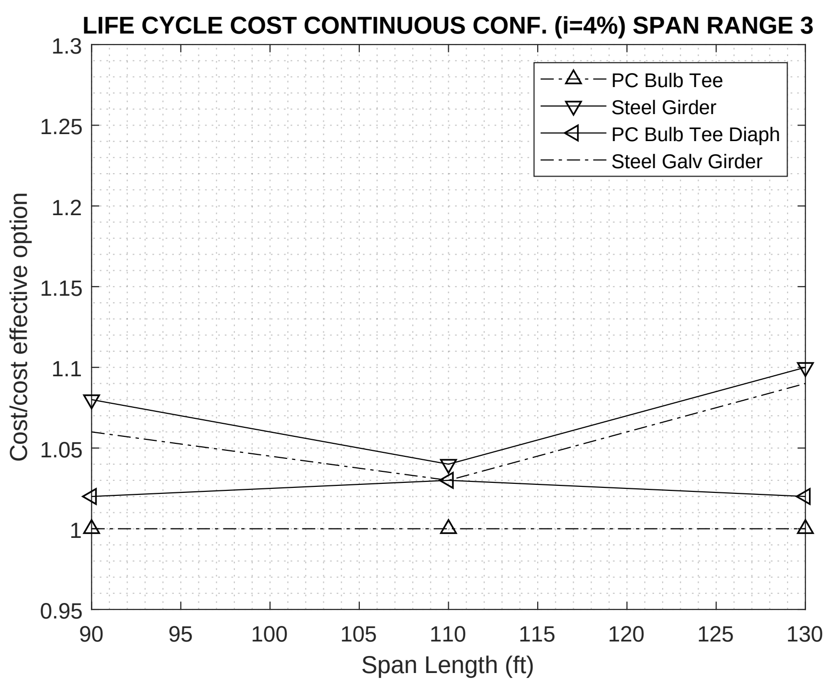


(b) LCCAP.

Fig. 7.5. Cost-effectiveness for continuous beams -Span Range 2- Deterministc Approach.



(a) Initial Cost.



(b) LCCAP.

Fig. 7.6. Cost-effectiveness for continuous beams -Span Range 3- Deterministc Approach.

FPG system needs a special discussion. As shown, the FPG option could be considered as a cost-effective solution depending on the span length of the structure. Nonetheless, a more accurate cost estimation of construction cost, not only for steel elements but also for prefabricated composite modules, is needed to demonstrate that viability of this system.

7.4 Life-cycle cost analysis -Stochastic analysis

The deterministic analysis could be useful as a first approximation to assess the superstructure selection. However, and as it is seen in Figures 7.1 to 7.6 the ER_{LCCAP} difference between the different alternatives is minimal and therefore a decision maker will not have a clear preference on a superstructure type. This is especially true given the fact that only average values where used. In other words, only a single case is examined but there is multiple other combinations that are omitted. As shown before, there is an inherent probabilistic nature associated with all the costs involved. Henceforth, a *Monte Carlo* Simulation (MCS) is used to assess this problem.

7.4.1 Monte Carlo Simulation (MCS)

As defined by Ang and Tang [66], "*simulation* is the process of replicating the real world based on a set of assumptions and conceived models of reality". When the problem is based on random variables with assumed or known probability distributions, the *Monte Carlo* simulation (MCS) method is very useful. Generally speaking, MCS method is a repetition of different simulations, using a different set of values of the random variable, based on the corresponding probability distribution. The deterministic analysis is then considered as a single simulation process. For computational purposes, random numbers are generated by a systematic procedure, those are known as *pseudo random.* However, its periodicity as well as their randomness feature can be assured using different algorithms, that are generally included in software packages.

Iterations

The number of iterations needed are highly dependant on the type of sampling chosen for the simulation process. There are various methods, including: random sampling, stratified sampling and latin hyperchube sampling. *Random sampling* is the most intuitive and was the most used during the early development of the MCS process. One of its drawbacks is that when few iterations are used, it is possible that not all the input PDF is sampled. In contrast, the *stratified sampling* techniques (the latin hypercube is a ramification of it) divides the PDF in strata (or hypercube) and then a sample is obtained randomly within the random selected strata. As a consequence, the PDF is better represented during the simulation with fewer iterations. Comparison between the different sampling techniques can be consulted in the work done by Beckman et al. [67]. Generally speaking, convergence is achieved faster using stratified methods.

Minimum iterations needed for *random sampling* depends on the confidence level desired, and is based in the *central limit theorem* (CLT), assuming that the final result will be represented most likely by the normal or lognormal distribution. On the other hand, *stratified sampling* convergence is achieved when additional iterations do not change greatly the statistics of the output distribution. In this case, variation on the output mean and output standard derivation is checked for a given example of LCCAP. Figure 7.7 presents the coverage test to determine the number of iteration needed for the simulation. It shows the normalized output mean and output standard deviation of the LCCAP example depending on the number of iterations (simply supported slab bridge, span=30-ft). As it is shown, output mean converges rapidly (close to 1,000 iterations), while the output standard deviation converges much more slowly, close to 10,000 iterations. The greater number of iterations needed for both statistic outputs is then chosen.

The MCS was done using the Microsoft excel complementary software called @RISK, developed by Palisade software company. The simulations used in this doc-

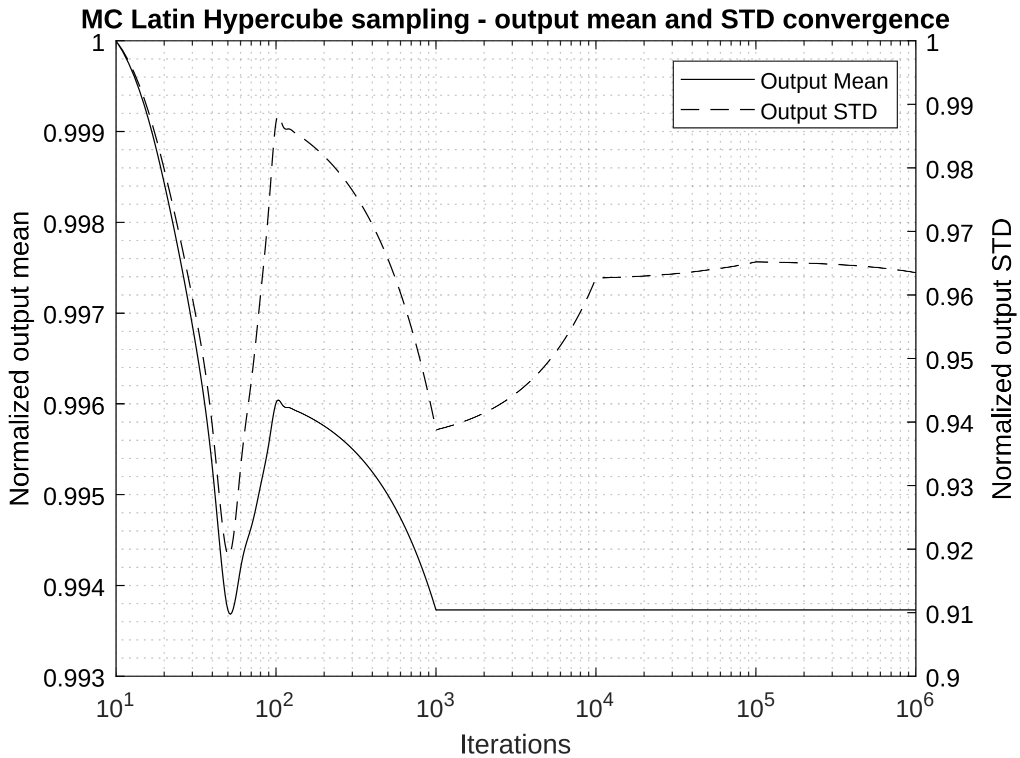


Fig. 7.7. Latin hypercube sampling for convergence of output mean and standard deviation of LCCAP SSB slab bridge, 30-ft

ument uses a random number generator based on *Mersenne Twister* pseudo random numbers, latin hypercube sampling and 10,000 iterations per simulation.

7.4.2 Stochastic dominance (SD)

The concept of *Stochastic Dominance* (SD) is used to define one of the methods used to categorize an alternative among others in a sense that "there will be one investment which is better than (or equal to) all of the other available investments" [68]. This categorization, when the decision makers only has partial information available, is also known as *partial ordering*, which is the case of the SD.

In order to categorize the alternatives, the process divides the *feasible set* (FS, i.e. a set composed by all possible alternatives), into the *efficient set* (ES) which is composed by all the alternatives that satisfy the decision requirement, and the *inefficient set* (IS) that contains the alternatives that are excluded from the decision rule. Those two sets are mutually exclusive and comprehensive, and are shown in Figure 7.8.

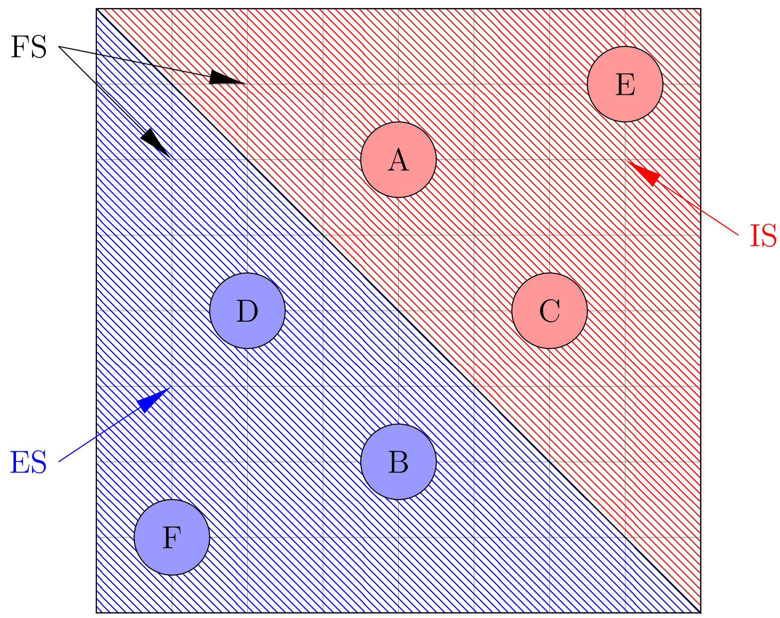


Fig. 7.8. Feasible (FS), Efficient (ES) and Inefficient (IS) sets.

The decision rule is based on the "dominance" of one alternative over the other. This dominance could be represented by first, second, third or nth degree of dominance. Namely, the general rule of thumb is to consider the ES as the group of none dominating alternatives, and the IS as the complementary group of the FS. This is especially true for cases in which a higher chance of positive revenues is intended when negative results are possible as the case of the utility functions (examples given in [68], [69], [70], [71], [72] and [73]). However, for the case of this study, the objective is to find the alternative with the lower price, i.e. the alternative with a higher probability to have a lower price. Henceforth, the decision rule used to divide the FS is such that the ES will conglomerate the alternative that is dominated by all the other alternatives and the IS the complementary options. The concept of "dominance" is better understood using the different levels of *stochastic dominance* explained as follows. Mathematical proof for all the stochastic dominance levels presented here can be found in the work done by Levy [68]. Additionally, if an alternative (A) is dominated by another (B), no matter the degree of dominance, it is considered that A is preferred to B, $A \succ B$. However, if the result does not conclude dominance, it is considered that there is no preference between the alternatives and the decision maker is indifferent between the options, $A \sim B$.

First stochastic dominance (FSD)

The FSD criterion is a way to establish if an alternative dominates (or not) another alternative if the only information available is that the set of utility functions (U) considered belongs to the set of non decreasing utility functions ($\mathbb{U}_1$), $U \in \mathbb{U}_1$, and the first derivative of U is grater than or equal to 0, $U' \geq 0$.

If F and G are the CDFs of two different alternatives, F dominates G (denoted as FD_1G where D_1 means first degree of dominance) if and only if $F(x) \leq G(x)$ for all values of x, and there is at least some x_0 for which a strong inequality holds. Let the difference of the two considered CDFs be $I(x)$ such as $I(x) = G(x) - F(x)$, then FD_1G if and only if $I(x) \geq 0$ for all x and $I(x) > 0$ for at least one x_0. Figure 7.9 shows an example of FSD for 75-ft simply supported beams. In this case, four painted structural steel rolled beams option (F) dominates the prestressed concrete AASTHO beams option (G) by first degree. Then, prestressed concrete AASTHO beams are preferred, $G{\succ_1}F$.

Second stochastic dominance (SSD)

In the case that two alternatives do not satisfy the FSD requirements, a stronger theorem is needed for investors most risk averse. For concave utility functions ($\mathbb{U}_2$), or in other words, utility functions (U) such that $U' \geq 0$, and $U'' \leq 0$, there must

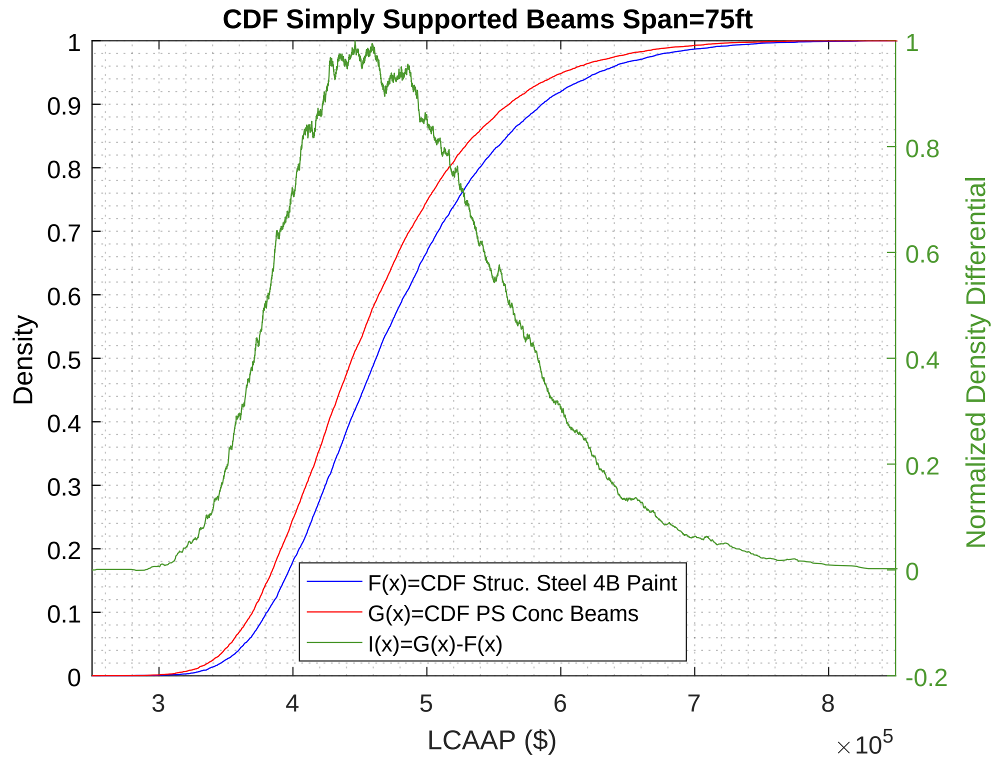


Fig. 7.9. FSD example for simply supported beams, span=75-ft.

be a utility function U such that, for two alternatives F and G, $\int_a^x G(t)dt \leq \int_a^x F(t)dt$. Then, F dominates G by SSD (denoted as FD_2G where D_2 means second degree of dominance) if and only if $I(x) \equiv \int_a^x [G(t) - F(t)]dt \geq 0$ for all $x \in [a, b]$ and there is at least one x_0 for which there is an strict inequality. Additionally, it is also stated that if FD_1G, then FD_2G. This is especially important when you have multiple alternatives to compare in order to minimize the number of comparisons.

Figure 7.10 presents an example of SSD for two simply supported beams with a span of 45-ft. As it can be seen clearly in the Figure, alternative F (concrete slab option) does not dominate alternative G (prestressed concrete bulb tee option) in first degree because both CDFs crossed each other in multiple points. Additionally, it is presented in the right y axis, the normalized area under the CDF differential, $I(x)$. It is proved that F dominates G in second degree, which means that a decision maker will be inclined to chose the prestressed concrete bulb tee option over the concrete slab option, $G \succ_2 F$.

Nonetheless, there are cases in which the comparison between different alternatives using FSD and SSD is not enough to clearly select an option among the others. In those cases, the concept of *almost stochastic dominance* proposed by Leshno and Levy [70] is useful. This method is intended to identify a preference of an alternative among others for "most" decision makers but not "all" of them.

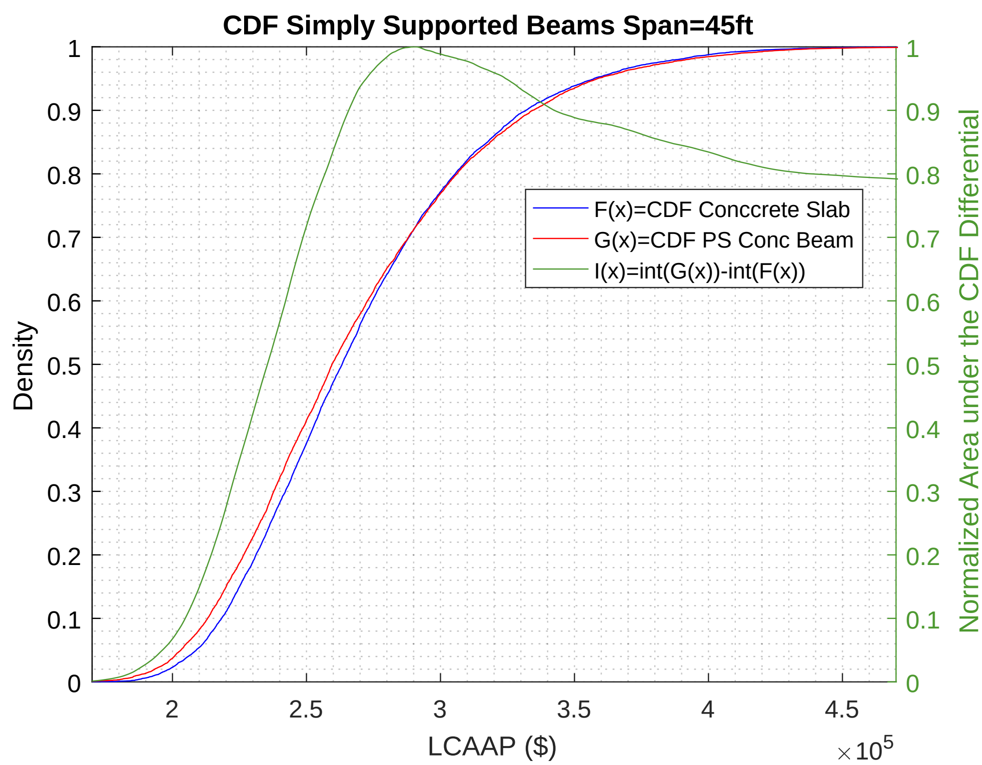


Fig. 7.10. SSD example for simply supported beams, span=45-ft.

Almost first stochastic dominance (AFSD)

AFDS rules are based on the definition of FDS. However, while FSD stated that for two alternatives with different CDFs denoted as F and G, the following inequity, $F(x) \leq G(x)$, must be true for all x and strictly true for at least one x_0. The AFSD, however, requires that the previous inequality must be true for “most” of the range S, allowing a small “violation” of the dominance. The violation range of the FSD (S_1) is denoted as

$$S_1(F,G) = \{x : G(x) < F(x)\} \tag{7.4}$$

then, the ratio between the violation area of the FSD and the total area between the CDF's (ε_1) is

$$\varepsilon_1 \equiv \frac{\int_{S_1} (F(x) - G(x))dx}{\int_{S} \mid F(x) - G(x) \mid dx} \tag{7.5}$$

Leshno and Levy [70] stated that *almost stochastic dominance* holds when ε_1 is lesser than 50%, $0 < \varepsilon_1 < 0.5$. The limiting value for ϵ is strictly related with the level of risk aversion, it means than the smaller the value, the higher the risk aversion and the stronger the dominance. This inequality is also related with the normal SD, when it holds, then the standard FSD or SSD holds.

Figure 7.11 illustrates the AFSD approach for simply supported beams with spans of 75-ft. Alternative F, prestressed concrete AASHTO beams, and alternative G, prestressed concrete bulb tee with intermediate diaphragms, are tested for dominance. The FSD violation area between the two CDF's is shaded in the Figure. The violation ratio, ε_1, is equal to 0.11 which is less than 0.50. Therefore, prestressed concrete AASHTO beams dominates prestressed concrete bulb tee with intermediate diaphragms option, $G{\succ^{almost}}_1 F$.

Almost second stochastic dominance (ASSD)

On the other hand, there are cases in which AFSD is not enough to select an option among others. For these cases, ASSD rules could be used. Consider again two different CDFs alternatives F and G, then, F almost dominates G in second degree

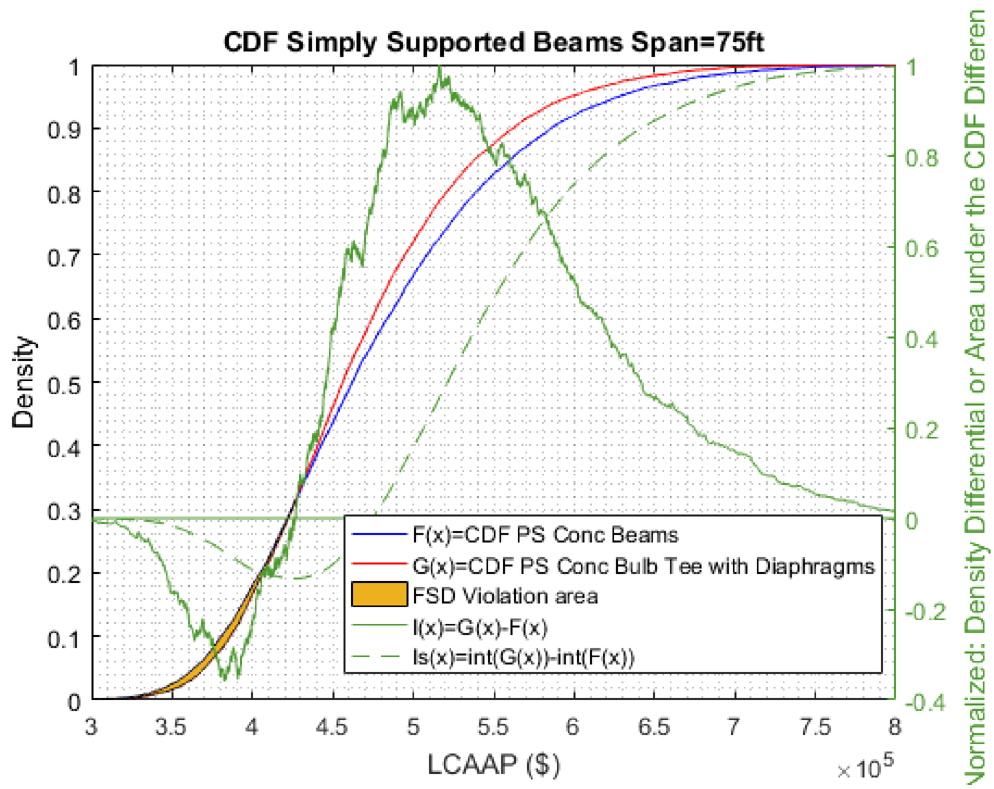


Fig. 7.11. AFSD example for simply supported beams, span=75-ft.

if $\int_a^x G(t)dt \leq \int_a^x F(t)dt$ for most of the range S, but not for all of it. The violation range of the SSD (S_2) is defined by

$$S_2(F,G) = \left\{ x : G(x) < F(x); \int_a^x G(t)dt \leq \int_a^x F(t)dt \right\} \tag{7.6}$$

then, the ration between the violation area of the SSD and the total area between the CDFs (ε_2) is, where $\bar{S}_2$ is the complement area of S_2, i.e $\bar{S}_2 \cap S_2 = S$

$$\varepsilon_2 \equiv \frac{\int_{S_2} (F(x) - G(x))dx}{\int_{\bar{S}_2} (G(x) - F(x))dx + \int_{S_2} (F(x) - G(x))dx} \tag{7.7}$$

As stated in AFSD, for ASSD, F almost dominate G in the second degree if $0 < \varepsilon_2 < 0.5$. However, for simplicity, Levy [69] proposed the Equation 7.7 for an easy programmable algorithm as follows:

$$\varepsilon_2 = \frac{\int\limits_{S_2}(F(x) - G(x))dx}{\mathbb{E}[F] + \mathbb{E}[G] + 2\int\limits_{S_2}(F(x) - G(x))dx} \tag{7.8}$$

Figure 7.12 illustrates the ASSD approach for simply supported beams with spans of 90-ft. Alternative F, galvanized structural steel girders, and alternative G, pre-stressed concrete bulb tee, are tested for dominance. The SSD violation area between the two CDFs is shaded in the Figure. The violation ratio, ε_2, is equal to 0.21 which is less than 0.50. Therefore, the prestressed concrete bulb tee alternative dominates the galvanized structural steel girders option, $F \succ^{almost}{}_2 G$.

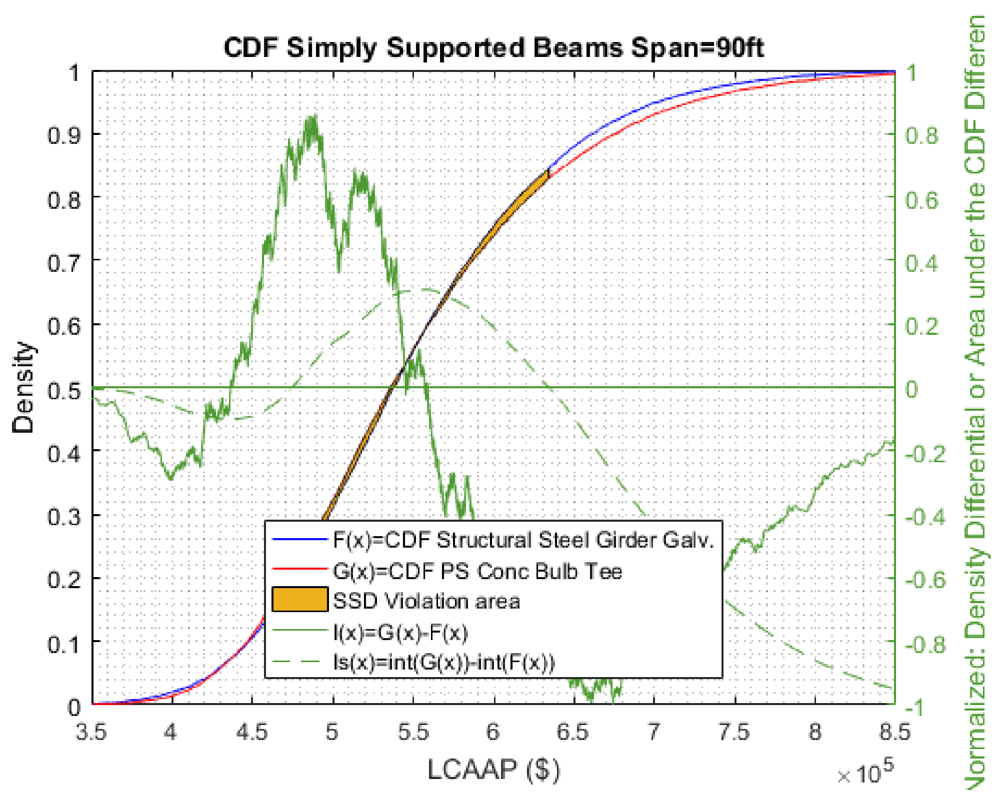


Fig. 7.12. ASSD example for simply supported beams, span=90-ft.

7.4.3 Superstructure selection

Mirroring the process made in the deterministic approach, the first step in order to select the most cost effective superstructure is to find the most cost effective life-cycle cost profile (LCCP) for each superstructure type. Since all LCCP for each type have the same SL, it is indifferent if the LCCA or the LCCAP is used to do the analysis. In this case, the LCCA is the base of the LCCP selection using SD. Additionally, there is no necessity to evaluate the cost effectiveness of all LCCP using all available spans since all factors only will vary on proportion but not in differences. Finally, and base on the LCCP presented in Appendix C, as a general approach, LCCPs can be grouped in main categories: concrete structures, prestressed concrete structures, prestressed concrete structures with diaphragms, structural steel painted structures and structural steel galvanized structures. Therefore, only one case is presented for each of the main groups. The results presented herein corresponds to 30-ft long bridges with simply supported beams and 60-ft long for continuous beams (SDCL and prestressed concrete options with diaphragms).

Table 7.2 summarizes the different alternatives considered for each group. Those alternatives are the combination of working actions such as: washing and cleaning of the deck surface (WC), sealing of the concrete deck surface (SC), full depth concrete patching (CP), concrete overlays (O), superstructure removal (BD), concrete slab reconstruction (DR), bearing replacements (BR), steel elements full painting (FP), steel elements spot painting (SP), and structural steel recycle (SR). As a general rule of thumb, the table presents the current INDOT working action procedure (called INDOT in the table), and different alternatives depending on the superstructure type and the working actions considered (in the table, alternatives A to E).

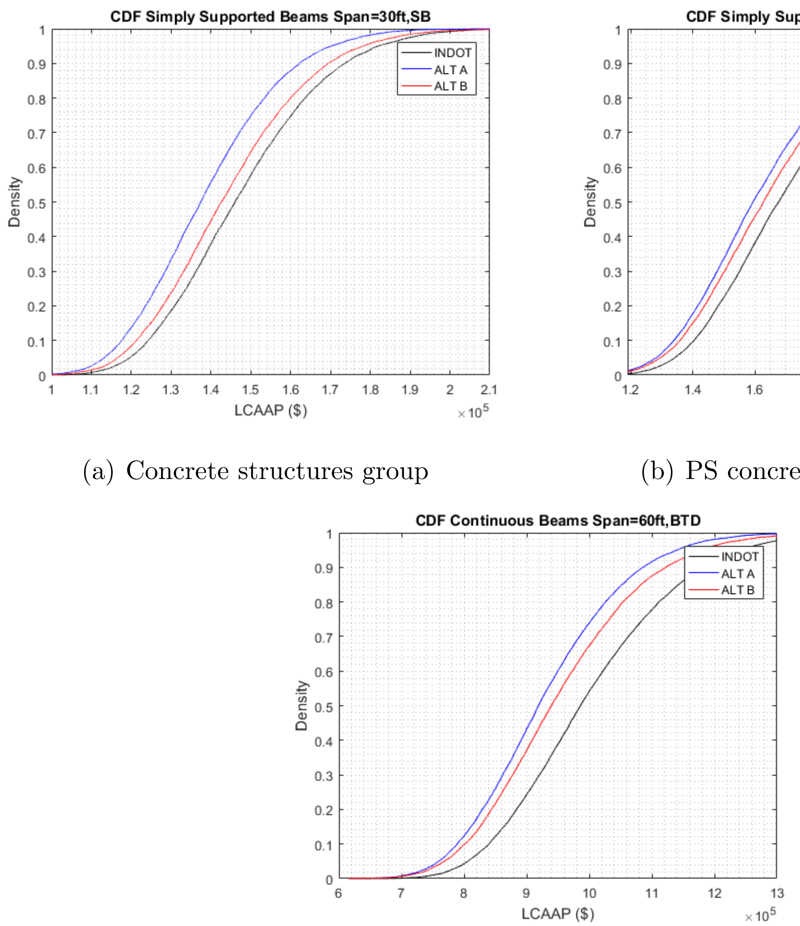
Figures 7.13 and 7.14 present the CDFs for the concrete and structural steel groups respectively. The SD analysis shows that for concrete structures Alt A $\succ$ Alt B $\succ_2$ INDOT, for prestressed concrete structures Alt A $\succ^{almost}{}_1$ Alt B $\succ_2$ INDOT, for prestressed concrete structures with diaphragms Alt A $\succ^{almost}{}_1$ Alt B $\succ$ INDOT,

Table 7.2.
Summary of alternatives considered for LCCP selection

Group	S L (ys)	Option	Working action (years of occurrence)									
			WC	SC	CP	O	BD	DR	BR	FP	SP	SR
Conc. Slab	58	INDOT	Y*	0	-	25,50	SL	-	-	-	-	-
		Alt A	Y*	5**	-	40	SL	-	-	-	-	-
		Alt B	Y*	0	10***	30	SL	-	-	-	-	-
PS Concrete	65	INDOT	Y*	0,45	-	25	SL	45	45	-	-	-
		Alt A	Y*	5**	-	-	SL	40	40	-	-	-
		Alt B	Y*	0,40	10***	-	SL	40	40	-	-	-
PSC Diap.	80	INDOT	Y*	0,40	-	25,65	SL	40	-	-	-	-
		Alt A	Y*	5**	-	-	SL	40	-	-	-	-
		Alt B	Y*	0,40	10***	-	SL	40	-	-	-	-
Steel Paint	80	INDOT	Y*	0,45	-	25,65	SL	45	45	35,65	-	SL
		Alt A	Y*	0,45	-	25,65	SL	45	45	-	10***	SL
		Alt B	Y*	5**	-	-	SL	40	40	30,60	-	SL
		Alt C	Y*	5**	-	-	SL	40	40	-	10***	SL
		Alt D	Y*	0,40	10***	-	SL	40	40	30,60	-	SL
		Alt E	Y*	0,40	10***	-	SL	40	40	-	10***	SL
Steel Galv.	100	INDOT	Y*	0,50	-	25,75	SL	-	50	-	-	SL
		Alt A	Y*	5**	-	-	SL	50	50	-	-	SL
		Alt B	Y*	0,50	10***	-	SL	50	50	-	-	SL
SDCL Pnt.	95	INDOT	Y*	0,50	-	25,75	SL	50	-	35,60,75	-	SL
		Alt A	Y*	0,50	-	25,75	SL	50	-	-	10***	SL
		Alt B	Y*	5**	-	-	SL	50	-	35,60,80	-	SL
		Alt C	Y*	5**	-	-	SL	50	-	-	10***	SL
		Alt D	Y*	0,50	10***	-	SL	-	50	30,60,80	-	SL
		Alt E	Y*	0,50	10***	-	SL	-	50	-	10***	SL
SDCL Galv.	115	INDOT	Y*	0,45,80	-	25,65,100	SL	45,80	-	-	-	SL
		Alt A	Y*	5**	-	-	SL	40,80	-	-	-	SL
		Alt B	Y*	0,40,80	10***	-	SL	40,80	-	-	-	SL

*Working action performed on a yearly basis.
**Working action performed every 5 years until the end of the service life.
***Working action performed every 10 years until the end of the service life.

for painted structural steel bridges Alt C $\succ^{almost}{}_1$ Alt B $\succ^{almost}{}_1$ Alt E $\succ_2$ Alt D $\succ$ Alt A $\succ^{almost}{}_1$ INDOT, for galvanized structural steel bridges Alt A $\succ^{almost}{}_1$ Alt B $\succ_2$ INDOT, for painted SDCL elements Alt C $\succ^{almost}{}_1$ Alt B $\succ^{almost}{}_1$ Alt E $\succ_2$ Alt D $\succ$ Alt A $\succ^{almost}{}_1$ INDOT, and finally, for galvanized SDCL beams Alt A $\succ^{almost}{}_1$ Alt B $\succ_2$ INDOT.



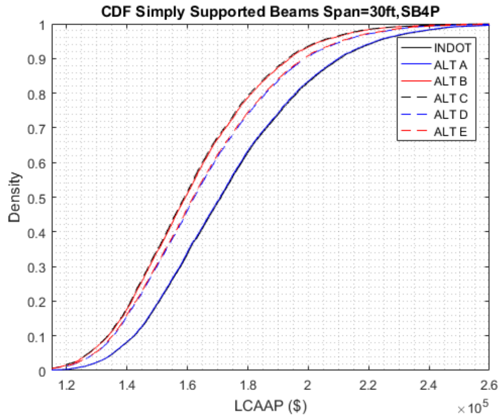
(a) Concrete structures group

(b) PS concrete structures group

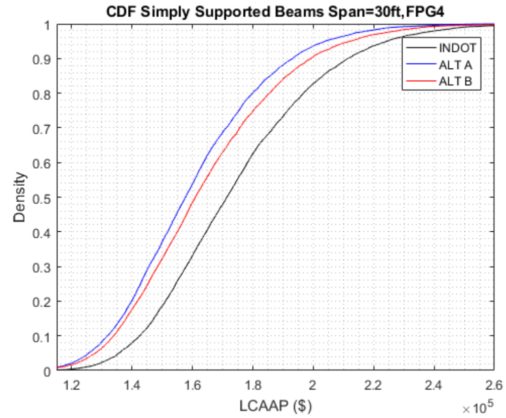
(c) PS concrete structures with diaphragms group

Fig. 7.13. LCCP selection - CDFs concrete groups

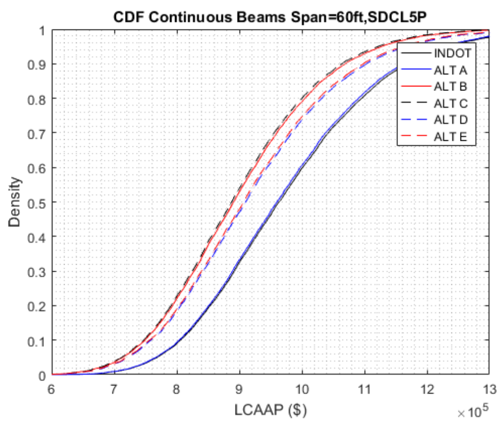
Based on these categorizations of different alternatives, two conclusions can be made. First, for deck treatments, it is more cost effective to sequence of cleaning and sealing of the deck surface periodically (every 5 years) than to perform full depth patching periodically, and the later is more preferable than the actual INDOT practice involving installation of overlays. Second, for steel structures, it is more cost effective to implement periodical spot painting rather than full painting of the complete element. However, if user costs are included, the results could vary depending on the road and traffic conditions inherent to the specific site specifications. Sensitivity analyses were done for each of the groups mentioned before using both, simply



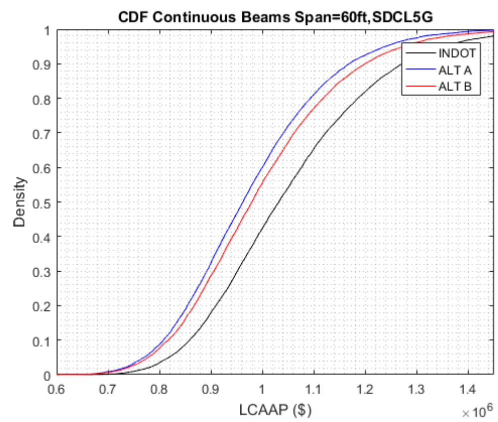
(a) Painted structural steel group



(b) Galvanized structural steel group



(c) Painted SDCL group

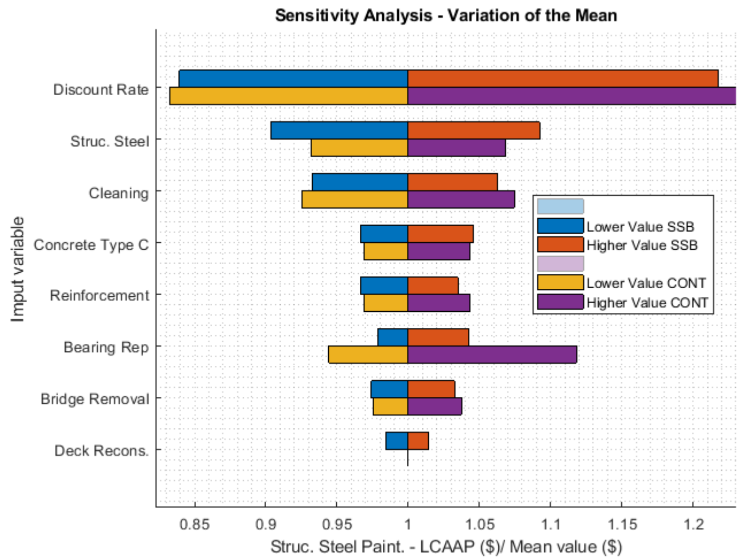


(d) Galvanized SDCL group

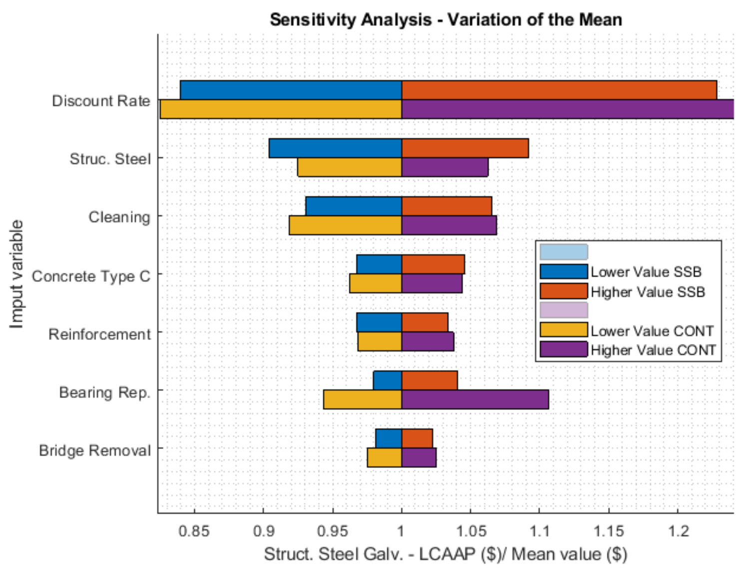
Fig. 7.14. LCCP selection - CDFs structural steel groups

supported and continuous beams results (Figures 7.15 and 7.17). As expected, the discount rate is the variable with the most impact into the results, having an inverse relationship with the LCCA (the higher the discount rate, the lower the LCCA). Additionally, generally speaking, the variables that have more impact in the mean are the ones related with the concrete deck, structural elements (i.e. concrete for slab bridges, prestressed concrete and structural steel), bearing replacement (for the options that consider it)), the washing and cleaning process and deck reconstruction.

These results are the basis for the definitive LCCP used for the Monte Carlo simulation and presented in Chapter 6. After selecting the LCCP for each superstructure

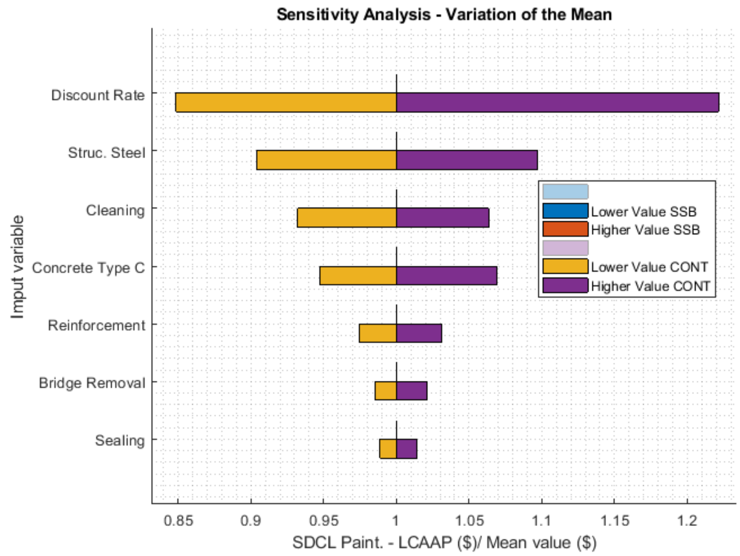


(a) Painted structural steel group

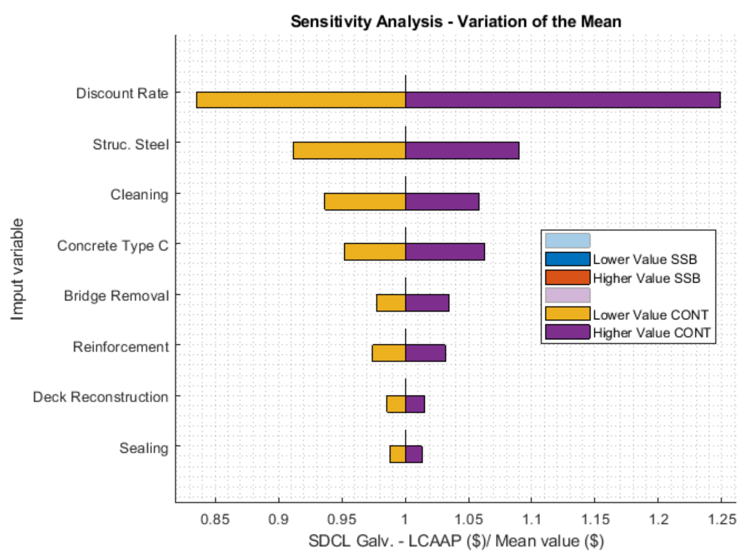


(b) Galvanized structural steel group

Fig. 7.15. LCCP selection - Sensitivity analysis structural steel groups

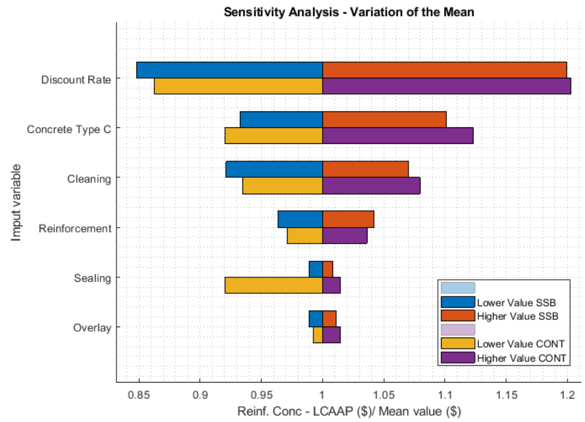


(a) Painted SDCL group

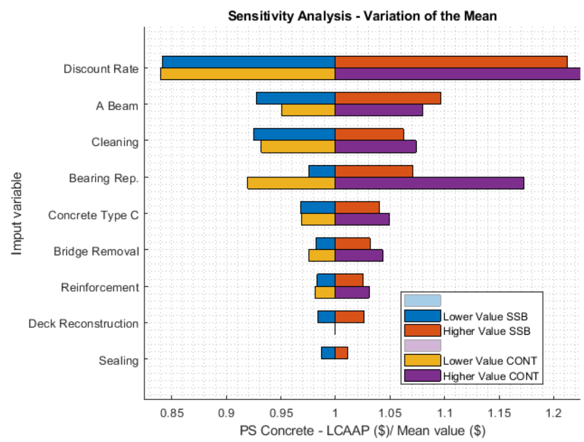


(b) Galvanized SDCL group

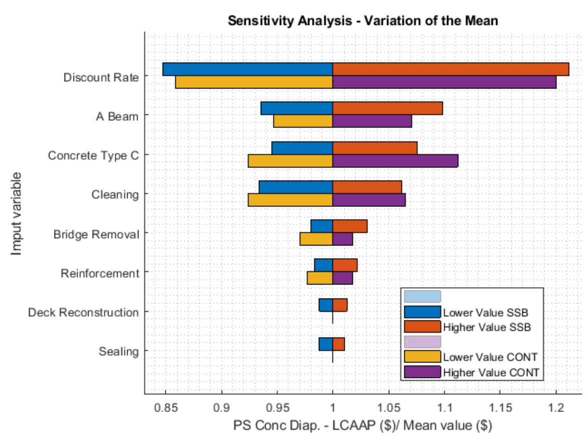
Fig. 7.16. LCCP selection - Sensitivity analysis SDCL groups



(a) Concrete structures group



(b) PS concrete structures group



(c) PS concrete structures with diaphragms group

Fig. 7.17. LCCP selection - Sensitivity analysis concrete groups

type, the simulation for each span length for both simply supported and continuous beams is performed. As stated before, ten thousand (10,000) simulations are used for each one of the cases. However, due to the multiple comparisons needed for each of the span lengths, a graphical representation for each case is insufficient to determine accurately the preference of any alternative among the others. Figure 7.18 shows one example of such graphical representations. To summarize the results, a summary table is used for superstructure selection for each case, an example is given in Table 7.3. In the table, an stochastic matrix selection is presented, in this, each cell shows a set of logical values composed of 4 figures, namely, first, second, almost first and almost second stochastic dominance. As a convention, 0 indicates not dominance of the option shown in the row to the option contrasted in the column, while 1 means dominance in any degree of the row alternative to the column option. For example, for row 6 of Table 7.3, galvanized 5 SDCL beams (SDCL5G), and column 2, prestressed concrete beams with diaphragms (ABD), the logical output is "0-0-1-1", meaning that SDCL5G almost dominates in first degree ABD, ABD $\succ^{almost}{}_1$ SDCL5G. In other words, ABD is most cost-effective than SDCL5G and is more preferable for that specific span length. Results for all cases are presented in Appendix F.

Table 7.3.
Stochastic dominance matrix - Continuous beams, span=75-ft

Alternative	**AB**	**ABD**	**SDCL4P**	**SDCL5P**	**SDCL4G**	**SDCL5G**	**BT**	**BTD**
AB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1
ABD	0-0-1-1	-	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	1-1-1-1
SDCL4P	0-0-1-1	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1
SDCL5P	0-0-1-1	0-0-1-1	0-0-1-1	-	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1
SDCL4G	0-0-1-1	0-0-1-1	1-1-1-1	1-1-1-1	-	0-0-0-0	1-1-1-1	1-1-1-1
SDCL5G	0-0-1-1	0-0-1-1	1-1-1-1	1-1-1-1	0-0-1-1	-	1-1-1-1	1-1-1-1
BT	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0
BTD	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-

The final objective is to find a column that is dominated by every row in the stochastic dominance matrix. In the previous case, the prestressed bulb tee alternative

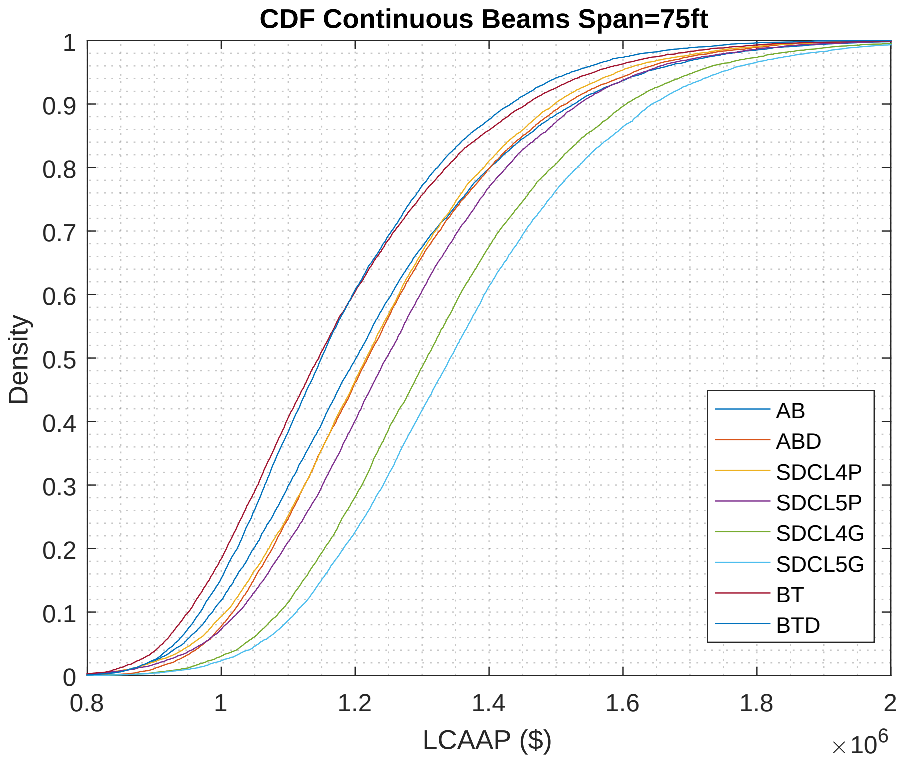


Fig. 7.18. Simulation results, CDFs continuous beams, span=75-ft.

(BT) is dominated by all the other options, then it is the most cost effective option for continuous beams with a 75-ft longer span. As a result of the analysis of the simulation results and the summarization of the stochastic dominance matrices, the superstructure selection for each case is as follows (and is summarized in Figures 7.18 and 7.19):

- Simply Supported Beams:
  - Span=30-ft

    SB $\succ^{almost}{}_1$ SB4G $\succ^{almost}{}_1$ FPG4 $\succ_2$ SB5G $\succ$ SB4P $\succ^{almost}{}_1$ FPG6 $\succ_2$ SB5P $\succ_1$ AB $\succ_2$ ABD $\succ_1$ CB
  - Span=45-ft

    FPG4 $\succ_1$ SB4G $\succ^{almost}{}_1$ SB5G $\succ^{almost}{}_1$ SB4P $\succ^{almost}{}_1$ SB5P $\succ_2$ AB $\succ_2$ SB $\succ_2$ ABD $\succ_1$ CB

- Span=60-ft

  SB4G $\succ^{almost}{}_1$ FPG4 $\succ^{almost}{}_1$ SB5G $\succ^{almost}{}_1$ SB4P $\succ^{almost}{}_1$ SB5P $\succ_2$ BT $\succ_2$ BTD $\succ^{almost}{}_1$ CB $\succ^{almost}{}_1$ AB $\succ_2$ ABD

- Span=75-ft

  BT $\succ_1$ BTD $\succ^{almost}{}_1$ AB $\succ^{almost}{}_1$ SB4G $\succ^{almost}{}_1$ SB4P $\succ_2$ ABD $\succ^{almost}{}_1$ SB5G $\sim$ SB5P

- Span=90-ft

  BT $\sim$ SPG5G $\succ^{almost}{}_1$ SPG5P $\succ^{almost}{}_2$ BTD $\succ^{almost}{}_1$ SB4P $\succ^{almost}{}_1$ SB4G $\succ^{almost}{}_1$ AB $\succ^{almost}{}_1$ SB5P $\succ_2$ SB5G $\succ_2$ ABD

- Span=110-ft

  BT $\succ^{almost}{}_1$ SPG5G $\succ^{almost}{}_1$ SPG5P $\succ_2$ BTD

- Span=130-ft

  BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ SPG5P $\succ_2$ SPG5G

• Continuous Beams

- Three spans, longer span=30-ft

  SB $\succ_1$ CB

- Three spans, longer span=45-ft

  SB4G $\succ^{almost}{}_1$ ABD $\succ^{almost}{}_1$ AB $\succ^{almost}{}_1$ SB5G $\succ^{almost}{}_1$ SB $\succ^{almost}{}_1$ SB4P $\succ^{almost}{}_1$ SB5P $\succ_1$ CB

- Three spans, longer span=60-ft

  SB4G $\succ^{almost}{}_1$ SDCL5P $\succ^{almost}{}_1$ SDCL4P $\sim$ AB $\succ^{almost}{}_1$ ABD $\succ^{almost}{}_1$ SB4P $\succ^{almost}{}_1$ SB5G $\succ^{almost}{}_1$ SB5P $\succ^{almost}{}_1$ BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ SDCL5G $\succ^{almost}{}_1$ SDCL4G $\succ_1$ CB

- Three spans, longer span=75-ft

  BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ AB $\succ^{almost}{}_1$ SDCL4P $\succ^{almost}{}_1$ ABD $\succ^{almost}{}_1$ SDCL5P $\succ_1$ SDCL4G $\succ^{almost}{}_1$ SDCL5G

- Three spans, longer span=90-ft

  BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ AB $\succ^{almost}{}_1$ ABD $\succ^{almost}{}_1$ SPG5G $\succ^{almost}{}_1$ SPG5P $\succ^{almost}{}_1$ SDCL4P $\succ^{almost}{}_1$ SDCL5P $\succ_1$ SDCL4G $\succ^{almost}{}_1$ SDCL5G

- Two spans, equal spans=90-ft

  BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ SPG5G $\succ^{almost}{}_1$ SPG5P

- Two spans, equal spans=110-ft

  BT $\succ^{almost}{}_1$ BTD $\sim$ SPG5G $\succ^{almost}{}_1$ SPG5P

- Two spans, equal spans=130-ft

  BT $\succ^{almost}{}_1$ BTD $\succ^{almost}{}_1$ SPG5P $\succ^{almost}{}_1$ SPG5G

Based on the results obtained through stochastic dominance, it is clear that the most cost effective options implicitly consider the least amount of working actions during their service lives. Specifically, in all the span lengths where structural steel options are the most cost effective, galvanized options are preferred among the painted options. This finding corroborates indirectly the assumptions of not taking into account user costs in the whole analysis.

Generally speaking, it can be inferred that the inclusion of user costs associated with the different working actions, will increase the working actions final cost proportionally with the duration and degree of intervention of the original structure. It is expected that specifically for minor deck interventions, such as washing and cleaning and deck sealing, the user costs will be low compared with the cost associated with major deck interventions such as the construction of overlays or reconstruction of the deck itself. In this sense, if user costs are considered, and an alternative considering overlays are more expensive than one without them, then the difference will increase and the preference of the later option will be reaffirmed. The same logical inference can be applied with other significant working actions. Hence, inclusion of minor working action user costs will not change the superstructure categorization concluded in this chapter.

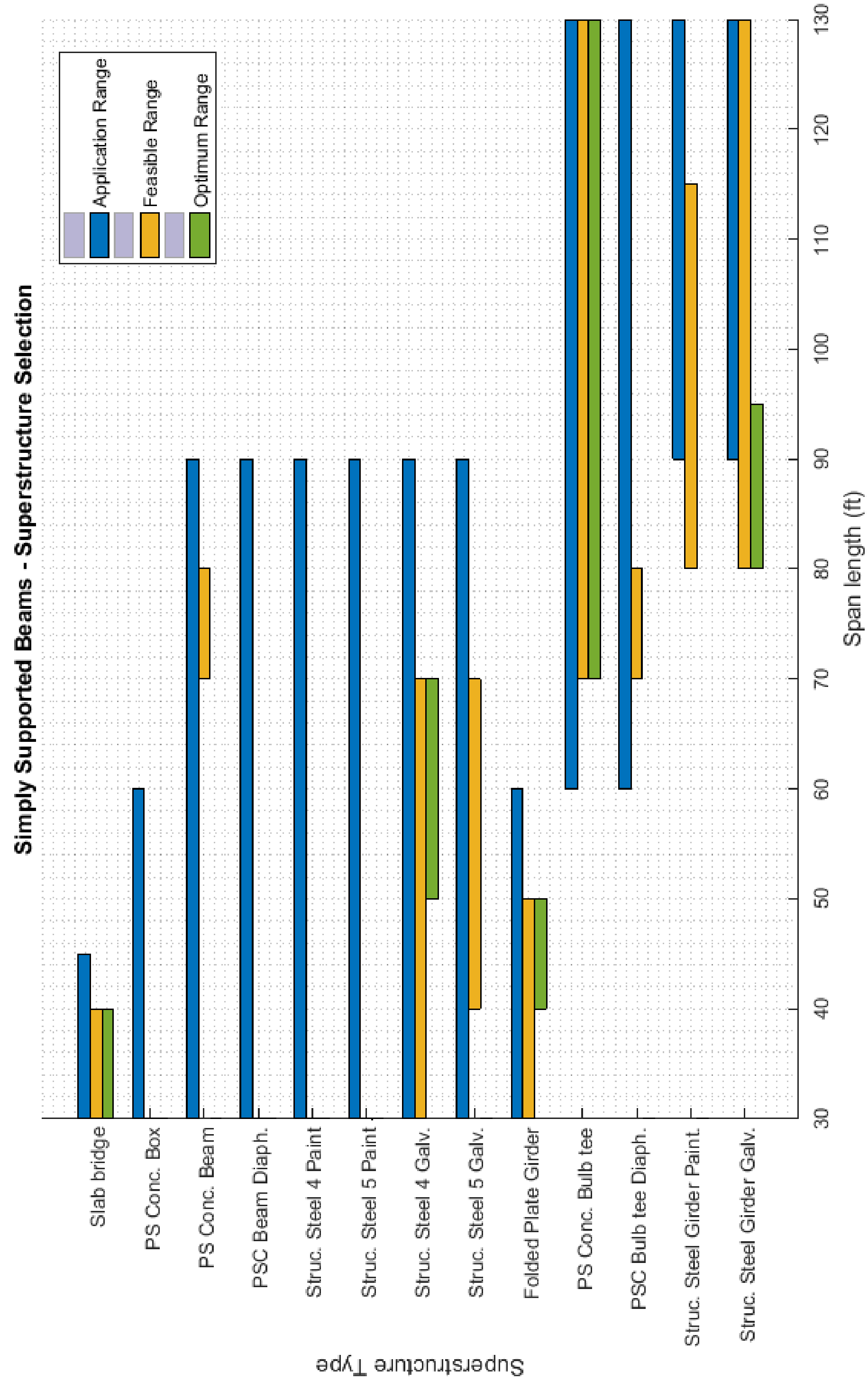


Fig. 7.19. Superstructure selection chart - Simply supported beams

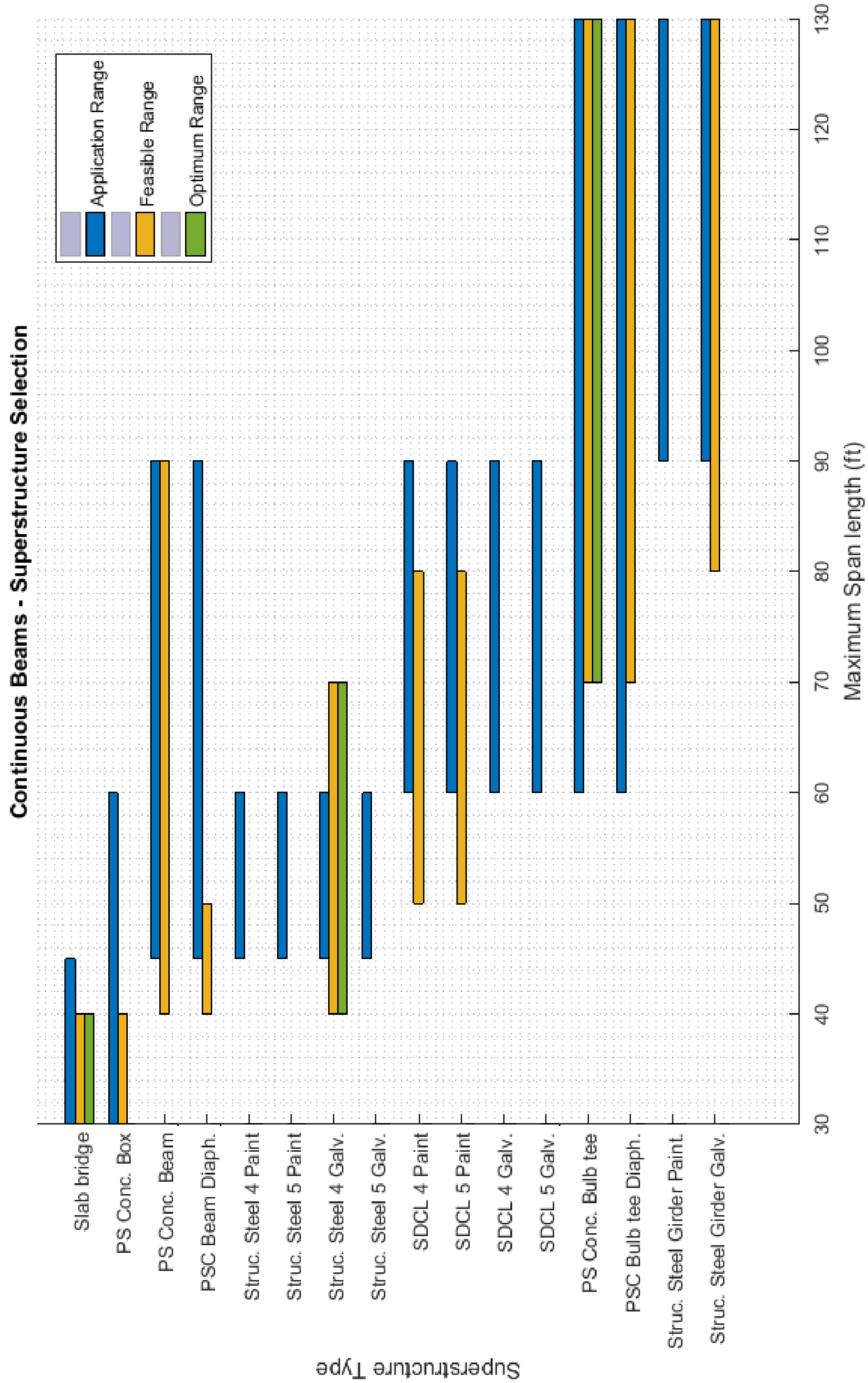


Fig. 7.20. Superstructure selection chart - Continuous beams

Figures 7.19 and 7.20 show the application, feasible and optimum ranges for simply supported and continuous beams respectively. The application range corresponds to the span lengths for the superstructure types originally considered due, to perceived structural efficiency and cost effectiveness (shown in Table 3.1). The feasible range is composed of the top three alternatives for each span length, resulting from the categorization made using stochastic dominance. Finally, the optimum range indicates the most cost effective option for each span length. As it can be seen, prestressed concrete bulb tee are preferable for spans longer than 70-ft for both continuous and simply supported spans. Additionally, slab concrete bridges are also preferable for span lengths shorter than 40-ft for both span configurations. However, structural steel galvanized structures dominates the span lengths between 40-ft and 70-ft for both span distributions.

And special discussion is needed for the SDCL and FPG alternatives. SDCL painted options are categorized in the top 3 feasible options for continuous spans between 50-ft and 70-ft. In this case, galvanized options are not preferred since the extension of the service life includes a second deck reconstruction which impact negatively its final cost effectiveness. Analysis showed that consider a shorter SL (same SL as the painted option) benefits the cost-effectiveness of this alternative, adding it to the feasible set for the applicable span lengths. On the other hand, FPG is the most cost-effective for simply supported beams with lengths between 40-ft and 50-ft. Additionally, it is considered as a feasible option for span lengths up to 50-ft. As mentioned in the deterministic analysis, a more accurate cost estimation of construction costs is needed to validate the viability of this structural system.

Even though comparing the results from the deterministic and the stochastic analyses may appear to suggest the same superstructure selection (see summary Table 7.4), it is important to differentiate the extent of both analyses. The consideration of only average values will be enough for risk-neutral investor. However, decision makers are generally risk-averse and therefore, superstructure selection need to be based on more informed analysis considering different scenarios. This statement is even more relevant for cases in which the deterministic analysis for some span lengths show cost-effectiveness ratios close to each other for different superstructure types. In the summary table, it is shown the top 3 cost-effective alternatives for the stochastic analysis along with the alternatives with a cost-effective ratio less than 1.05 (considered as the decision maker considerable alternatives) for the deterministic analysis. Additionally, the alternatives considered are ranked form most (rank 1) to least cost-effective. As it can be seen, the deterministic analysis results are more vague in terms of superstructure selection, giving a wide variety to the decision maker for the selection, while the stochastic analysis narrow the possibilities, facilitating the superstructure selection.

In order to exemplify this idea, consider the results for continuous beams with a maximum span of 75-ft. Both analyses indicate that prestressed concrete bulb tee beams are the most cost-effective option. Nonetheless, as shown in Figure 7.5(b), the deterministic analysis will not show a clear conclusion, since prestressed concrete beams and prestessed bulb tee beams with diaphragms have cost-effectiveness ratios within a range of 3% above the optimum. Therefore, a decision maker will not have information enough to clearly chose one option above the others. On the other hand, the stochastic analysis consider multiple scenarios that will give the sufficient information to the decision maker to select the most cost-effective option with enough confidence. However, results presented in this research are based on historical data obtained from the contractors database provided by INDOT that includes all bridge construction projects during the last 6 years. Improvements in construction methods for different superstructure types could lead to different results. Also, an update of not only designs but also costs with current data could be beneficial and could strengthen this analysis. If more updated data related to such parameters is available, all the alternatives presented in this document must be included in the future analysis.

Finally, it is important to underline that user costs need to be included if specifics on the project location are known, even though it was shown that inclusion of such in

Table 7.4.
Results summary - Deterministic and stochastic analysis comparison

Span Length (ft)[1]	**Deterministic Analysis**		**Stochastic Analysis**	
	SSB	**CONT**	**SSB**	**CONT**
30	1. SB	1. SB	1. SB	1. SB
	2. SB4G	-	2. SB4G	2. CB
	3. FGP4	-	3. FGP4	-
45	1. FPG4	1. SB4G	1. FPG4	1. SB4G
	2. SB4G	2. AB	2. SB4G	2. ABD
	3. SB5G	3. ABD	3. SB5G	3. AB
	-	4. SB5G	-	-
	-	5. SB	-	-
	-	6. SB4P	-	-
	-	7. SB5P	-	-
60	1. SB4G	1. SB4G	1. SB4G	1. SB4G
	2. FPG4	2. AB	2. FPG4	2. SDCL5P
	3. SB5G	3. SDCL5P	3. SB5G	3*. SDCL4P
	4. SB4P	4. SDCL5P	-	3*. AB
	5. BT	5. ABD	-	-
	6. SB5P	6. SB4P	-	-
	-	7. SB5G	-	-
75	1. BT	1. BT	1. BT	1. BT
	2. AB	2. BTD	2. BTD	2. BTD
	3. BTD	3. AB	3. AB	3. AB
	4. SB4G	4. SDCL4P	-	-
	-	5. ABD	-	-
90	1. BT	1. AB	1*. BT	1. BT
	2. SPG5G	2. BT	1*. SPG5P	2. BTD
	3. SPG5P	3. BTD	2. SPG5P	3. AB
	4. BTD	4. ABD	-	-
	-	5. SPG5G	-	-
	-	6. SPG5P	-	-
90-90	-	1. BT	-	1. BT
	-	2. BTD	-	2. BTD
	-	-	-	3. SPG5G
110	1. BT	1. BT	1. BT	1. BT
	2. SPG5G	2. BTD	2. SPG5G	2*. BTD
	3. SPG5P	3. SPG5G	3. SPG5P	2*. SPG5G
	4. BTD	4. SPG5P	-	-
130	1. BT	1. BT	1. BT	1. BT
	2. BTD	2. BTD	2. BTD	2. BTD
	-	-	3. SPG5P	3. SPG5P

[1] Maximum span length for continuous beams (CONT)
* No stochastic dominance between the two options

the working actions costs will not change the outcome of this study, they could have an important impact for computation of construction and removal costs.

8. SUMMARY AND CONCLUSIONS

8.1 Summary

A literature review was presented on innovative cost effective solutions for short to medium span bridges, deterioration curves and current approaches taken to conduct a Bridge Life Cycle Cost Assessment. Additionally, information obtained from the National Bridge Inventory (NBI) was used to summarize the most common structures within the state and generate a design plan for the structures to analyze. Designs covered the most common structures found in Indiana along with the innovative bridge systems presented in this document. Bridge types used are: slab bridges (constant thickness), prestressed concrete box beams, concrete AASHTO beams, concrete bulb tees, structural steel folded plate beams, rolled steel beams, steel plate girders, and finally, structural steel SDCL beams.

Three different span ranges were selected for further study. Range 1 includes bridges with spans between 30-ft and 60-ft. Range 2 included spans between 60-ft and 90-ft. Finally, Range 3 included span lengths between 90-ft to 130-ft. Design types were considered depending on their cost-effectiveness potential for each of the span ranges. Spread sheets that include applicable sections of the LRFD and the Indiana Design Manual specifications were created for every design option. The spread sheets were then used to develop designs for a *"typical"* bridge selection in the appropriate span ranges

Extensive cost allocations for agency costs were presented, including not only initial costs involved but also long-term costs depending on the material and superstructure type considered. No contingencies other than regular deterioration of the bridge were considered. However, it should be mentioned that maintenance or rehabilitation activities may affect user costs. Nonetheless, and in order to compute those costs, a thorough understanding of the traffic (quantities and type of vehicles), detour lengths, travel times and travel velocities is needed. As specified in this document, all bridge designs have no specific location along any specific road. In other words, traffic, velocity and detour assumptions are not made. Additionally, such assumptions are considered an oversimplification of the problem and could impact negatively the outcome of the LCCA comparison. Additionally, historical data were used to determine the probability distribution that better fit the data for each pay item considered. This information is crucial for the accurate stochastic analysis made for each superstructure type. A method of classification is presented based on the Kolmogorov-Smirnoff test and the Anderson-Darling test. For pay items without historical data available, a PERT distribution was specified with lower and maximum probable values equal to, respectively, 75% and 125% of the expected value. Finally, full parameterized probability distributions for each pay item are presented.

Deterioration curves for the Indiana state highway system from work conducted by Moomen et al. [17], Sinha et al. [20] and Cha et al. [38] were used to obtain the service lives for each alternative. Additionally, and considering the working actions along with the service life for each alternative, different LCCPs were proposed and the most cost-effective were used for the LCCA comparison for each superstructure type analyzed. In addition to the regular superstructure options described before, prestressed beam alternatives including integral abutments and intermediate diaphragms, as well as galvanized structural steel beams were considered, including the equivalent extension of the service life of each option. In order to compare all the alternatives considered, a life cycle present worth in perpetuity method is used. Two different approaches were utilized for the analysis, a deterministic and a stochastic method.

For the deterministic analysis, initial cost and LCCA comparison for all span ranges of simply supported beams and continuous beams are presented. It was shown that the inclusion of long term-costs using LCCA generally reduces the cost-effectiveness difference between all the alternatives for the same span length. This reduction could be an important factor if specific site conditions are considered during the analysis. If specific site conditions are known, multiple options for each span length must be considered before choosing the best alternative.

As mentioned before, deterministic analysis reduced the cost-effectiveness difference between all the alternatives. Under those circumstances, and considering only average values, only risk-neutral investors would take decision or whether use and alternative or not. Nonetheless, decision makers in general are risk-averse, which requires consideration of multiple scenarios in order to make a superstructure selection. Thereupon, the stochastic analysis is used to take more informed based superstructure selections.

A Monte Carlo simulation was used to consider multiple scenarios. Specifics on the method and assumptions made are presented. A latin hypercube sampling system is selected and ten thousand (10,000) simulations are considered for the analysis. Cumulative density functions are then obtained for every scenario used. The results of the simulation are then analyzed using the stochastic dominance principles. First, second, almost first and almost second stochastic dominance rules are explained and used for the categorization of the different superstructure types for each span length. As a result, the optimal superstructure option selected is the one that is dominated by all the other alternatives, composing in that way the efficient set and relegating the other options to the inefficient set.

Finally, charts showing the application, feasible and optimum ranges for simply supported and continuous beams are presented. The application range corresponds to the span lengths which the superstructure type is originally considered and structural efficient and cost effective possible (shown in Table 3.1). The feasible range is composed by the top three alternatives for each span length, resulted from the stochastic dominance analysis. Finally, the optimum range indicates the most cost effective option for each span length.

As a recommendation to practitioners, it is important to underline that user costs need to be included if specifics on the project location are known, even though it was shown that inclusion of such in the working actions costs will not change the outcome of this study, they could have an important impact for computation of construction and removal costs that need to be addressed.

8.2 Conclusions

Based upon the analysis conducted herein the following conclusions can be stated:

- The deterministic analysis, explicitly for simply supported beams, showed that for Span Range 1 the slab bridge is the most cost-effective solution either considering or not considering long-term costs for spans less than 45-ft. However, for spans longer than 35-ft, the inclusion of galvanized steel structures - specifically the four (4) beam configuration - provided the most cost-effective alternative. For Span Range 2, four (4) galvanized rolled steel beams are still cost-effective for spans shorter than 65-ft, while the prestressed concrete bulb tees became the optimal solution for longer spans. Additionally, Span Range 3 results show that including long-term costs suggests multiple cost-effective design solutions for spans up to 105-ft, with prestressed concrete bulb tees and galvanized steel plate girders being the two optimal solution. Beyond this point, bulb tees are the most cost effective solution.

- For continuous beams again for the deterministic analysis, it is shown for Span Range 1 that slab bridges and galvanized steel continuous beams are again the most cost effective solutions for the lower and upper parts of the span range, respectively. However, prestressed concrete AASHTO beams are also a competitive option for spans between 45 and 60-ft. In contrast, Span Range 2 suggests that the cost-effectiveness of the SDCL system for spans up to 90-ft is not considered as the optimum, however, it is among the most cost-effective alternatives. Additionally, it is noticeable that prestressed bulb tees and AASHTO beams become more attractive for longer spans. Finally, for Span Range 3, no variance in the cost-effectiveness of the bulb tee option is noticed between the initial cost comparison and the inclusion of long-term costs.

- The stochastic analysis showed that the most cost effective options implicitly consider the least amount of working actions during their service lives. Specifically, in all the span lengths where structural steel options are the most cost effective, galvanized options are preferred among the painted options. This finding corroborates indirectly the assumptions of not taking user costs into account in the analysis. it can be inferred that the inclusion of user costs associated with the different working actions, will increase the working actions final cost proportionally with the duration and degree of intervention of the original structure. Thus, if user costs are considered, and an alternative considering more intrusive actions is most costlier than one without them, then, the difference will increase and the preference of the later option will be reaffirmed. Consequently, inclusion of working action user costs will not change the superstructure categorization made.

- Stochastic analysis also showed that prestressed concrete bulb tee are preferable for spans longer than 70-ft for both continuous and simply supported spans. Additionally, slab concrete bridges are also preferable for span lengths shorter than 40-ft for both span configurations. However, structural steel galvanized structures dominates the span lengths between 40-ft and 70-ft for both span distributions. These charts and the information provided could help designers to chose the most cost effective alternative for a given span length based on LCCA instead of the initial cost approach, which is the base of the current procedure.

- Strictly speaking, results obtained form both analyses suggest the same superstructure selection (see summary Table 7.4) for every span length, however, it is important to remark that the consideration of only average values will be enough for risk-neutral investor. However, decision makers are generally risk-averse and therefore, superstructure selection need to be based on more informed analysis considering different scenarios, which are included in the stochastic analysis.

- Finally, the deterministic analysis results are more vague in terms of superstructure selection, giving a wide variety to the decision maker for the final selection, while the stochastic analysis narrow the possibilities, facilitating the superstructure choosing.

8.3 Future Work

Even though the superstructure selection presented in this document can be considered as an starting point to more informed base superstructure selection process, it also true that many others consideration and scenarios are needed to enrich the findings of this research. Some ideas fr future work based in this research are summarized herein:

i This study was based on bridge structures included in the National Highway System (NHS), locally owned or minor road structures could be used to expand this study. Inclusion of such structures may change not only the *"typical"* structures considered and span configurations but also the deterioration curves due to change in traffic conditions. As a consequence, working actions for such structures may vary as well as their periodicity, factors that could impact greatly the outcome of the BLCCA method.

ii Even though user costs consideration may not change the superstructure selection, their consideration could be beneficial to strengthen the conclusions of this research. To do so, a geographical division could be implemented (i.e. northern, central and southern Indiana regions) to simplify the assumptions on traffic. detours and truck composition upon many other factors.

iii This research was focused on Indiana infrastructure database, equivalent analysis could be made for other states, with the likelihood of obtaining different results. However, specifically for the pay item probability function selection, it could be possible that trends on specific PDFs depending on certain working actions could be analyzed as a regional behavior for colliding states. This could be beneficial for future studies, facilitating the assumptions made by the researcher.

iv The prediction of the service lives used in this document is based on historical data obtained from the NBI database. However, no information can be extracted regarding the intervention and maintenance of the existing bridges. Additional information of the history of existing bridges is needed in order to compute the actual bridge age for each condition rating record. This corrected age could change the prediction of the service life of different superstructure alternatives and then different superstructure selection results could be expected. Also, implementation of bridge element based inspections historical data could also change not only the service life of the alternatives but also the distribution and scheduling of different working actions. Additionally, since the NBI database relays on visual inspections that are subjective, inclusion of non-destructive testing complementary to the visual inspection could be beneficial to decrease its variability and the subsequent determination of deterioration curves needed. Studies evaluating the benefits not only on the accuracy of the deterioration curves but also possible economic benefits due to its implementation should be considered.

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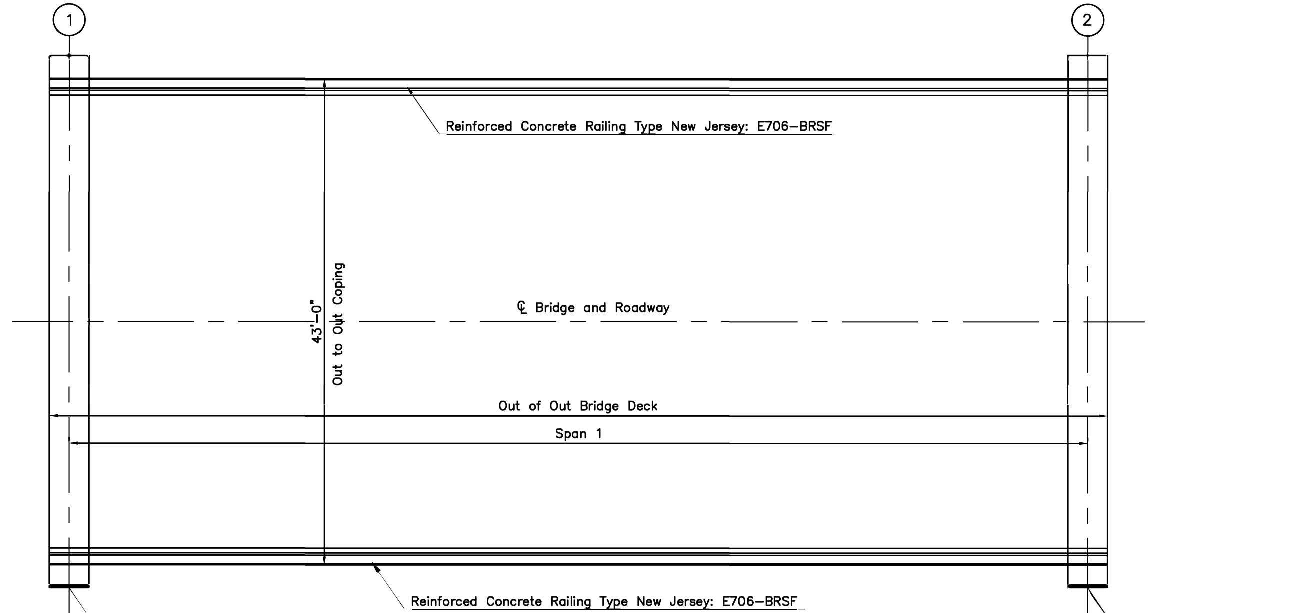
APPENDICES

A. BRIDGE DESIGN DRAWINGS

Drawings presented herein corresponded to the bridge designs developed for each of the options considered in the design plan. This involved the design of 64 bridges in total. Comparable design details were developed for each of the other options in the design plan. Summary information from the designs can be found in the design drawings in Appendix A. The detailed spread sheet designs for each bridge are available by request.



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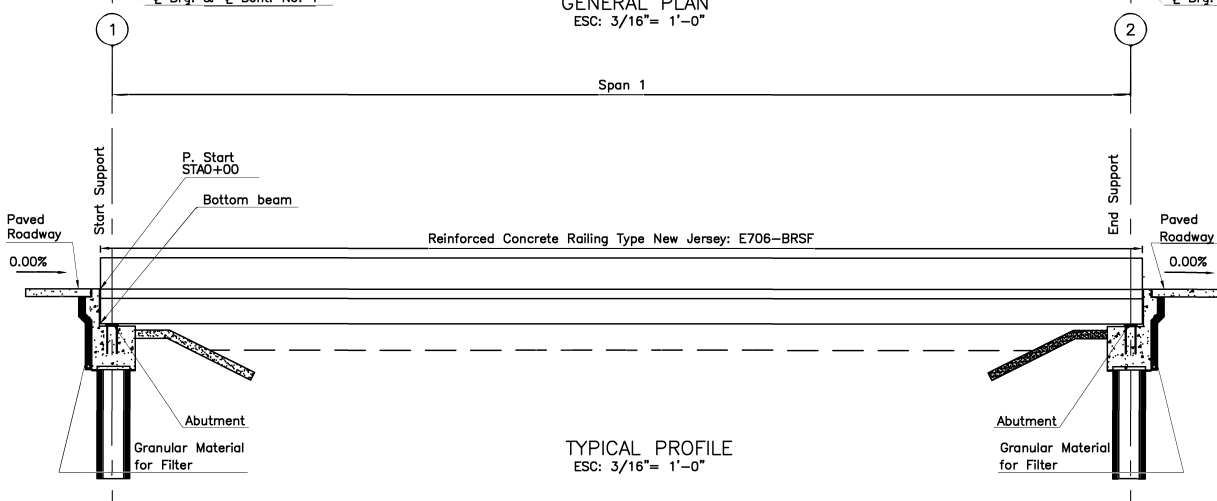

TYPICAL PROFILE
ESC: 3/16"= 1'-0"

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SIMPLY SUPPORTED BEAMS CONFIGURATION

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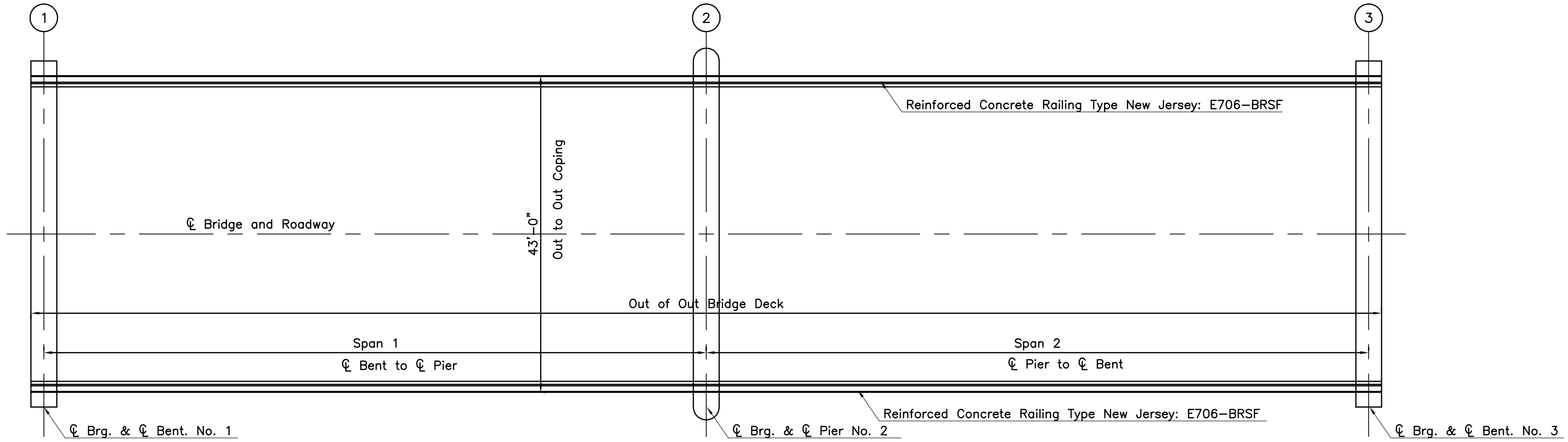
DESIGNED: SLLM — DRAWN: CJK
CHECKED: MDB — CHECKED: SLLM

INDIANA
DEPARTMENT OF TRANSPORTATION

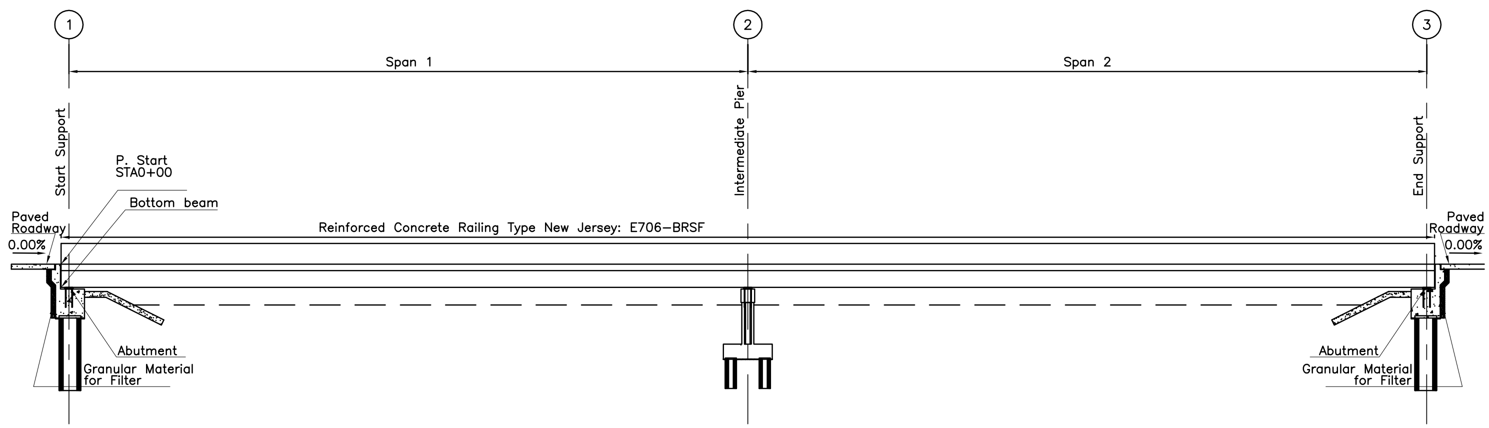
GENERAL PLAN

SCALE: AS NOTED
BRIDGE FILE
DESIGNATION
PAGE 1 of 12



GENERAL PLAN
ESC: 1/8"= 1'–0"

PLAN
Scale: $\frac{1}{8}$" = 1'-0"



TYPICAL PROFILE
ESC: 1/8"= 1'–0"

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TWO SPAN CONFIGURATION

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GENERAL PLAN

SCALE	BRIDGE FILE
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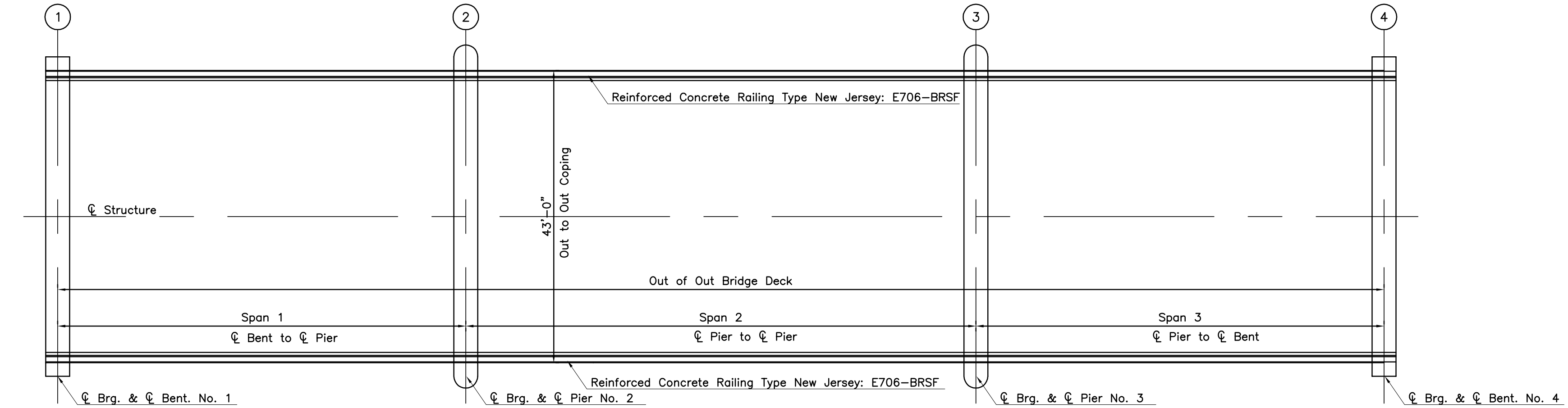
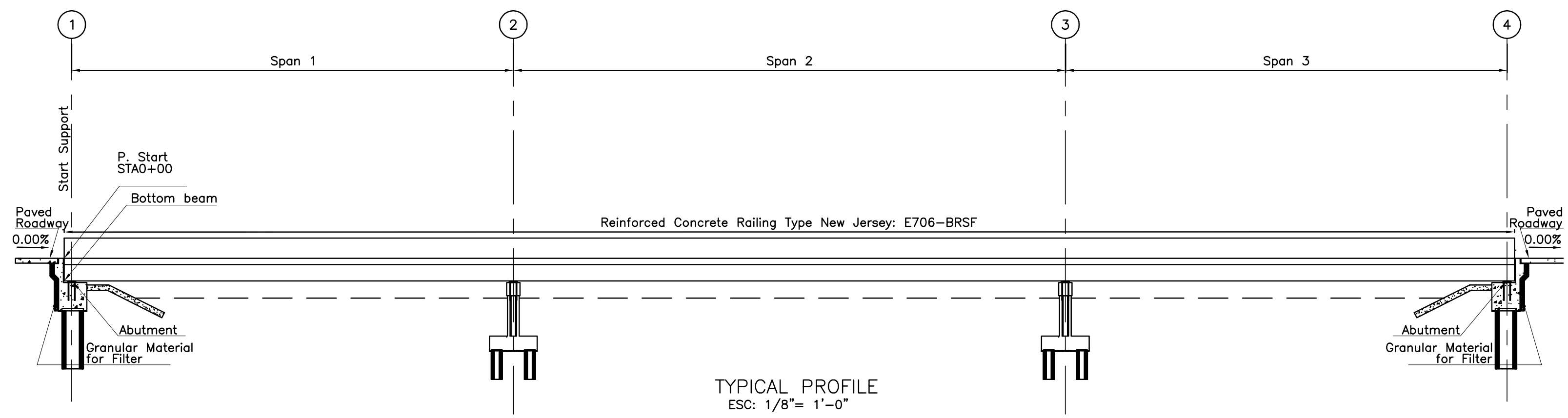
	PAGE 2 of 12





TYPICAL PROFILE
ESC: 1/8"= 1'-0"

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THREE SPAN CONFIGURATION

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GENERAL PLAN

SCALE: AS NOTED
BRIDGE FILE
DESIGNATION
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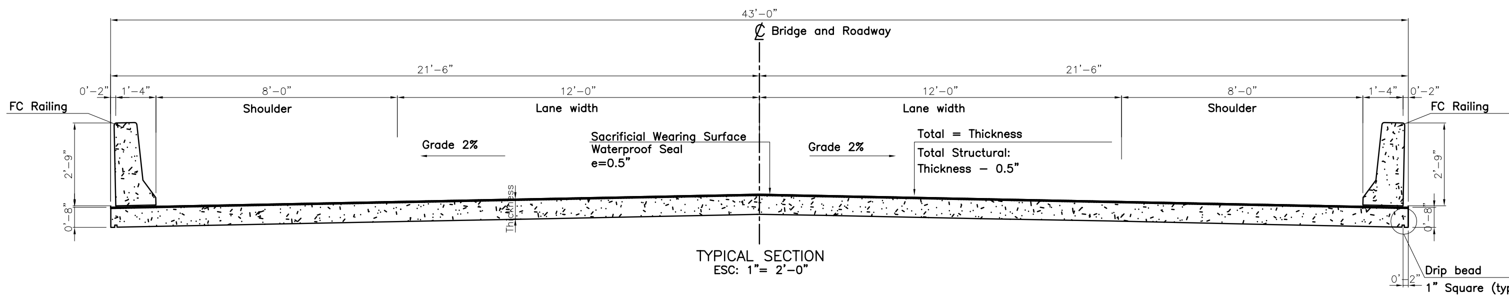

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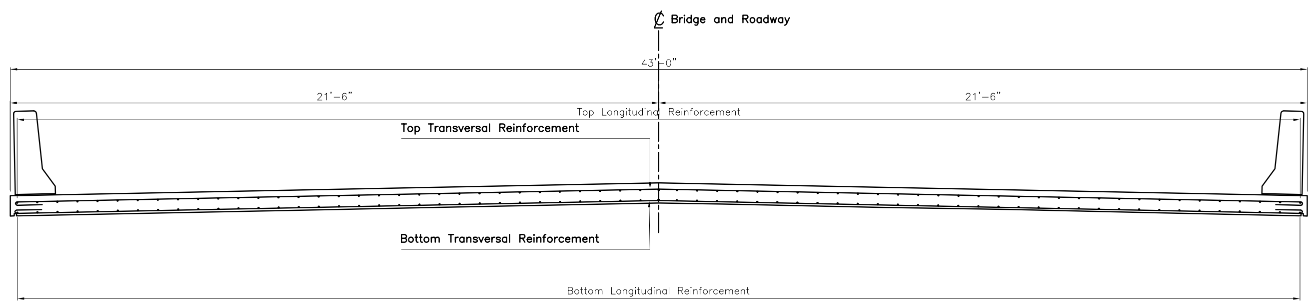

TYPICAL SECTION
ESC: 1"= 2'-0"

REINFORCED CONCRETE SLAB BRIDGE SUPERSTRUCTURE DESIGNS								
Total Thickness (in)	Number of Spans	Span Length(s) (ft)			Top Reinforcement		Bottom Reinforcement	
		Span 1	Span 2	Span 3	Longitudinal	Transversal	Longitudinal	Transversal
$17\frac{1}{2}$	1	30	-	-	# 5 @ 8"	# 5 @ 8"	# 8 @ 5"	# 5 @ 8"
24	1	45	-	-	# 5 @ 8"	# 5 @ 8"	# 9 @ 4"	# 5 @ 8"
14	3	25	30	25	#8 @ 6" + #6 @ 6" (Over Sup)	# 5 @ 6"	# 7 @ 6"	# 5 @ 6"
19	3	35	45	35	#9 @ 6" + #9 @ 6" (Over Sup)	# 5 @ 5"	# 8 @ 5"	# 5 @ 5"

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SLAB BRIDGES

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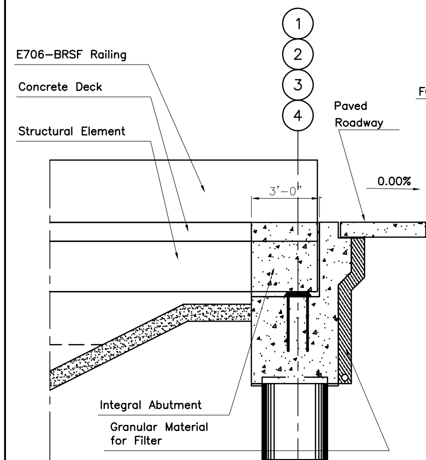
CHECKED: TRB CHECKED: SLLM

INDIANA
DEPARTMENT OF TRANSPORTATION

GENERAL DETAILS

SCALE: AS NOTED
BRIDGE FILE
DESIGNATION
PAGE 4 of 12



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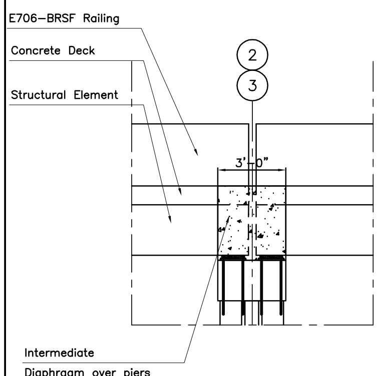

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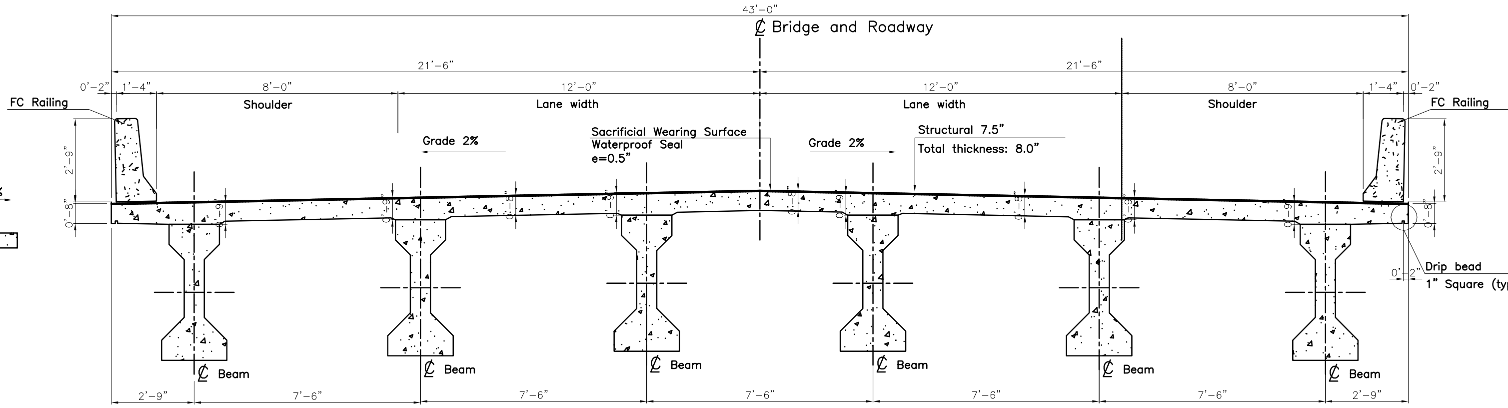

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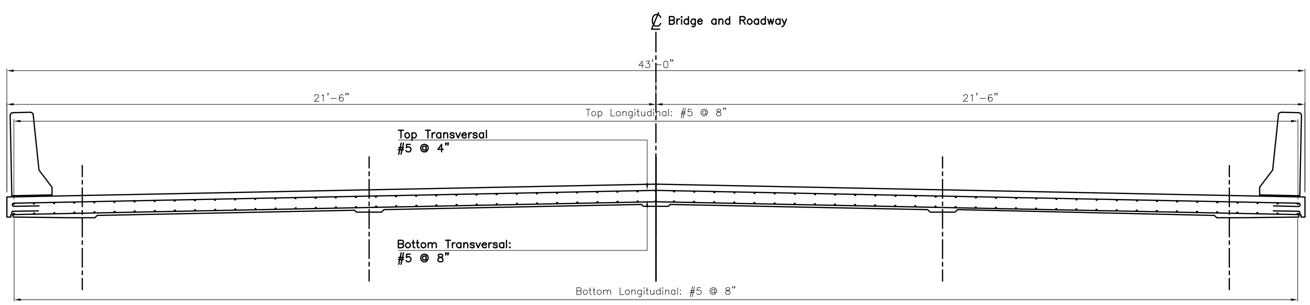
ESC: 1"= 2'–0"



TYPICAL SECTION
ESC: 1"= 2'–0"

PRESTRESSED CONCRETE AASHTO I-BEAM SUPERSTRUCTURE DESIGNS

Girder Type	Number of Spans	Span Length(s) (ft)			No. of Girders
		Span 1	Span 2	Span 3	
A Type I	1	30	-	-	6
A Type II	1	45	-	-	6
A Type III	1	60	-	-	6
A Type III	1	75	-	-	6
A Type IV	1	90	-	-	6
A Type I	3	35	45	35	6
A Type II	3	50	60	50	6
A Type III	3	60	75	60	6
A Type III	3	70	90	70	6

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PRESTRESSED CONCRETE AASTHO BEAMS

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DESIGNED: SLLM — DRAWN: CJK
CHECKED: MDB — CHECKED: SLLM

INDIANA
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GENERAL DETAILS

SCALE: AS NOTED — BRIDGE FILE — DESIGNATION

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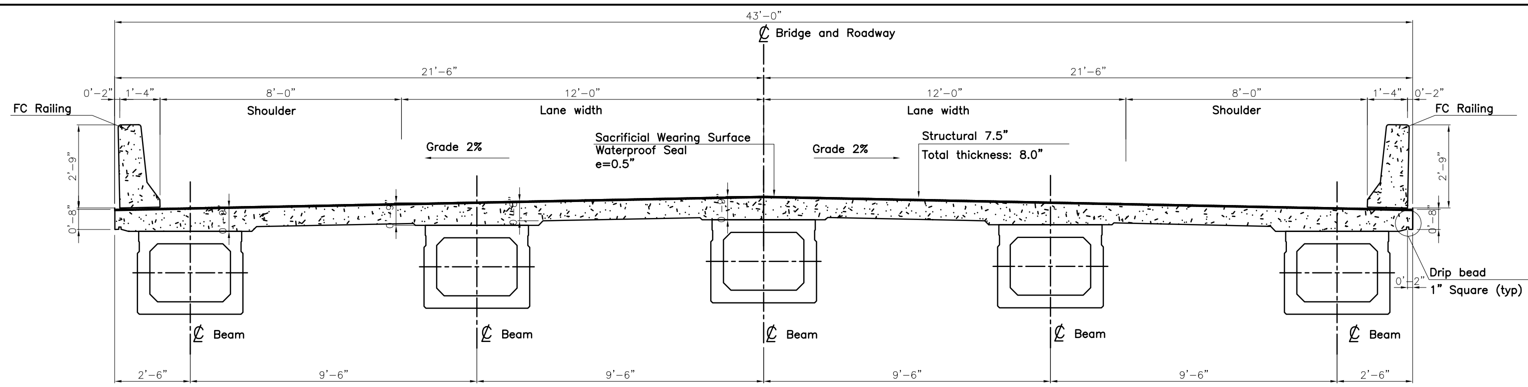

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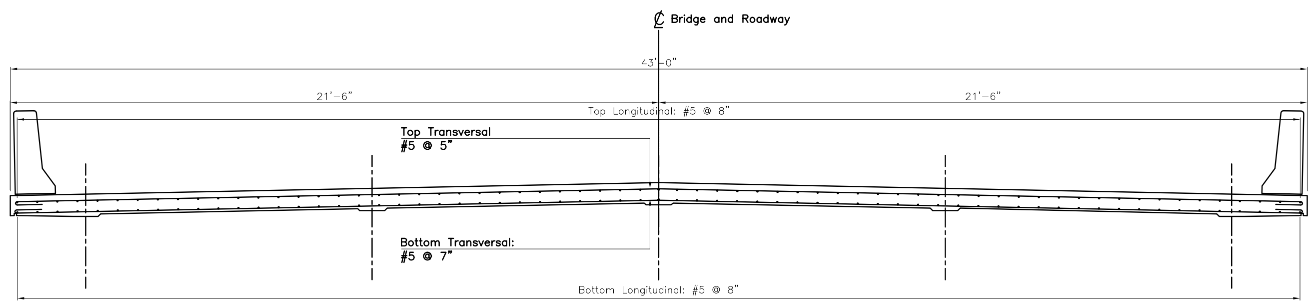
ESC: 1"= 2'-0"



TYPICAL SECTION
ESC: 1"= 2'-0"

PRESTRESSED CONCRETE BOX BEAM SUPERSTRUCTURE DESIGNS					
Girder Type	Number of Spans	Span Length(s) (ft)			No. of Girders
		Span 1	Span 2	Span 3	
CB 17x48	1	30	-	-	5
CB 27x48	1	45	-	-	5
CB 27x48	1	60	-	-	5
CB 17x36	3	25	30	25	5
CB 21x48	3	35	45	35	5
CB 27x48	3	50	60	50	5

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PRESTRESSED CONCRETE BOX BEAMS

RECOMMENDED FOR APPROVAL Stefan Leonardo Leiva Maldoando Winter/2018
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DESIGNED: SLLM DRAWN: CJK
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INDIANA
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GENERAL DETAILS

SCALE AS NOTED
BRIDGE FILE
DESIGNATION
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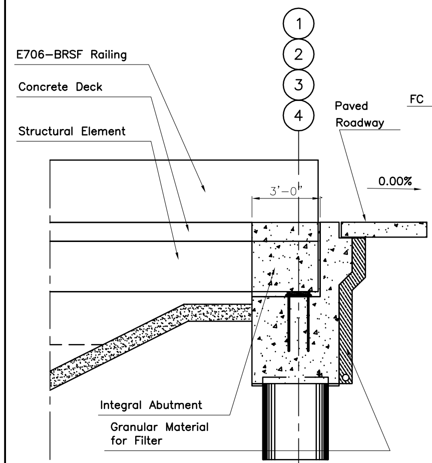

TYPICAL SECTION
INTEGRAL ABUTMENT
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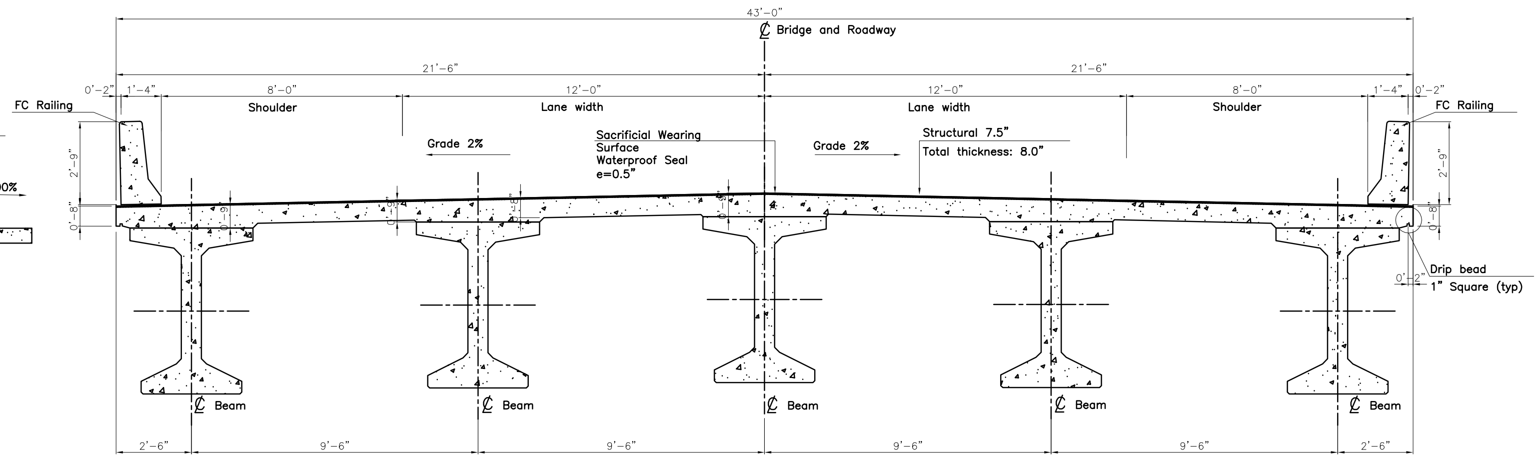

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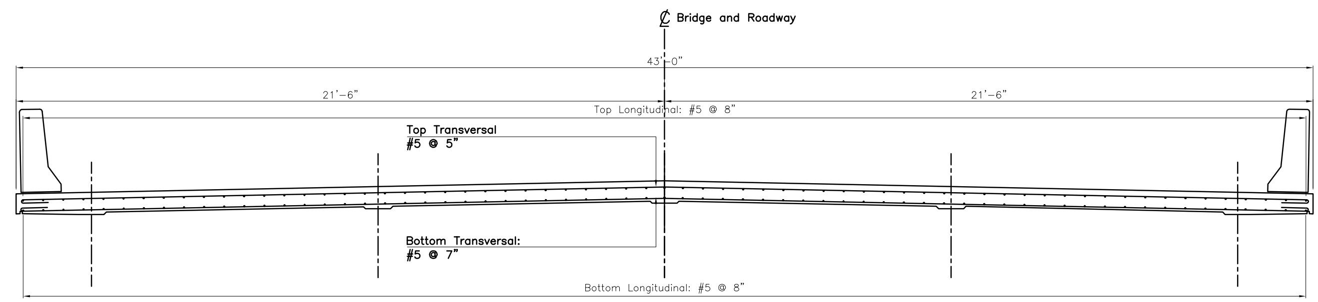

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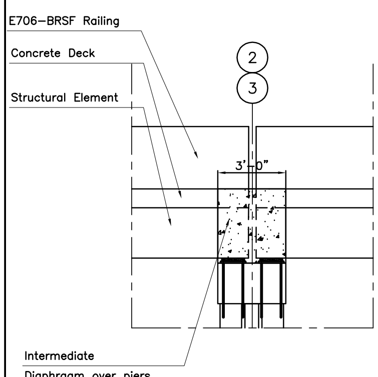

TYPICAL SECTION
INTERMEDIATE DIAPHRAGMS
ESC: 3/8"= 1'–0"

PRESTRESSED CONCRETE BULB TEE BEAM SUPERSTRUCTURE DESIGNS					
Girder Type	Number of Spans	Span Length(s) (ft)			No. of Girders
		Span 1	Span 2	Span 3	
HBT 36x49	1	60	-	-	5
HBT 42x49	1	75	-	-	5
HBT 48x49	1	90	-	-	5
HBT 60x49	1	110	-	-	5
HBT 72x49	1	130	-	-	5
HBT 48x49	2	90	90	-	5
HBT 66x61	2	110	110	-	5
HBT 72x61	2	130	130	-	5
HBT 36x49	3	50	60	50	5
HBT 36x49	3	60	75	60	5
HBT 48x49	3	70	90	70	5

E706–BRSF Railing

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PRESTRESSED CONCRETE BULB TEE

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INDIANA
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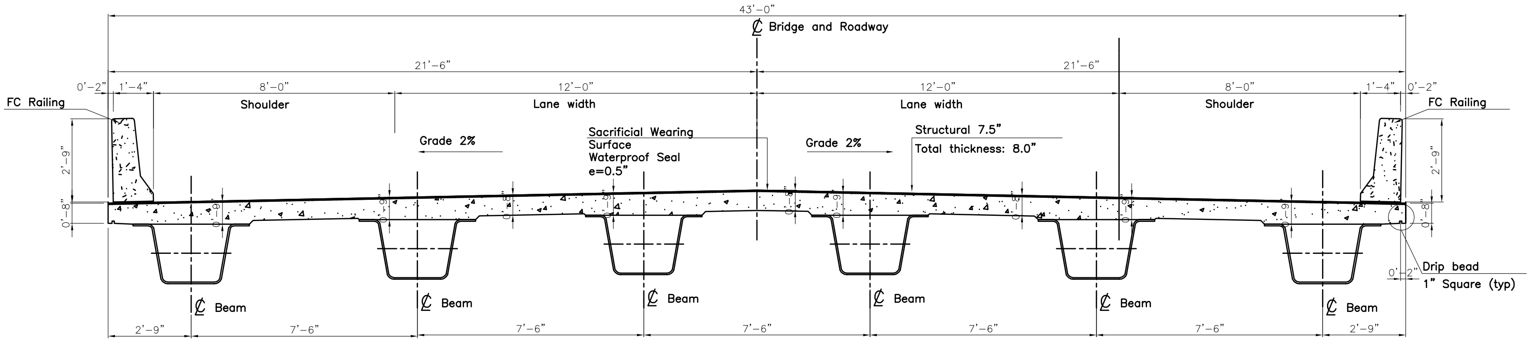
GENERAL DETAILS

SCALE: AS NOTED
BRIDGE FILE
DESIGNATION
PAGE 7 of 12



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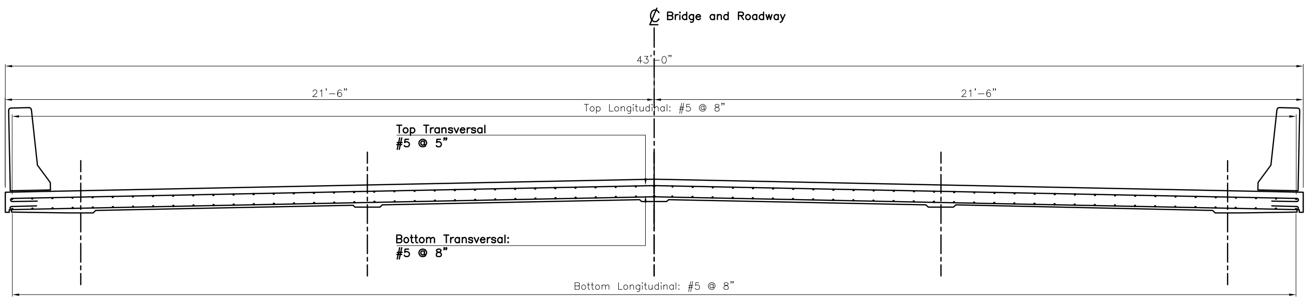

TYPICAL SECTION
ESC: 1"= 2'-0"

FOLDED STEEL PLATE GIRDER SUPERSTRUCTURE DESIGNS			
Girder Type	Number of Spans	Span Length (ft)	No. of Girders
FP 60x12x$\frac{1}{2}$	1	30	6
FP 84x23x$\frac{7}{16}$	1	45	6
FP 96x26x$\frac{7}{16}$	1	60	6

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STRUCTURAL STEEL FOLDED PLATE GIRDERS
(FPG)
6 BEAMS CONFIGURATION

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GENERAL DETAILS

SCALE: AS NOTED
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DESIGNATION
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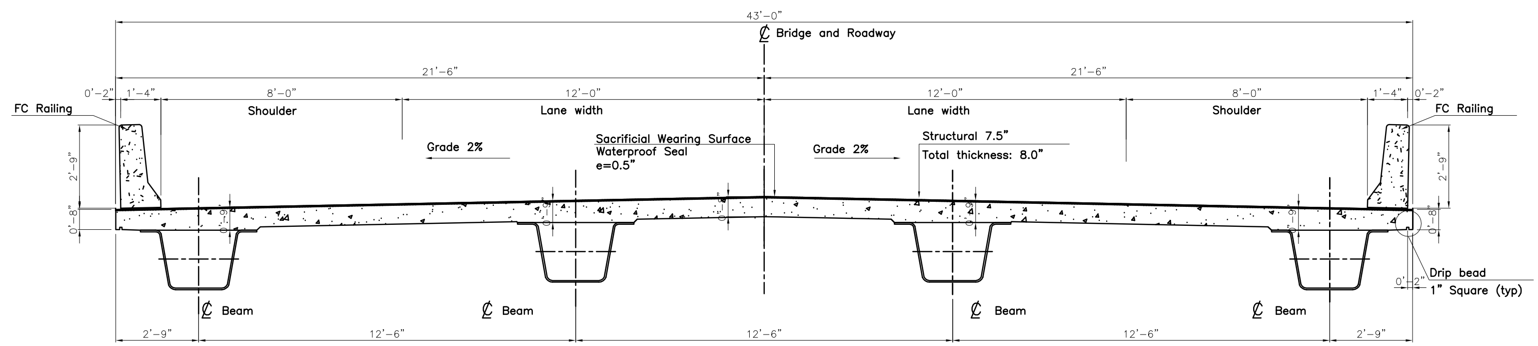
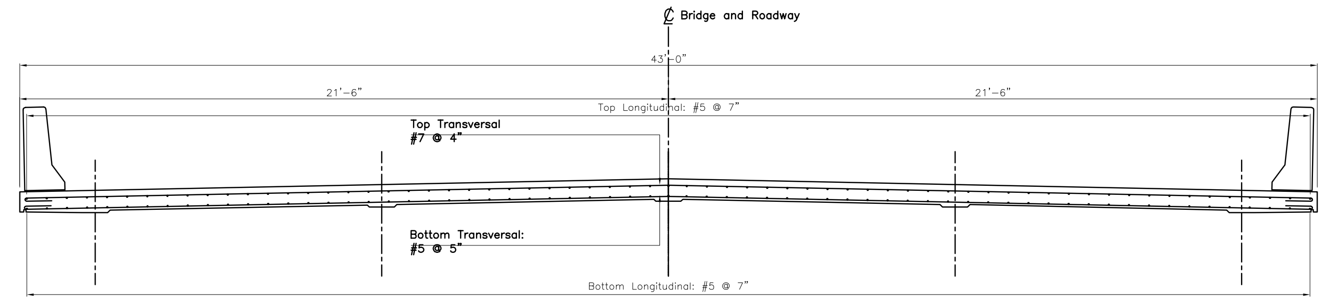


TYPICAL SECTION
ESC: 1"= 2'-0"



TYPICAL SECTION
ESC: 1"= 2'-0"

FOLDED STEEL PLATE GIRDER SUPERSTRUCTURE DESIGNS			
Girder Type	Number of Spans	Span Length (ft)	No. of Girders
FP 72x17x½	1	30	6
FP 96x26x½	1	45	6
FP 120x34x½	1	60	6

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WINTER/2018	Original Version (V,0)

STRUCTURAL STEEL FOLDED PLATE GIRDERS
(FPG)
4 BEAMS CONFIGURATION

RECOMMENDED FOR APPROVAL: Stefan Leonardo Leiva Maldoando, DESIGN ENGINEER — Winter/2018, DATE

DESIGNED: SLLM — DRAWN: CJK
CHECKED: MDB — CHECKED: SLLM

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DEPARTMENT OF TRANSPORTATION

GENERAL DETAILS

SCALE: AS NOTED
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DESIGNATION
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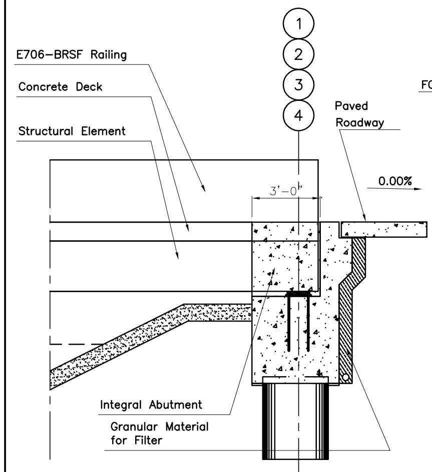
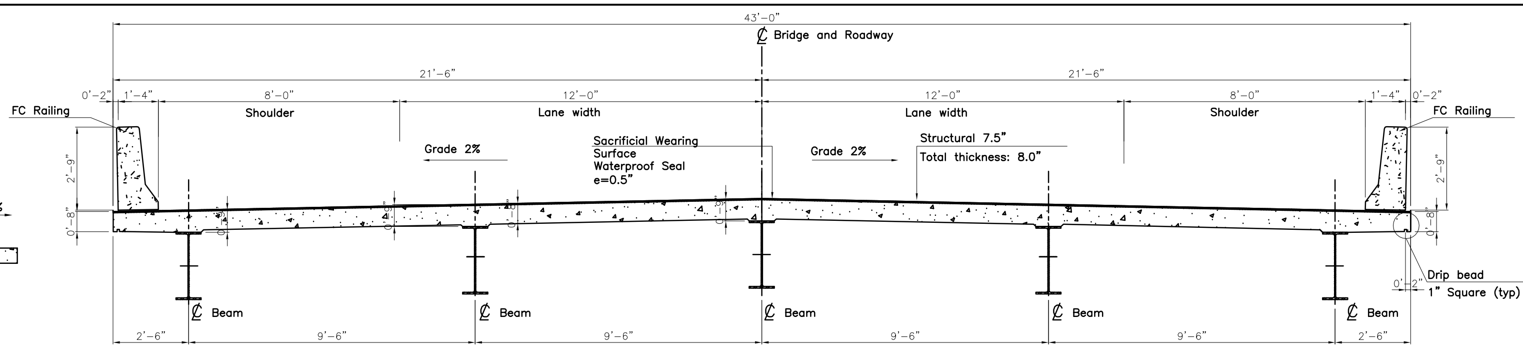
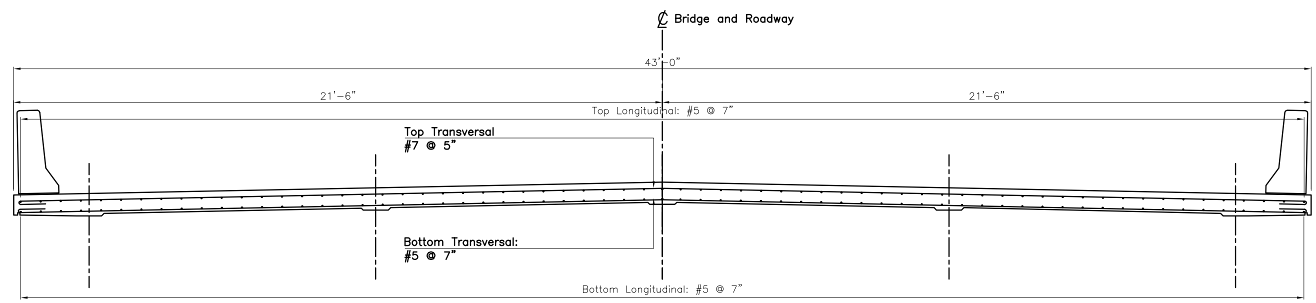


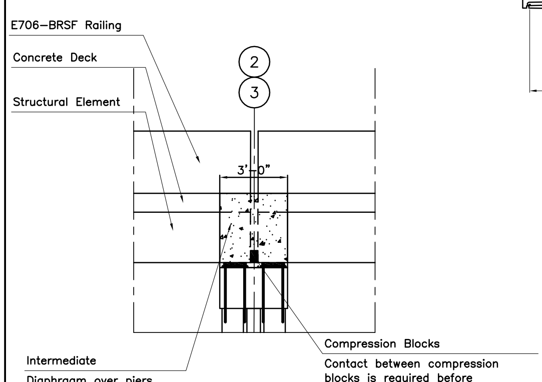
TYPICAL SECTION
INTEGRAL ABUTMENT
ESC: 3/8"= 1'–0"



TYPICAL SECTION
ESC: 1"= 2'–0"



TYPICAL SECTION
ESC: 1"= 2'–0"



TYPICAL SECTION
INTERMEDIATE DIAPHRAGMS
STEEL BEAMS SDCL SYSTEM
ESC: 3/8"= 1'–0"

STRUCTURAL STEEL ROLLED BEAM SUPERSTRUCTURE DESIGNS

Girder Type	Number of Spans	Span Length(s) (ft)			No. of Girders
		Span 1	Span 2	Span 3	
W24x68	1	30	-	-	5
W24x103	1	45	-	-	5
W33x130	1	60	-	-	5
W36x182	1	75	-	-	5
W40x235	1	90	-	-	5
W21x101	3	35	45	35	5
W33x130	3	50	60	50	5
W30x116 SDCL	3	50	60	50	5
W40x183 SDCL	3	60	75	60	5
W44x199 SDCL	3	70	90	70	5

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STRUCTURAL STEEL FOLDED ROLLED BEAMS
AND SIMPLY SUPPORTED FOR DEAD LOAD AND
CONTINUOUS FOR LIVE LOAD (SDCL)
5 BEAMS CONFIGURATION

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CHECKED: MDB — CHECKED: SLLM

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BRIDGE FILE
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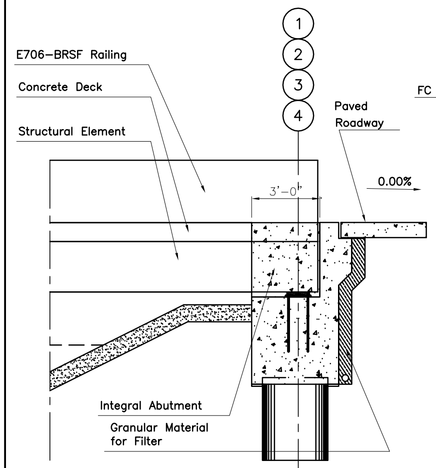
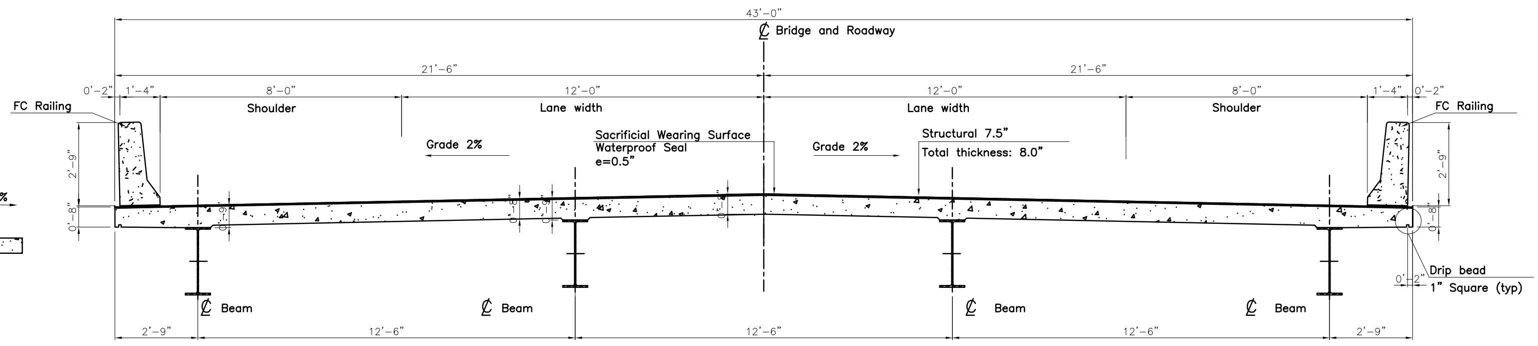
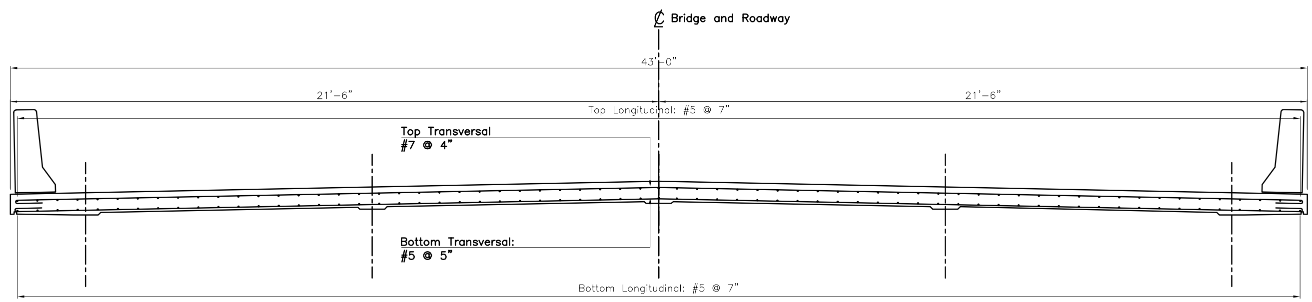


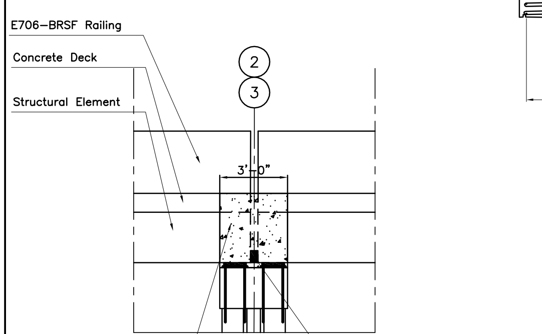
TYPICAL SECTION
INTEGRAL ABUTMENT
ESC: 3/8"= 1'-0"



TYPICAL SECTION
ESC: 1"= 2'-0"



TYPICAL SECTION
ESC: 1"= 2'-0"



TYPICAL SECTION
INTERMEDIATE DIAPHRAGMS
STEEL BEAMS SDCL SYSTEM
ESC: 3/8"= 1'-0"

STRUCTURAL STEEL ROLLED BEAM SUPERSTRUCTURE DESIGNS					
Girder Type	Number of Spans	Span Length(s) (ft)			No. of Girders
		Span 1	Span 2	Span 3	
W27x84	1	30	-	-	4
W30x116	1	45	-	-	4
W36x150	1	60	-	-	4
W40x211	1	75	-	-	4
W44x262	1	90	-	-	4
W21x111	3	35	45	35	4
W36x135	3	50	60	50	4
W33x130 SDCL	3	50	60	50	4
W40x199 SDCL	3	60	75	60	4
W44x230 SDCL	3	70	90	70	4

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DATE	REVISION
WINTER/2018	Original Version (V.0)

STRUCTURAL STEEL FOLDED ROLLED BEAMS
AND SIMPLY SUPPORTED FOR DEAD LOAD AND
CONTINUOUS FOR LIVE LOAD (SDCL)
4 BEAMS CONFIGURATION

RECOMMENDED FOR APPROVAL: Stefan Leonardo Leiva Maldonado, DESIGN ENGINEER — Winter/2018, DATE

DESIGNED: SLLM — DRAWN: CJK
CHECKED: MDB — CHECKED: SLLM

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GENERAL DETAILS

SCALE: AS NOTED
BRIDGE FILE
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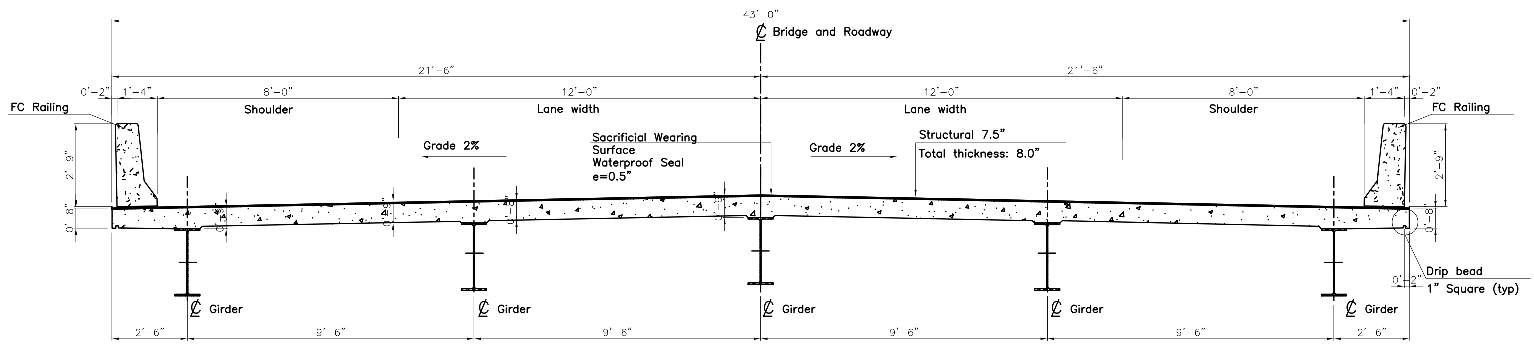
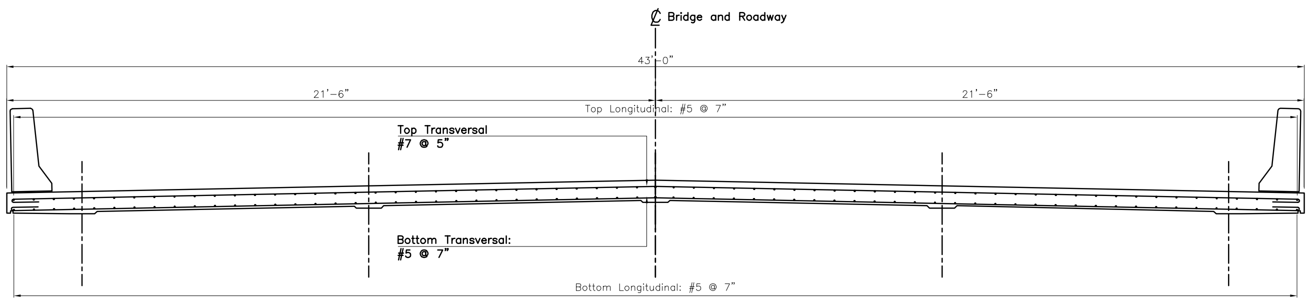


TYPICAL SECTION
ESC: 1"= 2'-0"



TYPICAL SECTION
ESC: 1"= 2'-0"

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Number of Spans	Span Length(s) (ft)			No. of Girders	Bottom Flange (in)			Top Flange (in)			Web (in)
	Span 1	Span 2	Span 3		Span 1	Span 2	Span 3	Span 1	Span 2	Span 3	
1	90	-	-	5	16 x $\frac{3}{4}$ (70%) - 17 x $\frac{7}{8}$ (30%)	-	-	16 x $\frac{3}{4}$ (70%) - 17 x $\frac{7}{8}$ (30%)	-	-	48 x $\frac{7}{16}$
1	110	-	-	5	16 x $\frac{3}{4}$ (50%) - 21 x 1 (50%)	-	-	16 x $\frac{3}{4}$ (50%) - 21 x 1 (50%)	-	-	50 x $\frac{7}{16}$
1	130	-	-	5	20 x 1 (50%) - 23 x 1 $\frac{1}{4}$ (50%)	-	-	20 x 1 (50%) - 23 x 1 $\frac{1}{4}$ (50%)	-	-	50 x $\frac{3}{4}$
2	90	90	-	5	14 x 1 (80%) - 14 x 1 $\frac{1}{2}$ (20%) - 14 x 2 $\frac{1}{4}$ (10%)	14 x 1 (80%) - 14 x 1 $\frac{1}{2}$ (20%) - 14 x 2 $\frac{1}{4}$ (10%)	-	14 x 1 (80%) - 14 x 1 $\frac{1}{2}$ (20%) - 14 x 2 $\frac{1}{4}$ (10%)	14 x 1 (80%) - 14 x 1 $\frac{1}{2}$ (20%) - 14 x 2 $\frac{1}{4}$ (10%)	-	46 x $\frac{1}{2}$
2	110	110	-	5	14 x 1 (60%) - 14 x 1 $\frac{3}{8}$ (30%) - 14 x 2 $\frac{5}{8}$ (10%)	14 x 1 (60%) - 14 x 1 $\frac{3}{8}$ (30%) - 14 x 2 $\frac{5}{8}$ (10%)	-	14 x 1 (60%) - 14 x 1 $\frac{3}{8}$ (30%) - 14 x 2 $\frac{5}{8}$ (10%)	14 x 1 (60%) - 14 x 1 $\frac{3}{8}$ (30%) - 14 x 2 $\frac{5}{8}$ (10%)	-	50 x $\frac{1}{2}$
2	130	130	-	5	14x1 (80%) - 14x1$\frac{1}{2}$(30%) - 14x2 $\frac{1}{4}$ (10%)	14x1 (80%) - 14x1$\frac{1}{2}$(30%) - 14x2 $\frac{1}{4}$ (10%)	-	14x1 (80%) - 14x1$\frac{1}{2}$(30%) - 14x2 $\frac{1}{4}$ (10%)	14x1 (80%) - 14x1$\frac{1}{2}$(30%) - 14x2 $\frac{1}{4}$ (10%)	-	60 x $\frac{1}{2}$
3	70	90	70	5	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	16 x $\frac{7}{8}$ (80%) - 16 x 1 $\frac{1}{4}$ (20%)	44 x $\frac{1}{2}$

DATE	REVISION
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STRUCTURAL STEEL PLATE GIRDERS
5 GIRDERS CONFIGURATION

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DRAWN: CJK
CHECKED: MDB
CHECKED: SLLM

INDIANA
DEPARTMENT OF TRANSPORTATION

GENERAL DETAILS

SCALE: AS NOTED
BRIDGE FILE
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B. BASIC CONCEPTS OF PROBABILITY

The aim of this section is to give definitions of basic concepts of probability that would be helpful to the reader to better understand the main body of the document. In addition, definitions of all probability distribution functions used in this study are summarized.

B.1 The Probabilty Density Function (PDF)

This section is based on the book by Morgan [74]. When $F(x)$ is a continuous funtion of x, with a continuous first derivative, then $f(x) = dF(x)/dx$ is called *the probability density function* of the random variable X. The pdf has the following properties:

i .

$$f(x) \geq 0 \tag{B.1}$$

ii .

$$\int_{-\infty}^{\infty} f(x)dx = 1 \tag{B.2}$$

iii .

$$\begin{aligned} Pr(a < X < b) = Pr(a \leq X < b) = Pr(a < X \leq b) = Pr(a \leq X \leq b) \quad (\text{B.3}) \\ = \int_a^b f(x)dx \end{aligned}$$

B.2 The Cumulative Density Function (CDF)

This section is based on the book by Morgan [74]. For any random variable X, the function F, given by $F(x) = Pr(X \leq x)$ is called *the cumulative density function* of X. We have:

$$\lim_{x \to \infty} F(x) = 1; \lim_{x \to -\infty} F(x) = 0 \tag{B.4}$$

$F(x)$ is a nondecreasing funtion of x, and $F(x)$ is continuous from the right (i.e. if $x > x_0$, $\lim_{x \to x_0} F(x) = F(x_0)$). The cdf can be expressed as the integral of its probability function $f(x)$ as follows:

$$F(x) = \int_{-\infty}^{c} f(x)dx \tag{B.5}$$

B.3 The Empirical Cumulative Density Function (ECDF)

This section is based on the book by D'Agostino and Stephens [30]. The *empirical cumulative distribution function* of a random sample $X_1, ..., X_n$ drawn from a distribution with a cdf F, is defined by:

$$F_n(x) = \frac{\#(X_j \leq x)}{n}, -\infty < x < \infty \tag{B.6}$$

where $\#(X_j \leq x)$ is read, the number of X_j's less than or equal to x. The ecdf is actually a step function with steps or jumps at the values of the variable that occur in the data. When the sample size is large, it is often not displayed as such.

B.4 Expectation

The *expectation* of a variable X for continuous and discrete variables respectively, is defined as (only exist if the defining sum or integral converges absolutely):

$$\mathbb{E}[X] = \int_{-\infty}^{\infty} xf(x)dx \qquad if \qquad \int_{-\infty}^{\infty} \mid x \mid f(x)dx < \infty \tag{B.7}$$

$$\mathbb{E}[X] = \sum_i x_i Pr(X = x_i) \qquad if \qquad \sum_i \mid x_i \mid Pr(X = x_i)x < \infty \tag{B.8}$$

The *variance* of a random variable X is defined as:

$$Var(X) = \mathbb{E}[(X - \mathbb{E}[X])^2] \tag{B.9}$$

and the *covariance* between random variables X and Y is defined as:

$$Cov(X, Y) = \mathbb{E}[(X - \mathbb{E}[X])(Y - \mathbb{E}[Y])] \tag{B.10}$$

B.5 Useful Probability Distributions

This section presents the probability density functions, cumulative density function and distribution parameters for each of the distribution mentioned in the main body of this document. [30] [74] [36] [75] [76]

B.5.1 The Normal distribution

Also known as the *Gaussian distribution*, it is the most widely used probability distribution. The parameters used for this distribution are the mean (μ) and the standard deviation (σ). A short notation for this distribution is $N(\mu, \sigma)$. The probability density function is as follows:

$$f(x \mid \mu, \sigma) = \frac{1}{\sigma\sqrt{2\pi}} e^{\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]} \tag{B.11}$$

and the cumulative density function as:

$$F(x \mid \mu, \sigma) = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{\frac{x-\mu}{\sigma}} e^{\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]} = \frac{1}{2}\left[1 + erf\left(\frac{x-\mu}{\sigma\sqrt{2}}\right)\right] \tag{B.12}$$

B.5.2 The Gamma distribution

If the occurrences of an event are considered a Poisson process (i.e. an event can occur at random at any time or any point in space), then the time until the kth occurrence of the event is governed by the *gamma distribution.* This distribution is also considered as a general purpose pdf. The parameters used for this distribution are the shape (κ) and the scale (θ, i.e. as the inverse of the *mean occurrence rate* υ, $\upsilon = 1/\theta$) parameters. The probability density function is as follows:

$$f(x \mid \kappa, \theta) = \frac{1}{\Gamma(\kappa)\theta^{\kappa}} x^{\kappa-1} e^{\frac{x}{\theta}} \tag{B.13}$$

and the cumulative density function as:

$$F(x \mid \kappa, \theta) = \frac{1}{\Gamma(\kappa)} \gamma \left(\kappa, \frac{x}{\theta}\right) \tag{B.14}$$

where $\gamma\left(\kappa, \frac{x}{\theta}\right)$ is defined as the *lower incomplete gamma function* also known as the *incomplete gamma function ratio.* However, there are some events in which the data skewness is significant. In those cases, a third parameter would be necessary. For these cases, the *shifted gamma distribution*, which is a three-parameter distribution, is useful. The extra parameter is called shift (ω). Then the pdf and the cdf are as follows:

$$f(x \mid \kappa, \theta, \omega) = \frac{1}{\Gamma(\kappa)\theta^{\kappa}} (x-\omega)^{\kappa-1} e^{\frac{x-\omega}{\theta}} \tag{B.15}$$

$$F(x \mid \kappa, \theta, \omega) = \frac{1}{\Gamma(\kappa)} \gamma \left(\kappa, \frac{x-\omega}{\theta}\right) \tag{B.16}$$

B.5.3 The Weibull distribution

It is named after the Swedish mathematician Waloddi Weibull, who described it in detail in 1951 as a result of his research on strength of materials and fatigue. he parameters used for this distribution are the shape (κ) and the scale (λ) parameters. The probability density function is as follows:

$$f(x \mid \kappa, \lambda) = \begin{cases} \frac{\kappa}{\lambda}\left(\frac{x}{\lambda}\right)^{\kappa-1} e^{-\left(\frac{x}{\lambda}\right)^{\kappa}} & x \geq 0 \\ 0 & x < 0 \end{cases} \tag{B.17}$$

and the cumulative density function as:

$$f(x \mid \kappa, \lambda) = \begin{cases} 1 - e^{-(\frac{x}{\lambda})^{\kappa}} & x \geq 0 \\ 0 & x < 0 \end{cases} \tag{B.18}$$

B.5.4 The Lognormal distribution

A random variable X has a *lognormal* pdf if $\ln X$ is normal. The parameters used for this distribution are the mean (λ) and the standard deviation (ζ) of $\ln X$. This distribution is specially useful where the values of the variate are known to be strictly positive. The probability density function is as follows:

$$f(x \mid \lambda, \zeta) = \frac{1}{\zeta x \sqrt{2\pi}} e^{\left[-\frac{1}{2}\left(\frac{\ln x - \lambda}{\zeta}\right)^2\right]} \tag{B.19}$$

and the cumulative density function as:

$$F(x \mid \lambda, \zeta) = \frac{1}{\zeta x \sqrt{2\pi}} \int_{-\infty}^{\frac{\ln x - \lambda}{\zeta}} e^{\left[-\frac{1}{2}\left(\frac{\ln x - \lambda}{\zeta}\right)^2\right]} = \frac{1}{2}\left[1 + erf\left(\frac{\ln x - \lambda}{\zeta\sqrt{2}}\right)\right] \tag{B.20}$$

B.5.5 The Logistic distribution

It resembles the *normal distribution* in shape but has heavier tails. The parameters used for this distribution are the location (μ, mean) and the scale (σ) parameters. The probability density function is as follows:

$$f(x \mid \mu, \sigma) = \frac{e^{-\frac{x-\mu}{\sigma}}}{\sigma\left(1 + e^{-\frac{x-\mu}{\sigma}}\right)^2} \tag{B.21}$$

and the cumulative density function as:

$$F(x \mid \mu, \sigma) = \frac{1}{1 + e^{-\frac{x-\mu}{\sigma}}} \tag{B.22}$$

B.5.6 The Inverse Gaussian distribution

Also known as the *Wald distribution*, it is widely used to model non negative positively skewed data. The parameters used for this distribution are the mean (μ) and the shape (λ) parameter. The probability density function is as follows:

$$f(x \mid \mu, \lambda) = \left[\frac{1}{x^3\sqrt{2\pi}}\right]^{\frac{1}{2}} e^{\left[-\frac{1}{2}\left(\frac{\lambda(x-\mu)^2}{x\mu^2}\right)^2\right]} \tag{B.23}$$

and the cumulative density function as:

$$F(x \mid \mu, \lambda) = \Phi\left[\sqrt{\frac{\lambda}{x}}\left(\frac{x}{\mu} - 1\right)\right] + e^{\frac{2\lambda}{\mu}}\Phi\left[\sqrt{-\frac{\lambda}{x}}\left(\frac{x}{\mu} + 1\right)\right] \tag{B.24}$$

where Φ is the standard normal distribution cdf.

B.5.7 The PERT distribution

The *Program Evaluation and Review Technique (PERT) distribution* is a special case of a *beta distribution*. The parameters used for this distribution are the minimum value (min), most likely value (ML) and the maximum value (max) which are the same parameters used for the *triangular distribution*. The probability density function is as follows:

$$f(x \mid min, ML, max) = \frac{(x - min)^{\alpha-1}(max - x)^{\beta-1}1}{B(\alpha, \beta)(max - min)^{\alpha+\beta-1}} \tag{B.25}$$

$$\alpha = \frac{4ML + max - 5min}{max - min} \tag{B.26}$$

$$\beta = \frac{5max - min - 4ML}{max - min} \tag{B.27}$$

where β corresponds to the Beta function and the cumulative density function as:

$$F(x \mid min, ML, max) = I_z(\alpha, \beta) \tag{B.28}$$

where I_z is the incomplete beta function with z as:

$$z = \frac{x - min}{max - min} \tag{B.29}$$

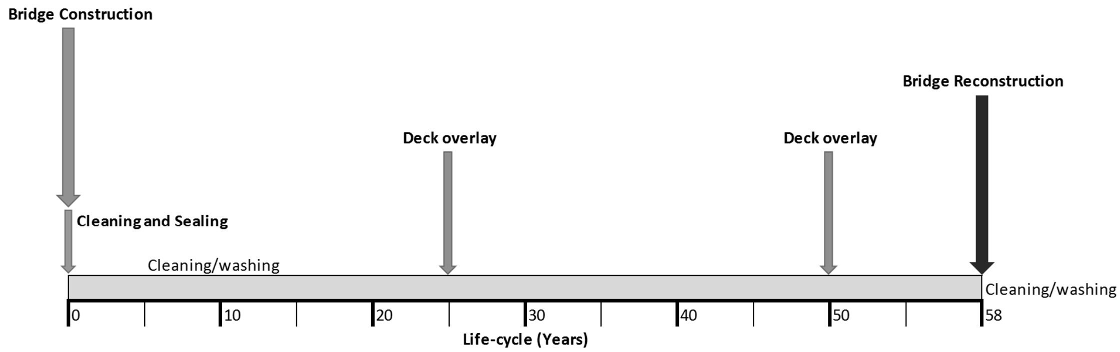
C. LIFE CYCLE PROFILES FOR INDIANA BRIDGES

This appendix presents the different life-cycle cost profiles considered for each one of the superstructures analyzed in this document. Those presented in Chapter 6 are the most cost-effective LCCP for each of the superstructure types used.

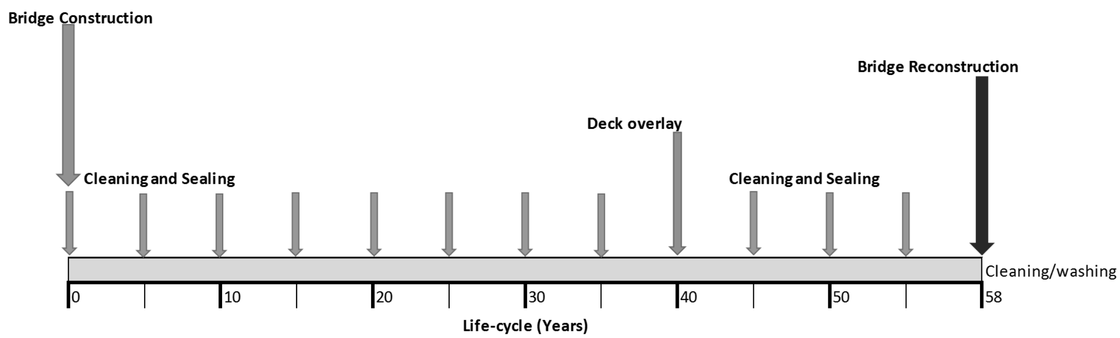
1. CONCRETE SLAB

Service Life: 58 years Moomen et al (2016)

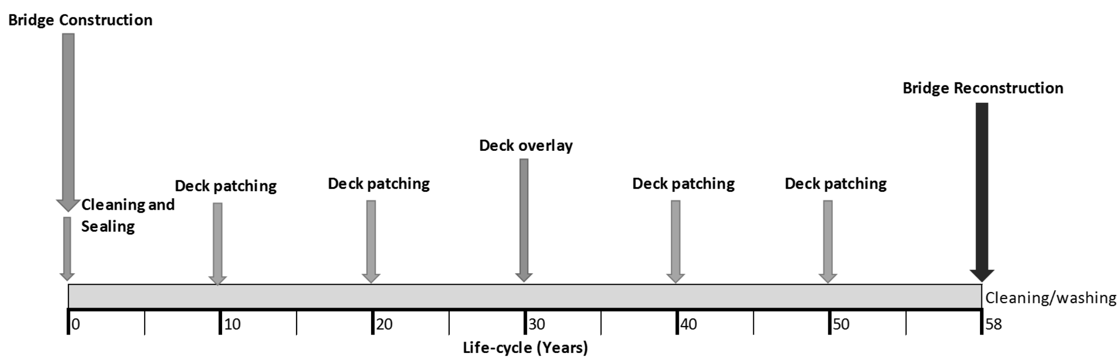
1.1. INDOT routine procedure



1.2. Alternative A: Modified INDOT routine procedure



1.3. Alternative B: Alternative INDOT routine procedure

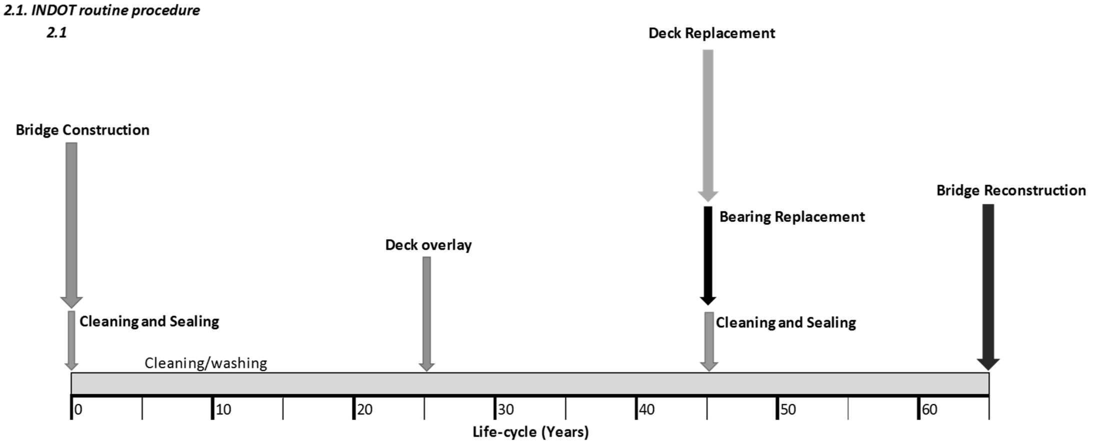


2. PRESTRESSED CONCRETE BEAM - BEARINGS INCLUDED

Service Life: 65 years Moomen et al (2016)

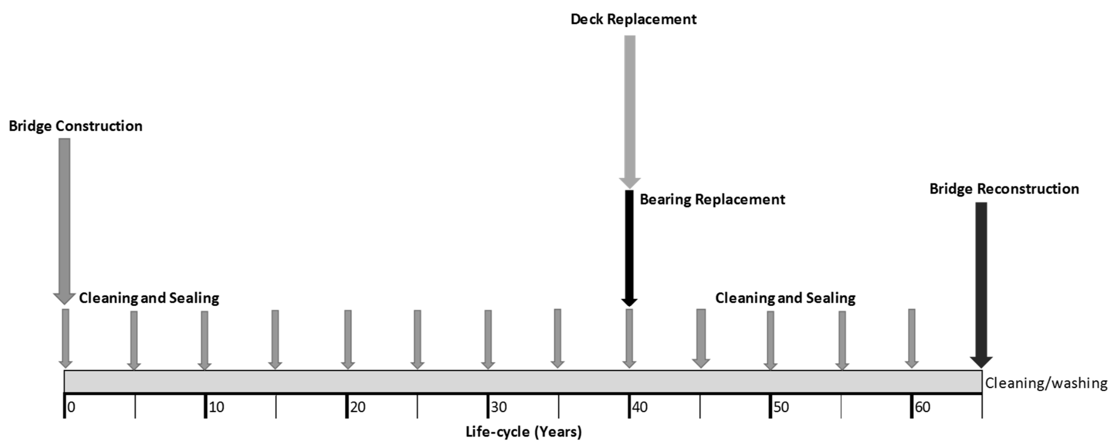
2.1. INDOT routine procedure

2.1



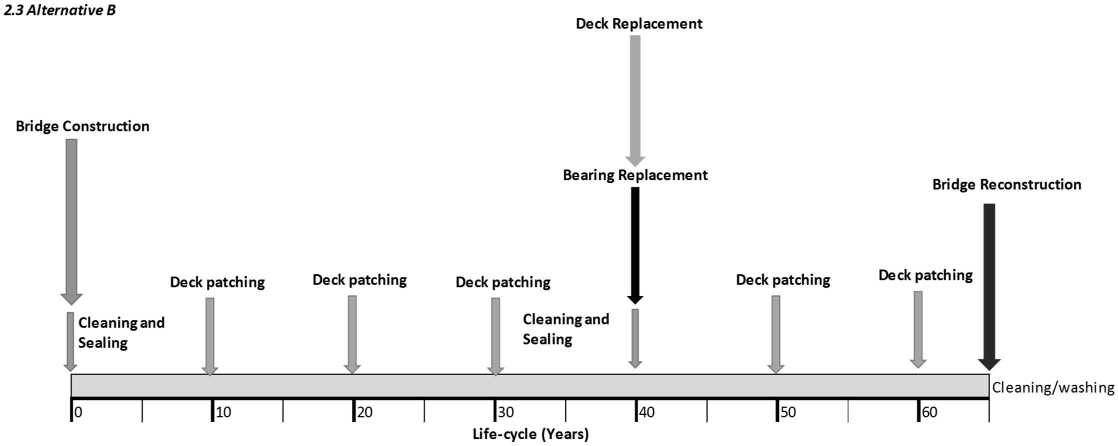
2.2. Modified INDOT routine procedure

2.2 Alternative A



2.3. Alternative INDOT routine procedure

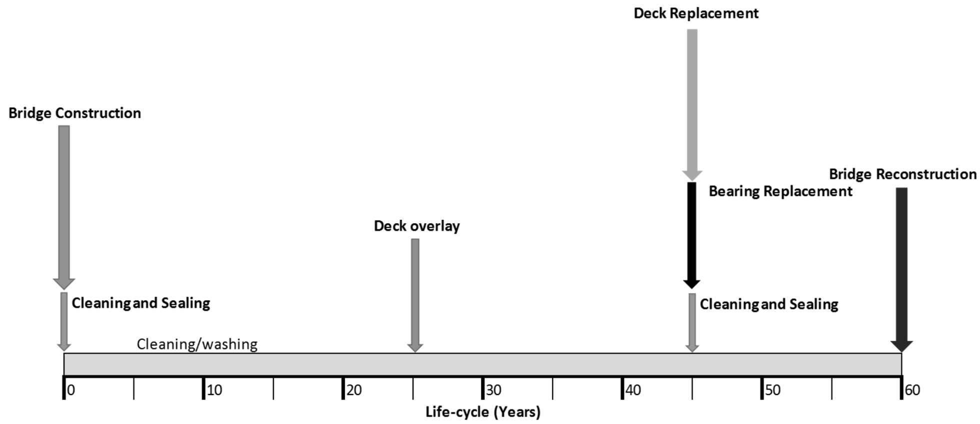
2.3 Alternative B



3. PRESTRESSED CONCRETE BOX

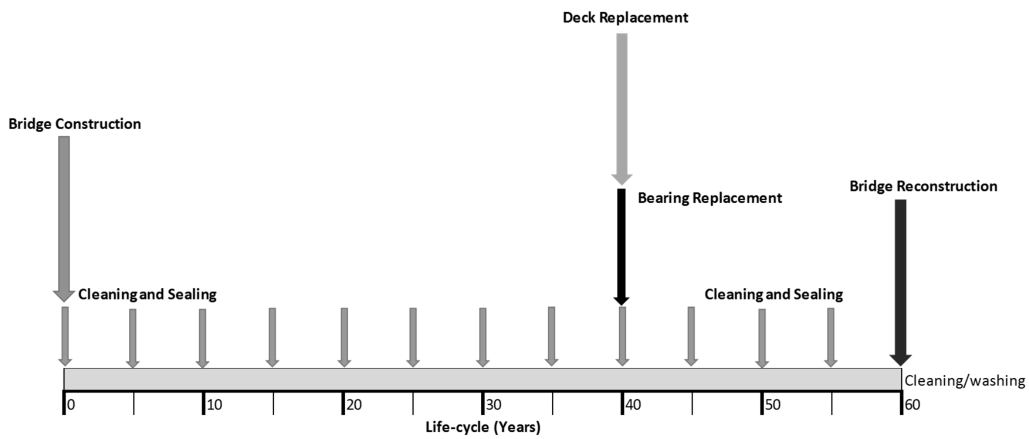
Service Life: 60 years Moomen et al (2016)

3.1. INDOT routine procedure



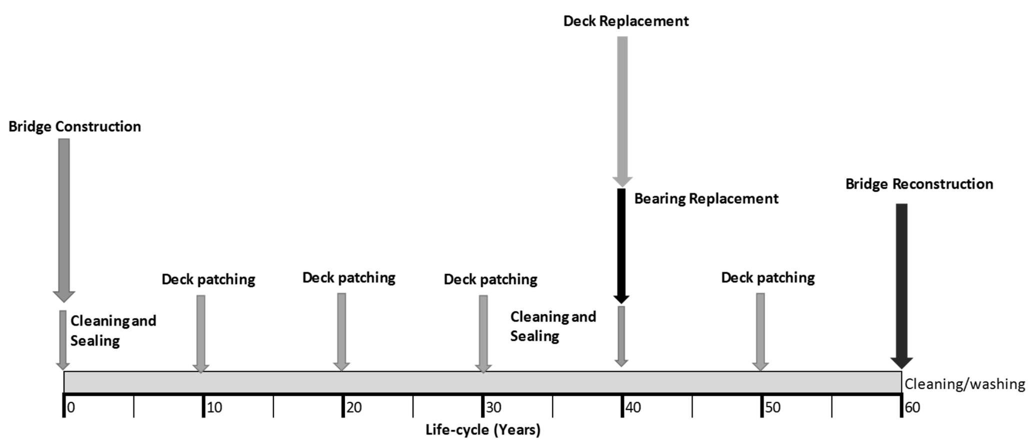
3.2. Modified INDOT routine procedure

3.2.1Alternative A



3.3. Alternative INDOT routine procedure

3.3 Alternative B

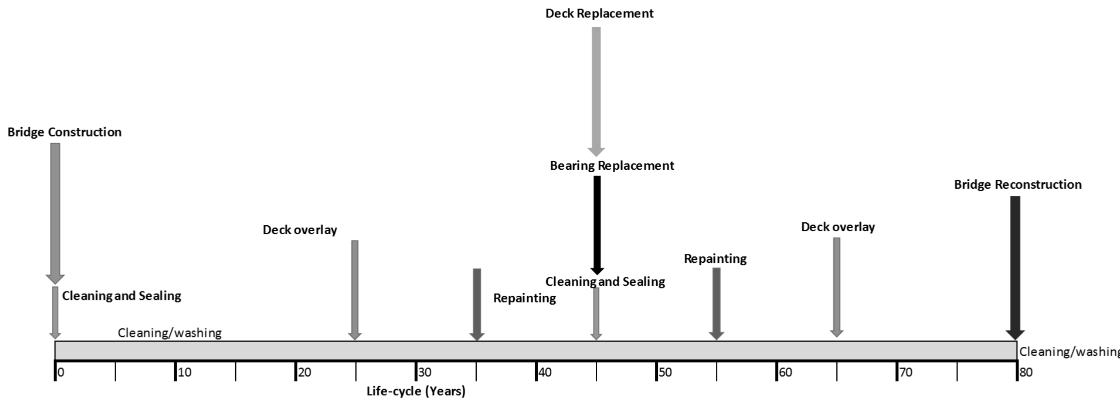


4. STRUCTURAL STEEL ELEMENTS - PAINTED CORROSION PROTECCTION

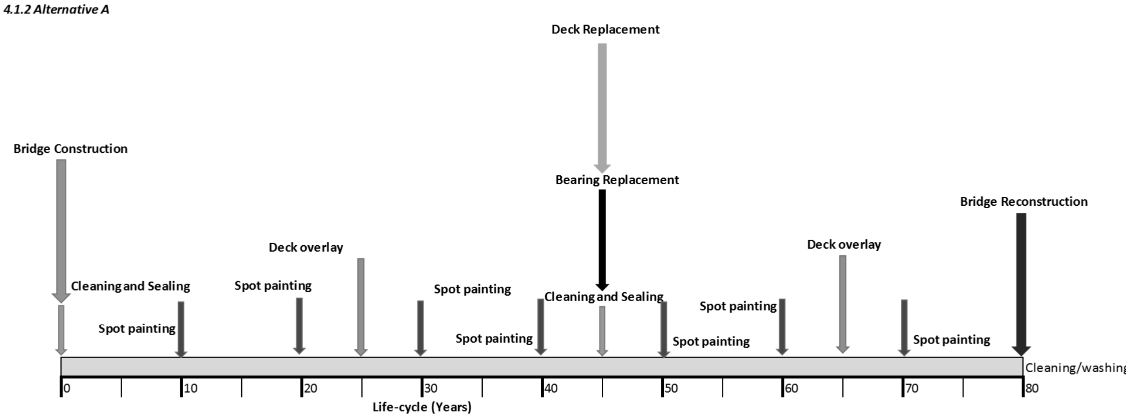
Service Life: 80 years Sinha et al (2009)

4.1. INDOT routine procedure

4.1.1.



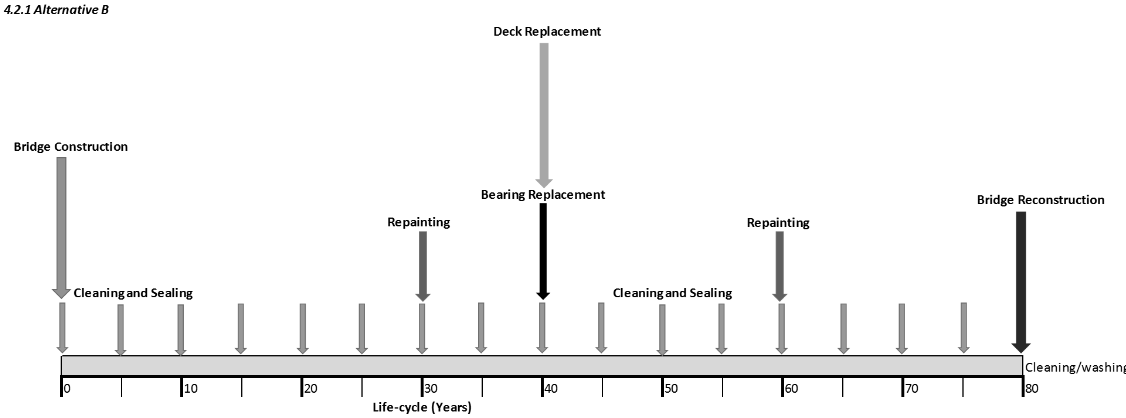
4.1.2 Alternative A



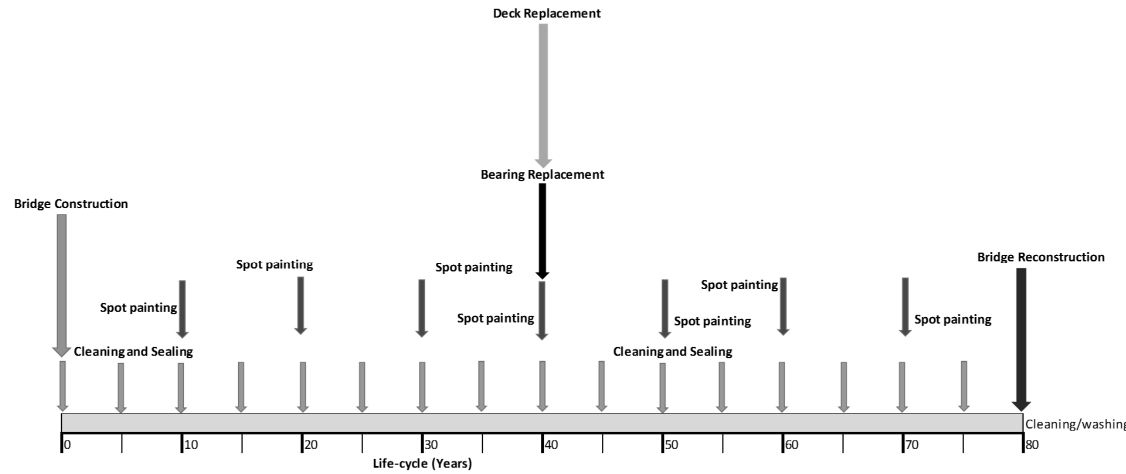
4.2. Modified INDOT routine procedure

4.2.1

4.2.1 Alternative B



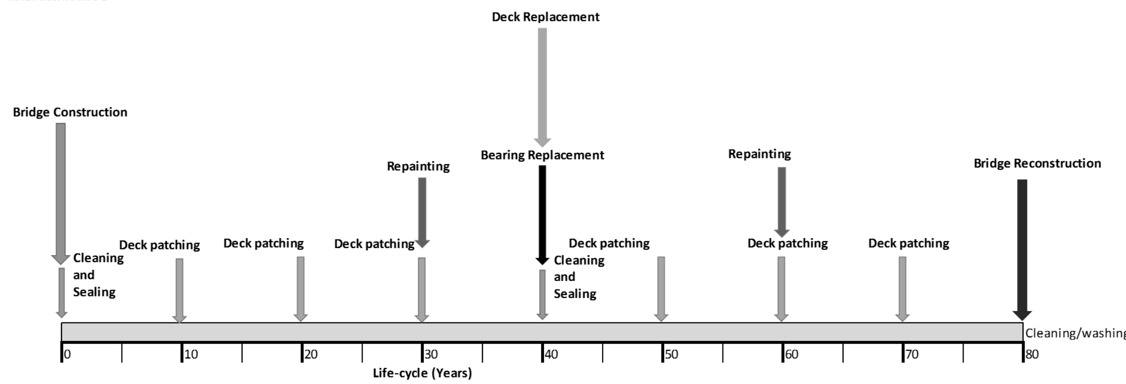
4.2.2 Alternative C



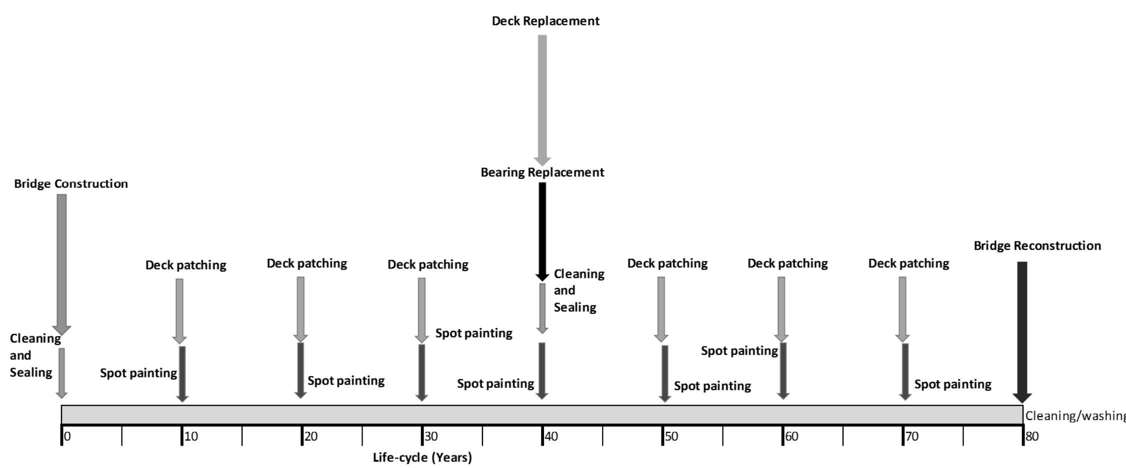
4.3. Alternative INDOT routine procedure

4.3.1

4.3.1 Alternative D



4.3.2 Alternative E

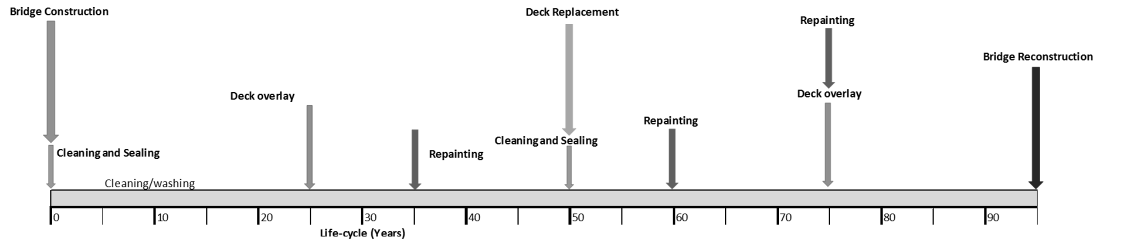


5. SIMPLY SUPPORTED FOR DEAD LOAD CONTINUOUS FOR LIVE LOAD BEAMS (SDCL) - PAINTED CORROSION PROTECTION

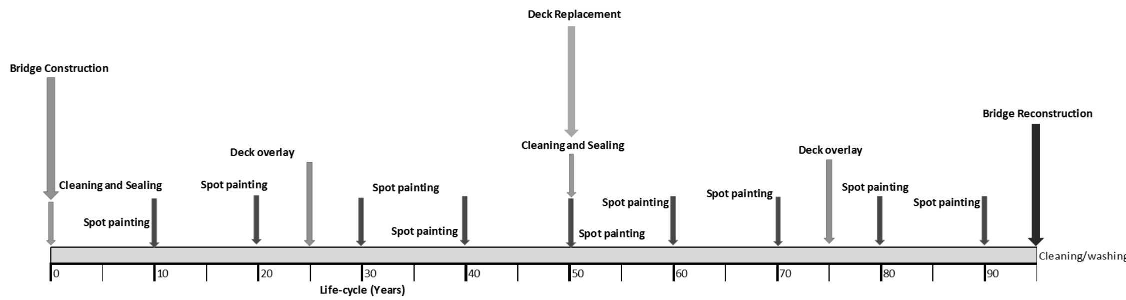
Service Life: 95 years Sinha et al (2009)

5.1. INDOT routine procedure

5.1.1.

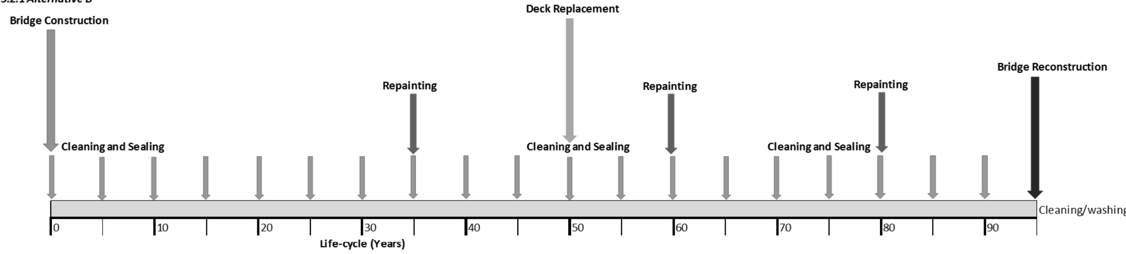


5.1.2 Alternative A

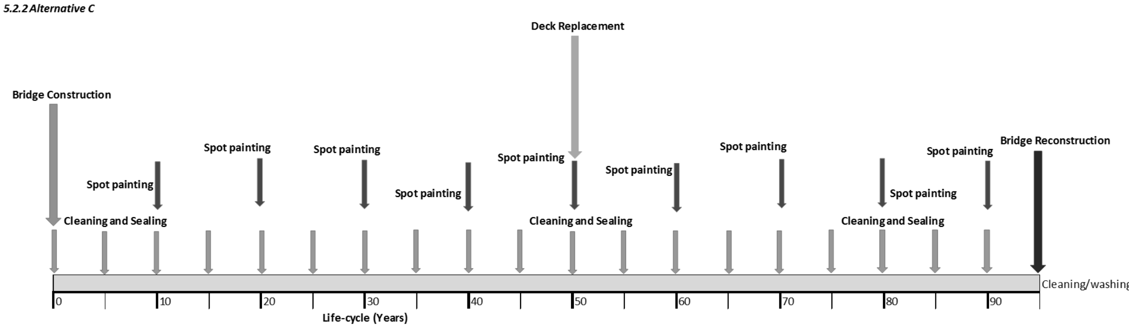


5.2. Modified INDOT routine procedure

5.2.1 Alternative B

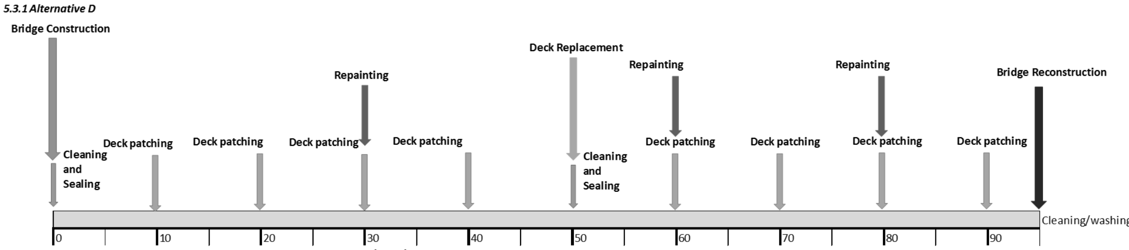


5.2.2 Alternative C

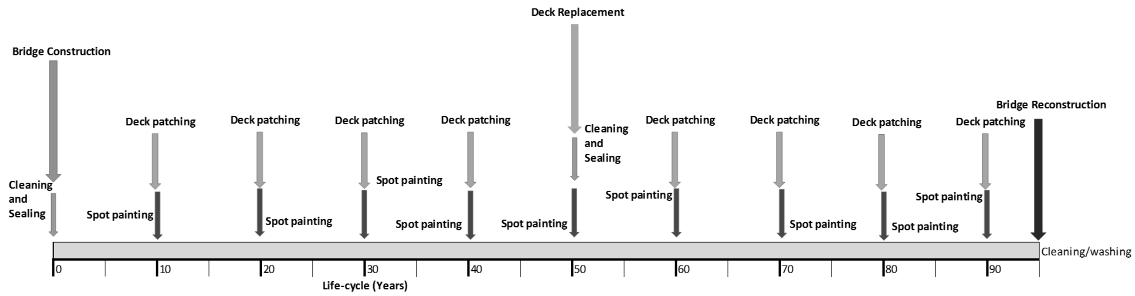


5.3. Alternative INDOT routine procedure

5.3.1 Alternative D



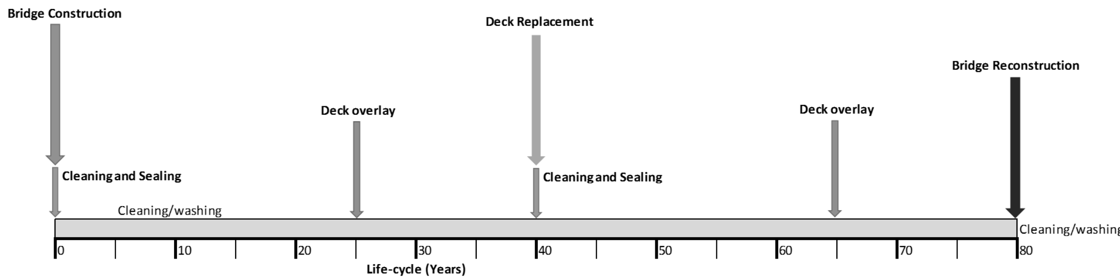
5.3.2 Alternative E



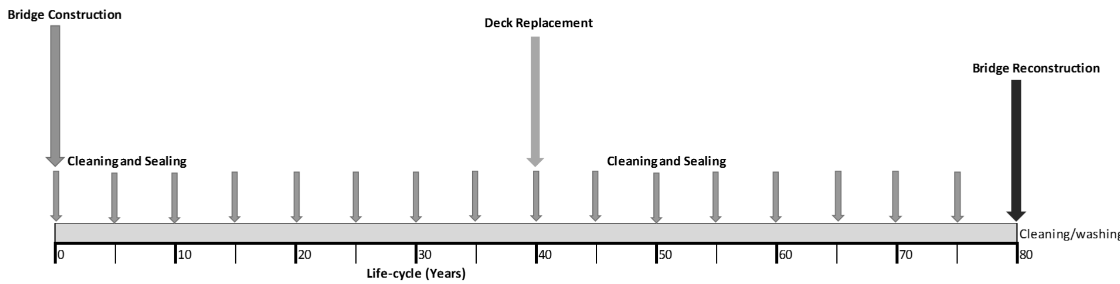
6. PRESTRESSED CONCRETE BEAM - DIAPHRAGMS AND INTEGRAL ABUTMENTS INCLUDED

Service Life: 80 years Moomen et al (2016)

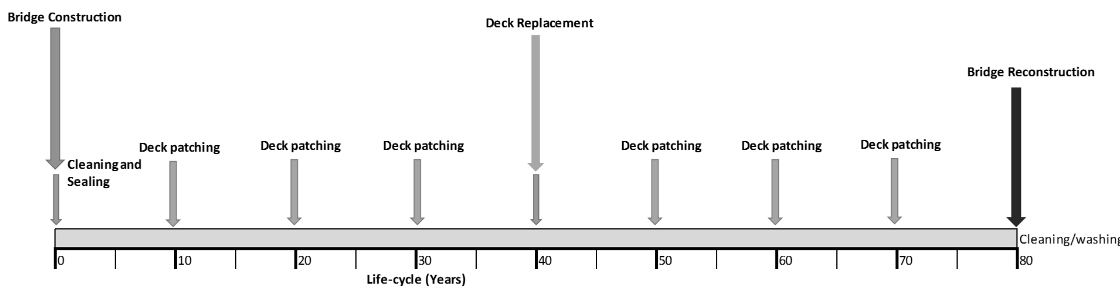
6.1. INDOT routine procedure



6.2. Alternative A: Modified INDOT routine procedure



6.3. Alternative B: Alternative INDOT routine procedure

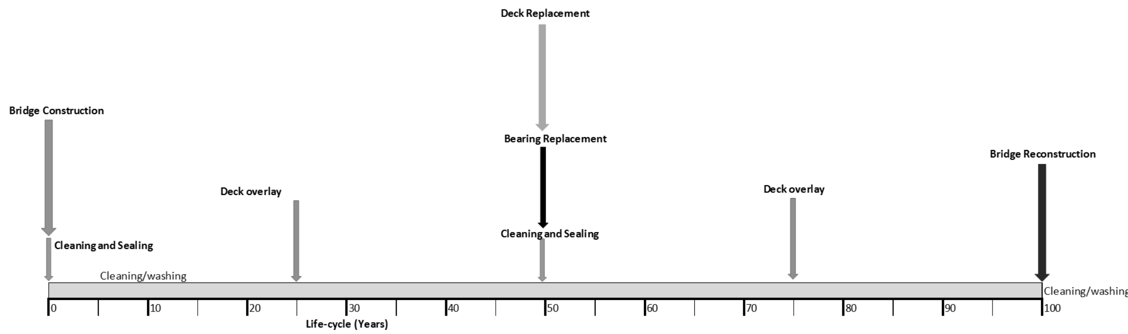


7. STRUCTURAL STEEL ELEMENTS - GALVANIZED CORROSION PROTECCTION

Service Life: 100 years

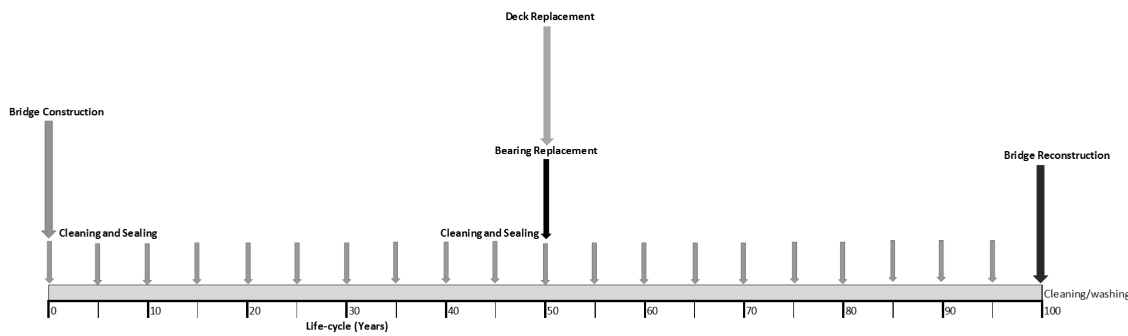
7.1. INDOT routine procedure

7.1.1



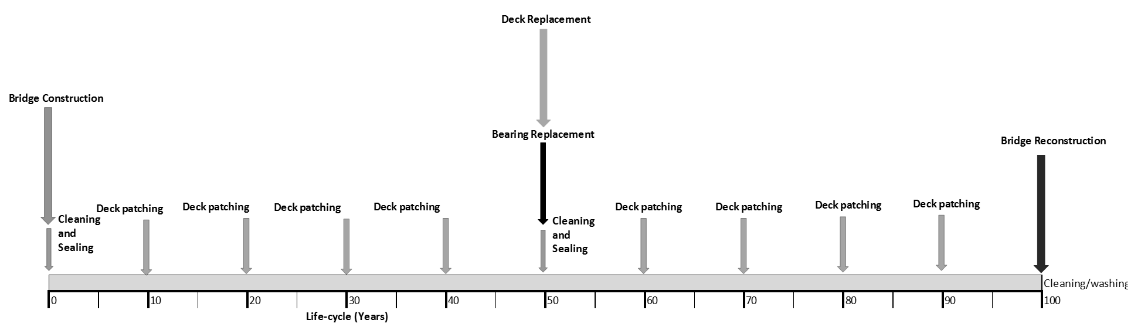
7.2. Modified INDOT routine procedure

7.2.1 Alternative A



7.3. Alternative INDOT routine procedure

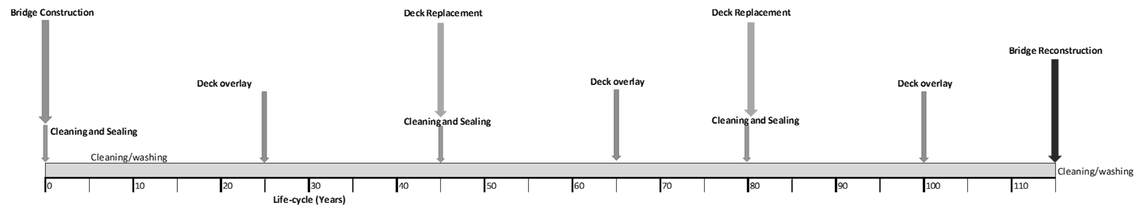
7.3.1 Alternative B



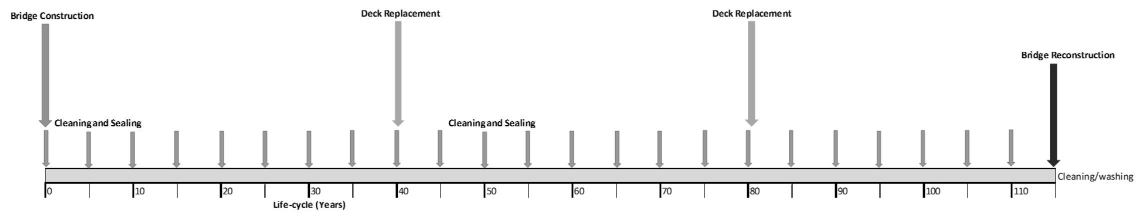
8. SIMPLY SUPPORTED FOR DEAD LOAD CONTINUOUS FOR LIVE LOAD BEAMS (SDCL) - GALVANIZED CORROSTION PROTECTION

Service Life: 115 years

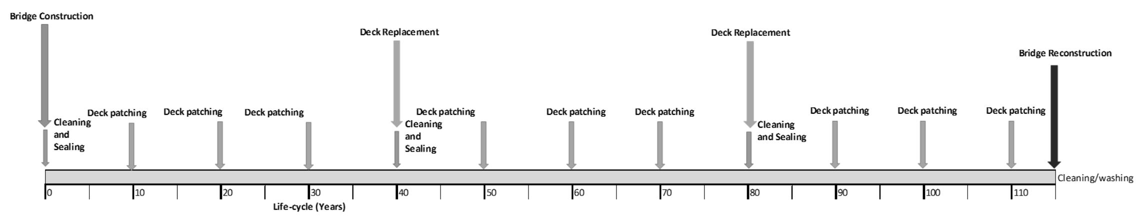
8.1. INDOT routine procedure



8.2. Alternative A Modified INDOT routine procedure



8.3. Alternative B Alternative INDOT routine procedure



D. INTEREST EQUATIONS AND EQUIVALENCES

According to Sinha and Labi [64], interest equations known also as equivalency equations are the relationships between amounts of money that occur at different points in time and are used to estimate the worth of a single amount of money or a series of monetary amounts from one time period to another to reflect the time value of money. All relationships involve some of the following five basic factors: P, initial amount; F, amount of money at a specified future period; A, a periodic amount of money; i,the interest rate or discount rate for the compounding period; and N, a specified number of compounding periods or the analysis period.

D.1 Single Payment Compound Amount Factor (SPACF)

Finding the future compounded amount (F) at the end of a specified period given the initial amount (P), the analysis period (N) and interest rate (i), is given by Equation D.1.

$$F = P \times SPACF \quad , \quad SPACF = (1+i)^N \tag{D.1}$$

D.2 Single Payment Present Worth Factor (SPPWF)

Finding the initial amount (P) that would yield a given future amount (F), at the end of an specified analysis period (N) given the interest rate (i), is given by Equation D.2.

$$P = F \times SPPWF \quad , \quad SPPWF = \frac{1}{(1+i)^N} \tag{D.2}$$

D.3 Sinking Fund Deposit Factor (SFDF)

Finding the uniform yearly amount (A) that would yield a given future amount (F), at the end of an specified analysis period (N) given the interest rate (i), is given by Equation D.3.

$$A = F \times SFDF \quad , \quad SFDF = \frac{i}{(1+i)^N - 1} \tag{D.3}$$

D.4 Uniform Series Compound Amount Factor (USCAF)

Finding the future compounded amount (F) at the end of a specified period given the annual payments (A), the analysis period (N) and the interest rate (i), is given by Equation D.4.

$$F = A \times USCAF \quad , \quad USCAF = \frac{(1+i)^N - 1}{i} \tag{D.4}$$

D.5 Uniform Series Present Worth Factor (USPWF)

Finding the initial amount (P) that is equivalent to a series of uniform annual payments (A), given the analysis period (N) and the interest rate (i), is given by Equation D.5.

$$P = A \times USPWF \quad , \quad USPWF = \frac{(1+i)^N - 1}{i(1+i)^N} \tag{D.5}$$

D.6 Capital Recovery Factor (CRF)

Finding the amount of uniform yearly payments (A) that would completely recover an initial amount (P), at the end of the analysis period (N) given the interest rate (i), is given by Equation D.6.

$$A = P \times CRF \quad , \quad CRF = \frac{(i(1+i)^N)}{(1+i)^N - 1} \tag{D.6}$$

E. EXAMPLE OF LIFE-CYCLE COST ANALYSIS - DETERMINISTIC APPROACH

This section describes the procedure used for the computation of the LCCA and the indicator of economic efficiency. Information needed is the following: Alternatives considered, bridge designs, service life depending on the superstructure type, life-cycle profiles and working action scheduling, agency costs and finally, the LCCA strategy including discount rate and comparison criteria as mentioned earlier in this Appendix.

As a general outline, this example is performed using the following procedure. First, computation of the initial cost for all the alternatives is assembled. Then a LCCA of different profiles for one superstructure alternative is conducted to show the procedure used for the selection of the definitive profile. After that, computation of the LCCA for the different superstructure type alternatives is done, followed by the estimation of the LCCAP of each one of them.

Following the design plan shown in Chapter 3, six different superstructure types are considered for the simply supported configuration in span range 1, and specifically for a span length of 30-ft. Types considered are the following: slab bridge, structural steel rolled beam bridge (5 beams configuration alternative), structural steel rolled beam bridge (4 beams configuration alternative), prestressed concrete AASHTO beams bridge, structural steel FPG bridge, and prestressed concrete box beam bridge. As mentioned before in this document, barriers and other miscellaneous elements are not considered in the initial cost estimation. Thus, the only costs considered are those for concrete for the superstructure (slab), reinforcing steel, structural steel, and prestressed concrete elements. The costs used are shown in Tables 4.2 to 4.4 and .Quantities were obtained from the designs drawings shown in the Appendix A. Critical features for each of the designs alternatives are noted below:

- Slab bridge: Total concrete slab thickness of 17.5-in including sacrificial surface. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 8 top and #8 @ 5 bottom. Transverse reinforcing steel (perpendicular to direction of the traffic): #5 @ 8.0 top and bottom.

- Structural steel rolled beams (5 beams): Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #7 @ 5.0 top and #5 @ 7.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 7.0 top and bottom. Five (5) W18x65 beams separated by 9.5-ft.

- Structural steel rolled beams (4 beams): Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #7 @ 4.0 top and #5 @ 5.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 7 top and bottom. Four (4) W18x76 beams separated by 12.5-ft.

- Prestressed concrete AASTHO beams: Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #5 @ 4.0 top and #5 @ 8.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 8.0 top and bottom. Six (6) type I AASHTO beams separated by 7.5-ft.

- Structural steel FPG (6 beams): Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #5 @ 5.0 top and #5 @ 8.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 8.0 top and bottom. Six (6) FP60x12x1/2 beams separated by 7.5-ft.

- Structural steel FPG (4 beams): Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #7 @ 4.0 top and #5 @ 5.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 7.0 top and bottom. Six (4) FP72x17x1/2 beams separated by 12.5-ft.

- Prestressed concrete box beams: Total concrete slab thickness of 8.0-in including sacrificial surface. Transverse reinforcing steel (perpendicular to direction of the traffic): #5 @ 5.0 top and #5 @ 7.0 bottom. Longitudinal reinforcing steel (parallel to direction of the traffic): #5 @ 8.0 top and bottom. Five (5) box beams CB17x48 separated by 9.5-ft.

Based on the descriptions of the design features for each of the alternatives, the construction costs can be obtained. The initial cost for all the alternatives is shown in Table E.1. Since the construction is considered at year 0, this value does not need to be discounted to a present value. However, if Equation 7.1 is to be used to calculate the LCCA, present values will be used to compute the single life-cycle cost of the alternative, then this amount is projected to the end of the service life using the SPACF, and finally the LCCAP is obtained (Equation 7.1).

Table E.1.
Initial cost Simply supported beam, span 30 ft.

Span (ft)	**Superstructure Type**	**Width (ft)**	**Total Cost ($)**
30	Slab Bridge	43	51,438
	Steel Beam Painted (4 Beams)	43	59,464
	Steel Beam Painted (5 Beams)	43	59,224
	Steel Beam Galvanized (4 Beams)	43	62,511
	Steel Beam Galvanized (5 Beams)	43	62,234
	PS Concrete AASHTO Beam (Bearings)	43	59,747
	PS Concrete AASHTO Beam (Diaph)	43	73,639
	Steel FPG Galvanized (6 Beams)	43	67,921
	Steel FPG Galvanized (4 Beams)	43	62,790
	PS Concrete Box Beams	43	75,404

Life-cycle profile selection and TLCC estimation

Different maintenance schedules were considered for each superstructure type that resulted in different life-cycle profiles. The minimum TLCC among all the different alternatives per superstructure type is then used for comparison with other superstructure types. Therefore, the lowest value corresponds to the most cost effective option for that specific span length. All the different profiles used can be seen in Appendix D. For this illustrative example, only one superstructure type is detailed (slab bridge). For the remaining types only the most cost-effective profile is shown.

Working actions considered for the slab bridges are described below, various combinations of all of them are presented in the life-cycle profiles shown in Figure 1.

- *Cleaning and washing of the deck*: Only the current INDOT practice is taken into account. The procedure is performed on a yearly basis.
- *Deck Overlay*: Two different alternatives were considered: Alternative A involves a first overlay after 25 years of original construction, then a 25 years of overlay service life. Due to the limited service life of this type of superstructure, only two overlays are considered. However, INDOT policies indicates that a slab bridge could stand up to three different overlays if needed until the end of its service life. Alternative B involves a single overlay after 40 years of construction along with a process of sealing and cleaning of the deck surface every 5 years.
- *Sealing and cleaning of the deck surface*: INDOT current policy contemplate the sealing and cleaning of the deck surface only after the construction/reconstruction of the deck, it means it is considered at year 0 exclusively for slab bridges.

Alternative practice involves performing this procedure every five years for the service life of the bridge.

- *Deck Patching*: Deck patching is considered for 10% of the total deck surface area. This working action is performed every 10 years.
- *Bridge reconstruction*: At the end of the service life (58 years),

Current INDOT practice. This option involves a deck overlay (OC) at 25 and 50 years, plus sealing and cleaning of the deck surface (SCC) at the beginning of the service life, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,INDOT} = IC + PV(WC) + SCC + PV(OC) + PV(BRC) \qquad [1]$$

$$\boldsymbol{IC} = \$\,\mathbf{51,438}$$

$$PV(WC) = wc \times USPWF(4\%, 58years) \qquad [2]$$

$$\boldsymbol{PV(WC)} = {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{58\,years} - 1}{4\%(1+4\%)^{58\,years}} = \$\mathbf{62,787}$$

$$\boldsymbol{SCC} = se \times Area = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft) = \$\mathbf{1,638} \qquad [3]$$

$$PV(OC) = o \times \boldsymbol{SPPWF}(4\%, 25) + o \times \boldsymbol{SPPWF}(4\%, 50) \qquad [4]$$

$$PV(OC) = {}^{\$39.64}/_{ft^2}\,(30ft \times 40ft)\,\frac{1}{(1+4\%)^{25}} + {}^{\$39.64}/_{ft^2}\,(30ft \times 40ft)\,\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(OC)} = \$\mathbf{24,537}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 58) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{1}{(1+4\%)^{58}} \qquad [5]$$

$$\boldsymbol{PV(BRC)} = \$\mathbf{1,474}$$

$$\boldsymbol{TLCC_{Alt\,INDOT}} = \$\mathbf{51,438} + \$\mathbf{62,787} + \$\mathbf{1,638} + \$\mathbf{24,537} + \$\mathbf{1,474} = \$\mathbf{141,874}$$

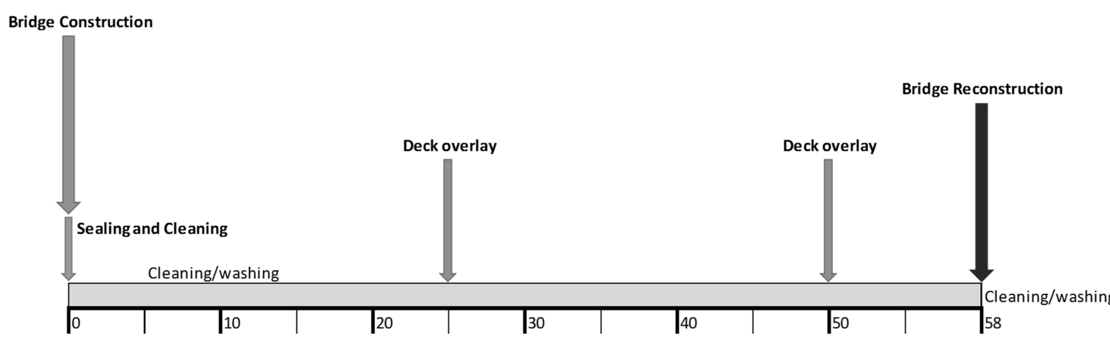
Alternative A - initial extended deterioration. This option involves a deck overlay (OC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ A} = IC + PV(WC) + PV(SCC) + PV(OC) + PV(BRC) \qquad [6]$$

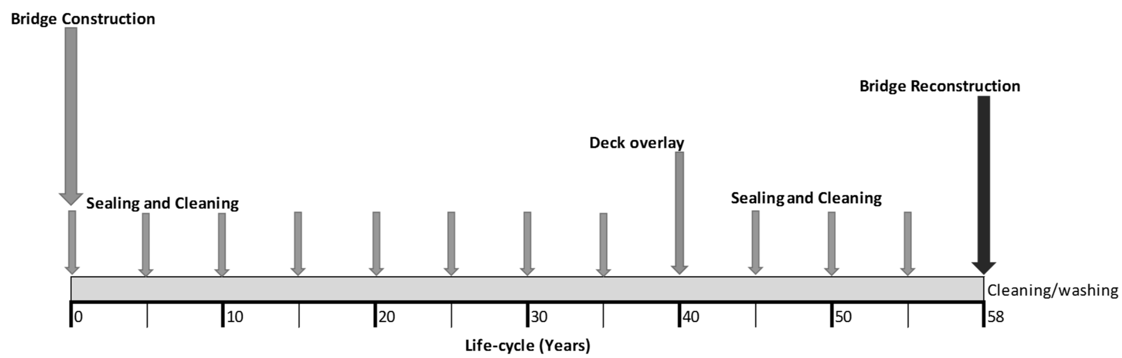
$$\boldsymbol{IC = \$\ 51,438}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 58years) = {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{58\ years} - 1}{4\%(1+4\%)^{58\ years}} \qquad [7]$$

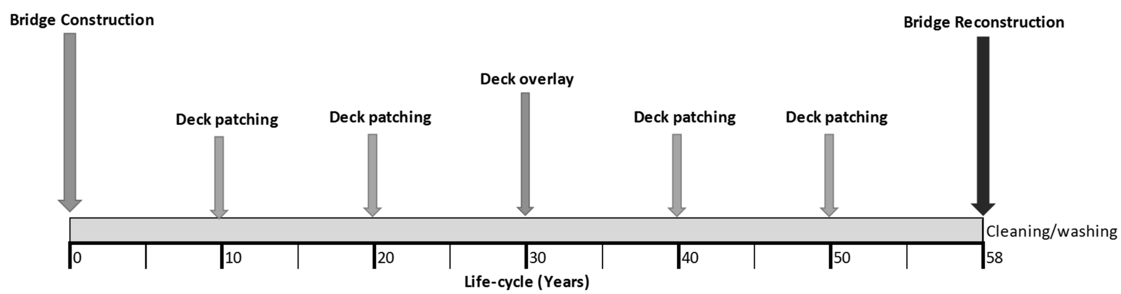
$$\boldsymbol{PV(WC) = \$62,787}$$



(a)



(b)



(c)

Figure 1. Slab bridge life-cycle profiles

(a) INDOT current practice, (b) Alternative A: initial extended deterioration and (c) Alternative B: Deck patching.

$$\mathrm{PV}(SCC) = \sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \begin{Bmatrix} 40 \\ SL \end{Bmatrix} \qquad [8]$$

$$PV(SCC) = \$1.27/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots + \frac{1}{(1+4\%)^{58}}\right) - \$1.27/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{40}} + \frac{1}{(1+4\%)^{58}}\right)$$

$$\boldsymbol{PV(SCC) = \$7,984}$$

$$PV(OC) = o \times \boldsymbol{SPPWF}(4\%, 40) \qquad [9]$$

$$PV(OC) = \$39.64/_{ft^2}\,(30ft \times 40ft)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(OC) = \$9,908}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 58) = \$11.11/_{ft^2}\,(30ft \times 43ft)\frac{1}{(1+4\%)^{58}} \qquad [10]$$

$$\boldsymbol{PV(BRC) = \$1,474}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$51,438 + \$62,787 + \$7,984 + \$9,908 + \$1,474 = \$133,591}$$

Alternative B: Deck patching. This option involves a deck overlay (OC) at 30 years, plus sealing and cleaning of the deck surface (SCC) at the beginning of the service life, plus full depth patching of the deck (PC) every 10 years since the bridge construction (10% of the deck surface), and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,B} = IC + PV(WC) + PV(PC) + PV(OC) + PV(SCC) + PV(BRC) \qquad [11]$$

$$\boldsymbol{IC = \$\ 51,438}$$

$$PV(WC) = wc \times USPWF(4\%, 58years) = \$2.17/_{ft^2}\,(30ft \times 43ft)\frac{(1+4\%)^{58\,years} - 1}{4\%(1+4\%)^{58\,years}} \qquad [12]$$

$$\boldsymbol{PV(WC) = \$62,787}$$

$$\mathrm{PV}(PC) = \sum_1^N pc \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n pc \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 10 \\ 20 \\ \vdots \\ SL \end{Bmatrix}, y_x = \begin{Bmatrix} 30 \\ SL \end{Bmatrix} \qquad [13]$$

$$PV(PC) = {}^{\$37.23}/_{ft^2}\,(30ft \times 40ft) \times 10\% \left(\frac{1}{(1+4\%)^{10}} + \frac{1}{(1+4\%)^{20}} + \cdots + \frac{1}{(1+4\%)^{58}}\right)$$

$$- {}^{\$37.23}/_{ft^2}\,(30ft \times 40ft) \times 10\% \left(\frac{1}{(1+4\%)^{30}} + \frac{1}{(1+4\%)^{58}}\right)$$

$$\boldsymbol{PV(PC) = \$6,616}$$

$$SCC = se \times Area = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft) = \$\mathbf{1,638} \qquad [14]$$

$$PV(OC) = o \times \boldsymbol{SPPWF}(4\%, 30) \qquad [15]$$

$$PV(OC) = {}^{\$39.64}/_{ft^2}\,(30ft \times 40ft)\frac{1}{(1+4\%)^{30}}$$

$$\boldsymbol{PV(OC) = \$14,666}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 58) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\frac{1}{(1+4\%)^{58}} \qquad [16]$$

$$\boldsymbol{PV(BRC) = \$1,474}$$

$$\boldsymbol{TLCC_{Alt\,B} = \$51,438 + \$62,787 + \$14,666 + \$6,616 + 1,638 + \$1,474}$$

$$\boldsymbol{TLCC_{Alt\,B} = \$136,981}$$

As it can be seen, no residual value or salvage value was included. Salvage value was only considered for the steel superstructures and it was included as a benefit. To conclude, it is shown that the most cost-effective profile for slab bridges corresponds to Alternative B.

Following the same principles for the remaining superstructure types, the most cost-effective life-cycle profiles were chosen. However, only the calculation of the definitive profiles for each of the superstructure types analyzed are shown below. Refer to Appendix D for all life-cycle profiles considered for all superstructure types.

Structural Steel Rolled Beam – 5 Beam Configuration: Alternative C: Bearing replacement, spot painting and sealing process. This option involves a deck reconstruction (DRC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, plus spot painting (SPC) every 10 years since the bridge construction (10% of the structural element surface),bearing replacements (BC) at 40 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). In addition, some details are needed regarding the structural steel beam elements. Firstly, the exposed perimeter of the beam is for spot painting 4.94 ft. Secondly, the total weight of the steel elements is 10,506 lb. Finally, a total price for the reinforcement steel of $12,365 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,C} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(SPC) + PV(BRC) + PV(SRC) \qquad [17]$$

$$\boldsymbol{IC = \$\,59,464}$$

$$PV(WC) = wc \times USPWF(4\%, 80years) = {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\frac{(1+4\%)^{80\,years} - 1}{4\%(1+4\%)^{80\,years}} \qquad [18]$$

$$\boldsymbol{PV(WC) = \$66,946}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \left\{\begin{matrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{matrix}\right\}, y_x = \{SL\} \qquad [19]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{80}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{80}}\right)$$

$$\boldsymbol{PV(SCC) = \$8,801}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 40) \qquad [20]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$12{,}365\right)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(DRC) = \$15,314}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 40) \qquad [21]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(5bm \times 2sup)\,\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(BC) = \$7,254}$$

$$\mathrm{PV}(\mathrm{SP}C) = \textstyle\sum_0^N spc \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n spc \times \boldsymbol{SPPWF}(4\%, y_x) \qquad [22]$$

$$\therefore y_i = \begin{Bmatrix} 10 \\ 20 \\ 30 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\}$$

$$PV(SPC) = {}^{\$2.19}/_{ft^2}\,(4.94ft \times 30ft \times 5bm \times 10\%)\left(\frac{1}{(1+4\%)^{10}} + \frac{1}{(1+4\%)^{20}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{80}}\right) - {}^{\$1.27}/_{ft^2}\,(4.94ft \times 30ft \times 5bm \times 10\%)\left(\frac{1}{(1+4\%)^{80}}\right)$$

$$\boldsymbol{PV(SPC) = \$316}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 80) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{1}{(1+4\%)^{80}} \qquad [23]$$

$$\boldsymbol{PV(BRC) = \$622}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 80) = {}^{\$0.08}/_{Lb}\,(5 \times 10{,}506lb)\,\frac{1}{(1+4\%)^{80}} \qquad [24]$$

$$\boldsymbol{PV(SRC) = \$182}$$

$$\boldsymbol{TLCC_{Alt\,C} = \$59,464 + \$66,946 + \$8,801 + \$15,314 + \$7,254 + \$316 + \$622 - \$182}$$

$$\boldsymbol{TLCC_{Alt\,C} = \$158,535}$$

Prestressed Concrete AASTHO Beams: Alternative A –modified INDOT routine procedure. This option involves a deck reconstruction (DRC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 45 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). Finally, a total price for the reinforcing steel of $9,086 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(BRC) \qquad [25]$$

$$\boldsymbol{IC = \$\ 59,747}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 65years) = {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\frac{(1+4\%)^{65\,years} - 1}{4\%(1+4\%)^{65\,years}} \qquad [26]$$

$$\boldsymbol{PV(WC) = \$64,515}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [27]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots \right.$$

$$\left. + \frac{1}{(1+4\%)^{65}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{65}}\right)$$

$$\boldsymbol{PV(SCC) = \$8,481}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 40) \quad [28]$$

$$PV(DRC) = \left({}^{\$47.41}\!/_{ft^2}\,(30ft \times 43ft) + \$9{,}086\right)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(DRC) = \$14,631}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 40) \quad [29]$$

$$PV(BC) = {}^{\$3{,}483}\!/_{unt}\,(6bm \times 2sup)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(BC) = \$8,705}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 65) = {}^{\$11.11}\!/_{ft^2}\,(30ft \times 43ft)\frac{1}{(1+4\%)^{65}} \quad [30]$$

$$\boldsymbol{PV(BRC) = \$1,120}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$59,747 + \$64,515 + \$8,481 + \$14,631 + \$8,705 + \$1,120}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$157,199}$$

Structural Steel Rolled Beam – 4 Beam Configuration: Alternative C: Bearing replacement, spot painting and sealing process. This option involves a deck reconstruction (DRC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, plus spot painting (SPC) every 10 years since the bridge construction (10% of the structural element surface),bearing replacements (BC) at 40 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). In addition, some details are needed regarding the structural steel beam elements. Firstly, the exposed perimeter of the beam is 5.76 ft. Secondly, the total weight of the steel elements is 10,382 lb. Finally, a total price for the reinforcement steel of $14,222 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ C} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(SPC) + PV(BRC) + PV(SRC) \quad [31]$$

$$\boldsymbol{IC = \$\ 59,224}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 80years) = {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{80\ years} - 1}{4\%(1+4\%)^{80\ years}} \quad [32]$$

$$\boldsymbol{PV(WC) = \$66,946}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \quad [33]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots + \frac{1}{(1+4\%)^{80}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{80}}\right)$$

$$\boldsymbol{PV(SCC) = \$8,801}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 40) \quad [34]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$14{,}222\right)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(DRC) = \$15,701}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 40) \quad [35]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(4bm \times 2sup)\,\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(BC) = \$5,803}$$

$$\text{PV(SP}C) = \sum_0^N spc \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n spc \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 10 \\ 20 \\ 30 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [36]$$

$$PV(SPC) = {}^{\$2.19}/_{ft^2}\,(5.76ft \times 30ft \times 4bm \times 10\%)\left(\frac{1}{(1+4\%)^{10}} + \frac{1}{(1+4\%)^{20}} + \cdots \right.$$

$$\left. + \frac{1}{(1+4\%)^{80}}\right) - {}^{\$1.27}/_{ft^2}\,(5.16ft \times 30ft \times 4bm \times 10\%)\left(\frac{1}{(1+4\%)^{80}}\right)$$

$$\boldsymbol{PV(SPC) = \$295}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 80) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\frac{\mathbf{1}}{(\mathbf{1} + 4\%)^{\mathbf{80}}} \qquad [37]$$

$$\boldsymbol{PV(BRC) = \$622}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 80) = {}^{\$0.08}/_{Lb}\,(4bm \times 10{,}382lb)\frac{1}{(1+4\%)^{80}} \qquad [38]$$

$$\boldsymbol{PV(SRC) = \$144}$$

$$\boldsymbol{TLCC_{Alt\,C} = \$59,224 + \$66,946 + \$8,801 + \$15,701 + \$5,803 + \$295 + \$622 - \$144}$$

$$\boldsymbol{TLCC_{Alt\,C} = \$157,248}$$

Prestressed Concrete Box Beams: Alternative A – modified INDOT routine procedure. This option involves a deck reconstruction (DRC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 40 years, washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). Finally, a total price for the reinforcement steel of $8,651 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,A} = IC + PV(WC) + PV(SCC) + PV(BC) + PV(DRC) + PV(BRC) \qquad [39]$$

$$\boldsymbol{IC} = \$\,\mathbf{75,404}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 60years)$$

$$= {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{60\,years} - 1}{4\%(1+4\%)^{60\,years}} \qquad [40]$$

$$\boldsymbol{PV(WC)} = \$\mathbf{63,330}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [41]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{60}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{60}}\right)$$

$$\boldsymbol{PV(SCC)} = \$\mathbf{8,326}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 40) \qquad [42]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$8{,}651\right)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(DRC)} = \$\mathbf{14,541}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 30) \qquad [43]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(5bm \times 2sup)\,\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(BC)} = \$\mathbf{7,254}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 60) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{1}{(1+4\%)^{60}} \qquad [44]$$

$$\boldsymbol{PV(BRC)} = \$\mathbf{1,362}$$

$$\boldsymbol{TLCC_{Alt\ A} = \$75,404 + \$63,330 + \$8,326 + \$14,541 + \$7,254 + \$1,362}$$

$$\boldsymbol{TLCC_{Alt\ A} = \$170,217}$$

Structural Steel Rolled Beam – 5 Beam Configuration Galvanized: Alternative A: Bearing replacement and sealing process. This option involves a deck reconstruction (DRC) at 50 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 50 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). Structural steel beam elements with an exposed perimeter of 4.94 ft. and a total weight of the steel elements of 10,506 lb. Finally, a total price for the reinforcement steel of $12,365 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(BRC) + PV(SRC) \qquad [45]$$

$$\boldsymbol{IC = \$\ 62,511}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 100years)$$

$$= {}^{\$2.17}\!/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{100\ years} - 1}{4\%(1+4\%)^{100\ years}} \qquad [46]$$

$$\boldsymbol{PV(WC) = \$68,597}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [47]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{100}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{100}}\right)$$

$$\boldsymbol{PV(SCC) = \$9,018}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 50) \qquad [48]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$12{,}365\right)\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(DRC) = \$10,346}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 50) \qquad [49]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(5bm \times 2sup)\,\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(BC) = \$4,901}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{1}{(1+4\%)^{100}} \qquad [50]$$

$$\boldsymbol{PV(BRC) = \$284}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$0.08}/_{Lb}\,(5 \times 10{,}506lb)\,\frac{1}{(1+4\%)^{100}} \qquad [51]$$

$$\boldsymbol{PV(SRC) = \$83}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$62,511 + \$68,597 + \$9,018 + \$10,346 + \$4,901 + \$284 - \$83}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$155,573}$$

Prestressed Concrete AASTHO Beams Diaphragms Included: Alternative A – Modified INDOT procedure. This option involves a deck reconstruction (DRC) at 40 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC) and the removal of the bridge cost (BRC). Finally, a total price for the reinforcement steel of $9,086 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BRC) \qquad [52]$$

$$\boldsymbol{IC = \$\ 73{,}639}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 80years)$$

$$= \$2.17/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{80\ years} - 1}{4\%(1+4\%)^{80\ years}} \qquad [53]$$

$$\boldsymbol{PV(WC) = \$66{,}946}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \left\{\begin{matrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{matrix}\right\}, y_x = \{SL\} \qquad [54]$$

$$PV(SCC) = \$1.27/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{80}}\right) - \$1.27/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{80}}\right)$$

$$\boldsymbol{PV(SCC) = \$8{,}801}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 40) \qquad [55]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$9{,}086\right)\frac{1}{(1+4\%)^{40}}$$

$$\boldsymbol{PV(DRC) = \$14,631}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 80) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{\mathbf{1}}{(\mathbf{1} + 4\%)^{\mathbf{80}}} \qquad [56]$$

$$\boldsymbol{PV(BRC) = \$622}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$73,639 + \$66,946 + \$8,801 + \$14,631 + \$622 = \$164,639}$$

Structural Steel Rolled Beam – 4 Beam Configuration Galvanized: Alternative A: Bearing replacement and sealing process. This option involves a deck reconstruction (DRC) at 50 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 50 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). Structural steel beam elements with an exposed perimeter of 5.76 ft. and a total weight of the steel elements of 10,382 lb. Finally, a total price for the reinforcement steel of $14,222 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\,A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(BRC) + PV(SRC) \qquad [57]$$

$$\boldsymbol{IC = \$\ 62,234}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 100years)$$

$$= {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{100\,years} - 1}{4\%(1+4\%)^{100\,years}} \qquad [58]$$

$$\boldsymbol{PV}(\boldsymbol{WC}) = \$\mathbf{68,597}$$

$$\mathrm{PV}(SCC) = \sum_{0}^{N} se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_{1}^{n} se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [59]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots + \frac{1}{(1+4\%)^{100}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{100}}\right)$$

$$\boldsymbol{PV}(\boldsymbol{SCC}) = \$\mathbf{9,018}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 50) \qquad [60]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$14{,}222\right)\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV}(\boldsymbol{DRC}) = \$\mathbf{10,607}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 50) \qquad [61]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(4bm \times 2sup)\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV}(\boldsymbol{BC}) = \$\mathbf{3,920}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\frac{1}{(1+4\%)^{100}} \qquad [62]$$

$$\boldsymbol{PV}(\boldsymbol{BRC}) = \$\mathbf{284}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$0.08}/_{Lb}\,(4bm \times 10{,}382lb)\frac{1}{(1+4\%)^{100}} \qquad [63]$$

$$\boldsymbol{PV}(\boldsymbol{SRC}) = \$\mathbf{66}$$

$$\boldsymbol{TLCC_{Alt\,A}} = \$\mathbf{62,234} + \$\mathbf{68,597} + \$\mathbf{9,018} + \$\mathbf{10,607} + \$\mathbf{3,920} + \$\mathbf{284} - \$\mathbf{66}$$

$$\boldsymbol{TLCC_{Alt\,A}} = \$\mathbf{154,594}$$

A special discussion is needed for the FPG system since it is a new system included in this study. As discussed in the literature review, there are two different configurations that can be addressed using FPGs, the regular closed section and the inverted option with the bottom open for inspection. The second option is a proprietary product, and its use involves an additional cost that depends of the holder of the patent. These hidden costs are not available to the public, and consequently it was decided to not include this option in this analysis. However, the closed section is an open technology that can be used without restriction, and therefore it is used as the alternative discussed in this report.

The FPG acts as a steel box section, and such sections are subjected to all the geometric and proportion requirements given by the AASHTO LFRD specification, in particular section 6.11. The requirement given by AASHTO LRFD Section 6.11.2.3 includes the maximum spacing between parallel elements in order to use the distribution factors proposed by the code. This requirement is based on the lateral distribution factors for steel box girders provided by Johnston and Mattock (1967).

Using the section properties available and the AASHTO requirements it is mandatory to use six (6) beams in the cross section of the bridge. The use of this additional beam (compared with the total elements needed for a regular rolled I steel beam) increases the initial cost of this alternative an amount that makes it not cost-effective. Nonetheless, a separate analysis was made using a four (4) beam arrangement. A conservative assumption was made regarding the distribution factors (considering the distribution factor as 1.00 for each beam), designing accordingly the beam elements. This change increases the unit weight of each supporting element, however, the final total weight is less than the six (6) beam alternative. Both LCCA are included herein, proving that the six (6) beam configuration is not cost-effective while the four (4) beam alternative is a competitive option. Further research is needed to explore the viability of 4 girders and the applicability of AASHTO 6.11.2.3 for FPG girders.

Structural Steel Folded Plate Beams – 6 Beam Galvanized Configuration: Alternative A: Bearing replacement, spot painting and sealing process. This option involves a deck reconstruction (DRC) at 50 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 50 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). Structural steel beam elements with an exposed perimeter of 3.60ft. and a total weight of the steel elements of 16,020 lb. Finally, a total price for the reinforcement steel of $8,375 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(BRC) + PV(SRC) \quad [64]$$

$$\boldsymbol{IC = \$\ 67,921}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 100years)$$

$$= {}^{\$2.17}/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{100\ years} - 1}{4\%(1+4\%)^{100\ years}} \quad [65]$$

$$\boldsymbol{PV(WC) = \$68,597}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [66]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots \right.$$

$$\left. + \frac{1}{(1+4\%)^{100}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{100}}\right)$$

$$\boldsymbol{PV(SCC) = \$9,018}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 50) \qquad [67]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$8{,}375\right)\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(DRC) = \$9,784}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 50) \qquad [68]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(6bm \times 2sup)\,\frac{\mathbf{1}}{(\mathbf{1} + 4\%)^{50}}$$

$$\boldsymbol{PV(BC) = \$5,881}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\frac{1}{(1+4\%)^{100}} \qquad [69]$$

$$\boldsymbol{PV(BRC) = \$284}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$0.08}/_{Lb}\,(6bm \times 16{,}020lb)\frac{1}{(1+4\%)^{100}} \qquad [70]$$

$$\boldsymbol{PV(SRC) = \$152}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$67,921 + \$68,597 + \$9,018 + \$9,784 + \$284 + \$5,881 - \$152}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$161,332}$$

Structural Steel Folded Plate Beams – 4 Beam Galvanized Configuration: Alternative A: Bearing replacement, spot painting and sealing process. This option involves a deck reconstruction (DRC) at 50 years, plus sealing and cleaning of the deck surface (SCC) every 5 years since the bridge construction, bearing replacements (BC) at 50 years, and washing of the deck surface (WC) on a yearly basis, plus the initial cost (IC), the removal of the bridge cost (BRC) and the salvage value represented by the benefit of selling the structural steel for recycling (SRC). Structural steel beam elements with an exposed perimeter of 4.17ft. and a total weight of the steel elements of 12,240 lb. Finally, a total price for the reinforcement steel of $14,222 which will be included together with the bridge deck reconstruction cost calculation. The present value of this alternative can be obtained as follows:

$$TLCC_{Alt\ A} = IC + PV(WC) + PV(SCC) + PV(DRC) + PV(BC) + PV(BRC) + PV(SRC) \qquad [71]$$

$$\boldsymbol{IC = \$\ 62{,}790}$$

$$PV(WC) = wc \times \boldsymbol{USPWF}(4\%, 100 years)$$

$$= \$2.17/_{ft^2}\,(30ft \times 43ft)\,\frac{(1+4\%)^{100\,years} - 1}{4\%(1+4\%)^{100\,years}} \qquad [72]$$

$$\boldsymbol{PV(WC) = \$68{,}597}$$

$$\mathrm{PV}(SCC) = \textstyle\sum_0^N se \times \boldsymbol{SPPWF}(4\%, y_i) - \sum_1^n se \times \boldsymbol{SPPWF}(4\%, y_x)$$

$$\therefore y_i = \begin{Bmatrix} 0 \\ 5 \\ 10 \\ \vdots \\ SL \end{Bmatrix}, y_x = \{SL\} \qquad [73]$$

$$PV(SCC) = {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^0} + \frac{1}{(1+4\%)^5} + \frac{1}{(1+4\%)^{10}} + \cdots\right.$$

$$\left. + \frac{1}{(1+4\%)^{100}}\right) - {}^{\$1.27}/_{ft^2}\,(30ft \times 43ft)\left(\frac{1}{(1+4\%)^{100}}\right)$$

$$\boldsymbol{PV(SCC) = \$9,018}$$

$$\mathrm{PV}(DRC) = dr \times \boldsymbol{SPPWF}(4\%, 50) \qquad [74]$$

$$PV(DRC) = \left({}^{\$47.41}/_{ft^2}\,(30ft \times 43ft) + \$14{,}222\right)\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(DRC) = \$10,607}$$

$$PV(BC) = bc \times \boldsymbol{SPPWF}(4\%, 50) \qquad [75]$$

$$PV(BC) = {}^{\$3{,}483}/_{unt}\,(4bm \times 2sup)\,\frac{1}{(1+4\%)^{50}}$$

$$\boldsymbol{PV(BC) = \$3,920}$$

$$PV(BRC) = br \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$11.11}/_{ft^2}\,(30ft \times 43ft)\,\frac{1}{(1+4\%)^{100}} \qquad [76]$$

$$\boldsymbol{PV(BRC) = \$284}$$

$$PV(SRC) = sr \times \boldsymbol{SPPWF}(4\%, 100) = {}^{\$0.08}/_{Lb}\,(4 \times 12{,}240lb)\,\frac{1}{(1+4\%)^{100}} \qquad [77]$$

$$\boldsymbol{PV(SRC) = \$78}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$62,790 + \$68,597 + \$9,018 + \$10,607 + \$284 + \$3,920 - \$78}$$

$$\boldsymbol{TLCC_{Alt\,A} = \$155,139}$$

F. STOCHASTIC DOMINANCE RESULTS FOR SUPERSTRUCTURE SELECTION

This appendix present the summary of the results obtained from the Monte Carlo simulation and used for the superstructure selection using stochastic dominance. A summary table is used for superstructure selection for each case. Each table shows the stochastic matrix selection, in this, each cell shows a set of logical values composed of 4 figures, namely, first, second, almost first and almost second stochastic dominance. As a convention, 0 indicates not dominance of the option shown in the row to the option contrasted in the column, while 1 means dominance in any degree of the row alternative to the column option. For example, if the logical output is "0-0-1-1", it means that the option shown in the row almost dominates in first degree the option presented in the column, Column $\succ^{almost}{}_1$ Row. In other words, the option in the column is most cost-effective than the option shown in the row and is more preferable for that specific span length. The final objective is to find a column that is dominated by every row in the stochastic dominance matrix.

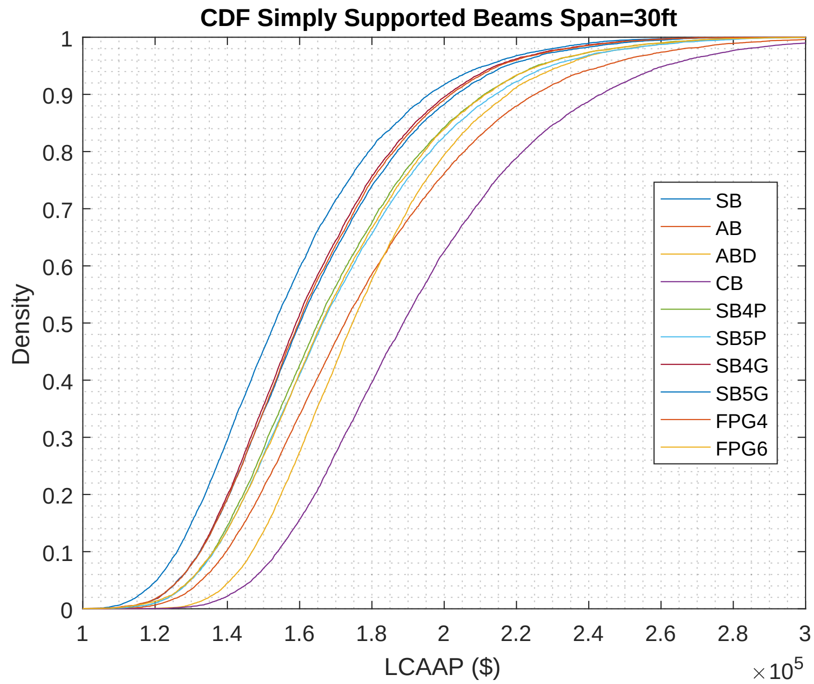


Fig. F.1. Simulation results, CDFs simply supported beams, span=30-ft.

Table F.1.
Stochastic dominance matrix - Simply supported beams, span=30-ft

Alt.	**SB**	**AB**	**ABD**	**CB**	**SB4P**	**SB5P**	**SB4G**	**SB5G**	**FPG4**	**FPG6**
SB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0
AB	1-1-1-1	-	0-0-0-0	0-0-0-0	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
ABD	1-1-1-1	0-1-1-1	-	0-0-0-0	1-1-1-1	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	0-1-1-1
CB	1-1-1-1	0-1-1-1	1-1-1-1	-	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
SB4P	0-1-1-1	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	1-1-1-1	1-1-1-1	1-1-1-1	0-0-0-0
SB5P	0-1-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	1-1-1-1	1-1-1-1	1-1-1-1	0-1-1-1
SB4G	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0
SB5G	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	0-1-1-1	0-0-0-0
FPG4	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0	-	0-0-0-0
FPG6	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0	0-0-1-1	0-0-1-1	0-0-1-1	-

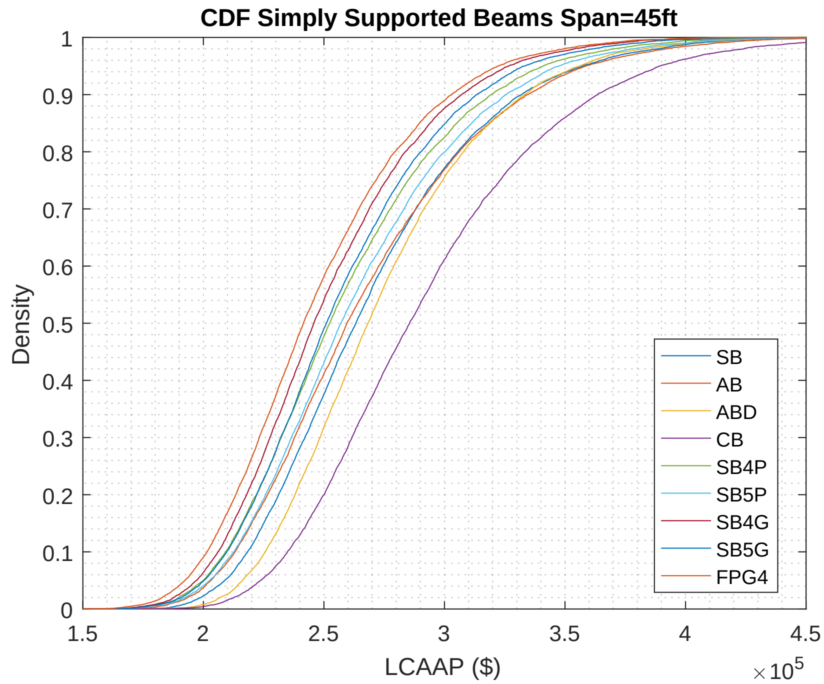


Fig. F.2. Simulation results, CDFs simply supported beams, span=45-ft.

Table F.2.
Stochastic dominance matrix - Simply supported beams, span=45-ft

Alternative	**SB**	**AB**	**ABD**	**CB**	**SB4P**	**SB5P**	**SB4G**	**SB5G**	**FPG4**
SB	-	0-1-1-1	0-0-0-0	0-0-0-0	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
AB	0-0-0-0	-	0-0-0-0	0-0-0-0	1-1-1-1	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
ABD	0-1-1-1	0-1-1-1	-	0-0-0-0	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
CB	1-1-1-1	1-1-1-1	1-1-1-1	-	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
SB4P	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-1-1	0-0-1-1	1-1-1-1
SB5P	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	0-0-1-1	0-0-1-1	1-1-1-1
SB4G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	1-1-1-1
SB5G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	1-1-1-1
FPG4	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-

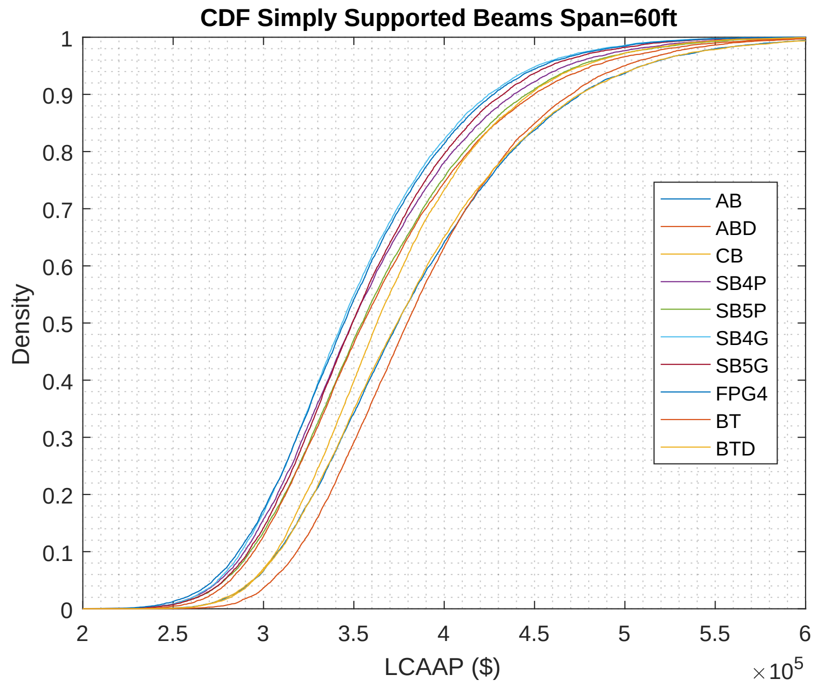


Fig. F.3. Simulation results, CDFs simply supported beams, span=60-ft.

Table F.3.
Stochastic dominance matrix - Simply supported beams, span=60-ft

Alter.	**AB**	**ABD**	**CB**	**SB4P**	**SB5P**	**SB4G**	**SB5G**	**FPG4**	**BT**	**BTD**
AB	-	0-0-0-0	0-0-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	0-1-1-1	0-1-1-1
ABD	0-1-1-1	-	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
CB	0-0-0-0	0-0-0-0	-	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	0-0-1-1
SB4P	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-1-1	0-0-1-1	1-1-1-1	0-0-0-0	0-0-0-0
SB5P	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	0-0-1-1	0-0-1-1	0-0-1-1	0-0-0-0	0-0-0-0
SB4G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0
SB5G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	-	1-1-1-1	0-0-0-0	0-0-0-0
FPG4	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0	-	0-0-0-0	0-0-0-0
BT	0-0-0-0	0-0-0-0	0-0-0-0	1-1-1-1	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	-	0-0-0-0
BTD	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	0-1-1-1	-

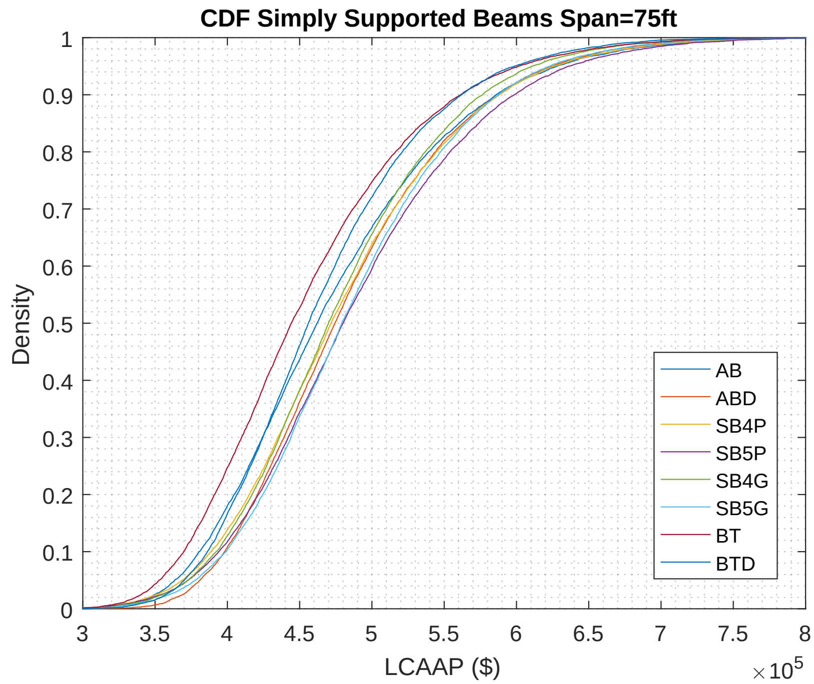


Fig. F.4. Simulation results, CDFs simply supported beams, span=75-ft.

Table F.4.
Stochastic dominance matrix - Simply supported beams, span=75-ft

Alternative	**AB**	**ABD**	**SB4P**	**SB5P**	**SB4G**	**SB5G**	**BT**	**BTD**
AB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-0-1-1
ABD	0-1-1-1	-	0-1-1-1	0-0-0-1	1-1-1-1	0-0-0-0	1-1-1-1	1-1-1-1
SB4P	0-0-1-1	0-0-0-0	-	0-0-0-0	0-0-1-1	0-0-0-0	0-0-1-1	0-0-1-1
SB5P	0-0-1-1	0-0-1-1	0-1-1-1	-	0-0-1-1	0-0-1-1	0-0-1-1	0-0-1-1
SB4G	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-1-1	0-0-1-1
SB5G	0-0-1-1	0-0-1-1	0-1-1-1	0-0-0-0	0-0-1-1	-	0-0-1-1	0-0-1-1
BT	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0
BTD	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	-

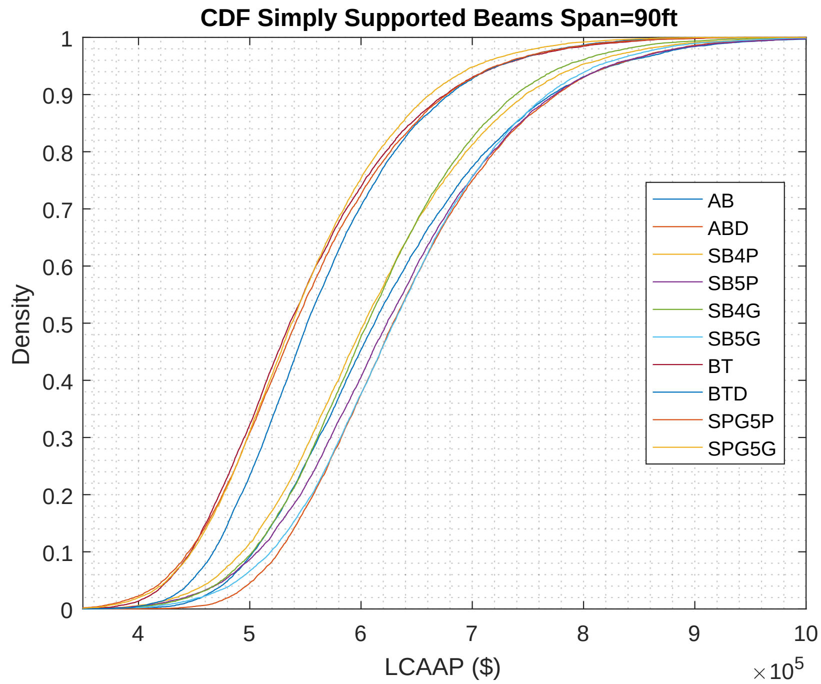


Fig. F.5. Simulation results, CDFs simply supported beams, span=90-ft.

Table F.5.
Stochastic dominance matrix - Simply supported beams, span=90-ft

Alter.	**AB**	**ABD**	**SB4P**	**SB5P**	**SB4G**	**SB5G**	**BT**	**BTD**	**SPG 5P**	**SPG 5G**
AB	-	0-0-0-0	1-1-1-1	0-0-0-0	0-1-1-1	0-0-0-0	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
ABD	0-1-1-1	-	1-1-1-1	0-1-1-1	1-1-1-1	0-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
SB4P	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1	1-1-1-1	1-1-1-1
SB5P	0-0-1-1	0-0-0-0	0-0-1-1	-	0-0-1-1	0-0-0-0	0-0-1-1	0-0-1-1	1-1-1-1	1-1-1-1
SB4G	0-0-0-0	0-0-0-0	0-1-1-1	0-0-0-0	-	0-0-0-0	0-1-1-1	0-0-1-1	1-1-1-1	1-1-1-1
SB5G	0-0-1-1	0-0-0-0	0-1-1-1	0-1-1-1	0-0-1-1	-	0-1-1-1	0-0-1-1	1-1-1-1	1-1-1-1
BT	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-0-1	0-0-1-1
BTD	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	-	0-1-1-1	1-1-1-1
SPG5P	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0	-	0-0-1-1
SPG5G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-1	0-0-0-0	0-0-0-0	-

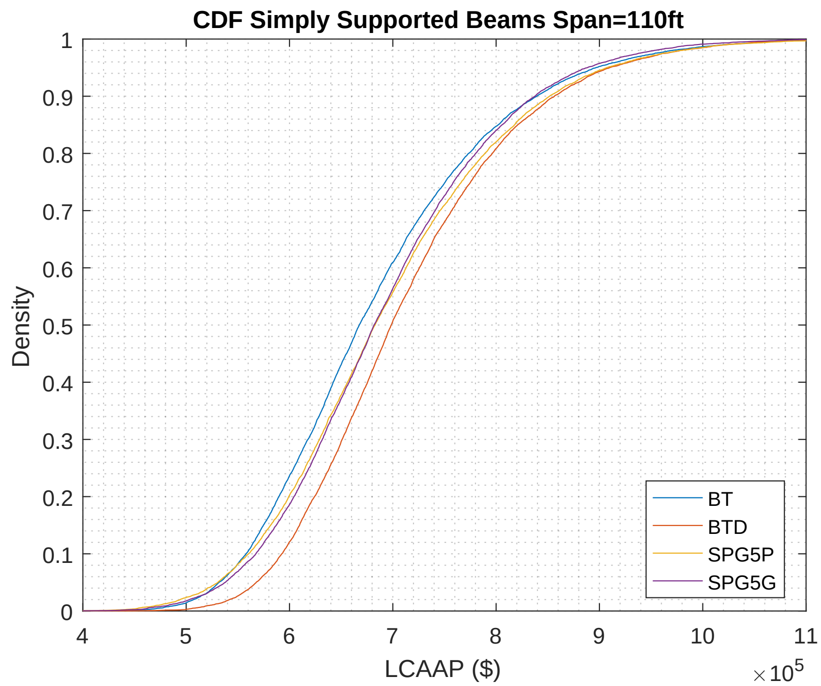


Fig. F.6. Simulation results, CDFs simply supported beams, span=110-ft.

Table F.6.
Stochastic dominance matrix - Simply supported beams, span=110-ft

Alternative	**BT**	**BTD**	**SPG5P**	**SPG5G**
BT	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-1-1-1	-	0-1-1-1	1-1-1-1
SPG5P	0-0-1-1	0-0-0-0	-	0-0-1-1
SPG5G	0-0-1-1	0-0-0-0	0-0-0-0	-

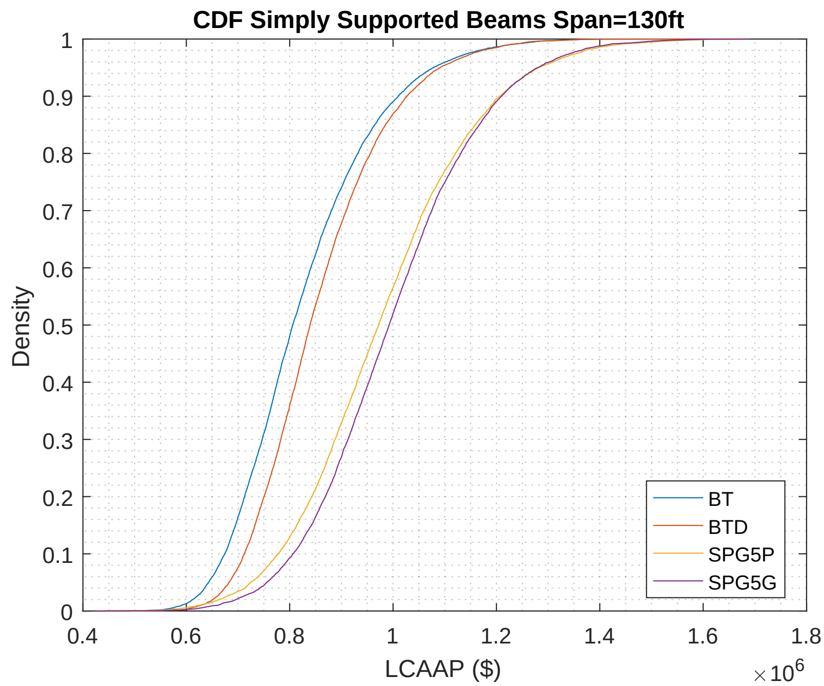


Fig. F.7. Simulation results, CDFs simply supported beams, span=130-ft.

Table F.7.
Stochastic dominance matrix - Simply supported beams, span=130-ft

Alternative	**BT**	**BTD**	**SPG5P**	**SPG5G**
BT	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-1-1-1	-	0-0-0-0	0-0-0-0
SPG5P	0-0-1-1	0-0-1-1	-	0-0-0-0
SPG5G	1-1-1-1	0-0-1-1	0-1-1-1	-

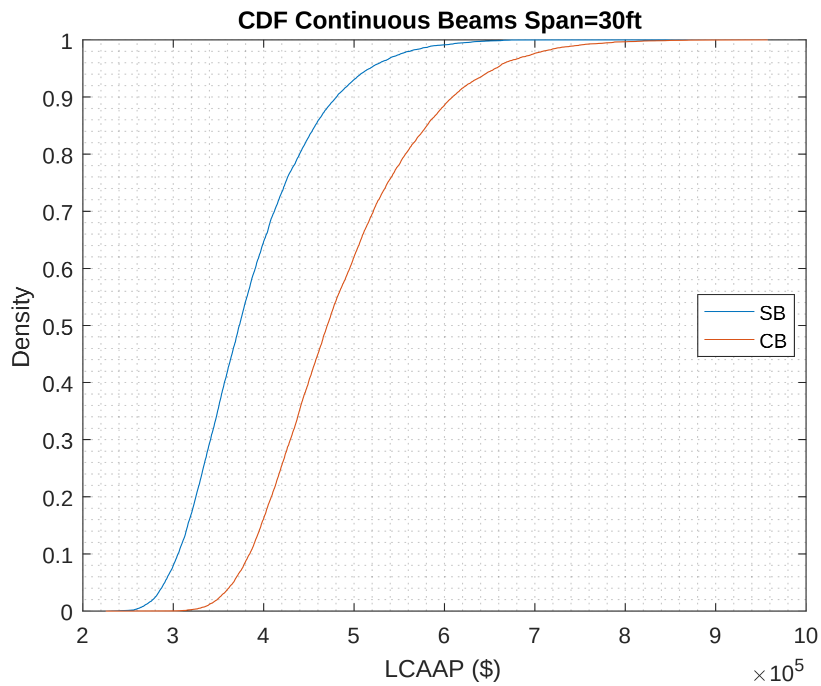


Fig. F.8. Simulation results, CDFs continuous beams, span=30-ft.

Table F.8.
Stochastic dominance matrix - Continuous beams, span=30-ft

Alternative	**SB**	**CB**
SB	-	0-0-0-0
CB	1-1-1-1	-

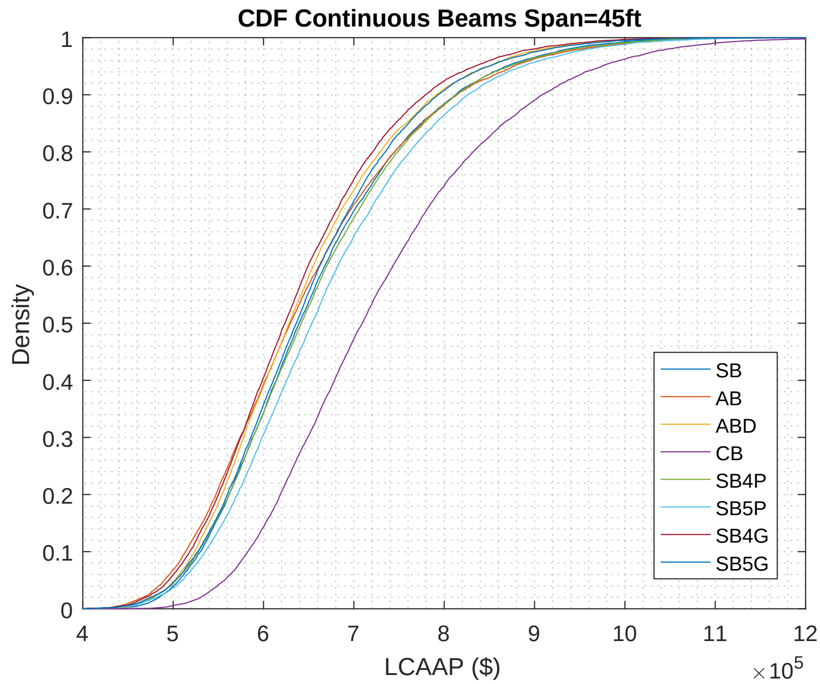


Fig. F.9. Simulation results, CDFs continuous beams, span=45-ft.

Table F.9.
Stochastic dominance matrix - Continuous beams, span=45-ft

Alternative	**SB**	**AB**	**ABD**	**CB**	**SB4P**	**SB5P**	**SB4G**	**SB5G**
SB	-	0-1-1-1	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-1-1-1
AB	0-0-0-0	-	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0
ABD	0-0-0-0	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-0-0-0
CB	1-1-1-1	1-1-1-1	1-1-1-1	-	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
SB4P	0-0-1-1	0-1-1-1	0-0-1-1	0-0-0-0	-	0-0-0-0	1-1-1-1	0-1-1-1
SB5P	0-0-1-1	0-0-1-1	0-0-1-1	0-0-0-0	0-0-1-1	-	0-1-1-1	0-1-1-1
SB4G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0
SB5G	0-0-0-0	0-0-1-1	0-0-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	-

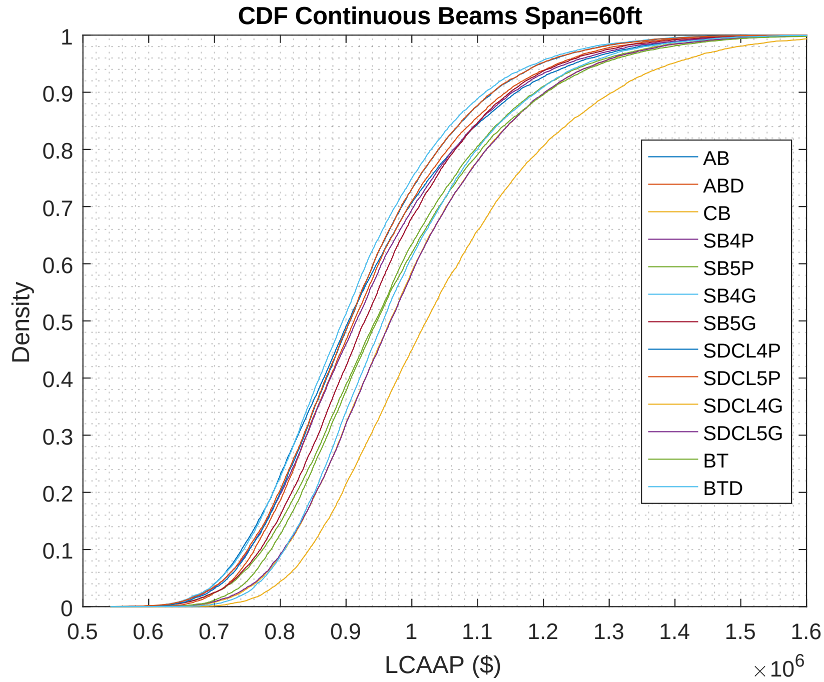


Fig. F.10. Simulation results, CDFs continuous beams, span=60-ft.

Table F.10.
Stochastic dominance matrix - Continuous beams, span=60-ft

Alternative	**AB**	**ABD**	**CB**	**SB4P**	**SB5P**	**SB4G**	**SB5G**
AB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-0-0
ABD	0-1-1-1	-	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-0-0-0
CB	1-1-1-1	1-1-1-1	-	1-1-1-1	1-1-1-1	1-1-1-1	1-1-1-1
SB4P	0-0-1-1	0-0-1-1	0-0-0-0	-	0-0-0-0	0-0-1-1	0-0-0-0
SB5P	0-0-1-1	0-0-1-1	0-0-0-0	0-0-1-1	-	0-0-1-1	0-0-1-1
SB4G	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0
SB5G	0-0-1-1	0-0-1-1	0-0-0-0	0-0-1-1	0-0-0-0	0-0-1-1	-
Alternative	**SDCL4P**	**SDCL5P**	**SDCL4G**	**SDCL5G**	**BT**	**BTD**	
SDCL4P	-	0-1-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	
SDCL5P	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	
SDCL4G	1-1-1-1	1-1-1-1	-	0-0-1-1	0-1-1-1	0-0-1-1	
SDCL5G	1-1-1-1	1-1-1-1	0-0-0-0	-	0-0-1-1	0-0-1-1	
BT	1-1-1-1	1-1-1-1	0-0-0-0	0-0-0-0	-	0-0-0-0	
BTD	1-1-1-1	1-1-1-1	0-0-0-0	0-0-0-0	0-1-1-1	-	

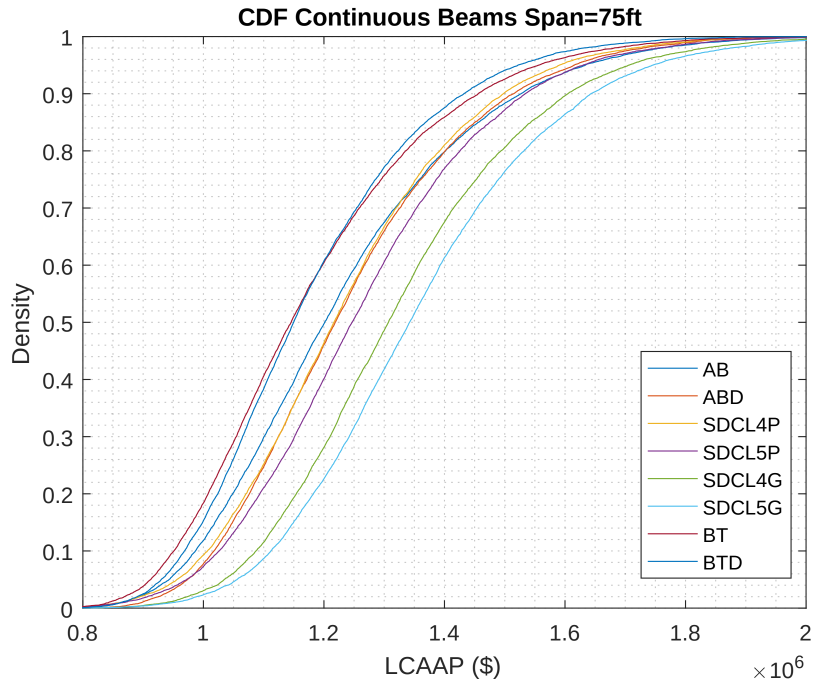


Fig. F.11. Simulation results, CDFs continuous beams, span=75-ft.

Table F.11.
Stochastic dominance matrix - Continuous beams, span=75-ft

Alternative	**AB**	**ABD**	**SDCL4P**	**SDCL5P**	**SDCL4G**	**SDCL5G**	**BT**	**BTD**
AB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	0-0-1-1
ABD	0-1-1-1	-	0-1-1-1	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	1-1-1-1
SDCL4P	0-0-1-1	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1
SDCL5P	0-0-1-1	0-0-1-1	0-0-1-1	-	0-0-0-0	0-0-0-0	0-0-1-1	0-0-1-1
SDCL4G	0-1-1-1	0-0-1-1	1-1-1-1	1-1-1-1	-	0-0-0-0	1-1-1-1	1-1-1-1
SDCL5G	0-0-1-1	0-0-1-1	1-1-1-1	1-1-1-1	0-0-1-1	-	1-1-1-1	1-1-1-1
BT	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	-	0-0-0-0
BTD	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0	0-1-1-1	-

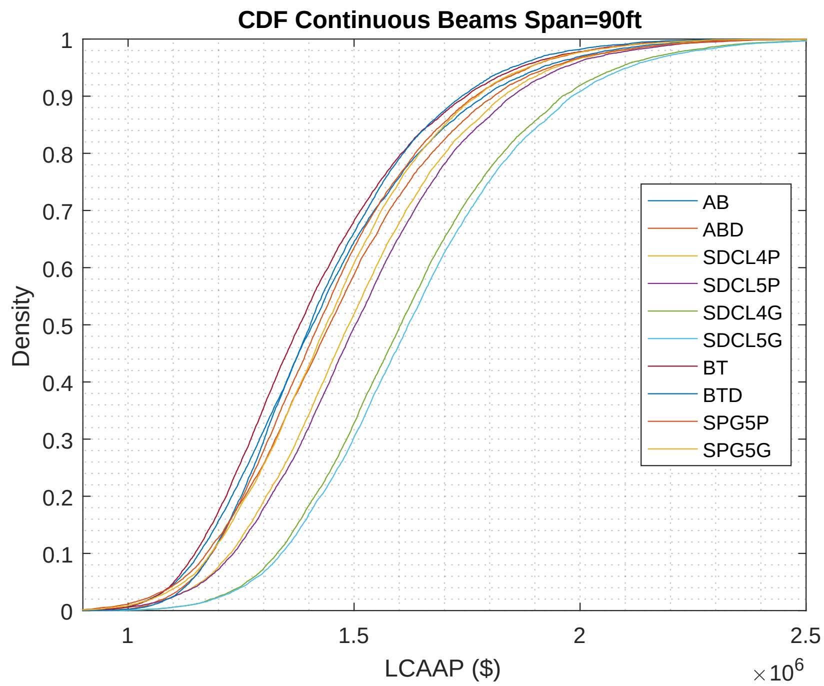


Fig. F.12. Simulation results, CDFs continuous beams, span=90-ft.

Table F.12.
Stochastic dominance matrix - Continuous beams, span=90-ft

Alternative	**AB**	**ABD**	**SDCL4P**	**SDCL5P**	**SDCL4G**
AB	-	0-0-0-0	0-0-0-0	0-0-0-0	0-0-0-0
ABD	0-1-1-1	-	0-0-0-0	0-0-0-0	0-0-0-0
SDCL4P	0-0-1-1	0-0-1-1	-	0-0-0-0	0-0-0-0
SDCL5P	0-0-1-1	0-0-1-1	0-0-1-1	-	0-0-0-0
SDCL4G	1-1-1-1	0-0-1-1	1-1-1-1	1-1-1-1	-
Alternative	**SDCL5G**	**BT**	**BTD**	**SPG5P**	**SPG5G**
SDCL5G	-	1-1-1-1	0-0-1-1	1-1-1-1	1-1-1-1
BT	0-0-0-0	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-0-0-0	0-1-1-1	-	0-0-0-0	0-0-0-0
SPG5P	0-0-0-0	0-0-1-1	0-0-1-1	-	0-0-1-1
SPG5G	0-0-0-0	0-0-1-1	0-0-1-1	0-0-0-0	-

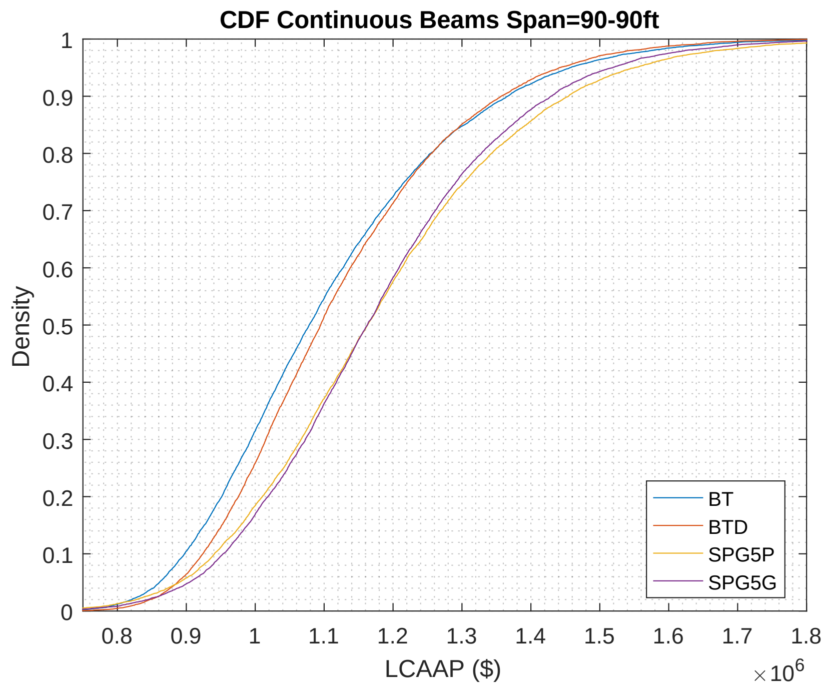


Fig. F.13. Simulation results, CDFs continuous beams, span=90-90-ft.

Table F.13.
Stochastic dominance matrix - Continuous beams, span=90-90-ft

Alternative	**BT**	**BTD**	**SPG5P**	**SPG5G**
BT	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-1-1-1	-	0-0-0-0	0-0-0-0
SPG5P	0-0-1-1	0-0-1-1	-	0-0-1-1
SPG5G	0-0-1-1	0-0-1-1	0-0-0-0	-

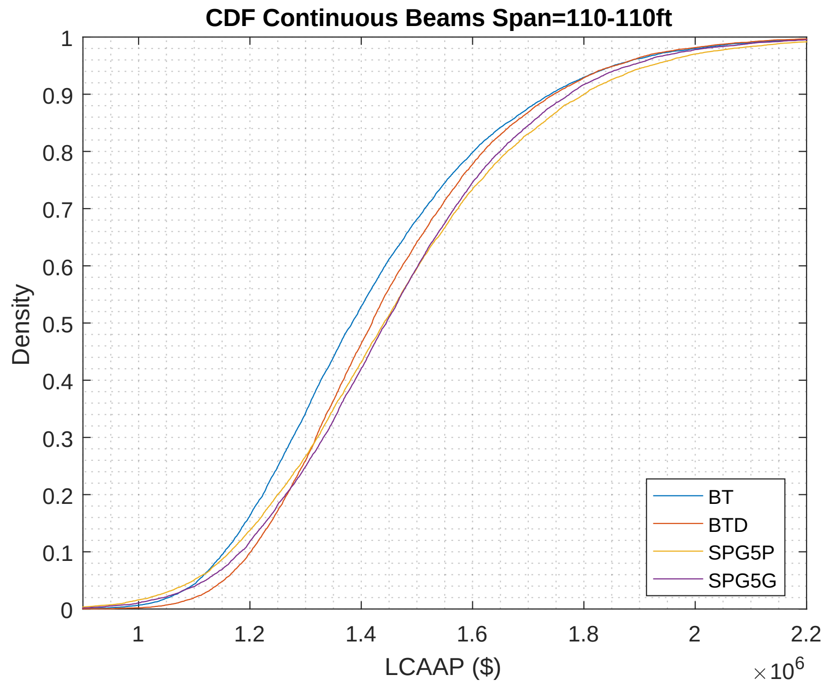


Fig. F.14. Simulation results, CDFs continuous beams, span=110-ft.

Table F.14.
Stochastic dominance matrix - Continuous beams, span=110-ft

Alternative	**BT**	**BTD**	**SPG5P**	**SPG5G**
BT	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-1-1-1	-	0-0-0-0	0-0-1-1
SPG5P	0-0-1-1	0-0-1-1	-	0-0-1-1
SPG5G	0-0-1-1	0-0-1-1	0-0-0-0	-

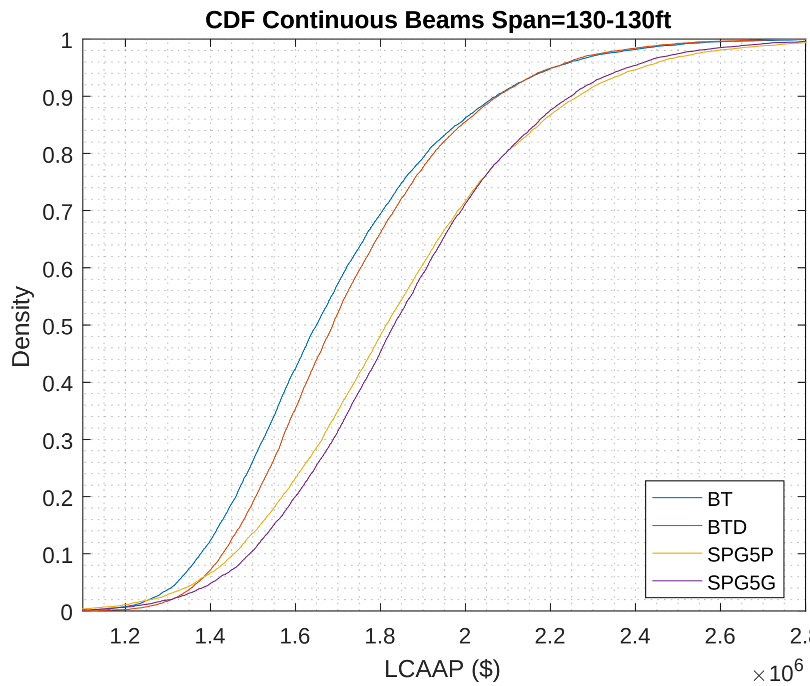


Fig. F.15. Simulation results, CDFs continuous beams, span=130-ft.

Table F.15.
Stochastic dominance matrix - Continuous beams, span=130-ft

Alternative	**BT**	**BTD**	**SPG5P**	**SPG5G**
BT	-	0-0-0-0	0-0-0-0	0-0-0-0
BTD	0-1-1-1	-	0-0-0-0	0-0-0-0
SPG5P	0-0-1-1	0-0-1-1	-	0-0-0-0
SPG5G	0-0-1-1	0-0-1-1	0-1-1-1	-

VITA

VITA

Stefan L. Leiva Maldonado was born in Bogota, Colombia on May 12, 1988. He obtained his undergraduate degree in Civil Engineering in March 2011 from Universidad Distrital Francisco Jos de Caldas, Bogot, Colombia. In August 2011 Stefan joined the Civil Engineering Masters program from Universidad Nacional, Bogot, Colombia, and obtained his Masters degree in 2015. During his studies Stefan has been working actively in the Colombian industry as structural designer. His experience in structural design in a wide range of projects and specialized in steel structures gave him the opportunity to join projects with huge impact in Colombian infrastructure as “El Quimbo dam”, “Sogamoso hydroelectric Project”, “CERROMATOSO S.A Mine workshop replacement”, and many others. In 2013 Stefan was awarded with the Fulbright-Colciencias fellowship to continue his studies in one of the most important universities in United States. Stefan joined Purdue University as PhD student in spring 2014 and worked in a project founded by the Indiana Department of Transportation (INDOT) focused on Life cycle cost analysis (LCCA) of concrete and steel bridges for short spans. He received his Ph.D. in civil engineering from Purdue University in August 2019.