Development of Secondary Route Bridge Design Plan Guides

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16. Abstract
The objective of this study is to develop a set of bridge plan guides for low-volume traffic roads. The purpose of the plans is to facilitate construction of new structures suitable for local agencies. The designs are to specifically address common local agency road, span, and site conditions. A series of initial bridge design concepts were first identified, and presented to MDOT and local agency bridge engineers, consultants, contractors, and fabricators for comments. Based on the comments received, the initial set was reduced to four designs: galvanized steel girders, spread box beams, side-by-side box beams, and bulb-tee girders. Deterministic and probabilistic life-cycle cost analyses (LCAA) were conducted to compare the long-term costs of the designs. After further input from the bridge engineering community, the plans were finalized and made available for use.

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EXECUTIVE SUMMARY

The advancing age and deterioration of local agency bridges leads to increasing maintenance concerns. In the near future, many of these bridges will require replacement. Some bridge replacement guidance is available by way of MDOT plans and design guides. However, these plans usually address larger structures with higher traffic volumes that are not always best suited for local agency use. Local agencies and their design consultants recognize this, and often base designs on previously constructed structures. This approach results in a wide variety of designs and details, some of which produce sub-optimal bridges. The objective of this study is to develop a design guide, in the form of bridge plans, for low-volume traffic roads. The designs are to specifically address common local agency road, span, and site conditions. Anticipated benefits from implementing the bridge plans include reduced design and construction uncertainties; bridges that are simple and practical to construct; improved quality control; and lower life cycle costs.

The first task of the research was to identify potential bridge types for plan development. The selection of initial concepts was guided by a review of plans developed by others, prior MDOT research documents, as well as an analysis of the local agency bridge inventory. From these assessments, twelve possible bridge concepts were identified. These concepts were presented to MDOT bridge engineers and a panel of local agency representatives, bridge consultants, contractors, and fabricators for comment. Additional comments were obtained from on-line and in-person surveys of a wider stakeholder audience. Based on these comments and additional analysis, four bridge types were identified to be most suitable for further development. These structures were galvanized steel, spread box beam, side-by-side box beam, and spread bulb-tee bridges. It was further determined that three bridge widths (30, 34, and 40 ft clear) suitable for lower traffic volumes were appropriate for development, with spans from 20-110 ft and skews from 0-30 degrees. After initial design, a deterministic and probabilistic life cycle cost analysis (LCCA) was conducted on the candidate designs. The LCCA includes costs and activity timing for initial construction, inspection, repair and maintenance, demolition, replacement, and associated user costs. The analysis found that these four types had similar initial and life cycle costs, with side by side box beams generally most expensive; steel most effective at shorter spans, and bulb tees most effective at longer spans. It was further determined that steel bridges were suitable for spans up to about 60 ft, bulb tees from spans greater than 70 ft, and box beams throughout the span range considered. Balancing needs for economy, constructability, and minimizing beam depths, recommended galvanized steel structures have 5-7 beams spaced approximately 6.3 ft; spread boxes have 4-6 beams spaced from 6.9-9.4 ft; bulb tees have 4-5 beams spaced from 7.9-9.2 ft, and side by side boxes have 8-13 beams, depending on deck width and span length. The refined designs were again presented to a panel of experts for further comment. Considering this final input, the designs were refined and finalized into plan sets for distribution.

The plans are to be released as design templates in a commonly available electronic format. Accompanying the plans is a document that discusses design considerations, provides initial depth/span length selection information, initial cost estimates, and instructions on use of the plans.

CHAPTER 1: INTRODUCTION

Statement Of The Problem

Local agencies own and maintain over 6000 bridges in Michigan, and approximately 15% of these structures have been rated as structurally deficient, where either the deck, superstructure, or substructure was rated in "poor" (4) condition or lower, or if the appraisal rating for adequacy indicates a high priority for replacement (2 or less) (FHWA 2011; MDOT 2015). Of these locally owned structures, most are steel or prestressed concrete girder bridges, and the large majority span from 30 to 100 ft in length (Nowak et al. 1998). As over 70% of locally-owned bridges were built from the 1920s-1980s (Nowak et al. 1998), many of these structures are nearing the end of their service life and require replacement, as evidenced by the significant number that have been found to be structurally deficient.

Although local agencies have traditionally relied upon MDOT standards for guiding bridge design, such plans, generally intended for trunkline use, are likely suboptimal for the conditions faced by local bridges, which are often found on lower volume roads. This is an important issue, as recent research found that traffic volume is a primary factor that influences the long term life cycle cost of Michigan bridges (Eamon et al. 2012). Given the number of local bridges which are nearing replacement age, it is desirable to formulate a new set of plan guides that have been developed with low life cycle costs for the specific conditions found on local roads and using local construction resources.

In recent years, various state DOTs and researchers have developed standard plans for bridge replacement, such as under Missouri's Safe & Sound bridge replacement program (MoDOT 2011), PennDOT's Rapid Bridge Replacement project (PennDOT 2015), and the Transportation Research Board's standard plans of prefabricated bridge components, which were used for replacing some structures in Iowa, Vermont, and New York, among others (Rossbach 2014; TRB 2014). However, such efforts, although useful, are not Michigan-specific. That is, local construction practices, material availability and costs, the familiarity and knowledge base of contractors, fabricators, and engineers, Michigan load effects on bridges, as well as geological and environmental conditions, may differ from the generic or other state-specific conditions that the developers of these existing plans had in mind. Such differences render this previous work sub-optimal for Michigan, and relying on existing plans without modification will result in less effective designs. To address this issue, there exists a need to develop a set of cost-effective bridge plan guides for use by local bridge owners in Michigan.

Objectives Of The Study

The goal of this study is to address the problem above. The research objectives are to:

1. Evaluate the life cycle costs of various alternative single span bridge designs suitable for secondary roads in Michigan.

2. Determine the most feasible and cost effective designs, considering constructability, long term serviceability, maintenance, and other important performance metrics.

3. Develop plan guidance and specifications for design and construction of these optimal structures.

4. Educate potential bridge owners and their design and construction partners on the benefits of the developed bridge plans and how to implement them.

Summary Of Research Tasks

This research is composed of the following tasks:

Task 1. State-of-the-art literature review.

Task 2. Identify a series of cost-effective candidate bridge designs and details suitable for low-volume roads in Michigan.

Task 3. Hold stakeholder focus groups to evaluate the feasibility of the candidate designs.

Task 4. Conduct detailed probabilistic life cycle cost analysis on the candidate designs.

Task 5. Develop plan guides and specifications for the optimal designs.

Task 6. Develop an education plan to inform stakeholders of the benefits and methods of implementation of the bridge designs.

Task 7. Prepare project deliverables.

CHAPTER 2: LITERATURE REVIEW

A literature review was conducted to identify relevant standards, research, and best practices pertaining to secondary route bridge plan standardization for low life cycle costs. The review included a broad search of technical engineering journals, conference proceedings, technical reports, and standards. Of particular interest were sources describing types of standardized bridge systems proposed and currently in use; the use of innovative but practical configurations, materials and construction methods; bridge systems that are cost effective, constructible and durable; and bridge LCCA in general. A brief summary of approximately 60 documents found to be most relevant to these topics are discussed below. In general, it was found that standardized plans focused on the use of precast components and alternative materials such as high performance concrete. Concepts that have been found to have low life cycle costs include double-tee members; the use of low maintenance or galvanized steel girders; high performance concrete deck materials; stainless steel or stainless-clad bars; and the minimization of maintenance events.

Plans and Design Approaches for Bridge Replacement

To reduce costs, increase quality control, and enhance the speed of construction, various agencies have proposed, developed, and implemented standard plans for bridge construction. Approaches have been proposed from a wide range of sources, from individual researchers to national organizations such as NCHRP as well as industry. Numerous DOTs have implemented these concepts, in addition to unique state-specific plans. A summary of these efforts is presented below.

In Report S2-R04-RR1, *Innovative Bridge Designs for Rapid Renewal*, under the Second Strategic Highway Research Program (SHRP2), the Transportation Research Board (TRB) developed a "design toolkit" for prefabricated bridge components aimed at state and local agencies (TRB 2014). As with other recent efforts of this kind, the toolkit focuses on standard bridge designs, and includes such plans for foundations, substructure, and superstructure systems meant for fast and efficient construction.

The plans were developed to allow construction of bridges that were structurally efficient and simple to erect (Starnes 2013). The toolkit was tested in 2011 by Iowa DOT in a trial program where a bridge was replaced in 2 weeks. The demonstration structure included a superstructure module of a precast concrete deck on steel beams; a substructure of precast pier columns and caps as well as abutment stem and wing walls; and a prefabricated bridge approach system of precast concrete panels and a sleeper slab. Figure 2.1 illustrates a sample bridge composed of prefabricated components that could be developed from the proposed plans.

Typical designs for superstructure and substructure modules have been grouped into the following spans: 40 to 70 ft; 70 to 100 ft; and 100 to 130 ft. The structures were designed for HL-93 load. Superstructure components made use of high performance concrete (HPC) with compressive strength of 8,000 psi. Also provided were construction drawings, details, and design calculations.

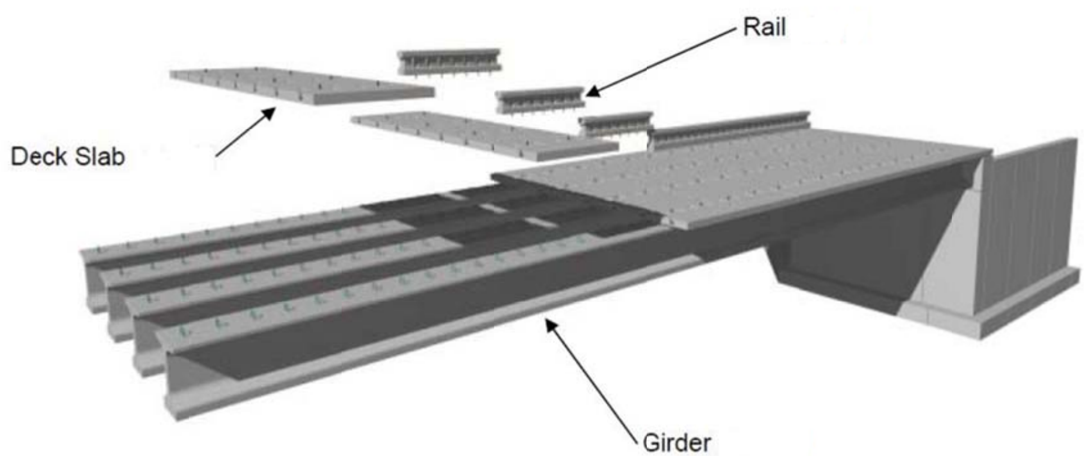

Figure 2.1. Components for Typical Substructure and Superstructure Construction (reproduced with permission from PCI).

Vermont DOT also made use of the SHRP2 plans, in conjunction with PCI Northeast Extreme Tee (NEXT) beams to replace 17 bridges damaged by Hurricane Irene (Figure 2.2) (Culmo and Seraderian 2010). These structures used steel H-pile foundation elements and precast abutments. Similarly, New York DOT used the SHRP toolkit and NEXT beams to replace the eastbound and westbound I-84 bridges over Dingle Ridge Road. These structures used ultra-high performance concrete (UHPC) closure pours to join the NEXT beams (TRB 2014). The NEXT beams have been used by various DOTs including those of CT, MA, ME, NH, NY, RI, VT, and PA. The beams are suitable for spans from 30 – 90 ft (Figure 2.3) and are available in depths of 24-36 inches, with web widths from 8-12 inches.

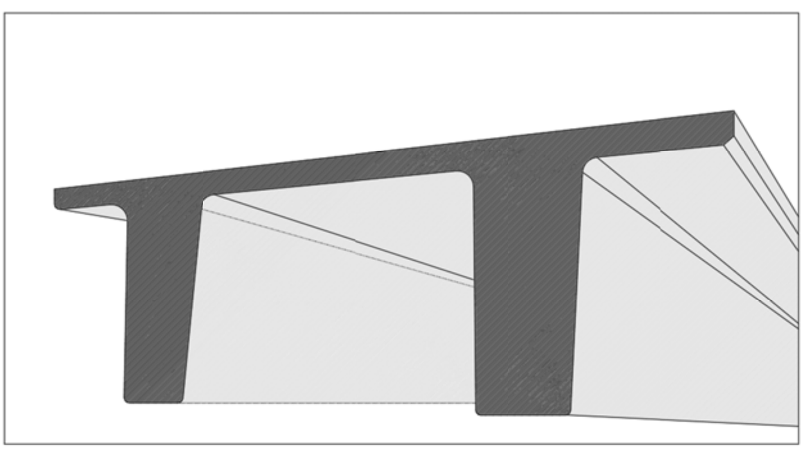

Figure 2.2. Precast NEXT Beams.

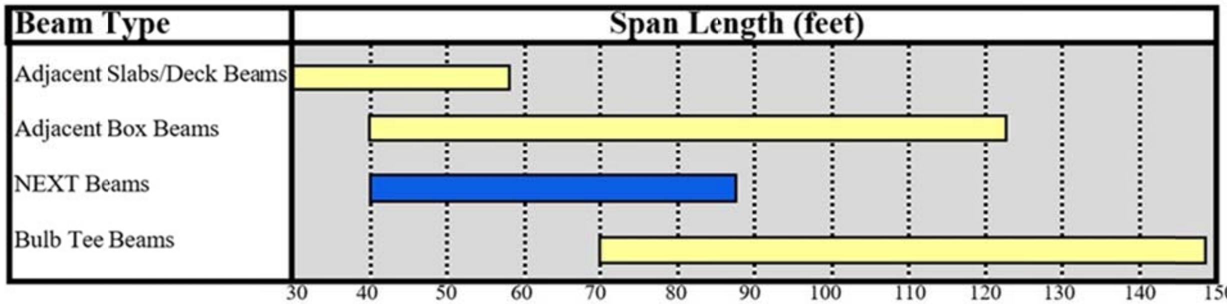

Figure 2.3. The Use of NEXT Beams Compared To Other Member Types (Redrawn from High Bridge Team 2016).

Pennsylvania DOT developed a set of standards using NEXT beams as well, providing options for precast and cast in place decks, as shown in Figures 2.4 and 2.5 (PennDOT 3013).

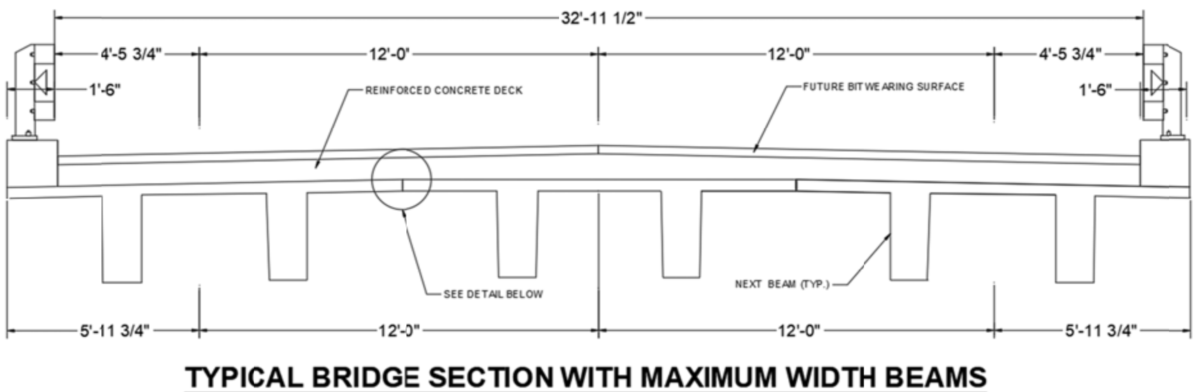

Figure 2.4. Beam Flange Serving As a Form for a CIP Deck (Redrawn from High Bridge Team 2016).

PCI published the *Guidelines for Accelerated Bridge Construction Using Precast/Prestressed Concrete Components* (PCI 2006), which were later summarized by Burak and Seraderian (2010). The manual was developed from consideration of the needs of bridge owners, engineers, and contractors, with respect to the design, fabrication, planning, and constructing prefabricated concrete highway bridges. The manual contains various recommendations for construction. For example: Non-skewed designs are preferred (Sec. 2.3.1), and PC or H-pile foundation elements are recommended (Sec. 3.1); and round columns are to be avoided in favor of rectangular elements (Sec 3.2.3).   It also suggests that several types of precast concrete retaining walls be considered, including approved proprietary walls; precast cantilever walls; the use of mechanically stabilized earth (MSE), and precast modular block gravity walls. To support girders, the manual provides suggestions for pier caps, integral abutment pile caps, and seat adjustment beams. It notes that prefabricated decks and the use of stay-in-place formwork are preferred (Sec. 3.3).

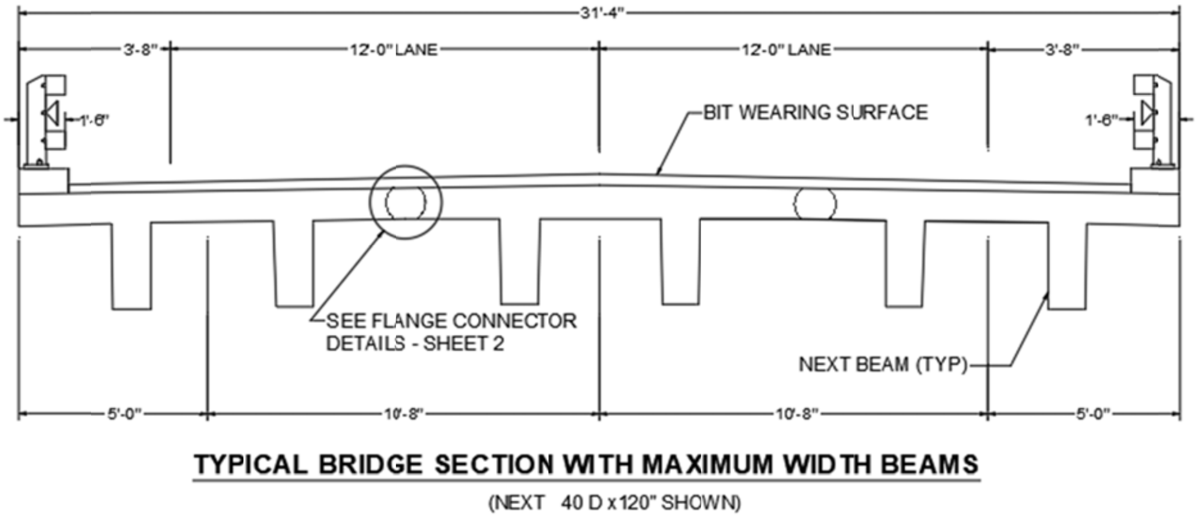

Figure 2.5. Compete Beam Flange Serves as Deck with Overlay (Redrawn from High Bridge Team 2016).

The manual also provides recommendations for joints (Sec 4.3), where embedded mechanical couplers, cast-in-place closure pours, and post tensioning with match-cast components are suggested to form moment connections. For shear connections, vertical or horizontally grouted keys and reinforced dowels are discussed.

The last three sections of this manual summarize case studies. The first case study concerns construction of a 65-ft single span bridge by Maine DOT using self-consolidating concrete with a precast box beam superstructure. The bridge uses pile-supported, integral abutments and was completed in four days. A detail of the abutment is given in Figure 2.6.

The second case study involved Maine DOT as well, where a single span bridge was constructed using all precast elements including abutments and wing walls. These were designed as post-tensioned units and rested on driven piles. The bridge was completed in 30 days with a budget of 1$ million. Figures 2.7-2.9 illustrate some details of this bridge.

The third case study discussed the Mill Street Bridge project, constructed in New Hampshire. The project won two awards; for the Best Bridge design for spans between 20 and 41 m, and for the Best All-Precast Solution.

Similarly, Iowa DOT research recently produced standard plans for prefabricated bridge components to speed construction time, reduce costs, and allow local contractors to assemble the structures (Rossbach 2014). The prefabricated bridge components were to be based on two common low volume roadway widths (24 and 30 ft), left and right skews of 0, 15, and 30 degrees, and span lengths from 30 to 70 ft, in 5 to 10 ft increments. To facilitate transportation and construction, the components were limited to 45 kips self-weight. Also included were cast-in-place and precast abutment options. In 2015, Pennsylvania DOT initiated the Rapid Bridge Replacement (RBR) project to replace over 500 structurally deficient bridges within three years.

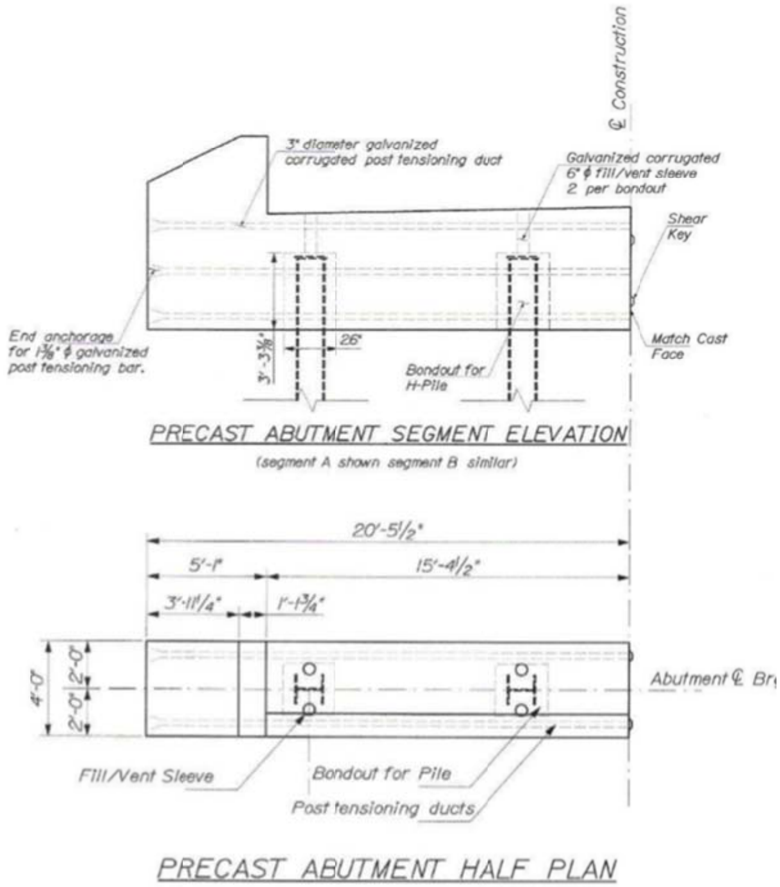

Figure 2.6. Abutment Details for Maine DOT Case Study Bridge (reproduced with permission from PCI).

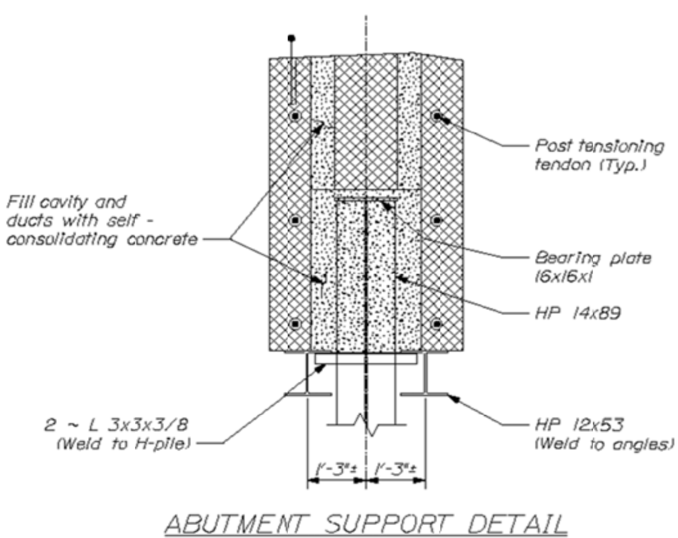

Figure 2.7. Precast Abutment Section Showing Void And PT Bar Locations (reproduced with permission from PCI).

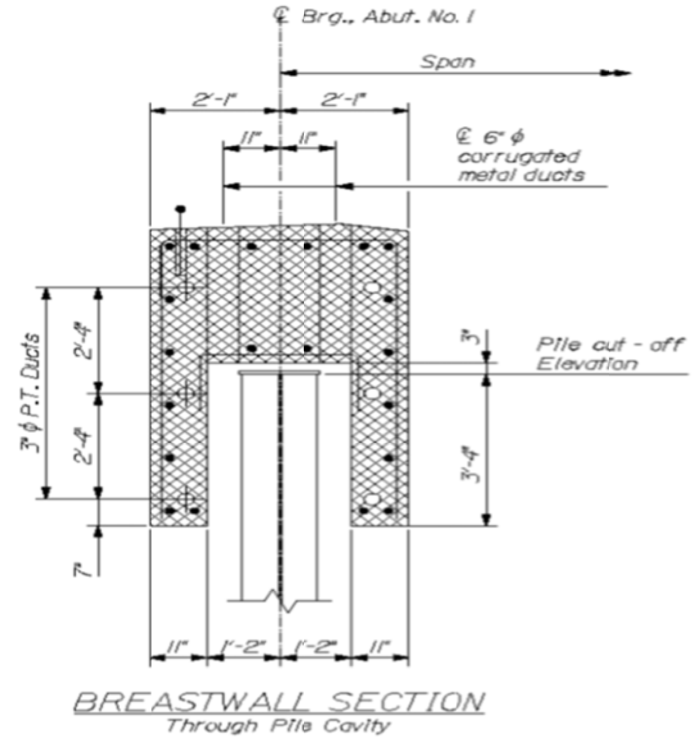

Figure 2.8. Precast Abutment Geometry (reproduced with permission from PCI).

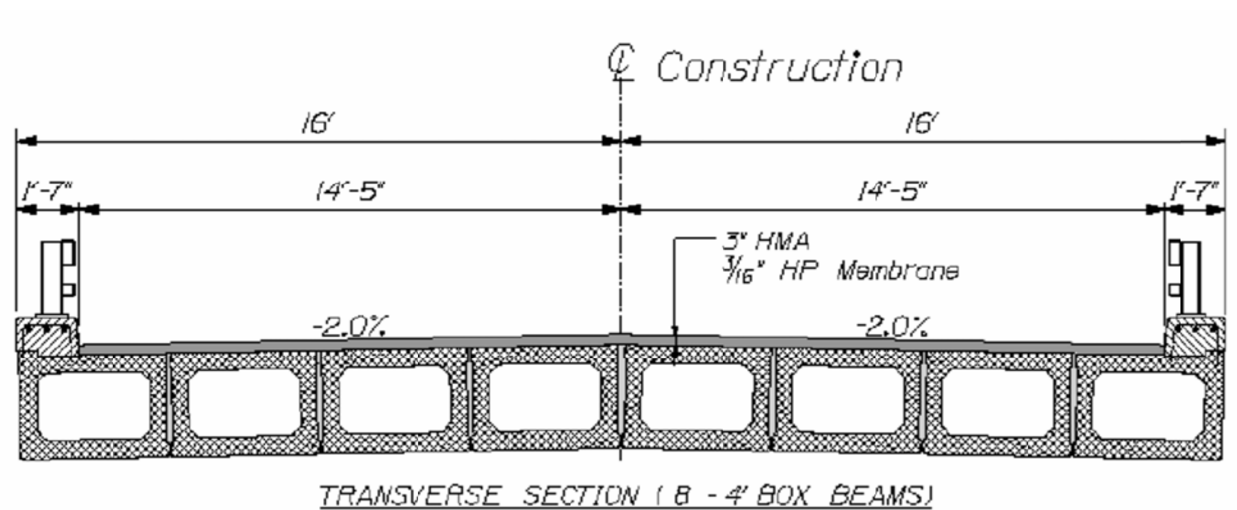

Figure 2.9. Superstructure Cross-Section (reproduced with permission from PCI).

Similar to Iowa and Missouri, a set of standardized bridge designs were developed based on prefabricated components to increase construction speed (PennDOT 2015). In 2013, Missouri DOT finished its “Safe & Sound” program, which involved developing standardized plans and completely replacing over 550 bridges, as well as replacing the decks of nearly 250 more structures, all within five years. Rapid construction was a priority for the program, where the average closure time was 42 days, and some single-span structures were replaced within 8 days

(MoDOT 2011). Related efforts include those by the South Dakota and Indiana DOTs to develop integral abutment bridges for standardized and fast construction, and those of New York DOT to develop guidelines for use of UHPC to join superstructure elements together for enhanced durability and rapid construction (Rossbach 2014). The Utah, Idaho, and Washington State DOTs have also developed sets of pre-engineered bridge plans, specifically for precast construction (TRB 2014).

Aktan et al. (2014) summarized a strategic plan for promoting accelerated bridge construction in Michigan, and discussed the guidelines needed for such an effort, including those for scoping, decision-making, planning and cost, structural analysis and design, contracting, equipment selection, scheduling, contingency plans, demolition methods, construction, and others.

Fowler (2006) summarized concrete bridge construction using prefabricated bridge elements and systems (PBES), and discussed potential advantages of PBES. These include: a reduction of the impact that bridge construction has on traffic as well as a decrease in life-cycle costs; an increase in construction safety; and improvement of the quality and constructability of bridge designs. Much of these advantages come from the removal of significant quantities in-situ work to the controlled environment of the fabricator. For illustration, several case studies were discussed, including four long-span bridges in San Juan, Puerto Rico, which are from 700-900 ft in total length. Another example presented is the Mitchell Gulch Bridge, a 40 ft single span Colorado DOT structure, where the foundation included driven piles which supported substructure units that were welded together after installation. Precast slabs were also used, where deck joints were grouted.

A similar use of precast elements was described by Ranasinghe (2014), who presented the design and construction of a structure with two-column straddle bents and post-tensioned segmental cantilever piers and wall piers. It was found that such use of precast elements were suitable for accelerated construction practices.

Sponsored by Colorado DOT, Outcal et al. (2014) documented the construction of single span bridge on State Highway 69 over Turkey Creek. This project concerned the use of accelerated construction techniques utilizing precast box beams and precast pier caps. This bridge used six disc bearings (three per abutment) to connect the pier beam to the column cap. These discs were meant to serve as pinned connections. The work involved installing precast pier caps, bearings and girders. The project was finished within 8 days. Construction did involve some unique techniques, however. Installing the precast pier caps required temporary stays to be used to fix the pier cap to the columns in a bent as shown in Figures 2.10 and 2.11.

Burgueno and Pavlich (2008) used finite element models to investigate prefabricated steel/concrete box girder and deck units, and found that these systems can be safe and viable for short-span highway bridges. It was also found that it is possible to maintain continuity between components for a range of spans and girder spacings without requiring to exceed AASHTO limits for transverse post-tensioning. Also using finite element methods for analysis were Chen and Bao (2014), who evaluated the effectiveness of precast diaphragms. They found that if constructed properly, such diaphragms could perform well even under seismic forces.

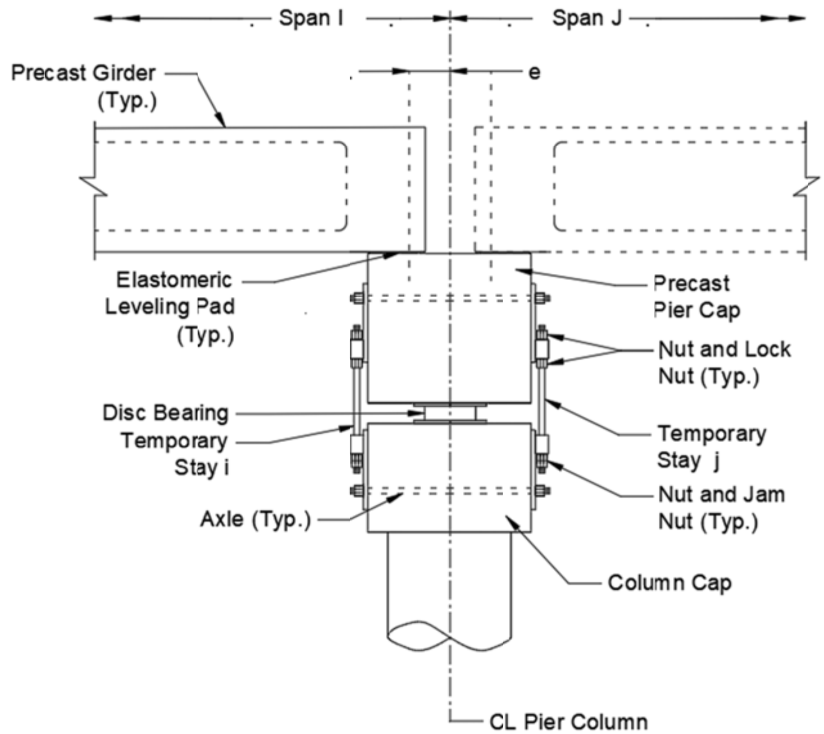

Figure 2.10. Typical Pier Section (Redrawn from Outcal et al. 2014).

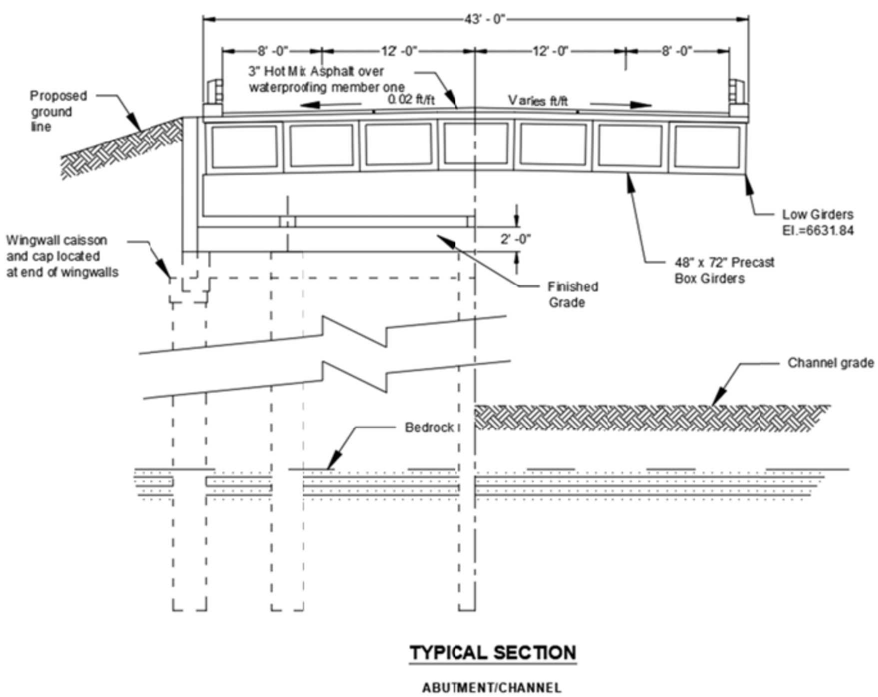

Figure 2.11. Bridge Cross-section (Redrawn from Outcal et al. 2014).

Pierce and Kirtan (2015) monitored and analyzed two Southington I-84 bridges in Connecticut built using precast components. The superstructures for the bridges were built offsite and then moved to the site for installation. The bridges were 100 ft long and composed of ten PCEF 47 prestressed concrete girders and an 8.5 inch deck. Both bridges used diaphragms at quarter point locations, and the decks were placed with a 6.25% cross slope. The authors conducted multiple finite element analyses of the construction process to ensure damage to the components would be avoided during construction. The two bridges were replaced within 36 hours, and met CTDOT's requirements for a 75-year design life.

NCHRP Report 733 (Cousins et al. 2013) suggested changes to the AASHTO LRFD Bridge Design Specifications (2010) and the AASHTO LRFD Bridge Construction Specifications (2010) to accommodate use of high-strength lightweight concrete girders and high-performance lightweight concrete decks, which are often used in conjunction with precast bridge components. The research focused on developing mixtures made with normal weight fine aggregates and manufactured lightweight shale, clay, or slate coarse aggregates to limit concrete density to 125 lb/ft^3. It was found that use of such material for precast girders and decks could result in a reduction of structural dead load and reduced construction, handling and transportation effort.

In NCHRP Report 679 (Shahrooz et al. 2011), recommendations were made to allow the use of high-strength reinforcing steels, with specified yield strengths up to 100 ksi. It was concluded that the existing AASHTO Design Specifications could be used without significant modification for steels up to 100 ksi yield strength, and that service-load strains are generally higher than those when conventional Grade 60 steel is used. However, based on consideration of flexural tests, deflections, and crack widths at service load levels, serviceability metrics were still predictable using current specifications, and designs using high strength steel could be within accepted serviceability limits.

Missouri DOT (Thiagaraja et al. 2013) studied cost efficient designs for bridge approach slabs, and three designs were recommended: 1) For new construction on major roads, a 20 ft cast-in-slab with sleeper slab design (CIP20SLP); 2) For replacement and new construction on major and minor roads, 25 and 20 ft prestressed slabs with a sleeper slab (PCPS20SLP); and for new cast in place construction on minor roads, a 25 ft modified slab without a sleeper slab (CIP25NOSLP). Details for these slabs are provided in Table 2.1. Figure 2.12 shows the PCPS20SLP design and a cross section of the sleeper slab which connects the slab and the roadway.

Table 2.1. Reinforcement Summary.

BAS Type	Span (ft)	Depth (in)	Cover (in)	Bottom Reinforcement (main/distribution)	Top Reinforcement (longitudinal/transverse)
CIP20SLP	20	12	3	#6@5" / #5@12"	#4@18" / #5@12"
CIP25NOSLP	25	12	3	#6@8" / #4@12"	#5@12" / #4@12"
PCPS20SLP	20	10	3.5	#6@12" + 12 PS strands / #5@18"	6 PS strands / #5@18"

Upon further study, it was found that the CIP designs allowed small rotations and longitudinal deflections under traffic, but such movements were not large enough to produce a hazardous condition for motorists. Visual inspections revealed no damage or distress in the PCPS slabs after eighteen months. However, the CIP slab on a bridge with no sleeper slab developed some differential movement between the slab and the asphalt pavement, generating a longitudinal crack. However, the slab remained flush with the bridge surface. Testing also indicated that the prestressed slab panels were fully supported on the base material, and failed at a capacity 44% higher than the design moment. It was further determined that cost savings resulting from the CIP designs ranged from 12.5 and 28.3 percent over the current design. For the PCPS design, the cost savings ranged from 3.3 and 15.3 percent.

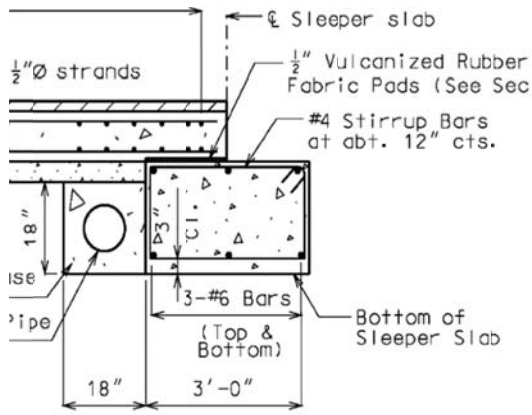

Figure 2.12. Placement of PCPS20SLP Panels (left) and Sleeper Slab Design (right) (reproduced with permission from Missouri DOT) .

Barth et al. (2013) developed standardized bridge plans for steel girder bridges to reduce design time, lower cost, provide a long life span, and lower maintenance needs. Three types of sections were recommended, depending on bridge span, and selections were pre-made for spans from 40 to 140 ft in 5 ft increments. These section types are rolled beams, homogeneous plate girders, and hybrid plate girders. Design selection charts were based on two approaches: the lightest weight possible and the lightest weight possible with a maximum allowed section depth. It was found that rolled sections are most efficient for spans from 40 to 100 ft; hybrid plate girders most effective from spans of 80 to 140 ft, and homogeneous plate girders from spans of 60 to 140 ft.

To briefly summarize the research discussed above, standardized plans appear to have focused on the use of precast components including girders, piers, columns, abutments, wing walls, and decks; simple span structures; high performance concrete pours to join components; double-tee spanning members; and stay-in-place formwork when site-casting is necessary.

Life Cycle Cost Analysis

It is well known that a design alternative with the lowest initial cost does not necessarily produce the lowest cost over the lifetime of the structure, particularly once the costs associated with maintenance and replacement are included. Life cycle cost analysis (LCCA) provides a formal method to quantify these costs as they accumulate over time. A extensive body of LCCA research exists in the literature. The following section provides a summary of the LCCA literature most relevant to bridges.

Total life cycle cost (LCC) includes costs and assessment of activity timing for initial construction, inspection, repair and maintenance, demolition, replacement, and, if desired, the associated user costs. As shown by various research (Eamon et al. 2012; Grace et al. 2012;

Zayed at al. 2002; TRB 2003), it is important to consider uncertainties in LCCA for accurate assessment, as significantly different results may be found if uncertainty information is neglected. For example, Zayed et al. (2002) applied deterministic LCCA on an example steel bridge girder to find that the least costly solution was to repaint at intermediate intervals, while when including uncertainties, a corresponding probabilistic LCCA determined that repainting is in fact best at the end of the paint life only. Therefore, neglecting uncertainties may lead the agency to adopt a sub-optimal solution. By using this approach, another advantage can be realized; probabilistic LCCA inherently quantifies the risk involved in adopting one design alternative over another, information which cannot be quantified with a deterministic LCCA. Important sets of random variables (RVs) for the probabilistic LCCA include those for maintenance activity timing, agency costs, and user costs. The LCCA process can be divided into 5 steps.

The first step is to determine activity timing. As suggested by TRB (2003), the analysis period must be long enough to include major rehabilitation actions for each reinforcement alternative. To satisfy this requirement, the LCCA should be conducted up to 75-100 years. However, results are available cumulatively for each year, so the LCC for any lesser period of time can be referenced. For consistent LCC comparison among cases, it is important that the maintenance actions are scheduled such that the expected bridge condition, at any year, is the same for all alternatives. Operation, maintenance, and repair (OM&R) strategies may be based on current and expected practices. RVs representing maintenance activity timing should include the timing (e.g. in years since construction or last event) of events such as: superstructure replacement; deck and beam replacement; deck overlay; beam end repair; deck patch, and/or others, as appropriate for the particular bridge system considered. Mean values for activity timing can be based on current maintenance scheduling practices, while coefficients of variation (COV) and an estimate of distribution type, also needed for the probabilistic analysis, can be calculated from a sample of bridges on secondary roads for which historic maintenance scheduling information is available. If innovative materials are considered in a bridge alternative, such as UHPC or corrosion-resistant reinforcement, appropriate OM&R strategies may not have been developed, and other sources with such experience must be consulted to estimate appropriate maintenance schedules (Eamon et al. 2012).

The second step is to determine agency costs. Agency costs and associated RVs include items such as material, personnel, and equipment costs corresponding to events such as: initial construction, routine and detailed inspections, deck patch, deck overlay, deck replacement, beam end repair, beam replacement, superstructure demolition, superstructure replacement, and others, as appropriate for the bridge system considered. Many of these costs are based on a combination of sub-costs. RV data for material and construction costs (mean, COV) can be computed from agency estimates and available records, as well as other sources if unavailable (Saito et al. 1988; Skitmore and Ng 2002; Sobanjo and Thompson 2001).

The third step is to determine user costs. During construction and maintenance work, traffic delays as well as increased accident rates occur. The resulting delay costs include the value of time lost due to increased travel time as well as the cost of additional vehicle operation. Therefore, mean user cost is taken as the sum of travel time costs, vehicle operating costs, and crash costs. Equations are available in the literature to estimate these costs, as a function of:

length of affected roadway, traffic speed during road work, normal traffic speed, number of days of road work, hourly time value of drivers, hourly vehicle operating cost, cost per accident, accident rate during construction and normal accident rate per million vehicle-miles (Eamon et al. 2012). Data for these parameters are available in the technical literature (Ehlen and Marshall 1996; Ehlen 1999; Huang et al. 2004; USDOT 2002; USDOT 1997; USDOT FHWA 2007; FHWA 2005). Other user costs to be potentially considered are those associated with emissions, environmental damage, noise pollution, and effects on local businesses, as possible. Thus, for each maintenance activity specified, an associated user cost RV should be characterized.

Once these cost models are developed, the life cycle cost can be determined. The total life cycle cost is the sum of all yearly partial costs. Because dollars spent at different times have different present values (PV), future costs at time *t*, C_t, are converted to consistent present dollar values by adjusting future costs using the real discount rate *r*, and then summing the results over *T* years (Ehlen 1999), per Eq. 2.1. The real discount rate reflects the opportunity value of time and is used to calculate the effects of both inflation and discounting. It is typically from 2-3%, and estimates can be obtained from the federal Office of Management and Budget (Eamon et al. 2012).

$$\mathrm{LCC} = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \tag{2.1}$$

As shown be Eamon et. al (2014), probabilistic outcomes can be determined by direct simulation such as Monte Carlo Simulation (MCS). For each bridge configuration considered, MCS is used to first generate a simulated activity timing. Then, simulated costs for each year are generated using MCS based on the activity timing. The cumulative cost at a given year is determined by converting previous yearly costs to present value and summing the results up to year *j* using eq. 1. To conduct the probabilistic analysis, a limit state function (g) is needed. The limit state function can be written in terms of cost, such as: $g_j = C_R - C_{alternative}$, where C_R is the cumulative cost of a reference (i.e. control) bridge configuration, and $C_{alternative}$ is the cumulative cost of an alternate considered. If $g_j < 0$, then C_R was found to be cheaper for that year considered for simulation *i*. This result (i.e. if $g_j > 0$ or $g_j < 0$) is recorded for each year *j*. For each year considered, the simulation is repeated a sufficient number of times for accurate statistical quantification of the results (for example, 100,000 simulations). The cost probabilities *(P)* for each year *j* (i.e. probability of the cost of the alternate exceeding the reference bridge) can then be determined with the traditional MCS process using Eq. 2.2.

$$P(C_R < C_{alternative})_j = \frac{(\#\,of\ times\ g < 0)_j}{(total\ simulations; 100{,}000)} \tag{2.2}$$

Similar recommendations are given in NCHRP Report 483 (TRB 2003), which presents an outline of how to conduct LCCA for bridges. The report covers step-by-step LCCA procedures for bridges, as well as how to apply and interpret life-cycle cost analysis for consideration of repair strategies for existing structures as well as the evaluation of construction alternatives for new structures.

Various LCCA efforts involved consideration of steel members. Iowa DOT (Fagen and Phares 2000) compared the life cycle costs of three superstructure types for low volume roads: 1) precast double-T units, involving the fabrication of two rolled steel sections joined compositely to a reinforced concrete deck; 2) a traditional PC beam; and 3) slab construction. It was found that, when using salvaged steel sections, the steel beam unit produced the lowest life cycle costs when using county work forces for construction, as shown in Table 2.2.

Table 2.2. Life Cycle Costs of Bridge Unit Alternatives (Fagen and Phares 2000).

Costs:	Steel 2-T	PC	Slab
Initial	111,590	126,750	136,500
Total LCC	295,000	301,900	311,000

Weyers and Goodwin (1999) studied the life-cycle cost of alternate corrosion-protection coating systems for steel structures in Virginia. They considered construction, replacement, and rehabilitation costs, and conducted sensitivity and risk analyses. For a 75-year service life, it was determined that painting resulted in life cycle costs of approximately 1.93 times the costs for galvanization.

More recently, Soliman et al. (2014) compared the life cycle maintenance costs of two types of steel girders in Pennsylvania: maintenance-free steel (ASTM A1010) and conventional painted carbon steel. The deterioration model used assumes severe chloride exposure due to the use of deicing salts, resulting in the requirement for frequent repairs and maintenance. The results showed that, depending on the real discount rate considered (from 0 to 0.03), maintenance-free steel, although most costly initially, resulted in lower overall costs beginning at 40-60 years after construction. Similarly, Wallbridge et al. (2013) compared conventional painted steel to weathering steel, and found that the overall costs of weathering steel were lower after about 15 years of service. It was also found that when using conventional steel, LCC was lowest if the girders were replaced without repainting.

Zayed et al. (2002) considered a deterministic economic analysis, as well as a stochastic Markov decision process, to analyze different bridge rehabilitation scenarios. After deterioration curves for painted steel bridge girders were developed, the above two analysis methods were studied. It was found that the deterministic and probabilistic approaches gave different results. The deterministic analysis favored conducting spot repairs on the paint, while the probabilistic method indicated conducting no maintenance until the end of paint life, with a complete repainting, resulted in lowest LCC.

A number of bridge LCCA research studies considered bridge deck and joint types, where much of this work concerns materials and protective coating alternatives. Daigle and Lounis (2006), for example, conducted LCCA for high performance concrete (HPC) and normal performance concrete (NPC) decks. In addition to typical agency and user costs, costs associated with environmental impact in terms of CO_2 emission and waste production were included. The results indicated that, for the same water-to-cement ratio, the HPC deck was expected to have a service life from 3 to 10 times the service life the corresponding NPC deck. Considering initial costs of the HPC deck from 1-1.5 times those of the NPC deck, it was found that the HPC deck had approximately 50-65% of the LCC of the NPC deck. It was also found that user costs associated with HPC were only about 30% of NPC. Additionally, it was found that the HPC deck yields a 65% reduction in CO_2 emissions compared to the NPC deck.

Keoleian et al. (2008) compared two conventional steel reinforced concrete (SRC) decks, but varied the joint detail. One deck used a typical mechanical steel expansion joint, while the other deck used an engineered cementitious composite (ECC) link slab to join the adjacent spans together. The life cycle model used to evaluate infrastructure sustainability indicators considered two integrated elements: 1) assessment of material production, use, repair, construction, and demolition; and 2) assessment of social and agency costs. Social costs consisted of pollution damage costs from vehicle congestion, agency activities, vehicle crash, user delay, and vehicle operation costs, while agency costs consisted of material, end-of-life, and construction costs. Environmental impacts that were assessed including energy and material resource consumption, solid waste generation, and air and water pollutant emissions. The analysis was conducted for as assumed 60-year service life with a traffic volume of 35,000 vehicles per day and a 4% real discount rate. As shown in Table 2.3, the results showed that the ECC link slab system preformed significantly better with regard to LCC, producing a 37% cost advantage over the conventional system. It was also found that, from an environmental perspective, the link slab produced 39% less carbon dioxide emissions and consumed 40% less total primary energy.

Table 2.3. Results of LCCA (Keoleian et al. 2008).

(Keoleian)	LCC ($k)	
Cost	Conventional	ECC
Agency	751	489
User	34,909	22,075
Environmental	43	23
Total LCC	35,703	22,587

Similar to the study of Keoleian et al., Kendall et al. (2008) developed an integrated life-cycle assessment and life-cost analysis model. The authors compared two bridge deck designs and assessed agency, environmental, and user costs. The competing deck designs considered a conventional concrete bridge deck and an alternative (ECC) link slab. As shown in Table 2.4, it was found that the link slab design resulted in lower life-cycle costs as well as decreased environmental impacts, although the initial costs were higher.

Table 2.4. Results of LCCA (Kendall et al. 2008).

(Kendall)	LCC ($k)	
Event	Conventional	ECC
Deck replacement	386	431
Deck resurfacing	185	101
Patching	7	7
Total LCC	578	539

Colorado DOT (Hearn and Xi 2007) studied four types of reinforced concrete bridge deck construction: 1) decks with a waterproofing membrane and uncoated reinforcing bars; 2) without a membrane and uncoated bars; 3) with a membrane and epoxy-coated bars; and 4) with concrete sealer and epoxy-coated bars. As shown in Table 2.5, it was found that the bare deck with uncoated bars resulted in the lowest equivalent uniform annual costs (EUAC), in dollars per square yard of deck surface.

Table 2.5. Results of LCCA (Hearn and Xi 2007).

Type of Deck Construction	EUAC ($/SY)
Membrane & uncoated bars	2.14
Bare deck & uncoated bars	1.89-1.92
Membrane & epoxy-coated bars	2.89
Concrete sealer & epoxy-coated bars	4.28

Colorado DOT also explored the LCC of different deck coatings (Liang et al. 2010): hot rubberized asphalt, spray-applied liquid preformed sheets, and torch applied coatings were considered. As shown in the last column of Table 2.6, it was found that hot rubberized asphalt, followed by spray-applied liquid membranes, had lowest LCC.

Table 2.6. Deck LCCA Results (Liang et al. 2010).

Type of Membrane	new deck cost ($/m^2)	1st repair cost ($/m^2)	1st time to repair (yrs)	2nd repair cost ($/m^2)	2nd time to repair (yrs)	3rd repair cost ($/m^2)	3rd time to repair (yrs)	4th repair cost ($/m^2)	4th time to repair (yrs)	Present value of costs ($/m^2)
Preform sheet	164	169	20	169	40	169	60			406
Spray applied	204	27	20	27	35	214	50	27	70	322
Torch applied	173	183	20	183	40	183	60			435
Hot asphalt	160	27	20	170	40	27	60			263

A significant body of additional LCC research specifically concerned reinforcing bars. Among this work, MDOT (Kahl 2007) compared epoxy-coated bars to chromium steel (MMFX) steel reinforcement. LCCA results are shown in Table 2.7 for a 90-year life cycle, and assuming 52 years for the initial repair for decks using MMFX and 25 years for the next repair. MDOT agency costs as well as South Dakota DOT costs were considered. It was found that use of MMFX steel provided lowest LCC.

Later, Kahl (2011) compared epoxy-coated bars with stainless-clad reinforcement. The corrosion-deterioration model assumed that stainless steel bars are able to provide more than 100 years of maintenance-free service whereas the epoxy-coated bar service life is 85 years. Although the initial cost of stainless steel clad bars are high, the LCC are lower overall, as shown in Table 2.8.

Table 2.7. Epoxy-Coated and MMFX LCC Results (Kahl 2007).

Reinforcement	New Cost ($/yd^2)		Repair Cost ($/yd^2)		Time to Initial Repair (years)	Time to Next Repair (years)	Time to Next Repair (years)	Present Value of Costs ($/yd^2) with 3.0 % Discount Rate	
	SDDOT	MDOT	SDDOT	MDOT				SDDOT	MDOT
Epoxy-coated	112	94	158	319	40	65	90	184	238
MMFX	124	106			52	77	103	174	207

Table 2.8. LCCA Present Value Costs (Kahl 2011).

Bridge deck reinforcement type	Epoxy	Stainless Clad/ Stainless
Bid price for reinforcement per lb.	$1.00	$3.21*
Initial construction cost	$3,587,016	$3,947,690
Rehabilitation recurrence interval (variance +/- 20%)	60 years	100 years
Rehabilitation cost**	$1,004,000	$271,000
Work zone user delay cost (ADT = 55,000)	$432,000	$121,000
Real discount rate (variance +/- 100%)	2.7%,	2.7%
Salvage value	$0	$0
Life cycle cost	$4,591,000	$4,219,000
EUAC (over 100 year period)	$49,350	$45,350

* Calculated from bid prices of stainless and stainless-clad reinforcement at $5.00/lb. and $1.75/lb. respectively, and a ratio of 45/55 percent based on the steel reinforcement quantities.
** Deck replacement costs are $70/SFT for ECR and $87.43/SFT for stainless steel reinforcement.

Although LCC was not considered, MDOT recently examined the performance of approximately 800 decks reinforced with epoxy coated steel, stainless steel, and fiber reinforced polymer (FRP) bars (Boatman 2010; Valentine 2015), and used linear regression and transition probabilities to estimate the time for future rating conditions. Due to the limited number of FRP (4) and stainless steel (13) reinforced decks, results could not be directly compared to decks with epoxy-coated bars. However, it was found that stainless decks required an average of 19 years to attain a deck surface rating of 7 (good); FRP decks required an average of 12 years to attain a rating of 6; and epoxy-coated bar decks required 86 years to obtain a rating of poor. The results indicate that the stainless steel decks appear to be performing better than epoxy coated, while FRP appears to be performing worse. A comparison of the projected deterioration trends is given in Figure 2.13.

Bhaskaran et al. (2006) conducted a LCCA on a long span prestressed concrete bridge, considering two corrosion protection alternatives: using galvanic cathodic protection and no such protection. For a 50-year design life, the annual cost of corrosion was approximately $43000 for no protection but using galvanic cathodic protection, the annual cost was reduced to $40000.

Considering carbon fiber reinforced polymer (CFRP) bars and strands, Eamon et al. (2012) conducted a probabilistic life-cycle cost analysis on prestressed concrete bridge superstructures using MDOT agency costs and maintenance practices, and compared results to uncoated bars using cathodic protection and epoxy-coated steel. Depending on the bridge case considered, the results indicated that using CFRP reinforcement has the potential to reach significant reductions in life-cycle cost, with a 95% probability to be the least expensive alternative beginning at year 23–77 after initial construction. Using CFRP reinforcement was found to be most effective on AASHTO PCI beam bridges in a high traffic volume area. It was also found that LCC and the cost effectiveness of alternatives are significantly affected by traffic volume. In particular, user cost savings for alternatives requiring less maintenance are most pronounced as traffic volume increases.

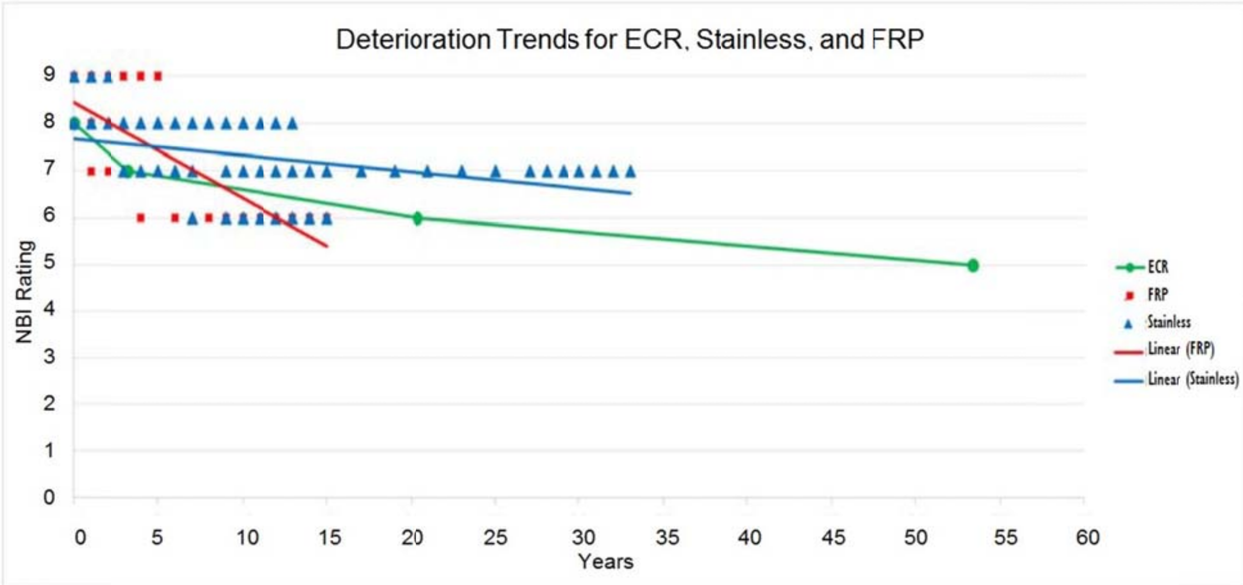

Figure 2.13. Projected Deterioration for MDOT Decks.

In addition to comparing deck types for Colorado DOT, Liang et al. (2010) also considered various reinforcing bar types. These included: uncoated steel; epoxy-coated; galvanized steel; stainless clad; and solid stainless steel bars. As shown in Table 2.9, stainless clad bars had lowest life cycle cost, as confirmed by Kahl (2011).

Table 2.9. LCCA Results for Reinforcing Bar Alternatives (Liang et al. 2010).

Type of Bar	new deck cost ($/m^2)	1st repair cost ($/m^2)	1st time to repair (yrs)	2nd repair cost ($/m^2)	2nd time to repair (yrs)	3rd repair cost ($/m^2)	3rd time to repair (yrs)	4th repair cost ($/m^2)	4th time to repair (yrs)	Present value of costs ($/m^2)
Black steel	125	259	25	259	50					379
Epoxy coated	135	259	30	259	55					365
Stainless	277									277
Stainless clad	155									155
Galvanized	150	259	27	259	52					394

Various LCCA research efforts considered the effect of different maintenance strategies. Wisconsin DOT (Huang et al. 2004) studied maintenance strategies for concrete bridge decks and how they affect life-cycle costs. Five different maintenance alternatives throughout the life of the deck were considered (followed by total LCC in $k): 1) concrete overlay - replace deck (560); 2) patch – patch – patch – replace deck (575); 3) patch – patch – patch – AC overlay without membrane – replace deck (600); 4) AC overlay with membrane (535); 5) replace deck (with no maintenance) (490). It was observed that early maintenance activities had a strong influence on the estimated service life, and that the deck replacement or asphaltic concrete overlay with membrane (without prior maintenance) options resulted in lowest LCC.

More recently, Wyoming DOT (Shim and Lee 2016) studied 53 bridge decks considering three types of bridge rehabilitation methods in order to develop the probable service life and determine probabilistic life cycle costs. The rehabilitation methods studied were application of: an asphalt-membrane; epoxy sealant, and; a silica fume modified concrete overlay. Monte Carlo simulation was used to estimate the present value of bridge deck replacement cost as well as the equivalent uniform annual present value costs. The results indicated that epoxy sealants were most expensive, while asphalt membranes generally resulted in lowest LCC.

At about the same time, Hatami and Morcous (2016) compared the effectiveness of several deck rehabilitation options by conducting deterministic and probabilistic LCCA approaches. For the analysis, the authors developed deck deterioration models and collected cost data for highway bridges in Nebraska. In the project, three deck overlays were considered: a silica fume overlay; an epoxy polymer overlay; and a polyester overlay, compared with a bare deck. Additionally, two alternative expansion joint replacement options were compared: relocating abutment expansion joints at the grade beam, and replacing abutment expansion joints at the same place. Finally, different deck widening schemes were analyzed. Of the various overlay options, the results indicated that applying an epoxy polymer overlay on a bare deck with a condition rating of 7 resulted in the lowest life cycle costs, whereas the bare deck resulted in the highest costs. It was also found that replacing an abutment expansion joint at the same place is more cost-effective in the long term than relocation, and deck widening was significantly more cost-effective than deck replacement in most cases.

Other related research efforts include those by Rafiq et al. (2005), who presented a comparison in life-cycle costs and safety characteristics for different bridge management strategies. It was found that the use of structural health monitoring data and regular inspections could significantly reduce the life cycle costs of concrete structures.

Hegazy et al. (2004) presented a bridge deck management system (BMS) for the project as well as the network level. The network level BMS provided repair priority to bridges using weighted cost-benefit analyses, and considered multi-year planning period and budget limits. The project level BMS provided an optimal repair strategy for components of a specific bridge, and, although usually done in isolation from the network analysis, the author developed a project and network integrated approach to minimize overall life cycle costs. It was found, however, that the optimization algorithm may fail due to the large problem size. Later, Harada and Yokoyama (2007) proposed the Real Options method to evaluate the cost effectiveness of bridge inspection procedures.

Other Research

El-Diraby and O'Connor (2001) proposed a method to evaluate the quality of bridge construction plans during the design phase considering five major factors: safety; carrying capacity; schedule; budget; and accessibility. Results were collected by reviewing project documentation of three major highway projects in Texas for a period of 18 months, and conducting formal and informal interviews with design and construction engineers. In the interview process, formal interviews with industry experts were first conducted, then a second part of the interview allowed for an open-ended discussion. Interviewees included eight experts from TxDOT, two from engineering consulting firms, and two from contracting firms.

MDOT analyzed projected bridge deterioration for decks, superstructures, and substructures (Kelley 2016). As shown in Figure 2.14, it was found that the deck deterioration rate was independent of time for ratings between 9 and 7, while deterioration slowed between ratings of 7 and 3. Moreover, for ratings of 7 or less, differences between the time periods considered widened, with earlier periods subjected to more rapid deterioration. This indicates that maintenance events may extend the life of the deck, decreasing deterioration rate. A similar trend occurs with superstructures, as shown in Figure 2.15. Comparing painted steel and prestressed concrete girders (Figure 2.16), it appears that both obtain a rating of 5 in about 30 years, then the PC deterioration rate increases beyond that of steel. Substructure trends are similar to deck and superstructure results, as shown in Figure 2.17.

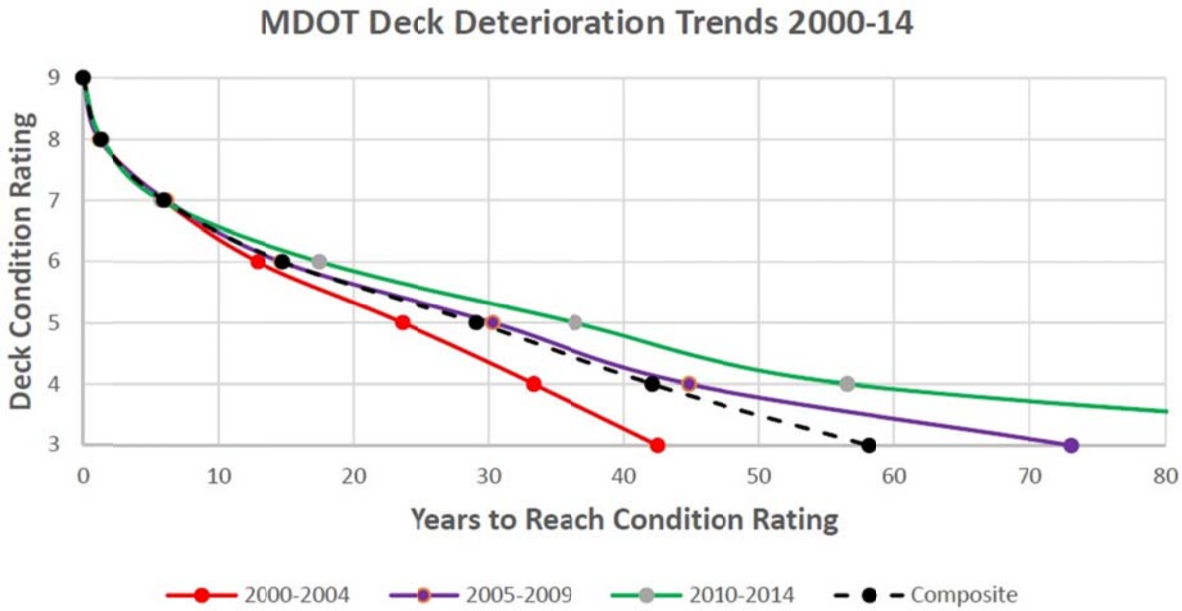

Figure 2.14. Projected Deck Deterioration (Kelley 2016).

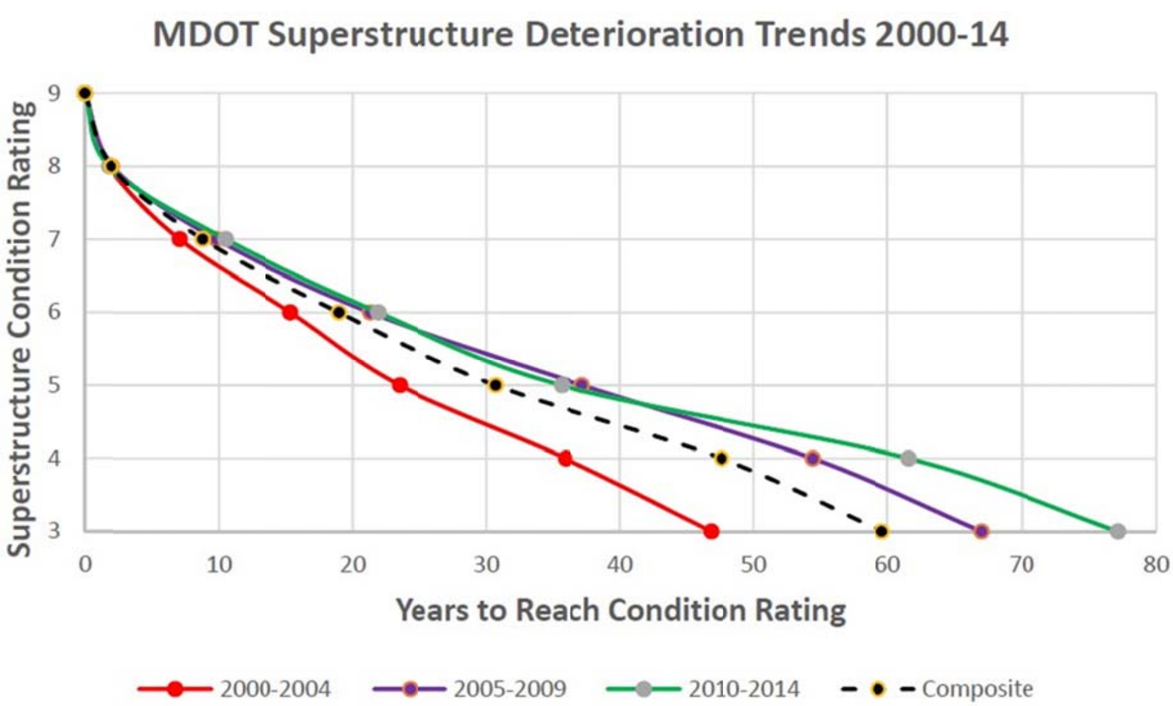

Figure 2.15. Projected Superstructure Deterioration (Kelley 2016).

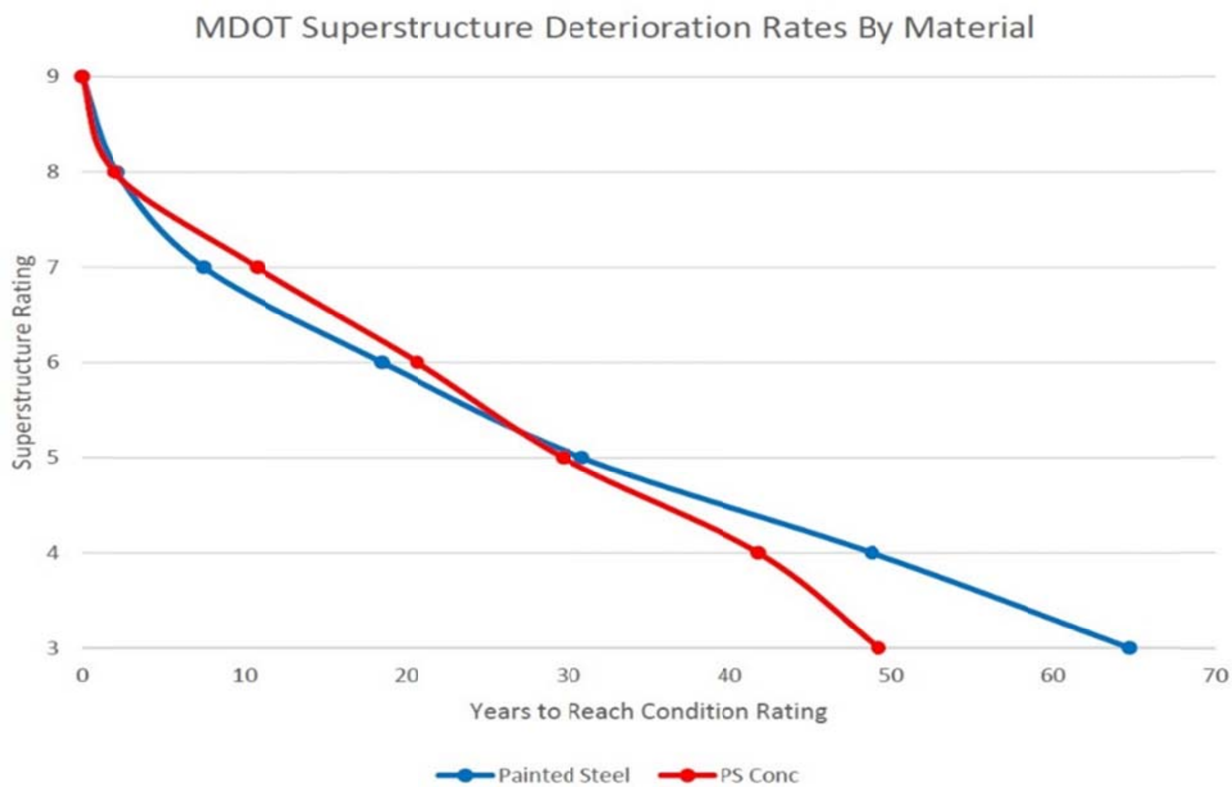

Figure 2.16. Comparison of Steel and PC Girders (Kelley 2016).

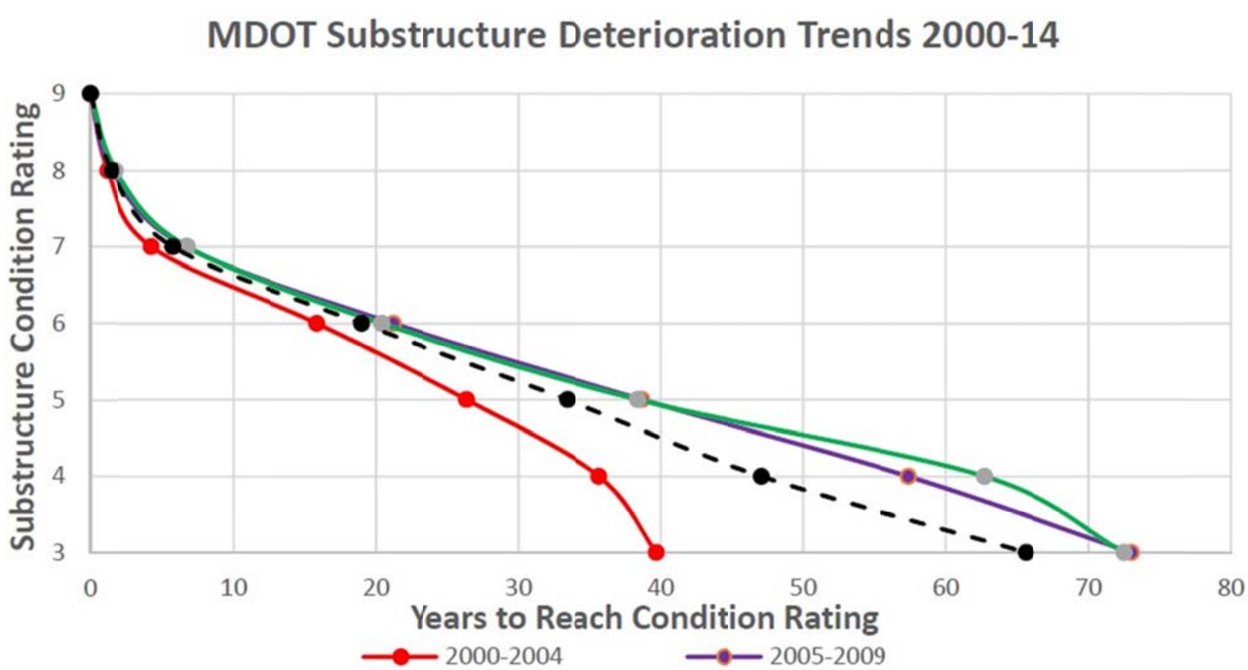

Figure 2.17. Projected Substructure Deterioration (Kelley 2016).

Summary

In summary, in the available literature, standardized plans for bridge construction focused on the use of several concepts, including: precast components beyond individual girders, such as precast piers, columns, abutments, wing walls, and decks; simple span rather than continuous structures; alternative connection details such as high performance concrete pours to join components; double-tee spanning members; and stay-in-place formwork when site-casting is necessary. Additionally, concepts that have been found to have low life cycle costs include double-tee members; the use of low maintenance or galvanized steel girders rather than painted steel; high performance concrete deck materials; the use of high performance concrete pours to form joints rather than traditional expansion joints; stainless steel or stainless-clad bars; and the minimization of maintenance events.

CHAPTER 3: EVALUATION OF LOCAL BRIDGE DATABASE

Introduction

The following chapter represents a brief statistical analysis of the contents of MDOT's local agency bridge database as of June, 2016. The database has approximately 7000 structures listed. Eliminating structures less than 20 ft in total length, as well as non-vehicle structures (i.e. railroad or pedestrian only, etc.) leaves approximately 6675 bridges. The data presented below in Tables 1 - 37 are based on this subset of structures. The tables provide the percentage (%) as well as the number (No.) of structures in each category. Note that some entries are not fully documented in the database, and are missing particular pieces of information. Therefore, the total number of structures in each table may differ slightly.

General Information

As shown in Table 3.1, most structures are in the Bay region (approximately 30%), while least appear in the North region (approximately 6%l). At the county level (Table 3.2), most bridges appear in Wayne County (5%), whereas significantly fewer are spread throughout the remaining counties; Oakland and Macomb have the next most number of bridges at less than 4% each. Approximately 87% of structures are owned by county highway agencies (Table 3.3).

Table 3.4 indicates that most structures (about 57%) have been built between the years of 1961-2000, while about 17% have been built later than 2000. In relation to material type (Table 3.5), most concrete bridges have been built since 1981 (52%); most continuous concrete from 1941-1960 (41%); most steel (any type) from 1921-1980 (about 80%); and nearly all prestressed concrete since 1961. As shown in Table 3.6, if a structure has been reconstructed, it is most likely to have occurred from 1981-2000.

Tables 3.7 and 3.8 indicate the type of service on and under the bridge, respectively. Approximately 88% of structures carry only vehicular traffic (no sidewalks), whereas 97% have a waterway under the bridge. About 44% of bridges carry a rural local road, while 27% carry rural major collector roads (Table 3.9). Overall, about 86% are rural, and most structures (about

63%) have average daily traffic (ADT) less than 1000, though approximately 7% have ADT from 5000-10,000 (Table 3.10).

As shown in Table 3.11, about 67% of bridges have been designed for the AASHTO Standard *HS20* (or "*HS20+Mod*") load, while very few appear to have been designed to AASHTO LRFD requirements (*HL-93*) (1.5%) or to MDOT's current *HL-93-mod* design load (3%), potentially indicated by the "*Greater than HL-93*" label. However, Table 3.12 indicates that the large majority of structures (84%) may carry Michigan vehicle configurations greater than 77 tons.

Geometry and Structural Information

Table 3.13 provides the types of bridge structures. As shown, 34% are box beam bridges, while 24% are culverts, 22% are girder bridges ("multi-stringer"), and about 9% are slab bridges. Most structures (43%) are prestressed concrete, while 30% are steel (Table 3.14). Of the steel structures, 30% are of Grade A36 and about 5% utilize weathering steel (Table 3.15) Most (25%) are galvanized, and about 13% are unpainted (Table 3.16). Most (12%) were painted from 1961-2000; however, most (78%) have no information on painting date (Table 3.17).

As shown in Table 3.18, most bridges (63%) have a single span, while 20% have 2 spans and 12% have three spans. Nearly all structures (96%) have no approach spans (Table 3.19). Most bridges (31%), as shown in Table 3.20, are less than 30 ft in total length, while about 58% are less than 50 ft long. Table 3.21 indicates that most structures (43%) contain a maximum single span length less than 30 ft, and the large majority (83%) contain a span with maximum length no greater than 60 ft. In Table 3.22, the total bridge lengths associated with different bridge types are shown. A similar length pattern exists for the common materials, where most lengths are less than 40 ft. However, continuous structures (prestressed concrete and steel), although few in number, tend to be associated with longer spans in greater proportion than non-continuous bridges. This is particularly so with continuous steel, for which 29% have lengths greater than 200 ft. In Table 3.23, bridge types are sorted by the counties that have the most bridges. It can be seen that the majority of continuous structures (concrete and steel) appear in Wayne county, while simple steel and simple prestressed structures are more evenly divided between Wayne, Kent, Oakland, Saginaw, Macomb, and Sanilac counties.

Bridge skew is shown in Table 3.24, where most (62%) are no skew and 89% have skew no greater than 30 degrees. Although Table 3.25 provides information on foundation type, few bridges (about 12%) have this information given in the database, and hence the numbers available may not reliably represent the entire population. However, of these, most are supported by steel H-piles.

Decks

In Table 3.26, it can be seen that most decks are cast-in-place concrete (44%), while 20% are precast concrete panels. Most decks are relatively narrow, with 40% from 30-40 ft wide (out-to-out) and about 69% less than 40 ft wide (Table 3.27). The type of rail placed upon the deck is given in Table 3.28. Most are either steel beam guardrails (40%) or open concrete parapets (17%). Relatively few are solid concrete barriers (less than 12%). As shown in Table 3.29, 50%

of decks have an asphalt wearing surface, while about 30% have a concrete topping. Most decks (73%) have no protective membrane; 10% have a preformed fabric membrane and 2% a built-up membrane (Table 3.30). About 60% of decks have no special corrosion protection, while 25% use epoxy coated bars (Table 3.31). Table 3.32 indicates when the latest deck overlay was performed, although the usefulness of this data is questionable since 84% of the structures have no information regarding overlay data. In Table 3.33, the type of approach pavement is given; about 56% of approaches are asphalt and 25% are gravel.

Ratings

Deck ratings are given in Table 3.34. Here, it can be seen that most (26%) are in Good condition; 16% are rated Very Good and 16% Satisfactory. Very few are rated Excellent or less than Fair. In Table 3.35, superstructure ratings are shown. Similar to decks, most are rated Good (24%), while 19% are Very Good and 15% Satisfactory. Substructure ratings (Table 3.36) follow the same trend as well, where 25% are rated Good, 19% Very Good, and 15% Satisfactory.

To identify any potential relationships between bridge characteristics and current rating, a multi-linear regression analysis was conducted with 15 variables: deck rating, superstructure rating, substructure rating, region ("district"), county, year built, maximum span, length, skew, material, design, deck structural type, deck membrane type, deck surface type, and deck protection. Table 3.37 presents the resulting correlation coefficients. Observing the values within the table, it can be seen that the deck rating, superstructure rating, and substructure are rather strongly correlated to one another (with correlation coefficients from 0.90-0.93). Ratings are also moderately strongly (negatively) correlated with bridge design (i.e. structure type; Table 3.13, with all condition correlation coefficients of about -0.84), while the remaining variables have quite low coefficient values, with most no greater than about 0.2 and none greater than about 0.5. This indicates that of all bridge parameters considered, only bridge structural type appears to be a strong predictor of condition rating. Additionally, it indicates that the condition of one component is highly related to the condition of the remaining components. To explore the relationship between bridge type and condition rating further, Table 3.38 presents average condition ratings for the different structural types. As shown, significant differences are evident, with division (i.e. deck, superstructure, or substructure) averages ranging from 4.9 to 8.0, depending on structure type, and an overall average of 6.3 for all divisions. Box beam type structures have the highest overall ratings, while the lowest are girder & floor beam system bridges, as well as special types (movable – bascule and truss – thru & pony).

Table 3.39 considers a subset of structures, with the unusual types (movable, arch, and truss, etc.) removed. Here again it can be seen that box beam bridges have the highest average condition rating overall (7-7.8), followed by slab and rigid frames (6.5), followed by multi-stringer and tee-beam bridges (6.1), with an overall average for all types of 6.7.

Table 3.1. Bridge Location by Region.

Region	%	No.
Superior	9.3	620
North	6.2	414
Grand	16.3	1085
Bay	29.5	1967
Southwest	9.6	644
University	16.7	1118
Metro	12.4	827

Table 3.2. Bridge Location by County.

County	%	No.
Wayne	5.0	332
Kent	3.5	233
Oakland	3.7	249
Saginaw	3.2	216
Macomb	3.7	246
Sanilac	2.2	146
Lenawee	2.9	196
Others	75.8	5058

Table 3.3. Bridge Owner.

Description	%	No.
County Highway Agency	87.0	5802
Other State Agencies	< 0.1	3
Other Local Agencies	0.3	20
City or Municipal Highway Agency	12.7	846

Table 3.4. Year of Build.

Year	%	No.
<1921	3.9	258
1921-1940	12.3	819
1941-1960	9.4	625
1961-1980	28.4	1898
1981-2000	28.7	1917
>2000	16.9	1125
No info.	0.5	33

Table 3.5. Year Built and Type of Bridge [percent (No. of bridges)].

Year built	Concrete	Concrete, continuous	Steel, simple or cantilever	Steel, continuous
<1921	5.4 (58)	1.3 (1)	6.7 (124)	1.1 (1)
1921-1940	24.9 (266)	28.0 (21)	20.0 (372)	19.8 (18)
1941-1960	8.0 (86)	41.3 (31)	17.7 (329)	25.3 (23)
1961-1980	9.2 (98)	26.7 (20)	40.5 (752)	37.4 (34)
1981-2000	25.4 (272)	2.7 (2)	12.6 (234)	7.7 (7)
>2000	27.0 (289)	0.0 (0)	2.5 (46)	8.8 (8)

Year built	Prestressed concrete (PC)	PC, continuous	Timber	Masonry	Aluminum, WI, CI
<1921	1.4 (41)	5.0 (1)	4.5 (28)	75.0 (3)	0.0 (0)
1921-1940	4.1 (118)	0.0 (0)	3.5 (22)	25.0 (1)	0.0 (0)
1941-1960	4.3 (122)	0.0 (0)	5.1 (32)	0.0 (0)	4.3 (2)
1961-1980	24.9 (710)	35.0 (7)	44.1 (274)	0.0 (0)	2.1 (1)
1981-2000	41.7 (1191)	40.0 (8)	29.9 (186)	0.0 (0)	36.2 (17)
>2000	23.5 (671)	20.0 (4)	12.9 (80)	0.0 (0)	57.4 (27)

Table 3.6. Year of Reconstruction.

Year	%	No.
<1921	0.0	0
1921-1940	0.1	8
1941-1960	0.7	50
1961-1980	2.3	154
1981-2000	4.7	316
>2000	3.5	232
No info.	88.6	5916

Table 3.7. Type of Service on the Bridge.

Description	%	No.
Highway	88.3	5894
Highway + pedestrian	11.7	781

Table 3.8. Type of Service under the Bridge.

Description	%	No.
Highway, with or w/o pedestrian	0.6	40
Railroad	1.3	86
Pedestrian-bicycle	0.2	13
Highway-railroad	0.1	9
Waterway	97.4	6504
Highway-waterway	0.1	5
Railroad-waterway	< 0.1	3
Highway-waterway-railroad	0.1	4
Other (non-highway)	0.2	11

Table 3.9. Functional Classification.

Description	%	No.
Rural - Minor Arterial	7.4	493
Rural - Principal Arterial - Other	< 0.1	2
Rural - Major Collector	27.3	1817
Rural - Minor Collector	7.1	474
Rural - Local	44.2	2940
Urban - Other Principal Arterial	2.0	132
Urban - Minor Arterial	1.1	75
Urban - Collector	5.2	343
Urban - Local	5.6	374

Table 3.10. Average Daily Traffic.

ADT total	%	No.
<1000	62.6	4218
1000-2000	11.4	769
2001-3000	5.4	363
3001-5000	5.5	369
5000-10000	7.0	474
10000-15000	3.5	239
15000-20000	2.1	144
>20000	2.3	157

Table 3.11. Type of Design Load.

Description	%	No.
H 10	0.7	47
H 15	5.6	375
HS 13.5	0.3	23
H 20	11.7	780
HS 20	34.7	2311
HS 20+Mod	31.9	2125
HS 25 or greater	3.7	247
Unknown	6.8	452
HL93	1.5	99
Greater than HL-93	3.1	204
Other	0.1	5

Table 3.12. Bridge Posting.

77 Ton MI vehicle	42 Ton MI vehicle	HS truck	%	No.
≥ 77 tons	≥ 42 tons	≥ 36 tons	83.7	5554
69.4 - 76.9 tons	37.8 - 41.9 tons	32.4 - 35.9 tons	2.3	151
61.7 - 69.3 tons	33.6 - 37.7 tons	28.8 - 32.1 tons	2.5	167
54.0 - 61.6 tons	29.4 - 33.5 tons	25.2 - 28.7 tons	3.0	200
53.9 - 46.3 tons	25.2 - 29.3 tons	21.6 - 25.1 tons	2.0	134
< 46.3 tons	< 25.2 tons	< 21.6 tons	6.5	433

Table 3.13. Structure Type.

Description	%	No.
Other	0.3	20
Slab	9.3	620
Multi-Stringer, W or I-beam	21.5	1432
Girder & floorbeam system	1.0	64
Tee beam	6.3	420
Box beam - multiple	34.4	2295
Box beam - single or spread segmental	1.5	100
Frame - rigid (except culverts)	0.4	24
Truss - deck	< 0.1	1
Truss - thru & pony	0.6	43
Arch - deck, filled spandrel	0.9	58
Arch - thru	0.1	8
Movable - bascule	0.2	10
Movable - swing	< 0.1	2
Culvert	23.5	1567

Table 3.14. Bridge Material.

Description	%	No.
Concrete	16.2	1079
Concrete continuous	1.1	75
Steel, simple or cantilever	28.2	1880
Steel, continuous	1.4	91
Prestressed concrete (PC)	42.8	2854
PC, continuous	0.3	20
Timber	9.3	622
Masonry	0.1	4
Aluminum, WI, CI	0.7	47
Other	< 0.1	3

Table 3.15. Type of Steel.

Description	%	No.
Carbon steel (A7) (A 373)	27.6	544
Carbon steel (A36)	29.8	587
Alloy steel (A 441)	8.4	165
Alloy-weather (A 588 - A 441 mod.)	4.9	96
Alloy (A 572)	2.2	44
No info.	27.1	535

Table 3.16. Type of Paint (steel structures only).

Description	%	No.
Unpainted	13.1	258
Lead-based paint	14.6	287
Non-lead paint	7.2	141
Galvanized	24.5	482
Urethane	3.2	64
Unknown	4.4	87
No info.	33.1	652

Table 3.17. Year Painted (steel structures only).

Year	%	No.
<1921	0.5	9
1921-1940	2.6	51
1941-1960	2.7	53
1961-1980	5.7	113
1981-2000	6.0	118
>2000	4.8	94
No info.	77.8	1533

Table 3.18. Number of Spans in Main Unit.

No. of spans	%	No.
1	63.7	4254
2	19.7	1316
3	12.3	824
4	2.2	149
5	0.8	56
6	0.4	25
7	0.3	20
8	0.1	9
9	< 0.1	1
>11	0.2	13
No info.	0.1	8

Table 3.19. Number of Approach Spans.

No. of spans	%	No.
0	95.6	6306
1	0.2	10
2	0.4	26
3,9,21	< 0.1	3
10	< 0.1	2
5	< 0.1	2
4	0.1	5
No info.	3.7	245

Table 3.20. Bridge Length.

Total length (ft)	%	No.
20-30’	31.1	2078
30-40	16.1	1074
40-50	11.2	747
50-60	8.9	595
60-70	6.7	445
70 -80	4.7	315
80-90	3.1	204
90-100	3.1	204
100-120	4.0	267
120-140	2.8	190
140-160	1.7	114
>160	6.1	408
No info.	0.5	35

Table 3.21. Maximum Span Length.

Max span (ft)	%	No.
<20	15.7	1051
20-30	27.4	1827
30-40	17.2	1146
40-50	12.6	844
50-60	10.0	670
60-70	6.7	450
70-80	4.1	273
80-90	2.3	151
90-100	1.5	102
100-120	1.3	88
120-140	0.6	43
140-160	0.1	4
>160	0.3	17
no info.	0.1	10

Table 3.22. Total Length and Type of Bridge [percent (No. of bridges)].

Total length (ft)	Concrete	Concrete, continuous	Steel, simple or cantilever	Steel, continuous
20-40	80.1 (853)	24.0 (18)	67.0 (1245)	7.7 (7)
40-60	10.4 (111)	14.7 (11)	14.9 (277)	4.4 (4)
60-80	3.3 (35)	17.3 (13)	5.6 (104)	12.1 (11)
80-100	3.3 (35)	20.0 (15)	2.5 (47)	13.2 (12)
100-120	1.0 (11)	9.3 (7)	2.1 (39)	13.2 (12)
120-150	0.8 (8)	6.7 (5)	1.8 (34)	8.8 (8)
150-200	0.6 (6)	5.3 (4)	1.9 (35)	12.1 (11)
>200	0.6 (6)	2.7 (2)	4.2 (78)	28.6 (26)

Total length (ft)	Prestressed concrete (PC)	PC, continuous	Timber	Masonry	Aluminum, WI, CI
20-40	21.2 (604)	0.0 (0)	60.1 (374)	75.0(3)	100.0 (47)
40-60	28.5 (813)	10.0 (2)	19.6 (122)	0.0 (0)	0.0 (0)
60-80	18.4 (524)	5.0 (1)	11.6 (72)	0.0 (0)	0.0 (0)
80-100	9.5 (272)	15.0 (3)	3.7 (23)	25.0 (1)	0.0 (0)
100-120	6.6 (188)	5.0 (1)	1.4 (9)	0.0 (0)	0.0 (0)
120-150	6.6 (188)	10.0 (2)	2.3 (14)	0.0 (0)	0.0 (0)
150-200	4.8 (136)	25.0 (5)	0.6 (4)	0.0 (0)	0.0 (0)
>200	4.5 (129)	30.0 (6)	0.6 (4)	0.0 (0)	0.0 (0)

Table 3.23. County and Type of Bridge [percent (No. of bridges)].

County	Concrete	Concrete, continuous	Steel, simple or cantilever	Steel, continuous
Wayne	10.9 (118)	56.0 (42)	3.5 (66)	34.1 (31)
Kent	4.6 (50)	2.7 (2)	5.0 (94)	2.2 (2)
Oakland	7.6 (82)	10.7 (8)	3.6 (67)	1.1 (1)
Saginaw	3.4 (37)	0.0 (0)	3.8 (72)	6.6 (6)
Macomb	4.7 (51)	1.3 (1)	4.6 (86)	2.2 (2)
Sanilac	0.2 (2)	0.0 (0)	4.5 (84)	0.0 (0)
Lenawee	1.7 (18)	1.3 (1)	1.4 (27)	2.2 (2)
Other	66.8 (721)	28.0 (21)	73.6 (1384)	51.6 (47)

County	Prestressed concrete (PC)	PC, continuous	Timber	Masonry	Aluminum, WI, CI
Wayne	2.5 (72)	5.0 (1)	0.0 (0)	0.0 (0)	0.0 (0)
Kent	2.7 (77)	0.0 (0)	0.5 (3)	0.0 (0)	10.6 (5)
Oakland	2.6 (74)	0.0 (0)	2.6 (16)	0.0 (0)	2.1 (1)
Saginaw	3.3 (93)	0.0 (0)	1.3 (8)	0.0 (0)	0.0 (0)
Macomb	3.4 (98)	5.0 (1)	1.0 (6)	0.0 (0)	2.1 (1)
Sanilac	1.7 (48)	0.0 (0)	0.8 (5)	0.0 (0)	14.9 (7)
Lenawee	3.3 (95)	0.0 (0)	8.5 (53)	0.0 (0)	0.0 (0)
Other	80.5 (2297)	90.0 (18)	85.4 (531)	100.0 (4)	70.2 (33)

Table 3.24. Skew.

Skew (deg)	%	No.
0	61.9	4130
1-30	27.0	1803
31-60	7.9	525
60-99	0.6	40
No info.	2.7	179

Table 3.25. Foundation Type.

Description	%	No.
Spread footing on soil	2.0	134
Footing/timber piles	0.4	29
Footing/steel H piles	3.0	202
Footing /steel tube piles	0.4	25
Footing on tremie	0.3	22
Pile bents	1.0	70
Caisson	0.1	4
Spread footing on rock	0.1	6
Footing in cofferdam - steel sheet piling left in place	0.1	10
Gravity type on Soil	0.1	4
Gravity type on timber piles	0.1	4
Gravity type on steel H-piles	3.4	226
Gravity type on concrete piles	1.0	67
Gravity type on rock	< 0.1	1
no info.	88.0	5902

Table 3.26. Deck Structural Type.

Description	%	No.
Concrete, cast-in-place	44.1	2941
Concrete, precast panels	20.3	1355
Open grating	0.2	15
Closed grating	< 0.1	2
Steel plate (includes orthotropic)	0.1	6
Corrugated steel	1.6	108
Aluminum	0.1	7
Wood or timber	10.2	683
Other	1.8	122
Not applicable	21.5	1435
No info.	< 0.1	1

Table 3.27. Bridge Deck Width.

Width (ft)	%	No.
<20	8.2	550
20-30	20.6	1375
30-40	40.2	2681
40-50	12.4	830
50-60	4.1	272
>60	8.3	557
no info.	6.1	410

Table 3.28. Type of Rail.

Description	%	No.
No railing or guardrail	8.7	555
Steel beam guard rail	40.2	2576
Concrete or stone balustrades	1.6	103
Aluminum tubular railing with cast aluminum posts (R10, 2 or 3 tubes) R13 structural aluminum tube.	2.7	176
Steel or aluminum fabricated panels with concrete or steel posts (R1 to R9) or similar	4.7	301
Concrete girder or solid reinforced concrete panel (1919-35+)	3.8	244
Concrete parapet (open) (R11 or R12) or similar	17.2	1102
Concrete parapet (solid R16) or similar	6.4	411
Concrete G.M. barrier (R15) or New Jersey (X17)	5.4	349
Other railing type	6.1	389
No info.	3.2	208

Table 3.29. Type of Wearing Surface.

Description	%	No.
None	12.5	823
Monolithic concrete	23.8	1563
Integral concrete	3.9	254
Latex concrete or similar additive	2.0	129
Low slump concrete	0.1	7
Epoxy overlay	0.7	47
Bituminous	50.3	3302
Wood or timber	2.5	166
Gravel	4.0	262
Other	0.2	15
Not applicable	< 0.1	2

Table 3.30. Type of Membrane.

Description	%	No.
None	73.0	4871
Built-up	2.4	160
Preformed Fabric	10.0	666
Epoxy	0.3	18
Unknown	0.8	53
Other	2.7	183
Not applicable	10.8	721
No info.	< 0.1	3

Table 3.31. Deck Protection.

Description	%	No.
None	60.3	4023
Epoxy Coated Reinforcing	25.2	1681
Galvanized Reinforcing	0.2	11
Other Coated Reinforcing	< 0.1	1
Cathodic Protection	< 0.1	1
Polymer Impregnated	0.2	16
Internally Sealed	0.1	6
Unknown	1.1	76
Other	1.8	123
Not applicable	11.0	736
No info.	< 0.1	1

Table 3.32. Year of Overlay.

Year of overlay	%	No.
<1921	< 0.1	2
1921-1940	< 0.1	3
1941-1960	< 0.1	2
1961-1980	1.4	95
1981-2000	8.8	584
>2000	5.8	389
No info.	83.8	5580

Table 3.33. Bridge Approach Pavement.

Approach Pavement, Type & Width	%	No.
Unimproved earth	1.4	96
Graded and drained earth	1.0	69
Gravel or similar	25.4	1714
Bituminous surface treated gravel	6.0	407
Mixed bituminous surface on gravel (≥1")	48.3	3256
Mixed bituminous on concrete, brick or black base (≥1")	8.3	559
Concrete	5.5	370
Brick	0.0	0
Freeway designed bituminous concrete on aggregate base	0.2	16
No info.	3.7	248

Table 3.34. Deck Rating.

Condition	%	No.
0 Failed	0.1	7
1 Imminent Failure	< 0.1	1
2 Critical	0.3	23
3 Serious	1.6	109
4 Poor	3.5	244
5 Fair	8.5	593
6 Satisfactory	15.8	1111
7 Good	26.4	1855
8 Very Good	16.1	1131
9 Excellent	1.9	130
N N/A (NBI)	25.4	1782
No info.	0.4	28

Table 3.35. Superstructure Rating.

Condition	%	No.
0 Failed	0.1	9
1 Imminent Failure	0.1	10
2 Critical	0.9	63
3 Serious	2.6	185
4 Poor	4.4	312
5 Fair	9.2	647
6 Satisfactory	15.2	1063
7 Good	23.7	1663
8 Very Good	18.6	1306
9 Excellent	2.1	144
N N/A (NBI)	22.5	1579
No info.	0.5	34

Table 3.36. Substructure Rating.

Condition	%	No.
0 Failed	0.1	6
1 Imminent Failure	0.1	6
2 Critical	0.6	39
3 Serious	2.2	154
4 Poor	4.8	335
5 Fair	9.1	639
6 Satisfactory	15.0	1055
7 Good	24.9	1746
8 Very Good	18.9	1329
9 Excellent	1.5	108
N N/A (NBI)	22.5	1577
No info.	0.3	21

Table 3.37. Correlation Coefficients.

		dkrating	suprating	subrating	district	county	yearbuilt	maxspan
Pearson Correlation	dkrating	1.000	.920	.898	.001	.027	.023	.486
	suprating	.920	1.000	.933	-.021	.007	.070	.507
	subrating	.898	.933	1.000	-.005	.011	.083	.535
	district	.001	-.021	-.005	1.000	.224	-.031	-.024
	county	.027	.007	.011	.224	1.000	.008	-.022
	yearbuilt	.023	.070	.083	-.031	.008	1.000	.182
	maxspan	.486	.507	.535	-.024	-.022	.182	1.000
	length	.228	.233	.239	.014	.019	.056	.594
	skew	-.016	-.006	.001	.063	-.022	.112	.112
	material	.487	.518	.493	-.059	-.044	.257	.196
	design	-.846	-.840	-.848	.003	-.011	.210	-.386
	dkstructype	.289	.271	.246	-.070	-.045	-.049	-.041
	dkmembtype	.136	.148	.170	-.019	-.043	.093	.089
	dksurftype	.135	.129	.151	-.112	-.025	-.106	-.083
	dkprotect	.196	.204	.193	.050	-.067	.136	.102

		length	skew	material	design	dkstructype	dkmembtype	dksurftype	dkprotect
Pearson Correlation	dkrating	.228	-.016	.487	-.846	.289	.136	.135	.196
	suprating	.233	-.006	.518	-.840	.271	.148	.129	.204
	subrating	.239	.001	.493	-.848	.246	.170	.151	.193
	district	.014	.063	-.059	.003	-.070	-.019	-.112	.050
	county	.019	-.022	-.044	-.011	-.045	-.043	-.025	-.067
	yearbuilt	.056	.112	.257	.210	-.049	.093	-.106	.136
	maxspan	.594	.112	.196	-.386	-.041	.089	-.083	.102
	length	1.000	.133	.111	-.207	-.030	.019	-.090	.062
	skew	.133	1.000	-.037	.037	-.105	.011	-.063	-.005
	material	.111	-.037	1.000	-.429	.473	.131	.173	.214
	design	-.207	.037	-.429	1.000	-.386	-.090	-.235	-.155
	dkstructype	-.030	-.105	.473	-.386	1.000	.057	.425	.178
	dkmembtype	.019	.011	.131	-.090	.057	1.000	.214	.218
	dksurftype	-.090	-.063	.173	-.235	.425	.214	1.000	.042
	dkprotect	.062	-.005	.214	-.155	.178	.218	.042	1.000

Table 3.38. Average Condition Ratings.

	Average Condition Rating			
Structural Type	Deck	Superstructure	Substructure	Overall
Other	6.2	6.0	5.9	6.0
Slab	6.6	6.6	6.3	6.5
Multi-Stringer, W or I-beam	6.2	6.0	6.0	6.1
Girder & floorbeam system	5.1	5.1	5.4	5.2
Tee beam	6.2	6.0	6.2	6.1
Box beam - multiple	7.0	6.9	7.1	7.0
Box beam - single or spread segmental	7.8	8.0	7.7	7.8
Frame - rigid (except culverts)	6.1	6.5	6.8	6.5
Truss - deck	6.0	8.0	6.0	6.7
Truss - thru & pony	5.5	4.9	5.2	5.2
Arch - deck, filled spandrel	6.2	6.0	5.8	6.0
Arch - thru	6.8	6.4	6.8	6.7
Movable - bascule	5.5	5.3	5.9	5.6
Movable - swing	7.5	7.0	7.0	7.2
Culvert	-	-	-	-
Overall Average:	6.3	6.3	6.3	6.3

Table 3.39. Average Condition Ratings, Typical Bridges.

	Average Condition Rating			
Structural Type	Deck	Superstructure	Substructure	Overall
Slab	6.6	6.6	6.3	6.5
Multi-Stringer, W or I-beam	6.2	6.0	6.0	6.1
Tee beam	6.2	6.0	6.2	6.1
Box beam - multiple	7.0	6.9	7.1	7.0
Box beam - single or spread segmental	7.8	8.0	7.7	7.8
Frame - rigid (except culverts)	6.1	6.5	6.8	6.5
Overall Average:	6.7	6.7	6.7	6.7

CHAPTER 4: INITIAL BRIDGE CONCEPTS AND FOCUS GROUP COMMENTS

Initial Concepts

After review of the technical literature and the characteristics of local agency bridges in Michigan, twelve initial bridge concepts were identified as potential choices. These were:

1) AASHTO/PCI beams (spans 20-140 ft)
2) Painted steel beams (spans 20 – 200 ft)
3) Galvanized steel beams (spans 20 – 60 ft)
4) Spread bulb tees (spans 70-180 ft)
5) Side-by-side bulb Tees (spans 70-180 ft)
6) Spread box beams (spans 20 – 130 ft)
7) Side-by-side box beams (spans 20 - 130 ft)
8) Precast double tees (span 20 - 50 ft)
9) Prefabricated steel/concrete double tees (spans 20 - 50 ft)
10) Slab (spans 30 – 50 ft)
11) 3-Sided prefabricated culvert (spans 20 – 60 ft)
12) 4-Sided prefabricated culvert (span < 25 ft)

Panelist Comments

A focus group meeting was held in July of 2016. After a brief introduction, twelve bridge concepts were presented to an invited panel of stakeholders for review and comment (see Appendix A). The morning session included a panel of 4 local agency representatives and 3 bridge engineering consultants, while the afternoon session included a panel of representatives from 4 bridge contractors and 2 fabricators (steel and prestressed concrete). The panelists were asked to consider performance, cost, constructability, durability/maintenance, as well as any other issues or concerns that they thought relevant, in the context of developing bridge plan guides for local agency use. Sessions were scheduled for 3 hours. In general, the morning session involved robust discussion and generated a significant number of comments, while the afternoon session had less discussion. The latter finished about an hour earlier than scheduled. A summary of the comments received are provided below.

Morning Session Comments

AASHTO/PCI Beam

The panel concluded that AASHTO PCI beams are not a good candidate for the plan guide. This is because AASHTO beams are inefficient for the shorter bridge spans which commonly appear in the local agency inventory, due to the limited availability of smaller beam depths. In addition to structural inefficiency, the availability of smaller depth beams is critical because the hydraulic requirements (i.e. width and height of clear opening below the bridge) often drives the bridge configurations that are possible.

Panelists also thought these beams to be durable, though deterioration of beam ends can be problematic for some joint conditions. However, other beam types are similarly subjected to beam end deterioration as well.

Painted Steel Beam

Although possibly useful for shorter spans, panelists were concerned with the long term maintenance costs of repainting steel beams. These beams not only require periodic repainting, but are difficult to repaint due to environmental considerations associated with paint removal. As such, the panel generally recommended to avoid this structure.

As an alternative, timber bridge structures were recommended for short spans. These can be low cost, are easy to construct, and have a preferred aesthetic for some natural locations. These structures can only be used for very short spans, however. Another concern is if a wood deck is used with an asphalt overlay, problematic cracking of the overlay surface may occur.

Galvanized Steel Beam

The panel recommended galvanized steel beam structures, as due the durability of galvanization, long term maintenance costs are thought to be significantly lower than those accompanying painted steel beams. Plate girders may also be used for longer spans, although these will require longer fabrication times.

A deck type to consider is corrugated steel decking used as stay-in-place formwork (spanning perpendicular to closely spaced beams), and filled with asphalt.

The panel did not have experience with other exotic steel beam shapes such as bent plate girders.

Spread Bulb Tee

The panel agreed that spread bulb tee beams are suitable for longer spans. Alternative shapes include the MI 1800 girder and the Indiana bulb tee. The beams are thought to be efficient to fabricate, and may be similar to spread box beams in terms of application.

It was noted that some local agencies desire to minimize deck reinforcement placement and concrete pouring, as concrete work is difficult to handle with a typical local agency workforce.

Side-by-Side Bulb Tee

The panel did not recommend side by side bulb tee structures.

Spread Box Beam

A spread box beam structure was recommended by the panel. It is commonly used and has shown to be a good choice overall. Some of its advantages include familiarity; availability of shallow beam depths; no post-tensioning (as opposed to side-by-side box beams); and its suitability for staged construction, although the latter is generally not a critical issue for low-volume local agency roads. It was noted that this configuration does have a potential deterioration problem with the exterior beams, however, if the method of deck drainage is not detailed properly.

Side-by-Side Box Beam

Side-by-side box beam structures were not well-liked by some panel members, mainly due to the common issue of longitudinal deck cracking along beam joints. Another disadvantage is that it is difficult to replace damaged beams, especially if transversely post-tensioned. It was noted that if detailed correctly, however, long term maintenance problems such as deck cracking can be eliminated or reduced to a reasonable level. The prime advantage of this system is the potential use of small beam depths. Ease of construction was also noted. Despite its disadvantages, due to the small beam depths possible with this system, it was recommended for consideration because in some cases it is the only bridge type that will reasonably work in some hydraulic situations.

Precast Double Tee

Due to its small web thickness, a precast double tee structure may have insufficient space to place prestress strands, limiting its ability to carry load. Another disadvantage of the thin webs is the lack of reserve capacity and cover due to surface deterioration. The panel did not recommended this structure.

Prefabricated Steel/Concrete Double Tee

Although few specific negative comments were provided, the panel generally did not react positively a prefabricated steel/concrete double tee structure. A concern raised was similar to that with side-by-side box beams; namely, the potential for longitudinal cracking between beam tees. Some potential solutions were the use of diaphragms as well as closure pours with high performance concrete rather than a cast-in-place deck. The overall consensus was not to consider this structure for plan development.

Slab

The panel did not recommend prestressed concrete slab structures.

3 and 4 Sided Prefabricated Culverts

Although expensive, culverts are commonly used. Three sided culverts are most suitable for short spans up to about 30-40 ft. Difficulties include the inclusion of skewed configurations and the increasing height requirement as span length increases. A issue of concern is that 3-sided culverts typically require pile foundations, although 4-sided culverts can utilize spread footings in many cases. Because of the prevailing standard culvert plans that already exist from manufacturers, these structures were not recommended for further plan development.

Other Comments

Several panelists expressed a desire to simplify the design process, particularly with regard to integrating DEQ environmental regulations, such that a prequalified design might be developed. Such an integration may be beyond the project scope, however.

It was suggested that it is generally better to provide bridge widths larger than initially needed to extend the structure's functional relevance in the future to carry larger traffic volumes.

It was expected that the bridge plan guide will provide minimal cost savings from design; most savings may occur from construction and a reduction in future maintenance costs.

Bridge ratings should be provided along with the designs.

Afternoon Session Comments

In general, contractors and fabricators indicated that any of the bridge types can be built without excessive costs or undue difficulties.

AASHTO/PCI Beam

The afternoon panel agreed with the morning panel that AASHTO/PCI beam structures are generally not efficient for shorter spans.

Painted Steel Beam

Due to material availability, the lead time for steel components is relatively long compared to concrete.

Galvanized Steel Beam

To allow galvanization of longer span beams (i.e. up to about 60 ft), they can be dipped twice in existing vats. However, such 'double-dipping' generally costs twice that of single-dipping.

Galvanized beams may be painted after galvanization if a particular aesthetic is desired.

Spread Bulb Tee

Spread bulb tees not difficult to fabricate, especially with a 49 in flange width, which is most common. The wider flanges provide a beam that is more stable during construction. However, these beams are more difficult to transport and erect compared to AASHTO beams.

Side-by-Side Bulb Tee

An advantage of the side-by-side bulb tee system is that it has the potential for faster construction than the spread bulb tee, due to a reduction in formwork requirements.

It was noted that stay-in-place formwork generally results in cheaper and faster construction. Another advantage is that the formwork prevents debris from deteriorating decks from falling below the bridge. However, most local agencies avoid stay-in-place forms because the deck underside cannot be inspected, and construction time is not a prime concern for most low-volume roads.

Spread Box Beam

Spread box beam structures are easy to construct, but the interior of the beams cannot be inspected. To reduce construction and fabrication costs, it is important that beam geometry and formwork variations are minimized. It was noted that 'decked' spread box beams (i.e. a tee-beam formed with a box beam in place of the girder) are difficult to work with and are not a good choice. It was suggested that most local agencies prefer cast-in-place decks over prefabricated decks.

Side-by-Side Box Beam

Increasing the number of post-tensioning strands (in order to minimize longitudinal joint cracking) increases the fabrication complexity of side-by-side box beams.

Precast Double Tee

Precast double tee structures were not recommended.

Prefabricated Steel/Concrete Double Tee

Precast double tee structures will likely be associated with longer construction times, due to the availability of steel as well as the steel/concrete prefabrication. They were not recommended.

Slab

Although no construction or fabrication difficulties were noted with a slab system, it was pointed out that there is no fabrication advantage over solid slabs as compared to voided slabs/boxes.

3 and 4 Sided Prefabricated Culverts

It was mentioned that culverts can be very quickly constructed.

Other Comments

Precast substructures can reduce construction time. Geosynthetic reinforced soil (GRS) abutments were recommended for consideration.

It is important that the plans are clear and simple to follow. A concern raised is that too much complexity in the guidelines may lead to a confusion of options; i.e. the choice of various different options should not be left to the contractor.

Typical reinforcement bar sizes and placement in slabs should be specified.

The level of detail in existing plans is generally sufficient.

On-Line Survey

An on-line survey was developed to engage local agencies in the plan guidelines development process as well as to receive additional feedback. The survey was advertised in The Bridge periodical, with an associated news article (see Appendices B and C).

A total of 10 responses were received for the survey. All were All CRC engineers or engineer/managers, representing the following Michigan counties: Allegan, Genesee, Baraga, Huron, Eaton, Ontonagon, Dickinson, Keweenaw, St. Clair, and Kalamazoo.

The respondents ranking of the various concepts presented is given in Table 4.1. In the ranking, "1" indicates the highest preference while "10" the lowest preference.

Table 4.1. On-line Survey Ranking.

	AASHTO/ PCI	Painted Steel	Galvanized Steel	Spread Bulb Tee	Side-by-Side Bulb Tee	Spread Box	Side-by-Side Box	Precast Double Tee	Prefab Steel/RC Double Tee	Prestressed Slab	Prefabricated Culvert	Timber
	5	10	1	5	5	1	3	5	5	5	2	5
	5	10	10	5	8	1	3	8	10	10	2	4
	2	4	3	5		1	9	8	7	10	10	6
	1	9	8	1	3	9	5	1	9	9	1	9
	1	6	7	1	8	10	9	2	3	4	5	2
	9		8	7	6	5	1	4		10	3	2
		6	7	8	9	5	1	6	4	2	3	10
	4	3	5	6	7	8	9	10	10	10	1	2
	6	7	2	5	4	8	10	10	3	10	1	9
	4	2	1	5	6	3	7	8	9	10	1	6
worst	9	10	10	8	9	10	10	10	10	10	10	10
best	1	2	1	1	3	1	1	1	3	2	1	2
mode (#)	1,4,5 (2 each)	6,10 (2 each)	1,7 (2 each)	5 (5)	8 (2)	1 (3)	9 (3)	8 (3)	10 (2)	10 (6)	1 (4)	2 (3)
mean	4.1	6.3	5.2	4.8	6.2	5.1	5.7	6.2	6.7	8	2.9	5.5

As shown in the table, the most disliked structures were: prestressed slab, painted steel, side-by-side bulb tee, precast double tee, and prefab steel/RC double tee, while the most liked were: prefab culverts, AASHTO/PCI, and spread bulb tees.

Three of the respondents indicated that their agency and/or their consultants used Microstation for plan development, while 8 used Autocad.

Survey of Michigan Bridge Conference Attendees

A similar survey was submitted to Michigan Bridge Conference attendees in March, 2017 (see Appendix D). This survey received 78 responses, 20 of which were local agency representatives and 29 of which were local agency consultants. A summary of these responses are given in Tables 4.2-4.7, while details are given in Appendices E and F. Of particular interest is the responses to the question: "Rank the following 12 structures given below in your order of preference for plan development ("1" being highest preference)".

Results from all respondents are given in Table 4.2, while detailed responses are given in Appendix E. Numbers given in the table indicate how many respondents ranked a particular bridge type a given rank value (for example, 16 respondents give the "AASHTO/PCI Beam" type a ranking of 1. In the table, a "10" ranking may indicate a provided rank of 10, 11, or 12, due to limitations of the ability of the on-line survey to record values greater than 10. Additionally, respondents were asked to indicate their level of experience with a particular bridge type, where None = N, Little = L, Moderate = M, and Substantial = S. The percentage of respondents indicating (moderate or substantial) experience, or (little or no) experience are given in the final columns of the table.

Table 4.2. Ratings For All Respondents.

Local Agency + MDOT Ranking

	Ranking												Experience (%)	
Bridge Type	1	2	3	4	5	6	7	8	9	10	mean	mode	M & S	L & N
AASHTO/PCI Beam	16	7	15	10	7	4	2	0	1	4	3.6	1	77	23
Painted Steel	6	5	7	5	3	6	6	8	5	12	6.0	10	65	35
Galvanized Steel	2	9	7	9	6	3	3	6	7	12	5.9	10	29	71
Spread Bulb Tee	14	6	8	7	6	7	3	5	3	6	4.6	1	42	58
SBS Bulb Tee	3	5	3	3	5	5	10	11	9	9	6.6	8	14	86
Spread Box Beam	9	11	9	6	3	6	4	5	5	9	5.0	2	71	29
SBS Box Beam	8	11	4	2	9	2	4	3	3	23	6.0	10	86	14
Precast Double Tee	1	4	1	5	2	9	8	7	12	14	7.2	10	13	87
Prefab Steel/RC DT	1	1	5	2	7	2	6	9	3	25	7.6	10	8	92
PC Slab	1	1	2	3	8	4	3	6	7	25	7.8	10	16	84
Culvert	14	9	5	13	6	6	6	1	2	9	4.5	1	79	21
Timber	1	5	7	4	7	8	5	1	2	29	7.0	10	33	67

The bridge types which were most and least liked, as compared to the previous on-line survey of 10 respondents, is given in Table 4.3.

Table 4.3. Comparison of Survey Results.

Perception	Bridge Conference Survey	On-line Survey
Liked	AASHTO/PCI Beam Culvert Spread Bulb Tee	Culvert AASHTO/PCI Beam Spread bulb Tee
Neutral-positive	Spread Box	Spread Box Beam SBS Box Beam Galvanized Steel Beam
Neutral-negative	Galvanized Steel Beam Painted Steel Beam SBS Box Beam	Timber SBS Bulb Tee Precast Double Tee
Disliked	Timber Everything else	PC Slab Prefab Steel/RC DT Painted steel

Results when considering only local agency representatives and their consultants are given in Table 4.4, while detailed results are given in Appendix F. Table 4.5 presents results when limiting responses only to local agency representiatives and their consultants who indicated that they have substantial or moderate experience with a particular bridge type that they ranked, while Table 4.6 presents mean rankings by bridge type considering the results from Tables 4.4, 4.5, and 4.6.

Table 4.4. Ratings for Local Agency Representatives and Their Consultants.

Local Agency Ranking

	Ranking												Experience (%)	
Bridge Type	1	2	3	4	5	6	7	8	9	10	mean	mode	M & S	L & N
AASHTO/PCI Beam	10	2	10	8	6	3	1	0	0	3	3.7	1, 3	75	25
Painted Steel	2	2	4	1	2	5	4	6	3	10	6.8	10	54	46
Galvanized Steel	1	4	4	6	4	3	3	3	4	9	6.2	10	39	61
Spread Bulb Tee	6	4	5	5	3	6	2	4	2	5	5.1	1, 6	50	50
SBS Bulb Tee	3	3	1	2	3	2	5	8	8	6	6.8	8, 9	39	61
Spread Box Beam	6	8	7	3	2	4	3	3	3	3	4.5	2	65	35
SBS Box Beam	7	11	3	2	3	1	2	0	3	12	5.2	10	88	12
Precast Double Tee	1	2	1	3	1	4	4	7	5	12	7.5	10	40	60
Prefab Steel/RC DT	0	1	2	0	4	1	4	3	2	21	8.3	10	20	80
PC Slab	1	1	2	2	7	2	2	3	4	14	7.3	10	44	56
Culvert	12	5	3	10	6	1	3	1	0	4	3.9	1	79	21
Timber	1	4	7	4	5	7	4	1	2	10	5.9	10	59	41

Table 4.5. Ratings for Local Agency Representatives and Their Consultants with Experience.

Local Agency Ranking With Substantial or Moderate Experience

Bridge Type	1	2	3	4	5	6	7	8	9	10	mean	mode
AASHTO/PCI Beam	8	1	9	7	3	1	1	0	0	2	3.5	3
Painted Steel	1	2	1	1	1	3	4	4	2	3	6.5	7, 8
Galvanized Steel	1	2	2	4	2	1	1	0	2	0	4.5	4
Spread Bulb Tee	4	3	0	3	1	1	1	0	0	1	3.6	1
SBS Bulb Tee	0	1	0	0	1	1	0	0	1	0	5.5	~
Spread Box Beam	6	4	4	2	0	1	2	0	1	3	4.0	1
SBS Box Beam	7	10	3	2	3	1	1	0	3	10	5.0	2, 10
Precast Double Tee	0	0	0	0	0	0	2	1	0	1	8.0	7
Prefab Steel/RC DT	0	0	0	0	0	0	0	1	0	1	9.0	~
PC Slab	0	0	1	0	1	0	1	1	0	2	7.2	10
Culvert	10	4	3	10	5	1	3	0	0	1	3.5	1, 4
Timber	1	2	2	2	1	4	1	1	0	2	5.2	6

Table 4.6. Mean Rankings.

Mean Ranking By Bridge Type

Local Agency + MDOT		Local Agency		Experienced Local Agency	
3.6	AASHTO/PCI Beam	3.7	AASHTO/PCI Beam	3.5	Culvert
4.5	Culvert	3.9	Culvert	3.5	AASHTO/PCI Beam
4.6	Spread Bulb Tee	4.5	Spread Box Beam	3.6	Spread Bulb Tee
5.0	Spread Box Beam	5.1	Spread Bulb Tee	4.0	Spread Box Beam
5.9	Galvanized Steel	5.2	SBS Box Beam	4.5	Galvanized Steel
6.0	Painted Steel	5.9	Timber	5.0	SBS Box Beam
6.0	SBS Box Beam	6.2	Galvanized Steel	5.2	Timber
6.6	SBS Bulb Tee	6.8	SBS Bulb Tee	5.5	SBS Bulb Tee
7.0	Timber	6.8	Painted Steel	6.5	Painted Steel
7.2	Precast Double Tee	7.3	PC Slab	7.2	PC Slab
7.6	Prefab Steel/RC DT	7.5	Precast Double Tee	8.0	Precast Double Tee
7.8	PC Slab	8.3	Prefab Steel/RC DT	9.0	Prefab Steel/RC DT

In summary, as shown in Table 4.6, only minor differences exist between groups. Comparing all responses (i.e. local agency and MDOT responses) to local agency responses, for local agency results: timber moved up 3 places, switching places with painted steel; galvanized steel moved down 2 places; PC slabs moved up 2 places; and two other bridge types changed only 1 place in the ranking.

Moreover, local agency and experienced local agency results showed little difference, where 4 bridge types changed in ranking by only one place, and galvanized steel moved up in ranking 2 spots in the experienced group. An overall, qualitative summary is given in Table 4.7. Comments that were provided on the surveys are given in Appendix G.

Table 4.7. Summary of Results.

Most Liked by All Groups	Most Disliked by All Groups
AASHTO/PCI beam	Precast Double Tee
Culvert	Prefab Steel/RC Double Tee
Spread Box Beam	PC Slab
Spread Bulb Tee	
Moderately Liked by Local Agency	**Moderately Disliked by Local Agency**
SBS Box Beam	SBS Bulb Tee
Timber	Painted Steel
Galvanized Steel	

Based on all comments received, the following concepts were recommended for further development:

Very Short Spans

- Timber. Applicable to spans near 40 ft. Cheap, easy to construct, and high aesthetic quality in natural settings.

Short to Medium Spans

- Galvanized steel. Viable up to about 60 ft, due to the limited size of galvanizing vats. Good long-term maintenance performance.

- Spread box beam. A familiar and generally well-performing choice up to about 70 ft.

- Side-by-side box beam. For spans up to about 70 ft, although not a first choice, useful when small beam depths are required and other options cannot meet this limitation.

Longer Spans

- Spread bulb tees. Best for spans greater than 70 ft.

Note that culverts and timber structures are not considered for plan development, as these structures are typically designed and prefabricated or available in 'kits' by manufacturers.

CHAPTER 5: LIFE CYCLE COST ANALYSIS

Bridge Designs

Based on the information presented in earlier chapters, the final bridge designs were developed. A summary of these designs are given in Appendix H.

Life Cycle Cost Analysis (LCCA) Process

A LCCA was conducted for each of the alternative bridge systems above. The LCCA includes costs and activity timing for initial construction, inspection, repair and maintenance, demolition, replacement, and, if desired, the associated user costs. LCCAs were conducted considering agency costs only, as well as both agency and user costs. Both deterministic as well as probabilistic LCCAs were completed. Important random variables (RVs) for the probabilistic LCCA include those for maintenance activity timing, agency costs, and user costs.

Activity Timing

As suggested by FHWA (2002), the analysis period must be long enough to include major rehabilitation actions for each reinforcement alternative. To satisfy this requirement, the LCCA should be conducted up to 75-100 years. However, results are available cumulatively for each year, so the LCC for any lesser period of time can be referenced. For consistent LCC comparison among cases, it is important that the maintenance actions are scheduled such that the expected bridge condition, at any year, is the same for all alternatives. Operation, maintenance, and repair (OM&R) strategies are based on current MDOT and expected practices. The assumed maintenance schedule and related information for all structures is given in Tables 5.1 and 5.2. Note that joint replacement at year 40 is for concrete bridges only.

Table 5.1. Maintenance Activities.

Activity	Year
Deck patch (& other CSM)	7, 18, 31, 46, 59, 72
Joint replacement	12, 25, 40, 52, 65
Epoxy overlay	12, 25, 52
Substructure repair	25, 52
Shallow overlay	40, 65
Beam end repair	40 (PCI); 25, 52 (Steel)
HMA overlay	80
Deck replacement	90

Table 5.2. Maintenance Parameters.

Maintenance Parameter	Value	Units
Length of affected roadway	4	miles
Length affected by detailed inspection	0.1	miles
Length affected by routine inspection	0.1	miles
Time for deck replacement	90	days
Time for deck patch	7	days
Time for shallow overlay	15	days
Time for detailed inspection	0.4	days
Time for routine inspection	0.4	days

Agency Costs

Agency costs and associated RVs include items such as material, personnel, and equipment costs corresponding to events such as: initial construction, routine and detailed inspections, deck patch, deck overlay, deck replacement, beam end repair, beam replacement, superstructure demolition, superstructure replacement, and others, as appropriate for the bridge system considered. Many of these costs are based on a combination of sub-costs. Agency costs are based on data obtained from MDOT as well as bridge construction rep. Variance information for RV data for material and construction costs, in terms of COV, is taken from the available technical literature (Eamon et al. 2014).

Assumed agency costs are given in Tables 5.3 and 5.4.

Table 5.3. Initial Construction Costs.

Item	Cost	Units	Assumptions
Deck + barriers	57	$/SF	
Steel diaphragm for PC beams	4	$/LB	
Prestressed concrete beam	287	$/FT	
Steel beam & diaphragms	5.7	$/LB	including galvanization
Substructure (abutments & wing walls)	90	$/CF	5000 CF; no foundations
Deck Joint	560	$/FT	2 joints replaced

Table 5.4 Maintenance Costs.

Item	Cost	Units	Assumptions
Detailed inspection	1	$/SF	Entire deck
Routine inspection	500	$	
Deck patch	38	$/SF	2% of deck area
Deck shallow overlay	30	$/SF	Entire deck
Deck epoxy overlay	3.8	$/SF	Entire deck
HMA overlay	5.5	$/SF	Entire deck
Deck replacement	75	$/SF	Entire deck
Joint replacement	560	$/FT	2 joints replaced
Beam end repair (Steel)	6000		4 beam ends repaired
Beam end repair (PCI)	4200	$/end	4 beam ends repaired
Substructure repair	310	$/CF	10 CF
Traffic control for deck patch, beam end repair	7500	$	
Traffic control for deck, beam replace, overlay	10%	of project	
Mobilization	10%	of project	

<u>User costs</u>

During construction and maintenance work, traffic delays as well as increased accident rates occur. The resulting delay costs include the value of time lost due to increased travel time as well as the cost of additional vehicle operation. Therefore, mean user cost is taken as the sum of travel time costs, vehicle operating costs, and crash costs. Equations 5.1 – 5.3 are used to calculate these costs (Ehlen 1999).

For travel through a construction zone:

$$\text{Travel time costs} = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) \times AADT \times N \times w \qquad (5.1)$$

$$\text{Vehicle operating costs} = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) \times AADT \times N \times r \qquad (5.2)$$

$$\text{Crash costs} = L \times AADT \times N \times \left(A_a - A_n \right) \times c_a \qquad (5.3)$$

where L = length of affected roadway over which cars drive; S_a = traffic speed during road work; S_n = normal traffic speed; N = number of days of road work; w = hourly time value of drivers; r = hourly vehicle operating cost; c_a = cost per accident; A_a and A_n = accident rate during construction and normal accident rate per million vehicle-miles, respectively.

For most agency bridges, it is assumed that the bridge is closed and the traffic is re-routed. In this case, for travel through a detour, the expressions above are modified as follows:

$$\text{Travel time costs} = \left(\frac{L}{S_n} \right) \times AADT \times N \times w \qquad (5.4)$$

$$\text{Vehicle operating costs} = \left(\frac{L}{S_n} \right) \times AADT \times N \times r \qquad (5.5)$$

$$\text{Crash costs} = L \times AADT \times N \times An \times c_a \qquad (5.6)$$

Assumed user cost parameters are given in Table 5.5.

Table 5.5. User Cost Parameters.

User Cost Parameters	Value	Unit
AADT	1000	vehicles/day
Initial AADT	1000	vehicles/day
Max AADT	20000	vehicles/day
AADT growth rate	2	%/ year
Accident rate	1.56	%/million vehicle miles
Driving speed	55	MPH
Driver cost	13.61	$/hr
Vehicle operating cost	11.22	$/hr
Cost per vehicle accident	99560	$

<u>Life cycle costs</u>

The total life cycle cost is the sum of all yearly partial costs. Because dollars spent at different times have different present values (PV), future costs at time t, C_t, are converted to consistent present dollar values by adjusting future costs using the real discount rate r, and then summing the results over T years (Ehlen 1999), per eq. 5.7. The real discount rate reflects the opportunity value of time and is used to calculate the effects of both inflation and discounting. It is typically from 2-3%, and estimates are taken from the federal Office of Management and Budget (Eamon et al. 2012). For this study, the initial construction cost occurs in year 0, while the first year after bridge construction is defined as year 1. The costs associated with any subsequent activity are presented in terms of present value considering the real discount rate. The real discount rate is taken as 3% while the inflation rate is taken as 4%.

$$\text{LCC} = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \qquad (5.7)$$

Probabilistic LCCA

For each bridge configuration considered, Monte Carlo Simulation (MCS) is used to simulate costs for each year based on the activity timing. The cumulative cost at a given year is determined by converting previous yearly costs to present value and summing the results up to year j using Eq. 5.7. To conduct the probabilistic analysis, a limit state function (g) is needed. In this research, the limit state function of interest is in terms of cost, and can be written as: $g_j = C_R - C_{alternative}$, where C_R is the cumulative cost of a reference (i.e. control) bridge configuration, and $C_{alternative}$ is the cumulative cost of an alternate considered. If $g_j < 0$, then C_R was found to be cheaper for that year considered for simulation i. This result (i.e. if $g_j > 0$ or $g_j < 0$) is recorded for each year j. For each year considered, the simulation is repeated a sufficient number of times for accurate statistical quantification of the results (for example, 100,000 simulations). The cost probabilities *(P)* for each year j (i.e. probability of the cost of the alternate exceeding the reference bridge) can then be determined with the traditional MCS process using Eq. 5.8.

$$P(C_R < C_{alternative})_j = \frac{(\#\,of\ times\ g < 0)_j}{(total\ simulations; 100{,}000)} \qquad (8)$$

Random variables in the probabilistic LCCA are given in Table 5.6. Mean values for the RVs are taken as the base costs, while COVs are taken from Eamon et al. (2014). All distributions are assumed to be normal.

Table 5.6. LCCA Random Variables.

Agency Cost RVs			User Cost RVs		
RV	Description	COV	RV	Description	COV
AC(1)	Bridge construction	0.20	UC(1)	Deck patch	*
AC(2)	Deck patch	0.40	UC(2)	Deck epoxy overlay	*
AC(3)	Deck epoxy overlay	0.40	UC(3)	Deck HMA overlay	*
AC(4)	Deck HMA overlay	0.40	UC(4)	Deck replacement	*
AC(5)	Deck replacement	0.20	UC(5)	Superstructure replacement	*
AC(6)	Joint replacement	0.40	UC(6)	Routine inspection	*
AC(7)	Beam end repair	0.60	UC(7)	Detailed inspection	*
AC(8)	Beam replacement	0.20	UC(8)	Beam paint	*
AC(9)	Beam paint	0.40	UC(9)	Routine inspection	*
AC(10)	Traffic control	0.60	UC(10)	Detailed inspection	*

*COV varies and is a function of COVs for travel time cost (0.12), operating cost (0.18), and crash cost (0.13).

Results

Initial deterministic costs as a function of span, as well as deterministic total life cycle costs (at 90 years) are given in Figures 5.1-5.16. Detailed yearly cost values are summarized in Appendix I. Note that foundation costs are not included.

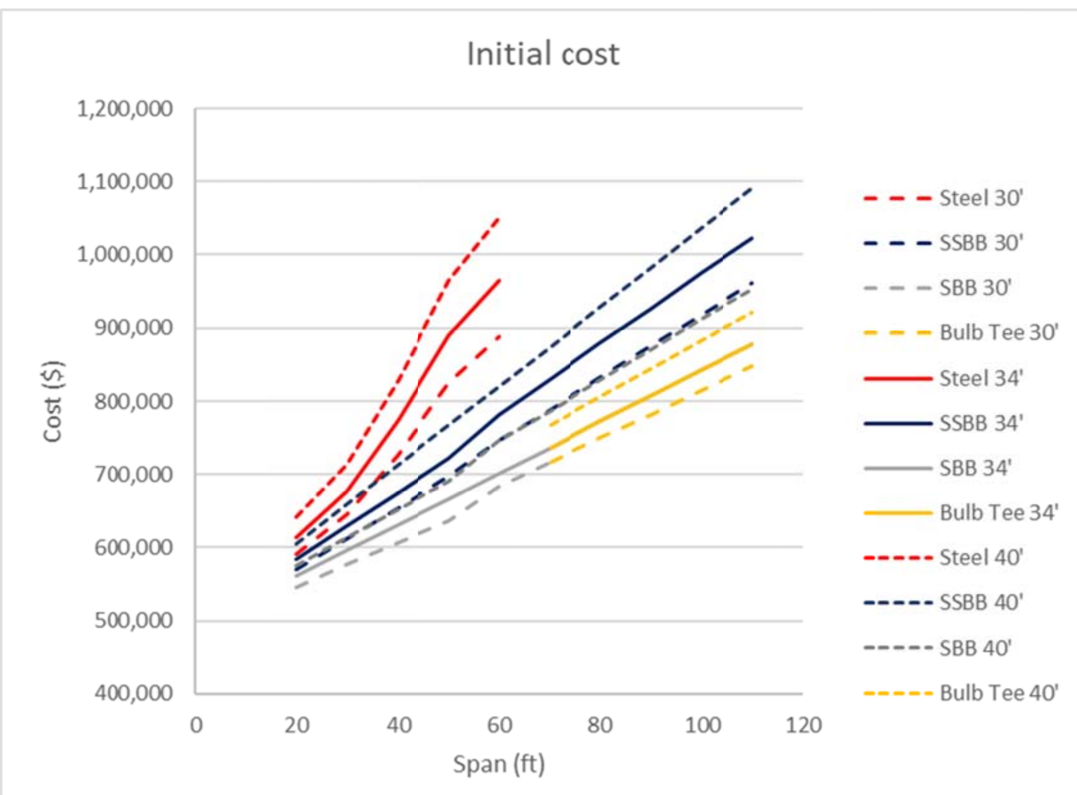

Figure 5.1. Initial Construction Costs.

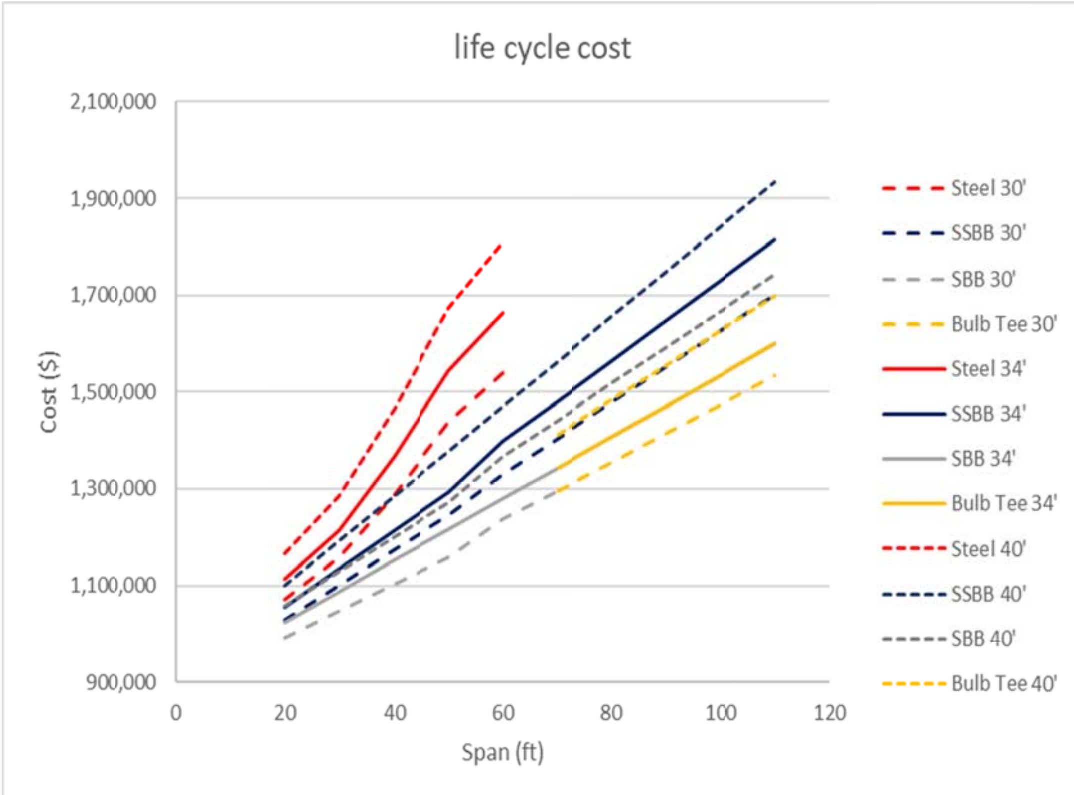

Figure 5.2. Total Life Cycle Cost at 90 Years.

An example of deterministic bridge costs as a function of year are given in Figures 5.3-5.12, while example probabilistic results are given in Figures 5.13-5.16.

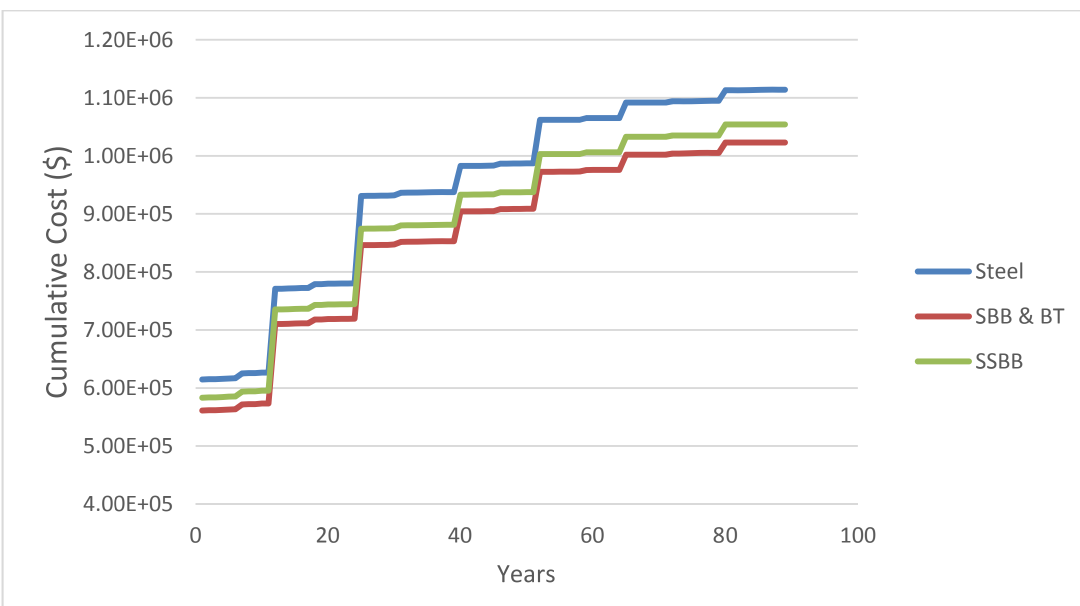

Figure 5.3. Cumulative Deterministic Cost, 20’ Span.

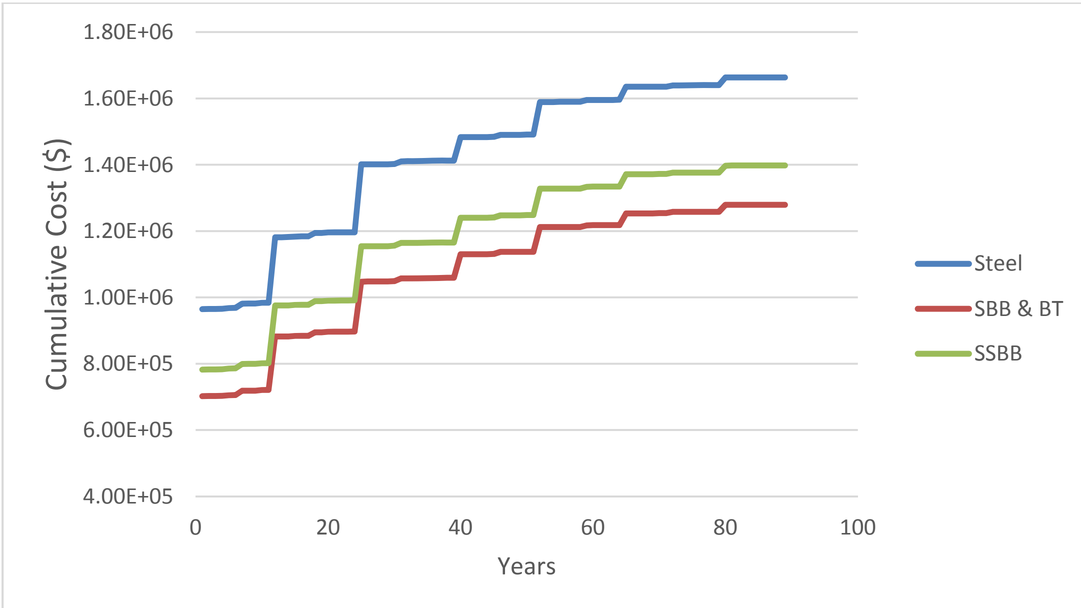

Figure 5.4. Cumulative Deterministic Cost, 60’ Span.

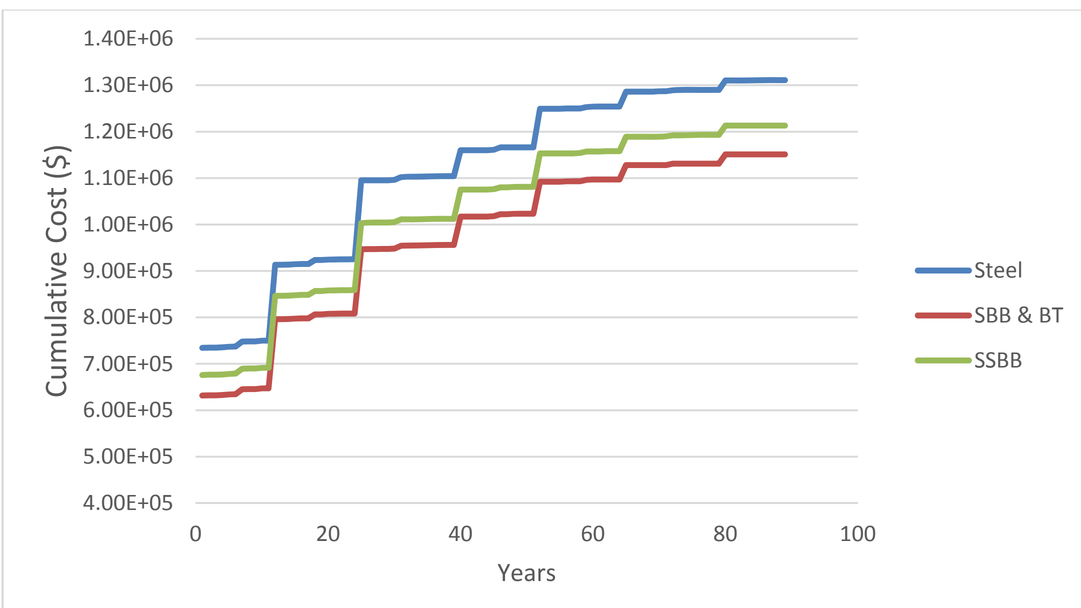

Figure 5.5. Cumulative Deterministic Cost, 40’ Span.

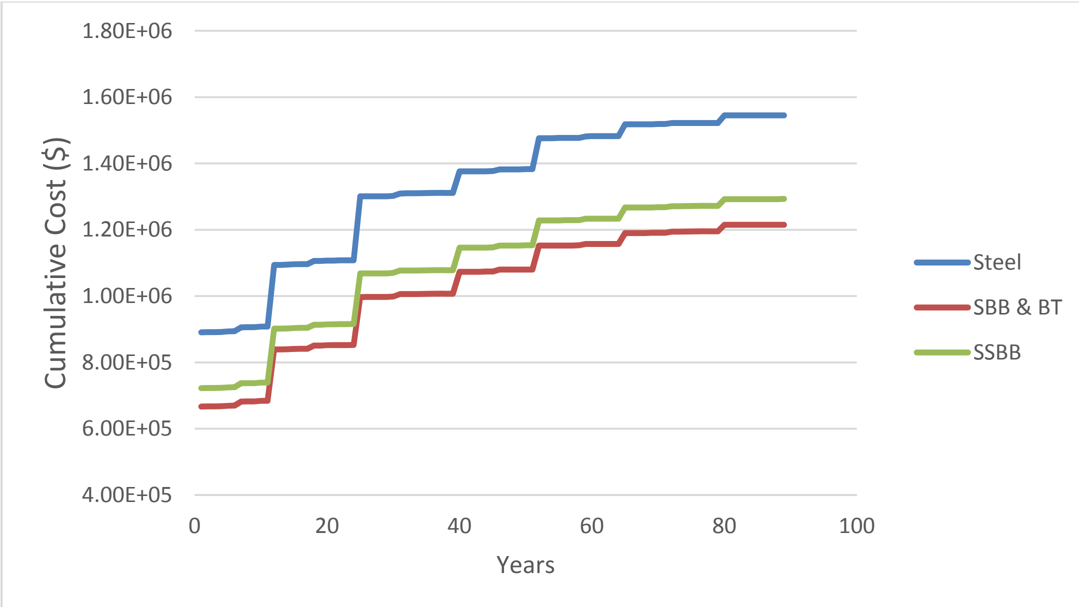

Figure 5.6. Cumulative Deterministic Cost, 50’ Span.

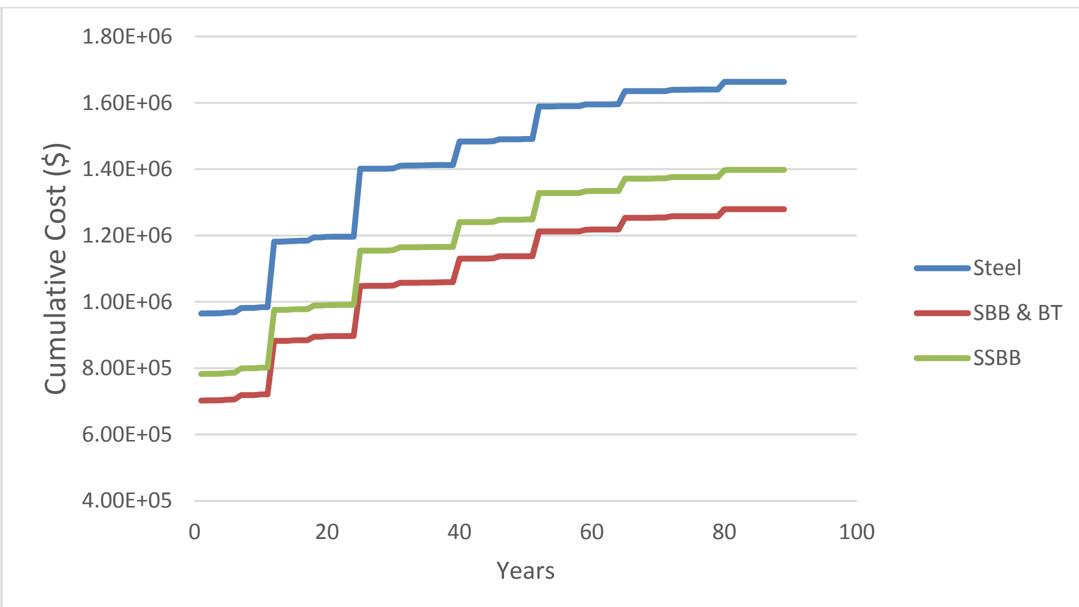

Figure 5.7. Cumulative Deterministic Cost, 60’ Span.

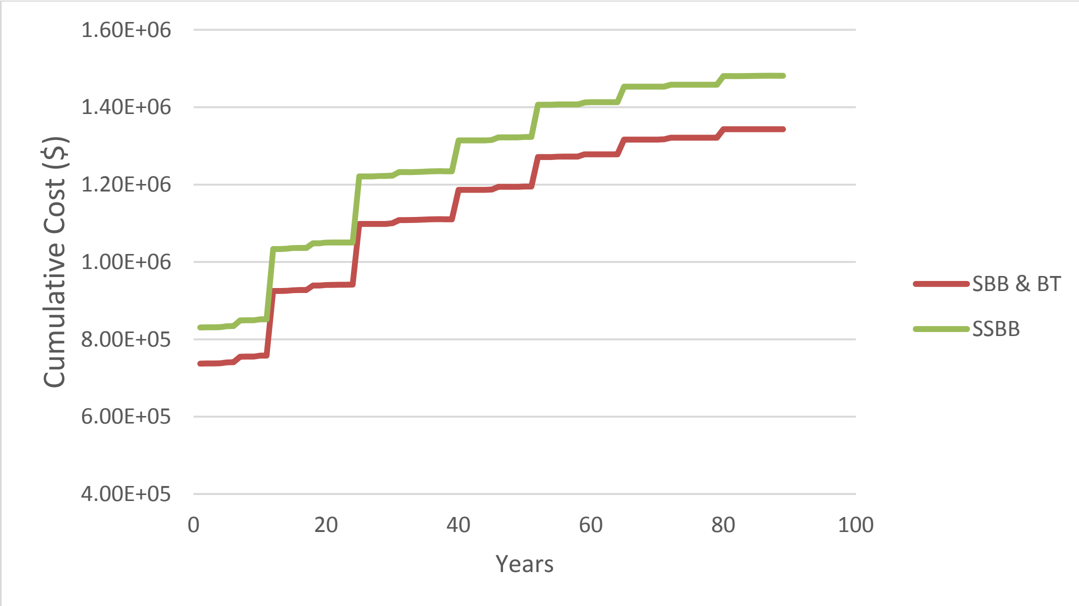

Figure 5.8. Cumulative Deterministic Cost, 70’ Span.

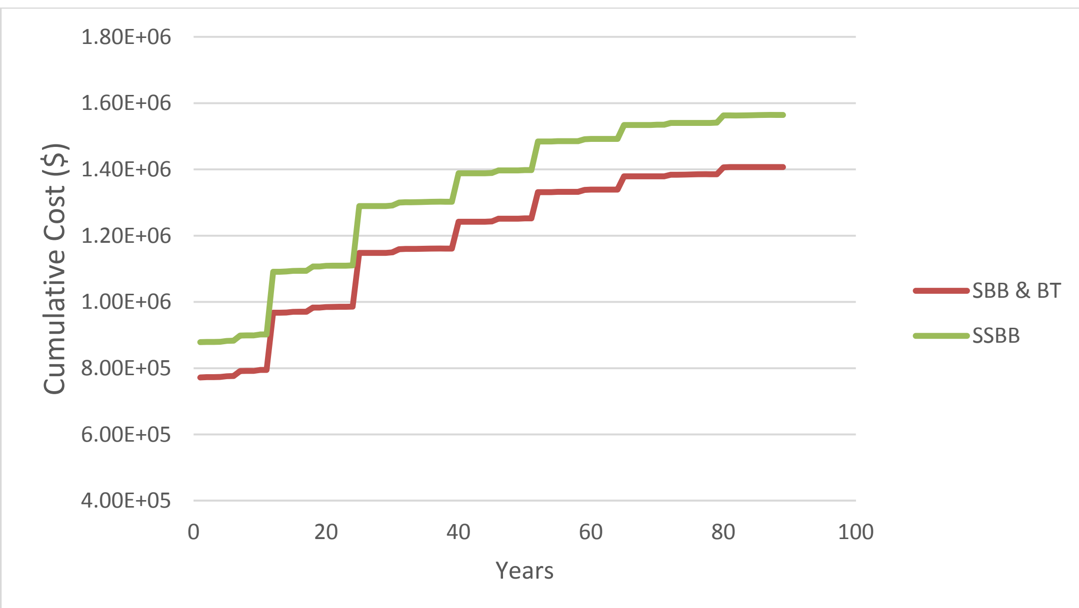

Figure 5.9. Cumulative Deterministic Cost, 80’ Span.

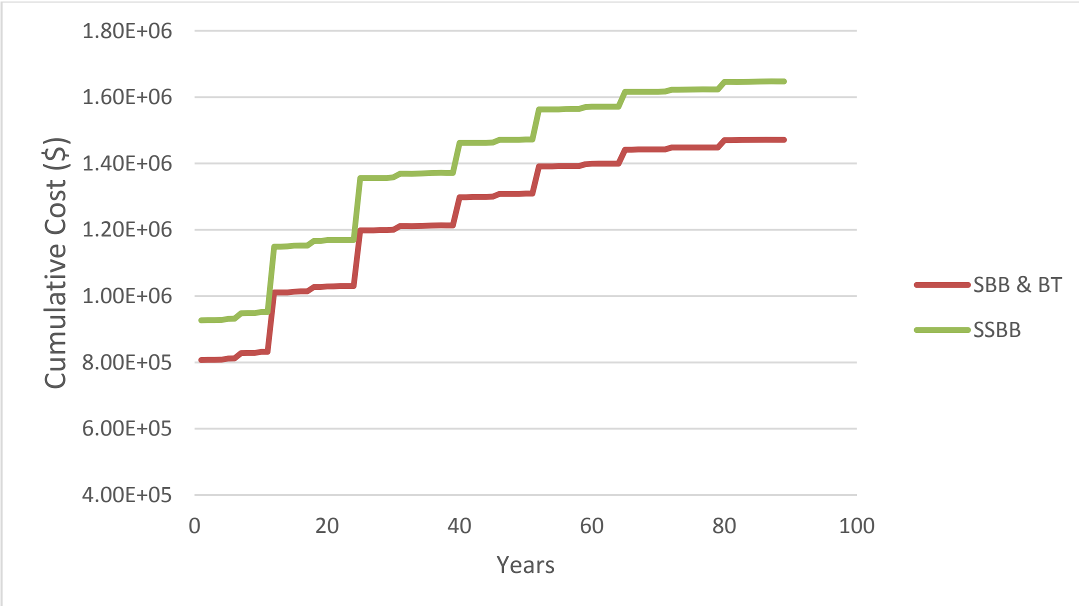

Figure 5.10. Cumulative Deterministic Cost, 90’ Span.

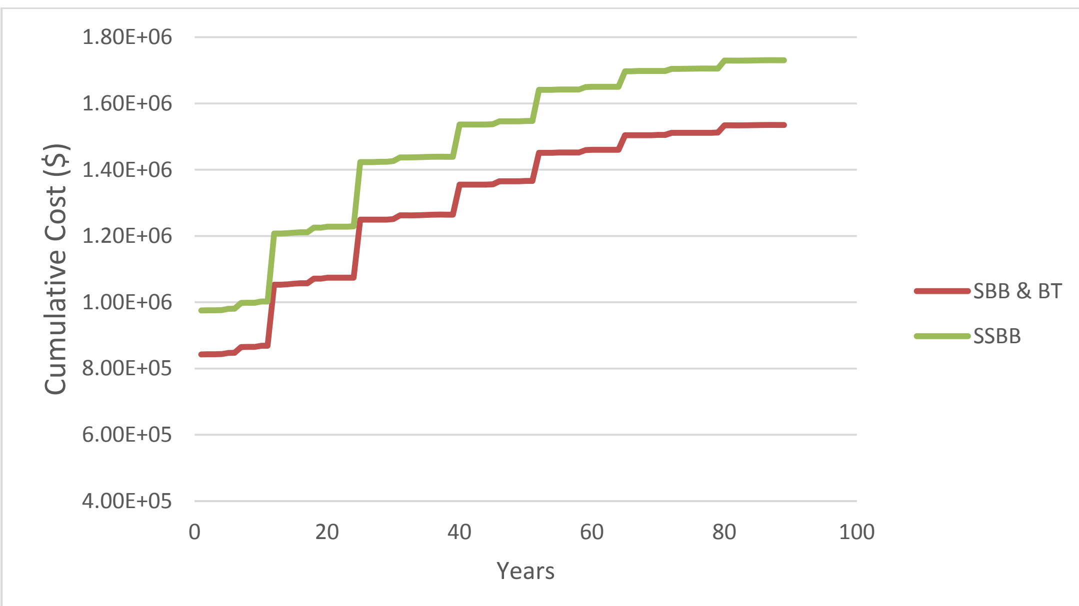

Figure 5.11. Cumulative Deterministic Cost, 100’ Span.

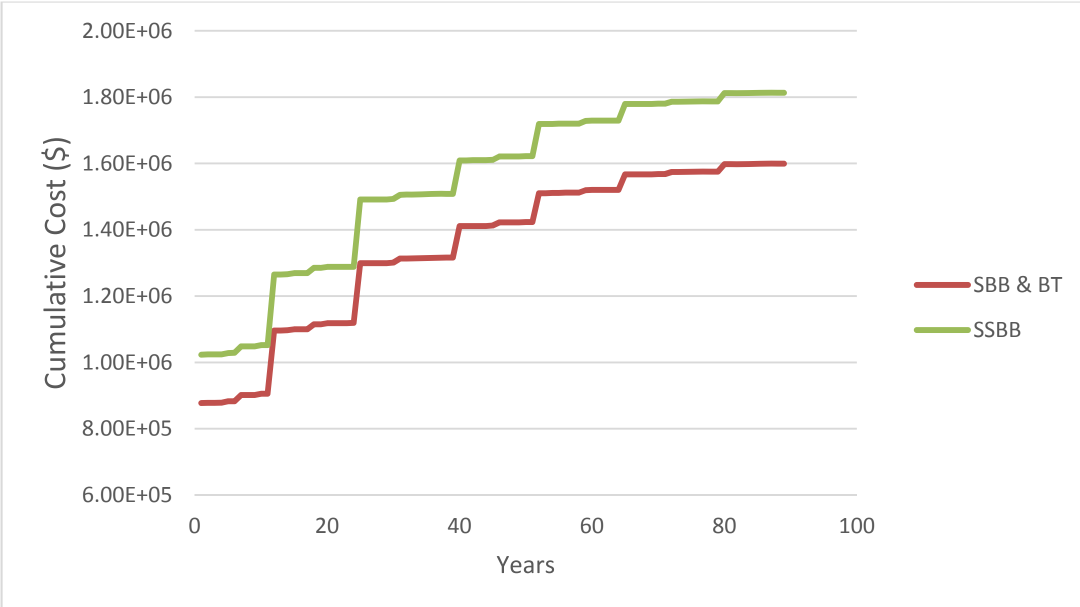

Figure 5.12. Cumulative Deterministic Cost, 110' Span.

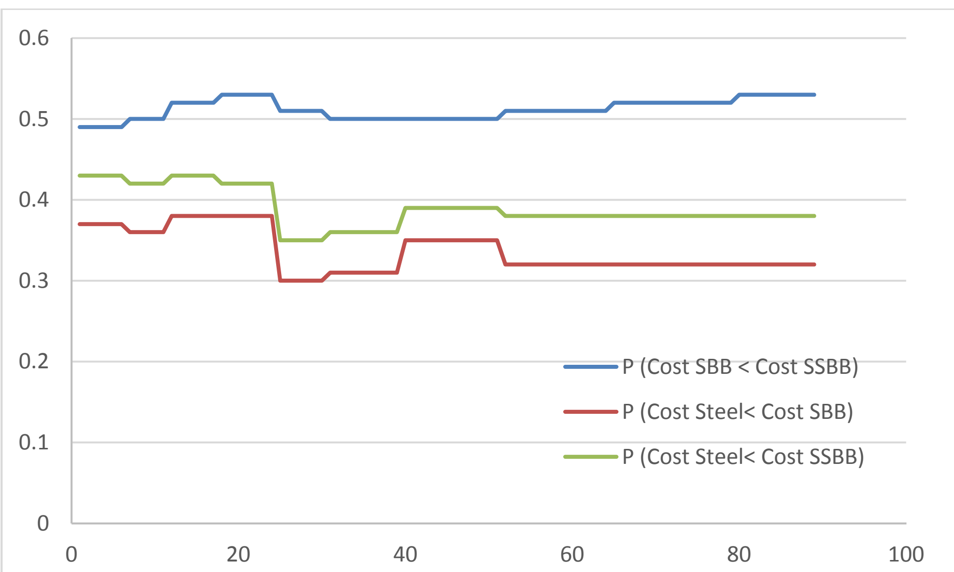

Figure 5.13. Alternative Cost Probabilities, 20' Span.

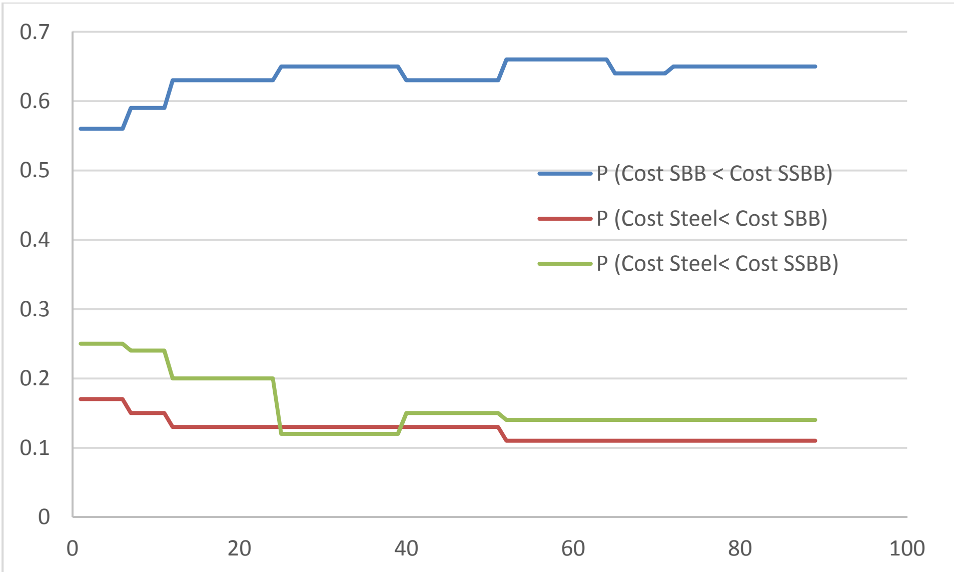

Figure 5.14. Alternative Cost Probabilities, 60' Span.

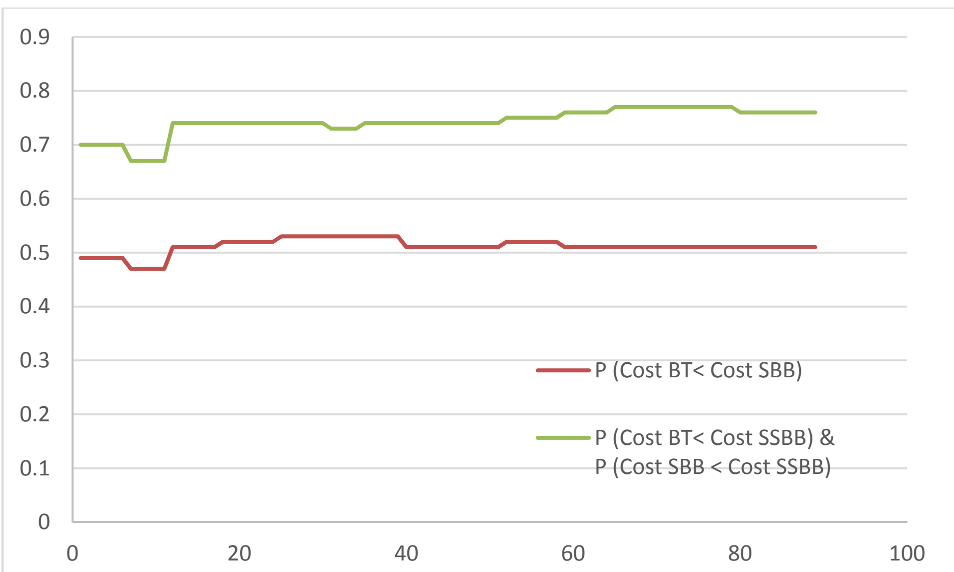

Figure 5.15. Alternative Cost Probabilities, 70' Span.

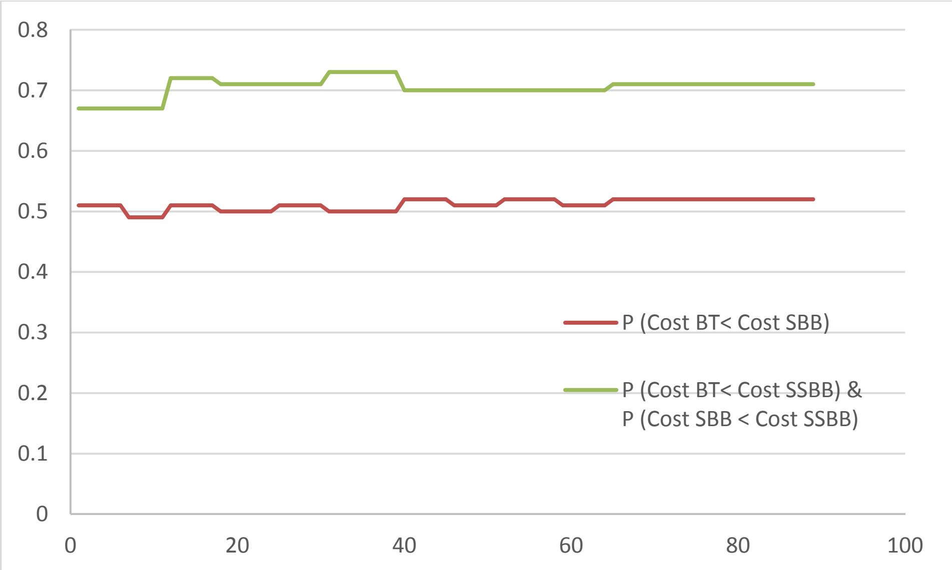

Figure 5.16. Alternative Cost Probabilities, 110’ Span.

CHAPTER 6: FOCUS GROUP COMMENTS ON DRAFT FINAL PLANS

A set of preliminary design templates for the four designs chosen for development (galvanized steel, spread box beam, side by side box beam, and bulb tee) were prepared and submitted to a second focus group meeting held in December of 2017. The session had 12 attendees (6 fabricators, 3 local agency representatives, and 3 bridge engineering consultants. In general, the concepts were well-received. Most of the comments concerned specific information to be presented on the plans. These comments and responses from the RAP are summarized in Appendix J.

CHAPTER 7: RECOMMENDATIONS

Recommended Bridge Plans and Details are given in Appendix L, while guidelines for use of the plans are given in Appendix K.

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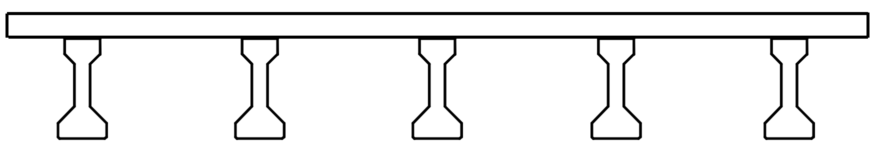
APPENDIX A: BRIDGE CONCEPTS PRESENTED TO PANELISTS

Panelists were given a short presentation introducing the project. In this presentation, it was noted that approximately 7000 Michigan roadway bridges are owned by local agencies, and of these, about 60% are on low volume roads (less than 1000 ADT) and about 60% are single spans less than 50 ft long. Over half of these bridges have been built prior to 1980, approximately one-fourth were built prior to 1960, and about 17% have major components rated less than satisfactory. Panelists were then informed that the purpose of project is to develop standardized plans suitable for local agency use, and that some desired characteristics of these bridges are: designed for low volume roads; suitable for MI geotechnical and hydraulic site conditions; low initial and long term costs; adequate capacity and serviceability; constructible using workforce, materials, and components available to local agencies; and acceptable aesthetics. If successfully developed, some anticipated benefits of using the standardized plans include: reduced design and construction uncertainties; simpler and faster to design and construct; improved quality control; and lower life cycle costs. Panelists were then informed that the purpose of the meeting was to: 1) introduce stakeholders to the project, and; 2) include stakeholders in bridge plan development. In particular, the meeting was to gain feedback in terms of suggestions and concerns for initial bridge concepts.

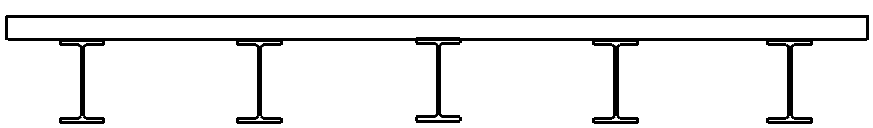
Panelists were then given a handout with the following bridge types. These figures were also shown to the panel on a projector to facilitate discussion.

(1)



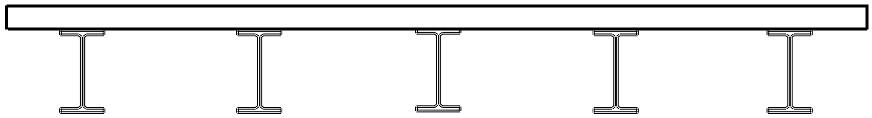
AASHTO/PCI Beam (spans 20 – 140 ft)

(2)



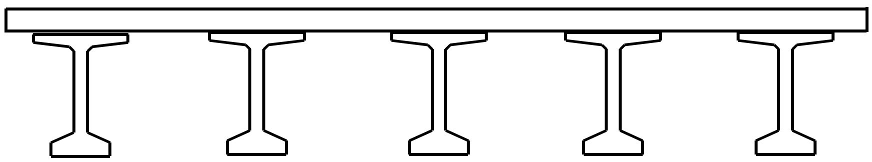
Painted Steel Beam (spans 20 – 200+ ft)

(3)



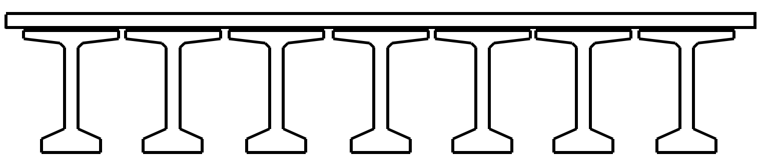
Galvanized Steel Beam (spans 20 – 60 ft)

(4)



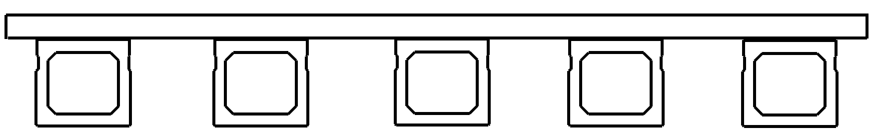
Spread Bulb Tee (spans 70 - 180 ft)

(5)



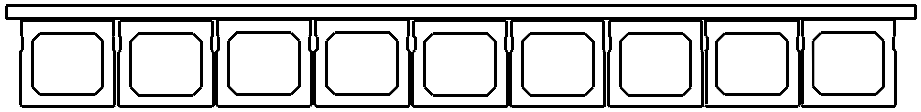
Side-by-Side Bulb Tee (spans 70 - 180 ft)

(6)



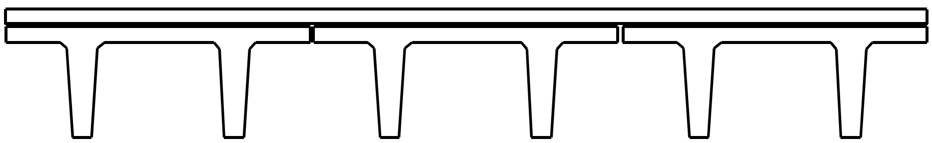
Spread Box Beam (spans 20 – 130 ft)

(7)



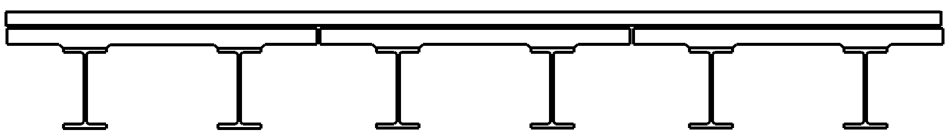
Side-by-Side Box Beam (spans 20 – 130 ft)

(8)



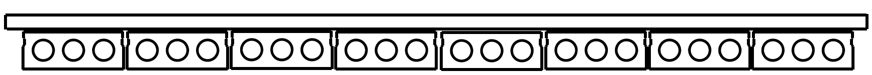
Precast Double Tee (spans 20 – 50 ft)

(9)



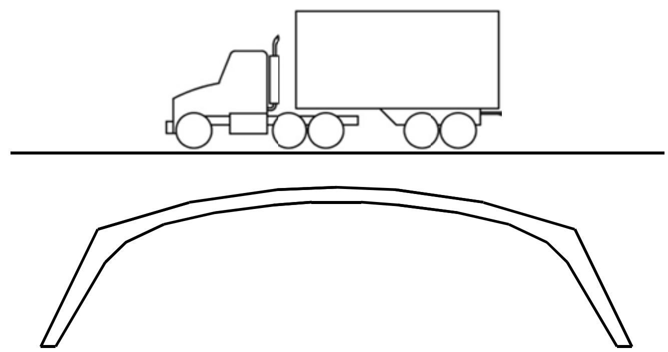
Prefabricated Steel/Concrete Double Tee (spans 20 – 50 ft)

(10)



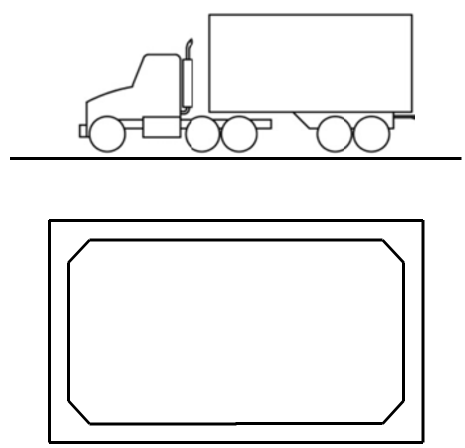
Slab (spans 30 – 50 ft)

(11)



3-Sided Prefabricated Culvert (spans 20 – 60 ft)

(12)



4-Sided Prefabricated Culvert (span ≤ 25 ft)

APPENDIX B: NEWS ARTICLE

As part of Task 6 (Prepare promotional and educational materials), a news article was developed for The Bridge periodical. The new article is as follows:

<u>Standard Bridge Plans for Michigan's Counties, Cities, and Villages</u>

One quarter of the bridges owned by counties, cities, and villages across the state are over 75 years old and will need replacement in the years ahead. Will funding be available? What can be done to decrease construction costs? Can the replacement bridges be designed to last longer?

Local agencies throughout Michigan are providing input on what these bridges of the future might look like. The Michigan Department of Transpiration's (MDOT's) local agency bridge program initiated a project with Wayne State University (WSU) to develop standard bridge plans suitable for local agency use. "The purpose of the standardized plans is to facilitate construction of low costs low maintenance readily constructible structures," says Keith Cooper of MDOT.

Approximately 7000 Michigan roadway bridges are owned by local agencies, and nearly 90% of these are managed by county highway agencies. In contrast to the typical highway bridges maintained by MDOT, most local agency bridges are smaller structures that carry lower traffic volumes; about 60% of local agency bridges are single span structures less than 50 ft long, and average fewer than 1000 vehicles per day.


A typical low-volume, short - span county bridge.

As with the rest of the civil infrastructure, these structures are aging. Over half of these bridges were built prior to 1980, while one-fourth were built prior to 1960. This advancing age leads to increasing maintenance concerns. 17% have superstructures and substructures rated less than satisfactory, and in the near future, many of these bridges will require replacement.

Some bridge replacement guidance is available to local agencies and engineering consultants by way of the standard plans that MDOT uses for its

structures. However, although optimal for MDOT, these existing bridge plans are not always best suited for local agency use. As many MDOT bridges are built for higher traffic volume roads, they tend to be wider and longer than the typical local agency bridge, often using girders such as PCI AASHTO-type beams meant for larger spans. MDOT bridges may also be challenging to efficiently construct by workforces readily available to many local agencies. Local agencies and their design consultants recognize this, and often base designs on previously constructed structures which were regarded as successful. Although reasonable, this approach results in a wide variety of designs and details, some of which produce sub-optimal bridges.

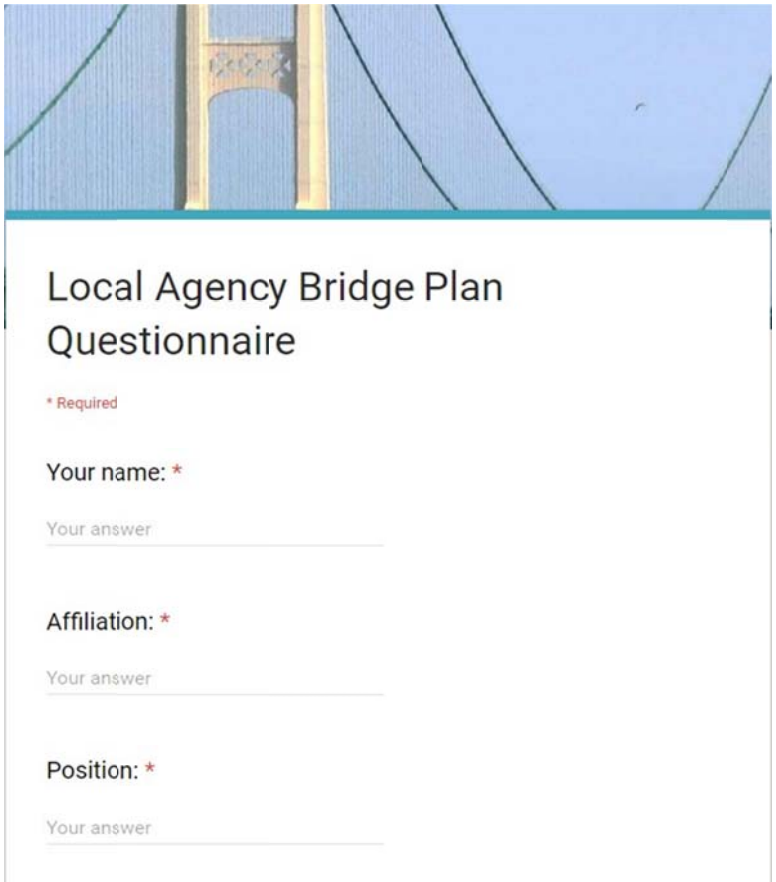
Given this concern, MDOT has recently initiated a project with WSU and Benesch Company to develop standard bridge plans ideal for local agency use. The purpose of the plans is to facilitate construction of suitable new structures. The designs are to specifically address common local agency road, span, and site conditions.

In July, MDOT held an initial panel discussion with local agencies, designers, and contractors to gather input on various bridge design concepts presented by WSU. Additionally, MDOT recently released a survey to local agencies for additional feedback on these structures.


Reconstructing a bridge deck.

Anticipated benefits from implementing the standardized bridge plans include reduced design and construction uncertainties; bridges that are simpler and faster to design and construct; improved quality control; and lower life cycle costs. The project is scheduled to finish, and final bridge plans made available to local agencies, by January, 2018. Comments and questions about this project can be submitted at any time to Chris Eamon (eamon@eng.wayne.edu).

APPENDIX C: BRIDGE CONCEPT SURVEY



Local Agency Bridge Plan Questionnaire

* Required

Your name: *

Your answer

Affiliation: *

Your answer

Position: *

Your answer

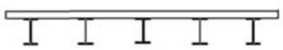
1. Rank the following structures in your order of preference for standardized plan development ("1" being highest preference):

AASHTO/PCI Beam

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○

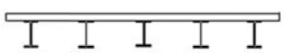
Painted Steel Beam

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○



Galvanized Steel Beam

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○

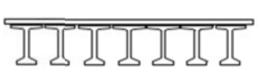


Spread Bulb Tee

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○

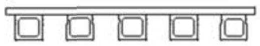
Side-by-Side Bulb Tee

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○



Spread Box Beam

1	2	3	4	5	6	7	8	9	10



Side-by-Side Box Beam

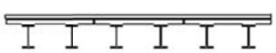
1	2	3	4	5	6	7	8	9	10

Precast Double Tee

1	2	3	4	5	6	7	8	9	10

Prefabricated Steel/Concrete Double Tee

1	2	3	4	5	6	7	8	9	10

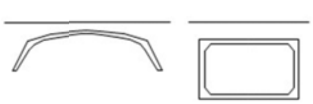


Prestressed Slab

1	2	3	4	5	6	7	8	9	10

3 and 4 Sided Prefabricated Culverts

1	2	3	4	5	6	7	8	9	10



Timber

1	2	3	4	5	6	7	8	9	10
○	○	○	○	○	○	○	○	○	○

2. What design software does your agency and its consultants use? *

- ☐ a. MicroStation
- ☐ b. AutoCAD
- ☐ Other: ____________

3. Are there any issues that you would like to see addressed?

Your answer ____________

We Appreciate Your Time and Patience

APPENDIX D: SURVEY SUBMITTED TO MICHIGAN BRIDGE CONFERENCE ATTENDEES

Standard Bridge Plans for Michigan's Counties, Cities, and Villages:
A Survey

The Michigan Department of Transpiration's Local Agency Bridge Program initiated a project to develop standard bridge plans suitable for local agency use. The purpose of the plans is to facilitate construction of low cost, low maintenance, readily constructible structures.

The majority of roadway bridges owned by local agencies (60%) are on low volume roads with less than 1000 average daily traffic and are single spans less than 50 feet long. Over half of local agency bridges were built prior to 1980, while one-fourth were built prior to 1960. Moreover, 17% have major components rated less than satisfactory, and in the near future, many may require replacement. Given this concern, we greatly value your suggestions for this project.

If you would like additional information about this effort please contact:

Christopher Eamon
Civil and Environmental Engineering
5050 Anthony Wayne Dr.
Wayne State University
Detroit, MI 48202

Phone (313) 577-3766
Fax (313) 577-3881
eamon@eng.wayne.edu

1. Do you represent:

(a) MDOT
(b) a local agency (specify which agency, if desired):
(c) an MDOT consultant
(d) a local agency consultant
(e) other (please specify):

2. What drafting software does your organization or its consultants use:

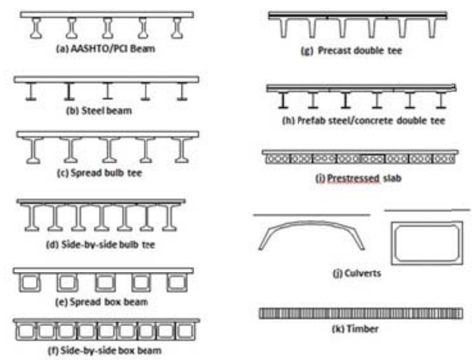
(a) AutoCAD
(b) MicroStation
(c) Other (please specify):

3. What types of bridge abutments do you typically use?

(a) Cantilever
(b) Curtain wall
(c) Integral (stub abutment on a single row of piles)
(d) GRS
(e) Other (please specify):

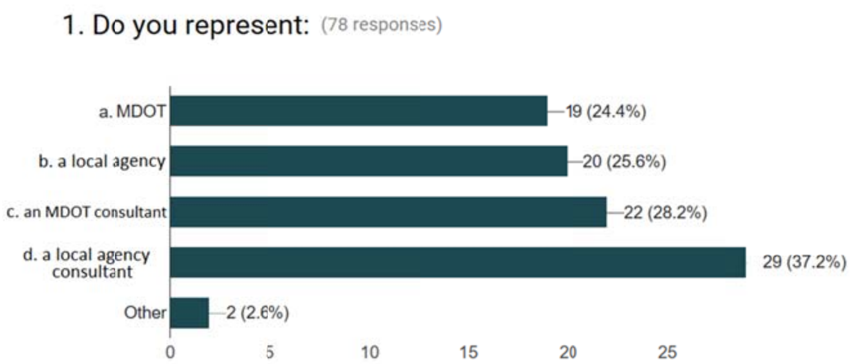
4. Rank the following 12 structures given below in your order of preference for standardized plan development ("1" being highest preference), and indicate your level of experience with each type:

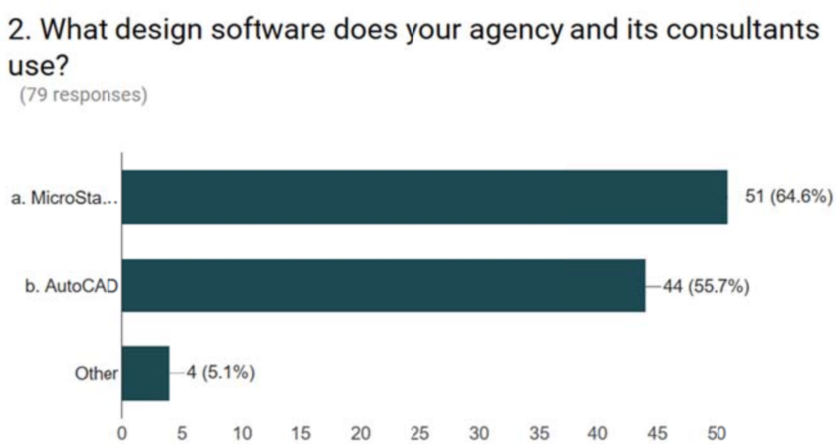
Your Ranking		Your Level of Experience (please indicate: None = N, Little = L, Moderate = M, Substantial = S)
____	(a) AASHTO/PCI Beam	______
____	(b) Painted Steel Beam	______
____	(b) Galvanized Steel Beam	______
____	(c) Spread Bulb Tee	______
____	(d) Side-by-Side Bulb Tee	______
____	(e) Spread Box Beam	______
____	(f) Side-by-Side Box Beam	______
____	(g) Precast Double Tee	______
____	(h) Prefabricated Steel/Concrete Double Tee	______
____	(i) Prestressed Slab	______
____	(j) 3 and 4 Sided Prefabricated Culverts	______
____	(k) Timber	______

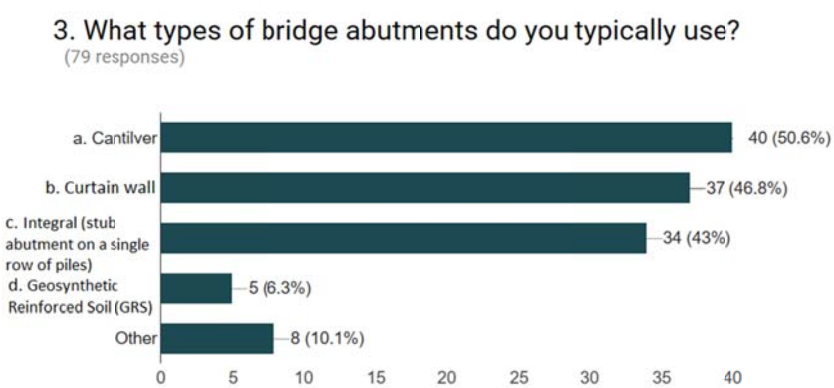


5. The objective of this project is to identify optimal types of bridge structures for local agency use, and produce standardized plans of these structures. What would you recommend to accomplish this?

APPENDIX E: MICHIGAN BRIDGE CONFERENCE SURVEY RESULTS FOR ALL RESPONDENTS





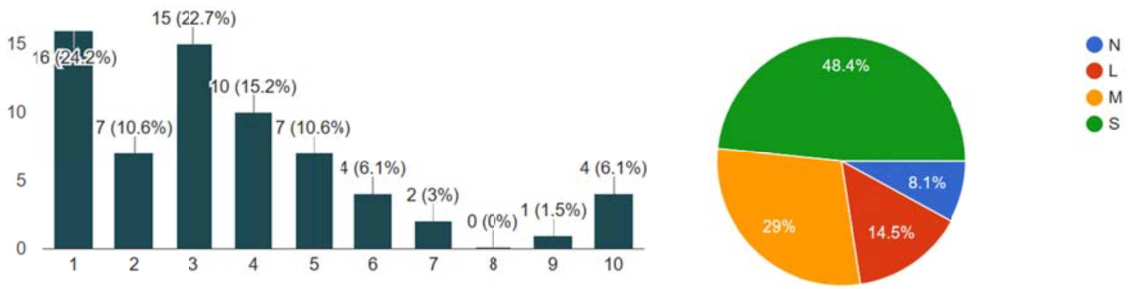


Rank the following 12 structures given below in your order of preference for standardized plan development ("1" being highest preference):

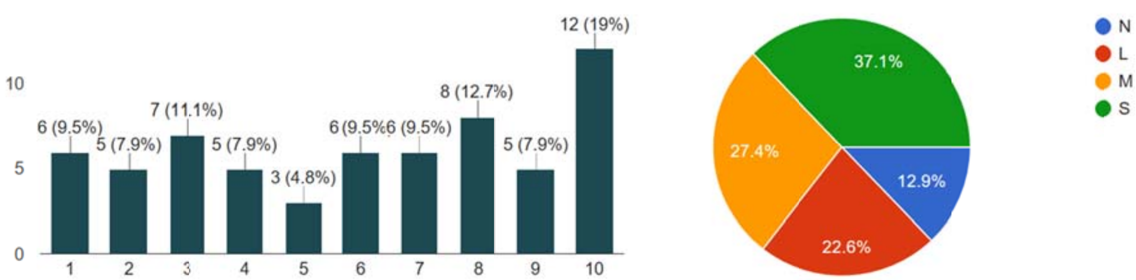
Level of Experience: None = N, Little = L, Moderate = M, Substantial = S

Note: “10” = includes ranks 10, 11, and 12.

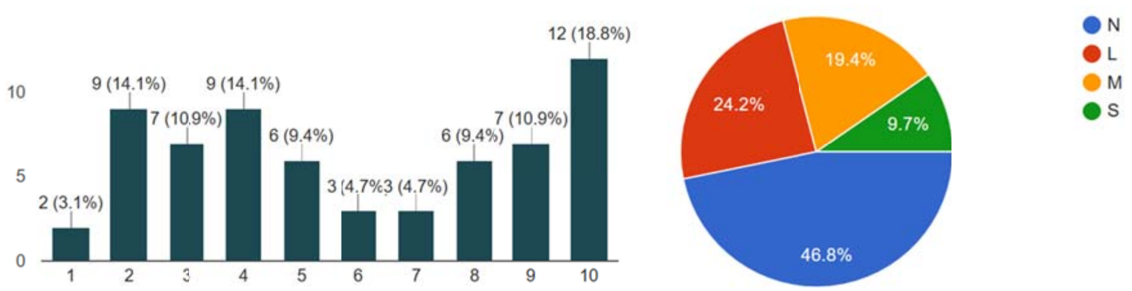
AASHTO/PCI Beam (66 responses)



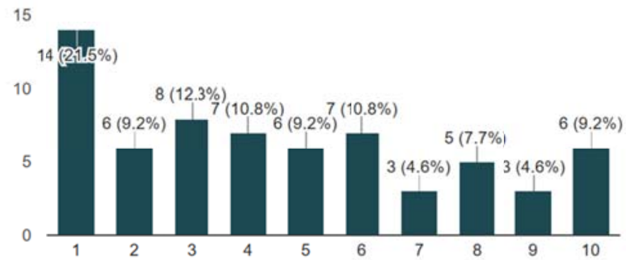
Painted Steel Beam (63 responses)

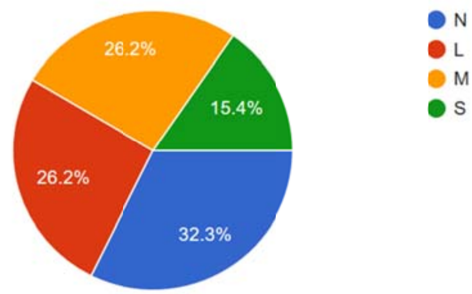


Galvanized Steel Beam (64 responses)

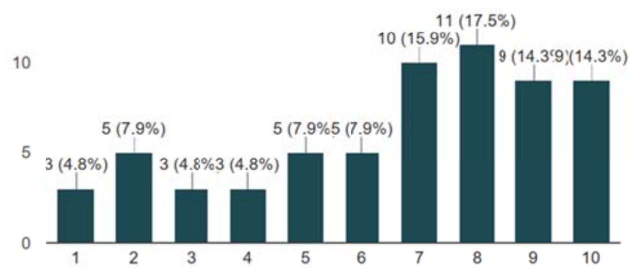


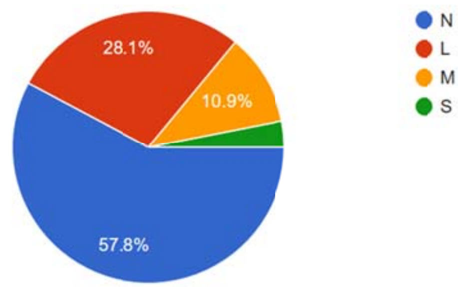
Spread Bulb Tee (65 responses)



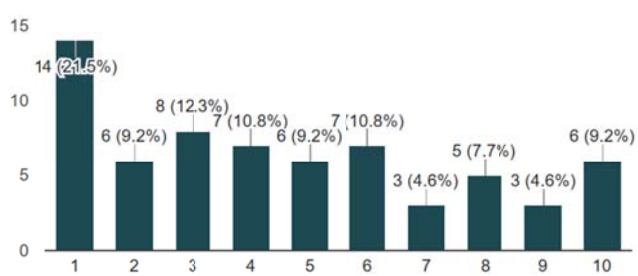


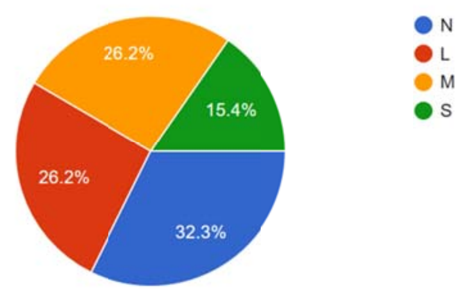
Side-by-Side Bulb Tee (63 responses)



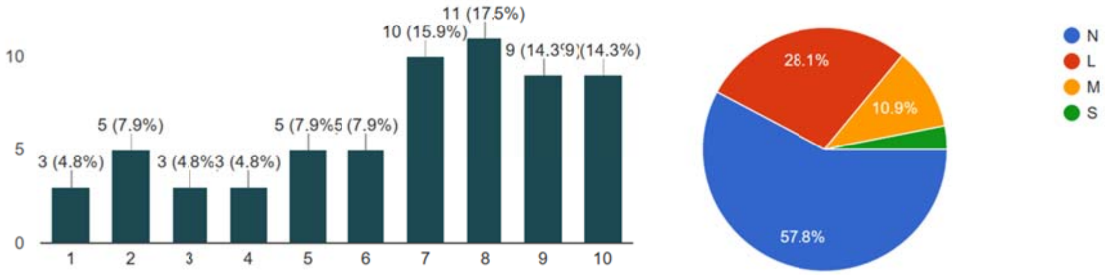


Spread Bulb Tee (65 responses)

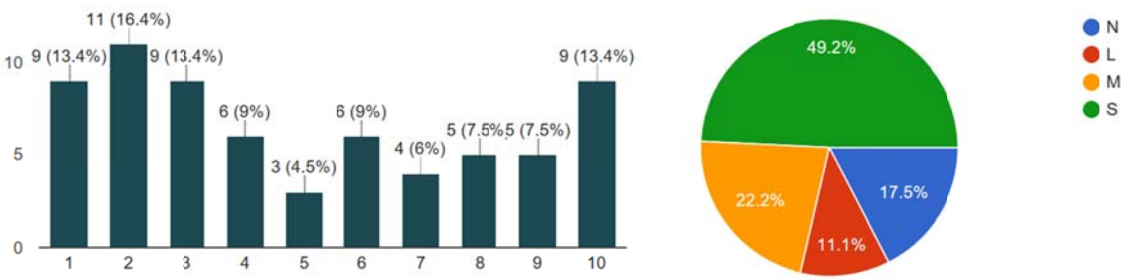




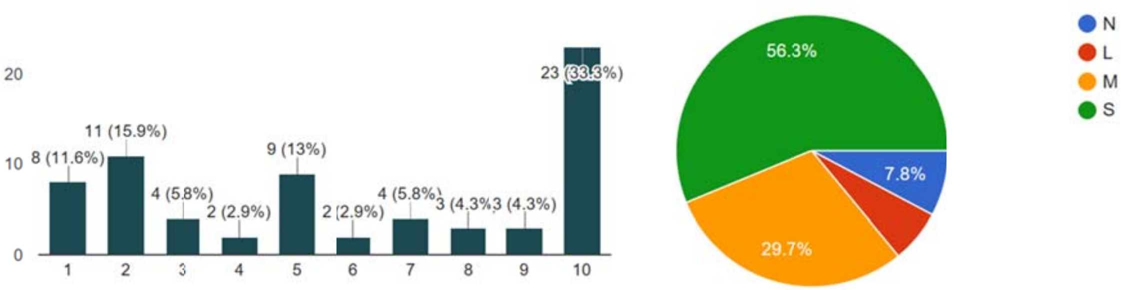
Side-by-Side Bulb Tee (63 responses)



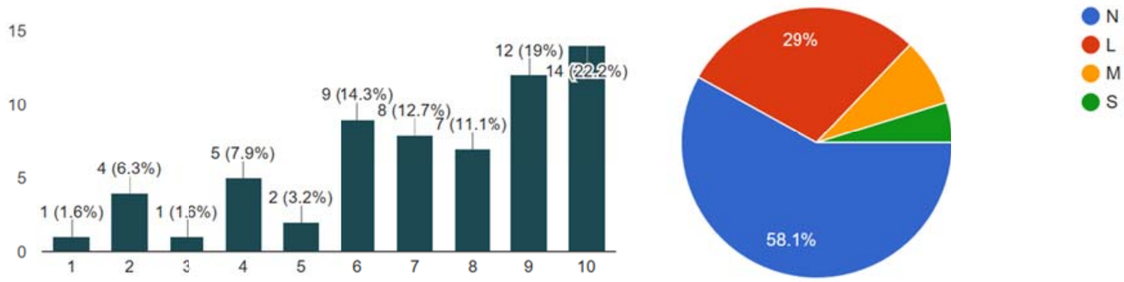
Spread Box Beam (67 responses)



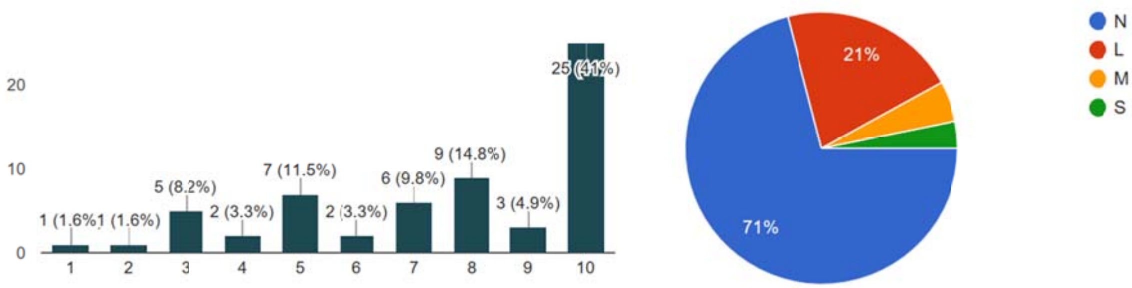
Side-by-Side Box Beam (69 responses)



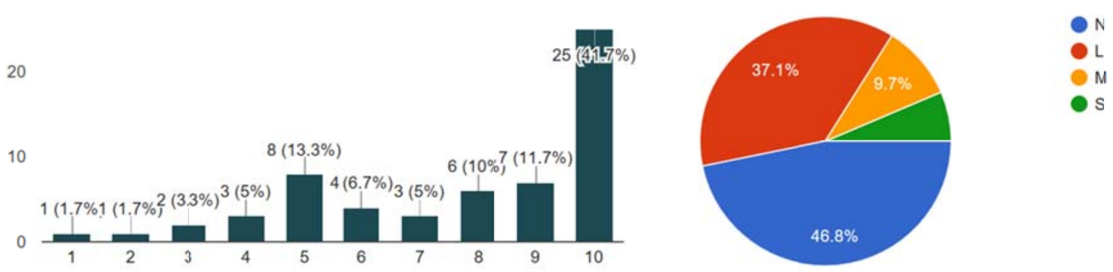
Precast Double Tee (63 responses)



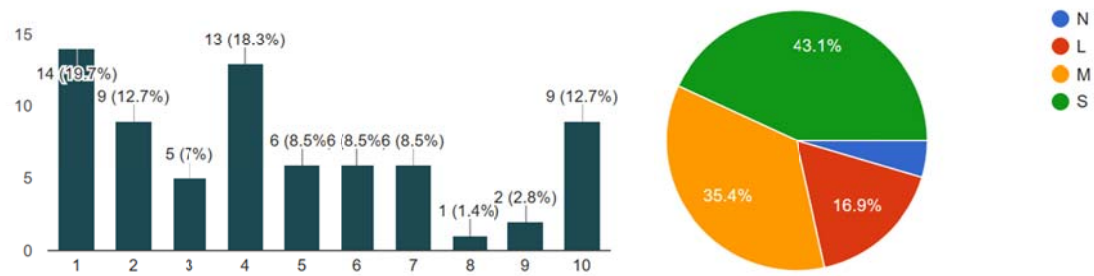
Prefabricated Steel/Concrete Double Tee (61 responses)



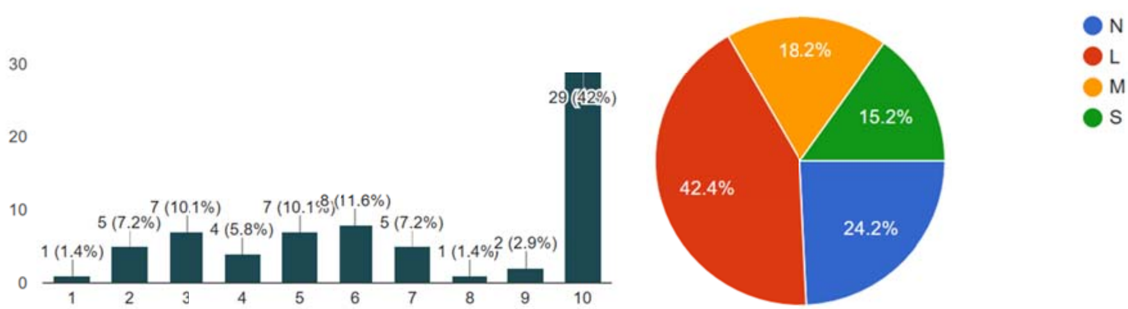
Prestressed Slab (60 responses)



3 and 4 Sided Prefabricated Culverts (71 responses)



Timber (69 responses)

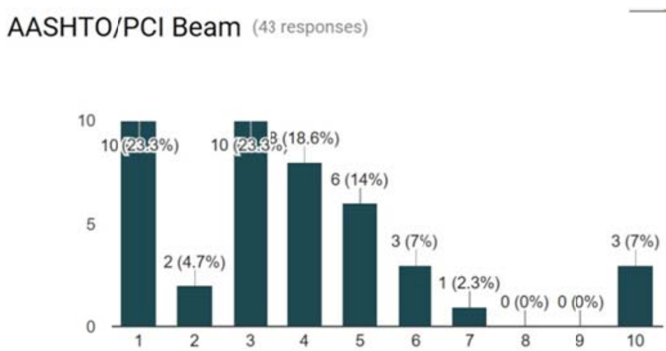


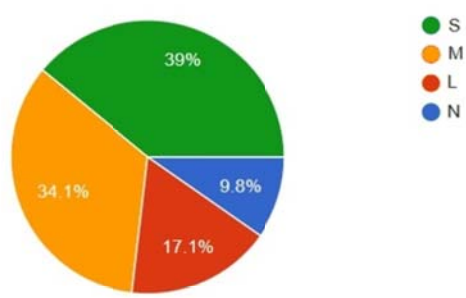
APPENDIX F: MICHIGAN BRIDGE CONFERENCE SURVEY RESULTS FOR LOCAL AGENCIES AND THEIR CONSULTANTS

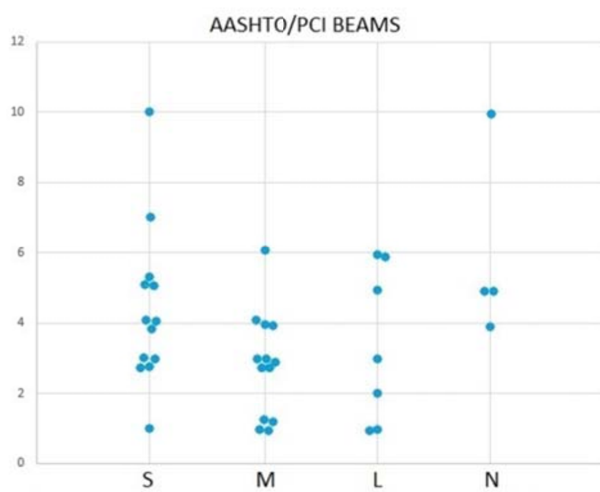
Rank the following 12 structures given below in your order of preference for standardized plan development ("1" being highest preference):

[Note: Do to limitations of the on-line survey, a "10" indicates a rank of either 10, 11, or 12].

<u>Experience Levels:</u>
S – Significant
M – Moderate
L – Little
N - None

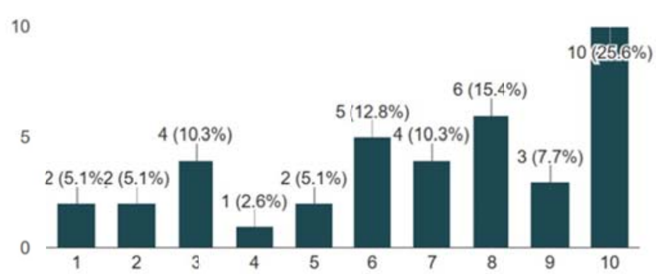


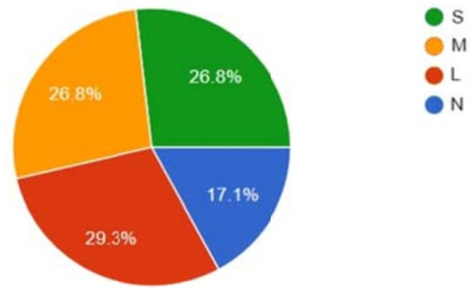




	S	M	L	N
Mean	4.6	2.8	3.4	6
Mode	3	3	1, 6	5

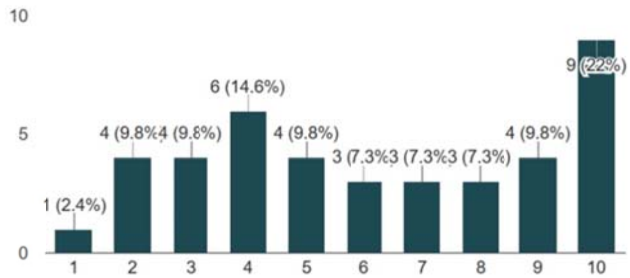
Painted Steel Beam (39 responses)

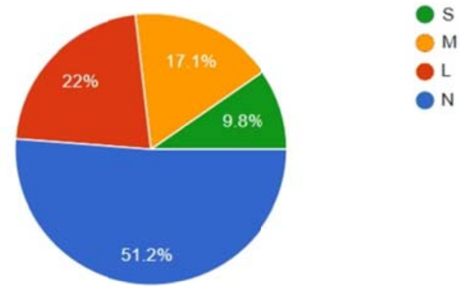


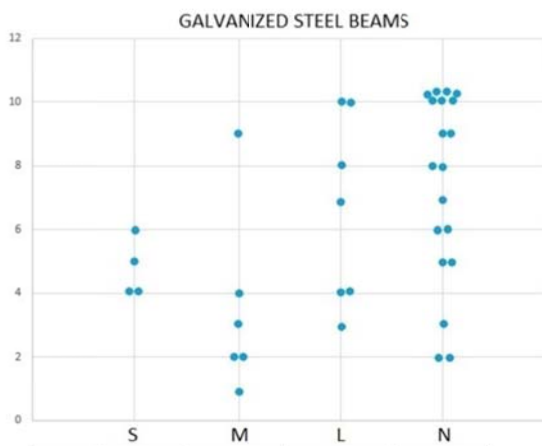


	S	M	L	N
Mean	6.9	5.3	6.4	8.7
Mode	8,9	2,7	10	10

Galvanized Steel Beam (41 responses)

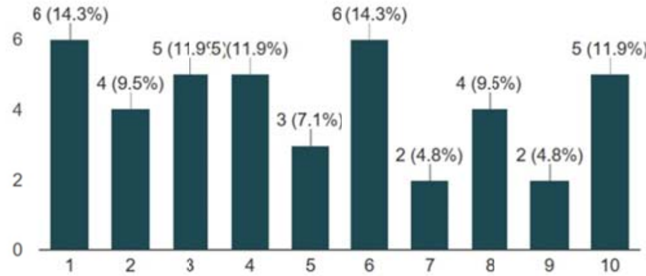


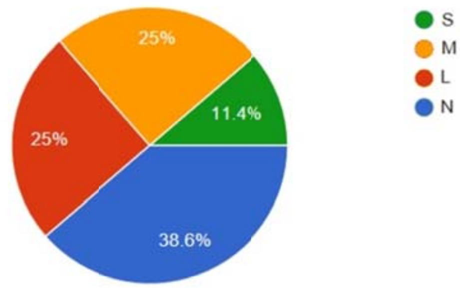


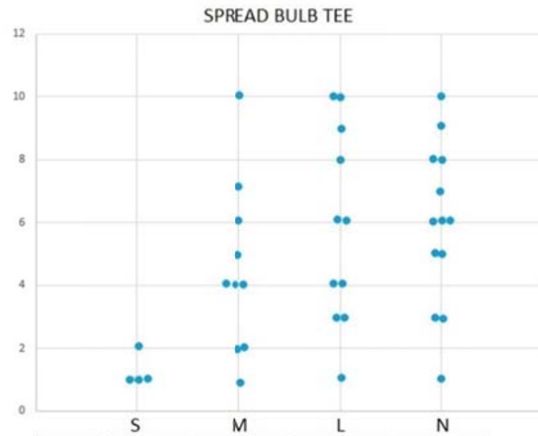


	S	M	L	N
Mean	4.75	3.5	6.6	7.4
Mode	4	2	4,10	10

Spread Bulb Tee (42 responses)

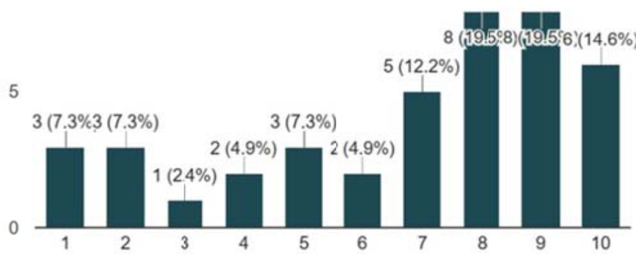


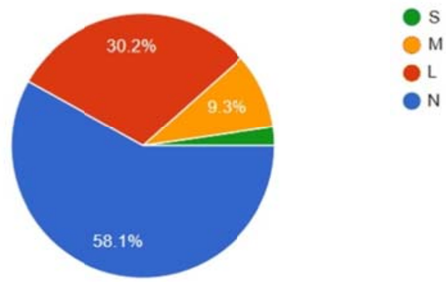


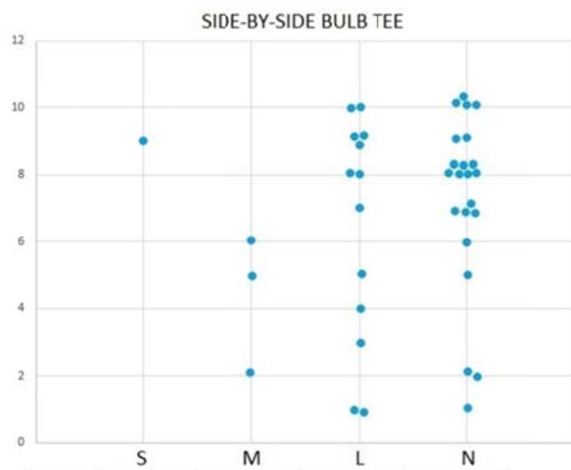


	S	M	L	N
Mean	1.3	4.5	6.1	5.9
Mode	1	4	3,4,6,10	6

Side-by-Side Bulb Tee (41 responses)

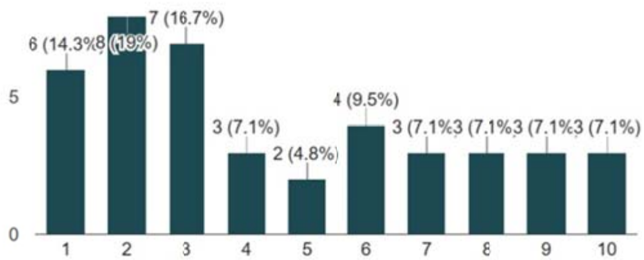


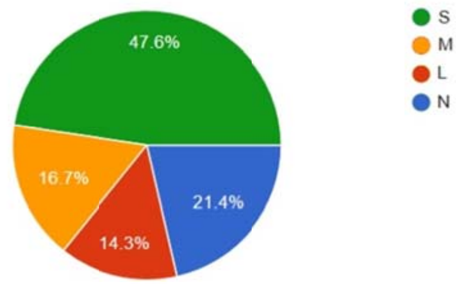


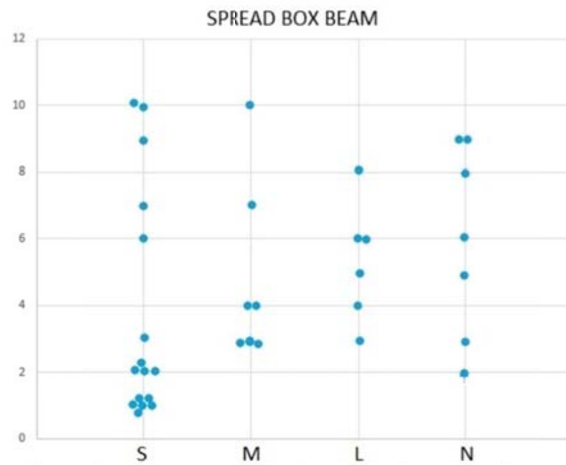


	S	M	L	N
Mean	9	4.3	6.5	7.2
Mode	-	-	9	8

Spread Box Beam (42 responses)

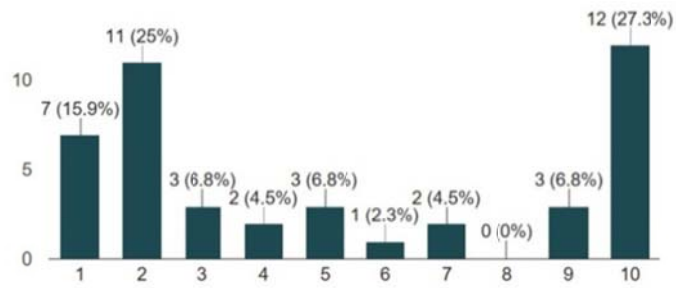


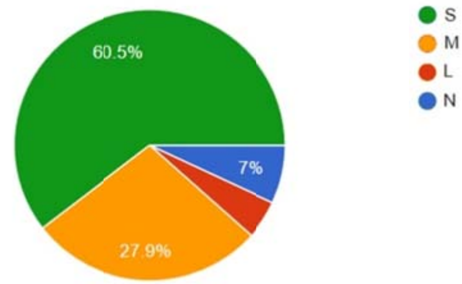


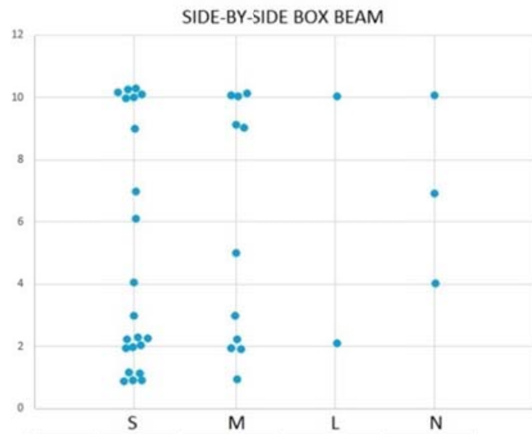


	S	M	L	N
Mean	3.9	4.9	5.3	6.0
Mode	1	3	6	9

Side-by-Side Box Beam (44 responses)

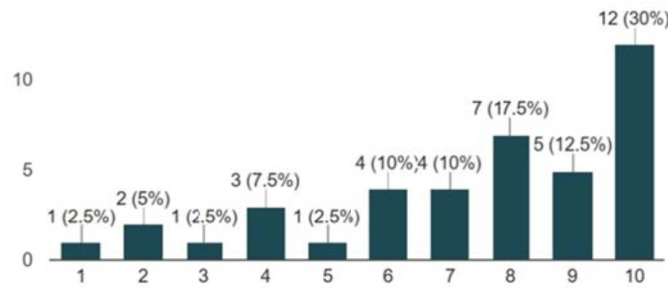


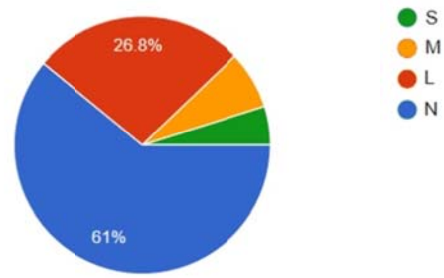


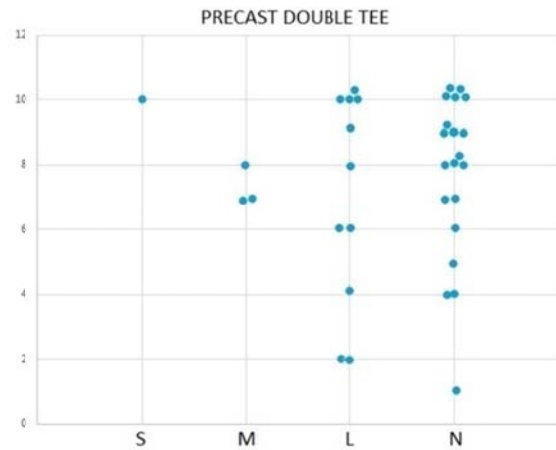


	S	M	L	N
Mean	4.8	5.7	6	7
Mode	2,10	2,10	-	-

Precast Double Tee (40 responses)

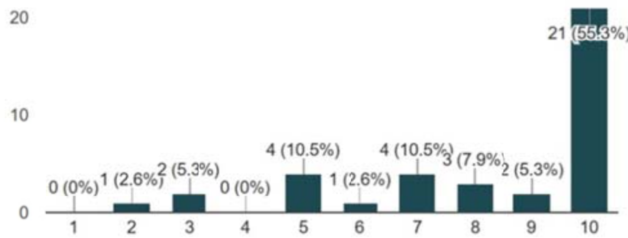


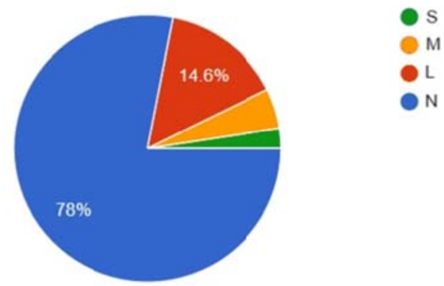




	S	M	L	N
Mean	10	7.3	7	7.6
Mode	-	7	10	10

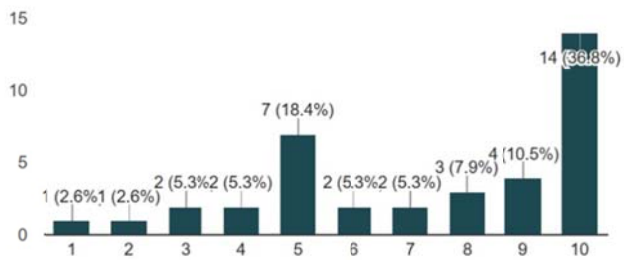
Prefabricated Steel/Concrete Double Tee (38 responses)

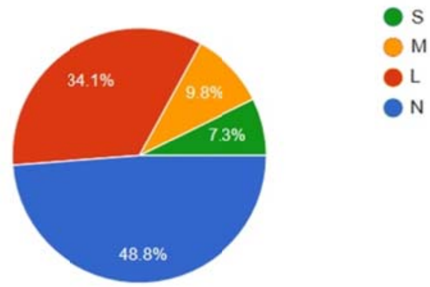


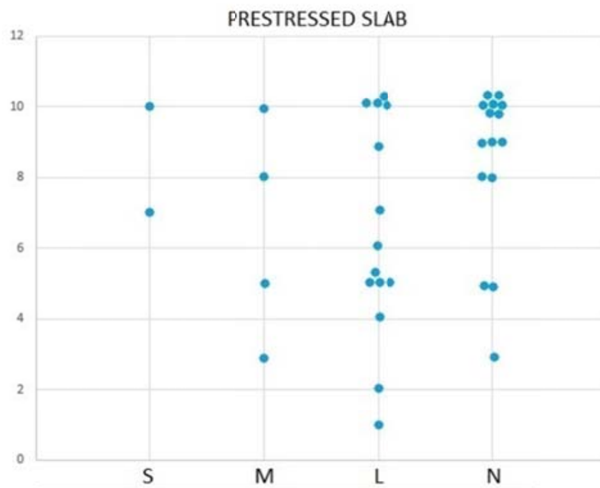


Mean	-	9	8.3	8.3
Mode	-	-	10	10

Prestressed Slab (38 responses)

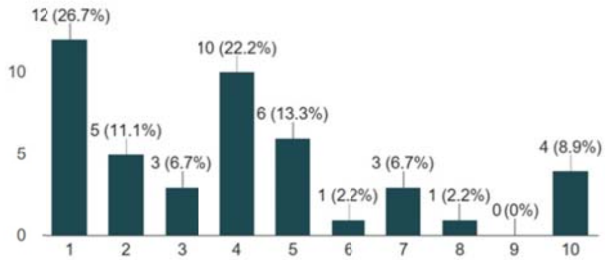


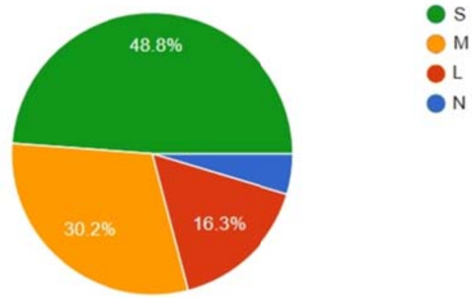


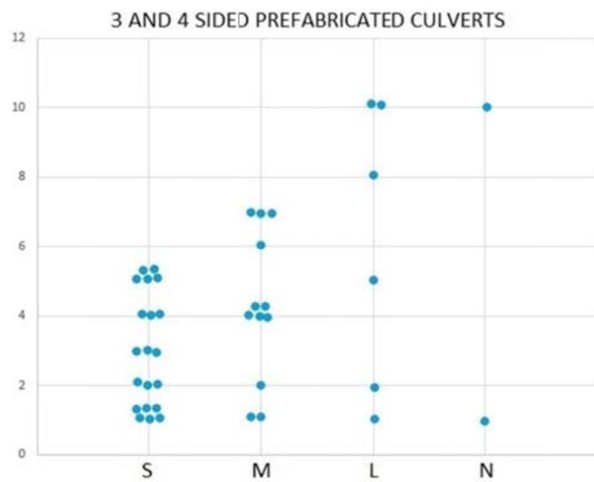


Mean	8.5	6.5	6.4	8.4
Mode	-	-	5,10	10

3 and 4 Sided Prefabricated Culverts (45 responses)

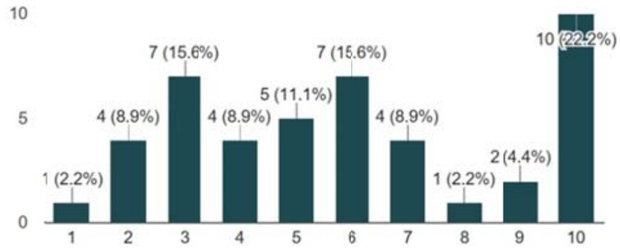


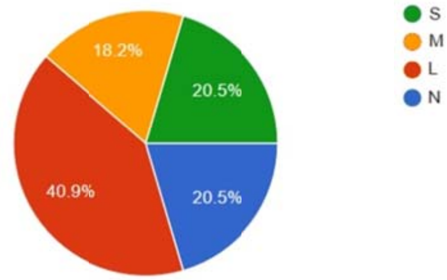


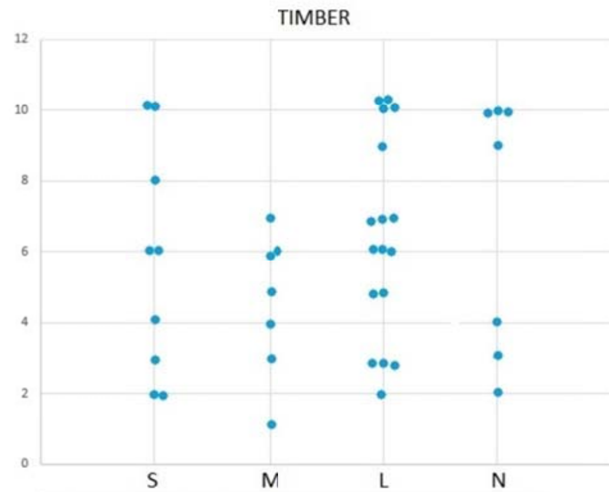


	S	M	L	N
Mean	2.9	4.3	6	5.5
Mode	1	4	10	-

Timber (45 responses)







Mean	5.7	4.6	6.4	6.9
Mode	2,6,10	6	10	10

APPENDIX G. SURVEY COMMENTS

Comments

- Work closely with County Road Association and the Local Bridge Advisory Board. Utilize local agency engineers, contractors and consultants for constructability issues."
- Reinforcing tables done by fabricators, reviewed by engineer-Engineer provides min sizing and spacing Tabular design of as many elements as possible, standardize approach slab design
- Analysis of the last 10 yrs and pick the top three types that are actually used
- Prefabricated steel superstructure
- Standardize beam designs for various length/spacing/skew
- Improve details to prevent water intrusion into strands for ends of box beams & PCI beams
- Consider galvanizing the steel whatever beams or rebar. I presented about rebar at the conference. I can talk with you about either method if you'd like. Shannon Pole 289.440-1886
- Consider pre-fabricated concrete bridges
- Keep it simple
- GRS wall concrete structures
- Standard Bms - Box PCI - PS - SLABS – TIMBER; Standard ABUTS - CURTAIN WALL; STND DECK - STND typical STND railings
- Microstation using MDOT workspace and pay items. Other states and federal park service has good details & design tables to look at as examples/template for local bridges
- 1) Provide alternatives one size doesn't fit all. 2) consider availability of materials (beams) and competition. 3) gather input from some stockholders (MDOT bridge unit, local agency engineers, etc.)
- Determine most widely utilized span lengths and standardize plans for the most efficiency structure type for these spans.
- Make sure everything is consistent with what MDOT produces and consultants produce. seems to be different requirements of each.
- Consider long term performance & end details to increase performance, ENS are usually wet & collect debris. Type F (side-by-side box beam) always end up with long cracks in deck. Culverts are cheap but hard to repair.
- Address skew & geotechnical considerations
- Provide tables for each section type for spans + width and associated reactions
- Side by side boxes in several lengths - 20,30,40,50 with skews 10,20,30
- Have various proven options so a local agency can utilize a variety of structure types throughout their system. Develop 4 sided prefab concrete box for larger spans (~30'+)
- Use joint details from the ODOT standard drawings. Support the steel joint on end cross frames per ODOT details. 1) strip seal 2) compression seal. "we can put a man on the moon, but we cannot design a bridge deck joint that does not leak" Martin P. Burke ODOT, 1985 Side by side box beam and prestressed slab bridges are really bad idea with HMA or lighter reinforced concrete deck. also, they have been proved to be a poor

superstructure type with 15-20 yrs design life. should be banned from use. Use structural steel. painted or not. composite. it lasts forever, and can be easily repaired.

- Start w list: survey, hydro, soils report, ADT-get your ducks in a row first. a lot of design requests come in for rapid repair/replace + no needed background so delays designs
- Curtain wall and box beams are "cookie cutter". There should be standard plans for each. The only design should be site specific like Geotech, grades, curtain wall height.

Other Comments:

- Any design with low cost/ high life.
- Is there an option to post tension culverts together to increase span?
- Let the bridge engineers determine what is best! who's going to seal these plans and be liable for them?
- Provide most cost effective option
- Longest lasting, lowest maintenance, cost efficient + most flexible design would be preformed option
- I don't think this will save much cost, especially in construction which represents by for the most significant cost of a project. Design costs are typically only 2% - 3% of construction cost, and you might only save about 15%-20% of design cost. if you really want to reduce costs, eliminate federal regulations and permits such as wetland mitigation, FWS, SHOP, etc. These create unnecessary costs and scheduling headaches/delays. also need to eliminate prevailing wage.
- There is still confusion on which loading to use on low cover box culverts. In some situations, HL-93 per standard specifications is NOT adequate to provide a >1.0 rating factor on all MDOT rating trucks. Manufactures do not like building for HL-93 Modified as the standard forms get congested with steel reinforcing. Forms need to be bigger but manufactures not willing to invest in new equipment.

APPENDIX H. SUMMARY OF BRIDGE DESIGNS

Design parameters common to all structures

The plans present design information for simple span structures of three widths and variable lengths in 10' increments designed to the 7th Edition of the AASHTO LRFD Specifications (2014), but using the HL-93-mod live load. Decks are designed using the strip method, while girder shear design is based on the General Procedure. The designs are valid for skews from 0-30 degrees (i.e. angle of crossing from 60-90 degrees), and satisfy Strength I, Service I and III (for prestressed concrete), Service II (for steel), and Fatigue I limit states, and meet a deflection limit of L/800. Note that these plans are for superstructures only.

Clear widths are based on AASHTO 2011 (A Policy on Geometric Design of Highways and Streets), as a function of average daily traffic (ADT). Width requirements are based on design speed (assumed to be 55 MPH) and ADT. Although very low ADT roads (<400) can use a two-lane bridge width of 26', most local agencies specify a minimum deck width of 30' to accommodate agricultural equipment; this width assumes two 11' lanes with 4' shoulders, and can be used for ADT up to 1500. From ADT of 1500-2000, the minimum bridge width is 34' (two 11' lanes and 6' shoulders); for ADT over 2000, a width of 40' is specified (12' lanes and 8' shoulders). All girder bridges have approximately 2.5' overhang, except the 30' clear steel beam bridge (all spans) and the 40' clear, 110' span bulb tee, which have approximately 3.5' overhang, measured from the center of the girder to edge of the deck.

Designs are provided for span lengths in increments of 10', while designs are presented for clear bridge deck widths of 30', 34', and 40'. When specifying beam sizes and girder spacing, the plans were prepared to provide a balance between economy and maximizing vertical clearance. Steel diaphragms are used throughout, although no diaphragms are specified for spread box beam bridges, which are not required for the geometries considered.

All bridges have a reinforced concrete composite deck with f'c = 4 ksi, that is 7.5" thick, with an additional 1.5" wearing surface (9" total depth), except for side by side box beam bridges, which have a 4.5" deck thickness with an additional 1.5" wearing surface (6" total depth). For girder design, a beam haunch of 4" is assumed for dead load, but no haunch was included in calculation of composite beam section properties. Decks are reinforced with 60 ksi, #5 bars throughout, where top bars are spaced 12" and bottom bars are spaced 8" on center. All bars are assumed to be epoxy coated.

For prestressed concrete girders, f'c = 7 ksi at release and 8 ksi in service. Prestress tendons are taken as 0.6" nominal diameter low relaxation strands, with ultimate strength 270 ksi. Stirrups are #4, and were designed as 40 ksi steel in 17" and 21" box beam sections in order to meet lap length requirements. Otherwise, stirrups are 60 ksi.

Spread Box Beam Structures

Spread Box beam bridge geometry is given in Table H.1. Additional bridge information is given in Tables H.2 and H.3. For Table H3,

Table H.1. Spread Box Beam Bridge Geometry.

Span (ft)	# of beams x spacing (ft)	Overhang (ft)	Total width (ft)	Clear width (ft)
20 - 50	4 x 9.17	2.5	32.5	30
60 - 110	5 x 6.87	2.5	32.5	30
20 - 110	5 x 7.87	2.5	36.5	34
20 - 50	5 x 9.37	2.5	42.5	40
60 - 110	6 x 7.5	2.5	42.5	40

Table H.2. Spread Box Beam Strand Information.

Beam			No. of strands*					
Span (ft)	Depth (in)	Width (in)	1st layer	2nd layer	Total strands	No. of debonded strands	(No. debonded strands) Debonded length (ft)**	Initial prestress force (kips)
20	21	36	7	-	7	N/A	N/A	307.6
30	21	36	11	-	11	N/A	N/A	483.4
40	21	36	13	3	16	2	(2)4	703.1
50	21	48	19	3	22	2	(2)4	966.7
60	21	48	19	15	34	8	(2)2,(4)8,(2)12	1494
70	27	48	19	15	34	6	(2)2,(2)4,(2)6	1494
80	33	48	19	15	34	6	(2)2,(4)4	1494
90	39	48	19	17	36	6	(2)2,(4)4	1581.9
100	39	48	19	21	46	10	(2)12, (2)14, (2)30 ,[2]12, [2]20	2021.4
110	48	48	19	21	44	10	(2)6, (2)10, (2)12, [2]2, [2]4	1933.5

*First strand layer is 2" from bottom of beam; second layer is 4" from bottom. The 110 ft span has a third layer of strands containing 4 strands, 6" from the bottom of beam.
**( ) refers to debonding in the first layer; [ ] refers to debonding in the second layer. All debonded strands are in pairs, symmetrically placed about the beam center.

Table H.3. Spread Box Beam Bridge Stirrup Requirements.

	Stirrup spacing (in), Endpoint of region (ft)*		
Span (ft)	A	B	C
20	3,10	-	-
30	3,15	-	-
40	3,3	6,20	-
50	3,4	6,25	-
60	3,9	6,30	-
70	3,6	6,35	-
80	3,6	6,40	-
90	3,6	6,36	12,45
100	3,6	6,44	12,50
110	3,6	6,43	12,55

*Region endpoint measured from leftmost side of beam. Stirrups symmetric about centerline.

Side by Side Box Beam Structures

Side by side box beam bridge geometry is given in Table H.4. Additional bridge information is given in Tables H.5 and H.6.

Table H.4. Side by Side Box Beam Bridge Geometry.

span (ft)	Clear width 30'		Clear width 34'		Clear width 40'	
	# of beams	Total width (ft)	# of beams	Total width (ft)	# of beams	Total width (ft)
20 – 50	9	32.5	10	36.54	12	42.5
60 - 110	7	33.75	8	37.88	9	42.5

Table H.5. Side By Side Box Beam Bridge Strand Information.

	Beam		No. of strands*					
Span (ft)	Depth (in)	Width (in)	1st layer	2nd layer	Total strands	No. of debonded strands	(No. debonded strands) Debonded length (ft)**	Initial prestress force (kips)
20	17	36	7	-	7	-	-	307.6
30	17	36	7	-	7	-	-	307.6
40	17	36	9	-	9	-	-	395.5
50	17	36	13	3	16	2	2(2)	703.08
60	21	48	20	-	20	2	2(2)	878.85
70	21	48	19	11	30	6	2(2), 4(4)	1318.3
80	27	48	19	9	28	2	2(2)	1230.4
90	27	48	19	17	36	6	2(2), 2(4), 2(10)	1581.9
100	33	48	19	15	34	4	2(2), 2(4)	1494.1
110	39	48	19	15	34	2	2(4)	1494.1

*First strand layer is 2" from bottom of beam; second layer is 4" from bottom.
**( ) refers to debonding in the first layer. All debonded strands are in pairs, symmetrically placed about the beam center.

Table H.6. Side By Side Box Beam Bridge Stirrup Requirements.

	Stirrup spacing (in), Length of region (ft)	
Span (ft)	A	B
20	3,10	-
30	6,6	12,15
40	6,3	12,20
50	3,6	6,25
60	3,6	6,30
70	3,6	6,35
80	3,6	6,40
90	3,6	6,45
100	3,6	6,50
110	3,6	8,55

*Region endpoint measured from leftmost side of beam. Stirrups symmetric about centerline.

Bulb-Tee Structures

Bulb tee girder bridge geometry is given in Table H.7. Additional bridge information is given in Tables H.8 and H.9.

Table H.7. Bulb Tee Girder Bridge Geometry.

Spans (ft)	# of beams x spacing (ft)	Overhang (ft)	Total width (ft)	Clear width (ft)
70 - 110	5 x 6.87	2.5	32.5	30
70-110	5 x 7.87	2.5	36.5	34
70 - 100	5 x 9.37	2.5	42.5	40
110	5 x 8.87	3.5	42.5	40

Table H.8. Bulb-Tee Bridge Strand Information.

	Beam		No. of strands*							
Span (ft)	Depth (in)	Width (in)	1st layer	2nd layer	3rd layer	4th layer	Total strands	No. of draped strands	No. draped strands, Position (in)**	Initial prestress force (kips)
70	36	49	17	11	-	-	28	3	3, 4-31	1230
80	36	49	17	19	3	-	39	6	3, 4-29; 3, 6-31	1714
90	42	49	17	19	3	2	41	8	3, 4-25; 3, 6-35; 3, 8-37	1802
100	48	49	17	19	7	-	43	6	3, 4-37; 3, 6-43	1890
110	48	49	17	19	13	3	54	12	3, 2-31; 3, 4-39; 3, 6-41; 3, 8-43	2373

*First strand layer is 2" from bottom of beam; second layer is 4" from bottom; third layer is 6" from bottom.
** First number refers to the number of draped strands; the second number is the strand position from the bottom near the beam midspan; the third number is the strand position from the bottom at the end of the beam. See Figure 2 below.

Table H.9. Bulb-Tee Bridge Stirrup Requirements.

	Stirrup spacing (in), Length of region (ft)	
Span (ft)	A	B
70	3,6	9,35
80	3,6	6,40
90	3,6	6,45
100	3,6	8,50
110	3,6	6,55

*Region endpoint measured from leftmost side of beam. Stirrups symmetric about centerline.

Steel Beam Structures

Steel beam bridge geometry is given in Table H.10. Each beam has 2 rows of shear studs spaced at 2'. Studs are ¾" diameter and no less than 4" tall.

Table H.10. Steel Beam Bridge Geometry.

Span (ft)	Beam Size	# of beams x spacing (ft)	Overhang (ft)	Total width (ft)	Clear width (ft)
20	W21 x 93	5 x 6.375	3.5	32.5	30
20	W21 x 93	6 x 6.25	2.625	36.5	34
20	W21 x 93	7 x 6.25	2.5	42.5	40
30	W21 x 93	5 x 6.375	3.5	32.5	30
30	W21 x 93	6 x 6.25	2.625	36.5	34
30	W21 x 93	7 x 6.25	2.5	42.5	40
40	W24 x 117	5 x 6.375	3.5	32.5	30
40	W24 x 117	6 x 6.25	2.625	36.5	34
40	W24 x 117	7 x 6.25	2.5	42.5	40
50	W30 x 173	5 x 6.375	3.5	32.5	30
50	W30 x 173	6 x 6.25	2.625	36.5	34
50	W30 x 173	7 x 6.25	2.5	42.5	40
60	W36 x 170	5 x 6.375	3.5	32.5	30
60	W36 x 170	6 x 6.25	2.625	36.5	34
60	W36 x 170	7 x 6.25	2.5	42.5	40

APPENDIX I. SUMMARY OF BRIDGE LCCA RESULTS

Table I.1. Deterministic Results.

Design	Span (ft)	Initial cost ($)	90 year cost ($)	Agency cost ($)	User cost ($)
Steel (Galvanized)	20	614,600	1,114,000	998,200	115,800
	30	648,700	1,176,000	1,057,000	119,000
	40	734,200	1,311,000	1,187,000	124,000
	50	890,700	1,545,000	1,418,000	127,000
	60	964,600	1,663,000	1,532,000	131,000
SBB	20	561,200	1,023,000	907,800	115,200
	30	596,300	1,088,000	967,900	120,100
	40	631,500	1,152,000	1,028,000	124,000
	50	666,700	1,216,000	1,088,000	128,000
	60	701,800	1,280,000	1,148,000	132,000
BT & SBB	70	737,000	1,343,000	1,208,000	135,000
	80	772,100	1,407,000	1,268,000	139,000
	90	807,300	1,471,000	1,328,000	143,000
	100	842,400	1,535,000	1,388,000	147,000
	110	877,600	1,599,000	1,449,000	150,000
SSBB	20	583,300	1,054,000	938,600	115,400
	30	629,600	1,134,000	1,014,000	120,000
	40	675,800	1,213,000	1,090,000	123,000
	50	722,100	1,293,000	1,166,000	127,000
	60	782,100	1,398,000	1,266,000	132,000
	70	830,400	1,481,000	1,345,000	136,000
	80	878,600	1,564,000	1,424,000	140,000
	90	926,900	1,647,000	1,503,000	144,000
	100	975,200	1,730,000	1,582,000	148,000
	110	1,023,000	1,813,000	1,661,000	152,000

Table I.2. Yearly Costs For 20’ Span.

Year	Steel	SSBB	SBB & Bulb-Tee
0	615000	583000	561000
1	615000	584000	562000
2	615000	584000	562000
3	616000	584000	562000
4	616000	585000	563000

5	617000	586000	563000
6	625000	594000	572000
7	626000	594000	572000
8	626000	594000	572000
9	627000	595000	573000
10	627000	595000	573000
11	771000	735000	710000
12	771000	735000	710000
13	771000	736000	711000
14	772000	736000	711000
15	772000	737000	711000
16	772000	737000	711000
17	779000	743000	718000
18	779000	743000	718000
19	780000	744000	719000
20	780000	744000	719000
21	780000	744000	719000
22	780000	744000	719000
23	780000	745000	719000
24	931000	874000	846000
25	931000	875000	846000
26	931000	875000	846000
27	931000	875000	847000
28	931000	875000	847000
29	932000	875000	847000
30	937000	880000	852000
31	937000	880000	852000
32	937000	880000	852000
33	937000	881000	852000
34	937000	881000	853000
35	937000	881000	853000
36	937000	881000	853000
37	938000	881000	853000
38	938000	881000	853000
39	983000	933000	904000
40	983000	933000	904000
41	983000	933000	904000
42	983000	933000	904000
43	983000	934000	905000
44	983000	934000	905000
45	987000	937000	908000
46	987000	937000	908000
47	987000	937000	908000

48	987000	937000	908000
49	987000	938000	909000
50	987000	938000	909000
51	1060000	1000000	972000
52	1060000	1000000	972000
53	1060000	1000000	973000
54	1060000	1000000	973000
55	1060000	1000000	973000
56	1060000	1000000	973000
57	1060000	1000000	973000
58	1070000	1010000	975000
59	1070000	1010000	976000
60	1070000	1010000	976000
61	1070000	1010000	976000
62	1070000	1010000	976000
63	1070000	1010000	976000
64	1090000	1030000	1000000
65	1090000	1030000	1000000
66	1090000	1030000	1000000
67	1090000	1030000	1000000
68	1090000	1030000	1000000
69	1090000	1030000	1000000
70	1090000	1030000	1000000
71	1090000	1040000	1000000
72	1090000	1040000	1000000
73	1090000	1040000	1000000
74	1090000	1040000	1010000
75	1090000	1040000	1010000
76	1100000	1040000	1010000
77	1100000	1040000	1010000
78	1100000	1040000	1010000
79	1110000	1050000	1020000
80	1110000	1050000	1020000
81	1110000	1050000	1020000
82	1110000	1050000	1020000
83	1110000	1050000	1020000
84	1110000	1050000	1020000
85	1110000	1050000	1020000
86	1110000	1050000	1020000
87	1110000	1050000	1020000
88	1110000	1050000	1020000
89	1110000	1050000	1020000

Table I.3. Yearly Costs For 30’ Span.

Year	Steel	SSBB	SBB & Bulb-Tee
0	649000	630000	596000
1	649000	630000	597000
2	649000	630000	597000
3	650000	631000	597000
4	651000	632000	598000
5	651000	632000	599000
6	661000	642000	608000
7	661000	642000	609000
8	661000	642000	609000
9	663000	643000	610000
10	663000	643000	610000
11	813000	791000	753000
12	813000	791000	753000
13	813000	791000	754000
14	814000	792000	754000
15	814000	792000	755000
16	814000	792000	755000
17	822000	800000	762000
18	822000	800000	762000
19	823000	801000	763000
20	823000	801000	763000
21	823000	801000	764000
22	823000	801000	764000
23	824000	802000	764000
24	980000	939000	896000
25	980000	939000	897000
26	980000	939000	897000
27	980000	939000	897000
28	980000	939000	897000
29	981000	940000	898000
30	987000	946000	903000
31	987000	946000	903000
32	987000	946000	903000
33	987000	946000	904000
34	987000	946000	904000
35	988000	947000	904000
36	988000	947000	904000
37	988000	947000	904000
38	988000	947000	904000

39	1040000	1000000	961000
40	1040000	1000000	961000
41	1040000	1000000	961000
42	1040000	1000000	961000
43	1040000	1000000	961000
44	1040000	1010000	961000
45	1040000	1010000	965000
46	1040000	1010000	965000
47	1040000	1010000	966000
48	1040000	1010000	966000
49	1040000	1010000	966000
50	1040000	1010000	966000
51	1120000	1080000	1030000
52	1120000	1080000	1030000
53	1120000	1080000	1030000
54	1120000	1080000	1030000
55	1120000	1080000	1030000
56	1120000	1080000	1030000
57	1120000	1080000	1030000
58	1120000	1080000	1040000
59	1120000	1080000	1040000
60	1120000	1080000	1040000
61	1120000	1080000	1040000
62	1120000	1080000	1040000
63	1120000	1080000	1040000
64	1150000	1110000	1070000
65	1150000	1110000	1070000
66	1150000	1110000	1070000
67	1150000	1110000	1070000
68	1150000	1110000	1070000
69	1150000	1110000	1070000
70	1150000	1110000	1070000
71	1160000	1110000	1070000
72	1160000	1110000	1070000
73	1160000	1110000	1070000
74	1160000	1110000	1070000
75	1160000	1110000	1070000
76	1160000	1110000	1070000
77	1160000	1110000	1070000
78	1160000	1110000	1070000
79	1180000	1130000	1090000
80	1180000	1130000	1090000
81	1180000	1130000	1090000

82	1180000	1130000	1090000
83	1180000	1130000	1090000
84	1180000	1130000	1090000
85	1180000	1130000	1090000
86	1180000	1130000	1090000
87	1180000	1130000	1090000
88	1180000	1130000	1090000
89	1180000	1130000	1090000

Table I.4. Yearly Costs For 40' Span.

Year	Steel	SSBB	SBB & Bulb-Tee
0	734000	676000	632000
1	735000	676000	632000
2	735000	676000	632000
3	735000	677000	633000
4	737000	678000	634000
5	737000	679000	634000
6	748000	689000	645000
7	748000	690000	645000
8	748000	690000	645000
9	750000	691000	647000
10	750000	691000	647000
11	913000	846000	796000
12	913000	846000	796000
13	913000	847000	796000
14	915000	848000	797000
15	915000	848000	798000
16	915000	848000	798000
17	923000	857000	806000
18	923000	857000	806000
19	925000	858000	808000
20	925000	858000	808000
21	925000	858000	808000
22	925000	858000	808000
23	925000	859000	808000
24	1100000	1000000	947000
25	1100000	1000000	947000
26	1100000	1000000	947000
27	1100000	1000000	947000
28	1100000	1000000	947000
29	1100000	1010000	948000

30	1100000	1010000	954000
31	1100000	1010000	955000
32	1100000	1010000	955000
33	1100000	1010000	955000
34	1100000	1010000	955000
35	1100000	1010000	956000
36	1100000	1010000	956000
37	1100000	1010000	956000
38	1100000	1010000	956000
39	1160000	1080000	1020000
40	1160000	1080000	1020000
41	1160000	1080000	1020000
42	1160000	1080000	1020000
43	1160000	1080000	1020000
44	1160000	1080000	1020000
45	1170000	1080000	1020000
46	1170000	1080000	1020000
47	1170000	1080000	1020000
48	1170000	1080000	1020000
49	1170000	1080000	1020000
50	1170000	1080000	1020000
51	1250000	1150000	1090000
52	1250000	1150000	1090000
53	1250000	1150000	1090000
54	1250000	1150000	1090000
55	1250000	1150000	1090000
56	1250000	1150000	1090000
57	1250000	1150000	1090000
58	1250000	1160000	1100000
59	1250000	1160000	1100000
60	1250000	1160000	1100000
61	1250000	1160000	1100000
62	1250000	1160000	1100000
63	1250000	1160000	1100000
64	1290000	1190000	1130000
65	1290000	1190000	1130000
66	1290000	1190000	1130000
67	1290000	1190000	1130000
68	1290000	1190000	1130000
69	1290000	1190000	1130000
70	1290000	1190000	1130000
71	1290000	1190000	1130000
72	1290000	1190000	1130000

73	1290000	1190000	1130000
74	1290000	1190000	1130000
75	1290000	1190000	1130000
76	1290000	1190000	1130000
77	1290000	1190000	1130000
78	1290000	1190000	1130000
79	1310000	1210000	1150000
80	1310000	1210000	1150000
81	1310000	1210000	1150000
82	1310000	1210000	1150000
83	1310000	1210000	1150000
84	1310000	1210000	1150000
85	1310000	1210000	1150000
86	1310000	1210000	1150000
87	1310000	1210000	1150000
88	1310000	1210000	1150000
89	1310000	1210000	1150000

Table I.5. Yearly Costs For 50' Span.

Year	Steel	SSBB	SBB & Bulb-Tee
0	891000	722000	667000
1	891000	723000	667000
2	891000	723000	667000
3	892000	723000	668000
4	894000	725000	669000
5	894000	725000	670000
6	906000	737000	682000
7	906000	737000	682000
8	906000	737000	682000
9	908000	739000	684000
10	908000	739000	684000
11	1090000	902000	839000
12	1090000	902000	839000
13	1100000	902000	839000
14	1100000	904000	841000
15	1100000	904000	841000
16	1100000	904000	841000
17	1110000	914000	851000
18	1110000	914000	851000
19	1110000	915000	852000
20	1110000	915000	852000

21	1110000	915000	852000
22	1110000	915000	852000
23	1110000	916000	853000
24	1300000	1070000	997000
25	1300000	1070000	997000
26	1300000	1070000	997000
27	1300000	1070000	998000
28	1300000	1070000	998000
29	1300000	1070000	999000
30	1310000	1080000	1010000
31	1310000	1080000	1010000
32	1310000	1080000	1010000
33	1310000	1080000	1010000
34	1310000	1080000	1010000
35	1310000	1080000	1010000
36	1310000	1080000	1010000
37	1310000	1080000	1010000
38	1310000	1080000	1010000
39	1380000	1150000	1070000
40	1380000	1150000	1070000
41	1380000	1150000	1070000
42	1380000	1150000	1070000
43	1380000	1150000	1070000
44	1380000	1150000	1070000
45	1380000	1150000	1080000
46	1380000	1150000	1080000
47	1380000	1150000	1080000
48	1380000	1150000	1080000
49	1380000	1150000	1080000
50	1380000	1150000	1080000
51	1480000	1230000	1150000
52	1480000	1230000	1150000
53	1480000	1230000	1150000
54	1480000	1230000	1150000
55	1480000	1230000	1150000
56	1480000	1230000	1150000
57	1480000	1230000	1150000
58	1480000	1230000	1160000
59	1480000	1230000	1160000
60	1480000	1230000	1160000
61	1480000	1230000	1160000
62	1480000	1230000	1160000
63	1480000	1230000	1160000

64	1520000	1270000	1190000
65	1520000	1270000	1190000
66	1520000	1270000	1190000
67	1520000	1270000	1190000
68	1520000	1270000	1190000
69	1520000	1270000	1190000
70	1520000	1270000	1190000
71	1520000	1270000	1190000
72	1520000	1270000	1190000
73	1520000	1270000	1190000
74	1520000	1270000	1200000
75	1520000	1270000	1200000
76	1520000	1270000	1200000
77	1520000	1270000	1200000
78	1520000	1270000	1200000
79	1550000	1290000	1220000
80	1550000	1290000	1220000
81	1550000	1290000	1220000
82	1550000	1290000	1220000
83	1550000	1290000	1220000
84	1550000	1290000	1220000
85	1550000	1290000	1220000
86	1550000	1290000	1220000
87	1550000	1290000	1220000
88	1550000	1290000	1220000
89	1550000	1290000	1220000

Table I.6. Yearly Costs For 60’ Span.

Year	Steel	SSBB	SBB & Bulb-Tee
0	965000	782000	702000
1	965000	783000	702000
2	965000	783000	702000
3	966000	783000	703000
4	968000	785000	705000
5	968000	786000	705000
6	981000	799000	718000
7	981000	799000	719000
8	981000	799000	719000
9	984000	802000	721000
10	984000	802000	721000
11	1180000	975000	882000

12	1180000	975000	882000
13	1180000	976000	882000
14	1180000	977000	884000
15	1180000	978000	884000
16	1180000	978000	884000
17	1190000	989000	895000
18	1190000	989000	895000
19	1200000	990000	896000
20	1200000	990000	896000
21	1200000	991000	897000
22	1200000	991000	897000
23	1200000	991000	897000
24	1400000	1150000	1050000
25	1400000	1150000	1050000
26	1400000	1150000	1050000
27	1400000	1150000	1050000
28	1400000	1150000	1050000
29	1400000	1160000	1050000
30	1410000	1160000	1060000
31	1410000	1160000	1060000
32	1410000	1160000	1060000
33	1410000	1160000	1060000
34	1410000	1170000	1060000
35	1410000	1170000	1060000
36	1410000	1170000	1060000
37	1410000	1170000	1060000
38	1410000	1170000	1060000
39	1480000	1240000	1130000
40	1480000	1240000	1130000
41	1480000	1240000	1130000
42	1480000	1240000	1130000
43	1480000	1240000	1130000
44	1480000	1240000	1130000
45	1490000	1250000	1140000
46	1490000	1250000	1140000
47	1490000	1250000	1140000
48	1490000	1250000	1140000
49	1490000	1250000	1140000
50	1490000	1250000	1140000
51	1590000	1330000	1210000
52	1590000	1330000	1210000
53	1590000	1330000	1210000
54	1590000	1330000	1210000

55	1590000	1330000	1210000
56	1590000	1330000	1210000
57	1590000	1330000	1210000
58	1600000	1330000	1220000
59	1600000	1330000	1220000
60	1600000	1330000	1220000
61	1600000	1330000	1220000
62	1600000	1330000	1220000
63	1600000	1330000	1220000
64	1640000	1370000	1250000
65	1640000	1370000	1250000
66	1640000	1370000	1250000
67	1640000	1370000	1250000
68	1640000	1370000	1250000
69	1640000	1370000	1250000
70	1640000	1370000	1250000
71	1640000	1380000	1260000
72	1640000	1380000	1260000
73	1640000	1380000	1260000
74	1640000	1380000	1260000
75	1640000	1380000	1260000
76	1640000	1380000	1260000
77	1640000	1380000	1260000
78	1640000	1380000	1260000
79	1660000	1400000	1280000
80	1660000	1400000	1280000
81	1660000	1400000	1280000
82	1660000	1400000	1280000
83	1660000	1400000	1280000
84	1660000	1400000	1280000
85	1660000	1400000	1280000
86	1660000	1400000	1280000
87	1660000	1400000	1280000
88	1660000	1400000	1280000
89	1660000	1400000	1280000

Table I.7. Yearly Costs For 70’ Span.

Year	SSBB	SBB & Bulb-Tee
0	830000	737000
1	831000	738000
2	831000	738000
3	831000	738000

4	834000	740000
5	834000	741000
6	849000	755000
7	849000	755000
8	849000	755000
9	852000	758000
10	852000	758000
11	1030000	925000
12	1030000	925000
13	1030000	925000
14	1040000	927000
15	1040000	927000
16	1040000	927000
17	1050000	939000
18	1050000	939000
19	1050000	941000
20	1050000	941000
21	1050000	941000
22	1050000	941000
23	1050000	941000
24	1220000	1100000
25	1220000	1100000
26	1220000	1100000
27	1220000	1100000
28	1220000	1100000
29	1220000	1100000
30	1230000	1110000
31	1230000	1110000
32	1230000	1110000
33	1230000	1110000
34	1230000	1110000
35	1230000	1110000
36	1230000	1110000
37	1230000	1110000
38	1230000	1110000
39	1310000	1190000
40	1310000	1190000
41	1310000	1190000
42	1310000	1190000
43	1310000	1190000
44	1320000	1190000
45	1320000	1190000
46	1320000	1190000

47	1320000	1190000
48	1320000	1190000
49	1320000	1200000
50	1320000	1200000
51	1410000	1270000
52	1410000	1270000
53	1410000	1270000
54	1410000	1270000
55	1410000	1270000
56	1410000	1270000
57	1410000	1270000
58	1410000	1280000
59	1410000	1280000
60	1410000	1280000
61	1410000	1280000
62	1410000	1280000
63	1410000	1280000
64	1450000	1320000
65	1450000	1320000
66	1450000	1320000
67	1450000	1320000
68	1450000	1320000
69	1450000	1320000
70	1450000	1320000
71	1460000	1320000
72	1460000	1320000
73	1460000	1320000
74	1460000	1320000
75	1460000	1320000
76	1460000	1320000
77	1460000	1320000
78	1460000	1320000
79	1480000	1340000
80	1480000	1340000
81	1480000	1340000
82	1480000	1340000
83	1480000	1340000
84	1480000	1340000
85	1480000	1340000
86	1480000	1340000
87	1480000	1340000
88	1480000	1340000
89	1480000	1340000

Table I.8. Yearly Costs For 80' Span.

Year	SSBB	SBB & Bulb-Tee
0	879000	772000
1	879000	773000
2	879000	773000
3	880000	773000
4	883000	776000
5	883000	776000
6	898000	791000
7	899000	792000
8	899000	792000
9	902000	795000
10	902000	795000
11	1090000	968000
12	1090000	968000
13	1090000	968000
14	1090000	970000
15	1090000	971000
16	1090000	971000
17	1110000	983000
18	1110000	983000
19	1110000	985000
20	1110000	985000
21	1110000	985000
22	1110000	985000
23	1110000	986000
24	1290000	1150000
25	1290000	1150000
26	1290000	1150000
27	1290000	1150000
28	1290000	1150000
29	1290000	1150000
30	1300000	1160000
31	1300000	1160000
32	1300000	1160000
33	1300000	1160000
34	1300000	1160000
35	1300000	1160000
36	1300000	1160000
37	1300000	1160000
38	1300000	1160000

39	1390000	1240000
40	1390000	1240000
41	1390000	1240000
42	1390000	1240000
43	1390000	1240000
44	1390000	1240000
45	1400000	1250000
46	1400000	1250000
47	1400000	1250000
48	1400000	1250000
49	1400000	1250000
50	1400000	1250000
51	1480000	1330000
52	1480000	1330000
53	1480000	1330000
54	1490000	1330000
55	1490000	1330000
56	1490000	1330000
57	1490000	1330000
58	1490000	1340000
59	1490000	1340000
60	1490000	1340000
61	1490000	1340000
62	1490000	1340000
63	1490000	1340000
64	1530000	1380000
65	1530000	1380000
66	1530000	1380000
67	1530000	1380000
68	1530000	1380000
69	1540000	1380000
70	1540000	1380000
71	1540000	1380000
72	1540000	1380000
73	1540000	1380000
74	1540000	1390000
75	1540000	1390000
76	1540000	1390000
77	1540000	1390000
78	1540000	1390000
79	1560000	1410000
80	1560000	1410000
81	1560000	1410000

82	1560000	1410000
83	1560000	1410000
84	1560000	1410000
85	1560000	1410000
86	1560000	1410000
87	1560000	1410000
88	1560000	1410000
89	1560000	1410000

Table I.9. Yearly Costs For 90’ Span.

Year	SSBB	SBB & Bulb-Tee
0	927000	807000
1	927000	808000
2	927000	808000
3	928000	808000
4	931000	812000
5	932000	812000
6	948000	828000
7	949000	829000
8	949000	829000
9	952000	832000
10	952000	832000
11	1150000	1010000
12	1150000	1010000
13	1150000	1010000
14	1150000	1010000
15	1150000	1010000
16	1150000	1010000
17	1170000	1030000
18	1170000	1030000
19	1170000	1030000
20	1170000	1030000
21	1170000	1030000
22	1170000	1030000
23	1170000	1030000
24	1360000	1200000
25	1360000	1200000
26	1360000	1200000
27	1360000	1200000
28	1360000	1200000
29	1360000	1200000
30	1370000	1210000

31	1370000	1210000
32	1370000	1210000
33	1370000	1210000
34	1370000	1210000
35	1370000	1210000
36	1370000	1210000
37	1370000	1210000
38	1370000	1210000
39	1460000	1300000
40	1460000	1300000
41	1460000	1300000
42	1460000	1300000
43	1460000	1300000
44	1460000	1300000
45	1470000	1310000
46	1470000	1310000
47	1470000	1310000
48	1470000	1310000
49	1470000	1310000
50	1470000	1310000
51	1560000	1390000
52	1560000	1390000
53	1560000	1390000
54	1560000	1390000
55	1560000	1390000
56	1560000	1390000
57	1560000	1390000
58	1570000	1400000
59	1570000	1400000
60	1570000	1400000
61	1570000	1400000
62	1570000	1400000
63	1570000	1400000
64	1620000	1440000
65	1620000	1440000
66	1620000	1440000
67	1620000	1440000
68	1620000	1440000
69	1620000	1440000
70	1620000	1440000
71	1620000	1450000
72	1620000	1450000
73	1620000	1450000

74	1620000	1450000
75	1620000	1450000
76	1620000	1450000
77	1620000	1450000
78	1620000	1450000
79	1650000	1470000
80	1650000	1470000
81	1650000	1470000
82	1650000	1470000
83	1650000	1470000
84	1650000	1470000
85	1650000	1470000
86	1650000	1470000
87	1650000	1470000
88	1650000	1470000
89	1650000	1470000

Table I.10. Yearly Costs For 100' Span.

Year	SSBB	SBB & Bulb-Tee
0	975000	842000
1	976000	843000
2	976000	843000
3	976000	844000
4	980000	847000
5	980000	847000
6	998000	865000
7	998000	865000
8	998000	865000
9	1000000	869000
10	1000000	869000
11	1210000	1050000
12	1210000	1050000
13	1210000	1050000
14	1210000	1060000
15	1210000	1060000
16	1210000	1060000
17	1230000	1070000
18	1230000	1070000
19	1230000	1070000
20	1230000	1070000
21	1230000	1070000
22	1230000	1070000

23	1230000	1070000
24	1420000	1250000
25	1420000	1250000
26	1420000	1250000
27	1420000	1250000
28	1420000	1250000
29	1430000	1250000
30	1440000	1260000
31	1440000	1260000
32	1440000	1260000
33	1440000	1260000
34	1440000	1260000
35	1440000	1260000
36	1440000	1260000
37	1440000	1260000
38	1440000	1260000
39	1540000	1360000
40	1540000	1360000
41	1540000	1360000
42	1540000	1360000
43	1540000	1360000
44	1540000	1360000
45	1550000	1370000
46	1550000	1370000
47	1550000	1370000
48	1550000	1370000
49	1550000	1370000
50	1550000	1370000
51	1640000	1450000
52	1640000	1450000
53	1640000	1450000
54	1640000	1450000
55	1640000	1450000
56	1640000	1450000
57	1640000	1450000
58	1650000	1460000
59	1650000	1460000
60	1650000	1460000
61	1650000	1460000
62	1650000	1460000
63	1650000	1460000
64	1700000	1500000
65	1700000	1500000

66	1700000	1500000
67	1700000	1500000
68	1700000	1500000
69	1700000	1510000
70	1700000	1510000
71	1700000	1510000
72	1700000	1510000
73	1700000	1510000
74	1710000	1510000
75	1710000	1510000
76	1710000	1510000
77	1710000	1510000
78	1710000	1510000
79	1730000	1530000
80	1730000	1530000
81	1730000	1530000
82	1730000	1530000
83	1730000	1530000
84	1730000	1540000
85	1730000	1540000
86	1730000	1540000
87	1730000	1540000
88	1730000	1540000
89	1730000	1540000

Table I.11. Yearly Costs For 110' Span.

Year	SSBB	SBB & Bulb-Tee
0	1020000	878000
1	1020000	878000
2	1020000	878000
3	1020000	879000
4	1030000	883000
5	1030000	883000
6	1050000	901000
7	1050000	902000
8	1050000	902000
9	1050000	905000
10	1050000	905000
11	1270000	1100000
12	1270000	1100000
13	1270000	1100000
14	1270000	1100000

15	1270000	1100000
16	1270000	1100000
17	1290000	1120000
18	1290000	1120000
19	1290000	1120000
20	1290000	1120000
21	1290000	1120000
22	1290000	1120000
23	1290000	1120000
24	1490000	1300000
25	1490000	1300000
26	1490000	1300000
27	1490000	1300000
28	1490000	1300000
29	1490000	1300000
30	1510000	1310000
31	1510000	1310000
32	1510000	1310000
33	1510000	1310000
34	1510000	1320000
35	1510000	1320000
36	1510000	1320000
37	1510000	1320000
38	1510000	1320000
39	1610000	1410000
40	1610000	1410000
41	1610000	1410000
42	1610000	1410000
43	1610000	1410000
44	1610000	1410000
45	1620000	1420000
46	1620000	1420000
47	1620000	1420000
48	1620000	1420000
49	1620000	1420000
50	1620000	1420000
51	1720000	1510000
52	1720000	1510000
53	1720000	1510000
54	1720000	1510000
55	1720000	1510000
56	1720000	1510000
57	1720000	1510000

58	1730000	1520000
59	1730000	1520000
60	1730000	1520000
61	1730000	1520000
62	1730000	1520000
63	1730000	1520000
64	1780000	1570000
65	1780000	1570000
66	1780000	1570000
67	1780000	1570000
68	1780000	1570000
69	1780000	1570000
70	1780000	1570000
71	1790000	1570000
72	1790000	1570000
73	1790000	1570000
74	1790000	1580000
75	1790000	1580000
76	1790000	1580000
77	1790000	1580000
78	1790000	1580000
79	1810000	1600000
80	1810000	1600000
81	1810000	1600000
82	1810000	1600000
83	1810000	1600000
84	1810000	1600000
85	1810000	1600000
86	1810000	1600000
87	1810000	1600000
88	1810000	1600000
89	1810000	1600000

APPENDIX J. SUMMARY OF COMMENTS FROM SECOND FOCUS GROUP MEETING

1) Fabrication may be difficult for shallow box beams using a 7 ksi concrete mix along with extensive reinforcement.

> Unfortunately it is difficult to lower concrete strength and/or reduce reinforcement while meeting strength and other performance goals. If this difficulty is encountered, the galvanized steel bridge might be considered as an alternative.

2) Allow reduction in the specified concrete strength and include optional top strands that may be used for quicker release and shipping/construction.

> The plans presented are suggested guidelines rather than requirements. Viable alternative strand layouts as well as other design approaches are not prohibited.

3) Consider a 27' wide bridge for lowest-volume roads.

> Ideally, more possibilities than those presented would be included. However, based on the resources available for the project, the scope was to focus on the majority of bridge geometries thought to be in need of replacement in the near future.

4) Redesign box beam reinforcement lap lengths, especially for 17' & 21" sections.

> Currently under consideration.

5) Consider different reinforcement detailing and bar size for the end wall/diaphragm, as this poses construction difficulties.

> Currently under consideration.

6) Provide camber/deflection/screed elevation information.

> The plans presented are for 10' increments of span length as well as for a minimum concrete strength. Providing deflection information for these idealized cases may have limited utility, and may be misleading, when an actual structure with a unique span length and concrete strength, generally higher than the minimum specified, is to be designed. These idealized values are therefore not provided, and should be calculated as needed by the design engineer.

7) How will different barriers be dealt with?

> A typical barrier detail is shown on the plans. Alternative barrier types that meet MDOT standards are not restricted from use.

8) Differential cambering may be a problem for side by side box beams when exterior beams have a thicker top flange than interior beams.

> The thicker flange for exterior beams is a necessity to secure the barriers. This problem, more prevalent on longer structures, has been traditionally solved by preloading. However, Alternative viable solutions, such as increasing deck thickness as needed, although not specified in the plans, are not prohibited.

9) Can a single crane with angled lines be used with the lift loops provided?

> This must be evaluated on a case-by-case basis; the contractor should contact the design engineer for approval.

10) The joint details do not seem sufficient to accommodate bridge expansion. Consider eliminating the sleeper slab and unreliable expansion joint and allow the HMA absorb the expansion/contraction.

> The joint detail was checked and verified to be sufficient for the expansion expected. Eliminating the joint completely and replacing with HMA is an alternative, but was deemed to be potentially more troublesome with respect to deterioration than the current solution.