MDOT RC-1602

Improving Bridges with Prefabricated Precast Concrete Systems

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16. Abstract

In order to minimize the impact of construction on the traveling public, MDOT utilizes innovative and specialized construction methods such as Accelerated Bridge Construction (ABC). Michigan, like other highway agencies in the region, has several challenges in specifying prefabricated bridge elements and systems (PBES) and accelerated bridge construction (ABC) techniques for bridge replacement projects. Among those challenges, the following are the most common: (1) justification of initial project costs, (2) defining a rational process for selecting ABC over conventional construction, (3) absence of PBES selection guidelines and proven standard and successful designs, (4) absence of constructability evaluation guidelines, and (5) uncertain durability performance of PBES and connections.

This research project was initiated with several objectives. They were achieved by (1) synthesizing the state-of-the-art practices, challenges, and lessons learned from the implemented ABC projects, (2) developing a Michigan-specific decision-making platform based on the site specific data to identify the optimal construction alternative between conventional construction and ABC, (3) developing a comprehensive list of connection details and cementitious materials for connections, (4) developing standard deck level longitudinal connection details for typical highway bridges, (5) developing a template of special provisions for grout selection and application procedures, (6) documenting construction procedures, equipment, and implementation limitations, (7) developing a constructability review checklist, and (6) providing recommendations for further research and implementation of PBES based on constructability, maintainability, repairability, and durability.

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Improving Bridges with Prefabricated Precast Concrete Systems

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EXECUTIVE SUMMARY

INTRODUCTION

Michigan has eleven corridors of National/International significance. The decision principles to guide the management, operation, and investments on these corridors include strategies to reduce delays and minimize construction impacts. MDOT's vision for transportation states that, "*MDOT will embrace technology and technological development. The department will use innovation in every aspect of what it builds, how it builds, and in every service that is provided*." In order to minimize the impact of construction on the traveling public, MDOT utilizes innovative and specialized construction methods such as ABC (MDOT 2007). The first such implementation of *Accelerated Bridge Construction* was in 2008. ABC was used to construct Michigan's first totally prefabricated full-depth deck panel bridge system, the Parkview Avenue Bridge. The bridge carries Parkview Avenue over US-131 freeway. MDOT has completed a few more ABC projects since then.

Michigan, like other highway agencies in the region, has several challenges stated below related to prefabricated bridge elements and systems (PBES) and accelerated bridge construction (ABC).

- Justification of initial project costs,
- A rational process for selecting ABC over conventional construction,
- Lack of access to PBES selection guidelines and proven standard and successful designs,
- Constructability evaluation guidelines, and
- Durability performance of PBES and connections.

This research project is designed for addressing the above challenges by documenting current national and international state-of-the-art practices in PBES design, construction, and demolition, and associated potentials and limitations. The process is to analyze the existing practices and systems and then to identify fully prefabricated precast concrete systems suitable for Michigan. The specific objectives of the study are as follows:

(1) Develop a comprehensive list of prefabricated bridge elements and systems (PBES) and associated potentials and limitations with attention to durability, repairability, and maintainability.

(2) Develop a Michigan-specific decision-making platform.
(3) Evaluate the performance of selected PBES bridges.
(4) Develop a comprehensive list of connection details and cementitious materials for durable connections and closures suitable for Michigan exposure conditions.
(5) Develop standard deck level longitudinal connection details for typical highway bridges.
(6) Document construction procedures, equipment, and implementation limitations; and develop recommendations for demolition of selected PBES bridges.
(7) Provide recommendations for further research and implementation of selected systems.

To achieve these objectives, this project was organized into five tasks: (1) review the state-of-the-art literature, (2) assess the performance, challenges and lessons learned, (3) develop a Michigan-specific ABC decision-making platform, (4) recommend PBES, connection details, and cementitious grout or closure material suitable for Michigan, and demolition procedures for selected PBES bridges, and (5) provide recommendations for further research and ABC implementation.

LITERATURE REVIEW

Literature on the following topics was reviewed:

1. The PBES currently being implemented under ABC and the potentials and limitations associated with each structural system,
2. Connection (joint) details between prefabricated elements or systems,
3. The grout materials for connections and their application procedures,
4. The accelerated construction and demolition methods and equipment,
5. The constructability benefits, implementation barriers, and essential elements of a constructability program, and
6. State-of-the-art decision making models.

ASSESSMENT OF THE PERFORMANCE, CHALLENGES AND LESSONS LEARNED

The durability performance of (a) full-depth deck panel systems, (b) the bridges constructed using Self Propeller Modular Transporters (SPMT), (c) the bridges constructed using slide-in techniques, and (d) side-by-side box-beam bridges were reviewed. The outcome of the review is (i) the causes of premature deterioration, (ii) potential measures to enhance durability performance, and (iii) recommendations for future research. In addition, a large number of ABC projects were reviewed, and the challenges and lessons learned were documented. The challenges and lessons learned were synthesized and categorized into three major groups.

1. Project planning and design
2. Precast element fabrication.
3. Construction operations and tolerances.

The outcome of this synthesis, in conjunction with the experience of the project team and review of constructability benefits, implementation barriers, and essential elements of a constructability program, led to the development of a constructability review checklist for ABC projects.

MICHIGAN-SPECIFIC ABC DECISION-MAKING PLATFORM

State-of-the-art decision making models were reviewed, and the deficiencies pertaining to the existing models were documented. To overcome the limitations in the available decision-making processes, a multi-criteria decision-making process and a guided software were developed. The software is named as the Michigan Accelerated Bridge Construction Decision-Making (Mi-ABCD) tool that evaluates the Accelerated Bridge Construction (ABC) vs. Conventional Construction (CC) alternatives for a particular project. The process incorporates project-specific data and available user-cost and life-cycle cost models to help the decision makers with quantitative data to make informed decisions on bridge construction alternatives. The software was developed using Microsoft Excel and Visual Basic for Applications (VBA) scripts. The multi-criteria decision-making process developed during this project provides solutions to many issues in ABC decision-making. The decision-making framework provides a *preference rating* of each construction alternative. The contribution of each parameter to the *preference ratings* is also provided. The decision-making platform developed in this project is an advancement over the available decision-making models by addressing their shortcomings.

RECOMMENDATIONSFOR PBES AND ASSOCIATED DETAILS

After synthesizing the state-of-the-art PBES practices in bridge construction and demolition, durability performance, and lessons learned, the PBES that can be readily implemented in Michigan are recommended without reservation by considering constructability, maintainability, repairability, and durability. The connection details between the PBES are selected based on the exposure conditions, load transfer mechanism, durability, constructability, dimensions and tolerances, and formwork requirement. Standard details for longitudinal connection at the deck level are presented and recommended for implementation.

The attributes of selected PBES, formwork for grouting of connections, constructability challenges, and other limitations are also presented. The demolition techniques and equipment for each of the selected PBES are also discussed.

Several challenges were identified during the grout selection process; therefore, to address those challenges, a template of special provision for grout selection and application is presented in this report. Further, the importance of developing a database of material properties suitable for establishing the connection between prefabricated components and making it available to the designers is discussed.

ABC CONSTRUCTABILITY REVIEW CHECKLIST

An ABC constructability review checklist is presented in this report. The checklist needs to be reviewed by the project development team and the project delivery team. Review of the checklist before initiating the design process will help to prevent repeated mistakes of the past, and to complete projects in most efficient and cost effective manner. This ABC constructability review checklist can be fine-tuned by monitoring the construction activities including prefabrication and by conducting a post-construction program to document the challenges and lessons learned.

SUMMARY AND CONCLUSIONS

The project was organized into five tasks: (1) review the state-of-the-art literature, (2) assess the performance, challenges and lessons learned, (3) development of a Michigan specific ABC decision-making platform, (4) recommend PBES, connection details, and cementitious grout or closure material suitable for Michigan, and demolition procedures for selected PBES bridges, and (5) provide recommendations for further research and ABC implementation.

A Michigan-specific decision-making process that was supported by a software platform was developed. The decision-making process was structured to allow the site-specific analysis of the optimal construction alternative decision between conventional construction and ABC. The decision-making process incorporates parameters that are evaluated based on site specific data. It also incorporates judgment of planning, design, transportation, and construction experts. To guide the experts in providing their judgments, supportive information on the site specific data is generated and made available. Mathematical fundamentals of the decision-making platform are based on principal eigenvector calculations to deal with the potential variability of expert judgments. The result is presented as a *preference rating* of each construction alternative. The contribution from each parameter to the *preference ratings* is also provided. The decision-making process and the platform developed in this project is an advancement over the available decision-making models by addressing their shortcomings.

The prefabricated bridge elements and systems, connection details, and grout or special mixes appropriate for the Michigan exposure provide a significant contribution to this project. After synthesizing the state-of-the-art practices and performance and lessons learned from ABC implementations, potential PBES for immediate implementation are identified. These PBES recommendations are based on constructability, maintainability, reparability, and durability (CMRD). The suitable connections between the PBES are identified considering the exposure conditions, load transfer mechanism, constructability, durability, dimensions and tolerances, and formwork requirements for grout or special mix placement. Also, standard details for the longitudinal deck level connection were developed for bridge superstructures with precast prestressed girders. The details in the MDOT Bridge Design Guide format is presented in Appendix I.

Nonspecific grout or special mix recommendations for a connection are not practical because the material selection is based on project parameters. The parameters are (1) site specific exposure conditions, (2) grout pocket dimensions, (3) application procedures and limitations, (4) curing requirements and also (5) grout properties such as compressive strength, volume stability, initial setting time or working time, and working temperature range. The grout materials need to be tested and evaluated for the particular application before field implementation. In order to address these difficulties, a template of special provisions for grout selection and application is presented in the report. In addition to that, a database of material properties suitable for the connection between prefabricated components is also provided in the report.

An ABC constructability review checklist is presented in the report. This checklist can be used to guide the project development and delivery teams in constructability assessments before initiating the design process. Moreover, the checklist will be useful to overcome mistakes documented in earlier ABC implementations. The checklist will also help with project management, scheduling and cost control.

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1 INTRODUCTION

1.1 OVERVIEW

Michigan has eleven corridors of National/International significance. The decision principles to guide the management, operation, and investments on these corridors include strategies to reduce delays and minimize construction impacts. MDOT's vision for transportation states that, "*MDOT will embrace technology and technological development. The department will use innovation in every aspect of what it builds, how it builds, and in every service that is provided*." In order to minimize the impact of construction on the traveling public, MDOT utilizes innovative and specialized construction methods such as ABC (MDOT 2007). The first such implementation of *Accelerated Bridge Construction* was in 2008. ABC was used to construct Michigan's first totally prefabricated full-depth deck panel bridge system, the Parkview Avenue Bridge. The bridge carries Parkview Avenue over US-131 freeway. MDOT has completed a few more ABC projects since then.

The National Bridge Inventory (NBI) of 2013 showed that there are 4,423 bridges (exceeding the span of 20ft) maintained by the Michigan Department of Transportation (MDOT) on the MDOT trunkline system, and among those the percentage of structurally deficient bridges is about 5.9%. The requirement of rehabilitation and repair with conventional approaches creates delays and safety conditions for the commuters. This can be affirmed by the 2010 road construction work zone crash statistics from the State of Michigan, which documented 5632 crashes, 1488 injuries, and 23 driver and/or passenger fatalities in highway work zones in 2010. According to AASHTO (2011) there was an increase of 500 crashes and 100 injuries from 2009 to 2010. The work zone safety guidelines provided by the Transportation Information Center (TIC) suggests making traffic safety, project duration, and construction quality an integral and high priority factor of every project (TIC 2006).

Michigan, like other highway agencies in the region, has several challenges in specifying prefabricated bridge elements and systems (PBES) and accelerated bridge construction (ABC) techniques for bridge replacement projects. Among those challenges, the following are the most common: (1) justification of initial project costs, (2) defining a rational process for selecting ABC over conventional construction, (3) absence of PBES selection guidelines and proven standard and successful designs, (4) absence of constructability evaluation guidelines, and (5) uncertain durability performance of PBES and connections (FHWA 2012).

Considering the current and future needs of the state as well as the local agency needs, MDOT initiated this project with the objective of identifying and documenting national and international best practices on accelerated bridge construction and demolition, identifying precast system configurations with attention to constructability, maintainability, repairability, and durability, and developing short-term and long-term plans for technology implementation.

1.2 PROJECT OBJECTIVES AND TASKS

This project is designed for documenting current national and international state-of-the-art practices in prefabricated bridge elements and systems design, construction, and demolition, along with associated potentials and limitations. The process is to analyze the existing practices and systems and then to identify fully prefabricated precast concrete systems suitable for Michigan. The specific objectives of the study are as follows:

(1) Develop a comprehensive list of prefabricated bridge elements and systems (PBES) and associated potentials and limitations with attention to durability, repairability, and maintainability.
(2) Develop a Michigan-specific decision-making platform.
(3) Evaluate the performance of selected PBES bridges.
(4) Develop a comprehensive list of connection details and cementitious materials for durable connections and closures suitable for Michigan exposure conditions.
(5) Develop standard deck level longitudinal connection details for typical highway bridges.
(6) Document construction procedures, equipment, and implementation limitations; and develop recommendations for demolition of selected PBES bridges.
(7) Provide recommendations for further research and implementation of selected systems.

To achieve these objectives, this project was organized into six tasks: (1) review the state-of-the-art literature, (2) assess the performance, challenges and lessons learned, (3) develop a

Michigan specific ABC decision-making platform, (4) recommend PBES, connection details, and cementitious grout or closure material suitable for Michigan, and demolition procedures for selected PBES bridges, (5) develop standard details for deck level longitudinal connection of decked bulb-tee and decked box-beams, and (6) provide recommendations for further research and ABC implementation.

1.3 REPORT ORGANIZATION

The report is organized in 8 chapters.

Chapter 2, the state-of-the-art literature review describes potentials and limitations of PBES, connection details between prefabricated elements, properties of cementitious grouts and special mixes and their application procedures, and accelerated construction along with demolition technologies. Further discussed are constructability evaluation benefits, implementation challenges, and elements of a constructability program.. Moreover, the state-of-the-art decision-making models/frameworks and their associated limitations are reviewed with respect to cast-in-place (CIP) and ABC methods.

Chapter 3 presents challenges and lessons learned from earlier ABC implementations and performance of in-service ABC bridges.

Chapter 4 describes the Michigan specific ABC decision-making platform.

Chapter 5 describes PBES, connection details, and grout and special mixes for Michigan exposure conditions. Also presented are the potential construction/demolition methods and equipment, and implementation challenges associated with the PBES.

Chapter 6 presents constructability review checklist that includes questionnaire developed through synthesizing the benefits, challenges, and essential elements of a constructability program.

Chapter 7 presents the comprehensive results, recommendations, and proposed further work on this topic.

Chapter 8 includes the cited references.

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2 STATE-OF-THE-ART LITERATURE REVIEW

2.1 OVERVIEW

2.1.1 ABC/ABR Definition

Accelerated Bridge Construction (ABC) is a project delivery process which minimizes on-site construction duration. ABC alleviates congestion, reduces environmental impacts, and improves safety. The on-site construction duration is reduced through several processes. Currently popular processes are (a) assembling the prefabricated bridge structural elements into place, (b) moving a bridge superstructure or a complete bridge from within right-of-way into place and (c) constructing a 'Replacement Bridge' on temporary supports adjacent to the bridge and sliding it in place following rapid demolition.

2.1.2 Objective and Approach

The literature review is conducted to identify, review, and synthesize information related to accelerated bridge construction. Concentration areas for the literature review are as follows:

- Prefabricated bridge configurations/elements/systems currently being used in ABC or the elements/systems that show a potential (These include prefabricated superstructure and substructure elements.),
- Connection (joint) details between prefabricated elements or systems,
- Cementitious grout or special concrete mixes and application procedures specified for the prefabricated element connections,
- Accelerated bridge construction and demolition methods and equipment,
- Constructability analysis benefits, implementation challenges, and essential elements of a constructability program, and
- State-of-the-art decision making models.

2.2 PREFABRICATED BRIDGE ELEMENTS AND SYSTEMS

2.2.1 Introduction

Prefabricated elements and systems are being used to minimize on-site bridge construction duration. In the meantime, innovative details and construction procedures are being developed. The literature presents cross-section details and span lengths of elements and systems as well as construction details. However, there is no comprehensive discussion on the benefits and limitations of these elements and/or systems in terms of span length, underclearance, durability, and repairability. Hence, this section of the report is developed with the following objectives:

1. Document the available prefabricated bridge elements and systems (PBES).
2. Discuss the benefits and limitations of each PBES used in bridge superstructures, and commonly used span ranges to facilitate selection of such elements or systems for a specific site.
3. Document the concrete mix designs to achieve the required strengths for specific spans.

In PBES, the bridge superstructure typically consists of (1) prefabricated girders and a cast-in-place concrete deck, or (2) prefabricated girders and precast deck panels with or without cast-in-place concrete deck, or (3) modular systems (e.g., single-tee, double-tee, segmental box girders, or (4) any other configuration where a continuous bridge superstructure is formed once the prefabricated elements are placed and connected through field cast joints. The prefabricated bridge substructure units typically consist of foundations (piles or footings), pile caps, columns, bent caps (or pier caps), abutments, and backwalls (RTA 2004). According to RTA (2004), widely recognized classifications of the bridge span ranges are these:

- Short-span: 20 ft to 60 ft,
- Short-to-medium span: more than 60 ft up to 130 ft,
- Medium span: more than 130 ft up to 260 ft,
- Medium-to-long span: more than 260 ft up to 980 ft, and
- Long span: more than 980 ft up to 2,600 ft.

The maximum span length of the standard prefabricated girder sections is given in the PCI Bridge Design Manual (PCI 2011) and the DOT documents (MDOT-BDM 2013; UDOT 2010b). The suitable standard sections for the required span can be identified from these manuals. However, in addition to the span limitations, the weight of prefabricated elements for transport and placement is a consideration. FHWA (2012) lists the transport weight and size limitations as one of the major concerns raised by the DOTs during regional peer-to-peer exchanges. The weight issue is addressed in the MDOT-BDM (2013) Section 7.01.19, recommending a limiting weight of prefabricated bridge element (PBE) to 80 kips (40 tons) for safe handling using conventional equipment. The ABC Toolkit developed under the SHRP2 R04 project (SHRP2 2012), on the other hand, recommends limiting weights to 160 kips (80 tons). Where site conditions allow, SHRP2 (2012) suggests using PBE up to 250 kips (125 tons). Increased weight limits allow building longer spans and wider bridges to further reduce construction duration. Initially, the weight limits were raised to accommodate the substructure components. However, weight limits need to be reviewed after selecting the girder types because the girder weights may exceed the limits specified for the substructure components. As shown in Table 2-1, the majority of girder spans are below the 80 kip weight limit.

Table 2-1. Girder Types and Span Length with 80 kips Weight Limit

Girder Type		Weight (kip/ft)	Standard Section Maximum Span (ft)*	Span Length with 80 kips Weight Limit (ft)**
PCI Decked Bulb-Tee with 6 in. thick flange	DBT-35	1.07	85	75
	DBT-53	1.19	135	67
	DBT-65	1.27	165	63
PCI Decked Bulb-Tee with 9.5 in. thick flange	DBT-35	1.42	*Not defined*	56
	DBT-53	1.54	*Not defined*	52
	DBT-65	1.62	*Not defined*	49
Decked PCI Box-Beam (48 in. wide) with 9.5 in. thick flange	BI	1.67	*Not defined*	48
	BII	1.73	*Not defined*	46
	BIII	1.80	*Not defined*	45
	BIV	1.83	*Not defined*	44
PCI Spread Box-Beam (48 in. wide)	BI	0.72	75	75
	BII	0.78	85	85
	BIII	0.85	95	95
	BIV	0.88	100	91
PCI I Beam	I	0.29	40	40
	II	0.38	65	65
	III	0.58	90	90
	IV	0.82	125	97
	V	1.06	140	76
	VI	1.13	150	71
PCI Bulb-Tees	BT-54	0.69	115	115
	BT-63	0.74	130	108
	BT-72	0.80	145	100
MI 1800		0.91	145	88

* Maximum span length is not defined for nonstandard sections
** Highlighted cells indicate when the component weight limits the usable span

In 1949, precast concrete girders were introduced to the U.S. during the construction of the Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania (PCI 1976). Prestressed concrete single-cell and multi-cell box-beams, which are used in the side-by-side box-beam bridge, are one of the first generation prefabricated girders used in short-span (20 ft to 60 ft) bridges. Use of prestressed concrete box-beams in Michigan bridges dates back to 1955 (Attanayake 2006). Precast concrete I-girders were later developed in 1956 for spans ranging from 24 ft to 70 ft (PCI 1976).

The use of precast concrete bridge deck panels dates back to early 1970's (Issa et al. 1995a). During the mid 1970's, the prestressed bridge deck panels were implemented in Illinois, Texas, Florida, Virginia, and Pennsylvania. Most of the precast bridge deck panels used in that era were neither prestressed nor post-tensioned. The details used at that time for connecting deck panels to the girders were not able to provide monolithic behavior of the deck-girder integrated section. Further, the details used at that time were not adequate to accommodate skew and deck crown (PCI 1976).

Another example of a prefabricated section is the modular superstructure element where the girder and the deck are prefabricated as a single monolithic unit. The double-tee section, which was designed for spans from 25 ft to 65 ft, is one of the first generation prefabricated modular elements. Other sections used during the 1970's were channel sections and tri-tee sections. The channel sections were designed for spans from 24 ft to 44 ft and tri-tee sections from 25ft to 40ft. These sections were specified for buildings and parking structures, and subsequently used in bridges with low traffic volume in the U.S. (PCI 1976).

In April 2004, Ralls et al. (2005) conducted a scanning tour covering five countries under the sponsorship of the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO). The purpose was to study precast structural elements that can be utilized in ABC. Moreover, for the substructure elements, various researchers and U.S. state departments of transportation (DOTs) developed configurations which could be prefabricated and transported to the site to accelerate the substructure construction (Stamnas and Whittemore 2005). Further, new sections for modular superstructure elements were developed to accelerate the onsite bridge construction process (Graybeal 2009).

PBES, as well as the bridges built using such elements or systems, are presumed to be durable. Unfortunately, data presented in literature is not encouraging. The durability problems are due to the quality of the prefabricated elements, defects during fabrication and erection, details and materials used for connecting prefabricated elements, and construction quality (Issa et al. 1995a; Aktan et al. 2002; Attanayake 2006; Culmo 2010). To promote successful implementation of PBES, compiling a library of elements and systems for designers to select from based on site-specific parameters is essential. In the element library, including information on benefits and limitations for implementation is desired for planning, design, constructability review, and scheduling. The overall performance of the bridges that are built using prefabricated elements and/or systems is discussed in Chapter 3. This chapter presents element specific durability problems and benefits and limitations for implementation.

The typical cross-section dimensions and span lengths of PBES for ABC are compiled from reviews of bridge plans, also from recent demonstration projects, and input from project engineers directly involved in ABC projects. The PBES are listed under four major groups: girders, decks, modular superstructure elements and systems, and substructure elements (Figure 2–1). The elements and systems, which are listed under the four major groups, are further categorized based on their use in accelerated bridge construction. The use categories are color coded as: common, limited, not used, and used in long span bridges (Figure 2–1). Bridge superstructure with a cast-in-place concrete deck is not classified as ABC; but shown in Figure 2–1 in a separate category. The elements or systems listed under the *limited* category either have been implemented no more than once or twice or are still under development. After careful analysis of the details and performance records available in literature, benefits and limitations of specifying such elements and systems in ABC projects are summarized. This chapter only provides a brief discussion of each element or system while the details are provided in Appendix A.

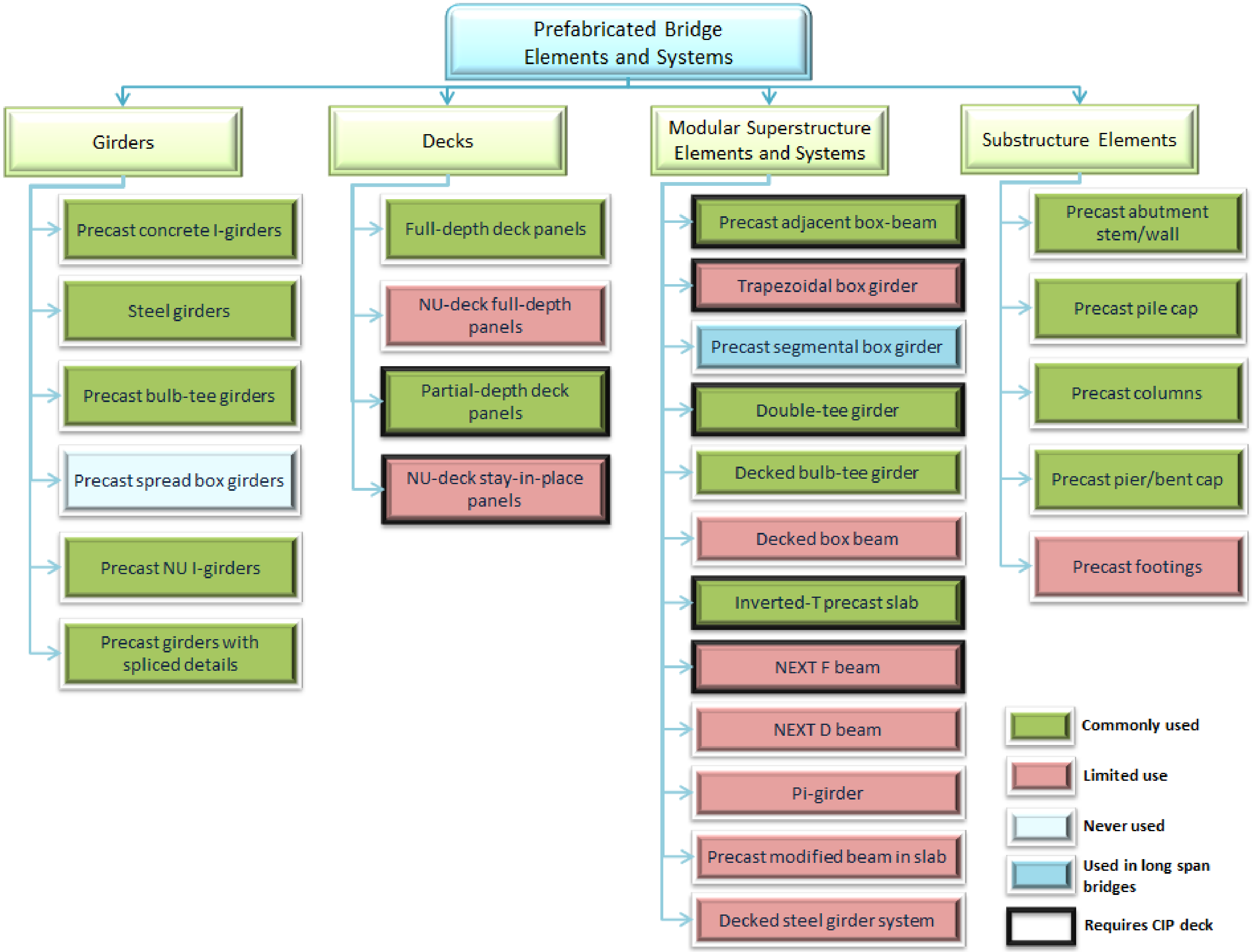


Figure 2–1. Prefabricated bridge elements and systems

2.2.2 Girders

Precast concrete girders are the most commonly specified among all the prefabricated structural elements. Girder types and sections are developed considering span, underclearance, aesthetics, loading (ADT and ADTT), and exposure. Use of these girders in ABC is limited because they can only be combined with partial-depth or full-depth deck panels to qualify for accelerated construction. Though the steel girder is listed in Figure 2–1, the discussion is limited because it is possible to design steel girders for most commonly used spans using rolled or built-up sections. On the other hand, prestressed concrete girders require testing and validation when they are different from commonly used sections and spans. Hence, commonly used spans and design strengths are provided with the prestressed girders to help designers specify sections for preliminary design based on site parameters.

Most of the precast girders listed below have been used in vast majority of the projects. A few of them are standardized, and the designers, fabricators, and contractors are familiar with the benefits and limitations. The girder types, the projects where they are utilized, information on cross-section dimensions and span lengths, applicable concrete strengths, and benefits and limitations of using the girders are summarized in Appendix A. The girder types reviewed during this study include

1. Precast concrete (PC) I-girders,
2. Steel girders,
3. Precast bulb-tee girders,
4. Precast spread box girders,
5. Precast NU I-girders, and
6. Precast girders with spliced details.

The tables given below (Table 2-2, Table 2-3, and Table 2-4) show the design strength and possible span ranges for standard I-girders, box-beams, girders with spliced span, and bulb-tee girders.

Table 2-2. Standard PC I-Girders, Spread Box Girders, and Girders with Spliced Details (Source: MDOT-BDM 2013; Castrodale and White 2004)

	Depth (in.)	Spans up to (ft)	28-day concrete strength (psi)
PC - I (type I – IV)	28 – 54	~114	5,000 – 7,000
PC – I (Wisconsin type)	70	~120	5,000 – 7,000
PC – I (MI 1800)	70.9	~145	5,000 – 7,000
Spread box-beam (36 in. wide)	42	~95	5,000 – 7,000
Spread box-beam (48 in. wide)	60	~140	5,000 – 7,000
Girders with spliced span	72 – 108	~220	9,000 – 10,000

Table 2-3. Depth and Span Range of Utah Bulb-Tee Girders (Source: UDOT 2010b)

	Depth (in.)	Spans up to (ft)		Diameter of prestressing strands (in.)	Number of strands
		28-day concrete strength of 6,500 psi	28-day concrete strength of 8,500 psi		
Utah bulb-tee girders spaced at 8 ft	42	~85	~98	0.6	N/A
	50	~97	~117		
	58	~112	~131		
	66	~124	~146		
	74	~140	~157		
	82	~150	~167		
	90	~164	~177		
	98	~169	~186		

Table 2-4. Depth and Span Range of NEBT Girders (Source: PCI NE 2011)

	Depth (in.)	Spans up to (ft)	Diameter of prestressing strands (in.)	Number of strands	28 day concrete strength (psi)
NEBT girders spaced at 8 ft	39.4	~85	0.6	60	10,000
	47.2	~98			
	55.1	~111			
	63	~121			
	70.9	~131			

The girders are specified considering span, capacity, efficiency, and benefits/limitations. Most girders are suitable for short and short-to-medium span bridges (up to 130 ft). The girder options are limited for medium span bridges (130 ft to 260 ft). Several efforts have been made to develop girders for medium span bridges (Geren and Tadros 1994). Another option for medium span bridges is girder splicing, which could potentially provide sections for spans up to 220 ft with post-tensioning (Castrodale and White 2004; Chung et al. 2008). Specifically, prestressed I- and bulb-tee girders can be redesigned to incorporate post-tensioning and/or spliced details to accommodate longer spans. Russell et al. (1997) performed a comprehensive study on effect of strand size and spacing on capacity and cost for high strength concrete girders. This study showed that 0.7 in. diameter strands at 2 in. spacing in a bulb-tee girder with 10,000 psi strength provide an economical solution for longer spans.

The NU-I girder series includes depths ranging from 30 in. to 95 in. and constant dimensions for top and bottom flanges, and includes depths for spans up to 300 ft with post-tensioning (Beacham and Derrick 1999). However, the girder web thickness needs to be increased when post-tensioning is used. Reinforcement details are standardized so that the amount of post-tensioning, girder span, or girder spacing does not affect the reinforcement pattern except the spacing (details of NU I-girder reinforcement are presented in Appendix A). Moreover, the large span-to-depth ratio allows for specifying these sections in lieu of steel girders without increasing the superstructure depth (Beacham and Derrick 1999). These girders have been used in many projects and had proven to be durable for continuous spans.

The NU 900 I-girder (35.4 in. deep) is the shallowest section of the series, which has been successfully implemented in several projects (Morcous et al. 2011). In 2009, two non-proprietary Ultra High Performance Concrete (UHPC) mixes were developed by the University of Nebraska-Lincoln and designated as NU-UHPC mix #4 and mix #5. A detailed discussion on these mixes is given in Tadros and Morcous (2009). A new configuration of the NU 900 I-girder was developed with the NU-UHPC mix #5 and 0.7 in. diameter prestressing strands. Research on the NU 900 I-girder verified the implementation with 2 in. strand spacing (Morcous et al. 2011). NU 900 I-girder spans, number of strands, strand size, and compressive strength of concrete are shown in Table 2-5. The typical NU I-girder series includes a wide range of depths and spans (Table 2-6).

Table 2-5. NU 900 I-Girder Specifications (Source: Morcous et al. 2011)

	Spans up to (ft)	Diameter of prestressing strands (in.)	Number of strands	Concrete strength at release (psi)
NU 900 I-girder (depth – 35.4 in.)	~90	0.5	60	6,000
	~110	0.6	60	8,500
	~90		36	
	~130	0.7	60	11,000
	~110		38	
	~90		26	

Table 2-6. NU I-Girder Series Specifications (Source: Hanna et al. 2010b)

	Depth (in.)	Spans up to (ft)	Diameter of prestressing strands (in.)	Number of strands	28 day concrete strength (psi)
NU I-girder	94.5	~200	0.6	60	12,000
	78.7	~180			8,000 – 12,000
	70.9	~172			8,000 – 12,000
	63.0	~155			8,000 – 12,000
	53.1	~135			8,000 – 12,000
	43.3	~118			8,000 – 12,000
	35.4	~110			8,000 – 12,000

2.2.3 Decks

Precast full-depth and partial-depth deck panel systems were reviewed. The systems that were reviewed include:

1. Full-depth deck panels with transverse prestressing and longitudinal post-tensioning,
2. Full-depth deck panels with only longitudinal post-tensioning,
3. Full-depth deck panels with only transverse prestressing,
4. NU-deck full-depth panels ,
5. Partial-depth deck panels, and
6. NU-deck stay-in-place panels.

Appendix A presents the specifications, benefits, and limitations of each system. The full-depth deck panels with transverse prestressing and longitudinal post-tensioning is the most specified deck panel system currently available for accelerated bridge construction. The primary limitations listed are related to grouting connections and repair and rehabilitation complexities of the post-tensioned system. A large number of grouted connections require selection of grout with specific durability and bonding properties. Also, grout properties are important in developing solid and tight fit between the components. Further, connection detailing and grout selection, preparation, application, curing and protection needs to be addressed in special provisions (see Section 2.4 for more details on this topic). With regard to limitations on repair and rehabilitation with the post-tensioning, it is best to implement this system at sites where girder damage (e.g., high-load hits) is unlikely. Based on the currently available data, deck panel systems without longitudinal post-tensioning could not fulfill the durability performance expectations. New partial and full-depth deck panel systems have been developed. These are NU-deck panels (1st and 2nd generation – full-depth) (Badie et al. 2006; Hanna et al. 2010a), the modified NU-deck panel (full-depth) (Wipf et al. 2009b), and the NU-deck stay-in-place (SIP) panels (Badie et al. 1998; Versace and Ramirez 2004). These systems use unprotected prestressing and post-tensioning strands, which will not result in a durable deck assemblage. Considering all the benefits and limitations, full-depth deck panels with transverse prestressing and longitudinal post-tensioning are still the best choice for Michigan bridges where substantial winter maintenance is required.

2.2.4 Modular Superstructure Elements and Systems

Prefabricated elements that are placed side-by-side to form a bridge superstructure and connected by shear and/or flexure-shear transfer details are referred to as modular superstructure elements. Examples are single-cell rectangular box-beams specified in adjacent box-beam bridges, trapezoidal box girders, single-cell or multi-cell sections for segmental box girder bridges, tee-beams, double-tee girders, and deck integrated sections. The decked single-cell rectangular box-beam was developed in 2010 and fabricated in 2012 for the M-25 bridge over the White River in Michigan (MDOT M-25 bridge plans 2010).

Prefabricated modular systems, such as the INVERSET™ and decked steel girder system, are developed by combining multiple girders and a precast slab. The decked steel girder system design standards and design examples are provided in the SHRP 2 Project R04 publications (SHRP2 2012). The decked steel girder system has been used in the I-93 Fast 14 project in Medford, MA (MassDOT 2011), and the Keg Creek bridge replacement project in Pottawattamie County, IA (IowaDOT 2011).

Information related to prefabricated elements and systems are presented in Appendix A. Sometimes, both prefabricated elements and systems are referred to as modules. The information summarized in the appendix includes the projects where these elements or systems were implemented, along with the attributes, benefits, and limitations of each modular element or system.

The *prefabricated modular superstructure elements* reviewed during this project include

1. Precast adjacent box-beam,
2. Trapezoidal box girder,
3. Precast segmental box girder,
4. Double-tee girder,
5. Decked bulb-tee girder,
6. Decked box-beam,
7. Inverted-T precast slab,
8. NEXT F beam,
9. NEXT D beam,
10. Pi-girder, and
11. Precast modified beam in slab.

The *prefabricated modular systems* reviewed during this project include

1. The INVERSET™ system, and
2. The decked steel girder system.

The modular superstructure elements and systems, except the segmental box-beam section, are suitable for short-span bridges (i.e., 20 ft to 60 ft) and *up to* short-to-medium span bridges (i.e., 60 ft to 130 ft).

2.2.4.1 Precast Adjacent Box-Beam

Adjacent box-beam bridges have been designed and constructed very efficiently since the mid 1950s. This system has been implemented with and without a cast-in-place concrete deck. The longitudinal cracking at the surface reflecting from the beam joints prompted the inclusion of a 6 in. thick cast-in-place deck and increased transverse post-tensioning. However, reflective deck cracking persisted. Also, inspection of concealed girder faces is still a challenge. The adjacent box-beam depth range, span, and compressive strength of concrete used in Michigan are shown in Table 2-7 (MDOT-BDM 2013).

Table 2-7. Attributes of Adjacent Box-beams (Source: MDOT-BDM 2013)

	Depth range (in.)	Spans up to (ft)	28 day concrete strength (psi)
Box-beam (36 in. wide)	17 – 42	~120	5,000 – 7,000
Box-beam (48 in. wide)	21 – 60	~150	5,000 – 7,000

2.2.4.2 Trapezoidal Box Girder

The trapezoidal box girder was developed in 1998 for bridges up to short-to-medium spans. The girder was developed in two cross-sections: (1) a closed trapezoidal box and (2) an open section requiring a cast-in-place concrete deck. Considering the difficulty in the casting of a closed trapezoidal box section, an open-top was preferred (Badie et al. 1999). The attributes of an open-top trapezoidal box girder are shown in Table 2-8. Based on the data currently available, this particular section has not been specified for any project.

Table 2-8. Attributes of Trapezoidal Box Girders (Source: Badie et al. 1999)

	Depth range (in.)	Spans up to (ft)	28 day concrete strength (psi)
Trapezoidal box (totally closed)	23.5 – 31.5	~95	7,500
Trapezoidal box (open-top)	20 – 28	~86	9,000

2.2.4.3 Double-Tee and Decked Bulb-Tee Girders

The standard double-tee girder system has been available for many decades (PCI committee 1983). This system was originally developed for building and parking structure floor systems. Web thickness is the limiting factor in the prestressed girder design. Further, developing a moment connection detail at the flange with two layers of reinforcement is difficult due to limited flange thickness. Standard double-tee sections require a cast-in-place concrete deck. Hence, the use of these girders is limited to short-span bridges with low-traffic volume (Bergeron et al. 2005; Chung et al. 2008; Li 2010).

Due to the documented limitations of the standard double-tee girders, decked bulb-tee sections were developed (Shah et al. 2006; PCI 2011). Increased web thickness of decked bulb-tee sections accommodates post-tensioning to develop continuity details over the supports. This system is suitable for bridges up to short-to-medium span. As with any system, durability performance is a concern. The increased flange thickness of the decked bulb-tee section is suitable for developing durable flexure-shear transfer connection details (Graybeal 2010a; UDOT 2010b; CPMP 2011; Culmo 2011).

2.2.4.4 Inverted-T Precast Slab

Inverted-T precast slab, which also provides a platform for the construction and formwork for the cast-in-place concrete deck, is suitable for short-span bridges with underclearance issues. The limitation of this system is the additional time required to place and cure the cast-in-place concrete deck. The deck requires 7-day wet curing . Further, reflective deck cracking is a concern similar to observed on adjacent box-beam bridge decks.

A recent NCHRP project (French et al. 2011) investigated three aspects of the inverted-T precast slab: (1) stresses in the end zones of the precast section, (2) transverse reinforcement spacing at the connection, and (3) compatibility with AASHTO (2010) design specifications. The project concluded that AASHTO (2010) design specifications are not conservative for deep inverted-T sections (i.e., depth greater than 22in.), because more reinforcement is required than specified. This NCHRP project (French et al. 2011) developed a design guide for the inverted-T precast slab. However, the section with the incorporated new details has not been specified yet, so the reflective cracking cannot be assessed.

2.2.4.5 NEXT Beam

The NEXT F beam system requires an 8 in. thick cast-in-place concrete deck on the typical 4.5 in. thick flange. Both the NEXT F and D beams are suitable for short and up to short-to-medium span bridges with a cast-in-place deck. As with any prefabricated system, joint durability is a concern. However, the use of flexure-shear transfer connections may improve joint durability. These connections need further investigation.

2.2.4.6 Pi-Girder

The pi-girder is a shallow section with a thin deck. At the current state of practice, this system is costly with the use of proprietary materials and requiring special forms for casting.

2.2.4.7 Precast Modified Beam in Slab System

The precast modified beam in slab system has steel girders embedded in concrete to protect against corrosion. This system is suitable for short-span bridges in corrosive environments. Durability performance of the longitudinal joints needs to be investigated.

2.2.4.8 Decked Steel Girder System

The proprietary INVERSET™ system is designed for short and short-to-medium span bridges in non-corrosive environments. Even though the system is costlier than other systems, the specific manufacturing process precompresses the deck, which helps eliminate/reduce deck cracking. However, replacement or overlays to a precompressed deck is a challenge.

The non-proprietary decked steel girder module that was developed under the SHRP2 Project R04 (SHRP2 2012) utilizes conventional designs and manufacturing processes. Therefore, this system could be economically specified for short and short-to-medium span bridges in non-corrosive environments. The *weathering steel* could be utilized to address the corrosion issue. *Weathering steel* has chemical compounds which enable the surface to create a protective layer by weathering. This protective layer, if retained, reduces the progression of corrosion (CCI 2004). Yet low-rate steel corrosion is present (Tozier Ltd. 2011). *Weathering steel* is not corrosion proof; and if deicing salts are allowed to accumulate, the corrosion rate sharply increases. In salt laden environments, the protective layer may not stabilize, and corrosion can progress more rapidly (Tozier Ltd. 2011). In zones with deicing usage, *weathering steel* is not suitable, and steel must be protected using high-quality paint.

2.2.4.9 *Summary*

In summary, the bridge superstructures using trapezoidal box, double-tee, inverted-T, or NEXT F beams require a cast-in-place concrete deck; hence project duration is extended. Generally, cast-in-place concrete decks require 7-day wet curing.

Rectangular box-beams for adjacent box-beam bridges, decked bulb-tee beams, NEXT D beams, Pi-girders, INVERSET^{TM}, and decked steel girder systems do not require cast-in-place deck. Therefore, as the wearing surface on these elements or systems, a hot-mix asphalt (HMA) layer with a waterproofing membrane, epoxy overlay, or latex modified concrete overlay is considered by many states. There have been records of poor HMA overlay performance, which require further investigation. Adequately designed flexure-shear transfer details need to be implemented for improved durability. Moreover, suitable grout material is needed to prevent cracking or debonding at the interfaces. The majority of these elements or systems were specified in several projects, and performance data may be available with respective DOTs.

2.2.5 Substructure Elements

Widely used prefabricated substructure elements are precast pier caps and bent caps (Ralls et al. 2004). Following the charge by the Technology Implementation Group (TIG) of AASHTO to promote further development of PBES, precast columns, precast segmental abutment stems, and precast pile caps were developed and implemented in several projects. Highway agencies in the U.S. designed and constructed innovative structural systems along with conducting research to standardize these substructure elements for high traffic-volume bridges (Matsumoto et al. 2001; Billington et al. 2001; Ralls et al. 2004; Restrepo et al. 2011). A summary of properties, benefits, and limitations of these prefabricated substructure elements is given in Appendix A. A majority of the prefabricated substructure elements, which were specified in ABC projects, are not standardized. A complete set of prefabricated substructure elements, as listed below, are available and can be specified for projects with changes to fit the project requirements. Based on the site constraints and cost, alternatives can be selected for a particular application.

The prefabricated substructure elements discussed in Appendix A include

1. Precast abutment stem/wall,
2. Precast pile cap,
3. Precast columns,
4. Precast segmental columns,
5. Precast pier/bent cap, and
6. Precast footings.

A detailed discussion of each of these elements is provided in Appendix A. Benefits and limitations of selected elements are provided in Chapter 5.

2.2.5.1 Reduced-Weight Bent/Pier Cap

As discussed in Section 2.2.1, the weight of prefabricated elements for transport and placement is a limitation. Generally, substructure elements are heavier than the superstructure elements. A bent cap is one such element that created many challenges during placement (Attanayake et al. 2012). Various bent and pier cap configurations and details are implemented to reduce the weight (Culmo 2009; Restrepo et al. 2011; Billington et al. 1999; Klaiber et al. 2009). The details of each of these elements are provided in Appendix A. The reduced-weight configurations presented in Appendix A include

1. Inverted U-section (Culmo 2009),
2. Precast reinforced bent cap with cavities (Culmo 2009),
3. Tapered cantilever section (Restrepo et al. 2011),
4. Precast inverted T-section (Billington et al. 1999),
5. Steel-concrete composite section (Klaiber et al. 2009), and
6. Precast segmental columns with precast templates (Billington et al. 1999).

Benefits and limitations of these selected elements are provided in Chapter 5.

2.2.6 Miscellaneous

2.2.6.1 Additional Accelerated Bridge Construction Technologies

Construction technologies used in ABC such as SPMT and slide-in are presented in Appendix A. A list of attributes, limitations, and details of the selected projects where these technologies are specified is also presented.

2.2.6.2 High Performance Concrete

High performance concrete has been specified and used in ABC projects. Mix designs, documented strengths, and other mix parameters are given in Appendix A. As with any other materials, prior to specifying their use, trial mixes and performance testing should be required to evaluate the material for the specific application. Additionally, documenting challenges and lessons learned during mixing, transporting, placing, handling, and curing will be helpful.

2.3 CONNECTION DETAILS

Prefabricated bridge elements and systems are discussed in Section 2.2. Even though elements and systems are prefabricated, continuity and the load transfer mechanisms are established by field cast connections. The connections are classified as superstructure and substructure. The superstructure and substructure connections are further classified into sub-categories as shown Figure 2–2. The details under each category are provided in Appendix B.

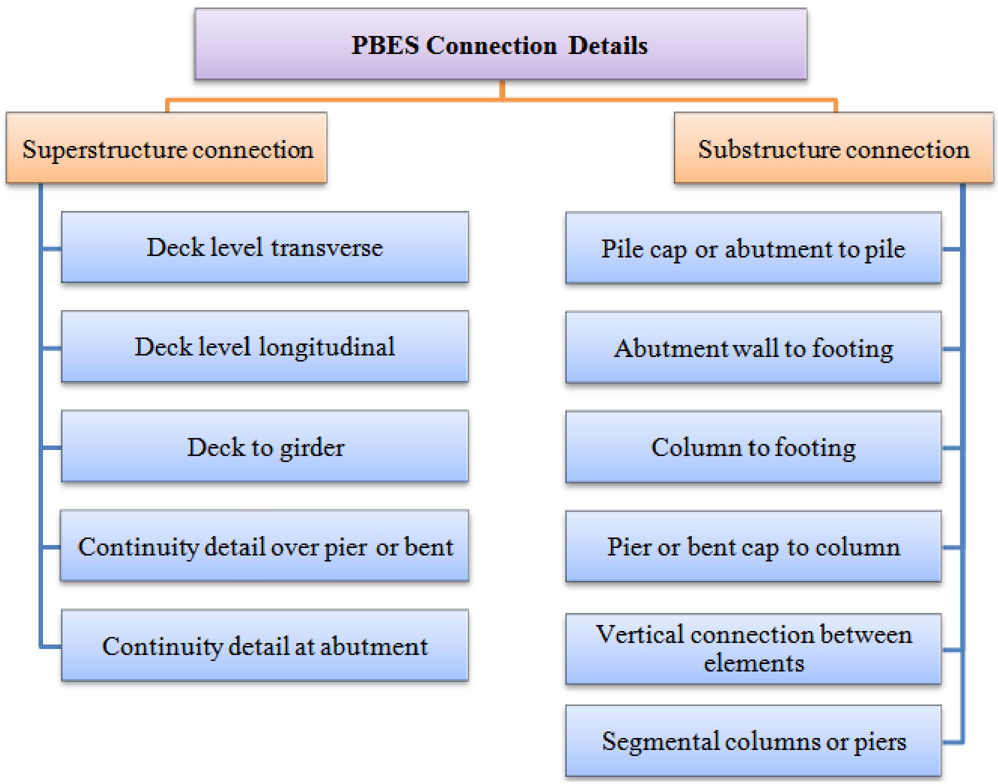


Figure 2–2. Classification of prefabricated element connection details

The durability performance of PBES is a critical consideration. The PBES durability is primarily controlled by the performance of field cast connections. In order to assure durability under the exposure conditions of Michigan, transverse and longitudinal connections of bridge superstructures and other connections used in bridge substructures are required to transfer moment and shear as well as tension, compression and torsion. Crack width limitations would be the means of quantifying durability in the ABC design process.

The post-tensioning or specific details used for connecting substructure elements in regions with high seismic activities are not included in Appendix B. The additional information presented in the Appendix includes review comments and projects where the specific details are implemented.

Durability performance of ABC methodologies that are currently being implemented were reviewed and presented in Chapter 3. In addition to the exposure conditions, other factors that need to be considered are (1) load transfer mechanism, (2) constructability, and (3) connection dimensions and tolerances (to ensure construction quality).

The connection details presented in Appendix B are reviewed considering load transfer mechanism, constructability, and durability. The most suitable details for implementation in Michigan are presented in Chapter 5.

2.3.1 Load Transfer Mechanism

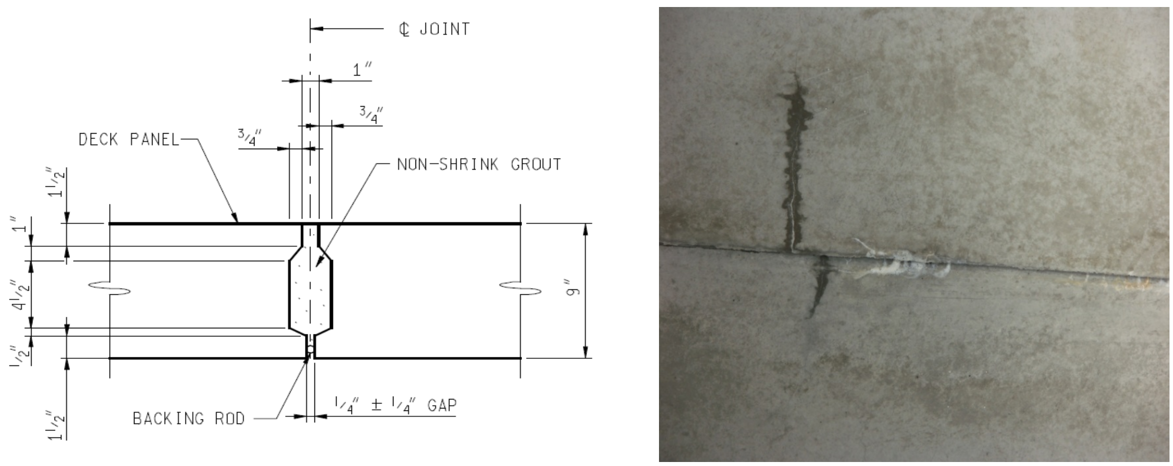
Connections are often required to transfer moment, shear, tension, compression, and torsion forces. Longitudinal and transverse connections that can accommodate moment and shear transfer under all loading conditions are recommended for Michigan. A typical flexure-shear transfer connection can be developed with two layers of reinforcement (French et al. 2011). However, unreinforced joints can also be designed to transmit moment and shear with appropriate post-tensioning design.

2.3.2 Constructability

Constructability is a critical aspect that needs special attention when selecting connection details. Inadequate details that lead to constructability issues compromise durability as well increase or delay the project schedule. Constructability is a broad topic, and a detailed discussion is provided in Section 2.6. However, constructability issues related to connection details are also discussed within this chapter, where applicable. Potential constructability issues need to be identified during design. In that regard, issues related to constructability also need to be documented in post-construction reports for assuring continuous improvement.

2.3.3 Connection Dimensions and Tolerances

Adequate space needs to be provided for field welding, coupling post-tension tendons, placing necessary rebars, and grouting. In addition, component tolerances need to be specified to avoid any unaccounted load transfer through a connection. As an example, Figure 2–3a shows a typical transverse connection detail of a full-depth deck panel system that allows the panels to be placed against each other (a tolerance of ¼ in. +/− ¼ in. can result in zero gap between the panels as shown in Figure 2–3b). Hence, the grout is not compressed, and the load is transferred through contact points of the panels under post-tensioning, causing splitting cracks (Ulku et al. (2011)).



(a) (b)

Figure 2–3. (a) Typical transverse connection detail of a full-depth deck panel system, and (b) zero tolerance leading to cracking at panels after post-tensioning

The connection needs to be adequately detailed to assure construction quality. When other alternatives are available, the connections with confined space, narrow access, and sharp corners or edges need to be avoided to minimize potential for developing voids during grouting. When a confined grouted pocket is to be filled, a proper air vent system is necessary. Connections with large cavities cannot be filled with neat grouts and require extended grouts or special mixes. However, slow strength development of extended grouts and other special mixes may slow the construction. Additionally, fit issues with connection reinforcement are reported, and those required field bending to make the components fit (Figure 2–4). Field bending is not desirable and can also damage epoxy coating.

The process of specifying material for the grout pockets, application procedures, and ensuring quality of grouted connections are discussed in Section 2.4. To assure the quality of field cast connections, component tolerances and cavity dimensions need to be designed considering the dimensional growth that may limit the space provided for adjacent component placement. This process must also specify grout or special mixes at connections, material application procedures, constructability, project schedule, and construction quality assurance.

Figure 2–4. Field bending of reinforcement to overcome space issues (Source: http://www.flickr.com/photos/iowadot)

2.4 GROUT MATERIALS FOR CONNECTIONS AND APPLICATION PROCEDURES

2.4.1 Literature Review Objective

The objective of this review is to develop a library of grout and special mixes that are suitable for precast concrete component connections. Connection examples are shear keys, transverse connections, and haunches, column to footing, and pile and abutment stem. Grout types and properties are compiled from the manufacturer data sheets and available laboratory or field test data are presented for comparison purposes. Further, special mixes discussed in literature are summarized. The properties considered in this study are as follows:

- Compressive strength at 1, 3, 7, 28 days,
- Freeze/thaw resistance,
- Non-shrink properties (Change in height/volume as per ASTM C1090),
- Initial setting time,
- Grout pocket dimensions,
- Working temperature range,

- Site constraints and limitations, and
- Curing requirements.

2.4.2 Commercially Available Grouts

A list of commercially available grouts and associated properties are presented in Table 2-9 and Table 2-10.

Table 2-9. Summary of Commercial Grout Properties

		Set 45[+]	EUCO SPEED MP*	SS Mortar	Masterflow 928	747 Rapid Setting Grout	S Grout	Sonogrout 10k	SikaGrout 212	Five Star Grout	Construction Grout
Compressive strength (ksi) (min. 5.0 ksi @ 24 hrs. as per AASHTO 2010)	1 day	6.0	6.0	4.0	4.0	7.6	3.5	1.6	3.5	4.0	1.5
	3 days	7.0	6.5	5.4	5.0	8.2	5.0	3.8	-	5.5	5.0
	7 days	-	7.0	7.0	6.7	10.6	6.0	5.1	5.7	6.5	6.0
	28 days	8.5	7.5	11.0	8.0	12.6	8.0	6.2	6.2	8.0	7.0
Initial setting time (min)		15	12	30	3 hrs	26	45	3 hrs	5 hrs	45	6 hrs
Fill depth/thickness for neat grout (in.)	Min	0.5	0.5	-	1	-	-	0.5	0.5	1	-
	Max	2	1	-	6	2	2	2	2	6	-
Working temperature (^oF)	Min	-	-	50	45	40	40	65	45	40	50
	Max	85	85	90	90	85	85	75	70	90	90
Freeze/thaw resistant		YES	YES	-	YES	-	YES	YES	YES	-	-
Change in Height/ Volume (as per ASTM C1090)	28 days	-	-	-	0.06%	-	-	0.04%	0.03%	0.30%	0.08%
Extend with aggregate		-	YES	-	YES	YES	YES	YES	YES	YES	YES

* EUCO SPEED MP HW shall be used for temperature above 85 ^{0}F and the properties are same as the EUCO SPEED MP

[+] SET 45 HW shall be used for temperature above 85 ^{0}F and the properties are same as the SET 45. SET 45 HW can be extended with aggregates for larger fill depth.

Table 2-10. Summary of Commercial Grout Properties

		Sure-Grip	Speccrete Superb Grout 611	CRYSTEX	1107 Advantage Grout	Conspec 100	DURAGROUT	Enduro 50	Multi-Purpose Grout	PRO GROUT 90	Quickrete Non-shrink Precision Grout	Meadows Sealtight 588-10K
Compressive strength (ksi) (min. 5.0 ksi at 24 hrs. as per AASHTO 2010)	1 day	5.0	5.0	4.6	2.5	3.8	2.3	-	2.1	4.7	3.0	4.5
	3 days	-	7.0	6.5	5.0	5.4	-	4.5	4.6	5.6	9.0	5.5
	7 days	8.0	8.5	8.2	6.0	7.7	7.0	5.8	6.7	6.6	9.5	6.5
	28 days	10.0	11.0	10.2	8.0	8.4	8.3	8.0	9.3	7.8	12.5	9.2
Initial setting time (min)		30	70	60	30	35	30	-	4 hrs	4 hrs	25	3 hrs
Fill depth/thickness for neat grout (in.)	Min	-	-	-	-	-	-	-	-	-	-	-
	Max	3	2	4	3	3	-	3	3	-	-	-
Working temperature (°F)	Min	45	45	45	45	40	40	40	45	-	50	-
	Max	90	70	-	90	-	-	-	90	-	90	-
Freeze/thaw resistant		-	-	-	-	-	-	-	-	YES	-	-
Change in Height/ Volume (as per ASTM C1090)	28 days	-	0.03%	0.02%	-	0.03%	0.03%	0.03%	0.13%	-	0.20%	0.14%
Extend with aggregate		YES	YES	YES	YES	YES	YES	YES	YES	-	-	YES

2.4.2.1 Compressive Strength

Non-shrink and high early strength grout are appropriate for precast component connections in order to prevent shrinkage, cracking and debonding at the grout-component interface while expediting the construction process. The compressive strength requirements are stipulated in the AASHTO (2010), ASTMs, and agency specific specifications (Table 2-11).

Compressive strength of the grouts is categorized based on three levels of workability: plastic, flowable, and fluid. The compressive strengths given in Table 2-9 and Table 2-10 correspond to the flowable consistency that is required for most of the grouting operations. According to the manufacturer data sheet, different consistency is achieved by increasing the water content. Increasing water content to reach fluid consistency will result in the lowest compressive strength compared to flowable and plastic. However, the strength reduction can be overcome by utilizing water reducing admixtures to achieve required consistency without increasing water content.

Considering Michigan standard specifications and ASTM C1107 specification requirements, almost all of the available materials qualify for the grouting operation. Whereas, only three cementitious grout materials meet the requirements under AASHTO (2010) stipulations. These are *747 Rapid Setting Grout*, *Sure-Grip® High Performance Grout*, and *Speccrete® Superb Grout* with the 1-day strength for flowable consistency of 7.6 ksi, 5.0 ksi, and 5.0 ksi, respectively. Remaining cementitious grouts gain a strength of approximately 5.0 ksi in 3 days, which does not adhere to the AASHTO (2010) stipulations.

Table 2-11. General Requirement for Grouting Materials

Properties	Value	Specification
Compressive strength	5.0 ksi @ 1 days	AASHTO (2010)
	1.0 ksi @ 1 day 2.5 ksi @ 3 days 3.5 ksi @ 7 days 5.0 ksi @ 28 days	ASTM C1107 (performance requirement)
Early age height change [+]	Maximum @ Final set: + 4.0%	ASTM C1107 and C827
Height change of moist cured hardened grout at 1, 3, 14 and 28 days [++]	Maximum: +0.3% Minimum: 0.0%	ASTM C1107 and C1090

[+]Early age height change is the measurement of change in height from the time of placement until the specimen is hardened, and it is measured as per ASTM C827

[++] Height change is either increase or decrease in the vertical dimension of the test specimen, and its measure as per ASTM C1090

2.4.2.2 Freeze/Thaw Resistance

Grout materials with freeze/thaw resistant performance characteristics are required in Michigan. From the list provided in Table 2-9 and Table 2-10, only seven are suitable for freeze/thaw exposure conditions:

- Set 45 or Set 45 HW,
- EUCO-SPEED MP or EUCO-SPEED MP HW,
- Masterflow 928,
- S Grout,
- Sonogrout 10k,
- SikaGrout 212, and
- PRO GROUT 90.

The grouts listed above are suitable for freeze/thaw exposure. Yet, only magnesium phosphate grouts, such as Set 45 and EUCO-SPEED MP, can develop the strength required by the AASHTO (2010). The AASHTO requirement for the grout is to achieve 5 ksi in 24 hours.

2.4.2.3 Non-Shrink Grout

The non-shrink property of the grout is important to assure the durability performance of the joint. This is because shrinkage may result in grout cracking and grout-component interface debonding. Abating and control of cracking and debonding is necessary in specific climate zones where deicing salts are used or the structures are in chloride rich environments. Cracks allow rapid penetration of chloride laden surface water resulting in initiating and accelerating the corrosion of reinforcing steel. Non-shrink grouts exhibit expansion after the initial hardening phase by the presence of shrinkage control additives such as g*as generating (e.g. Al powder)* and *air release* in the mix (Culmo 2009). Air release occurs when the additive reacts with water to release entrapped air and trigger expansion of the grout (Culmo 2009). This additive is incorporated into *Five Star® Grout* to control shrinkage. For hydraulic-cement grout to be qualified as a non-shrink grout, the maximum early age expansion and hardened state expansion must meet ASTM C1107 requirements on maximum and minimum height change (Table 2-11).

2.4.2.4 Initial Setting Time

Magnesium ammonium phosphate grouts achieves a high early strength of up to 5.0 ksi within a very short period of 6 hours after initial setting, and they are consequently suitable for rapid bridge construction. During the strength gain process, these grouts generate high temperature. Also, rapid setting leaves very little time for transport, handling, and placing compared to cementitious grouts. This also poses difficulty in grouting larger cavities. As an example, Set 45 and EUCO-SPEED MP have initial setting times of 15 and 12 min., respectively (Table 2-9). Cementitious grout materials allow sufficient working time ranging from 30 min. to 6 hours, thus they are suitable for filling large cavities. However, strength gain is low and requires longer curing duration to attain desired properties. The setting time and rate of strength gain is controlled by exposure conditions. Setting time and strength gain test data needs to be provided in order to validate the suitability of using such material for a specific project.

2.4.2.5 Grout Pocket Dimensions

The largest dimension of a grout pocket will define the criticality of shrinkage or heat of hydration properties. The largest fill dimension is sometimes referred to as the fill depth. The fill depth provided in Table 2-9 and Table 2-10 is for neat grout. Grouts can be extended with sand or pea gravel to fill large pockets. Extended grout properties such as consistency, workability, strength, and setting time are not the same as the data presented in manufacturers' data sheets. As a result, the rate of strength gain of extended grouts will be lower than values given in the manufacturers' data sheet. As per the information provided in the manufacturers' data sheet, certain grout types such as magnesium phosphate based EUCO-SPEED MP grout is limited to a maximum fill depth of 8 in. This fill depth limitation is valid even for extended grout.

2.4.2.6 Working Temperature Range

The material properties are obtained under specific exposure conditions as stipulated in the standard specifications. For most of the grouts, the compressive strength provided in the material data sheet is for a particular working temperature of 73 ^{0}F. The recommended temperature range for application depends on grout type (Table 2-9 and Table 2-10). As an example, in Table 2-10 the application temperature range of SS Mortar is given as 50 – 90 ^{0}F, but Set 45 shows only an upper limit of 85 ^{0}F. Magnesium phosphate grout has special types for hot weather grouting such as Set 45 HW and EUCOSPEED MP HW. Therefore, these grouts can be applied in ambient temperature greater than 85 ^{0}F. Another example is the grouting with SikaGrout, which has a working temperature range of 40 ^{o}F to 70 ^{o}F. It is advised to follow practices similar to hot and cold weather concreting when this grout is used under temperature conditions beyond the specified range. It is recommended to add cold water for a temperature greater than 70 ^{o}F and warm water for a temperature less than 40 ^{0}F. These observations further strengthen the need for testing under specific exposure conditions and compiling data available in order to develop a knowledge base.

2.4.2.7 Curing Requirements

The cementitious grouts require wet curing for at least 1 day and further curing or prevention of moisture loss for at least 3 days. Also, some grouts do not require wet curing. As an example, magnesium phosphate grouts, as recommended in the material data sheet, require protection only for moisture loss for at least 3 hours following application. Another example is the UHPC, which also requires protection of moisture loss for about three days. UHPC is often protected with an insulated board cover such as plywood (Perry et al. 2010). This requirement needs to be considered at sites with time constraints for on-site construction.

Based on the information provided in Table 2-9 and Table 2-10 and discussions provided in the above sections, it is recommended that selection of grout for a specific connection needs to be considered at the design stage. In addition to the fresh and hardened grout properties provided by the manufacturers that are listed in Table 2-9 and Table 2-10, additional third party test data is compiled and presented in the section below.

2.4.3 Commercial Grout Properties Documented in the Literature

Grout properties provided in the material data sheet are suitable for preliminary selection of the materials for a specific application. Yet, the data presented by manufacturers is for neat grout. In most cases, use of extended grout is required due to the size of the grout pocket. For this reason, documenting the variability due to exposure, mixing procedures, and grout constituents is required. Experimental data for four different commercially available cementitious and magnesium phosphate based grouts is presented in Table 2-12. Scholz et al. (2007) conducted testing for neat grout and extended grout to evaluate the strength development. The data shows a significant reduction in compressive strength for extended grouts compared to the neat grouts.

Table 2-12. Experimental Data for Neat and Extended Grouts (Source: Scholz et al. 2007)

		Neat Grout				Extended grout			
		ThoRoc 10-60 Rapid Mortar	SikaQuick 2500	Five Star Highway Patch	Set 45 Hot Weather	ThoRoc 10-60 Rapid Mortar	SikaQuick 2500	Five Star Highway Patch	Set 45 Hot Weather
Aggregate Extension, % by weight		-	-	-	-	50	50	80	60
Compressive strength per ASTM C109 (psi)	1 hr.	2700	1700	910	420	1860	1020	-	-
	2 hrs.	3030	2250	2810	2050	2370	1170	2730	230
	1 day	5210	3540	5080	4930	3150	1900	4490	2650
	7 days	6380	4710	5820	4930	5040	2550	5440	4180
Shrinkage per ASTM C157 (%)	28 days	0.076	0.080	0.029	0.034	0.064	0.089	0.036	0.018
Initial set time (min)		16	24	30	44	19	29	26	35

Compressive strength data documented in literature are compared to the materials data sheets shown in Table 2-13. In the majority of cases, the compressive strength presented in the literature is significantly different than those presented in the material data sheets by the manufacturer. This highlights the importance of conducting mock-up testing in order to evaluate the application procedures and material behavior under anticipated exposure conditions. Also, compiling this test data will help designers' understand the application limitations and property variability of grout. This will, perhaps, allow designers the opportunity to modify connection details to be compatible with the material selected for the particular application.

Table 2-13. Comparison of Manufacturer Data and Laboratory Test Data

	1 day Strength (ksi)				3 days Strength (ksi)			7 days Strength (ksi)				
	a	b	C	d	a	b	c	a	b	c	d	e
Set 45	6.0	-	3.8	-	7.0	-	4.3	-	-	5.5	-	-
Set 45 HW	6.0	6.3	-	4.9	7.0	7.4	-	-	8.3	-	4.9	7.3
EUCO SPEED MP	6.0	5.6	-	-	6.5	6.3	-	7.0	6.9	-	-	-
Set Grout	-	-	2.8	-	3.0	-	5.1	5.0	-	6.3	-	5.9
ThoRoc 10-60	6.5	-	-	5.2	-	-	-	-	-	-	6.4	-

a. Manufacturer data sheet
b. Oesterle and El-Remaily 2009
c. Issa et al. 2003
d. Scholz et al. 2007
e. Gulyas et al. 1995

2.4.4 Non-Commercial Grout and Mortar

A review of literature was performed to document potential non-proprietary grouts and special mixes.

2.4.4.1 Ultra High Performance Concrete (UHPC)

Ultra high performance concrete is often suitable for joints in precast construction. As an example, UHPC was successfully used by the Iowa DOT for grouting of dowel pockets at the longitudinal connection between the pi-girders (Figure 2–5). UHPC contains a premix of silica fume, ground quartz, sand, and cement with a mix of brass coated high tensile steel fibers. The fiber diameter is 0.008 in. with a length of 0.55 in. A high range water reducing admixture is added to improve the workability of UHPC. The characteristic design strength of 14.5 ksi is achieved at 96 hours (4 days) after casting. During this period of strength development, ambient vibrations adversely affect strength development by perhaps disturbing fiber orientation. The exposed surface of the UHPC connection often requires grinding because there is a tendency of steel fibers protruding out of the surface (Perry et al. 2010).

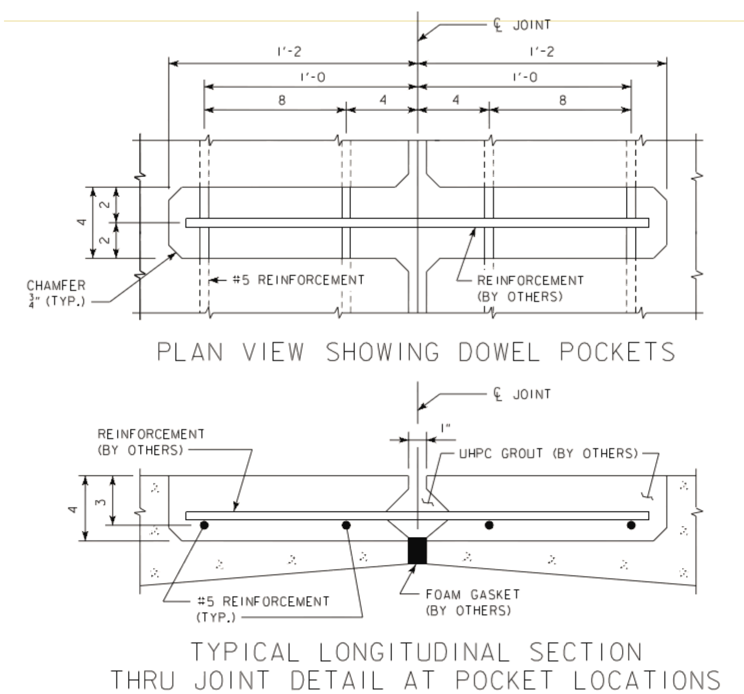


Figure 2–5. UHPC grouted dowel pockets in UHPC girders (Source: Bierwagen 2009)

An example of typical field-cast UHPC materials mix design and their properties is given in Table 2-14 and Table 2-15.

Table 2-14. UHPC Mix Design (Source: Graybeal 2010a)

Material	Quantity (lb/yd^3)
Portland cement	1200
Fine sand	1720
Silica fume	390
Ground quartz	355
Superplasticizer	51
Steel fibers	263
Water	218

Table 2-15. UHPC Material Properties (Source: Graybeal 2010a)

Material characteristic	Average results
Density	155 lb/yd3
Compressive strength (ASTM C39; 28-day strength)	18.3 ksi
Modulus of elasticity (ASTM C469); 28-day modulus)	6200 ksi
Long-term shrinkage (ASTM C157; initial reading after set)	555 microstrain
Total shrinkage (Embedded vibrating wire gage)	790 microstrain
Freeze-thaw resistance (ASTM C666A; 600 cycles)	RDM = 112%

2.4.4.2 Cement Mortar Specified in Michigan

Type R-2 Grout class is often specified for the shear keys of side-by-side box-beams. (Aktan et al. 2009). Various R-2 mixes were evaluated in an earlier study (Table 2-16). Also, a set of samples was collected from an MDOT bridge project that is labeled as BB in Table 2-16. The mix design of the grout obtained from the MDOT bridge project is also given in Table 2-17. Remaining samples were prepared based on the proportions given in the MDOT Standard Specification for Constructions. As seen in Table 2-18, R-2 grout strength development is slow and not suitable for most accelerated construction projects.

Table 2-16. Type R-2 Grout Description (Source: Aktan et al. 2009)

Mix ID	Description
BB	Type 1 Portland cement. Provided by Consumer's Concrete and used in Oakland over I-94 bridge
BBA	Type 1 Portland cement – Laboratory mix
BBM	Type M masonry cement - Laboratory mix
BBN	Type 1 Portland cement and Type N masonry cement – Laboratory mix
BBS	Type 1 Portland cement and Type S hydrated lime - Laboratory mix

Table 2-17. Type R-2 Grout Mix Design

Mix ID	Materials	Quantity
BB	Portland cement ASTM C150 (Type I), lbs	930
	Fine aggregate MDOT (#4 - #100), lbs	1956
	Water, lbs (gal)	416 (49.8 gal.)
	Air entraining ASTM C260, oz	46.0
BBA	Portland cement (Type I), lbs	930
	Fine aggregate (#4 - #100), lbs	1996
	Water, lbs	415
BBM	Masonry cement (Type M), lbs	930
	Fine aggregate (#4 - #100), lbs	2137
	Water, lbs	415
BBN	Portland cement (Type I), lbs	468
	Masonry cement (Type N), lbs	349
	Fine aggregate (#4 - #100), lbs	1991
	Water, lbs	415
BBS	Portland cement (Type I), lbs	828
	Hydrated lime, lbs	75
	Fine aggregate (#4 - #100), lbs	2016
	Water, lbs	415

Table 2-18. Compressive Strength of Type-R-2 Grout (Source: Aktan et al. 2009)

	Strength (psi)				
	BB	BBA	BBM	BBN	BBS
Age (Days)					
3	3,730	2,693	2,125	1,916	2,470
7	3,651	3,668	2,358	2,693	2,899
14	4,385	4,256	2,646	3,377	3,403
28	4,859	4,309	2,677	3,680	3,626

2.4.4.3 *High Performance Concrete*

Five different high performance concrete mixes were developed for the NCHRP 10-71 project (Table 2-19). These mixes are recommended for closure pours at longitudinal connections between the flanges of deck bulb tee sections (DBTs) and between precast panels. The mixes require a 7-day wet curing. Only three out of five mixes were tested, and the 7-day compressive strengths were recorded as 6.5 ksi, 4.1 ksi, and 5.1 ksi. High performance concrete may not be suitable for accelerated bridge construction projects due to slower strength development compared to commercial non-shrink grout materials.

Table 2-19. Mix Design for High Performance Concrete (Source: French et al. 2011)

		Mix-1	Mix-2	Mix-3	Mix-4		Mix-5	
w/c Ratio		0.31	0.35	0.31	0.32		0.35	
Cement type		I	I	II	I/II		I/II	
Cement Quantity, lb/yd^3		750	474	490	563		431	
Fly Ash Type C Quantity, lb/yd^3		75	221	210	75		58	
Slag Quantity, lb/yd^3					113		86	
Fine Aggregate, lb/yd^3		1400	1303	1365	1161		1308	
Coarse Aggregate Maximum Size, in		0.5	1	1.25	1.5	0.5	1.5	0.5
Coarse Aggregate Quantity, lb/yd^3		1400	1811	1900	1530	270	1520	380
Air Entrainment, fl oz/yd^3		5	-	3.1	3		2.3	
Water reducer, fl oz/yd^3		30	-	-	-		-	
Retarder, fl oz/yd^3		-	22	28	-		-	
High-Range Water Reducer, fl oz/yd^3		135	122	156	60		46	
Shrinkage Reducing Admixture, fl oz/yd^3		-	-	-	32		24.7	
Compressive strength (psi) per ASTM C39 Modified	7-day	6494	-	-	4112		5058	
	28-day	8895	-	-	5269		7309	

2.4.5 Grouting Operation

In addition to exposure conditions and strength requirements, connection details should also be considered in specifying the grouting. The connection geometry is critical in specifying the grout. This is because the fill depth of most grouts is limited, and grouts require extending for larger volume. Reinforcement details are also important for proper workability and sufficient consolidation. For a particular application, conducting mock-up tests on potential grouts would be useful to evaluate the mixing and placement procedures, as well as strength development under anticipated exposure conditions. Mock-up tests can also be useful in training the grouting crew.

All grouting operations require wetting the precast element surfaces to attain saturated-surface-dry condition before placing the grout or special concrete. Generally, wetting of the component surfaces should start at least 4 hours before the grout placement. However, most grout material data sheets recommend a wetting process to start 24 hours before placement.

Surface preparation is important and is a critical factor for the bonding between the grout and the precast elements. The surface should be cleaned from any foreign materials, and the joints should be roughened or mechanically abraded to allow forming a mechanical bond between the grout and the precast elements. Reinforcement at the joints should be thoroughly cleaned and free from rust. Cementitious grout with non-shrink properties is often recommended in precast construction due to assumed material compatibility of the grout with precast elements. The material data sheet for magnesium phosphate grouts indicates the need for special surface preparation to enhance bonding at the grout - component interface. Once the surface is prepared, the magnesium phosphate grout will provide desired bonding properties as per the manufacturer data sheet.

Another factor that promotes grout cracking and failure at the grout - component interface bond is ambient vibrations propagating from traffic or other construction operations. Some grouts, mostly those requiring longer setting time, are sensitive to the structural vibration. One such example is ultra-high performance concrete (UHPC) (Perry et al. 2010). Vibration impact should be considered for staged construction.

Grout placement methods include dry packing, gravity flow (pouring), and pumping. Dry packing is commonly used for shear keys. Grout mixed at flowable and fluid consistency can be pumped into tight spaces and sharp corners of the joint cavities (Figure 2–6, Figure 2–7). The pumping process requires a leak proof formwork that can withstand the pressure. Joints are normally sealed with a foam backer rod, which is flexible but not sufficient for pressure grouting operation.

Figure 2–6. Grouting adjacent box-beam shear keys using type R-2 grout (Oakland Drive over I-94, MI)

Figure 2–7. Pumping W.R. Meadows Sealight CG-86 non-shrink grout (Source: Oliva et al. 2007)

Figure 2–8. Grouting of full-depth deck panel connections (Source: Courtesy of MDOT)

Connections with narrow access for grouting should have a vent pipe to prevent entrapped air voids. Care is required while grouting connection cavities with sharp edges and corners (Figure 2–9).

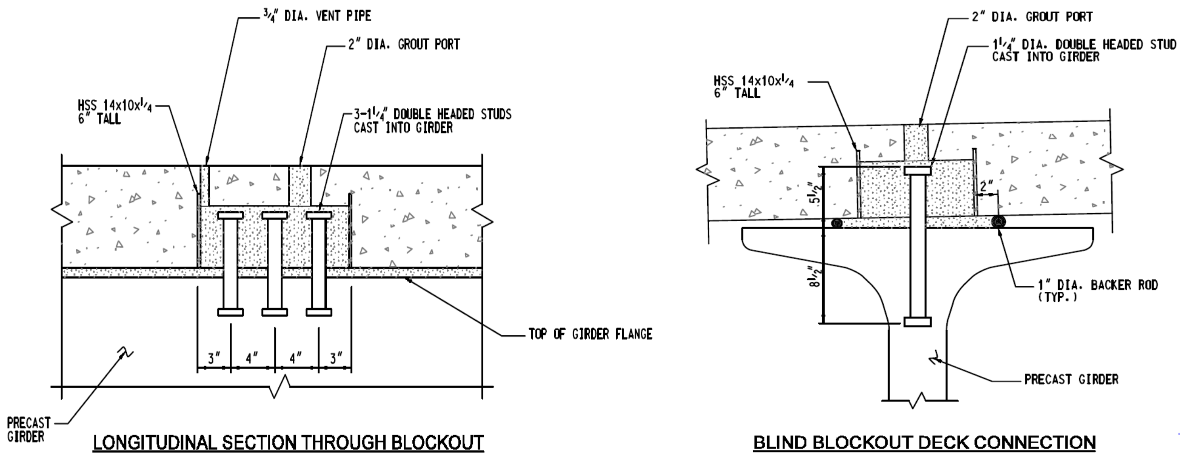


Figure 2–9. Joint details of panel-to-prestressed concrete I-girder connection with confined space, sharp edges, and corners (Source: Culmo 2009)

As indicated earlier, properties of neat grout may be different than those stated in the data sheets. Hence, the following steps will provide evidence to assure the compatibility of the material for a specific connection, application procedure, exposure conditions, curing requirements, and project schedule: (1) compiling available data of extended and neat grout properties, (2) evaluating grout properties through appropriate testing under various exposure conditions, and (3) evaluating grout application procedures using mock-up specimens.

2.4.6 Summary and Conclusions

A grout can be specified based on properties such as compressive strength, freeze/thaw resistance, non-shrink properties, initial setting time or working time, grout pocket dimensions, working temperature range, application procedures and limitations, and curing requirements. After identifying the grout based on the information in the material data sheet, it should be tested and evaluated for the particular field application before implementation.

2.5 ACCELERATED BRIDGE CONSTRUCTION AND DEMOLITION METHODS AND EQUIPMENTS

2.5.1 Accelerated Bridge Construction Methods and Equipments

Literature documents numerous innovative and creative techniques used by various agencies to accelerate the bridge construction process. These techniques have been developed through the process of exchanging ideas between design, fabrication, and construction teams. The "International technology scanning" program was a joint effort by the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO), in 2002, to review and identify global innovations for leveraging to the U.S. transportation infrastructure industry. Several innovations were identified and implemented by contractors to accelerate the construction process. Ralls et al. (2005) reports the popular techniques used for accelerated construction in various parts of the world as these:

- Vertical lifting of prefabricated components with large cranes to assemble the bridge system,
- Longitudinal incremental launching of bridges above existing highways,
- Moving a complete built system with a series of vehicles known as Self-Propelled Modular Transporters (SPMTs),
- Moving a complete built system by horizontal skidding or sliding into place,
- Building bridges alongside an existing roadway and rotating them into place, and
- Vertical lifting of the complete built system and placing into required alignment.

Several construction projects (case studies) that are listed below were reviewed to understand and document these technologies.

- Oakland Eastbound I-580 Connector in San Francisco Bay Area, California
- Russian River Bridge in Geyserville, California
- San Francisco Yerba Buena Island Viaduct in Oakland, California
- I-70 over Eagle Canyon, Utah
- I-215; 4500 South Bridge in Salt Lake City, Utah
- Five bridges on OR-38 between Drain and Elkton, Oregon

- U.S. 15/29 over Broad run in Prince William County, Virginia
- State highway 86 over Mitchell Gulch in Douglas, Colorado
- I-80 State street to 1300 east in Salt Lake City, Utah
- Mill Street Bridge, New Hampshire
- Tucker Bridge in Utah
- Lewis and Clark Bridge between Washington and Oregon
- Sam White Bridge in American Fork, Utah
- Parkview Bridge, in Kalamazoo, Michigan
- 120th Street Bridge in Boone County, Iowa
- Route 99/120 separation bridge in Manteca, California
- Skyline Drive Bridge over West Dodge Road in Nebraska
- I-215 East Bridge over 3760 South in Utah
- MD Route 24 Bridge over Deer Creek in Maryland
- I-40 Bridge in southeastern California
- Replacement of a bridge in a high seismic zone of western Washington State
- Fast 14 Project in Medford, Massachusetts
- M-25 over the White River in Huron County, Michigan

2.5.2 Accelerated Bridge Demolition Methods and Equipments

Numerous types of demolition techniques are available. They are demolition by machine, by chemical agents, and by the use of hand-held tools, etc. (BSI 2000). Nevertheless, some of these techniques may pose difficulties in practice due to complicated site conditions and various constraints on noise, dust, and vibration. According to Abudayyeh et al. (1998), the parameters that need to be considered in defining the bridge demolition techniques include the following:

- Location and accessibility,
- Shape and size of the structure,
- Time constraints,
- Transportation consideration,
- Financial constraint,

- Recycling consideration,
- Environmental consideration,
- Health and safety,
- Client specification,
- Stability of structure,
- Presence of hazardous material, and
- Degree of confinement.

New techniques are being proposed for bridge demolition. The following sections will highlight some of the new developments and will list the techniques that are currently in practice for bridge demolition.

2.5.2.1 Accelerated Bridge Demolition Techniques

New demolition methods and equipment are now available for a fast and safe demolition of bridges. One example is the Self Propelled Modular Transportation (SPMT) systems that are used for transporting massive objects such as bridges and buildings (Ardani et al. 2009). The Hyspec (hydraulic, self-powered, and electronically controlled) is one SPMT system that has been tested for removal and transport of the Warren Farm bridge structure to an adjacent set-down area without creating any debris (Anumba et al. 2003). The Warren Farm Bridge carrying over M1, located in Nottinghamshire, UK, was successfully removed using a Hyspec SPMT system (Anumba et al. 2003). Before the closure of the bridge, the exact location of lifting and maneuver positions were defined on M1. The computer on the Hyspec system was programmed with the accurate coordinates of adjacent set-down area before the closure of M1. A non-abrasive hydrodemolition technique was used for making the concrete cuts close to the abutment. The reduction of shear capacity because of removal of the concrete at the end of the bridge deck was accounted for prior to the jacking process. A diamond saw was used to cut the reinforcement. Once the Hyspec system was positioned under the bridge, and the jacking frame was positioned at the bottom of the bridge for lifting (Figure 2–10). The structure that was detached at the abutments and column foundations was transported to the set-down area adjacent to the bridge and was then jacked down onto temporary supports. Hydraulic hammers were then used to remove the reinforced concrete bridge deck and the bridge abutment.

Figure 2–10. Transportation of the Warren Farm Bridge (Source: Anumba et al. 2003)

Another example is the accelerated demolition of the 4500 South Bridge in Utah using a remotely-operated SPMT system (Ardani et al. 2009). The SPMT system was equipped with two sets of 16 axles and a hydraulic system that can lift and lower the structures within a vertical range of 24 inches. Prior to removal of the structure, the asphalt overlay, bridge railings, concrete median, and approach slabs of the structure were sawed off. The SPMT was used to lift and move the two-span superstructure of the bridge to a demolition area (Figure 2–11). The SPMT made two trips (one per span) and took 4 hours to complete the removal of the superstructure. After removing the existing superstructure to the demolition area, the existing columns and bent caps were demolished using hydraulic hammers.

Figure 2–11. Removal of the bridge using SPMT system (Source: Ardani et al. 2009)

The demolition of Silas N. Pearman bridge over Cooper River in Charleston, South Carolina was also accomplished in an accelerated manner using the hydraulic jacking method (Singh et al. 2008). The hydraulic jack system was attached by cables to the main span from the upper girder through the lower girder (Figure 2–12a and Figure 2–12b). The upper and lower girders of the main span were disconnected from the east and west cantilever sections. The hydraulic jacks were computer controlled and displayed the position of the jacks. After cutting the spans, the truss section was lowered on barges on the river positioned under the main span. After lowering the main span, the cables were detached from the truss section. The main span was then shipped to a yard for demolishing (Starmer and Witte 2006). The removal of the span was completed in few hours.

(a)

(b)

Figure 2–12. Hydraulic jack attached on the bridge (Source: Singh et al. 2008; Starmer and Witte 2006)

Crossings 3 and 4 on Oregon Highway 38 were removed using the accelerated hydraulic sliding system (HSS) method (Ardani et al. 2010a). The HSS includes hydraulic jacks with sliding rails and hydraulic pumps for lifting the superstructure (Figure 2–13). Temporary supports were constructed next to the bridge. Sideways were constructed to translate the superstructure of the bridge. The asphalt overlay and approach slabs were cut and crushed using hydraulic hammers and pulverizers. Hydraulic pumps generate power to move the hydraulic jacks that slide the superstructure on the rails. After removal of the superstructure, the temporary support systems and superstructure were dismantled and removed off-site. This HSS was also used for the Capilano River bridge replacement in West Vancouver, Canada and the Milton-Madison bridge replacement over the Ohio River in Indiana.

Figure 2–13. Hydraulic Sliding System (Source: Ardani et al. 2010a)

2.5.2.2 Traditional Demolition Methods

In this section, traditional bridge demolition methods and equipment are discussed.

- *Ball and Crane*: The ball and crane technique uses a steel ball suspended from the end of a crane (Transportation Research Board 1991) to demolish a structural component. This technique is used to break the concrete structure into pieces by either dropping the ball vertically onto the structure, or by swinging the ball on to the side of the structure.
- *Hydraulic Attachments*: Hydraulic attachments can be mounted on cranes and excavators to cut steel and crush the concrete structure. Hydraulic attachments have to be identified based on the materials that are being demolished. The most common hydraulic attachments are these:
    - Impact Hammer: An impact hammer is used to demolish masonry and concrete structures by applying force to a point (Figure 2–14a). It may be pneumatically or hydraulically operated (BSI 2000).
    - Hydraulic Hammer: Hydraulic hammers are mounted on excavators for demolition of bridge decks, piers, slabs, and pavements (Abudayyeh et al. 1998).
    - Hydraulic Shears and Pulverizer: Metal and reinforced concrete sections can be cut using shear jaws. A shear attachment can be mounted to an excavator for cutting (Figure 2–14b).

Demolition of the Parkview Avenue bridge in Kalamazoo, Michigan was accomplished using hydraulic hammers, excavators, and cranes. Two hydraulic hammers were used to demolish the old reinforced concrete deck (Figure 2–14*c*). The debris was removed to the side and crushed using excavators (Figure 2–14*d*). After demolition of the deck, the girders were removed using a crane by attaching cables to the ends of the girders for lifting. The piers were demolished using hydraulic hammers. The demolition of the bridge was completed in 8 days.

(a) Hydraulic Impact Hammer
(Source: IMECO 2011)

(b) Hydraulic Shear

(c) Hydraulic hammer

(d) Excavator

Figure 2–14. Equipment used in traditional bridge demolition process

- *Remotely Controlled Machines and Robotic Devices*: Remotely controlled machines and robotic devices are used when there are potential hazardous conditions. These machines can be controlled remotely at a safe distance from the demolition site (BSI

2000). They can be equipped with buckets, hydraulic hammers, a pulverizer, and drilling equipment (Figure 2–15).

Figure 2–15. Remote demolition machine Brokk 330 (Source: http://www.brokkinc.com/brokk-330.html)

- *Drilling and Sawing*: Drilling and sawing techniques are used to create holes or cut a portion of a structure. Diamond core drill and diamond floor sawing can also be used in demolition work. The diamond core drilling method is a vibration free method that creates holes in concrete. It can be powered by electricity, hydraulics or compressed air. The diamond floor sawing method is used for cutting trenches, expand joints, or remove slabs (BSI 2000).
- *Explosives*: Explosion methods have been used for complete demolition of concrete structures. They have many advantages such as cost effectiveness, time saving, and eliminating the need to use heavy machinery. They are also useful in cases where site access is limited (BSI 2000). For large structures, an experienced explosive engineer should manage the planning and execution process. Explosives may include gels, granules, powders, cord, liquids, plastics, and dynamite. For safety considerations, the specifications of all explosives must be analyzed before using.
- *Barge Mounted Crane*: The demolition of the Grace Memorial and Silas N. Pearman bridges in Charleston, South Carolina were accomplished using explosive techniques (Singh et al. 2008). The deck sections of the bridge were cut by saw and excavators with hoe rams. After removing the deck sections and the steel girders using a barge-mounted crane, the superstructure truss sections were demolished using explosive techniques.

- *Bursting*: This technique is used for the demolition of concrete, masonry, and rock. Gas expansion bursters and expanding demolition agents are two technologies in use for demolishing structures (BSI 2000).
    - Expanding demolition agents: This technique is used for reinforced concrete cutting. The chemical powder is mixed with water before pouring it into drilled holes (Archer Company 2011). The chemical composition of the agent includes calcium hydroxide that expands when the mixture hydrated. The 18,000psi pressure generated by the chemical mixture can break reinforced concrete without noise, vibration, or dust.
    - Gas expansion bursters: A gas burster is inserted into drilled holes. After being energized, high pressure fractures the component (BSI 2000).
- *Hydrodemolition:* This technique is used to cut concrete from steel reinforcing bars. The water mixture includes additives and abrasives to increase the pressure of water in the demolition process (BSI 2000). Hydrodemolition equipment consists of water-pumps, high-pressure hoses, high-pressure water nozzles, and a mobile housing unit for the water nozzles (Abudayyeh et al. 1998).

2.5.3 Safety Issues in Bridge Demolition

The process of bridge demolition requires careful planning, execution, and inspection to establish and maintain a safe work environment (Abudayyeh 1997). Bridges that cross environmentally sensitive waterways may need to be demolished using methods that do not create debris (Abudayyeh et al. 1998). Before selecting the demolition technique, the contractor should consider workers' protection, the safety of the public, adjacent structures, existing utilities, and the environment.

2.5.3.1 Protecting Workers and Safety of the Public

To ensure adequate protection to the workers and the public, the contractor and the owner should follow these steps:

- Develop proper demolition plans showing the demolition sequence, staging, equipment location, restraints and falsework for structural stability, and traffic control.

- Develop a comprehensive "Code of Safe Practice" that includes a plan for the use of personal protective equipment (hard hats, safety glasses, construction boots, tie-off, protective clothing, seat belts, and canopies).
- Develop a maintenance plan for keeping all pieces of equipment on the job in good working condition for the duration of the project.
- Develop a dust control plan (e.g., use of water sprays).
- Develop a plan to prevent debris from injuring the public and the workers (i.e., use debris nets).
- Develop a plan to control noise (i.e., observe work-hour schedules and monitor vibration and noise levels).

2.5.3.2 Protecting Utilities

Two types of utilities may exist in the vicinity of a demolition project: underground and overhead. Underground utilities may include gas mains, water pipes, and sewer lines. Overhead utilities may include power and telephone lines. To protect underground utilities, a number of measures can be taken:

- Debris piles may be built on top of such lines to provide a cushion against impact from falling objects.
- Steel plates may also be used as covers to protect against impact.
- High-pressure water lines should be shut down within the demolition zone.
- No large size debris should be allowed to fall freely.

To protect overhead utilities, the contractor and the owner should work closely with the responsible agency to arrange for a temporary shutdown and removal of utility lines in the immediate vicinity of the demolition site. Accurate schedules should always be sent to utility agencies to minimize service disruption and inconvenience to the public.

2.5.3.3 Protecting Adjacent Structures

One of the major challenges during a bridge demolition project is the protection of adjacent structures. Some of the measures that can be taken include the following:

- All hinges on the spans of a bridge should be restrained using steel cables or rods to prevent a premature collapse by slipping off the hinge seat.
- All possible loads on a bridge should be analyzed to establish a safe loading range before demolition starts to ensure that spans do not become overloaded with debris and/or heavy equipment.
- All columns should be restrained to prevent a premature collapse in the direction of adjacent structures.
- A monitoring program may also be established to prevent vibrations from exceeding the maximum limits for adjacent structures.

2.6 CONSTRUCTABILITY AND ELEMENTS OF A CONSTRUCTABILITY PROGRAM

2.6.1 Constructability

Knowledge and experience from previous construction projects are extremely valuable to improve constructability of an upcoming project. Therefore, a constructability review is the key to improving the project's buildability, bidability, and reduction of errors; thus, reducing contract change orders. Constructability may also reduce the life-cycle cost of a project. Constructability is formally defined as (AASHTO 2000):

"*a process that utilizes construction personnel with extensive construction knowledge early in the design stages of projects to ensure that the projects are buildable, while also being cost-effective, biddable, and maintainable*. "

The above definition is complemented by the definition given in Gambatese et al. (2007) "*the integration of construction knowledge and experience in the planning, design, procurement, construction, operation, maintenance, and decommissioning phases of projects consistent with overall objectives."*

A survey was conducted on constructability in the states of Washington, Oregon, and Nevada during 1998 and 2003. Ninety-nine of the 106 designers (93%) and 39 of the 52 constructors (75%) who completed the surveys indicated that constructability was formally considered. Additionally, a formal review process was reported by 71% of the respondents as the most commonly used practice in addressing constructability issues on construction projects. Other practices that were reported included plan reviews, project meetings, and value engineering reviews. In response to when the constructability reviews were conducted during a project life cycle, the constructors responded that these activities took place during the preliminary engineering phase, design, and construction. With regard to the members of the project team who need to address constructability, the responses included the involvement of constructors and designers as important constituents in addressing constructability. The survey also explored success measures in addressing constructability. Responders listed final construction cost, constructor feedbacks, and number of change orders as the most important metrics they use in evaluating the performance of constructability (Dunston et al. 2005).

Another survey performed within a project revealed that only a small percent of the state transportation agencies implemented a constructability review process (CRP) in their projects. The reasons were given as CRP requires significant time, cost, and effort (Dunston et al. 2005). The project concluded that flexible CRP implementation guidelines and effective involvement of contractors are crucial for successful implementation. Despite the apparent additional costs associated with the implementation of a constructability program, employing constructability reviews in projects includes many advantages such as the reduction in overall project and construction costs, the decrease in the number of construction change orders, improvements in project quality and safety, and shorter construction schedules. The maximum benefit from a constructability review is achieved through the early involvement of individuals with construction knowledge and experience in the design of a project. There are also challenges to including constructability reviews that need to be overcome. These topics will be detailed in the following subsections.

2.6.2 Constructability Benefits

Tangible and measurable benefits must be recognizable for the successful implementation of a constructability program. Promoting a unified vision for the agency must be set as a goal, and widely communicating the benefits of constructability will substantially increase the buy in of all parties involved. Below is a list of benefits of implementing constructability programs (Russell et al. 1994, Griffith and Sidwell 1997):

- Improved problem avoidance,
- Improved safety,
- Improved site layout,
- Reduced amount of re-work,
- Reduced change orders,
- Better communication,
- Increased commitment from team members,
- Better conceptual planning,
- Effective procurement,
- Improved design,
- Appropriate construction methods,

- Accomplished site management,
- Effective team work,
- Greater job satisfaction,
- Increased project performance,
- Enhanced recognition,
- Reduced engineering cost,
- Reduced construction cost,
- Reduced delays, and
- Shorter and more accurate schedules.

2.6.3 Constructability Implementation Challenges

In order to implement a constructability program; mutual trust, credibility, and respect between designers, project planners, and contractors is essential. However, there are some challenges with the implementation of a constructability program. Below is a summary of such challenges (Arditi et al. 2002; Uhlik and Lores 1998; Jargeas and Van der Put 2001; O'Connor and Miller 1994):

- Traditional contracting practices pose difficulties in implementing constructability programs. This is true in competitive bidding environments, where opportunities for collaboration between owners, designers, and constructors are lacking. In fact, the adversarial nature of such environments prohibits any collaboration possibilities due to the limitations in open communication between the parties involved with a project.
- Lack of initiative on the part of owners to commit funds and/or the resources (i.e., personnel) needs to improve for proper management of a constructability program.
- Lack of construction experience and fundamental knowledge is a problem that causes designers to become reluctant to include contractors in the constructability review.
- Rigid specifications pose difficulties as they limit design flexibility, reducing their ability to propose alternatives that will improve constructability.
- Agencies are reluctant to invest time or money in training the project personnel on constructability.

- Designers may resist the implementation of constructability programs, particularly with concerns about discovering inadequate designs, drawings lacking detail, or incomplete specifications.

2.6.4 Essential Elements of a Constructability Program

The constructability review process identifies potential conflicts that may lead to change orders, disputes, cost overruns, and delays during the construction phase of a project. To achieve the goal of any constructability program, a few key elements must be present, including the following (CII 1993):

- **Commitment to Constructability:** To demonstrate a strong commitment to constructability and to become proficient in understanding its concepts, agency leadership needs to develop a constructability implementation policy. The policy needs to include clearly defined organizational goals and training requirements of the project personnel through seminars and training courses. The distinction between constructability and value engineering must be understood.
- **Establish a Constructability Team:** The second element for constructability implementation is establishing a constructability program team. A typical team consists of the construction project manager, the agency's project manager, and project design engineers. These members are responsible for the approval of constructability suggestions. They also arrange for the participation of other potential constructability team members such as subcontractors, construction superintendents, permit and utility representatives, regulatory representatives, railroad representatives, fabricators, material suppliers, and other specialty contractors in order to provide input into areas requiring specific construction expertise. The team should have deep technical expertise and strong communication skills.
- **Define Constructability Objectives and Measures:** Constructability objectives can be defined after the team is identified. Typically, the main constructability objectives include improving quality, enhancing safety, and reducing project schedule and life-cycle costs. The team should develop a list of project objectives. Appropriate measures for assessment of each objective should be identified. These measures may

be capital dollars, construction dollars, direct field labor hours, labor productivity, design rework work-hours, shut-down duration, jobsite accessibility, etc.

- **Select Project Contracting Strategy:** The contracting strategy has an important influence on constructability and needs to be selected to facilitate the collaboration of all parties involved with a project. An agency should consider construction expertise that can increase the success of the constructability effort. The agency may review existing resources and perform a self-assessment to determine what constructability expertise is needed.
- **Develop Constructability Procedures:** Constructability activities need to be developed for every phase of the project and are best integrated with a project schedule. Below is a list of selected constructability concepts to consider when implementing the constructability program:
    - The constructability effort begins with the conceptual planning phase and continues through design, procurement, construction, and turnover phases.
    - The constructability team needs to develop a schedule for the various constructability studies and design inputs.
    - Constructability procedures and checklists can be developed utilizing lessons learned from past projects.
    - Experts, when needed, can be included in the meetings.
    - Professional estimating and scheduling support may be needed for complex analyses.
    - Design details need to be verified prior to the release of the design package.
    - At project acceptance, the constructability team needs to assess the project performance and evaluates the lessons learned for use by constructability teams of future projects.

Document Lessons Learned: Gaining knowledge from previous experiences is the key for continuous improvement of the constructability process. Lessons learned need to be well documented to prevent oversights in future projects, and they need to be documented during the design and construction phases and included in a database for use in future projects.

2.7 STATE-OF-THE-ART DECISION-MAKING MODELS

Traditionally, bridge construction, rehabilitation or a repair decision is made by a team of experts utilizing scoping reports of planned sites. Decisions are mostly influenced by bridge condition, available funding, and mobility. Understanding the need for selecting the construction alternative between cast-in-place and accelerated construction, Ralls (2005) developed the first decision-making model. Later, alternative models were developed to complement the simplifications of the first model.

Reviews of decision-making models used in other disciplines as well as the pertinent mathematical models are presented in the following sections. Also, the evolution of accelerated bridge construction (ABC) decision-making models as well as their limitations are discussed in the following sections.

2.7.1 Decision-Making for Outsourcing and Privatization of Vehicle and Equipment Fleet Maintenance

This decision-making model was developed by Wiegmann and Sundararajan (2011) and described in the *National Cooperative Highway Research Program (NCHRP) Report 692*. The model helps the fleet management agencies with outsourcing or insourcing decisions on fleet maintenance and business environment. Three outsourcing options are available in the decision making model. They are equipment class, maintenance service type, and organizational unit. The model defines process in phases, activities, decision points, and relevant evaluation criteria in a logical sequence for arriving at various outsourcing decisions. In Figure 2–16, high-level processes *(1 to 5)* are arranged in a sequence. Each *process* is described using secondary-level flowcharts that consist of activities and decision points. Following the flowchart will drive the decision-maker towards the decision. The secondary-level flow chart for *process-1* is shown in Figure 2–17. Each section of this flowchart provides the relevant evaluation criteria to be documented during the decision-making process.

The limitations of this model with potential to use in ABC decision-making are the following:

- A common strategy of answering *Yes/No* is used without any relative significance of critical criteria.
- Contemplating the flowchart without a mathematical model is considered obsolete in engineering and computing practice.

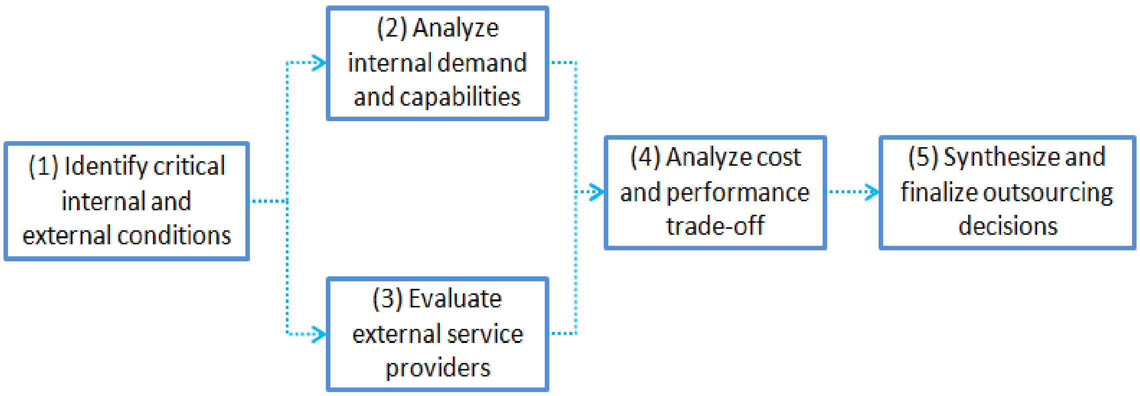


Figure 2–16. High-level outsourcing decision process (Source: Wiegmann and Sundararajan 2011)

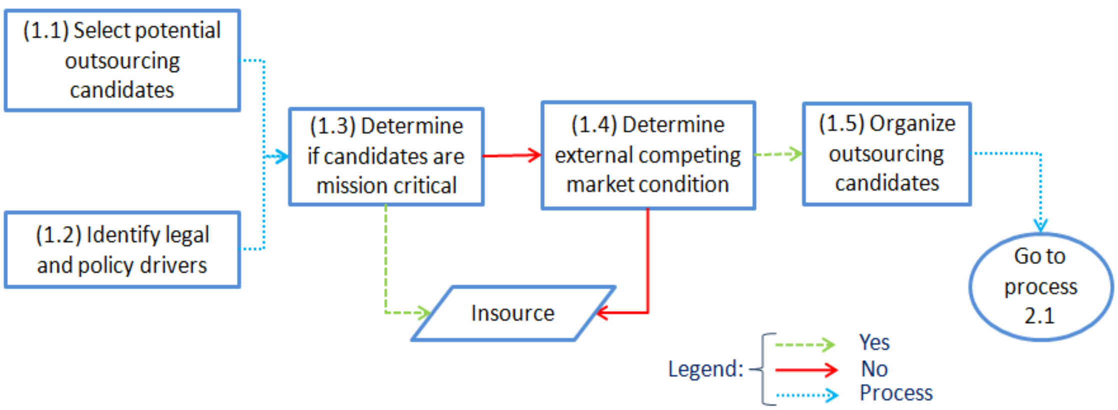


Figure 2–17. The secondary-level flow chart for process 1 to identify critical internal and external conditions (Source: Wiegmann and Sundararajan 2011)

2.7.2 Matrix–Based Decision Support Model for Pavement Rehabilitation Activities on High-Volume Roads

Carson et al. (2008) developed a matrix-based decision support model to assist state highway administrators in determining effective *strategies* on pavement rehabilitation activities. The activities are: i) construction, ii) traffic management, and iii) pubic information (outreach). The model consists of three sets of decision-support matrices, which are: (1) a preliminary strategy selection matrix, (2) secondary strategy selection matrices that are focused on construction, traffic management, and public information (outreach), and (3) an interdependency matrix that evaluates the relative level of interdependence between the strategies. Highway administrators evaluated the decision-support matrices in three steps to identify the *effective strategies*. Several strategies were considered for developing the decision-support matrices. The strategies were grouped as such:

- Contract administration,
- Planning-scheduling,
- Project management,
- Constructability,
- Construction practices,
- Traffic control, and
- Public relation (or outreach).

The construction traffic management and public information priorities are represented in the preliminary strategy selection matrix by the columns (Figure 2–18). The strategies that can be used to address those priorities are represented by the rows (Figure 2–18). In order to explain the process, only *contract administration*, *planning-scheduling*, *project management*, and *constructability* strategy groups are presented in the Figure 2–18. Three data sources, literature, case studies, and opinions from experts, were used to identify the best strategies to address construction and traffic management and public information priorities. Support from three data sources (i.e., literature, case studies, and opinions from experts) is represented as three equal segments of a circle. When all three data sources support a strategy, the circle is fully shaded and placed in the respective location of the matrix. As an example, all three data sources support the strategy to conduct a *formal constructability review* under the *constructability* strategy group and to address the priority of *Minimized Traffic Impacts* listed under traffic management and public information (Figure 2–18, row CO1 and column 25). Similarly, when only one data source supports a strategy to address a particular priority, only one-third of a circle is shaded and placed at the respective location of the matrix. As an example, only one data source supports the strategy of having *on-site agency decision makers* under the *project management* strategy group to address the early *project completion* priority listed under construction (Figure 2–18, row PM3 and column 4). A blank cell in the matrix is used to represent that there is no relation among the corresponding *strategy* and *motivation or concern*.

During the initial step of the decision-making process, based on the support from a majority of the data sources, the highway administrators may contemplate the preliminary strategy selection matrix to select the strategies relevant to the specific pavement rehabilitation project.

	Motivations or Concerns																								
	Construction													Traffic Management and Public Information											
	Owner-contractor cooperation	Common owner-contractor objectives	Worker safety	Early project completion	Set construction window	Short construction segments	Resource intensive, complex construction	High concrete production rates	High concrete quality	Exclusion of risky contractors	Reduced project costs	Alternative selection using design life	Minimized concrete haul times	High public visibility	High traffic volume	Predominant weekday commuters	Predominant weekend traffic	Directional traffic	Impacted local residents	Impacted local businesses	Alternate route availability	Safety of diverted traffic	Emergency services provisions	Real-time traffic information	Minimized traffic impacts
Strategy	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Contract Administration																									
CA1–contractor screening							◕			◔															
CA2–partnering	◔	◔					◔																		
CA3–incentives-disincentives				◔										◔		◔	◔			◔		◔	◔		
CA4–A+B contracting					◔											◕				◕		◕			
CA5–lane rental					◔										◕	◕	◕			◕					
CA6–production-based contract length																									◕
Planning–Scheduling																									
PS1–joint schedule review meetings																									◕
PS2–accelerated schedule																◔	◔		◔	◔					◔
PS3–detailed hourly schedule					◔																				
PS4–dynamic project schedule																									◕
PS5–weekend construction															◔	◔									
PS6–nighttime construction															◔	◔									
PS7–early utility work completion				◕																					
PS8–contingency plan																									◔
PS9–responsive adverse weather plan					◔																				◔
PS10–rest days			●																						
Project Management																									
PM1–daily construction meetings	◕	◕																							
PM2–effective communication							◔																		
PM3–on-site agency decision makers				◔																					◔
Constructability																									
CO1–formal constructability review					●		●								●										●
CO2–optimization software					●		●								●										●

Figure 2–18. Preliminary strategy selection matrix (Source: Carson et al. 2008)

The secondary strategy selection matrices are detailed matrices focused on the relative benefit of construction, traffic management, and public information (Figure 2–19). The strategy groups, *contract administration*, *planning-scheduling*, *project management*, *constructability*, and *construction practices*, are considered in the secondary strategy selection matrix of construction (Figure 2–19a). The *traffic control* strategy group is considered in the secondary strategy selection matrix of traffic management (Figure 2–19b). The *public relation* strategy group is considered in the secondary strategy selection matrix of public information (Figure 2–19c). The data source of *case studies* was used in these matrices to assess the relative benefit of each strategy for construction, traffic management, or public information activities. The relative benefit is represented in terms of i) low (L), ii) medium (M), or iii) high (H). A blank cell in the matrix is used to represent that there is no relation among the corresponding *strategy* and *relative benefit*.

During the secondary strategy selection stage, highway administrators need to refine the strategies that were selected during the preliminary strategy selection stage. This process requires evaluating the relative benefit of each strategy in terms of low, medium, and high and eliminate the strategy with low (L) relative benefit in all activities. This step will provide highway administrators with the *refined strategies* for the planned pavement rehabilitation project.

(a)

Strategy	Relative benefit: Communication-coordination	Speed-efficiency	Construction quality	Work zone safety
Contract Administration				
CA1–contractor screening		M		
CA2–partnering	M	L		
CA3–incentives-disincentives	L	H		
CA4–A+B contracting	L	H		
CA5–lane rental	L	H		
CA6–production-based contract length				
Planning–Scheduling				
PS1–joint schedule review meetings	H			
PS2–accelerated schedule		M		
PS3–detailed hourly schedule	M			
PS4–dynamic project schedule		M		
PS5–weekend construction		L		L
PS6–nighttime construction		L		L
PS7–early utility work completion		M		
PS8–contingency plan		M		
PS9–responsive adverse weather plan		M		
PS10–rest days				M
Project Management				
PM1–daily construction meetings	H			
PM2–effective communication	M			
PM3–on-site agency decision makers	H	H		
Constructability				
CO1–formal constructability review			M	
CO2–optimization software		M		L
Construction Practices				
CP1–concrete accelerator admixture		M		

(b)

Strategy	Relative benefit: Impact reduction	Demand reduction	Work zone traffic flow	Work zone safety
Traffic Control				
TC1–traffic pattern analysis	M	M		
TC2–alternate route planning		L-H		
TC3–project-special event coordination				
TC4–dynamic traffic control plan	M		M	
TC5–full roadway closures				
TC6–permanent lane closures				
TC7–ramp closures		M	M	M
TC8–movable concrete barriers			H	L-M
TC9–speed reductions				L
TC10–static signs-barrels-barriers			L-M	L
TC11–changeable message signs		M	M-H	M
TC12–highway advisory radio			M	
TC13–traffic management center-CCTV			H	
TC14–incident management program				M-H
TC15–use of local police				M-H
TC16–business access (signs-flaggers)		H	M	M
TC17–neighborhood traffic restrictions	M			
TC18–transit incentives-improvements		M		

(c)

Strategy	Relative benefit: General traffic	Weekday commuters	Weekend-recreational traffic	Local residents	Local businesses	Truck traffic
Public Relation						
PR1–contracted PR service						
PR2–slogans-logo	M-H					
PR3–elected-community leader buy-in	M					
PR4–ribbon-cutting ceremony		H		H		
PR5–community-public meetings	M-H	M		M	M	
PR6–targeted outreach	H			H	M	M-H
PR7–e-mail-fax database	M				M	
PR8–informational brochures	M			M-H	L-H	L
PR9–media press releases-PSAs	L-H	M	M	H	H	
PR10–telephone hotline						
PR11–website	M					
PR12–billboards	H					
PR13–pre-and post-construction surveys		H	H			

Figure 2–19. Secondary strategy selection matrix for (a) construction, (b) traffic management, and (c) public information (Source: Carson et al. 2008)

The third decision-support matrix, which is the interdependency matrix, considers the relative level of interdependencies among *all the strategies*. Only a part of the interdependency matrix is shown in Figure 2–20 to illustrate the decision-making process. The interdependency matrix was developed by pair-wise comparison of all strategies and represents the interdependencies as i) low (L), ii) medium (M), or iii) high (H). The data source of *opinions from experts* was used in this matrix to obtain the interdependencies. A blank cell in the matrix is used to represent the pair-wise comparison of the strategies that do not have any interdependency, and the cells with *1* represent the pair-wise comparison of same strategies.

The administrators, during the final step of decision-making process, evaluate the interdependency matrix to identify the strategies that are of high (H) interdependence to the *refined strategies* (from the second step). The *identified strategies* may provide additional synergistic benefits if simultaneously applied with the *refined strategies*. Therefore, the *identified strategies* can be included among the *refined strategies* to generate a set of *effective strategies*, which could be implemented during the project (Carson et al. 2008).

	Relative Level of Interdependence (Low, Medium, or High)															
Strategy	CA1	CA2	CA3	CA4	CA5	CA6	PS1	PS2	PS3	PS4	PS5	PS6	PS7	PS8	PS9	PS10
Contract Administration																
CA1–contractor screening	1	**H**						**H**		**H**						
CA2–partnering	**H**	1						**H**								
CA3–incentives-disincentives			1	**M**		**M**		**M**								
CA4–A+B contracting			**M**	1		**H**		**M**								
CA5–lane rental					1			**M**	**M**							
CA6–production-based contract length			**M**	**H**		1		**M**								
Planning–Scheduling																
PS1–joint schedule review meetings							1	**M**								
PS2–accelerated schedule	**H**	**H**	**M**	**M**	**M**	**M**	**M**	1					**H**	**H**	**H**	
PS3–detailed hourly schedule					**M**				1							
PS4–dynamic project schedule	**H**									1						
PS5–weekend construction											1					
PS6–nighttime construction												1				
PS7–early utility work completion								**H**					1			
PS8–contingency plan								**H**						1		
PS9–responsive adverse weather plan								**H**							1	
PS10–rest days																1

Figure 2–20. Part of the interdependency matrix (Source: Carson et al. 2008)

The limitations of this model with respect to ABC decision-making are the following:

- The entire process utilizes qualitative data,
- Access to literature on priorities and potential strategies related to ABC is limited, and
- Access to detailed case studies on ABC with relevant information is limited.

2.7.3 Linear Programming for Decision-Making

Linear programming or linear optimization is a mathematical model used to find an optimal solution for a given objective function with associated constraints. Linear programming is used in many disciplines such as transportation, manufacturing, and telecommunication. The applications include network optimization in transportation and telecommunication industries and production scheduling in manufacturing. The process requires developing a mathematical formulation of each specific problem. The mathematical formulation includes the objective function and its relevant constraints. There are many approaches within linear programming such as the simplex method, integer linear programming, decision-making, programming with and without probabilities, and data envelopment analysis. Based on the approach, the objective function is modeled, and the constraints are developed. The formulation is solved to obtain a feasible solution. The feasible solution provides the objective function value for the optimal solution (Anderson et al. 2005). Further, for a situation where the decision is to choose an efficient alternative among many, the data envelopment analysis approach is adopted. In this approach, a hypothetical composite alternative is assumed, which is composed of all virtuous characteristics of all potential alternatives, and its efficiency is assumed to be unity. The linear programming approach is customized to calculate the efficiency of each alternative and compared with the efficiency of the hypothetical composite alternative. The alternative with efficiency close to *unity* will be the decision alternative (Anderson et al. 2005).

In linear programming, the parameters of the decision alternatives are correlated with the objective function and constraints. This requirement imposes a major limitation when qualitative parameters are involved in the decision-making process.

2.7.4 Scoring Model for Decision-Making

The scoring model is a quick and easy approach to identify an optimal decision alternative from a set of alternatives. The process involves (1) identifying the appropriate parameters, (2) assigning weights to each parameter *"i"* out of 100% (i.e., w_i), (3) assigning a score to each parameter *"i"* on an ordinal scale with respect to each decision alternative *"j"* (i.e., r_{ij}), and (4) calculating the final score *"S_j"* for each decision alternative *"j"* using Eq.2-1. The decision alternative with the maximum *final score* will be the optimal decision alternative (Anderson et al. 2005).

$$S_j = \sum_i w_i \, r_{ij} \tag{2-1}$$

El-Diraby and O'Conner (2001) developed a decision-making process using the scoring model, the *Bridge Construction Plan (BCP) evaluation process*. The BCP evaluation process was designed to select an optimal BCP among the appropriate alternatives. The BCP evaluation process is discussed in the following section.

2.7.4.1 Bridge Construction Plan (BCP) Evaluation Process

The BCP evaluation process involves several parameters with six categories of i) safety (S), ii) accessibility (A), iii) carrying capacity (C), iv) schedule (T), v) budget (B), and vi) additional parameters (Q). Then an objective matrix similar to Figure 2–21 can be developed to evaluate the BCPs using the scoring model methodology.

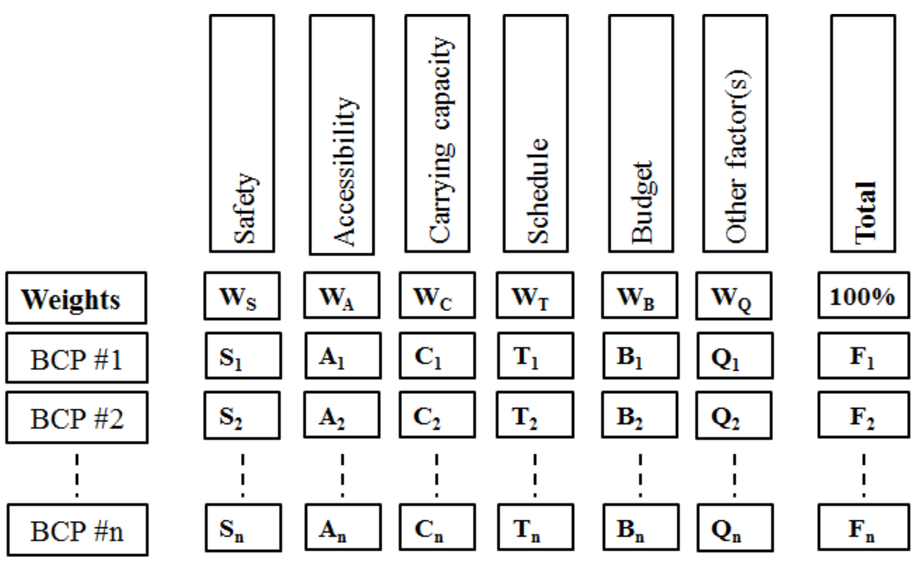


Figure 2–21 BCP comparison objective matrix (Source: El-Diraby and O'Conner 2001)

The steps that a decision maker needs to follow during a BCP evaluation process include the following:

1. Assign percentage weights (W_i) to each category. As an example, the "i" represents S, A, C, T, B, and Q categories shown in Figure 2–21. The sum of all the weight is 100% (i.e.$\Sigma W_i = W_S + W_A + W_C + W_T + W_B + W_Q = 100\%$).
2. Rate the parameters listed under the Safety (*S)* group using an ordinal scale with respect to BCP#*1*.
3. Calculate the total of all the parameter ratings listed under the Safety (*S)* group to obtain a total score value of S_1.
4. Repeat steps 2 and 3 for the remaining five groups *A*, *C*, *T*, *B*, and Q, to obtain the total scores A_1, C_1, T_1, B_1, Q_1, respectively.
5. Repeat steps 2 to 4 for the remaining BCP alternatives (i.e., BCP#*2* to BCP#*n*), and
6. Finally, calculate the *final score "F_j"* (where *"j"* is *1* to *n*) using Eq.2-2 for each BCP alternative.

$$F_j = W_S \times S_j + W_A \times A_j + W_C \times C_j + W_T \times T_j + W_B \times B_j + W_Q \times Q_j \qquad (2\text{-}2)$$

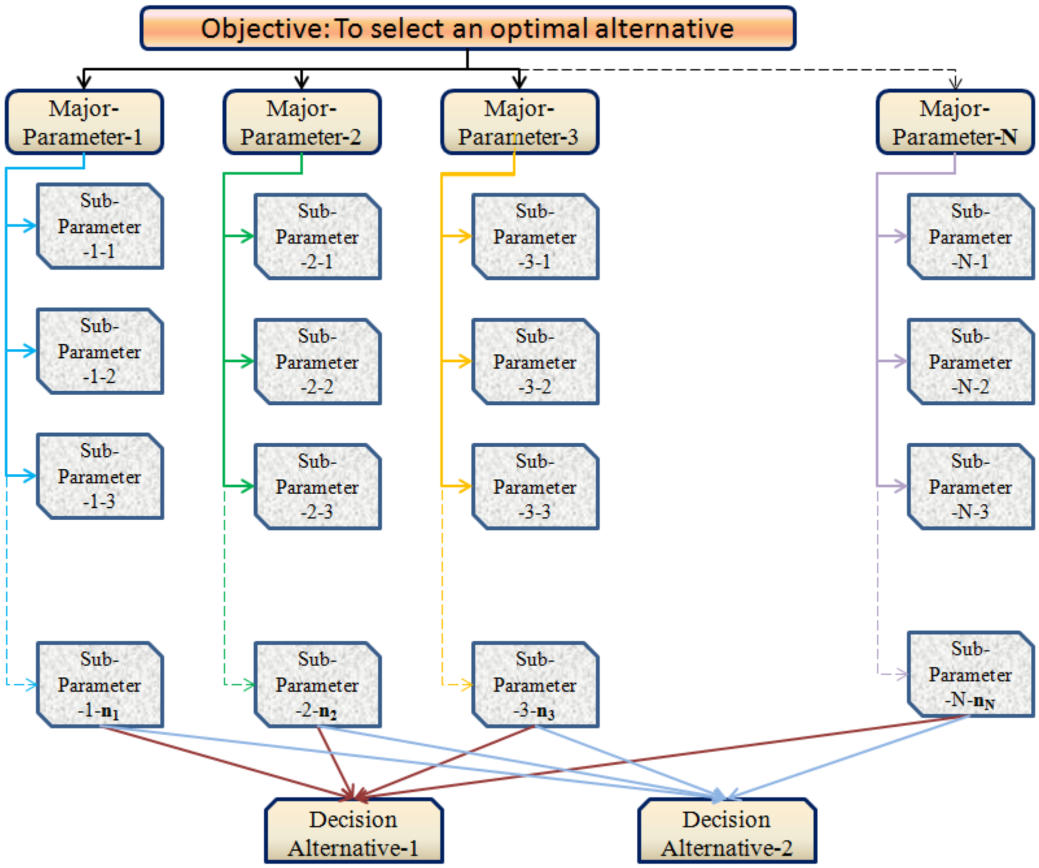
The BCP with the highest final score represents the optimal BCP alternative. The limitations of this model are as follows:

- Multiple decision makers do not have a consistent method to assign weights and preference ratings for each parameter. This may lead to biased results.
- A uniform ordinal scale is not provided for the preference ratings.
- The model does not account for the relative importance of the parameters that are involved in the evaluation process.

2.7.5 Analytic Hierarchy Process (AHP) for Decision-Making

The AHP process enables breaking down a complex, unstructured situation into multi-level hierarchical order by defining major-parameters and sub-parameters for each level.

The first step in AHP is to develop a graphical representation (hierarchy) of a problem in terms of the overall goal, its parameters, and the decision alternatives as shown in Figure 2–22. In Figure 2–22, the top level shows the objective, the second level shows the major-parameters, the third level shows the sub-parameters related to each major-parameter, and lowest level shows the decision alternatives.



→*Multiple decision alternatives existing for a project are rated against each sub-parameter.*

Figure 2–22. Graphical representation of the AHP

The first step is the pair-wise comparison of the major-parameters by assigning a preference rating using a nine-point ordinal scale. Numerical values are assigned to subjective preferences based on the relative importance of each parameter as shown in Table 2-20; this process helps the decision makers to maintain uniform consideration while moving towards a decision. From the major-parameter pair-wise comparison results, a matrix is developed which is described as the pair-wise comparison matrix. The eigenvectors calculated from the pair-wise comparison matrix define the local priorities of the major-parameters (Saaty 1980).

Table 2-20. Fundamental Scale for AHP Pair-Wise Comparison (Source: Saaty 1995)

Rating scale	Definition	Description
9	Extreme importance	The evidence favoring one activity over another is of the highest possible order of affirmation.
7	Very strong or demonstrated importance	An activity is favored very strongly over another; its dominance demonstrated in practice.
5	Strong importance	Experience and judgment strongly favor one activity over another.
3	Moderate importance	Experience and judgment slightly favor one activity over another.
1	Equal importance	Two activities contribute equally to the objective.
2,4,6,8	For compromise between the above values	Sometimes one needs to interpolate a compromise judgment numerically because there is no good word to describe it.
Reciprocals of above	If activity 'i' has one of the above nonzero numbers assigned to it when compared with activity 'j,' then 'j' has the reciprocal value when compared with 'i.'	A comparison mandated by choosing the smaller element as the unit to estimate the larger one as a multiple of that unit.

Next, a pair-wise comparison of the sub-parameters of each major-parameter is performed, again using the ordinal scale shown in Table 2-20. Pair-wise comparison matrices are developed for all the sub-parameters under the same major-parameter. Then local priorities are obtained for the pair-wise comparison matrices of sub-parameters by calculating the corresponding eigenvectors. Finally, relative preferences of the decision alternatives with respect to each sub-parameter are obtained. A third set of pair-wise comparison matrices are developed in which each matrix corresponds to the pair-wise comparison of decision alternatives with respect to each sub-parameter. The local priorities are obtained by calculating the corresponding eigenvectors. The three sets of the local priorities [that are i) local priorities of the major-parameters, ii) local priorities of the sub-parameters, and iii) local priorities of the decision alternatives with respect to each sub-parameter] are finally integrated using AHP formulation to calculate the final *priority* of each decision alternative. The decision alternative with the highest *priority* will be the optimal solution (Saaty 1980). This process allows incorporating both qualitative and quantitative parameters into the decision-making process. An example implementation of the AHP was completed by Moghadam et al. (2009), which was *the decision-making process in selecting container yard operating equipment*. This process is discussed in the following section.

2.7.5.1 Decision-Making in Selection of Container Yard Operating Equipment

The decision-making process for selecting the best container yard operating equipment utilized the AHP process (Moghadam et al. 2009). Five components of AHP, which provided an ideal platform for this problem, were i) finite set of options, ii) trade-offs between parameters, iii) heterogeneity of qualitative and quantitative parameters, iv) matrix of pair-wise comparisons, and v) the decision matrix. The AHP methodology was applied to find the best container yard operating equipment among the *selection options*, which were (1) semi-automated straddle carriers (SC), (2) rubber tired gantry cranes (RTG), and (3) automated and semi-automated rail mounted gantry cranes (RMG).

Many significant parameters for decision-making were identified for this problem. The major-parameters and their associated sub-parameters were organized in a container yard handling equipment decision tree as shown in Figure 2–23. The sub-parameters, which were both qualitative and quantitative, were brought under one category by using the AHP scale of 1 to 9 (Table 2-20). The preference ratings from the experts were collected following the three-step procedure of AHP (discussed in the previous section). The final priorities of the *selection candidates* were obtained by integrating the preference values of all the parameters and the *selection candidates* through the AHP formulation. The study concluded that the RMG system was the optimal alternative.

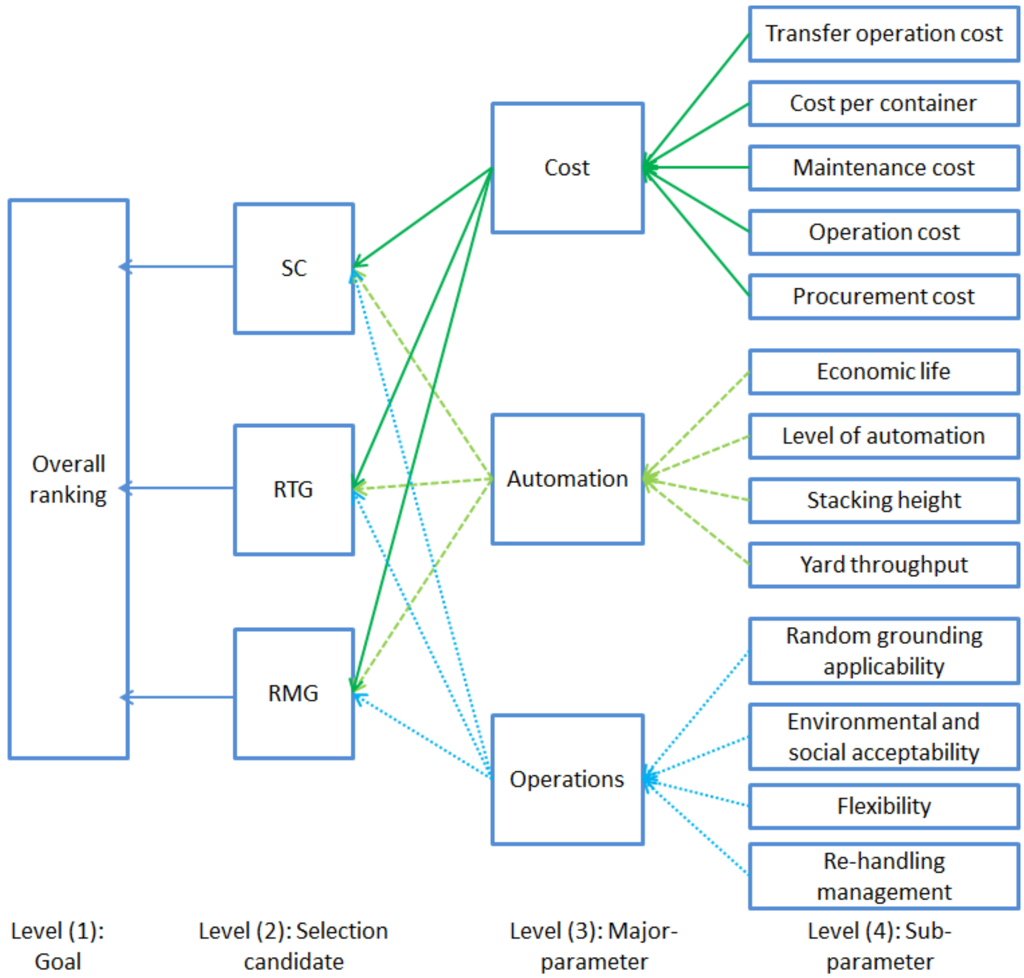


Figure 2–23. Container yard handling system decision tree (Source: Moghadam et al. 2009)

The advantages of AHP are these:

- Both qualitative and quantitative evaluation procedures can be incorporated with a constant rating scale.
- It provides consistency to the decisions from multiple decision makers.
- It uses an ordinal scale to represent qualitative judgments and priorities are calculated through a mathematical process (eigenvector). .
- AHP can incorporate both qualitative and quantitative parameters in the decision-making process.
- Finally, the method has been used in various disciplines and in different formats; hence, it is a tested and validated.

The limitations of AHP are these:

- The process is time consuming when the number of parameters is large.
- Processing manual AHP evaluation requires keeping track of AHP formulation as well as the consistency ratio, which is reported as tedious.
- For higher order matrices (i.e., large number of major-parameters or sub-parameters), consistency in the experts' judgments may be difficult to achieve. This is because the number of transitive rules to be satisfied is proportional to matrix rank square. Therefore, priority calculation methods without *the computing aid* will be impractical (Tam et al. 2006; Buckley et al. 2000; Moghadam et al. 2009).

2.7.6 Multi-Criteria Decision-Making Approach for Highway Slope Hazard Management

A linguistic fuzzy theory is a category of the multi-criteria decision-making procedure. This procedure was used to establish the hazard level associated with each hazard assessment criteria for managing potential highway slope failures in Ohio. In fuzzy set theory, the expert preference is expressed by a quantitative value by using a membership function [$\mu_A(x)$] that takes a real value between 0 and 1 (Zadeh 1965; Liang and Pensomboon 2010). The fuzzy approach can be evaluated with triangular fuzzy numbers and trapezoidal fuzzy numbers. This process will derive fuzzy weights from group evaluations, the max-min aggregation, and center-of-gravity defuzzification. The process proceeds in parallel with AHP with a difference of incorporating fuzzy mathematical calculations. As in the AHP, preference ratings are provided on a fixed scale of 1 to 9. In the fuzzy approach, the ratings are provided with respect to fuzzy set theory as shown in Eq.2-3. The perspective of the membership functions and linguistic values used is shown in Figure 2–24. Six hazard assessment criteria identified from the literature were considered for this study. The hazard assessment criteria includes i) slope failure location and its impact on highway safety, ii) extent of pavement damage due to the slope movement, iii) the maintenance response requirement due to slope movement, iv) Decision Sight Distance (DSD), which is the ratio of the actual sight distance to American Association of State Highway and Transportation Officials (AASHTO) standard sight distance, v) Average Daily Traffic (ADT), which is the average number of vehicles passing a landslide location per day, and vi) slope displacement magnitude.

The first three criteria require judgment from the assessor; whereas, the remaining three criteria can be measured directly. Linear optimization is carried out to obtain the decision value (D) based on hazard assessment criteria (Eq.2-4). The slope failure sites with higher decision value will be those requiring corrective activities with a higher priority (Liang and Pensomboon 2010).

$$\text{Fuzzy set } A = \{[x, \mu_A(x)]\},\ x \in X \tag{2-3}$$

$$D = \max\,(D^p{}_m) \tag{2-4}$$

where:

[μA(x)] is the membership function, which takes value between 0 and 1.

$D^p{}_m$ is the sum of *'m'* membership function averages for *'p'* criteria.

The limitations of this model are

- There is difficulty involved in intricate mathematical calculations.
- The manual version of evaluation is tedious, and programming in a computer structure is time consuming and costly.

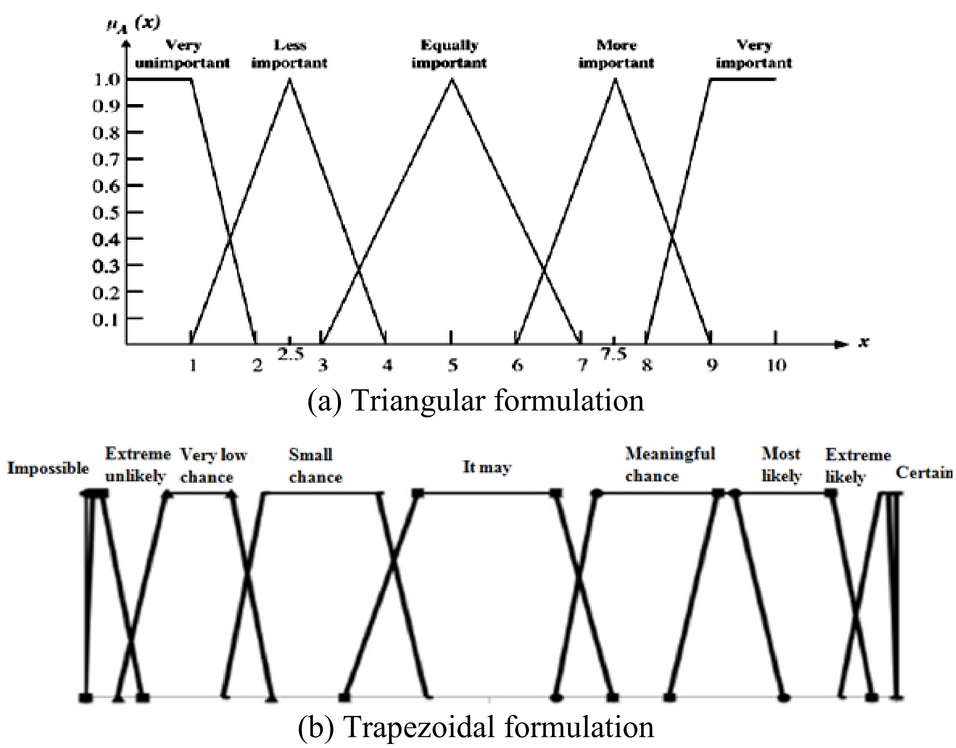


Figure 2–24. Membership functions and linguistic values (Source: Liang and Pensomboon 2010)

2.7.7 Application of the AHP to Select Project Scope for Video-Logging and Pavement Condition Data Collection

Highway video-logging involves the use of electronic equipment mounted in a large van at highway speed for capturing a range of accelerations, laser measurements, digital imagery, and precise positioning (Larson and Forman 2007). Virginia DOT's asset management division obtained support from executive management to expand video-logging and pavement data collection statewide. This required an optimal number of highway miles to be covered annually for video-logging. The division considered a few approaches in deciding the annual mileage of highway logging, which were i) collecting data on all hard-surfaced roads every year, ii) collecting only what is needed for supporting pavement maintenance and ignoring the right-of-way images. To arrive at an optimal strategy, Virginia DOT used AHP to formalize the decision process and 'Expert Choice' economic modeling software to assist AHP evaluation process. The AHP decision tree was formulated by considering distinct aspects as well as difficulties of each alternative. The aspects were organized into hierarchical levels so that the aspects with greater influence were classified into objectives (the first level of the hierarchy) and their respective sub-objectives (the second level of the hierarchy), while those with low influence were classified into objectives in general (the first level of the hierarchy) (Larson and Forman 2007). A schematic view of the decision tree from the Expert Choice economic modeling software is shown in Figure 2–25.

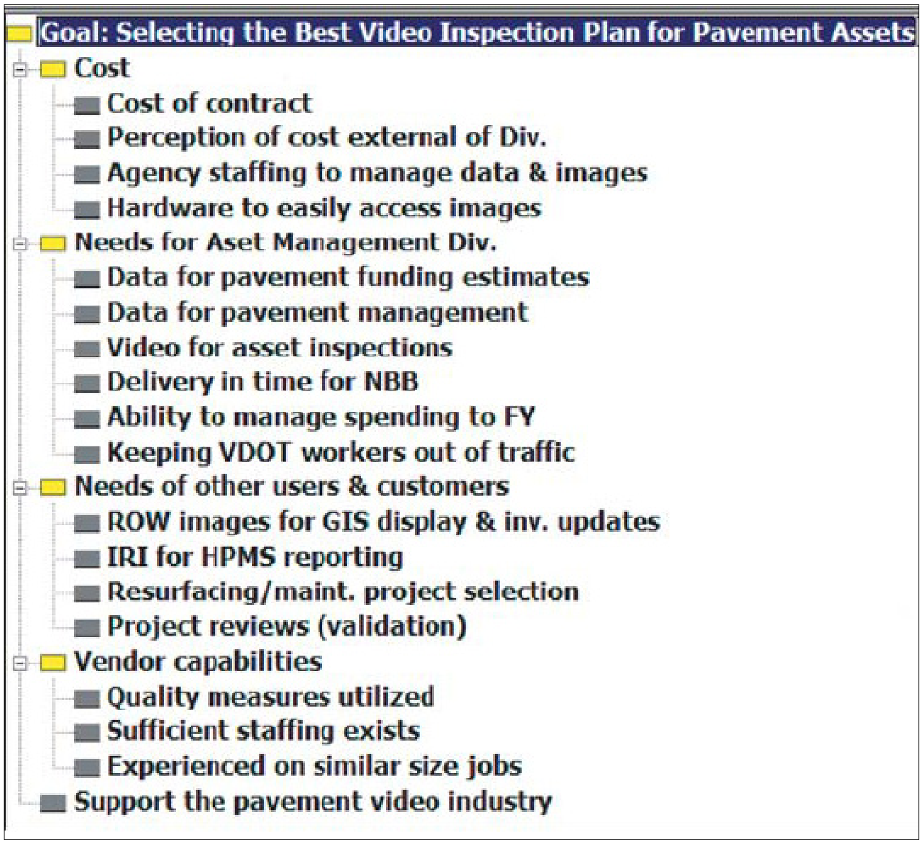


Figure 2–25. Decision tree from expert choice software (Source: Larson and Forman 2007)

The initial task was limited to pavement data collection. Thus, the decision goal was to select the best video inspection plan for pavement assets. Five alternatives were identified as i) perform video-logging for all pavements every year, ii) perform video-logging for interstate, primary, and one-third of secondary highways every year, iii) perform video-logging for interstate and one-third of secondary highways every year and all primary highways every two years, iv) perform video-logging for interstate, primary, and one-fifth of secondary highways every year, and v) perform video-logging for only interstate and primary highways every year (Larson and Forman 2007).

The preliminary AHP evaluation was performed using only the first four alternatives, and the optimal alternative was (i) *performing video-logging for all pavements.* This alternative was not feasible; thus, a sensitivity analysis of the alternatives was performed. The sensitivity analysis results (Figure 2–26) revealed that alternative (i) diverges significantly from other alternatives. For that reason, a fifth alternative was added to expand the range of preferences. The preferred alternative selected from the modified AHP was alternative (iv) *performing* *video-logging for interstate, primary, and one-fifth of secondary highways every year*. The inference from this model is that there is a need for precision and computing tools to obtain a desired consistency in the results, and sensitivity analysis is a virtuous technique to validate the obtained results.

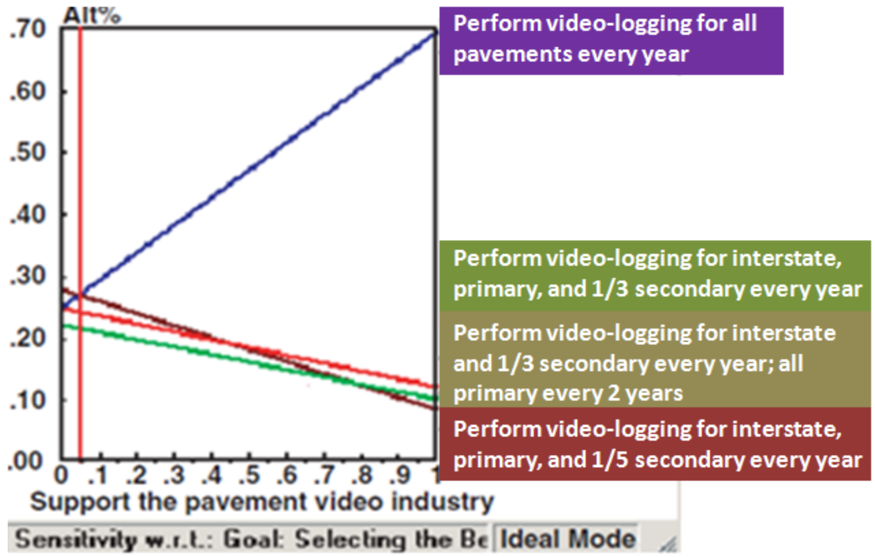


Figure 2–26. Sensitivity analysis results for corresponding alternatives from expert choice software (Source: Larson and Forman 2007)

2.7.8 Accelerated Construction Decision-Making Process for Bridges

Salem and Miller (2006) developed a decision-making model to identify the best construction procedure for a given bridge construction site. This study was funded by the Ohio DOT and the U.S. DOT. In the model, the potential of each construction strategy for achieving the goal is evaluated with respect to parameters of Cost (C), Traffic flow (T), Safety (S), Economic parameter (B), Social parameter (P) and Environmental parameter (E). These parameters were considered as major-parameters and consisted of sub-parameters. The data required for this decision-making model was obtained from a survey of 25 U.S. DOTs and industry experts with accelerated bridge construction expertise. The survey inquired about the share of each of six major-parameters used in accelerated construction decision-making. The survey data was averaged with 95% confidence intervals to obtain mean percentages for each parameter as shown in (Table 2-21). Similarly, the sub-parameters were assigned mean percentages.

Table 2-21. Mean Weights of Parameters from the Survey (Source: Salem and Miller 2006)

Parameter	Confidence interval (95%)	Mean percentage weight
Cost	16-31	25
Traffic flow	11-29	20
Safety	11-31	20
Economy	8-18	15
Social	7-16	10
Environment	7-13	10
Total		100

The mean percentage weights from major- and sub-parameters were back-calculated to form pair-wise comparison matrices of AHP (Table 2-22). Local priorities of major- and sub-parameters were then calculated, and each construction alternative was evaluated for its effectiveness: "e" with respect to each sub-parameter (Figure 2–27). The effectiveness was then multiplied with the mean percentage weights to calculate final priorities of construction alternatives (Figure 2–27).

Table 2-22. AHP Pair-Wise Comparison Matrix Developed through Back-Calculation from Mean Percentage Weights (Source: Salem and Miller 2006)

Pair-wise comparisons	Cost	Traffic flow	Safety	Economy	Social	Environmental
Cost	1	3/1	3/1	4/1	4/1	5/1
Traffic flow	1/3	1	1	3/1	4/1	4/1
Safety	1/3	1	1	3/1	4/1	4/1
Economy	¼	1/3	1/3	1	3/1	3/1
Social	¼	1/4	1/4	1/3	1	1
Environmental	1/5	1/4	1/4	1/3	1	1

$$\begin{array}{r} \text{Alternative x} \\ \text{Alternative y} \\ \text{Alternative z} \end{array} \begin{bmatrix} e_{1x} & e_{2x} & e_{3x} & e_{4x} & e_{5x} \\ e_{1y} & e_{2y} & e_{3y} & e_{4y} & e_{5y} \\ e_{1z} & e_{2z} & e_{3z} & e_{4z} & e_{6z} \end{bmatrix} \begin{bmatrix} w_1 \\ w_2 \\ w_3 \\ w_4 \\ w_5 \end{bmatrix}$$

Figure 2–27. Final evaluation of construction alternatives (Source: Salem and Miller 2006)

The limitations of this model are these:

- The model is not capable of adequately addressing specific sub-parameters, which can further affect the final decision.
- The pair-wise comparison matrices are not developed directly from expert opinions; rather pair-wise comparison matrices are formed by back-calculating the survey data percentages. Thus, the capabilities of the AHP are not effectively implemented in this decision-making model.

2.7.9 PBES Decision-Making Model

Implementation of innovative bridge systems and construction technologies requires addressing many variables. These are the applicability of the design, availability of skilled workforce (i.e., contractors' and suppliers' abilities to deliver a successful project), project site access and space for equipment placement, the effect of construction process on cost and schedule, the owner's and contractor's willingness to share responsibility and risk, and commitment of all the parties to successful completion of the project. Ralls (2005) developed a model addressing these variables to evaluate the potential and effectiveness of using an ABC for a particular site. This model consists of three main sections: a flow chart (Figure 2–28), a matrix (Table 2-23), and a considerations section. The flowchart is a tool that provides an overview of parameters that need to be considered in decision-making. The matrix of the questionnaire consists of detailed questions requiring a selection of *Yes/No/Maybe* answers. The dominance of a type of answer determines the optimal construction alternative. The questions presented in the matrix are all focused on ABC. For example, if *Yes* is dominant, then the site is feasible for ABC. The last part of this model, the *considerations section,* includes parameter descriptions in detail along with various definitions. Further details can be obtained from the report *Prefabricated Bridge Elements and Systems Decision-Making* by Ralls (2005).

The three sections in this model can be used independently or jointly, depending on the desired depth of evaluation. Even though the flowchart helps in arriving at the decision, the relative importance of different parameters is not considered. The matrix refers to the questionnaire having its implications for some tangible parameters and suggests answering the questionnaire. This is not that different from assigning random importance to parameters.

This approach is not quantitative and lacks a process to allow further refinements of the decision (Salem and Miller 2006).

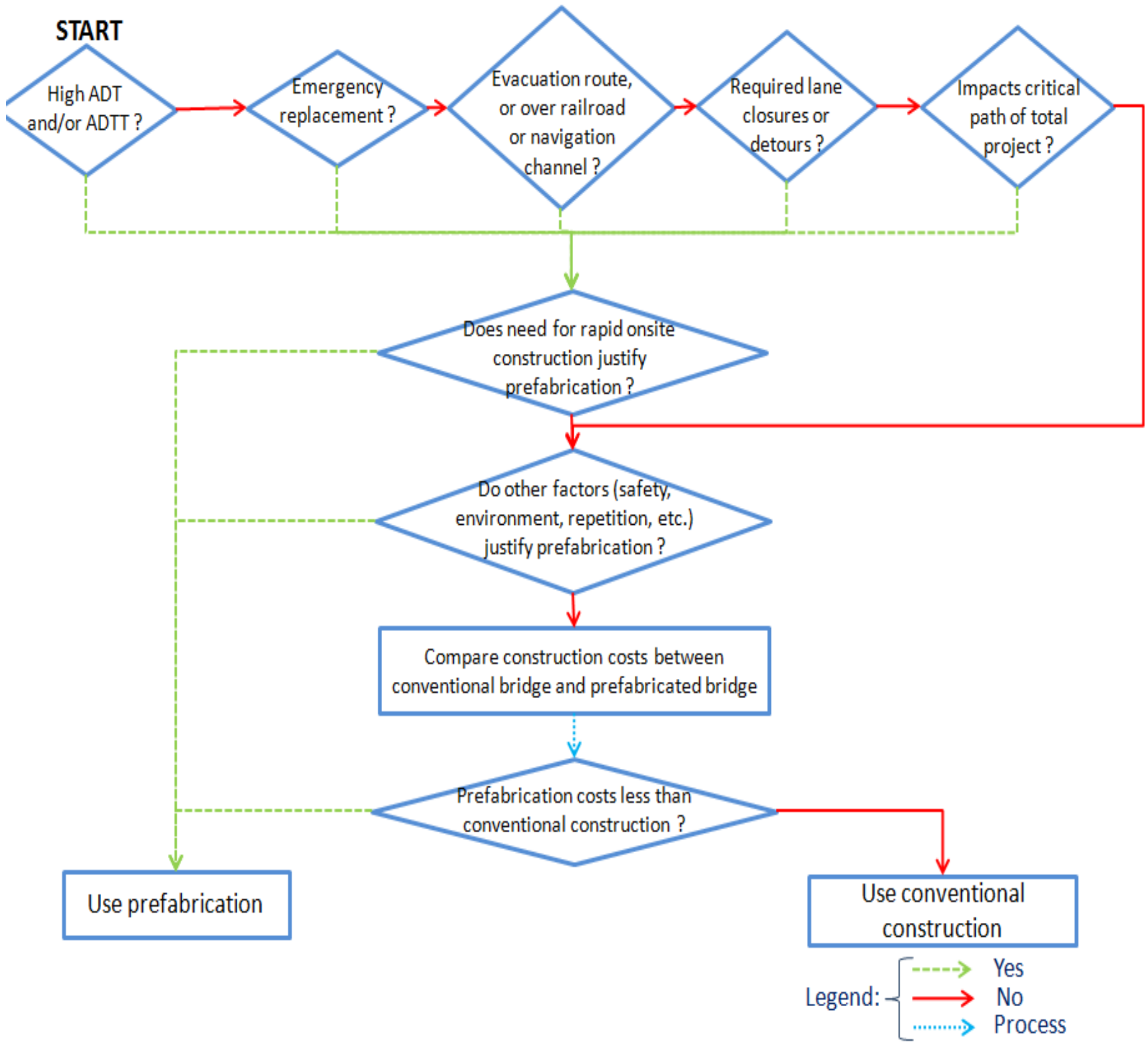


Figure 2–28. Flowchart for high–level decision-making on PBES (Source: Ralls 2005)

Table 2-23. Matrix for High–Level Decision-Making on PBES (Source: Ralls 2005)

Question	Yes	Maybe	No
Does the bridge have high average daily traffic (ADT) or average daily truck traffic (ADTT) or is it over an existing high-traffic-volume highway?			
Is the bridge over a railroad or navigable waterway, or is it on an emergency evacuation route?			
Will traffic be subject to back-ups when using the bridge during construction, or be subject to excessive detours during construction of the bridge?			
Is this project an emergency bridge replacement?			
Must traffic flow be maintained on the bridge during construction?			
Can the bridge be closed during off-peak traffic periods, e.g., nights and weekends?			
Does the bridge have multiple identical spans?			
Can the bridge be grouped with other bridges for economy of scale?			
Will roadway construction activities away from the bridge be completed quickly enough to make rapid installation of prefabricated bridge a cost effective solution?			
Can adequate time be allocated from project award to site installation to allow for prefabrication of components to occur concurrently with site preparation?			
Do worker safety concerns at the site limit conventional methods e.g., adjacent power lines or over water?			
Is the site in an environmentally sensitive area requiring minimum disruption (e.g., wetlands, air quality, noise, etc.)?			
Is the bridge location subject to construction time restrictions due to adverse economic impact?			
Are there natural or endangered species at the bridge site that necessitate short construction time windows or suspension of work for a significant time period, e.g., fish passage or peregrine falcon nesting?			
If the bridge is on or eligible for the national register of historic places, is prefabrication feasible for replacement/rehabilitation per the memorandum of agreement?			
Is the bridge site accessible for delivery of prefabricated components or use of heavy lifting equipment?			
Does the location of the bridge site create problems for delivery of ready-mix concrete?			
Does the local weather limit the time of year when cast-in-place construction is practical?			
Does the height of substructures make use of formwork to construct them inconvenient or impractical?			
Are fabricators available to economically manufacture and deliver the required prefabricated components?			
Are there contractors available in the area with sufficient skill and experience to perform prefabricated bridge construction?			
Does the height of the bridge above ground make false work uneconomical or impractical?			
Totals:			

2.7.10 Utah DOT ABC Decision-Making Process

UDOT developed a model, which is an extended version of the PBES model developed by Ralls (2005). The flowchart by Ralls (2005) (Figure 2–29) was modified by incorporating additional parameters. The *Yes /No* option selection was retained. Selecting one *Yes* choice on a critical parameter can lead to ABC implementation decision. Again, in this procedure, quantitative and informed judgment as to the relative importance of parameters was absent. To overcome this, UDOT in 2010 developed a scoring table with a modified flowchart (UDOT 2010a). In this new model, the mathematical method of a *scoring model* was utilized.

UDOT, in their decision-making model, focused on only one set of parameters rather than grouping data as major-parameters and sub-parameters. The parameters were divided into site-specific options with an ordinal scale of 0 to 5 (Table 2-24). Predefined weights were assigned to each parameter (Table 2-25, column b). For ABC decision evaluation, at a specific site, the site specific options will be assigned values depending on the site characteristics. The values entered by experts are multiplied with predefined weights and then summed to obtain a total score (Table 2-25, column c). This total score, with a maximum score (Table 2-25, column e), is assigned as the ABC rating (Eq.2-5). Finally, the ABC rating is used in the modified flowchart (Figure 2–30) for the final decision.

$$\text{ABC rating} = \frac{\text{Total score}}{\text{Maximum score}} \times 100 \tag{2-5}$$

Even though the UDOT procedure is an improvement to the PBES decision-making model, adequate descriptions for the predefined weights are not provided (Table 2-25, column b). The process is not flexible; thus project specific features cannot be addressed. Some tangible parameters, which will have a greater impact on the decision, are not included (e.g., impact on surrounding communities, contractor or precast plant experience, etc.). Moreover, in the modified flowchart, the decision box "Administrative decision by region/PD directors" is not clearly described. This decision box may switch the decision even when the rating for a construction alternative is within *0 to 20* (Figure 2–30).

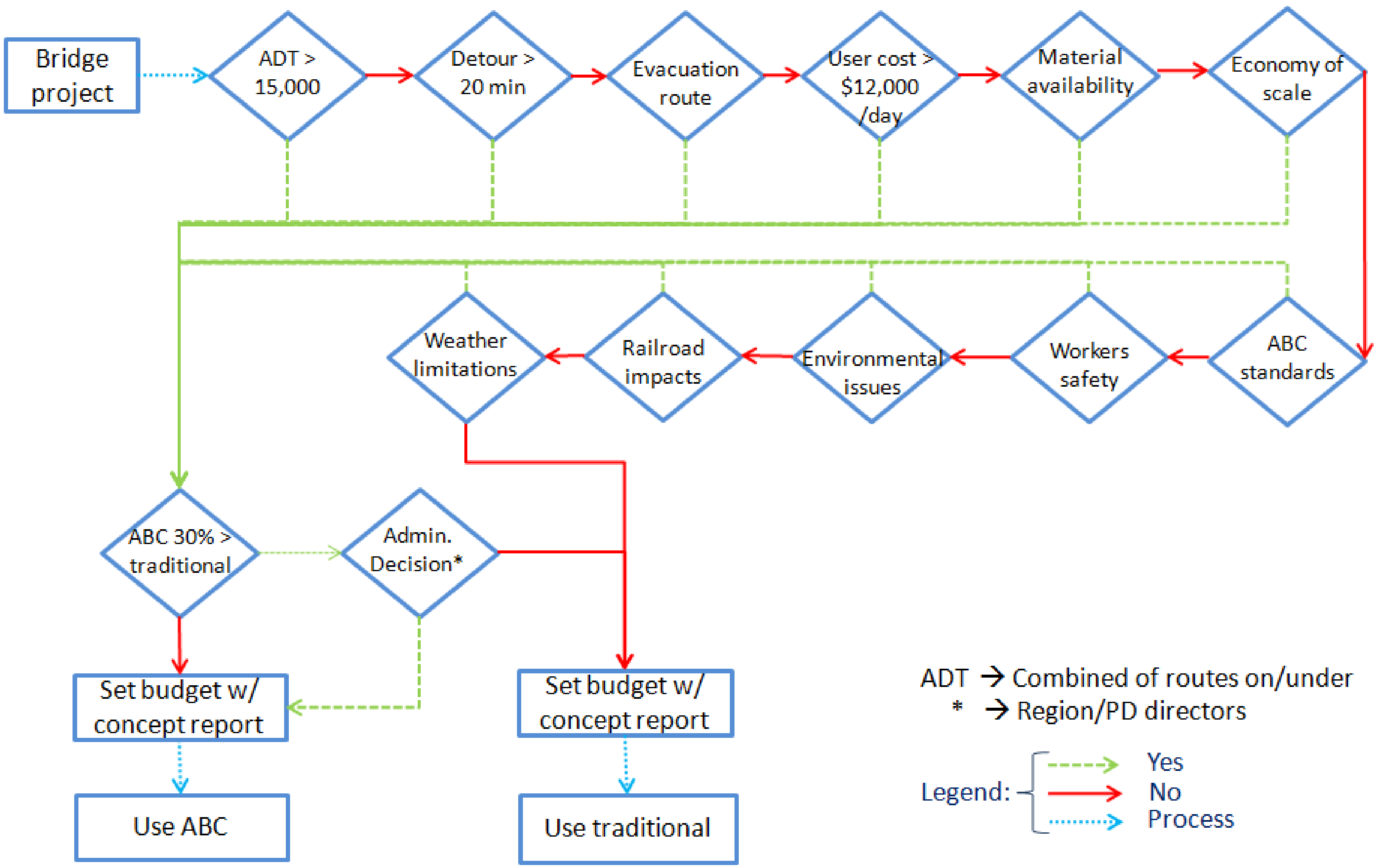


Figure 2–29. UDOT ABC decision chart (Source: Ralls 2008)

Table 2-24. UDOT ABC Scoring Sheet (Source: UDOT 2010a)

ABC rating procedure: Enter values for each aspect of the project. Attach back-up data if applicable			
Average daily traffic	X1	0	No traffic impacts
Combined on and under		1	Less than 5000
Enter 5 for Interstate Highways		2	5000 to 10000
		3	10000 to 15000
		4	15000 to 20000
		5	More than 20000
Delay time	X2	0	No delays
		1	Less than 5 minutes
		2	5-10 minutes
		3	10-15 minutes
		4	15-20 minutes
		5	More than 20 minutes
Bridge classification	X3	1	Normal Bridge
		3	Essential Bridge
		5	Critical Bridge
User costs	X4	0	No user costs
		1	Less than $10,000
		2	$10,000 to $50,000
		3	$50,000 to $75,000
		4	$75,000 to $100,000
		5	More than $100,000
Economy of scale	X5	0	1 span
(total number of spans)		1	2 to 3 spans
		2	4 to 5 spans
		3	More than 5 spans
Etc.		…….	

Table 2-25. Total Score Calculation of UDOT ABC Parameters (Source: UDOT 2010a)

Parameter	Score (a)	Weight (b)	Adjusted score (c)	Maximum score (d)	Maximum adjusted score (e)
Average daily traffic	X1	10	X1 * 10	5	50
Delay time	X2	10	X2 * 10	5	50
Bridge classification	X3	4	X3*4	5	20
User costs	X4	10	X4*10	5	50
Economy of scale	X5	3	X5*3	3	9
Use of typical details	X6	3	X6*3	5	15
Safety	X7	8	X7*8	5	40
Railroad impacts	X8	5	X8*5	5	25
Weather limitations	X9	3	X9*3	5	15
			Total score = ∑		Max. score = 274

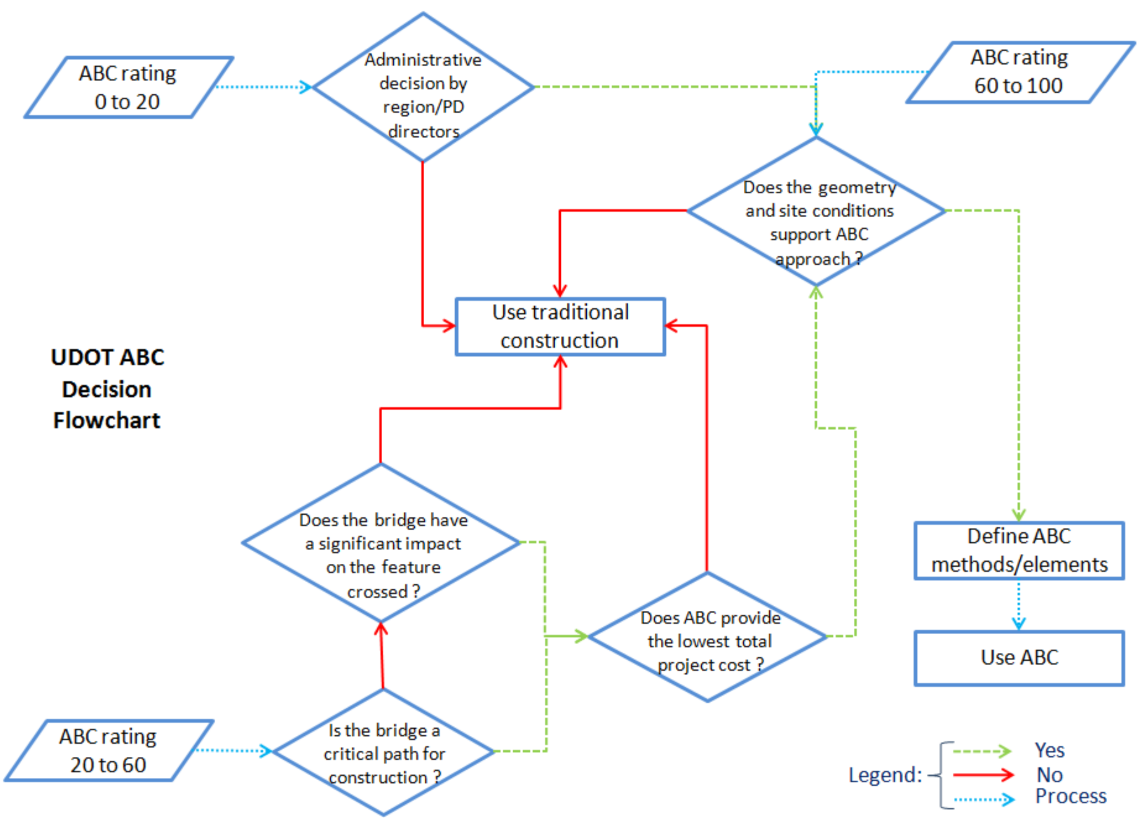


Figure 2–30. Modified UDOT ABC decision chart (Source: UDOT 2010a)

2.7.11 A Planning Phase Decision-Making Software for ABC

The decision-making software, which was developed by Doolen (2011) under the FHWA-sponsored pool fund study, considers relative appraisal of ABC parameters. This decision-making software utilizes AHP to quantify the qualitative trade-offs between the parameters to calculate the overall priority of respective construction alternatives. The decision-making platform is formally known as the *AHP decision-making environment*. It is developed in a Microsoft Visual Studio.NET application to evaluate between conventional and ABC alternatives.

The parameters (criteria) for this decision-making platform are gathered from interviews with various State Department of Transportation officials. The parameters are grouped into 5 major-parameters and associated sub-parameters. These parameters are arranged in a hierarchical format (Figure 2–31). The platform allows customization of major-parameters and sub-parameters with respect to the site-specific conditions. The graphical user interface allows users to navigate between four tabs (Figure 2–32) which are i) decision hierarchy, ii) pair-wise comparison, iii) results, and iv) cost weighted analysis. The three major steps are accessed by the first two tabs. In the first tab, the user has the option to add or delete sub-parameters (Figure 2–32). In the second tab, pair-wise comparison of the major-parameters, sub-parameters, and construction alternatives are performed qualitatively on a fixed ordinal scale of 1 to 9 (Figure 2–33). The pair-wise comparison matrices are generated and evaluated to calculate the local priorities using the *approximate method* developed by Saaty (1980). The *approximate method* involves forming normalized matrices from the pair-wise comparison matrices. Then each element of the normalized matrix is divided by a corresponding column total to form a resultant matrix. The rows of that resultant matrix are averaged to obtain the local priorities. A similar procedure is performed for all three steps of analysis. The final priority values of the construction alternatives are obtained after integrating local priorities from the three AHP steps. The construction alternative with highest priority will be the preferred one.

This *AHP decision-making platform* is developed to be used by a single user at a time (Doolen 2011). In the case of multiple users, each has to execute the program separately and discuss the choice with each other without a defined process.

The AHP calculates priority values for the alternatives. These priority values may change significantly with slight deviations in major-parameters' or sub-parameters' preferences. To account for this inconsistency, sensitivity analysis is recommended by Forman and Selly (2000). The *ABC decision-making platform* does not address the sensitivity analysis. Generally, a sensitivity analysis for any AHP evaluation is performed by varying one parameter preference, without changing other parameter preference ratings from their actual values. The sensitivity analysis can be performed by evaluating the process for multiple trials. In each trial, preference of parameters are varied (major-parameter and sub-parameter) independently. The results, when plotted on a bar chart, will show the sensitivity of priority value of alternatives with respect to each parameter (major-parameter or sub-parameter).

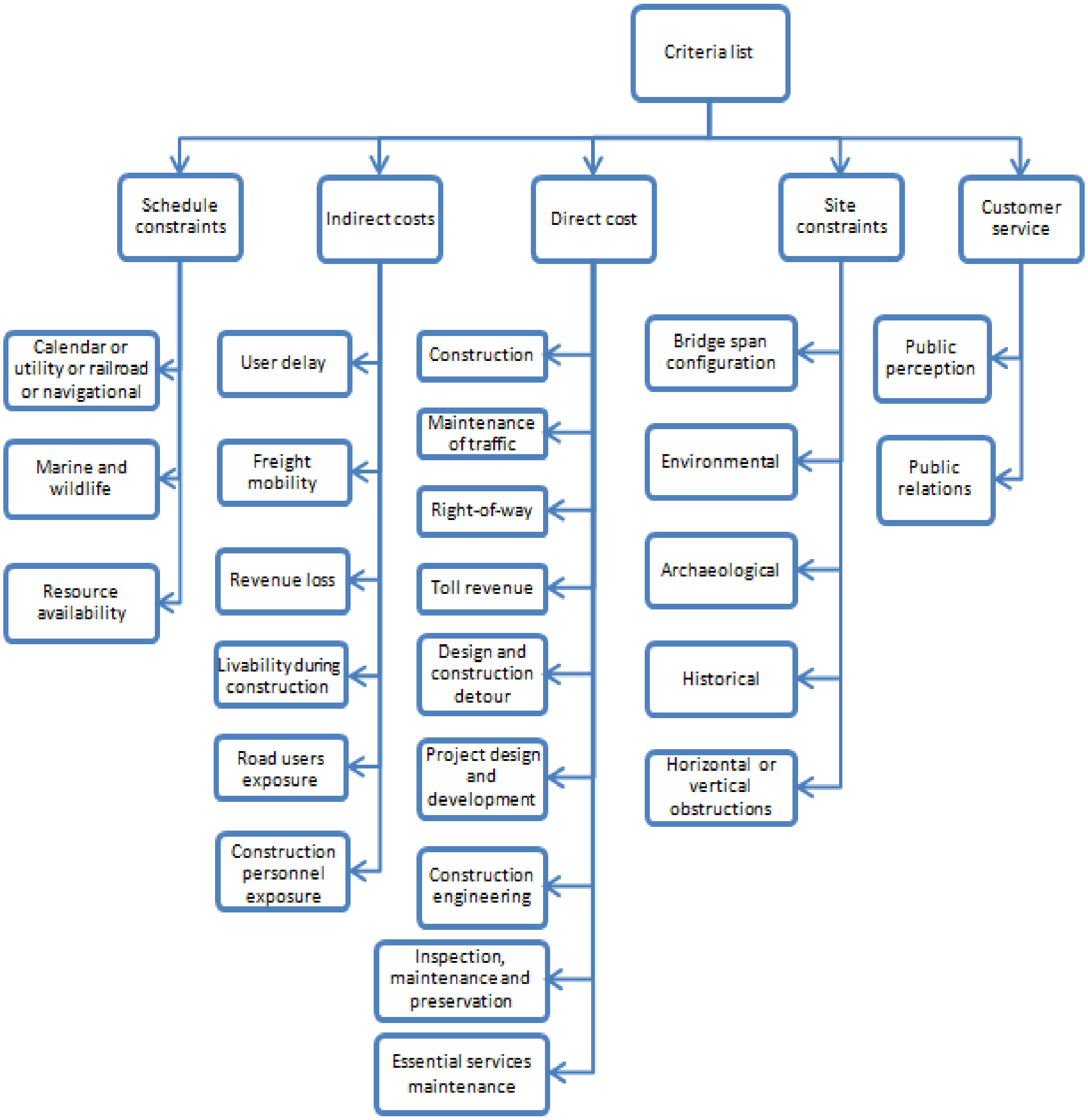


Figure 2–31. Default criteria hierarchy of the AHP decision-making software (Source: Doolen 2011)

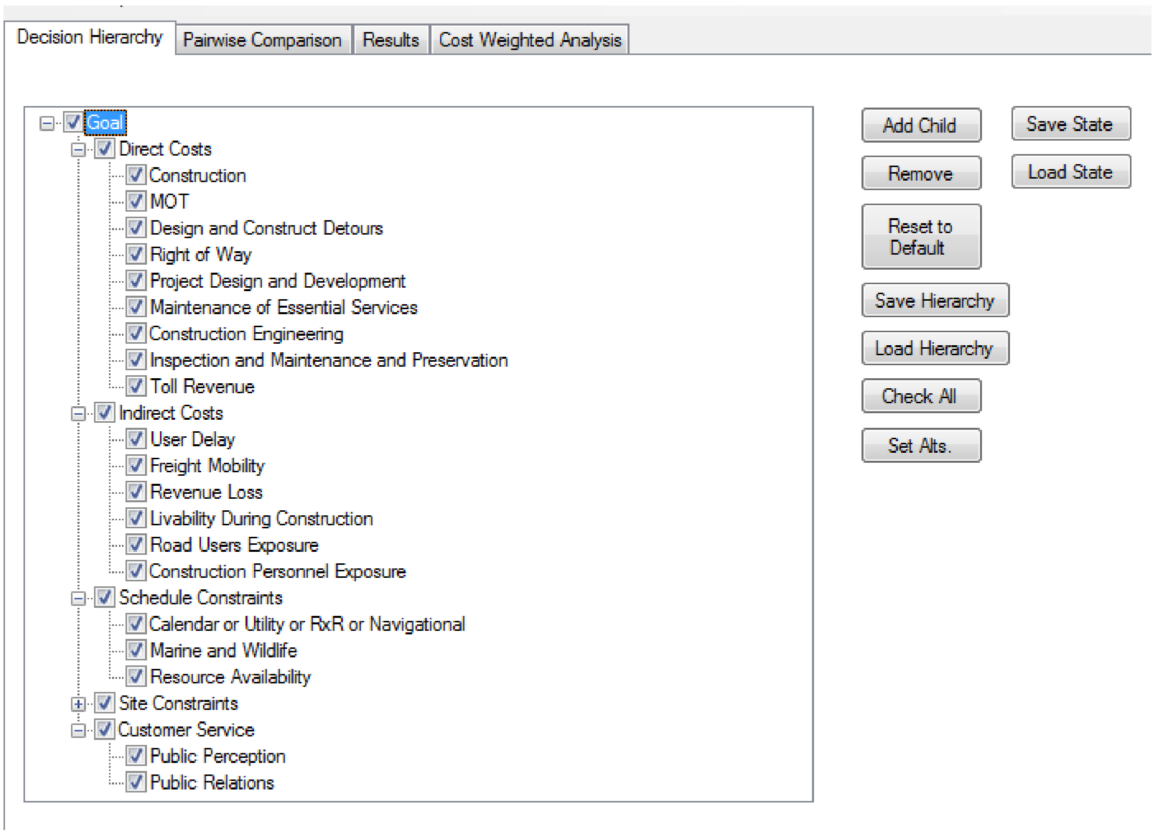


Figure 2–32. Graphical user interface of the AHP decision-making software (Source: Doolen 2011)

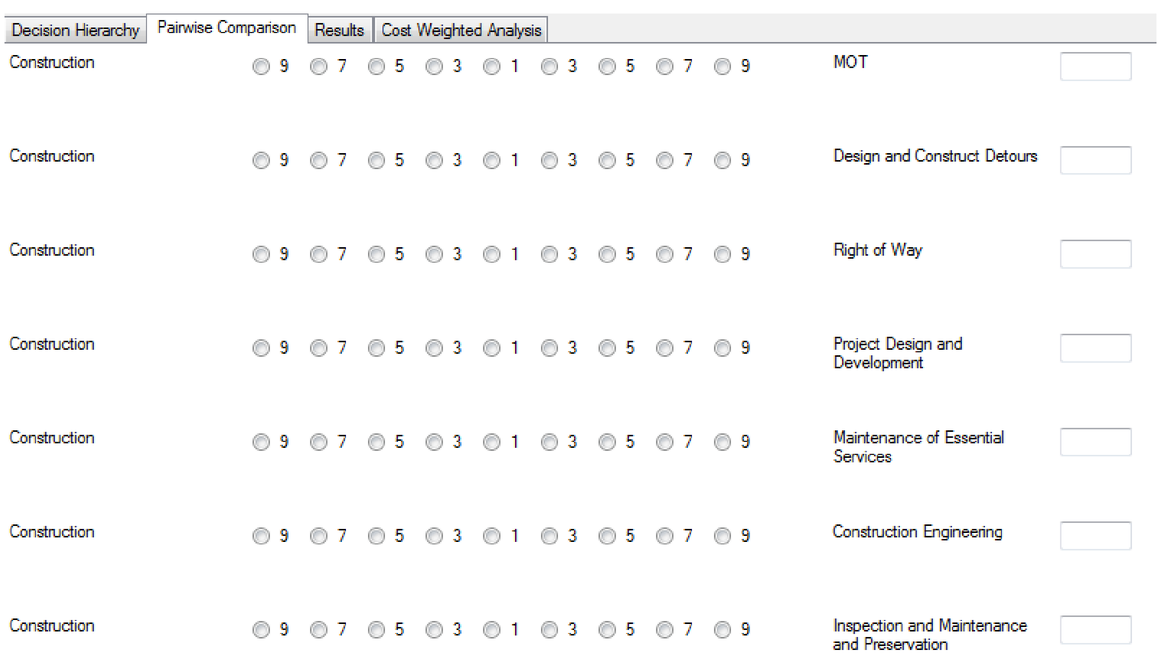


Figure 2–33. Qualitative pair-wise comparisons in AHP decision-making software (Source: Doolen 2011)

The *ABC decision making platform utilizing AHP* is superior to the previously discussed models. A sensitivity analysis was performed for its validation using a prototype bridge site. The *decision making platform* was used to evaluate among the alternatives viz., conventional cast-in-place construction (CIP) and ABC. Major-parameters were identified as duration, environment, safety, site condition, traffic, and cost. These major-parameters included several sub-parameters. In the example, the site was assumed to be in a rural area with low traffic volume of both facility carried and feature intersected (i.e., for sensitivity analysis with respect to the *Traffic* parameter). Generally, the major-parameters are pair-wise compared and assigned a preference rating based on the expert's experience and knowledge; whereas, the sub-parameters are pair-wise compared and assigned a preference rating with respect to site-specific conditions. To perform the sensitivity analysis, the major-parameters were assigned equal preference ratings (Figure 2–34, no.1); whereas, the sub-parameters were assigned preference ratings with respect to the site specific conditions (Figure 2–34, no.2). Most of the sub-parameters such as low ADT, short detour length, and low significance of the Level of Service (LOS) are biased towards conventional cast-in-place construction in the decision making process. Finally, *platform* was executed for evaluation, and the results showed that the ABC alternative is preferred (Figure 2–34, no.3). The analysis of the results show that i) ABC is governed for a site in a rural area with low ADT, short detour length, and low significance of LOS, which seems an unlikely preference, and ii) although the sub-parameters have different local priorities (Figure 2–34, no. 2), the results show constant values of each alternative under all major-parameters (in Figure 2–34, no. 4 red circle mark). This cannot be correct, because for AHP calculation, the values in Figure 2–34 no.4 should be calculated by integrating the major-parameter local priorities with their sub-parameter local priorities.

Further, the sensitivity plot for this analysis is generated by altering ratings only for the *ADT sub-parameter* (Figure 2–35). The sensitivity plot shows that the decision alternatives (i.e., conventional CIP and ABC) have equal values for each major-parameter. This cannot be correct because the major-parameter preference ratings are kept constant while the sub-parameter preference ratings are changed. The red and blue lines in Figure 2–35, showing the decision alternatives having equal weight corresponding to each major parameter, should take different values because of different sup-parameter preference ratings. This error is the result of *decision-making software* not checking the consistency ratio, while the local priorities are calculated using the approximate method.

Therefore, the drawbacks pertaining in this *decision-making procedure* are as follows:

- Project specific data are not provided to users during pair-wise comparison, leaving users to rely upon their choices without any supportive information (i.e., if the user has access to quantitative data on the parameters, then preference ratings will be more consistent).
- The failure in addressing the consistency ratio generates erroneous results (as shown by the sensitivity analysis of an example site). This is because multiple sub-parameters increase the number of pair-wise comparisons, thus the rank of the pair-wise comparison matrix. As mentioned by Saaty (1980), more variables in the pair-wise comparisons create consistency issues when the approximate method is used for calculating local priorities. A consistency ratio of less than 10% is required, or the pair-wise comparisons should be balanced (Saaty 1995).
- A complete understanding of the project and related data is impractical if it is to be obtained from a single source; a negotiation process among multiple decision makers will improve the accuracy of the final decision. The *platform* does not facilitate incorporating decisions from multiple decision makers.
- Incorporating further automation in the preference rating process will increase the consistency of the process.

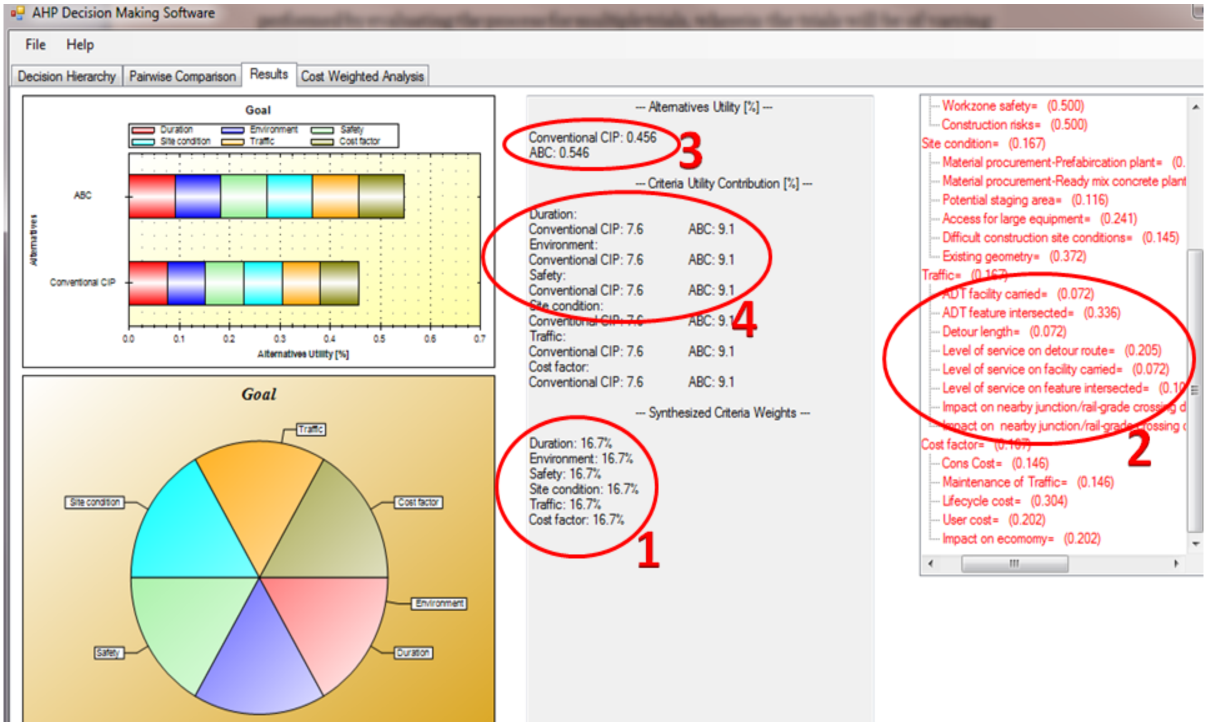


Figure 2–34. AHP decision-making software evaluation result for a prototype site

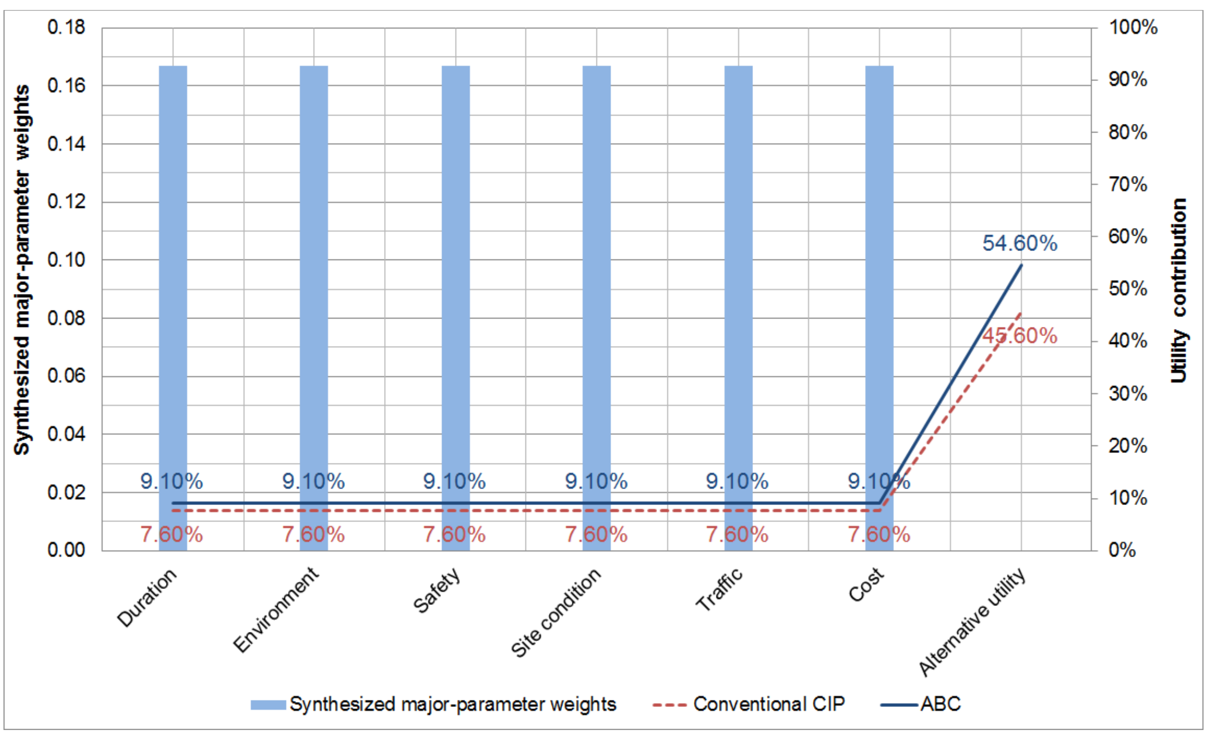


Figure 2–35. Plot of sensitivity analysis results from AHP decision-making software for a prototype site

2.7.12 Summary of Limitations in the Available Decision-Making Models

Several decision-making models reviewed during this study use the common strategy of requiring *Yes* or *No* inputs without any relative significance of critical parameters. Other decision-making processes are not really practical as they require a significant amount of survey and research. Also, a few of the decision-making models do not include a consistent method to assign weights and preference ratings for the parameters.

Some of the advanced analysis-based decision-making models (e.g., linear programming, fuzzy AHP, etc.) that include increased complexity as intricate mathematical calculations are required. Moreover, in some cases relating objective function and constraints to the qualitative parameters can get very complex.

Project specific data (quantitative data) is required to support the decision in the decision-making process. This would prevent the users to rely upon their subjective decision.

For the models using AHP methodology, the inability to address the consistency ratio while using the approximate method for calculating the local priorities is a major shortcoming. This may lead to erroneous results. Moreover, for higher order matrices, consistency may be difficult to achieve without *a computing aid*, because the number of transitive rules to be satisfied increases in a quadratic order.

The pair-wise comparison matrix is an in-depth process required in AHP. For an ideal AHP calculation, the *final weight* of each alternative at the level of each major-parameter should be calculated by integrating the major-parameter local priorities with their sub-parameter local priorities. The *final weight* should have a different value if any of the major-parameter or sub-parameter preference ratings are different.

Furthermore, the decision-making models developed are for a single user who is expected to know all the facts and data related to the project. Thus, there is a need to develop a collaborative decision-making model and a tool which allows preference ratings from multiple users.

Above all, the decision making models presented so far lack the use of project specific quantitative data. Therefore, there is a need to develop an ABC decision-making model that incorporates project specific data and available user-cost and life-cycle cost models to facilitate users with necessary quantitative data to make informed and accurate decisions. Moreover, some of the decision-making processes imply the need of precision for consistent results. To assure consistency and accuracy, the decision-making model can incorporate further automation to improve usability along with addressing the sensitivity of results. The decision-making model could be implemented using available programming platforms such as Microsoft Excel/ Visual Basic/ Mathcad/ Matlab.

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